E1 INTRODUCTION

Storm drainage system design includes the design of a minor system (storm sewers) and a major system (overland flow routes, stormwater management ponds, etc.).

The design of the storm drainage system shall be based on an accepted Stormwater Management Report, in accordance with the City of Markham's "Stormwater Management Guidelines" and the "Stormwater Management Pond Safety and Maintenance Criteria".

Site plan (Industrial, Commercial, Condo, etc.) developments shall be designed in accordance with the on-site detention (OSD) requirements of the City of Markham "Design of On-Site Detention (OSD)' manual.

Quantity control criteria for river flood protection are according to TRCA requirements (TRCA Criteria as of 2009 are summarised in Section E10), while criteria to protect downstream drainage systems will be determined on a site specific basis and may require over control to prevent impacts.

Water balance criteria shall be in accordance with TRCA and MNR requirements. The proposed water balance measures shall be discussed with the City staff and must be to the satisfaction of the Director of Engineering.

Design of the minor system shall be in accordance with the criteria in the following sections.

E2 STORM SEWER DESIGN

E2.1 Storm Sewer Flows

Storm sewers (minor system) shall be designed to accommodate a 5-Year design flow and shall operate without surcharge. Minor and major systems drainage analyses shall be provided in a report and this shall preferably be carried out using established computer models (e.g. PCSWM, OTTSWMM, etc.) accepted by the Director of Engineering.

A 100-Year Hydraulic Grade Line (HGL) analysis shall be performed and provided in a tabular format.

For Greenfield developments, the basement slab elevations shall be set minimum 0.5 m above the 100-Year (HGL) and shall be indicated on the Plan and Profile drawings.

For Infill developments, where HGL information is not readily available or determined, then the HGL shall be estimated to be minimum 1.8 m below the road centreline elevation, provided the municipal sewer is located at the standard 2.5 m depth. Therefore, the minimum basement slab elevation shall be set at maximum 1.3 m depth from the road centreline elevation. Sump pump shall be installed if the basement elevation is lower than 1.3 m from the centreline elevation of the road.

The minimum basement slab elevations shall be shown on all lots where HGL is above obvert of the pipe.

Inlet control devices (ICDs) shall only be used to control flow into the sewer to reduce 100-Year HGL.

Storm sewer calculations shall be completed on the design sheets as per the City's Standard Format (attached) and the final design sheets shall be included in the Engineering Drawings.

E2.2 Runoff Calculations

Storm sewers shall be designed based on the Rational Method. The Stormwater Management Guidelines shall be referred to for further details and principles.

Rational Method

$$Q = KRCIA$$

Where:

Q = Design flow (m^3 / sec)
K = Conversion factor (0.00278)
R = Return period factor
C = Runoff coefficient
I = Rainfall intensity (mm / hour)
A = Contributing drainage area (ha)

Runoff Coefficient (C)

The following runoff coefficient shall be used for the design purposes:

Area Types	Run-of-Coefficient (C)
Asphalt, Concrete, Roof Areas, Gravel Areas and Parking Lots	0.90
Grassed Areas, Parkland	0.25
Commercial	0.90
Industrial	0.90
Institutional (Schools and Churches)	0.75
Residential	
Single Family	0.65
Semi-detached, Duplex	0.70
Row Housing, Townhouse	0.75
Apartments / Mix Used	0.85

To calculate the corresponding Runoff Coefficient for existing development or where coefficients may be lower than standard values, the following formula may be used:

$C = 0.25 (1 - i) + 0.9 i$

Where,

C = Runoff Coefficient

i = Imperviousness Ratio

Supporting calculations demonstrating the calculated Imperviousness Ratio (i) must be provided. Lower Runoff Coefficients (C) values may be considered where lot-level best management practices detain 50% or more of the runoff from the City's 5-Year design event. Values must be accepted by the Director of Engineering.

Return Period Factor (R)

The following return period factor shall be used for design purposes:

Return Period	Return Period Factor (R)
Up to 10-Year	1.00
25-Year	1.10

50-Year	1.20
100-Year	1.25

When applying the above factors, the inlet capture and by-pass rates may be considered in determining the effective peak flow rate in the minor system.

Rainfall Intensity (I)

The intensity of rainfall shall be determined using the following equation:

I (mm / hr) = A / (T+B)^{C} , where T is Time of Concentration in minutes.

The values of A, B and C for the various storms are as follows:

Return Period	A	B	C
2-Year Storm	651.63	3.75	0.80
5-Year Storm	1045.41	4.90	0.83
10-Year Storm	1331.42	5.26	0.84
25-Year Storm	1817.88	6.22	0.87
50-Year Storm	1918.97	6.00	0.86
100-Year Storm	2167.43	6.03	0.86

The minimum initial time of concentration shall be 10 minutes.

Contributing Drainage Area: Drainage systems shall be designed to accommodate all upstream drainage areas for interim and ultimate conditions, as determined by contour mapping and drainage plans.

Pre-Development: To calculate the initial time of concentration (T) for upstream, undeveloped lands, the following formulae may be used: Bransby Williams, HYMO / OTTHYMO, SCS Upland Method, Airport Formula, etc. The most appropriate method shall be determined at the discretion of the Director of Engineering.

Post-Development: To calculate the initial external time of concentration (T) for external lands that are scheduled for future development, a straight line shall be drawn from the furthest point within the watershed to the proposed inlet. The top 50.0 m shall have an initial T of 10 minutes and the remainder shall have a T as if the velocity in the sewer is 2.0 m / s. The summation of the two T's will give the future external time of concentration.

E2.3 Storm Sewer Requirements

Minimum Size

The minimum size for a storm sewer, excluding FDC sewer, shall be 300 mm.

Sewer Capacity

Manning's formula (see Section D) shall be used in determining the capacity of all storm sewers. The capacity of the sewer shall be determined on the basis of the pipe flowing full. Design flow calculations shall be completed on the City's Standard Format for Storm Sewer Design Sheets.

Flow Velocities

Flow velocities shall be determined using Manning's Equation.

For circular pipes, the minimum and maximum flowing full velocities shall be 0.60 m/s and 3.70 m/s respectively.

The minimum grade of all sewers shall not be less than 0.3%.

Velocity change from one pipe to another in a manhole shall not exceed 0.6 m / s for mainline sewers.

Sewer Slope

The first leg of all sewers shall have a minimum grade of 1.0% and a maximum grade of 3.0%.

Depth of Storm Sewers

Sewers shall be designed with a minimum cover of 2.50 m between the road centerline and the sewer obvert, allowing sufficient depth for foundation drains.

For industrial / commercial Subdivisions, a minimum depth below 2.50 m may be considered provided that all tributary areas can be serviced.

A minimum cover of 1.2 m shall be provided at all times for frost protection.

