CHAPTER NINE
Managing Urban Runoff

9.1 INTRODUCTION

Impairments caused by urban storm water runoff range from reduced water quality due to the accumulation and transport of pollutants to the degradation of stream channels and localized flooding caused by increased volumes and rates of runoff. Preventing and managing these impacts are challenging, in part, because they are transient and elusive (e.g., contaminant concentrations), and the sources of problems are not easily identified. Furthermore, impacts are often interrelated and cumulative. For example, both degraded water quality and increased peak discharges combine to negatively affect aquatic habitat and biological resources.

The mitigation of such complex problems requires a thoughtfully-prepared watershed management plan. Formulating this plan begins by identifying potential environmental and health impacts through an inventory of natural resources (e.g., waterways, wetlands, and wildlife). For each resource, possible users (e.g., commercial navigation, recreation, or potable water supply) should be designated, existing conditions should be quantified, and, to the extent possible, current and future potential sources of pollution should be identified. This initial step will serve to define aspects of the problem and will serve as a benchmark to measure the effectiveness of future mitigation efforts. Also at this stage, modeling studies will be useful to better understand existing hydrologic behavior of the region, and existing regulatory or private planning processes (e.g., land use ordinances and flow monitoring programs) should be recognized in order to minimize duplication of efforts and maximize the use of available information. Moreover, representative stakeholders (e.g., regulatory agencies, land owners, city planners, and other private organizations) should be invited to become part of planning efforts at this early phase. In many cases, the success of the management plan will hinge upon the understanding and active participation of these stakeholders. Otherwise, the legitimacy of the plan may be questioned in the future.

After specific problems have been identified, corresponding mitigation objectives can be developed. These may be based on a singular criterion, such as reducing runoff for a residential complex to pre-development conditions, or more typically, can be defined based on multiple factors. An example of the latter might include the attenuation of peak flows and suspended solids concentrations at multiple locations. In either case, objectives are likely to vary among stakeholders. Thus, it is important for these stakeholders to convene and collectively evaluate what is envisioned for the future of the watershed, how that vision can be realized, and who will fund corresponding efforts.

At this point, best management practices (BMPs) should be selected and implemented in a phased approach. The term BMP broadly represents a range of possible abatement procedures, devices, activities, or restrictions that can be implemented to achieve a variety of specified objectives. As part of implementation, it is important that a single lead agency or entity assume the role of advocate or facilitator among community representatives and regulatory personnel and that roles and responsibilities of other stakeholders are well defined. Doing so will promote the efficient use of resources and maximize the potential for success of the plan.

After BMPs have been implemented, the management plan should be revisited every three to six years. Monitoring data for the corresponding period will demonstrate whether the initial problem is being resolved and reveal whether new problems have occurred. This information can be used to reassess the problem, objectives, and necessary control measures. This process essentially comprises an adaptive management framework whereby stakeholders acknowledge the inevitable uncertainty that existed in the planning phase with regard to data collection, hydrologic modeling, and the design of BMPs, as well as the variability of nature itself. Updated plans should consider recent changes in drainage controls, land use, population, and other factors that may affect future planning efforts.

The discussion that follows focuses on the selection and design of BMPs for reaching watershed management goals, and it is not meant to be an all inclusive guide for engineers and planners. Over the last decade, a significant amount of literature has surfaced with respect to various aspects of effective watershed management; the information ranges from technical design guidelines for BMPs to more qualitative analyses of related social, political, and economic issues. A number of case studies have also been published, some of which detail success stories and others that describe failed attempts at watershed management. The information provides the reader with an opportunity to learn from past experiences to more effectively control the impacts of urban runoff.

9.2 BEST MANAGEMENT PRACTICES

BMPs can generally be categorized as being either non-structural or structural control measures. The former focuses on the control and prevention of storm water problems at their source, whereas the latter refers to constructed, passive-treatment units. Some of these measures are designed to focus on either peak flow reduction or pollution control; however, many BMPs can be considered dual-purpose in that they provide both water quality and quantity benefits.

Furthermore, BMPs can be designed and implemented separately or can be applied in a locally-strategic combination (i.e., treatment train) to meet related goals more cost-effectively.

To enable the transfer of new BMP technology to practitioners, the United States Environmental Protection Agency (USEPA) and the American Society of Civil Engineers (ASCE) recently initiated the National Storm Water BMP Database project (ASCE, 2006). The initial database documented over 800 BMPs, and figures for more are being added regularly as part of this ongoing project. Representative data for BMPs includes test site location, watershed characteristics, climate statistics, design parameters, monitoring data, and equipment needs. This information can be used as a template in the design and implementation of future control measures.

9.2.1 Non-Structural BMPs

Once runoff enters receiving waters, it is more difficult and costly to mitigate its impact. As a result, the USEPA has placed a priority on implementing control measures that reduce possible pollutant discharges or peak flows at the source. Some of the more common examples of these measures include public education, land use planning, and improved landscaping techniques.

9.2.1.1 Public Education and Outreach

The public is often unaware of the effects their actions have on pollution. Proper education on day-to-day activities that can be undertaken (e.g., recycling of automotive fluids, household chemical and lawn fertilizer use and disposal, and animal waste control) is an effective method for limiting the amount of pollutants that enter receiving streams (USEPA, 1999a). Education can be formalized (i.e., workshops) or can be achieved through the distribution of pamphlets and other types of public service announcements.

9.2.1.2 Land Use Planning and Conservation

Prior to urban development, thoughtful planning that considers multiple objectives, including reducing environmental impacts, is an important preemptive step in controlling problems associated with urban runoff. Low-impact development strategies may involve minimization of directly-connected impervious surfaces (i.e., those draining directly to the basin outlet), application of site depressions and rain gardens, implementing tax incentives and zoning ordinances for directed growth, and formalizing the protection of sensitive areas. In most cases, high-density, clustered urban development is preferred over low-density, suburban development as the former tends to limit the creation of extensive impervious cover on a broad regional scale.

9.2.1.3 Landscaping and Vegetative Cover

Landscape planning should avoid long, steep slopes and provide terracing, contouring, and drainage channels as needed to limit runoff and erosion. Bare areas should be covered with vegetation or material such as hay or mulch. Doing so not only reduces erosion potential, but the increased surface roughness will improve infiltration to underlying soil and reduce runoff velocity. In areas where bare areas are temporarily unavoidable (i.e., construction activities), straw bales, check dams, and silt fences are often used as sediment traps to prevent eroded material from entering nearby drainage systems or streams.

Additionally, cities or drainage districts often obtain drainage easements for smaller channels to be modified as part of urbanization efforts and commonly install vegetative or riprap liners within channels. The reader should refer to methods described in Chapter 8 for designing and evaluating the suitability of flexible-lined channels. Alternatively, for riprap liners, the Federal Highway Administration (Normann, 1975) more simply recommends that the median diameter, D_{50}, of riprap on the bed of a sloping channel should be approximately

$$D_{50} = \frac{\gamma y S}{5} \tag{9-1}$$

where D_{50} is in ft; γ is the specific weight of water in lbs/ft^3; y is the maximum stable depth of flow in ft; and S is the slope of channel. Riprap is then typically graded in size from $0.2 \times D_{50}$ to $2.0 \times D_{50}$ (Wurbs and James, 2002).

9.2.1.4 Maintenance and Housekeeping

After urbanization has occurred, a number of ongoing practices are important in preventing pollutants from entering drainage facilities and receiving waters. These practices include street sweeping, implementing formal waste collection programs, removal of trash and other debris from catch basins, maintenance and stabilization of roadways and ditches, and use of improved roadway deicing methods.

9.2.2 Structural BMPs

Structural BMPs, or passive-treatment controls, are those that generally do not require active operational control, but only routine maintenance. These BMPs rely upon key mechanisms of settling, filtration or infiltration, sorption, biodegradation, and/or evapotranspiration. Examples include various storm water basins (e.g., detention, retention, and infiltration), filter strips, sand filters, and swales. For a variety of reasons (e.g., cost, site suitability, and proven performance), some of these systems are used far more widely than others. In particular, detention and retention facilities are most common for attenuating peak flows and limited treatment needs. Modeling of these systems typically relies on first principles of hydrology and environmental engineering (i.e., unit processes); it is important to note, however, that there still exists significant room for improving the prediction accuracy of current methods and models.

