APPENDIX C – AECOM'S DRAFT LAND DRAINAGE REPORT

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Technical Memorandum

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CC	Grant Mohr, Andy Nagy		
Subject	Plessis Underpass Land Drainage Study and Other City-Owned Utilities		
From	David Enns		
Date	May 4, 2012	Project Number	60248130 (506)

1. Introduction

In response to a request from the City of Winnipeg Public Works Department, AECOM provided a letter proposal dated January 17, 2012, to examine the impact of the proposed Plessis Underpass on land drainage and other City-owned (wastewater collection and water distribution) utilities in the vicinity of the proposed underpass.

The proposed Plessis Underpass will provide grade separation between the currently at-grade railway crossing between Plessis Road and the CNR Redditt Subdivision railway line. Figure 1 shows the proposed underpass is located on the western fringe of Transcona. The CNR Redditt Subdivision is considered the boundary between North and South Transcona. The Plessis Road crossing is one of two major crossing points between North and South Transcona with the other major crossing located at Ravenhurst Street approximately 4.1 km east of Plessis Road.

Figure 2 shows the preliminary road plan and profile of the Plessis Road portion of the depressed road provided by the Public Works Department. At the future rail bridge, Figure 2 shows that the bottom of the underpass will be approximately 7 m below existing grade.

Figure 3 shows the preliminary road plan and profile of the Dugald Road portion of the depressed road provided by the Public Works Department. Around the intersection with Plessis Road, Figure 3 shows that Dugald Road will be approximately 2.6 m below existing grade.

Figure 4 shows the approximate property requirements for the proposed underpass design. Figure 4 also shows that there are several properties surrounding the proposed underpass that are already owned by the City.

Figure 5 shows the existing water, sewer and land drainage utilities within the proposed underpass limits shaded in magenta. The proposed underpass area shaded in magenta was developed based on the preliminary design of the road network shown in Figure 2 and Figure 3. The area within the magenta shading is the approximate area which will be depressed from existing grade.

Figure 5 shows that there are many existing water, wastewater and land drainage elements within the limits of the proposed underpass including a 750 mm secondary interceptor wastewater sewer, a 450/500 mm diameter watermain, and the Dugald Drain.

All of the utilities shown on Figure 5 will likely require some relocation or remedial protection as discussed in Section 3 for the water distribution system and Section 4 for wastewater collection system. However, the most complicated of the existing City-owned utilities that conflict with the underpass is the Dugald Drain and the general issue of drainage. Due to the generally poor level of drainage service provided by the Dugald Drain and the Deep Pond Drainage system (which currently service the area around the proposed underpass), the drainage system servicing the underpass will need to be configured such that it will not negatively impact either system. There are also few feasible ways to allow the Dugald Drain to continue to function with the underpass in place. The drainage issue will thus be discussed at the greatest length in this document in the next section.

2. Land Drainage

The area around the current at-grade crossing of the CNR Redditt Subdivision and Plessis Road is serviced by the Dugald Drain and the Deep Pond Drainage System as shown on Figure 6. The at-grade crossing straddles the two land drainage service areas. Section 2.1 will describe the drainage patterns in the two districts as they relate to the proposed underpass.

2.1 Existing Drainage Patterns and Future Plans

As shown on Figure 6, the proposed underpass straddles the Deep Pond Land Drainage Area to the north and the Dugald Drain Land Drainage Area to the south of the railway line. Although these drainage areas are defined as two separate districts, the Deep Pond system takes a relatively large amount of runoff from the Dugald Drain due to an interconnecting ditch located along the east side of Plessis Road. This interconnection is located in the proposed underpass area and is shown on Figure 7.

The following sections describe the existing drainage patterns and future plans for the Dugald Drain and Deep Pond Land Drainage Areas.

2.1.1 Deep Pond Land Drainage Area

The 996 Ha Deep Pond Land Drainage District shown on Figure 6 services the northern area of the proposed underpass. About three quarters of the district is fully developed. The southern and eastern parts of the district consist of single family homes in the Mission Gardens, Regent Park and the Kildonan Meadows subdivisions. The western and south central parts of the district are comprised of large commercial areas including Kildonan Place Shopping Centre and various big box stores such as Best Buy and Canadian Tire. The remaining northwest area of the district is currently undeveloped agricultural lands or low density large lot residential properties along Ravelston Avenue West and Almey Avenue near Lagimodiere Boulevard. This land will likely develop over the coming years.

The land drainage sewers servicing the Deep Pond system lead to stormwater retention basin (SRB) 4-7, referred to as the Deep Pond, located in the middle of the district. The Deep Pond is dewatered using the 1.8 m^3/s of total pumping capacity at the Ravelston Land Drainage Pumping Station located at the eastern tip of SRB 4-7 shown by the yellow star on Figure 6. The pumping station is the only means of dewatering SRB 4-7, as it does not have a gravity outlet. The Ravelston Pumping Station discharges to the main trunk of the 907 Ha Kildare Land Drainage Area, located east of the Deep Pond Land Drainage Area. The pumps in the Ravelston Pumping Station are only turned on once the Kildare LDS Trunk is no longer surcharged at Hoka Street. The Kildare LDS Trunk flows from west to east and ultimately discharges to the Red River Floodway.

The Deep Pond Land Drainage System is considered to be problematic relative to other land drainage systems in the City primarily because it does not have a gravity outlet (i.e. the outlet relies on pumps). During the May 28 to 30, 2010 storms about 135 mm of rain fell at Bernie Wolf School, which is in the Deep Pond Land Drainage Area. In addition to the large amount of runoff, the pumps at the Ravelston Pump Station malfunctioned which resulted in very high water levels in the system as well as significant risk of property damage if additional rainfall would have occurred.

In addition to the problems at the Deep Pond, the Kildare LDS Outfall is also problematic during floods. The Kildare LDS system is dewatered during high water levels in the floodway at the Kildare LDS Flood Pump Station. The pump station only has about 2 m^3/s of total flood pumping capacity, which is significantly less than what the Kildare LDS system could generate and about the same capacity as the Ravelston Land Drainage Pump Station at the Deep Pond.

The May 28 to 30 storms showed that there is little extra capacity in the Deep Pond system to accept additional runoff, even though a quarter of the district currently remains undeveloped. The Deep Pond rose about 4 m above the high water level, or 6.2 m above normal water level. The Kildonan Meadows SRB's (4-6 and 4-5) were also well above the high water levels and took a very long time to return to normal water levels. Thus the Water and Waste Department has indicated that no additional runoff should enter the Deep Pond system from the proposed Plessis Underpass.

2.1.2 Dugald Drain Land Drainage Area

The 1178 Ha Dugald Drain Land Drainage District is shown on Figure 6 and is located south of the CNR Redditt Subdivision, east of Lagimodiere Boulevard, north of the CNR Symington Yards and St. Boniface Road, and west of Murdock Road. Figure 8 zooms in on the drainage area and shows the various drainage sub-areas within the district.

The main drainage element in the Dugald Drain Land Drainage District is the Dugald Drain itself. The drain is largely located on the south side of Dugald Road starting at Murdock Road. A plug was placed at this location during development of the South Transcona Lake to ensure runoff east of Murdock Road could not enter the South Transcona area. The drain flows from east to west along the south side of Dugald Road to Dawson Road. At Dawson Road it turns south and runs in behind a number of industries before discharging to the Seine River near the Marion Street Bridge.

The Dugald Drain runs through the Mission Combined Sewer District, but it only accepts drainage from a small strip of land on either side of the drain (i.e. they are not connected) within the limits of the Mission District.

The land use in Dugald Drain catchment area varies throughout the district. East of Plessis Road, the area is largely rural in nature with large lot residential clustered around the small hamlet around McFadden Avenue. The area east of Plessis Road is split into two drainage sub-areas, the 210 Ha

Esselmont Area and the 330 Ha Dugald Area. Prior to about 2001, the Esselmont Area went to the Dugald Drain.

During the summer of 1993 the hamlet in South Transcona experienced significant flooding. The area had suffered from chronic flooding for several decades prior to 1993 but due to additional development occurring without proper stormwater controls in the area, the City decided that something had to be done to improve the drainage.

Shortly after the 1993 flooding, the Esselmont Drain was constructed to relieve the overloaded Dugald Drain. The Esselmont Drain essentially diverted the area south of Esselmont Avenue to the stormwater retention basins (SRB's) in the St. Boniface Industrial Park. Drainage from the area north of Esselmont continues north and goes directly to the Dugald Drain. Although the drainage servicing has improved since the 1993 flooding, the area is still has a generally low level of service.

Between Plessis Road and Lagimodiere Boulevard there is a mix of urban industrial/commercial, single family residential and vacant/agricultural land use. The area is dominated by the St. Boniface Industrial Park (which includes the Waterside Estates residential subdivision), The Waters Business Park and Terracon Place.

The area has an urban style of land drainage system (i.e. closed conduit land drainage sewers leading to SRB's). The SRB's from the three areas are pumped out to the Dugald Drain when levels in the drain allow. Each sub-area shown on Figure 8 have their own pumping system that dewater the SRB's to the Dugald Drain. The St. Boniface Industrial Park pump is owned and operated by the City, whereas the other two are privately owned.

West of Plessis Road, the Dugald Drain directly services only the land that fronts on Dugald Road. The area to the south of the St. Boniface Industrial Park and the GWWD Railway is officially a part of the drainage district, however the drainage on the lands are generally poor with little to no definition. Thus they are likely effectively disconnected from the Dugald Drain for all but the largest of storm events.

The Esselmont Drain, which enters the St. Boniface Industrial Park system near Camiel Sys Street and Plessis Road, can cause high water levels in the St. Boniface Industrial Park SRB's under major storms. Like the Deep Pond, the system does not have a gravity outlet. The St. Boniface Industrial Park SRB's are pumped to the Dugald Drain using a 0.42 m^3/s (15 cfs) pump that is turned on only when levels in the Dugald Drain are low. In many cases this is several days after a storm has occurred.

During the May 28 to 30, 2010 storms the SRB 5-1 was 2.3 m above high water level due to the inflows from the Esselmont Drain. This is approximately only 0.3 m below the low LDS manhole rim in the district. Although very high, the industrial park part of the catchment area is largely made up of slab on grade structures that are not vulnerable to basement flooding (i.e. there are no walk-out basements). It is only after those storms that new home construction has begun in the Waterside Estates subdivision.

Although the areas serviced by the Dugald Drain generally have a low level of service, the drain will be the short term outlet for underpass land drainage system. In the short term the underpass system will be configured like the other developed areas of the district with temporary storage designed to be pumped to the Dugald Drain when levels subside.

The ultimate and fully urbanized land drainage servicing scheme for the area is found on Figure 9 as modeled as a part of the ongoing Mission Combined Sewer Relief Study. The long term plans for the Dugald Drain Land Drainage District is to construct a new gravity land drainage sewer trunk discharging to the Seine River. The currently undeveloped parts of the district will be serviced with new SRB's discharging to the new LDS trunk and the existing SRB's connected to the new trunk. The LDS trunk will also be designed to relieve a large part of the Mission Combined Sewer District by land drainage sewer separation and service the redeveloped Public Markets brownfield site.

The new LDS trunk will make use of a part of the now abandoned Dugald Interceptor from Lagimodiere Boulevard to Mazenod Road. The interceptor was replaced with a new pipe in 2007 to 2009. The old interceptor pipe, although in poor condition, was abandoned such that can be rehabilitated with a liner. Re-lining the existing interceptor represents a significant cost savings over new construction. The abandoned interceptor extends well east of Plessis Road so it could be rehabilitated to Mazenod Road and used as a discharge pipe from the underpass drainage system once the LDS Trunk from the Seine River to Mazenod Road is constructed.

The intention of the new LDS system is to reduce the reliance on the Dugald Drain. The drain itself will likely never be completely abandoned, but its importance as a regional land drainage element will be reduced over time.

2.2 Hydraulic Modelling

InfoWorks CS hydraulic modelling software by Innovyze Inc. was used to examine the performance of various land drainage configurations for the Plessis Underpass. This included a detailed model of the Dugald Drain Land Drainage District and a coarse model of the Deep Pond Land Drainage District.

A future conditions model of the Dugald Drain Land Drainage District was created as a part of the ongoing Mission Combined Sewer Relief study including the land drainage sewer separation scheme for the Mission District itself. The model assumed that most of the district was urbanized (i.e. with underground land drainage sewers) and thus was not representative of existing conditions (i.e. semi-rural with ditches).

For this study, the future conditions model of the Dugald Drain was modified such that it was representative of the existing conditions within the district. This included configuring the privately owned pumps that discharge from the various SRB's between Plessis Road and Lagimodiere Boulevard, reconfiguring some of the ditch sections, and changing some of the catchments to a rural pattern of runoff.

The coarse Deep Pond model was created to examine the impact of different interconnection schemes that send flow from the Dugald Drain into the Deep Pond System. The model was also used to examine the hydraulic grade line (HGL) in the Deep Pond System near the underpass to get an idea of the current level of service and to aid in the reconfiguration of the system north of the railway line should it be required.

The hydraulic model uses typical input parameters for Manning's roughness coefficients, catchment imperviousness, and other parameters. The open channel sections generally use a Manning's roughness coefficient of 0.050, which represents a moderately clean channel. The City of Winnipeg 1974 by MacLaren intensity-duration-frequency curves were used to create the 'Chicago' style hyetographs (rainstorms) in the InfoWorks model. Rural catchments were configured to according to volumes from N. Harden's "Runoff from Small Rural Watersheds" for the Province of Manitoba's Water Resources Branch in May 1983. Peak flows from rural catchments are approximately equal to the Value Added discharge formula.

The summer of 1993 and May 2010 rainstorms were also used with the InfoWorks hydraulic models. During the summer of 1993, two major storms occurred. During the July 24-25 storm event 195.2 mm of rain was recorded at the District 5 Building in St. Vital. During the August 8 storm event, 93.2 mm of rain was recorded at the North End Water Pollution Control Centre. The rainfall recordings from the two major rainstorms and locations were used in conjunction with the Dugald Drain and Deep Pond InfoWorks hydraulic models to examine the reactions of those systems. The May 2010 storms refer to the May 28-30, 2010 period where 135.2 mm of rain was recorded at Bernie Wolfe School which is on the boundary of the Dugald Drain and Deep Pond Land Drainage Districts. Three rainstorms fell during this period.

2.3 Dugald Drain

Although the long-term plans for South Transcona and the St. Boniface Industrial Park ultimately envision the diversion of the area to new land drainage sewers and stormwater retention basins, the Dugald Drain will continue to operate until those elements are constructed. This section examines two options that were investigated as part of this study to provide interim drainage servicing for the lands east of Plessis Road.

The first option, which is the recommended option, would be to construct a siphon for the Dugald Drain under the underpass section. This option has the advantage of allowing the Dugald Drain to continue to function as it is currently configured. However, the siphon does have some additional operation and maintenance complexities.

The second option, examines the diversion of the Dugald Drain to the Esselmont Drain and the SRB's in the St. Boniface Industrial Park. The second option requires the new South Transcona LDS Trunk through the Mission Combined Sewer District as depicted in Figure 9 because the St. Boniface Industrial Park SRB's do not have a gravity outlet and are only pumped by a small dewatering pump. The second option also requires the first SRB east of Plessis Road be constructed. The existing SRB's are already operating at their design range and thus additional inflows without providing outlet improvements and additional storage would stress the system beyond its capacity.

2.3.1 Dugald Drain Siphon under Plessis Underpass (Option 1)

Figure 10 shows the conceptual underpass land drainage configuration including the recommended 1500 mm Dugald Drain siphon in orange. The siphon allows the Dugald Drain to continue to function in its current configuration. To examine the impact of the siphon this section will begin by describing the existing conditions in the Dugald Drain as estimated by the InfoWorks hydraulic model.

2.3.1.1 Existing Conditions

Figure 11 contains the peak HGL's arising from the 5, 25 and 100 year, summer 1993 and May 2010 storms in the Dugald Drain from the Seine River (at left) to Murdock Road (at right) under existing conditions. For the most part the drain itself is within the banks of the channel, however due to a relative lack of grade and some low lying areas, other parts of the ditch systems are flooded for the larger events.

The profile on Figure 11 shows that the existing 1550x900 mm arch pipe under Plessis Road creates a backwater effect upstream of the crossing in the Dugald Drain. The estimated flows for the 5, 25 and 100 year design storms are shown on Figure 12. Figure 13 contains a graph showing the estimated flow in the arch pipe for the summer 1993 events and Figure 14 contains the same information but for the May 2010 storms.

Figure 12 shows that the flows in the Dugald Drain peak at about 1.0 m^3/s (35 cfs) for the 5 year storm, 1.2 m^3/s (42 cfs) for the 25 year storm, and 1.3 m^3/s (46 cfs) for the 100 year storm. Similar peak flows to the 100 year design storm are estimated to occur in the system arising from the summer of 1993 events and May 2010 events.

Figure 12 shows that the total volume of flow in the Dugald Drain at Plessis Road is estimated to range from 59,904, 92,135, and 109,023 m^3 for the three design storms. Figure 13 shows that the two 1993 events produce a total volume of 390,026 m^3 in the Dugald Drain (278,516 m^3 for July and 111,375 m^3 for the August events). Figure 14 shows that the May 2010 storms resulted in a total volume of 164,292 m^3 passing by the intersection.

The three graphs on Figure 12 to Figure 14 all show that there is about 0.05 m^3/s of flow reversing through the culvert under Plessis Road. This is in fact the pumped flows from the St. Boniface Industrial Park SRB's reversing back to the Deep Pond outlet. This flow represents about 10% of the pumped discharge from SRB 5-1.

As described in Section 2.1 and shown on Figure 7, there is a 600 mm culvert connecting the Dugald Drain to the Deep Pond. This interconnection was made shortly after the 1993 summer storms. This interconnection has a positive impact on the Dugald Drain by removing a relatively significant amount of runoff from the Dugald Drain to the Deep Pond, although it negatively impacts on water levels in the Deep Pond.

The inverts and cross-sections of the interconnection were estimated based on typical ditch elevations in the area and as-built drawings from the inlet to the Deep Pond. The information for the interconnecting drain was not in the City's GIS database, however many of the other surrounding drains are in the database. They should be confirmed prior to detailed design.

Figure 15 contains a graph showing the flows from the Dugald Drain to the Deep pond for the 5, 25 and 100 year design storms. The figure show that under design storm conditions that the land drainage system just north of the railway line at Plessis Road is overloaded and backs up into the interconnecting ditch at a rate of about 0.32 m^3/s for all the design events as indicated by the negative flow rates at the very beginning of the simulation.

However, the system eventually reverses and accepts significantly more runoff volume from the Dugald Drain system due to the longer discharge time of the semi-rural ditch system. The Deep Pond system accepts a peak flow of about 0.60 m^3/s but the sustained peak flow is more in the range of 0.30 to 0.40 m^3/s. A total volume of 35,433, 54,450, and 65,651 m^3 transfers from the Dugald Drain to the Deep Pond for the 5, 25 and 100 year design storms.

Figure 16 contains a graph that shows the flows that leave the Dugald Drain for the Deep Pond under the 1993 summer storms condition. Figure 17 contains a graph that shows the flows that leave the Dugald Drain for the Deep Pond under the May 2010 storms. The figures show that the interconnection behaves in a similar way as the design storms. A total of 191,540 m^3 would be transferred to the Deep Pond from the Dugald Drain under the two major 1993 summer events (104,104 m^3 for July 24-26 and 81,131 m^3 for August 8). About 99,708 m^3 of runoff will have transferred to the Deep Pond from the Dugald Drain as a result of the May 2010 events.

The Table 1 summarizes the volumetric flow split. It shows that roughly 30 to 40% of the volume that enters the Dugald Drain upstream (east) of Plessis Road ends up in the Deep Pond system.

Table 1 - Estimated Flow Split between Dugald Drain and Deep Pond Systems

Event	**Volume to Dugald Drain (m^3)**	**Volume to Deep Pond (m^3)**	**Total Volume (m^3)**	**Volume to Dugald Drain (%)**	**Volume to Deep Pond (%)**
5 Year	59,904	35,433	95,337	62.8%	37.2%
25 Year	92,135	54,450	146,585	62.9%	37.1%
100 Year	109,023	65,651	174,674	62.4%	37.6%
July 24-26, 1993	278,516	104,104	382,620	72.8%	27.2%
August 8, 1993	111,375	81,131	192,506	57.9%	42.1%
May 28-30, 2010	164,292	99,708	264,000	62.2%	37.8%

Figure 18 to Figure 20 shows the impact of the Deep Pond outlet in terms of water levels in the Dugald Drain itself for the 100 year, summer of 1993 and May 2010 storms. The figures show that peak water levels in the Dugald Drain would rise by about 0.097 to 0.17 m (4 to 6 inches) in the reach between Lagimodiere Boulevard and Plessis Road if the Deep Pond outlet were not functioning.

2.3.1.2 Siphon Design and Impact on Dugald Drain Operations

The intention of the siphon configuration shown on Figure 10 is to continue to allow the Dugald Drain to operate in a similar manner as it is currently configured. This includes using the existing LDS inlet to the Deep Pond.

A 1500 mm diameter pipe is appropriate for the siphoning the Dugald Drain through the underpass as shown on Figure 10. Figure 21 contains the profile of the siphon under the underpass under the three design storms, the 1993 summer storms and the May 2010 storms.

The 1500 mm diameter pipe is larger than the 1550x900 mm arch pipe currently under Plessis Road. The existing arch pipe creates a backwater effect upstream of Plessis Road as shown on Figure 11. The larger siphon pipe will theoretically remove this restriction but will increase water levels in the Plessis Road to Lagimodiere Boulevard reach of the drain. The section through the depressed area will also be pressurized as shown in Figure 21. Any access hatches within the depressed area of the siphon will need to be sealed to ensure the siphon does not flood out the underpass area while it is operating.

Figure 22 shows the estimated HGL in the Dugald Drain from the InfoWorks model for the three design storms plus the summer of 1993 and May 2010 scenarios with the proposed siphon. This profile can be compared to the profile in Figure 11 showing the existing conditions in the drain. Generally the siphon lowers water levels upstream of Plessis Road and increases them slightly downstream in the Plessis Road to Lagimodiere Boulevard reach.

Table 2 summarizes the change in water levels along the Dugald Drain with and without the proposed siphon. Both models assume the interconnection with the Deep Pond system.

Table 2 - Change from Existing Conditions Water Levels in Dugald Drain with Plessis Siphon

Location (U/S)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	Aug 1993 (m)	May 2010 (m)
Marion	0.007	-0.005	-0.008	0.001	0.000	-0.005
Lagimodiere	0.044	0.090	0.075	0.081	0.037	0.056
Mazenod	0.023	0.063	0.064	0.088	0.062	0.053
Plessis	-0.263	-0.298	-0.319	-0.238	-0.314	-0.303
Murdock	0.000	-0.014	-0.030	-0.061	-0.041	-0.039

Table 2 shows that water levels in the Plessis to Lagimodiere reach of the Dugald Drain will increase by about 23 to 90 mm (1 to 3.5 inches) relative to existing conditions. Immediately upstream of Plessis Road, water levels will decrease by about 263 to 319 mm for the five events. This change in water levels is due to the 1500 mm siphon being larger than the existing 1550x900 mm arch pipe under Plessis Road.

The profile of the siphon on Figure 21 and Figure 22 shows that the 1500 mm siphon has a steep grade down on the east side of the underpass. The grade of the pipe then follows minimum grade for the remaining stretch before it essentially comes out of a standpipe type configuration.

In the middle of the section of the siphon the profile shows that the pipe size changes to a 1800x900 mm box section which has about the same hydraulic properties of the 1500 mm circular pipe section. The box section is required because the 300 mm wastewater sewer on the west side of Plessis Road and south of Dugald Road conflicts with the grade of the siphon. The grade of the 300 mm wastewater sewer cannot change because it is already installed at minimum grade and at minimum depths further south. It also cannot be made deeper, as the existing 750 mm secondary interceptor is also relatively shallow.

Siphons are prone to plugging up with debris and require regular cleaning. The grade of the siphon was designed such that it could be regularly cleaned out at its west end (i.e. at the standpipe end). There are other maintenance issues that will need to be addressed at detailed design including the preferred cross sectional shape of any transition or non-circular sections. Another concern is how the siphon will operate in winter and how to prevent it from freezing. It may require dewatering before the onset of winter and/or other maintenance throughout the winter.

Velocity in the siphon is also important because higher velocities will help flush out debris. The velocity in a pipe is inversely proportional to the cross-sectional area of the pipe. Thus too large a pipe will yield a lower velocity and less self-cleaning ability.

The velocities in the siphon for the three design storm events range depending on the pipe section and design storms. For the 5 year storm, the velocities range from 0.74 m/s for the steeply graded section to about 0.54 m/s for the less steeply graded section. For the 25 year storm, the velocities range from 0.92 m/s to 0.67 m/s respectively. Ideally the velocities could be a bit higher (around 0.6 m/s for the 5 year storm) which could be achieved with a smaller pipe; however the size of the 1500 mm diameter pipe makes the siphon less vulnerable to plugging with debris and thus a reasonable compromise.

2.3.1.3 Impact of Deep Pond Outlet on Dugald Drain

Maintaining the interconnection to the Deep Pond is a delicate balancing act given the constraints in the Deep Pond system (i.e. no additional volume or higher peak flows are to enter this system due to the limited outlet capacity of the existing pumping station and outlet works). The hydraulic modelling indicates that if similar volumes and peak flows are to be sent to the Deep Pond from the Dugald Drain, the connection must continue to operate as an open channel drain at a similar elevation as the current interconnecting ditch or with a control structure. An underground land drainage system connection between the siphon and the Deep Pond system would result in a far larger volume of runoff being sent to the Deep Pond than through an open channel. This would be unacceptable considering the already poor level of service provided by the Deep Pond. These two configurations will be examined in this section.

The design shown on Figure 10 assumes that the East Side Storage facility at #2125 Dugald Road (not to be confused with a stormwater storage facility) will remain after the construction of the underpass. Figure 10 shows that an underground land drainage/shallow bury culvert system connecting to the siphon could be required to service the property after the installation of the underpass (discussed in Section 2.5). This drainage system could be connected to accept flows from the shallow drain behind the properties on the north side of Dugald Road and south of the railway line. The inlet to the Deep Pond system would then be connected across the railway line to the shallow drain in behind the properties on the north side of Dugald Road. This ditch configuration would behave in a similar manner as the existing ditched cross-connection.

Figure 23 shows the HGL from the five events in the Dugald Drain with the proposed siphon, but no outlet (interconnection) to the Deep Pond. It can be directly compared to Figure 22 (siphon and Deep Pond outlet) and Figure 11 (existing conditions). Table 3 summarizes the change in water levels in the Dugald Drain with the siphon but without the Deep Pond outlet over existing conditions.

Table 3 - Change from Existing Conditions Water Levels in Dugald Drain with Siphon and without Deep Pond Outlet

Location (U/S)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	Aug 1993 (m)	May 2010 (m)
Marion	0.016	-0.003	-0.007	0.003	0.002	0.008
Lagimodiere	0.235	0.283	0.262	0.281	0.172	0.247
Mazenod	0.187	0.244	0.252	0.295	0.242	0.253
Plessis	-0.041	-0.023	-0.025	0.025	-0.022	0.005
Murdock	0.000	-0.001	-0.004	0.019	-0.007	0.000

Table 3 shows that water levels in the Lagimodiere to Plessis Road section increase by about 187 to 295 mm (7 to 12 inches) over existing conditions. Upstream of Plessis Road, the water levels generally stay about the same as existing conditions rather than decreasing with the siphon and interconnection in place thus confirming that the 1500 mm diameter for the siphon is roughly the correct size whether the interconnection remains in place or is ultimately removed.

Figure 24 shows the estimated flows under the three design storms that are estimated to enter the Deep Pond system using the ditch interconnection depicted on Figure 10. The figure shows that the peak flows have increased over existing conditions. The total volume entering the Deep Pond system has increased by roughly 50% over existing conditions (the volume is shown at the bottom right corner of the figure). Figure 25 and 26 show the same information but for the summer of 1993 and May 2010 events.

Figure 27 shows the estimated flows under the three design storms that are estimated to enter the Deep Pond system from the Dugald Drain but by connecting a 600 mm land drainage sewer between the two systems instead of using the back lot ditch (i.e. connect the Deep Pond system directly to the Dugald Drain with a land drainage sewer). The figure shows that the more efficient land drainage sewer connection removes flow from the Dugald Drain for a longer period of time. As a result the volume of runoff entering the Deep Pond has increased by about 100% over existing conditions.

A sensitivity analysis was conducted to see if a smaller LDS pipe connection would provide similar hydraulic results as the ditch interconnection. The results from the 100 year storm with varying interconnecting pipe sizes are shown on Figure 28. The graph shows that with the exception of peak flows, the total volume being sent to the Deep Pond does not change significantly for connections ranging from 375 mm to 600 mm.

The volume of runoff to the Deep Pond does not change significantly based on the range of diameters analyzed because the deep LDS connection is active for a longer period of time than the ditch connection. As long as there is flow in the siphon, flow would be entering the LDS interconnection. However if the interconnection is configured as a ditch and close to the surface, the HGL in the siphon would not drive the interconnection for as long a period of time as the piped connection due to the higher elevation of the ditch.

Figure 29 shows the same sensitivity analysis but for the ditch interconnection via the shallow bury culvert through #2125 Dugald Road. The pipe under the railway line is varied from a 600 mm to a

375 mm diameter. The graph on Figure 29 shows that varying the pipe under the railway line does not significantly change the volume of runoff entering the Deep Pond similar to the LDS connection results.

The interconnection in the future underpass configuration designed to take flow from the Dugald Drain to the Deep Pond needs to be designed carefully such that it functions similarly in elevation as the existing ditch connection. Changing the configuration from a ditch connection to a land drainage sewer connection without a control structure (i.e. a weir structure) would send more runoff to the Deep Pond. Sending more runoff to the Deep Pond system is not acceptable; however the capacity already afforded by the outlet to the Deep Pond should continue to be considered, at least until the South Transcona LDS Trunk to SRB 5-1 is constructed.

The exact configuration of the ditch interconnection will need to be determined during the detailed design phase as it will depend on the ultimate disposition of the East Side Storage facility at #2125 Dugald Road. If the facility remains, then a shallow bury culvert system may be required to provide internal drainage servicing to the property and the interconnection to the Deep Pond. If the property is purchased outright and the facilities are demolished then a simpler ditch connection may be acceptable.

The estimated response in the Deep Pond (SRB 4-7) under a 100 year summer storm is found in the graph on Figure 30. The graph shows existing conditions (blue), existing conditions without the connection to the Dugald Drain (green), conditions with the 600 mm diameter LDS connection to the siphon (brown) and conditions with the 600 mm diameter ditch connection (gold).

With the exception of existing conditions without the Dugald Drain connection, the peak water levels in the Deep Pond remain the same for the three interconnection scenarios. The drawdown time for the ditch connection is about 4 hours longer than existing conditions. Likewise the LDS connection is about 10.5 hours longer than existing conditions. The drawdown times assume only the firm capacity of the Ravelston Lift Station (i.e. two out of three pumps operating). The small amount of additional drawdown time is likely acceptable, however the proposed ditched interconnection could likely be shrunk down to better match the existing volumes once a decision has been made regarding #2125 Dugald Road.

The estimated flows at the upstream end of the proposed siphon from the three design storms are found in Figure 31. Figure 32 and Figure 33 show the estimated flow in upstream end of the proposed siphon for the summer of 1993 events and the May 2010 events respectively.

The figures show that the flows generally peak at around 1.8 m^3/s. However, this flow includes the component that goes to the Deep Pond system. The peak flows are reduced to around 1.5 m^3/s downstream of the Deep Pond interconnection for the larger events.

Table 4 below summarizes the volumetric flow split with the proposed siphon and 600 mm ditch interconnection. It shows that the volume of flow to the Deep Pond is modestly increased such that the flow split ratio is roughly 50/50 rather than 35/65. The final design of the interconnecting ditch should reduce the volumetric flow split depending on the disposition of #2125 Dugald Road.

Table 4 - Estimated Flow Split between Dugald Drain and Deep Pond Systems with Proposed Siphon and Interconnecting Ditch and 600 mm Inlet

Event	Volume to Dugald Drain (m^3)	Volume to Deep Pond (m^3)	Total Volume (m^3)	Volume to Dugald Drain (%)	Volume to Deep Pond (%)
5 Year	47,305	47,056	94,361	50.1%	49.9%
25 Year	73,048	73,449	146,497	49.9%	50.1%
100 Year	86,779	87,820	174,599	49.7%	50.3%
July 24-26, 1993	250,658	144,556	395,214	63.4%	36.6%
August 8, 1993	81,893	109,325	191,218	42.8%	57.2%
May 28-30, 2010	131,989	131,515	263,504	50.1%	49.9%

The estimated water levels in the Dugald Drain at either end of the siphon are found in Table 5 below. The table assumes that flow is removed from the Dugald Drain via the ditched interconnection.

