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PROPOSED RESIDENTIAL TOWNHOUSE DEVELOPMENT
2532 ARGYLE ROAD
CITY OF MISSISSAUGA

PROJECT No. : 18201

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

Prepared For:

Plazacorp Investments Ltd.

Prepared By:

The Odan/Detech Group Inc.

Original: October 19th, 2018

TABLE OF CONTENTS

DESCRIPTION page

LIST OF FIGURES

LIST OF TABLES

APPENDIX A

Existing Site	Aerial view of Site and surrounding area
Site Plan & Statistics	by architectureunfolded

APPENDIX B

Argyle Road storm sewer design sheet
Visual OTTHYMO Model Output 2-year storm, 10-year storm & 100-year storm

APPENDIX C

Functional Servicing Plan
Functional Grading Plan

1.0 INTRODUCTION

The property under study is a 0.661 Ha (1.6 acre) site located at 2532 Argyle Road in the City of Mississauga. The site is bound by the following:

- Argyle Road to the east
- An existing residential highrise development to the north
- A landscaped area within the adjacent highrise development to the west
- Existing detached houses to the south and on the opposite side of Argyle Road

Refer to the Key Plan in Appendix A for the site's layout and adjacent developments.

The site presently comprises three existing detached houses in three separate lots.

It is proposed to demolish the three existing houses. It is proposed to construct a townhouse development comprising a common one-level below-grade parking structure and four blocks of four-storey stacked townhouses with a total of 92 townhouse units. Refer to the architectural Site Plan in Appendix A.

For detailed topography of the existing site conditions, as of January 15, 2017, refer to the topographic survey prepared by R. Avis Surveying Inc.

This report evaluates the serviceability of the site with respect to sanitary waste water, water and storm water management (SWM) and will implement the City of Mississauga's SWM requirements and criteria.

2.0 SCOPE OF WORK

THE ODAN/DETECH GROUP INC. was retained by **Plazacorp Investments Ltd.** to review the Site, collect data, evaluate the Site for the proposed commercial use and present the findings in a Functional Servicing and Storm Water Management Report in support of a Zoning Bylaw Amendment application. The scope of work in brief involves the following:

a) Collecting existing servicing drawings from the CITY in order to establish availability and feasibility of Site servicing;
b) Meetings/conversations with CITY Engineers and Design Team.
c) Evaluation of the data and presentation of the findings in a FSR and Storm Water Management Design Brief in support of the Zoning Bylaw Amendment application.

3.0 SANITARY SEWERS

i) Existing Infrastructure

There is an existing 250mm sanitary sewer flowing southerly beneath Argyle Road adjacent to the site's east frontage. This sewer continues easterly beneath Dunbar Road and then discharges into a 675mm sanitary sewer – which is assumed to be the trunk sewer – at the intersection of Dunbar Road and Rugby Road.

ii) Proposed Sanitary Servicing

The proposed townhouse development will be serviced for sanitary flows by a proposed 150mm sanitary service connection to the 250mm sanitary sewer beneath Argyle Road.

Sanitary flow calculations are based on the following criteria provided in the Region of Peel's manual: *Public Works Design, Specifications & Procedures Manual – Linear Infrastructure – Sanitary Sewer Design Criteria (Rev. July 2009)*.

- flow rate = 302.8 L/person/day per capita
- Infiltration to be 0.0002m^{3}/sec/ha
- for residential areas, population of 3.5 persons per unit is to be used (row dwellings)
- The Harmon formula will be used for the peaking factor

The pre-development sanitary flows are as follows. Refer to the detailed calculation on the following pages.

TABLE 1 – Pre-Development Sanitary Flow

Component	Population (P)	Average Flow (l/s)	Peak Sanitary Flow (l/s)	Inflow & Infiltration (l/s)	Total Flow (l/s)
Ex 3 x DTH	33	0.12	0.50	0.13	0.64

The post-development sanitary flows are as follows. Refer to the detailed calculation on the following pages.

A unit population of 3.5 persons/unit has been adopted in the Post-Development flow calculation, rather than the Region standard of 175 persons/Ha for townhouses, because the Region standard would result in a population of approximately 1.0 person/unit for the proposed development. This is not realistic, therefore a unit population of 3.5 persons/unit has been used as this was used in other similar developments in Mississauga.

TABLE 2 – Post-Development Sanitary Flow

Component	Population (P)	Average Flow (l/s)	Peak Sanitary Flow (l/s)	Inflow & Infiltration (l/s)	Total Flow (l/s)
PROP TH's	322	1.13	4.59	0.13	4.72

The peak sanitary flow from the proposed development is ***4.72 L/s***, as shown above.

RESIDENTIAL SANITARY FLOW CALCULATIONS

Sanitary flow calculations as per Region of Peel Public Works Design Criteria Manual - Sanitary Sewer

PROJECT: 2532 Argyle Road Residential Townhouse Development

SCENARIO: PRE-DEVELOPMENT

COMMERCIAL SITE AREA (ha) =

RESIDENTIAL SITE AREA (ha) = 0.661

TOTAL SITE AREA (ha) = 0.661

LAND USE	NUMBER OF UNITS	SITE AREA, (ha)	GROSS FLOOR AREA, m2	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW l/sec	PEAKING FACTOR, M	TOTAL FLOW FROM LAND USE, l/sec
Single family (>10m frontage), using 50 person/hectare		0.66		33	10008	0.12	4.35	0.50
Single family (<10m frontage), using 70 persons/hectare				0	0	0.00	4.50	0.00
Semi-Detached, using 70 persons/hectare				0	0	0.00	4.50	0.00
Row Dwellings, using 175 persons/hectare				0	0	0.00	4.50	0.00
Apartments, using 475 persons/hectare				0	0	0.00	4.50	0.00
RESIDENTIAL Townhomes, using 3.5 persons/unit				0	0	0.00	4.50	0.00
TOTAL RESIDENTIAL								0.50
COMMERCIAL, Using 50 persons/ha				0	0	0.00	4.50	0.00
TOTAL COMMERCIAL								**0.00**
	0			0				

TOTAL

V1= 10008

Q1= 0.50
Q2= 0.00
Qinfil 0.13
Qtot 0.64

Q = (MqP/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Land use + infiltration)

V1= Total Volume from Land Use in liters

where : P is population
q = 302.8 L/person/day for proposed residential

A = gross site area
i = 0.20 L/sec/ha (infiltration rate)
Peaking Factor M = 1 + [14 / (4 + (P/1000,1/2))]

RESIDENTIAL SANITARY FLOW CALCULATIONS

Sanitary flow calculations as per Region of Peel Public Works Design Criteria Manual - Sanitary Sewer

PROJECT: 2532 Argyle Road Residential Townhouse Development

SCENARIO: POST-DEVELOPMENT

COMMERCIAL SITE AREA (ha) =

RESIDENTIAL SITE AREA (ha) = 0.661

TOTAL SITE AREA (ha) = 0.661

LAND USE	NUMBER OF UNITS	SITE AREA, (ha)	GROSS FLOOR AREA, m2	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW l/sec	PEAKING FACTOR, M	TOTAL FLOW FROM LAND USE, l/sec
Single family (>10m frontage), using 50 person/hectare				0	0	0.00	4.50	0.00
Single family (<10m frontage), using 70 persons/hectare				0	0	0.00	4.50	0.00
Semi-Detached, using 70 persons/hectare				0	0	0.00	4.50	0.00
Row Dwellings, using 175 persons/hectare				0	0	0.00	4.50	0.00
Apartments, using 475 persons/hectare				0	0	0.00	4.50	0.00
RESIDENTIAL Townhomes, using 3.5 persons/unit	92			322	97502	1.13	4.07	4.59
TOTAL RESIDENTIAL								4.59
COMMERCIAL, Using 50 persons/ha				0	0	0.00	4.50	0.00
TOTAL COMMERCIAL								**0.00**
	92			0				

TOTAL

V1= 97502

Q1= 4.59
Q2= 0.00
Qinfil 0.13
Qtot 4.72

Q = (MqP/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Land use + infiltration)

V1= Total Volume from Land Use in liters

where : P is population
q = 302.8 L/person/day for proposed residential

A = gross site area
i = 0.20 L/sec/ha (infiltration rate)
Peaking Factor M = 1 + [14 / (4 + (P/1000,1/2))]

4.0 WATER DISTRIBUTION

Design Considerations

There is an existing 300mm ductile iron watermain beneath Argyle Road adjacent to the site's east frontage. There is also an abandoned 150mm watermain beneath Argyle Road. Refer to the Functional Servicing Plan for the layout of the existing bordering watermains. They also appear on the following Fire Separation Distance Plan.

It is proposed to connect to the existing 300mm watermain for domestic water and fire protection. Refer to the Functional Servicing Plan for the proposed domestic water and fire services. The proposed incoming fire service is to be connected to the sprinklers provided in the underground parking garage and a proposed private hydrant within the private laneway. Refer to the Functional Servicing Plan. The proposed townhouses will not be sprinklered. They will be served by hydrants as follows.

The proposed townhouse units will be served for fire protection by the existing hydrant on Dunbar Road adjacent to the site's southeast corner. Townhouse Blocks A and D are more than 90m from the existing hydrant and therefore require a new hydrant within the site. A new hydrant is proposed as shown on the Functional Servicing Plan.

The unit rate and peaking factors of water consumption, minimum pipe size and allowable pressure in line were established from the City Design Manual Standards. 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 water demand for the proposed townhouse development is as follows. Domestic flow calculation criteria is given Tables 1 and 2 in the Region of Peel's *Public Works Watermain Design Criteria* manual (2009). The criteria is as follows. Table 2 in the Region manual is adopted as the criteria in this development as it is the more stringent criteria intended for new development.

a)	Average Day domestic demand -	using 409L/cap/day (322 persons – Table 2)	1.5 L/sec
b)	Max day demand -	2.0 x daily demand	3.0 L/sec
c)	Peak hour demand -	3.0 x daily demand	4.5 L/sec
d)	Fire flow as per FUS 1999 manual		333 L/sec

TABLE 3 – Total Water Demand

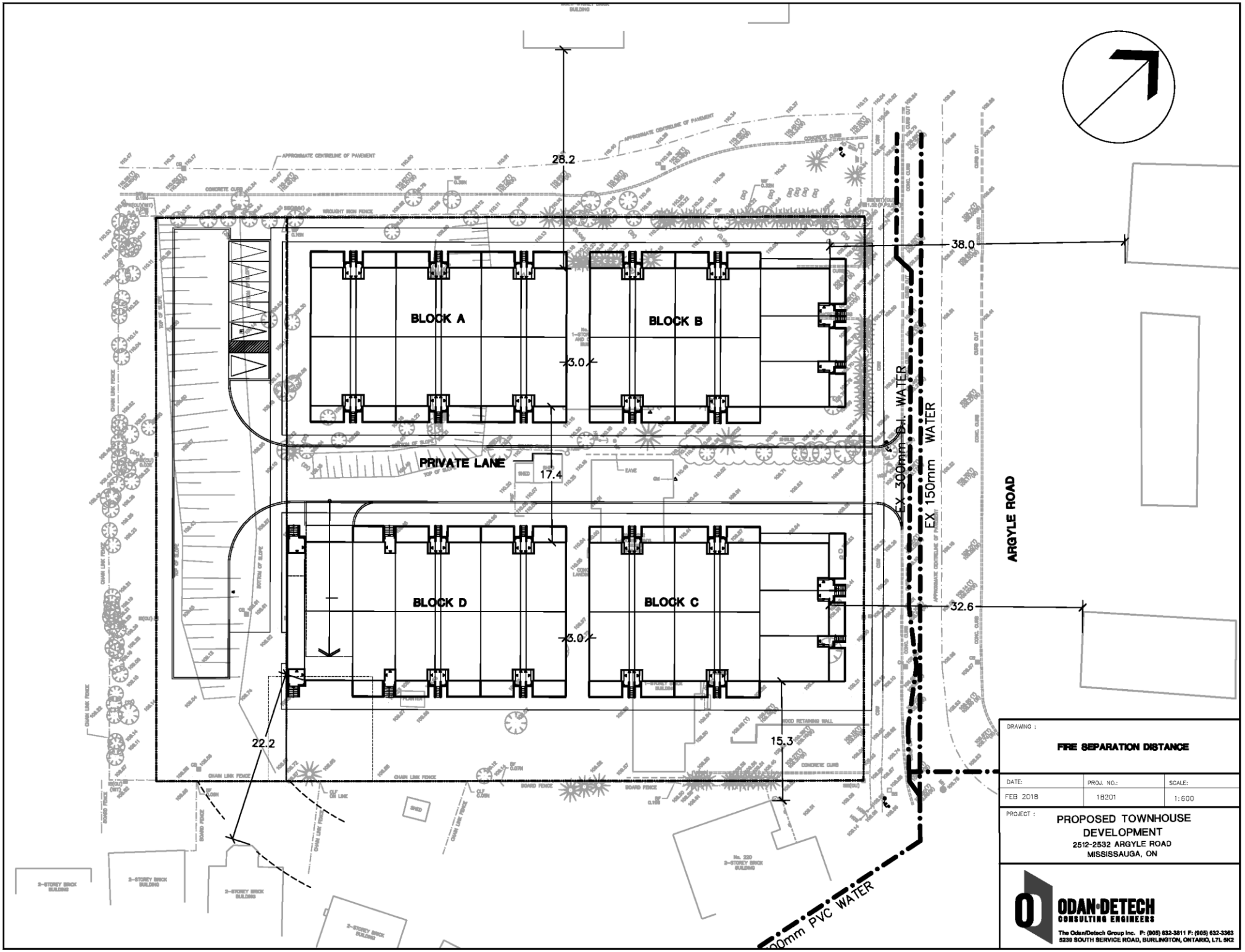
	L/sec	USGM
Max Day Demand	3.0	48
Fire Flow Demand (TH Block D)	333	5019
Total Water Demand	336	5067

The following assumptions are made in the following Fire Underwriters' Survey fire flow calculation.

