C7. Stormwater Management Report

Note: The Stormwater Management assessment for the study area is documented in two reports.

The August 2009 Report pertains to the original study limits which extended from Queen Street northerly to Mayfield Road.

The June 2011 Report pertains to the extended study area from Mayfield Road northerly approximately 2 km.

AECOM

Region of Peel

Dixie Road Class Environmental Assessment Queen Street to Mayfield Road

Stormwater Management Report

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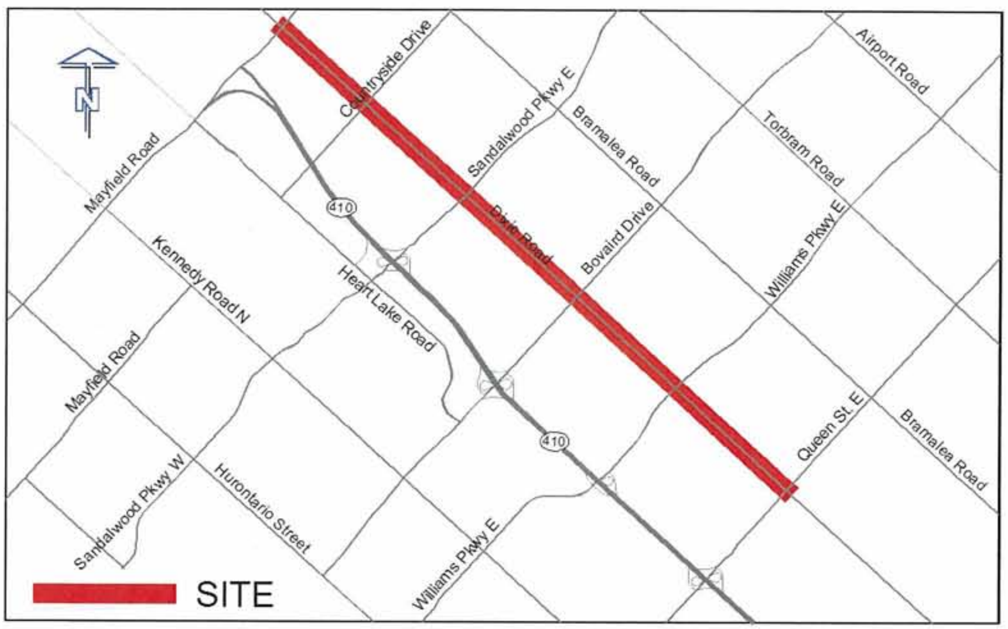
1. Introduction

AECOM was retained by the Region of Peel to complete an Environmental Assessment for improvements to Dixie Road. This Stormwater Management Report has been prepared in support of the Environmental Assessment. It examines existing drainage conditions, evaluates the impact of the various alternatives on stormwater quality, quantity and flooding, and recommends measures to mitigate any impacts associated with the preferred road design alternative.

The study area is illustrated in Figure 1.1. It includes Dixie Rd. from the north limit of Mayfield Road to the south limit of Queen Street East. The study area is in the City of Brampton and falls entirely within the jurisdiction of Toronto Region Conservation Authority, with the site draining to tributaries of Etobicoke Creek.

Figure 1. Study Area



2. Existing Conditions

2.1 Land Use

Along Dixie Road from Queen Street East to just south of Countryside Drive the land use is predominantly residential with some retail plazas, gas stations and small commercial lots. From just south of Countryside to Mayfield Road the land is mainly used for agricultural purposes. One key location along Dixie Road is the stormwater management pond on the west side of Dixie Road between Bovaird Drive and Peter Robertson Blvd. A portion of Dixie Road drains to that pond. The Brampton Soccer Centre is located on the southwest corner of Dixie Road and Sandalwood Parkway. This is a large building with a sizeable parking lot. There is also a very small water park on site for children.

The residential developments near the south end of the site were constructed during the late 1960's and 1970's. The developments farther north, near North Park Drive were constructed in the 1980's. The residential developments south of Bovaird are within the community of "Bramalea". And the developments north of Bovaird Drive, within the community of "Springdale" were constructed in the late 1990's and into the new millennium.

2.2 Soils

Detailed soils information was not obtained, but agricultural soils mapping indicates that the study area is a combination of Chinguacousy clay loam, with Oneida clay loam, and a small amount of Caledon loam (Hoffman & Richards, 1953). Chinguacousy clay loam makes up the majority of the site soil, and is considered to have imperfect natural drainage. According to the Ministry of Transportation's Soil Classification Chart it is within the Hydrology Soil Groups C indicating minimal infiltration. Along the watercourses the soil type was found to be Oneida clay loam, which has good natural drainage and is within the Hydrology Soil Group D. This indicates negligible infiltration along watercourse tributaries that intersect the site.

2.3 Topography

Dixie Road at Mayfield Road slopes south at an average rate of 0.7% to Countryside Drive. Ditches on either side of the road convey stormwater south. There are fairly flat fields on either side of Dixie Road. During large storm events drainage from the west side of Dixie Road may cross Dixie Road and continue east.

From Countryside Drive to Sandalwood Parkway, Dixie Road continues to slope south at approximately 0.5% to a low point at Sandalwood Parkway. There is a watercourse on the west side of Dixie north of Sandalwood that runs through a golf course. The watercourse continues to flow south along the west side of Dixie Road to the stormwater management pond at Dixie and Bovaird.

South of Sandalwood Parkway near Springtown Trail there is a local high point, sloping north towards Sandalwood Parkway at 0.4%. From Springtown Trail, Dixie Road slopes south to a local low point between Springtown Trail and Peter Robertson Blvd at 0.5%. Dixie Road slopes north from a local high point north of Peter Robertson Blvd to the local low point at a rate of 0.5%. This low point outlets into the adjacent watercourse that continues to flow south alongside Dixie Road. From the high point north of Peter Robertson Blvd, Dixie Road slopes south at a rate varying between 0.4% and 1.1% to a low point at Bovaird Drive. Stormwater is conveyed to the large stormwater management pond on the northwest corner of Bovaird Drive and Dixie Road. At the southeast corner of the pond there lies an elaborate outlet structure which is connected to a box culvert that crosses diagonally under the Bovaird Drive and Dixie Road intersection. The watercourse continues on the southeast side of the intersection and flows southeast away from Dixie Road.

There is a local high point just south of North Park Drive, which slopes north at 0.4 – 0.5% until just south of Bovaird Drive where it slopes northward at 3.0% to the low point. From the high point south of North Park Drive, Dixie Road slopes south at 3.0% which reduces to 1.8% until the low point at Williams Parkway.

Williams Parkway slopes from east to west, and drains to a small unnamed watercourse that flows southward adjacent to Williams Parkway, west of Dixie Road.

There is a high point south of Williams Parkway which slopes north at 1.0% to the low point at Williams Parkway. South of the high point, Dixie Road slopes south at varying slopes ranging from 0.4% to 2.5% to the low point at the crossing culvert just north of Queen Street.

Dixie Road has a very wide boulevard along either side of the road which slopes inward. Beyond the road right-of-way, the topography slopes away from Dixie Road within the adjacent residential subdivisions. These wide grassed boulevards will easily allow for the proposed expansion of the roadway.

2.4 Storm Drainage

At the north end of the study area, north of Countryside Drive, Dixie Road is a two-lane arterial roadway with ditch drainage and partial storm sewer drainage. South of Countryside Drive to Bovaird Drive, Dixie Road is a four-lane arterial roadway with curb and gutter cross-section and grassed boulevard. There are several long islands that divide northbound and southbound lanes, and intersections have additional turning lanes. From Bovaird Drive to Queen Street, Dixie Road remains 4-lanes with a curb and gutter cross-section and grassed boulevards. There are only small islands at intersections and some turning lanes at intersections.

The current Region of Peel standards state that the storm sewers should be based on a 10-year design storm.

Existing storm sewer data was obtained from as-built drawings and design sheets from the Region of Peel and the City of Brampton. Older portions of the existing storm sewer network were designed based on a 5-year or 2-year return period storm, where the current standards require 10-year return storm capacity. The newer storm sewer systems were based on the 10-year event. It is assumed that no foundation drains are connected to the existing sewer.

2.4.1 Mayfield Road south to Countryside Drive

At the north end of the study area there are ditches on either side of the roadway which convey runoff south. South of Mayfield Road there is a crossing culvert that conveys runoff from the west field across Dixie Road to the east field in large storm events. Approximately 850 metres south of Mayfield Road there is a crossing culvert which conveys ditch flow from the east side of the roadway to the west, and is seen in Figure 2. The ditch on the west side of Dixie Road captures runoff from the west half of the roadway and continues to convey runoff south, while a storm sewer on the east side of the right-of-way captures flow from the east half of the roadway where it is conveyed south. Approximately 240 metres south of the crossing culvert there is a ditch inlet catch basin (DICB) which captures flow from the west ditch and connects it to the storm sewer system, as seen in Figure 3. The storm sewer downstream of this connection has a diameter of 675 mm. There is also a crossing culvert at this location, however, it is assumed that this crossing culvert only functions during large storm events where runoff from the field west of the site crosses Dixie Road by means of the culvert and flow continues southeast across the east field. The inverts of the crossing culverts are quite a bit higher than the invert of the ditches which is why there would have to be a large depth of flow for the crossing culvert to come into effect. South of this culvert a small sliver of the right-of-way is captured by a ditch on the east side of the site, with the majority captured by the storm sewer, which is realigned to the centre of the roadway. Approximately 200 metres south of the crossing culvert there is a DICB which conveys flow from the east ditch to the storm sewers. The storm sewer continues to convey stormwater south with a diameter of 675 mm at Countryside Drive. See Drawing 1 for an illustration of the storm sewers.

Figure 2. 600mm Crossing Culvert

Figure 3. West ditch & DICB

The storm sewers in this section were analysed and it was determined that they were designed to capture and convey the 10-year design storm as per the Region of Peel design standards. Design Sheet information can be found in Appendix B.

2.4.2 Countryside Drive to Sandalwood Parkway

South of Countryside Drive, Dixie Road has a curb and gutter cross-section with catch basins on either side of the road that capture runoff into the storm sewer system that has continued from the north. At the intersection of Father Tobin Road there are double catch basins on the north and southeast corners to capture runoff from a portion of Father Tobin Road. Just north of Octillo Boulevard there is a large box culvert that crosses Dixie Road. The culvert conveys runoff from the residential areas east of Dixie Road to the watercourse on the west side of the road. This watercourse flows south to the stormwater management pond at Dixie Road and Bovaird Drive. The west end of the crossing culvert can be seen in Figure 4 below.

Figure 4. West end of Crossing Culvert

Figure 5. West Watercourse at Sandalwood Pkwy

Available design drawings indicate that this cross culvert was constructed some time after the storm sewers on Dixie Road were installed. The plans suggest that the 900mm diameter storm sewer on Dixie Road is intercepted into the crossing culvert. The remainder of the 900mm storm sewer continues south from the crossing culvert, eventually discharging to the west watercourse south of Sandalwood Parkway. The watercourse crossing at Sandalwood Parkway is seen in Figure 5 above. The sewer connection to the crossing culvert should be verified at the detailed design stage.

The storm sewers in this system have enough capacity to adequately convey the 10-year design storm under existing conditions. Please refer to Drawing 1 for a storm sewer diagram and Appendix B for storm sewer design sheets.

2.4.3 Sandalwood Parkway to Bovaird Drive

South of Sandalwood Parkway at Springtown Trail there is a high point in the topography and in the storm sewer system. A storm sewer runs north from Springtown Trail and outlets just south of Sandalwood into the west watercourse. South of the high point another storm sewer system conveys flow south to a low point between Springtown Trail and Peter Robertson Boulevard. Just north of Peter Robertson Boulevard is the upstream end of another storm sewer with the downstream end meeting up with the other storm sewer at the common low point. The storm sewer outlets into the west watercourse at the low point.

Another storm sewer conveys flow from Peter Robertson Boulevard, south on Dixie and outlets into the stormwater management pond on the north side of Bovaird Drive shown in Figure 6. The outlet structure of the stormwater management pond is shown in Figure 7 below.

The storm sewers in this area were analysed and it was found that most sections do not have capacity for the 10-year storm event. It appears that the storm sewers were designed for a 5-year event. Refer to Drawings 2 and 3 for storm sewer illustrations, and Appendix B for storm sewer design sheets.

Figure 6. SWM Pond

Figure 7. SWM Pond Outlet Structure

2.4.4 Bovaird Drive to Williams Parkway

The upstream end of a storm sewer system is just south of North Park Drive. This storm sewer starts at a diameter of 300mm and conveys flow north to the outlet on the southeast corner of Bovaird Drive and Dixie Road with an outlet diameter of 525mm. See Figure 8 for a view of the road cross-section at the upstream end of this storm sewer, facing north.

Farther south, there is another storm sewer which also has an upstream end just south of North Park Drive. It conveys flow south to Williams Parkway. It is believed that a storm sewer on Williams Parkway is connected to the system on Dixie Road and conveys flow west to a nearby watercourse. This should be confirmed at the detailed design stage. The Dixie Road and Williams Parkway intersection is seen in Figure 9.

Figure 8. At North Park Dr. facing North

Figure 9. At Williams Pkwy facing North-East

An analysis was completed for the storm sewers in the area and it was found that not all storm sewer sections have capacity for the 10-year design storm. It seems that the storm sewers were designed to accommodate a 5-year storm event. Please refer to Drawings 3 and 4 for storm sewer illustrations and Appendix B for storm sewer design sheets.

2.4.5 Williams Parkway to Queen Street East

There is a small storm sewer system south of Williams Parkway that conveys flow north to the connection at Williams Parkway and Dixie Road. South of that sewer are two catchbasins and a DICB that capture runoff which is conveyed west down Lascelles Boulevard. Only the upstream storm sewer section of this network was analysed as part of this project.

The next storm sewer system has the upstream end near Lascelles Boulevard and transfers flow south. Storm sewers on Crescent Hill Drive are connected to Dixie Road. It is assumed that since the lots on Crescent Hill Drive are so large the relative percent impervious is very low. The majority of runoff reaches its time of concentration much later than the road surface runoff, which is why the lot area was not included in the storm sewer design sheets.

The storm sewer system continues to convey flow south on Dixie Road and outlets to the crossing culvert just north of Queen Street East. The storm sewer diameter is 600mm at the outlet. Please refer to Figures 10 and 11 to view Dixie Road north and south of the crossing culvert.

Figure 10. Crossing Culvert facing North

Figure 11. At Crossing Culvert facing South

The storm sewer systems between Williams Parkway and Queen Street East were analysed and it was found that not all storm sewer sections have capacity for a 10-year storm event. The storm sewers on Crescent Hill Drive and Lascelles Boulevard do have capacity for the 10-year event. The remaining storm sewers have capacity for the 5-year storm event, which is what they were likely designed to contain. Refer to Drawing 4 for an illustration of the storm sewers, and Appendix B for storm sewer design sheets.

2.5 Etobicoke Creek Crossings

One watercourse crossing along the study area is Etobicoke Creek crossing under the Dixie Road – Bovaird Drive intersection. The intersection is seen in Figure 12 below. The crossing consists of twin 3.0m x 2.7m concrete box culverts, which seem to be in good condition and are shown in Figure 13 below.

Figure 12. Bovaird Dr. & Dixie Rd.

Figure 13. NW (Upstream) End of Crossing

Upstream of the stormwater management pond the watercourse is a wide-bottom trapezoidal channel with dense vegetation and straight path adjacent to Dixie Road. Downstream of the crossing the watercourse has similar characteristics as upstream; however it has a more natural section and meandering path through a series of parks.

This culvert will not be impacted by the proposed improvements to Dixie Road. A hydraulic analysis of the structure was not conducted as part of our study.

Another main watercourse crossing is Dixie Road just north of Queen Street East. The current crossing is a concrete box culvert, 3.05 m wide by 2.45 m high. The crossing culvert is approximately 29m long and at an angle of 71 degrees to Dixie Road. A previous report "Flood Study: Tributary of Etobicoke Creek – Dixie Road / Hillside Drive; City of Brampton", prepared by Aquafor Beech Limited in 2004 explains that the existing crossing culvert is able to convey the flows for the 2-year to 100-year storms, inclusive. The Regional storm, however, would spill over the roadway. The culvert seems to be in relatively good condition. Upstream of the crossing the channel is a concrete lined trapezoidal channel approximately 2-3 m deep and a top width of 8 – 10 m with vegetation adjacent to the concrete banks. Just upstream of the crossing there is a weir with a depth of approximately 0.5m.

Figure 14. Weir upstream of Crossing

Figure 15. Upstream End of Crossing Culvert

Downstream of the crossing the channel is similar to upstream. After veering to the south with a natural cross-section, the watercourse becomes a very straight trapezoidal channel adjacent to Dixie Road. Photos of the downstream end of the crossing can be found in Figures 16 and 17.

Figure 16. Downstream End of Crossing Culvert

Figure 17. Channel Downstream of Culvert

2.5.1 Existing Conditions Modelling

A hydraulic model of the creek was obtained from a previous flood study prepared by Aquafor Beech Limited. The model covers the Etobicoke Creek tributary from upstream of Hazelwood Drive to downstream of the Dixie Road crossing.

The hydraulic model from the Aquafor Beech study was compared against other available information regarding the crossing. At the Dixie Road crossing the model assumes a culvert obvert at an elevation of 216.54 m, and a minimum road elevation of 217.32 m. This compares well with the as-built drawings provided by the Region. The dimensions of the culvert (3.05 m wide x 2.44 m wide) in the model also agree with other records.

In addition to the main rectangular culvert opening, there is a 900 mm diameter pipe crossing under Dixie Road at the crossing location (See Figure 16). This additional opening was not included in the HEC-RAS hydraulic model prepared by Aquafor Beech Limited. AECOM staff revised the HEC-RAS hydraulic model to include the 900 mm diameter opening.

The output from the original and updated models are provided in Table 1. Adding the 900 mm diameter opening resulted in a 5 cm to 10 cm drop in upstream flood levels.

Table 1. Existing Hydraulic Model Results

Storm Event	Original Model Water Level	Updated Model Water Level	Clearance	Freeboard
5-year	215.37	215.32	1.22	2.00
10-year	215.63	215.54	1.00	1.78
25-year	215.94	215.84	0.70	1.48
50-year	216.16	216.06	0.48	1.26
100-year	216.39	216.29	0.15	1.03
Regional	217.75	217.72	N/A	OVERTOPPED

According to the City of Brampton Subdivision Design Manual, the criterion for culverts and bridges under arterial roads is that they must contain all storm events including the Regional without overtopping the roadway. The Ministry of Transportation Highway Design Standards state that the design storm for freeways and arterial roads with a culvert span less than 6m is a 50-year event. The minimum freeboard for this storm is 1.0m and the minimum clearance is 0.3m.

Clearance and freeboard were calculated for all storm events (based on the updated model) and are included in Table 1. The existing culvert provides more than adequate clearance and freeboard for storm events up to and including the 100 year storm event, but is predicted to be overtopped by a depth of approximately 0.4 m during the Regional storm.

Overtopping of Dixie Road during the Regional storm event could be eliminated if the existing structure was replaced with a larger culvert or bridge. However, the existing culvert is in good condition, with a considerable remaining service life. Furthermore, the existing structure should not require extensive modifications to accommodate the proposed improvements to Dixie Road. It was therefore concluded that replacement of the existing culvert is not warranted. More information on the required modifications to the structure is provided in Section 3.2.5.

