18 PRINCIPLES OF QUANTITY CONTROL

18.1 INTRODUCTION

Stormwater quantity control facilities can be classified by function as either detention or retention facilities. The term control facilities is used in this manual to describe any combination or arrangement of detention and retention facilities in urban stormwater management systems. A classification system based on the location of the facility is defined herein which will be used throughout the manual.

The detention concept is most often employed in urban stormwater drainage systems to limit the peak outflow rate for a specific range of flood frequencies to that which existed from the same catchment before development. The primary function of detention facilities is to reduce peak discharge by the temporary storage and gradual release of stormwater runoff by way of an outlet control structure or other release mechanism. Detention facilities, which provide for slow release of stormwater over an extended period of several days or more, are referred to as extended detention facilities.

True retention facilities reduce runoff volume, and possibly peak discharge, by the temporary storage of stormwater runoff, which is subsequently released via evaporation and infiltration only.

Detention and retention facilities can reduce the peak and volume of runoff from a given catchment (Figure 18.1), which can reduce the frequency and extent of downstream flooding. Detention/retention facilities have been used to reduce the costs of large stormwater drainage systems by reducing the size required for such systems in downstream areas.

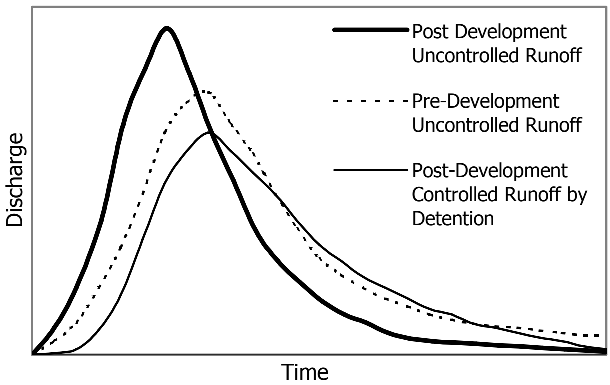

Figure 18.1 Hydrograph Schematic

The reduced post-development runoff hydrograph is typically designed so that the peak flow is equal to or less than the pre-development peak flow rate. Additionally, in some instances, the volume of the post-development hydrograph is required to be reduced to the same volume as the pre-development runoff hydrograph. This latter requirement will necessitate the use of retention facilities to retain the difference in volume between the post and pre-development hydrographs.

Retention facilities are commonly sized to provide only a reduction in the volume of stormwater runoff generated from an urban area. However, peak flow reduction can also be achieved in minor storm events if the storage volume is large enough to capture the peak flow before the storage is filled, i.e. the time to fill the basin is longer than the time to peak of the inflow hydrograph.

Detention and retention storages may be classified on the basis of their location and size as follows (see Figure 18.2);

on-site storage : small storages constructed on individual residential, commercial, and industrial lots

community storage : storage facilities constructed in public open space areas, or in conjunction with public recreation and sporting facilities

regional storage : large community storage facilities constructed at the lower end of catchments prior to discharge to receiving waters

Facilities can also be categorised as:

on-line storage : a facility that intercepts flow directly within a conveyance system. On-line storage occasionally is provided as an on-site facility, though it is more often a community or regional facility

off-line storage : the diversion of flow from a conveyance system into a separate storage facility. A typical example is a side channel spillway that diverts flows from the conveyance system into an adjacent storage facility

conveyance storage : Detention may also be provided as conveyance storage. This is an often-neglected form of storage, because it is dynamic and requires channel storage routing analysis to identify. Slower-flowing conveyance caused by flatter slopes or rougher channels can markedly retard the build up of flood peaks and alter the time response of the tributaries in a catchment.

18.2 DETENTION FACILITIES

The most common type of storage facilities used for controlling peak flow are 'dry facilities', which release all the runoff temporarily detained during a storm. Other facilities which are becoming more commonly used are detention 'ponds', which incorporate a permanent pool of water for water quality control as well as provision for the temporary storage and release of runoff for flood control.

Detention facilities should be provided only where they are shown to be beneficial by hydrologic, hydraulic, and cost analyses.