- The proposed townhouse blocks (above-grade) are of wood-frame construction
- The Fire Underwriters' Survey calculation considers above-grade floors, not below-grade floors. The above-grade townhouse units are not sprinklered, therefore the FUS calculation is completed accordingly.
- The building's contents (residences) will be non-combustible in nature
- The setbacks from the adjacent buildings are shown on the following Fire Separation Distance Plan

Townhouse Block D has the largest fire flow demand and is taken as the development's fire flow demand. Refer to the following FUS calculations.

The Region is requested to provide boundary conditions with which to conclude the available flow in the receiving watermain. Hydrant flow tests will be provided in the future.



DRAWING :

FIRE SEPARATION DISTANCE

DATE:	PROJ. NO.:	SCALE:
FEB 2018	18201	1:600

PROJECT :

PROPOSED TOWNHOUSE DEVELOPMENT
2512-2532 ARGYLE ROAD
MISSISSAUGA, ON

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WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY
GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOWS

$F = 220 \times C \times \sqrt{A}$
Where:

F = required fire flow in liters per minute
C= Coefficient related to the type of construction
A = the total floor area in square meters (excluding basements) in the building considered

Coefficient related to type of construction	
1.5	Wood Frame
1	Ordinary
0.8	Non combustible
0.6	Fire Resistive

LOCATION: 2532 Argyle Road - Block A
OBC OCCUPANCY: Residential
BUILDING FOOT PRINT (m2): 562
OF STOREYS: 4

PROJECT: 2532 Argyle Road, Mississauga
PROJECT No 18201

Contents	Charge
Non-Combustible	-25%
limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Buring	25%

CONSTRUCTION CLASS: Wood Frame

AUTOMATED SPRINKLER PROTECTION

		Credit	Total
NFPA 13 sprinkler standard	no	0%	0%
Standard Water Supply	no	0%	
Fully Supervised System	no	0%	
		0%	

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

Separation	Charge
0-3 m	25%
3.1 -10 m	20%
10.1 - 20 m	15%
20.1 - 30 m	10%
30.1 - 45	5%
> 45 m	0%

EXPOSURE 1 (south) TH Block D	Distance to Exposure Building (m)	17	15%
	Length - Height		
EXPOSURE 2 (east) TH Block C	Distance to Exposure Building (m)	3	25%
	Length - Height		
EXPOSURE 3 (west) NA	Distance to Exposure Building (m)	>45	0%
	Length - Height		
EXPOSURE 4 (north) EX Apartment Bldg	Distance to Exposure Building (m)	28	10%
	Length - Height		
		Total:	50%

no more than 75%

ARE BUILDINGS CONTIGUOUS: Yes

FIRE RESISTANT BUILDING Are vertical openings and exterior vertical communications protected with a minimum one (1) h — No

CALCULATIONS

C =	1.5		Wood Frame
A =	2248	m2	
F =	15646	L/min	
Round to Nearest 1000 L/min F =	16000	L/min	must be > 2000 L/min

STOREY AREAS m2	
562	Semi-UG
562	1st
562	2nd
562	3rd

CORRECTION FACTORS:

OCCUPANCY	-3200	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	12800	L/min
REDUCTION FOR SPRINKLER	0	L/min
EXPOSURE CHARGE	6400	L/min

REQUIRED FIRE FLOW

F =	19200	L/min		
Round to Nearest 1000 L/min **F =**	19000	L/min	5019	usgm
F =	317	L/sec		

WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY
GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOWS

$F = 220 \times C \times \sqrt{A}$
Where:

F = required fire flow in liters per minute
C= Coefficient related to the type of construction
A = the total floor area in square meters (excluding basements) in the building considered

Coefficient related to type of construction	
1.5	Wood Frame
1	Ordinary
0.8	Non combustible
0.6	Fire Resistive

LOCATION: 2532 Argyle Road - Block B
OBC OCCUPANCY: Residential
BUILDING FOOT PRINT (m2): 527
OF STOREYS: 4

PROJECT: 2532 Argyle Road, Mississauga
PROJECT No 18201

Contents	Charge
Non-Combustible	-25%
limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Buring	25%

CONSTRUCTION CLASS: Wood Frame

AUTOMATED SPRINKLER PROTECTION

		Credit	Total
NFPA 13 sprinkler standard	no	0%	0%
Standard Water Supply	no	0%	
Fully Supervised System	no	0%	
		0%	

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

Separation	Charge
0-3 m	25%
3.1 -10 m	20%
10.1 - 20 m	15%
20.1 - 30 m	10%
30.1 - 45	5%
> 45 m	0%

EXPOSURE 1 (south) TH Block C	Distance to Exposure Building (m)	17	15%
	Length - Height		
EXPOSURE 2 (east) Ex House	Distance to Exposure Building (m)	38	5%
	Length - Height		
EXPOSURE 3 (west) TH Block A	Distance to Exposure Building (m)	3	25%
	Length - Height		
EXPOSURE 4 (north) Ex Apartment Bldg	Distance to Exposure Building (m)	26	10%
	Length - Height		
		Total:	55%

no more than 75%

ARE BUILDINGS CONTIGUOUS: Yes

FIRE RESISTANT BUILDING Are vertical openings and exterior vertical communications protected with a minimum one (1) h: No

CALCULATIONS

	C =	1.5		Wood Frame
	A =	2108	m2	
	F =	15151	L/min	
Round to Nearest 1000 L/min	F =	15000	L/min	must be > 2000 L/min

STOREY AREAS m2	
527	Semi-UG
527	1st
527	2nd
527	3rd

CORRECTION FACTORS:

OCCUPANCY	-3000	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	12000	L/min
REDUCTION FOR SPRINKLER	0	L/min
EXPOSURE CHARGE	6600	L/min

REQUIRED FIRE FLOW

	F =	18600	L/min	
Round to Nearest 1000 L/min	**F =**	19000	L/min	5019 usgm
	F =	317	L/sec	

WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY
GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOWS

$F = 220 \times C \times \sqrt{A}$
Where:

F = required fire flow in liters per minute
C= Coefficient related to the type of construction
A = the total floor area in square meters (excluding basements) in the building considered

Coefficient related to type of construction	
1.5	Wood Frame
1	Ordinary
0.8	Non combustible
0.6	Fire Resistive

LOCATION: 2532 Argyle Road - Block C
OBC OCCUPANCY: Residential
BUILDING FOOT PRINT (m2): 527
OF STOREYS: 4

PROJECT: 2532 Argyle Road, Mississauga
PROJECT No 18201

Contents	Charge
Non-Combustible	-25%
limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Buring	25%

CONSTRUCTION CLASS: Wood Frame

AUTOMATED SPRINKLER PROTECTION

		Credit	Total
NFPA 13 sprinkler standard	no	0%	0%
Standard Water Supply	no	0%	
Fully Supervised System	no	0%	
		0%	

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

EXPOSURE 1 (south) Ex House	Distance to Exposure Building (m)	15	0%
	Length - Height		
EXPOSURE 2 (east) Ex House	Distance to Exposure Building (m)	32	15%
	Length - Height		
EXPOSURE 3 (west) TH Block D	Distance to Exposure Building (m)	3	25%
	Length - Height		
EXPOSURE 4 (north) TH Block B	Distance to Exposure Building (m)	17	15%
	Length - Height		
		Total:	55%

no more than 75%

Separation	Charge
0-3 m	25%
3.1 -10 m	20%
10.1 - 20 m	15%
20.1 - 30 m	10%
30.1 - 45	5%
> 45 m	0%

ARE BUILDINGS CONTIGUOUS: Yes

FIRE RESISTANT BUILDING Are vertical openings and exterior vertical communications protected with a minimum one (1) h: No

CALCULATIONS

	C =	1.5		Wood Frame
	A =	2108	m2	
	F =	15151	L/min	
Round to Nearest 1000 L/min	F =	15000	L/min	must be > 2000 L/min

STOREY AREAS m2	
527	Semi-UG
527	1st
527	2nd
527	3rd

CORRECTION FACTORS:

OCCUPANCY	-3000	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	12000	L/min
REDUCTION FOR SPRINKLER	0	L/min
EXPOSURE CHARGE	6600	L/min

REQUIRED FIRE FLOW

	F =	18600	L/min	
Round to Nearest 1000 L/min	**F =**	19000	L/min	5019 usgm
	F =	317	L/sec	

WATER SUPPLY FOR PUBLIC FIRE PROTECTION , FIRE UNDERWRITERS SURVEY
GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOWS

$F = 220 \times C \times \sqrt{A}$
Where:

F = required fire flow in liters per minute
C= Coefficient related to the type of construction
A = the total floor area in square meters (excluding basements) in the building considered

Coefficient related to type of construction	
1.5	Wood Frame
1	Ordinary
0.8	Non combustible
0.6	Fire Resistive

LOCATION: 2532 Argyle Road - Block D
OBC OCCUPANCY: Residential
BUILDING FOOT PRINT (m2): 562
OF STOREYS: 4

PROJECT: 2532 Argyle Road, Mississauga
PROJECT No 18201

Contents	Charge
Non-Combustible	-25%
limited Combustible	-15%
Combustible	0%
Free Burning	15%
Rapid Buring	25%

CONSTRUCTION CLASS: Wood Frame

AUTOMATED SPRINKLER PROTECTION

		Credit	Total
NFPA 13 sprinkler standard	no	0%	0%
Standard Water Supply	no	0%	
Fully Supervised System	no	0%	
		0%	

CONTENTS FACTOR: Non Combustible — CHARGE: -20%

Separation	Charge
0-3 m	25%
3.1 -10 m	20%
10.1 - 20 m	15%
20.1 - 30 m	10%
30.1 - 45	5%
> 45 m	0%

EXPOSURE 1 (south) Ex House	Distance to Exposure Building (m)	22	10%
	Length - Height		
EXPOSURE 2 (east) TH Block C	Distance to Exposure Building (m)	3	25%
	Length - Height		
EXPOSURE 3 (west) N/A	Distance to Exposure Building (m)	>45	10%
	Length - Height		
EXPOSURE 4 (north) TH Block A	Distance to Exposure Building (m)	17	15%
	Length - Height		
		Total:	60%

no more than 75%

ARE BUILDINGS CONTIGUOUS: Yes

FIRE RESISTANT BUILDING Are vertical openings and exterior vertical communications protected with a minimum one (1) h — No

CALCULATIONS

C =	1.5		Wood Frame
A =	2248	m2	
F =	15646	L/min	
Round to Nearest 1000 L/min F =	16000	L/min	must be > 2000 L/min

STOREY AREAS m2	
562	Semi-UG
562	1st
562	2nd
562	3rd

CORRECTION FACTORS:

OCCUPANCY	-3200	L/min
FIRE FLOW ADJUSTED FOR OCCUPANCY	12800	L/min
REDUCTION FOR SPRINKLER	0	L/min
EXPOSURE CHARGE	7680	L/min

REQUIRED FIRE FLOW

F =	20480	L/min		
Round to Nearest 1000 L/min **F =**	20000	L/min	5283	usgm
F =	333	L/sec		

5.0 STORM WATER MANAGEMENT & DRAINAGE PROPOSAL

i) Background Information & Existing Infrastructure

Presently the following existing separated storm sewers are adjacent to the subject site. Refer to the Functional Servicing Plan and the Pre-Development Drainage Plan on the following page for the existing storm sewers adjacent to the subject site and the existing site drainage patterns.