Proposed Conditions

The proposed design for the study area has been planned for two phases, one in 2021 and the next in 2031. The first phase involves expanding Dixie Road from Mayfield Road to Countryside Drive from two lanes to four lanes. This also includes adding turning lanes at intersections. The second phase planned for 2031 involves expanding the entire study area from four lanes to six, including turning lanes at every intersection. The storm sewer analysis completed for proposed conditions assumes the ultimate 2031 design. Under existing conditions the paved road width varies from 9 metres at the north end of the site to 22 metres where there is a median. Under proposed conditions, the average paved road width varies from 23 metres to over 30 metres wide.

The alignment of the proposed roadway is very similar to the existing alignment for the north portion of the study area resulting in a symmetrical expansion of the roadway. The alignment changes at Northcliffe Drive where it starts to shift east. At North Park Drive the proposed curb on the west side is very close to the location of the existing curb, and on the east side the proposed curb is approximately two lane widths out from the existing curb. At the Northampton Street / Mansion Street intersection the alignment of the proposed design is slightly east of the existing alignment, resulting in more expansion on the east side. This shifted alignment continues south to Williams Parkway. South of which, the proposed alignment shifts slightly to the west. By Crescent Hill Drive the east proposed curb lines up with the existing curb, and the expansion is almost entirely on the west side of Dixie Road. This alignment continues south to Howden Boulevard, where the alignment starts to shift more to the east. By the south end of the site the proposed alignment coincides with the existing alignment once more.

At the north end of the site the cross-section of the road right-of-way will change from ditches to curb and gutter. The drainage area width will also be expanded from existing conditions. South of Countryside Drive, the road right-of-way is very wide compared to the existing pavement width. There is a significant grassed boulevard on either side of the roadway. Due to the existing wide drainage area, the proposed drainage areas are practically the same as existing since the pavement expansion will occur within the existing boulevards. The percent impervious and runoff coefficients for proposed analyses will change significantly.

The potential storm drainage impacts resulting from the proposed construction are mainly due to the increase in pavement (impervious) area. More impervious area generally results in less infiltration, higher peak flow, and decreased water quality relative to existing conditions. Higher peak flows can result in an increase in stream bank erosion and flooding, and a decline in water quality can affect the natural habitat of the creek. This study area can be divided into two sections, each draining to a separate tributary of Etobicoke Creek. The portion of the study area draining to the Dixie Road and Bovaird Drive crossing has an additional paved area of 5.8 hectares. The drainage area of the tributary at the crossing culvert location is approximately 1,400 hectares. The additional paved area is only 0.4% of the drainage area, and will therefore not have an impact on peak flow rates.

The south portion of the study area draining to the crossing culvert just north of Queen Street East has an additional 2.2 hectares of paved area under proposed conditions. This increase in pavement area includes drainage area to the Williams Parkway outfall and the increase pavement at Lascelles Blvd. The drainage area of the tributary is approximately 370 ha at that location according to the Aquafor Beech Flood Study Report completed in March 2004. The increase in pavement is therefore less than 0.6% of the drainage area. It can be assumed that the additional pavement to the study area will not impact peak flow rates.

There is the potential for a slight decrease in water quality due to the increase in pavement area, and therefore some sort of water quality measure will be required. Improvements to the existing storm sewers through the study area will also be required to convey the local increase in peak flow rates.

The crossing culvert at Bovaird Drive will not be impacted by the expansion. The grassed boulevards at the intersection will be used to expand the roadway, but the existing property lines will remain resulting in no changed to the crossing itself.

The crossing culvert just north of Queen Street East will likely have to be extended due to the widened roadway. Hydraulic modelling will be undertaken to ensure that water levels do not exceed existing conditions.

More detail on the potential impacts and alternative mitigation measures are provided in the following sections.

2.6 Stormwater Management Criteria

The stormwater management criteria as per the TRCA guidelines includes Level 1 (Enhanced) water quality, no increase in downstream erosion, and the proposed peak flow must not be greater than the existing peak flow.

The City of Brampton and Region of Peel storm sewer criteria were also referenced in the proposed conditions analysis. The storm sewer system must be designed for the 10 year return period storm event. The storm sewers can be sized based on the 5 year return period storm event if the 10 year standard cannot be achieved and there are no foundation drains connected to the storm sewer system.

As previously stated, the City of Brampton criterion for watercourse crossings is that they contain all storm events, including the Regional event. The Ministry of Transportation guidelines state that for an arterial road with a culvert less than 6m wide, during a 50-year event there must be at least 1.0m of freeboard and 0.3m clearance.

2.7 Stormwater Management Alternatives

A wide range of best management practices are available to mitigate the impacts of road runoff on receiving watercourses. These are generally classified into source, conveyance and end-of-pipe treatment alternatives.

Source control measures address precipitation where it falls, typically through infiltration. Source control measures for roadways are generally limited to permeable pavement, and reducing the pavement area to the extent feasible. Permeable pavement is generally not appropriate for urban arterial roadways. The permeable pavement structure may not withstand the volume of traffic and heavy vehicle traffic expected on Dixie Road. There are also concerns regarding potential for groundwater contamination, and the maintenance requirements to prevent clogging of the permeable pathways through the pavement. **Neither permeable pavement nor any other source control measures have been carried forward for evaluation as a treatment alternative for Dixie Road.**

Conveyance control measures treat storm runoff as it flows from the source to the receiving watercourse. The most common application to road drainage is the enhanced roadside swale, with a wide, flat bottom (< 0.75 m) that acts to slow and infiltrate road runoff. The flat bottom also facilitates the establishment of vegetation which further slows runoff and traps pollutants through filtering and nutrient uptake. Through much of the study area, space is not available in the Dixie Road right of way for enhanced swales. Regardless, **enhanced vegetated swales have been carried forward as a potential measure to address the quality of storm runoff from Dixie Road.**

Pervious pipe systems are another conveyance control alternative applicable to road drainage. The clay soil type found in the area limits the potential effectiveness of a pervious pipe drainage system for Dixie Road. Furthermore, for a well travelled roadway such as Dixie Road, extensive pre-treatment would be required to prevent clogging of the system with fine particles and to prevent groundwater contamination, particularly in the event of a spill of fluid from an automobile or transport. **Pervious pipe systems** are much more applicable to smaller residential areas, and **have not been considered further for application on Dixie Road**.

End-of-pipe measures include oil-grit separators and stormwater management wet ponds and wetlands. Oil grit separators are commonly applied to road drainage systems, either as stand-alone devices or as components of larger stormwater management systems. While oil-grit separators cannot reduce the rate or volume of runoff, they have the potential to trap and hold both particulate matter and floatables, including oil and grease. Stormwater management ponds have been applied extensively to manage urban storm runoff and have been shown to significantly reduce peak flow rates and pollutant loadings. **Both oil-grit separators and end-of-pipe stormwater management facilities have been carried forward for potential application along Dixie Road.**

Stormwater management alternatives for Dixie Road are assessed in more detail in the following sections.

2.7.1 Mayfield Road to Sandalwood Parkway

The proposed widening of this section of Dixie Road will increase the total paved area from 4.7 ha to 8.6 ha in the ultimate design scenario. The increase in paved area has the potential to overwhelm the existing storm drainage systems, and increase pollutant loadings delivered to Etobicoke Creek.

An analysis of the existing storm sewer system was completed to evaluate its capacity relative to the runoff from the widened roadway. The storm sewers between Mayfield Road and Sandalwood Parkway do not have capacity for a 10-year storm event under proposed conditions. This network of storm sewers will therefore need to be upgraded for proposed conditions. The table below indicates details of the storm sewer system upgrades. Further detail on proposed storm sewers is located in Appendix B. A plan showing the locations of the proposed upgraded sewers is included in Drawing 5.

There is currently a ditch drainage system near the north end of the site, and once the roadway is expanded the minor drainage will be captured and conveyed by an extended storm sewer instead. The existing storm sewer will be extended from manhole A1 up to Mayfield Road.

Table 2. Proposed Storm Sewer Diameters

Street	From MH	To MH	Drainage Area (ha)	Ex. Diam (mm)	Prop. Diam (mm)
Dixie Road	X0	X1	0.93		525
	X1	X2	0.48		600
	X2	X3	0.42		675
	X3	X4	0.40		675
	X4	X5	0.39		750
	X5	X6	0.38		750
	X6	A1	0.34		750
	A0	A1	0.37	375	REMOVED
	A1	A2	0.38	675	750
	A2	A3	0.40	675	750
	A3	A4	0.14	675	750
@ Countryside Drive	A4	A5	0.91	675	750
	A5	A6	0.45	675	825
	A6	A7	0.46	750	825
	A7	A8	0.57	750	900
@ Father Tobin Road	A8	A9	0.83	825	900
	A9	A10	0.64	825	975
Outlets at Crossing Culvert	A10	A11	0.72	900	975
	A11	A12	0.74	900	900
	A12	A13	1.11	900	900
	A13	A14	0.75	1050	1050
	A14	A15	0.75	1050	1050
@ Sandalwood Parkway	A15	A16 (outlet)	0.20	1050	1050

This section of the project has two phases, expanding Dixie Road from Countryside Drive to Mayfield Road from two lanes to four lanes in 2021, followed by an expansion of Dixie Road to six lanes in 2031. During the first phase of the project the storm sewers between Countryside and Mayfield will be upgraded to the ultimate proposed conditions design, however, the storm sewers downstream of Countryside Drive will not be upgraded until the second phase. A design sheet analysis was completed to determine storm sewer capacity between construction phases. During a 10-year storm event only two storm sewer sections downstream of Countryside would be slightly undersized for the 10-year event. During a 5-year event all storm sewer sections would have more than enough capacity. This storm sewer scenario is just temporary and impact would be negligible so downstream storm sewers should be upgraded during the second construction phase as planned.

The proposed improvements to this section of Dixie Road will result in an additional 3.9 ha of pavement area. This has the potential to increase peak flow rates and impair water quality in the receiving drainage systems. However, the storm sewers currently servicing this section of Dixie Road discharge to a trunk storm sewer and an open channel located just west of the road right-of-way. The trunk sewer and open channel lead to the on-line stormwater management pond at the north-west corner of Dixie Road and Bovaird Drive.

The Dixie-Bovaird stormwater management pond provides water quality, flood and erosion control for the entire 1400 ha drainage area to the pond, which includes Dixie Road. We cannot confirm that the design of the pond considered the potential ultimate configuration of Dixie Road, but it can be concluded that the existing pond can easily accommodate the small additional pavement area associated with the proposed roadway improvements.

It is therefore concluded that stormwater management for this portion of Dixie Road can be provided by the existing Dixie-Bovaird SWM pond, and no further stormwater management measures are warranted.

2.7.2 Sandalwood Parkway to Bovaird Drive

The proposed widening of this section of Dixie Road will increase the total paved area from 3.8 ha to 5.2 ha in the ultimate design scenario.

An analysis of the existing storm sewer system was completed to evaluate its capacity relative to the runoff from the widened roadway. The storm sewers between Sandalwood Parkway and Bovaird Drive do not have capacity for a 10-year storm event under proposed conditions. This network of storm sewers will therefore need to be replaced for proposed conditions. The table below indicates details of the storm sewer system. Further detail on proposed storm sewers is located in Appendix B. A plan showing the locations of the proposed upgraded sewers is included in Drawings 6 and 7.

Table 3. Proposed Storm Sewer Diameters

Street	From MH	To MH	Drainage Area (ha)	Ex. Diam (mm)	Prop. Diam (mm)
Dixie Road					
@ Springtown Trail (N)	B0	B1	0.66	375	450
	B1	B2	0.49	450	600
	B2	B3	0.50	450	600
	B3	B4	0.28	450	600
@ Springtown Trail (S)	C0	C1	0.64	375	450
	C1	C4	0.49	450	600
	C2	C3	0.49	375	450
	C3	C4	0.44	450	525
	C4	Outlet	0.26	375	525
@ Peter Robertson Blvd.	D0	D1	1.06	375	525
	D1	D2	0.71	450	675
	D2	D3	0.62	525	750
	D3	D4	0.88	525	750
	D4	D5	0.96	900	900
	D5	Outlet	0.00	900	900

The proposed improvements to this section of Dixie Road will result in an additional 1.4 ha of pavement area. Runoff from this area also drains to the Dixie-Bovaird stormwater management pond. The pond can easily accommodate and treat the small additional pavement area from this section of Dixie Road.

For the same reasons discussed in the previous section, it is concluded that stormwater management for this portion of Dixie Road can be provided by the existing Dixie-Bovaird SWM pond, and no further stormwater management measures are warranted.

2.7.3 Bovaird Drive to S. of Williams Parkway

The paved area between Bovaird Drive and Williams Parkway is designed to expand from 3.2 ha to 4.5 ha by 2031.

The storm sewers in this area were analysed and it was found that they do not have capacity for proposed conditions, and so upgraded storm sewers will have to be implemented during construction to meet the Region of Peel design standards. The proposed storm sewer sizes are listed in Table 4 below. An illustration of proposed storm sewers can be found in Drawings 7 & 8.

Table 4. Proposed Storm Sewer Diameters

Street	From MH	To MH	Drainage Area (ha)	Ex. Diam (mm)	Prop. Diam (mm)
Dixie Road					
	E0	E1	0.76	300	525
@ North Park Dr.	E1	E2	0.33	375	600
	E2	E3	0.23	450	600
	E3	E4	0.30	450	600
	E4	E5	0.30	525	675
@ Northcliffe / Moregate	E5	E6	0.32	525	675
	E6	E7	0.36	525	675
	E7	E8	0.32	525	750
	E8	E9	0.32	525	750
	E9	Outlet	0.00	525	750
S. of North Park Dr.	F0	F1	0.60	300	375
	F1	F2	0.36	300	450
N. of Mansion	F2	F3	0.52	375	450
S. of Mansion	F3	F4	0.39	375	525
	F4	F5	0.44	450	600
@ Williams Pkwy	F5	F6	0.53	450	600
S. of Williams	G0	G1	0.60	300	450
	G1	F6	0.50	375	525

The existing storm sewer on Williams Parkway downstream of MH "F6" that conveys flow from Dixie Road to the outfall west of Dixie road is currently a 525mm diameter sewer. This section of storm sewer was not analysed as it is not within AECOM's scope of work. Since storm sewers upstream of MH "F6" will be upgraded, the 525mm storm sewer will be undersized under proposed conditions and will have to be upgraded. It is recommended that this section of storm sewer be analysed as part of the detailed design phase to determine the proposed diameter.

Drainage from this section of Dixie Road is split between 2 outlet locations.

Approximately 3.25 ha of Dixie Road drain to a sewer that discharges to Etobicoke Creek downstream of the stormwater management pond. The proposed improvements to Dixie Road will increase the total paved area to this outfall from 1.30 ha to 1.80 ha.

Alternatives were explored to mitigate the impacts of the proposed improvements on the quality and quantity of water delivered to Etobicoke Creek.

Enhanced vegetated swales were considered. Existing residential development abuts both sides if Dixie Road, and there is therefore no room to implement roadside swales through this section. The outlet location was examined to determine if a vegetated swale could be constructed from the sewer outlet for a suitable length before discharging to Etobicoke Creek. Etobicoke Creek flows through a city park in this area, which could facilitate a treatment swale. However, the park corridor west of the watercourse is relatively steeply sloped, and there is no area available for a reasonable treatment swale, nor for a stormwater management pond.

It is concluded that vegetated swales and stormwater management ponds are not feasible for the section of Dixie Road between North Park Drive and Bovaird Drive. Oil-grit separators are the only remaining feasible alternative to manage runoff from this section. An oil-grit separator could be installed upstream of the outlet to Etobicoke Creek. This would provide water quality treatment for the runoff from Dixie Road, but would not reduce the rate of volume of runoff delivered to the creek.

The increase in paved area of 0.5 ha represents 0.04 % of the total drainage area of 1400 ha at the sewer outlet. It is therefore concluded that an oil-grit separator will provide adequate water quality treatment for this area, and the proposed additional pavement area will have a negligible impact on flooding and erosion in Etobicoke Creek.

Runoff from the southern half of this section of Dixie Road empties into an existing storm sewer on Williams Parkway. At Williams Parkway, sewers on Dixie Road both north and south of Williams Parkway combine with a sewer from east of Dixie Road. From the junction manhole, a 525 mm diameter leads westward to a smaller tributary of Etobicoke Creek. The receiving watercourse is confined to a concrete lined channel upstream of Williams Parkway, and is piped from north of Williams Parkway to south of Leacrest Street.

Of the 3.94 hectares of Dixie Road draining to tributary of Etobicoke Creek at the Williams Parkway outlet, it is proposed that the pavement be increased from 1.90 ha to 2.71 ha. This may cause negative impacts on the receiving watercourse in regards to quality and quantity. There is no room for vegetated swales along Dixie Road due to the abutting residential developments. Limited space would also make it difficult to construct a stormwater management pond. The only possible pond location would be at the culvert outlet south of Leacrest Street. This is not an ideal location for a pond since space is limited and so the pond would be smaller than necessary to treat stormwater to quality and quantity levels required. A pond in this location would also destroy existing habitat along the natural watercourse. It is therefore not recommended to construct a pond in this location. Oil grit separators can be installed at the downstream ends of both storm sewer systems on Dixie Road to treat water quality. The drainage area to the watercourse tributary at that location is approximately 275 hectares. The 0.81 ha increase in pavement represents less than 0.3% of the tributary drainage area to that point. With such a relatively small increase in pavement it is not expected that peak flows will have a noticeable increase in the future.

2.7.4 S. of Williams Parkway to Queen Street East

In the ultimate design scenario the paved area along the storm sewer network from south of Lascelles Boulevard to the Queen Street culvert would increase from 2.93 ha to 3.65 ha.

The storm sewers were analysed and it was determined that not all the existing sewer sections have capacity for the 10-year storm under proposed conditions. Table 5 below lists the required diameters for the increased runoff. Further detail on storm sewer is located in Appendix B. A plan showing the locations of the proposed upgraded sewers is included in Drawing 8.

Table 5. Proposed Storm Sewer Diameters

Street	From MH	To MH	Drainage Area (ha)	Ex. Diam (mm)	Prop. Diam (mm)
Dixie Road					
S. of Lascelles Blvd.	H0	H2	0.73	375	450
Crescent Hill Dr. N	H1	H2	0.53	375	375
	H2	H3	0.36	375	525
	H3	H5	0.29	450	525
Crescent Hill Dr. S	H4	H5	0.99	450	450
	H5	H6	0.31	525	675
	H6	H7	0.45	525	675
	H7	H8	1.01	525	750
	H8	H9	0.69	600	825
Culvert on Queen St. E	H9	Outlet	0.80	600	825
Lascelles Blvd.	I0	I1	0.72	375	375

The 0.72 ha of increased pavement compares to the tributary drainage area to that point of approximately 370 hectares, representing less than 0.2%. This is not likely to produce any increase in peak flows. Water quality, however, may decrease as a result of the increased pavement, and so an oil grit separator should be installed just upstream of the outfall to maintain water quality. The area was evaluated to see if it would be feasible to implement a stormwater management pond or vegetated swales, however, the space constraints prevent either of these from being constructed and so an oil-grit separator remains the ideal solution for quality control.