Table 5 - Estimated Water Levels in Dugald Drain at Siphon Inlet and Outlet with Ditched Outlet to Deep Pond System

Event	Upstream (Road El = 233.0 m)		Downstream (Road El = 232.75 m)	
	Water Level (m)	Freeboard (m)	Water Level (m)	Freeboard (m)
5 Year	232.155	0.845	231.960	0.790
25 Year	232.431	0.569	232.138	0.612
100 Year	232.576	0.424	232.245	0.505
July 24-26, 1993	232.939	0.061	232.536	0.214
August 8, 1993	232.684	0.316	232.339	0.411
May 28-30, 2010	232.614	0.386	232.310	0.440

Table 5 indicates that for the 100 year event there is an estimated 424 mm of freeboard at the upstream end and 505 mm of freeboard at the downstream end of the Dugald Drain siphon from the preliminary road elevations. Although this is less than the standard 600 mm (2 feet) it should be enough to prevent the Dugald Drain from overflowing into the underpass drainage system for all but the very largest storm (July 1993).

If the Deep Pond outlet is unavailable, the InfoWorks model estimates the water levels shown in Table 6.

Table 6 - Estimated Water Levels in Dugald Drain at Siphon Inlet and Outlet without Outlet to Deep Pond System

Event	Upstream (Road El = 233.0 m)		Downstream (Road El = 232.75 m)	
	Water Level (m)	Freeboard (m)	Water Level (m)	Freeboard (m)
5 Year	232.389	0.611	232.104	0.646
25 Year	232.711	0.289	232.311	0.439
100 Year	232.872	0.128	232.430	0.320
July 24-26, 1993	233.201	-0.201	232.743	0.007
August 8, 1993	232.978	0.022	232.520	0.230
May 28-30, 2010	232.925	0.075	232.510	0.240

Table 6 shows that there will be only 128 mm of freeboard at the upstream and 320 mm of freeboard at the downstream end of the siphon under the 100 year condition. Under the July 1993 scenario, the underpass would be susceptible to being flooded out due to the Dugald Drain overtopping its existing banks if the interconnection were not in place. Careful grading of the drain and the siphon works could likely eliminate this possibility, however.

Water levels in the Plessis Road to Lagimodiere Boulevard reach of the Dugald Drain will increase with the proposed 1500 mm siphon. These levels will increase further without the interconnection to the Deep Pond. Therefore, the interconnection to the Deep Pond should be considered after the construction of the underpass in order to maintain approximately existing water levels on the Dugald Drain and to provide the proposed underpass and Dugald Road with the highest level of service that is reasonably achievable.

In extreme cases when the Deep Pond system is vulnerable, the interconnection could be fitted with a positive gate to prevent the Dugald Drain from flowing into the Deep Pond. This gate could be closed when the Deep Pond is vulnerable (e.g. when pumps have malfunctioned), as there is likely more damage that could be done in the limits of the relatively developed Deep Pond system versus the relatively undeveloped Dugald Drain Land Drainage District.

2.3.1.4 Connect Plessis Siphon to South Transcona LDS Trunk and SRB 5-1

As shown on Figure 9, the long term drainage plan for the area south of the railway line is to connect the proposed South Transcona LDS Trunk up to the existing SRB's in the St. Boniface Industrial Park (i.e. the Dugald Drain Land Drainage District). When the South Transcona LDS trunk is constructed, the Dugald Drain siphon at Plessis Road could be connected to the new LDS Trunk and SRB 5-1. This assumes that the land east of Plessis Road remains largely rural when the South Transcona LDS Trunk is initially constructed (i.e. the SRB system east of Plessis is not yet in place).

One possibility could be to rehabilitate the abandoned 1050 mm interceptor that runs from Mazenod Road to Plessis Road. The new South Transcona LDS trunk will use the rehabilitated 1350/1200 mm abandoned interceptor from Mazenod Road to Lagimodiere Boulevard as a cost saving measure. The 1050 mm pipe is undersized to take the full flows from the Dugald Drain upstream of Plessis Road; however, it would significantly help to reduce water levels along Dugald Road downstream (west) of Plessis Road and improve the level of service provided to Dugald Road and the various cross-streets that currently flood under larger rainstorms. It would also be more cost-effective than installing a brand new pipe. The Dugald Drain (and the siphon outlet) would not be abandoned, but would simply take excess flow that the 1050 mm pipe cannot handle.

The proposed bottom elevation of the siphon is lower than the invert of the abandoned 1050 mm interceptor due to the 250/300 mm wastewater sewer coming from the southern leg of Plessis Road. This means that it will always operate as a siphon if it is connected to the abandoned interceptor, but it would be a shallower siphon and thus likely less problematic. Additionally, three short and shallow siphons will be required along the abandoned interceptor between the proposed siphon and Mazenod Road if it is rehabilitated. These are all due to wastewater sewers with conflicting grades that were previously connected to the now abandoned interceptor.

The preliminary hydraulic modeling indicates good performance of the St. Boniface Industrial Park SRB's (SRB 5-1) with respect to drawdown times and peak water levels with the 1050 mm re-used as a land drainage sewer. However, the South Transcona LDS trunk through the Mission District is unlikely to be constructed in the near future (i.e. not within the next 10 years) so the details of the configuration will not be discussed here.

However, the siphon configuration should take into account a future connecting pipe that could be as large as a 1500 mm diameter to allow the Dugald Drain to be diverted to the future gravity LDS trunk and SRB 5-1.

2.3.2 Dugald Drain Diversion to Esselmont Drain (Option 2)

The Esselmont Diversion, shown on Figure 34, diverts the Dugald Drain south to the Esselmont Drain along the east side of the Transcona Golf Course. This configuration was examined primarily to show that the Dugald Drain cannot be diverted to the Esselmont Drain and the St. Boniface Industrial Park SRB's without a proper gravity outlet (i.e. the South Transcona LDS Trunk) for these SRB's or without a large amount of additional storage (i.e. the future 2.7 Ha SRB consistent with the long term drainage plans).

2.3.2.1 Background

The Esselmont Drain was constructed in the early 2000's as a result of severe flooding in South Transcona from the 1993 rainstorms. Prior to the drain's construction, the area was directly serviced by the Dugald Drain. The Esselmont Drain effectively cut the drainage area serviced by the Dugald Drain east of Plessis Road in half, diverting the southern part to the St. Boniface Industrial Park SRB's.

The St. Boniface Industrial Park SRB's are able to function with the Esselmont Drain component because they were constructed to handle back to back 25 year summer events or approximately 130 mm (5 inches) of rainfall with only standard water level rises for their original service area. This is a very high standard, but was primarily chosen because the SRB's originally had no permanent pumped outlet, but only a temporary pump that was brought out after a rainstorm.

When the St. Boniface Industrial Park was originally constructed in the mid to late 1970's the SRB's were dewatered to the Dugald Drain using only a small temporary pump that had to be transported to site. Eventually a permanent 0.14 m^3/s (5 cfs) pump was installed. The 0.42 m^3/s (15 cfs) pump was installed around the time the Esselmont Drain was constructed due to the additional rural inflows. The 0.42 m^3/s dewatering pump is manually turned on by City crews once the levels in the Dugald Drain are low enough to accept them.

As a result of the lack of permanent gravity outlet and large inflows from the Esselmont Drain, water levels on the St. Boniface Industrial Park SRB's can get very high. During the May 2010 storms, SRB 5-1 rose to an elevation of 231.538 m, which is about 4.2 m above normal water level, or 2.3 m above the high water level. Although very high, it was not known to cause property damage. However, if the levels got much higher due to additional inflows, they could cause damage and street flooding as the low LDS manhole elevation in the area is around 231.8 m (S-MH50009540 on Paquin Road east of Beghin Avenue).

There are knife gates on each of the three catchbasin leads that convey flows from the Esselmont Drain to the 1050 mm trunk located east of Plessis so as to stop flow from the Esselmont Drain from entering the St. Boniface Industrial Park system if water levels on the SRB's get too high. This would cause overland flooding around the Esselmont Drain inlet area which is less likely to cause as much damage as flooding part of the industrial park.

2.3.2.2 Existing Conditions in the Esselmont Drain and St. Boniface Industrial Park SRB's

This section will describe the existing performance of the St. Boniface Industrial Park SRB system and the Esselmont Drain as estimated by the InfoWorks model.

Figure 35 shows the estimated water levels in the Esselmont Drain and the St. Boniface Industrial Park SRB system under existing conditions for the 5, 25, 100 year, summer of 1993, and May 2010 events. The profile shows very high levels on the entire system. The water levels upstream of the shallow bury culvert on the Esselmont Drain are higher than would actually be seen due to a lack of field storage and overland flow conduits configured in the hydraulic model.

The estimated maximum rises (and peak water levels) are found in Table 7.

Table 7 - Estimated Peak Rises in St. Boniface Industrial Park SRB's

Element	NWL (m)	HWL (m)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	227.38	229.21	1.82	2.77	3.14	4.87	4.09	3.70
SRB 5-2	228.75	229.97	0.82	1.50	1.83	3.53	2.77	2.38
SRB 5-3	228.75	229.97	1.01	1.62	1.91	3.56	2.81	2.42
Waterside Estates	230.20	231.11	0.45	0.81	1.11	2.09	1.39	0.93

Table 7 shows that with the exception of SRB 5-1 the rises in the SRB's meet the City's maximum rise criteria of 1.8 m for SRB's in industrial areas (SRB 5-2 and 5-3) and 1.2 m rise criteria for residential areas (Waterside Estates) under a 25 year storm. The expected rises in SRB 5-1 are acceptable however, because the lake has a relatively low normal water level and is intended to be operated in such a way.

Table 8 contains the estimated drawdown times for the SRB's in the St. Boniface Industrial Park.

Table 8 - Estimated Drawdown Times in St. Boniface Industrial Park SRB's

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	115	190	227	N/A	321	314
SRB 5-2	76	149	185	N/A	280	275
SRB 5-3	75	150	186	N/A	280	275
Waterside Estates	8	12	13 to 68	296	174	159

Table 8 shows that the drawdown times for the St. Boniface Industrial Park SRB's, with the exception of the Waterside Estates SRB, do not meet the City's drawdown criteria of 48 hours for a 5 year storm and 120 hours for a 100 year storm. Any additional inflows would add further drawdown time for the system.

The Waterside Estates SRB discharges fast but only as far as SRB 5-1. Under a 100 year storm, SRB 5-1 actually backs up into the Waterside Estates SRB. The drainage from the intended service area discharges to SRB 5-1 in only 13 hours under a 100 year storm. However, the rural component of flow entering SRB 5-1 causes high enough water levels to back up into the Waterside Estates SRB. It then takes 68 hours for this rural flow component to fully leave the Waterside Estates SRB.

The drawdown time for the July 24-25, 1993 event is listed as not applicable because the SRB has not yet finished drawing down before the August 8, 1993 event occurs.

The drawdown times shown in Table 8 are based on the pump turning on and dewatering SRB 5-1. The pump is automatically configured in InfoWorks to only turn on when levels in SRB 5-1 and the Dugald Drain warrant it. The model assumes that if levels at the downstream end of the Mazenod Road box culvert are less than 231.75 m (or roughly 3 to 4 feet of water in the drain) then the dewatering pump can be turned on. This is done automatically by the InfoWorks model, however in reality it is done manually by City crews as they are able to get there. Thus the drawdown times predicted by the hydraulic model in Table 8 may be a bit less than what would actually be observed.

2.3.2.3 Estimated Performance of St. Boniface Industrial Park System with Dugald Drain Diversion

This section will discuss the performance of the land drainage system if the Dugald Drain is diverted into the Esselmont Drain.

This scenario assumes that the diversion drain along the east side of the Transcona Golf Course is constructed. Additionally, the downstream part of the Esselmont Drain from the diversion drain to the inlet structure is re-graded since the grading of the diversion requires it. The outlet pumps to the Dugald Drain remain in the same configuration.

Figure 36 shows the estimated HGL in the diverted part of the Dugald Drain, the Esselmont Drain and the St. Boniface Industrial Park SRB system to the Dugald Drain outlet for the 5, 25, 100 year design storms and the real summer of 1993 and May 2010 events.

The peak water levels on the St. Boniface Industrial Park SRB's generally increase as do the drawdown times with the Dugald Drain upstream of Plessis Road diverted to the Esselmont Drain. Table 9 summarizes the estimated peak rises in the St. Boniface Industrial Park SRB system with the Dugald Drain Diversion.

Table 9 - Estimated Peak Rises in St. Boniface Industrial Park SRB System with Dugald Drain Diversion to Esselmont Drain

Element	NWL (m)	HWL (m)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)
SRB 5-1	227.38	2.17	3.05	3.45	5.15	4.44	4.14
SRB 5-2	228.75	1.22	1.87	2.23	3.85	3.16	2.88
SRB 5-3	228.75	1.54	2.13	2.43	3.93	3.28	3.02
Waterside Estates	230.20	0.47	0.84	1.15	2.41	1.72	1.43

Table 10 summarizes the increase in rise in the SRB system with the diversion over existing conditions. It shows that peak water levels increase by about 0.28 to 0.60 m (1 to 2 feet) over existing conditions in most of the SRB's.

Table 10 - Estimated Increase in Peak Rises in St. Boniface Industrial Park SRB System with Dugald Drain Diversion over Existing Conditions

Element	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	0.35	0.28	0.32	0.28	0.35	0.44
SRB 5-2	0.40	0.37	0.39	0.32	0.40	0.50
SRB 5-3	0.53	0.52	0.53	0.37	0.47	0.60
Waterside Estates	0.02	0.03	0.04	0.32	0.33	0.50

Table 11 summarizes the estimated drawdown times of the St. Boniface Industrial Park SRB system with the Dugald Drain diverted to the Esselmont Drain.

Table 11 - Estimated Drawdown Times of St. Boniface Industrial Park SRB's with Dugald Drain Diversion to Esselmont Drain

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	141	221	265	N/A	358	372
SRB 5-2	103	183	227	N/A	320	334
SRB 5-3	104	183	226	N/A	320	334
Waterside Estates	9	12 to 66	14 to 110	316	204	218

Table 12 summarizes the increase in drawdown times over existing conditions with the Dugald Drain diverted to the Esselmont Drain. It shows that, based on the existing pumping rates, the drawdown times would increase by about 26 to 59 hours depending on the storm event.

Table 12 - Increase in Drawdown Times of St. Boniface Industrial Park SRB's with Dugald Drain Diversion over Existing Conditions

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	26	31	38	N/A	37	58
SRB 5-2	27	34	42	N/A	40	59
SRB 5-3	29	34	40	N/A	40	59
Waterside Estates	1	N/A	N/A	20	30	59

The increase in drawdown times and peak water levels in the St. Boniface Industrial Park SRB system with the Dugald Drain diversion would add significant additional risk of overland flooding to the system. It is thus not a recommended reconfiguration of the Dugald Drain for the Plessis Underpass.

The water levels along the Esselmont and the diverted part of the Dugald Drain also increase as a result of the diversion. The Esselmont Drain enters the St. Boniface Industrial Park SRB system via three 'beehive' style catchbasins each with a 375 mm catchbasin lead that connect to a 1050 mm land drainage sewer. Both the catchbasin leads and 1050 mm LDS will cause significant backwater on the downstream parts of the system due to the large amount of additional flow from the Dugald Drain catchment upstream of Plessis Road. Table 13 summarizes the increase in water levels along both the Dugald and Esselmont Drains.

Table 13 - Estimated Change in Water Levels on Dugald and Esselmont Drains due to Dugald Drain Diversion over Existing Conditions

Location	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
Esselmont Drain						
LDS Inlet Structures	0.54	0.23	0.19	0.14	0.04	0.30
West of Bates (at Esselmont X-ing)	0.22	0.19	0.16	0.13	0.02	0.27
Foch (Inlet of Shallow Bury Culvert)	0.01	0.02	0.02	0.08	-0.01	0.08
Symington	-0.01	0.00	0.00	0.06	-0.01	0.02
Dugald Drain						
East of Plessis	-0.04	0.23	0.31	0.70	0.45	0.43
Bates	-0.09	0.12	0.19	0.57	0.32	0.31
Foch	-0.07	0.04	0.07	0.35	0.14	0.15
Symington	-0.05	0.00	0.02	0.25	0.07	0.09
Fuller	-0.01	-0.01	0.00	0.22	0.04	0.06
Murdock	0.00	-0.01	0.00	0.21	0.03	0.04

Table 13 shows that the levels on the western part of the diverted Dugald Drain and Esselmont Drain increase significantly over existing conditions. Considering the already low level of service for these areas, improvements would be required to either the LDS leading to the St. Boniface Industrial Park or in the form of a new stormwater retention basin. Additional improvements in this area will not, however, reduce the drawdown times on the St. Boniface Industrial Park SRB's.

The next section will examine the impact on water levels on the diverted Dugald Drain and Esselmont Drain if the future 2.7 Ha SRB is installed as a part of the diversion drain.

2.3.2.4 Estimated Performance of LDS with Dugald Drain Diversion and 2.7 Ha SRB

As shown on Figure 34, the future 2.7 Ha SRB that is required for the ultimate land drainage servicing plan in South Transcona could be phased in as a part of the diversion drain. The SRB would help reduce water levels on the diverted Dugald Drain and Esselmont Drain. However, the additional storage would not help reduce drawdown times in the St. Boniface Industrial Park SRB's.

Figure 37 shows the estimated HGL in the diverted Dugald Drain and St. Boniface Industrial Park SRB's with the future 2.7 Ha SRB connected to the Esselmont Drain. The profiles show that the water levels in the Esselmont and Dugald Drains have decreased over conditions without the SRB and over existing conditions. The profiles in Figure 37 show that backwater effects occur on the downstream part of the Esselmont Drain for storms greater than a 5 year return period, which is an improvement over the conditions shown on Figure 36 without the 2.7 Ha SRB and existing conditions. The difference in water levels over existing conditions is summarized in Table 14.

Table 14 - Estimated Difference in Water Levels on Dugald and Esselmont Drains with Dugald Drain Diversion and 2.7 Ha SRB

Location	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
Esselmont Drain						
LDS Inlet Structures	-0.59	-0.51	-0.48	-0.26	-0.45	-0.20
West of Bates (at Esselmont X-ing)	-0.26	-0.53	-0.50	-0.27	-0.47	-0.22
Foch (Inlet of Shallow Bury Culvert)	-0.02	-0.12	-0.17	-0.11	-0.19	-0.14
Symington	-0.01	-0.03	-0.04	-0.05	-0.06	-0.06
Dugald Drain						
East of Plessis	-0.50	-0.49	-0.35	0.29	-0.04	-0.06
Bates	-0.36	-0.51	-0.41	0.18	-0.14	-0.16
Foch	-0.11	-0.20	-0.24	0.04	-0.15	-0.16
Symington	-0.07	-0.12	-0.15	0.00	-0.13	-0.15
Fuller	-0.01	-0.06	-0.09	-0.02	-0.10	-0.12
Murdock	0.00	-0.03	-0.08	-0.02	-0.08	-0.08

The 2.7 Ha SRB is thus largely effective in reducing water levels upstream of the St. Boniface Industrial Park SRB's.

The estimated rises from the InfoWorks model are shown in Table 15. With the added storage, the rises on the SRB's in the St. Boniface Industrial Park system generally meets the 1.8 m maximum rise criteria for the 25 year storm. The exception is the 2.7 Ha SRB has a 2.61 m rise for a 25 year storm. This rise would be considered marginally acceptable however since the SRB would be located in an undeveloped and generally inaccessible area.

Table 15 - Estimated Rises in St. Boniface Industrial Park SRB System with Dugald Drain Diverted to Esselmont Drain and 2.7 Ha SRB

Element	NWL (m)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	227.38	2.03	2.90	3.27	4.85	4.28	3.88
SRB 5-2	228.75	0.95	1.65	2.00	3.55	2.99	2.59
SRB 5-3	228.75	1.17	1.82	2.14	3.63	3.09	2.71
Waterside Estates	230.20	0.47	0.84	1.15	2.10	1.54	1.15
South Transcona 1	229.59	1.74	2.61	2.93	3.87	3.36	3.25

Table 16 summarizes the difference in rises between the Dugald Drain diversion and 2.7 Ha SRB versus existing conditions. The rises generally increase by about 0.15 to 0.30 m (6 to 12 inches).

Table 16 - Estimated Difference in Rises in the St. Boniface Industrial Park SRB System with the Dugald Drain Diversion and 2.7 Ha SRB versus Existing Conditions

Element	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	0.22	0.13	0.14	-0.03	0.19	0.18
SRB 5-2	0.13	0.15	0.16	0.01	0.22	0.22
SRB 5-3	0.16	0.21	0.23	0.07	0.28	0.30
Waterside Estates	0.02	0.03	0.04	0.01	0.15	0.22

Although the additional 2.7 Ha SRB added east of Plessis Road helps reduce water levels in both the influent drains and the SRB systems, the added storage does little to improve drawdown times. The estimated drawdown times are summarized in Table 17.

Table 17 - Estimated Drawdown Times in St. Boniface Industrial Park SRB's with Dugald Drain Diversion to Esselmont Drain and 2.7 Ha SRB

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	146	231	276	N/A	391	385
SRB 5-2	108	193	238	N/A	352	348
SRB 5-3	108	193	239	N/A	352	347
Waterside Estates	8	12 to 61	14 to 107	319	221	216
South Transcona 1	58	125	171	N/A	286	281

The drawdown times in Table 17 have generally increased over existing conditions and over the diversion drain only scenario (Table 12). Table 18 summarizes the increase in drawdown times over existing conditions.

Table 18 - Increase in Drawdown Times of St. Boniface Industrial Park SRB's with Dugald Diversion and 2.7 Ha SRB over Existing Conditions

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	31	41	49	N/A	70	71
SRB 5-2	32	44	53	N/A	72	73
SRB 5-3	33	44	53	N/A	72	72
Waterside Estates	0	N/A	N/A	23	47	57

Therefore, considering the already long drawdown times on the St. Boniface Industrial Park SRB's, for Dugald Diversion Option to be feasible, a more reliable and higher capacity outlet is required than the current 0.42 m^3/s pump station. This will be discussed in the context of the proposed South Transcona LDS Trunk in the next section.

2.3.2.5 Estimated performance of LDS with South Transcona LDS Trunk, 2.7 Ha SRB and Dugald Drain Diversion

In order to improve the drawdown times in the St. Boniface Industrial Park SRB's with the Dugald Drain diversion and the 2.7 Ha SRB the South Transcona LDS Trunk is needed. The South Transcona LDS Trunk is a large diameter land drainage sewer that is proposed as a part of the Mission Combined Sewer District Relief scheme. It has the dual purpose of relieving part of the Mission District by land drainage sewer separation and acting as a gravity outlet for the lands currently serviced by the Dugald Drain. The route of the proposed LDS trunk is shown on Figure 38 and is consistent with the ultimate drainage plan for the area shown on Figure 9 like the 2.7 Ha SRB on the lands east of Plessis Road.

Figure 38 shows that the South Transcona LDS Trunk will use new construction through the Mission District. At Lagimodiere Boulevard and Dugald Road the LDS trunk will use the former 1350 mm Dugald Interceptor which will be rehabilitated by relining. The former 1350/1200 mm interceptor will be used as the outlet for the St. Boniface Industrial Park SRB's, the other existing SRB's in the district and the future systems located in the undeveloped parts of the area serviced by the Dugald Drain. A relatively short stretch of new pipe on Mazenod Road will be required to connect SRB 5-1 with the rehabilitated interceptor on Dugald Road.

Table 19 summarizes the estimated rises in the SRB system with the South Transcona LDS Trunk, the 2.7 Ha SRB and the diverted Dugald Drain. The table shows that the rises are generally acceptable according to City guidelines.

Table 19 - Estimated Peak Water Levels in St. Boniface Industrial Park SRB's with Dugald Diversion, 2.7 Ha SRB and LDS Trunk

Element	NWL (m)	5 Year (m)	25 Year (m)	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	227.38	1.66	2.32	2.64	3.78	2.78	3.10
SRB 5-2	228.75	0.92	1.55	1.85	3.02	1.99	2.34
SRB 5-3	228.75	1.24	1.91	2.22	3.42	2.35	2.72
Waterside Estates	230.20	0.62	1.04	1.35	1.57	1.59	0.90
South Transcona 1	229.59	2.04	2.90	3.18	4.16	3.29	3.55

Table 20 summarizes the difference in peak rises in the St. Boniface Industrial Park SRB system with the Dugald Drain diversion, the 2.7 Ha SRB and the South Transcona LDS Trunk. The table shows that the peak rises on SRB 5-1 are generally lower than existing conditions for the three real events but higher for the design events. This is partially due to the peaky nature of the design events, but the increase is also a result of the model assuming the remainder of the area west of Plessis Road has developed and is discharging to the SRB system (i.e. the lands south of the GWWD railway line).

Table 20 - Estimated Difference in Peak Water Levels with Dugald Diversion, 2.7 Ha SRB and LDS Trunk versus Existing Conditions

Element	5 Year (m)	25 Yea (m)r	100 Year (m)	July 1993 (m)	August 1993 (m)	May 2010 (m)
SRB 5-1	-0.16	-0.44	-0.50	-1.10	-1.31	-0.60
SRB 5-2	0.10	0.05	0.02	-0.52	-0.78	-0.04
SRB 5-3	0.23	0.30	0.31	-0.14	-0.46	0.31
Waterside Estates	0.16	0.23	0.23	-0.52	0.20	-0.03

Table 21 summarizes the estimated drawdown times of the SRB's in the St. Boniface Industrial Park with the three improvements. The table shows that the drawdown times for the SRB's in the system generally meets the 120 hour criteria for a 100 year storm. However, the system does not meet the 48 hour drawdown time guideline for the 5 year storm. This is partially due to the long drawn out nature of the rural runoff component, but also the system takes a long time to discharge the remaining 0.30 m (1 foot) or so of rise.

Table 21 - Estimated Drawdown Times of St. Boniface Industrial SRB's with Dugald Diversion, 2.7 Ha SRB and LDS Trunk

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	99	130	145	249	153	204
SRB 5-2	71	100	116	221	123	175
SRB 5-3	72	101	117	221	121	176
Waterside Estates	10	13	15 to 69	173	76	129
South Transcona 1	60	82	97	199	103	154

Table 22 summarizes the difference in drawdown times with the South Transcona LDS Trunk, 2.7 Ha SRB and Dugald Drain diversion to the Esselmont Drain in the St. Boniface Industrial Park SRB system. The table shows that drawdown times are reduced significantly over existing conditions especially for the larger storms (indicated by the "–" sign). In the case of the July 25-26 1993 storms, the SRB's are actually able to draw down fully before the August 8, 1993 event. The existing conditions model indicates that the St. Boniface Industrial Park SRB's would not fully draw down from the July storm before the August storm occurred.

Table 22 - Estimated Difference in Drawdown Time of St. Boniface Industrial Park SRB's with Dugald Diversion, 2.7 Ha SRB and LDS Trunk versus Existing Conditions

Element	5 Year (hours)	25 Year (hours)	100 Year (hours)	July 1993 (hours)	August 1993 (hours)	May 2010 (hours)
SRB 5-1	-16	-60	-82	N/A	-168	-110
SRB 5-2	-5	-49	-69	N/A	-157	-100
SRB 5-3	-3	-49	-69	N/A	-159	-99
Waterside Estates	2	1	N/A	-123	-98	-30

Thus if the Dugald Drain east of Plessis Road is to be diverted away from the Plessis Underpass area to the Esselmont Drain, the South Transcona LDS Trunk and the 2.7 Ha SRB are required for the system to operate with a reasonable level of service.

2.4 Conceptual Underpass Drainage System

This section will discuss the preliminary design of the land drainage system servicing the underpass. The underpass system will eventually be pumped to the Dugald Drain or discharge by gravity to the future land drainage system installed in the district. However to maintain a high level of service the system must not directly connect to the external drainage systems as they cannot provide the required level of service.

In order to provide an adequate level of drainage service to the proposed underpass, the underpass must be serviced by a lift station. Gravity sewers convey the land drainage from the depressed road to the lift station. The preliminary design of the underpass drainage system is found on Figure 10.

This is a fairly typical configuration for newer underpasses in Winnipeg due to the relative lack of hydraulic grade. In the case of the proposed Plessis Road Underpass, the bottom of the proposed underpass is about 2 m below the closest land drainage sewer just north of the railway line making gravity drainage impossible.

The underpass lift station is envisioned to discharge to a dry pond located immediately next to the lift station. The dry pond stores the discharge until such time it can be discharged to the Dugald Drain. The dry pond is pumped down using a small 0.1 m^3/s pump and a 375 mm gravity discharge line connected to the Dugald Drain siphon. The discharge line essentially acts as a part of the siphon that could ultimately be connected to a land drainage sewer on Dugald Road.

2.4.1 Level of Drainage Servicing

As per Table A-2 of the City of Winnipeg Culvert and Drainage Inlet/Outlet Safety Guidelines, the Plessis Underpass (which has an arterial roadway classification) should be able to handle a 25 year minor storm and a 50 year major storm.

This means designing the catchbasin inlets and land drainage sewers to accommodate a 25 year design storm with a minimum of pavement encroachment. For the system to adequately handle a 50 year major storm, this means having at least one lane passable by vehicular traffic in each direction.

The criteria usually translates into a firm capacity of the lift station (i.e. with the largest single pump in the lift station being out of service) which is able to accommodate a 25 year design storm event and a total capacity of the lift station (i.e. all pumps operating) which is able to handle the 50 year storm.

2.4.2 Land Drainage Sewers

The preliminary underpass drainage design on Figure 10 was developed using the preliminary design provided by the Public Works Department shown on Figure 2 and Figure 3. The catchment area is estimated to be approximately 9 Ha; however this area may shrink as the design of the underpass is refined.

The preliminary underpass drainage design on Figure 10 was sized using a Rational Method spreadsheet analysis before being input to the InfoWorks model. The Rational Method analysis determined that a 25 year storm produces a peak instantaneous flow of 2.0 m^3/s and a 50 year storm produces a peak instantaneous flow of 2.3 m^3/s to the lift station assuming a free discharge.

2.4.3 Lift Station

The Bishop Grandin Boulevard Lift Station located at the grade separated intersection with Pembina Highway has a similar capacity as may be required at the Plessis Underpass. It has four submersible pumps rated at approximately 0.60 m^3/s for a firm capacity of 1.8 m^3/s and total capacity of 2.2 m^3/s. It services about 15 Ha of depressed road, however it makes use of a storage element in the underpass itself to significantly reduce the required lift station capacity. The four 0.60 m^3/s pump configuration similar to Bishop Grandin Boulevard at Pembina Highway was thus used in the InfoWorks model to examine the impact on the existing drainage systems.

2.4.4 Dry Pond

The Dugald Drain or any other nearby land drainage system cannot handle the large peak flows required to provide the underpass with adequate land drainage servicing. The peak flows must therefore be attenuated using a storage element, preferably as close to the lift station as possible so as to not have to convey a large peak flow a long distance.

The City owned property at the northwest corner of Plessis Road and Dugald Road and south of the railway line is an ideal location for a storage element as shown on Figure 10. It will be located immediately next to the proposed lift station, which is located as close to the low point of the underpass as reasonably possible so as to minimize the depth (and installation cost) of the gravity land drainage sewers servicing the underpass. There are other options for the storage element such as on the City owned golf course at the southeast corner of the Dugald and Plessis Road intersection or the Waterside Estates SRB. However, these locations are considerably further away than the proposed lift station location and would require an expensive forcemain or gravity sewer to convey 2.2 m^3/s to them.

The two City owned parcels of land at the northwest corner of Plessis and Dugald Roads are an appropriate size to store the volume of discharge coming from the underpass. However, in order to fit them in to the City owned parcels, the storage element will likely be a dry pond with 5:1 side slopes rather than the 7:1 standard. Using the steeper side slopes is reasonable for an installation such as this one as it is generally isolated, can be fenced off and is located in an industrial type setting. The approximate storage available in the dry pond below ground elevation (i.e. assuming no freeboard) is 12,500 m^3.

Alternately the dry pond could be moved further west onto land at #2049 Dugald Road. Some of the City owned land may be required for the road and rail detours while the underpass is being constructed.

The two City owned parcels currently have the 450/500 mm watermain running across them. Portions of this watermain will likely have to be relocated due to depth of cover issues in the south leg of the underpass (as will be discussed in Section 3). This may require the remainder of the 450/500 mm watermain through the two City owned parcels to be relocated further west onto #2049 Dugald Road as well.

Regardless, the 450/500 mm watermain could be moved closer to the south and west property lines of the parcel they are currently in to make more space for the dry pond. The relocation of watermain would be less costly than building a forcemain or gravity land drainage sewer to convey the underpass flows further away. A 1500 mm diameter gravity sewer would be the approximate requirement to convey the underpass peak flows by gravity.

2.4.5 Dry Pond Discharge Line

The configuration of the discharge line from the dry pond depends on the configuration of the Dugald Drain both in the short term and long term (discussed in Section 2.3). The recommended configuration of the Dugald Drain is to use the 1500 mm siphon under the underpass rather than diverting it to the St. Boniface Industrial Park SRB system.

Assuming the siphon is installed, the discharge line from the dry pond can be configured as a 375 mm gravity pipe by connecting it to the deep invert of the siphon. The 375 mm discharge line would essentially be another branch of the same siphon.

The hydraulic grade line (HGL) of the Dugald Drain as it functions today and for the near future under larger storm events is generally higher than the levels anticipated in the dry pond. The conceptual design of the dry pond has the invert of the dry pond at 230.0 m, with existing ground approximately at 233.0 m. The invert of the Dugald Drain just downstream of the siphon is about 230.9 m, or roughly 0.9 m (3 ft) higher than the bottom of the dry pond and would thus require a pump to dewater the bottom portion of the dry pond.

