Chapter 6

On-Site Runoff Control For Industrial Parks: A Case Study

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The proposed development of an industrial park in Québec City has been analyzed to evaluate the feasibility of systematic on-site detention and the possible benefits it could provide. At first, SWMM simulations for different return periods were completed to evaluate the conditions before and after development, without detention. Various scenarios for on-site detention were then investigated, using different types of standardized facilities (parking lots or basins) and different detention facilities throughout the park. Expected inconvenience caused by the detention in parking lots and the impact of different detention policies were evaluated by considering, with subsequent calculations, the level and duration of the surcharge as a function of the specified maximum discharge released by unit area. A simplified approach was also elaborated to provide a design aid for the local authorities and to facilitate implementation. Using pseudo-continuous simulations with extreme rainfall events extracted from a 28-year record, regression equations for the peak discharges and runoff volumes were developed as a function of the contributing area for

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different return periods. Combined with other equations relating the volume and the flooded area to the depth of ponding, these equations can be used to rapidly evaluate the applicability of different detention scenarios to a specific subdivision.

6.1 Introduction

The high degree of imperviousness which characterizes industrial areas can have a significant impact upon peak discharges and runoff volumes after the development is completed. Both regional and on-site detention have been used to minimize undesirable hydrological effects. Although it is generally accepted that regional facilities are theoretically preferable in many cases, consideration of institutional aspects and costs can force the selection of on-site detention as the optimum runoff control technique.

The fundamental objective of stormwater management is essentially to minimize the effects of hydrologic changes caused by urbanization. Historically, in order to achieve this objective, the typical design constraint for detention basins was to formulate simple rules stipulating zero runoff increase above pre-development levels, based on a high intensity storm (most often for a return period of 100 years). With time and experience however, it became evident that little control was provided for smaller floods and that this approach could in fact produce increased downstream erosion, as formative discharges for receiving streams are more likely to correspond to a return period of approximately two years (Leopold, 1968). Moreover, as it is sometimes difficult to agree on what exactly are the pre-development conditions, some local authorities have preferred to recommend a fixed post-development release rate for a given land use. Although this approach is surely the easiest to implement, it should be recognized that the aforementioned fundamental objective will, most likely, not be met by applying simplistic regulations to every case.

The proposed development of an industrial park in Québec City has been analysed with a global approach to evaluate the feasibility of systematic on-site detention and the possible benefits it could provide. Detention in parking lots or in small grassed basins was considered and practical aspects were taken into account to elaborate different detention alternatives. Very often, municipal regulations recommend a fixed maximum release rate per unit area. For the specific case of parking lot detention, this maximum release rate can have a significant impact on

level and duration of ponding, which could in some cases become intolerable. The impact of different release rates and their relation with pre-development conditions must therefore be investigated, in order to obtain a realistic design constraint.

Finally, recognizing that implementation of the proposed detention facilities could be a problem in practice, a simplified approach to evaluate the necessary storage to meet the design constraints was developed. Simple equations relating runoff discharges and volumes to impervious areas are very useful for local authorities, to help them work with developers, and to rapidly evaluate the applicability of different detention scenarios to a specific subdivision.

6.2 Site Description

The industrial park Armand-Viau is located in the northern area of Québec City and has a total area of 76.9 ha for its southern sub-basin (86% of the surface being pervious) and 25.7 ha (100% pervious) for its northern basin. The two main access roads with their infrastructures are already built and only five lots are presently occupied (all of them in the southern sub-basin), the remaining parts being in a relatively natural state with open fields and light bush. The entire southern sub-basin drains toward the only exit on Armand-Viau street, with slopes averaging 2.5%.

For future conditions, secondary streets will be 12 m wide and 85% of the surface will be impervious (only 15% minimum with grass, the other surfaces being parking lots or roofs).

6.3 Conceptual Approach

Following a preliminary investigation which concluded that one or two global basins were not economically feasible due to the presence of rock, this analysis focused essentially on a runoff control exclusively by on-site detention. Two major types of detention facilities were considered: detention in the parking lots and detention in a small grassed basin, most likely to be located near the lateral property line. In the specific case of the parking lots, different layouts for the location of the grate could be defined, the most common situation (and most efficient in terms of interception) being with the grate located in the middle of the parking lot.