1. There is an existing 2400mm x 3600mm at 2.0% box culvert conveying Mary Fix Creek in an approximately 17m wide easement in the west side of the subject site. This culvert commences at an inlet north of the site and south of Dundas Street. The operating condition of this culvert – which provides major system storm outlet for the subject site – defines the manner in which major storms are addressed in the site's stormwater quantity controls.

    1.1. The culvert was constructed in the 1970's and 1980's to provide conveyance for major storms, evidently based on hydrological calculations available at the time.

    1.2. A *HEC-RAS* floodplain model of Mary Fix Creek was completed in 2000 by Credit Valley Conservation (CVC), thereby establishing the reaches of Mary Fix Creek upstream of the inlet to this culvert in a CVC regulated area. ***The subject site is not presently in the regulated area.***

    1.2.1. Aside: the existing *HEC-RAS* analysis does not include the section of Mary Fix Creek in the culvert within the subject site, therefore the operating condition of the existing culvert in the 100-year storm cannot be concluded on the basis of that analysis.

    1.3. CVC staff are presently undertaking a new hydrologic and hydraulic analysis of the Mary Fix Creek floodplain which is conveyed by the subject culvert within the site. The analysis will determine present flow condition of the culvert in the critical 100-year storm. CVC staff have stated that the analysis is to be completed late-2018.

    1.3.1. Further to CVC direction – in anticipation of CVC's future analysis – The Odan/Detech Group undertook an independent analysis of the subject culvert and surface conveyances relative to the 100-year storm. The preliminary results were provided in a memo titled *Site Flood Analysis* by The Odan/Detech Group Group, dated June 25, 2018.

        1.3.1.1. The June 25/2018 memo addresses flow conditions in the subject 2400mm x 3600mm culvert and on the surface which are relevant for site grading and stormwater management design. There are two critical scenarios/flow conditions which were modelled:

        - Scenario 1: scenario where flows leaving the existing storm pond north of Dundas Street are considered to be attenuated by the pond – flow at culvert inlet is ***74.2 L/s***.

        - Scenario 2: scenario where flows are not considered as attenuated – flow at culvert inlet is ***43.1 L/s***.

1.3.1.2. The hydrographs for the two scenarios were provided by CVC from their ongoing hydrological study and are provided in Appendix B of the June 25/2018 *Site Flood Analysis* memo.

1.3.1.3. The results of The Odan/Detech Group's flood conveyance analysis are provided in the two foregoing scenarios in Tables 1 and 2 (page 27) of the memo. It was generally concluded that the subject culvert is flowing at or near full in either scenario and that there is a spill onto the surface in the scenario where the flows are unattenuated (Scenario 1). The spill onto the surface results in stormwater from the culvert passing through the subject site on the surface at a flow depth of up to 0.48 m as shown in Table 1 of the June 25/18 memo.

1.3.1.4. It has not presently been concluded which scenario will apply to this site if CVC's analysis is completed and registered before this development's zoning is approved; whether a spill condition is deemed to exist on the basis of the upstream stormwater not being attenuated by the pond or else if the stormwater is attenuated leaving the pond, whereby all stormwater is conveyed by the culvert. The site's design has been conservatively prepared to address both scenarios.

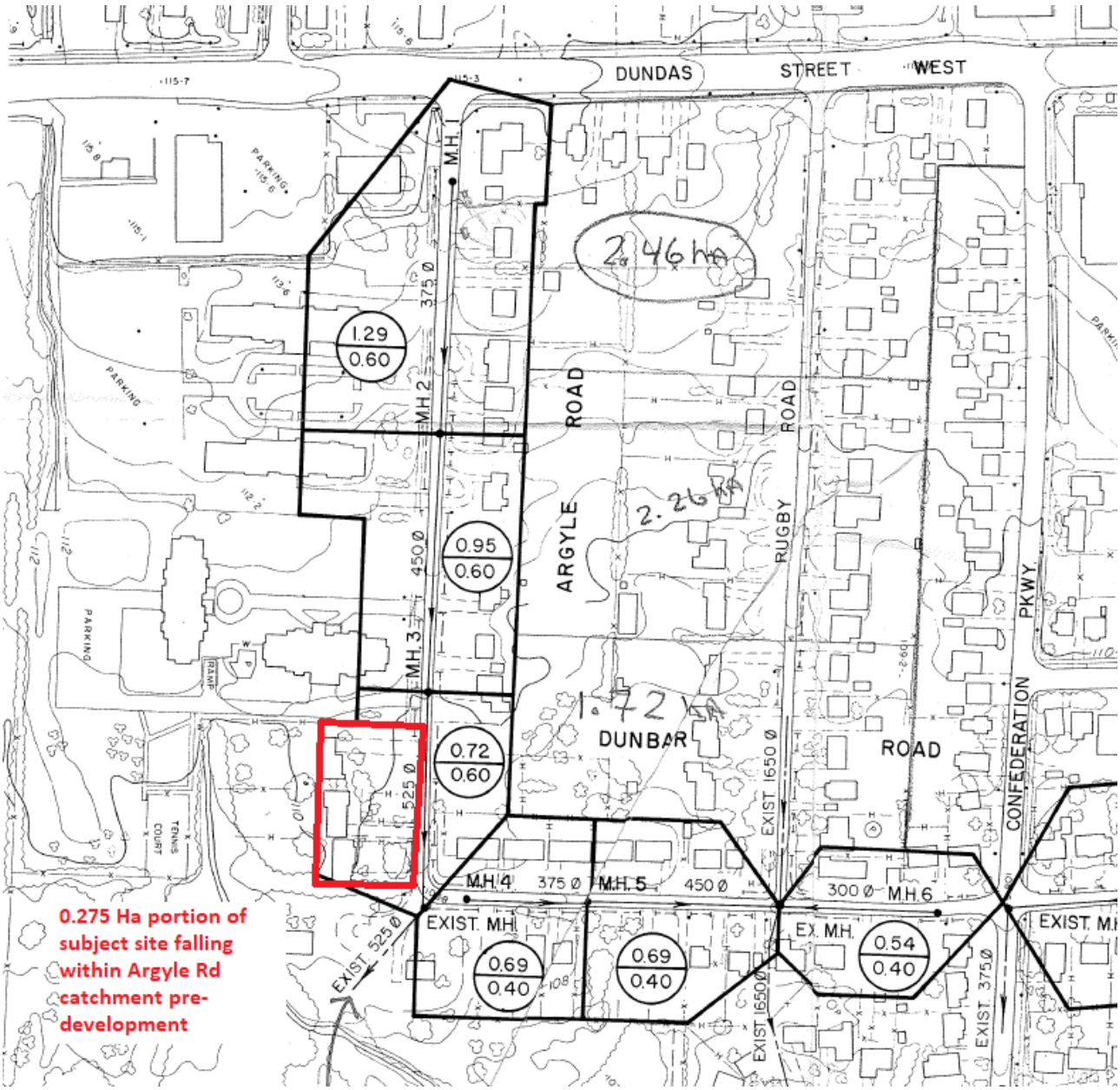
2. There is an existing 525mm storm sewer flowing southerly beneath Argyle Road, which discharges into the foregoing culvert downstream of the subject site.

   2.1. This sewer was designed for the 10-year storm with a C-value of 0.60, based on sewer design sheets and catchment plans provided by the City of Mississauga.

   2.2. A 0.275 Ha portion of the site was allocated to drain into this sewer based on the excerpt from the sewer's drainage plan, shown in Figure 1, below.

3. Design criteria for storm drainage design based on the foregoing conveyances are discussed below.

Figure 1 - Excerpt from Argyle Road storm drainage plan showing area in site with allocation



ii) Design Criteria

The City of Mississauga's *Development Requirements Manual (Effective September 2016)* provides criteria for stormwater management design. Table 2.01.03.03c therein states that developments in the Mary Fix Creek watershed should control 10-year post-development to 2-year pre-development storms. Note 1 on that table states that storm sewer capacity constraints may govern. Note 2 on that table states that pre-development C-value should be no greater than 0.50.

City staff have stated that 5mm rainfall event retention is required and that a best-effort to implement LID should be provided.

City staff have stated that stormwater quality control shall be provided by way of development charges, therefore no quality control measures are specified.

Design storm data for the City of Mississauga 2 year, 10 year and 100 year storms are shown below.

$i_2 = \frac{610}{(t_c+4.6)^{0.78}}$, $i_{10} = \frac{1010}{(t_c+4.6)^{0.78}}$, $i_{100} = \frac{1450}{(t_c+4.9)^{0.78}}$

where: *i* = intensity (mm/hr)
t = time of concentration (15min)

iii) Proposed Drainage & Allowable Discharge Flow Rate

The proposed development will drain storm flows to two outlets, and the pre-development or allowable discharge to each is established below based on the relevant criteria for each outlet.

1. Existing 3600mm x 2400mm culvert in easement.

    a. The western area of the development (western/rear drive aisle area; area of easement) will drain into this culvert by overland flow into the two existing catchbasins (EX CB1 and EX CB2).

    b. This is necessary to maintain surface flow conveyance for spill/flood water from Mary Fix Creek if the governing criteria is Scenario 1 (above) where a spill occurs through the site. The site has conservatively been designed to accommodate the Scenario 1 spill scenario. Design considerations arising from Scenario 1 condition:

        i. The topography is such that the spill/flood water flows through the site, from external of the site, approximately from the northwest to southwest corner.

        ii. It is not possible to control only the subject site's runoff in isolation in this scenario because there would be significant flows entering the site from elsewhere and it is therefore not feasible to control only the subject site's runoff among the flood/spill-water also passing through the site in Scenario 1.

2. Existing 525mm Argyle Road storm sewer.

    a. There is allocation in the Argyle Road storm sewer for a portion of the subject site as evidenced in Figure 1. The site will drain into this sewer based on the existing allocation, the sewer's capacity and the criteria for quantity control prescribed by the City of Mississauga and described above.

The site's allowable discharge rate into the two foregoing outlets is as follows in Table 4. The design criteria for the discharge to the Argyle Road storm sewer is as follows.

1. The 2-year pre-development flow with C=0.50, in accordance with the foregoing City criteria (as per Table 4), as well as:

2. Receiving storm sewer capacity – maintaining the pre-development flow conditions in the receiving 525mm storm sewer beneath Argyle Road (as per the below discussion and sewer design sheets)

TABLE 4 – Allowable Flow Rate

Receiving Outlet	Run-off Coefficient	Rainfall Intensity (mm/hr)	Area (ha)	Site Allowable Discharge (L/s)
Argyle Road 525mm storm sewer	0.50	59.9 mm/hr (2-Y Storm)	0.546 Ha	**45.5 (2-Y Storm)**
3600mm x 2400mm Mary Fix Creek Culvert	0.30*	59.9 mm/hr (2-Y Storm)	0.386 Ha (Catchment EX-A)*	**19.3 L/s (2-Y)**
		140.7 mm/hr (100-Y Storm)		**45.3 L/s (100-Y)**

**Refer to the Pre-Development Catchment Plan on the following page for Catchment EX-A, the portion of the site which drained into the Mary Fix Creek Culvert pre-development*.

Note that the foregoing allowable release rate to the Argyle Road 525mm sewer is based on an area of 0.546 Ha, rather than the site's total area of 0.661 Ha. This is because the use of the site's total area (0.661 Ha) in this calculation would result in the site's impact on the receiving storm sewer infrastructure being greater than in pre-development conditions.

Refer to the storm sewer design sheets on the following pages showing the pre-development and post-development impact on the receiving storm sewer in 10-year storms. The storm sewer was originally designed to convey the 10-year storm based on the storm sewer design sheet provided by City staff, provided here in Appendix B.

By the foregoing controlled release rate criteria, the proposed development causes no additional impact on the receiving Argyle Road storm sewers. The receiving segments are flowing at a lesser percentage of capacity in post-development as compared to pre-development conditions.