Location

Storm sewers shall be located as shown on the Standard Drawings. This standard location is generally 1.5 m offset from the centre line of the roadway. If sewers are in a common trench, the minimum horizontal separation between two sewers shall be 1.0 m, as shown in the Standard Drawings.

Sewer Alignment

Storm sewers shall be laid in a straight line between manholes unless a radius pipe (675 mm and above) has been designed. Joint burial (common trenching) with sanitary sewers will be considered when supported by the recommendations of a Soil Report prepared by a Geotechnical Engineer.

Clearances

A minimum barrel to barrel clearance of 0.5 m for a sanitary sewer and a storm sewer shall be provided.

See Section G - Composite Utility Plans for the minimum clear separation between storm sewers and other utilities / sewers.

Radius Pipes

Radius pipe shall be allowed for storm sewers 675 mm diameter and larger. The minimum centre line radius allowable shall be in accordance with the minimum radii table as provided by the manufacturers.

Limits of Construction

Sewers shall be terminated with a manhole at the Subdivision limits when external drainage areas are considered in the design. The design of the terminal manholes must allow for possible future extension of the sewer.

Temporary sewer stubs (maximum length of one full pipe) may be permitted between the phases of a development at the discretion of the Director of Engineering.

Changes in Pipe Size

No decrease of pipe size from a larger size upstream to a smaller size downstream shall be allowed regardless of the increase in grades. Exceptions may be made for stormwater management controls as accepted by the Director of Engineering.

E3 MANHOLE REQUIREMENTS

Manholes may be either precast or poured / cast-in-place and shall be designed and constructed in accordance with the Standard Drawings and Ontario Provincial Standard Drawings and Specifications. Precast manholes shall conform to CSA A257.4.

E3.1 Location and Spacing

Manholes shall be located at each change in alignment, grade or pipe material, at all pipe junctions, at the beginning or end of radius pipe sections and at intervals along the pipe to permit entry for maintenance to the sewer.

Maximum spacing of manholes shall be:

- Sewers 600 mm or less in diameter 120 m
- Sewers 675 mm or greater in diameter 170 m

Where a non-standard manhole configuration is required, it shall be designed with reinforced concrete. Such designs shall be detailed on the Engineering Drawings.

E3.2 Manhole Details

- Manhole chamber openings shall be located on the side of the manhole parallel to the flow for straight run manholes, or on the upstream side of the manhole at all junctions
- The change in the direction of flow in any manhole shall not exceed 90^0
- Where manhole depths exceed 5.0 m, safety grating as per the OPSD shall be incorporated into the manhole. Safety grating shall not be more than 5.0 m apart. Whenever practical, a safety grating shall be located 0.5 m above the drop structure inlet pipe
- The obverts on the upstream side of manholes shall not be lower than the obvert of the outlet pipe
- Where the difference in elevation between the obvert of the inlet and outlet pipes exceed 0.6 m, a drop pipe shall be provided in accordance with the Standard Drawings
- Manholes shall be benched to the obvert of the outlet pipe on a vertical projection from the spring line of the sewer
- Benching between the channel edge and the inside wall of the manhole shall be a minimum of 250 mm in width
- Manholes shall be located with a minimum of 1.5 m clearance away from the face of curb and / or any other service

E3.3 Head Losses and Drops

Suitable drops shall be provided across manholes to compensate for the loss in energy due to the change in flow velocity and for the difference in the depth of flow in the sewers. The change in velocity between the inlet and outlet pipes along mainline sewers shall not exceed 0.6 m / s.

Hydraulic calculations may be required for manholes where, in the opinion of the Director of Engineering, there may be insufficient invert drop provided across any manhole.

Regardless of the invert drop across a manhole as required by calculations, obvert of the outlet pipe shall not be higher than obvert of the inlet pipe at any manhole location.

The minimum drops across manholes shall be as follows:

Change of Direction	Minimum Drop (mm)
0°	20
1° to 45°	50
46° to 90°	80

The Consulting Engineer shall ensure that drops through manholes are sufficient to accommodate hydraulic losses.

Where pipe sizes change at manholes, the downstream sewer obverts should match the upstream obvert or be lower.

Drop structures shall be avoided, if possible. Drop structures shall be provided if drop is more than 0.6 m. Joints and gaskets shall conform to CSA B 182.1 and CSA B 182.2.

E3.4 Pipe Head Losses Calculations

Pipe head losses shall be calculated using the following formula:

$$h_f = f\,\frac{L\,V^2}{D\,2g} \quad \text{or} \quad k\,\frac{V^2}{2g} \quad \text{or} \quad \frac{124.5\,n^2\,LV^2}{D^{4/3}\,2g}$$

Where h_f = Pipe head loss (m) (ie frictional loss through a pipe)

f = Darcy-Weisbach friction factor = $\dfrac{8\,g}{(1 / n \times R^{1/6})^2}$

n = Manning's friction factor

L = Length of pipe (m)

V = Flow velocity (m / s)

R = Hydraulic radius (m)

D = Pipe diameter (m)

g = Acceleration of gravity (m / s^2)

k = Head loss coefficient ($f \times \frac{L}{D}$)

E3.5 Manhole Head Losses and Bend Losses Calculations

Manholes Head Losses

For losses through manholes, the applicable k (head loss coefficient) varies with the structure and the type of junction.

In a straight-through manhole with one incoming and one outgoing pipes, k = 0.05 and the resulting manhole loss (h_m) is:

$$h_m = 0.05 \frac{V_2^2}{2g} \quad (m)$$

Where V_2 = outflow velocity (m / s)

For a manhole that has incoming and outgoing main pipes (ie manhole on mainline) with one or more lateral pipes, k is calculated based on the velocities of the mainline pipes only (not the laterals) and the angle of lateral pipes to the mainline. The head losses at manholes are calculated as follows or as given in “Design and Construction of Urban Stormwater Management Systems” prepared by ASCE:

$$90^0 \text{ Lateral, } h_m = 0.75 \frac{V_2^2}{2g} \quad (m)$$

$$60^0 \text{ Lateral, } h_m = 0.65 \frac{V_2^2}{2g} \quad (m)$$

$$45^0 \text{ Lateral, } h_m = 0.50 \frac{V_2^2}{2g} \quad (m)$$

$$22.5^0 \text{ Lateral, } h_m = 0.25 \frac{V_2^2}{2g} \quad (m)$$

Bend Losses

Bend losses in pipes can be estimated by using the bend loss coefficients in conjunction with the established equations in hydraulic engineering practice.