Typical layouts for a particular lot are shown in Figure 6.1. Three distinct parking areas can usually be defined for an industrial or a

Figure 6.1
Typical physical layouts for industrial lots.

commercial area: the first area identified in the figure is usually reserved for the clients, the second one is often allocated for the employees and the last one, in the backyard, is usually for truck loading and unloading. Practically speaking, the owner will prefer to have the ponding located away from where the clients would park, i.e. in parts 2 or 3. Due to the relatively steep topography in the present case, it would not be very practical to drain entire parking lots toward only one grate inlet, as it would impose undue limitations on the site planning.

For the grate inlets, the discharges are controlled by a special diamond-shaped orifice, developed in Canada, installed directly on the outlet pipe, in the catchbasin. Standard capacities with a 1.22 m head are 20 l/s, 28.6 l/s and 36.8 l/s: the actual release rates from each lot were established with a combination of these standard capacities. For the grassed basins, the diameter of the outlet pipe (from 100 mm to 250 mm) controls the release rate.

The SWMM hydrologic model took into account a total of 85 detention facilities just for the southern sub-basin, i.e. one on each proposed lot. The northern basin, with its independent outlet, was not analysed with detention facilities.

6.4 Rainfall Data Base

6.4.1 Synthetic Design Storms

The general behavior of the system was first analysed with synthetic storms developed for the Québec City airport weather station according to the procedures recommended by Hogg (1980, 1982). The design storms for different return periods are based on the total precipitation as given by the IDF curves, which is distributed in time according to a cumulative rainfall pattern developed by Hogg, specifically for the southern part of the Québec province.

6.4.2 Actual Rainfall Series

Meteorological data for the Québec City airport weather station were also analyzed to obtain major historic storms for a 28-year period (1961 to 1991; the data for 1975, 1982 and 1985 were missing). The hourly rainfall data obtained directly from Environment Canada were first examined with the RAIN block of SWMM, with a minimum inter-event time of three hours, in order to find for each year the rainfall event which had the highest hourly intensity. Assuming that these selected rainfall events were the most critical for the relatively small impervious areas being considered, the actual rainfall charts were then obtained from Environment Canada and a rainfall series was developed with a 5 minute time step. Table 6.1 shows selected characteristics of these major historic storms. It should be noted that, in the particular case of industrial parks and commercial areas, the antecedent moisture conditions are not critical due to the high percentage of imperviousness. Of course, this would not be the case if a rural area or a low-density residential area were being studied.

Following the procedure described by Walesh (1979), a pseudo-continuous simulation using the rainfall series was then completed for a small, typical impervious area, in order to identify which rainfall events could be associated with specific return periods. The pseudo-continuous process consists of the following steps:

1. Input the selected hyetographs to the event model and obtain corresponding direct runoff hydrographs;
2. Obtain annual peak discharges or annual maximum volumes from the model output and perform statistical analyses resulting in a discharge-probability or volume-probability relationship.

Table 6.1
Selected characteristics of major historic storms.

Event # 1	Date	Duration (hrs)	Total P (mm)	Max. int. for 1 hr (mm/hr)	Precipitation in the preceding 5 days (mm)
1	Sept. 15 - 61	0.58	15.3	15	34.1
2	Sept. 10 - 62	6.0	63.7	25.4	30.2
3	July 29 - 63	6.0	37.9	20.6	8.5
4	Aug. 31 - 64	3.5	27.6	18.8	23.8
5	July 14 - 65	0.33	17.8	17.8	5.4
6	Nov. 10 - 66	5.0	23.3	12.4	25.2
7	Aug. 10 - 67	1.0	30.4	20.8	8.3
8	July 15 - 68	0.5	11.2	11.2	0
9	Aug. 24 - 69	3.83	35.2	20.8	11.5
10	June 5 - 70	3.0	23.9	14.7	12.1
11	Aug. 3 - 71	1.33	28.0	26.7	26.2
12	July 1 - 72	2.25	33.0	16.8	10.4
13	Aug. 10 - 73	6.08	49.9	32.0	46.2
14	July 8 - 74	6.0	27.5	18.3	32.6
15	June 14 - 76	1.67	24.1	19.3	33.4
16	Sept. 1 - 77	0.67	13.8	13.8	12.6
17	June 19 - 78	4.25	34.6	19.3	8.2
18	June 16 - 79	2.83	78.0	48.7	0
19	Sept. 20 - 80	2.5	58.8	29.5	4.2
20	June 22 - 81	8.0	60.3	19.6	0.2
21	July 5 - 83	1.33	13.7	12.9	7.7
22	July 15 - 84	10.5	50.1	12.2	40.1
23	Aug. 21 - 86	1.67	25.2	20.0	0.2
24	July 24 - 87	2.83	31.1	24.6	9.6
25	July 10 - 88	5.0	26.4	17.8	2.6
26	June 27 - 89	2.0	41.5	23.3	1.1
27	July 19 - 90	1.0	27.8	27.1	14.6
28	July 7 - 91	0.83	19.8	19.5	6.2