At higher water levels in the dry pond, the discharge line could either drain by gravity (depending on water levels in the Dugald Drain) or with a small dewatering pump. The discharge line should be configured with a flap gate such that the Dugald Drain will not flow back into the dry pond.

The model indicates that with a 0.1 m^3/s pump (3.5 cfs) the dry pond can be drawn down within acceptable timeframes using only the dewatering pump provided the Dugald Drain has the capacity.

Ultimately there will likely be a gravity land drainage sewer on Dugald Road that could provide a true gravity outlet for the dry pond and service Dugald Road between Mazenod Road and Plessis Road. The discharge line from the underpass dry pond would only have to be connected to the new land drainage sewer to operate and the dewatering pump would no longer be required.

The alignment of the dry pond discharge line shown on Figure 10 is set up such that the dewatering pump is located in the underpass lift station. This simplifies operations and reduces capital costs by placing the entire pump infrastructure in one location, rather than having the dry pond dewatering pump in a different pump station from the underpass lift station. There are other alignments for the 375 mm discharge line that could be explored once the road design has been refined further such as those that minimize the land acquisition from #2049 Dugald Road.

2.4.6 Estimated System Response

Figure 39 shows the estimated peak water levels in the underpass LDS system from the InfoWorks model leading to the lift station and underpass dry pond arising from the 5, 25 and 100 year design storms. The profile runs from the underpass dry pond (at right) to the high end of the underpass LDS system about 320 m south of Dugald Road (at left). The figure also shows the approximate siphon location relative to the LDS, which is not connected to the underpass LDS system but the grade of the LDS system could conflict.

The summer of 1993 and May 2010 storms are not plotted because they generally result in lower HGL's in the underpass system since the peak intensities are less than those in the design storms. However they do result in higher dry pond levels because they produce a very large amount of volume.

Figure 40 contains a graph of the underpass dry pond response for the 5, 25 and 100 year events with only the 0.1 m^3/s dewatering pump operating. Figure 41 contains a graph showing the underpass dry pond response to the two summer 1993 events (July 24-25 and August 8). Figure 42 shows the estimated dry pond response to the May 28-30, 2010 events recorded at Bernie Wolf School. Table 23 summarizes the peak water levels, rises and drawdown times of the proposed dry pond servicing the underpass lift station.

Table 23 - Estimated Peak Water Levels, Rises, and Drawdown Times for Proposed Underpass Dry Pond

Criteria	5 Year	25 Year	100 Year	July 1993	August 1993	May 2010
Water Level (m)	231.40	231.91	232.18	233.00	232.31	232.72
Rise (m)	1.40	1.91	2.18	3.00	2.31	2.72
Drawdown Time (hours)	38	54	64	128	68	111

Table 23 shows that the drawdown times and peak rises are acceptable and generally conform to city standards. The dewatering pumps are generally delayed by at least 1 day after the end of the various storm events in the simulations. The 1.91 m rise for the 25 year storm is a bit beyond the 1.8 m standard, however the dry pond will be in a generally inaccessible area and should be fenced off to prevent unauthorized access.

Figure 43 shows the profile of the discharge line from the underpass dry pond through to the Dugald Drain via the siphon. The profile shows the 375 mm discharge line connecting to the siphon with the dewatering pump pumping from the dry pond into the discharge line. The HGL of the discharge line is essentially at ground level. This is similar to how the pump at SRB 5-1 operates, as the water level at the pump discharge is at ground level.

The profile on Figure 43 also shows how the 375 mm discharge line could be connected to a future gravity land drainage sewer as well as the siphon.

2.5 Drainage on Properties around Underpass

This section will discuss drainage on the properties that surround the proposed Plessis Underpass. The reconfigured drainage systems are shown on Figure 10 like the rest of the proposed underpass drainage system.

The properties that border on the Plessis Underpass are currently configured to drain to the existing ditch system which will be removed when the underpass is constructed. With the removal of the ditches due to the underpass, the drainage patterns will need to be reconfigured such that drainage from these lands is captured before it enters the underpass.

Starting with #2049 Dugald Road (i.e. northwest corner of Plessis and Dugald), the existing drainage patterns can be maintained for the most part. A new swale along the front of the property could be constructed to discharge to a cross culvert leading to the siphon or to the ditch to the west. However, if the property develops further it would be required to provide additional on site storage.

At the southwest corner of Dugald and Plessis, the residential properties that front onto Dugald Road could be reconfigured with a shallow swale with catchbasins leading directly into the siphon. This assumes that the swale is only about 0.3 to 0.6 m below existing prairie and not much deeper.

The remaining properties at the southwest corner that front onto Plessis Road could be configured with a shallow swale and LDS line that ties back into the LDS leading to the Waterside Estates SRB. The Waterside Estates servicing plan shows that the property behind the frontage on Dugald and Plessis is to be serviced by the Waterside Estates System with future LDS conduits. These are shown by the dashed blue line on Figure 10. This land is privately owned and the LDS would require a permanent easement. The preferred tie-in point of the LDS servicing the properties that front onto Dugald Road is the Waterside Estates system since it can provide a higher level of service than the Dugald Drain.

At the southeast corner of Dugald and Plessis, the Transcona Golf Club at #2120 Dugald Road golf course lands could be re-graded with a new swale leading to the golf course's internal drainage system. The Transcona Country Club parking lot at #2070 Dugald Road has an internal drainage system that is pumped into the Dugald Drain which could be reconfigured to discharge to the underpass siphon rather than the Dugald Drain.

Drainage for the East Side Storage facility at #2125 Dugald Road will be difficult to reconfigure due to the distance between the storage buildings and the Dugald Road property line. There is little space to install an underground drainage system, however it could be done with a shallow bury culvert system. The shallow bury culvert system can be configured to provide a connection to the Deep Pond system as well as taking runoff from the backlot drain behind properties on the north side of Dugald Road. This shallow bury culvert system will likely have to be installed in an easement if the storage facility remains since the City's system will rely on it to improve the level of drainage servicing in the area.

North of the railway line and east of Plessis Road the 600 mm interconnection inlet pipe is shown increasing in size to a 900 mm and a 1050 mm pipe. This is essentially replacing what is currently in place at this location. The larger pipes are to service the undeveloped railway lands east of Plessis Road and the private drainage system servicing #1199 Plessis Road.

On the west side of Plessis Road and north of the railway line, the 525 mm pipe on the small drainage system servicing the streets to the west of the underpass may require realignment. Based on the HGL in the Deep Pond System the area has a low level of servicing and changing this pipe size much would do little to improve the level of service. However, if the downstream system is ever upgraded to a better level of service than a 600 mm pipe should be sufficient to keep the HGL below ground surface with the additional length due to the re-routing of the pipe.

The 600 mm re-aligned pipe is shown hugging the western top of slope of the underpass. The slope that this pipe is installed in should generally have a 1.3 factor of safety as is typical for Water and Waste Department requirements from past underpass projects for land drainage infrastructure.

3. Water Distribution

Figure 5 shows the existing water distribution infrastructure within the proposed Plessis Underpass limits. The figure shows that there are a number of watermains around the underpass including a 450/500 mm sub-feedermain line, and a 300 and 200 mm local distribution watermains on Plessis Road and Dugald Road.

Figure 44 shows the proposed watermain realignments required for the underpass design shown on Figure 2 and Figure 3. Most of the water distribution infrastructure may have to be moved due to the lower underpass grade. Note that at this stage, the watermain as-built drawings have not been reviewed to assess depth of cover. This should be done at the functional design stage, however it is expected that changes in cover of more than 0.5 m will likely result in relocation of the mains.

The 450/500 mm watermain that is located on the west side of Plessis Road south of Dugald Road and cuts through to the small subdivision north of the railway line is a relatively important element in the water distribution system in Transcona. Although not officially classed as a feedermain, it provides an important interconnection between the North Transcona and South Transcona Feedermains. Therefore it should be considered as critical infrastructure.

Since the 450/500 mm watermain may be considered a feedermain, it may have to be re-aligned to stay out of the underpass altogether for better access to facilitate maintenance activities. All watermains within the underpass limits should be located such that there is easy access to them in case of a breakage. Figure 44 shows the possible re-alignment around the underpass limits. Note that the 450/500 mm watermain realignment could also replace the existing 300 mm watermain already on Dugald Road. The 450/500 mm watermain has service connections to the properties that front onto Plessis Road unlike feedermains which typically do not have service connections.

The 450/500 mm watermain may also require re-alignment to accommodate the dry pond for the underpass drainage system. The watermain is located on the most conveniently located land for the dry pond and replacing this watermain (which may be required already) is likely the most cost-effective way to store the underpass drainage. Locating the drainage storage element further away would likely be quite costly since the size of discharge pipe required to convey the full amount of the lift station discharge would be quite large.

Also shown on Figure 44 is the possibility of relocating the 200 mm watermain along Plessis Road from Dugald Road to the utility corridor leading to the small subdivision west of Plessis Road and north of the railway line. This will have to be confirmed with a small EPANET hydraulic model (used for water distribution modelling) at the functional design stage. It would be more convenient to tie it back in to the 450/500 mm watermain rather than to the 300 mm watermain. This configuration would yield a hydraulically similar connection comparable to its current configuration via the 300 mm watermain (to be confirmed with the EPANET model).

Slopes or retaining walls that contain any watermains should have a safety factor of at least 1.3. The 450/500 mm watermain should be in a slope or behind a retaining wall that has a factor of safety of no less than 1.5 since that infrastructure is more critical to the overall water distribution system. The 1.5 factor of safety has typically been used for feedermains in other underpass situations such as the 900 and 600 mm diameter feedermains at the Kenaston Boulevard underpass. Location of major infrastructure must also consider maintainability.

Relocation of sub-feedermain and reconnection to existing systems will also need to be considerate of Water and Waste scheduling and timing criteria. A review of other major feedermain projects in the area should be undertaken to ensure schedule conflicts that may result in less than desirable service pressures are avoided, and effects of system flow reversals are considered to minimize “dirty water” complaints.

4. Wastewater Collection

Figure 5 shows the existing wastewater collection infrastructure in the area of the proposed underpass. There are four wastewater sewers located in the underpass limits, the 750 mm secondary interceptor along Dugald Road, the 375 mm wastewater sewer north of Dugald Road on Plessis Road, and the 250 mm gravity and 150 mm forcemain wastewater sewer which combine to a 300 mm gravity wastewater sewer south of Dugald Road.

Figure 45 shows the required wastewater sewer realignments. The figure shows that only the 375 mm wastewater sewer to the north of Dugald Road along Plessis Road needs to be realigned due to the underpass.

The 750 mm secondary interceptor going through the underpass limits provides regional wastewater servicing for lands north and south of Dugald Road and east of Plessis Road. It was installed in 2008 and replaced the now abandoned 1050 mm (west of Plessis Road) and 900 mm (east of Plessis Road) interceptor sewer located just south of the existing Dugald Road alignment.

The invert of the new 750 mm interceptor is about 227.33 m making the top of pipe roughly 228.20 m including the wall thickness. This elevation is approximately 2.2 m below the proposed Dugald Road low point of 230.40 m.

The new 750 mm interceptor was constructed using class 5 precast concrete pipe. Based on a 2.2 m depth to top of pipe, the pipe appears to have enough structural strength to remain in its current alignment and grading.

The depth of cover is a bit shallow relative to the frost depth of 2.4 m (8 ft) depth. The interceptor may require some insulation depending on the final grade of the road.

The wastewater collection infrastructure to the south (i.e. the 150 mm forcemain, 250 mm and 300 mm gravity sewer) is deep enough to stay in place with the proposed underpass road grade. However, the infrastructure in some spots may have less than the 2.4 m minimum depth of cover. The invert of the 300 mm wastewater sewer is at approximately 228.23 m at the south property line of Dugald Road. This means that there may be about 1.8 m depth of cover depending on the final road grading. If this situation occurs, the wastewater sewers should be insulated in the areas with less than the minimum depth of cover to prevent freezing.

The gravity wastewater collection infrastructure (250 and 300 mm diameter sewers) to the south is also installed at minimum depth and minimum slopes. And the system must also tie into the 750 mm secondary interceptor. There is little opportunity to change the vertical grade of the pipe to accommodate further depth without a small lift station (not recommended).

The 150 mm forcemain provides wastewater servicing to structures around Symington Yard which is more than 1.6 km (1 mile) south of Dugald Road. The forcemain is a private connection and was constructed in 1972 and has reportedly been problematic and leaky. The City's database indicates the material type is unknown. The forcemain may require outright replacement or diversion to the newer 250/300 mm wastewater sewer. Further investigations (e.g. as-built drawings or even inspections) should be undertaken if the forcemain is to be left under new pavement or in a generally inaccessible location. There may also be overburden loading constraints depending on the pipe material.

The 375 mm wastewater sewer north of Dugald Road will need to be realigned as the grade of the pipe will be above the 7 m deep low point of the underpass. The most likely location for realignment is in the west slope of the underpass although the east slope may be feasible pending the final elevations of the road. The slopes will require a 1.3 safety factor for the wastewater infrastructure.

The 375 mm wastewater sewer cannot realistically be diverted away from the 750 mm Dugald Interceptor to the north. The 375 mm wastewater sewer system only services the small subdivision north of the railway line, and west of Plessis Road as well as the Manitoba Housing complex at #1184 Plessis Road and frontage properties north of the railway line.

The wastewater system to the north leads to the small Pandora Wastewater Pump Station (located at the southeast corner of Pandora Avenue West and Plessis Road). The pump station likely has little to no capacity to accept additional flows since its service area has been fully built out. Additionally, a gravity system is preferred over a pump station due to the additional maintenance and costs required for the pump station.

With the watermain relocations, the wastewater, land drainage and water infrastructure will be in close proximity to one another in the west bank of the underpass. It will likely take a relatively large amount of space and careful planning to accommodate the infrastructure in the same side slope of the underpass. Depending on the final underpass road grade, the wastewater sewer could be moved to the east side of the underpass.

Various wastewater sewer service pipes may also require insulation as depth of cover may be an issue. However, these will not be entirely known until construction begins (and services are encountered or even accidentally broken).

With any major change in cover of gravity flow pipe systems, the pipeline structures should be reviewed to ensure change in the loading condition does not adversely affect the utility.

5. Cost Estimates

The estimated construction costs are summarized in Table 24 below.

Table 24 - Estimated Construction Costs for City Owned Utilities around Proposed Plessis Underpass

Item	Estimated Construction Cost (2012 $)
Dugald Drain Siphon (Option 1)	$4,500,000
Dugald Drain Diversion incl. South Transcona LDS Trunk (Option 2)	$44,800,000
Underpass LDS System	$11,500,000
LDS Diversions from Lands Surrounding the Underpass (incl. Deep Pond Outlet)	$2,200,000
Watermain Relocations	$1,300,000
Wastewater Sewer Relocations	$400,000
Total Option 1 (Dugald Drain Siphon)	**$19,900,000**
Total Option 2 (Dugald Drain Diversion)	**$60,200,000**

The costs summarized in Table 24 are considered Class 5 estimates as defined by the American Association of Cost Engineers International (AACEi) and are based on the utility relocations discussed as a part of this memo. The costs are considered appropriate for conceptual use with an expected accuracy range of -50% to + 100% of actual construction costs. The costs do not include burdens such as contingencies, engineering and financials.

The estimated $44.8 million for the Dugald Drain Diversion to the Esselmont Drain is largely due to the South Transcona LDS Trunk construction (about $39.5 million) and the construction of the 2.7 Ha SRB east of Plessis Road (about $3.5 million).

The funding for constructing the South Transcona LDS Trunk and the future 2.7 Ha SRB will largely be driven by other sources of funding including but not limited to the Basement Flooding Relief Program, Trunk Sewer Rates, and the Combined Sewer Overflow Reduction Program.

6. Conclusions and Recommendations

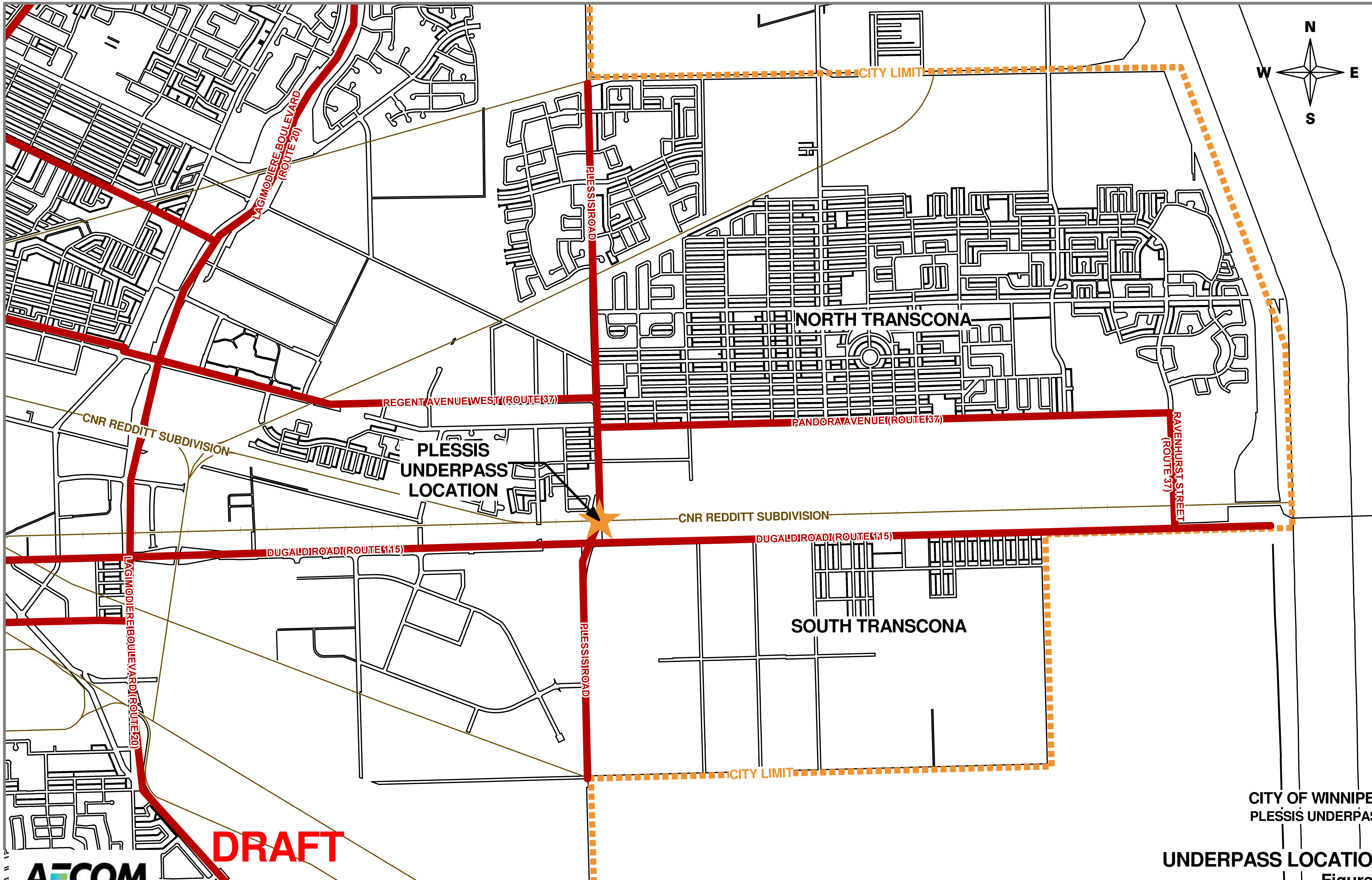
Based on the discussion in the previous sections, we conclude and recommend the following:

1. The Dugald Drain should be maintained using a 1500 mm siphon that can be tied in to a future land drainage sewer leading to the future South Transcona LDS Trunk as shown on Figure 10.
2. The west end of the siphon should be configured for a future 1500 mm land drainage sewer connection to the South Transcona LDS Trunk when it is constructed.
3. Maintaining the flow split between the Dugald Drain and the Deep Pond system as shown on Figure 10 would maintain approximately the existing level of service on the Dugald Drain.
4. The 1500 mm siphon with an LDS connection to SRB 5-1 and the South Transcona LDS Trunk would improve the level of service on the Dugald Drain without the Deep Pond outlet.
5. If the Deep Pond outlet is included as a part of the underpass reconfiguration, it should be afforded a positive gate so as to block flow from entering the Deep Pond from the Dugald Drain when it is vulnerable.
6. The Deep Pond outlet can not be configured as an underground land drainage sewer connection to the siphon without a control structure (i.e. weir) as it would significantly increase the volume contribution to the Deep Pond.
7. Diverting the Dugald Drain to the Esselmont Drain and the St. Boniface Industrial Park system is not feasible unless the South Transcona LDS Trunk and the 2.7 Ha SRB east of Plessis Road is constructed first.
8. The Dugald Drain Diversion to the Esselmont Drain (Option 2) is not feasible due to the large amount of construction cost required to provide the South Transcona LDS Trunk outlet.
9. The underpass will require a lift station discharging to a storage element exclusively servicing the underpass system.
10. The underpass storage element should discharge to the Dugald Drain using a 0.1 m^3/s pump configured to turn on once water levels in the Dugald Drain subside.
11. The underpass storage element should be located as close as reasonably possible to the underpass lift station.
12. The City owned land to the west of the underpass and south of the railway line appears to be large enough to install a dry pond to store the underpass lift station discharge if the 450/500 mm watermain already on the lands is moved.
13. The underpass storage element should be configured to discharge to a future gravity land drainage sewer when the South Transcona LDS Trunk is constructed.
14. The underpass minor drainage system should be able to handle a 25 year event.
15. The underpass major drainage system should be able to handle a 50 year event.
16. Drainage from lands around the underpass will need to be reconfigured to prevent it from entering the underpass drainage system as shown on Figure 10.
17. Most of the watermains within the underpass limits will likely have to be moved as per Figure 44.
18. Only the 375 mm wastewater sewer north of Dugald Road will have to be moved as a result of the current road grading plan.
19. The remaining wastewater sewer infrastructure may be deep enough to remain in place however the parts that are less than 2.4 m deep to obvert may have to be insulated to prevent freezing.

Respectfully Submitted,

David M. R. Enns, P.Eng.
Municipal Engineer
Community Infrastructure

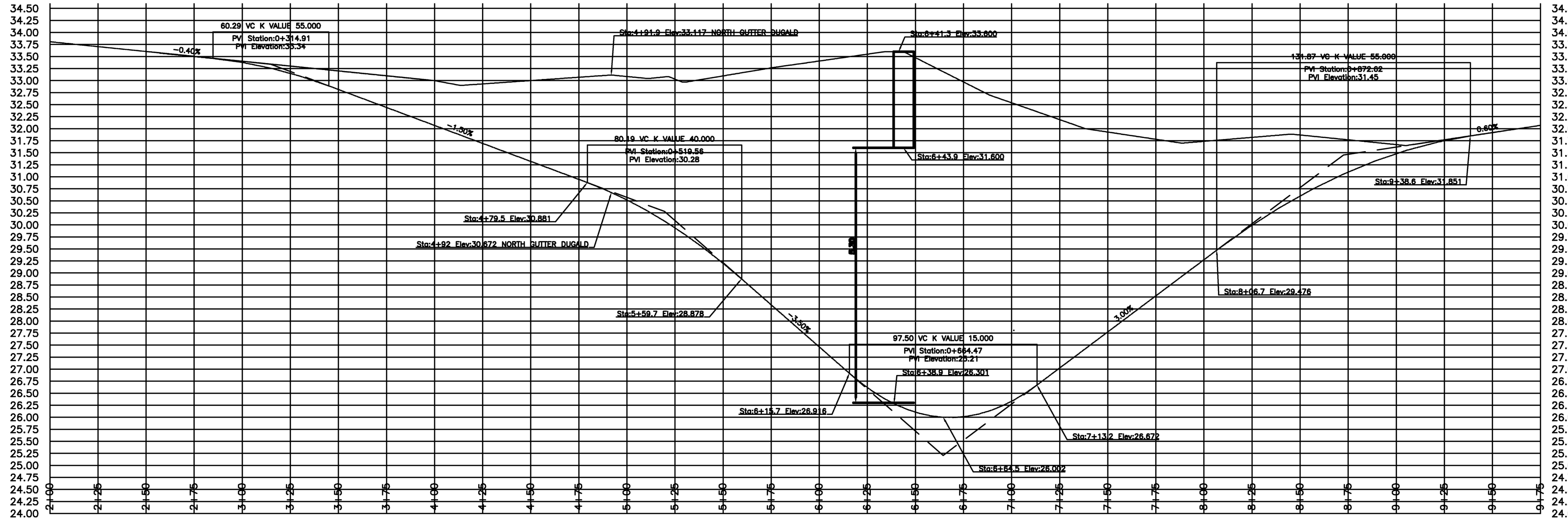
FIGURES

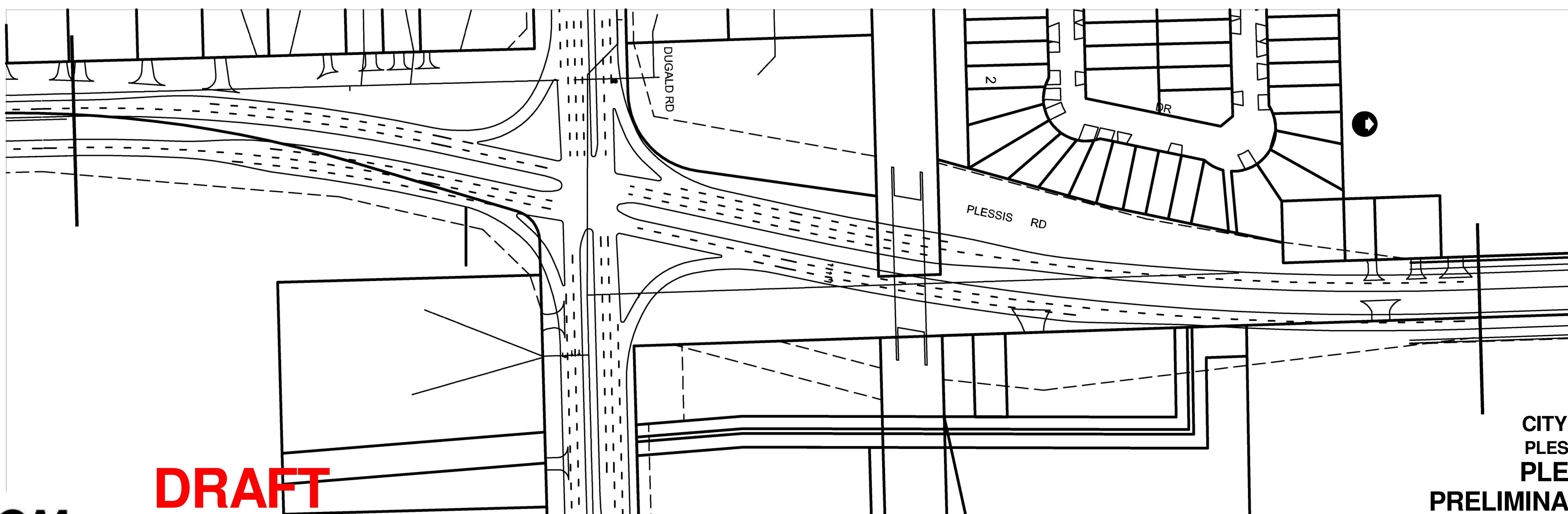


CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS LOCATION

Figure 1





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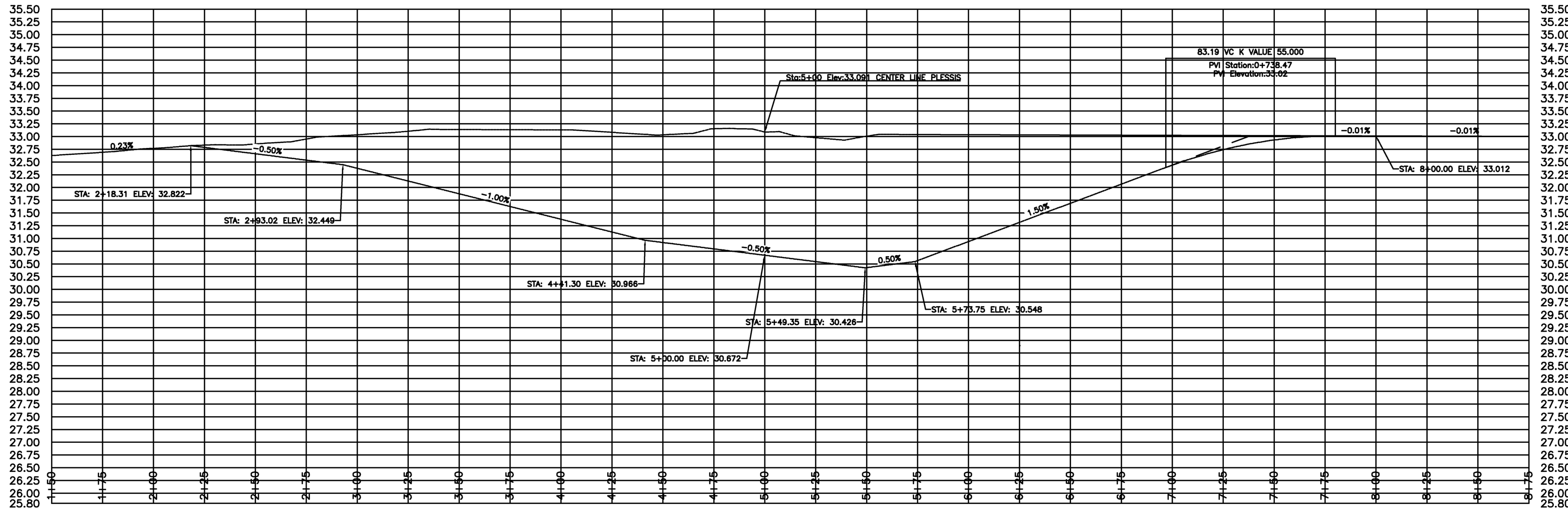
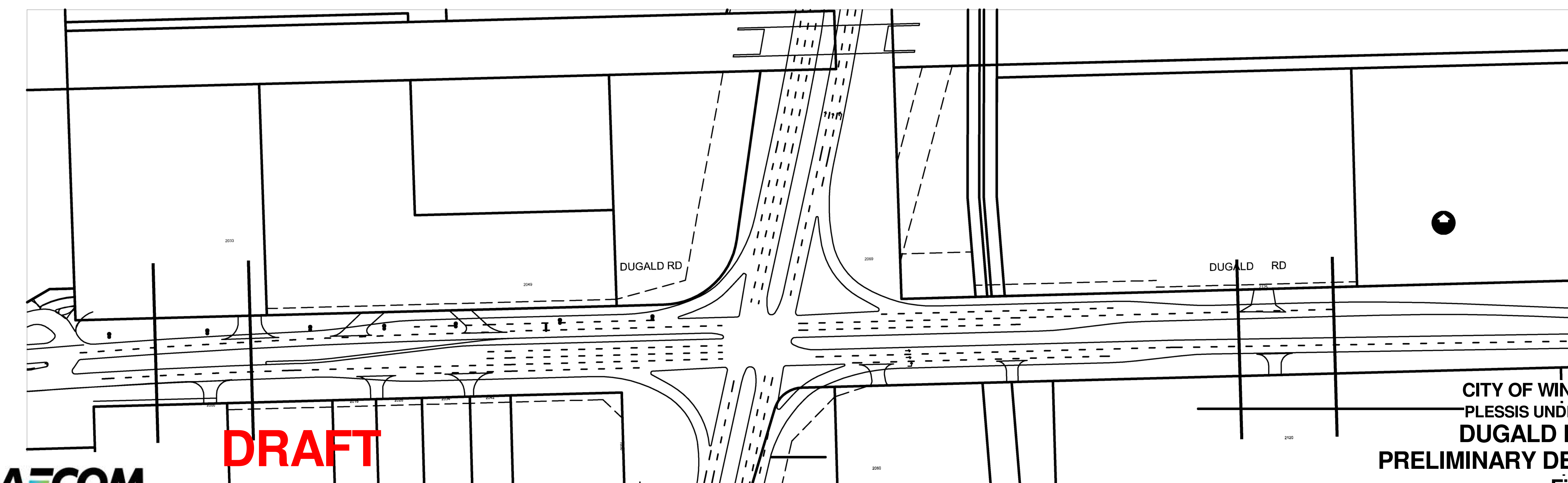
CITY OF WINNIPEG
PLESSIS UNDERPASS
PLESSIS ROAD
PRELIMINARY DESIGN
Figure 2





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CITY OF WINNIPEG
PLESSIS UNDERPASS
DUGALD ROAD
PRELIMINARY DESIGN
Figure 3

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LEGEND:

