McGill University, Macdonald Campus

Detention Ponds in Agricultural Fields

Final Project report

Engineering design 3
Bree 495

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Executive Summary:

As it has been exposed by the recent media coverage of events such as the blue-green algae bloom, the euthrophication of the different lakes and rivers is becoming a bigger concern for the Quebec society. As new method are being researched to reduce nutrient loss from agricultural land, some of those methods lack of clear guideline and tool for their application, this is the case for detention pond.

By looking at the different guidelines available for urban and rural pond, information was collected in order to create a comprehensive but user friendly list of the design criteria required to design a detention pond. This list include elements such as: Field conditions, Volume Calculation, Shape of the pond, Bank stabilization, Design of the water outlet, Design of Culvert, and Estimation of Sedimentation. That information was combined in order to create a technical sheet that would prioritize simplicity while maintaining accuracy. Because of this some information had to be simplified or left out, this report contains the summary of the research that was made on those different aspects as well as the reasoning behind the simplification.

Also, in conjunction with the technical sheet, a simulation program was created to let the user test a pond design under different conditions. The program allows the user to input criteria such as the conditions of the field, the type of precipitations, the shape of the pond and the size of the water outlet. From such criteria, the program simulates a hydrological event and provides the user with information such as the theoretical peak flow reduction and overflow time. This report contains the user guide for the simulator as well as a section-by-section overview of the different calculations that are made by the program.

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1-Introduction:

The implementation of surface and sub-surface drainage structures is important in the agricultural history because it allowed farmers to achieve higher crop productivity. At the same time though, runoff is taking less and less time to get back to rivers, causing floods, higher erosion rate and nutrients removal from the land. It has been observed that the quality of fresh water streams in the St-Lawrence basin is extremely low.(Stephane, Prevost, & Thomas, 2010) Occurrence of blue-green algae has increased tremendously between 2004 and 2007(Stephane et al., 2010). Even though many factors are contributing to these problems, they can somewhat be relate to high phosphorus levels. Recently, the Quebec government has taken a more aggressive stance to address these problems. For example, in 2005 regulation has been changed and they are using the P index to ensure farmers reduce application of fertilizers on the field. Also Quebec government initiated the Plan Prime Vert in 2008 which will allocate 145 $M over a period of 10 years to reduce phosphorus level(MAPAQ, 2009).

There are several best management practices (BMPs) that can be put into practice to reduce level of phosphorus downstream; the installation of buffer strips, reduction of phosphorus application, no-till practices and crop rotation are only a fraction of the different methods available to a farmer to reduce nutrient loss. Some of these practices are regulated by law, such as the buffer zones, however in many cases those laws are difficult to enforce and many violators can be found while touring an agricultural region. While some are regulated, most of the Best Management Practices (BMPs) rely on the cooperation of the farmer and the incentive of program such as Prime-Vert, a two-year monetary incentive that gives to farmers that implement no-tillage practices(MAPAQ, 2009).

While the practice that are currently being applied has been proven to be effective, research is currently being done to develop better ways to control the nutrients loss. An idea currently being considered is the use of small detention ponds on agricultural field as a way to reduce the peak flow and increase the sedimentation. As each part of land is precious on an agricultural field, a pond needs to be the most efficient possible and use the least amount of land possible.

While a lot of information is available on detention ponds in general, this information mostly address the design in urban area where detention ponds are used primarily as flood control. While conducting a pilot scale project on detention pond, expert from the MAPAQ found that there was a lack of design information on the adaptation of the pond in agricultural fields. This was especially true as they found that no information was available on how to adapt those ponds to Quebec conditions.

Based on those needs, the goals of this project were set as:

- Produce detention pond design guidelines that have been adapted to the agricultural Quebec conditions. Those guidelines should be produced under a technical sheet format , the information should be clear and easy to understand
- Produce a simulation program that will allow the user to test a design produced with the technical sheet under different field conditions such as varying precipitations and field sizes.

The technical sheet and the simulation program were built based on the information collected and exposed in the mid-project report that was produced in December 2010. Since the definition of the detention pond concept and the objectives were already outlined in detail in the mid-project report, this report will focus mainly on the description of the different design step that were taken to produce the technical sheet and the simulation program.

2-Technical sheet

The technical sheet that has been elaborated with this report presents most of the components listed in the technical sheet outline provided with the mid-project report. The purpose of this field guide is to provide the user with a brief description and step-by-step information for the design of detention ponds on agricultural fields. The content of this technical sheet has been selected with simplicity as the main objective: the goal is to provide readily available tools to the user to enable him to get a first good estimation for sizing the wet detention pond and estimate sedimentation efficiency based the chosen volume for the pond. Therefore every estimation tool is the simplest available in the literature while still providing an accurate enough estimation of the design components to enable the user to make preliminary decisions before undertaking more in-depth analysis of the components of the wet detention pond.

The technical sheet presented along with this report is final and complete in its content. However the images included in the guide are presented as placeholders. Since those images and diagrams have been borrowed from varied sources such as published guides, textbooks and internet, they cannot be kept if this technical sheet was to be published.

2.1 Detention pond overview

This section of the technical sheet provides background information on the different types of ponds available. It is aimed at helping the user to get familiar with the ponds properties, purpose, advantages and disadvantages. The information provided in this section was taken from mid-project report.

2.2 Field conditions

Wet detention ponds installation is intended as the last line of defense for capturing sediment when erosion has already occurred. The detention pond should be at a location where it intercepts as much of the runoff as possible from the field. The user should choose a location that would minimizes the number of entry points for runoff into the basin and interference with farming activities. (NRCS, 2010). This section also contains a complete list of information that the user needs to gather to use the technical sheet effectively

2.3 Volume calculation

The size of the pond is one of the most important characteristic of a detention basin.It influences the amount of water that can be retained during each precipitation event, as well as the total sedimentation and more importantly it influences the cost of the ponds since the machinery and the man power required to dig the pond is the most costly part of a detention pond.

Because of this, it is important to let the users calculate the minimum required volume to achieve their own flow reduction goals. As explained in the mid-project report a great variation between the different volume designs was found in the different guidelines consulted. Also, it was found that none of the guideline took into consideration the characteristics of the field used such as the soil type or the slope; instead values of volume in function of the area were given. To obtain a volume calculation method that would take into account those factors, it was necessary to obtain a hydrograph curve for a typical inflow. As explained in the mid-project report, the standard SCS dimensionless hydrograph curve was chosen as a sample inflow graph and the formula for this hydrograph was obtained from(Haan, Barfield, & Hayes, 1994)

A Matlab code, shown in Annex 1, was used to obtain the integrated values that represent the area under the curve between two hydrograph curves as shown in Annex 2. The total volume required to achieve a specific flow reduction can be obtained by multiplying the dimensionless integrated value by Q_p (Peak flow, m^3/h) and T_p (Time to peak, h).

Reduction wanted (%)	Dimensionless integrated Values
20	0.1538
30	0.2281
40	0.3024
50	0.3767
60	0.4510

Table 1 : Values in function of reduction wanted (Dimensionless Hydrograph curve)

Before applying those factors to any real volume calculation, it is important to consider the assumptions that were taken in order to obtain those results:

1. The inflow curve follow the SCS dimensionless hydrograph curve (Q(t))
2. The first part of the outflow curve for T=0..Tp follow a dimensionless hydrograph curve multiplied by a reduction factor R (Q = R * Q(t))
3. Once the maximal flow has been reached, at time T_p, the outflow will stay constant until it reach the inflow curve, at which point the basin will start to empty itself.

The first assumption can be assumed to be fairly safe as the dimensionless hydrograph method was designed to be used in this way; however it is important to note that the method was designed to simulate instantaneous precipitation is less accurate for precipitation that goes on for a long time. The

third assumption was made to simplify the integration calculation, however under real conditions, the outflow would keep increasing until the equilibrium point is reached (Q_{out}=Q_{in}). This means that the volume calculated would be larger than the real volume needed, however it would also mean that the real flow reduction would be smaller than the reduction that is aimed for. However both the volume and the reduction change should be fairly minor so this assumption seemed reasonable. The second assumption was the one that seemed to cause the most problems as it assumed an outflow that would perfectly follow a hydrograph curve. While the outflow is heavily correlated to the outflow, it would be foolish to presume that a perfect fit could be attained on the field. Because of this uncertainty and following Professor Prasher recommendation, it was decided that a second wave of calculation should be made with the following assumption:

1. The outflow follow a linear function : Q_{out} (t) = (1-R)*Q_p/T * t where :
    - R = Reduction factor * 100
    - T is the time when the outflow equal to (1-R)*Q_p i.e. Q(T)=(1-R)*Q_p

The code shown in Annex 3 was used to obtain the following results:

Reduction wanted (%)	Dimensionless integrated Values
20	0.3420
30	0.4743
40	0.5680
50	0.6349
60	0.6796

Table 2. Values in function of reduction wanted (Linear)

While the first method represented the perfect case scenario with a constant reduction during most of the event, this second scenario represents a relatively inefficient scenario where excessive flow reduction is reached until (1-r)*Q_p is reached. On a field, it would be possible to predict that the flow follow a curve located between these two extremes. However since thebehavior of the flow is heavily dependent on the design and condition of the water outlet (such as clogging) it was decided that the safest action would be to use the linear assumption in the volume calculation. It was also decided to add a safety factor of 25 % to the number obtained to assure a good efficiency, after some rounding the following values were obtained.

Minimal volume of the Detention storage area:

$$Vs = Qp \times Tp \times k$$

Where:
Q_p : Maximal flow (Suggested method, Modified SCS)
T_p : Time to peak (Suggested method, Mockus)
K: Flow factor in function of the reduction wanted

Reduction (%)	K
20	0.45
30	0.6
40	0.7
50	0.8
60	0.85

Table 3. K factor in function of the reduction to be achieved

The required volume obtained from using this formula will provide an volume estimation that accounts for both the return period of a precipitation and field conditions. This is possible since the term Qp used in the formula is derived from return period of a precipitation while the Tp is derived from the field conditions.. To calculate Q_p, the user will need to use a storm intensity, which will require the obtention IDF curve for the region where the pond will be located. A design return period will also have to be selected. From the literature it was found that this type of pond should be designed for return period varying from 2 to 10 years, with an aim at a return period of 5 year and a minimum of 2 year (NRCS, 2010; Pitt, 2004). Once the return period is selected, a duration equal to the time to peak should be used to obtain the intensity of the storm. With those parameters, it should be easy to calculate the volume required for any reduction wanted.

Once the storage area has been calculated, the sedimentation and flood storage should be calculated. From the NRCS detention pond design guide(NRCS, 2010), it was found that the sediment storage should be about 25 % of the detention storage which give the following formulas:

$$Vse = 0.25 * Vs$$

$$Vtotal = Vs + Vse$$

Once the total volume required has been calculated it is possible to proceed to the design of the pond itself. This volume should allow the pond to achieve the desired flow reduction.

2.4 Shape of the pond

Pond width to length ratio

When looking through different construction guides available in the literature (NRCS, 2010; USEPA, 1999),it was found that there was no consensus on the ratio to respect for the design of the detention pond. To achieve the highest peak flow reduction, the wet detention pond volume should be

as large as the field would allow it without taking up too much space from the field that is allocated to crop cultivation.

To achieve the highest sediment trapping efficiency, the length to width ratio should be as large as possible based on the selected volume for the pond as a higher ratio has been shown to improve sedimentation. It was chosen to recommend an ideal length to width ratio of 5:1 and a minimal of 2:1 based on the information found in the different guide.(NRCS, 2010; Pitt, 2004; USEPA, 1999). Those values are given as guidance, however the best situation would be to have ratio as large as possible while maintaining the other criteria such as minimum water depth.

Pond depth

In urban pond the height of the permanent water depth can have a big impact on the efficiency of the pond as stratification and anoxic condition can be reached.(USEPA, 1999) However, in the case of agricultural detention pond this situation should not pose any problem as they tend to occurs at depth higher than 6 meter(USEPA, 1999). However a permanent water depth that is too low could pose severe problem as it will not trap the sediment efficiently and will require a higher maintenance rate as sediment accumulate in the pond. Based on the different urban guide available, it was found that the pond should be between 3 to 9 feet of depth(Pitt, 2004), however this depth took in account parameter such as the safety of fish, which can be disregarded in the case of agricultural design. Because of those factors, it was decided to give a recommended depth of 1 meter for the different design of pond.