Head losses applied at the beginning of bend (h_b) are as follows or as given in “Design and Construction of Urban Stormwater Management Systems” prepared by ASCE:

$$90^0 \text{ Bend, } h_b = 0.50 \frac{V_2^2}{2g} \quad (m)$$

$$60^0 \text{ Bend, } h_b = 0.43 \frac{V_2^2}{2g} \quad (m)$$

$$45^0 \text{ Bend, } h_b = 0.35 \frac{V_2^2}{2g} \quad (m)$$

$$22.5^0 \text{ Bend, } h_b = 0.20 \frac{V_2^2}{2g} \quad (m)$$

E4 CATCHBASIN REQUIREMENTS

E4.1 Catchbasins Types

Typical details for single, double, and rear lot type catchbasins are shown in the Standard Drawings and O.P.S.D. Standards.

Any special catchbasins and inlet structures proposed shall be fully designed and detailed by the Consulting Engineer in the Engineering Drawings for acceptance by the Director of Engineering.

Double catchbasins shall be installed at the low point of any road where drainage is collected from two or more directions. Single catchbasins may be acceptable at low points approaching intersections where drainage is mostly from one direction.

Catchbasins shall be precast and shall be designed and constructed in accordance with the Standard Drawings, O.P.S.D. and O.P.S.S. requirements.

Catchbasins in rear yards and other grassed areas such as parks shall not contain sumps.

Catchbasin manholes are not accepted in main sewer line. However, a catchbasin manhole may be accepted where it is connected to a catchbasin on the other side of the road where there is no main sewer line.

E4.2 Location and Spacing

Catchbasins shall be selected, located, and spaced in accordance with the conditions of design. The design of the catchbasin location and type shall take into consideration the lot areas, the lot grades, pavement widths, road grades, and intersection locations. No catchbasins shall be located in walkways.

Maximum spacing for catchbasins shall be as follows:

Road Grades	Maximum Spacing	
	Two Lane Road	**Four Lane Road**
0.7% to 4.0%	110 m	60 m
> 4.0% up to 6.0%	75 m	45 m

NOTE: *For cul-de-sacs, the distance shall be measured along the gutter.*

Catchbasins shall be generally located upstream of sidewalk crossings at intersections and upstream of all pedestrian crossings.

Rear lot catchbasins shall be located to drain a maximum of 0.1 ha or 4 rear yards, whichever is smaller.

E4.3 Catchbasin Leads

The minimum size and slope of catchbasin leads shall be:

Catch Basin Type	Minimum Connection Size (mm)	Minimum Grade (%)
Single Road CB	200	1.0
Double Road CB	300	1.0
Rear Lot CB	250	0.5

Catchbasins leads shall be connected to the storm sewer and not directly to manholes.

Rear lot catchbasin leads shall be installed as follows:

- Rear lot catchbasins and leads shall be constructed using concrete pipe and shall be within one lot, with the centre of the lead 0.5 m off the lot line
- Where the concrete pipe lead goes between houses, concrete encase the lead between the front building line and the rear building line
- Where PVC pipe is used, concrete slab (minimum 1.0 m x 0.15 m thick) shall be provided for safety of the pipe within the lot limits

E4.4 Frames and Grates

The frame and cover for catchbasins shall be as detailed in the Standard Drawings and O.P.S.D. Standards. In general, catchbasin grates shall be square flat grade type (OPSD 400.100, OPSD 400.110) for catchbasins located in roadway, parking or walkway areas.

Bird cage frames and covers, flat top frames and covers, or ditch inlet catchbasins shall be used for all parks and school grassed open areas, as required by the users (parks / schools). Beehive covers shall be used for rear lot catchbasins.

E4.5 Inlet / Outlet Structures

A ditch inlet catchbasin may be used where more substantial drainage areas shall be drained into a storm sewer system. Where additional inlet capacity is required, inlet structures shall be designed specifically for the required application. Ditch inlet grating sizing shall be designed assuming 50% blockage.

Inlet grates shall generally consist of inclined parallel bars or rods set in a plane at approximately 45^0 with the top away from the direction of flow. Gabions, Rip-Rap or concrete shall be provided at all inlets to protect against erosion and to channel the flow to the inlet structure. Storm sewer headwalls shall be constructed in accordance with the OPSD Standards. All headwalls shall be equipped with a grating over the outlet end of the pipe and a 1.2 m chain-link fence across the top of the headwall and along its sides for the protection of the public.

Directional change shall be accomplished within the sewer upstream of the outfall in order to minimize erosion within the watercourse.

Erosion protection shall be indicated on the Engineering Drawings and shall be dependent upon the velocity of the flow in the storm sewer outlet, the soil conditions, the flow in the existing watercourse and site conditions. Materials shall be selected based upon the recommendations in an accepted Stormwater Management Report. Erosion protection calculations shall be provided.

E5 BEDDING & PIPE SELECTION

The type and classification of storm sewer and the sewer bedding type shall be clearly indicated on all plan & profile drawings for each sewer length.

All storm sewers shall conform to the requirements of the Canadian Standards Association.

E5.1 Bedding

The class of pipe and the type of bedding shall be selected to suit loading and proposed construction conditions.

All pipes attached to manholes shall be supported from manhole to first pipe joint as per OPSD 708.020.

Storm and FDC sewer bedding shall be as per OPSD-802.010 for flexible pipes and OPSD-802.030 Class ‘B’ for rigid circular pipes and OPSD-802.050 Class ‘B’ for rigid elliptical pipes unless otherwise specified by the Geotechnical Engineer.

Storm and FDC sewer bedding in water bearing sand and silt (wet trench condition) shall consist of minimum 20 mm crusher-run limestone as detailed in Engineering Drawings. The necessity for implementing these measures can be assessed at the time of trench excavation by a Geotechnical Engineer.

The width of trench at the top of the pipe shall be carefully controlled to ensure that the maximum trench width is not exceeded unless additional bedding or higher strength pipe is used (refer OPSS 514).

E5.2 Polyvinyl Chloride Pipe (PVC)

The maximum allowable deflected pipe diameter 7.5% of the base inside diameter of the pipe. Deformation gauge (Mandrel) test shall be required for all sewers prior to Acceptance.

For PVC pipe, the initial maximum allowable deflection of PVC pipe under load shall be in accordance with the pipe manufacturer’s specifications. The pipe class shall be selected in accordance with the bedding type, depth of sewer, trench width, and soil conditions. The Consulting Engineer may be required to submit pipe loading calculations in support of their design. These calculations shall be based on the Modified Iowa Formula.

Storm sewers 375 mm in diameter or smaller shall be constructed either from PVC or concrete. Sewers 450 mm diameter and greater shall be concrete.

PVC gravity sewer and fittings shall conform to CSA B 182.1 or CSA B 182.2. The pipe shall have a maximum Standard Dimension Ratio (SDR) of 35 and a minimum pipe stiffness of 320 kPa. Storm sewers (mainline pipe) shall be green in colour while service connection pipe shall be white in colour.

Sewers, fittings, joints, and gaskets shall be fabricated in accordance with CSA B182.1, CSA B182.2 and CSA B182.4.