- PROPOSED STREET RIGHT-OF-WAY
- PROPERTY REQUIREMENT FOR STREET RIGHT-OF-WAY
- CITY OWNED PROPERTY

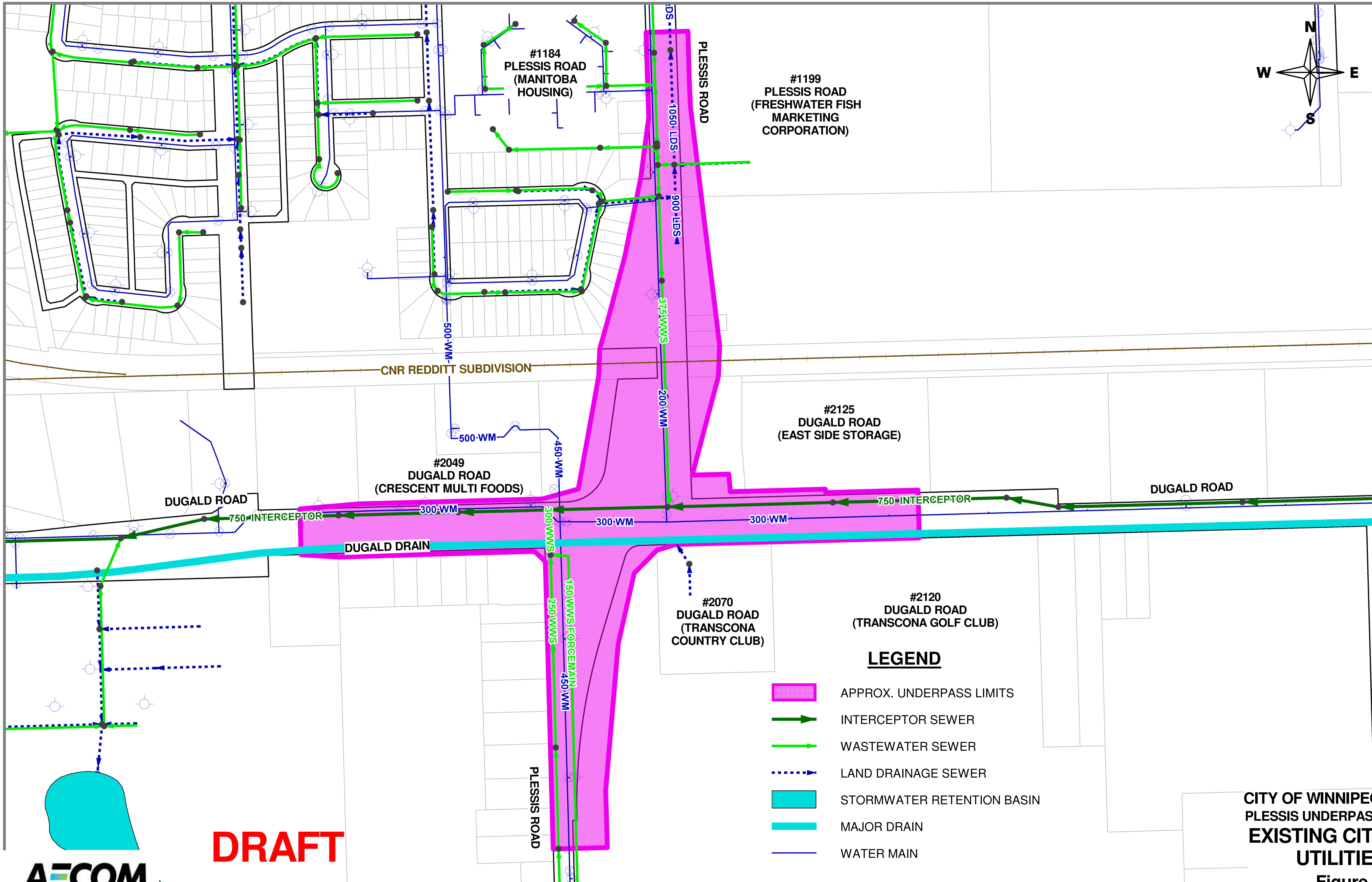
THE CITY OF WINNIPEG
PUBLIC WORKS DEPARTMENT
TRANSPORTATION DIVISION

DRAWN BY: AP
DATE: 2012-02-15
SCALE: 1:3000

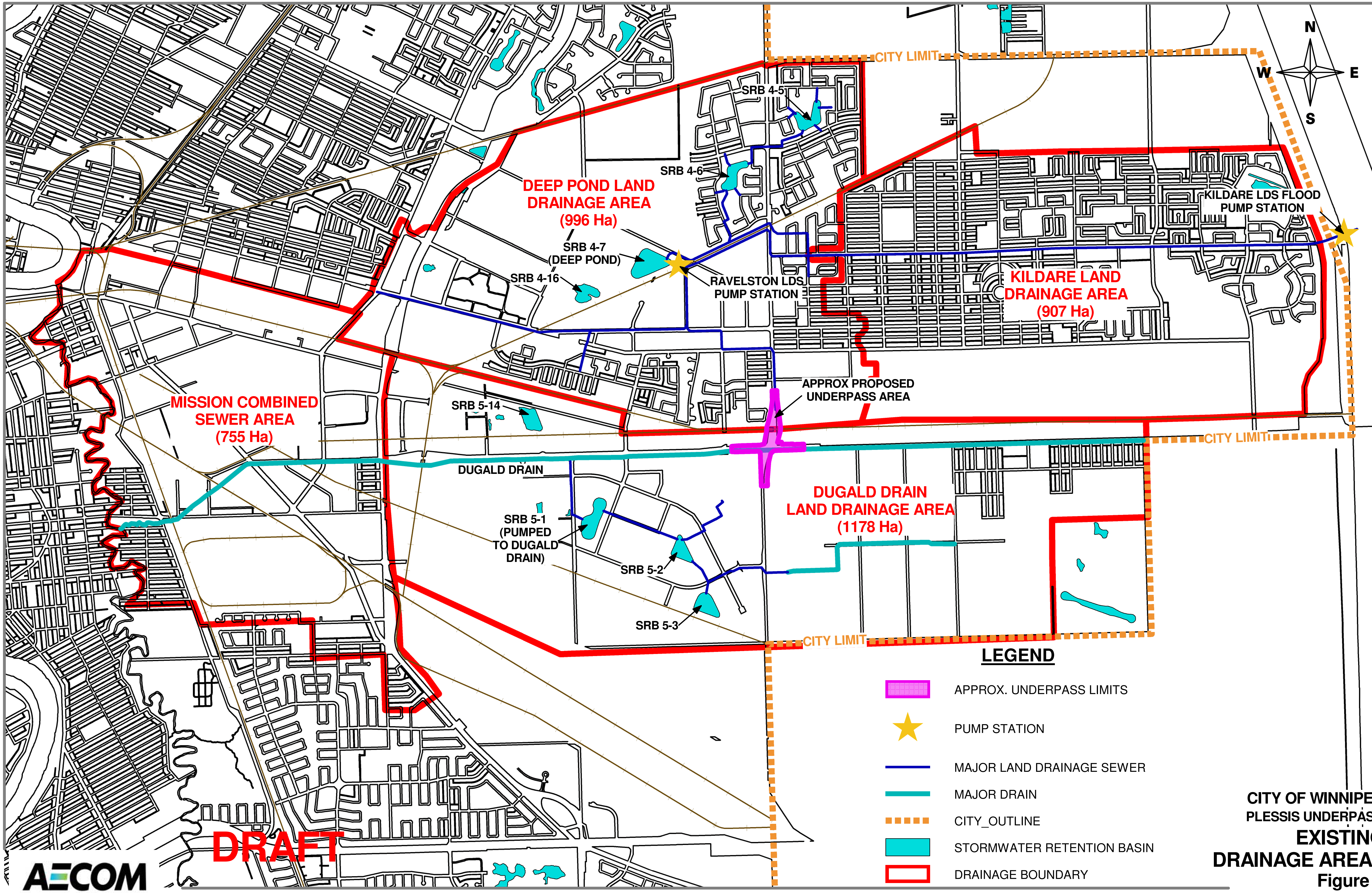
PLESSIS ROAD UNDERPASS
CONCEPTUAL PROPERTY REQUIREMENT
PLESSIS RD AND DUGALD RD

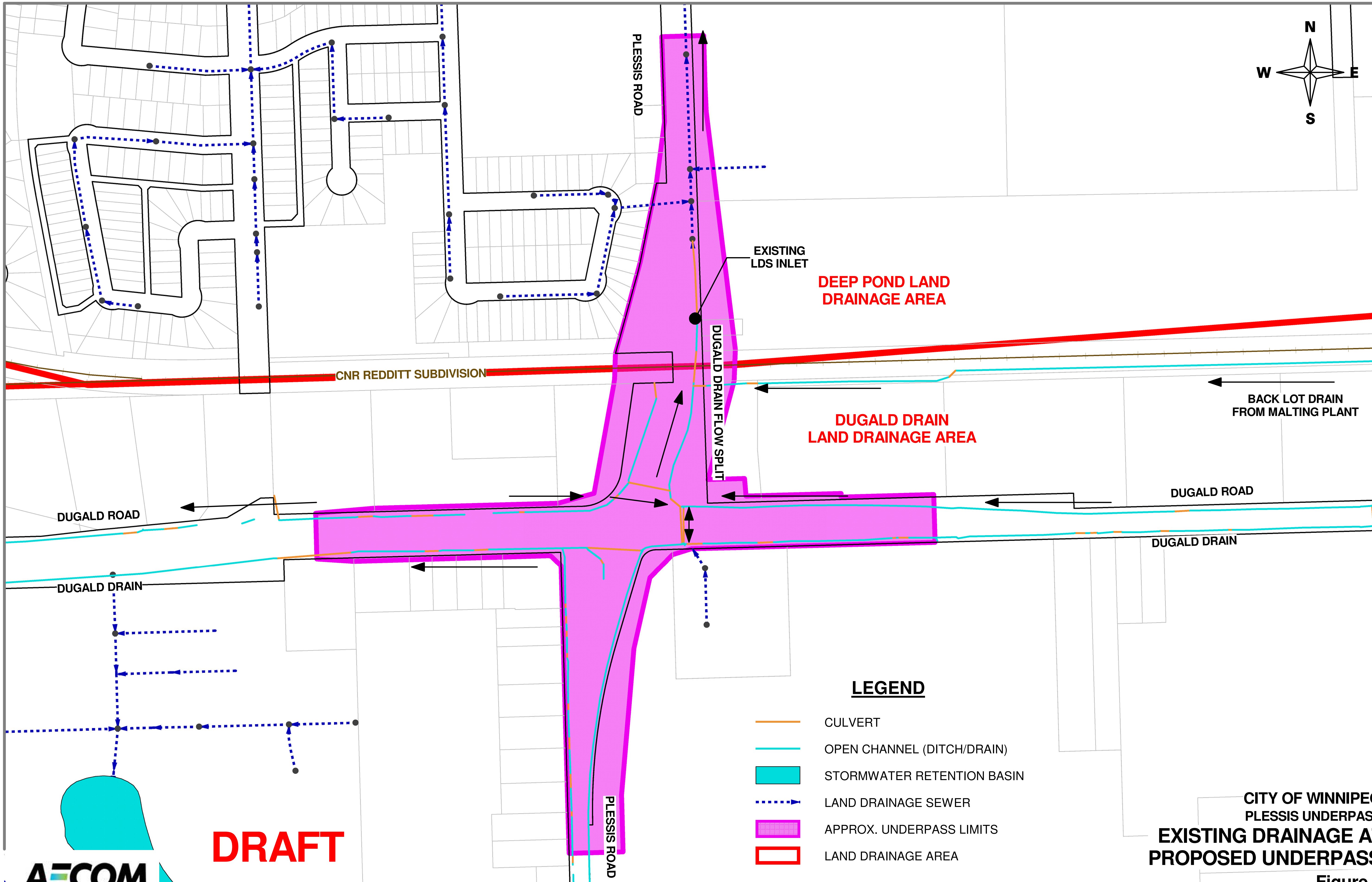
DRAWING NO.

CITY OF WINNIPEG
PLESSIS UNDERPASS
PROPERTY REQUIREMENTS
Figure 4

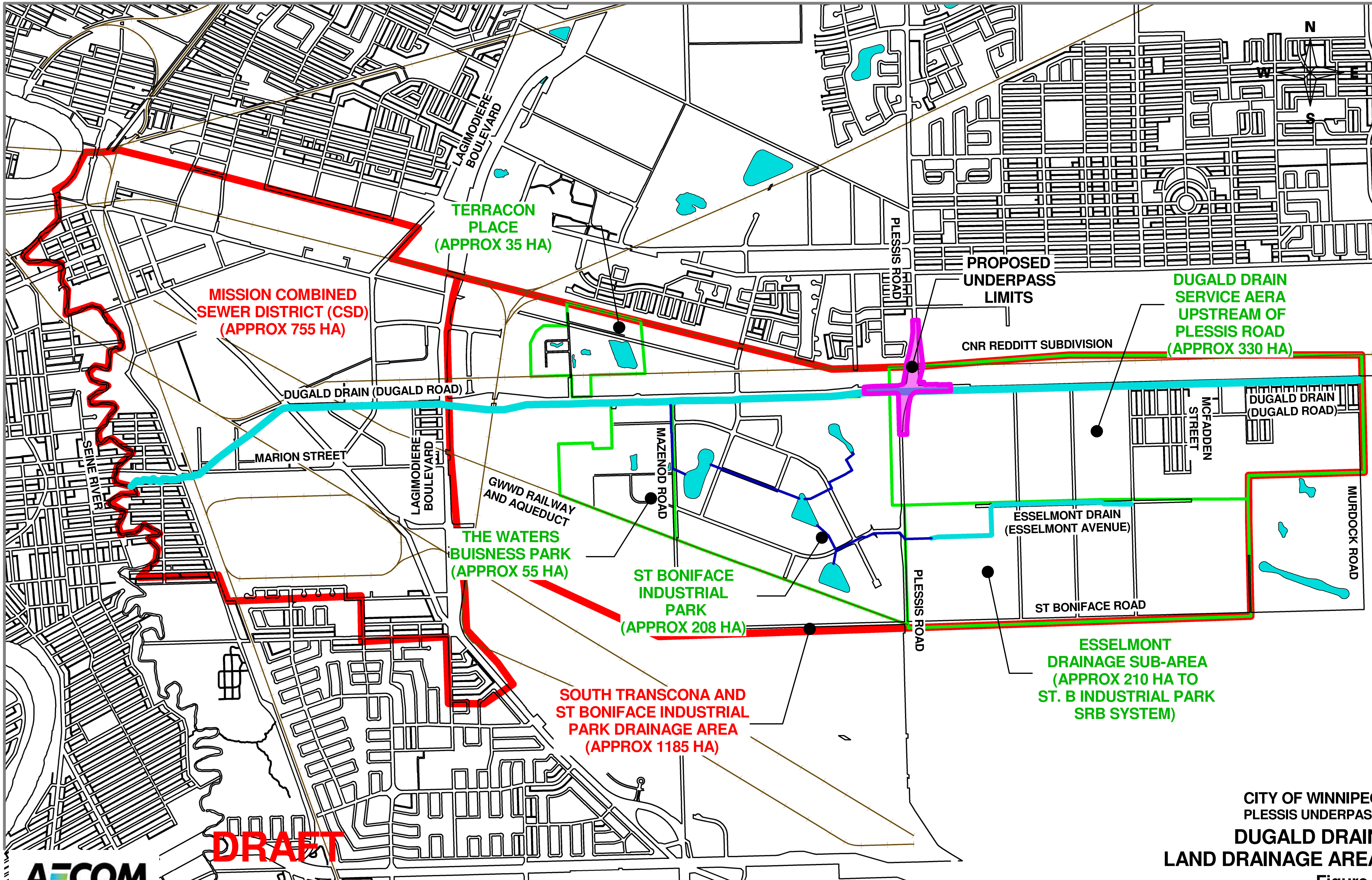


CITY OF WINNIPEG
PLESSIS UNDERPASS
EXISTING CITY UTILITIES
Figure 5


CITY LIMIT
SRB 4-5
SRB 4-6
DEEP POND LAND
DRAINAGE AREA
(996 Ha)
SRB 4-7
(DEEP POND)
SRB 4-16
RAVELSTON LDS
PUMP STATION
KILDARE LDS FLOOD
PUMP STATION
KILDARE LAND
DRAINAGE AREA
(907 Ha)
APPROX PROPOSED
UNDERPASS AREA
MISSION COMBINED
SEWER AREA
(755 Ha)
SRB 5-14
CITY LIMIT
DUGALD DRAIN
DUGALD DRAIN
LAND DRAINAGE AREA
(1178 Ha)
SRB 5-1
(PUMPED
TO DUGALD
DRAIN)
SRB 5-2
SRB 5-3
CITY LIMIT
N
W
E
S
DRAFT
LEGEND
APPROX. UNDERPASS LIMITS
PUMP STATION
MAJOR LAND DRAINAGE SEWER
MAJOR DRAIN
CITY_OUTLINE
STORMWATER RETENTION BASIN
DRAINAGE BOUNDARY
AECOM
CITY OF WINNIPEG
PLESSIS UNDERPASS
EXISTING
DRAINAGE AREAS
Figure 6



CITY OF WINNIPEG
PLESSIS UNDERPASS
EXISTING DRAINAGE AT PROPOSED UNDERPASS
Figure 7



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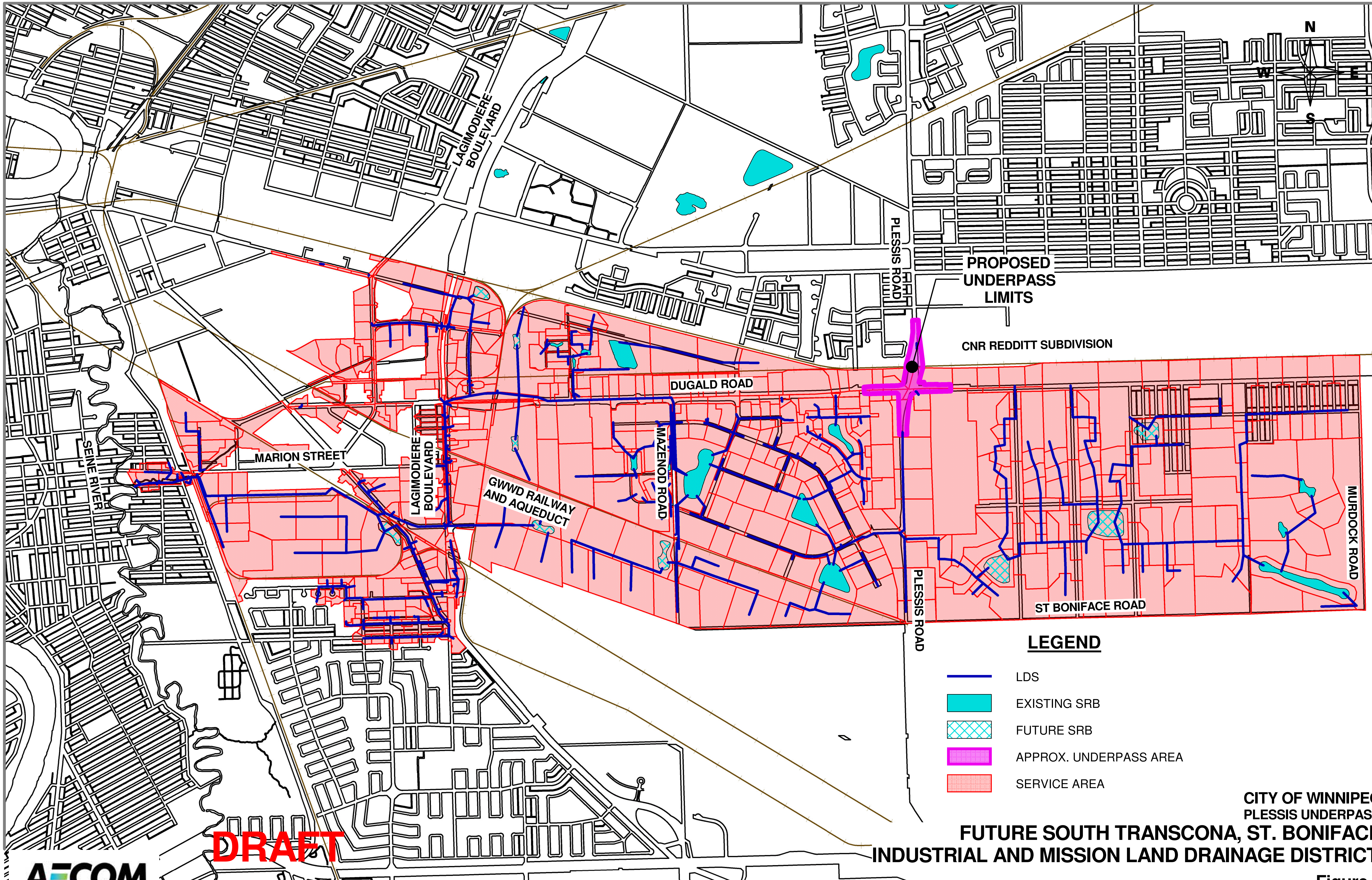
CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DRAIN LAND DRAINAGE AREA

Figure 8



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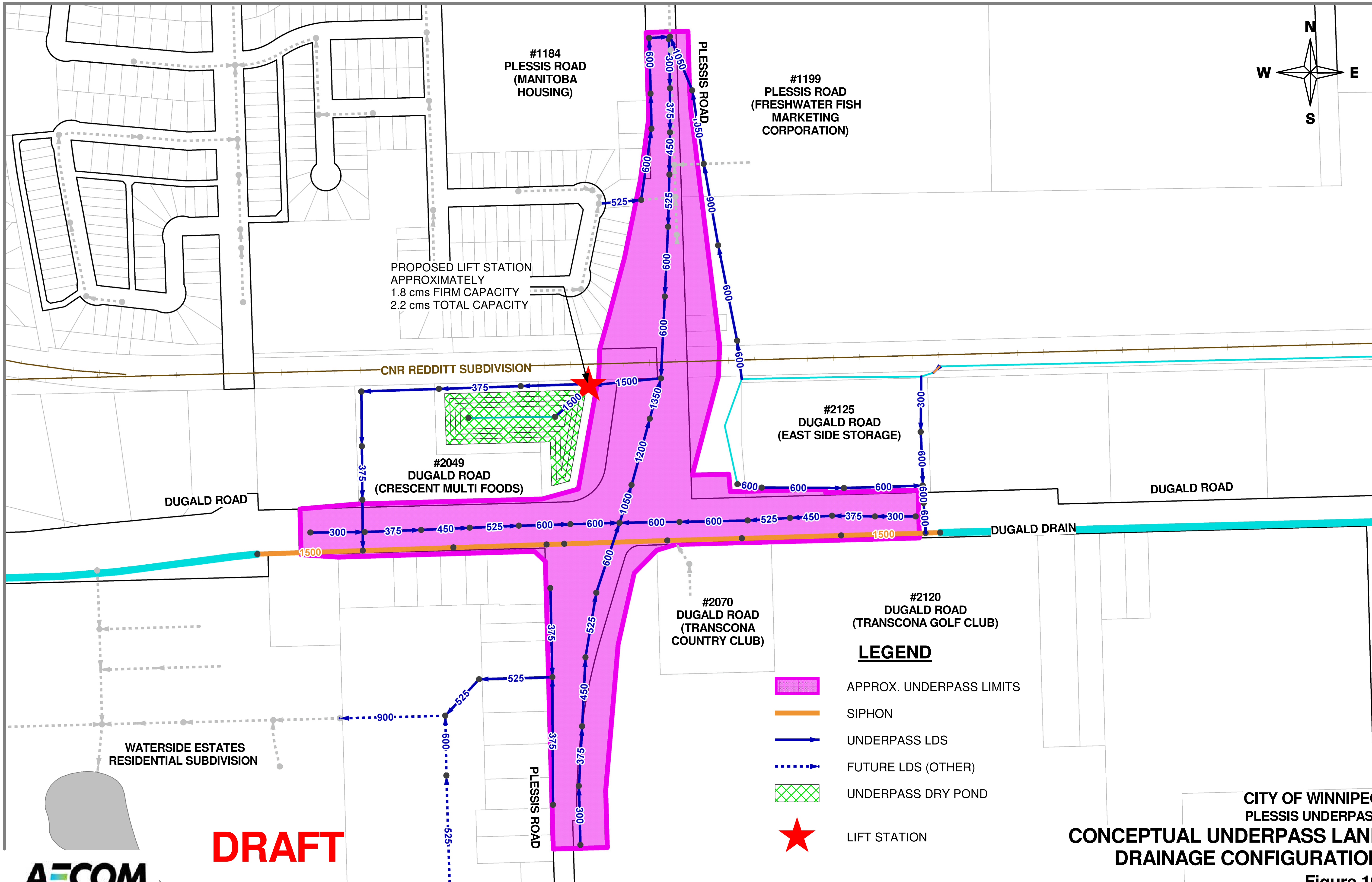
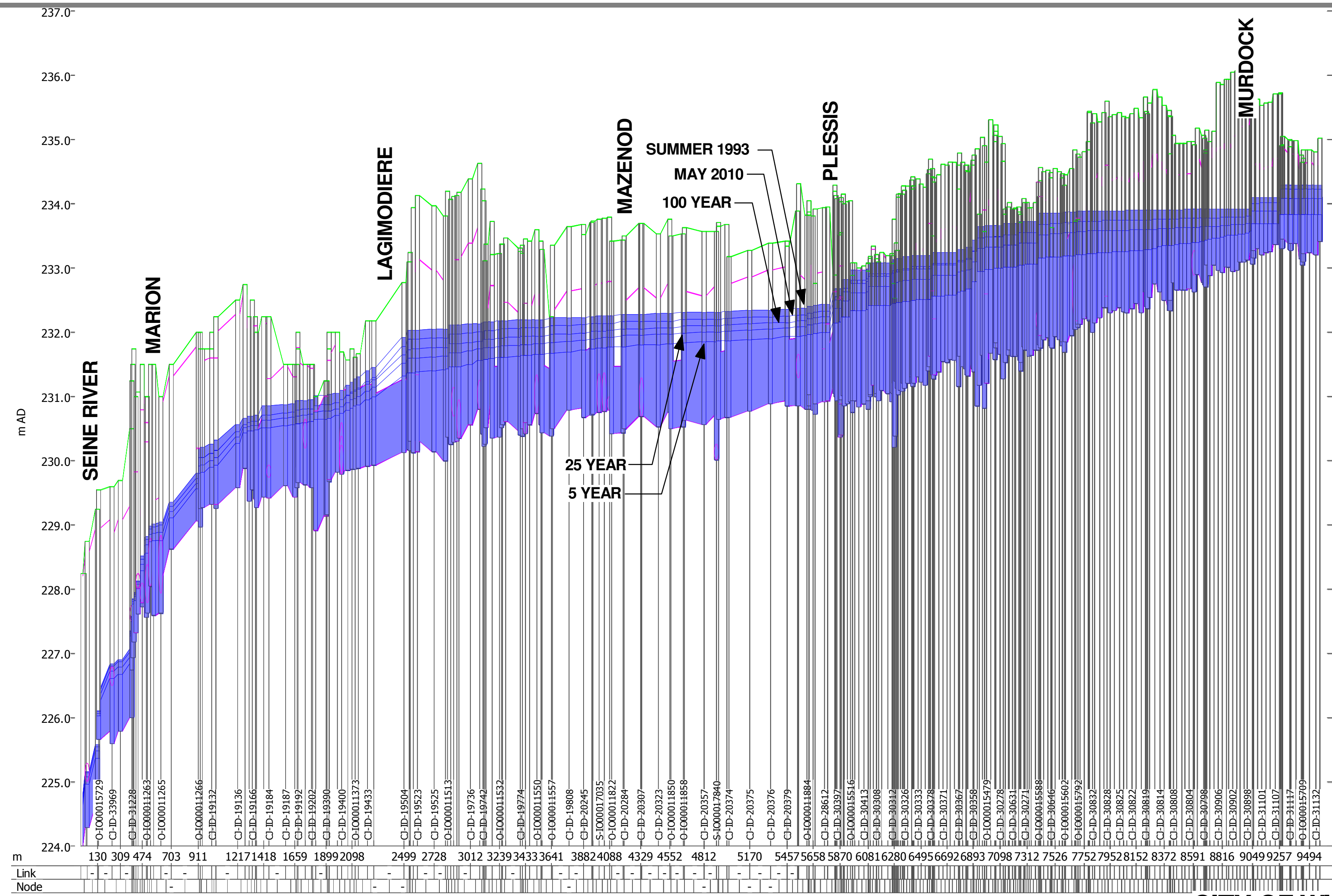
CITY OF WINNIPEG
PLESSIS UNDERPASS

FUTURE SOUTH TRANSCONA, ST. BONIFACE INDUSTRIAL AND MISSION LAND DRAINAGE DISTRICT

Figure 9



CITY OF WINNIPEG
PLESSIS UNDERPASS
CONCEPTUAL UNDERPASS LAND DRAINAGE CONFIGURATION
Figure 10



CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DRAIN EXISTING CONDITIONS

Figure 11

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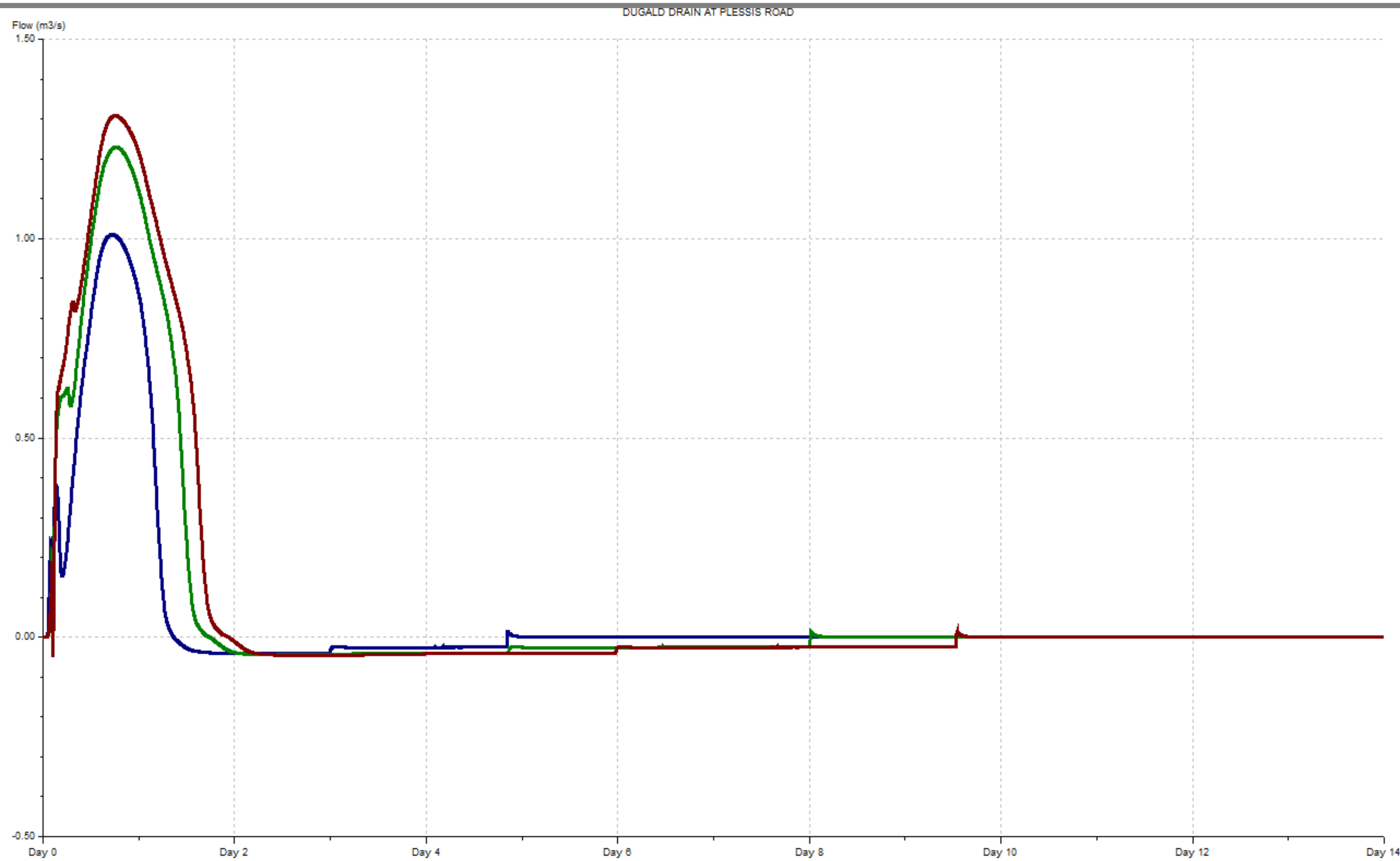
AECOM



	Flow (m3/s)		
	Min	Max	Volume (m3)
5 Year Storm	-0.040	1.010	59904.264
25 Year Storm	-0.044	1.229	92135.581
100 Year Storm	-0.052	1.308	109023.200

