Chapter 16

A Continuous Simulation Technique for the Assessment of Infiltration Basins

Neil R. Thomson

Department of Civil Engineering
University of Waterloo
Waterloo, Ontario, N2L 3G1

This chapter provides an overview of a simulation technique that can be used to assess the performance of an infiltration basin facility. The developed model provides the necessary quantity and quality linkage between the surface and groundwater regime, and allows for the continuous simulation of the basin which is required for a long-term site-specific design assessment. Simulation results from a conceptual infiltration basin design are presented and discussed for both a single and a multiple event hydrograph and pollutograph.

16.1 Introduction

Over the last few years, a wide range of *Best Management Practices* has been developed in order to aid in the protection of the water quality of receiving water bodies in need of assistance due to actual or, potential, impacts of stormwater discharges.

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Some of the more desirable structural *Best Management Practices* are the induced infiltration alternatives (e.g., infiltration basins, infiltration trenches, and porous asphalt pavements) since they are capable of mitigating the stormwater impact of flooding, streambank erosion, and receiving water quality problems (Md WRA, 1984). More importantly, however, induced infiltration may maintain, or perhaps enhance, stream baseflow thereby allowing a stream to increase its ability to assimilate contaminants, and thus strengthen the potential for aquatic life. Unfortunately, the implementation of infiltration alternatives is only possible for areas where the underlying hydrogeological conditions are favourable.

The most promising of the induced infiltration alternatives for servicing large drainage areas or developments is the infiltration basin. An infiltration basin is constructed by excavation of a large dugout, or formation of earth berms or embankments in order to create a relatively large open surface storage area. A typical infiltration basin which may be used to infiltrate high-frequency runoff events is presented in Figure 16.1 (Schueler, 1987).

Figure 16.1: Typical infiltration basin.

For this configuration, stormwater runoff enters the basin through the inlet structure and passes over the riprap apron which reduces the energy capacity of the runoff. This reduction in runoff velocity is required to reduce the potential for erosion of the basin floor. The basin floor is turfed and is as level as physically possible in order to distribute the storm runoff volume evenly over the entire basin floor area. The basin stores a specified volume of water which is allowed to infiltrate into the surrounding subsurface regime through the floor and side walls of the basin. An outlet structure is provided to allow less frequent storms to be released, while major events can be discharged over the emergency spillway.

Infiltration basins are thought to be very effective in removing fine particulate matter and soluble contaminants present in urban stormwater runoff; however, little field data exists to substantiate the effectiveness of this removal process (Schueler, 1987). The quantity of sediment build-up within an infiltration basin is problematic since it reduces the maximum infiltration rate of the underlying soil strata. Sediment build-up rate is a function of the storage capacity of the infiltration basin, the volume of inflow into the basin, and the sediment loading. It has been recommended by Md WRA (1984) that an infiltration basin be designed to drain completely within 3 days (72 hours) following the occurrence of a storm event. The primary purpose for this design requirement is to encourage aerobic conditions for bacteria growth within the upper portions of the soil profile.

16.2 Potential Problems

A serious concern with the use of infiltration basins is the potential for groundwater and soil contamination. This concern has arisen because of the potential impact on aquifers which are used as a primary water supply source. An illustration of this situation is presented in Figure 16.2 and 16.3. In Figure 16.2, an infiltration basin has developed a zone of groundwater

Figure 16.2: Components of the continuous simulation model.

contamination that threatens a water supply source, while in Figure 16.3 the zone of groundwater contamination will eventually migrate into a surface water body thus degrading this aquatic ecosystem.

Unfortunately, to date, data relating to the long-term impact of infiltration basins on the subsurface regime is not very extensive. Several monitoring studies have been conducted in the U.S. (e.g., Nightingale 1987a,b; Yousef *et al.*, 1987), and have shown that the concentration of heavy metals quickly decreased with soil depth. However, it should be noted that these studies are site specific and variations between land use, soil type, runoff water quality characteristics, and the groundwater flow regime have not been investigated.

The impact of infiltration basins on the subsurface regime is extremely important since the remediation of groundwater contamination problems can be very expensive, and groundwater contamination problems are usually considered long-term.

16.3 Continuous Simulation Model

In order to assess the impact of a proposed, or existing, infiltration basin on the subsurface regime, a continuous

Figure 16.3: Subsurface configuration.

simulation model has been proposed. This model provides the necessary linkage between the infiltration basin and the subsurface regime for the interchange of both fluid and contaminants which is required for a long-term site specific performance assessment. This long-term performance assessment allows for the simulation of actually observed multiple storms where the sequence of events is important, rather than basing the infiltration basin performance on a generic or hypothetical set of conditions (Marshall Macklin Monaghan Ltd., 1991).

