CHAPITRE 3

ARTICLE 1 : « IMPROVED RATIONAL HYDROGRAPH METHOD »

3.1 Introduction

Stormwater management practices, such as sewer design and evaluation, combined sewer overflow (CSO) attenuation, on-line and off line storage, and real and off-time management, require the availability of computed runoff hydrographs.

Runoff hydrograph at the outlet of an urban catchment depends on the space-time variation of rainfall, the rainfall-runoff process over the catchment area, and the hydrograph routing in pipes. With the help of computers, complex models can be used to simulate detailed runoff hydrographs and provide flow rates and hydraulic gradient lines at the various nodes of the sewer network. The more sophisticated models, such as SWMM (Huber and Dickinson., 1988) or MOUSE (DHI, 2000) require considerable data inputs and effort compared to simpler approach. Such models are well suited for research purposes or for extensive hydrological analysis of large urban areas. Nevertheless, most engineers are unwilling to use such complex models to compute runoff hydrographs for their current practices (O'Loughlin *et al.*, 1996).

Simple and comprehensive models require limited effort and data input to compute accurate runoff hydrographs. Consequently, most engineers prefer using simple and comprehensive models than complex models for their current practices. Simple models do not mean "black box" models. Indeed, "black box" models do not describe the mechanisms involved in the rainfall-runoff process. Consequently, they are not that much useful for most of stormwater management practices (Nix, 1994).

The simplest comprehensive rainfall-runoff model is the rational method developed by Mulvaney (1851) and Kuichling (1889). The method is rational in the sense that it relates runoff peak discharge to rainfall intensity as opposed to purely empirical techniques that correlate peak discharge to catchment characteristics. The rational method has European equivalents that are similarly successful overseas: Lloyd Davies' (1906) formula is used in England, Caquot's (1941) formula in France, and Imhoff's (1964) formula in Central Europe. These models compute the peak flow rate at the outlet of a catchment for a given rainfall intensity.

Despite its simplicity, the rational method has been the most popular method used to design drainage facilities in North America. It is still strongly favored by engineers, since it requires few parameters, all of which are physical and easily obtained from site surveys (Yen and Akan, 1999). Recent developments have transformed the rational method in a model able to compute complete runoff hydrographs. Rossmiller (1983) developed a rational hydrograph formula based on the assumptions of the rational method to compute hydrographs using a constant rainfall intensity deduced from the intensity-frequency–duration curves. Smith and Lee (1984) developed a rational hydrograph method that can simulate the runoff corresponding to a variable rainfall intensity. His method appears to be limited by the difficulty of accurately computing the time of concentration and the runoff coefficient. Guo (2001) improved the rational hydrograph method by deriving a new formula to compute the time of concentration. Unfortunately, the formula was derived from limited data and therefore cannot be used with confidence. Moreover, his rational hydrograph method was unable to simulate the initial abstraction on impervious areas and the infiltration on pervious areas.

The main goal of the present paper is to offer engineers an improved rational hydrograph method able to compute accurate runoff hydrographs at the outlet of an urban catchment. The paper aims to show how it is possible to improve the rational hydrograph method and overcome its limitations while keeping its simplicity. The following items are of particular interest: 1- the capacity to use variable intensity rainfalls; 2- a proposed alternative to the lumped runoff coefficient by introducing infiltration for pervious areas and initial abstraction for impervious areas; 3- the sensitivity analysis of the improved rational hydrograph method; 4- a sequential approach to calibrate the parameters involved; 5- the validation of the improved rational hydrograph method by comparing simulated runoff to measured runoff and to runoff computed with the nonlinear reservoir model.

3.2 Rational hydrograph method and non linear reservoir model

3.2.1 IRH method

The improved rational hydrograph (IRH) method is based on the linear system theory described by Chow *et al.* (1988). Moreover, the following physical assumptions are considered:

1- the impulse response function of a catchment area is rectangular-shaped and ends at the time of concentration;

2- the time of concentration corresponds to the time difference between the end of the rainfall and the end of the direct runoff;

3- rainfall intensity is uniform on the catchment;

4- the runoff on impervious areas is independent of the runoff on pervious areas.

Consequently, the runoff at a time t, due to a variable rainfall intensity, is given by the following convolution product between the rainfall intensity and the impulse response function of the catchment:

$$Q(t) = \int_0^t \left[(I(\tau) - dp(\tau))u_{imp}(t-\tau)\right]d\tau + \int_0^t \left[(I(\tau) - f(\tau))u_{per}(t-\tau)\right]d\tau \quad (3.1)$$

where Q is the runoff (m^3/s), I is the rainfall intensity (mm/h), u_{imp} is the impulse response function of the directly connected impervious area, u_{per} is the impulse response function of the pervious area, dp is the initial abstraction capacity (mm/h), f is the infiltration capacity (mm/h).

Figure 4 shows that the duration of the impulse response function for directly connected impervious areas and for pervious areas is equal to the time of concentration.