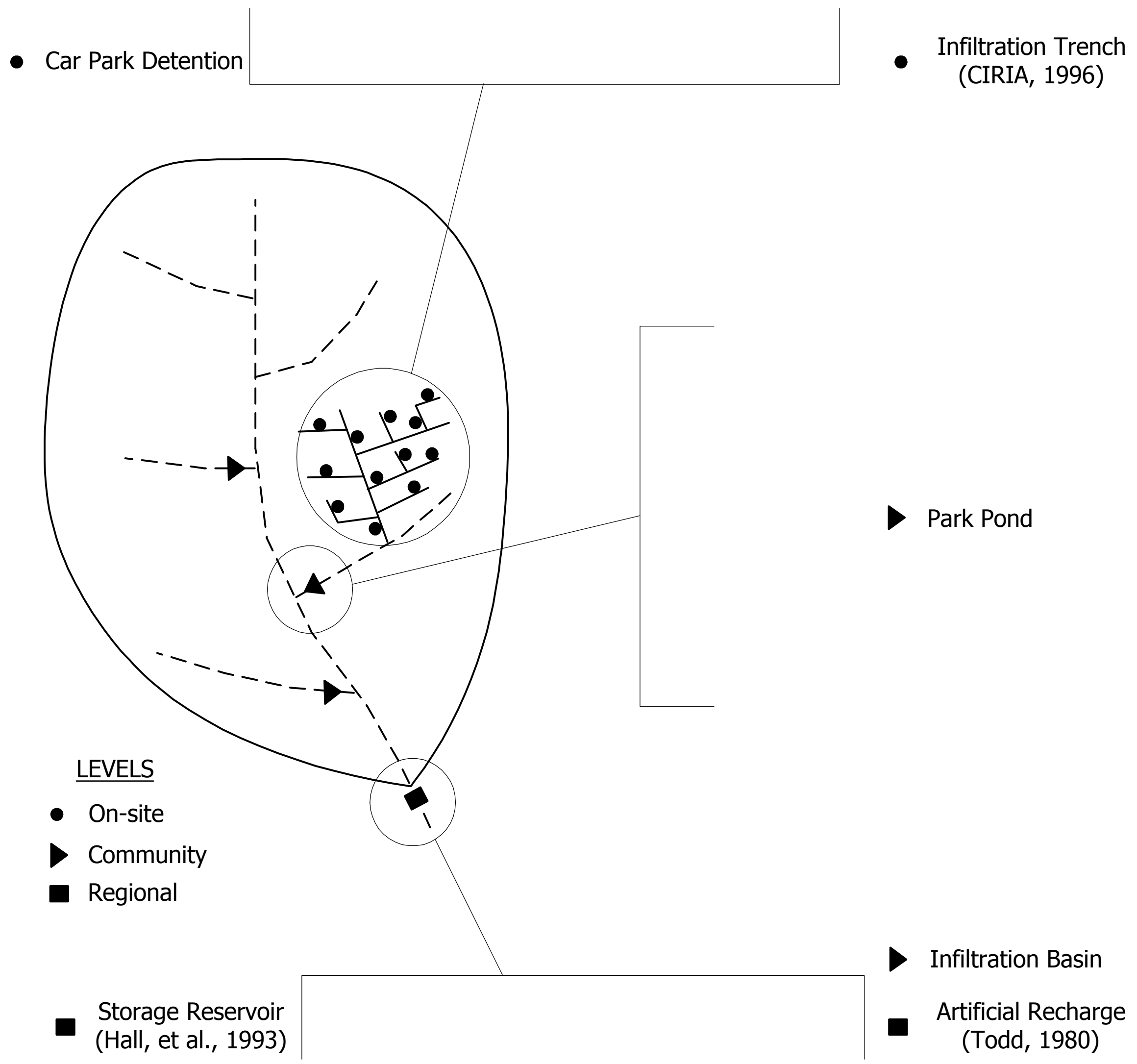

Figure 18.2 Detention/Retention Storage Classifications

18.2.1 On-site Detention

On-site detention (OSD) may be provided as above-ground storages, below-ground storages, or a combination of both. The main advantages of above-ground storages are they can generally be easily incorporated into the site by slight regrading or modification to the design of surface features and are relatively inexpensive compared to below-ground storages. The main advantages of below-ground storages are they are out of sight, occupy less physical space, and will not cause any inconvenience with ponding of water that could result using above-ground storage. The common types of above and below-ground storages used are illustrated in Figure 18.3.

(a) Above-ground Storages

(i) Landscaped areas

Landscaped areas such as lawns and garden beds offer a wide range of possibilities for providing surface storage and can enhance the aesthetics of a site. Careful consideration must be given to the design of the storage outlet as landscaped storages are most susceptible to blockage by wash off of leaves and lawn clippings.

(ii) Impervious areas

Car parks, driveways, paved storage yards, and other paved surfaces may be used for stormwater detention. The depth of ponding on these surfaces should be limited to minimise nuisance conditions for residents and water damage to vehicles.

(iii) Flat Roofs

Stormwater can be detained on flat roofs provided that adequate protection against leakage is provided in the structural design of the building. This type of storage has limited application in residential areas and is more suited to commercial and industrial buildings where flat roofs are more common. Careful consideration must also be given in design for the anticipated design hydraulic loadings and adequate provision made for bypassing flows in the event of a blockage of roof outlets.

(iv) Surface Tanks

Surface tanks may be provided to reduce flows from building roofs. If a rainwater harvesting tank is to be utilised for detention, the detention storage must be in addition to the storage provided for rainwater harvesting.

(b) Below-ground Storages

(i) Underground Tanks

Typical storages are circular or rectangular tanks in form and normally constructed of reinforced concrete. Due to their structural nature, tanks can be configured into almost any geometrical plan shape. Tanks are the relatively expensive compared to above-ground storages due to high construction costs. The main advantage with tanks is that they can take up less area of the site and therefore can minimise the size of drainage reserves.

(ii) Pipe Packages

A *pipe package* consists of one or more parallel rows of buried pipes connected by a common inlet and outlet chamber. The configuration of the pipes can be shaped to suit any available site space by providing horizontal bends. Pipe packages are less expensive to construct than tanks as commercially available pipes may be used.

(c) Combined Storages

With combined storages, a proportion of the total storage is provided as below-ground storage, whilst the remainder of the storage is provided as above-ground storage. The frequency of ponding as well as depth and extent of ponding in the above-ground storage is thus limited to minimise inconvenience to property owners or occupiers. Flooding which causes occasional inconvenience is generally acceptable, provided it does not cause any damage to property.