As seen in the following storm sewer design sheets, the 450mm downstream pipe segment is flowing at 120% of it's capacity in post-development conditions. The sewer was therefore modelled using *XPSWMM 2017.1.1* by XP Solutions to identify the existing surcharging. The catchment areas were inputted as shown on the sewer design sheets.

The HGL plot generated by XPSWMM, Figure 2, below, shows that there is minor surcharging at *EX MH*, downstream of the subject site, in the existing and proposed condition. The storm sewer post-dates the houses on Argyle Road therefore it is assumed that any foundation drains that the houses might have are not connected to the storm sewer. It is additionally noted that no laterals into the storm sewer from houses appears on the storm sewer's plan-profile drawing (City drawing no. C-28466). On this basis, the surcharging does not pose a risk of basement flooding.

The proposed development's controlled discharge rate to the 525mm storm sewer beneath Argyle Road will be 45.5 L/s based on the foregoing calculations and discussion.

The *Pre-Development Drainage Plan* on the following page shows the pre-development drainage patterns in accordance with the City Argyle Road storm sewer drainage plan.

PRE-DEVELOPMENT STORM SEWER DESIGN SHEET - Existing Argyle Road Storm Sewer

Site location: Argyle Road, Mississauga

Ref# PN 18201

Location				Segment Tributary Area	Accumulative Tributary Area	Time of Concentration	10-year Rainfall Intensity	Segment Catchment Area C-Value	Segment A x C	Accumulative A x C	Accumulative 10-Year Storm Flow	Pipe						
Segment Storm Trib ID	Street Name	US Node	DS Node	(Ha)	(Ha)	(minutes)	(mm/hr)		(Ha)	(Ha)	(L/s)	Length L (m)	Size D (mm)	Slope S (%)	Shape	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	% Full Q(d)/Qcap
	External Downstream Storm Sewers																	
	Argyle Rd	MH1	MH2	1.29	1.29	15.000	99	0.60	0.77	0.77	213	116.00	375	1.60	circle	221.78	2.01	96.21%
	Argyle Rd	MH2	MH3	0.95	2.24	15.963	96	0.60	0.57	1.34	357	118.00	450	1.60	circle	360.63	2.27	98.97%
	Argyle Rd	MH3	EX MH	0.72	2.96	16.830	92	0.60	0.43	1.78	457	96.00	525	1.60	circle	543.99	2.51	83.95%
	Argyle Rd	EX MH	Culvert	-	-	17.467	90	0.60	0.00	1.78	446	41.40	450	1.69	circle	370.64	2.33	120.43%

Flow Calculation Criteria

Q=2.78CiA

Mississauga 10-Year Storm IDF data:

$I10 = 1010.00 / (4.60 + t)^{0.78}$

$n = 0.013$

Note: Tributary Area and C-value as given in City of Mississauga Drawing: Argyle Rd. - Dunbar Rd. Storm Drainage Areas, May 1991

POST-DEVELOPMENT STORM SEWER DESIGN SHEET - Existing Argyle Road Storm Sewer

Site location: Argyle Road, Mississauga

Ref# PN 18201

Location				Segment Tributary Area	Accumulative Tributary Area	Time of Concentration	10-year Rainfall Intensity	Segment Catchment Area C-Value	Segment A x C	Accumulative A x C	Accumulative 10-Year Storm Flow	Pipe						
Segment Storm Trib ID	Street Name	US Node	DS Node	(Ha)	(Ha)	(minutes)	(mm/hr)		(Ha)	(Ha)	(L/s)	Length L (m)	Size D (mm)	Slope S (%)	Shape	Full Flow Capacity Qcap (L/s)	Full Flow Velocity V (m/s)	% Full Q(d)/Qcap
	External Downstream Storm Sewers																	
	Argyle Rd	MH1	MH2	1.29	1.29	15.000	99	0.60	0.77	0.77	213	116.00	375	1.60	circle	221.78	2.01	96.21%
	Argyle Rd	MH2	MH3	0.95	2.24	15.963	96	0.60	0.57	1.34	357	118.00	450	1.60	circle	360.63	2.27	98.97%
Subject Site Trib	Argyle Rd	MH3	EX MH	0.55	0.55	16.830	60	0.50	0.28	0.28	46							
Argyle Rd Trib				0.42	2.66	16.830	92	0.60	0.25	1.60	456	96.00	525	1.60	circle	543.99	2.51	83.86%
	Argyle Rd	EX MH	Culvert	-	-	17.467	90	0.60	0.00	1.60	447	41.40	450	1.69	circle	370.64	2.33	120.58%

Flow Calculation Criteria

Q=2.78CiA

Mississauga 10-Year Storm IDF data:

$I10 = 1010.00 / (4.60 + t)^{0.78}$

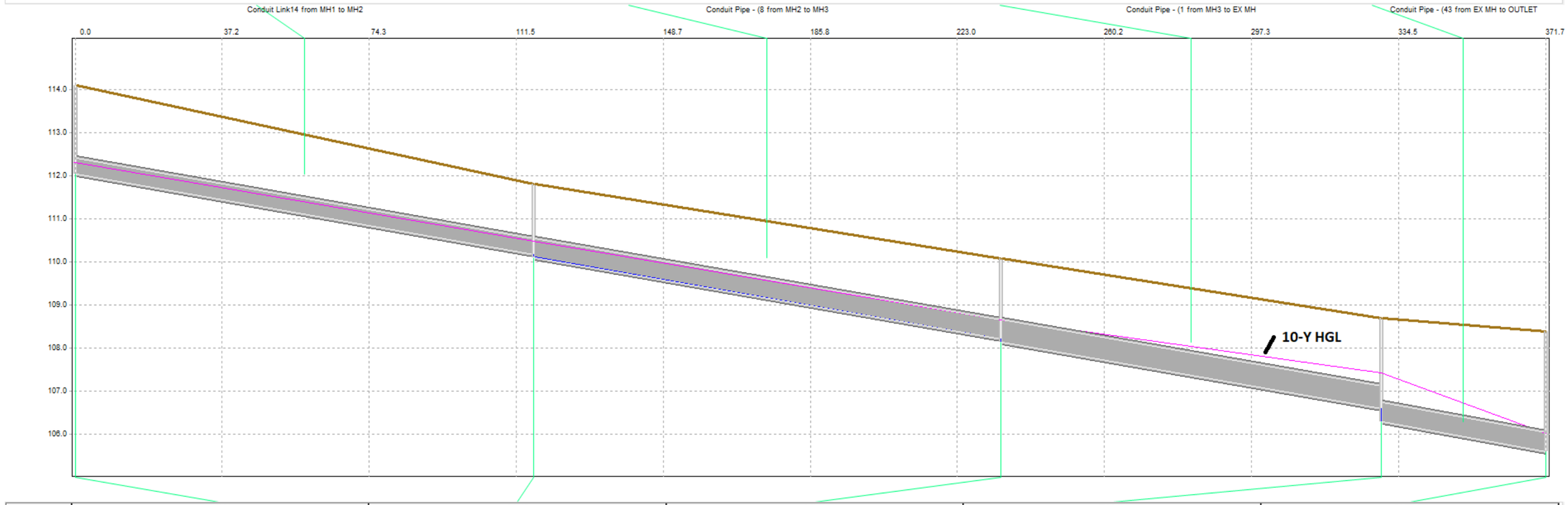
$n = 0.013$

Note: Tributary Area and C-value as given in City of Mississauga Drawing: Argyle Rd. - Dunbar Rd. Storm Drainage Areas, May 1991

Figure 2 - XPSWMM HGL Plot showing 10-Year Post-Development HGL

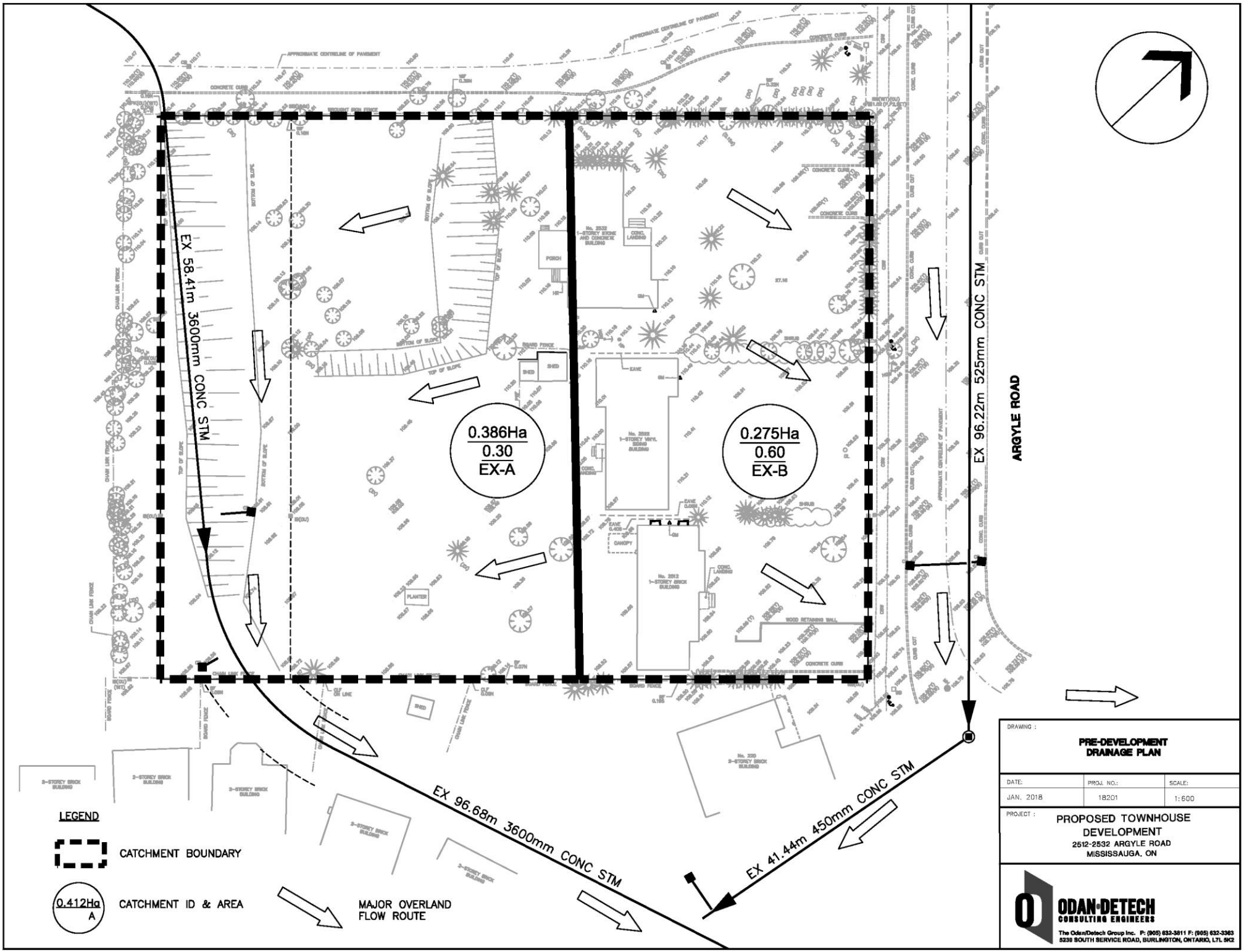
PIPES

	Link14	Pipe - (8	Pipe - (1	Pipe - (43
Storm	MISSISSAUGA10Y	MISSISSAUGA10Y	MISSISSAUGA10Y	MISSISSAUGA10Y
Max Flow	0.192	0.337	0.450	0.449
Diameter (Hei	0.375	0.450	0.525	0.450



	MH1	MH2	MH3	EX MH	OUTLET
Storm	MISSISSAUGA10Y	MISSISSAUGA10Y	MISSISSAUGA10Y	MISSISSAUGA10Y	MISSISSAUGA10Y
Ground Elevati	114.100	111.810	110.090	108.700	108.400
Freeboard	1.780	1.310	1.420	1.270	2.370
Max Water Ele	112.319	110.497	108.667	107.432	106.029
Invert Elevatio	112.050	110.110	108.139	106.300	105.600

MH



DRAWING :

PRE-DEVELOPMENT DRAINAGE PLAN

DATE:	PROJ. NO.:	SCALE:
JAN. 2018	18201	1:600

PROJECT :

PROPOSED TOWNHOUSE DEVELOPMENT
2512-2532 ARGYLE ROAD
MISSISSAUGA, ON

ODAN•DETECH
CONSULTING ENGINEERS
The Odan/Detech Group Inc. P: (905) 832-3811 F: (905) 832-3383
5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

iv) Post Development Flow Analysis

The proposed development will control the post development flows to the allowable flow rate calculated above. On-site stormwater storage will be required for the portion of the site draining to the Argyle Road 525mm culvert (Catchment Areas 'C', 'D' and 'E'). The site grading has been designed such that the portion of the site draining to the Mary Fix Creek 3600mm x 2400mm culvert produces no more runoff than in pre-development conditions, therefore no attenuation or storage is proposed for this catchment (Catchment Areas 'A' and 'B').