Once the length and width of the pond have been decided, the permanent water depth should be checked by dividing the sedimentation volume (Vse) by the area of the length (L) and the width (W). This number should provide an estimation of the permanent depth require satisfying the sedimentation volume. It is important to note that this formula will tend to slightly overestimate the water depth required.

$$Hpdepth = \frac{Vse}{W * L}$$

Pond side slopes

Pond side slopes are important to consider as they impact the safety, aesthetics and reduce erosion when the water enter the pond. They are also important in minimizing mosquito problems and excessive rooted plant growths.

The recommendations for simple pond side slope are the following and were included in the technical sheet (NRCS, 2010):

- 3 horizontal to 1 vertical or flatter for the section above the permanent water line

- 2 horizontal to 1 vertical for side slopes below the permanent water line

For more elaborated design:

An underwater shelf near the pond edge can be planted with rooted aquatic plants to prevent children's access to deep water. The following general dimensions for pond side slopes are suggested: (USEPA, 1999)

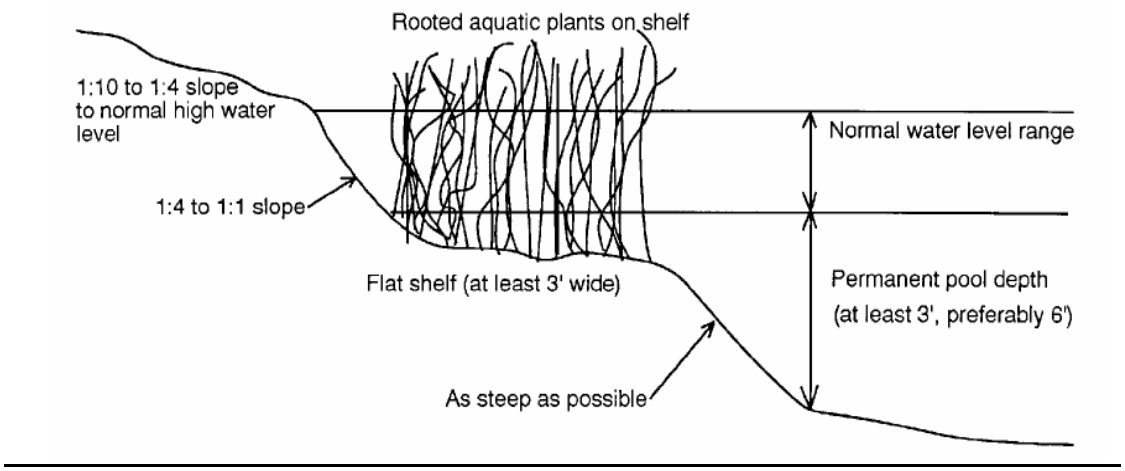

Figure 1 : Slope of the field from (USEPA, 1999)

The first set of recommendation were included in the technical sheet as they provided clearer guideline, also while a underwater shelf would provide a great increase in the safety of the pond it would also increase the area require by the pond. Since the goal of the technical sheet is to maximize the efficiency in function of the area and since it most case, the pond should not be installed in heavily frequented area, it was judged that an underwater shelf should not be mentioned in the technical sheet.

2.5 Bank stabilization

This section regroups all the features of a wet detention pond that play a role in the banks stabilization. It include specifications on the grassed waterway which provides cover to the soil surrounding the pond . The grassed waterway alsol play a role of cohesion in retaining the topsoil of the bank from eroding into the pond and a role of sediments catchment that are transported toward the pond. This section also include information on the use of rip-rap which will help to keep important structures into place and prevent erosion of critical area. Finally specifications on the installation of geotextile are given, as it installation might be required if the soil is considered too permeable.

2.6 Design of water outlet

The water outlet, with the volume of the basin is one of the most important components of a detention basin, however due to its complexity; it is hard to provide a comprehensive design guide about it. Because of the complexity of the fluid mechanics involve and the different stage of the flow, there would be enough information to make a report only on the design of the best water outlet. Because of the lack of time and knowledge in this area, it was decided to let the design of the water outlet itself to the engineer designing the pond; however the flow requirement would be given to the users.

Before going further in this section, let's define some variable that will be used:

H_p = the permanent height of the water

H_s = the maximal height of the detention area

H_e = the maximal height of the emergency storage, will also be considered as the point of overflow

$H_{critical}$ = the critical height at which point structure, road or property is endangered

R = Flow reduction factor, see volume calculation

Q_p(Re) = Peak inflow of a storm where Re is the return period of the storm, consider a 6 hour storm

Qout(H) = Outflow at height H

The first step in designing a good water outlet is to set the boundary condition; they can be described as follow:

Qout($H<=H_p$) = 0

Qout(H_s) = (1-R)Qout(R_{design}), R_{design} is the return period used to design the volume of the pond in the volume calculation section

Qout(H_e+0.1) = Qout(10 year), in this case the 10 cm values is given as a general guideline, the emergency spillway should be able to handle the peak flow of a 10 year return period storm within an acceptable water height to prevent crop damage

Qout($H_{critical}$)= Qout(15-50 years) the return period should be chosen by taking into consideration the type of structure endangered and the cost of possible damage.

A water outlet that is able to satisfy all of those condition should be safe for the field and its surrounding. An important aspect of a detention pond is that it should allow for an outflow that is steadily increasing in function of the height of the water. The goal of a detention basin is to reduce the peak flow of all precipitation regardless of the return period, this mean that there shouldn't be any sharp increase in the outflow. Because of this the outlets should be kept small and reasonably spaced. A

good example of an efficient water outlet would be an elevated riser with holes perforated around the pipes at every 10 cm step between the permanent water height and the critical height. However as some clogging problem was experienced with the elevated riser, the Mapaq engineers are currently looking to adapt flashboard riser to use in a detention pond. Another important aspect of the flashboard riser is that the spacing can easily be modify even after the installation.

Because the flow pattern of a multi-stage water outlet can be quite complex, it is heavily suggested that the designer should use simulation program to obtain the best approximation of the water outlet used. A water outlet that cannot meet the flow requirement could cause overflow problem which could lead to large efficiency drop during extreme event, while an outlet that allow bigger flow than it should, will not be as efficient to reduce the peak of the smaller event.

2.7 Design of culvert

In the case where a Culvert is required in the design of the pond, for example if the water has to cross a road before reaching a stream, the culvert should be able to handle the maximal flow of the water outlet ($Q_{out}(H_{critical})$) plus a certain safety factor. Culvert design information was not added to the technical sheet as it was considered a design by itself and giving a simplification in the design sheet could lead to trouble.

2.8 Estimation of sedimentation

The second main function a wet detention pond is to promote sedimentation of the soil that has been eroded and transported into the pond by runoff. There are three steps to follow to estimate the sedimentation in the pond:

1. Estimate the soil erosion using the RUSLEFAC
2. Estimate the sediment yield using the Forest Service method to predict the delivery ratio
3. Calculate the sediment trapping efficiency using the model for quiescent flow

2.8.1 Estimation of gross erosion

The first step in evaluating the sedimentation occurring inside the wet detention pond is to find out the amount of sediments in the water inflow after precipitation has occurred. To achieve this, the soil erosion caused by a precipitation can be estimated with the Revised Universal Soil Loss Equation (RUSLE). The RUSLE was first developed for the US climate conditions in mind, and since the goal is to adapt the installation of wet detention pond to Quebec climate conditions, the RUSLEFAC guide published by Agriculture and Agri-Food Canada which is the Revised Universal Soil Loss Equation For Application in Canada had to be used. This guide contain adjustments for the different factors required to evaluate soil erosion by taking into account the soil characteristics for each provinces. It also take into account winter and spring conditions since snow cover and snow melt can greatly influence the soil erosion rate.

The RUSLE equation is as follow: (Wall et al., 2002)

$$A = R \times K \times L \times S \times C \times P$$

A is the average annual soil loss (tonnes per hectare per year)

R is the rainfall/runoff factor (MJ mm ha^{-1} h^{-1})

K is the soil erodibility factor (t h MJ^{-1} mm^{-1})

L is the slope length factor (dimensionless)

S is the slope steepness factor (dimensionless)

C is the cover and management factor (dimensionless)

P is the supporting conservation practice factor (dimensionless)

For more details/precisions and explanations about the conditions influencing each factors involved in the calculation of the RUSLE, the user is referred to the RUSLEFAC guide. Only a brief summary of the utilisation of the RUSLEFAC specifically for Eastern Canada and Quebec region is provided in the technical sheet to enable the user to obtaina first estimate of the soil erosion by water.

2.8.1.1 Selection of the Rainfall Energy Factor R

The RUSLEFAC provides isoerodent map showing annual R values for all Canada provinces and also tables for monthly distribution of R values for certain regions. It also provides an Erosivity index and monthly distribution in percentage for important cities and regions of Canada. Figure 2 and Table 4 have been selected for the technical sheet as they provide R values for Quebec regions.

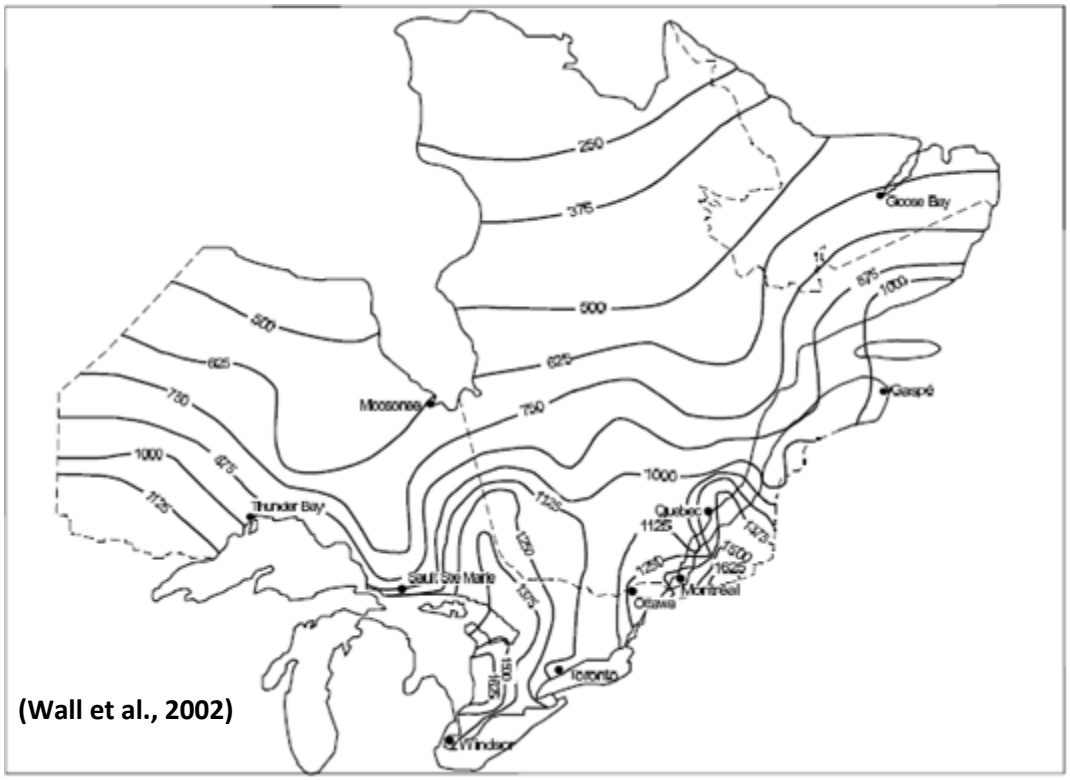

Figure 2 : Isoerodent map showing R values for Ontario and Quebec

Table 4: Monthly distribution of rainfall and runoff erosivity index (%) for selected areas in Ontario and Quebec

Region	Monthly percentage of annual precipitation											
	J	F	M	A	M	J	J	A	S	O	N	D
Southwestern Ontario	4	4	4	9	7	13	17	14	11	7	5	5
Eastern Ontario-Western Quebec	0	0	5	10	8	15	19	16	13	8	4	2
Southern Quebec	0	0	5	10	9	14	16	12	10	6	5	4
Eastern Quebec	0	0	8	11	10	14	18	16	9	8	6	0

(Wall et al., 2002)

2.8.1.2 Selection of the Erodibility Factor K

The RUSLEFAC does have a step-by-step guide for the calculation specific K value of a soil, however to keep things simple, the Soil Erodibility values K for common surface textures were added to the technical sheet. However it is highly suggested to refer to the RUSLEFAC for more accurate K estimation since the K values in table 5 were based on information obtained on 1600 samples collected in Southern Ontario by Ontario Institute of Pedology surveyors.