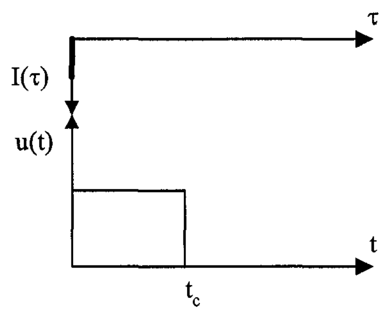


Figure 4 Impulse response function of the catchment

The IRH method uses the time of concentration to take into account the physical characteristics of a catchment. Indeed, the time of concentration is a lumped parameter related to the slope, roughness and flow path length of a catchment (Chow *et al.*, 1988).

Rainfall intensity is always sampled in discrete time. Consequently, equation (3.1) should be expressed as follows:

$$Q(m) = \overbrace{\sum_{j=1}^{m\leq m_r}\left[\left(I(j)-dp(j)\right)u_{imp}(m-j+1)\right]\cdot \Delta t}^{Q_{imp}} + \overbrace{\sum_{j=1}^{m\leq m_r}\left[\left(I(j)-f(j)\right)u_{per}(m-j+1)\right]\cdot \Delta t}^{Q_{per}} \quad (3.2)$$

with

$$u_{imp}(m-j+1) = K_c \, IMP \, A \frac{1}{t_c} \quad \text{when} \quad 1 \le (m-j+1)\Delta t \le t_c \tag{3.3}$$

and

$$u_{per}(m-j+1) = K_c \, (1-IMP) A \frac{1}{t_c} \quad \text{when} \quad 1 \le (m-j+1)\Delta t \le t_c \tag{3.4}$$

where I is the average rainfall intensity during Δt (mm/h), A is the catchment area (ha), IMP is the ratio of directly connected impervious area, t_c is the time of concentration (min), K_c is a constant equal to 0,0028 in metric units or 1 for English units, Δt is the time step (min), m_r is the last index of rainfall vector, j and m are time indices.

The conditions $I(j) - dp(j) \ge 0$, $I(j) - f(j) \ge 0$ and $\Delta t / t_c \le 1$ must be respected in equation (3.2). Moreover, u_{imp} and u_{per} are equal to 0 when $(m-j+1)\Delta t > t_c$. The notation $m \le m_r$ as the upper limit of the summation indicates that the terms are summed for $j = 1$ to m when $m \le m_r$, whereas for $m > m_r$, the summation is limited to $j = 1$ to m_r.

The first term Q_{imp} on the left side of equation (3.2) represents the contribution from directly connected impervious areas. These are mainly roads, as well as roofs that are directly connected to the storm sewer system. The second term Q_{per} on the right side of equation (3.2) represents the contribution from the pervious areas and from the indirectly connected impervious areas. These are wastelands and grass-covered lawns and, in the case of indirectly connected impervious areas, roofs draining towards pervious areas. The IRH method assumes that indirectly connected impervious areas are equivalent to pervious areas. Indeed, the rainfall that falls on indirectly connected impervious areas pass through pervious areas. Consequently, this rainfall is subject to infiltration as the rainfall falling on pervious areas.

Like the rational hydrograph of Guo (2001), the accuracy of the IRH method diminishes steadily when the catchment area is greater than 100 ha. Indeed, the flow routing in pipes becomes significant in large catchments and the catchment response can no longer be assumed linear.

To illustrate the properties of the IRH method, three runoff hydrographs were computed with equation (3.2) using rainfalls of constant intensity and variable duration. The conditions $dp(j)=0$, $I(j)=I$ and $f(j)=f$ were assumed for the computation of the three hydrographs. Figure 5 presents the three different runoff hydrographs.

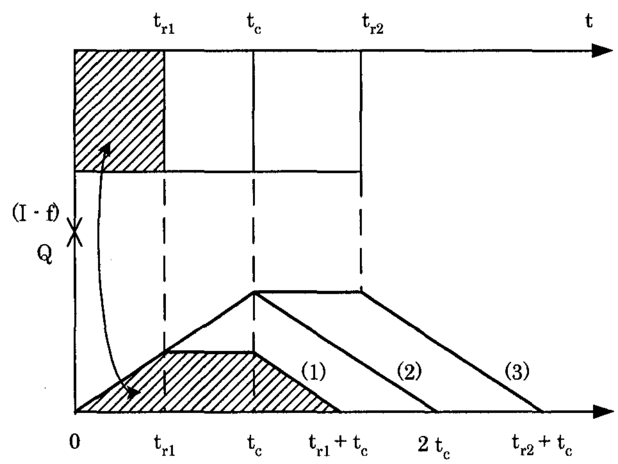


Figure 5 Runoff hydrographs of the IRH method for various rainfall durations

The hydrograph (1) is obtained for a rainfall having a duration t_{r1} lower than t_c. In this case, the runoff increases up to t_{r1} when a proportion t_{r1}/t_c of the catchment contributes to the outlet. Then, the runoff kept constant, up to t_c, owing to the contribution of the rainfall felt in the upper portion of the catchment. The hydrograph (2) is obtained for a rainfall having a duration equal to t_c. In this case, the runoff increases up to t_c. At this point, the runoff is maximum because the whole catchment area contributes at the outlet. The hydrograph (3) is obtained for a rainfall having a duration t_{r2} greater than t_c. In this case, the runoff increases up to t_c. Then, the runoff remains constant up to t_{r2} owing to the rainfall that continues to fall on the overall catchment area. In all patterns, the runoff always ends after a period of time equal to the sum of the rainfall duration and the time of concentration. Moreover, runoff volume is always equal to net rainfall volume. Maximum runoff peak flow occurs at t_c for a rainfall duration upper or equal to t_c. The value of the maximum peak flow computed with equation (3.2) is expressed as follows:

$$Q(t_c) = Q_p = K_c\, IMP\, I\, A + K_c(1 - IMP)(I - f)A \tag{3.5}$$

rearranging equation (3.5) gives:

$$Q_p = K_c I\left[IMP\,A + \frac{I-f}{I}(1 - IMP)A\right] = K_c I\left(C_{imp}A_{imp} + C_{per}A_{per}\right) = K_c CIA \tag{3.6}$$

with

$$C = \frac{C_{imp}A_{imp} + C_{per}A_{per}}{A} \tag{3.7}$$

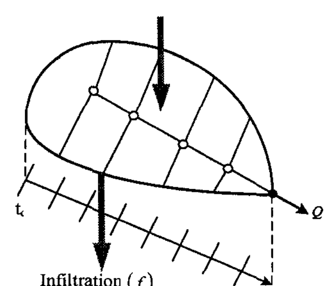
where C is the runoff coefficient, C_{imp} is the runoff coefficient for the directly connected impervious area, C_{per} is the runoff coefficient for the pervious area, A_{imp} is the impervious area (ha), A_{per} is the pervious area (ha).

Equation (3.6) is the rational method formula used in current practice to compute peak flow at the outlet of an urban catchment. This formula appears to be a special case of the IRH method.

3.2.2 NLR model versus IRH method

The well-tried nonlinear reservoir model (NLR model) of SWMM (Huber and Dickinson, 1988) is used as a basis of comparison to emphasize the advantages of the IRH method. The nonlinear reservoir model and the IRH method are shown side by side in Figure 6.

IRH method



$$Q(m)=Q_{imp}(m)+Q_{per}(m)$$

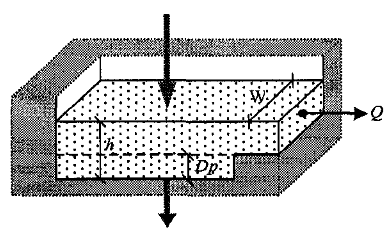
Impervious Area

$$Q_{imp}(m)=\sum_{j=1}^{m\le m_r}\left[\left(I(j)-dp(j)\right)u_{imp}(m-j+1)\right]\cdot \Delta t$$

Pervious Area

$$Q_{per}(m)=\sum_{j=1}^{m\le m_r}\left[\left(I(j)-f(j)\right)u_{per}(m-j+1)\right]\cdot \Delta t$$

NLR model



$$Q(m)=Q_{imp}(m)+Q_{per}(m)$$

Impervious Area

$$Q_{imp}(m)=W\frac{1}{n_{imp}}\left(h(m)-Dp\right)^{5/3}S_o^{1/2}$$

$$\frac{h(m)-h(m-1)}{\Delta t}=I+W\frac{S_o^{1/2}}{n_{imp}A_{imp}}\left[h(m-1)+\frac{1}{2}\left(h(m)-h(m-1)\right)-Dp\right]^{5/3}$$

Pervious Area

$$Q_{per}(m)=W\frac{1}{n_{per}}h(m)^{5/3}S_o^{1/2}$$

$$\frac{h(m)-h(m-1)}{\Delta t}=\left(I(m)-f(m)\right)+W\frac{S_o^{1/2}}{n_{per}A_{per}}\left[h(m-1)+\frac{1}{2}\left(h(m)-h(m-1)\right)\right]^{5/3}$$

Figure 6 Schematic representation of the IRH method and the NLR model

The NLR model conceptualizes the urban catchment as a reservoir having the rainfall as input and, rainfall abstractions and runoff as output. The depth of water in the reservoir is found by coupling the continuity equation and Manning's equation. Expressed in terms of finite differences, the continuity equation becomes:

$$\frac{h(m)-h(m-1)}{\Delta t} = I + W\frac{S_o^{1/2}}{nA}\left[h(m-1)+\frac{1}{2}\left(h(m)-h(m-1)\right)\right]^{5/3} \tag{3.8}$$

where h is the depth of water in the catchment (m), W is the catchment width (m), n is the Manning's coefficient, S_o is the average slope of the ground (m/m), A is the area of the catchment (m^2), I is the rainfall intensity (m/s).

The successive depths of water (h) in the NLR model are determined at each time step with the Newton-Raphson iterative method and the corresponding flow rates at the outlet of the reservoir are computed with Manning's equation.

The NLR model and the IRH method simulate the runoff at the outlet of an urban catchment by adding the runoff computed respectively on directly connected impervious areas and pervious areas. The runoff on pervious areas starts when the volume of rainfall exceeds the initial abstraction and the potential storage of the soil. The initial abstraction for pervious areas is due to the interception of rainfall by the surface cover. Losses by initial abstraction are much less significant than infiltration and are therefore disregarded. The infiltration process on pervious areas is represented with Horton's model (Horton, 1940) and the amount of potential storage remaining in the soil is taken into account through the use of the moving curve concept (Huber and Dickinson, 1988). The Horton model is widely used on urban catchments because it is not a physically based approach. Moreover, its calibration necessitates few field data. The runoff on directly connected impervious areas starts when the rainfall depth has reached the initial abstraction depth Dp. For directly connected impervious areas, the initial abstraction Dp is the water depth retained in surface depressions.

The volume of runoff computed with the NLR model and the IRH method is controlled by the ratio of directly connected impervious areas IMP, the initial abstraction depth Dp of directly connected impervious areas and the infiltration capacity f of pervious areas. IMP is never known with a high level of accuracy. Consequently, it has to be adjusted in a calibration procedure. The initial infiltration capacity f_o of Horton model is far more sensitive than the final infiltration capacity f_∞ and the decay rate K (Liong *et al.*, 1991). Consequently, f_o has to be calibrated and a default value can be chosen for the final infiltration capacity and the decay rate (Maidment, 1993). Dp controls the starting time of the hydrograph. A default value of Dp can be estimated using the SCS method (Maidment, 1993).

The peak flow amplitude and the rise time of a runoff hydrograph computed with the NLR model are controlled by Manning's coefficient for pervious areas n_{per}, Manning's coefficient for directly connected impervious areas n_{imp}, the ground slope S_o and the catchment width W. In the case of the IRH method, peak flow and rise time are only controlled by t_c. A representative value of n_{per}, n_{imp} and S_o can be obtained from a site survey. Nevertheless, W for the NLR model and t_c for the IRH method have to be calibrated because any reliable relationships can give an accurate estimation of these two parameters (McCuen, 2005).

3.3 Sensitivity Analysis

A sensitivity analysis was carried out on the IRH method in order to assess practical use and limitations. The sensitivity analysis involves determining the change of the IRH method response to the change of its parameters. The set of parameters presented in

Table I is assigned to the IRH method in order to compute a reference runoff hydrograph.

Table I

Reference parameters for sensitivity analysis of the IRH method

Paramètres	A	IMP	t_c	Dp	f_o	f_∞	K
Valeurs	100	0,5	32	2	160	16	4

The level of change between simulated and reference runoff hydrograph is evaluated with the following three performance criteria:

- the *Nash* coefficient (Nash and Sutcliffe, 1970):

$$Nash = 1 - \frac{\sum_{j=1}^{m} \left(Q_{ref}(j) - Q_{sim}(j)\right)^2}{\sum_{j=1}^{m} \left(Q_{ref}(j) - \overline{Q}_{ref}\right)^2} \tag{3.9}$$

where $Q_{ref}(j)$ is the reference flow at time j, $Q_{sim}(j)$ is the simulated flow at time j, $\overline{Q}_{ref}$ is the reference mean flow.

The *Nash* coefficient evaluates the agreement between a simulated and a reference runoff hydrograph. A *Nash* of 1 indicates a perfect agreement between simulated and reference runoff.

- the peak flow rate:

$$R_v = \frac{Q_p^{sim}}{Q_p^{ref}} \tag{3.10}$$

As in the case of the *Nash* coefficient, a R_p value of 1 indicates that the simulated peak flow is equal to the reference peak flow.

- the ratio of the simulated over the reference runoff volume:

$$R_v = \frac{\sum_{j=1}^{m} Q_{sim}(j) \cdot \Delta t}{\sum_{j=1}^{m} Q_{ref}(j) \cdot \Delta t} \tag{3.11}$$

A R_v value of 1 indicates that the simulated runoff volume is equal to the reference runoff volume.

Two rainfalls of constant intensity were used:
Rainfall (1): $I = 10$ mm/h; $t_r = 240$ min.
Rainfall (2): $I = 80$ mm/h; $t_r = 20$ min.

Rainfall (1) is of low intensity whereas rainfall (2) is of high intensity. Consequently, the runoff generated by rainfall (1) exclusively comes from the directly connected impervious areas and the runoff generated by rainfall (2) comes from the pervious and impervious areas.

The results of the sensitivity analysis carried out with rainfall (1) have shown that R_v and R_p change linearly with *IMP*. Consequently, an error in the estimation of *IMP* leads to an equivalent error in the simulated peak flow and runoff volume. Moreover, the initial abstraction *Dp* has a very limited impact on the change of R_v, R_p and *Nash* and the time of concentration has no impact on R_v and R_p. Nevertheless, Figure 7 shows that *IMP* has a major impact on the *Nash* and t_c has a limited impact on the *Nash*.

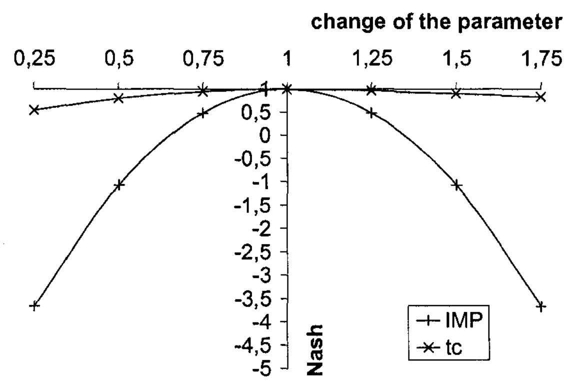


Figure 7 Changes in *Nash* with IRH parameters for rainfall (1)

The results of the sensitivity analysis carried out with rainfall (2) are shown in Figure 8 (a), (b) and (c).

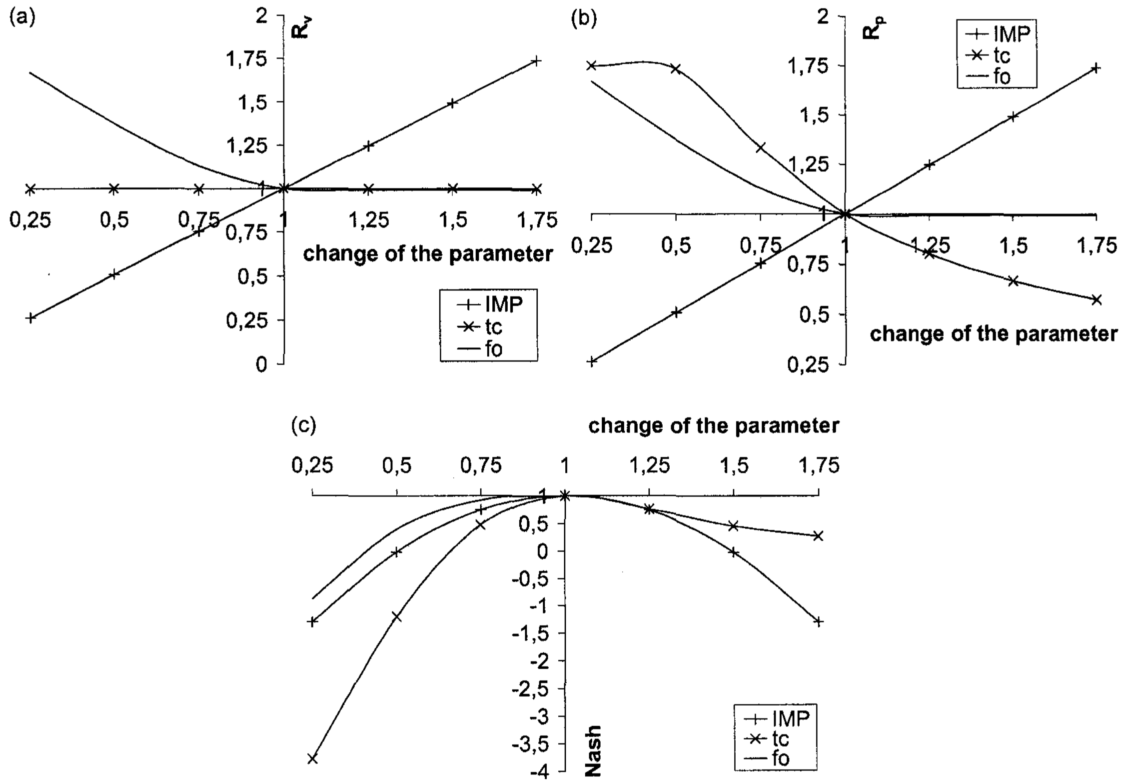


Figure 8 Changes in (a) $R_{v,}$, (b) R_p, and (c) *Nash* with IRH parameters for rainfall (2)

Three important statements can be made:

- *IMP* has a huge influence on R_v, R_p and *Nash* change.

- t_c has the greatest impact on R_p and *Nash* when t_c is lower than the rainfall duration. The impact of t_c decreases significantly and drops lower than the impact of *IMP* when t_c becomes greater than the rainfall duration.

- f_o has a major impact on the R_v, R_p and *Nash* change. The final infiltration capacity and the decay rate of the Horton model have a limited impact on the R_v, R_p and *Nash* change. Consequently, these two parameters are not represented in Figure 8 (a), (b) and (c). This observation correlates with the results obtained by Liong *et al.* (2001) for the sensitivity analysis of the NLR model. Moreover, an underestimation of f_o leads to a greater change on R_v, R_p, and *Nash* than an overestimation.

3.4 Calibration procedure

Two calibration approaches can be used to calibrate rainfall-runoff models (Duan *et al.*, 1994). The first approach is to calibrate rainfall-runoff models with automatic global search algorithms. This approach is generic and requires minimum user intervention. Practically, a set of monitored rainfall-runoff events are used to calibrate automatically all the parameters simultaneously. The second approach is to use knowledge-based calibration procedures. The philosophy of this approach is to guide and help the user during the model calibration. Practically, the user has to follow a predefine series of steps to calibrate its model. At each step, the user calibrates automatically or manually a specific set of parameters. Knowledge-based calibration is specific to each rainfall-runoff model and requires the user intervention. The two calibration approaches were compared in different studies (Gupta *et al.*, 1999), (Madsen *et al.*, 2002). No significant differences of efficiency were noted between the two approaches.

The runoff simulated with the NLR model and the IRH method comes from directly connected impervious areas during rainfalls of low intensity and from both pervious and impervious areas during rainfalls of high intensity. Consequently, it appears advantageous to use the rainfalls of low intensity to calibrate the parameters associated to directly connected impervious areas and the rainfalls of high intensity to calibrate the parameters associated to pervious areas. Rainfalls of low intensity can be differentiated from rainfalls of high intensity using the Horton model. Default values for the parameters of the Horton model are given in the literature for different type of soil (Maidment, 1993).

A knowledge-based calibration in three sequential steps is proposed in Figure 9 to calibrate the parameters of the NLR model and the IRH method.

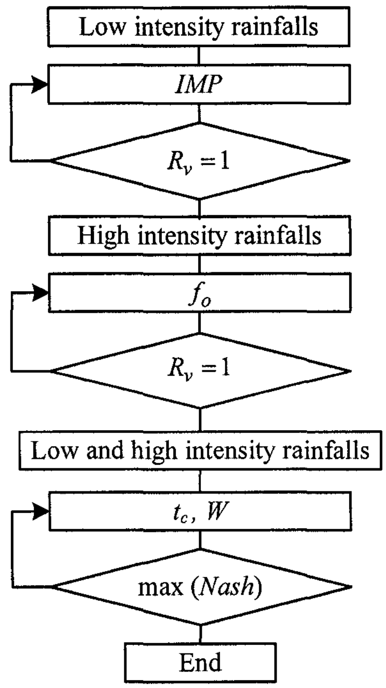


Figure 9 Calibration procedure for the NLR model and the IRH method

The first step is to calibrate *IMP* to both the NLR model and the IRH method using rainfalls of low intensity. The second step is to calibrate f_o to both the NLR model and the IRH method using rainfalls of high intensity. The third step is to calibrate t_c for the IRH method and W for the NLR model. The same storm events used in the first and second step are applied to calibrate t_c and W. The parameters n_{imp}, n_{per}, and S_o of the NLR model, and, the parameters Dp, f_∞ and K of the NLR model and the IRH method are not calibrated. Default values are given to these parameters.

The first step starts by assigning a default value to *IMP*. A default value of *IMP* can be obtained by a survey of the catchment occupation. Then, the optimal value of *IMP* is computed with the Nelder-Mead simplex method (Lagarias *et al.*, 1998) by equalizing the simulated runoff volume to the measured runoff volume.

The second step starts by assigning a default value to f_o. Default values of f_o are given in the literature. Then, the optimal value of f_o is computed with the Nelder-Mead simplex method by equalizing the simulated runoff volume to the measured runoff volume.

The third step starts by assigning a default value to W and t_c. A default value of W and t_c can be estimated using various formulae proposed in the literature (Huber and Dickinson, 1988), (Mays, 2005). Then, t_c and W are adjusted with the Nelder-Mead simplex method in order to maximize the *Nash* coefficient.

3.5 Application

3.5.1 The sites

The calibration and validation of the IRH method was carried out with 5 rainfall events monitored in the subcatchment (1) of the Verdun borough (Canada) and with 36 rainfall events (Maximovic and radojkovic, 1986) monitored in the urban catchments of East York (Canada), Sample Road and Fort Lauderdale in Broward County (USA), Malvern in Burlington (Canada), Gray Haven in Baltimore (USA) and Saint Marks Road in Derby (Great Britain). Runoff at the outlet of the urban catchment was monitored for each rainfall event. The physical characteristics of the seven selected catchments are given in Table II.

Table II

Physical characteristics of the seven selected urban catchments

Catchments	Area (ha)	Ratio of impervious area	Ground slope (m/m)
Verdun	177,0	0,53	0,005
East York	155,8	0,38	0,011
Sample Road	23,6	0,17	0,003
Malvern	23,3	0,34	0,020
Gray Haven	9,4	0,45	0,010
Saint Marks Road	8,6	0,55	0,003
Fort Lauderdale	8,3	0,98	0,001

The seven selected urban catchments cover a large range of values for each physical characteristic, which makes it possible to conduct an exhaustive validation of the IRH method. The ratio of impervious areas corresponds to the estimated ratio of directly and indirectly connected impervious areas for the Verdun and Saint Marks Road catchments and to the estimated ratio of directly connected impervious areas for the other catchments.

3.5.2 Model calibration and validation

The NLR model and the IRH method were calibrated using one rainfall of low intensity and one rainfall of hygh intensity for each catchment. Parameters obtained after calibration for each catchment are shown in Table III.

Table III

Calibrated parameters of the NLR model and of the IRH method

Catchments	A (ha)	IMP (%)	t_c (min)	W (m)	S_0 (m/m)	n_{imp}	n_{per}	Dp (mm)	f_o (mm/h)	f_∞ (mm/h)	K (h^{-1})
Verdun	177,0	0,40	36	2970	0,005	0,014	0,025	1,0	50	15	2
East York	155,8	0,40	30	2000	0,011	0,014	0,025	1,0	45	15	2
Sample Road	23,6	0,20	29	260	0,003	0,014	0,025	1,0	230	15	2
Malvern	23,3	0,37	10	1380	0,020	0,014	0,025	1,0	50	15	2
Gray Haven	9,4	0,43	15	310	0,010	0,014	0,025	1,0	95	15	2
St. Mark Road	8,6	0,30	25	460	0,003	0,014	0,025	1,0	35	15	2
Fort Lauderdale	8,3	1,00	18	1340	0,001	0,014	0,025	1,0	undefined		

IMP values are close to the ratio of impervious areas presented in Table II for the East York, Sample Road, Malvern, Gray Haven and Fort Lauderdale catchments. Indeed, the ratio of impervious areas presented in Table II corresponds to the ratio of directly connected impervious areas for the East York, Sample Road, Malvern, Gray Haven and Fort Lauderdale catchments. Consequently, the calibration results validate the fact that *IMP* in the NLR model and IRH method is a representative parameter of the directly connected impervious areas. In the case of the Verdun and Saint Marks Road catchments, *IMP* values are different to the ratio of impervious areas presented in Table II. Indeed, the ratio of impervious areas presented in Table II for the Verdun and Saint Marks Road catchments corresponds to the ratio of total impervious areas.

The validation of the NLR and IRH method consists of testing the ability of both models to simulate runoff hydrographs. Consequently, the NLR model and the IRH method were used to compute the runoff of the 27 rainfall events that were not considered during the calibration procedure. The *Nash*, R_v and R_p values for each computed event are shown in Table IV.

Table IV

Nash, R_v and R_p value after simulation of the 27 registered runoff events with the NLR model and the IRH method

Events	Rainfall depth (mm)	Runoff depth (mm)	Rainfall duration (min)	I_{max} 5 min (mm/h)	Q_{max} (m^3/s)	*Nash*		R_v		R_p	
						NLR	IRH	NLR	IRH	NLR	IRH
V. 13-10-99	20,80	9,95	300	21,6	1,88	0,61	0,53	0,91	0,97	0,78	1,11
V. 23-08-00	10,00	8,60	240	14,4	1,81	0,73	0,78	0,96	0,99	0,85	1,01
V. 22-06-01	10,10	5,90	250	20,4	2,51	0,71	0,76	0,97	1,01	0,71	0,88
E. Y. 13-08-76*	5,58	2,15	20	58,0	3,34	0,40	0,41	0,99	1,33	0,54	0,81
E. Y. 01-09-76	5,27	1,75	59	19,7	1,78	0,69	0,45	0,81	0,98	0,55	0,86
E. Y. 25-06-77*	17,14	8,63	82	61,0	6,24	0,75	0,76	0,90	1,01	0,56	0,93
E. Y. 10-08-77*	11,13	5,23	35	54,8	5,98	0,48	0,60	0,87	1,11	0,54	0,76
S. R. 29-05-76*	52,00	12,14	168	85,9	1,04	0,85	0,83	0,83	0,84	0,63	0,66
S. R. 29-05-76	13,30	2,20	132	28,0	0,20	0,92	0,85	1,06	1,12	1,14	1,25
S. R. 04-06-76	9,96	2,20	72	28,9	0,24	0,74	0,89	0,75	0,81	0,53	0,75
S. R. 07-06-76	16,96	3,44	197	52,7	0,33	0,78	0,82	0,89	0,93	0,73	0,94
M. 23-09-73*	9,14	3,25	126	31,2	0,72	0,88	0,86	0,94	0,94	1,00	0,89
M. 05-05-74	7,62	2,22	164	7,6	0,14	0,89	0,91	1,08	1,10	1,18	1,08
M. 28-09-74	15,24	4,40	87	24,4	0,43	0,87	0,80	1,19	1,20	0,97	1,16
M. 20-11-74	4,57	1,46	56	11,7	0,20	0,17	0,29	0,66	0,65	0,58	0,73

Table IV (continued)

Events	Rainfall depth (mm)	Runoff depth (mm)	Rainfall duration (min)	I_{max} 5 min (mm/h)	Q_{max} (m^3/s)	*Nash*		R_v		R_p	
						NLR	IRH	NLR	IRH	NLR	IRH
G. H. 05-06-63*	55,88	37,94	53	13,1	2,26	0,81	0,94	0,81	0,89	0,87	0,97
G. H. 10-06-63*	50,80	37,09	53	103,0	2,21	0,75	0,86	0,77	0,79	0,76	0,78
G. H. 20-06-63*	37,08	15,27	72	81,1	0,83	0,87	0,88	1,10	1,14	0,95	0,98
G. H. 29-06-63*	30,23	14,85	175	78,6	0,76	0,85	0,87	0,94	0,96	0,87	1,03
S. M. 02-10-75	6,97	1,52	192	6,2	0,05	0,86	0,83	1,13	1,17	0,76	0,70
S. M. 15-11-75	5,64	1,64	139	10,8	0,08	0,75	0,84	0,75	0,81	0,64	0,72
S. M. 22-09-76	7,05	1,45	352	12,6	0,06	0,83	0,86	1,18	1,23	0,81	0,83
S. M. 25-09-76	13,63	3,49	290	14,2	0,11	0,80	0,88	1,05	1,10	0,86	0,85
F. L. 20-06-75	7,23	7,14	218	23,8	0,46	0,62	0,83	0,82	0,88	0,40	0,71
F. L. 23-06-75	44,34	44,61	314	91,0	1,47	0,97	0,94	0,95	0,97	1,23	1,15
F. L. 04-07-75	22,13	21,39	152	70,9	1,28	0,93	0,84	0,96	0,99	0,96	0,80
F. L. 05-07-75	15,48	19,81	177	74,2	1,04	0,70	0,72	0,72	0,73	0,64	0,65

*Events for which impervious and pervious areas contribute

The *Nash* coefficient is over 0,7 for 22 events simulated with the IRH method and for 21 events simulated with the NRL model. Consequently, there is a good agreement between simulated and measured runoff for a large number of simulated events. The *Nash* value for the NLR model and IRH method varies with the size of the lag time between simulated and measured runoff as shown in Figure 10 (a) and (b).

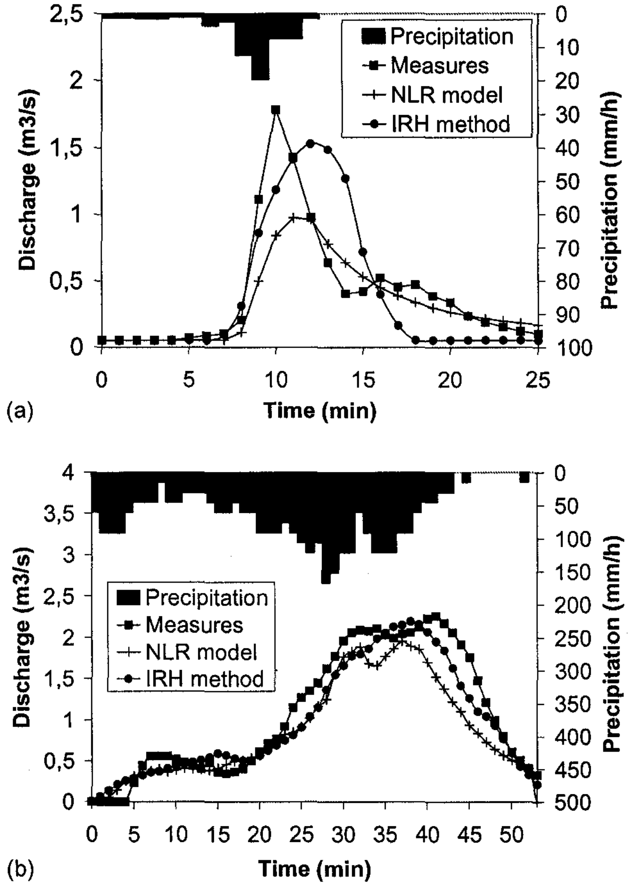


Figure 10 Runoff computed with the NLR model and the IRH method for rainfalls (a) E. Y. 01-09-76 and (b) G.H. 05-06-63

The error on the simulated runoff volume is less than 15% for 59% of the events simulated with the NLR model and for 63% of the events simulated with the IRH method. Therefore, the two models give an acceptable forecast of the runoff volume. However, the two models have a peak flow error greater than 15% for 67% of the events simulated with the NLR model and for 52% of the events simulated with the IRH method. The low R_p values illustrate the accuracy limits of the two hydrologic models.

The reduction of the Manning coefficients used in the NLR model may slightly improve R_p values. Unfortunately, these new Manning values will have no more physical significance.

The average and standard deviation computed with the *Nash*, R_v and R_p data are presented in Table V for the NLR model and the IRH method.

Table V

Average and standard deviation of
Nash, R_v and R_p for the NLR model and the IRH method

Model	*Nash*		R_v		R_p	
	Average	Standard deviation	Average	Standard deviation	Average	Standard deviation
NLR model	0,75	0,18	0,93	0,14	0,78	0,21
IRH method	0,77	0,17	0,99	0,16	0,89	0,17

The average of *Nash*, R_v and R_p data for the IRH method appears to be greater than the average of *Nash*, R_v and R_p data for the NLR model. Moreover, the standard deviation of *Nash* and R_p data for the IRH method is lower than the standard deviation of *Nash* and R_p data for the NLR model. Consequently, the IRH method seems to give better results than the NLR model. Nevertheless, statistical t-tests carried out on the average of the *Nash*, R_v and R_p data have failed to detect significant differences in accuracy between the NLR model and the IRH method.

3.6 Conclusion

The IRH method represents the urban catchment as a linear system. The originality of the IRH method is based on the explicit consideration of the contribution of pervious and impervious areas, the variability of rainfall intensity, as well as losses due to infiltration and initial abstraction. Moreover, the rational method formula appears to be a special case of the IRH method.

Sensitivity analysis reveals that for rainfalls of high intensity and short duration, the IRH method is particularly sensitive to the time of concentration. In the other case, the catchment area and the ratio of impervious area are the most influencing parameters.

All the parameters are calibrated with a sequential procedure using rainfalls of low and high intensity. The validation of the IRH method carried out with 41 rainfall events gauged in 7 different North American and European urban catchments shows that the rational hydrograph method can simulate runoff with a high level of accuracy. Moreover, a comparative study between the IRH method and the NLR model indicates that the IRH method gives equivalent results to those of the more sophisticated NLR model. Practically, the IRH method can easily be implemented with an Excel spreadsheet or with a programming language.

New developments are required to take into account the routing effect in the sewer network and to extend the use of the IRH method to large catchments.