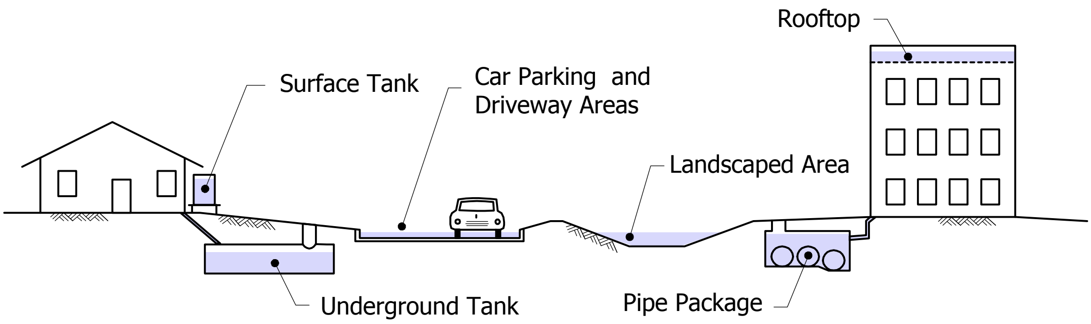

Figure 18.3 Typical OSD Storage Facilities

18.2.2 Community and Regional Detention

Community and regional detention facilities are larger facilities than OSD which are provided in public areas outside of private properties. Facilities are commonly formed by the construction of an embankment across a stream or stormwater conveyance and/or the excavation of a basin storage area.

The storage basin is usually drained via an ungated outlet through the base of the embankment. An overflow spillway set near the top of the embankment is required to safely discharge larger storms that exceed the basin capacity.

Community and regional detention facilities should be analysed using a hydrograph estimation technique and preferably using a computer model (refer Chapter 14 and 17). Under no circumstances shall such a facility be sized using hydrographs estimated by the Rational Method.

Dry basins and flood storage reservoirs are the types of detention facilities normally provided for community and regional detention (refer Figure 18.4).

(a) Dry Basins

These are basins that are normally dry or empty when not in operation, i.e. they do not have a permanent water pond or pool associated with them. Dry detention basins are particularly suited to multi-purpose use and sites may be selected to incorporate passive or active recreation areas such as public parks and open space, or sporting facilities such as soccer fields.

(b) Flood Storage within Ponds and Lakes

The primary function of water quality control ponds and urban lakes is to provide water quality control by means of a permanent pond area, but they may also be designed to incorporate temporary flood storage above the permanent pond for flood control purposes. In such facilities, the permanent pond is termed *permanent storage* while the flood storage component is termed *active storage*.

18.3 RETENTION FACILITIES

Some developed countries have elected to encourage or mandate local disposal of stormwater at its source of runoff. This is done by having a portion of the stormwater infiltrate or percolate into the soil. Typically, it has been used to control stormwater from individual residential lots. This approach has received greater attention locally. The advantages often cited for the use of local disposal include:

1. recharge of groundwater
2. reduction in the settlement of the land surface in areas of groundwater depletion
3. control of saline water intrusion
4. preservation and/or enhancement of natural vegetation
5. reduction of pollution transported to the receiving waters
6. reduction of downstream flow peaks
7. reduction of basement flooding in underground drainage systems
8. smaller storm drains at a lesser cost

Equally good arguments are made against the use of local disposal systems. As a result, the use of local disposal systems should be addressed on a site-specific basis within each urban community.

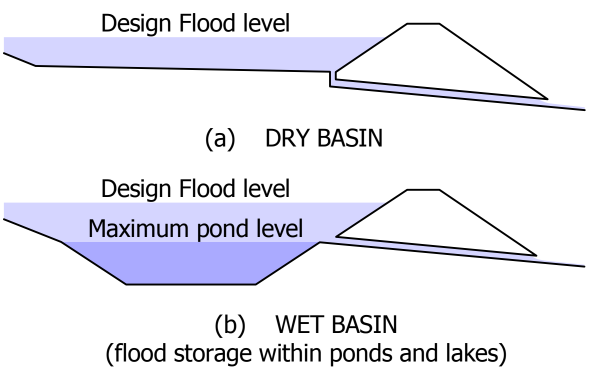

Figure 18.4 Community and Regional Basin Types

Retention facilities as defined here include extended detention facilities, infiltration structures, recharge wells and swales. In addition to stormwater storage, retention may be used for water supply, recreation, pollutant removal, aesthetics, and/or groundwater recharge. Infiltration facilities provide significant water quality benefits, and although groundwater recharge is not a primary goal of stormwater management, the use of infiltration basins and wells can provide this secondary benefit. Retention facilities are typically designed to provide the dual functions of stormwater quantity and quality control. These facilities may be provided at one or more locations and may be both above-ground or below-ground. These locations may exist as collection and conveyance facilities (swales or perforated conduits), and on-site facilities such as parking lots and roadways using porous pavements.

Design criteria for retention facilities are the same as those detention facilities except that it may not be necessary to remove all runoff after each storm.

18.3.1 On-Site and Community Retention

The main types of retention/infiltration techniques are infiltration trenches, soakaway pits, porous pavement and infiltration basins.