(Wall, Coote, Pringle, Shelton, & (Editors), 2002)

TEXTURAL CLASS	ORGANIC MATTER CONTENT		
	< 2 %	> 2 %	AVERAGE
Clay	0.032	0.028	0.029
Clay Loam	0.044	0.037	0.040
Coarse Sandy Loam	-	0.009	0.009
Fine Sand	0.012	0.008	0.011
Fine Sandy Loam	0.029	0.022	0.024
Heavy Clay	0.025	0.020	0.022
Loam	0.045	0.038	0.040
Loamy Fine Sand	0.020	0.012	0.015
Loamy Sand	0.007	0.005	0.005
Loamy Very Fine Sand	0.058	0.033	0.051
Sand	0.001	0.003	0.001
Sandy Clay Loam	-	0.026	0.026
Sandy Loam	0.018	0.016	0.017
Silt Loam	0.054	0.049	0.050
Silty Clay	0.036	0.034	0.034
Silty Clay Loam	0.046	0.040	0.042
Very Fine Sand	0.061	0.049	0.057
Very Fine Sandy Loam	0.054	0.044	0.046

Table 5 : Soil erodibility values (K) for common surface textures

2.8.1.3 Selection of the Length and Slope Factors L and S

The RUSLEFAC provides four tables for the selection of the Length Factor L and Slope Factor S. Each of those table covering one or two of the following conditions:

- Values for topographic factor LS for low ratio of rill-inter-rill erosion, such as consolidated soil conditions with cover and rangeland (applicable to thawing soils where both inter-rill and rill erosion are significant)
- Values for topographic factor LS for moderate ratio of rill-inter-rill erosion such as for row-cropped agricultural soils and other moderately consolidated conditions with little to moderate cover
- Values for topographic factor LS for high ratio of rill inter-rill erosion, such as highly disturbed soil conditions and freshly prepared construction sites with little or no cover
- Values for topographic factor LS for thawing soils where most of the erosion is caused by surface flow
- Soil loss factors for irregular slopes
- USLE values for LS for specific combinations of slope lengths and steepness

The following table 6 has been selected and included in the technical sheet as it is the most appropriate to obtain a first estimation. The table named : the RUSLEFAC " for moderate ratio of rill-inter-rill erosion for row-cropped agricultural soils with little to moderate cover" might come in useful to obtain a more in-depth calculation while evaluating soil erosion for a wet detention pond. However, that table was rejected to keep the sheet simple.

	slope length (m)											
%slope	2	5	10	15	25	50	75	100	150	200	250	300
0.2	0.046	0.055	0.063	0.069	0.076	0.088	0.095	0.101	0.109	0.116	0.121	0.125
0.5	0.055	0.066	0.076	0.083	0.092	0.105	0.114	0.121	0.131	0.139	0.145	0.151
0.8	0.065	0.078	0.090	0.098	0.108	0.124	0.135	0.143	0.155	0.164	0.172	0.178
2	0.089	0.117	0.144	0.162	0.189	0.233	0.263	0.287	0.324	0.353	0.377	0.399
3	0.127	0.167	0.205	0.232	0.270	0.333	0.376	0.410	0.463	0.504	0.539	0.570
4	0.134	0.194	0.256	0.301	0.369	0.487	0.573	0.643	0.756	0.848	0.928	0.998
5	0.137	0.217	0.306	0.375	0.484	0.685	0.839	0.969	1.187	1.370	1.532	1.678
6	0.172	0.272	0.385	0.472	0.609	0.861	1.054	1.217	1.491	1.722	1.925	2.109
8	0.254	0.401	0.568	0.695	0.898	1.270	1.555	1.795	2.199	2.539	2.839	3.110
10	0.351	0.554	0.784	0.960	1.240	1.753	2.147	2.479	3.037	3.506	3.920	4.294
12	0.462	0.731	1.033	1.265	1.633	2.310	2.829	3.267	4.001	4.620	5.165	5.658
14	0.588	0.929	1.314	1.609	2.078	2.938	3.598	4.155	5.089	5.876	6.570	7.197
16	0.727	1.149	1.626	1.991	2.570	3.635	4.452	5.140	6.296	7.270	8.128	8.903
18	0.880	1.391	1.967	2.409	3.110	4.398	5.386	6.219	7.617	8.795	9.833	10.772
20	1.045	1.652	2.336	2.861	3.694	5.223	6.397	7.387	9.047	10.447	11.680	12.795

(Wall et al., 2002)

Table 6 :USLE values for LS for specific combinations of slope lengths and steepness

2.8.1.4 Selection of Cover Factor C

Generalized Cover Factor C tables have been generated for each Canada province by the RUSLEFAC guide. For more precision, the RUSLEFAC has also generated C factors based on the region, season, crop and tillage practices.

The following table 7 explains definitions that the RUSLEFAC had used to elaborate their C factors tables and table 8 shows an example of one of these tables:

1. Tillage practices used in preparation for crop, prior to planting including primary and secondary

Seasons:	**F** – fall	S – spring	
Tillage type:	**C** – cultivate	MP – mouldboard plough	TD – tandem disc
	S – spring	CH – chisel	NT- no-till
	D – disc	OD – offset disc	H – harrow
	P – pack		

Table 7 : Variable definitions for table 8

2. Cropping practices
    a. Underseeded
        refers to whether or not a forage crop is underseeded into the main crop

b. Post-crop residue
residue treatment after harvest (left or removed)

3. Previous crop
refers to crop grown immediately prior to main crop
2nd year after hay
indicates that a hay crop was grown 2 years before current or main crop (some residual benefits of the hay still exist)

Table 8 represents only a small part of the initial and very detailed table presented in the RUSLEFAC which presents C values for over 40 different crops in different conditions:

Field Crop	Management Practices			Previous Crop	C Values				
	Tillage	Cropping			Region				
		underseeded	post-crop residue		1	2	3	4	Quebec
Beans (white)	F MP	N	L	beans, canola				0.62	0.62
	F MP	N	L	corn, grain				0.54	0.54
Canola (spring)	F MP / S C (x2-3)	N	L	beans				0.43	0.43
	F MP / S C (x2-3)	N	L	corn, grain				0.39	0.39
	- followed by no-till	N	L	field crops			0.45		
	- followed by no-till	N	L	Field crops (2nd yr. after hay)			0.41		
	- followed by no-till	N	L	hay			0.23		
	- followed by F MP	N	L	field crops			0.53		
	- followed by F MP	N	L	Field crops (2nd yr. after hay)			0.49		
	- followed by F MP	N	L	hay			0.29		
Canola (winter)	F MP	N	L	field crops		0.24			
	F MP	N	L	Field crops (2nd yr. after hay)		0.20			
	F MP	N	L	hay		0.13			

Table 8: C values for the Great Lakes/St-Lawrence region – Part I (Wall et al., 2002)

In the interest of the technical sheet, table 9 has been selected for its simplicity and generalized C values for Quebec. For C value selection of a more specific crop grown in Quebec, users have been referenced to table 8 taken from the RUSLEFAC which presents C values for the Great Lakes/St-Lawrence region.

Crop	Conventional Till	Conservation Till	No Till
Spring Grain	0.41	0.36	0.15
Fall Grain	0.27	0.22	-*
Corn (grain)	0.37	0.32	0.15
Corn (silage)	0.51	0.44	0.21
Soybeans, buckwheat, dry peas, dry beans	0.46	0.40	0.28
Hay (alfalfa)	0.02	0.02	0.02
Hay (all other)	0.004	0.004	0.004
Potatoes	0.45	0.40	-
Tobacco	0.49	0.44	-
Vegetables	0.56	0.42	-
Tree fruits	0.04	0.04	0.04
Berries, grapes	0.36	0.10	-
Nursery products	0.20	0.20	0.20

* - not applicable

Table 9: Generalized C values for Quebec (Wall et al., 2002)

2.1.8.5 Selection of Conservation Support Practice P Factor

From the tables present in the RUSLEFAC guide, table 10, which contain general P values, was selected.

Support practice	P-value
No support practice	1.00
Cross slope farming	0.75
Contour farming (3-8%slopes)	0.50
Strip cropping, cross slope (3-8%)	0.38
Strip cropping, on contour (3-8% slopes)	0.25

Table 10: General P values (Wall et al., 2002)

For more specific P values, users have been referenced to the RUSLEFAC guide as it provides the following three pertinent tables:

1. P values and topographic limits for contouring

   Provides P values based on land slope (%) and maximum slope length (m)

2. P values and topographic limits of contouring strip cropping

Provides P values based on land slope (%) and maximum slope length (m) strip width length (m)

3. P values for terracing

   Provides P values based on horizontal terrace interval (m), closed outlets, open outlets with percent grade varying from 0.1 to 0.8

2.8.2 Estimation of the sediment yield

Once applied the RUSLEFAC equation gives an estimation of the amount of soil loss from a field, this give a value representing the gross erosion that occurred over a certain period of time (Yearly or Monthly). However the RUSLE model assumes no deposition. Not all the soil that has been eroded from the field will end up in the water stream or in this case, in the wet detention pond at the end of the field. Part of the sediments being transported will be deposited. For example, eroded soil can be trapped by vegetation or any obstacles found between the field and the final point of deposition. Therefore sediment yield is only a fraction of the gross erosion estimated by the RUSLEFAC equation.

To quantify the sediment yield from the RUSLEFAC gross erosion, a sediment delivery ratio relationship is needed. By consulting Design Hydrology and Sedimentology for Small Catchments textbook, two graphical methods for predicting delivery ratios were suggested:

- Area-Delivery Ratio Relationship (Boyce, 1975)
- Forest Service Sediment Delivery Index Model (1980)

Both these delivery ratio curves use erosion estimates that were based on the simple USLE, therefore they can be use in conjunction with gross erosion estimated from the USLE/RUSLE or in our case RUSLEFAC.

The Area-Delivery Ratio Relationship illustrated in Figure 3 is said to have limited applicability due the large variability of the data used in its elaboration therefore it can only provide a very rough estimation of the sediment yield. It is also stated that this method is not recommended when predicting sediment yield of single storm. Since the goal is to predict sedimentation efficiency of wet detention pond for different precipitations scenario, this method was not retained for the technical sheet.