16.3.1 Conceptualization

Conceptually, the developed continuous simulation model can be illustrated as shown in Figure 16.4. For a given drainage area and meteorological conditions, a continuous profile of the surface runoff hydrograph and pollutograph can be generated. These profiles are routed into the infiltration basin using a variable time stepping scheme, and the appropriate overflow and infiltration rates are determined. Since the time frame for each storm event is, in general, small compared to the infiltration time frame (hours as compared to days), the portion of inflow over the design

Figure 16.4: Boundary conditions.

volume of the infiltration basin (if any) is removed as overflow. The remaining volume within the infiltration basin is available for infiltration. The actual infiltration rate is determined by the depth of water in the basin and the transient pressure head distribution within the subsurface. For simplicity, the subsurface has been conceptualized as a two-dimensional spatial domain which is assumed to coincide with the regional groundwater flow direction. Although a fully three-dimension representation would capture more completely the actual performance of the basin, the two-dimensional representation should be adequate for most situations (Fipps *et al.*, 1988).

In general, the following steps are involved for each simulation time step:

1. determine added fluid and pollutant mass to the infiltration basin;
2. remove the overflow portion (if required);
3. determine the infiltration rate into the subsurface; and
4. determine the mass transport of contaminant in the

subsurface regime.

The calculation of the infiltration rate into the subsurface (Step 3) and the subsequent loss of fluid and contaminant mass from the infiltration basin is perhaps the most computationally burdensome step. This is due in part to the nonlinear flow equation that governs the transient pressure head distribution in the subsurface, and also due to the iterative procedure required to determine the depth of water in the basin at the end of each time step.

A basin model and a subsurface model are the two basic components of this continuous simulation model and are described in the following two sections.

16.3.2 Basin Model

The basin model is essentially a level pool routing algorithm (Chow *et al.*, 1988). The fluid mass balance equation for an infiltration basin is given by

$$\frac{dS(h_w^b)}{dt} = I(t) - O(t, h_w^b) - E(t, h_w^b) \quad (16.1)$$

where $S(h_w^b)$ is the storage within the pond, h_w^b is the depth of fluid in the basin, $I(t)$ is the continuous fluid inflow, $O(t, h_w^b)$ is the basin overflow, and $E(t, h_w^b)$ is the infiltration rate from the basin into the subsurface. The basin storage is determined from a basin stage-storage curve which is input into the developed model as a set of tabular data. The basin overflow is determined based on a stage-discharge curve which is also input into the model as a set of tabular data. Finally, the infiltration rate $E(t, h_w^b)$ is determined implicitly as part of the calculation of the pressure head distribution within the subsurface.

A similar mass balance equation can be formulated for the mass of contaminants in the infiltration basin. In this formulation it is assumed that the contaminant of concern is completely mixed within the basin.

16.3.3 Subsurface Model

Within the subsurface domain two distinct zones can be defined. The first zone is the saturated zone where all of the available pore space is occupied by water, while the second zone is the unsaturated zone where all the available pore space is occupied by water and soil air. Assuming that the soil air within the unsaturated zone is infinitely mobile, then the governing equation which describes the continuous two-dimensional transient pressure distribution in the subsurface may be expressed as:

$$S_w S_s \frac{\partial h_w}{\partial t} + \phi \frac{\partial S_w}{\partial t} = \frac{\partial}{\partial x_i} \left[k_{rw} K_{ij} \frac{\partial}{\partial x_j} (h_w + e_j) \right] \quad (16.2)$$

where S_w is the water saturation, S_s is the specific storage, t is time, h_w is water pressure head, ϕ is porosity, K_{ij} is the saturated hydraulic conductivity tensor, k_{rw} is the relative permeability with respect to water, and e_j is the unit vector in the direction of the vertically upward component of the coordinate system. Equation (16.2) represents a modified version of Richard's equation (Richard's 1931). Associated with (16.2) is an appropriate set of initial and boundary conditions. The initial conditions define the water pressure head distribution prior to the start of simulation, while the boundary conditions are defined around the external boundary of the spatial domain. There are two types of boundary conditions that apply to (16.2): the first denoted as Type-1, represents a known water pressure head along a portion of the boundary, and the second denoted as Type-2, represents a known Darcy flux along a portion of the external boundary (Huyakorn and Pinder, 1983). To complete the statement of the governing equation (16.2), constitutive relationships relating water saturation to capillary pressure, and relative permeability to water saturation must be provided. One of the most widely used set of constitutive relationships are those developed by van Genuchten (1980). Assuming that water saturation is a single valued