(a) Infiltration Trench

An infiltration trench is a trench in which the permeable fill material extends to the ground surface and overland flow discharges onto the top of the trench along its length i.e. there is no traditional pipe and gully drainage system collecting and conveying stormwater to the trench. The top of the trench must retain an infiltration rate sufficient to allow for inflows from the surface at the design rainfall intensity multiplied by the surface area ratio (i.e. ratio of the area draining to the trench to the top surface area of the trench). Figure 18.5 illustrates one form of trenches.

(b) Soakaway Pit

A soakaway pit has been the traditional method of disposal of stormwater in many western countries where no drain or conveniently close watercourse existed. Often on single dwellings, the soakaway consisted of a roughly dug pit filled with rubble or hardcore to which the storm drain discharged. There was often no means of cleansing or access to the end of the drain and blockage around the outfall into the pit was common. Before the production of geotextiles, the soakaway pits were subject to infilling by soil from above or from the sides as local percolation of groundwater transported material into the pits. (see Figure 18.6).

Precast concrete ring unit soakaways are now in common use with the advantage that they retain access for cleansing and monitoring of performance. The access cover provides evidence of the location of the soakaway, a fact not always known with rubble-filled pits.

(c) Porous Pavement

A porous pavement is an 'engineered' construction allowing stormwater to infiltrate into the pavement generally across the whole surface. This type of pavement allows the water to percolate to the subgrade for recharge of groundwater. There is a range of alternative methods for disposing of the water entering the pavement (incorporating either on or off-site disposal or re-use of the water, with or without further treatment before disposal (see Figure 18.7).

(d) Infiltration Basin

An infiltration basin is an area of land surrounded by a bank or berm, which retains the stormwater until it has infiltrated through the base of the basin. The basin is frequently excavated in the ground surface, but occasions do occur where berms are used to enclose an area of land on the ground surface, or on one side where the basin is constructed on sloping ground. There are examples where combined attenuation/infiltration processes can be established: the principal mode of operation is stormwater detention to attenuate the discharge hydrograph, but ground conditions are such that some measure of infiltration occurs during storage (see Figure 18.8).

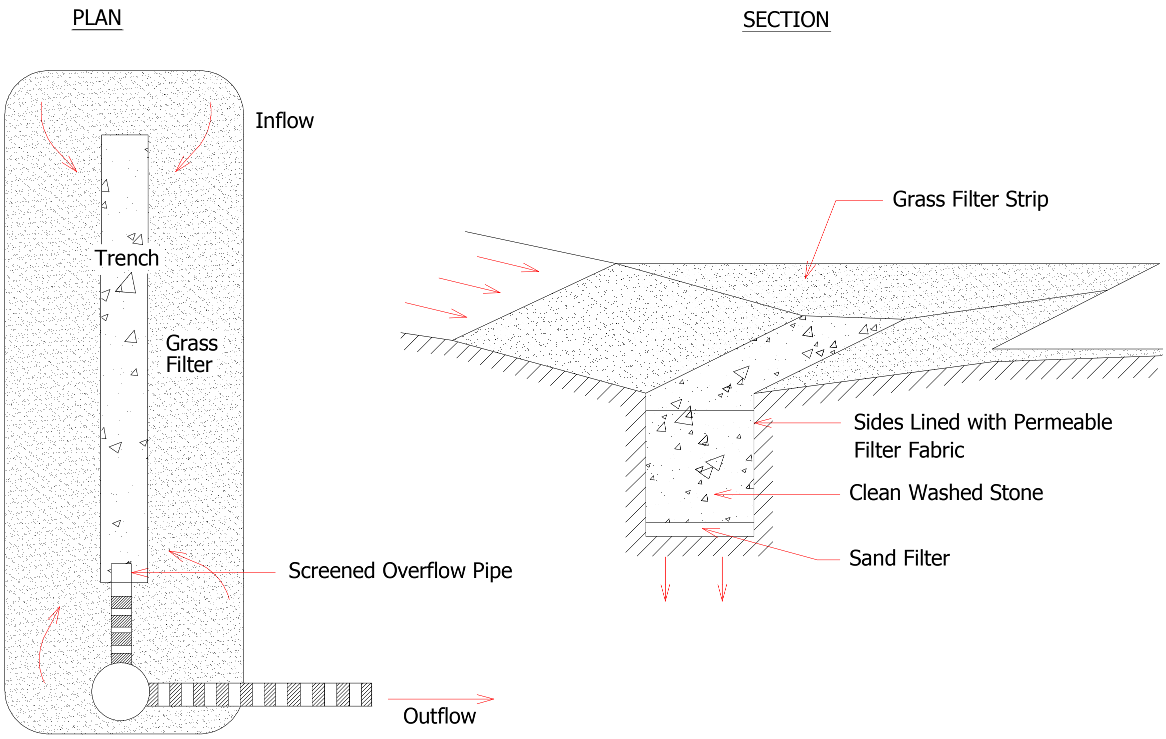

Figure 18.5 Infiltration Trench (Schueler, 1987)