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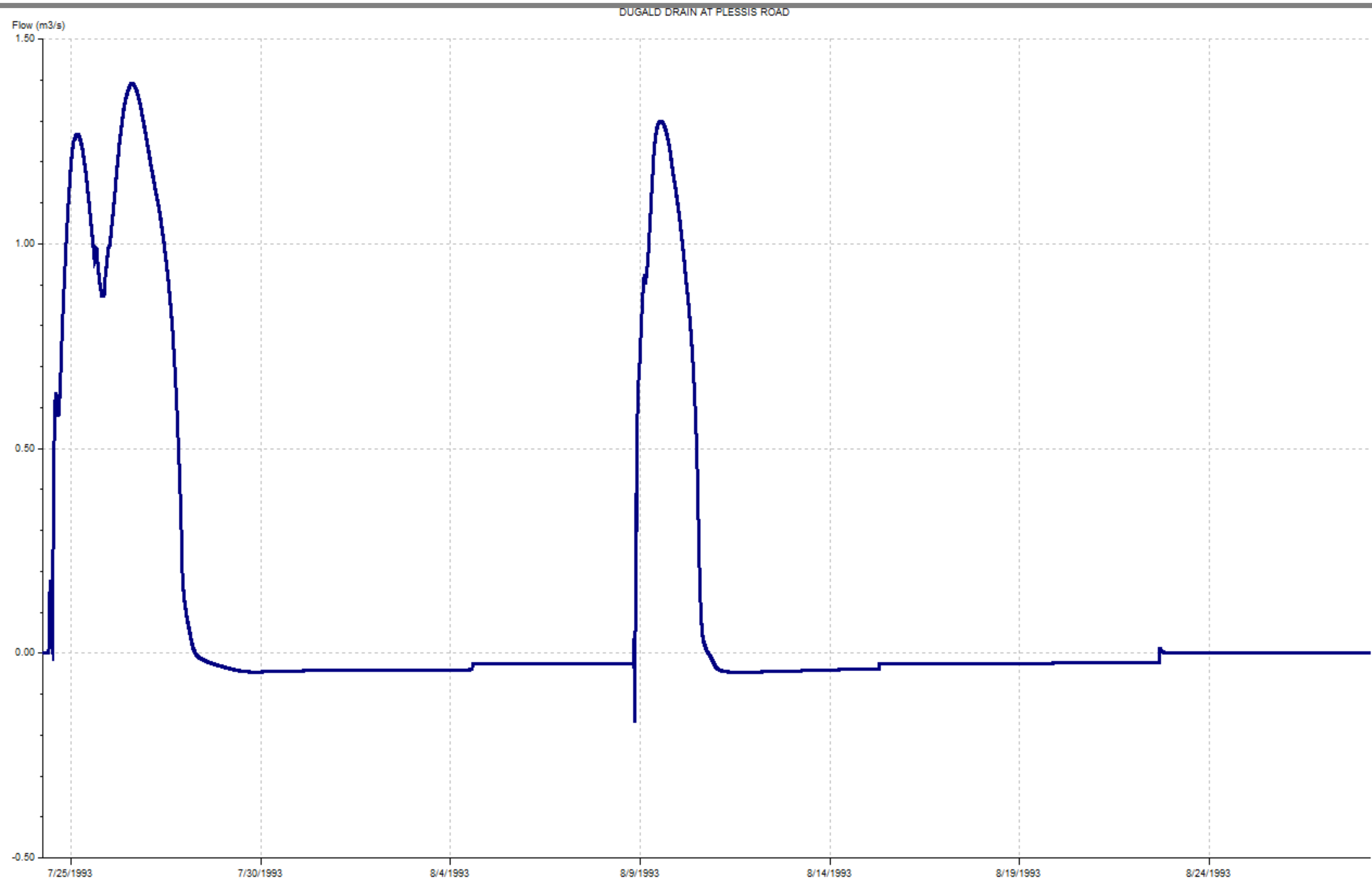
CITY OF WINNIPEG
PLESSIS UNDERPASS

DESIGN STORM FLOWS IN DUGALD DRAIN AT PLESSIS

Figure 12



	Flow (m3/s) Min	Flow (m3/s) Max	Flow (m3/s) Volume (m3)
Summer 1993 Storms	-0.171	1.391	390026.650

CITY OF WINNIPEG
PLESSIS UNDERPASS

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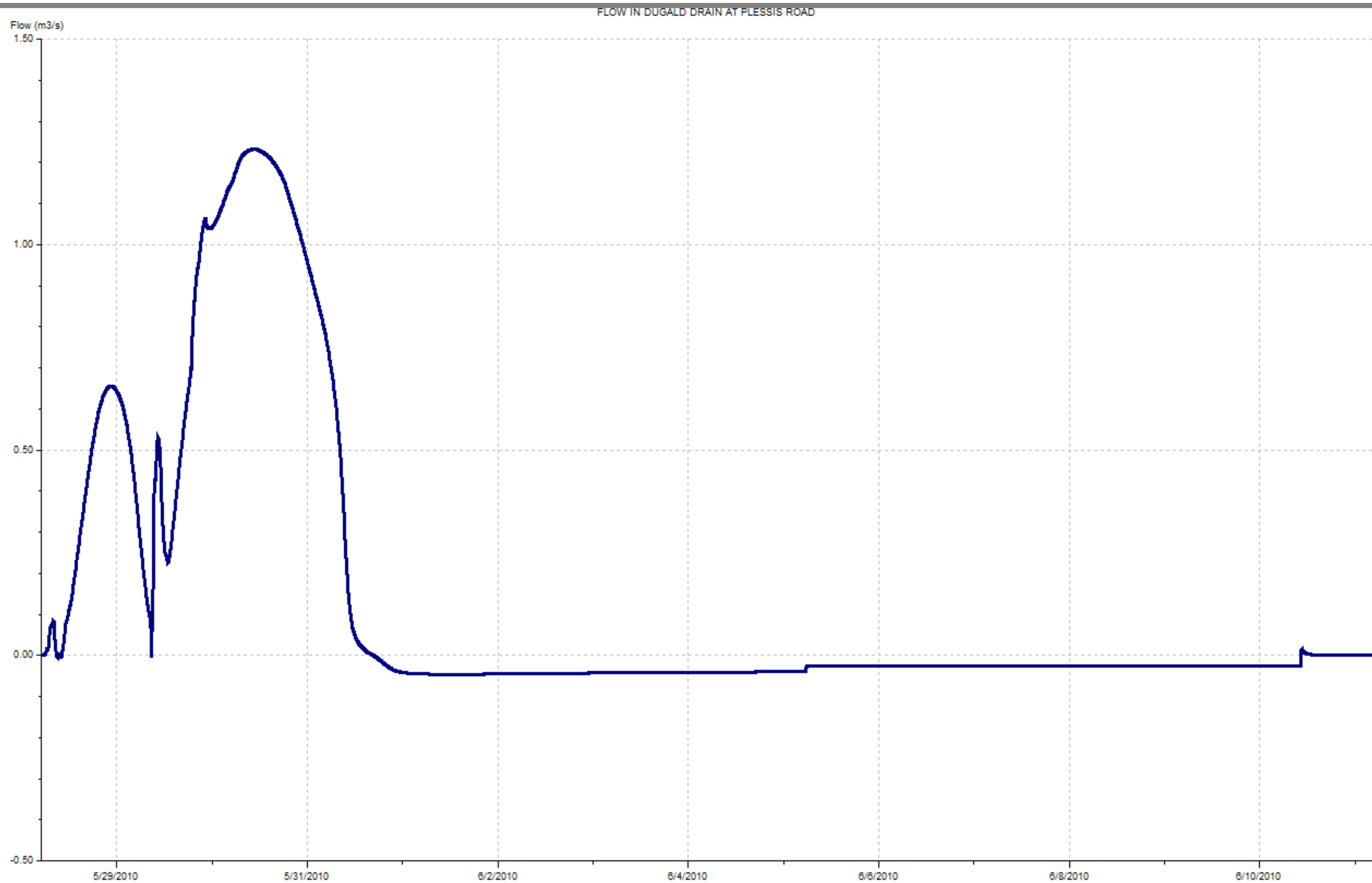
AECOM

SUMMER 1993 STORM FLOWS IN DUGALD DRAIN AT PLESSIS

Figure 13



	Flow (m3/s)		
	Min	Max	Volume (m3)
May 2010 Storms	-0.046	1.232	164291.784

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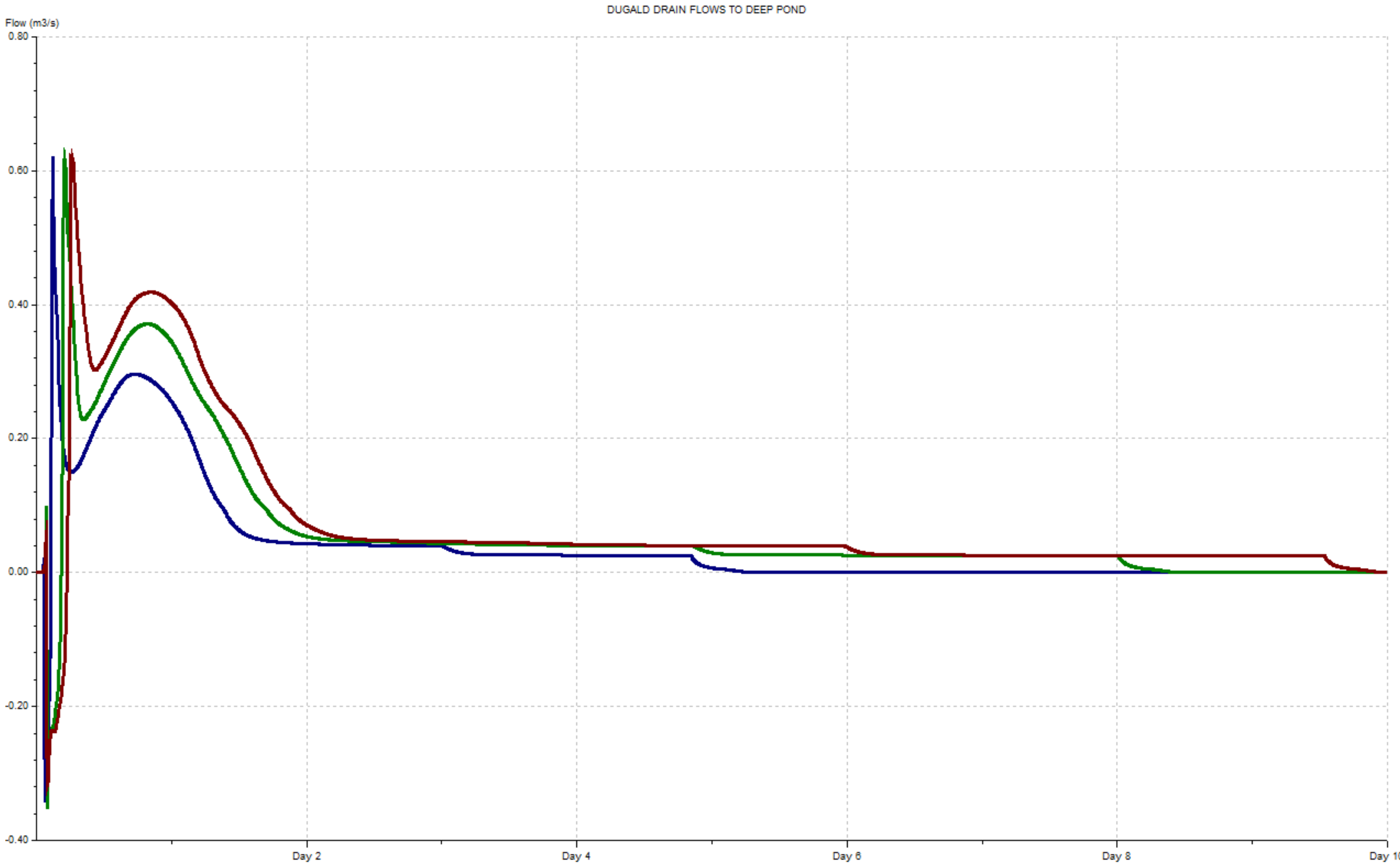
CITY OF WINNIPEG
PLESSIS UNDERPASS

MAY 2010 STORM FLOWS IN DUGALD DRAIN AT PLESSIS

Figure 14



		Flow (m3/s)		
		Min	Max	Volume (m3)
5 Year Storm		-0.320	0.598	35432.944
25 Year Storm		-0.329	0.617	54450.273
100 Year Storm		-0.317	0.621	65651.346

CITY OF WINNIPEG
PLESSIS UNDERPASS

DESIGN STORM FLOWS FROM DUGALD DRAIN TO DEEP POND

Figure 15

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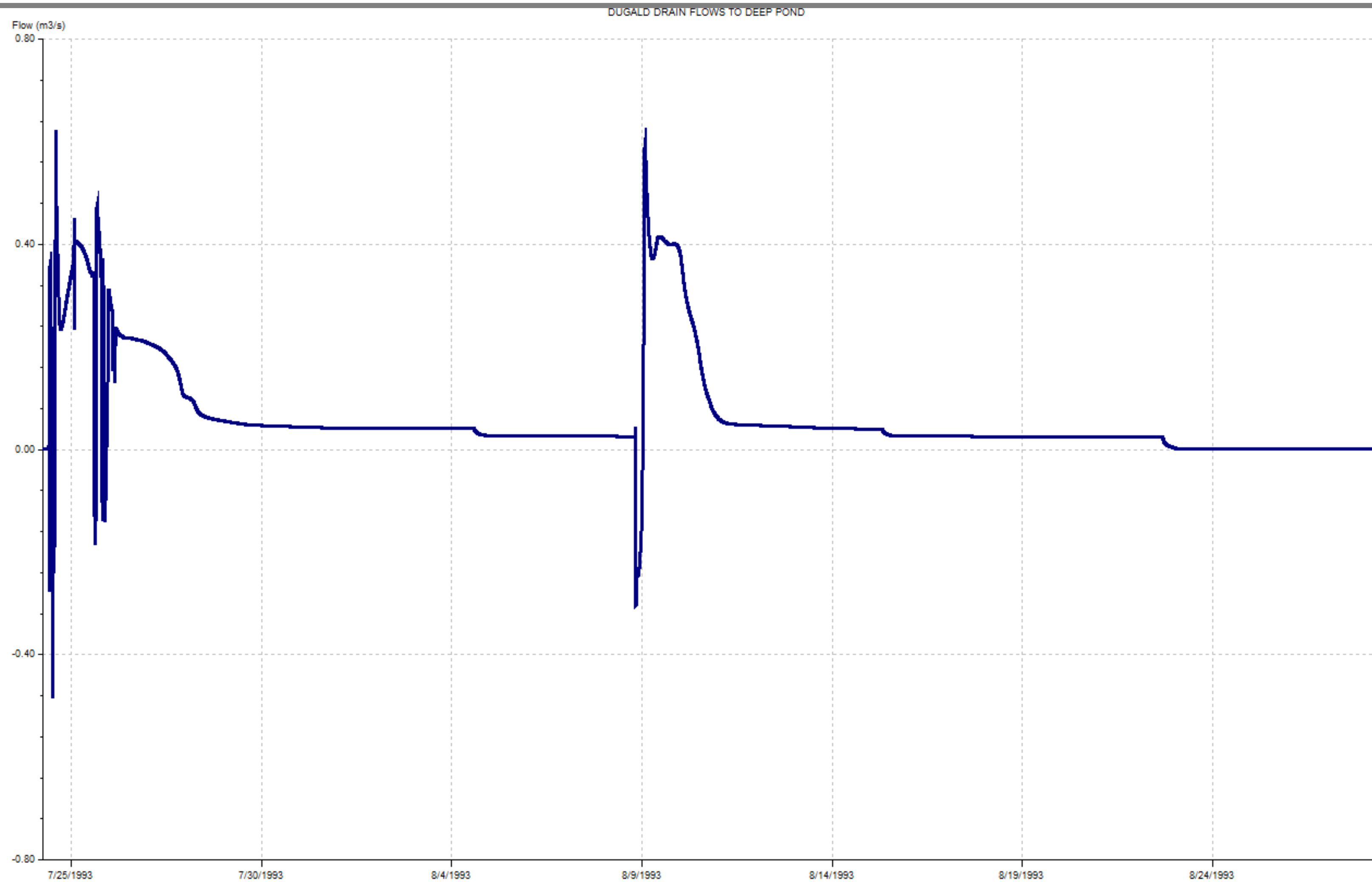
AECOM



	Flow (m3/s)		
	Min	Max	Volume (m3)
Summer 1993 Storms	-0.454	0.623	191539.540

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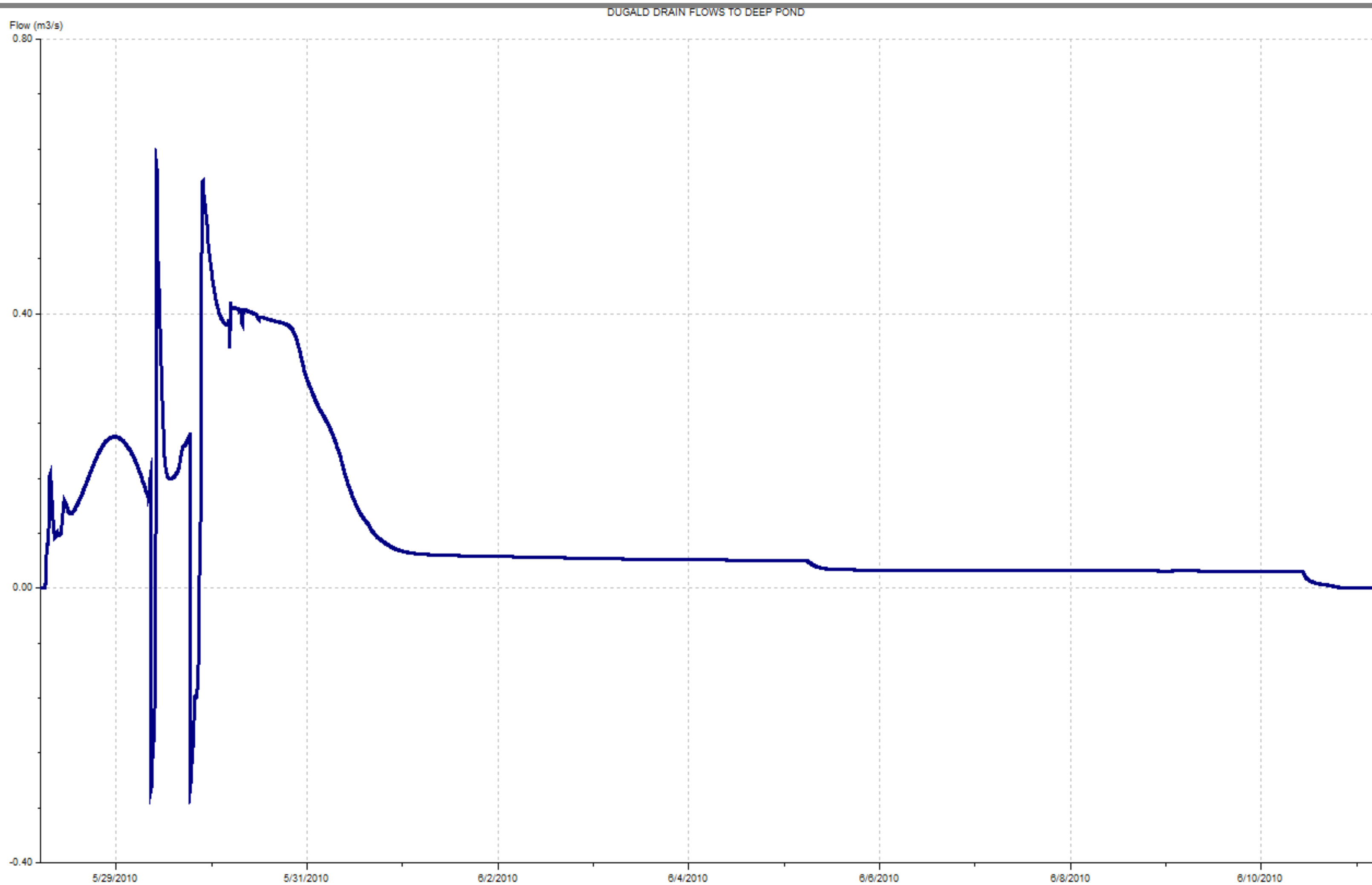
CITY OF WINNIPEG
PLESSIS UNDERPASS

SUMMER 1993 STORM FLOWS FROM DUGALD DRAIN TO DEEP POND

Figure 16



	Flow (m3/s) Min	Flow (m3/s) Max	Flow (m3/s) Volume (m3)
May 2010 Storms	-0.295	0.619	99707.950

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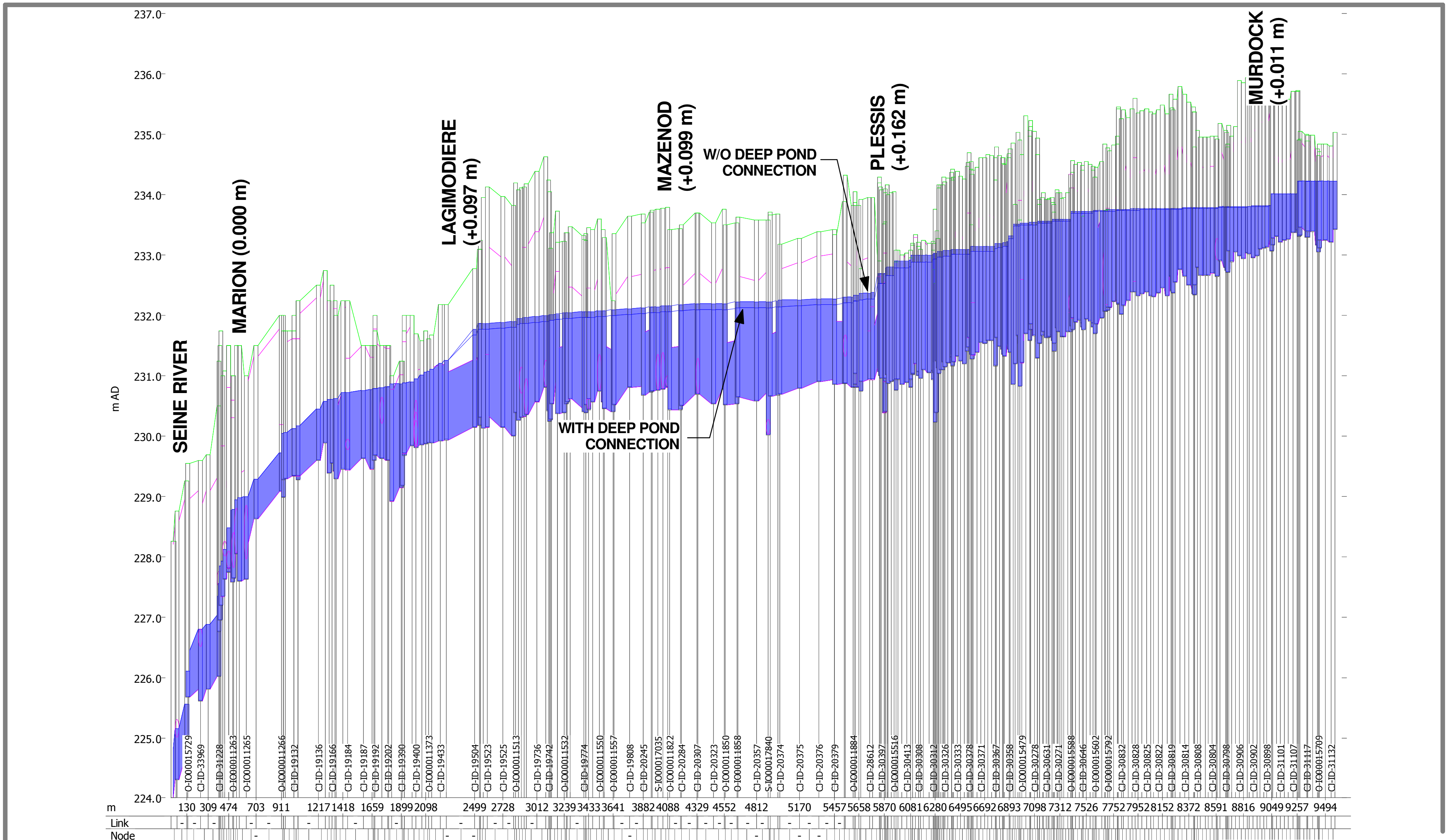
CITY OF WINNIPEG
PLESSIS UNDERPASS

MAY 2010 STORM FLOWS FROM DUGALD DRAIN TO DEEP POND

Figure 17



CITY OF WINNIPEG
PLESSIS UNDERPASS

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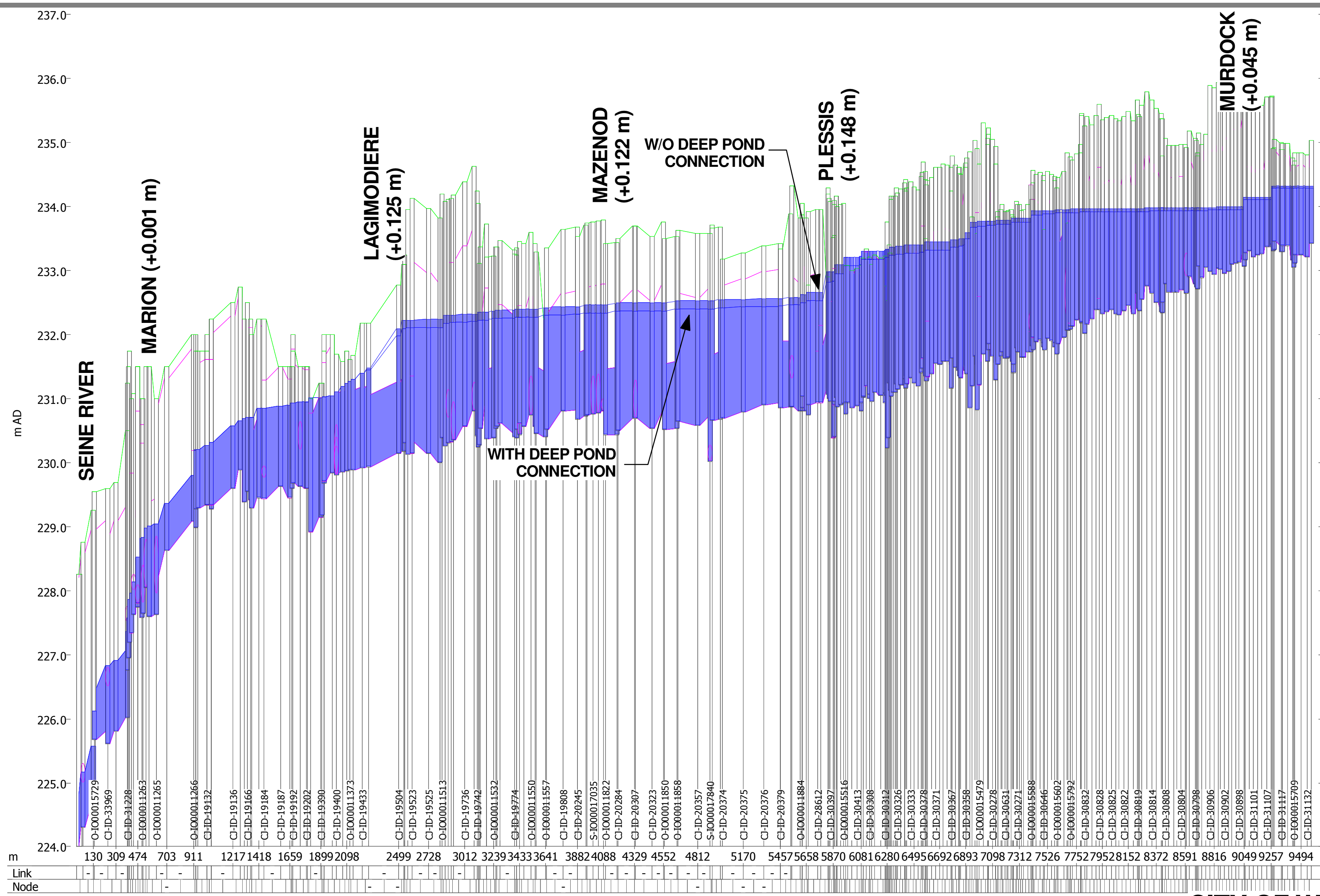
DUGALD DRAIN EXISTING CONDITIONS 100 YEAR STORM DEEP POND OUTLET IMPACT

Figure 18

AECOM



CITY OF WINNIPEG

PLESSIS UNDERPASS

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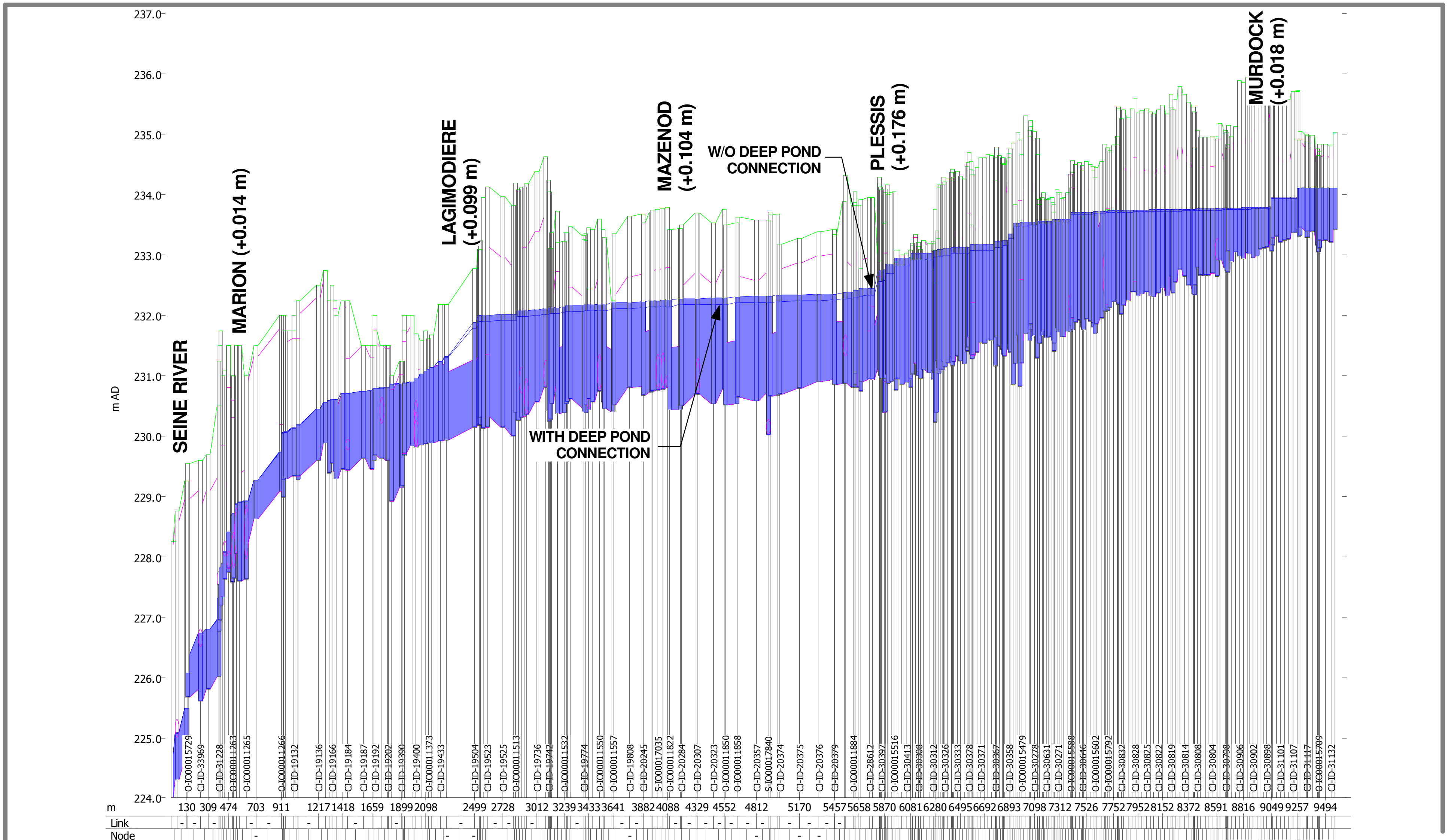
DUGALD DRAIN EXISTING CONDITIONS SUMMER 1993 STORMS DEEP POND OUTLET IMPACT