E5.3 High Density Poly Ethelene Pipe (HDPE) (375 mm or smaller)

HDPE pipe and fittings for storm sewers shall conform to CSA B 182.6 and a minimum pipe stiffness of 320 kPa. HDPE pipe shall have a light coloured interior to facilitate CCTV inspections. HDPE pipes may be used at the sole discretion of the Director of Engineering.

E5.4 Rigid Pipe

The type and classification of sanitary sewer and the sewer bedding type shall be clearly indicated on all plan & profile drawings for each sewer length.

Non-reinforced concrete sewers and fittings less than 300 mm in diameter shall be fabricated in accordance with CSA-A257.1, minimum Class 3 or latest amendment unless otherwise noted.

Reinforced concrete sewers and fittings 300 mm in diameter and greater shall be fabricated in accordance with CSA-A257.2 or latest amendment unless otherwise noted.

Joints and gaskets shall conform to CAN / CSA-A257.3.

All Tees and Wyes shall be pre-manufactured.

E5.5 Others

Any other sewer materials shall first be submitted to the Director of Engineering and can only be used if accepted by the Director of Engineering.

E6 ROOF LEADERS AND FOUNDATION DRAINS

E6.1 Roof Leaders

Roof leaders shall be discharged onto splash pads directing water away from house and towards drainage swales. No roof leader connection shall be allowed directly to the storm sewer (or sanitary sewer).

E6.2 Foundation Drains Collectors

A foundation drain collector (FDC) or “third pipe” system shall be considered where warranted by flat grades and minimal available outfall depth at the discretion of the Director of Engineering.

Foundation drains shall be connected by gravity to the storm sewer system provided that the elevation of the bottom of the basement floor slab is at least 1.0 m above the elevations of the storm sewer obvert at that point or minimum 0.5 m above the 100-Year HGL elevation.

Minimum size for a FDC shall be 200 mm.

Calculations shall be provided supporting recommended pipe sizes.

Separate manholes are required, not combined with any other system.

FDC shall not be laid on top of main Storm Sewers. Appropriate horizontal clearance shall be provided to ensure proper maintenance of both the systems.

FDC shall be designed on the basis of continuous flow rate of 0.10 l / s per residential (typical lot size of upto 400 m^2) lot plus infiltration or actual measured groundwater flow. For commercials / condominiums and other uses, FDC shall be designed based on actual measured groundwater flow and potential need.

E6.3 Sump Pumps

Where the above provisions for gravity connection of foundation drains cannot be met, a sump pump system shall be installed in the building and discharge to the storm sewer connection. All lots / blocks requiring sump pumps shall be identified clearly in the Storm Drainage Plans and Engineering Drawings.

The basement shall have 450 mm diameter cast-in-place non-reinforced concrete sump with 100 mm thick walls and base.

Foundation drainage tiles shall be cast into the sump walls and depth of sump below lowest inlet tile shall be 300 mm.

Provide a custom fabricated aluminum cover plate over the sump C / W for discharge piping and electrical conductors. Discharge piping shall be PVC schedule 40. Provide a check valve on discharge piping.

Typical sump pump would be a Myers Model S25, 1 / 4 horsepower submersible sump pump, 115 V, 9 AMP, single phase, 60 Hz or equivalent. Float controls shall be integral.

Where a storm sewer system is not available, the sump pump discharge shall be onto a 600 x 600 mm pre-cast concrete splash pad graded away from the foundation to the side-yard swale(s). Discharge shall be located at front yard (typical) ensuring that its discharge does not, in any way, affect the neighboring property.

The above information is only provided as a guideline. The Consulting Engineer shall design appropriate pump as per the specific situation.

E7 SERVICE CONNECTIONS

See Section M - Service Connections for information regarding storm service connections.

E8 OVERLAND FLOW

Overland flow routes shall be designed to convey flows in excess of the capacity of the minor storm sewer system. Overland flow routes shall be continuous either within the road right-of-way or by walkways to the nearest outlet, such as river, stormwater management pond, etc.

Maximum depth of flow shall be 250 mm in accordance with the City of Markham Stormwater Management Guidelines.

Where super catchbasins are to be installed to capture the major overland flow, the catchbasin inlet capacity shall be designed considering 50% blockage.

Where major flow (100-Year) is required to be captured in storm sewer through catchbasins, an unobstructed emergency flow route must be provided at this location to cater for events beyond 100-Year. The emergency flow route shall be designed with proper erosion protection works to safely convey 100-Year flow considering no attenuation. The Director of Engineering, at his discretion, may require an easement / block to be dedicated to the City for emergency flow route.

E8.1 Inlet Control Device (ICD)

Should the Consulting Engineer requires to use ICDs to control the ingress of runoff into the minor system, the ICDs shall be sized and spaced to limit runoff in excess of 5-Year.

Catchbasins shall be equipped with IPEX Inlet Control or approved equivalent where shown on plan / profile drawings.

ICD ratings shall be as follows (assumes 950 mm depth to orifice centerline, plus 250 mm maximum ponding at curb; total 1,200 mm head):

- Type ‘A’ 19.8 l / s
- Type ‘B’ 28.3 l / s
- Type ‘C’ 36.8 l / s

E9 STORMWATER MANAGEMENT

E9.1 Stormwater Management Guidelines

In general, the document “**Stormwater Management Guidelines**”, **by Paul Wisner & Associates - January 1995** for the Town of Markham and the MOE’s “**Stormwater Management Planning and Design Manual” - March 2003** shall be followed in the design and treatment of runoff quality (enhanced protection) and quantity control measures. The City guidelines may be obtained from the Engineering Department. The 1995 Stormwater Management Guidelines are currently being updated.

E9.2 SWM Pond Design Policy

Included in this section are the City policies specific to the design of SWM pond facilities, which are in addition to the Guidelines referenced in E9.1.

Stormwater runoff from new development shall be managed to achieve appropriate levels of quantity, quality, and erosion controls in order to minimize any adverse effects to downstream watercourses. Stormwater management ponds are frequently used to provide the necessary controls.

On-site control shall not be assumed for school site block when sizing a SWM pond facility.

The location of a SWM pond shall be based on site specific conditions and an appropriate analysis of environmental, technical (safety, maintenance, and operations), economic and social considerations and be subject to relevant the City and other approval authorities policies.

Stormwater management ponds shall be designed to provide a reasonable level of safety, both in terms of stormwater management function and in relation to potential use of the pond area by members of the public. Additional safety provisions may be required in areas where an increased level of public access may be anticipated, such as ponds that are integrated with adjacent to parks and pathways.

Stormwater management ponds shall be designed to facilitate ease of maintenance.

Stormwater management ponds shall also emulate a passive natural feature and provide a visual amenity for surrounding development. This can be achieved through a basic level of landscaping which is required to support stormwater management functions, ground stability, and safety.