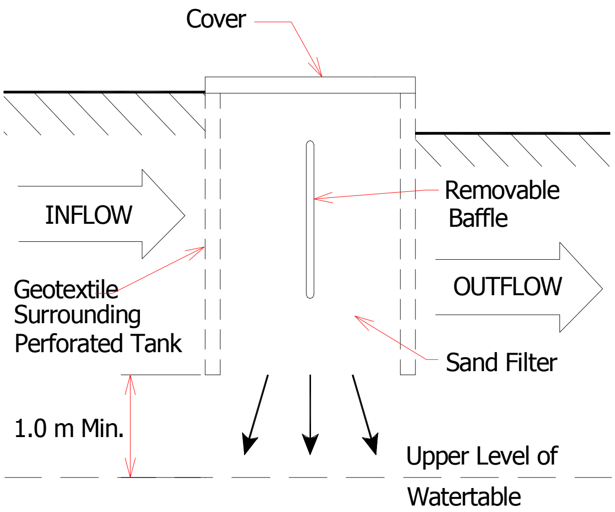

Figure 18.6 Soakaway Pit

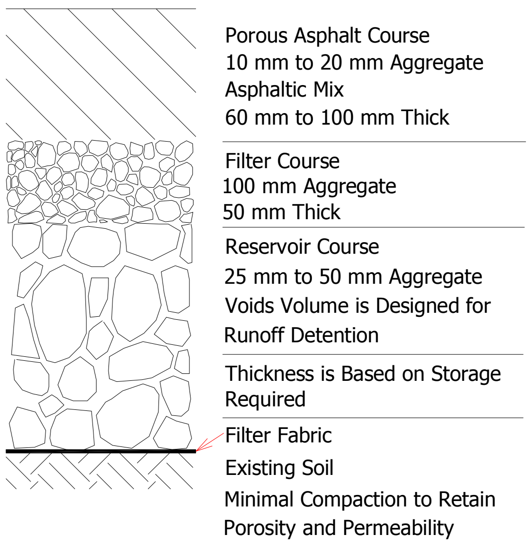

Figure 18.7 Porous Pavement

18.3.2 Regional Retention

It is more cost-efficient to implement large scale stormwater retention facilities in conjunction with artificial groundwater recharge/groundwater management programme of a community. The main methods recommended are by spreading, recharging and integrated pumping schemes (including recharge). Spreading methods may be classified as basin, stream/channel, ditch and furrow flooding and irrigation while recharge methods include pits and injection wells. Pumping sometimes is required to lower water table/piezometric level for inducing infiltration from spreading or well techniques. The scheme is gaining more popularity in managing/conserving municipal water resources as practised in developed countries such as the Netherlands. Multiple basins provide for continuity of operation when certain basins are removd from services for drying and maintenance. Furthermore, where streamflow from storm runoff is being spread, a series of basins have the advantage that upper basins can be reserved for settling silt.

(a) Basin Method

Water may be recharged by releasing it into basins formed by construction of dykes or levees or by excavation. Generally, basin sizes and shapes are adapted to land surface slope. Silt-free water aids in preventing sealing of basins during submergence. Most basins require periodic maintenance to improve infiltration rates by scarifying, disking, or scraping the bottom surface when dry. Where local storm runoff is being recharged, a single basin will normally suffice, but when larger drain flow or streamflow is being diverted for recharge, a series of basins, become advantageous. Figure 18.9 illustrates a typical plan of a multiple basin recharge project.

(b) Ditch-and-Furrow Method.

In this method water is distributed to a series of ditches, or furrows, that are shallow flat-bottomed, and closely spaced to obtain maximum water-contact area. One of three basic layout is generally employed : (1) contour, where the ditch follows the ground contour and by means of sharp switchbacks meanders back and forth across the land; (2) tree-shaped, where the main canal successively branches into smaller canals and ditches; and (3) lateral, where a series of small ditches extend laterally from the main canal. On very steep slopes checks are sometimes placed in ditches to minimise erosion and to increase the wetted area.

Gradients of major feeder ditches should be sufficient to carry suspended material through the system. Deposition of fine-grained material clogs soil surface openings. Although a variety of ditch plans have been devised, a particular plan should be tailored to the configuration of the local area. A collecting ditch is needed at the lower end of the site to convey excess water back into the main stream channel. Figure 18.10 shows typical spreading ditches on an alluvial plain.

(c) Flooding Method

In relatively flat topography, water may be diverted to spread evenly over a large area. In practice, canals and earth distributing gullies are usually needed to release the water at intervals over the upper end of the flooding area. It is desirable to form a thin sheet of water over the land, which moves at a minimum velocity to avoid disturbing the soil. Tests indicate that highest infiltration rates occur on areas with undisturbed vegetation and soil covering. Compared with other spreading methods, flood spreading costs least for land preparation. In order to control the water at all times, embankments or ditches should surround the entire flooding area.