The results for the pseudo-continuous simulations are shown on Figure 6.2, where it can be seen that the same event can have a different return period for discharge and volume. For example, the 1989 event has

a return period evaluated to be one in ten years for the discharge and approximately one in five years for the volume. On the other hand, this does not mean necessarily that the 1981 event, which has a 10-year return period if we consider the runoff volume, is more critical than the 1989 event for the detention analysis; the 1981 event was in fact characterized by a relatively long duration with low intensity rainfall which did not produce peak discharges larger than the outlet capacities. It is therefore the combination of discharge and volume which must be considered in the selection of design events.

For the sake of comparison, the discharges and volumes generated by the synthetic storms are also indicated on the figure and we can see that use of synthetic storms produced acceptable peak discharges when compared with real storms, whereas volumes are underestimated. This reinforces the point that problems involving detention and volume estimation should be analysed most preferably with real rainfall events, as some independent studies have pointed out (Marsalek, 1978; Geiger, 1984; Arnell, 1987).

6.5 Simulated Conditions Before and After Development

Table 6.2 shows in its first two columns the peak discharges for the conditions before and after development, without any runoff control. The simulations for the future conditions were completed with the design mode activated in SWMM, so that the under-capacity of the pipes already in place under the two main access roads would not affect the actual discharges and volumes at the outlet. Different points can be made from these results:

1. the discharge increase is more pronounced for frequent events, the ratios between the discharges before and after development being 5.3 for the 2-year return period and about 2.1 for the most intense event on record;
2. the ratio between the discharges for the 2-year return period and the 10-year return period for the conditions after development is significant (1.9); the collector which will have to be built from the exit of the park to the river outfall being approximately 1.2 km long, significant cost savings could be expected if the minor system was designed to carry only the discharge for the 1 in 2 year return period.

Figure 6.2 Statistical analysis for pseudo-continuous simulations (for 500m² surface, 85% impervious).

Table 6.2
Simulated peak discharges for before and after development conditions, with
different on-site controls (values at outlet of southern sub-basin).

Rainfall Event	Q^1 (m ³ /s)	Q^2 (m ³ /s)	Q^3 (m ³ /s)	Q^4 (m ³ /s)	Q^5 (m ³ /s)	Q^6 (m ³ /s)
AES 1/2	1.57	8.41	7.88	7.14	-	5.53
AES 1/5	2.66	-	-	8.55	-	-
AES 1/10	3.54	16.29	10.44	9.09	-	8.64
AES 1/100	6.32	27.05	-	-	-	-
89-06-27	-	-	-	8.62	7.55	-
79-06-16	8.52	17.66	9.67	9.16	8.0	9.31

where:

- Q^1 : Actual conditions
- Q^2 : Future conditions - no control (obtained with design mode)
- Q^3 : Future conditions -existing sewers and future conduits designed for 2-year return period
- Q^4 : On-site control with grate inlets (average release rate: 103 l/s/ha)
- Q^5 : On-site control with grate inlets (average release rate: 54 l/s/ha)
- Q^6 : On-site control with grassed basins
- AES: Synthetic design storm (Hogg, 1980)

6.6 Simulations with Detention

6.6.1 Description of Computer Runs

Different computer runs (using synthetic design storms and selected real rainfall) were then completed to evaluate the system responses under different scenarios (see Table 6.2 for results):

- for all the runs with detention, the pipes already in place were left as they are, whereas the future pipe system were designed with a 2-year return period (after development);
- for the fourth and fifth runs, detention facilities in parking lots were added for each lot. The average maximum release rates, obtained by using a combination of standard restricting devices (with 1, 2, 3 or 4 grates depending on the surface areas), were 103 l/s/ha and 54 l/s/ha in the second and third run respectively;

- in the next simulation, trapezoidal grassed basins with a total length of 25 m (lateral slopes of 2V:1H) were used in place of the restriction devices in catchbasins. The diameters for the pipe outlets were set at 250 mm for a lot area greater than 0.85 ha and 200 mm for areas smaller than 0.85 ha;
- other supplementary simulations, not shown in the table, were carried out to evaluate the impact of having only certain portions of the park equipped with detention facilities.