In addition to safety and maintenance requirements, the Consulting Engineer shall also consider the latest MOE guidelines regarding storage requirements, maximum or minimum water depths, configuration, temperature mitigation, etc. and consult with the Director of Engineering on their applicability.

SAFETY CRITERIA

DESIGN FEATURE	OBJECTIVE	CRITERIA
Pond Depth (Difference between top of bank elevation and permanent pool elevation)	Provide barriers to prevent access to the permanent pool	Provide enhanced vegetative barriers and 3.0 m wide flat terraces at approximately mid-depth for ponds with total depths of 6.0 – 9.0 m. Terraces may be integrated with maintenance access roads.
Slope Grades	Reduce risk of uncontrolled fall	Slopes to be varied between 3:1 to 7:1, however 3:1 slopes shall be avoided in areas expected to have greater exposure to the public, otherwise consideration of enhanced vegetative barriers and / or terracing shall be required.
Tableland Buffer	Provide barrier to uncontrolled falls	Minimum 2.0 m wide buffer between top of the slope and the edge of the ROW or the edge of the pathway
Water Edge Treatment	Provide ease of egress from water	6:1 terrace at permanent pool edge, 3.0 m wide either side of permanent pool
Vegetative Barriers	Prevent falls	Ponds within residential areas shall be provided with enhanced vegetative barriers
Signage	Warn the public of potential hazards	All wet ponds must have the information / warning signage shown in the Standard Drawings
Safety Equipment	Facilitate rescues	Provide, in areas with greater exposure to public and, as required by the Director of Engineering

Clay Liner	Prevent interaction between the stormwater and the groundwater and to maintain the permanent pool level	Provide a minimum of 1.0 m thick compacted clay liner extended to the permanent pool or the seasonal high groundwater lever (whichever is higher) + 0.5 m
Chain Link Fence	Provides public safety	Provide a 1.5 m high black vinyl chain link perimeter fencing along the property lines of residential, commercial, industrial or institutional lands where they abut a stormwater management facility block Gates along fences shall not be allowed

MAINTENANCE CRITERIA

DESIGN FEATURES	OBJECTIVE	CRITERIA
Maintenance Roads	Facilitate access for maintenance vehicles to critical pond features	Roads shall be constructed on a granular base, covered with grass and minimum topsoil, 4.0 m wide within a 5.0 m “no shrub / tree” zone, 2% cross-fall, 10% gradient with maximum 15% gradient. Refer to Standard Drawing MP4.
Access To Pond Inlet / Outlet	Facilitate maintenance of pond inlets / outlets	Create routes, accessible by personnel and maintenance vehicles, to top and bottom of inlet and outlet structures.
Access To Sediment Forebay	Facilitate removal of sediments	Grade of ramp shall be 10% with maximum 15% gradient maintenance access above permanent pool.
Sediment Forebay Bottom Treatment	Provide adequate bearing capacity for maintenance vehicles removing sediment	4.0 m wide ramp of adequate bearing capacity shall continue to the bottom of the permanent pool.
Vegetation	Stabilize ground surface, enhance stormwater control effectiveness, safety, and aesthetics	Vegetation shall be native species requiring minimal maintenance and suited to variations in water levels experienced in ponds (ie see MOE guidelines). For pond depths < 6.0 m, basic slope landscaping shall contain grasses and shrubs of adequate density to discourage public access and geese.
Sediment Dewatering Area	Dewater sediment	Temporary dewatering areas for sediment shall be provided within the SWM block if there is no adjacent park.

E10 WATERSHED FLOOD CONTROL CRITERIA

This section details the watershed flood control criteria related to the Rouge River, Don River, Highland Creek, Duffins Creek, and Petticoat Creek Watersheds in the City of Markham (Figure 1). The following criteria are intended to manage riverine-based flood risks related to design flows that affect flood hazards along watercourses and at watercourse crossings. The following generalized watershed criteria are suitable for greenfield development within the City of Markham and have been derived from the TRCA 2009 Criteria Document.