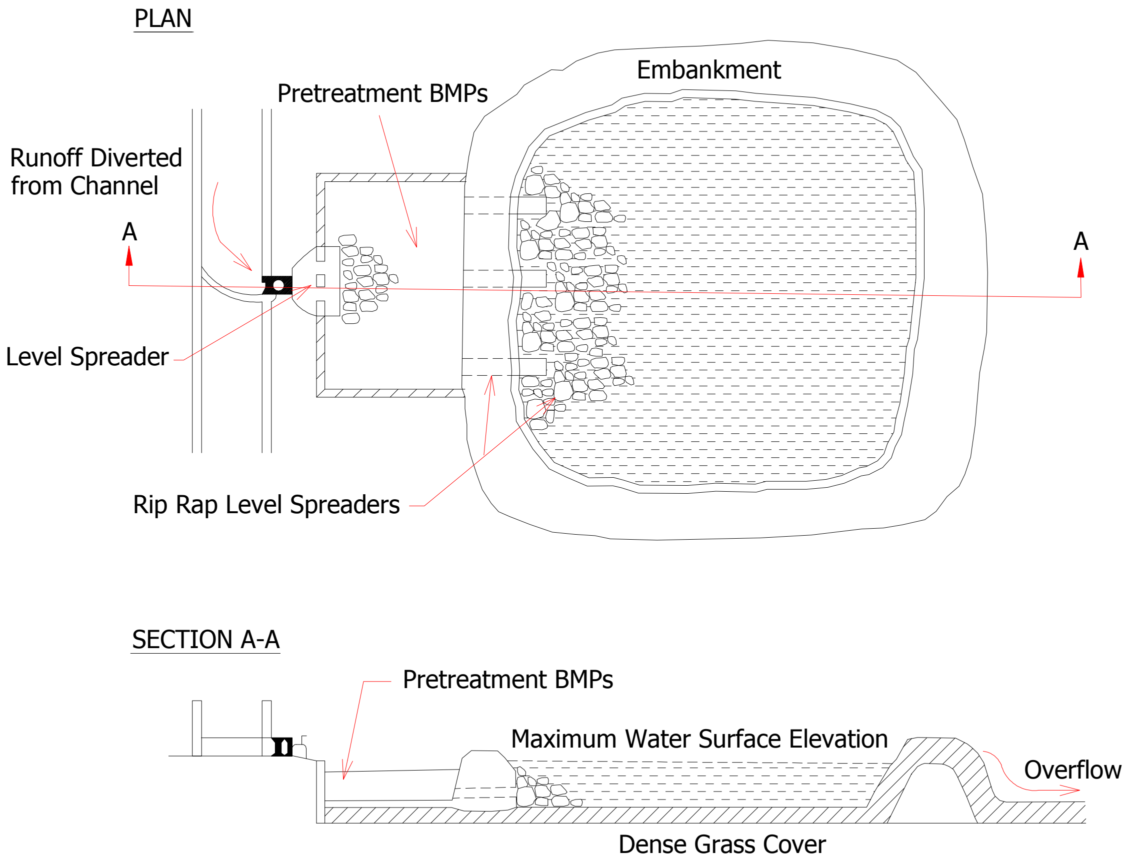

Figure 18.8 Infiltration Basin (Schueler, 1987)

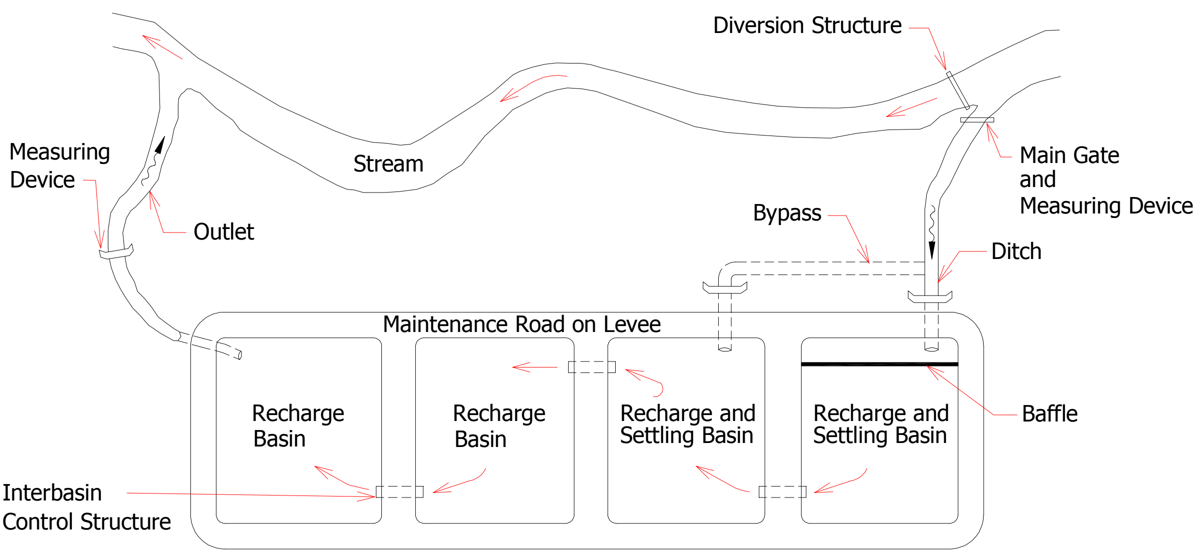

Figure 18.9 Multiple Basin Recharge Facilities (ASCE, 1996)

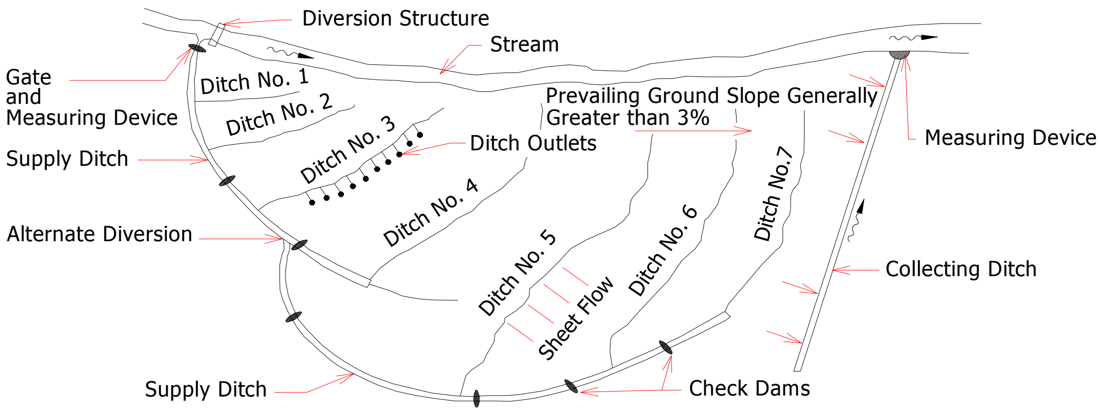

Figure 18.10 Typical Spreading Ditches

(d) Irrigation Method

In irrigated areas water is sometimes deliberately spread by irrigating cropland with excess water during dormant or non-irrigating seasons. The method requires no additional cost for land preparation because the distribution system is already installed. Even keeping irrigation canals full will contribute to recharge by seepage from the canals. Where a large portion of the water supply is pumped, the method has the advantage of raising the water table and consequently reducing power costs.

(e) Recharge Well Method

A recharge well may be defined as a well that admits water from the surface to freshwater aquifers. Its flow is the reverse of a pumping well, but its construction may or may not be same. Well recharging of stormwater is practical where deep, confined aquifers to be recharged exist, or where economy of space, such as in urban areas, is an important consideration.

If stormwater is admitted into a well, a cone of recharge will be formed that is similar in shape but is the reverse of a cone of depression surrounding a pumping well.

By comparing the discharge equations for pumping and recharge wells, it might be anticipated that the recharge capacity would equal the pumping capacity of a well if the recharge cone has dimensions equivalent to the cone of depression. Field measurements, however, rarely support this reasoning; recharge rates seldom equal pumping rates. The difficulty lies in the fact that pumping and recharging differ by more than a simple change of flow direction.

Recharge wells (Figure 18.11), like spreading areas, may show initial large intake rates followed by nearly constant or slowly decreasing values. Greatest intake rates are found in extremely porous formation such as lime-stones and lavas. It should be noted that supply wells can alternate as recharge wells. Although considerable rubbish is carried into the wells, the cavernous limestone formation seldom become clogged. Because these formations do not provide adequate filtration, a danger of pollution exists to any nearby water supply wells.

Studies have also demonstrated the feasibility of temporary storage of fresh water in saline water aquifers through wells first recharged and later pumped. The efficiencies of the procedures increase with each recharge-storage-withdrawal cycle. The technique thus has application in flat coastal areas underlain by saline water aquifers where no surface reservoir sites are available to provide freshwater supplies on a year-round basis.

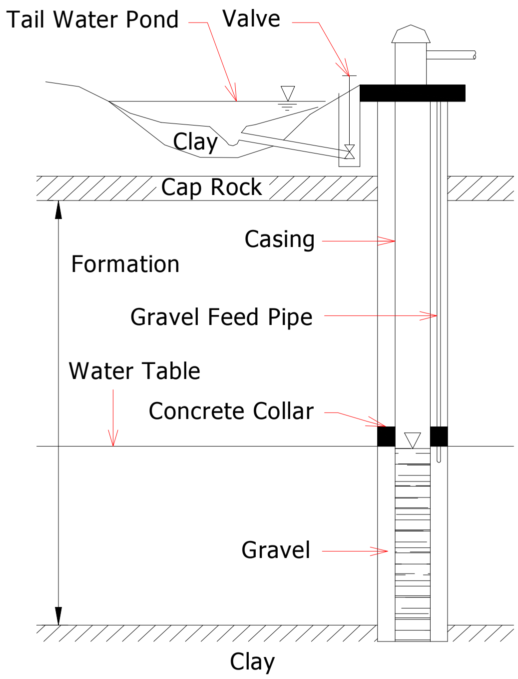

Figure 18.11 Typical Recharge Well

18.4 GENERAL DESIGN CONSIDERATIONS

18.4.1 On-site Detention

Detention on development sites has been seen as the solution to problems of established areas where additional development or redevelopment is occurring. Generally, it is not possible, either physically or financially, to progressively enlarge drainage systems as redevelopments that increase impervious areas and runoff rates and volumes occur.

Regulations, which put the onus on developers to restrict flows, are therefore attractive to drainage authorities. Flows can be limited by the use of various OSD facilities. These methods were popularised by Poertner (1974) and the American Public Works Association (1981) in the US, and by Phillips (1985,1986,1993) in Australia. They employ design procedures based on the Rational Method.

Simplified hydrographs are combined with an assumed outlet relationship to determine a critical volume of water to be stored. Often several cases are considered, to allow for different storm durations. A storage is then to be provided for this critical volume.

Permissible site discharge (PSD) and site storage requirement (SSR) are used for a OSD development. There are two basic approaches that may be used for determining the required PSD and SSR.