There is a slight increase in paved area within the Lascelles drainage area, however the storm sewer has capacity for the increased flow, therefore it can be assumed that storm sewers downstream also have capacity. The storm sewer system along Lascelles Boulevard discharges into the same tributary as the other storm sewers, only north of Queen Street. With a 0.13 ha increase in pavement it is unlikely that this would result in any changes in quality or quantity and so no measures are recommended for this area.

2.7.5 Etobicoke Creek Crossings

The Etobicoke Creek crossing at Bovaird Drive will not be impacted by the proposed design. The pavement expansion will remain within the existing property lines and therefore the existing crossing culvert will not need to be extended.

There is a crossing culvert at the south end of the study area, just north of Queen Street East. The existing culvert is approximately 29 metres long, crossing Dixie Road at a 71 degree angle. The proposed width of the road right-of-way at the location is wider than that of the existing culvert. The crossing culvert therefore must be extended to a total length of up to 38 m.

2.7.6 Proposed Conditions Modelling

The existing conditions hydraulic model was modified to account for the extended culvert at Dixie Road. The culvert was extended to 38 m and the distance between cross-sections was also modified to take the extended culvert into consideration. The updated model was run and the results are listed in the table below.

The results indicate an increase in upstream water levels by 1-2 cm in all storm events other than the Regional event which remained the same. This increase can be considered insignificant and is not expected to cause any noticeable change to the creek, the crossing or surrounding properties.

Table 6. Proposed Hydraulic Model Results

Storm Event	Ex. Water Level	Prop. Water Level	Clearance	Freeboard
5-year	215.32	215.33	1.21	1.99
10-year	215.54	215.56	0.98	1.76
25-year	215.84	215.86	0.68	1.46
50-year	216.06	216.08	0.46	1.24
100-year	216.29	216.30	0.24	1.02
Regional	217.72	217.73	N/A	OVERTOPPED

The water levels just upstream of the watercourse crossing are all 1-2cm higher than in existing conditions. This negligible amount will not have any impact on nearby properties. The City of Brampton criteria is not met as the Regional storm overtops the road as it does in existing conditions. The MTO criteria for freeboard and clearance are still met after the crossing culvert has been extended.

2.8 Assessment of Mitigation Measures

The quality and quantity measures for this study area will benefit the watercourse downstream. The existing stormwater management pond at Dixie Road and Bovaird Drive allows the north portion of the site to be treated without any additional measures. The storm sewer networks downstream of the stormwater management pond will not be treated with any existing measures.

Ideally, another stormwater management pond would be designed to treat the south portion of the study area, but the expansion in pavement consumes the remainder of available space in the road right-of-way and so this is not possible. The area is so developed that there is not even space for vegetated swales to treat some of the runoff. It is therefore recommended that oil-grit separators are implemented at the downstream end of storm sewer networks that are downstream of the stormwater management pond. This should be sufficient to mitigate the impact of the proposed widening on water quality.

The increase in peak flow for the storm sewer networks downstream of the stormwater management pond will have some impact on the receiving watercourses. However, compared to the relative size of the receiving watershed to that of the increased pavement, the impact will be negligible. This small impact does not warrant the cost or disruption of implementing a stormwater management measure. The tributary at Bovaird Drive downstream of the stormwater management pond meets up with the tributary that crosses Williams Parkway and Dixie Road farther south near Orenda Road east of Dixie Road with a combined area of over 2,900 hectares. These tributaries are only a portion of the large area that makes up the Etobicoke Creek watershed, and so 1.9 hectares of increased pavement will not make a significant impact on peak runoff rates.

3. Summary and Conclusions

Throughout the study area Dixie Road is currently a four-lane roadway with an urban cross section, except for the very north portion which is a two-land roadway with ditches. Storm drainage is provided by a combination of storm sewers, ditch flow and overland flow along the roadway. For the entire site the pavement area is increased from 15.2 ha to 22.9 ha. Of the 7.7 ha of increased pavement, 5.3 ha will drain to the existing stormwater management pond at Dixie Road and Bovaird Drive. This will treat the increased runoff quality and quantity. The 2.4 ha downstream of the pond will have oil-grit separators installed to treat water quality upstream of outfalls or connections. The increased quantity produced by the expanded pavement will be negligible and therefore does not warrant the cost or inconvenience of treatment since the lack of space renders a treatment solution to be drastic, expensive and disruptive to the public.

The section of storm sewer between Williams Parkway and the outfall west of Dixie Road will be undersized during proposed conditions, and should be analysed during the detailed design phase to determine proposed upgrade diameter.

The crossing culvert just north of Queen Street will have to be extended by up to 8m to allow it to span the width of the widened roadway. The hydraulic analysis found that the impact to upstream water levels would be negligible.

AECOM

4. References

Aquafor Beech Limited, “Flood Study – Tributary of Etobicoke Creek – Dixie Road /Hillside Drive; City of Brampton”. March 2004.

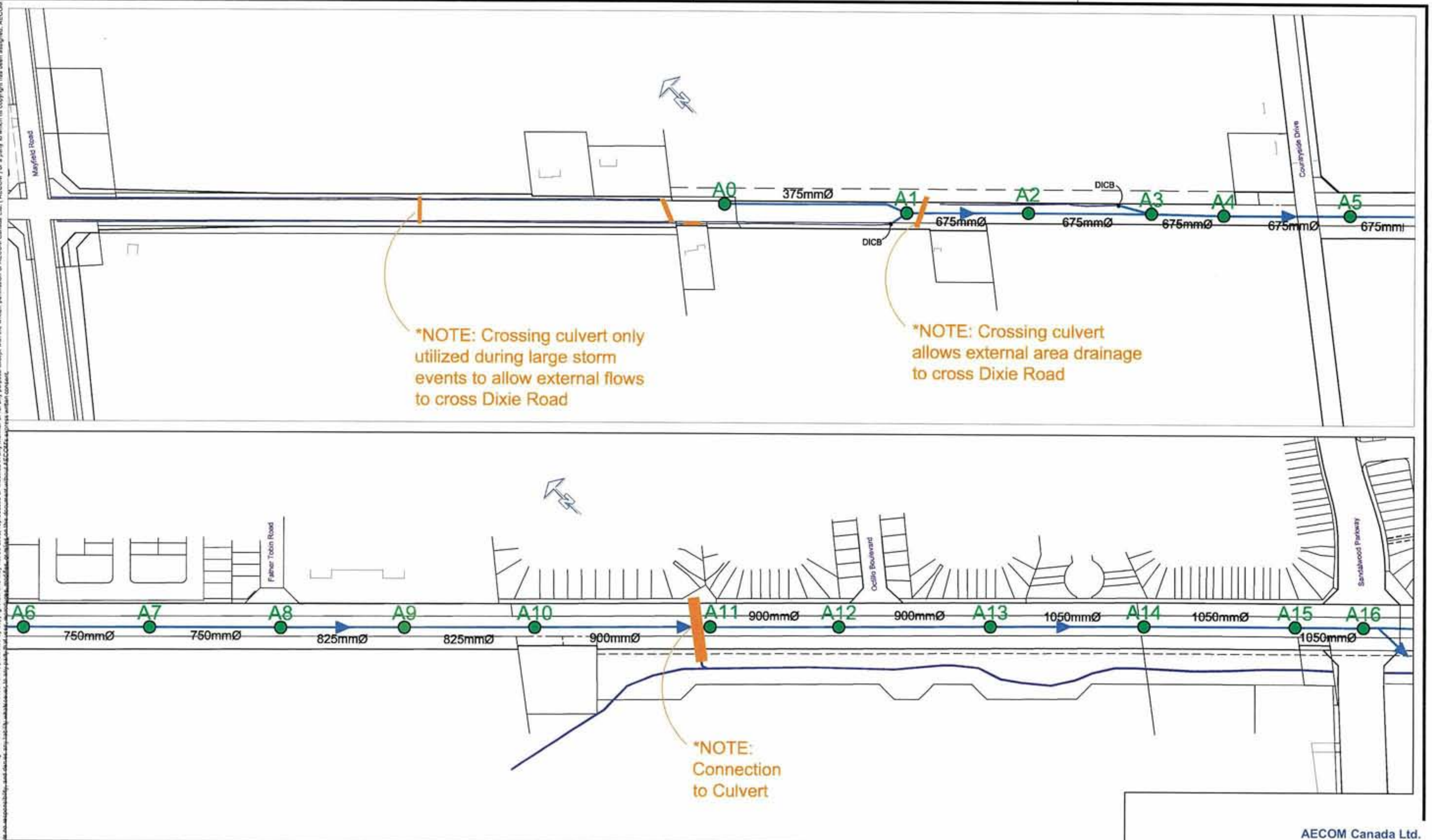
Corporation of the City of Brampton. “Subdivision Design Manual.” November, 2006

Hoffman, D.W. and Richards, N.R, “Soil Survey of Peel County – Report No. 18 of the Ontario Soil Survey.” Experimental Farms Services, Canada Department of Agriculture and the Ontario Agriculture College, 1953.

J.F. Sabourin and Associates Inc. “Evaluation of Roadside Ditches and Other Related Stormwater Management Practices”. February, 2000.

Ministry of Transportation Ontario. “Highway Drainage Design Standards”, January, 2008.

Ontario Ministry of the Environment. “Stormwater Management Planning and Design Manual.”. March 2003.



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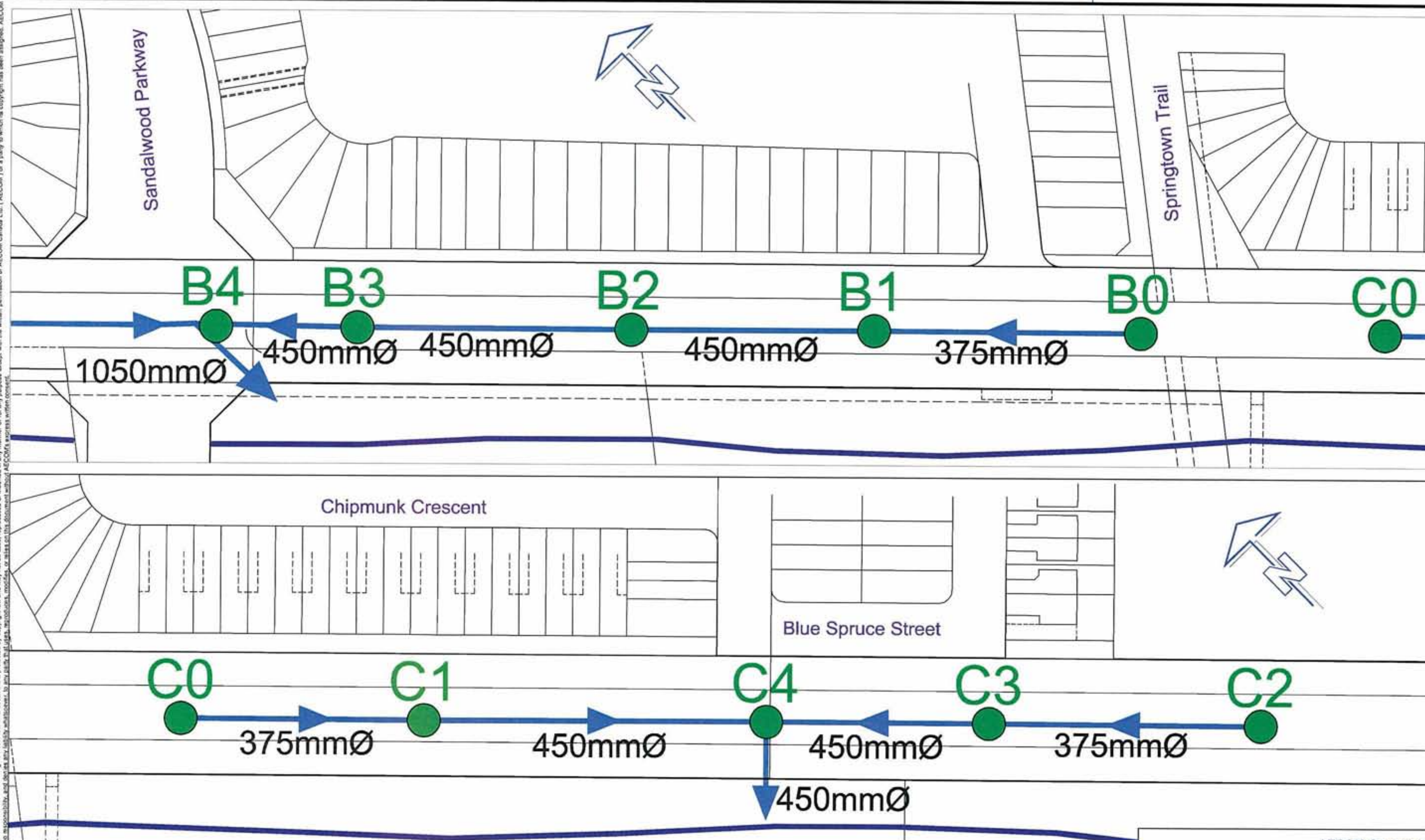
Dixie Road Class Environmental Assessment

Existing Conditions
Storm Sewers - Section 1
Drawing 1



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Existing Conditions

Storm Sewers - Section 2

Drawing 2

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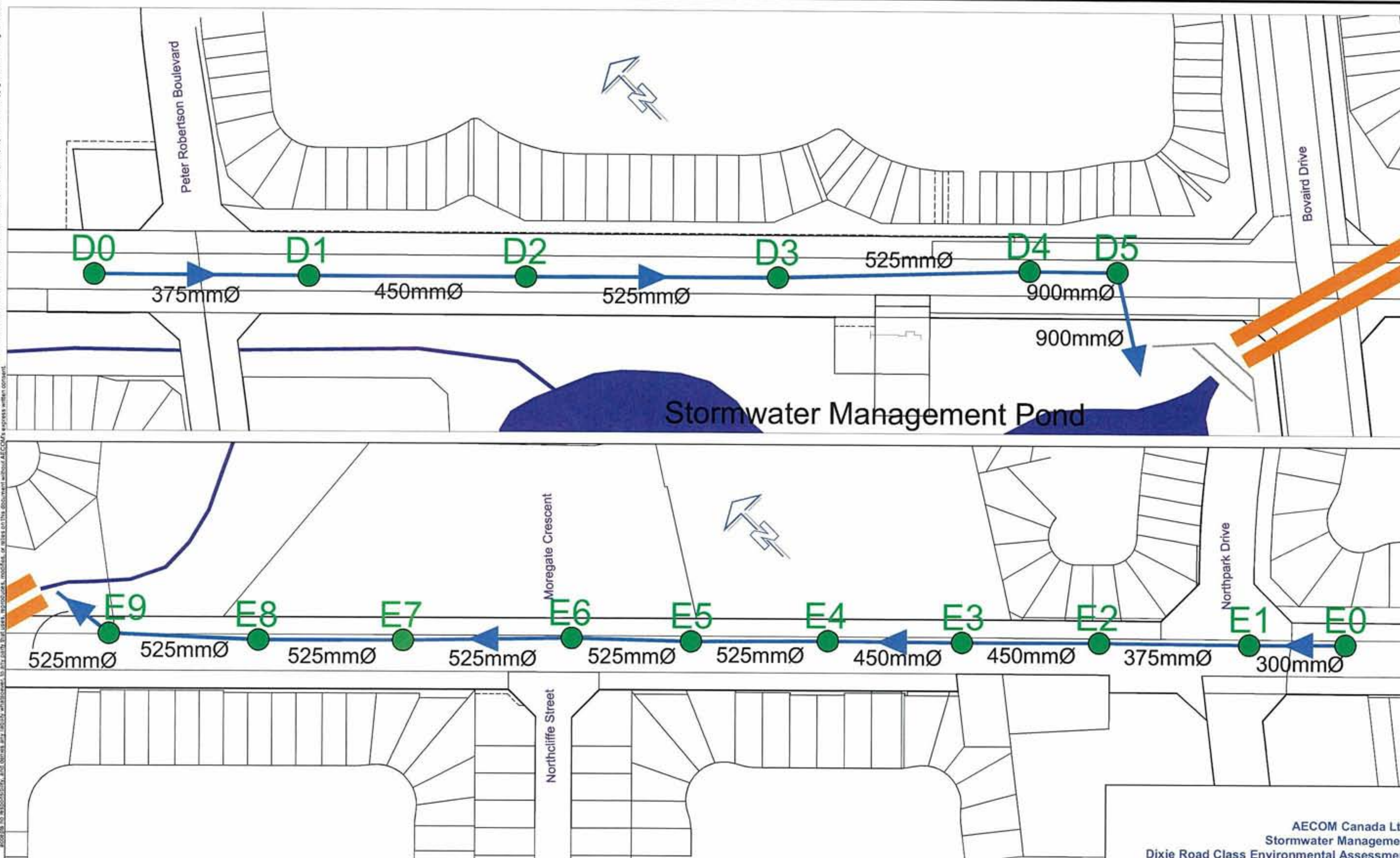

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Existing Conditions
Storm Sewers - Section 3
Drawing 3

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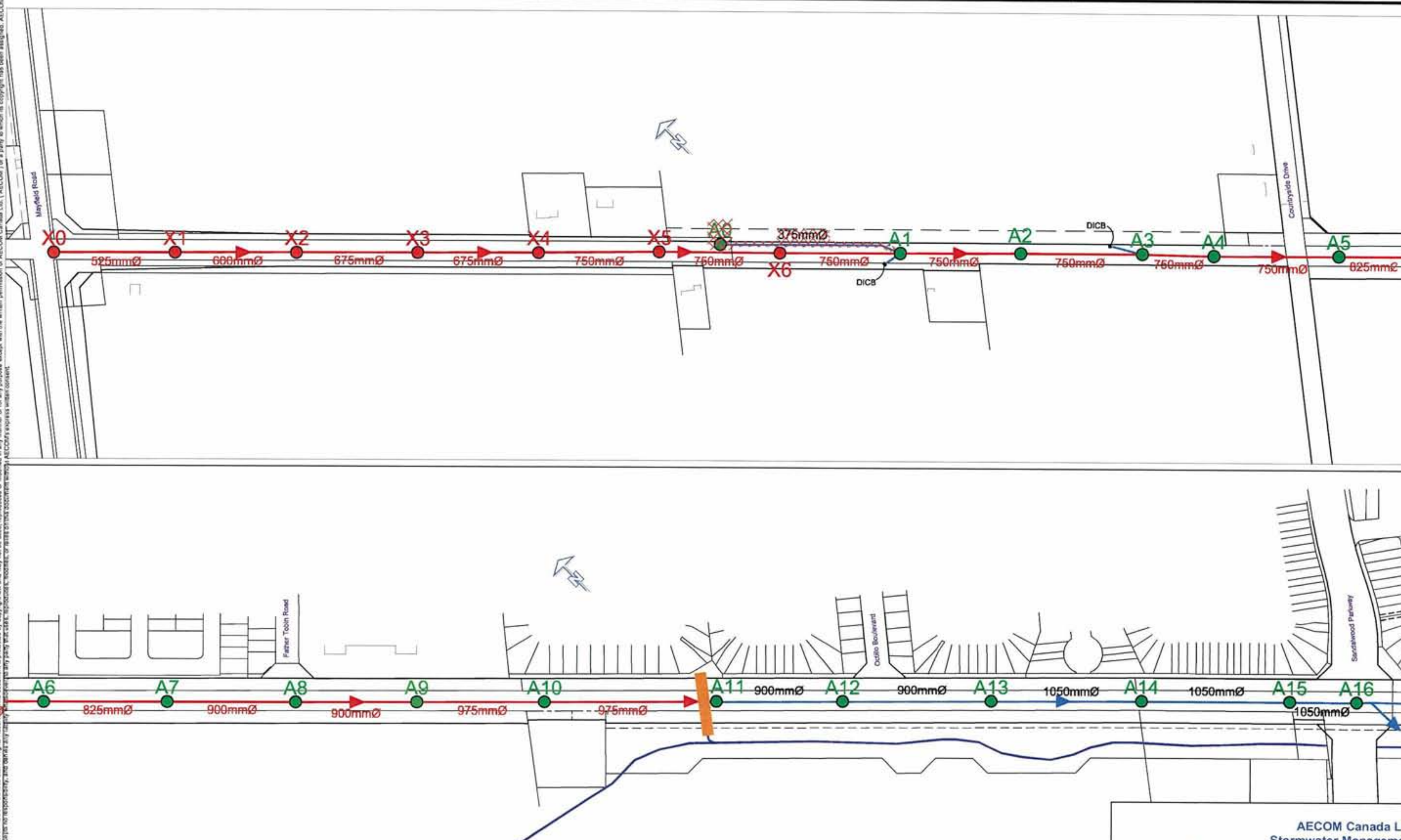
Dixie Road Class Environmental Assessment