Refer to the Post-Development Catchment Plan on the following page for the post-development catchment areas.

The adjacent developments and the adjacent Argyle Road have self-contained storm drainage and do not drain into the subject site, with the exception of major system flood/spill water that might enter and pass through the western portion of the site in Scenario 1, above. Runoff from adjacent developments is therefore not considered in the development's stormwater calculations.

Visual OTTHYMO 2.3.2. will be used to model and determine the detention volume required. For drainage areas with significant imperviousness the calculation of effective rainfall in Visual OTTHYMO is accomplished using the "Standhyd" method. This method is used in urban watersheds to simulate runoff by combining two parallel standard unit hydrographs resulting from the effective rainfall intensity over the pervious and impervious surfaces. For pervious surfaces, losses are calculated using the SCS modified CN method.

The following parameters were used in Visual OTTHYMO to characterize the post development catchment areas.

TABLE 5 - Catchment Characteristics for the Post-Developed Site

Area I.D.	Area (ha)	Hydrograph Method	% impervious	imperviousness directly connected %	Loss Method for Pervious Area	CN for Pervious Area	Initial Abstraction for Pervious (mm)	Time to peak (T_p)
A – Impervious (to culvert)	0.079	StandHyd	90	90	SCS	80	1	-
B – Pervious (to culvert)	0.051	NashHyd	-	-	SCS	80	5	0.11
C - Roofs	0.287	StandHyd	99	99	SCS	80	1	-
D - Impervious	0.077	StandHyd	90	90	SCS	80	1	-
E - Pervious	0.167	NashHyd	-	-	SCS	80	5	0.11

The Visual OTTHYMO Model showing flows in 2-year and 10-year storms is as follows. Refer to the Visual OTTHYMO output in Appendix B for further details.

Figure 3 - Post-Development Visual OTTHYMO Model (2-Year Storm Flows)

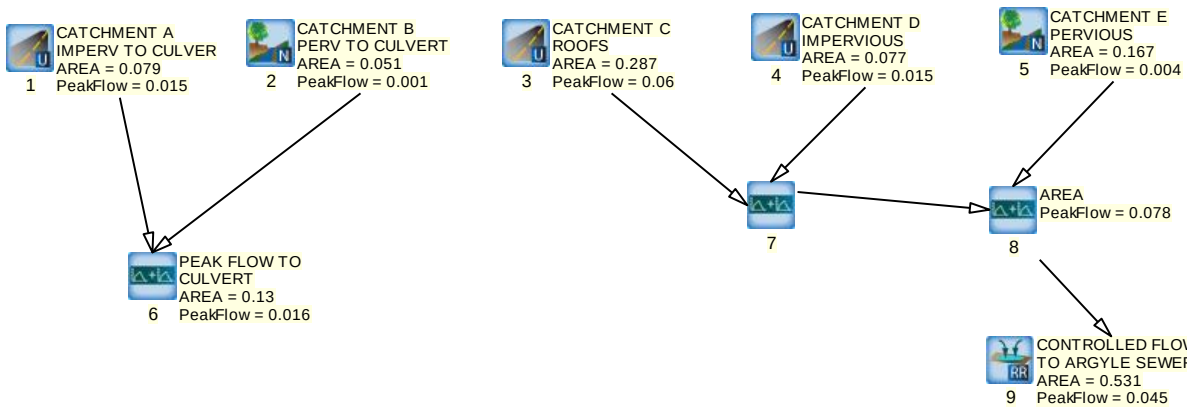
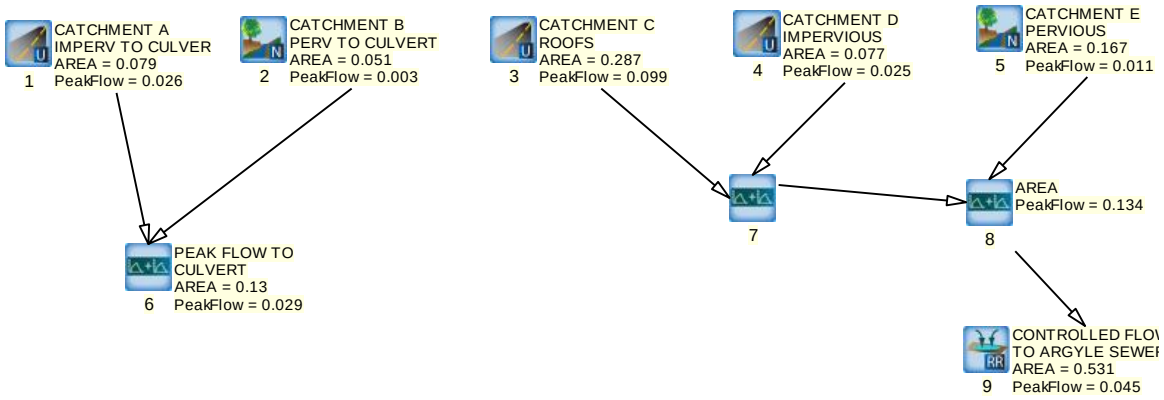


Figure 4 - Post-Development Visual OTTHYMO Model (10-Year Storm Flows)



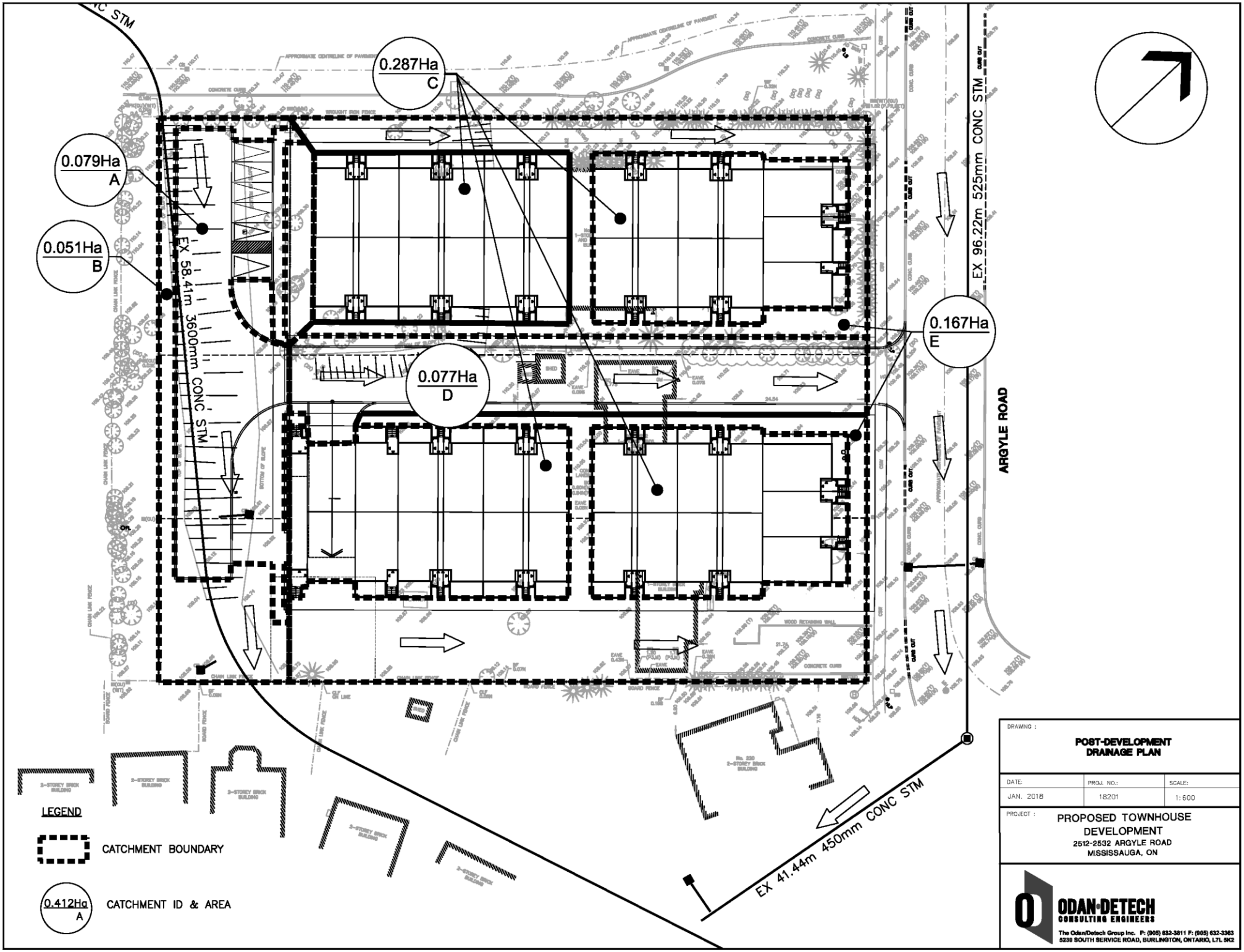
The discharge criteria is thus satisfied as follows.

TABLE 6 - Summary of Stormwater Control & Storage Scenarios

Discharge Outlet	Storm	Allowable Release Rate (L/s) (Table 4)	Proposed Release Rate (L/s)	Stormwater Storage Volume
Mary Fix Creek 3600mm x 2400mm culvert	2-Year	19.3	16	N/A
	100-Year	45.3	43	N/A
Argyle Road 525mm storm sewer	10-Year	45.5	45	53 m^3

As shown above, no stormwater attenuation or storage is required to meet discharge criteria by overland flow into the Mary Fix Creek culvert – Catchment Areas ‘A’ and ‘B’.

Stormwater falling on Catchment Areas ‘C’, ‘D’ and ‘E’ will be controlled to the allowable release rate and subsequently 53m^3 of storage will be required. A stormwater storage tank will be provided accordingly as shown on the Functional Servicing Plan.



DRAWING :

POST-DEVELOPMENT DRAINAGE PLAN

DATE:	PROJ. NO.:	SCALE:
JAN. 2018	18201	1:600

PROJECT : PROPOSED TOWNHOUSE DEVELOPMENT
2512-2532 ARGYLE ROAD
MISSISSAUGA, ON

ODAN·DETECH
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5230 SOUTH SERVICE ROAD, BURLINGTON, ONTARIO, L7L 5K2

v) Water Balance

City staff have stated that the criteria for this site is to retain 5mm rainfall events on the site.

A strategy will be developed at the SPA stage whereby 5mm rainfall events will be retained on site. The site will comprise landscaped areas which will require irrigation – a potential source of water reuse. Greywater reuse is another feasible mechanism for stormwater reuse, to be developed in the future at the SPA stage.

vi) Water Quality

City staff have stated that stormwater quality may be addressed by development charges.

6.0 CONCLUSIONS

From the foregoing investigation, the site is serviceable utilizing existing sanitary, storm and watermain infrastructure within and adjacent to the site. Storm water management can be accommodated with on-site storage as described in this report.

The following table summarizes the SWM and Servicing components of the proposed development.