9.2.2.1 Detention Systems

Surface detention basins, either extended or dry ponds, are small impoundments usually serving drainage areas of 10 acres (4 ha) or less. Although a number of variations are possible, Figure 9-1 illustrates a typical basin profile. These systems are designed to store a portion of runoff and, through the use of restricting, gravity-flow outlet works, empty slowly following a storm event. In addition to attenuating peak runoff rates by redistributing discharge hydrographs, detention of storm water provides an opportunity for settling of suspended solids (e.g., sediment) and other pollutants. Removal rates generally increase with detention time, defined by Haan et al. (1994) as the time difference between the centroids of the inflow and outflow hydrographs. Brown et al. (1996) report pollutant removal rates as high as 90 percent if storm water is detained for 24 hours or more. Detention basins may also create benefits in terms of recreational use (e.g., golf course or parks) of seldomly-inundated portions of the pond and the promotion of wetland and wildlife habitat. Despite their versatility, detention basins may occupy up to 20 percent of the corresponding basin's area (ASCE, 2001), and they require routine maintenance, including mowing, debris removal, and the eventual removal of sediment. Otherwise, they may become an aesthetic nuisance with respect to odor, debris, and insect-breeding.

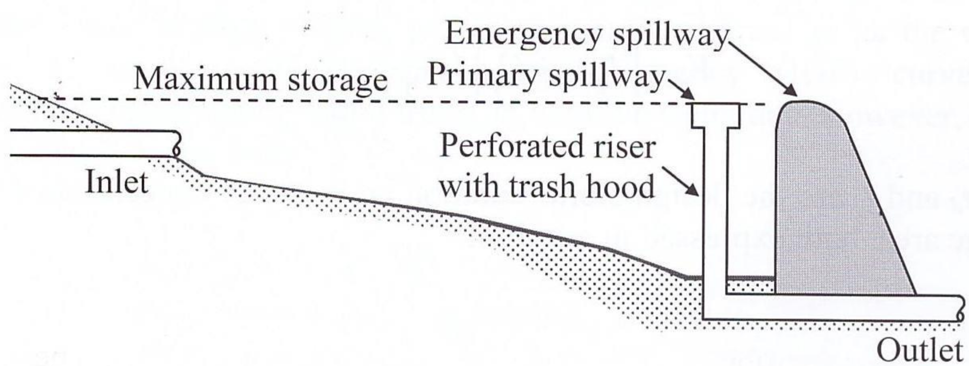

Figure 9-1: Typical detention basin

Sizing a detention basin begins by estimating the volume of pond storage required. Storage criteria, along with the period over which the associated volume should be released, vary among regulatory or management authorities. Brown et al. (1996), however, recommend that, at a minimum, a basin store the equivalent of 0.5 in (13 mm) of runoff spread uniformly over the drainage area for water quality purposes. This volume is often referred to as the water quality volume (WQV). From a water quantity perspective, larger rainfall events (e.g., five- or ten-year storms) should be used for design. The required volume is used to prepare a trial layout, including a desired outlet configuration. Note that basic layout dimensions will often depend on site limiting factors such as topography, utilities, and geology of underlying soils. In addition, layout and outlet works may be adjusted so that the basin drains within a typical interstorm period of 72 hours, or 24 to 36 hours for landscaped basins. Once a trial design has been selected, storage-outflow and stage-outflow (i.e., water surface elevation-discharge) relationships are used to route an inflow hydrograph through the basin. Based on routing results, the size and detention time can be iteratively refined until a suitable design is attained. ASCE (1998) further recommends increasing this final volume by 20 percent to allow for sediment accumulation in the pond.

Table 9-1 lists a number of methods available for estimating a required detention storage volume, V_s. As an alternative, the modified rational method is commonly used for basins that are less than 30 acres (12 ha) in size (Chow et al., 1988). While there are several variations of the method, it generally assumes that the inflow hydrograph for the basin is trapezoidal in shape and can be constructed such that the period associated with the rising and recession limbs equals the time of concentration for the drainage basin. The peak inflow, I_p, can be computed using the rational formula (see Chapter 5). The approach also assumes that the rising limb of the outflow hydrograph is linear and that its peak, O_p, falls on the recession limb of the inflow hydrograph, as shown in Figure 9-2. With similar assumptions, Aron and Kibler (1990) showed that the volume of required storage can be expressed as

$$V_s = \left[I_p t_d - O_p \left(\frac{t_d + t_c}{2} \right) \right] \times 60 \tag{9-2}$$

where t_d and t_c are the design storm duration and time of concentration for the drainage area, both expressed in minutes.

Table 9-1: Formulae for trial storage volumes

Relationship	Formula	Comments
Abt and Grigg (1978)	$\frac{V_s}{V_r} = \left(1 - \frac{O_p}{I_p}\right)^2$	Triangular inflow hydrograph and trapezoidal outflow hydrograph
Wycoff and Singh (1986)	$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{O_p}{I_p}\right)^{0.753}}{\left(t_b/t_p\right)^{0.411}}$	Based on regression analysis
ASSHTO (1991)	$V_s = \frac{1}{2} t_b \left(I_p - O_p\right)$	Inflow and outflow hydrographs are triangular (i.e., $t_p = t_b/2$).
Kessler and Diskin (1991)	$\frac{V_s}{V_r} = 0.932 - 0.792 \frac{O_p}{I_p}$	For weir outlet and constant basin surface area; valid for $0.2 < \left(O_p/I_p\right) < 0.9$
Kessler and Diskin (1991)	$\frac{V_s}{V_r} = 0.872 - 0.861 \frac{O_p}{I_p}$	Orifice outlet and constant basin surface area; valid for $0.2 < \left(O_p/I_p\right) < 0.9$

Note: V_s is the required storage volume; I_p and O_p are the peak inflow and outflow (i.e., allowable outflow) rates of the basin, respectively; V_r is the total volume of runoff; t_b is the base time of the inflow hydrograph, defined for the Wycoff and Singh (1986) method as the time from the beginning of rise to a point on the recession limb where flow is five percent of I_p; and t_p refers to time to peak inflow.

For the modified rational method, the design duration is that which maximizes basin storage volume and is frequently referred to as the critical duration. If using local intensity-duration-frequency (IDF) curves, its corresponding value can be found using an iterative technique. However, if IDF data is available in the form

$$i = \frac{a}{\left(t_d + b\right)^c} \tag{9-3}$$

where i is the average rainfall intensity and a, b, and c refer to empirical constants determined using curve fitting or by using published data (e.g., see Chapter 2; Table 2-1), then the critical duration can be computed by differentiating Equation 9-2 and equating it to zero. Doing so yields

$$\frac{a\left[t_d(1-c)+b\right]}{\left(t_d+b\right)^{c+1}}-\frac{O_pK_r}{2CA}=0 \qquad (9\text{-}4)$$

where, for time and intensity data given in minutes and in/hr or mm/hr, respectively, O_p is in cfs or m^3/s; C is a dimensionless runoff coefficient (see Chapter 5; Table 5-3); A is the drainage area in ac or ha; and K_r is a conversion constant equal to 1.0 in U.S. customary units and 360 in S.I. units. Equation 9-4 can be solved for t_d, which is then used to evaluate the associated rainfall intensity and peak rate of flow entering the basin. Subsequently, Equation 9-2 can be used to evaluate a trial basin storage volume.

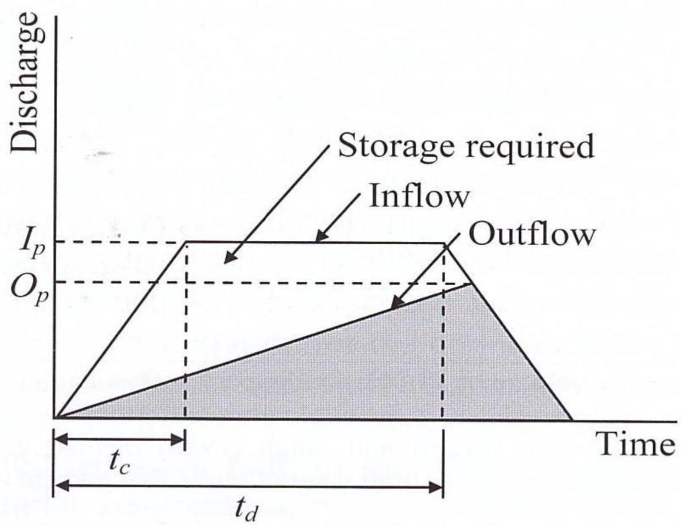

Figure 9-2: Inflow and outflow hydrographs for trial storage volume

Another commonly used method for determining basin storage volume is that of the Federal Aviation Administration (FAA, 1970). Very similar to the modified rational method, the FAA approach incorporates the duration that produces the maximum basin storage volume. Alternatively expressed, the volume required is the maximum difference between cumulative inflow and outflow volumes to the basin, or

$$V_s=\max\left(I_pt_d-mO_pt_d\right) \qquad (9\text{-}5)$$

loading. In this way, the permanent pool captures and treats the smaller and more frequently-occurring runoff. Additional storage volume, above the permanent pool, is used to limit peak discharges caused by design storm events. Similar to detention facilities, criteria for determining the size of the permanent pool varies between management authorities. Brown et al. (1996), however, recommend that the pool be at least three times the WQV specified for detention basins.