Existing Conditions
Storm Sewers - Section 4
Drawing 4



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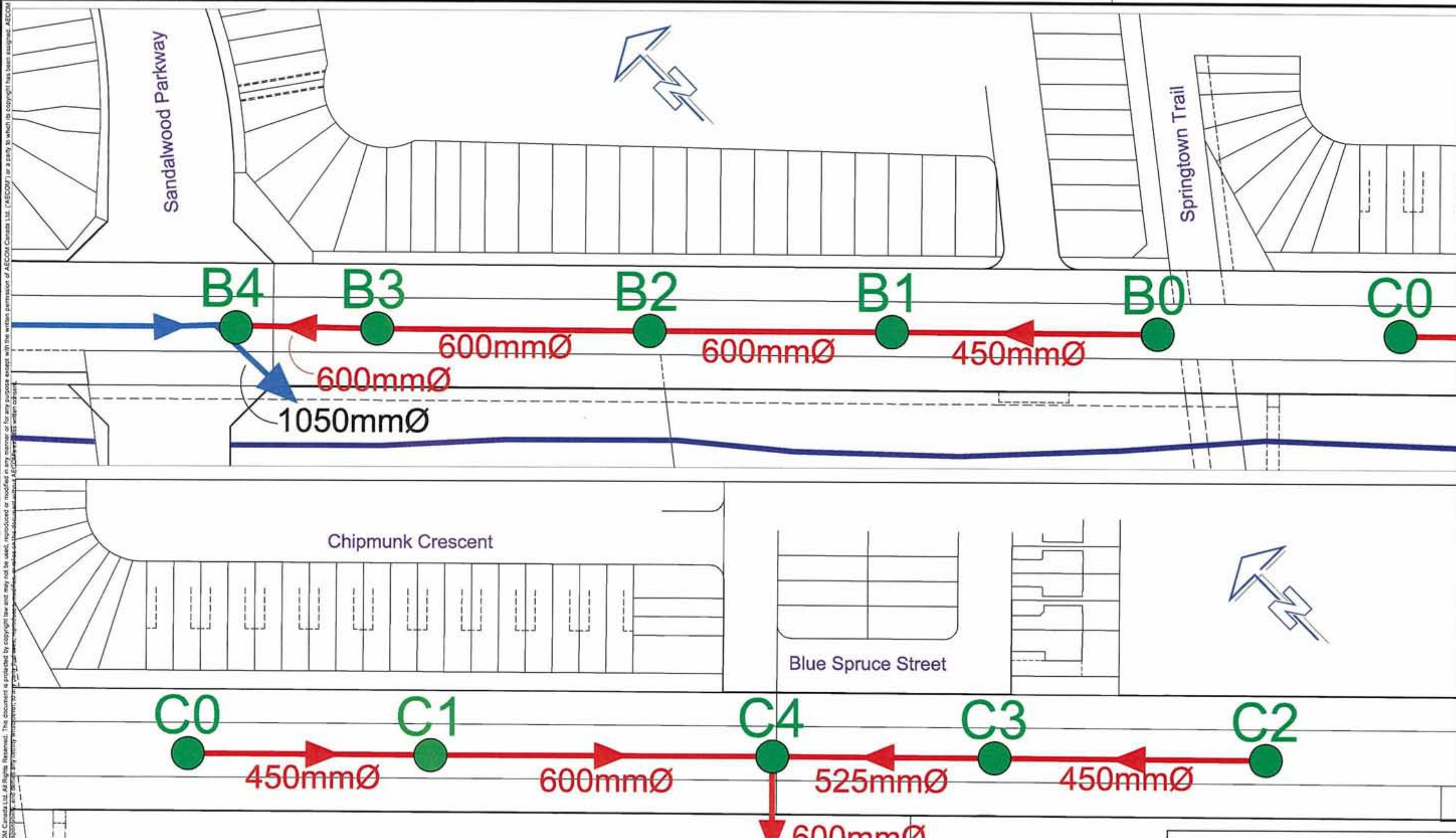
Proposed Conditions
Storm Sewers - Section 1
Drawing 5



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Proposed Conditions
Storm Sewers - Section 2
Drawing 6

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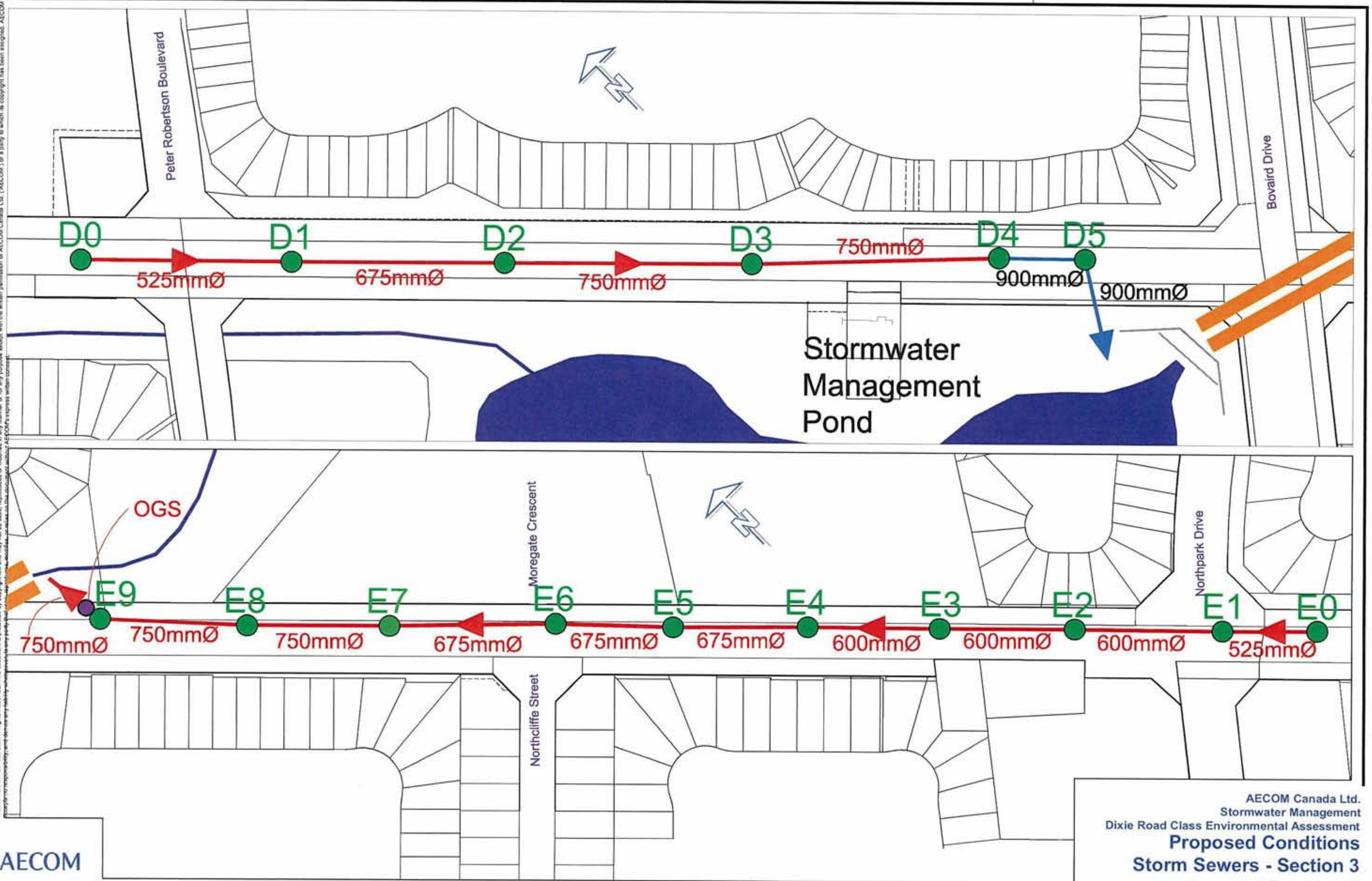

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Proposed Conditions
Storm Sewers - Section 3
Drawing 7

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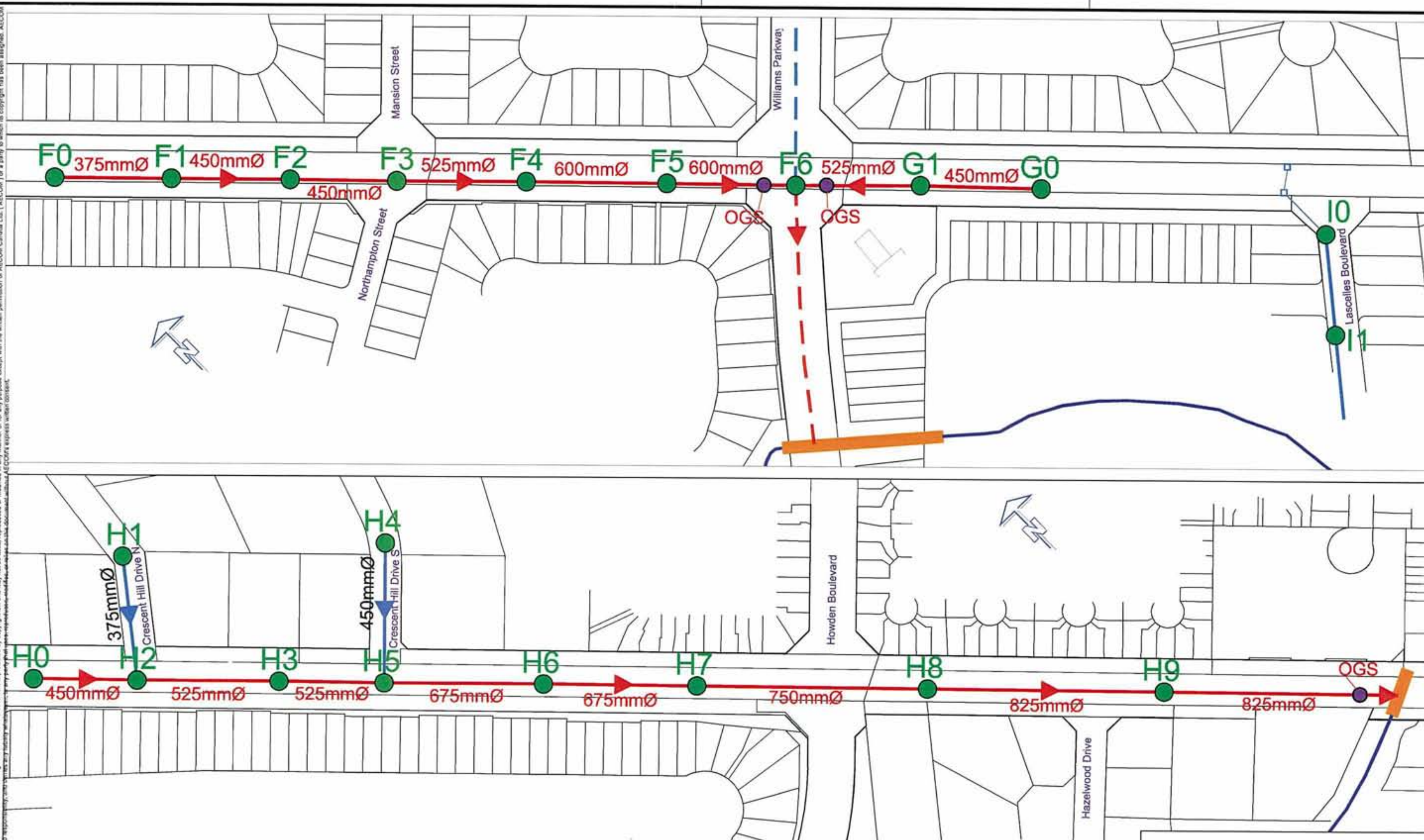

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Proposed Conditions
Storm Sewers - Section 4
Drawing 8

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Appendix A – Dixie Road Preliminary Design (Preferred Alternative)

See Preferred Design Drawings in Environmental Study Report, Section 10.

Appendix B – Storm Sewer Design Sheets (Existing & Proposed)

Storm Sewer Design Sheets - Existing 10-year

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope	Pipe	Velocity	Time	
Street	MH From	MH To	Area (ha)	C	AC						(m)	(m)	(m/m)	Capacity	(m/s)	Pipe (min)	Total (min)
	Ditch	A1	2.06	0.42	0.87	0.87											10
	A0	A1	0.15	0.90	0.14	0.14	121.93	0.05	180	375	248.97	247.72	0.69%	0.15	1.32	2.27	12.27
	A1	A2	0.21	0.75	0.16	1.16	105.79	0.34	120	675	247.20	246.66	0.45%	0.56	1.58	1.27	13.54
	Ditch	A3	0.07	0.2	0.01	0.01											
	A2	A3	0.21	0.75	0.16	1.34	98.79	0.37	120	675	246.63	246.06	0.47%	0.58	1.62	1.24	14.77
	A3	A4	0.08	0.75	0.06	1.40	92.97	0.36	72	675	246.03	245.68	0.49%	0.59	1.64	0.73	15.51
@ Countryside Drive	A4	A5	0.72	0.75	0.54	1.94	89.90	0.48	125	675	245.65	244.77	0.70%	0.70	1.97	1.06	16.56
	A5	A6	0.45	0.60	0.27	2.21	85.86	0.53	100	675	244.73	243.99	0.75%	0.73	2.03	0.82	17.39
	A6	A7	0.46	0.60	0.27	2.48	83.02	0.57	125	750	243.80	242.99	0.65%	0.90	2.04	1.02	18.41
	A7	A8	0.57	0.60	0.34	2.83	79.79	0.63	125	750	242.87	242.25	0.50%	0.78	1.77	1.17	19.58
@ Father Tobin Road	A8	A9	0.83	0.60	0.50	3.32	76.43	0.71	121	825	242.17	241.58	0.48%	1.00	1.87	1.08	20.66
	A9	A10	0.64	0.60	0.38	3.71	73.63	0.76	125	825	241.56	240.94	0.49%	1.01	1.88	1.11	21.77
Outlets at Crossing Culvert	A10	A11	0.72	0.60	0.43	4.14	71.01	0.82	168	900	240.83	240.05	0.47%	1.24	1.94	1.44	23.21
	A11	A12	0.74	0.60	0.45	0.45	121.93	0.15	130	900	239.99	239.37	0.47%	1.25	1.96	1.11	11.11
	A12	A13	1.11	0.60	0.67	1.11	113.35	0.35	150	900	239.33	238.63	0.47%	1.24	1.94	1.29	12.39
	A13	A14	0.75	0.60	0.45	1.56	105.04	0.46	150	1050	238.53	237.80	0.48%	1.90	2.19	1.14	13.53
	A14	A15	0.75	0.60	0.45	2.01	98.81	0.55	150	1050	237.81	237.01	0.53%	1.99	2.30	1.09	14.62
@ Sandalwood Pkwy	A15	A16 (Outlet)	0.20	0.60	0.12	2.13	93.64	0.55	67	1050	236.85	236.68	0.27%	1.41	1.63	0.69	15.31

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope	Pipe	Velocity	Time	
Street	MH From	MH To	Area (ha)	C	AC						(m)	(m)	(%)	Capacity	(m/s)	Pipe (min)	Total (min)
																	10.00
@ Springtown Trail (N)	B0	B1	0.66	0.53	0.35	0.35	121.93	0.12	100	375	237.66	237.01	0.66%	0.14	1.29	1.29	11.29
	B1	B2	0.49	0.51	0.25	0.60	112.04	0.19	90	450	236.94	236.59	0.39%	0.18	1.12	1.34	12.63
	B2	B3	0.50	0.52	0.26	0.85	103.66	0.25	100	450	236.55	235.98	0.56%	0.21	1.34	1.24	13.87
@ Sandalwood	B3	B4	0.28	0.62	0.17	1.03	97.12	0.28	52	450	235.97	235.63	0.65%	0.23	1.44	0.60	14.47
@ Springtown Trail (S)	C0	C1	0.64	0.49	0.31	0.31	121.93	0.11	90	375	237.41	236.82	0.65%	0.14	1.28	1.17	11.17
	C1	C4	0.49	0.49	0.24	0.56	112.89	0.17	127	450	236.73	236.13	0.47%	0.20	1.23	1.72	12.90
	C2	C3	0.49	0.54	0.27	0.27	121.93	0.09	100	375	237.08	236.58	0.50%	0.12	1.12	1.49	11.49
	C3	C4	0.70	0.49	0.34	0.61	110.74	0.19	83	450	236.56	236.15	0.50%	0.20	1.27	1.09	12.58
	C4	Trunk	0.00	0.00	0.00	1.17	102.18	0.33	10	450	236.17	234.37	17.96%	1.21	7.60	0.02	12.92
@ Peter Robertson Blvd.	D0	D1	1.06	0.61	0.64	0.64	121.93	0.22	120	375	237.02	236.33	0.57%	0.13	1.20	1.66	11.66
	D1	D2	0.71	0.47	0.33	0.98	109.58	0.30	120	450	236.22	235.67	0.46%	0.19	1.22	1.64	11.64
	D2	D3	0.62	0.49	0.30	1.28	109.69	0.39	140	525	235.59	234.86	0.52%	0.31	1.43	1.63	11.63
	D3	D4	0.88	0.45	0.39	1.68	109.76	0.51	140	525	234.84	233.19	1.18%	0.47	2.15	1.08	11.08
	D4	D5	0.96	0.44	0.42	2.10	113.52	0.66	88	900	232.65	231.77	1.00%	1.81	2.85	0.52	10.52
	D5	Outlet	0.00	0.00	0.00	2.10	117.75	0.69	47	900	231.70	231.00	1.49%	2.21	3.47	0.23	10.23