TABLE 7 - Summary

	Proposed Development
Peak Sanitary Discharge (L/s)	4.72
Proposed Sanitary Service	150mm @ 2.0%
Receiving Sanitary Sewer	250mm sanitary sewer – Argyle Road
Development Water Demand (Fire + Domestic)	5067 USGM
Proposed Fire Service	150mm
Proposed Domestic Service	Branch 100mm
Allowable release rate from site (L/s)	TBC – table 4
Stormwater Quality	Not applicable
Quantity Control	Orifice pipe (to be designed at SPA)

7.0 REFERENCES

1. Region of Peel **"Public Works Design Criteria Manual – Sanitary Sewer"**, 2009.

2. Region of Peel **"Public Works Design Criteria Manual – Watermain"**, 2009.

3. Storm water Management Planning and Design Manual, Ontario Ministry of the Environment, March 2003.

4. New Jersey Storm Water Best Management Practices Manual, April 2004.

5. Visual OTTHYMO v2.0 Reference Manual, July 2002

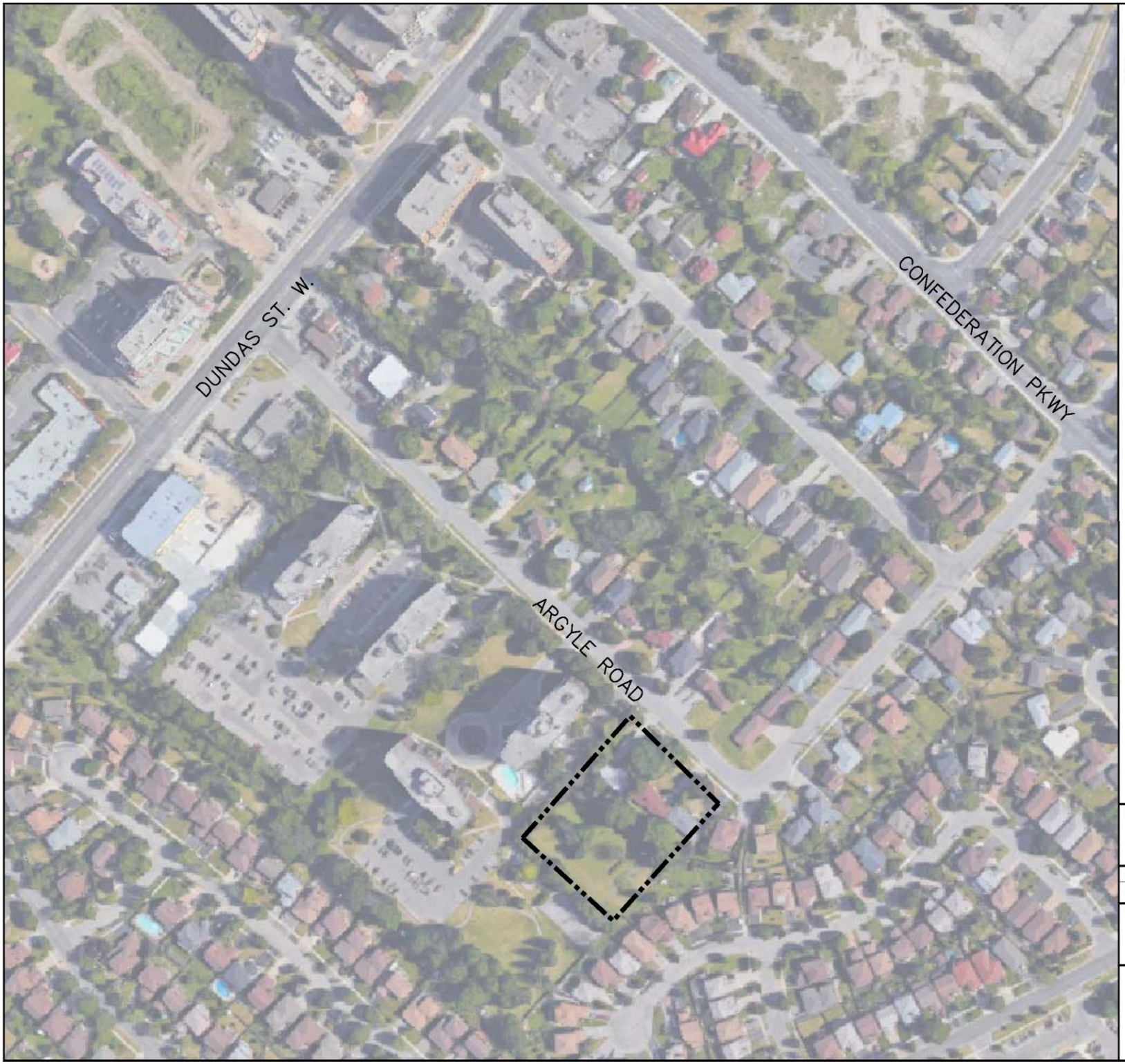
Respectfully Submitted;
The Odan Detech Group Inc.

Daniel Bancroft, P.Eng.

John Krpan, MSCE, P.Eng.

APPENDIX A

Existing Site	Aerial view of Site and surrounding area
Site Plan & Statistics	by architectureunfolded



LEGEND

PROPERTY LINE

DRAWING :

KEY PLAN

DATE:	PROJ. NO.:	SCALE:
JAN. 2018	18201	N.T.S.

PROJECT :

PROPOSED TOWNHOUSE DEVELOPMENT
2532 ARGYLE ROAD
MISSISSAUGA, ON

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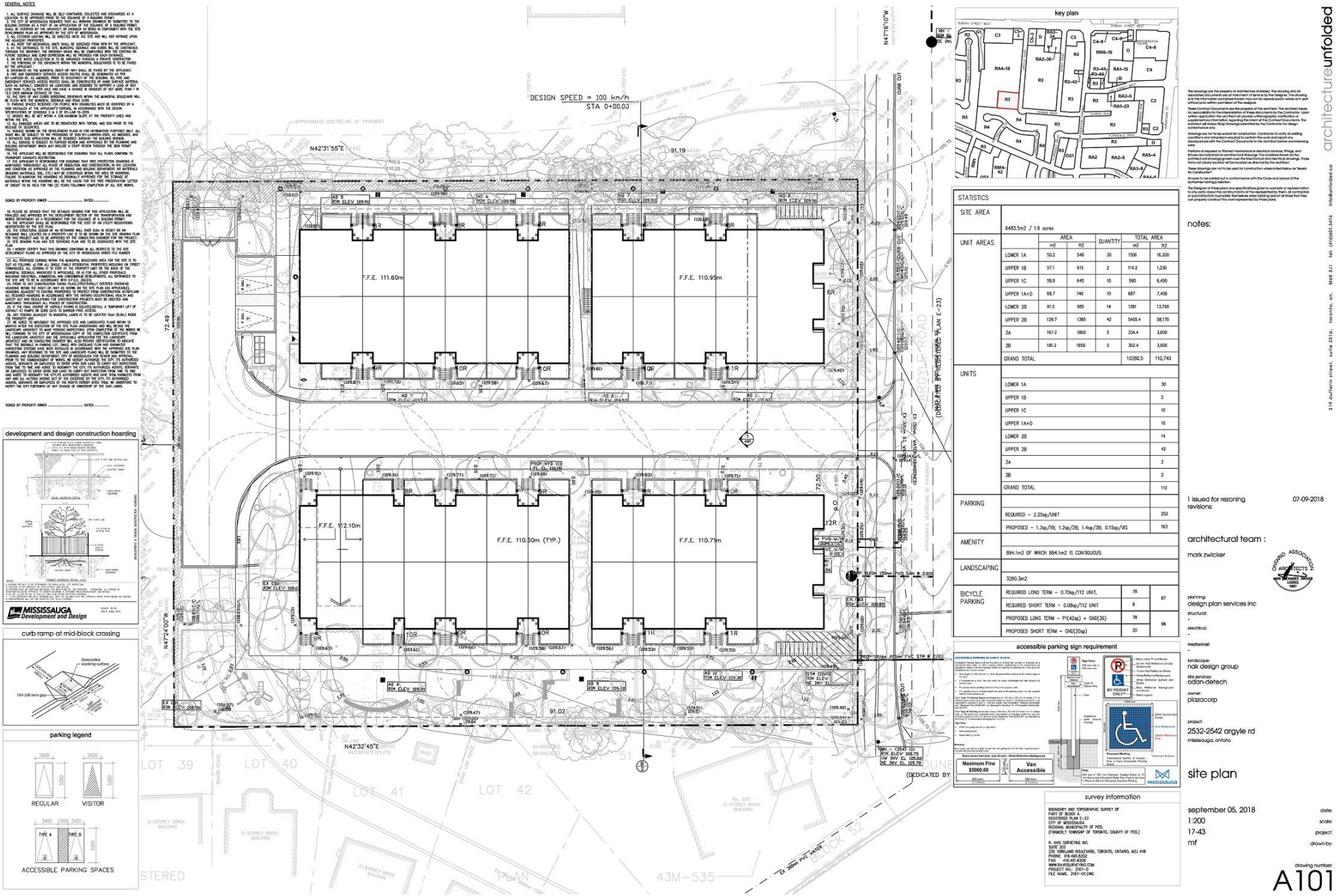
PROPOSED TOWNHOUSE DEVELOPMENT – 2532 ARGYLE ROAD



STATISTICS						
SITE AREA	6483.5m2 / 1.6 acres					
UNIT AREAS		AREA		QUANTITY	TOTAL AREA	
		m2	ft2		m2	ft2
	LOWER 1A	50.2	540	30	1506	16,200
	UPPER 1B	57.1	615	2	114.2	1,230
	UPPER 1C	59.9	645	10	590	6,450
	UPPER 1A+D	68.7	740	10	687	7,400
	LOWER 2B	91.5	985	14	1281	13,790
	UPPER 2B	128.7	1385	42	5405.4	58,170
	3A	167.2	1800	2	334.4	3,600
	3B	181.2	1950	2	362.4	3,900
	GRAND TOTAL				10280.5	110,740
UNITS						
	LOWER 1A					30
	UPPER 1B					2
	UPPER 1C					10
	UPPER 1A+D					10
	LOWER 2B					14
	UPPER 2B					42
	3A					2
	3B					2
	GRAND TOTAL					112
PARKING						
	REQUIRED – 2.25sp/UNIT					252
	PROPOSED – 1.3sp/1B; 1.2sp/2B; 1.4sp/3B; 0.15sp/MS					163
AMENITY	894.1m2 OF WHICH 894.1m2 IS CONTIGUOUS					
LANDSCAPING	3280.3m2					
BICYCLE PARKING	REQUIRED LONG TERM – 0.70sp/112 UNIT,			78		87
	REQUIRED SHORT TERM – 0.08sp/112 UNIT			9		
	PROPOSED LONG TERM – PI(42sp) + GND(36)			78		98
	PROPOSED SHORT TERM – GND(20sp)			20		

1 issued for rezoning 07-09-2018
revisions:

architectural team :
mark zwicker

planning:
design plan services inc
structural:
-
electrical:
-
mechanical:
-
landscape:
nak design group
site services:
odan-detech
owner:
plazacorp
project:
2532-2542 argyle rd
mississauga, ontario

site plan

september 05, 2018 — date:
1:200 — scale:
17-43 — project:
mf — drawn by:

drawing number:
A101

architecture unfolded
219 dufferin street, suite 201b, toronto, on, M6K 3J1 tel: (416)901.6416 info@unfolded.ca

APPENDIX B

Argyle Road Storm Sewer design sheet
Visual OTTHYMO Model Output (2-Year, 10-Year & 100-Year storms)

SUBDIVISION ARGYLE RD & DUNBAR RD
CONSULTANT
MAJOR DRAINAGE AREA

CITY OF MISSISSAUGA

STORM DRAINAGE DESIGN CHART FOR CIRCULAR DRAINS FLOWING FULL

SHEET No. 1 OF 1 DATE APR 1991
PROJECT No. 90-124
DESIGNED BY J.C.M.