Figure 1

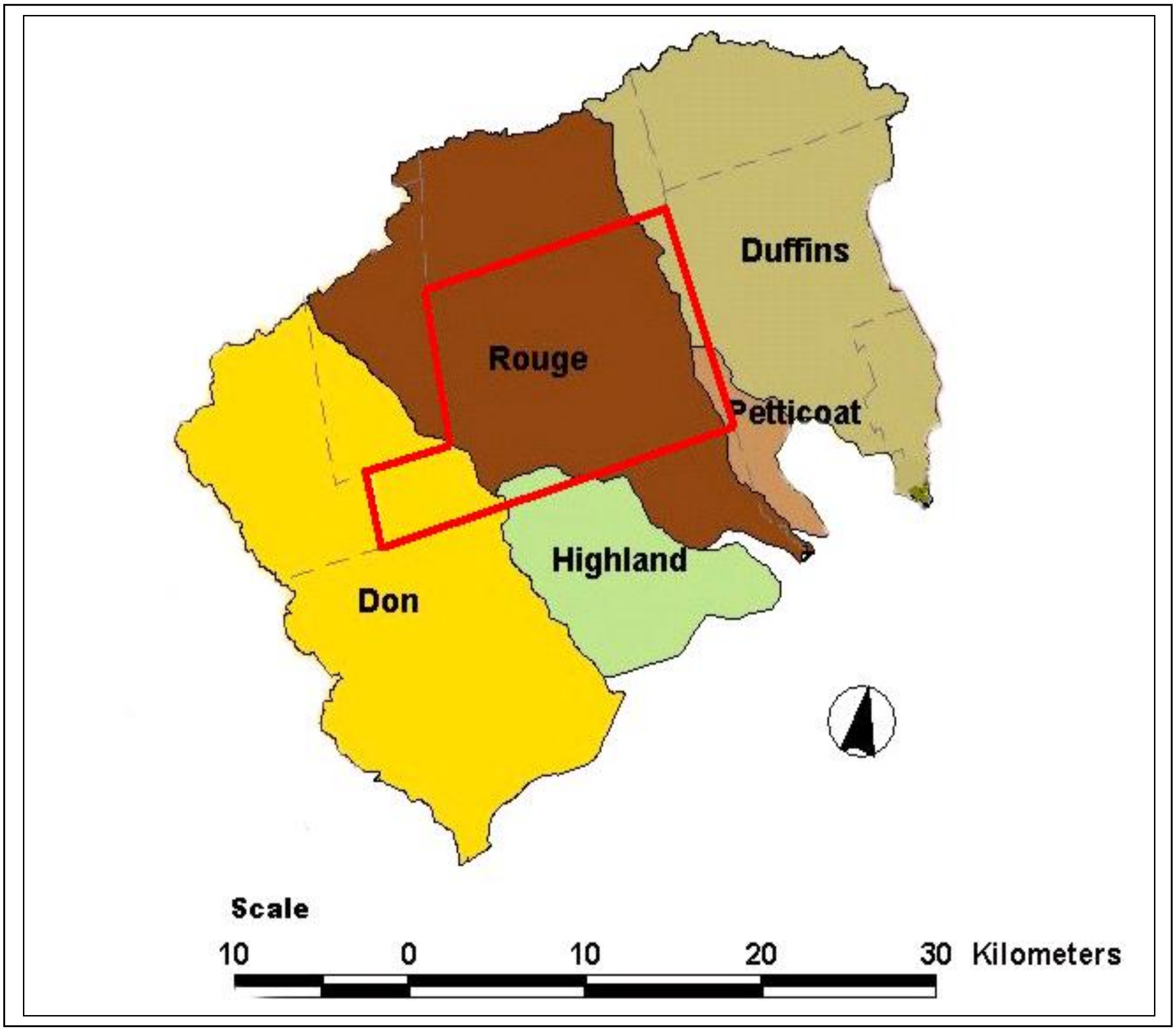


Table: Flood (Quantity) Control Criteria

Watershed	Flood (Quantity) Control Criteria	References & Notes
Rouge River	• Control post-development peak flows to pre-development levels for all storms up to and including the 100-Year storm (i.e. 2, 5, 10, 25, 50 and 100-Year storms), for the following: ➢ Rouge River (main channel) and tributaries upstream of Major Mackenzie Drive ➢ Leslie Street Tributary upstream of Major Mackenzie Drive ➢ Beaver Creek (upstream of 16th Avenue) ➢ Carlton Creek (upstream of Warden Avenue) ➢ Burndenett Creek, Robinson Creek, and Exhibition Creek (all upstream of 16th Avenue) ➢ Box Grove Tributary, Morningside Tributary ➢ Katabokokonk Creek ➢ Kennedy Road Tributary, McCowan Road Tributary of the Little Rouge River ➢ Bruce Creek upstream of 16th Avenue ➢ Berczy Creek upstream of Warden Avenue ➢ Hwy 48 Tributary ➢ Carlton Creek • No flood flow Control requirements for: ➢ Main Rouge - downstream of Major Mackenzie Drive ➢ Little Rouge River (downstream of the confluence of Kennedy Road, McCowan, and HWY 48 Tributaries) near Elgin Mills Road ➢ Beaver Creek (downstream 16th Avenue) ➢ Berczy Creek (downstream of Warden Avenue) ➢ Bruce Creek downstream of 16th Avenue ➢ Burndenett Creek, Robinson Creek, and Exhibition Creek (all downstream of 16th Avenue) ***Note:*** *Further study is required to determine the appropriate level of control for lands draining to contributing tributaries of the above noted watercourses*	• Hydrologic Model: Visual OTTHYMO (V2.0)-Return period peak flows based upon 12 hour AES distribution • Hydrology Study: "Rouge River Watershed Hydrology Update" (Marshall Macklin Monaghan, October 2001)
Don River	• Control post-development peak flows to pre-development levels for all storms up to and including the 100-Year storm (i.e. 2, 5, 10, 25, 50 and 100-Year storms). • Unit flow relationships representing pre-development levels (ie flow targets on a per area basis) have been established and should be used for all sites located north of Steeles Avenue that are greater than 2 ha. See Figure 2.	• Hydrologic Model: Visual OTTHYMO-Return period peak flows based on 12 hour SCS event • Hydrology Study: *Don River Hydrology Update* (Marshall Macklin Monaghan Ltd., Dec. 2004)

<table>
<tr>
<td>Highland Creek</td>
<td>
Control post-development peak flows to pre-development levels for all storms up to and including the 100-Year storm (i.e. 2, 5, 10, 25, 50 and 100-Year storms)
</td>
<td>

Hydrologic Model: Visual OTTHYMO
Return period peak flows based on 6hour AES event
Hydrology Study: Highland Creek Hydrology Update (Aquafor Beech Ltd., December 2004)

</td>
</tr>
<tr>
<td>Petticoat Creek</td>
<td>
Control post-development peak flows to pre-development levels for all storms up to and including the 100-Year storm (i.e. 2, 5, 10, 25, 50 and 100-Year storms)
</td>
<td>

Hydrologic Model: Visual OTTHYMO (Version 2.0)
Return period peak flows based upon 12hour AES event
Hydrology Study: "Petticoat Creek Watershed Hydrology Update" (Greenland Consulting Engineers, 2005)

</td>
</tr>
<tr>
<td>Duffins Creek</td>
<td>

Control post-development peak flows to pre-development levels for all storms up to and including the 100-Year storm (i.e. 2, 5, 10, 25, 50 and 100-Year storms) except for the main branches of the East and West Duffins where no quantity control is required
Unit flow relationships have been established representing pre-development levels (ie, flow targets on a per area basis) have been established and should be used for all sites located in the Duffins Creek Watershed. For the portion of Duffins Creek within the boundaries of the City of Markham, unit flow rates can be calculated based on the following equations. Should future development be proposed beyond those assumed for the official plan scenario in the 2002 Duffins Creek Hydrology Update, “post-to-pre” runoff controls may be required regardless of the location within the watershed and an assessment will also be required to determine whether Regional Storm quantity controls will be necessary for such developments.

Unit flow Relationships for Duffins Creek in the City of Markham
<table>
<tr><th>Return Period</th><th>Equation – [note: Q (l / s); Drainage Area (ha)]</th></tr>
<tr><td>2- Year</td><td>Q_2 = 6.125 – 0.675 * LN (Drainage Area)</td></tr>
<tr><td>5-Year</td><td>Q_5 = 8.601 – 0.890 * LN (Drainage Area)</td></tr>
<tr><td>10-Year</td><td>Q_{10} = 11.032 – 1.168 * LN (Drainage Area)</td></tr>
<tr><td>25-Year</td><td>Q_{25}= 14.199 – 1.530 * LN (Drainage Area)</td></tr>
<tr><td>50-Year</td><td>Q_{50} = 15.580 – 1.612 * LN (Drainage Area)</td></tr>
<tr><td>100-Year</td><td>Q_{100} = 17.972 – 1.870 * LN (Drainage Area)</td></tr>
</table>
</td>
<td>

Return period peak flows based on the AES - 6 hour design storm
hydrology study: “Duffins Creek Hydrology Update" (Aquafor Beech Ltd., May 2002)
Example: 100-Year pre-development flow for a 40 hectare development:
- Q_{100} = 17.972 – 1.870 * LN (40), where LN is the natural logarithm function
- Q_{100} = 17.972 – (1.870 * 3.69)
= 11.1 l / s