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC						(m)	(m)				Pipe (min)	Total (min)
																	10
	E0	E1	0.75	0.49	0.37	0.37	121.93	0.12	53	300	238.78	238.49	0.54%	0.07	1.01	0.88	10.88
@ North Park Dr.	E1	E2	0.33	0.63	0.21	0.58	114.97	0.18	84	375	238.40	237.99	0.49%	0.12	1.11	1.26	12.15
	E2	E3	0.23	0.64	0.15	0.73	106.52	0.21	76	450	237.89	237.55	0.45%	0.19	1.20	1.06	13.21
	E3	E4	0.30	0.52	0.15	0.88	100.50	0.25	76	450	237.49	237.10	0.51%	0.20	1.28	0.98	14.19
	E4	E5	0.30	0.49	0.15	1.03	95.60	0.27	76	525	237.02	236.61	0.54%	0.31	1.45	0.88	15.07
@ Northcliffe / Moregate	E5	E6	0.32	0.54	0.18	1.20	91.70	0.31	67	525	236.60	236.19	0.61%	0.34	1.56	0.71	15.78
	E6	E7	0.36	0.58	0.21	1.41	88.80	0.35	94	525	236.17	235.53	0.69%	0.36	1.65	0.95	16.73
	E7	E8	0.32	0.46	0.15	1.56	85.28	0.37	80	525	235.40	234.92	0.60%	0.33	1.54	0.87	17.59
	E8	E9	0.32	0.46	0.15	1.71	82.33	0.39	80	525	233.93	233.16	0.96%	0.42	1.94	0.69	18.28
	E9	outlet	0.00	0.00	0	1.71	80.17	0.38	40	525	233.00	232.50	1.25%	0.48	2.22	0.30	18.58

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC						(m)	(m)				Pipe (min)	Total (min)
																	10
S. of North Park Dr.	F0	F1	0.60	0.43	0.26	0.26	121.93	0.09	76	300	238.19	236.18	2.65%	0.16	2.23	0.57	10.57
	F1	F2	0.36	0.46	0.17	0.42	117.35	0.14	77	300	236.02	234.11	2.49%	0.15	2.16	0.59	11.16
N. of Mansion	F2	F3	0.52	0.54	0.28	0.71	112.99	0.22	69	375	234.04	232.25	2.60%	0.28	2.56	0.45	11.61
S. of Mansion	F3	F4	0.39	0.49	0.19	0.89	121.93	0.30	84	375	232.23	230.73	1.79%	0.23	2.13	0.66	12.26
	F4	F5	0.44	0.45	0.20	1.09	109.94	0.33	91	450	230.71	228.99	1.89%	0.39	2.46	0.62	12.88
	F5	F6	0.53	0.57	0.30	1.39	102.27	0.40	84	450	228.95	227.55	1.67%	0.37	2.32	0.60	13.48
	G0	G1	0.59	0.49	0.29	0.29	121.93	0.10	79	300	229.74	228.96	0.99%	0.10	1.36	0.97	10.97
S. of Williams	G1	F6	0.50	0.53	0.27	0.56	114.36	0.18	81	375	228.96	228.17	0.98%	0.17	1.57	0.86	11.83

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC						(m)	(m)				Pipe (min)	Total (min)
																	10.00
S. of Lascelles Blvd.	H0	H2	0.59	0.49	0.29	0.29	121.93	0.10	67	375	229.67	228.98	1.03%	0.18	1.61	0.69	10.69
CrescentHill Dr. N	H1	H2	0.53	0.52	0.28	0.28	121.93	0.09	80	375	230.75	229.16	1.99%	0.25	2.24	0.60	0.60
	H2	H3	0.36	0.53	0.19	0.76	116.41	0.25	92	375	228.93	227.39	1.68%	0.23	2.06	0.74	11.43
	H3	H5	0.29	0.52	0.15	0.91	111.10	0.28	68	450	226.68	225.39	1.90%	0.39	2.47	0.46	11.89
CrescentHill Dr. S	H4	H5	0.99	0.52	0.51	0.51	121.93	0.17	74	450			1.80%	0.38	2.41	0.51	10.51
	H5	H6	0.31	0.54	0.17	1.59	108.10	0.48	103	525	225.23	224.03	1.17%	0.47	2.15	0.80	12.69
	H6	H7	0.45	0.53	0.24	1.83	103.33	0.52	100	525	224.03	222.34	1.69%	0.56	2.59	0.64	13.33
	H7	H8	1.00	0.59	0.59	2.42	99.84	0.67	152	525	222.33	219.30	1.99%	0.61	2.80	0.91	14.24
	H8	H9	0.68	0.59	0.40	2.82	95.38	0.75	152	600	219.23	217.09	1.41%	0.73	2.58	0.98	15.22
Culvert on Queen St. E	H9	Outlet	0.72	0.55	0.39	3.21	91.06	0.81	152	600	217.06	214.93	1.40%	0.73	2.57	0.99	16.21
Lascelles Blvd.	I0	I1	0.72	0.50	0.36	0.36	121.93	0.12	76	375			1.00%	0.18	1.59	0.80	10.80

Storm Sewer Design Sheets - Existing 5-year

Location			Runoff			Accum AC	5 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC											Pipe (min)	Total (min)
																	10.00
@ Springtown Trail (N)	B0	B1	0.66	0.53	0.35	0.35	104.99	0.10	100	375	237.66	237.01	0.66%	0.14	1.29	1.29	11.29
	B1	B2	0.49	0.51	0.25	0.60	96.41	0.16	90	450	236.94	236.59	0.39%	0.18	1.12	1.34	12.63
	B2	B3	0.50	0.52	0.26	0.85	89.13	0.21	100	450	236.55	235.98	0.56%	0.21	1.34	1.24	13.87
@ Sandalwood	B3	B4	0.28	0.62	0.17	1.03	83.47	0.24	52	450	235.97	235.63	0.65%	0.23	1.44	0.60	14.47
@ Springtown Trail (S)	C0	C1	0.64	0.49	0.31	0.31	104.99	0.09	90	375	237.41	236.82	0.65%	0.14	1.28	1.17	11.17
	C1	C4	0.49	0.49	0.24	0.56	97.14	0.15	127	450	236.73	236.13	0.47%	0.20	1.23	1.72	12.90
	C2	C3	0.49	0.54	0.27	0.27	104.99	0.08	100	375	237.08	236.58	0.50%	0.12	1.12	1.49	11.49
	C3	C4	0.44	0.49	0.22	0.48	95.27	0.13	83	450	236.56	236.15	0.50%	0.20	1.27	1.09	12.58
	C4	Outlet	0.26	0.00	0.00	1.04	87.85	0.25	10	450	236.17	234.37	17.96%	1.21	7.60	0.02	12.92
@ Peter Robertson Blvd.	D0	D1	1.06	0.61	0.64	0.64	104.99	0.19	120	375	237.02	236.33	0.57%	0.13	1.20	1.66	11.66
	D1	D2	0.71	0.47	0.33	0.98	94.27	0.26	120	450	236.22	235.67	0.46%	0.19	1.22	1.64	11.64
	D2	D3	0.62	0.49	0.30	1.28	94.36	0.34	140	525	235.59	234.86	0.52%	0.31	1.43	1.63	11.63
	D3	D4	0.88	0.45	0.39	1.68	94.42	0.44	140	525	234.84	233.19	1.18%	0.47	2.15	1.08	11.08
	D4	D5	0.96	0.44	0.42	2.10	97.69	0.57	88	900	232.65	231.77	1.00%	1.81	2.85	0.52	10.52
	D5	Outlet	0.00	0.00	0.00	2.10	101.36	0.59	47	900	231.70	231.00	1.49%	2.21	3.47	0.23	10.23

Location			Runoff			Accum AC	5 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC											Pipe (min)	Total (min)
																	10
	E0	E1	0.75	0.49	0.37	0.37	104.99	0.11	53	300	238.78	238.49	0.54%	0.07	1.01	0.88	10.88
@ North Park Dr.	E1	E2	0.33	0.63	0.21	0.58	98.95	0.16	84	375	238.40	237.99	0.49%	0.12	1.11	1.26	12.15
	E2	E3	0.23	0.64	0.15	0.73	91.61	0.18	76	450	237.89	237.55	0.45%	0.19	1.20	1.06	13.21
	E3	E4	0.30	0.52	0.15	0.88	86.39	0.21	76	450	237.49	237.10	0.51%	0.20	1.28	0.98	14.19
	E4	E5	0.30	0.49	0.15	1.03	82.15	0.23	76	525	237.02	236.61	0.54%	0.31	1.45	0.88	15.07
@ Northcliffe / Moregate	E5	E6	0.32	0.54	0.18	1.20	78.77	0.26	67	525	236.60	236.19	0.61%	0.34	1.56	0.71	15.78
	E6	E7	0.36	0.58	0.21	1.41	76.25	0.30	94	525	236.17	235.53	0.69%	0.36	1.65	0.95	16.73
	E7	E8	0.32	0.46	0.15	1.56	73.20	0.32	80	525	235.40	234.92	0.60%	0.33	1.54	0.87	17.59
	E8	E9	0.32	0.46	0.15	1.71	70.66	0.34	80	525	233.93	233.16	0.96%	0.42	1.94	0.69	18.28
	E9	outlet	0.00	0.00	0	1.71	68.79	0.33	40	525	233.00	232.50	1.25%	0.48	2.22	0.30	18.58

Location			Runoff			Accum AC	5 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC											Pipe (min)	Total (min)
																	10
S. of North Park Dr.	F0	F1	0.60	0.43	0.26	0.26	104.99	0.07	76	300	238.19	236.18	2.65%	0.16	2.23	0.57	10.57
	F1	F2	0.36	0.46	0.17	0.42	101.01	0.12	77	300	236.02	234.11	2.49%	0.15	2.16	0.59	11.16
N. of Mansion	F2	F3	0.52	0.54	0.28	0.71	97.23	0.19	69	375	234.04	232.25	2.60%	0.28	2.56	0.45	11.61
S. of Mansion	F3	F4	0.39	0.49	0.19	0.89	94.58	0.23	84	375	232.23	230.73	1.79%	0.23	2.13	0.66	12.26
	F4	F5	0.44	0.45	0.20	1.09	91.01	0.28	91	450	230.71	228.99	1.89%	0.39	2.46	0.62	12.88
	F5	F6	0.53	0.57	0.30	1.39	87.93	0.34	84	450	228.95	227.55	1.67%	0.37	2.32	0.60	13.48
	G0	G1	0.59	0.49	0.29	0.29	104.99	0.09	79	300	229.74	228.96	0.99%	0.10	1.36	0.97	10.97
S. of Williams	G1	F6	0.50	0.53	0.27	0.56	98.41	0.15	81	375	228.96	228.17	0.98%	0.17	1.57	0.86	11.83

Location			Runoff			Accum AC	5 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC											Pipe (min)	Total (min)
																	10.00
S. of Lascelles Blvd.	H0	H2	0.69	0.49	0.34	0.34	104.99	0.10	67	375	229.67	228.98	1.03%	0.18	1.61	0.69	10.69
CrescentHill Dr. N	H1	H2	0.53	0.52	0.28	0.28	104.99	0.08	80	375	230.75	229.16	1.99%	0.25	2.24	0.60	10.60
	H2	H3	0.36	0.53	0.19	0.81	100.20	0.22	92	375	228.93	227.39	1.68%	0.23	2.06	0.74	11.43
	H3	H5	0.29	0.52	0.15	0.96	95.59	0.25	68	450	226.68	225.39	1.90%	0.39	2.47	0.46	11.89
CrescentHill Dr. S	H4	H5	0.99	0.52	0.51	0.51	104.99	0.15	74	450			1.80%	0.38	2.41	0.51	10.51
	H5	H6	0.31	0.54	0.17	1.64	92.99	0.42	103	525	225.23	224.03	1.17%	0.47	2.15	0.80	12.69
	H6	H7	0.45	0.53	0.24	1.88	88.85	0.46	100	525	224.03	222.34	1.69%	0.56	2.59	0.64	13.33
	H7	H8	1.00	0.59	0.59	2.47	85.82	0.59	152	525	222.33	219.30	1.99%	0.61	2.80	0.91	14.24
	H8	H9	0.68	0.59	0.40	2.87	81.95	0.65	152	600	219.23	217.09	1.41%	0.73	2.58	0.98	15.22
Culvert on Queen St. E	H9	Outlet	0.72	0.55	0.39	3.26	78.21	0.71	152	600	217.06	214.93	1.40%	0.73	2.57	0.99	16.21

Estimated Values

Storm Sewer Design Sheets - Proposed

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope (m/m)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC							(m)	(m)				Pipe (min)	Total (min)
																		10
	X0	X1	0.93	0.90	0.84	0.84	121.93	0.28	120		525	251.61	251.01	0.50%	0.30	1.40	1.42	11.42
	X1	X2	0.48	0.90	0.43	1.27	111.16	0.39	120		600	250.98	250.38	0.50%	0.43	1.54	1.30	12.73
	X2	X3	0.42	0.90	0.38	1.65	103.12	0.47	120		675	250.35	249.75	0.50%	0.59	1.66	1.20	13.93
	X3	X4	0.40	0.90	0.36	2.00	96.84	0.54	120		675	249.72	249.12	0.50%	0.59	1.66	1.20	15.13
	X4	X5	0.39	0.90	0.35	2.35	91.42	0.60	120		750	249.09	248.49	0.50%	0.79	1.78	1.12	16.26
	X5	X6	0.38	0.90	0.34	2.70	86.99	0.65	120		750	248.46	247.86	0.50%	0.79	1.78	1.12	17.38
	X6	A1	0.34	0.90	0.31	3.01	83.04	0.69	120		750	247.83	247.23	0.50%	0.79	1.78	1.12	18.50
	A1	A2	0.38	0.90	0.34	2.45	103.12	0.70	120	675	750	247.20	246.66	0.45%	0.75	1.69	1.18	13.91
	A2	A3	0.40	0.90	0.36	2.81	96.94	0.76	120	675	750	246.63	246.06	0.47%	0.77	1.74	1.15	15.06
	A3	A4	0.14	0.90	0.13	2.94	91.73	0.75	72	675	750	246.03	245.68	0.49%	0.78	1.76	0.68	15.74
@ Countryside Drive	A4	A5	0.91	0.90	0.82	3.75	88.95	0.93	125	675	750	245.65	244.77	0.70%	0.93	2.11	0.99	16.73
	A5	A6	0.45	0.90	0.40	4.16	85.27	0.98	100	675	825	244.73	243.99	0.75%	1.24	2.32	0.72	17.45
	A6	A7	0.46	0.90	0.41	4.57	82.81	1.05	125	750	825	243.80	242.99	0.65%	1.16	2.17	0.96	18.41
	A7	A8	0.57	0.90	0.52	5.08	79.79	1.13	125	750	900	242.87	242.25	0.50%	1.27	2.00	1.04	19.45
@ Father Tobin Road	A8	A9	0.83	0.90	0.74	5.83	76.80	1.24	121	825	900	242.17	241.58	0.48%	1.26	1.98	1.02	20.47
	A9	A10	0.64	0.90	0.58	6.40	74.12	1.32	125	825	975	241.56	240.94	0.49%	1.57	2.10	0.99	21.46
Outlets at Crossing Culvert	A10	A11	0.72	0.90	0.65	7.05	71.72	1.40	168	900	975	240.83	240.05	0.47%	1.53	2.05	1.36	22.82
	A11	A12	0.74	0.90	0.67	0.67	111.16	0.21	130	900	900	239.99	239.37	0.47%	1.25	1.96	1.11	12.53
	A12	A13	1.11	0.90	1.00	1.67	104.24	0.48	150	900	900	239.33	238.63	0.47%	1.24	1.94	1.29	13.82
	A13	A14	0.75	0.90	0.67	2.34	97.40	0.63	150	1050	1050	238.53	237.80	0.48%	1.90	2.19	1.14	14.96
	A14	A15	0.75	0.90	0.68	3.02	92.17	0.77	150	1050	1050	237.81	237.01	0.53%	1.99	2.30	1.09	16.04
@ Sandalwood Pkwy	A15	A16 (Outlet)	0.20	0.90	0.18	3.20	87.78	0.78	67	1050	1050	236.85	236.68	0.27%	1.41	1.63	0.69	16.73

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s	Invert d/s	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC							(m)	(m)				Pipe (min)	Total (min)
																		10.00
@ Springtown Trail (N)	B0	B1	0.66	0.90	0.59	0.59	121.93	0.20	100	375	450	237.66	237.01	0.66%	0.23	1.45	1.15	11.15
	B1	B2	0.49	0.90	0.44	1.03	113.08	0.32	90	450	600	236.94	236.59	0.39%	0.38	1.36	1.10	12.25
	B2	B3	0.50	0.90	0.45	1.48	105.89	0.43	100	450	600	236.55	235.98	0.56%	0.46	1.63	1.02	13.27
	B3	B4	0.28	0.90	0.25	1.73	100.14	0.48	52	450	600	235.97	235.63	0.65%	0.49	1.75	0.50	13.77
@ Springtown Trail (S)	C0	C1	0.64	0.90	0.58	0.58	121.93	0.20	90	375	450	237.41	236.82	0.65%	0.23	1.45	1.04	11.04
	C1	C4	0.49	0.90	0.45	1.02	113.85	0.32	127	450	600	236.73	236.13	0.47%	0.42	1.49	1.42	12.46
	C2	C3	0.49	0.90	0.44	0.44	121.93	0.15	100	375	450	237.08	236.58	0.50%	0.20	1.27	1.32	11.32
	C3	C4	0.44	0.90	0.40	0.84	111.89	0.26	83	450	525	236.56	236.15	0.50%	0.30	1.40	0.98	12.30
	C4	Outlet	0.26	0.90	0.23	1.08	104.65	0.31	120	375	600	237.02	236.33	0.57%	0.47	1.65	1.21	13.68
@ Peter Robertson Blvd.	D0	D1	1.06	0.90	0.95	0.95	121.93	0.32	120	375	525	237.02	236.33	0.57%	0.33	1.51	1.33	11.33
	D1	D2	0.71	0.90	0.64	1.59	111.81	0.49	120	450	675	236.22	235.67	0.46%	0.57	1.59	1.26	12.58
	D2	D3	0.62	0.90	0.56	2.15	103.94	0.62	140	525	750	235.59	234.86	0.52%	0.80	1.81	1.29	12.62
	D3	D4	0.88	0.90	0.79	2.93	103.75	0.85	140	525	750	234.84	233.19	1.18%	1.21	2.73	0.85	12.18
	D4	D5	0.96	0.90	0.86	3.80	106.31	1.12	88	900	900	232.65	231.77	1.00%	1.81	2.85	0.52	11.84
	D5	Outlet	0.00	0.90	0.00	3.80	108.41	1.14	47	900	900	231.70	231.00	1.49%	2.21	3.47	0.23	11.55

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC												Pipe (min)	Total (min)
																		10
	E0	E1	0.76	0.90	0.68	0.68	121.93	0.23	53	300	525	238.78	238.49	0.54%	0.32	1.46	0.61	10.61
@ North Park Dr.	E1	E2	0.33	0.90	0.30	0.98	117.04	0.32	84	375	600	238.40	237.99	0.49%	0.43	1.51	0.92	11.53
	E2	E3	0.23	0.90	0.21	1.19	110.43	0.37	76	450	600	237.89	237.55	0.45%	0.41	1.45	0.88	12.41
	E3	E4	0.30	0.90	0.27	1.46	104.96	0.42	76	450	600	237.49	237.10	0.51%	0.44	1.56	0.81	13.22
	E4	E5	0.30	0.90	0.27	1.73	100.43	0.48	76	525	675	237.02	236.61	0.54%	0.62	1.72	0.74	13.96
@ Northcliffe / Moregate	E5	E6	0.32	0.90	0.29	2.02	96.70	0.54	67	525	675	236.60	236.19	0.61%	0.66	1.84	0.60	14.56
	E6	E7	0.36	0.90	0.33	2.35	93.89	0.61	94	525	675	236.17	235.53	0.69%	0.70	1.95	0.80	15.37
	E7	E8	0.32	0.90	0.29	2.64	90.46	0.66	80	525	750	235.40	234.92	0.60%	0.86	1.95	0.68	16.05
	E8	E9	0.32	0.90	0.29	2.93	87.77	0.71	80	525	750	233.93	233.16	0.96%	1.09	2.46	0.54	16.59
	E9	outlet	0.00	0.00	0	2.93	85.77	0.70	40	525	750	233.00	232.50	1.25%	1.24	2.82	0.24	16.83