6.6.2 Discussion

The first point that can be made is that a design with a 2-year return period for the future pipe system produces a 36% reduction in peak discharge if we consider a 10-year return period (10.44 m³/s as compared to 16.29 m³/s with the design mode). Considering the 1% slope which could be available for the 1.2 km long collector to be built, the diameter could then be reduced from 2100 mm to 1800 mm.

If we now examine the discharges for the 2-year return period in the second and third column in Table 6.2, it can be seen that the discharge produced with the design mode (column 2) is slightly higher than the discharge with the evaluation mode with the future pipe system designed with a 2-year return period (column 3). This anomaly could be explained by the fact that certain pipes already in place will not have sufficient capacity to carry the anticipated 2-year discharge, due to changes in subdivisions and secondary streets layout. The surcharges produced would however be minimal.

Comparing the results in columns 4 and 5 can provide an indication of the impact of selecting different minimum release rates for detention in parking lots: a reduction by approximately 50% of the allowable release rate (from 103 l/s/ha down to 54 l/s/ha) produced only a mean 12.5% reduction in peak discharge for the entire southern sub-basin, if we consider the two real storms (that of June 1989, which has a return period of 1/10 years -see Table 6.1- and that of June 1979, which is the most severe storm in the data base). It should be remembered that runoff from streets, which can represent 8 to 15% of the total area, was not controlled in this analysis; that can explain in part why a significant reduction in allowable discharge does not have such an impact on total peak discharge.

This slight reduction in peak discharge has however important secondary effects. First of all, the level and duration of ponding will

obviously be increased if a more severe maximum release rate is applied. With a release rate of 103 l/s/ha and the most intense rainfall on record, calculations indicate that the maximum depth of ponding would be less than 150mm, lasting about fifteen minutes. On the other hand, Figure 6.3 shows the effect of using a release rate of 54 l/s/ha for a typical lot, when the June 27th, 1989 storm is considered; it can be seen that unacceptable level and duration of ponding are then created. Secondly, with a lower release rate and the corresponding longer duration of ponding, it is likely that smaller discharges (which will anyhow be close to the discharges produced with a less restrictive release rate) will last longer, which could have in turn a direct effect on erosion.

Column 6 in Table 6.2 gives the results with grassed basins installed on each lot as detention facilities. The discharges produced with this arrangement are comparable to those computed with grate inlets, except for the 2-year return period, where the computed discharge is significantly lower with grassed basins. This can be explained by the fact that the restriction imposed by the pipe outlet is more severe than the restriction imposed by a controlled grate inlet for lower discharges.

6.6.3 Critical Evaluation of Design Constraints

Ideally, the discharges after development should be kept at the same levels as the discharges before development, for a range of return period. In this particular case, this would mean that the maximum release rates for return periods of 2, 10 and 100 years should be respectively 20 l/s/ha, 35 l/s/ha and 82 l/s/ha. On the other hand, simulations indicate that, without control, the unit release after development for the same return periods would be 109 l/s/ha, 212 l/s/ha and 352 l/s/ha.

Considering these numbers and the analysis for different maximum release rates, it does not seem practical for this case to reduce post-development discharges to pre-development levels using only parking lot detention: not only the level, duration and recurrence of ponding (which would then happen more often) would likely be intolerable to the parking users but, more importantly, this would imply for example that an entire one ha lot would have to drain towards a single grate inlet. This would likely impose unnecessary constraints for site planning and would also increase the chances of clogging the grate inlets (see Wisner, 1985). Therefore, a global basin which would control runoff from the streets and the difference in discharges for the conditions before and after development would be needed. The option with individual grassed basins could

Figure 6.3
Depth and duration of ponding - release rate of 54 l/s/ha.

also be used, as a somewhat more severe control is possible in this case without nuisance for the parking lot users; it should however be recognized that maintenance costs could be more important in that situation.