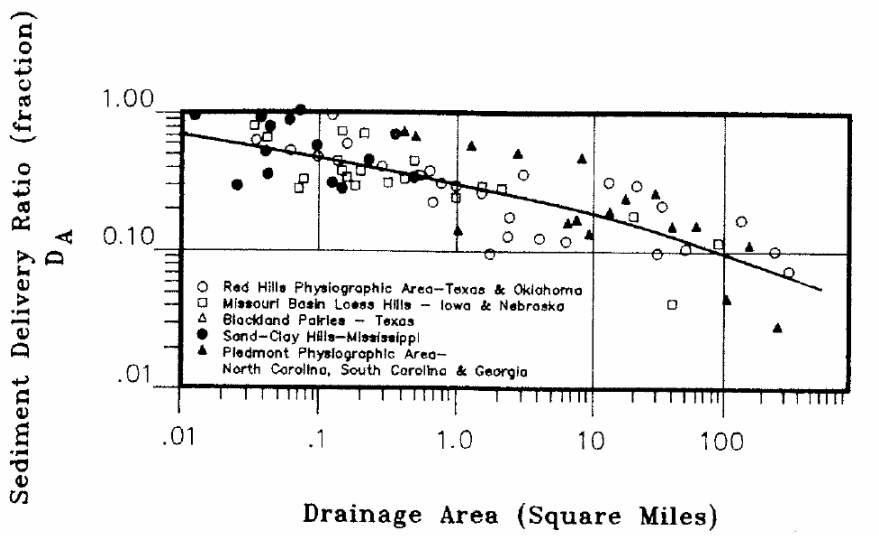

Figure 3: Sediment delivery ration versus drainage area size for use with USLE/RUSLE (after Boyle, 1975) (Haan et al., 1994)

On the other hand, the Forest Service is a method that can predict sediment delivery ratio for a single storm and canbe used only in conjunction with the gross erosion estimated from USLE/RUSLE. This method is more accurate and specific to the field studied as it predicts the sediment yield by taking into account the following parameters:

Delivery distance from the slope to stream	Flow distance in feet between the source area and channel
Slope shape	Refers to slope shape between the source area and channel. This factor that accounts for the impact of concave or convex surfaces on sediment delivery. A factor of zero represents a convex shape and four a concave shape.
Percentage of ground cover	Percentage ground cover in the flow path between the source area and the stream. Ground water in this case is defined as cover such as litter that is in contact with the surface. Zero indicates no cover and 100 mean complete cover.
Texture of eroded material	Parameter that will define the impact of particle size on delivery. The parameter is the percentage finer than 0.05 mm (silt size and finer). A value of 100% would mean that all particles are silt size and smaller.
Surface runoff	Quantity of water available to transport sediments from a storm given in a peak discharge in cfs/ft. If peak discharge information is unobtainable, the runoff factor can be estimated by the following equation :

	$$F = 2.31 \times 10^{-5} \sigma L$$ With F = runoff rate per foot of slope width (cfs/ft) σ = rainfall excess (in/hr) L = length of the disturbed area (ft) ; values greater than 0.1 are assumed to be 0.1
Slope gradient	average slope between the source area and the receiving channel
Surface roughness	it is a subjective value to be chosen by the user. A value of 0 indicates a smooth surface and 4 represents a very rough surface

Table 11 : Influencing Factor of the Forest Service method

Those parameters should be estimated and inputted into the Stiff diagram illustrated in figure 4.Each parameter is plotted on its assigned axis. Then all the points are connected to form a polygon as illustrated in Figure 8.29. The ratio of the area within the polygon on the total area of the rectangle is used in Fig 8.28 to predict the sediment delivery ratio.

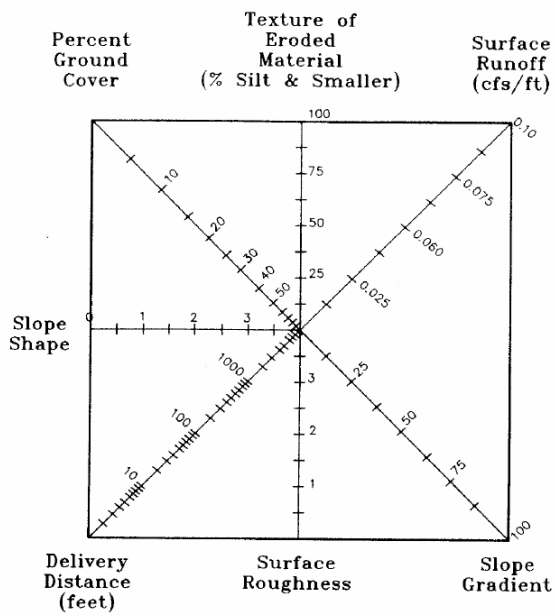

Figure 4 : Stiff diagram for estimating sediment delivery with the USLE/RUSLE (after Forest Service, 1980) (Haan et al., 1994)

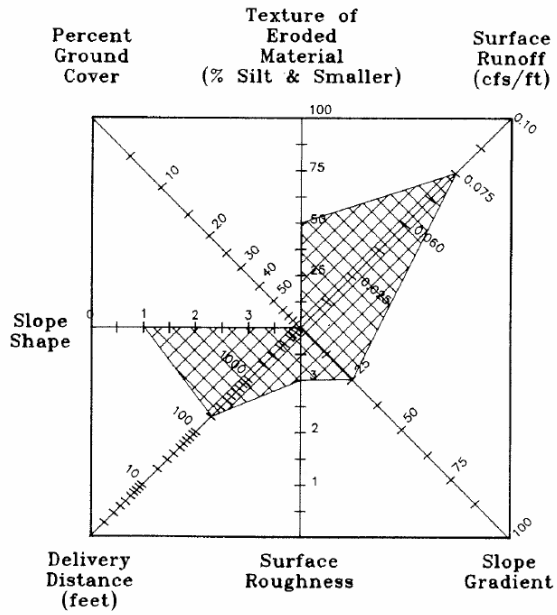

Figure 5 : Application of Forest Service Stiff diagram (Haan et al., 1994)

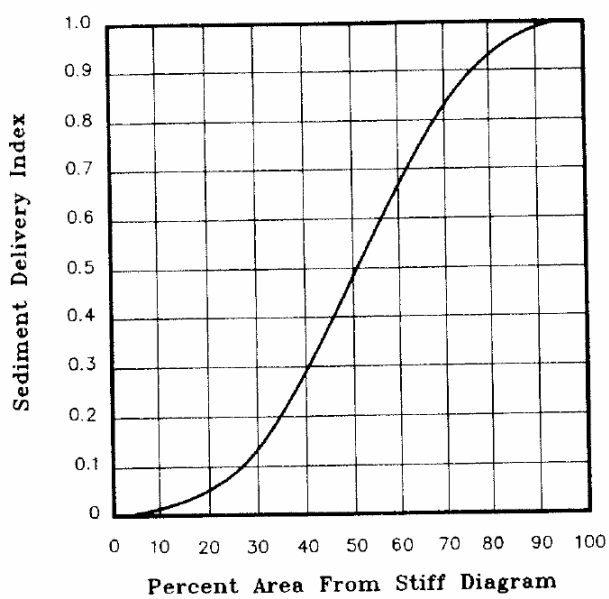

Figure 6 : Diagram to convert percentage of area in Fig. 4 to sediment delivery ratio (after Forest Service, 1980)

Once the delivery ratio has been obtained from figure 6, it can be used in the following equation to estimate the sediment yield:

$$Y = D \times (gross\ erosion)$$

With

Y = sediment yield from a watershed expressed in tons
Gross erosion = erosion per unit area occurring on the watershed (obtained from USLE/RUSLE) expressed in tons/acre

Once the sediment yield in mass unit has been obtained, the sediments concentration in the runoff can be estimated.

The runoff volume is:

$$Q = runoff\ depth\ \times area$$

With

Q = runoff volume in ft^3
Runoff depth is expressed in ft
Area is expressed in ft^2

Sediment concentration is the mass of sediment divided by the mass of water:

$$C = \frac{Y\ \times (2000\ lb/ton)}{(62.4\ lb/ft^3)\ \times Q}$$

The sediment concentration obtained will have units of lb/lb and can be converted to mg/L if needed.

2.8.3 Sediment trapping efficiency

Once the amount of sediment getting in the pond has been calculated, it is possible to estimate the sedimentation that can occur during a rainfall event.

The Assumptions that were taken in order to simplify the process were:

- Quiescent flow
- No resuspension of sediment
- Rectangular shaped reservoir
- Steady flow

First the overflow rate needs to be calculated and is given by:

$$V_c = \frac{Q}{A}$$

With
V_c = critical settling velocity in ft/sec
Q = inflow rate in ft^3/sec
A = surface area of the rectangular pond in ft^2

Assuming a temperature of 20°C, the corresponding equivalent diameter sphere corresponding to V_c can be derived from Stokes equation:

$$d_c = \left(\frac{V_c}{2.81}\right)^{1/2}$$

The trapping efficiency can then be obtained with:

$$E = (1 - X_c) + \sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$$

With
E = total trapping efficiency of a basin in percentage %
X_c = fraction of particles with settling velocity less than Vc ; can be found on a Size distributions chart as shown on figure 7. Usually the owner of an agricultural field should be able to provide size distribution charts and other pertinent information specific to his field soil.
V_s/V_c= fraction trapped for a particle given size
ΔX = differential element of fraction of particles finer than given size

The summation term $\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$ represents the contribution of each given particle size that is smaller than the size associated with critical velocity V_c and is obtained using the following excel spreadsheet model:

Particle size range (mm)	Diameter d (mm)	ΔX	V_{si} (ft/sec)	$(V_{si}/V_c)\Delta X_i$	$(1 - V_{si}/V_c)\Delta X_i$
0.0001-0.00095	0.00075		1.58×10^{-6}		
0.00095-0.0016	0.0014		5.51×10^{-6}		
0.0016-0.0025	0.0020		1.24×10^{-5}		
0.0025-0.0036	0.0030		2.53×10^{-5}		
0.0036-0.0045	0.0042		4.96×10^{-5}		
0.0045-0.0064	0.0056		8.81×10^{-5}		
				$\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$	$\sum_{i=1}^{n} 1 - \frac{V_{si}}{V_c} \Delta X_i$

Table 12 : Tabulations to obtain term $\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$

The particle size range is a list of all the size smaller than critical equivalent diameter sphere d_c. For each of these particle ranges, the diameter size d is an average over the range. ΔX (differential element of fraction of particles finer than given size) is obtained from a size distribution chart such as in figure 7 using the diameter d values for each particle size.

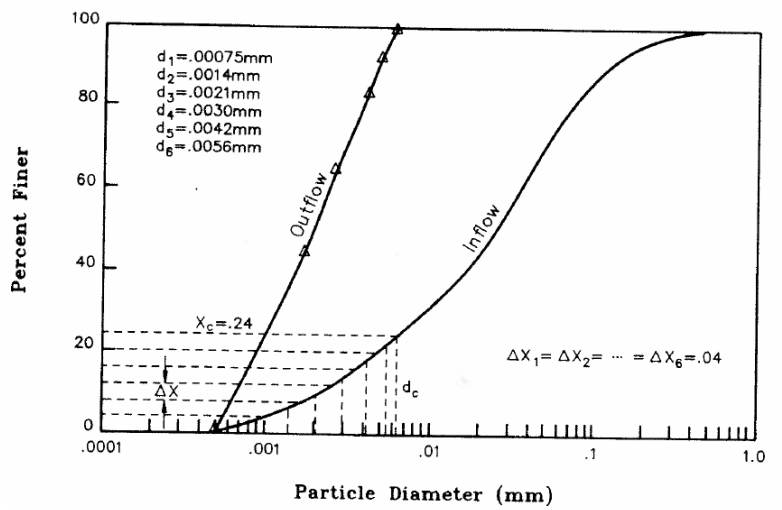

Figure 7: Example of size distribution chart (Haan et al., 1994)

The V_{si} column is provided by $V_{si}=d^2$

The two last columns are simply obtained with $(V_{si}/V_c)\Delta X_i$ and $(1 - V_{si}/V_c)\Delta X_i$ for each particle size range.

This lead to the term $\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$ by summation of column 5.

However in the technical sheet, it was provided as the shortened version of the equation:

$$E = (1 - X_c)$$

In the technical sheet, the term $\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$ is removed from the equation since it is laborious to calculate to obtain a first sedimentation estimate calculation .The purpose was, once again, to obtain a good estimation while keeping the calculations simple to the user. That second term contribution is fairly low in comparison to the first term $(1 - X_c)$ which is why the second term $\sum_{i=1}^{n} \frac{V_{si}}{V_c} \Delta X_i$ was removed. While the effect of this term should not be neglected in precise calculation, it was felt that it was not required to provide a first estimate of the sedimentation potential.

The detailed calculations for obtaining the sediment trapping efficiency for a steady-state overflow rate model for turbulent flow were not included in the technical sheet either. This process was judged too rigorous and detailed for the purpose of the technical sheet. The users have been referred to book Design Hydrology and Sedimentology for Small Catchments to obtain the procedure for more in depth analysis of the sediment trapping efficiency.

By calculating the sediment trapping efficiency for quiescent flow, the user has already obtained the maximum value for efficiency that the pond will be able to provide, which provide an acceptable estimation. . In reality the sediment trapping efficiency with quiescent flow will be very close to the sediment trapping efficiency with turbulent flow until it reaches a certain ratio of setting velocity V_s/V_c equal to 1.0. As shown on figure 8, from that point on, the difference between quiescent settling and fully turbulent flow becomes important. By using this graph, the user will know right away to expect a difference of 10-15% between the quiescent flow sediment trapping efficiency and the turbulent flow sediment trapping efficiency which will allow him to account for factor.