Figure 19

AECOM



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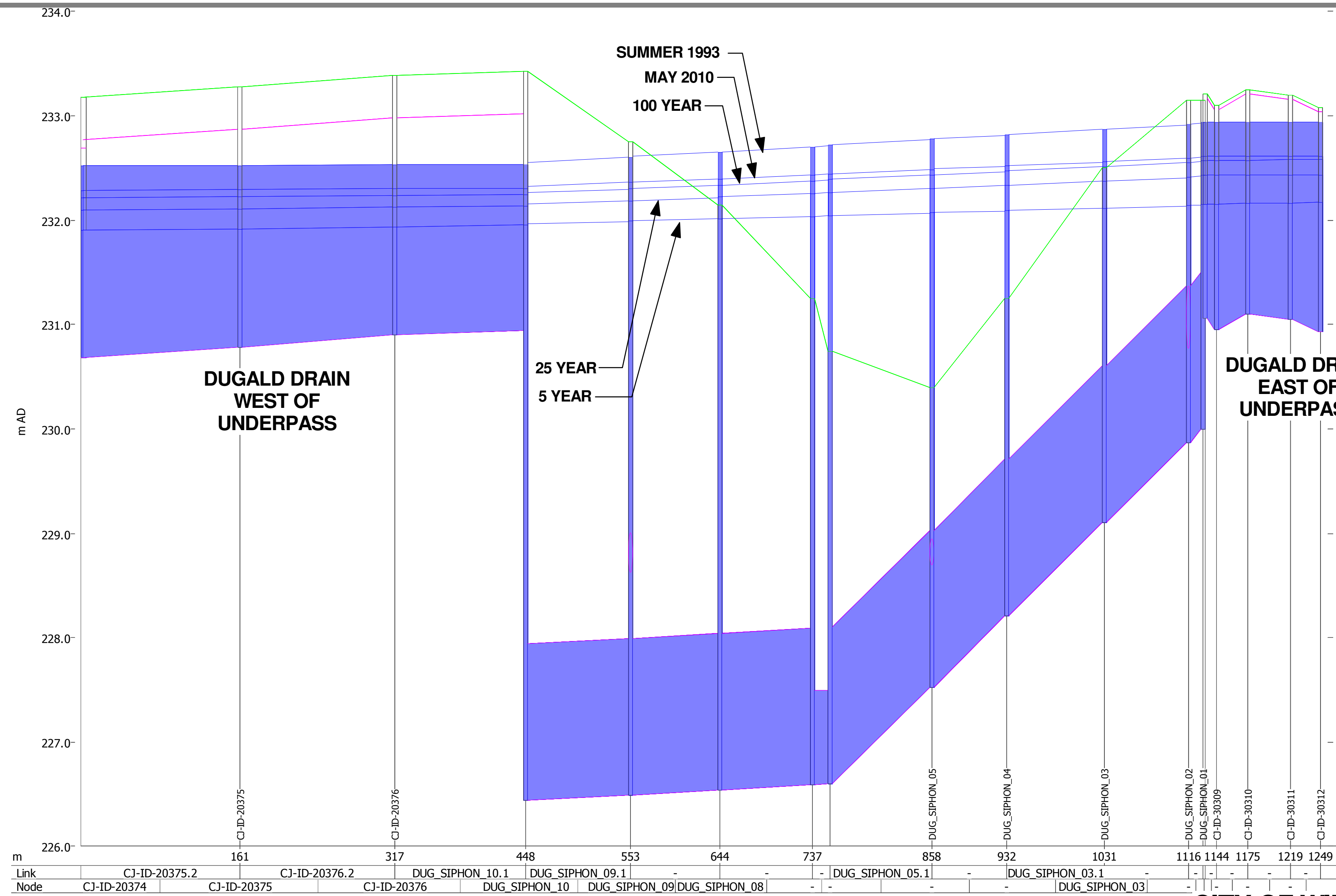
AECOM

CITY OF WINNIPEG

PLESSIS UNDERPASS

DUGALD DRAIN EXISTING CONDITIONS MAY 2010 STORMS DEEP POND OUTLET IMPACT

Figure 20



CITY OF WINNIPEG
PLESSIS UNDERPASS

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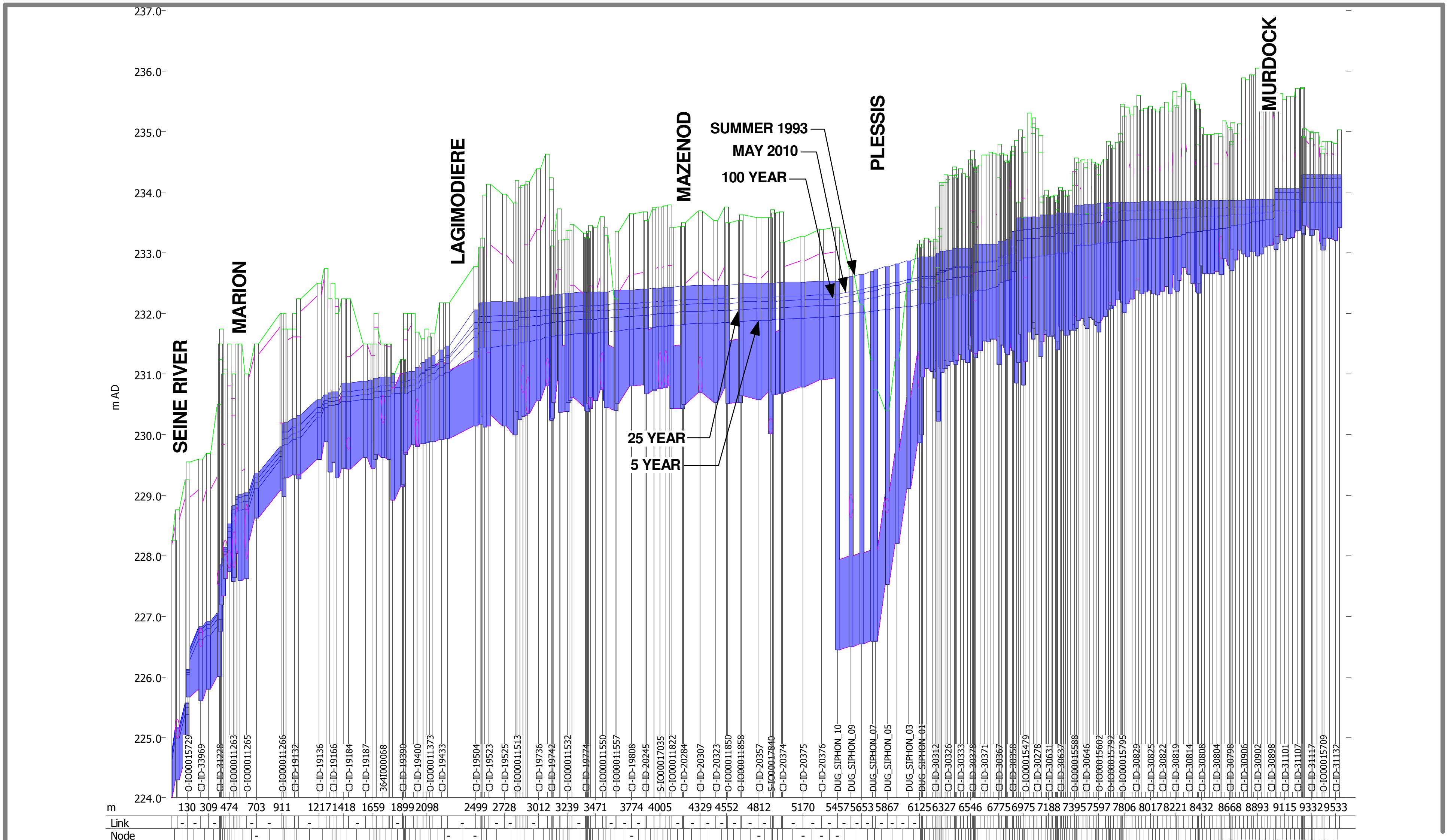
SIPHON PROFILE WITH INTERCONNECTION

Figure 21

AECOM



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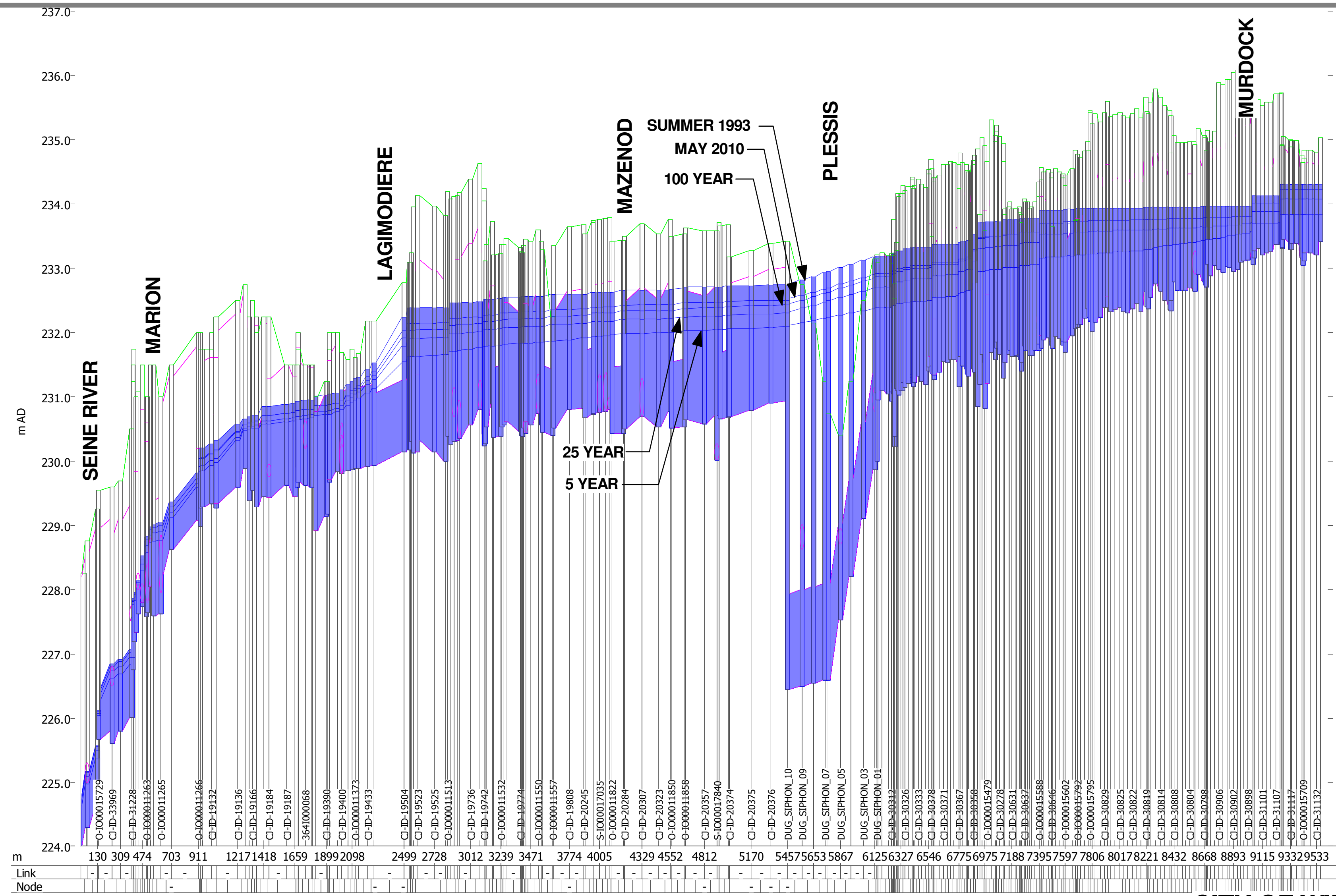
CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DRAIN WITH SIPHON

Figure 22



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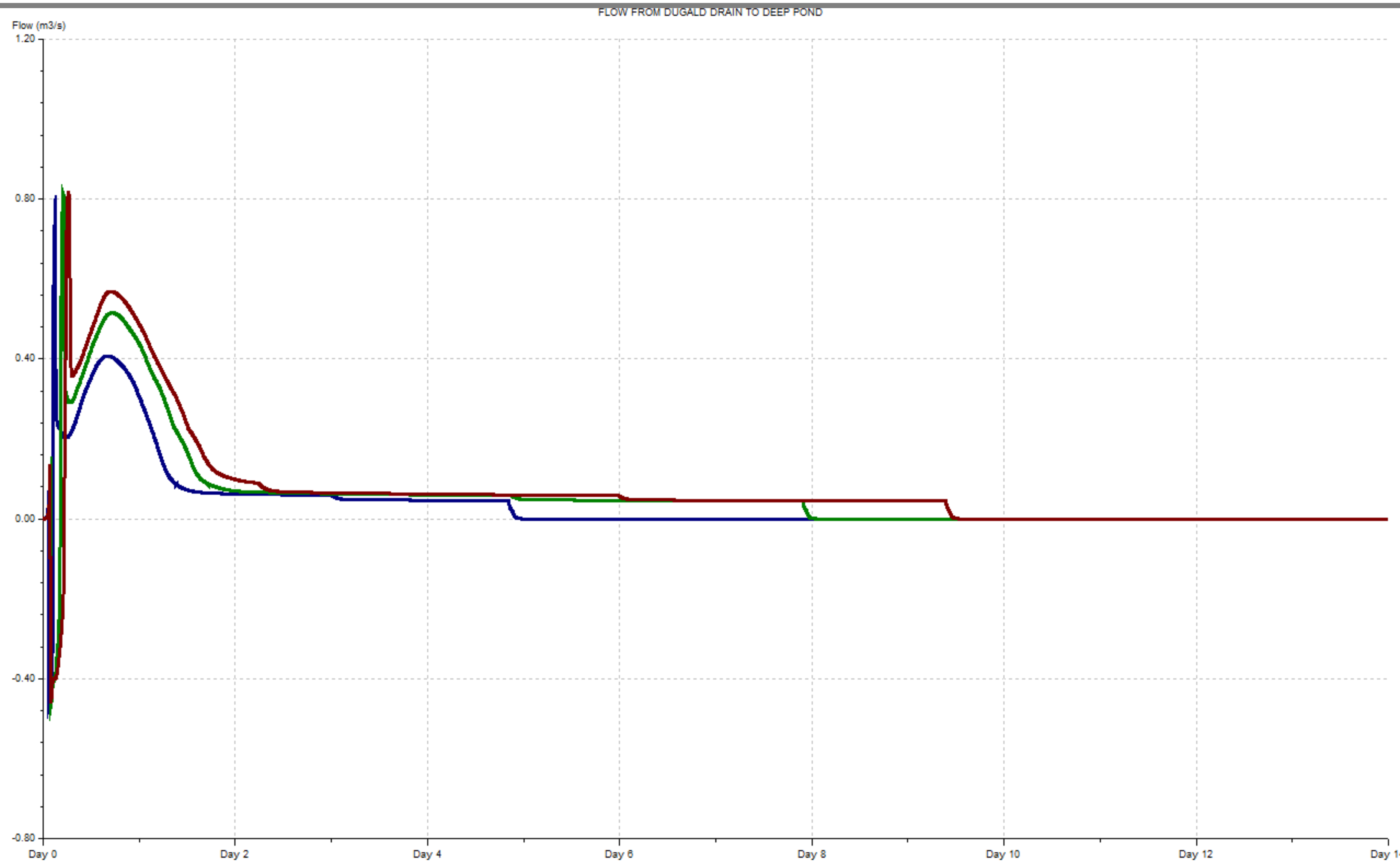
CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DRAIN WITH SIPHON
NO DEEP POND CONNECTION

Figure 23



		Flow (m3/s)		
		Min	Max	Volume (m3)
5 Year Storm		-0.471	0.770	47056.170
25 Year Storm		-0.467	0.817	73449.463
100 Year Storm		-0.464	0.820	87819.936

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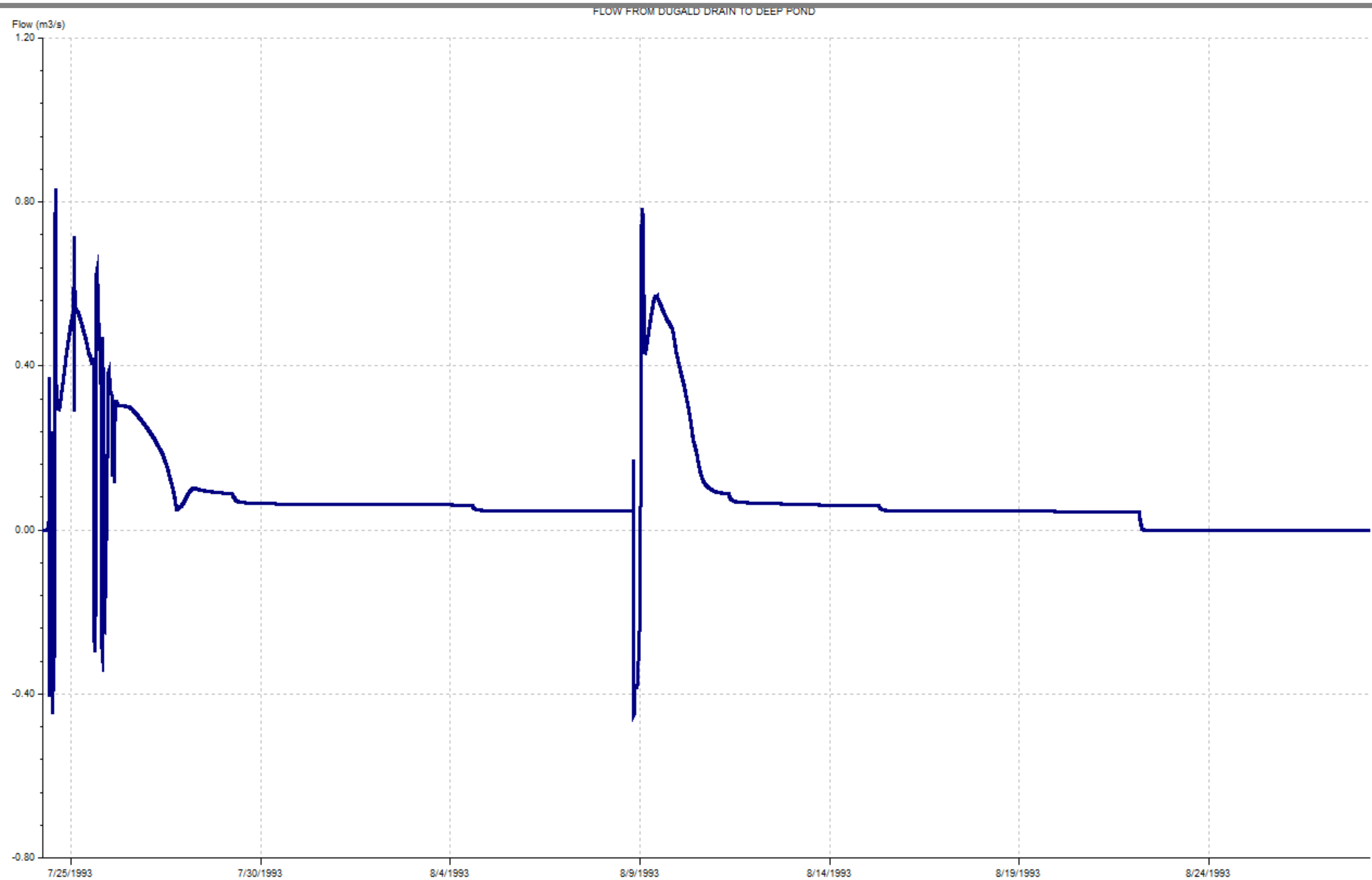
CITY OF WINNIPEG
PLESSIS UNDERPASS

DESIGN STORM FLOWS FROM DUGALD DRAIN TO DEEP POND - DITCH INTERCONNECTION

Figure 24



	Flow (m3/s)		
	Min	Max	Volume (m3)
Summer 1993 Flows	-0.454	0.833	253578.593

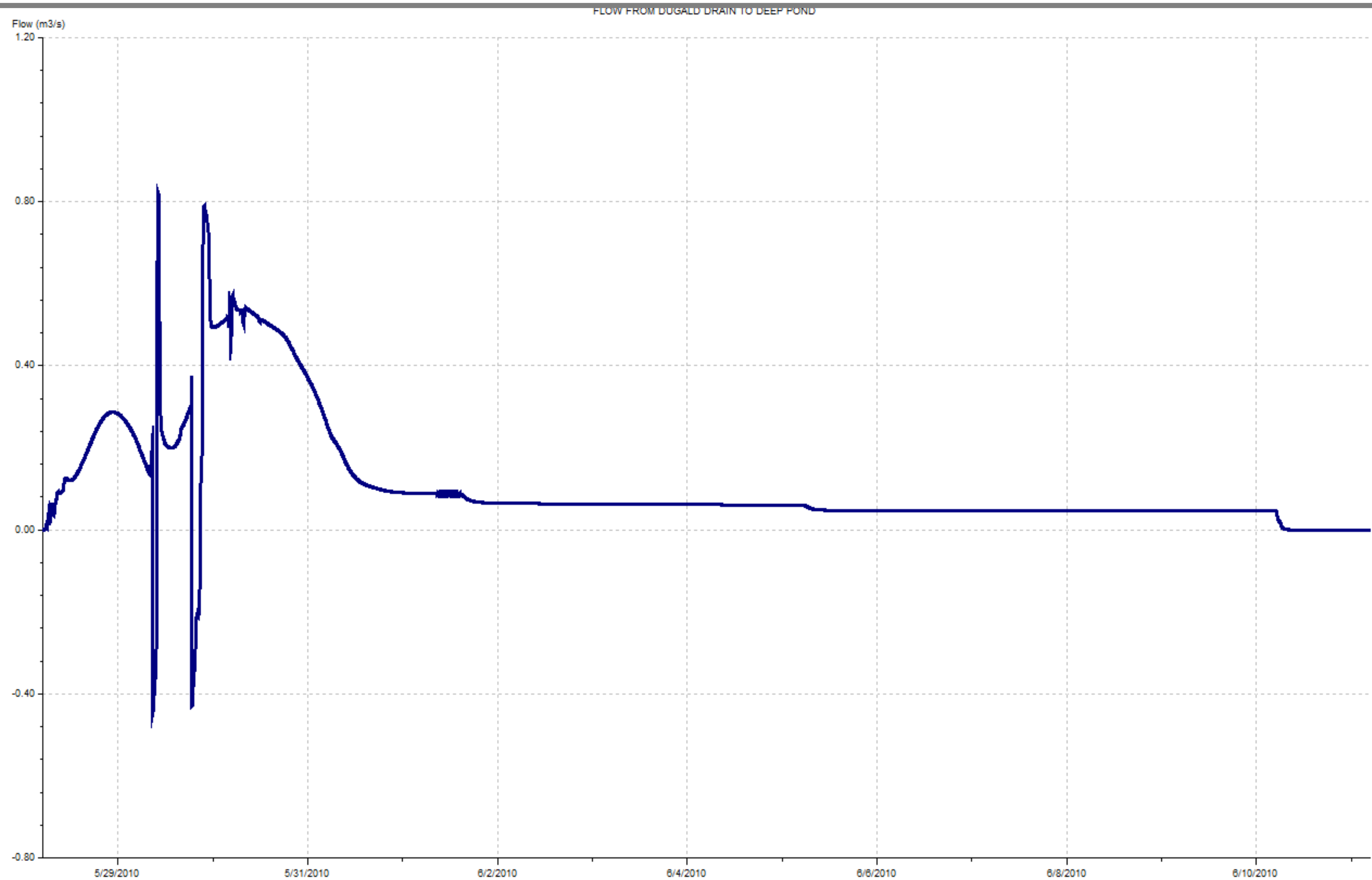
CITY OF WINNIPEG
PLESSIS UNDERPASS

SUMMER 1993 FLOWS FROM DUGALD DRAIN TO DEEP POND - DITCH INTERCONNECTION

DRAFT

AECOM

Figure 25



	Flow (m3/s)		
	Min	Max	Volume (m3)
May 2010 Flows	-0.452	0.825	131515.072

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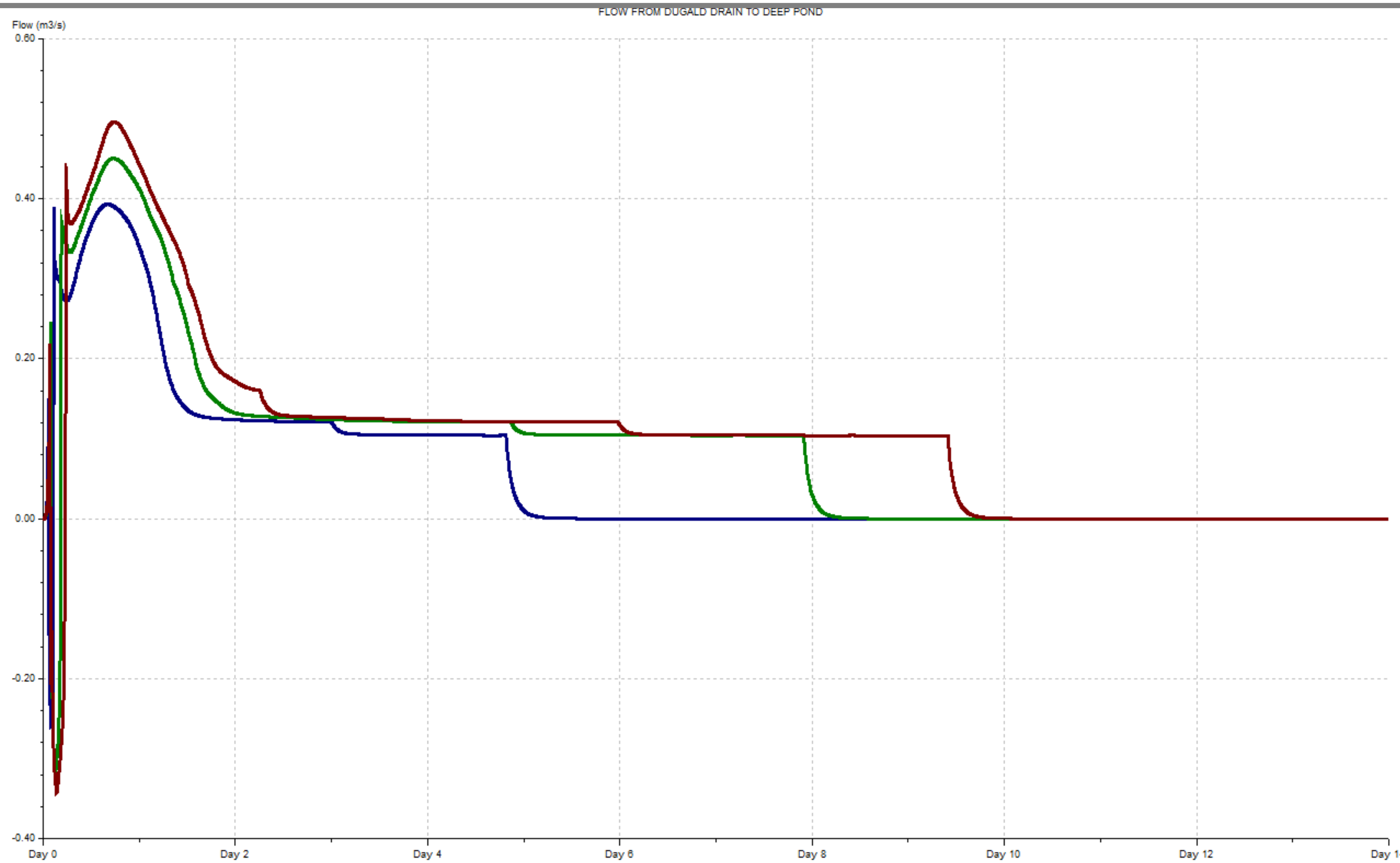
CITY OF WINNIPEG
PLESSIS UNDERPASS

MAY 2010 STORM FLOWS FROM DUGALD DRAIN TO DEEP POND - DITCH INTERCONNECTION

Figure 26



		Flow (m3/s) Min	Max	Volume (m3)
5 Year Storm		-0.259	0.393	68883.178
25 Year Storm		-0.315	0.451	106890.365
100 Year Storm		-0.342	0.496	126880.137

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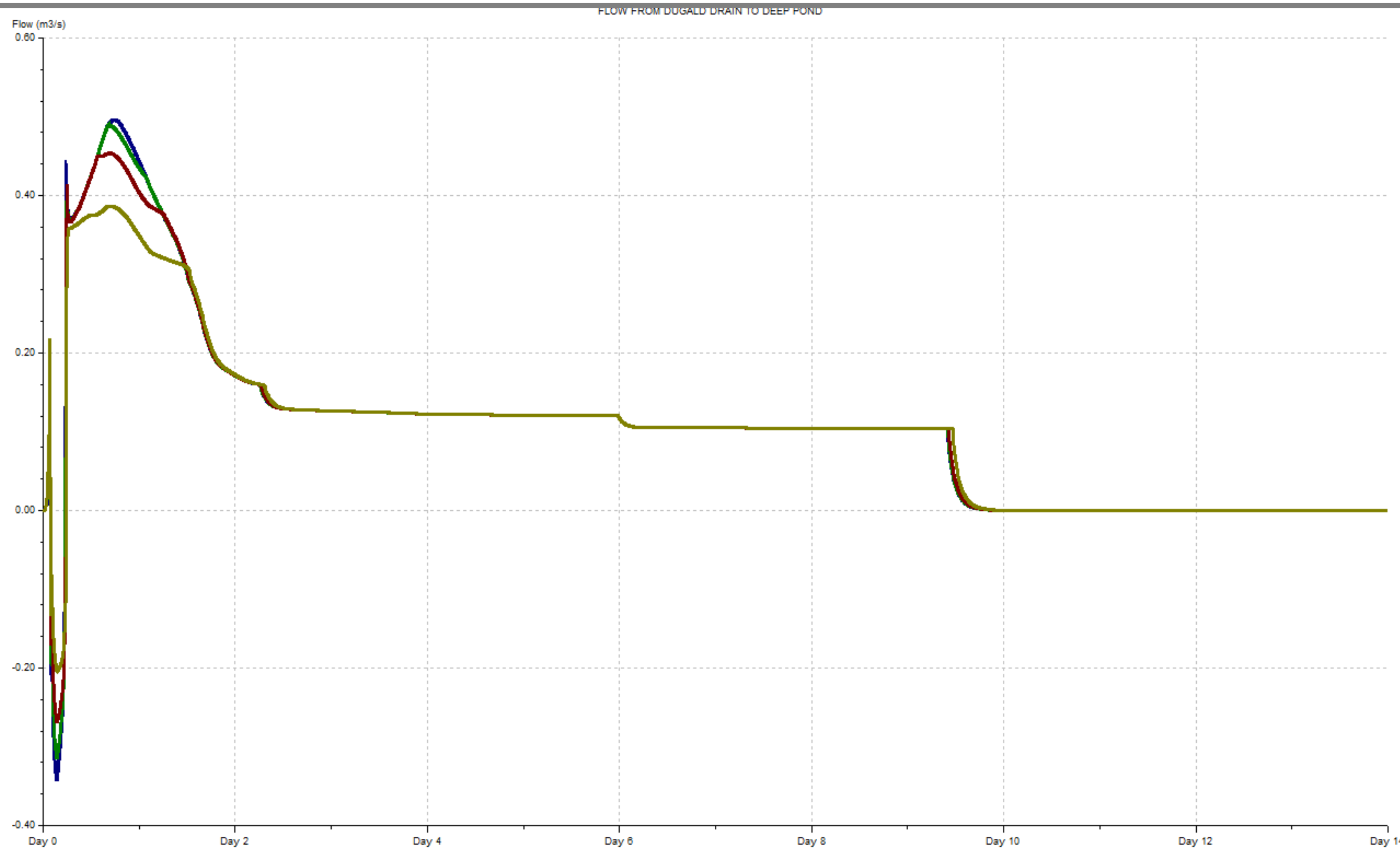
CITY OF WINNIPEG
PLESSIS UNDERPASS

DESIGN STORM FLOWS FROM DUGALD DRAIN TO DEEP POND - LDS INTERCONNECTION

Figure 27



	Flow (m3/s) Min	Max	Volume (m3)
600 Interconnection	-0.342	0.496	126880.137
525 Interconnection	-0.314	0.490	126694.665
450 Interconnection	-0.269	0.454	125634.974
375 Interconnection	-0.204	0.387	121656.311

CITY OF WINNIPEG
PLESSIS UNDERPASS

100 YEAR STORM FLOWS FROM DUGALD DRAIN TO DEEP POND - LDS INTERCONN SENSITIVITY ANALYSIS

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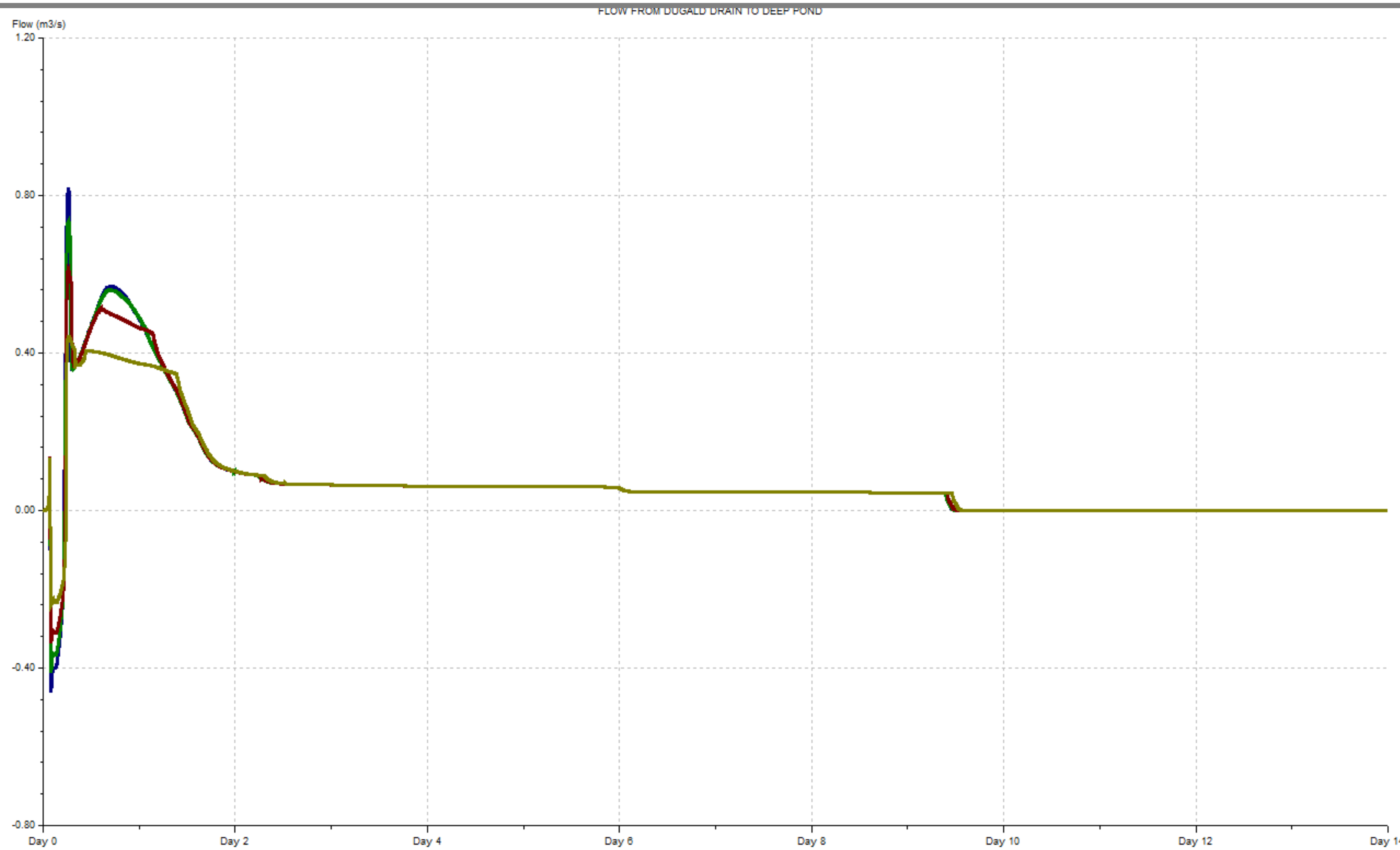
Figure 28



		Flow (m3/s)		
		Min	Max	Volume (m3)
600 Interconnection		-0.464	0.820	87819.936
525 Interconnection		-0.415	0.736	87737.743
450 Interconnection		-0.339	0.611	86308.047
375 Interconnection		-0.243	0.443	81007.468

CITY OF WINNIPEG
PLESSIS UNDERPASS

100 YEAR STORM FLOWS FROM DUGALD DRAIN TO DEEP POND - DITCH INTERCONN SENSITIVITY ANALYSIS

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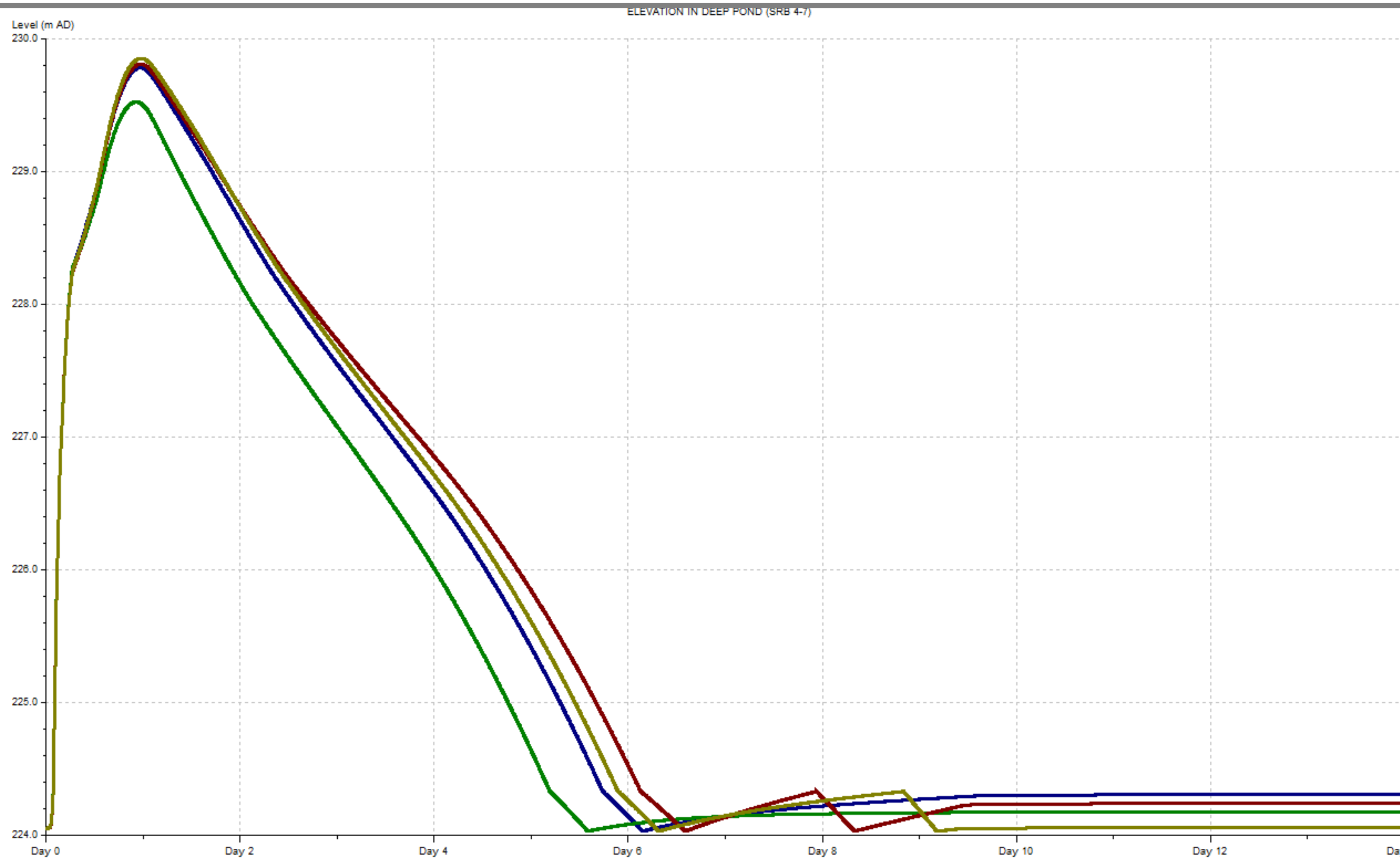
AECOM

Figure 29



		Level (m AD)	
		Min	Max
Existing Conditions		224.031	229.783
Existing Conditions (No Dugald Drain Interconnection)		224.030	229.524
600 LDS Connection to Siphon		224.031	229.810
600 Ditch Connection to Siphon		224.030	229.854

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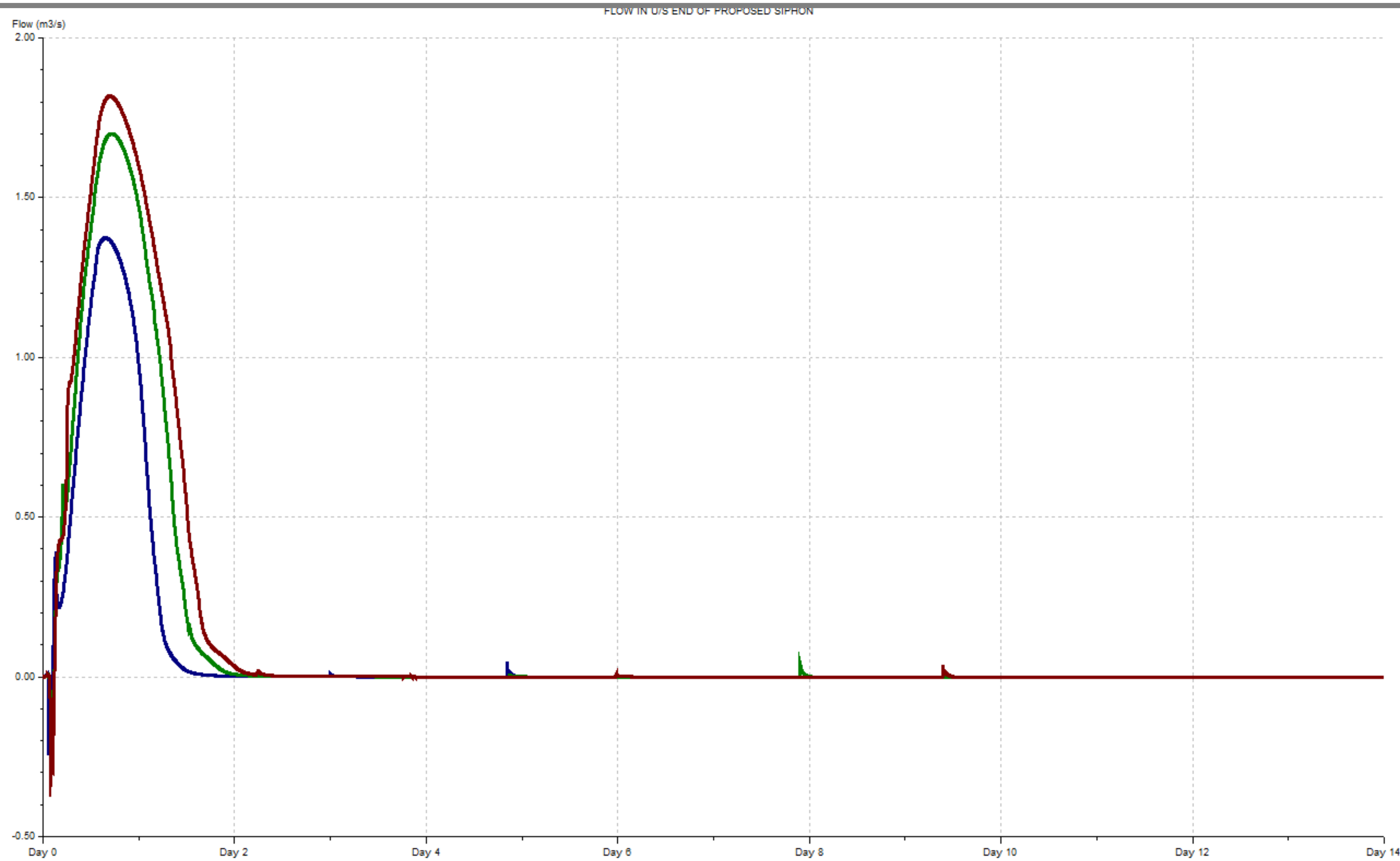
CITY OF WINNIPEG
PLESSIS UNDERPASS

100 YEAR STORM RESPONSE IN DEEP POND (SRB 4-7)

Figure 30



	Flow (m3/s) Min	Max	Volume (m3)
5 Year Summer Storm	-0.196	1.374	87904.331
25 Year Summer Storm	-0.260	1.701	135826.151
100 Year Summer Storm	-0.327	1.818	161125.655

CITY OF WINNIPEG
PLESSIS UNDERPASS

DESIGN STORM FLOWS IN SIPHON

Figure 31

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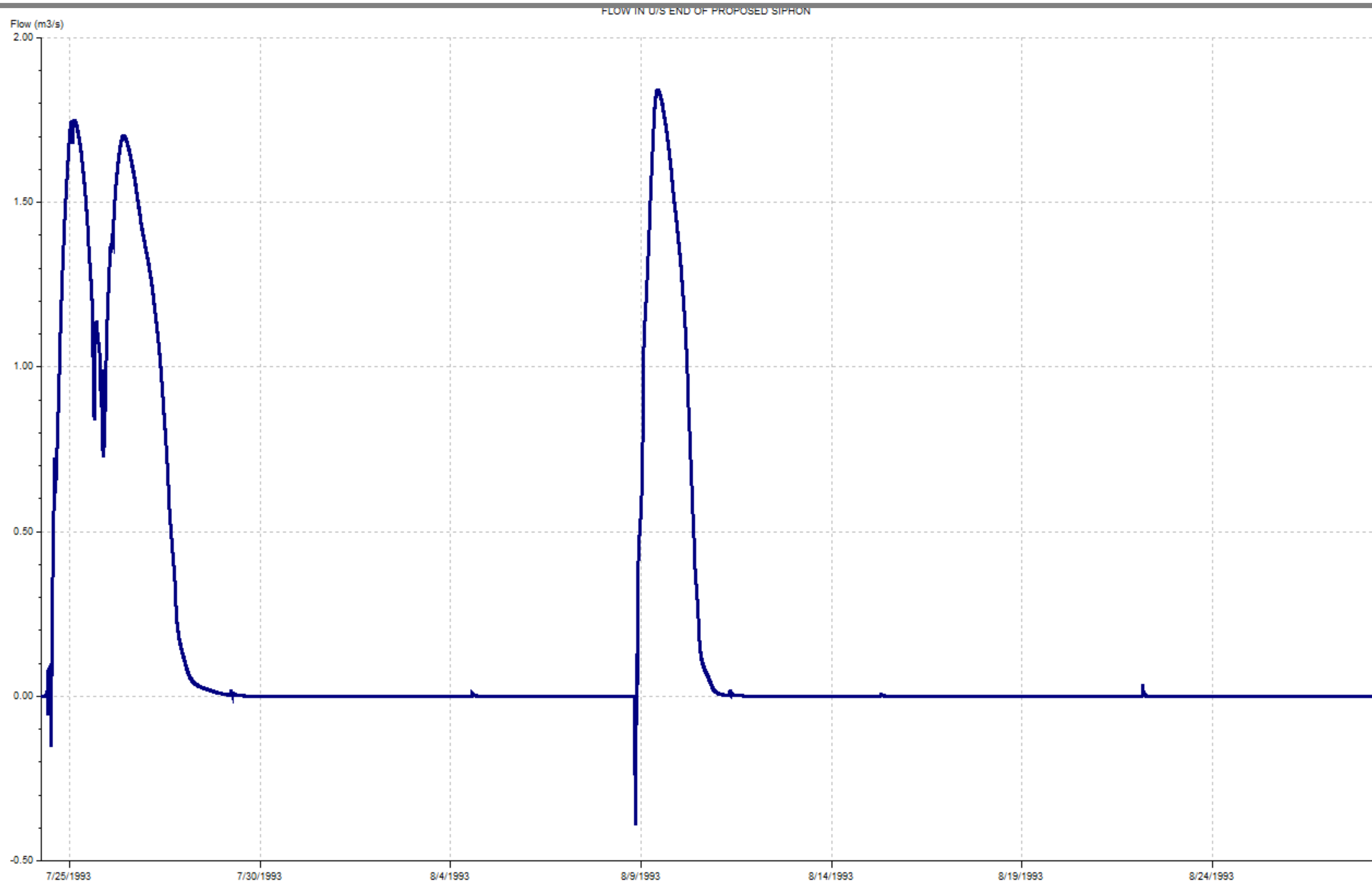
AECOM



	Flow (m3/s) Min	Max	Volume (m3)
Summer 1993 Flows	-0.393	1.841	537202.087

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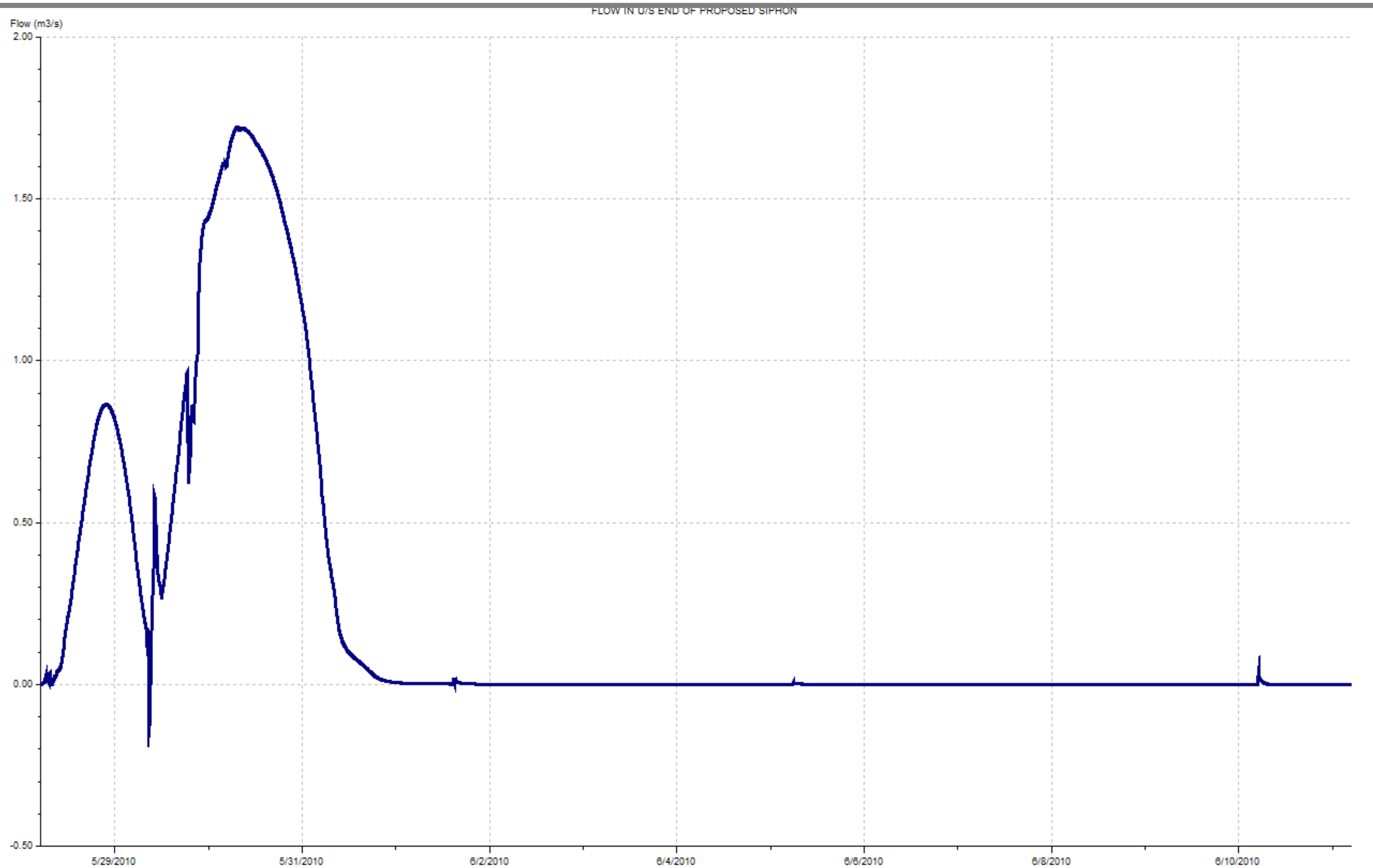
CITY OF WINNIPEG
PLESSIS UNDERPASS

SUMMER 1993 FLOWS IN SIPHON

Figure 32



	Flow (m3/s) Min	Max	Volume (m3)
May 2010 Events	-0.146	1.724	243711.217

CITY OF WINNIPEG
PLESSIS UNDERPASS

MAY 2010
FLOWS IN SIPHON
Figure 33

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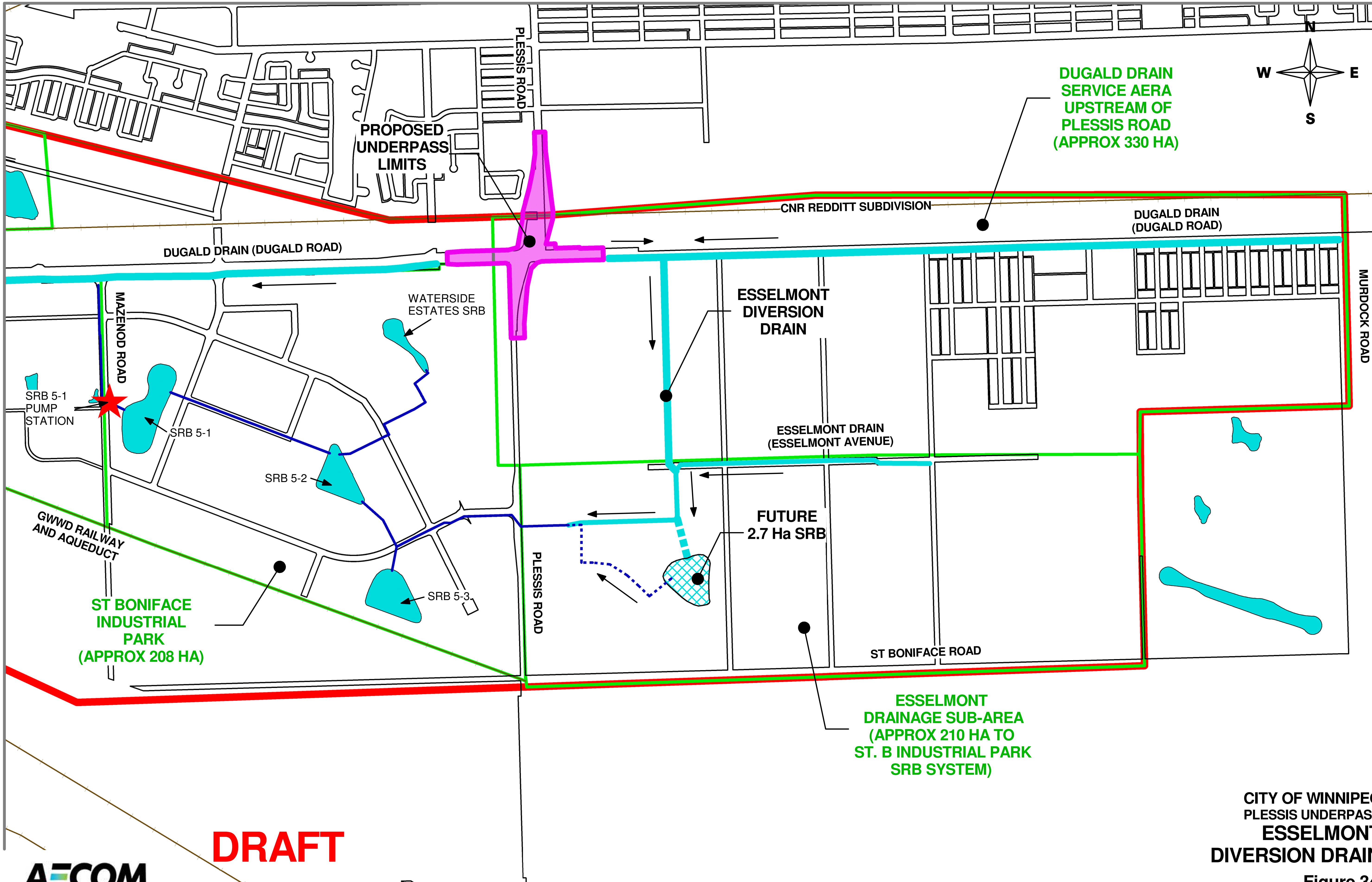
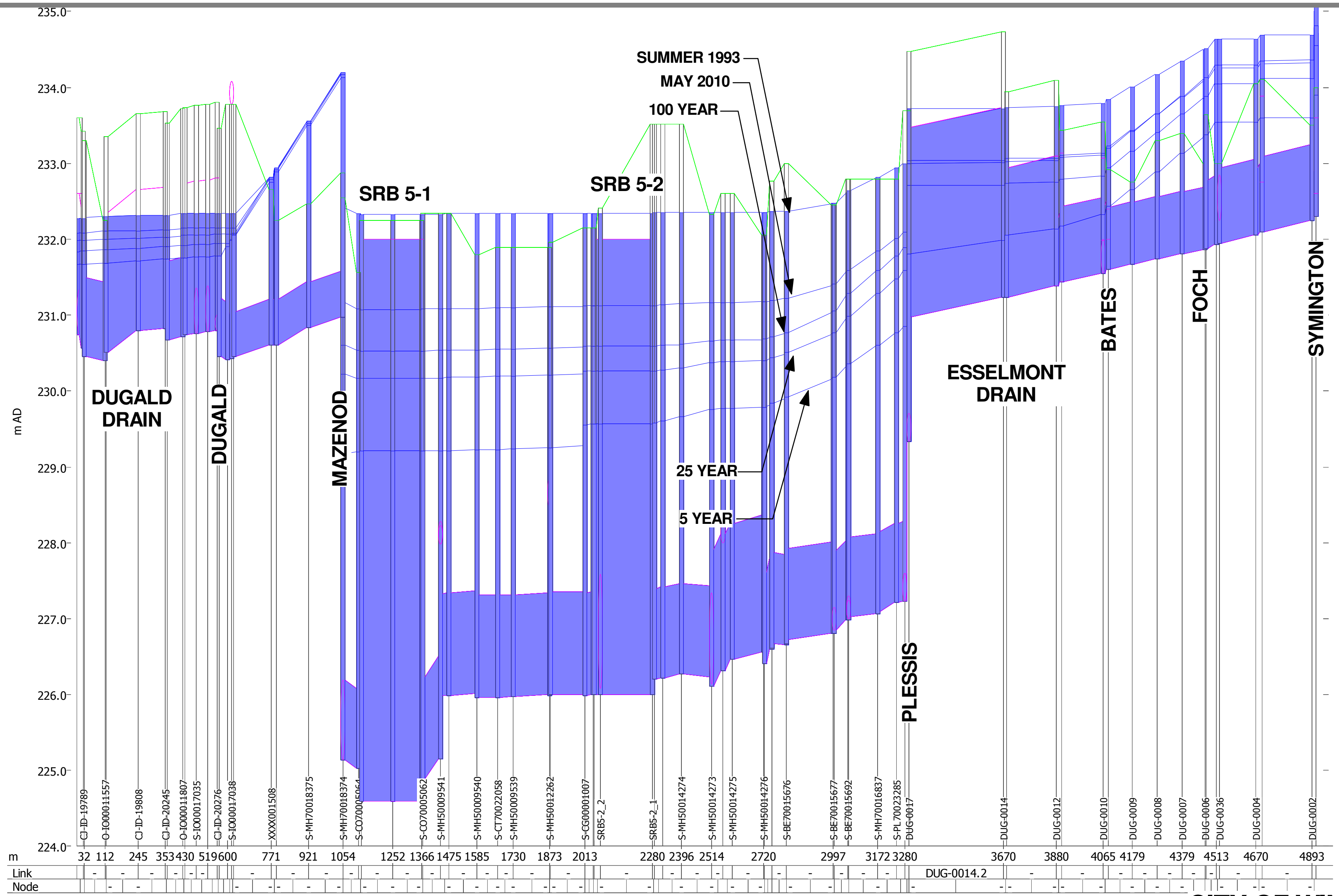
AECOM



CITY OF WINNIPEG
PLESSIS UNDERPASS
ESSELMONT
DIVERSION DRAIN
Figure 34

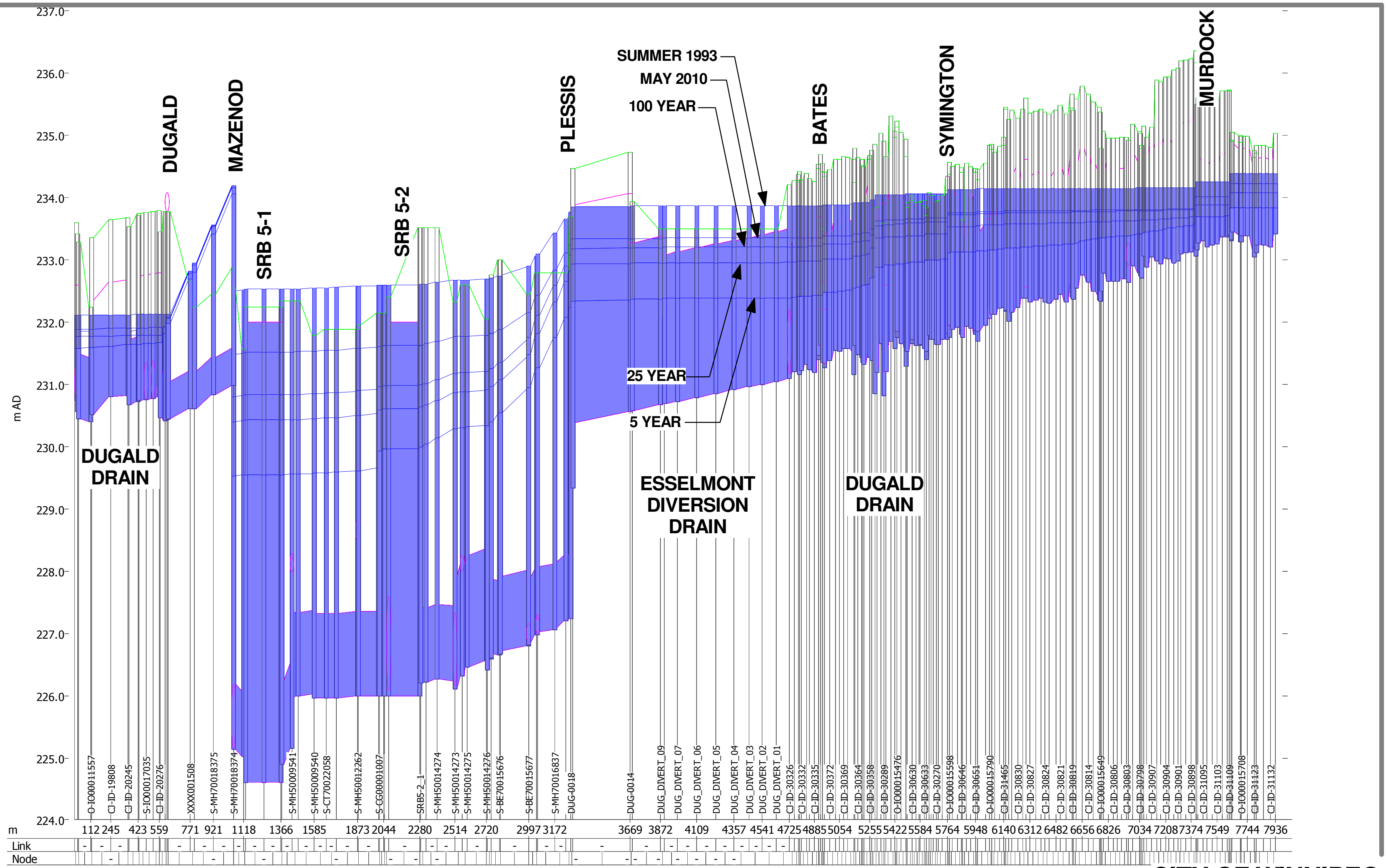


DRAFT

CITY OF WINNIPEG
PLESSIS UNDERPASS

ESSELMONT DRAIN

Figure 35



CITY OF WINNIPEG
PLESSIS UNDERPASS

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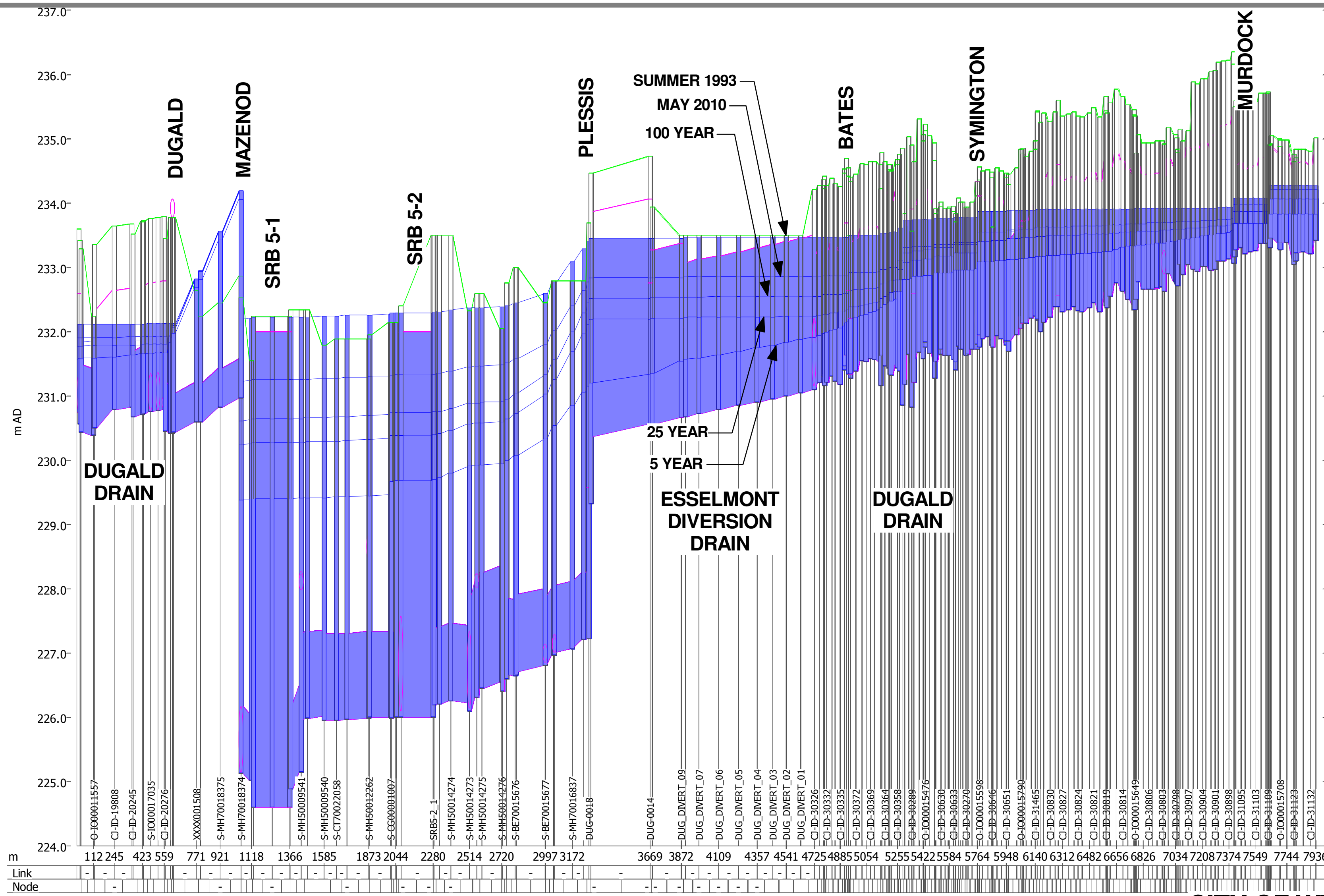
DUGALD DRAIN DIVERSION TO ESSELMONT DRAIN