function of the capillary pressure head, van Genuchten developed the following empirical relationship

$$S_{we}(h_w) = \left(1 + (\alpha h_{aw})^n\right)^{-m} \quad (16.3)$$

with

$$S_w = (1 - S_{wr}) S_{we} + S_{wr} \quad (16.4)$$

and

$$h_{aw} = h_a - h_w \quad (16.5)$$

where S_{we} is the effective water saturation; S_{wr} is the residual water saturation; h_{aw} is the capillary pressure; h_a is the soil air pressure in equivalent units of water pressure head; and α , n , and m are empirical constants. Based on the theoretical work of Mualem (1976), van Genuchten (1980) derived the following empirical relationship for relative permeability using (16.3)

$$k_{rw}(S_{we}) = S_{we}^{\frac{1}{2}} (1 - [1 - S_{we}^{\frac{1}{m}}]^m)^2 \quad (16.6)$$

To represent the mass transport of a contaminant within the subsurface domain, the following governing equation is employed in this model

$$\frac{\partial}{\partial t}(\phi S_w C_w + \rho_s(1-\phi)C_s) = \frac{\partial}{\partial x_i} \left(\phi S_w D_{ij} \frac{\partial C_w}{\partial x_j} \right) - v_i \frac{\partial C_w}{\partial x_i} \quad (16.7)$$

where C_w is the concentration of the dissolved constituent of concern in the water phase, ρ_s is the soil particle density, C_s is the concentration of the sorbed constituent in the solid phase, D_{ij} is the hydrodynamic dispersion tensor and accounts both for mechanical mixing and diffusion, and v_i is the Darcy velocity determined from the water pressure distribution. Equation (16.7) accounts for the transport processes of advection, dispersion, diffusion, and sorption (de Marsily, 1986). Also associated with (16.7) is a set of initial and boundary conditions. The initial conditions define the spatial distribution of both the water phase concentration and soil phase concentration prior to the start of simulation. Three types of boundary conditions are possible: the first, Type-1, defines a known concentration along a portion of the domain boundary, the second, Type-2, defines a specified diffusive mass flux along a portion of the domain boundary, while the third, Type-3, defines a known total mass flux along a portion of the domain boundary. All three of these boundary conditions are used to represent various portions of the spatial domain boundary for the infiltration basin configuration discussed in Section 16.4.

The solution of the governing equations (16.2) and (16.7) in conjunction with their appropriate constitutive relationships, and boundary and initial conditions were solved numerically by the finite element method. Full advantage was taken of the influence coefficient approach in forming the element matrix equations. Fluid mass was conserved in the solution of (16.2) due to the direct application of the finite element method to the mixed form of Richard's equation instead of the conventional pressure head form. A full Newton-Raphson iterative scheme was employed in order to converge to the required solution at each timestep. The linear system of equations was solved by an iterative, preconditioned orthomin solution method (Mendoza, 1991). To reduce numerical oscillations that are inherent in the solution of mass transport equation (16.7), an upwind finite element scheme was implemented (Huyakorn and Pinder, 1983). A variable timestepping scheme was also implemented in the model to improve the computational efficiency. This scheme allows the

time step increment to be automatically selected depending on the convergence criteria, the maximum change in water pressure head and/or concentration, and the time to the next inflow event.

16.4 Example Results

To demonstrate the applicability of the developed continuous simulation model the results from a single event simulation and a multiple event simulation are presented. For each of these simulations, the basin geometry and subsurface configuration are identical.

The infiltration basin considered in these analyses has a bottom width of 25 m, a length of 60 m, side slopes of 3:5 (v:h), and a maximum ponding depth of 2.4 m. The subsurface configuration consists of the 35 m deep by 700 m long two-dimensional spatial domain as presented in Figure 16.5. This spatial domain was discretized into 3864 triangular elements and 2030 nodes. Sand was used as the soil medium and assigned a saturated hydraulic conductivity of 0.001 cm/s, a porosity of 0.3, a longitudinal dispersivity of 5 m, and a transverse dispersivity of 2 m. For the constitutive relationships, (16.3)-(16.6), a value of α of 0.2/m, n of 2.2, $m=1-1/n$, and S_{wr} of 0.2 were employed. Although in this example one soil zone was employed, the developed methodology can be applied to subsurface domains which are comprised of more than one soil type.