9.2.2.3 *Infiltration Systems*

Infiltration of a portion of the runoff through coarse media and underlying soils reduces peak surface discharges to receiving streams and simultaneously recharges groundwater. It also allows for removal of fine and soluble pollutants through adsorption, filtering, and microbial decomposition of soluble and particulate pollutants. Commonly-used systems that rely heavily on infiltration include infiltration basins, infiltration trenches, and porous pavements.

Note that the effectiveness of infiltration practices can be particularly poor if soil infiltration capacities are low or if sediments have not been adequately removed prior to groundwater recharge; in both cases, problems with media clogging can often occur quickly (Wurbs and James, 2002; Field and Sullivan, 2003). To prevent such problems, a filter strip can be installed around the perimeter of the trench. In addition, if groundwater in the area is used as water supply, care should be taken so that infiltration does not worsen the negative health impacts due to the migration of contaminants.

Infiltration Basins: These basins are essentially depressed areas, either natural or excavated, into which storm water is conveyed and allowed to percolate through underlying soil. Their appearance and construction is similar to that of detention ponds, although they can generally accommodate larger drainage areas of up to 50 ac (20 ha) (Brown et al., 1996). In fact, some facilities are designed as combined infiltration/detention basins. The major advantages of infiltration basins are that they replenish groundwater and preserve the natural water balance locally, and generally they are more cost effective than other BMPs. They do, however, experience a high failure rate due to unsuitability of soils and clogging, and they require frequent maintenance to prevent operational and nuisance problems.

The suitable operation of infiltration basins requires that the facility is capable of capturing a design storm water load and that subsurface geometry and geology can sustain infiltration. With regard to the former, the basin can essentially be designed as a detention pond, using previously described methods. Since water percolates slowly, however, a much larger surface area is required to disperse storm water. Guo (2001) showed that the minimum basin surface area, A_{min}, can be estimated as

$$A_{min} = \frac{V_c}{Z(\phi - \theta)} \tag{9-7}$$

where ϕ is the soil porosity; θ is the specific retention, or initial water content, of the soil; Z is the distance between the basin bottom and the groundwater; and V_c is the volume of runoff to be captured based on surface hydrology (e.g., WQV or that required to meet pre-development conditions). This relationship is based on a conservative approach in which it is assumed that the water storage volume available in the soil pores beneath the basin is greater than V_s. The maximum basin depth is equivalent to the denominator of Equation 9-7, and the associated drain time, or drawdown time, T_D, of the basin is (Guo and Hughes, 2001)

$$T_D = \frac{Z(\phi - \theta)}{f} \tag{9-8}$$

where f is the final (i.e., saturated) infiltration rate of the soil. As a general rule, for the basin to be feasible, soils lying at depths up to 5 ft (1.5 m) beneath the basin should have a minimum infiltration rate of 0.5 in/hr (13 mm/hr) (Brown et al., 1996); otherwise, the basin area required to limit peak runoff is excessive. In addition, the bottom of the basin should lie at least 5 to 10 ft (1.5 to 3 m) above the seasonally-high groundwater level and 5 ft (1.5 m) above bedrock (Guo, 2001).

Infiltration Trenches: Trench systems, as shown in Figure 9-3, can serve drainage basins as large as 10 ac (4 ha) and consist of an excavated, permeable-lined trench that has been backfilled with stone having a median diameter of 1.5 to 3 in (40 to 80 mm) and porosity of 30 to 40 percent (Akan and Houghtalen, 2003). Runoff should either drain from the trench into native soil or enter an underdrain within 24 hours (i.e., T_D) for less permeable soils or within up to 72 hours for more pervious soils. Trench depths often range from 3 to 8 ft (1.0 to 2.5 m) for water quality control purposes and can be larger if water quantity is a design factor (Schueler, 1987; ASCE 2001). The maximum allowable depth, d_{max}, is approximately (Akan and Houghtalen, 2003)

$$d_{max} = \frac{fT_D}{\phi} \tag{9-10}$$

where f and ϕ are for surrounding soils and stone fill, respectively. Simultaneous consideration, however, should be given to the guideline that the bottom of the trench should lie at least 4 ft (1.2 m) above bedrock and at least 2 ft (0.6 m) above the high groundwater level (ASCE, 2001). Once the depth is determined, the remaining dimensions of the trench can be estimated, noting that the trench should store the runoff volume and rain water falling directly on the trench, minus the quantity of water infiltrated into surrounding native soil while the trench is filled. For a rectangular trench having a bottom area of $W \times L$, this criterion can be expressed as

$$\phi(LWd) = V_c + (LW)P_e - (LW)fF_T \tag{9-11}$$

where V_c is the volume of runoff to be captured; P_e is the depth of rainfall excess; and F_T is the trench fill time. If the trench is designed for water quality purposes only (i.e., capture of smaller, more frequent storms), only the capture volume need be considered on the right-hand side of the expression.

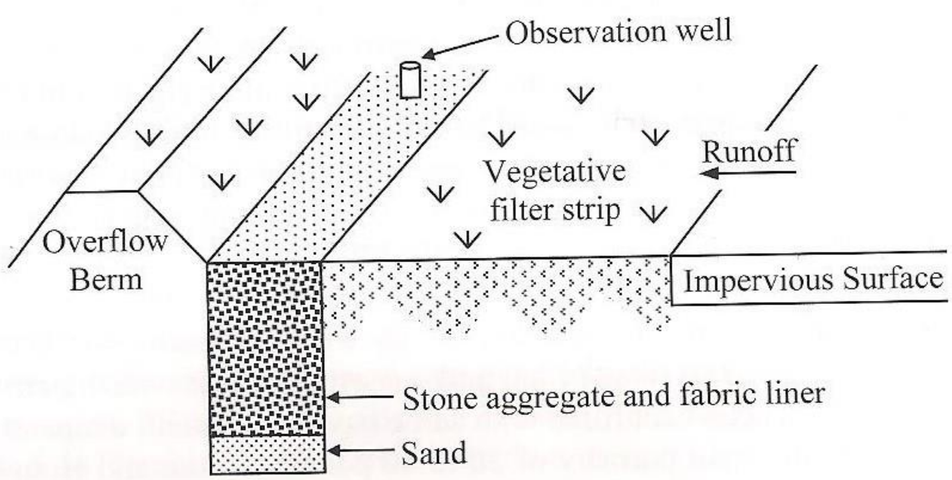

Figure 9-3: Typical infiltration trench (adapted from USEPA, 1999a)

Porous pavements: These pavements can be used in parking areas and roads having relatively low traffic volumes. They consist of aggregates having very high void ratios that permit temporary storage of runoff and incident rainfall. Over time, this volume will infiltrate deeper into the surrounding and underlying soil. The stone sub-base for porous pavements can be designed using the method outlined previously for infiltration trenches.

9.2.2.4 Filter Strips

Filter strips, as shown in Figure 9-4, represent zones of dense vegetation ranging from grass to forest that are planted on contoured or riparian areas. Also referred to as buffer strips, they are placed perpendicular to overland flow in order to attenuate discharge and filter particulates from runoff. Pollutant removal in these systems occurs through detention, vegetative filtration, and infiltration into underlying soils. Filter strips are generally limited to drainage areas of 5 ac (2 ha) or less and are often used as pre-treatment for other BMPs (Brown et al., 1996). Special care should be taken so that flow does not prematurely form a concentrated channel and thus short-circuit the filter.