Figure 36

AECOM



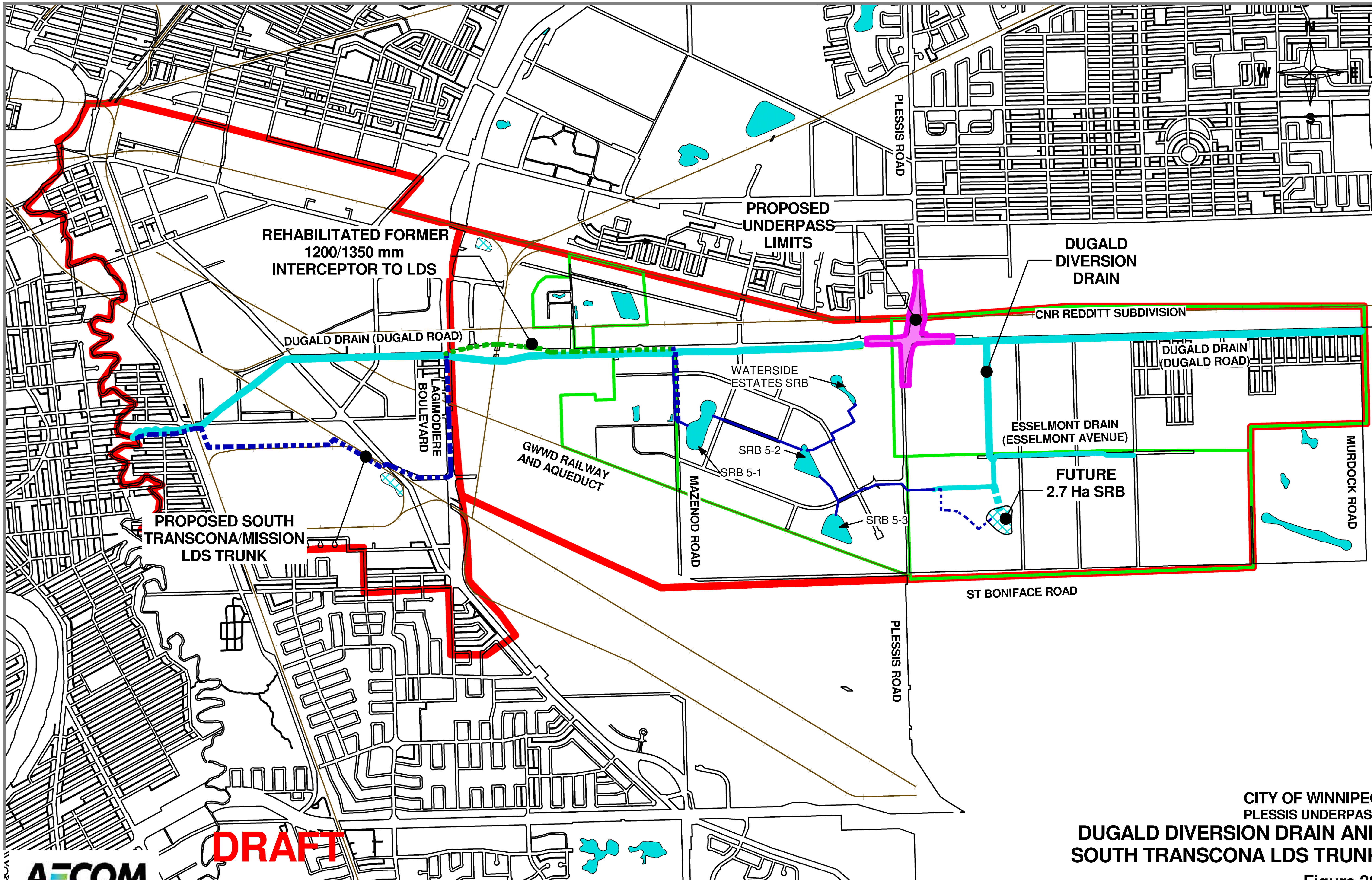
CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DRAIN DIVERSION TO ESSELMONT DRAIN WITH FUTURE SRB

Figure 37

DRAFT

AECOM

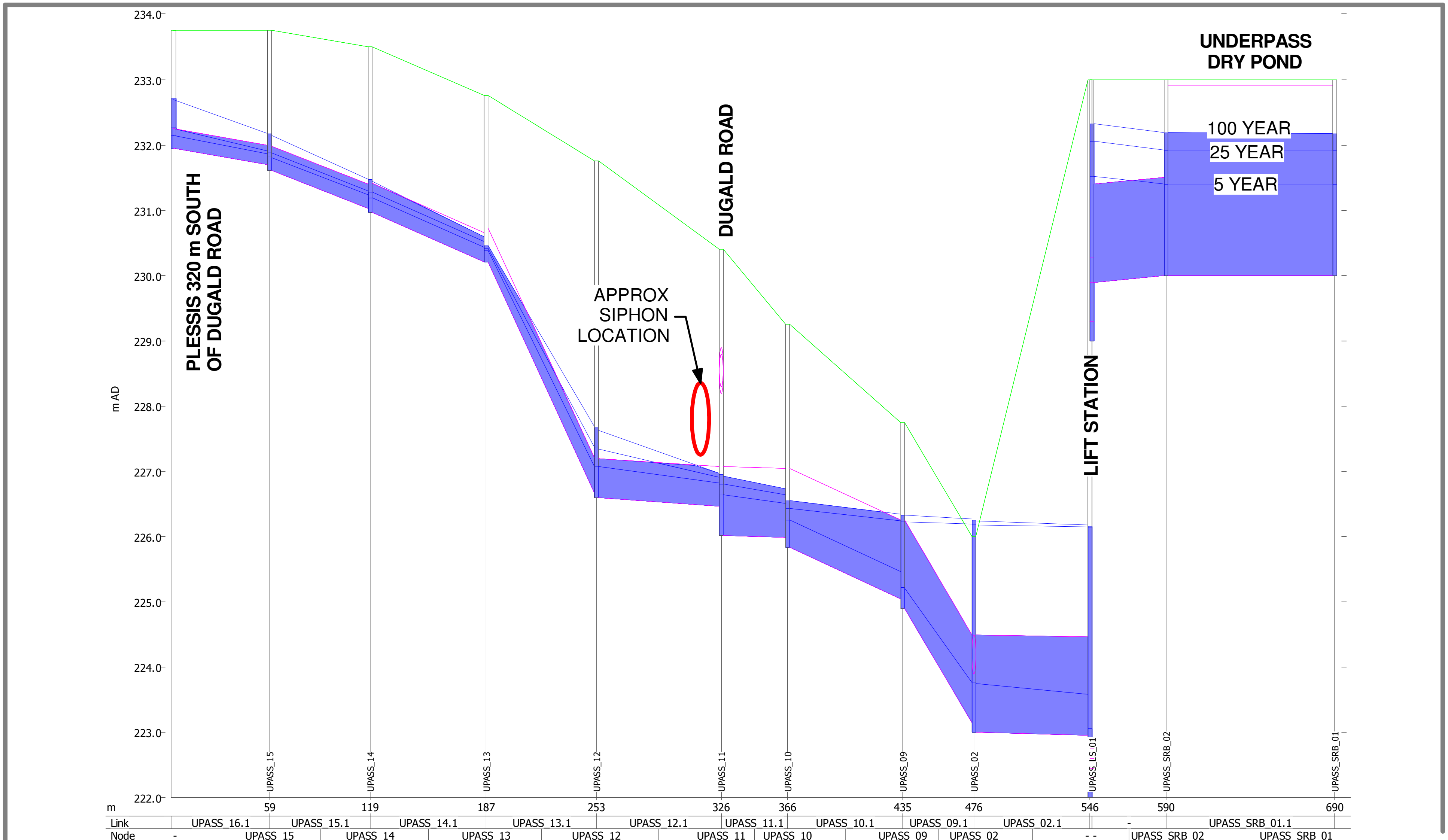


AECOM

CITY OF WINNIPEG
PLESSIS UNDERPASS

DUGALD DIVERSION DRAIN AND SOUTH TRANSCONA LDS TRUNK

Figure 38



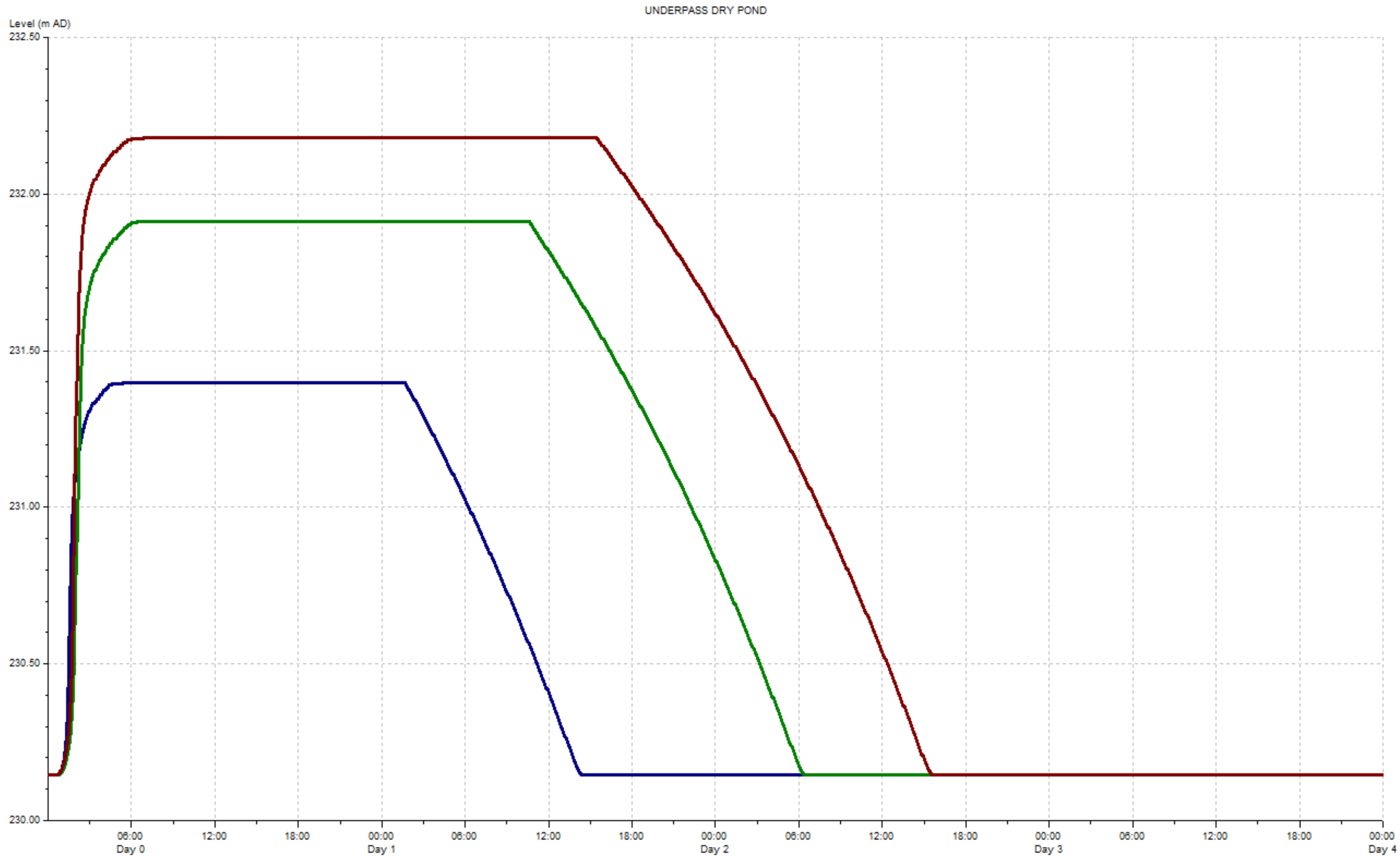
CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS LDS PROFILES - SOUTH OF RAILWAY LINE

Figure 39

DRAFT

AECOM



		Level (m AD)	
		Min	Max
5 Year Storm		230.145	231.399
25 Year Storm		230.145	231.913
100 Year Storm		230.145	232.179

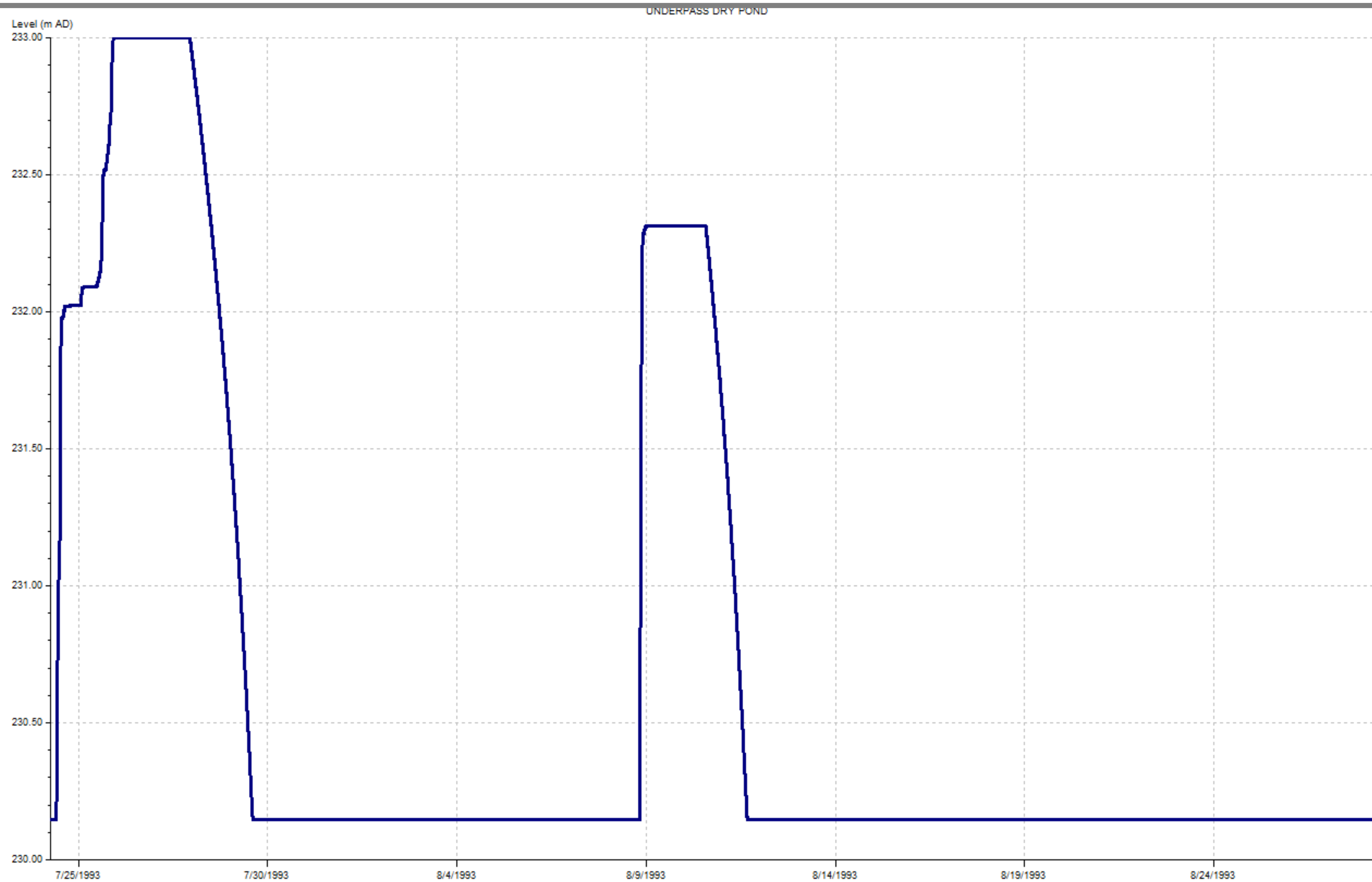
CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS DRY POND DESIGN STORM PERFORMANCE

Figure 40

DRAFT

AECOM



	Level (m AD) Min	Level (m AD) Max
Summer 1993 Storms	230.145	233.000

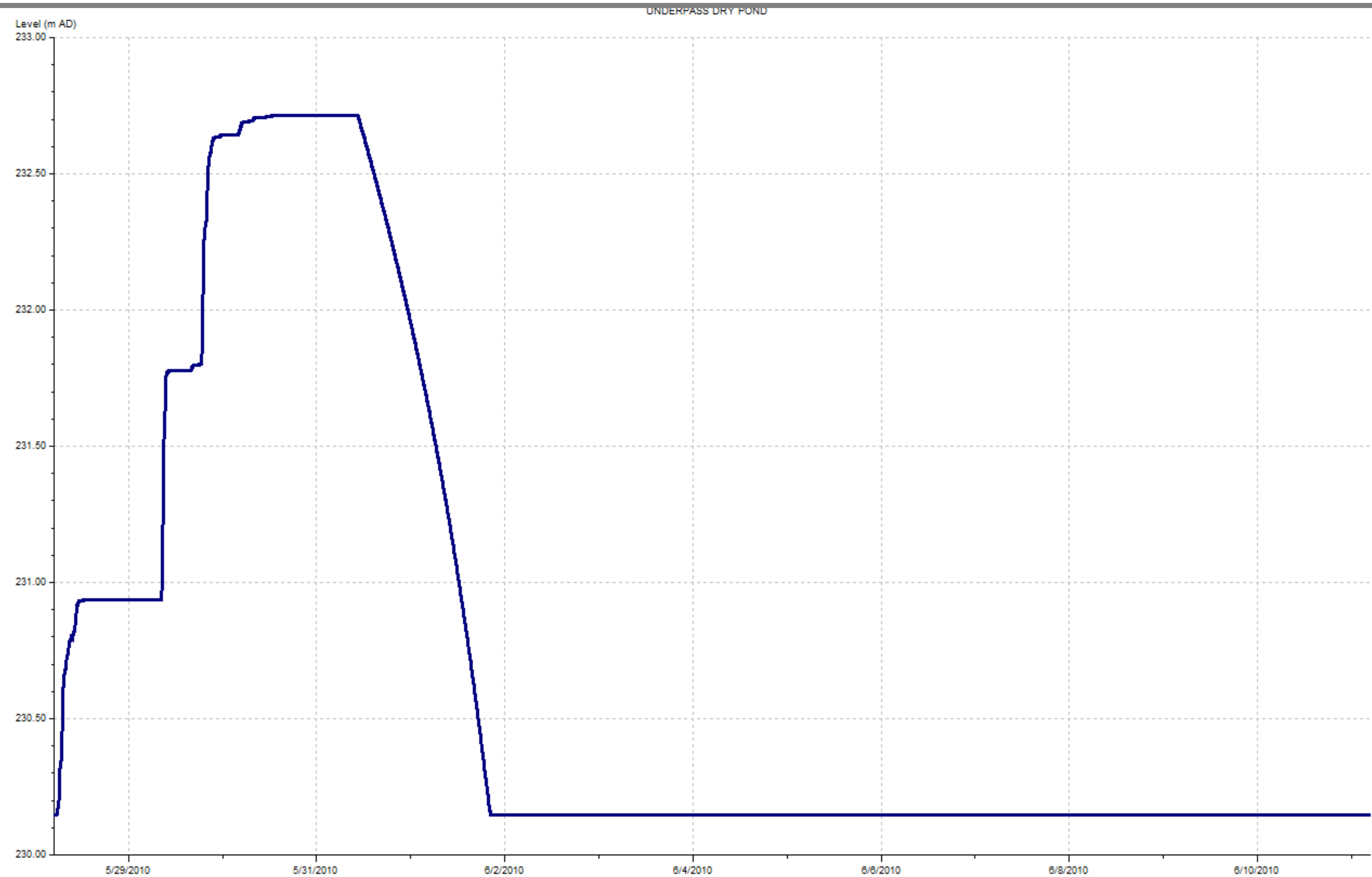
CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS DRY POND SUMMER 1993 PERFORMANCE

Figure 41

DRAFT

AECOM



	Level (m AD) Min	Level (m AD) Max
May 2010 Storms	230.145	232.713

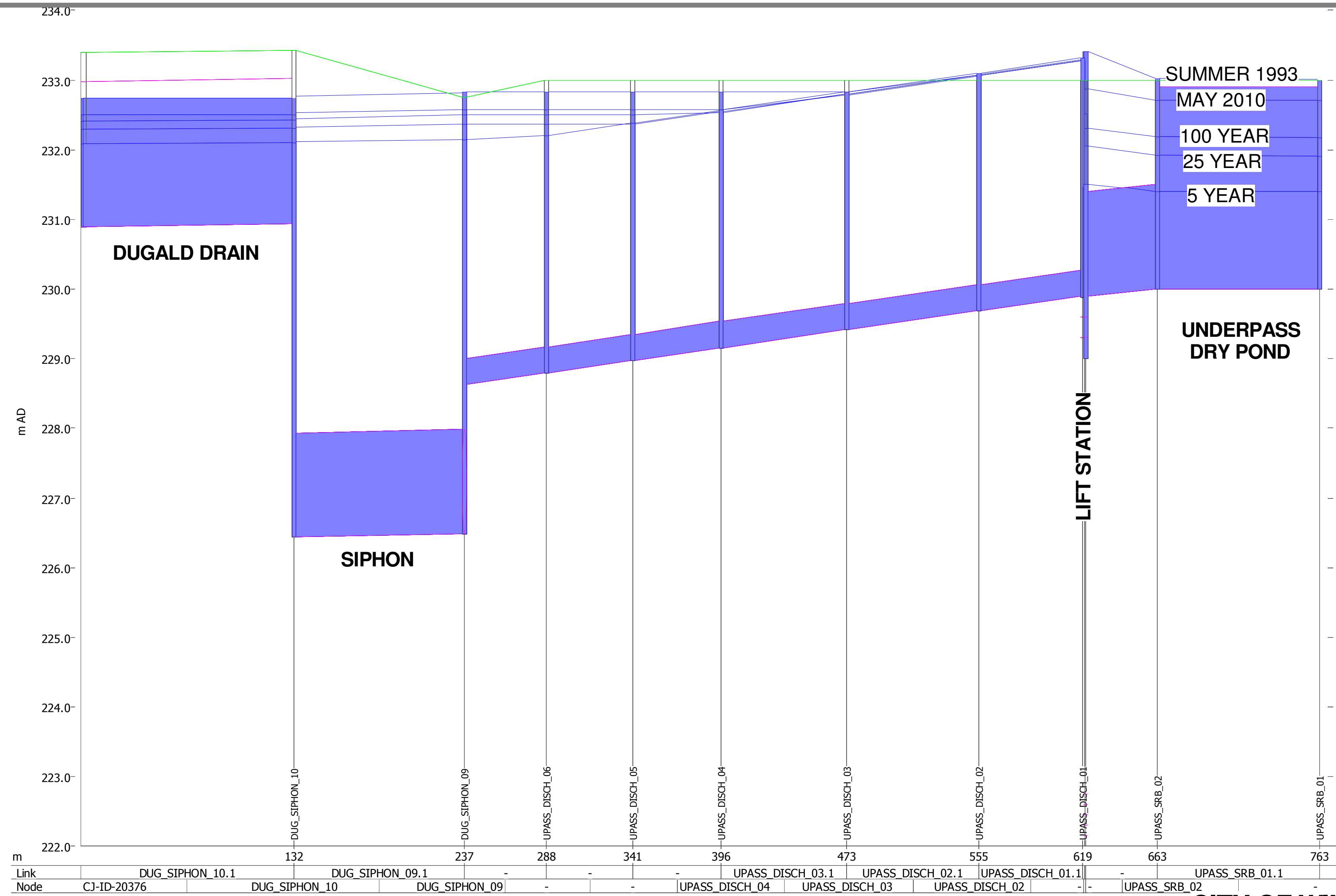
DRAFT

AECOM

CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS DRY POND
MAY 2010 PERFORMANCE

Figure 42



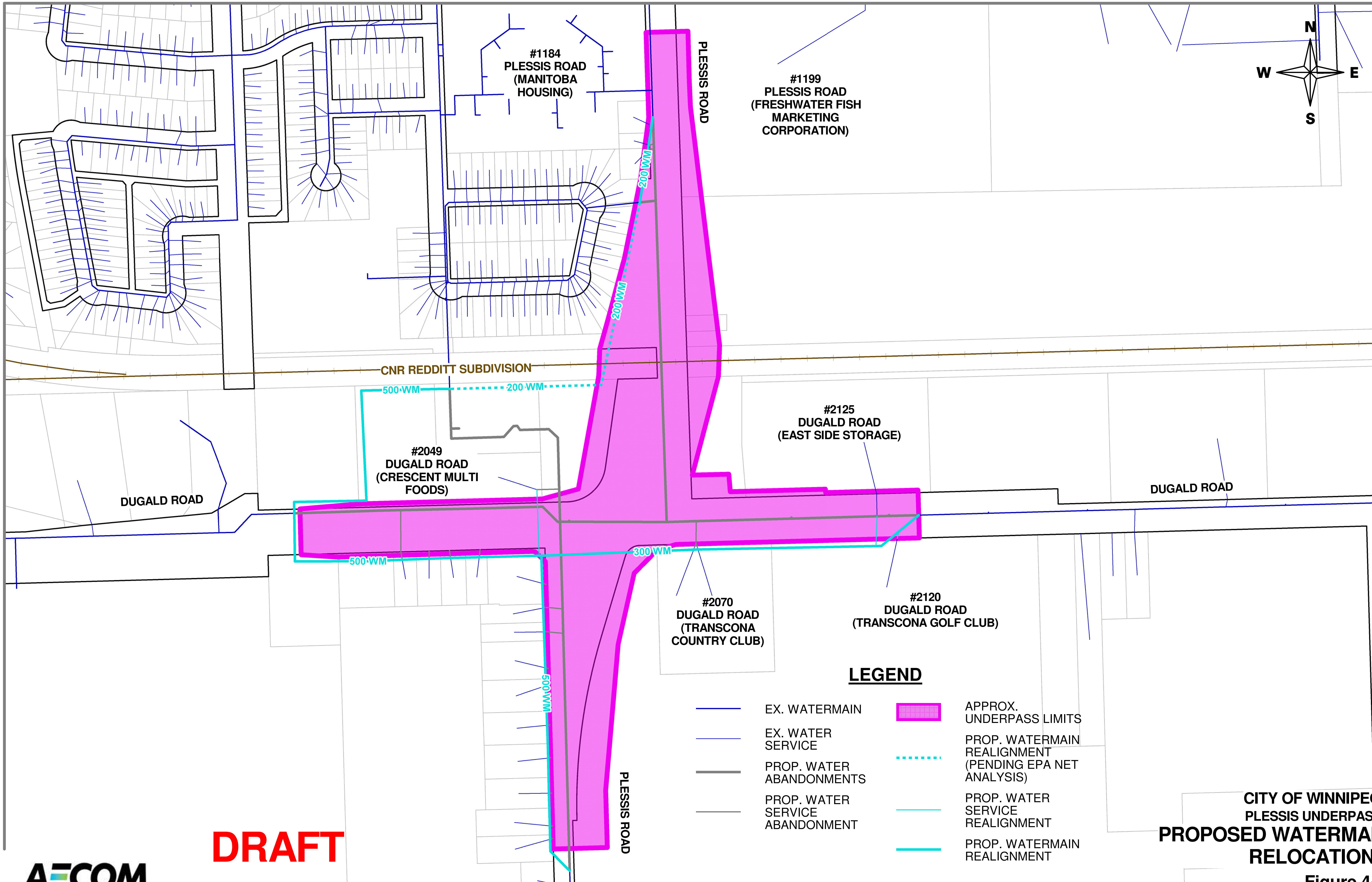
CITY OF WINNIPEG
PLESSIS UNDERPASS

UNDERPASS DRY POND DISCHARGE LINE

Figure 43

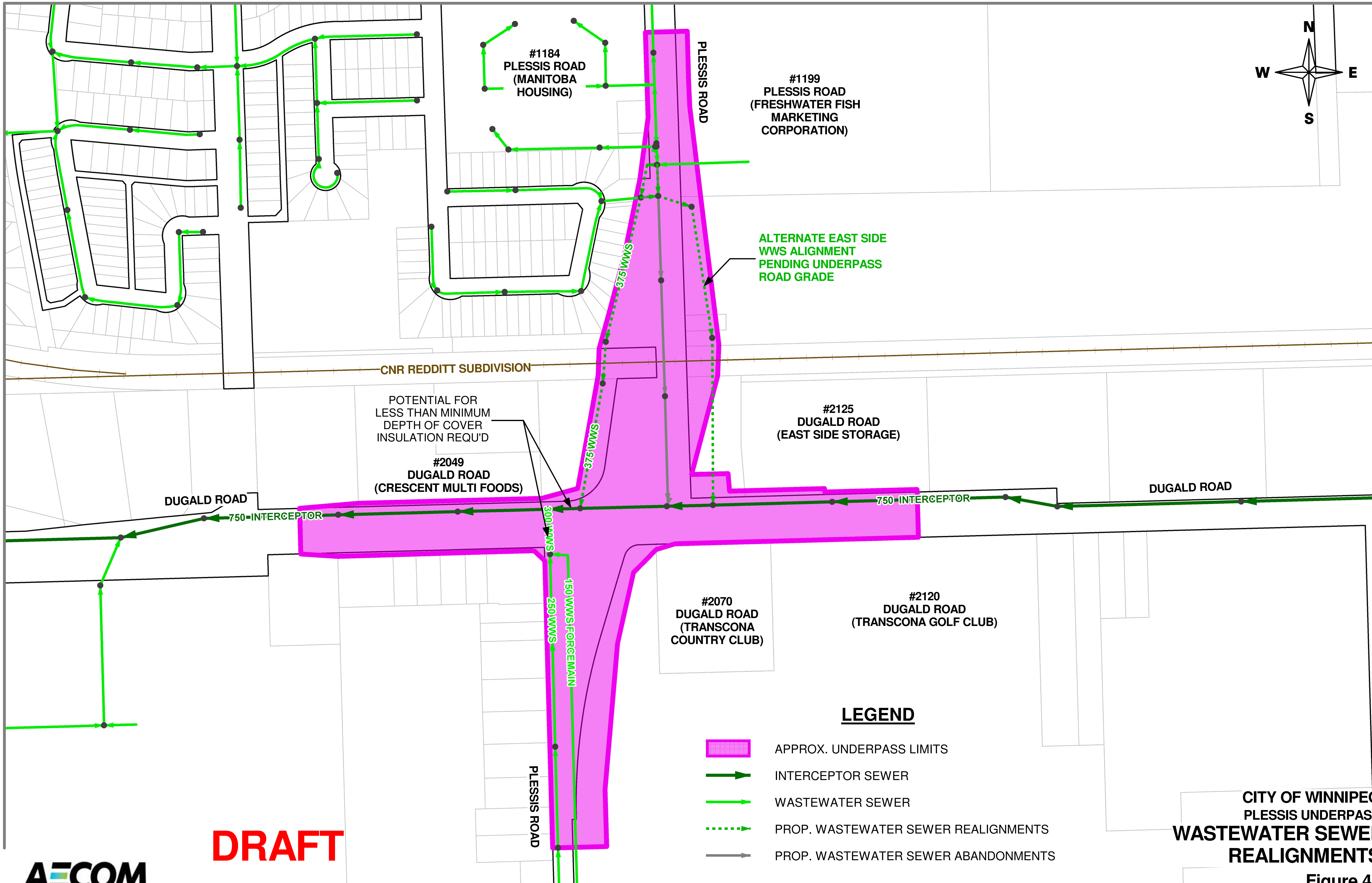
DRAFT

AECOM



AECOM

CITY OF WINNIPEG
PLESSIS UNDERPASS
PROPOSED WATERMAIN
RELOCATIONS
Figure 44



N
W
E
S
#1184
PLESSIS ROAD
(MANITOBA
HOUSING)
PLESSIS ROAD
#1199
PLESSIS ROAD
(FRESHWATER FISH
MARKETING
CORPORATION)
ALTERNATE EAST SIDE
WWS ALIGNMENT
PENDING UNDERPASS
ROAD GRADE
375 WWS
CNR REDDITT SUBDIVISION
POTENTIAL FOR
LESS THAN MINIMUM
DEPTH OF COVER
INSULATION REQU'D
375 WWS
#2125
DUGALD ROAD
(EAST SIDE STORAGE)
#2049
DUGALD ROAD
(CRESCENT MULTI FOODS)
DUGALD ROAD
750 INTERCEPTOR
300 WWS
750 INTERCEPTOR
DUGALD ROAD
150 WWS FORCEMAIN
250 WWS
#2070
DUGALD ROAD
(TRANSCONA
COUNTRY CLUB)
#2120
DUGALD ROAD
(TRANSCONA GOLF CLUB)
LEGEND
APPROX. UNDERPASS LIMITS
INTERCEPTOR SEWER
WASTEWATER SEWER
PROP. WASTEWATER SEWER REALIGNMENTS
PROP. WASTEWATER SEWER ABANDONMENTS
PLESSIS ROAD
DRAFT
AECOM
CITY OF WINNIPEG
PLESSIS UNDERPASS
WASTEWATER SEWER
REALIGNMENTS
Figure 45