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC												Pipe (min)	Total (min)
																		10
S. of North Park Dr.	F0	F1	0.60	0.90	0.54	0.54	121.93	0.18	76	300	375	238.19	236.18	2.65%	0.29	2.59	0.49	10.49
	F1	F2	0.36	0.90	0.33	0.86	117.96	0.28	77	300	450	236.02	234.11	2.49%	0.45	2.83	0.45	10.94
N. of Mansion	F2	F3	0.52	0.90	0.47	1.33	114.56	0.42	69	375	450	234.04	232.25	2.60%	0.46	2.89	0.40	11.34
S. of Mansion	F3	F4	0.39	0.90	0.35	1.68	121.93	0.57	84	375	525	232.23	230.73	1.79%	0.58	2.66	0.52	11.86
	F4	F5	0.44	0.90	0.40	2.08	111.76	0.65	91	450	600	230.71	228.99	1.89%	0.84	2.98	0.51	12.37
	F5	F6	0.53	0.90	0.47	2.55	105.18	0.75	84	450	600	228.95	227.55	1.67%	0.79	2.81	0.50	12.87
	G0	G1	0.60	0.90	0.54	0.54	121.93	0.18	79	300	450	229.74	228.96	0.99%	0.28	1.78	0.74	10.74
S. of Williams	G1	F6	0.50	0.90	0.45	0.99	116.05	0.32	81	375	525	228.96	228.17	0.98%	0.42	1.96	0.69	11.43

Location			Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (%)	Pipe Capacity	Velocity (m/s)	Time	
Street	MH From	MH To	Area (ha)	C	AC												Pipe (min)	Total (min)
																		10.00
S. of Lascelles Blvd.	H0	H2	0.73	0.90	0.66	0.66	121.93	0.22	67	375	450	229.67	228.98	1.03%	0.29	1.82	0.61	10.61
CrescentHill Dr. N	H1	H2	0.53	0.90	0.48	0.48	121.93	0.16	80	375	375	230.75	229.16	1.99%	0.25	2.24	0.60	0.60
	H2	H3	0.36	0.90	0.33	1.46	117.01	0.47	92	375	525	228.93	227.39	1.68%	0.56	2.57	0.59	11.20
	H3	H5	0.29	0.90	0.26	1.72	112.67	0.54	68	450	525	226.68	225.39	1.90%	0.59	2.74	0.41	11.62
CrescentHill Dr. S	H4	H5	0.99	0.90	0.89	0.89	121.93	0.30	74	450	450			1.80%	0.38	2.41	0.51	10.51
	H5	H6	0.31	0.90	0.28	2.89	109.86	0.88	103	525	675	225.23	224.03	1.17%	0.91	2.54	0.68	12.29
	H6	H7	0.45	0.90	0.40	3.29	105.63	0.97	100	525	675	224.03	222.34	1.69%	1.09	3.06	0.54	12.84
	H7	H8	1.01	0.90	0.91	4.20	102.51	1.20	152	525	750	222.33	219.30	1.99%	1.57	3.55	0.71	13.55
	H8	H9	0.69	0.90	0.62	4.82	98.72	1.32	152	600	825	219.23	217.09	1.41%	1.70	3.19	0.80	14.35
Culvert on Queen St. E	H9	Outlet	0.80	0.90	0.72	5.54	94.88	1.46	152	600	825	217.06	214.93	1.40%	1.70	3.18	0.80	15.14
Lascelles Blvd.	I0	I1	0.72	0.90	0.65	0.65	121.93	0.22	76	375	375			2.00%	0.25	2.25	0.56	10.56

AECOM

Region of Peel

Dixie Road Class Environmental Assessment Stormwater Management Report Addendum: North of Mayfield Road

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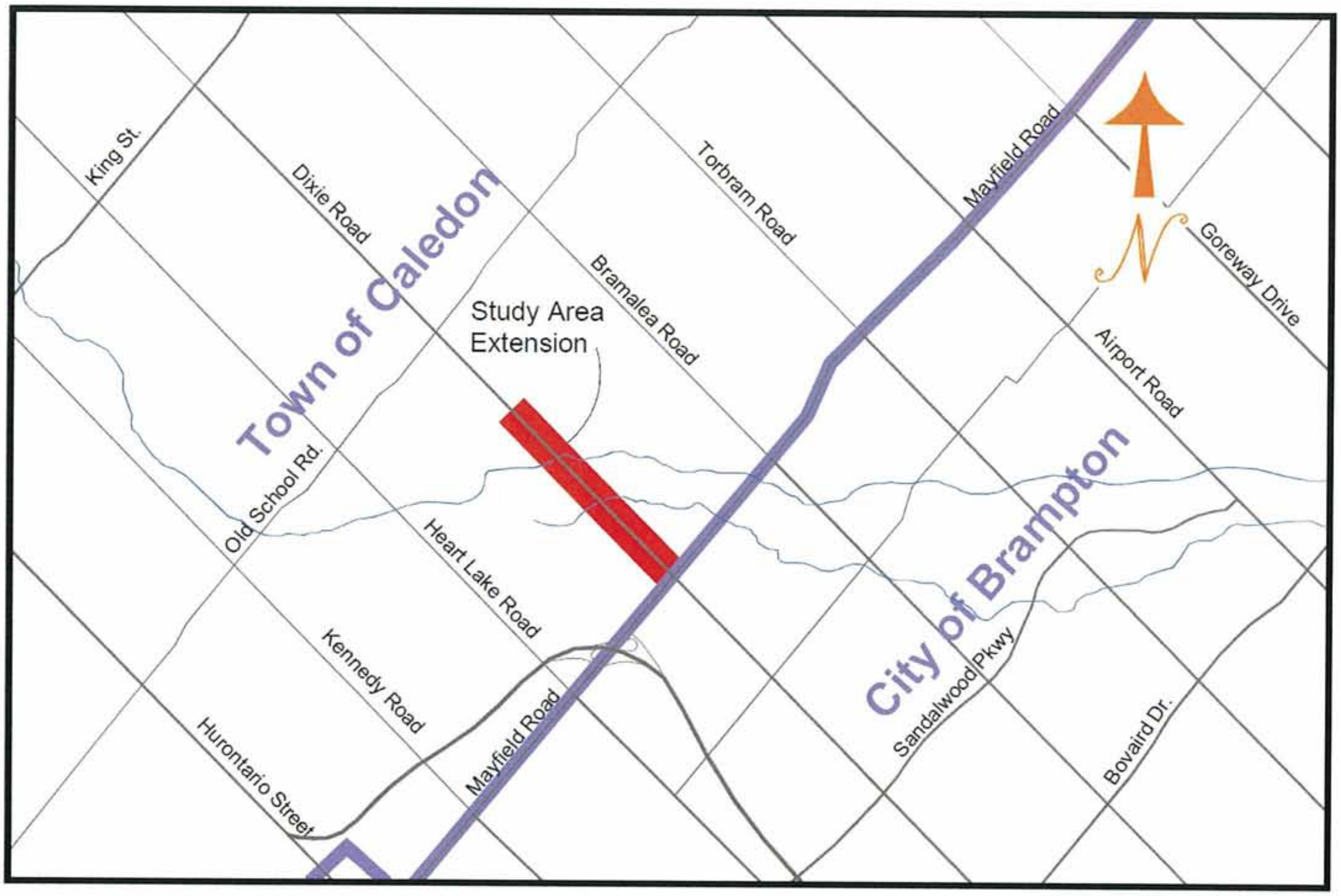
1. Introduction

AECOM was retained to complete the Environmental Assessment (EA) for the Dixie Road Expansion project for the Region of Peel. The original study area was from Queen Street East to Mayfield Road within the City of Brampton. The scope of the project has been extended to 2.1 km north of Mayfield Road within the Town of Caledon. See Figure 1 for an illustration of the project study area extension. The extended study area drains to the Humber River (West Branch) as well as Etobicoke Creek, both within the jurisdiction of the Toronto Region Conservation Authority (TRCA).

Figure 1. Additional Study Area



A stormwater management report was previously prepared by AECOM for the original study area. This report addresses only the extended study area north of Mayfield Road. The existing drainage conditions will be reviewed, and the proposed alternatives to address stormwater management and flooding impacts will be evaluated.

2. Existing Conditions

2.1 Land Use

The existing land use of the study area is considered agricultural with some rural residences and farms along Dixie Road. The lands west of Dixie Road and north of Mayfield Road are within the Mayfield West Secondary Plan. Planning is underway for a proposed industrial development fronting onto Dixie Road within the subject study area.

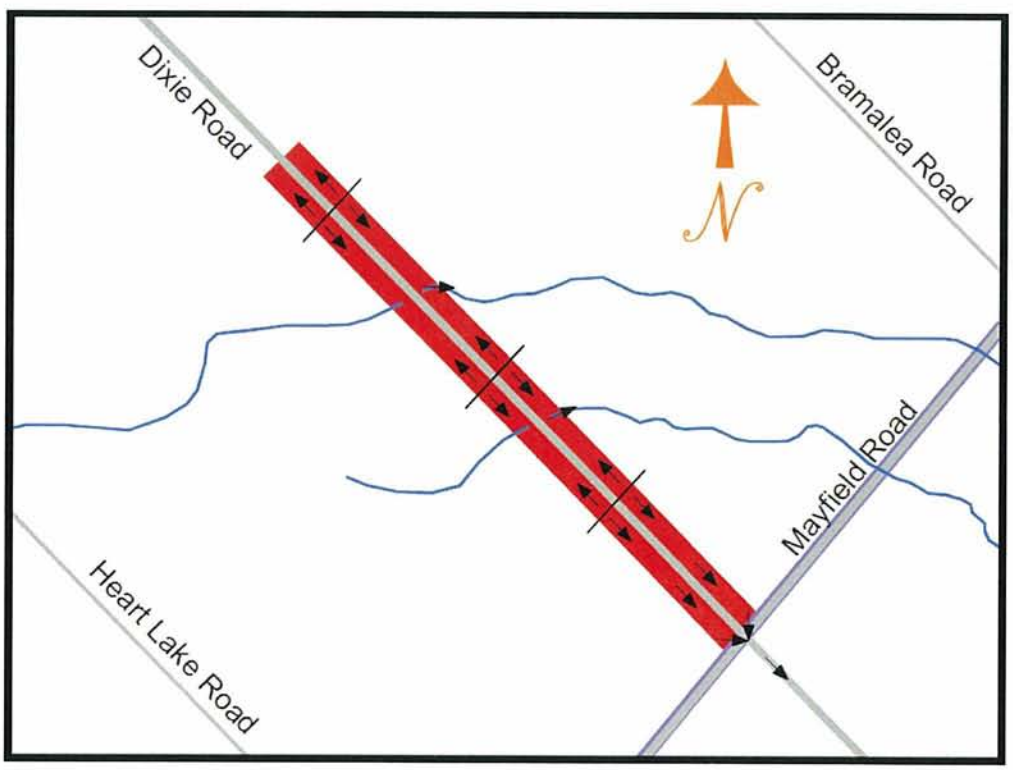
2.2 Soils

According to the Soils Survey of Peel County the soils within the study area are mostly Oneida clay loam which is within Ministry of Transportation's Hydrologic Soils Group D. This indicates minimal infiltration capabilities. The remainder of the study area is Chinguacousy clay loam, which is within Ministry of Transportation's Hydrologic Soil Type C indicating limited infiltration.

2.3 Topography

The study area north of Mayfield Road has three high points along the existing roadway. The first high point is approximately 590 metres north of Mayfield Road. The area south of the high point drains south towards Mayfield Road. The next high point is 540 metres north with a low point at a small tributary crossing between high points. The third high point is 730 metres north of the second and the low point between them is at a regulated watercourse crossing. The third high point is 230 metres from the north limit of the study area. The area north of the third high point will not be expanded and so will not be analysed within this report. A basic sketch of the drainage divides is seen in Figure 2 below.

Figure 2. Existing Topography



2.4 Storm Drainage

Dixie Road north of Mayfield Road has a rural cross-section. Throughout the study area, stormwater runoff is mainly conveyed through ditches alongside the roadway. There are no curbs or storm sewers north of the Mayfield Road intersection.

The drainage area immediately north of Mayfield Road flows south by means of drainage ditches towards Mayfield Road. See Figure 3 for an upstream view of the west roadside ditch. An 800mm diameter CSP culvert beneath Dixie Road on the north side of Mayfield Road (seen in Figure 4) captures flow from both east and west ditches, and is connected to a storm sewer conveying south down Dixie Road. The storm sewer outlets to the existing ditch on the east side of Dixie Road approximately 150m south of Mayfield Road. Farther south on Dixie Road both east and west ditches are captured by a storm sewer system which discharges runoff to a swale on the west side of Dixie Road north of Sandalwood Parkway. The swale conveys runoff south to the stormwater management pond at Bovaird Drive for treatment. Please refer to the AECOM Stormwater Management Report for Dixie Road south of Mayfield Road for further information.

Figure 3. West Ditch facing Upstream

Figure 4. Crossing Culvert

Further north, drainage ditches on both sides of Dixie Road drain to one of two watercourses that cross Dixie Road. The two watercourses that cross Dixie Road north of Mayfield Road flow from west to east and are headwaters for the Humber River. Both of these watercourses are described in more detail in the following section. The small northernmost drainage area flows north to a watercourse crossing outside of the study area, but which also flows to the Humber River.

2.5 Watercourse Crossings

There are two significant watercourse crossings within the study area. The northern-most crossing, hereafter called Crossing 1, is a large watercourse that is regulated as flood plain by the TRCA. The southern-most crossing did not initially appear on the TRCA's regulation mapping. However, through the review of the studies for the Mayfield West Secondary Plan and for the proposed ProLogis industrial development on the west side of Dixie Road, the TRCA has concluded that the watercourse merits protection, and has preserved the corridor upstream (west) of Dixie Road. This crossing has therefore been included in our analysis, and is referred to as Crossing 2.

There is a third culvert, located approximately 150 m south of Crossing 1. The 900 mm diameter CSP culvert drains a small area west of Dixie Road. Staff at the TRCA have indicated that previous plans for the proposed development west of Dixie Road proposed to divert the area draining to this culvert to a stormwater management pond discharging to Crossing 2. More recently, TRCA staff indicated that revised plans for the proposed development have been submitted that preserve some or all of the drainage to this culvert. Additional investigation of this culvert is recommended at the detailed design stage, when plans and timing for the development west of Dixie Road are better understood.

Crossing 1 crosses Dixie Road at a 90 degree angle through a 4.9 metre by 3.0 metre concrete box culvert approximately 34 metres long. See Figures 5 and 6 below. The existing culvert extends past the roadway on both sides and appears to be in excellent condition. Upstream of the crossing the watercourse meanders through fields and then straightens out as it flows past two ponds where it is surrounded by dense vegetation. Downstream of the crossing it meanders through a field and passes another pond. There is some sparse vegetation just downstream of the crossing.

Figure 5. Crossing Culvert 1

Figure 6. Downstream of Crossing 1

The south watercourse crossing consists of two 1200mm diameter CSP culverts that convey runoff from the west side of Dixie Road to the east side at a skewed angle. The culverts are in good condition, except for the upstream side of one of the culverts which is slightly crushed. See Figures 7 and 8 below. Upstream of the crossing the watercourse flows southeast across an open field and along a roadside ditch to the crossing. Downstream of the crossing the watercourse is more defined and continues to flow southeast through another field. Preliminary plans for the industrial development west of Dixie Road propose to preserve the watercourse in a re-aligned corridor.

Figure 7. Upstream End of Crossing Culvert 2

Figure 8. Crossing Culvert 2

2.5.1 Existing Conditions Modelling

An existing hydraulic model was obtained from the TRCA for Crossing 1. The Regional flow rate in the model at the crossing location is 38.53 m^3/s. The crossing is modelled as a bridge, 4.9 metres wide and 3.0 metres high. The low point of the roadway is set at an elevation of 260.30, which is 1.3 metres above the culvert obvert. The model indicates that the crossing contains the Regional event and meets all other hydraulic design criteria. Output from the existing model can be found in Table 1 below.

Table 1. Existing Hydraulic Model Results for Crossing 1

Storm Event	Flow Rate (m^3/s)	Water Surface Level (m)	Clearance (m)	Freeboard (m)
2-year	3.23	256.95	2.05	3.35
5-year	5.03	257.13	1.87	3.17
10-year	6.34	257.23	1.77	3.07
25-year	8.07	257.34	1.66	2.96
50-year	9.38	257.41	1.59	2.89
100-year	10.74	257.48	1.52	2.82
Regional	38.53	259.40	N/A	0.90

There was no existing hydraulic model for Crossing 2, and so one was created. Existing flowrates were obtained from the TRCA, and culvert and road elevations were determined from the latest survey data. The existing crossing contains two 1200mm diameter CSP culverts and has a Regional peak flowrate of 10.05 cubic metres per second. The road low point is 261.15 which is 0.55m from the culvert obverts. The culvert data was input into a new CulvertMaster analysis to determine if the criteria are met. The model indicates that the 100-year design storm is contained within the crossing culverts. The Regional event overtops the road with a maximum depth of 16cm. Detailed results for major storm events are in the following table.

Table 2. Existing Hydraulic Model Results for Crossing 2

Storm Event	Flow Rate (m^3/s)	Water Surface Level (m)	Clearance (m)	Freeboard (m)
50-year	2.38	260.56	N/A	0.59
100-year	2.78	260.65	N/A	0.50
Regional	10.05	261.31	N/A	N/A

The clearance and freeboard requirements are not met and the road would overtop during a Regional event, therefore the hydraulic design criteria are not met.

3. Proposed Conditions

It is proposed to expand Dixie Road north of Mayfield Road from one lane in each direction to two lanes in each direction with a centre median / turn lane. The roadway will be widened from 8.5 metres to 22 metres to accommodate the additional lanes. The proposed cross-section will have curb and gutter and will also have a sidewalk on either side of Dixie Road within the boulevard. Figure 9 below illustrates the proposed plan for the study area, and Table 3 compares existing and proposed paved areas.

AECOM has been informed that an industrial development is currently being planned on the west side of Dixie Road north of Mayfield Road. This large industrial development will require several intersections with Dixie Road, as illustrated in Figure 9 below. Stormwater management ponds are also proposed to manage the runoff from the industrial development. There is some potential for the stormwater management ponds for the proposed industrial development to also treat the runoff from Dixie Road. However, for the purpose of this EA for Dixie Road, an independent solution is required for stormwater management. Regardless, it is recommended the Region work co-operatively with the developer of the lands west of Dixie Road to determine if the stormwater management ponds can be modified to accept runoff from Dixie Road.

Figure 9. Proposed Dixie Road Plan

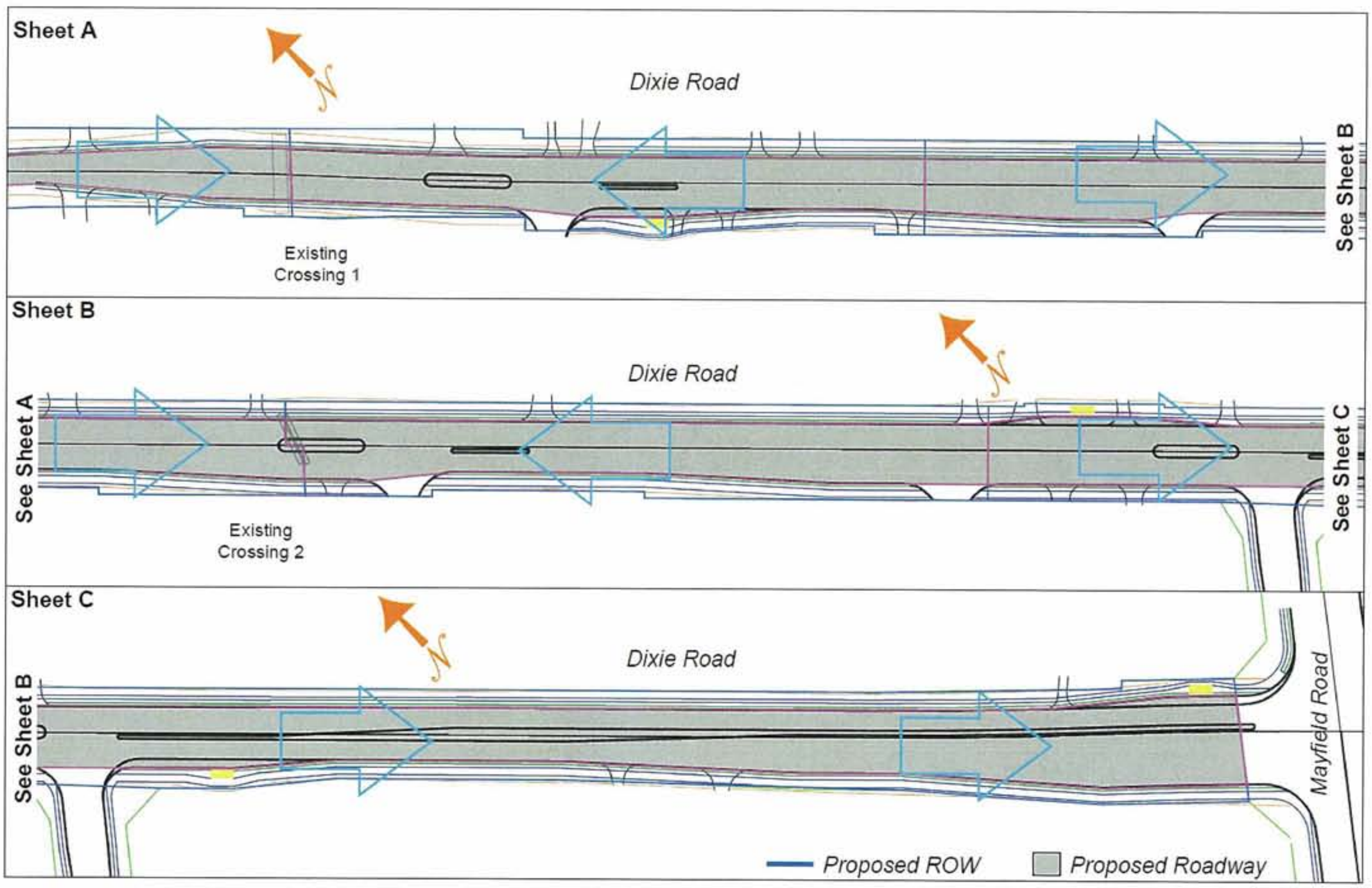


Table 3. Existing vs. Proposed Paved Areas

Section	ROW Drainage Area	Existing Paved	Proposed Paved	% Increase
Drainage Area to Watercourse Crossing 1	2.61	0.69	1.17	18%
Drainage Area to Watercourse Crossing 2	2.04	0.48	1.25	38%
Drainage Area to Mayfield Road	2.32	0.55	1.53	42%

3.1 Stormwater Management Criteria

The TRCA has criteria involving runoff quality, quantity, erosion protection, water balance, and fish habitat protection. Water quality control measures should be implemented for proposed developments, especially those with increased impervious area where vehicular traffic would be increased. Enhanced treatment should be achieved, removing 80% of the total suspended solids (TSS) from the flow.

Water quantity control should also be addressed for proposed works. In most cases, development involves increasing the amount of impervious area and potentially increasing runoff flows. The increased impervious area and resulting peak flow rates should be evaluated and compared to existing. If a significant increase is likely to occur then measures need to be implemented to prevent increased flows. The Humber River Unit Rates must be used to determine allowable release rates for watercourses within the Humber River watershed.

There should be no increase in erosion downstream of the watercourse crossing after development. The extended detention of the runoff from a 25mm storm event is a common erosion control measure to protect the downstream watercourse. It is also important to evaluate water balance to preserve groundwater recharge to the extent possible.

Fish habitats also need to be protected from increased pollutants, erosion and temperature changes. The Humber River tributaries that cross the study are managed for redside dace and so increases in runoff temperature should be avoided.

The current Region of Peel standards state that the storm sewers should be based on a 10-year design storm. The Town of Caledon guidelines express that a minimum 10 minute time of concentration shall be used as an inlet time for storm sewers. For external areas proposed for development, the time of concentration shall be based on the sum of a 10 minutes inlet time for the upstream 50 metres of a direct path to the outlet, and the time resulting from the remaining length traveled at a velocity of 2 m/s. The Rational Method shall be used to design storm sewers. The "Caledon Development Standards, Guidelines and Policies – 2006" should be referenced to obtain all criteria.

There are also design criteria for watercourse crossings. The Town of Caledon design guidelines state that alterations to a watercourse crossing shall not increase water surface elevations upstream, and the design flood frequency for an arterial road is a 1:100 year event. The Ministry of Transportation Highway Drainage Design Standards are an excellent reference for watercourse crossings and state that freeboard and clearance should be at least 1.0 m for arterial roads for the design storm.

3.2 Stormwater Management Alternatives

A wide range of best management practices are available to mitigate the impacts of road runoff on receiving watercourses. These are generally classified into source, conveyance and end-of-pipe treatment alternatives.

Source Control
Source control measures address precipitation where it falls. Common source control measures include roof top storage (including green roofs), cisterns for rainwater harvesting, soak-away pits, and permeable pavement. There are few, if any, source control practices that could be applied to the proposed Dixie Road. Permeable pavement is better suited to laneways and parking areas with lower traffic volumes and few heavy vehicles. It is not intended for urban arterial roadways where there are larger traffic volumes, heavy vehicles and higher potential for groundwater contamination (i.e. tanker truck spills). Neither permeable pavement nor any other source control measures have been carried forward for evaluation as a treatment alternative for the Dixie Road study area.

Conveyance Control
Conveyance control measures manage the quality and quantity of storm runoff as it travels from where it falls to the system outlet. There are several conveyance control measures that can be applied to transportation infrastructure, given the relatively long, narrow configuration of transportation corridors.

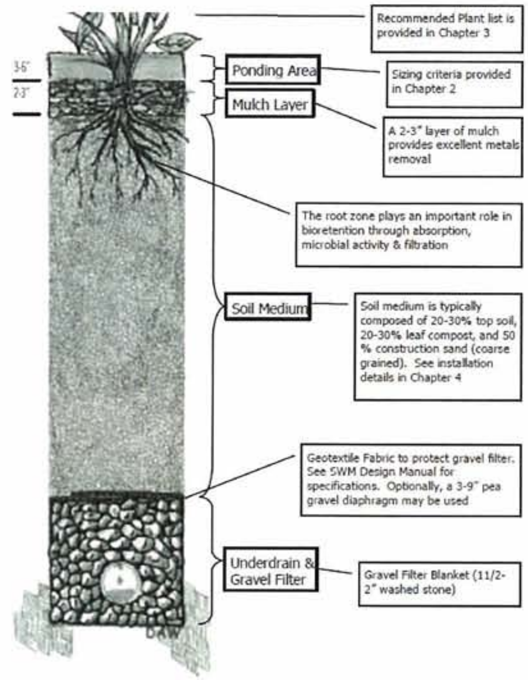
Potential conveyance control stormwater management alternatives for the Dixie Road study area include the following:

<u>Enhanced Vegetated Swales:</u> Where property is available, enhanced vegetated swales can be constructed along one or both sides of the roadway. The swales are typically constructed with a wide, flat base to reduce flow velocity and maximize infiltration, filtration through vegetation and nutrient uptake by vegetation. Stormwater treatment can be further enhanced by adding rock check dams to slow and/or pond water in the swales, soil amendments to improve growing conditions and hold water for infiltration/evapotranspiration, and granular trenches below the swale to promote infiltration and cool stormwater before it is released to the receiving drainage course.

<u>Bioretention Swales</u>: Bioretention swales are specifically designed to filter stormwater and promote infiltration and evaporation. A typical cross section through a bioretention swale is shown below. It generally consists of a surface swale to accept runoff from adjacent paved areas, which is underlain by a layer of planting soil. Stormwater passing through the planting soil will drain down through a layer of sandy soil, and a sub-drain is generally provided at the base of the trench to collect the filtered runoff. A cross section for a typical bioretention swale is included as Figure 10.

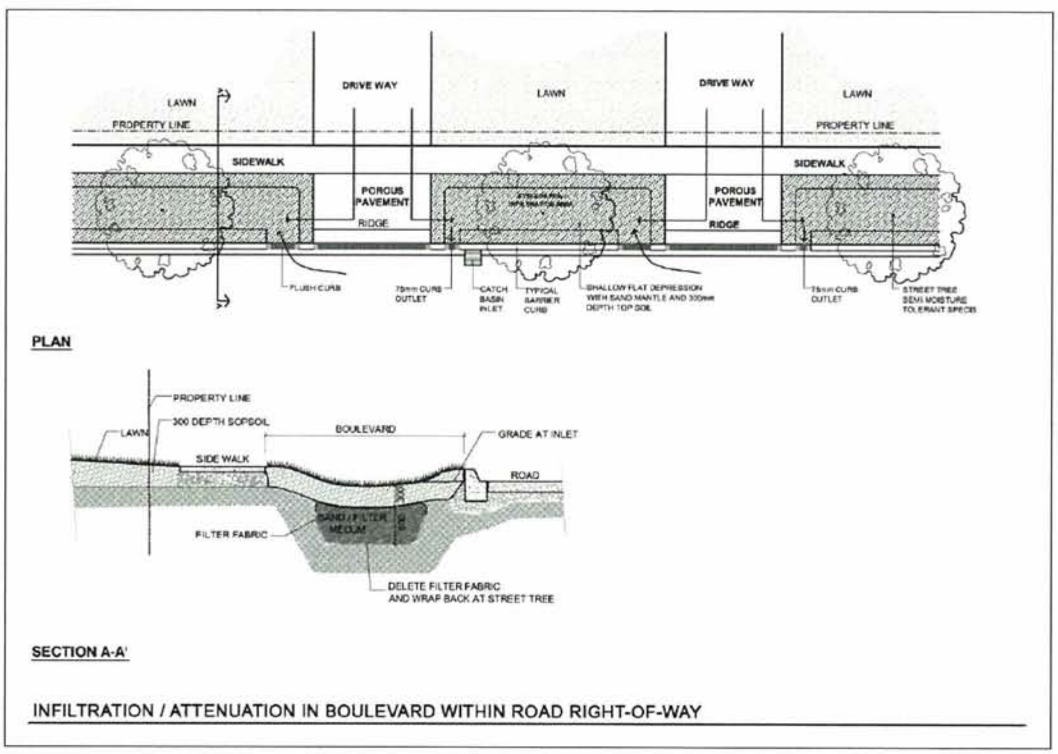
<u>Bioretention Depressions</u>: Bioretention depressions are similar in form to bioretention swales, but are in the form of local depression areas at the edge of the road instead of a continuous drainage element. The bioretention depressions replace a typical catchbasin at the edge of the curb on roads with an urban cross section. Runoff enters a depression in the boulevard (between the roadway and the sidewalk) by means of a cut in the curb. Water then ponds in the depression until it reaches the top of a raised catchbasin in the depression, or flows back onto the roadway through a second curb cut. The depressed area includes vegetation over a granular medium that holds runoff for infiltration and evapotranspiration. An example is illustrated in Figure 11.

Figure 10. Typical Bioretention Swale Section



Adopted from 'The Bioretention Manual', Prince George's County, Maryland

Figure 11. Bioretention Depression in the Boulevard



Taken from the Low Impact Development Manual (Aquafor Beech, 2008)

Pervious Pipe (Exfiltration) Systems: These systems are constructed in place of or in addition to a traditional storm sewer system. Exfiltration systems include a perforated pipe within a stone trench under the roadway. Catchbasins are connected to the perforated pipe, and road runoff infiltrates into the soil surrounding the trench during and following rainfall events. A relief storm sewer conveys runoff from larger storm events to the outlet once the exfiltration trench is full. Exfiltration systems are generally not appropriate for heavily travelled urban arterials. Extensive pre-treatment is required to prevent the system from clogging, and there is an increased risk of contamination from spills and polluted runoff. Pervious pipe systems are not recommended for the Dixie Road study area.

End of Pipe Control

The final alternatives explored for the Dixie Road study area are end-of-pipe measures.

Stormwater Management Ponds: Stormwater management ponds are a commonly applied end-of-pipe measure to manage runoff from new development areas. These are large ponds that contain a permanent pool of water to slowly settle out fine sediments, and an active storage zone above the permanent pool to store and slowly release the runoff from moderate to large storm events. The active storage zone typically includes an extended detention zone to hold the runoff from small storm events and slowly discharge it over a period of 24 to 48 hours to improve water quality and reduce erosion in the receiving watercourse. The remainder of the active storage zone attenuates peak flows from larger storm events and controls the discharge from the pond to prevent downstream flooding.

Stormwater management ponds typically require a drainage area of 5 ha or more to be effective, and require a significant block of property beyond the road right of way. The Dixie Road study area is bisected by two watercourses. There are no sections of roadway in the study area that will drain a total area of more than 5 ha to the receiving drainage courses. Stormwater Management Ponds are therefore not recommended for the Dixie Road study area.

It may be possible to work cooperatively with the landowner on the west side of Dixie Road to discharge runoff from Dixie Road to the centralized stormwater management ponds for the future development. However, at this time, a standalone stormwater management plan is required for the Dixie Road EA project.

Oil-Grit Separators: These treatment devices are often installed to treat storm runoff where space is not available for a stormwater management pond, and in combination with other treatment measures to make up a treatment train approach. Oil grit separators can be installed on-line on the storm sewer system, and can remove both sediments and floatables such as oil from storm runoff. However, oil-grit separators do not have a significant storage volume, and are unable to reduce flow rates to mitigate potential flooding and erosion impacts.

3.3 Assessment of Mitigation Measures

The feasible stormwater management alternatives described above will be applied to the study area based on local opportunities and constraints.

Enhanced Vegetated Swales

Since the study area is within a rural, currently undeveloped area there are few constraints for solutions such as vegetated swales. There are some residences along both sides of Dixie Road, and an industrial development proposed on the west side of Dixie Road. The buildings that are along the study area are set back quite a ways from the road, and so it would be possible to purchase a narrow easement outside of the current property limits for vegetated swales along the majority of the roadway. The roadway would be designed to have an urban cross-section with a curb and gutter. Catchbasins or curb cuts would capture road runoff and discharge directly into the adjacent swale(s). The swales would treat water quality through filtration, and nutrient uptake as well as providing some quantity control by reducing the velocity of the runoff due to the vegetation.

According to the *Low Impact Development Stormwater Management Planning and Design Guide (LID Guide)*, vegetated swales could be able to remove approximately 76% of total suspended solids and reduce flowrates via infiltration by 10%. This would benefit groundwater recharge as well as minimize erosion. These statistics would depend on many conditions, and would be increased if pre-treatment or the implementation of check-dams was applied. The addition on check-dams would facilitate increased infiltration by ponding runoff. If properly designed, ponding should only be for a 24-hour period after a storm. Purchasing a narrow easement along the majority of the study area could get quite costly. Also to maintain these swales regular mowing would be required as well as occasional sediment removal. If these swales were to be adjacent to private property, landowners would have to be educated on regular maintenance.

Bioretention

Through the majority of the study area, a median will separate the two directions of traffic. A bioretention swale can be constructed within the median to control storm runoff from a portion of the roadway. The inside lanes can be sloped to drain to the centre median, where cuts in the median curb can allow runoff to enter a bioretention swale in the median. During small storm events, runoff would drain vertically through the swale to be collected in the sub-drain. The sub drain will outlet either directly or indirectly to a watercourse. During large storm events, runoff would flow along the surface of the bioretention swale to ditch inlet catchbasins connected to the storm sewer system.

Along Dixie Road the median tapers at times to allow for turn lanes and breaks at road intersections. Bioretention swales will not be possible near intersections, where the road cross section must slope to the outside to match the intersecting road profile.

It may also be possible to construct a bioretention swale or bioretention depressions between the sidewalk and curb. This would allow the swales to be within the road right-of-way but not be affected by the limitations of the median. At each catchbasin location a curb cut would allow road runoff to flow into a small bioretention swale. Beneath the swale would be a sub-drain to convey treated runoff to the outfall. In large storm events where the swale and sub-drain would be overwhelmed, runoff would spill into the catchbasin and flow through a typical storm sewer to the outfall and may also be conveyed along the roadway. The main concern with this location for bioretention swales is that utilities are usually located within the road boulevard and may conflict with the swale and subdrain.

This type of treatment would be constructed within the road right-of-way, not requiring additional property. The LID Guide states that bio-retention swales remove 55-100% of total suspended solids (based on previous studies), and reduce peak flows for 45% in swales with sub-drains via infiltration. Another benefit to this type of treatment is that by conveying flows through the sub-drain and surrounding material the stormwater temperature will remain cool to the outlet.

The width of the median on Dixie Road does not remain constant, and is quite narrow along turn lanes and may not be wide enough to construct a bioretention swale. A bioretention swale between the curb and sidewalk may conflict with utilities that are usually located within the boulevard. Also, these swales require maintenance to maintain vegetation and check for clogging.