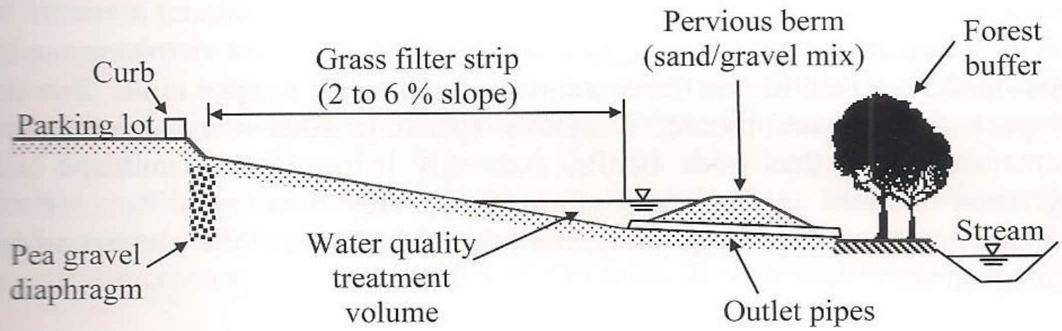

Figure 9-4: Typical filter strip (adapted from Claytor and Schueler, 1996)

As a guide, filter strips should have hydraulic retention times of five minutes or more; should not incur flow velocities greater than 0.9 fps (0.3 m/s); should have a Manning roughness coefficient, n, in the range of 0.2 to 0.24; and should maintain flow depths that are less than 1.0 inch (25 mm) (ASCE, 1998). These criteria can be used to determine the minimum filter dimensions. First, the maximum unit-width loading, q, is determined using the Manning equation for overland flow, expressed as

$$q = \frac{K_m}{n} y^{5/3} S^{1/2} \qquad (9\text{-}12)$$

where q is the unit width flow; K_m is a constant equal to 1.49 in U.S. customary units and 1.0 in S.I. units; n is the Manning roughness coefficient (see Chapter 5; Table 5-1); y is the flow depth; and S is the longitudinal slope of the strip. The minimum width can be then computed by dividing the design flow rate by q, and the corresponding required length, L_f, can be estimated by rearranging the kinematic wave equation for overland flow as follows:

$$L_f = \frac{t_c^{5/3} i^{2/3} S^{1/2}}{K_f n} \tag{9-13}$$

where L_f is in ft (m); t_c is the time of concentration through the filter in minutes, assumed to be equivalent to the retention time; i is rainfall intensity in in/hr (mm/hr); and K_f is a constant equal to 0.899 in U.S. customary units and 25.55 in S.I. units. Since Equation 9-13 assumes a constant Manning n and a homogeneous catchment and rainfall intensity, results are only approximate. Other time of concentration relationships for overland flow could be used to estimate length as well (see Chapter 5).

Note that regardless of the value computed using Equation 9-13, lengths should be no less than 20 ft (6 m) (ASCE, 1998). Furthermore, if runoff is heavily polluted and the filter is in a public area (e.g., park or playground), steps should be taken so that the contaminated sediments trapped in the filter do not pose an exposure hazard. Generally speaking, filter strips have lower maintenance costs than other BMPs, especially if the natural landscape and vegetation is used in design. Unfortunately, they have significant space requirements, sometimes consuming more than 25 percent of the corresponding drainage area.

9.2.2.5 *Sand Filters*

Although variations exist, sand filter systems generally utilize a bed of sand or similar media to filter sediment and pollutants from runoff prior to infiltration to native soils or being returned to a stream through underdrains (Brown et al., 1996). The overall design typically includes a pre-treatment (i.e., sedimentation) basin and flow spreader to capture runoff and distribute it slowly to the filter. Filters are generally very adaptable – they can be used in areas with thin soils, high evaporation rates, low infiltration rates, and limited space – and can serve watersheds up to 30 ac (12 ha) in size, depending on the design (ASCE, 2001). In application, the top of the filter media must be completely horizontal to prevent disproportionate use of the media, and the sand bed should be at least 1.5 ft (0.5 m) deep. The minimum required surface area of the bed, A_{min}, can be computed as (Debo and Reese, 1995)

$$A_{min} = \frac{V_c Z_b}{f(y_{max} + Z_b) T_D} \tag{9-14}$$

where Z_b is the depth of the sand bed, and y_{max} is the maximum depth of water over the filter surface. ASCE (2001) recommends a target drawdown time of 40 hours.

Similar to application of infiltration systems, the use of sand filters has been questioned in recent years due to the high cost of operation. Even with pre-treatment facilities, sediments can still quickly accumulate in the filter and must be removed to maintain filtration capacity. In addition, since the transport capacity of the downstream channel will generally exceed the sediment content of the effluent, the use of sand filters may actually increase downstream sediment degradation (i.e., bed erosion) (Wurbs and James, 2002). This latter problem may be experienced with other BMPs, such as detention and retention basins, as well.

It is also worth mentioning that filtration devices such as sand filters have a major tradeoff: they can be highly effective, but they cannot accommodate large flow quantities. If used alone, flow must pass through the filter to be effectively treated. They are, therefore, limited to small- to medium-sized drainage areas and relatively low flows. Larger flows, caused by high intensity storms, will tend to short-circuit or bypass the filter. In some cases, simultaneous application of other BMPS (i.e., detention facilities), can remedy this situation. For flow that is passed through the filter, a layer of organic matter, or biomass, will develop within the BMP, and the adsorption potential and removal efficiency of pollutants will actually improve over time as this biomass increases. This is especially true in areas of warmer climate.

9.2.2.6 Bioretention Basins and Wetlands

These BMPs rely primarily on biological processes for pollutant control. On average, they are highly effective for the removal of soluble pollutants (e.g., nutrients), suspended solids, and metals, but generally provide limited overall water quantity control. In operation, the combination of a shallow, depressed pool and rich, emergent vegetation allow a natural food chain to develop. Thus, pollutant removal can occur through biological plant uptake, microbial decomposition, volatilization, filtering, settling, and other processes (Brown, et al., 1996). If maintained properly, they can promote wildlife habitat and are generally more aesthetically pleasing than other BMPs.

Wetlands and bioretention basins are similar in many respects; however, wetlands require more vegetation, may incur slightly larger depths, and are designed as a small-scale network of permanent micro-pools, channels, and marsh areas (Novotny, 2003). Thus, for urban areas, bioretention and bioinfiltration basins (see Figure 9-5) are generally more suitable than wetlands as they can be placed in parking lot islands and around buildings. Basin dimensions should be at least 15 ft (4.6 m) wide by 40 ft (12.2 m) long with side slopes of no steeper than 4H:1V. The minimum surface area required can be computed using Equation 9-14 if Z_b is the depth of planting soil and y_{max} is the maximum depth of water in the basin. The size of drainage area treated by one bioretention cell should not exceed one acre (0.4 ha). In addition, ponding depths should be no greater than 6 inches (15 cm) above the filter bed. Bioretention basins are not appropriate at locations where the water table is within 6 ft (1.8 m) of the ground surface or where surface slopes exceed approximately 20 percent. Significant attention must be paid to careful selection of plants and conditioning of special planting soils for development of adequate vegetation and promotion of infiltration (USEPA, 1999b). Generally, planting soils should extend to a depth between 1 and 4 ft (0.3 and 1.2 m) below the filter surface and should be covered with a layer of mulch. In many cases, an underdrain is used to release the retention volume being infiltrated. Note that both bioretention basins and wetlands have moderately high maintenance (e.g., watering, plant upkeep, and litter removal) requirements. They may also require pre-treatment (e.g., filtering) facilities to remove coarse sediments that would degrade system performance. Currently, bioretention designs vary among applications, but as more data is collected over time, better and more cost-effective designs and standards should emerge.

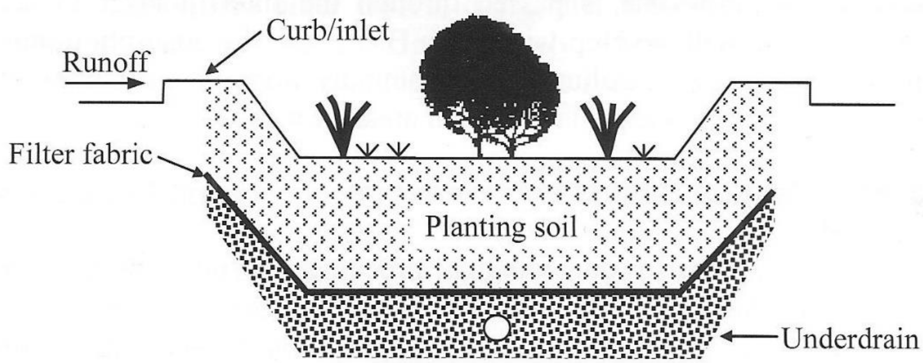

Figure 9-5: Typical bioretention cell (adapted from USEPA, 1999a)

9.2.2.7 Swales

A swale is a broad, shallow channel with dense stands of vegetation covering its side slopes and bottom. They are wider than normal storm water channels in order to accommodate design flows at depths below the height of vegetation. The vegetation serves to trap particulate pollutants and reduce flow velocities, in turn lowering peak runoff rates and providing a means for increased infiltration. Most swales are only effective, however, for drainage areas up to approximately 10 ac (4 ha). As such, they are often applied as a pre-treatment unit for other BMPs, and they may have filter media and an underdrain system added to further promote higher infiltration rates and enhanced treatment. Swales tend to be particularly common in residential areas and highway medians, where standing water poses a hazard or public nuisance. They are not recommended, however, when runoff velocities exceed 3 fps (0.9 m/s).