</td>
</tr>
</table>

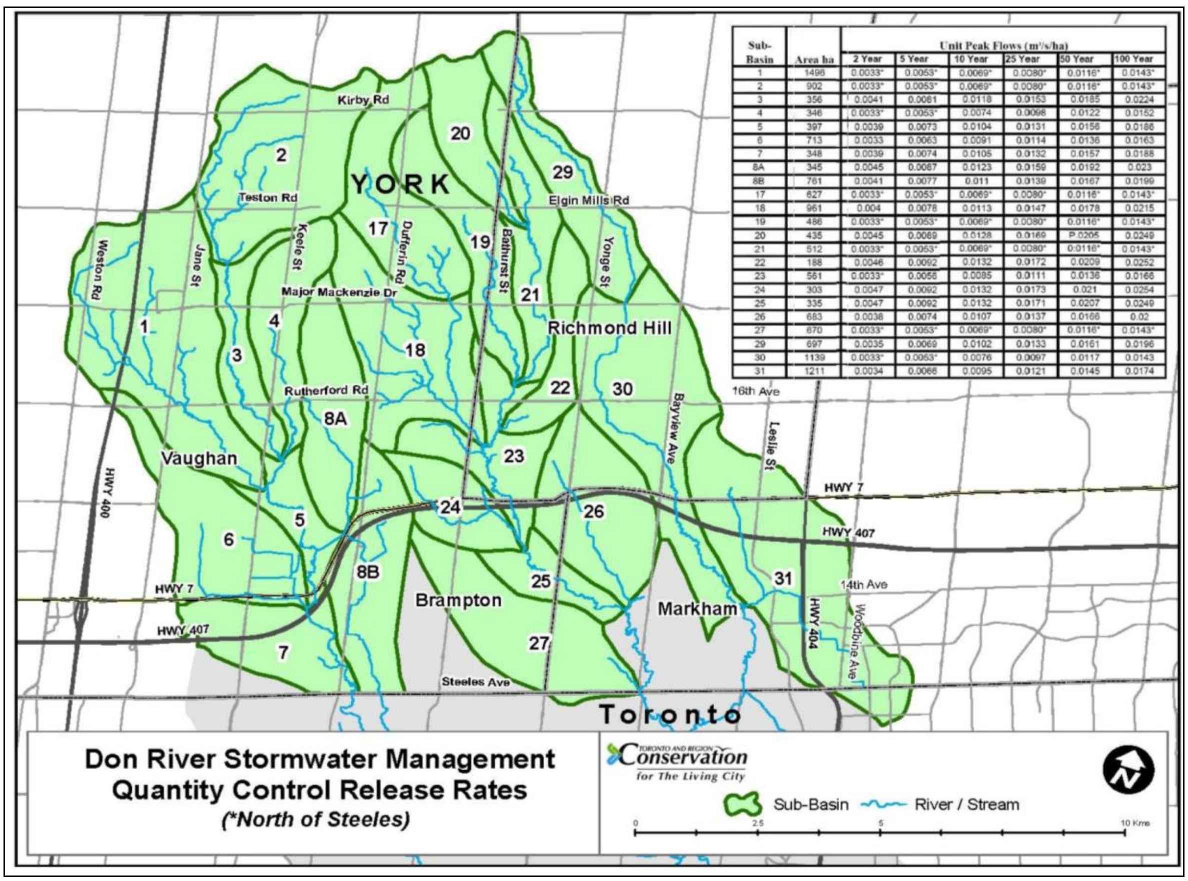
***(**Source: 2009 TRCA Criteria Document)*
Refer to the latest TRCA requirements

The watershed flood criteria outlined in the Table above may be superseded by:

- Local constraints such as:
    - Flood vulnerable areas
        - Special Policy Areas (SPA)
        - Flood Damage Centers
    - Active valley land uses and
    - River or creek crossings
- Results of continued local studies that have identified flood vulnerable areas (e.g., Don Mills Channel Class EA, watershed plans)
- Local flood control requirements considering downstream drainage system capacity.

In all cases the proponents should consult with the City of Markham and the TRCA staff to confirm the criteria to be utilized. The same design storm distribution as used in the approved hydrology model may be considered when addressing flow management criteria. Any proposed distributions must first be reviewed with respect to the storm's time step discretization in relation to the hydrologic response time of the specific development. Where appropriate the watershed distribution may be updated to provide a suitable time step, or the City's design storm distribution may be considered for the purpose of local analysis. In some instances both the watershed and City distributions must be used to select the governing requirements.

Figure 2



Sub-Basin	Area ha	Unit Peak Flows (m³/s/ha)					
		2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
1	1496	0.0033*	0.0053*	0.0069*	0.0080*	0.0116*	0.0143*
2	902	0.0033*	0.0053*	0.0069*	0.0080*	0.0116*	0.0143*
3	356	0.0041	0.0081	0.0118	0.0153	0.0185	0.0224
4	346	0.0033*	0.0053*	0.0074	0.0098	0.0122	0.0152
5	397	0.0039	0.0073	0.0104	0.0131	0.0156	0.0186
6	713	0.0033	0.0063	0.0091	0.0114	0.0136	0.0163
7	348	0.0039	0.0074	0.0105	0.0132	0.0157	0.0188
8A	345	0.0045	0.0087	0.0123	0.0159	0.0192	0.023
8B	761	0.0041	0.0077	0.011	0.0139	0.0167	0.0199
17	627	0.0033*	0.0053*	0.0069*	0.0080*	0.0116*	0.0143*
18	961	0.004	0.0078	0.0113	0.0147	0.0178	0.0215
19	486	0.0033*	0.0053*	0.0069*	0.0080*	0.0116*	0.0143*
20	435	0.0045	0.0089	0.0128	0.0169	P.0205	0.0249
21	512	0.0033*	0.0053*	0.0069*	0.0080*	0:0116*	0.0143*
22	188	0.0046	0.0092	0.0132	0.0172	0.0209	0.0252
23	561	0.0033*	0.0056	0.0085	0.0111	0.0136	0.0166
24	303	0.0047	0.0092	0.0132	0.0173	0.021	0.0254
25	335	0.0047	0.0092	0.0132	0.0171	0.0207	0.0249
26	683	0.0038	0.0074	0.0107	0.0137	0.0166	0.02
27	670	0.0033*	0.0053*	0.0069*	0.0080*	0.0116*	0.0143*
29	697	0.0035	0.0069	0.0102	0.0133	0.0161	0.0196
30	1139	0.0033*	0.0053*	0.0076	0.0097	0.0117	0.0143
31	1211	0.0034	0.0066	0.0095	0.0121	0.0145	0.0174

E11 GUIDELINES FOR DEWATERING APPLICATIONS (INTERIM POLICY)

E 11.1 Permanent Dewatering

Permanent Dewatering is not allowed in any project in the City, where negative impacts to surface watercourses and natural features occur and where these impacts are not mitigated.