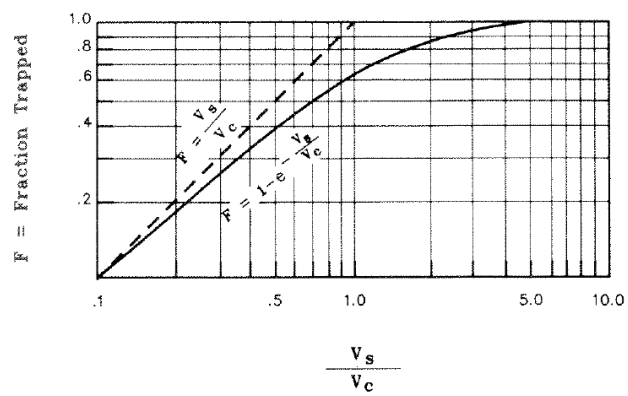

Figure 8 : Trap efficiency versus ratio of setting velocity to overflow rate for a high-turbulence model (Chen, 1975) (Haan et al., 1994)

2.9 Future work

The focus in the elaboration of this technical sheet has mainly been the pond sizing and the sedimentation estimation since they are the critical components in the efficiency of a wet detention pond. Both of these sections have been researched in the literature compared to the other components since they are involved in the two important purposes in the installation of a wet detention pond which are peak runoff reduction and sedimentation trapping. Therefore the information provided in the technical sheet on these two aspects is able to assist the user in estimating the pond sizing and sedimentation rate. Some aspects could be added to the technical sheet to make it more complete and they are the vegetation component and regulations involved in wet detention pond installation.

Since most of the procedure for the estimation of sedimentation has been borrowed from Design Hydrology and Sedimentology for Small Catchment 1994, there is a concern about copyright issues and those would need to be address that issue if the technical sheet was to be published. Is it allowable to use that much material from a reference book and make it publically available ?

3-Simulation sheet:

The second goal of this project was to create a simulation tool that would allow a user to test different designs of detention pond. As exposed in the mid-project report, the program Excel^{tm} was chosen because of prior knowledge and accessibility. The goal of the simulator is to allow its user to test a pond design (size and spillway design) in function of different field conditions (runoff, precipitations and field characteristics). The program should only be used as a tool to test an existing design and identify possible problems. Since the simulation sheet functions based onmany different assumptions, the design itself should be made using different design criteria such as the one explained in the technical sheet. It was assumed that the user would have a minimal background in hydrogeology, which is knowledge of the basic calculation formula for peak flow, time to peak and the different field conditions.

This section will start with a brief overview on how the program should be used and what result should be expected. The following section will present a tab by tab overview of the program explaining every step that was taken. The last section will conclude with the possible modifications and improvements that could be done.

3.1-User Guide

As many other simulation tools, the program works under a fairly simple principle: the user inputs the wanted condition and the program will do the required calculations and give the results. Results evaluations are not given, as they should be done by a professional using a critical eye. Here is a step by step method on how the program should be used:

1. Under the “field” tab the user will have to input the maximum length of the field (meters), the area of the field (hectares), the average slope (% m/m), the average runoff coefficient of the field and the Cn of the soil. A table of the different Cn values for different soil types can be found in the “Cn table” tab. Once the values are inputted, the program calculates the time of concentration and time to peak of the field. The user should make sure that those two values make sense before continuing.

2. Under the “Precipitation input” tab, the user will be asked to input the type of to be simulated. Using the first scrolling bar the user has to select between the 3 types of input possible and enter the duration of the precipitation. (Note : In the case of hourly and 5-minute input, the duration of precipitation will only affect the calculation of the average intensity, it has no real effect on the simulation)

    a. If the “IDF curve” option is selected, the user will have to input the different IDF Intensity values and the duration of the storm it want to simulate. The last step will be

to select the desired intensity using the scrolling bar.(Note : The maximum duration is 24 hours)

b. If the “Input data Hourly “ option is selected, the user will simply have to enter the different precipitation value in the corresponding section, the program allow for a maximum of 24 different values to be entered using this function
c. If the “Input data (5 min)” is selected the user will have to provide precipitation values for every 5 min interval of a precipitation that can last up to 24 hours. The information has to be inputted in the “input data (5 min) “section.

3. Under the “Pond Design” tab the user has to input the different shape parameter of the pond as shown in the sketch shown on the tab. It is important to note that the program only allows for single slope pond. The “Initial water height” represents the height of water present in the ponds at the start of the precipitation event (meter). The “test” section of the tab can be used to see the area calculated by the program at any height. This should be used to determine if the assumption made by the program can fit with the field.

4. The last input tab is the “Water outlet” tab. On this tab the user has to enter information related to the water control structure. The different height values of every plank of a flashboard riser have to be inputted following the sketches that are present on the tab. To simulate a wet detention pond simply set H1 as 0 and H2 as the height of your permanent water depth and input your values starting from H2. Finally the length of the flashboard riser should be entered (m)
    a. On the bottom of the page the different values for the interference factor can be found. The values found while first using the simulation tools are theoretical values that were obtained under heavy assumption, those values should be checked before implementing a design based on those calculation

5. While the information are inputted, the program does all the necessarily calculation in real time, once the water outlet information are inputted, the user can then see the simulation in the “result” tab.
    a. The first graph seen is the inflow the ponds and outflow of the pond as a function of time. The hourly precipitation is also shown on an inverse axis.
        i. Any sharp jump in the graph represents an overflow event, those overflow values are only given as estimation. No design should be based upon those values before prior calculation
    b. The second graph shown is the height of the water in the pond as a function of time
    c. Some information are also available on the side such as Maximal flow in, Maximal flow out, Peak flow reduction and maximal height of the water during the precipitation. There is also the option to check the amount of time passed above a certain height (hb), simply enter the values for hb and the result will be calculated.

i. Note: it is recommended to allow a certain margin of error for hb if it is used to calculate amount of time for an overflow. For example if the pond is designed for a maximum height of 4 m, it is suggested to set hb to 3.95 to have an accurate estimation of the overflow time.

6. The “Precipitation Output” and “Inflow” tab can also provide some additional information and graph on the precipitation and the inflow respectively.

3.2-Simulation sheet Explanation

This section will provide a step by step explanation of all calculations that are made by the simulation sheet. The description will start with the different input tabs and will introduce the different calculation tabs has they are used by the program. This section will also explain the different challenges encountered while programming those tabs. While reading this section, it should be assumed that every tab follow the different command and formula that were described in the mid-project report and in the flowchart found in Annex 4, all changes that were made to formula are described in the corresponding section.

3.2.1-Field

The first tab of the simulation sheet, the “Field” tab is pretty simple, it is mainly used to obtain the information required to calculate the time of concentration and the time to peak of the field. An average runoff coefficient for the field is also asked. While it is known that the runoff coefficient for any field is not constant during any precipitation event as a dry soil is more permeable than a wet one, an average value has to be taken to keep the simulation sheet relatively simple.

The method used to calculate the time of concentration is the Mockus method and the formula used to obtain the time of concentration is $T_p*0.6 = T_c$. It should be noted that the Mockus method is mostly accurate on low slope field (Slope < 1 %) that range from 4 to 1000 ha. (Mario & Jacques, 1989)

3.2.2-Precipitation Input

The second input tab that should be approached by the user is the “Precipitation Input” tab. The first important aspect of this tab is the “Type of precipitation” scrolling menu, which control the input precipitation that will be taken into consideration for the calculation. The scrolling bar has no direct effect on this tab but affect the selection of the “Precipitation output” tab (See next section).

The second scrolling menu is located in the IDF curves section; it serves as a quick way to change between different return periods. The goal of this scrolling menu is to allow the user to change quickly between different return periods and durations once the IDF curves numbers have been entered. The duration input was kept to give the possibility to simulate non-standard precipitations, for example a storm with 2 year and 6 hour intensity lasting for 7 hour.

The second part of the IDF box is the IDF intensity table. This section should be filled using theIDF curve obtained from a meteorological station located close to the design area. Those IDF curves can be obtained for free on the Environmental Canada website and they are available for most of the meteorological stations located across Canada. By default the values found in this table are the intensity values for the station of Ste-Anne-de-Bellevue, Montreal.

Once a duration/frequency option is selected and the intensity has been inputted, an hourly precipitation event is simulated. The simulation simply takes in account a constant intensity for the amount of time selected. However a constant intensity precipitation is nearly impossible in nature so the result obtained from this option should take this into account. An option for improvement would be to obtain the standard distribution for a type II precipitation event, once the distribution is obtained it would simply require to apply the distribution to the formula present on the sheet.

The two other precipitation options follow a similar input pattern. If one of those options has been selected, the user simply has to enter the precipitation value (in mm) in the corresponding section. Hourly precipitation data are available for most Canadian weather stations, so this option should be reserved to test detention design using past precipitations. For example a user could use the precipitation from the previous year storm to see how the pond would have handled it. While an precipitation interval of 5 minutes might seem excessive, this option was added as to the simulation since it was easy to program.

3.2.3-Precipitation Output

Once the precipitation and field data have been entered it is possible to create the simulated precipitation and start the calculation of the inflow. The precipitation output tabs take the information from the Precipitation Input tab to create a 5 min interval simulated precipitation. If the 5 min interval precipitation was selected in the Precipitation Input tab, it simply import the value entered. However, if hourly data were inputted or IDF was selected, the program will convert the hourly intensity to 5 min precipitation by dividing it by 12.

$I_{hourly}/12 = Precipitation_{5min}$

Once the depth of precipitation for every 5 min interval has been obtained, the program can calculate the maximum flow due to that precipitation. As explained in the mid-project report, the initial plan to calculate the maximal inflow, was to use the rational method derived from Mockus equation.

However this method was developed to estimate the maximum flow of an entire event and not an "instantaneous" precipitation and it ended up overestimating the real flow greatly. The modified SCS hydrograph method was also tried and similar results were obtained .

Because of this, a method to calculate the peak flow was obtained by deriving the Dimensionless hydrograph curve and making it equal to the volume of flow (C*I*A). The method used is outlined in Annex 5.

This tab also presents a hydrograph of the hourly precipitation as well as some information that could be useful to the user: duration of the precipitation event, total precipitation, maximal intensity (hourly), maximal intensity (5 min interval) and average intensity. It is important to note that the average intensity is calculated with the duration input in the field tab, if the hourly of 5 minute interval precipitation is used, this value has to be input manually by the user to obtain a good calculation of the average intensity.

3.2.4-Qin Calculation Part 1 and 2

Once the maximum flow for each 5 minute instantaneous precipitation has been calculated, it is now possible to create a flow curve for each of those precipitations. First, the maximum flow calculated in the Precipitation Output tab has to be transposed to a separate calculation sheet. Because of size restriction, the first 12 hour were transposed on the first part (Qin part 1) and the last 12 hours on the second part (Qin part 2). Once this is done, following the SCS dimensionless hydrograph theory and the formula provided by: (Charles T., Billy J., & J. C., 1994) it is possible to calculate the flow in function of the time.

$$Q(t) = Qp * (\frac{t}{Tp} * e^{\left(1-\frac{t}{Tp}\right)})\hat{}k$$

Where :

Q(t) = flow in function of time

Q_p = Peak Flow

T_p = Time to peak

K = Field shape factor

Because the dimensionless hydrograph method assumes that nearly all precipitation have evacuated through as runoff after an amount of time equal to four times the time to peak, it was assumed that a duration of 48 hours after the last rainfall would be sufficient for the simulation. This

allow the simulation to be accurate for field that has a time to peak of less than 12 hours, which is the equivalent of a relatively flat field with a maximal length of 3500 meters (Slope of 0.035 %).

The program calculate Q values for every 5 minute interval after an “instantaneous” precipitation. Once the calculation are done, the flows are added for every 5 minute interval which create of total inflow curve that can be found in the inflow tab.

3.2.5-Inflow

The Inflow tab does the summation of the different flows calculated in the Qin calculation Tab; the first 12 hours are taken only from the first part, while the other 60 hours are calculated by adding the value obtained in part 1 and part 2. Once this is done, a graph of the flow in function of the time is displayed.

The results obtained by the calculation sheet were tested using the Modified SCS Desforges and Hoang modified methods. The program was asked to calculate the total inflow for different intensities and durations. The results obtained were compared to the value obtained by the Modified SCS method.