Swales should generally have longitudinal slopes as close to zero as the drainage design will permit, although 2.0 to 6.0 percent slopes are common. Lengths and widths should be at least 100 ft (30 m) and 2 ft (0.6 m), respectively, and swales should have residence times greater than five minutes. In addition, side slopes should be 3H:1V or flatter, and underlying soils should have a relatively high infiltration capacity (Schueler, 1987; ASCE, 2001).

The methods presented in Chapter 8 for the design and analysis of flexible-lined channels are applicable to swales. In addition, based a mass balance of inflows and outflows, the length of swale, L, required for full infiltration of the design storm, can be expressed as

$$L = \frac{Q}{Pf} \tag{9-15}$$

where Q is the average flow rate, and P is the wetted perimeter. Thus, for any cross-sectional shape, P can be related to Q via the Manning equation. For example, for trapezoidal swales, Wanielista et al. (1997) used the concept of the most efficient cross section to show that

$$L = \frac{K_u Q}{f}\left[b + 2\left\{ \frac{K_n n Q\left(1+z^2\right)^{1/3}}{S^{1/2} z^{2/3}\, 2\left[\left(1+z^2\right)^{1/2} - z\right]} \right\}^{3/8} \left(1+z^2\right)^{1/2} \right]^{-1} \tag{9-16}$$

where K_u and K_n are constants equal to 43,200 and 1.068, respectively, in U.S. customary units and 360,000 and 1.0 in S.I. units; Q is in cfs (m^3/s); b is the bottom width in ft (m); n is the Manning roughness coefficient, which can be taken as approximately 0.20 for routinely-mowed swales and 0.24 for infrequently-mowed swales (ASCE, 1998); z is the horizontal component of the side slope (i.e., zH:1V); and f is in units of in/hr (cm/hr). For triangular swales,

$$L = \frac{K_t Q^{5/8} z^{5/8} S^{3/16}}{n^{3/8}\left(1+z^2\right)^{5/8} f} \tag{9-17}$$

where K_t assumes a value of 21,032 in U.S. customary units and 151,361 in S.I. units. In some cases, the length required will be excessive or greater than the site allows. To overcome such a dilemma, small check dams or berms may be installed at intervals throughout the channel; this creates a wet swale which ultimately enhances treatment processes (i.e., settling, adsorption, uptake by vegetation, etc.) and provides additional storage for flow attenuation. Check dams might consist of several gabions (i.e., pervious, rock-filled baskets) with a weir overflow. Material used in construction of check dams should generally have a median diameter of 1 to 3 in (25 to 75 mm), and spacing of dams should not exceed the horizontal distance from the toe of the upstream dam to the same elevation at the top of the downstream dam.

9.2.2.8 Other Devices

Other devices and systems for controlling the impacts of urban runoff range from riprap and concrete flow spreaders, which dissipate kinetic energy at outfall locations, to water quality inlets and inserts (e.g., silt screens and oil/grit separators) that remove certain pollutants and floatable debris before they enter a collection system. Baffle boxes, installed in collection pipes for in-line removal of solids, are also considered within this category. These devices are generally limited to serving smaller basins that are less than 5 ac (2 ha) in size. However, they are relatively simple to implement and can greatly improve storm water control and pollutant removal efficiencies, particularly when used in combination with other BMPs.

9.2.3 Selection Guidelines

A variety of factors affect the screening and selection of BMPs to achieve specified runoff quantity and quality goals. These commonly include:

- local regulations or requirements;
- desired pollution removal efficiency;
- political and public support;
- condition and designated use of receiving waters;
- existence of nearby substitute, unimpaired waters;
- drainage area size and corresponding land uses;
- climate, including average rainfall frequency, duration, and intensity;
- soil types;
- depth to groundwater;
- site topography;
- soil types and geologic character (e.g., bedrock and karst formations);
- availability of land;
- future development and land uses;

- availability of supplemental water to support vegetative BMPs;
- susceptibility to freezing;
- safety;
- ability of the BMP to treat multiple pollutants and problems; and
- periodic and long-term maintenance needs, including accessibility.

Two of the most influential factors that are not explicitly mentioned in this list are cost and benefits to be earned from selected BMPs. Ideally, selection of BMPs should be based on the relationship between these two factors, or a cost-benefit analysis. To aid in the analysis, the USEPA (1999a) has compiled information from the literature that provides estimates for capital and annual maintenance costs for structural BMPs (see Tables 9-2 and 9-3). Unfortunately, while the data presented in these tables is useful for planning purposes, it is insufficient for making a comprehensive comparison of the cost effectiveness of different practices. The realized cost will likely include a variety of additional indirect and intangible costs. For example, it might consider resolution of local and regional social, economic, and political constraints, as well as a broad range of secondary environmental impacts.

Preparing a comparable list of specific benefits is equally difficult due to the lack of a clear and consistent definition of BMP performance and insufficient data; most data has focused on characterization of pollutant or discharge problems and not the quantifiable effectiveness of control measures. The USEPA has recently provided partial guidance with regard to assessing BMP effectiveness and, thus, benefits. Table 9-4 summarizes reported pollutant removal efficiencies for some of the more common structural BMPs (USEPA, 1999a). It should be noted, however, that attempts to broadly extend such data and related conclusions should be avoided since performance is heavily dependent on a host of site-specific factors such as the concentration of targeted pollutants, climate, seasonal temperature variation, and others. Similarly transferable data regarding BMP performance from a water quantity (i.e., flow reduction) perspective is even more uncommon, primarily due to the variability of watershed response to precipitation. The information that does exist, however, at least confirms a direct correlation between urbanization and degradation of receiving streams. Generally speaking, the degradation is particularly accelerated when impervious area in the watershed exceeds approximately five to ten percent of the total area (Pitt and Clark, 2003). In some cases, it may be possible to investigate the local level of control necessary to compensate for impervious areas greater than these critical values.

Table 9-2: Typical capital costs for BMPs

BMP	Cost [a] ($ per ft^3 of treated water)	Comments
Retention and detention basins	0.5 – 1.0	Cost range reflects economies of scale in designing BMP. The lowest unit cost represents approxmately150,000 ft^3 of storage, while the highest is approximately 150,000 ft^3. Dry detention basins are the least expensive design option.
Infiltration basin	1.3	Represents typical costs for a 0.25-acre infiltration basin.
Filter strip	0.0 – 1.3	Based on cost per square foot, assuming 6 inches of storage in the filter. The lowest cost assumes that the buffer uses existing vegetation, and the highest cost assumes that sod was used.
Sand filter	3.0 – 6.0	The range in costs is largely due to different possible designs. Surface and underground filters are among the most costly.
Bioretention basin	5.3	This BMP is relatively constant in cost because it is usually designed as a constant fraction of the total drainage area.
Constructed wetland	0.60 – 1.25	Costs are assumed to be approximately 25 percent more expensive than retention basins, primarily due to plant selection and sediment pre-treatment requirements.
Grass swale	0.50	Based on cost per square foot, assuming 6 inches of storage in the filter.

[a] 1997 U.S. dollars
Source: Adapted from USEPA (1999a)

For most non-structural BMPs, distinct measures of effectiveness have not been reported because they are generally unattainable. Surrogate measures of effectiveness, such as the degree of change exhibited in the population's habits or the degree of reduction in source pollutants, are more typically used. These measures might include the number of public workshops held per year, periodic surveys of residents' recycling habits, or the volume of material recycled each month.

Table 9-3: Annual maintenance costs for BMPs

BMP	Annual maintenance cost (percent of construction cost)
Detention basin	< 1
Retention basin	3 – 6
Sand filter	11 – 13
Bioretention basin	5 – 7
Constructed wetland	2
Grass swale	5 – 7
Infiltration basin	1 - 10

Source: Adapted from USEPA (1999a)

Table 9-4: Estimated pollutant removal efficiency

	Typical pollutant removal in percent				
	Suspended solids	Nitrogen	Phosphorus	Pathogens	Metals
Detention basin	30 -85	15 - 45	15 - 50	< 30	15 – 70
Retention basin	50 - 98	30 - 98	30 - 98	< 30	50 – 98
Constructed wetland	50 - 80	< 30	15 - 45	< 30	50 – 80
Infiltration basin	50 - 80	50 - 80	50 - 80	65 - 100	50 – 80
Grassed swale	30 - 83	< 45	15 - 45	< 30	15 – 45
Filter strip	50 - 80	50 - 80	50 - 80	< 30	30 – 65
Sand filter	50 - 95	< 47	41 - 80	< 30	20 – 80

Source: Adapted from USEPA (1999a) and ASCE (2001)

9.3 WATERSHED-SCALE PLANNING

Historically, the approach to water quality protection and the maintenance of pre-development runoff rates has been a patchwork of legislation and control measures that focus on individual sources of problems and individual properties. More recently, however, emphasis has been placed on formulating watershed management plans at broader, hydrologically-based scales. Specifically, decisions are based on spatial scales that correspond to entire watersheds. Thus, it provides a more logical and more effective basis for water resource management decisions than political boundaries. In particular, non-structural BMPs are most effective when implemented at such scales. This type of holistic approach also encourages more extensive stakeholder involvement and allows concerns to be addressed synergistically with greater economy and efficiency.