Permanent dewatering can have a significant and profound impact on the terrestrial and aquatic environment by lowering the groundwater table or reversing groundwater flow that would ordinarily sustain vegetation and sensitive fishery habitats. Lowering of groundwater levels may impact the performance of existing water wells, if any, within the Zone of Influence (ZOI). Experience in some areas of Ontario has shown that permanent dewatering could cause normally perennial flowing creeks and watercourses to run dry during summer seasons due to loss of base flow. The Rouge River watershed Plan notes that some local tributaries receive most of their base flow from local shallow groundwater sources.

The City and TRCA would, however, consider the application of site-specific groundwater dewatering / depressurization to occur where appropriate mitigation measures are implemented that counteract the negative impacts to surface watercourses and natural features.

Site-specific hydrogeologic / geotechnical investigations must be undertaken that characterize existing soil and groundwater conditions but also identify and quantify potential negative impacts to watercourses and natural features. The hydrogeologic study must also identify and characterize the nearby watercourse(s) and natural features (ie wetlands) within the ZOI that may be impacted by permanent dewatering / depressurization activities. An ecological study must be performed that identifies all sensitive natural features that could be impacted by the proposed works. The mitigation measures must be presented that counteract the identified negative impacts and must be in conformation with the EIA Report. A monitoring program must be developed to assess the effectiveness of the mitigation measures implemented.

Inventory of the existing wells within the ZOI shall be identified and well monitoring and mitigation plan, as required, shall be provided for City review and acceptance as outlined in Section E 11.2 (4).

E 11.2 Temporary Dewatering

Temporary Dewatering is accepted during the construction periods subject to the following conditions:

1) The Geotechnical Engineer or Hydrogeologist shall complete a field test pit, borehole test or a hydrogeological study to determine the actual pumping rate that will occur during the construction periods.
2) A report, duly signed and stamped by a qualified Geotechnical Engineer / Hydrogeologist shall be submitted to the City to include at a minimum the following information:
    a) Test results and expected pumping rate and ZOI
    b) Identify any impact on adjacent wells and natural environmental features (e.g. watercourse, wetland, wood lot, etc.) and its Mitigation Plan
    c) State any impacts from temporary dewatering will be mitigated *during and after* completion of the dewatering work. Appropriate parameter threshold values, target levels, and mitigation strategies for the project will be developed and can be incorporated into an Environmental Impact Assessment (EIA) study
    d) A plan showing the discharge locations, flow rate, storm capacity if discharging to a storm sewer, water quality control measures, etc. and Site Alteration Plans. Approvals must be obtained from the appropriate City's department and if required, York Region before discharging to any outlet

e) Discharge must be sampled and conform with applicable By-laws

f) Environmental Mitigation Plan, if required, conforming to the accepted EIA report

3) If the test pumping indicates that the temporary pumping rate is greater than 50,000 l / d, an MOE's Permit-To-Take Water (PTTW) is required, copy of which shall be submitted to the City before construction starts. No PTTW is required if the pumping rate is below 50,000 l / d, but a report as identified above will be required.

4) If a well monitoring and mitigation plan is required as a result of the above findings, a qualified Geotechnical Engineer or Hydrogeologist shall submit to the City a well monitoring and mitigation report for review and acceptance, as outline below. The report shall be subject to peer review at Owner's cost:

a) Detail hydrogeological investigation regarding aquifers and groundwater conditions

b) Inventory of existing wells in the ZOI which may be impacted by the dewatering activities

c) Baseline survey for water quality and quantity in the existing wells prior to any construction activities

d) Carry out water quality and quantity survey during construction and post construction for at least one year after construction is completed, at regular intervals (minimum once in three months)

e) Prepare a short term and long term mitigation / contingency plan which include 24 hour emergency contacts and investigation protocols

f) Communicate monitoring program and mitigation plan to the owners of wells

g) The Owner shall deposit a Letter of Credit at the rate of $20,000 per private well in the ZOI up to a maximum of $200,000 for ensuring mitigation measures. The Letter of Credit shall be released following acceptance by the City of a post-construction monitoring report demonstrating that water level and water quality parameters have returned to their pre-construction conditions

E12 MOE'S ENVIRONMENTAL COMPLIANCE APPROVALS

MOE's Environmental Compliance Approvals (ECA) for Municipal and Private Sewage Works and Dry Pond (only for quantity control) is required prior to starting any servicing at site. The sumbission shall be reviewed by the City under the Transfer of Review program.

MOE's ECA for Stormwater Management Ponds and / or Oil Grit Separators is required prior to starting any related works. This is a direct submission to MOE, where the City only signs the forms.

Refer to Engineering Submissions Required Documents (Annex 1) for details.

STORM SEWER DESIGN (5-YR)

PROJECT NAME ______________________________

SHEET NO: ______________________
JOB NO: ______________________
DATE: ______________________
CONSULTANT: ______________________
AMANDA NUMBER:______________

Design Return Period = 5 Yrs, n = 0.013

Rainfall Intensity 'I' = $A/(t + B)^c$
A =
B =
C =
Starting 't' = 10 min

Location			Runoff				Rainfall Intensity I	Cumm Flow	Pipe Data			Full Capacity	Full Velocity	Qact/Qcap	Time (Entry 10 min.)	
Street Name	Manhole No. From	Manhole No. To	Area(A) (ha)	(R)	(A x R)	Cumm (A x R)	(mm/hr)	(m^3/s)	Length (m)	Diameter (mm)	Slope (%)	(m^3/s)	(m/s)		Sect. (min)	Accum. (min)

Notes

Prepared By:

STORM SEWER DESIGN / HYDRAULIC GRADE LINE (HGL) ANALYSIS (100-YR)

PROJECT NAME ______________________________

SHEET NO: ____________________
JOB NO: ____________________
DATE: ____________________
CONSULTANT: ____________________
AMANDA NUMBER: ______________

Design Return Period = 100 Yrs, n = 0.013

Rainfall Intensity 'I' = A/(t + B)^c

A =

B =

C =

Starting 't' = 10 min

Location			Runoff				Rainfall Intensity	Cumm Flow	Pipe Data					Full Capacity	Full Velocity	Time (Entry 10 Min.)		Flow (Act/Full)	Hydraulic Grade Line Analysis					
	Manhole No.		Area(A)			Cumm	I		Length	Diameter	Slope					Sect.	Accum.		U/S	D/S	Surcharging (m)	Basement	U/S HGL less	Verification
Street Name	From	To	(ha)	(R)	(A x R)	(A x R)	(mm/hr)	(m^3/s)	(m)	(mm)	(%)			(m^3/s)	(m/s)	(min)	(min)		HGL (m)	HGL (m)	Above U/S Obv.	Elevation (m)	Basement Ele(m)	

Notes:

Prepared By: