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CHAPITRE 4

ARTICLE 2 : « IMPROVEMENT AND SCALE EFFECT STUDY OF THE RATIONAL HYDROGRAPH METHOD »

4.1 Introduction

The runoff flow at the outlet of urban drainage systems is used for design or management purposes. The widely used rainfall-runoff model is indisputably the rational method (Mulvaney, 1851). This method was introduced in North America by Kuichling (1889). The rational method has equivalents in Europe such as the Lloyd-Davies (1906) method in England, the Caquot (1941) formula in France, and the Imhoff (1964) formula for Central Europe. The traditional rational formula gives the peak flow at the catchment outlet according to the area, the runoff coefficient and the average rainfall intensity evaluated for a specific return period. In spite of its simplicity, the rational method was used to design the majority of Northern American sewer networks and still receives a great recognition among the engineer's community. The rational method was transformed, by Smith and Lee (1984), into a rational hydrograph method able to simulate complete runoff hydrographs. Then, Guo (2001) developed a formula to compute the time of concentration, of the rational hydrograph method. More recently, Bennis and Crobeddu (2005) brought new developments to the rational hydrograph method by taking into account the runoff contribution of pervious and impervious areas, the initial abstraction and the infiltration losses.

The rational hydrograph method simulates accurately the rainfall-runoff process on small catchments. Nevertheless, it seems to be limited by the urban catchment area. Indeed, Guo (2001) has shown that the rational hydrograph method undergoes a decrease of accuracy for catchment areas greater than 60 hectares. Moreover, validation tests of the rational hydrograph method carried out by Bennis and Crobeddu (2005) on a 170 ha urban catchment emphasized a temporal shift in the occurrence of the peak flow.

The objective of this paper is to develop a new rational hydrograph method valid for large urban catchment area. The following items are of particular interest: 1- the introduction of new theoretical developments to the rational hydrograph method of Bennis and Crobeddu (2005) in order to improve its accuracy; 2- a scale effect study of the rational hydrograph method; 3- the validation of the new rational hydrograph method on monitored runoff data and its comparison to the old rational hydrograph method.

4.2 Rational hydrograph methods

4.2.1 IRH1 method

The improved rational hydrograph (IRH1) method is based on the linear system theory described by Chow *et al.* (1988). The runoff flow at the outlet of an urban catchment is expressed in discreet time by the following convolution product :

$$Q(m)=\overbrace{\sum_{j=1}^{m\le m_r}\left[(I(j)-dp(j))u_{imp}(m-j+1)\right]\Delta t}^{Q_{imp}}+\overbrace{\sum_{j=1}^{m\le m_r}\left[(I(j)-f(j))u_{per}(m-j+1)\right]\Delta t}^{Q_{per}} \quad (4.1)$$

with

$$u_{imp}(m-j+1)=K_c\, IMP\, A\frac{1}{t_c} \quad \text{for} \quad 1\le(m-j+1)\Delta t\le t_c \quad (4.2)$$

and

$$u_{per}(m-j+1) = K_c(1-IMP)A\frac{1}{t_c} \quad \text{for} \quad 1 \le (m-j+1)\Delta t \le t_c \tag{4.3}$$

where Q represents the runoff (m^3/s); Q_{imp}, the runoff on impervious areas, (m^3/s); Q_{per}, the runoff on pervious areas, (m^3/s); I, the rainfall intensity (mm/h); u_{imp}, the impulsional response of impervious areas; u_{per}, the impulsional response of pervious areas; dp, the initial abstraction capacity (mm/h); f, the infiltration capacity (mm/h); A, the catchment area (ha); IMP, the ratio of impervious areas; t_c, the time of concentration; K_c, constant equal to 0,0028 in metric units or 1 for English units; Δt, the time step (min); m_r, the last index of rainfall vector; j and m, 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 (4.1). 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 IRH1 method is conceptually represented in Figure 11 (a).

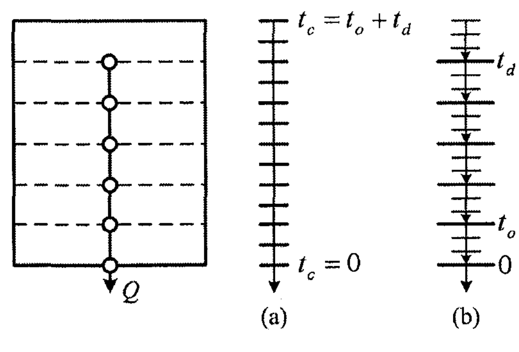

Figure 11 Conceptualized urban catchment

The conceptualized urban catchment represented in Figure 11 (a) has a time of concentration t_c defined as follows:

$$t_c = t_o + t_d \tag{4.4}$$

where t_o represents the overland travel time (min); t_d, the drainage network travel time (min).

The overland travel time t_o can be computed with empirical formulae (Viessman, 2003) or with a physically based formula (Morgali and Linsley, 1965) derived from the kinematic wave theory and formulated as follows:

$$t_o = \frac{(L_o\, n)^{0,6}}{S_o^{0,3}\; I^{0,4}} \tag{4.5}$$

where I represents the rainfall intensity (m/s); L_o, the overland flow pass length (m); S_o, the overland flow path slope (m/m); n, the Manning roughness factor.

The drainage network travel time is computed with the Manning formula expressed as follows:

$$t_d = \frac{L_d\, n}{A\, R_h^{0,66}\; S_d^{\;0,5}} \tag{4.6}$$

where A represents the wet area (m^2); R_h, the hydraulic radius (m); L_d, the drainage flow path length (m); S_d, the slope of the drainage flow path (m/m).

The conceptual urban catchment represented in Figure 11 (a) implicitly integrates the overland travel time and the drainage network travel time by the mean of a global time of concentration. A more realistic conceptual urban catchment is shown in Figure 11 (b). This conceptual urban catchment explicitly considers the overland travel time and the drainage network travel time.

4.2.2 IRH2 method

The assumptions of the IRH2 method, ensued from the conceptual urban catchment shown in Figure 11 (b) and from the linear system theory, are the followings:

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

2- the impulse response function of a sewer network is rectangular-shaped and ends at the drainage network travel time;

3- the overland travel time is the time difference between the end of the rainfall and the end of the direct runoff;

4- the drainage network travel time is the time taken by a water particle, entering into the sewer network at the far distant point of the network, to reach the outlet;

5- the runoff from impervious and pervious areas are independent phenomena.

Consequently, the runoff at the catchment outlet is given by the following double convolution product:

$$Q(t) = \int_0^t \Pi(t-\tau') \int_0^t \left(I(\tau) - dp(\tau)\right) u_{imp}(t-\tau) + \left(I(\tau) - f(t)\right) u_{per}(t-\tau)\, d\tau\, d\tau' \quad (4.7)$$

The convolution product, in discrete time, of the IRH2 method is formulated as follows:

$$Q(m) = Q_{imp}(m) + Q_{per}(m) \quad (4.8)$$

where

$$Q_{imp}(m) = \sum_{k=1}^{m \le m_r + m_o} \left(\sum_{j=1}^{m \le m_r} \left[\left(I(j,k) - dp(j,k) \right) u_i(m - j + 1) \right] \right) \Pi(m - k + 1)\, \Delta t \quad (4.9)$$

and

$$Q_{per}(m) = \sum_{k=1}^{m \le m_r + m_o} \left(\sum_{j=1}^{m \le m_r} \left[\left(I(j,k) - f(j,k) \right) u_p(m - j + 1) \right] \right) \Pi(m - k + 1)\, \Delta t \quad (4.10)$$

The impulsional response on impervious and pervious areas are defined as follows:

$$u_{imp}(m - j + 1) = K_c\ IMP\ A \frac{1}{t_o\ t_d} \qquad \text{for} \qquad 1 \le (m - j + 1)\Delta t \le t_o \qquad (4.11)$$

and

$$u_{per}(m - j + 1) = K_c\ (1 - IMP) A \frac{1}{t_o\ t_d} \qquad \text{for} \qquad 1 \le (m - j + 1)\Delta t \le t_o \qquad (4.12)$$

The Heaviside function Π is defined as follows:

$$\Pi(m - k + 1) = 1 \qquad \text{for} \qquad 1 \le (m - k + 1)\Delta t \le t_d \qquad (4.13)$$

where m_o represents the last index of the overland travel time vector; k, a time index.

The conditions $\Delta t / t_o \le 1$ and $\Delta t / t_d \le 1$ must be respected in equation (4.9) and (4.10). The notation $m \le m_r + m_o$ as upper limit of the summation indicates that the terms are summed for $k = 1$ to m when $m \leq m_r + m_o$, whereas for $m > m_r + m_o$, the summation is limited to $k = 1$ to $m_r + m_o$.

The infiltration capacity of the IRH2 method is computed with the Horton (1940) formula. Nevertheless, the Horton formula can be replaced by another infiltration model that describes the time evolution of the infiltration capacity.

The formulation of the IRH2 method is equivalent to the formulation of the IRH1 method for $t_d = 1$. Consequently, the IRH1 method is a special case of the IRH2 method. Moreover, the IRH2 method takes into account the spatial variability of rainfall. Indeed, new rainfall intensities can be chosen at each travel time step. Nevertheless, the IRH2 method does not simulate the overland flow routing and the pipe flow routing.

4.3 Scale effect study of the IRH1 and IRH2 methods

4.3.1 Synthetic catchments design

A series of synthetic urban catchments were designed in order to study the scale effect on the validity of the IRH1 and IRH2 methods. The synthetic catchments and sewer networks were designed by varying the value of the catchment area, the ratio of connected impervious areas and the overland time of concentration.

The effect of the catchment area on the accuracy of the IRH1 and IRH2 methods was studied with synthetic urban catchments having 5 different areas ranged from 60 ha to 480 ha.

The effect of connected impervious areas was studied by the mean of the drainage density defined as follows:

$$D = \frac{L_{pipe}}{A} \tag{4.14}$$

where D is the drainage density (m/ha); L_{pipe}, total length of pipe (m).

Sewer networks are divided into five class of density according to the range of the drainage density (Kamal and Bennis, 2005):

Class 1: $D < 30$

Class2: $30 \leq D < 45$

Class 3: $45 \leq D < 60$

Class 4: $60 \leq D < 75$

Class 5: $D \geq 75$

Drainage density varies from bellow 30 for the class 1 to over 75 for the class 5. A sewer network in class 1 has a low density whereas a sewer network in class 5 has a high density.

A direct correlation exists between the ratio of connected impervious areas and the drainage density. Indeed, the ratio of connected impervious areas reflects the density of urbanization and consequently, the density of the sewer network. Consequently, the synthetic urban catchments were elaborated by taking the drainage density equal to the ratio of connected impervious areas. Finally, five different drainage densities ranged from 25% to 85% were chosen in order to define five different sewer network configurations.

The effect of the time of concentration was studied by the mean of the slope which is the only independent parameter in the time concentration formula (4.4). Indeed, the flow path length is linked to the catchment area because large urban catchments have longer flow paths than small catchments. Moreover, the overland Manning roughness factor is linked to the ratio of impervious areas. Indeed, impervious areas have a lower Manning coefficient than pervious areas. Consequently, five different slopes ranged from 0,002 to 0,05 were used for the design of the synthetic urban catchments. Furthermore, the catchment slopes were assumed equal to the sewer pipe slopes.

The Table VI presents the physical characteristics of the synthetic urban catchments.

Table VI

Physical characteristics of the synthetic urban catchments

A (ha)	D (m/ha)	S_o (m/m)
60	0,25	0,002
120	0,4	0,005
240	0,55	0,01
360	0,7	0,03
480	0,85	0,05

The combination of the three physical characteristics conduct to 125 different catchments. In fact, the number of synthetic catchments can be reduced considering that the three physical characteristics are independent. Consequently, the effect of each characteristics was evaluated using 30 different catchments divided into two groups:

- Group 1: 25 catchments having A = 60 ha, D = 0,25; 0,4; 0,55; 0,70; 0,85 and S_o = 0,002; 0,005; 0,01; 0,03; 0,05 m/m
- Group 2: 5 catchments having A = 60; 120; 240; 360; 480 ha, D = 0,50 and S_o = 0,01 m/m

In order to have realistic synthetic urban catchments, the 30 different sewer networks were designed using the current conception practice (Brière, 1997), (Mays, 1999). Figure 12 shows two synthetic urban catchments with their sewer network.

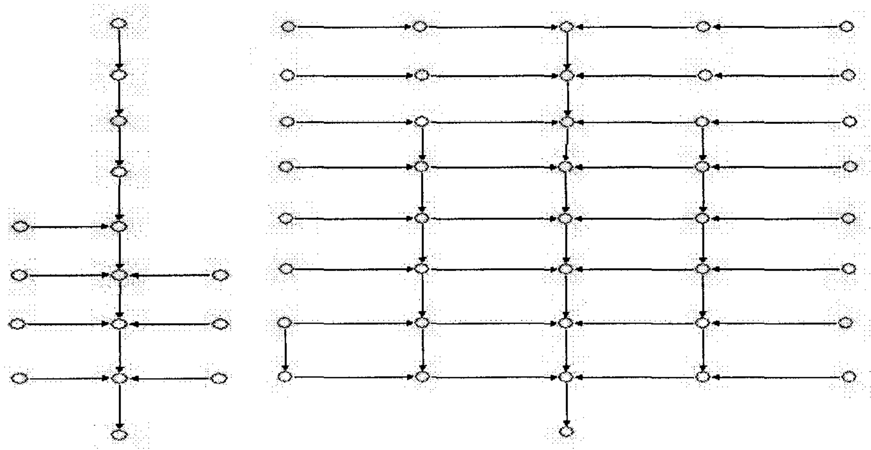

Figure 12 Two synthetic urban catchments with their sewer network of respectively class 1 and 5

The pipe diameters of the sewer networks were designed with a centered Chicago rainfall having a five year return period and a 5 minutes time step pattern. Moreover, the total length of pipe was computed for each catchment with the help of the drainage density. The length of pipe between two inlets was 100 meters for the 60 hectare catchments, and 300 meters for the other catchment areas.

4.3.2 Test procedure of the IRH1 and IRH2 methods

The scale effect study was conducted in three steps with the Chicago rainfall used for the design of pipes. Moreover, the contribution of pervious areas were not considered in order to avoid the interference of infiltration and the initial abstraction on impervious areas was set to zero.

The first step was to compute a reference runoff at the outlet of the synthetic urban catchments. To achieve this goal, the runoff of each subcatchments associated to an inlet was computed using the IRH1 method. Then, the runoff was routed into the sewer network with the full Saint-Venant equation incorporated in the EXTRAN module (Huber and Dickinson, 1988) of the XP-SWMM software.

The second step was to compute the catchment runoff with the help of the IRH1 and IRH2 methods in order to fit with the reference runoff computed with XP-SWMM. To achieve this goal, the time of concentration of the IRH1 method, the overland travel time and the drainage network travel time of the IRH2 method were calibrated in order to maximize the agreement between simulated and reference runoff hydrographs. The calibration was carried out with the simplex algorithm (Lagarias *et al.*, 1998) implemented in MATLAB, by maximizing the *Nash* criterion (Nash and Sutcliffe, 1970) defined as follows:

$$Nash = 1 - \frac{\sum_{i=1}^{m}\left(Q_{ref}(i) - Q_{sim}(i)\right)^2}{\sum_{i=1}^{m}\left(Q_{ref}(i) - \overline{Q}_{ref}\right)^2} \tag{4.15}$$

where $Q_{ref}(i)$ = reference flow at time i; $Q_{sim}(i)$ = simulated flow at time i; $\overline{Q}_{ref}$ = reference average flow.

A *Nash* of 1 indicates a perfect agreement between the simulated and the reference hydrograph. In urban hydrology, a good agreement between a simulated and a reference hydrograph is achieved when the *Nash* is greater than 0,7.

The differences between the simulated and the reference runoff hydrograph were also evaluated with the peak flow ratio, R_p, and with the peak flow synchronism, ΔT_p.

The final step was to study the evolution of the *Nash*, R_p and ΔT_p for the synthetic catchments of group 1 and 2.

4.3.3 Results

Firstly, the IRH1 method was applied to the 25 synthetic urban catchments of the group 1. The results are presented in Figure 13.

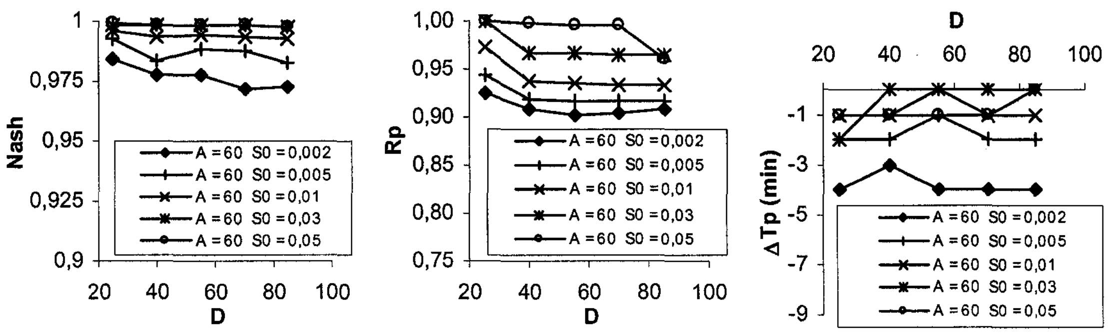

Figure 13 Variation of (a) *Nash*, (b) R_p, (c) ΔT_p with D for the IRH1 method

The *Nash*, R_p and ΔT_p criteria presented in Figure 13 (a), (b) and (c) have optimal values. Consequently, a good agreement was achieved between simulated and reference hydrographs. Moreover, Figure 13 (a), (b) and (c) show steady variation of the *Nash*, R_p and ΔT_p with the drainage density. Consequently, the drainage density has an insignificant effect on the IRH1 method accuracy. Nevertheless, Figure 13 (a), (b) and (c) show respectively a 2,5% decrease of *Nash,* a 10% drop of R_p and a maximal ΔT_p of 4 minutes with the decrease of the catchment slope. Consequently, the catchment slope has a slight effect on the *Nash* decrease but a significant effect on the R_p and ΔT_p decrease.

Secondly, the IRH2 method was applied to the 25 urban catchments of the group 1. The results are presented in Figure 14.

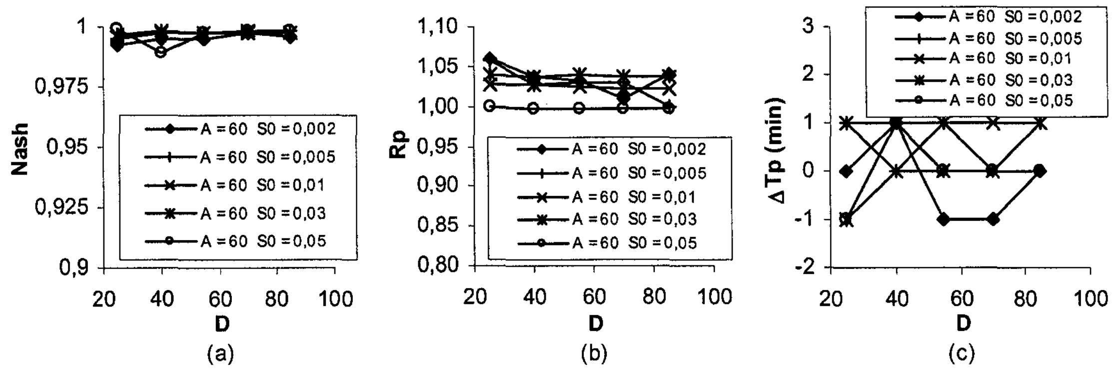

Figure 14 Variation of (a) *Nash*, (b) R_p and (c) ΔT_p with D for the IRH2 method

The *Nash*, R_p and ΔT_p criteria shown in Figure 14 (a), (b) and (c) have optimal values. Consequently, a good agreement was achieved between simulated and reference hydrographs. Moreover, the *Nash*, R_p and ΔT_p criteria experiences steady variation with the increase of the drainage density. Consequently, the drainage density has an insignificant effect on the accuracy of the IRH2 method. Figure 14 (a), (b) and (c) also show a *Nash* greater than 0,99; an R_p ranged between 1 and 1,05; and a ΔT_p lower than 1 minutes for the five catchment slopes. Consequently, the catchment slope has a slight effect on the accuracy of the IRH2 method.

Finally the IRH1 and IRH2 methods were applied to the 5 urban catchments of the group 2. The results are presented in Figure 15.

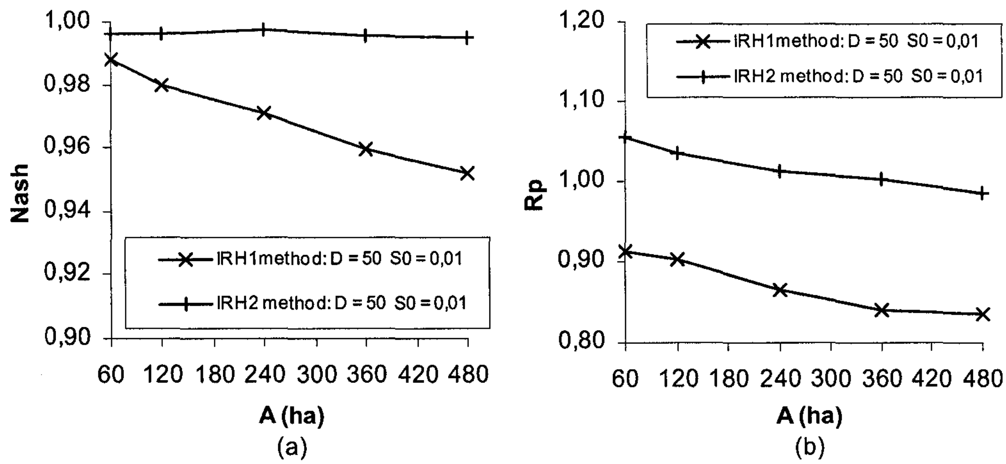

Figure 15 Variation of (a) *Nash* and (b) R_p with A for the IRH1 and IRH2 methods

The *Nash* and R_p criteria for the IRH1 method decreases respectively of 5% and 10% with an area increase of 420 hectares. Consequently, the catchment area has a significant effect on the accuracy of the IRH1 method. On the contrary, the *Nash* and R_p of the IRH2 method decreases respectively of 1% and 5% with an area increase of 420 hectares. Consequently, the catchment area has a lower effect on the accuracy of the IRH2 method than on the accuracy of the IRH1 method.

The scale effect study clearly has clearly shown an accuracy diminution of the IRH1 and IRH2 methods with a decreasing slope or an increasing catchment area. Nevertheless, the loss of accuracy is lower for the IRH2 method than for the IRH1 method, justifying the concept used for the formulation of the IRH2 method.

4.4 Validation of the IRH2 method

The validation of the IRH2 method was carried out with 5 rainfall events monitored in the subcatchment (1) of the Verdun borough (Canada), and 36 rainfall events

(Maximovic and Radojkovic, 1986) monitored in the urban catchments of East York (Canada), Malvern in Burlington (Canada), Sample Road and Fort Lauderdale in Broward County (USA), Gray Haven in Baltimore (USA), Saint Marks Road in Derby (Great Britain).

The physical characteristics of the seven selected catchments are given in Table VII.

Table VII

Physical characteristics of the seven selected urban catchments

Catchments	Area (ha)	Ratio of Impervious Area	Drainage Slope
Verdun	177	0,39	0,005
East York	155,8	0,38	0,007
Sample Road	23,6	0,17	0,002
Malvern	23,3	0,34	0,009
Gray Haven	9,4	0,45	0,009
Saint Marks Road	8,6	0,55	0,012
Fort Lauderdale	8,26	0,98	0,002

The ratio of impervious areas corresponds to the estimated ratio of directly and indirectly drained impervious areas for the Verdun and Saint Marks Road catchments and to the estimated ratio of directly drained impervious areas for the other catchments.

4.4.1 Calibration of the IRH1 and IRH2 methods

The four parameters needing a calibration for the IRH1 method are *IMP*, t_c, the initial abstraction depth *Dp*, and the initial infiltration capacity f_o. The four parameters needing a calibration for the IRH2 method are *IMP*, t_o, *Dp* and f_o. The asymptotic infiltration parameter f_∞ and the decay rate *K*, of the Horton formula, are evaluated without calibration. A default value (Maidment, 1993) was chosen for f_∞ and *K*. Moreover, t_d, for the IRH2 method, is computed with formula (4.6) using a Manning roughness of 0,014, the longest drainage flow path length and the weighted average of pipe diameters along the flow path.

The IRH1 and IRH2 methods were calibrated following the procedure described in Figure 16.

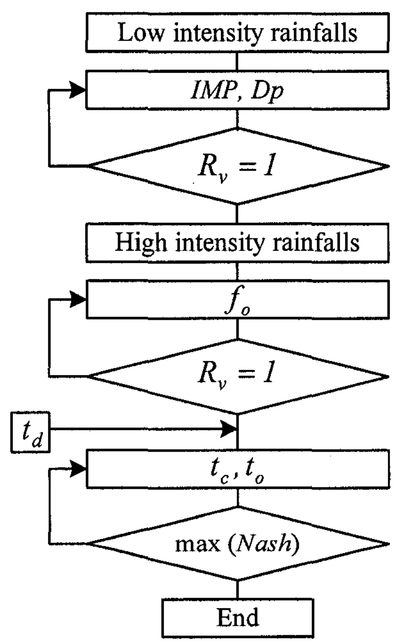

Figure 16 Calibration procedure of the IRH1 and IRH2 methods

The calibration procedure starts with the calibration of *IMP* using a low intensity rainfall event which only makes contribute the impervious areas. The initial value of *IMP* is given by the study of the catchment occupation. The final value of *IMP* is estimated by balancing the ratio of the simulated over the measured runoff volume. This ratio is given by the following formula:

$$R_v = \frac{\sum_{i=1}^{m} Q_{sim}(i) \cdot \Delta t}{\sum_{i=1}^{m} Q_{meas}(i) \cdot \Delta t} \tag{4.16}$$

Then, a default value of *Dp* is estimated using the SCS method (Maidment, 1993). The initial value of *Dp* is adjusted to fit the beginning of the simulated runoff, to the beginning of the measured runoff. The modification of *Dp* alters the ratio of simulated over measured runoff volume. Thus, the coefficient *IMP* must be readjusted using the following formula:

$$IMP_{final} = IMP_{initial} \frac{P - Dp_{initial}}{P - Dp_{final}} \tag{4.17}$$

where *P* represent the cumulative rainfall.

The second step is the calibration of f_o with a high intensity rainfall. Indeed, pervious areas contribute during high intensity rainfalls. A default value of f_o is chosen to start the calibration. The final value of f_o is estimated by balancing the ratio of the simulated over the measured runoff volume.

The third step is the calibration of t_c and t_0. An initial value of t_c is given by equation (4.4). Then, a final value of t_c and t_0 is estimated by maximizing the *Nash* coefficient between the simulated and the measured runoff hydrograph. The *Nash* is automatically maximized using the simplex algorithm implemented in MATLAB.

The calibration of the IRH1 and IRH2 method was carried out with two monitored rainfall events per catchment. The calibrated parameters for each catchments are showed in Table VIII.

Table VIII

Calibrated parameters of the IRH1 and IRH2 methods

Catchments	A (ha)	IMP (%)	t_C (min)	t_o (min)	t_d (min)	Dp (mm)	f_o (mm/h)	f_∞ (mm/h)	K (h^{-1})
Verdun	177,0	0,41	37	28	13	0,7	55	15	2
East York	155,8	0,39	30	17	17	1,0	60	15	2
Sample Road	23,6	0,20	35	24	14	0,5	170	15	2
Malvern	23,3	0,35	11	7	6	0,5	50	15	2
Gray Haven	9,4	0,43	15	13	3	0,4	80	15	2
St. Mark Road	8,6	0,28	24	16	12	1,0	40	15	2
Fort Lauderdale	8,3	1,00	19	10	10	0,2	undefined		

The values of *IMP* are close to the percentage of directly drained impervious areas validating the assumption that *IMP* corresponds to the ratio of connected impervious areas.

4.4.2 Application of the IRH1 and IRH2 methods

The IRH1 and IRH2 methods were applied to the 27 rainfall events that have not been used for the calibration procedure. The Table IX shows the *Nash*, R_v, R_p and ΔT_p values after simulation of the 27 monitored runoff events with the IRH1 and IRH2 methods.

Table IX

Nash, R_v, R_p and ΔT_p value after simulation of the 27 monitored runoff events with the IRH1 and IRH2 methods

Events	Rainfall depth (mm)	Runoff depth (mm)	Rainfall duration (min)	I_{max} 5 min (mm/h)	*Nash*		R_v	R_p		ΔT_p *(min)*	
					IRH1	IRH2		IRH1	IRH2	IRH1	IRH2
V. 13-10-99	20,80	9,95	300	21,6	0,50	0,53	1,00	1,16	1,31	5	0
V. 23-08-00	10,00	8,60	240	14,4	0,76	0,79	1,01	1,03	1,07	10	0
V. 22-06-01	10,10	5,90	250	20,4	0,75	0,77	1,05	0,89	1,06	15	5
E. Y. 13-08-76*	5,58	2,15	20	58,0	0,57	0,83	1,15	0,69	1,04	5	0
E. Y. 01-09-76	5,27	1,75	59	19,7	0,47	0,10	0,96	0,84	1,05	10	10
E. Y. 25-06-77*	17,14	8,63	82	61,0	0,80	0,93	0,93	0,84	1,05	15	5
E. Y. 10-08-77*	11,13	5,23	35	54,8	0,64	0,90	0,95	0,65	0,89	15	5
S. R. 29-05-76*	52,00	12,14	168	85,9	0,74	0,69	0,91	0,58	0,72	21	13
S. R. 29-05-76	13,30	2,20	132	28,0	0,87	0,87	1,17	1,17	1,15	5	3
S. R. 04-06-76	9,96	2,20	72	28,9	0,84	0,90	0,86	0,69	0,86	11	6
S. R. 07-06-76	16,96	3,44	197	52,7	0,78	0,86	0,96	0,86	1,02	16	8
M. 23-09-73*	9,14	3,25	126	31,2	0,91	0,92	0,94	0,79	0,94	1	2
M. 05-05-74	7,62	2,22	164	7,6	0,90	0,89	1,12	1,05	1,10	3	1
M. 28-09-74	15,24	4,40	87	24,4	0,80	0,75	1,17	1,15	1,35	4	1
M. 20-11-74	4,57	1,46	56	11,7	0,69	0,67	0,73	0,66	0,59	29	4

Table IX (continued)

Events	Rainfall depth (mm)	Runoff depth (mm)	Rainfall duration (min)	I_{max} 5 min (mm/h)	*Nash*		R_v	R_p		ΔT_p *(min)*	
					IRH1	IRH2		IRH1	IRH2	IRH1	IRH2
G. H. 05-06-63*	55,88	37,94	53	13,1	0,95	0,95	0,96	1,03	1,06	3	3
G. H. 10-06-63*	50,80	37,09	53	103,0	0,90	0,92	0,86	0,83	0,84	4	3
G. H. 20-06-63*	37,08	15,27	72	81,1	0,81	0,81	1,24	1,08	1,10	2	2
G. H. 29-06-63*	30,23	14,85	175	78,6	0,83	0,84	1,04	1,13	1,17	3	3
S. M. 02-10-75	6,97	1,52	192	6,2	0,86	0,82	1,09	0,91	0,88	7	5
S. M. 15-11-75	5,64	1,64	139	10,8	0,79	0,85	0,75	0,68	0,74	2	3
S. M. 22-09-76	7,05	1,45	352	12,6	0,86	0,91	1,15	0,79	0,87	2	1
S. M. 25-09-76	13,63	3,49	290	14,2	0,86	0,90	1,03	0,80	0,81	6	4
F. L. 20-06-75	7,23	7,14	218	23,8	0,82	0,84	0,99	0,80	0,91	3	1
F. L. 23-06-75	44,34	44,61	314	91,0	0,94	0,91	0,99	1,12	1,19	4	3
F. L. 04-07-75	22,13	21,39	152	70,9	0,80	0,82	1,02	0,76	1,00	7	6
F. L. 05-07-75	15,48	19,81	177	74,2	0,72	0,81	0,78	0,67	1,03	0	3

*Events for which impervious and pervious areas contribute

Results presented in Table IX show a *Nash* coefficient above 0,7 for more than 80% of simulated events. Consequently, a good agreement is achieved between simulated and measured runoff hydrographs. Moreover, the high values of *Nash* indicates that the IRH1 and IRH2 methods describe accurately the runoff process on urban catchments. The low *Nash* values are explained by the lag existing between simulated and measured runoff hydrographs. The error on runoff volume is less than 15% for 78% of simulated events. Therefore, the two models give an accurate estimation of the runoff volume. The error of estimation for the peak flow is less than 15%, for 63% of the events simulated with the IRH1 method, and for 77% of the events simulated with the IRH2 method. Consequently, the IRH2 method has better estimated the peak flow than the IRH1 method.

A t-test was conducted on the average *Nash,* R_p of the IRH1 and IRH2 methods in order to identify a significant difference of accuracy between both methods. The Table X presents the average and standard deviation for the *Nash* and R_p data of the IRH1 and IRH2 methods.

Table X

Average and standard deviations of *Nash*, R_p for the IRH1 and IRH2 methods

	Nash		R_p	
Model	Average	Standard deviation	Average	Standard deviation
IRH1 method	0,78	0,12	0,88	0,18
IRH2 method	0,81	0,17	0,99	0,17

The t-test has not been able to detect a significant difference of average *Nash*, between the IRH1 and IRH2 methods. Consequently, the agreement between simulated and measured runoff hydrographs is equivalent, as regard *Nash*, for the IRH1 and IRH2 methods. On the contrary, the t-test has revealed a significant difference of average R_p between the IRH1 and IRH2 methods. Indeed, the average R_p of the IRH2 method is 10% closer to 1 than the average R_p of the IRH1 method. Consequently, the IRH2 method gives a better estimation of the peak flow than the IRH1 method.

4.5 Conclusion

A new improved rational hydrograph method, named IRH2 method was derived from the conceptual representation of an urban catchment operation. The originality of the IRH2 method is based on the explicit consideration of the overland runoff, using the overland travel time, and of the translation process of runoff hydrographs in sewer networks, using the drainage network travel time. The IRH2 method is an extension of the IRH1 method which uses a global time of concentration to represent catchment runoff. The theoretical study of the IRH1 and IRH2 methods, carried out with the help of 30 synthetic urban catchments, has shown that the accuracy of the IRH1 and IRH2 methods deteriorates with the increase of catchment area and with the decrease of catchment slope. Moreover, the accuracy of the IRH1 method deteriorates faster than the accuracy of the IRH2 method. The validation of the IRH2 method was carried out with 41 rainfall events monitored in 7 North American and European urban catchments. The IRH1 and IRH2 methods have accurately simulated runoff hydrographs. Moreover, the IRH2 method has given a better estimation of the peak flow than the IRH1 method.

CHAPITRE 5

ARTICLE 3 : « MODÈLE DE LESSIVAGE DES MATIÈRES EN SUSPENSION EN MILIEU URBAIN »

5.1 Introduction

Les municipalités cherchent de plus en plus à minimiser les impacts liés aux rejets de polluants au milieu naturel en interceptant la masse maximale de pollution. La capacité de traitement des eaux pluviales dans un réseau d'assainissement unitaire est limitée, en général 2 à 10 fois le débit de temps sec, alors que le ruissellement en temps de pluie peut dépasser de 100 fois ce débit (Bennis *et al.*, 2001). La connaissance du pollutogramme associé à un événement pluvieux apparaît indispensable afin de maximiser l'interception de polluants dans le cas d'une gestion en temps réel du réseau ou afin de connaître la quantité de polluants rejetés au milieu naturel dans le cas d'une gestion en temps différé.

En général, les matières en suspension (MES) sont le vecteur principal de la pollution des eaux de ruissellement en milieu urbain (Chebbo et Bachoc, 1992), (Jack *et al.*, 1996). Ainsi, l'estimation des quantités de matière en suspension lessivées à l'exutoire d'un bassin versant permet d'estimer le niveau de pollution des eaux de ruissellement (Tsihrintzis et Hamid, 1997).

Le cycle des particules solides sur un bassin urbain est un processus complexe. Ce cycle comprend une phase d'accumulation des particules solides sur le bassin en période de temps sec et une phase de lessivage des particules solides en temps de pluie. Les modèles de lessivage des eaux de ruissellement s'attachent à décrire séparément ou conjointement ces deux phases avec un niveau de complexité variable.

Les modèles d'accumulation les plus utilisés sont asymptotiques. Ils dérivent le plus souvent du modèle exponentiel de Alley (1981) employé dans le logiciel SWMM (Storm Water Management Model) mais peuvent également être linéaires (Servat, 1984).

Le lessivage des sols en milieu urbain est un phénomène complexe et difficile à appréhender qui a été principalement modélisé à l'aide d'approches conceptuelles. Le modèle conceptuel le plus simple est le modèle de « rating curve » (Huber et Dickinson, 1988), (Temimi et Bennis, 2002) qui relie la charge de matière en suspension au débit de ruissellement. Le modèle exponentiel du logiciel SWMM (Huber et Dickinson, 1988) calcule la charge de matière en suspension lessivée à l'aide du débit de ruissellement et de la masse de particules accumulées. Le modèle de lessivage du logiciel STORM (US Army Corps of Engineers, 1977), le modèle développé par Moys *et al.* (1988) pour le logiciel MOSQUITO, le modèle de lessivage du logiciel HSPF (Bicknell *et al.*, 1997) et le modèle développé par Zug *et al.* (1999) pour le logiciel HORUS prennent également en compte la masse de particules accumulées. Des développements récents ont conduit à l'élaboration de modèles déterministes (Deletic *et al.*, 1997), (Deletic *et al.*, 2000). Ces modèles présentent l'avantage de décrire les phénomènes physiques impliqués dans le lessivage des sols. En revanche leur utilisation réclame des données qui sont rarement disponibles ou coûteuses à obtenir. Les charges de matière en suspension peuvent également être calculées à l'aide de modèles statistiques (Driver et Troutman, 1989). Malheureusement, ces modèles sont uniquement valides pour les sites où ils ont été développés (Jewell et Adrian, 1978).

Cet article présente un nouveau modèle conceptuel de lessivage des MES en milieu urbain. Le modèle proposé sera validé à l'aide de mesures de MES effectuées sur le terrain. Les résultats du modèle proposé seront comparés aux résultats du modèle exponentiel et du modèle de « rating curve ». De plus, les paramètres du modèle proposé feront l'objet d'une analyse de sensibilité.

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