LOCATION OF SECTION	FROM UPSTREAM	TO DOWNSTREAM	ADJACENT CONTRIBUTARY AREA	RUNOFF COEFFICIENT		ACCUMULATIVE AREA DRAINED BY SECTION	ACCUMULATIVE AREA TIMES RUNOFF COEFFICIENT FOR SECTION	FLOW TIME TO SECTION (FROM EXTREME UPSTREAM INLET)	INITIAL TIME OF CONCENTRATION AT EXTREME UPSTREAM INLET	TIME OF CONCENTRATION AT UPSTEAM END OF SECTION	INTENSITY OF RAINFALL	QUANTITY OF FLOW TO BE ACCOMMODATED IN SECTION	TYPE OF PIPE	MANNINGS ROUGHNESS COEFFICIENT	SLOPE	DIAMETER	LENGTH OF SECTION	VELOCITY OF FLOW WITH PIPE FLOWING FULL	CAPACITY OF PIPE FLOWING FULL	PIPE INVERT AT UPSTREAM M.H.	PIPE INVERT AT DOWNSTREAM MH	TIME OF FLOW IN SECTION
	MH#	MH#	A_A	C_A	$A_A \times C_A$	$A = \Sigma A_A$	$A \times C = \Sigma A_A \times C_A$	t_{c_f}	t_{c_1}	$t_c = t_{c_1} + t_{c_f}$	I	$Q = \frac{1}{360} AIC$		n	S	D	L	V	Q			$t = \frac{L}{V \times 60}$
			(ha)			(ha)		(min)	(min)	min	mm/hr	m^3/sec			%	mm	m	m/sec	m^3/sec	m	m	min
ARGYLE RD	1	2	1.29	0.60	0.77	1.29	0.77	–	15	15	99	.213	CONC.	.013	1.60	375	116	2.03	.231	112.05	110.19	0.95
"	2	3	0.95	0.60	0.57	2.24	1.34	0.95	15	15.95	96	.357	"	"	1.60	450	118	2.29	.376	110.11	108.22	0.86
"	3	EXIST.	0.72	0.60	0.43	2.96	1.77	0.86	15.95	16.81	92	.452	"	"	1.60	525	96	2.54	.568	108.14	106.60	0.63
DUNBAR RD	4	5	0.69	0.40	0.28	0.69	0.28	–	15	15	99	.077	CONC.	.013	0.5	375	55	1.13	.129	106.83	106.55	0.81
"	5	EXIST.	0.69	0.40	0.28	1.38	0.56	0.81	15	15.81	96	.149	"	"	0.5	450	90	1.28	.210	106.47	106.02	1.17
"	6	EXIST.	0.54	0.40	0.22	0.54	0.22	–	15	15	99	.061	CONC.	.013	0.5	300	75	0.98	.071	106.76	106.38	1.28
"	7	EXIST.	1.15	0.40	0.46	1.15	0.46	–	15	15	99	.127	CONC.	.013	0.5	375	110	1.13	.129	107.36	106.81	1.62

EFFECTIVE DATE 79-09 ... NO 2-1-5

Visual OTTHYMO Output (2-year, 10-year & 100-year storm)

```
==========================================================================================================================

       V    V   I    SSSSS  U   U    A    L
       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

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


  Input   filename: C:\Program Files (x86)\Visual OTTHYMO 2.3.3\voin.dat
  Output  filename: P:\2018\18201\Visual OTTHYMO\Rev0.1\18201 site swm\post dev.out
  Summary filename: P:\2018\18201\Visual OTTHYMO\Rev0.1\18201 site swm\post dev.sum


DATE: 8/31/2018                           TIME: 11:44:14 AM

USER:


COMMENTS: _____________________________________________________

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

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| CHICAGO STORM    |    IDF curve parameters: A= 610.000
| Ptotal= 33.44 mm |                          B=   4.600
--------------------                          C=    .780
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  4.00 hrs
                       Storm time step    = 10.00 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
                 .17    2.24 |  1.17   16.92 |  2.17    5.18 |  3.17    2.65
                 .33    2.56 |  1.33   75.36 |  2.33    4.43 |  3.33    2.47
                 .50    3.00 |  1.50   22.14 |  2.50    3.88 |  3.50    2.31
                 .67    3.67 |  1.67   11.74 |  2.67    3.46 |  3.67    2.17
                 .83    4.80 |  1.83    8.14 |  2.83    3.14 |  3.83    2.05
                1.00    7.21 |  2.00    6.30 |  3.00    2.87 |  4.00    1.95


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| CALIB            |
| NASHYD    (0002) |   Area    (ha)=    .05   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .001 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=   8.550
     TOTAL RAINFALL   (mm)=  33.439
     RUNOFF COEFFICIENT   =    .256

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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CALIB					
STANDHYD (0001)	Area (ha)= .08				
ID= 1 DT= 5.0 min	Total Imp(%)= 90.00	Dir. Conn.(%)= 90.00			

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.07	.01
Dep. Storage	(mm)=	1.00	1.00
Average Slope	(%)=	1.00	2.00
Length	(m)=	22.90	40.00
Mannings n	=	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	2.24	1.083	16.92	2.083	5.18	3.08	2.65
.167	2.24	1.167	16.92	2.167	5.18	3.17	2.65
.250	2.56	1.250	75.36	2.250	4.43	3.25	2.47
.333	2.56	1.333	75.36	2.333	4.43	3.33	2.47
.417	3.00	1.417	22.14	2.417	3.88	3.42	2.31
.500	3.00	1.500	22.14	2.500	3.88	3.50	2.31
.583	3.67	1.583	11.74	2.583	3.46	3.58	2.17
.667	3.67	1.667	11.74	2.667	3.46	3.67	2.17
.750	4.80	1.750	8.14	2.750	3.14	3.75	2.05
.833	4.80	1.833	8.14	2.833	3.14	3.83	2.05
.917	7.21	1.917	6.30	2.917	2.87	3.92	1.95
1.000	7.21	2.000	6.30	3.000	2.87	4.00	1.95

Max.Eff.Inten.(mm/hr)=		75.36	23.15	
over (min)		5.00	5.00	
Storage Coeff. (min)=		1.18 (ii)	4.46 (ii)	
Unit Hyd. Tpeak (min)=		5.00	5.00	
Unit Hyd. peak (cms)=		.33	.23	
				TOTALS
PEAK FLOW	(cms)=	.01	.00	.015 (iii)
TIME TO PEAK	(hrs)=	1.33	1.33	1.33
RUNOFF VOLUME	(mm)=	32.44	10.97	30.28
TOTAL RAINFALL	(mm)=	33.44	33.44	33.44
RUNOFF COEFFICIENT	=	.97	.33	.91

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

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 80.0 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

CALIB					
NASHYD (0005)	Area (ha)= .17		Curve Number (CN)= 80.0		
ID= 1 DT=10.0 min	Ia (mm)= 5.00		# of Linear Res.(N)= 3.00		
	U.H. Tp(hrs)= .20				

NOTE: RAINFALL WAS TRANSFORMED TO 10.0 MIN. TIME STEP.

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.167	2.24	1.167	16.92	2.167	5.18	3.17	2.65
.333	2.56	1.333	75.36	2.333	4.43	3.33	2.47
.500	3.00	1.500	22.14	2.500	3.88	3.50	2.31
.667	3.67	1.667	11.74	2.667	3.46	3.67	2.17
.833	4.80	1.833	8.14	2.833	3.14	3.83	2.05
1.000	7.21	2.000	6.30	3.000	2.87	4.00	.00

Unit Hyd Qpeak (cms)=		.032
PEAK FLOW	(cms)=	.004 (i)
TIME TO PEAK	(hrs)=	1.500
RUNOFF VOLUME	(mm)=	8.399
TOTAL RAINFALL	(mm)=	33.114
RUNOFF COEFFICIENT	=	.254

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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| CALIB            |
| STANDHYD  (0004) |   Area    (ha)=    .08
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .07          .01
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      22.70        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                          ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    2.24 | 1.083   16.92 | 2.083    5.18 |  3.08    2.65
                .167    2.24 | 1.167   16.92 | 2.167    5.18 |  3.17    2.65
                .250    2.56 | 1.250   75.36 | 2.250    4.43 |  3.25    2.47
                .333    2.56 | 1.333   75.36 | 2.333    4.43 |  3.33    2.47
                .417    3.00 | 1.417   22.14 | 2.417    3.88 |  3.42    2.31
                .500    3.00 | 1.500   22.14 | 2.500    3.88 |  3.50    2.31
                .583    3.67 | 1.583   11.74 | 2.583    3.46 |  3.58    2.17
                .667    3.67 | 1.667   11.74 | 2.667    3.46 |  3.67    2.17
                .750    4.80 | 1.750    8.14 | 2.750    3.14 |  3.75    2.05
                .833    4.80 | 1.833    8.14 | 2.833    3.14 |  3.83    2.05
                .917    7.21 | 1.917    6.30 | 2.917    2.87 |  3.92    1.95
               1.000    7.21 | 2.000    6.30 | 3.000    2.87 |  4.00    1.95

     Max.Eff.Inten.(mm/hr)=      75.36        21.70
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.18 (ii)    4.46 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .23
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .01          .00          .015 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      32.44        10.97        30.28
     TOTAL RAINFALL   (mm)=      33.44        33.44        33.44
     RUNOFF COEFFICIENT   =        .97          .33          .91

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (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)=    .29
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .28          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      43.70        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=      75.36       108.49
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.74 (ii)    3.00 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .32          .28
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .06          .00          .060 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33         1.33
     RUNOFF VOLUME    (mm)=      32.44        10.97        32.21
     TOTAL RAINFALL   (mm)=      33.44        33.44        33.44
     RUNOFF COEFFICIENT   =        .97          .33          .96

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
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```
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| ADD HYD   (0006) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0002):       .05    .001     1.50     8.55
        + ID2= 2 (0001):       .08    .015     1.33    30.28
          ================================================
          ID = 3 (0006):       .13    .016     1.33    21.66

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0004):       .08    .015     1.33    30.28
        + ID2= 2 (0003):       .29    .060     1.33    32.21
          ================================================
          ID = 3 (0007):       .36    .074     1.33    31.81

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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--------------------
| ADD HYD   (0008) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0005):       .17    .004     1.50     8.40
        + ID2= 2 (0007):       .36    .074     1.33    31.81
          ================================================
          ID = 3 (0008):       .53    .078     1.33    24.33

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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--------------------
| RESERVOIR (0009) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0451       .0055
                           .0450      .0001   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0008)       .531       .078        1.33       24.33
     OUTFLOW: ID= 1 (0009)       .531       .045        1.42       24.34

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 57.97
                  TIME SHIFT OF PEAK FLOW         (min)=  5.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0019

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

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1010.000
| Ptotal= 55.37 mm |                          B=   4.600
--------------------                          C=    .780
                       used in:   INTENSITY =  A / (t + B)^C

                       Duration of storm  =  4.00 hrs
                       Storm time step    = 10.00 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
                 .17    3.71 |  1.17   28.02 |  2.17    8.58 |  3.17    4.39
```

.33	4.23	1.33	124.77	2.33	7.33	3.33	4.08
.50	4.97	1.50	36.65	2.50	6.42	3.50	3.82
.67	6.07	1.67	19.43	2.67	5.74	3.67	3.60
.83	7.95	1.83	13.47	2.83	5.19	3.83	3.40
1.00	11.94	2.00	10.43	3.00	4.75	4.00	3.22

CALIB					
NASHYD (0002)	Area (ha)= .05	Curve Number (CN)= 80.0
ID= 1 DT=10.0 min	Ia (mm)= 5.00	# of Linear Res.(N)= 3.00
	U.H. Tp(hrs)= .20	

Unit Hyd Qpeak (cms)= .010

PEAK FLOW (cms)= .003 (i)
TIME TO PEAK (hrs)= 1.500
RUNOFF VOLUME (mm)= 21.688
TOTAL RAINFALL (mm)= 55.365
RUNOFF COEFFICIENT = .392

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

CALIB			
STANDHYD (0001)	Area (ha)= .08		
ID= 1 DT= 5.0 min	Total Imp(%)= 90.00	Dir. Conn.(%)= 90.00	

		IMPERVIOUS	PERVIOUS (i)
Surface Area	(ha)=	.07	.01
Dep. Storage	(mm)=	1.00	1.00
Average Slope	(%)=	1.00	2.00
Length	(m)=	22.90	40.00
Mannings n	=	.013	.250

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

---- TRANSFORMED HYETOGRAPH ----

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.083	3.71	1.083	28.02	2.083	8.58	3.08	4.39
.167	3.71	1.167	28.02	2.167	8.58	3.17	4.39
.250	4.23	1.250	124.77	2.250	7.33	3.25	4.08
.333	4.23	1.333	124.77	2.333	7.33	3.33	4.08
.417	4.97	1.417	36.65	2.417	6.42	3.42	3.82
.500	4.97	1.500	36.65	2.500	6.42	3.50	3.82
.583	6.07	1.583	19.43	2.583	5.74	3.58	3.60
.667	6.07	1.667	19.43	2.667	5.74	3.67	3.60
.750	7.95	1.750	13.47	2.750	5.19	3.75	3.40
.833	7.95	1.833	13.47	2.833	5.19	3.83	3.40
.917	11.94	1.917	10.43	2.917	4.75	3.92	3.22
1.000	11.94	2.000	10.43	3.000	4.75	4.00	3.22

Max.Eff.Inten.(mm/hr)=	124.77	52.44		
over (min)	5.00	5.00		
Storage Coeff. (min)=	.97 (ii)	3.65 (ii)		
Unit Hyd. Tpeak (min)=	5.00	5.00		
Unit Hyd. peak (cms)=	.34	.25		
			TOTALS	
PEAK FLOW (cms)=	.02	.00	.026 (iii)	
TIME TO PEAK (hrs)=	1.33	1.33	1.33	
RUNOFF VOLUME (mm)=	54.37	25.08	51.43	
TOTAL RAINFALL (mm)=	55.37	55.37	55.37	
RUNOFF COEFFICIENT =	.98	.45	.93	