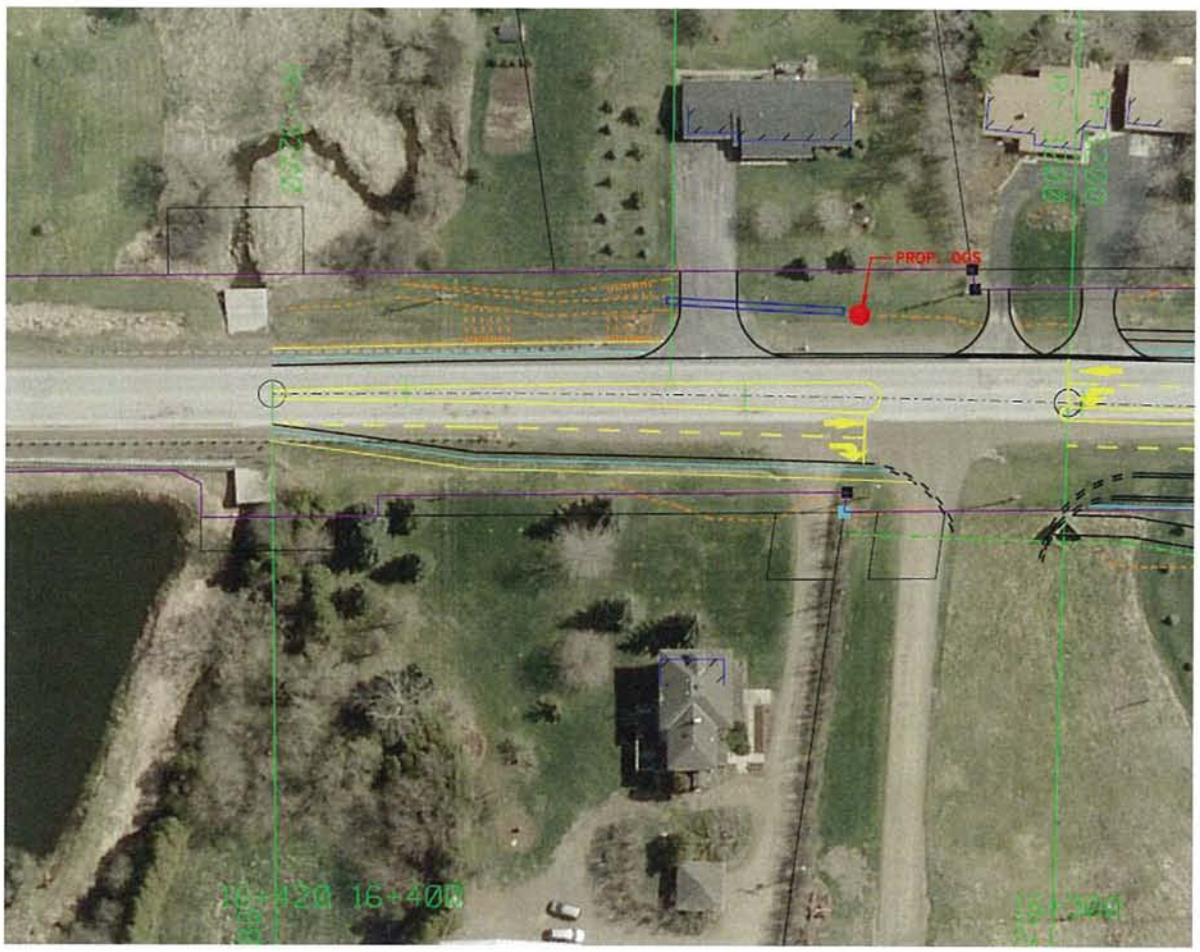
Oil-Grit Separators

In cases where no feasible alternative will independently manage storm runoff from the proposed widened roadway, it would be possible to use oil-grit separators within a treatment train approach where more than one quality control measure is used in series to fully treat road runoff.

3.3.1 Drainage Area to Watercourse Crossing 1

It is proposed to have storm sewers capture and convey flow from this drainage area to a treatment system. Of the few possible mitigation measures, the preferred solution for this area is to treat runoff with an oil-grit separator and then have it flow through an enhanced vegetated swale for additional treatment prior to flowing into the receiving watercourse. This treatment system would be constructed on the south side of the crossing. The swale would have a 2m base width and side slopes as steep as 2:1 to match surrounding ground. Ideally there would be a continuous swale along the roadway to treat roadway runoff; however, for this study area there are residences along the roadway, and so to minimize costs and impacts to existing residents the proposed swales will act more as an end of pipe treatment measure rather than a conveyance measure. The shorter vegetated swale will not provide as much quality or quantity control as a continuous swale. The oil-grit separator will pre-treat the runoff prior to flows discharging into the swales, and so the combination of treatment measures will provide greater quality treatment than a single measure. Also, the vegetated swale may not require much long-term maintenance compared to a bio-retention swale. See Figure 12 for a schematic of the proposed solution.

Figure 12. Proposed Swale near Crossing 1

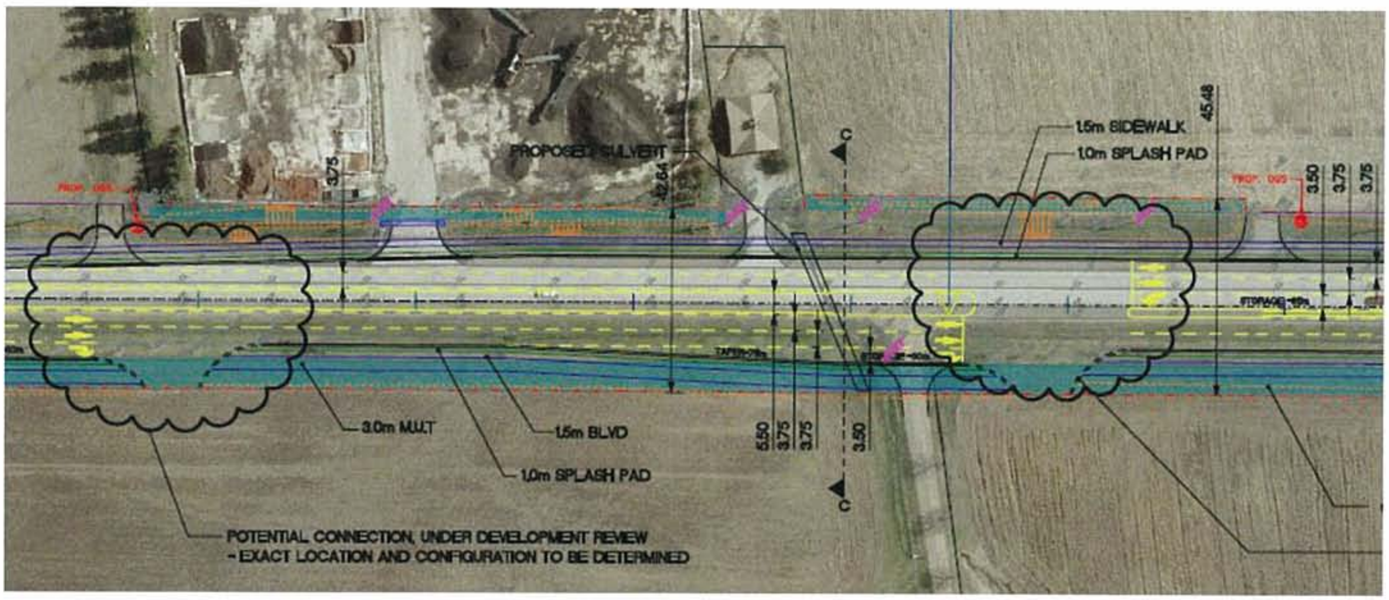


As stated in Table 3, under proposed conditions there will be an additional 0.48 ha of impervious area within this drainage area. The total drainage area to the watercourse upstream of the crossing is approximately 1300 ha. The increase in impervious area represents less than 0.05% of the total drainage area to Crossing 1, and so the improvements to Dixie Road will not impact peak flowrates or water surface elevations at Crossing 1. The proposed vegetated swales will cause flows to spread out along the base of the swale, which will cause the velocity of the flow to decrease. Despite the relatively short length of the swale compared to the drainage area, some reduction in peak flow will occur, although none is warranted.

3.3.2 Drainage Area to Watercourse Crossing 2

The same treatment system determined for the drainage area to watercourse Crossing 1 would be applied to the drainage area to watercourse Crossing 2. Storm sewers would convey flow to vegetated swales on the east side of Dixie Road after being pre-treated by oil-grit separators. The combination of oil-grit separators and vegetated swales will provide adequate quality treatment for the drainage area. See Figure 13 for a schematic of the proposed solution.

Figure 13. Proposed Swales near Crossing 2



The increase in impervious area, as stated in Table 3, is 0.77 ha. The existing drainage area to the watercourse crossing is greater than 40 ha. The increase in impervious area represents less than 2% of the total drainage area to the crossing location. This insignificant increase will not impact water surface elevations and so quantity control will not be required.

Staff at the TRCA have indicated that the proposed development on the west side of Dixie Road may alter the total drainage area to this watercourse. During detailed design, the latest plans for the proposed development should be obtained and reviewed to assess the cumulative impact of the road widening and upstream development on flooding and erosion in the downstream watercourse.

3.3.3 Drainage Area North of Mayfield

The drainage area just north of Mayfield Road is the largest within the study area. This area flows south and eventually drains to the stormwater management pond at Dixie Road and Bovaird Drive. Since there are no protected watercourses between the study area and the pond, it can be assumed that quality control will take place in the Dixie-Bovaird pond. Since quantity control will be addressed at the stormwater management pond, no other quantity treatment is required. Minor runoff will be conveyed through storm sewers which will be sized according to the proposed minor peak flowrates from the area. At the detailed design stage it will be determined which the most cost-effective solution is; implimenting a new single storm sewer system, or addition a parallel storm sewer to the existing one. Major runoff will flow overland south along Dixie Road to the stormwater management pond at Bovaird Drive.

The stormwater management pond at Dixie Road and Bovaird Drive was designed to service a 740 ha development with an average percent impervious of 45%. Including external areas, the total drainage area to the pond is 1350 ha. The additional 2.2 hectares of impervious area proposed for Dixie Road north of Mayfield represents less than 0.2% of the total drainage area. It can be concluded that pond at Dixie Road and Bovaird Drive will have capacity for the additional impervious area and be able to adequately treat additional runoff.

Refer to Drawing 3 for a concept plan and Appendix A for design sheets for the proposed storm sewer system to the outlet north of Sandalwood Parkway.

3.4 Watercourse Crossings

3.4.1 Crossing 1

The existing box culvert at Watercourse Crossing 1 spans beyond the current roadway and will likely not require an extension to allow for the road widening. The proposed additional impervious area is 1.21 ha compared to the total 1300 ha drainage area to the watercourse at the crossing location. Since the peak flowrates will not be impacted by the widened roadway, the hydraulic model will remain the same. Flows are contained within the crossing, so the slight change in roadway profile will not have any impact on water surface elevations. It can therefore be assumed that the existing crossing culvert will satisfy proposed conditions and so no modelling was required for proposed conditions.

3.4.2 Crossing 2

The existing CulvertMaster file was updated for proposed conditions. The existing culverts were extended in the model and the proposed road profile was input. The proposed road low point is 261.31m, 16cm higher than existing. As previously stated, the proposed flows can be considered the same as existing. The results indicate that the proposed crossing contains the 100-year event but the Regional event continues to overtop the roadway with a maximum depth of 16cm. Clearance and freeboard criteria are also not met. The culvert extension would impact upstream water surface elevations and so the culvert is recommended for replacement.

Considering the proposed low point, there are some limitations as to the size of the proposed culvert. The maximum depth of culvert possible, with minimum cover, would be 1.2 m. A 3600 x 1200mm box culvert was input into the CulvertMaster model. The results indicate that the Regional flow would be contained within the crossing; however, the clearance criterion still is not met. Considering the limited vertical distance between the watercourse inverts (259.6 m) and the proposed road low point (261.3 m) it is not feasible to meet clearance and freeboard requirements. This proposed design reduces upstream water surface elevations and provides safe passage during a Regional storm. More details can be found in Table 4.

Table 4. Proposed Hydraulics at Crossing 2 – 3.6 m x 1.2 m Concrete Box Culvert

Storm Event	Flow Rate (m^3/s)	Existing Water Surface Level (m)	Proposed Water Surface Elevation (m)	Clearance (m)	Freeboard (m)
50-year	2.38	260.56	260.22	0.57	1.09
100-year	2.78	260.65	260.29	0.50	1.02
Regional	10.05	261.31	261.27	N/A	0.04

The TRCA has indicated that a HEC-RAS analysis of Crossing 2 has been prepared to support the proposed development west of Dixie Road, and is currently under review. At the detailed design stage, the latest HEC-RAS hydraulic model for this watercourse should be obtained from the TRCA and updated to confirm that the recommended culvert will be adequate.

The 900 mm diameter culvert south of Crossing 1 has been excluded from the analysis due to the small upstream drainage area and the potential for the drainage area to be reduced or eliminated when the area west of Dixie Road is developed. Additional investigation is recommended at the detailed design stage, when plans for the upstream drainage area should be finalized. Inlet improvements, such as a mitred inlet or standard pre-fabricated end section will be sufficient to mitigate any potential impact on upstream water levels due to the small extension that may be required for the proposed roadway widening.

4. Summary & Conclusion

The Region of Peel has retained AECOM to complete an environmental assessment for improvements to Dixie Road from Queen Street in Brampton to north of Mayfield Road in Caledon. A stormwater management report was previously prepared for the original study area south of Mayfield Road. This study has addressed stormwater management for the extended study area north of Mayfield Road.

The preferred alternative to manage future traffic volumes is to widen Dixie Road from its existing 2 lane rural configuration to a 4 lane urban configuration with a continuous center turning lane/median. The proposed widening of Dixie Road has the potential to impact the quality and quantity of runoff delivered to the receiving watercourses. An extension is also required to a significant culvert crossing under Dixie Road, which has the potential to impact upstream flooding.

An evaluation of stormwater management alternatives to mitigate these potential impacts was completed. The preferred solution is described below, and will be further refined during the detailed design stage.

Crossing 1 – West Humber River Tributary:
An enhanced vegetated swale is recommended to manage the runoff from the section of Dixie Road discharging to the northernmost culvert crossing in the study area. The vegetated swale will not be continuous through the entire section, but will instead be limited to a relatively short length between the proposed storm sewer outfalls and the watercourse. An oil-grit separator is recommended as part of the treatment train approach to quality control for this section. The small additional pavement area will have a negligible impact on peak flow rates in the watercourse, and no alterations are required to accommodate the limited widening of the road platform at this location.

Crossing 2 – West Humber River Tributary:
Enhanced vegetated swales are also recommended to manage the runoff from the section of Dixie Road discharging to the southern culvert crossing in the study area. The vegetated swales will not be continuous through the entire section, but will instead be limited to relatively short lengths between the proposed storm sewer outfalls and the watercourse. Oil-grit separators are recommended as part of the treatment train approach to quality control for this section.

The existing twin 1200 mm diameter CSP culverts are undersized relative to the Regional storm peak flow at the crossing, and must be replaced to prevent upstream flood impacts. A 3.6 m wide x 1.2 m high concrete box culvert is recommended to prevent upstream impacts and provide safe passage during a Regional storm event.

Mayfield Road Drainage Area:
No water quality or quantity control measures are warranted for this section, as drainage from this area eventually reaches the large, on-line stormwater management pond near Dixie Road and Bovaird Drive.

A storm sewer is proposed to convey minor runoff from north of Mayfield Road to the crossing culvert north of Sandalwood Parkway. At the detailed design stage a cost-benefit comparison should be done for the implementation of a new storm sewer, or the addition of a parallel storm sewer to the existing one. Major runoff is proposed to flow overland along Dixie Road to the stormwater management pond at Bovaird Drive.

It is also recommended that the Region co-ordinate future design phases for Dixie Road with the development of the industrial lands on the west side of Dixie Road. It may be possible to work co-operatively with the developer(s) to have the runoff from Dixie Road treated in the stormwater management ponds required for the adjacent development. This would eliminate the need for the recommended enhanced vegetated swales, oil-grit separators and underground storage.

AECOM

AECOM Canada Ltd.
Stormwater Management
Dixie Road Class Environmental Assessment

Proposed Conditions
Storm Sewers
Drawing 1

Storm Sewer Design Sheets - Proposed

Location				Runoff			Accum AC	10 yr Intensity (mm/hr)	Flow (cms)	Length of Pipe (m)	Ex. Diam (mm)	Pipe Diam (mm)	Invert u/s (m)	Invert d/s (m)	Pipe Slope (m/m)	Fall in metres	Pipe Capacity	Velocity (m/s)	Time	
Street		MH From	MH To	Area (ha)	C	AC													Pipe (min)	Total (min)
																				10
External Area (not incl.)				12.12	0.25	3.03	3.03	21.96	0.18											117.8
	15469	Y0	Y1	1.05	0.90	0.95	0.95	121.93	0.32	120		600	258.54	257.92	0.51%		0.44	1.55	1.29	11.29
	15349	Y1	Y2	0.44	0.90	0.40	1.34	112.08	0.42	120		600	257.17	256.56	0.51%		0.44	1.55	1.29	12.58
	15229	Y2	Y3	0.48	0.90	0.43	1.78	103.96	0.51	120		675	255.81	255.20	0.51%		0.60	1.68	1.19	13.77
	15109	Y3	X0	0.34	0.90	0.31	2.09	97.62	0.57	120		675	254.45	253.84	0.51%		0.60	1.68	1.19	14.96
@ Mayfield Road	14989	X0	X1	0.93	0.90	0.84	2.93	92.14	0.75	120		750	253.09	252.48	0.51%	0.01	0.80	1.80	1.11	16.08
	14869	X1	X2	0.48	0.90	0.43	3.36	87.67	0.82	120		825	251.73	251.11	0.51%	0.01	1.03	1.92	1.04	17.12
	14749	X2	X3	0.42	0.90	0.38	3.74	83.92	0.87	120		825	250.36	249.75	0.51%	0.01	1.03	1.92	1.04	18.16
	14629	X3	X4	0.40	0.90	0.36	4.09	80.54	0.92	120		825	249.14	248.53	0.51%	0.01	1.03	1.92	1.04	19.20
	14509	X4	X5	0.39	0.90	0.35	4.44	77.48	0.96	120		825	248.50	247.89	0.51%	0.01	1.03	1.92	1.04	20.25
	14389	X5	X6	0.38	0.90	0.34	4.78	74.68	0.99	120		825	247.86	247.25	0.51%	0.01	1.03	1.92	1.04	21.29
	14269	X6	A1	0.34	0.90	0.31	5.10	72.12	1.02	120		900	247.17	246.56	0.51%	0.01	1.29	2.03	0.98	22.27
	14149	A1	A2	0.38	0.90	0.34	5.44	69.89	1.06	120	675	900	246.53	245.92	0.51%	0.01	1.29	2.03	0.98	23.26
	14029	A2	A3	0.40	0.90	0.36	5.80	67.82	1.09	120	675	900	245.89	245.27	0.51%	0.01	1.29	2.03	0.98	24.24
	13909	A3	A4	0.14	0.90	0.13	5.92	65.89	1.08	72	675	900	245.24	244.88	0.51%		1.29	2.03	0.59	24.83
@ Countryside Drive	13837	A4	A5	0.91	0.90	0.82	6.74	64.80	1.21	125	675	900	244.85	244.21	0.51%	0.01	1.29	2.03	1.03	25.86
	13712	A5	A6	0.45	0.90	0.40	7.14	63.00	1.25	100	675	900	244.18	243.67	0.51%	0.01	1.29	2.03	0.82	26.68
	13612	A6	A7	0.46	0.90	0.41	7.55	61.65	1.29	125	750	900	243.64	243.00	0.51%	0.01	1.29	2.03	1.03	27.70
	13487	A7	A8	0.57	0.90	0.52	8.07	60.06	1.35	125	750	975	242.93	242.29	0.51%	0.01	1.60	2.14	0.97	28.68
@ Father Tobin Road	13362	A8	A9	0.83	0.90	0.74	8.81	58.63	1.44	121	825	975	242.26	241.64	0.51%	0.01	1.60	2.14	0.94	29.62
	13241	A9	A10	0.64	0.90	0.58	9.39	57.33	1.50	125	825	975	241.61	240.98	0.51%		1.60	2.14	0.97	30.59
Outlets at Crossing Culvert	13116	A10	A11	0.72	0.90	0.65	10.03	56.06	1.56	168	900	1050	240.90	240.05	0.51%	0.01	1.95	2.25	1.24	31.83