Intensity (mm/hr)	Duration (hours)	Simulator (m^3/s)	Modified SCS Method (m^3/s)
1.666	6	0.02126672	0.0220583
3.33	6	0.04255004	0.0436929
7.5	6	0.09573855	0.09930206
3.33	12	0.06454271	0.0436929
7.5	12	0.14668798	0.09930206

Table 13 : Evaluation of the accuracy of the simulator

(Those results were obtained using the following parameter: Length of the field 1300m, Cn values 80, slope of the field 0.035 % and area 20 hectares and a runoff coefficient of 0.4 and a K shape factor of 2.23)

Those results are interesting as they are somewhat close to each other but were obtained using two completely different calculation techniques. It should be noted that the modified SCS method does not take into account the length of the precipitation and has been known to under-evaluate to peak flow of long event.(Mario & Jacques, 1989). In this example, the time to peak of the field is close to 6 hours. This mean that the estimation for 6 hour precipitations are the most accurate for the SCS method.

In case that the maximal flow simulated does not seem to match the calculated or observed maximal flow, it is possible that it is due to the shape of the field. To obtain a better accuracy the user could modify the K factor found on this page. The K factor represents a shape factor for the field, it influences the shape of the dimensionless hydrograph curve without changing the maximal flow or the total volume for a 5 minute precipitation curve. However, since it changes the shape of the curve it has an influence on the total maximal flow; see Annex 6 for the influence of the K factor on a standard precipitation.

In general the K values of a field should be between 2 and 4. For the best results, it is suggested that the user run a constant precipitation event, such as the one created by the IDF option for a duration equal to the time to peak of the field, rounded to the closest integer. The program will provide an average K values obtained from two different peak flow estimation method. After testing,, it seems that it is the most reliable method to obtain a good K estimation, however more tests should be done on this aspect of the sheet.

While it is important that the simulation program should obtain flow values close to the standard peak flow calculation method, it is also important to remember the limitation of those methods. For example the Modified SCS and the triangular hydrograph method work under the assumption that the peak precipitation occur for a duration equal to the time to concentration which mean that in the case of burst precipitation (Example : 2,3,22,3,4 hourly mm), those methods will tend to greatly overestimate the maximal flow for field with a long time to concentration. Those formulas should not be used as standard in those cases. This also means that there are few options available to test the simulation program under those conditions short of using field data that has already been collected or other simulation program. Because of time constraints and the lack of knowledge and access to such programs, it was decided that the program would be tested with hydrological data from the St-Samuel detention pond test site that were provided by Agriculture Canada. In the example below, the precipitation and field data were entered and two inflow simulations were created for the field, one using the hourly precipitation and the second using 5 minute interval precipitation. As seen in annex 7, by using a standard K value of 3 (the default value of the program), both simulation seemed to achieve accurate results.

Type of data	Peak Flow (m^3/s)
Field Data	0.06124
Hourly simulation	0.06034556
5 minute interval simulation	0.06440211

Table 14: Simulation using Field data

As seen in Table 14 the simulated peak flows are both inside a 10 % range of the collected field data which seem promising. However, more data would be required to validate the accuracy of the simulation sheet. While a good accuracy was achieved using the data of the St-Samuel field, it would be important to test those results with other available flow data and proved simulation programs.

3.2.6-Pond Design:

Once the inflow in the pond is calculated, it is now required to calculate the outflow; the first step is to input the dimension of the basin. For simplicity, the program only allows the user to input dimension for one type of basin, rectangular basin with one constant side slope. This choice was made becauserectangular basin seem to be the most common, in the literature. Also it should be possible to

obtain a decent approximation for a circular shaped pond as explained in Annex 8. However it should be noted that circular basin should not be used since they reduce the sedimentation efficiency.

Once the different slopes and length of the ponds are entered, it is possible to calculate the surface area of the water as it was exposed in the mid-project report or in Annex 4. The calculation made to obtain the areawhile within the limit of the pond is simple. However, when the pond overflows, the area will be heavily dependent on the field and the location of the pond. The simulation work under the assumption that water will only be able to spread from two sides of the pond and that this water rise will follow the slope of the field, this give the following formula for overflowing water :

$$A(h) = \left(\frac{h - Hpond}{\frac{S}{100}} \right) * (S1 + S2) + \ A(Hpond)$$

Where:

h = Height of the water
Hpond = Maximum height of the pond (m)
S = Slope of the field (%)
S1,S2 = Length of the side of the pond (m)
A(Hpond)= Surface Area of the water when the ponds is at the maximum height (m^2)

This assumption was chosen as it seemed to be the safest, in many cases, it is possible that it will overestimate the rise of the water since the water will be able the spread from more than 2 sides of the pond. Also it is possible the area close to the pond will not follow the general slope of the field, this should be taken into consideration when looking at the results provided by the simulation sheet.

3.2.7-Water Outlet

The final input tab adresses the water outlet dimension. Following the sketches found on the tab, the user has to input the different height values for the water outlet. It is possible to create a wet detention pond by inputting a value of 0 for h1 and the starting height for h2. This choice of input form was made to allow the user to be able to simulate dry and wet retention and detention ponds while keeping the program relatively simple. It is important to note that the program needs a value for each of the ten input cells to make correct calculations. In the case of an outlet with less release space that the simulator ask for, the user will have to enter the same value for the start and the end of the release area. For example a wet detention basin with only three release spaces could have H8=H9 to allow the simulator to proceed.

The different outflow calculations also use different K values to represent the flow reduction from weir flow, the shape factor and the interference due to multiple flow paths. Using fluid mechanics reference, it is possible to estimate the K_{weir} and the K_{shape} with a good confidence has this shape and the assumed material, which is wood, has been researched on many different occasion. However because of

the lack of time and resource it was not possible to test the different values for the $K_{interference}$, following discussion with Dr. Raghavan, the best approximation that could be made was that the interference would vary between 0 to 50% flow reductions . The default values were entered following this assumption, which mean that the default formula present on this sheet should not be used to design the spacing of a water outlet. However, this can be used to obtain a general idea of the behavior of the water.

Once all the height values for the water outlet are entered, the simulator uses the formula that were outlined in the mid-project report (See annex 4) to calculate the outflow at any height. The first and last parts of the water outlet are assumed to follow the rule for weir flow while the other alternate between weir and pressured flow. Since there are 11 different formulas that can be used to calculate the outflow, it makes the sheet a bit complicated. Since the IF command only allows for a maximum of 8 conditions, a LOOKUP function had to be used. This means that each of the 11 calculation cells have to be present in order to allow the calculation of the outflow at any height. An example of the calculation required to produce a graph of the outflow in function of the height can also be found on this page.

3.2.8-Flow calculation

Once all the information about the basin and the water outlet have been entered, it is now possible to proceed to the calculation that will give the height and the outflow in function of the time. Following the procedure exposed in the mid-project report (see annex 4), the simulation will proceed by iteration to find a correct approximation for the Height at any time. The initial assumption was that 3 iterations would be sufficient to obtain a good accuracy. However this was not the case, since the results kept varying greatly after 4 or 5 iterations. Since the calculation tend to underestimate the outflow for every odd iteration and overestimate it for every even iteration, it was decided to increase the number of iterations to five and to take the average outflow of the fourth and the fifth iteration to obtain the final height values.

The difference between the average outflow and the outflow obtained from the final height value was plotted for a typical 6 hour IDF precipitation. Since the graph shows that the difference between the two values is minimal and it was judged acceptable.

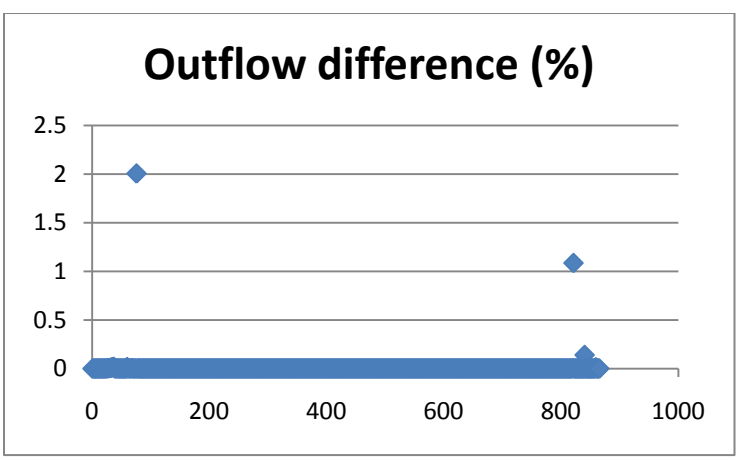

Figure 9 : Outflow Difference

3.2.9-Results

Once all the flow calculation has been done and the inflow and outflow for every 5 minute interval has been calculated, it is possible to proceed to the final result tab. This tab simply shows most of the data that has been calculated in the previous tab in a user friendly format. Information such as the maximum inflow and outflow and water height are displayed for the user convenience. Also, the last cell of the table can be used to calculate the time the water pass above a certain level of water. This will mostly be used to calculate the overflow time occurring in the pond during a certain event.

There are also two graphs that can be found on this tab: the first one shows the variation of the water height in function of time, while the second one shows the inflow and outflow in function of the time. Addition of the precipitation as an inverse bar graph to the inflow and outflow graph was attempted, however excel seemed to pose problem with this and the idea could not applied due to time constraint.

Using this tab the user should be able to determine if the design input is appropriate for this type of field and precipitation. By looking at the maximum outflow and water height, the safety of the design can be judged by identifying if any critical height has been reached or if the outflow goes above what is allowed by the culvert design.

Also the effectiveness of the design can be evaluated by looking at the total peak flow reduction, this should provide a good estimate of the effectiveness but the user should also combine those information to sedimentation calculations to obtain a better estimate of the efficiency of the pond.

When using the different values found on this tab, it is important to keep in mind the different assumptions that were made during the process.Those simulated numbers are onlyprovided for estimation purposes and should not be taken as fact during the conception of a basin.

3.3-Future Work:

In conclusion, the simulation program seems to satisfy the different objectives that were set at the start of the project: it can simulate the inflow from any precipitation, simulate the outflow in the pond and calculate the peak flow reduction and the height variation in function of the time. While the tests made using the simulation sheet were all within an acceptable range of the theoretical or measured values, more tests would be required to confirm the accuracy of the simulator. It would be important to determine the circumstances the simulation sheet will provide accurate data. For example, while the sheet had been designed for small watershed use, would it be possible to use it for larger area?

For those tests, real field data for different types and shapes of field would be required; also it would be interesting to test the program under different conditions such as urban and forested area. To execute this test, inflow field data would be required, the best way to obtain those would be to look at the previous hydrological studies that were conducted within the department. This should allow enough data to test the accuracy of the inflow simulation. If the access to such data is impossible, commercial precipitation and flow simulator could be used, such as SWMM^{tm}.

Another part of the simulation sheet that require more testing is the fluid mechanic equation behind the outflow calculation, as explained in the water outlet section, the interference factors used have simply been assumed. This mean that the translation between the height values entered and the outflow curve is most likely inaccurate. The only way to solve this problem would be to replicate a flashboard riser under monitored conditions and determine the different interference coefficients by creating an outflow curve based on the lab result. While the outflow curve given by the program might not correspond exactly to the real outflow curve of a riser with the inputheight, it can be assumed that a similar curve would be achievable by changing some of the height values. This means that the outflow simulation can still be considered accurate for any water outlet that would follow the outflow curve given in the water outlet section.

Besides testing, there are also some improvements that could be made to the simulation program. Following recommendations made by Dr. Prasher, a fourth type of precipitation input could add. An SCS type II precipitation input could use a standard Type II precipitation distribution with which, it could create a standard storm by simply asking the user to input a total rainfall and storm duration. However, this option would require some complex programming if it was to allow a storm duration ranging from 1 to 24 hours.

Another possible addition would be the automatic calculation of the possible sedimentation that would occur during the simulated precipitation, however due to the complexity of sedimentation formula and the type of input that would be required (ie. Soil curve), it would be extremely complex to create this in Excel^{tm}.

One of the major limitations to the improvement of the various aspects of the simulation was the Excel^{tm} program itself. While Excel^{tm} suited perfectly the requirements to create a basic simulation program, it does not allow the programming of options that would be required to achieve a commercial grade product; such as save and load feature, being an executable etc…. While Excel^{tm} simplicity and availability corresponded to the need of this project, a real programming language would be required to bring it to the next level.

Closing Statement:

This concludes the overview the work that has been done for our design project. While it might not be ready for publication and there is still work to be done, we believe that the goals that were set have been achieved.