The boundary conditions for the subsurface configuration are presented in Figure 16.6. A no-flow condition was assigned along the bottom of the spatial domain, and along the portions of the left and right boundaries corresponding to the unsaturated zone. For the remaining portions of the left and right boundaries a prescribed water pressure head corresponding to a hydrostatic pressure profile was assigned. The water table was positioned approximately 3 m below the base of the infiltration basin at a one percent slope. Across the top of the spatial domain, a prescribed flux of 2.5 cm/year with a concentration of zero was

Figure 16.5: Subsurface configuration.

prescribed, except for the portion of the boundary corresponding to the location of the infiltration basin which was handled in the following iterative fashion for each time step:

- 1) The water elevation in the basin is estimated based on the added fluid mass to the basin,
- 2) The submerged portion of the boundary is prescribed a water pressure head equal to the hydrostatic profile in the basin while the remaining portion is prescribed a zero water flux condition,
- 3) Using this set of boundary conditions the transient water pressure head distribution is determined and the total infiltration rate $E(t, h_w^b)$ in (16.1) is calculated,
- 4) This infiltration rate is used in conjunction with (16.1) to determine the updated water elevation in the infiltration basin and then this process is repeated until convergence is attained.

Once convergence has been attained for the pressure head distribution, the concentration of the pollutant in the fluid contained within the infiltration basin is assigned to the submerged portion of the basin boundary, while the remaining

portion of the basin boundary is assigned a zero diffusive flux value. The concentration in the fluid entering the spatial domain through the left boundary is assigned a value of zero, indicating that this fluid is devoid of the pollutant.

Figure 16.6: Boundary conditions.

The initial condition for the water pressure head was determined from a steady state flow analysis using the above defined boundary conditions for flow, and a prescribed uniform flux of 2.5 cm/year across the top of the spatial domain. It was assumed the initial water and soil phase concentrations are zero throughout the spatial domain.

16.4.1 Single Event

This single event simulation consists of one event routed into the infiltration basin and the subsequent water pressure head and concentration distribution determined within the subsurface regime. The hydrograph and pollutograph which are used as input into the infiltration basin are presented in Figure 16.7. For simplicity, a triangular shape and time base of 3 hours was used for each of these profiles. Over the 3-hour period a total of 4000 m³ of water and 3000 kg of a conservative contaminant (e.g., chloride) were routed into the infiltration basin.

Figure 16.7: Input hydrograph and pollutograph for the single event simulation.

The pressure head and concentration distributions are presented in Figures 16.8 through 16.11 for a simulation time of 0.25, 15, 36, and 72 hours, respectively. Note that the concentration contour increment in all of these figures is 200 mg/L. In Figure 16.8 (0.25 hours) it can be observed that the water table (which is approximated by the zero water pressure head contour) is responding to infiltration from the basin, and that a contaminant plume has been formed below the infiltration basin. At 15 hours into the simulation (Figure 16.9) the subsurface regime below the infiltration basin is saturated and a substantial plume exists. By 36 hours (Figure 16.10) into the simulation the infiltration basin is almost empty, while at 72 hours (Figure 16.11) the water table is returning to its initial position. Notice that the morphology of the contaminant plume below the basin is almost identical at 36 and 72 hours, which suggests that the early portion of the infiltration event contributes predominantly to the migration of the plume.

A single event simulation approach can also be used effectively to analyze a proposed infiltration basin during the hydraulic design phase. A brief study of the variations in infiltration rate with maximum ponding depth, antecedent soil moisture conditions, the location of the water table, and the physical soil characteristics of the underlying soil has been reported elsewhere by Graham and Thomson (1991).

Figure 16.8: Pressure head and concentration distribution at 0.25 hours.

Figure 16.9: Pressure head and concentration distribution at 15 hours.

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Figure 16.10: Pressure head and concentration distribution at 36 hours.

Figure 16.11: Pressure head and concentration distribution at 72 hours.

Figure 16.12: Cumulative inflow to infiltration basin.

Figure 16.13: Cumulative chloride mass inflow to infiltration basin.

16.4.2 Multiple Event

The multiple event simulation consists of a continuous hydrograph and pollutograph routed into the infiltration basin for a period of one year. The hydrograph and pollutograph employed in this analysis were determined from STORM (HEC, 1977) using the coefficient method for a 20 hectare drainage area. Hourly precipitation and daily temperature data were used to generate rainfall/runoff or snowmelt/runoff information as shown in Figure 16.12. To indicate the upper-end member of the migration

Figure 16.10: Pressure head and concentration distribution at 36 hours.

Figure 16.11: Pressure head and concentration distribution at 72 hours.

Figure 16.14: Chloride distribution after one year of simulation.

The proposed approach appears to be useful for both single and multiple event simulations; however, a comprehensive validation of the developed model using actual field observations should be undertaken to ensure that the necessary processes have been taken into consideration.

Additional research into soil clogging, soil freezing and a fully three-dimensional representation of the infiltration basin need to be addressed.

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