Since the early- to mid-1990s, the USEPA has publicly recognized the importance of watershed-scale planning as part of water quantity and quality control efforts, and applications of the approach are becoming more common. Unfortunately, in many cases, describing the approach is far easier than its successful implementation. Conflicting stakeholder objectives, entrenched opinions and past relationships, funding difficulties, and numerous uncertainties are only a few of the constraints that must be overcome if benefits are to be realized.

9.4 THE ROLE OF OPTIMIZATION

As focus shifts toward watershed-scale management of urban storm water, recent research has shown that optimization and search methods provide unique benefits in the cost-effective design of certain BMPs. The majority of previous work is based on the coupling of various hydrologic and environmental simulation models with evolving search techniques. The search method iteratively, but strategically, determines decision variable values that maximize pollutant reduction goals or minimize cost required to meet water quality criteria. Here, decision variables can represent land use allocation, types of control measures to be implemented, or BMP sizing and placement criteria. The simulation model is then called automatically each time the search method requires information about the state (e.g., pollutant concentration in receiving streams or discharge quantities at specified locations) of the watershed in response to a prescribed set of decision variables.

The benefits of this integrative approach are discussed by Nicklow (2000) and include a reduced size and complexity of the overall design and optimization problem since the simulation model implicitly solves governing hydrologic and environmental constraints. At the same time, the complexity of the physical system is maintained and additional model simplifications are not typically necessary to reach an optimal or near-optimal result. Historically, the use of traditional, gradient-based optimization techniques within this overall approach was problematic; such techniques typically required significant derivative information about the simulation equations that was difficult to obtain efficiently, if at all. However, when used with evolutionary search algorithms (i.e., Genetic Algorithms, Simulated Annealing, and Strength Pareto Evolutionary Algorithms) that require no gradient information, the integrative approach is relatively efficient and can lead to highly-effective, basin-wide storm water control.

Major contributions in this field include that of Dorn et al. (1995), Harrell and Ranjithan (2003), Perez-Pedini et al. (2004), Zhen et al. (2004) and several others who have demonstrated a variety of methods that can be used to evaluate the most cost-effective, system-wide configuration of detention basins or similar BMPs to meet watershed pollutant removal levels. Likewise, researchers have applied multiobjective search methods to directly generate solutions for minimizing system cost while maximizing the effects of detention systems (Yeh and Labadie, 1997; Dorn and Ranjithan, 2003).

From a land-use planning perspective, Nicklow and Muleta (2001) developed a computational model to identify landscapes (i.e., land use and land cover) and management practices, in both a spatial and temporal sense, that could be implemented to minimize erosion. The method was subsequently expanded to yield multiple tradeoff solutions for the multiobjective case of maximizing landowner profit and minimizing non-point source pollution (Muleta and Nicklow, 2005; Bekele and Nicklow, 2005). With anticipated improvements in computer speed and capacity, in hydrologic and environmental simulation models, and in optimization and search methods, it is expected that the application of optimization to BMP design and to watershed management will become more common over time.

9.5 SOLVED PROBLEMS

Problem 9.1 Detention volume (Abt and Grigg)

The peak runoff and time to peak discharge for a 20-ac watershed are 120 cfs and 35 minutes, respectively. The excess rainfall from the design storm event is estimated to be 2.8 in. Use the Abt and Grigg (1978) method to estimate the detention basin storage required to limit the peak discharge to 35 cfs.

Solution

The volume of runoff is computed using the depth of excess rainfall,

$$V_r = \left(\frac{2.8}{12}\right)\left(20 \times 43{,}560\ ft^2/ac\right) = 203{,}280\ ft^3 = 4.67\ ac\text{-}ft$$

Using the Abt and Grigg (1978) formula (see Table 9-1),

$$\frac{V_s}{V_r} = \left(1 - \frac{O_p}{I_p}\right)^2 = \left(1 - \frac{35}{120}\right)^2 = 0.50$$

Then, the volume of storage required is

$$V_s = 0.50(4.67) = \underline{\underline{2.34\ \ ac\text{-}ft}}$$

Problem 9.2 Detention volume (Wycoff and Singh)

Solve Problem 9.1 using the Wycoff and Singh (1986) method.

Solution

The base time of the inflow is evaluated as the time from the beginning of runoff to a point on the recession limb where flow is five percent of I_p. Thus, for the triangular inflow hydrograph,

$$t_b = (2 \times 35) - 35\left[\frac{0.05(120)}{120}\right] = 68.25\ min.$$

Using the Wycoff and Singh (1986) formula,

$$\frac{V_s}{V_r} = \frac{1.291\left(1 - \frac{O_p}{I_p}\right)^{0.753}}{\left(t_b/t_p\right)^{0.411}} = \frac{1.291\left(1 - \frac{35}{120}\right)^{0.753}}{(68.25/35)^{0.411}} = 0.76$$

so that the volume of storage required is

$$V_s = 0.76(4.67) = \underline{\underline{3.55\ \ ac\text{-}ft}}$$

Problem 9.3 Detention volume (AASHTO)

Solve Problem 9.1 using the AASHTO (1991) method.

Solution

With the base time defined as twice the time to peak discharge,

$$V_s = \frac{1}{2} t_b \left(I_p - O_p\right) = \frac{1}{2}(70 \times 60)(120 - 35) = 178{,}500\ ft^3 = \underline{\underline{4.09\ ac\text{-}ft}}$$

Problem 9.4 Detention volume (Kessler and Diskin)

Assuming a weir outlet, solve Problem 9.1 using the Kessler and Diskin (1991) method.

Solution

From the information given in the problem statement, $\left(O_p / I_p\right)$ = 0.3 and is within the acceptable range of 0.2 to 0.9. Then

$$\frac{V_s}{V_r} = 0.932 - 0.792\left(\frac{35}{120}\right) = 0.70$$

and the volume required is expressed as

$$V_s = 0.70(4.67) = \underline{\underline{3.27\ ac\text{-}ft}}$$

Problem 9.5 Detention volume (Kessler and Diskin)

Assuming an orifice outlet, solve Problem 9.1 using the Kessler and Diskin (1991) method.

Solution

Using the corresponding formula from Table 9-1 yields

$$\frac{V_s}{V_r} = 0.872 - 0.861\frac{O_p}{I_p} = 0.872 - 0.861\left(\frac{35}{120}\right) = 0.62$$

and the volume required is expressed as

$$V_s = 0.62(4.67) = \underline{\underline{2.90\ ac\text{-}ft}}$$

which is slightly lower than similar computations for a weir outlet. Note that storage values range from 2.34 to 4.09 ac-ft, depending on the method used.

Problem 9.6 Detention volume (Modified rational method)

Using the modified rational method, estimate the maximum detention basin storage required for a 30-ac drainage area having a runoff coefficient of 0.9 and a time of concentration of 25 minutes. The basin should reduce the peak discharge to 25 cfs. Assume that rainfall intensity for the area is expressed as $\frac{101.2}{(t_d + 15.6)^{0.92}}$ in/hr, where t_d is the rainfall duration in minutes.

Solution

Rainfall intensity is expressed in a form similar to that of Equation 9-3. Therefore, the fitting coefficients are a = 101.2, b = 15.6, and c = 0.92. Equation 9-4 can be expressed as

$$\frac{a[t_d(1-c)+b]}{(t_d+b)^{c+1}} - \frac{O_p K_r}{2CA} = \frac{101.2[t_d(0.08)+15.6]}{(t_d+15.6)^{1.92}} - \frac{25}{2(0.9)(30)} = 0$$

Solving this expression by trial-and-error or by using numerical analysis yields a critical storm duration of 64.71 minutes. The intensity associated with this duration is

$$i = \frac{101.2}{(64.71+15.6)^{0.92}} = 1.79 \ in/hr$$

and, using the rational equation, the peak inflow is

$$I_p = CiA = (0.9)(1.79)(30) = 48.3 \ cfs$$

The maximum volume of storage required can then be evaluated using Equation 9-2, or

$$V_s = \left[(48.3)(64.71) - 25\left(\frac{64.71+25}{2}\right)\right] \times 60 = 120{,}247 \ ft^3 = \underline{\underline{2.76 \ ac\text{-}ft}}$$

Problem 9.7 Detention volume (FAA)

Solve Problem 9.6 using the FAA (1970) method. Use the rational method to estimate peak runoff conveyed to the basin.

Solution

The FAA (1970) method used here integrates the modifications suggested by Guo (1999). The corresponding solution is based on Equation 9-5 and is summarized in Table P9-7. For different assumed durations, the rainfall intensity is computed and used with the rational equation to evaluate runoff volume (i.e., $I_p t_d \times 60$) in Column 3. The outflow volume in Column 5 is computed as ($m O_p t_d \times 60$), noting that the value of m varies with duration as indicated by Equation 9-6.

Table P9-7

(1)	(2)	(3)	(4)	(5)	(6)
Duration (min)	Rainfall Intensity (in/hr)	Runoff Volume (ft^3)	m	Outflow Volume (ft^3)	Storage (ft^3)
40	2.51	162,648	0.81	48,600	114,048
45	2.32	169,128	0.78	52,650	116,478
50	2.16	174,960	0.75	56,250	118,710
55	2.02	179,982	0.73	60,225	119,757
60	1.89	183,708	0.71	63,900	119,808
65	1.78	187,434	0.69	67,275	120,159
68	1.72	189,475	0.68	69,360	120,115
69	1.71	191,144	0.68	70,380	**120,764**
70	1.69	191,646	0.68	71,400	120,246
75	1.60	194,400	0.67	75,375	119,025

For example, at a duration of 40 minutes, intensity and runoff volume are evaluated as

$$i = \frac{101.2}{(40 + 15.6)^{0.92}} = 2.51 \ in/hr$$

$$I_p t_d = CiA = (0.9)(2.51)(30)\left(40 \times 60 \frac{sec}{min}\right) = 162{,}648 \ ft^3$$

The corresponding adjustment factor, m, and the outflow volume are

$$m = \frac{1}{2}\left(1 + \frac{25}{40}\right) = 0.81$$

$$mO_p t_d = 0.81(25)\left(40 \times 60 \frac{sec}{min}\right) = 48{,}600\ ft^3$$

The difference between inflow and outflow volumes, or 162,648 – 48,600 = 114,048 ft^3 in this case, is tabulated in Column 6. The maximum difference occurs at a duration of 69 minutes and is equal to 120,764 ft^3, or 2.77 ac-ft. This is approximately the same answer obtained for the previous problem, which is expected given the similarity of methods and the assumptions used.

Problem 9.8 Detention time

The table below describes inflow and outflow from a storm water detention basin. Determine the detention time of the facility.

Time (min)	Inflow (m^3/s)	Outflow (m^3/s)	Time (min)	Inflow (m^3/s)	Outflow (m^3/s)
0	0.0	0.0	240	2.4	4.0
30	4.8	0.2	270	1.2	3.6
60	9.6	1.3	300	0.0	3.1
90	8.4	2.5	330	0.0	2.6
120	7.2	3.4	360	0.0	2.2
150	6.0	3.9	390	0.0	1.8
180	4.8	4.2	420	0.0	1.5
210	3.6	4.1	450	0.0	1.3

Solution

From Haan et al. (1994), the detention time is the temporal change between the centroids of the inflow and outflow hydrographs. The centroid of the inflow hydrograph can be approximated as

$$\overline{t_I} = \frac{\sum_{j=1}^{J} t_j I_j}{\sum_{j=1}^{J} I_j}$$

where J refers to the total number of data points on the hydrograph. For the current problem, the denominator of this expression is equal to the sum of the inflows, or 48 m^3/s, and the numerator is

$$[(30)(4.8)+(60)(9.6)+(90)(8.4)+...+(270)(1.2)]\times 60$$

which yields 345,600 m^3/s. Then,

$$\frac{345{,}600}{48}\times\frac{1}{60}\ min/sec = 120\ \ min$$

Using a similar expression for the outflow hydrograph,

$$\overline{t_O} = \frac{\sum_{j=1}^{J} t_j O_j}{\sum_{j=1}^{J} O_j}$$

yields a centroid at 235 minutes. Thus, the detention time of the basin is 235 – 120, or $\underline{\underline{115\ \text{min}}}$.

Problem 9.9 Infiltration basin

A planned infiltration basin is to have a bottom elevation of 102 ft and capture a volume of 0.3 ac-ft. Field measurements indicate that the soil underlying the basin has a porosity of 0.39 and an initial water content of 0.10. If the groundwater table lies at elevation 91 ft, determine the minimum bottom area and maximum depth of the basin.

Solution

Using the data provided, the minimum basin area is computed from Equation 9-7, expressed as

$$A_{min} = \frac{0.3}{(102-91)(0.39-0.10)} = 0.094\ ac = \underline{\underline{4{,}097\ ft^2}}$$

which is approximately equal to a 64-ft square. The maximum basin depth is

$$(102-91)(0.39-0.10) = \underline{\underline{3.2\ ft}}$$

Problem 9.10 Infiltration basin

If the basin described in Problem 9.9 has a saturated infiltration rate of 5.0 in/hr, determine the associated drain time of the basin.

Solution

From Equation 9-8,

$$T_D = \frac{(102-91)(0.39-0.10)}{5.0/12} = \underline{\underline{7.65\ hrs}}$$

This value is well within the typical interstorm period. Note, however, that if clogging occurs, drain time increases accordingly. For example, if infiltration is reduced by 50 percent, drain time is doubled.

Problem 9.11 Infiltration trench

An infiltration trench will be used to control the quality of runoff from a 120 ft × 200 ft parking lot. The porosity of backfill material is 0.40 and the surrounding soil has an infiltration rate of 1.2 in/hr. Assuming a desired drawdown time of 48 hrs and assuming the depth to high groundwater is 15 ft, determine the required trench dimensions.

Solution

The WQV to be captured is evaluated as

$$V_c = \left(\frac{0.5}{12}\right)(120)(200) = 1{,}000\ ft^3$$

The maximum trench depth can be determined using Equation 9-10,

$$d_{max} = \frac{(1.2/12)(48)}{0.40} = 12\ ft$$

which is sufficiently above the groundwater table. Since only water quality concerns are considered, Equation 9-11 can be expressed as

$$\phi(LWd) = V_c$$

Then, assuming a width of 6 ft, the required length is computed using

$$0.40(L \times 6 \times 12) = 1{,}000$$

or $L = 34.7$ ft. Thus, a trench that is 6 ft × 12 ft × 35 ft should effectively treat the runoff. A filter strip should be considered for the perimeter of the trench, and trench aggregate should range from 1.5 to 3.0 inches in diameter.

Problem 9.12 Filter strip

Determine the dimensions of a filter strip to capture a design flow of 0.5 cfs at a constant rainfall intensity of 0.17 in/hr. The filter is to have a longitudinal slope of 0.045 ft/ft and a Manning roughness coefficient of 0.20 (i.e., combination of very dense grass and shrubs).

Solution

Assuming a maximum depth of 1.0 inch, the unit-width flow is evaluated using Equation 9-12, as follows:

$$q = \frac{1.49}{0.20}\left(\frac{1.0}{12}\right)^{5/3} 0.045^{1/2} = 0.025 \ cfs/ft$$

The minimum filter width, W, is then

$$W = \frac{Q}{q} = \frac{0.5}{0.025} = 20 \ ft$$

Using a conservative ten-minute retention time, the corresponding length is computed as

$$L_f = \frac{t_c^{5/3} i^{2/3} S^{1/2}}{K_f n} = \frac{(10)^{5/3}(0.17)^{2/3}(0.045)^{1/2}}{(0.899)(0.20)} = 16.8 \ ft$$

which is less than the minimum suggested length of 20 ft. Therefore, the filter should be at least 20 ft × 20 ft, and, if land is available, an even larger length (i.e., 20 ft wide × 50 ft) could be specified to improve treatment performance.

Problem 9.13 Sand filter

Design a sand filter to treat the water quality volume associated with a 4-ha, 75 percent impervious, commercial development. A sedimentation basin is to be used to pre-treat runoff and steadily releases flow to the filter over a 24-hr period. Assume a drawdown time of 1.5 days and a coefficient of permeability of 1.1 m/d.

Solution

Assuming a 0.5 in (0.013 m) runoff for the water quality volume, the capture volume, V_c, is computed as

$$V_c = (0.013)(0.75)(40{,}000) = 390\ m^3$$

The sedimentation basin releases flow at a rate of 390/24, or 16.25 m^3/hr, and flow will infiltrate through the sand at 390/36, or 10.83 m^3/hr. The daily volume of runoff that will accumulate on the sand surface is equal to

$$(16.25 - 10.83)(24) = 130\ m^3$$

at a maximum depth of $130/A_{min}$. Assuming that the average depth is half of the maximum depth and that the bed is 0.5 m deep, the minimum surface area of the bed can be computed as

$$A_{min} = \frac{(390)(0.5)}{1.1\left(\dfrac{130/A_b}{2} + 0.5\right)1.5} = 106.4\ m^2$$

Thus, a 1.5 m wide × 75 m long × 0.5 m deep filter would satisfy the requirements of the site.