First, a technical sheet has been produced which contain simplified information and technique on the design of a detention pond. This sheet contains information from different sources as well as equations and guidelines that were developed specifically for this project. The sheet might not be of professional grade yet, but we feel that the largest part of the work has been done.

Secondly, a simulation program was created based on different hydrologic concepts found across the literature, it was found to be accurate under the conditions on which it was tested. While the calculations behind the simulator might be complicated the program itself should be simple enough so that a person with little to no knowledge in hydrology might be able to use it.

We hope you appreciated our work

Sincerely

Jonathan Martel-Gagnon

Minh-Vy Le

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Annex

Annex -1 : Volume calculation Curve (Matlab)

```
syms x k

k = (0.2)^(1/3.77)
f = @(x) x*exp(1-x)-k
A2=fzero(f,1.4);

k = (0.3)^(1/3.77)
f = @(x) x*exp(1-x)-k
A3=fzero(f,1.4) ;

k = (0.4)^(1/3.77)
f = @(x) x*exp(1-x)-k
A4=fzero(f,1.4) ;

k = (0.5)^(1/3.77)
f = @(x) x*exp(1-x)-k
A5=fzero(f,1.4) ;

k = (0.6)^(1/3.77)
f = @(x) x*exp(1-x)-k
A6=fzero(f,1.4) ;

A2
A3
A4
A5
A6
```

Answer

A2 = 2.2280

A3 = 2.0249

A4 = 1.8678

A5 = 1.7347

A6 = 1.6146

Where A = t/Tp

```
A2 =  2.2280
A3 =    2.0249
A4 =    1.8678
A5 =    1.7347
A6 =    1.6146
syms x
```

```
Q = (exp(3.77*(1-x))*x^3.77)
R = 0.2
V1 =int(Q,0,A2)
B = (exp(3.77*(1-A2))*A2^3.77)*R
Qo = B*x/A2
V2 =int(B,0,A2)
Vf2= V1 -V2

R = 0.3
V1 =int(Q,0,A2);
V2 =R *int(Q,0,1);
V3 =(A2-1)*(1-R);
Vf3= V1 -V2-V3

R = 0.4
V1 =int(Q,0,A2);
V2 =R *int(Q,0,1);
V3 =(A2-1)*(1-R);
Vf4= V1 -V2-V3

R = 0.5
V1 =int(Q,0,A2);
V2 =R *int(Q,0,1);
V3 =(A2-1)*(1-R);
Vf5= V1 -V2-V3

R = 0.6
V1 =int(Q,0,A2);
V2 =R *int(Q,0,1);
V3 =(A2-1)*(1-R);
Vf6= V1 -V2-V3
```

R = 0.2000

Vf2 = 0.1538

R = 0.3000

Vf3 = 0.2281

R = 0.4000

R = 0.5000

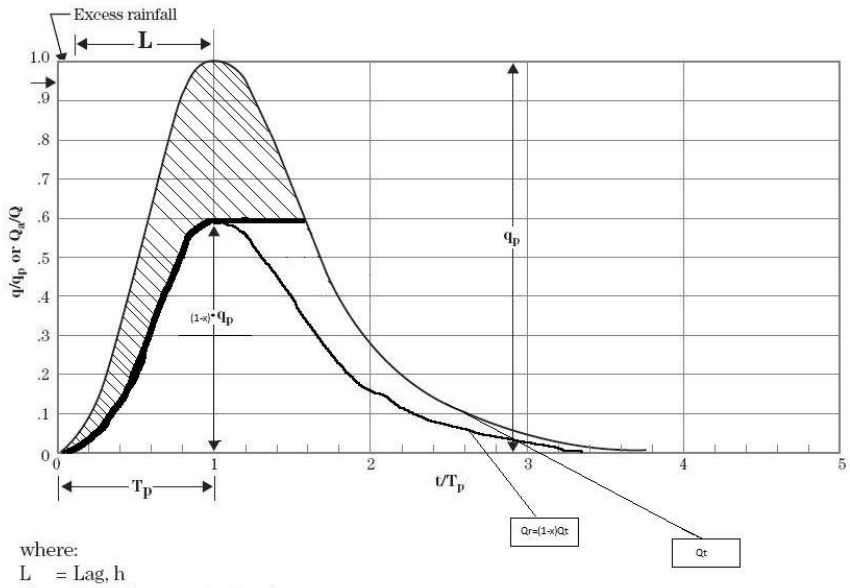
Vf5 = 0.3767

R = 0.6000

Vf6 = 0.4510

Vft= 1.32

Annex-2 : Volume Calculation Sketch



where:
L = Lag, h
T_c = time of concentration, h
T_p = time to peak, h
ΔD = duration of excess rainfall, h
t/T_p = dimensionless ratio of any time to time to peak
q = discharge rate at time t, ft^3/s
q_p = peak discharge rate at time T_p, ft^3/s
Q_a = runoff volume up to t, in
Q = total runoff volume, in

Annex-3 : Volume Calculation Linear (Matlab)

```
A2 =  2.2280
A3 =    2.0249
A4 =    1.8678
A5 =    1.7347
A6 =    1.6146
syms x
Q = (exp(3.77*(1-x))*x^3.77);
R = 0.2
V1 =int(Q,0,A2);
B = (1-R);
Qo = B*x/A2;
V2 =int(Qo,0,A2);
Vf2= V1 -V2;
Vf2= double(Vf2)

Q = (exp(3.77*(1-x))*x^3.77);
R = 0.3
V1 =int(Q,0,A3);
B = (1-R);
Qo = B*x/A3;
V2 =int(Qo,0,A3);
Vf3= V1 -V2;
Vf3= double(Vf3)

Q = (exp(3.77*(1-x))*x^3.77);
R = 0.4
```

```
V1 =int(Q,0,A4);
B = (1-R);
Qo = B*x/A4;
V2 =int(Qo,0,A4);
Vf4= V1 -V2;
Vf4= double(Vf4)

Q = (exp(3.77*(1-x))*x^3.77);
R = 0.5
V1 =int(Q,0,A5);
B = (1-R);
Qo = B*x/A5;
V2 =int(Qo,0,A5);
Vf5= V1 -V2;
Vf5= double(Vf5)

Q = (exp(3.77*(1-x))*x^3.77);
R = 0.6
V1 =int(Q,0,A6);
B = (1-R);
Qo = B*x/A6;
V2 =int(Qo,0,A6);
Vf6= V1 -V2;
Vf6= double(Vf6)
```

Annex -4: Flow chart

1.
INPUT Rain Data
Input precipitation from IDF curves (Average precipitation (mm/h)
OR
Input custom precipitation (mm/h) for every hour or 5 min
Input D= Duration (h)

3.
INPUT Field Data
S=Average Slope (%)

CN of soil

L=Length of field (m)

A= Area (m^2)

7.
INPUT Area Curve of Pond

Input dimension of the basin
L1, L2, Sl1, Sl2, H
Or
Input custom Area curve

2.
Compute Hydrograph

Create a hydrograph for precipitation every 5 min.

If hourly inputed

$P_{hourly}/12= P_{5min}$

4.
Compute Hydrologic properties

Compute
T_p = timpe to peak Fct (S,CN,L)
$Q_{max(n)}$= Maximum runoff for precipitation n Fct(S,CN,L,P_{5min},A)

For every n such as n = 0.. D*60/5

To 8.

5.
Create Flow Table

Tevent(m*5)/ Tprecipitation(n*5)	5	10	15	20	25	…	D	…	48*60
5	$Q_{1,1}$	$Q_{1,2}$	$Q_{1,3}$	$Q_{1,4}$	$Q_{1,5}$		$Q_{1,D/5}$		$Q_{1,(48*60/5)}$
10	0	$Q_{2,2}$	$Q_{2,3}$	$Q_{2,4}$	$Q_{2,5}$		$Q_{1,D/5}$		$Q_{2,(48*60/5)}$
15	0	0	$Q_{3,3}$	$Q_{3,4}$	$Q_{3,5}$		$Q_{2,D/5}$		$Q_{3,(48*60/5)}$
20	0	0	0	$Q_{4,4}$	$Q_{4,5}$		$Q_{3,D/5}$		$Q_{4,(48*60/5)}$
25	0	0	0	0	$Q_{5,5}$		$Q_{4,D/5}$		$Q_{5,(48*60/5)}$
…									
D	0	0	0	0	0		$Q_{D/5,D/5}$		$Q_{D/5,(48*60/5)}$

$Q_{(n,m)}= Q_{max(n)}(m*5/T_p)e^{(-0.5((m*5/T_p)^2-1)}$ From unitless hydrograph curve

6.
Create inflow Table

Tevent	$Q_{in(m)}$
5	$Q_{i(1)}$
10	$Q_{i(2)}$
15	$Q_{i(3)}$
20	$Q_{i(4)}$
25	$Q_{i(5)}$
…	
D	$Q_{i(D/5)}$
…	
48*60	$Q_{i(48*60/5)}$

$Q_{in(m)}$=(Summation of $Q_{(m,n)}$ for n= 0..D/5)

To 11.

9.
INPUT Water outlet design specification

Input Heigth H1..H10 , L

10.
Compute Outflow in function of water height
Using $Q_{(h)}=K_{int}K_{shape}SQRT(2Gh)*A$
$Q_{(h)}$= Outflow at height h(m) (m^3/s)
K_{int}= flow reduction due to flow interference, estimated between 0.5 to 1
K_{shape}= Flow reduction due to shape of the outtake. To be determined for flow between water plank (Assumed to be 0.5)
k_{weir} = Flow reduction due to the friction of the weir flow
G = Gravity 9.81 m/s^2
A = Area of flow

For h smaller than	Outflow (m^3/s)
H1	$Q_{(h)}=(2/3)k_{weir}$ SQRT(2Gh)*h*L
H2	$Q_{(h)}=K_{shape}$SQRT(2Gh)*H1*L
H3	$Q_{(h)}=K_{int}K_{shape}$SQRT(2Gh)* H1*L+ (2/3) k_{weir} SQRT(2G(h-H2))*(h-H2)*L
H4	$Q_{(h)}=K_{int}K_{shape}$SQRT(2Gh)* H1*L+ $K_{int}K_{shape}$SQRT(2G(h-H2))*(H3-h2)*L
…	….

From 7.

8.
Compute Area Curve of Pond

A(h)=(L1+(2*h*Sl1))* (L2+(2*h*Sl2))

From 6.

11.
Compute Delta(H)

m	T	Q_{in}	H1	Q_{out1}	H2	Q_{out2}	H3	Q_{out3}	H(m)	$Qout_{(m)}$

m= 0.. 48*60/5 T = time (min) * m*5 Q_{in} = $Q_{in(m)}$ from step 7. (m^3)

$H1 = H_{(m-1)} + ((Q_{in}(m) *5*60-Q_{out}(H_{(m-1)})*5*60))/A(H_{(m-1)})$ (m)

$H2 = H1 + ((Q_{in}(m) *5*60-Q_{out}(H1)*5*60))/A(H1)$ (m)

$H3 = H2 + ((Q_{in}(m) *5*60-Q_{out}(H2)*5*60))/A(H2)$ (m)

$H(m) = H3 + ((Q_{in}(m) *5*60-Q_{out}(H3)*5*60))/A(H3)$ (m)

Annex-5 : Maximal Inflow Calculation Method

From(Haan et al., 1994) the following formula for the dimensionless hydrograph curve was obtained

$$Q(t) = Qp * (\frac{t}{Tp} * e^{\left(1-\frac{t}{Tp}\right)})\text{^}k$$

By definition:

$$\int_0^\infty Q(t) = CIA = Total\ runoff\ volume$$

With matlab the integration was made using the following code :

```
syms x
syms qp tp k positive
q=qp*(x/tp*exp(1-x/tp))^k
int(q,0,inf)
F=(x/tp)^k
B=(exp(1-x/tp))^k
C= qp
int(C*B*F,0,inf)
```

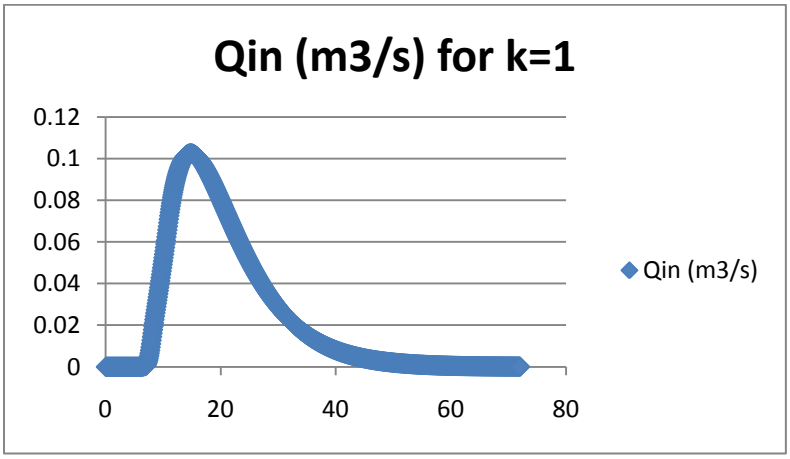
this give the following result :

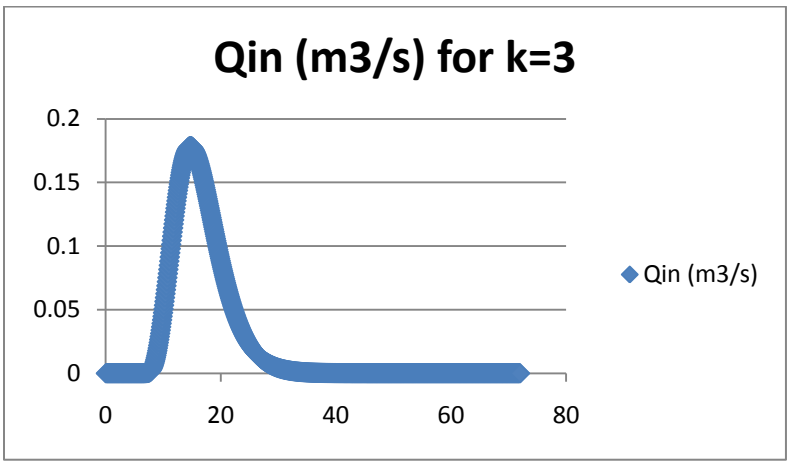
$$\int_0^\infty Q(t) = Qp * tp * e^k * \frac{gamma(k)}{k^k} = CIA$$

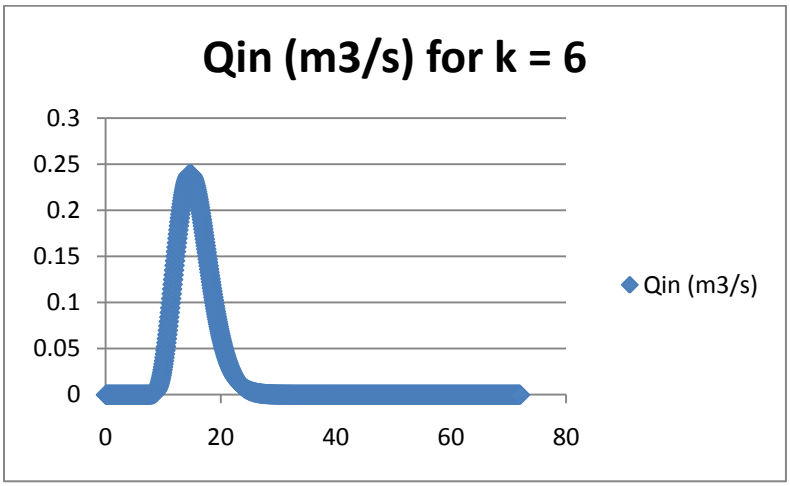
From which the following formula can be obtained :

$$Qp = \frac{CIA}{tp * e^k * \frac{gamma(k)}{k^k}}$$

Annex-6 : Influence of the shape factor on the total Inflow Curve

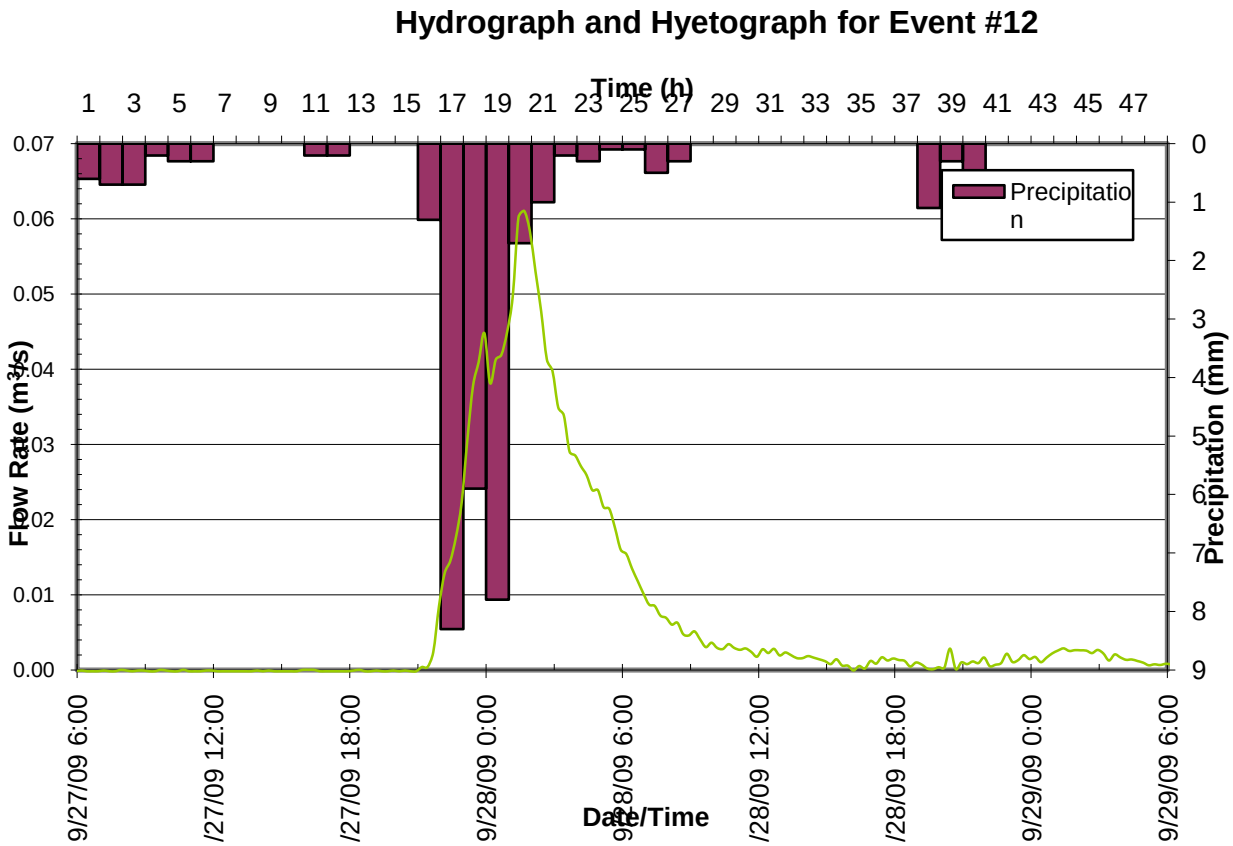




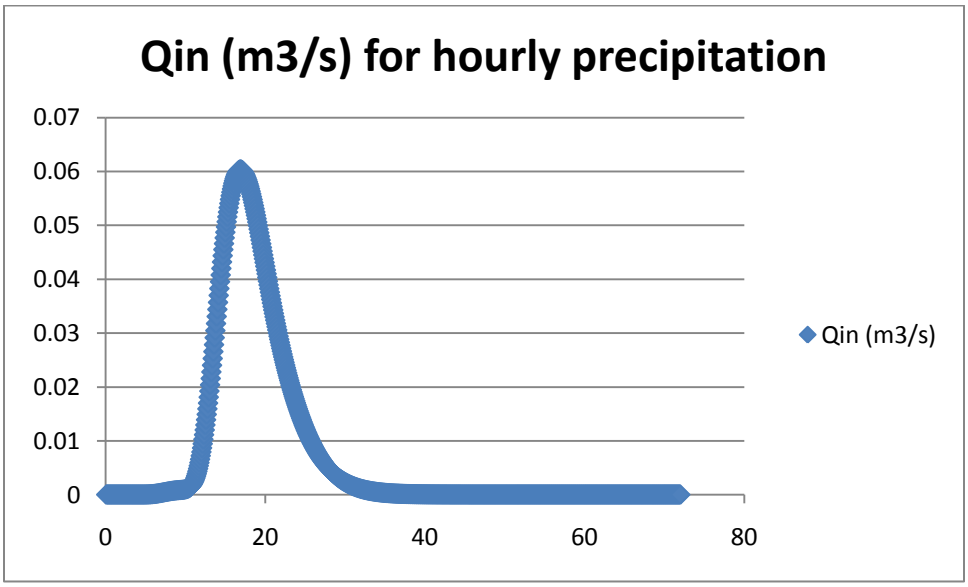


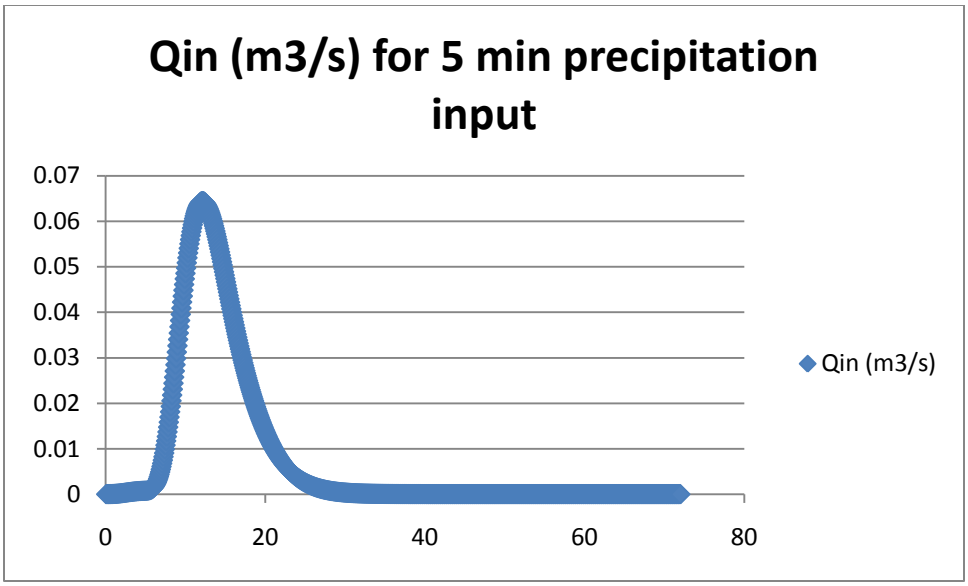
The following input were used to create those graph: Length = 2500 m Area = 35 hectare, Slope = 0.08 %, Cn= 80, Runoff coefficient = 0.4. Hourly Input of 1,22,4,8,9 mm

Annex-7 : Use of hydrological Data to simulate a inflow curve



Inflow graph for Event 12 of the year 2009 for the pond 2 of the St-Samuel field site





Annex-8 : Use of a square basin to approximate a circular one

Pond 1 : Circular Basin of Radius R and height H, the slope are S/H

Area formula:As(h) (R + h*S/H)^2*pi = pi*(R^2+2RhS/H+h^2S^2/H^2)

where h is the height of the water

Approximated pond:

Square pond with a side of C=sqrt(pi*r^2)=1.7725R

Area formula = Ar(h)= $(1.7725R+h*Sc/H)^2$

$Ar(H)= (R+Sc)^2*pi =(1.7725R+Sc)^2$

Find Sc as a function of S

$(2SR+S^2)*pi = 3.545ScR+Sc^2$

Assume $S^2*pi = Sc^2$

SC =(2*pi *S/ 3.545) = sqrt(pi)*S

$Ar(H)= (R+h*sqrt(pi)*S/H)^2*pi = (R^2 +2 RhS/H+ h^2S^2/H^2)*pi$

As(h)-Ar(h) = 0

At h = H (the maximal difference this)

As(H)-Ar(H) = 0

To approximate a circular pond with the rectangular simulation

L1=L2 = sqrt(pi)*R

S1=S2= sqrt(pi)*S

H = H