In conclusion, if it could be ascertained by subsequent analyses that significant erosion and pollution in the receiving stream would result from the development of the industrial park, a major basin to control these parameters would therefore have to be recommended. Otherwise, a maximum release rate corresponding to the post-development discharges for a 2-year return period (i.e. zero ponding on parkings), as controlled by on-site detention facilities without a major basin, appears to be a reasonable design constraint.

6.7 Simplified Approach

For preliminary estimates and analysis of on-site detention facilities, a simplified approach has also been developed to assist the local

authorities with their relations with developers. The simple steps to obtain a preliminary design are:

1. measurement of the total impervious areas for which runoff will be controlled;
2. computation of runoff discharges and volumes;
3. computation of the storage needed to limit the release rate to the selected design constraint;
4. analysis of the impacts the selected detention can have for the users, with the help of figures relating flooded volumes and areas to depth of ponding;
5. selection of the most appropriate detention facilities, considering the acceptable level of nuisance.

For the second step, discharges and volumes can be rapidly evaluated with regression equations which have been developed for a percentage of imperviousness of 100% and results of simulations with actual rainfall of given return periods, as indicated by the statistical analysis involving the pseudo-continuous simulations. For this particular case, these equations take the following simple forms (A being the impervious area in m²):

Discharge (L/s)

$$\begin{aligned} Q_{1/2 \text{ years}} &= 0.0124A \\ Q_{1/5 \text{ years}} &= 0.022A \\ Q_{1/10 \text{ years}} &= 0.0232A \\ Q_{1/100 \text{ years}} &= 0.0418A \end{aligned}$$

Volumes (m³)

$$\begin{aligned} Q_{1/2 \text{ years}} &= 0.0272A \\ Q_{1/5 \text{ years}} &= 0.0406A \\ Q_{1/10 \text{ years}} &= 0.0594A \\ Q_{1/100 \text{ years}} &= 0.0761A \end{aligned}$$

These equations can be used to determine the following parameters:

- the maximum discharge to be released toward the minor system (in this case the discharge corresponding to a 2-year return period),
- maximum discharges to control (1-10 or 1-100 year return periods), and
- volumes corresponding to the maximum discharges to be controlled.

Note: S is the necessary volume and
I is the runoff volume to be controlled.

Figure 6.4
Approximate methods for storage determination (after Boyd, 1981).

Using these computed values, the storage necessary to meet the design constraint can then be calculated with approximate equations which does not involve tedious routing calculations. Many such equations exist (McEnroe, 1992) and most of them relate ratios of the peak discharge to the maximum release rate and the runoff volume to the necessary storage to meet the maximum release rate. Two equations, reported by Boyd (1981) and reproduced in Figure 6.4, have been suggested as appropriate for the type of detention facilities analyzed here.

The storage volume being now determined, the depth of ponding corresponding to this detention volume can then be determined from a graph such as Figure 6.5. Finally, with a second graph relating the depth to the flooded area (Figure 6.6), the designer has all the necessary information for a preliminary analysis of the detention facility.

Figure 6.5
Available storage in parking lot as a function of depth of ponding - case of centrally located grate inlet.

6.8 Conclusions

This case study was presented essentially to illustrate that the definition of design constraints is not as straightforward as it may appear, and that a case-by-case analysis should be recommended instead of applying arbitrarily to all cases a simple zero-runoff-increase criteria. This approach is not new as it has been included for some time in drainage policies of some municipalities in Canada: this is not however as generalized as it should be.

Apart from the obvious case where the maximum release rate from a development is imposed by the capacity of existing pipes, definition of acceptable design constraints should as much as possible consider local effects produced by the detention facilities (level and duration of

Figure 6.6
Flooded area in parking lot as a function of depth of ponding - case of centrally located grate inlet.

ponding in the present case) and, for the receiving stream, potential erosion, flooding and increased pollution. Ideally, if the hydrological impacts of development are to be minimized as dictated by the fundamental objective of stormwater management, all these aspects should be analyzed and taken into account; practically, however, economic considerations very often prevent optimal solutions being implemented and engineering judgment has to be involved to select the most acceptable solution in terms of tangible and intangible benefits.

Finally, continued use of synthetic design storms for volume determination and detention design should be seriously questioned, as continuous or pseudo-continuous simulations can be completed rather easily nowadays with powerful micro-computers; simulations with historic storms selected appropriately will likely provide a much more reliable design.

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