Problem 9.14 Bioretention basin

Determine the size of bioretention basin necessary to capture and treat 0.05 ac-ft of runoff. The planting soil to be used has an infiltration capacity of 1.0 in/hr and extends to a depth of 4 ft below the filter bed.

Solution

Assuming a maximum ponding depth of 0.5 ft and that drawdown should occur within 40 hrs, the minimum area of the bed (i.e., invert) can be computed as

$$A_b = \frac{(0.05 \times 43{,}560)(4)}{(1/12)(0.5 + 4)(40)} = 580\ ft^2$$

So a 15 ft × 40 ft (i.e., 600 ft^2) basin would seem adequate. However, the temporary basin volume above the mulch layer should be checked to ensure that the capture volume (i.e., 0.05 × 43,560 = 2,178 ft^3) can be accommodated at reasonable depth. For a trapezoidal basin having a length L, width W, and side slopes of zH:1V, the following expression can be solved for depth:

$$V = 2{,}178 = LWD + (L + W)zD^2 + \frac{4}{3}z^2D^3$$

Using this expression with $W = 15$ ft, $L = 40$ ft, and $z = 8$ yields a depth of 1.5 ft which exceeds the maximum ponding limit of 0.5 ft, so the basin dimensions must be increased. For example, a basin that is 45 ft × 85 ft, with 8H:1V side slopes will store the capture volume at a satisfactory depth of 0.5 ft.

Problem 9.15 Grass swale

Design a grass swale to accommodate a design runoff of 0.5 m^3/s. Mowed Bermuda grass (retardance class D) is to be used, and the depth should be no more than 0.3 m. Based on topography, the swale's longitudinal slope is to be 2.0 percent.

Solution

Using a trapezoidal section with side slopes of 5H:1V, the area and hydraulic radius for a depth of 0.3 m can be expressed as

$$A = (b + zy)y = [b + (5)(0.3)]0.3 = 0.3b + 0.45$$

and

$$R = \frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}} = \frac{0.3b + 0.45}{b + (2)(0.3)\sqrt{1 + 5^2}} = \frac{0.3b + 0.45}{b + 3.06}$$

Substituting these parameters into the Manning equation (see Chapter 8) yields

$$Q = 0.5 = \frac{1.0}{n}\frac{(0.3b + 0.45)^{5/3}}{(b + 3.06)^{2/3}}0.02^{1/2}$$

This relationship can be solved for b according to assumed values of n. The value of n, however, must be consistent with Equation 8-50 and 8-51, written for class D retardance. For example, assuming a roughness coefficient of 0.04, the channel bottom width is computed as 0.14 m using the Manning equation. From the previous relationship for hydraulic radius, R is evaluated as 0.154 m, or 0.505 ft. An updated roughness coefficient can then be computed by combining Equations 8-50 and 8-51, or

$$n = 0.505^{1/6}\left[34.6 + 19.97\log\left\{\left(0.505^{1.4}\right)\left(0.02^{0.4}\right)\right\}\right]^{-1} = 0.070$$

Thus, a second iteration is necessary in which n = 0.070 is used to solve for b. The following summarizes the iterative solution, which yields a final bottom width of 0.83 m.

Assumed n	b (m)	R (m)	R (ft)	Computed n
0.04	0.14	0.154	0.505	0.07
0.07	1.03	0.186	0.61	0.061
0.061	0.77	0.178	0.584	0.063
0.063	**0.83**	0.18	0.591	**0.063 (OK)**

Velocity can then be checked by

$$V = \frac{Q}{A} = \frac{0.5}{\left[0.83 + (5)(0.3)\right]0.3} = 0.72\,\frac{m}{s}$$

which is in an acceptable range. Using a minimum retention time of 5 minutes, the length of the swale should be at least

$$L = (0.72)(5 \times 60) = 216\ m$$

The swale should have a trapezoidal cross section, be at least 216 m long, and have a bottom width of 0.83 m and side slopes of 5H:1V.

Problem 9.16 Grass swale

Determine the minimum length of triangular grass swale necessary to drain a section of highway. The swale should fully infiltrate the design flow of 1.0 cfs, which occurs over a period of 30 minutes. Assume a routinely-mowed surface having a slope of 0.02 ft/ft and a saturated infiltration rate of 4.0 in/hr.

Solution

Assuming side slopes are 5H:1V and a roughness coefficient of 0.20, the required length can be computed from Equation 9-17, expressed as

$$L = \frac{K_t Q^{5/8} z^{5/8} S^{3/16}}{n^{3/8}\left(1+z^2\right)^{5/8} f} = \frac{(21{,}032)(1.0)^{5/8}(5)^{5/8}(0.02)^{3/16}}{(0.20)^{3/8}\left(1+5^2\right)^{5/8}(4.0)} = 1{,}648\ ft$$

Problem 9.17 Check dam

Assume that the swale length computed in Problem 9.16 exceeds the physical limitations of the site. Specify the additional storage volume that must be created by a series of check dams if the length of the swale is to be no longer than 1,000 ft.

Solution

The additional storage volume required can be evaluated by first letting L assume a value of 1,000 ft and solving Equation 9-17 for Q as follows:

$$Q = \left[\frac{Ln^{3/8}\left(1+z^2\right)^{5/8} f}{K_t z^{5/8} S^{3/16}}\right]^{8/5} = \left[\frac{(1{,}000)(0.20)^{3/8}\left(1+5^2\right)^{5/8}(4.0)}{(21{,}032)(5)^{5/8}(0.02)^{3/16}}\right]^{8/5} = 0.45\ cfs$$

The additional storage that should be created through use of the dams is

$$(1.0 - 0.45)\left(30 \times 60 \frac{sec}{min}\right) = \underline{\underline{990\ ft^3}}$$

Site and swale geometry should be evaluated to ensure that this volume exists without causing overflow and to determine the number of check dams needed.

Problem 9.18 BMP selection

Consider a highly-urbanized, 5 acre (2 ha) drainage area that is 80 percent impervious. The site houses two gas stations with large, paved parking areas. The available space for constructing treatment BMPs is limited to two to three percent of the total basin, so subsurface BMPs would generally be desirable. Unfortunately, low-permeability soils in the area limit the use of infiltration systems. Evaluate and select BMP alternatives to reduce suspended solids, oil, and grease loadings to the existing storm sewer and eventually to a nearby watercourse.

Solution

Important considerations and limitations for the site include:

- Subsurface BMPs with a modular design should be utilized to the extent possible so that land overlying the BMP can be utilized for other purposes;
- Infiltration systems should not be used due to the low permeability of surrounding soils;

- The selected BMP should be capable of operating under a range of hydraulic heads since the existing storm drain will limit the amount of hydraulic drop (i.e., elevation change) that can be made available within the new BMP; and
- Oil and grease will attach to suspended solids and float on top of storm water runoff.

Structural BMPs most likely to meet these criteria are an underground sand filter - likely to achieve a 50 to 90 percent removal of suspended solids - and an inlet device such as an oil/grit separator. These measures must, however, be examined in light of funding that is available for construction, operation, and maintenance. Note that sand filters are among the most costly alternatives with respect to both capital and maintenance expenditures. If funding is insufficient, other BMPs might need to be considered, even if they provide a lower level of performance. In addition to treatment units, non-structural BMPs in the form of education and training of site personnel, sweeping of streets and parking areas, and on-site containment measures should be considered. These will limit the amount of larger debris and solids that enter the sewer, minimize the amount of fuel spilled, and improve response actions in the event of a spill.

Problem 9.19 BMP selection

Consider a 1,000-ft length of four-lane, metropolitan highway that drains to a major river system. Flooding in the river is not a significant concern, but phosphorus and suspended solids loadings have reached critical levels and BMPs are being considered. The area for control measures is limited to the median and shoulder, and groundwater levels are in close proximity to the surface. The site is relatively flat and is highly visible to passersby. Identify a suitable BMP or combination of BMPs to reduce solids and phosphorus yields.

Solution

BMPs that are not well suited include detention and retention basins, wetlands, filter strips, and infiltration systems. Those that could be considered are bioretention cells, grass swales, and sand filters. Although bioretention facilities could potentially achieve high levels of treatment, they may be difficult to size and place given physical limitations of the site. The combination of a median swale and sand filter, however, would fit well into a linear highway configuration and should yield an adequate level of treatment. This combination is also quite aesthetically acceptable, although costs for the controls would likely be moderate to high. These BMPs should be implemented simultaneously with a litter control program (i.e., adopt-a-highway).

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