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

(i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
CN* = 80.0 Ia = Dep. Storage (Above)
(ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
THAN THE STORAGE COEFFICIENT.
(iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

CALIB					
NASHYD (0005)	Area (ha)= .17	Curve Number (CN)= 80.0

```
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .20

         NOTE:  RAINFALL WAS TRANSFORMED TO  10.0 MIN. TIME STEP.


                              ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .167    3.71 | 1.167   28.02 | 2.167    8.58 |  3.17    4.39
                .333    4.23 | 1.333  124.77 | 2.333    7.33 |  3.33    4.08
                .500    4.97 | 1.500   36.65 | 2.500    6.42 |  3.50    3.82
                .667    6.07 | 1.667   19.43 | 2.667    5.74 |  3.67    3.60
                .833    7.95 | 1.833   13.47 | 2.833    5.19 |  3.83    3.40
               1.000   11.94 | 2.000   10.43 | 3.000    4.75 |  4.00     .00

     Unit Hyd Qpeak  (cms)=    .032

     PEAK FLOW       (cms)=    .011 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  21.336
     TOTAL RAINFALL   (mm)=  54.828
     RUNOFF COEFFICIENT   =    .389

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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-------------------
| CALIB           |
| STANDHYD  (0004) |   Area    (ha)=    .08
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
-------------------
                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       .07          .01
     Dep. Storage     (mm)=      1.00         1.00
     Average Slope     (%)=      1.00         2.00
     Length            (m)=     22.70        40.00
     Mannings n           =      .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                              ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    3.71 | 1.083   28.02 | 2.083    8.58 |  3.08    4.39
                .167    3.71 | 1.167   28.02 | 2.167    8.58 |  3.17    4.39
                .250    4.23 | 1.250  124.77 | 2.250    7.33 |  3.25    4.08
                .333    4.23 | 1.333  124.77 | 2.333    7.33 |  3.33    4.08
                .417    4.97 | 1.417   36.65 | 2.417    6.42 |  3.42    3.82
                .500    4.97 | 1.500   36.65 | 2.500    6.42 |  3.50    3.82
                .583    6.07 | 1.583   19.43 | 2.583    5.74 |  3.58    3.60
                .667    6.07 | 1.667   19.43 | 2.667    5.74 |  3.67    3.60
                .750    7.95 | 1.750   13.47 | 2.750    5.19 |  3.75    3.40
                .833    7.95 | 1.833   13.47 | 2.833    5.19 |  3.83    3.40
                .917   11.94 | 1.917   10.43 | 2.917    4.75 |  3.92    3.22
               1.000   11.94 | 2.000   10.43 | 3.000    4.75 |  4.00    3.22

     Max.Eff.Inten.(mm/hr)=     124.77        52.44
               over (min)         5.00         5.00
     Storage Coeff.  (min)=        .96 (ii)     3.64 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .25
                                                      *TOTALS*
     PEAK FLOW       (cms)=        .02          .00          .025 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      54.37        25.08         51.43
     TOTAL RAINFALL   (mm)=      55.37        55.37         55.37
     RUNOFF COEFFICIENT   =        .98          .45           .93

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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| CALIB            |
| STANDHYD  (0003) |   Area    (ha)=    .29
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .28          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      43.70        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     124.77       262.21
               over (min)         5.00         5.00
     Storage Coeff.  (min)=       1.42 (ii)    2.45 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .30
                                                        *TOTALS*
     PEAK FLOW       (cms)=        .10          .00           .099 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      54.37        25.08         54.06
     TOTAL RAINFALL   (mm)=      55.37        55.37         55.37
     RUNOFF COEFFICIENT   =        .98          .45           .98

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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| ADD HYD   (0006) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0002):      .05    .003     1.50    21.69
        + ID2= 2 (0001):      .08    .026     1.33    51.43
          ================================================
          ID = 3 (0006):      .13    .029     1.33    39.60

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0004):      .08    .025     1.33    51.43
        + ID2= 2 (0003):      .29    .099     1.33    54.06
          ================================================
          ID = 3 (0007):      .36    .124     1.33    53.51

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0008) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0005):      .17    .011     1.50    21.34
        + ID2= 2 (0007):      .36    .124     1.33    53.51
          ================================================
          ID = 3 (0008):      .53    .134     1.33    43.20

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0009) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          .0000      .0000    |    .0451       .0055
                          .0450      .0001    |    .0000       .0000

                                AREA      QPEAK       TPEAK       R.V.
```

```
                                (ha)       (cms)        (hrs)        (mm)
     INFLOW : ID= 2 (0008)       .531        .134         1.33       43.20
     OUTFLOW: ID= 1 (0009)       .531        .045         1.50       43.21

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 33.74
                   TIME SHIFT OF PEAK FLOW         (min)= 10.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=   .0053

--------------------------------------------------------------------------------
  ****************************
  ** SIMULATION NUMBER:   3 **
  ****************************

--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1450.000
| Ptotal= 79.41 mm |                          B=   4.900
--------------------                          C=    .780
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 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
                 .17    5.34 |  1.17   40.65 |  2.17   12.41 |  3.17    6.33
                 .33    6.10 |  1.33  176.31 |  2.33   10.59 |  3.33    5.89
                 .50    7.17 |  1.50   53.15 |  2.50    9.28 |  3.50    5.51
                 .67    8.77 |  1.67   28.20 |  2.67    8.28 |  3.67    5.18
                 .83   11.49 |  1.83   19.53 |  2.83    7.49 |  3.83    4.89
                1.00   17.30 |  2.00   15.10 |  3.00    6.86 |  4.00    4.64


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0002) |   Area     (ha)=    .05   Curve Number    (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia       (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

     Unit Hyd Qpeak  (cms)=    .010

     PEAK FLOW       (cms)=    .006 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  39.085
     TOTAL RAINFALL   (mm)=  79.409
     RUNOFF COEFFICIENT   =    .492

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0001) |   Area     (ha)=    .08
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------

                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       .07          .01
     Dep. Storage     (mm)=      1.00         1.00
     Average Slope     (%)=      1.00         2.00
     Length            (m)=     22.90        40.00
     Mannings n           =      .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                          ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    5.34 | 1.083   40.65 | 2.083   12.41 |  3.08    6.33
                .167    5.34 | 1.167   40.65 | 2.167   12.41 |  3.17    6.33
                .250    6.10 | 1.250  176.31 | 2.250   10.59 |  3.25    5.89
                .333    6.10 | 1.333  176.31 | 2.333   10.59 |  3.33    5.89
                .417    7.17 | 1.417   53.15 | 2.417    9.28 |  3.42    5.51
                .500    7.17 | 1.500   53.15 | 2.500    9.28 |  3.50    5.51
                .583    8.77 | 1.583   28.20 | 2.583    8.28 |  3.58    5.18
                .667    8.77 | 1.667   28.20 | 2.667    8.28 |  3.67    5.18
                .750   11.49 | 1.750   19.53 | 2.750    7.49 |  3.75    4.89
                .833   11.49 | 1.833   19.53 | 2.833    7.49 |  3.83    4.89
                .917   17.30 | 1.917   15.10 | 2.917    6.86 |  3.92    4.64
```

```
               1.000   17.30 | 2.000   15.10 | 3.000    6.86 |  4.00    4.64

     Max.Eff.Inten.(mm/hr)=     176.31        92.61
               over (min)         5.00         5.00
     Storage Coeff.  (min)=        .84 (ii)    3.18 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .27
                                                      *TOTALS*
     PEAK FLOW       (cms)=        .03          .00          .037 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      78.41        43.32         74.89
     TOTAL RAINFALL   (mm)=      79.41        79.41         79.41
     RUNOFF COEFFICIENT   =        .99          .55           .94

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0005) |   Area    (ha)=    .17   Curve Number   (CN)= 80.0
|ID= 1 DT=10.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=    .20

         NOTE:  RAINFALL WAS TRANSFORMED TO  10.0 MIN. TIME STEP.


                          ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .167    5.34 | 1.167   40.65 | 2.167   12.41 |  3.17    6.33
                .333    6.10 | 1.333  176.31 | 2.333   10.59 |  3.33    5.89
                .500    7.17 | 1.500   53.15 | 2.500    9.28 |  3.50    5.51
                .667    8.77 | 1.667   28.20 | 2.667    8.28 |  3.67    5.18
                .833   11.49 | 1.833   19.53 | 2.833    7.49 |  3.83    4.89
               1.000   17.30 | 2.000   15.10 | 3.000    6.86 |  4.00     .00

     Unit Hyd Qpeak  (cms)=    .032

     PEAK FLOW       (cms)=    .021 (i)
     TIME TO PEAK    (hrs)=   1.500
     RUNOFF VOLUME    (mm)=  38.507
     TOTAL RAINFALL   (mm)=  78.635
     RUNOFF COEFFICIENT   =    .490

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0004) |   Area    (ha)=    .08
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .07          .01
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      22.70        40.00
     Mannings n           =       .013         .250

         NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.


                          ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                .083    5.34 | 1.083   40.65 | 2.083   12.41 |  3.08    6.33
                .167    5.34 | 1.167   40.65 | 2.167   12.41 |  3.17    6.33
                .250    6.10 | 1.250  176.31 | 2.250   10.59 |  3.25    5.89
                .333    6.10 | 1.333  176.31 | 2.333   10.59 |  3.33    5.89
                .417    7.17 | 1.417   53.15 | 2.417    9.28 |  3.42    5.51
                .500    7.17 | 1.500   53.15 | 2.500    9.28 |  3.50    5.51
                .583    8.77 | 1.583   28.20 | 2.583    8.28 |  3.58    5.18
                .667    8.77 | 1.667   28.20 | 2.667    8.28 |  3.67    5.18
                .750   11.49 | 1.750   19.53 | 2.750    7.49 |  3.75    4.89
```

```
                .833   11.49 | 1.833   19.53 | 2.833    7.49 |  3.83    4.89
                .917   17.30 | 1.917   15.10 | 2.917    6.86 |  3.92    4.64
               1.000   17.30 | 2.000   15.10 | 3.000    6.86 |  4.00    4.64

     Max.Eff.Inten.(mm/hr)=     176.31        92.61
               over (min)        5.00         5.00
     Storage Coeff.  (min)=        .84 (ii)    3.17 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .34          .27
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .03          .00          .036 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      78.41        43.32         74.89
     TOTAL RAINFALL   (mm)=      79.41        79.41         79.41
     RUNOFF COEFFICIENT   =        .99          .55           .94

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (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)=    .29
|ID= 1 DT= 5.0 min |   Total Imp(%)=  99.00   Dir. Conn.(%)=  99.00
--------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=        .28          .00
     Dep. Storage     (mm)=       1.00         1.00
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      43.70        40.00
     Mannings n           =       .013         .250

     Max.Eff.Inten.(mm/hr)=     176.31       463.07
               over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.24 (ii)    2.13 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=        .33          .31
                                                          *TOTALS*
     PEAK FLOW       (cms)=        .14          .00          .140 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      78.41        43.32         78.05
     TOTAL RAINFALL   (mm)=      79.41        79.41         79.41
     RUNOFF COEFFICIENT   =        .99          .55           .98

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  80.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD   (0006) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0002):      .05    .006     1.50    39.09
        + ID2= 2 (0001):      .08    .037     1.33    74.89
          ================================================
          ID = 3 (0006):      .13    .043     1.33    60.60

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0007) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0004):      .08    .036     1.33    74.89
        + ID2= 2 (0003):      .29    .140     1.33    78.05
          ================================================
          ID = 3 (0007):      .36    .176     1.33    77.38
```

```
     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

--------------------
| ADD HYD   (0008) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0005):      .17    .021     1.50    38.51
        + ID2= 2 (0007):      .36    .176     1.33    77.38
          ================================================
          ID = 3 (0008):      .53    .195     1.33    64.89

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0009) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           .0000      .0000   |    .0451       .0055
                           .0450      .0001   |    .0000       .0000

                                 AREA      QPEAK       TPEAK       R.V.
                                 (ha)      (cms)       (hrs)       (mm)
     INFLOW : ID= 2 (0008)       .531       .195        1.33       64.89
     OUTFLOW: ID= 1 (0009)       .531       .045        1.58       64.89

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 23.23
                  TIME SHIFT OF PEAK FLOW         (min)= 15.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=   .0105

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 FINISH
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```

APPENDIX C

Functional Servicing Plan
Functional Grading Plan