(a) Site-based Methods

With these methods, the PSD and SSR values to be applied to a particular development site are determined by hydrologic analysis of the development site only, without any consideration of the effect of site discharges on the downstream catchment. The PSD is the estimated peak flow for the site prior to development for a selected design storm. The only concern is that post-development site discharges are reduced to pre-development levels. PSD values may be determined using either the Rational Method or a hydrograph estimation method (refer Chapter 14).

Site-based methods do not consider the effects of post-development discharges on the downstream catchment since it is assumed that reducing discharges to pre-development levels is sufficient to prevent increases in downstream flooding.

(b) Catchment-based Methods

With these methods, the PSD and SSR values are determined from an analysis of a total catchment instead of a single site. Catchment modelling is undertaken to determine the maximum values of PSD and SSR for a selected design storm that will not cause flooding at any location within the catchment. These are general values that may be applied to any site within the catchment.

OSD storages may be analysed using any hydrograph estimation technique, but the Rational Method is the most popular. Rational Method hydrograph techniques are acceptable for OSD as development sites are relatively small and any errors introduced will most likely be minor. The effort involved with more sophisticated computer modelling techniques is not normally warranted. The Swinburne Method recommended in Chapter 19 is based on the Rational Method.

18.4.2 Community and Regional Detention

(a) Design and Analysis

In designing a community or regional detention facility to meet the flow control objectives stated in Section 4.5, it is necessary to consider the behaviour of the storage by examining:

- the degree of reduction of flows from the catchment
- the depths of ponding in the basin (for safety reasons)
- the frequency at which the overflow spillway comes into operation, or the embankment overtops
- the duration of ponding (for interruption to other uses and possible harmful effects on grass and other vegetation in the basin)

As well as considering the required levels of operation, it may be necessary to examine low ARIs (e.g. 6 months) to assess maintenance requirements and to design underdrains beneath storages. At the other end of the scale, it may be necessary to consider extreme events such as probable maximum precipitation (PMP) storms or to examine the effects of a dambreak failure.

Design processes can be different from analysis when simple procedures are used, which fall short of a reasonable simulation of a system. The Rational Method is such a simplified procedure. However, for complex systems such as larger detention basins, design should be carried out as a series of analyses. A trial basin arrangement is set and then modified as different storms are modelled.

Design and analysis involve the following processes:

- hydrological calculations to determine the flowrate to be handled by the storage
- hydraulic calculations to route the flows through the storage, determining the reduction in flowrates
- geotechnical, structural, and other design processes
- preparation of drawings, specifications, and contract documents before moving on to construction

Hydraulic routing calculations are considered in detail in Chapter 14 and computer models are presented in Chapter 17.

(b) Release Timing

The timing of releases from detention facilities can be critical to the proper functioning of overall stormwater systems. As illustrated in Figure 18.1, a stormwater detention structure can provide a significant reduction in peak flow, but in so doing, the time to peak and total flow duration are both increased. Whilst a reduction in the peak flow is the desired result for the flow tributary immediately downstream of an individual detention facility, this shifting of the flow peak time and total flow duration in some instances can cause adverse flow effects further downstream.

For example, where the drainage area being controlled is in a downstream portion of a larger catchment, delaying the peak and extending the recession limb of the hydrograph may result in a higher peak on the main channel. As illustrated in Figure 18.12, this can occur if the reduced peak from the controlled catchment is delayed in such a way that it reaches the main stream at or near the time of the main stream peak. On occasions, it has also been observed that in locations where multiple detention facilities have been installed within developing cathments, downstream stormwater flooding problems continue to be noticed. In both of these cases, the natural timing characteristics of the catchment are not being considered, and certainly are not being duplicated by the uncoordinated use of randomly located detention facilities. It is critical that release timing be considered in the analysis of stormwater control facilities to ensure that the desired result is obtained.

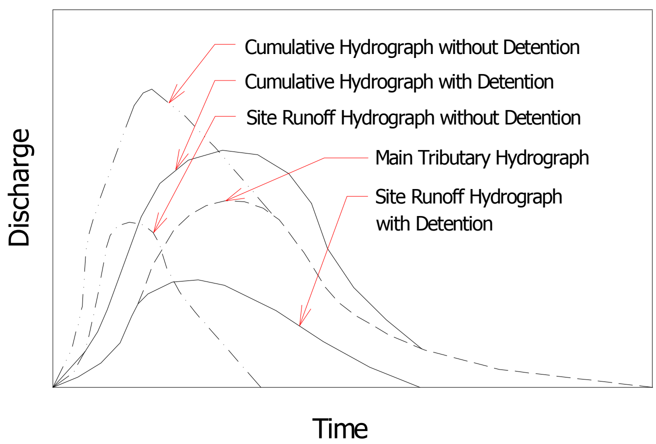

Figure 18.12 Example of Cumulative Hydrograph with and without Detention

(c) Calculations with Multiple Design Storms

Unfortunately, it is not possible to determine easily the particular rainfall pattern which provides the worst case for a detention basin. This has to be found by trial and error, by modelling basin behaviour during storms of different durations and magnitudes.

A computer model is necessary to handle this large workload. Even with this, the task is a large one. Some models allow rainfall patterns to be "stacked", so that several storms can be handled in a single run, and critical flows are automatically identified.

Rather than calculating results for all storms, it is probably more efficient to concentrate on key storms which are most likely to be the worst cases. The range of storms, which might be considered, are shown in Figure 18.13.

Assuming that the main design is to be for the major drainage system ARI, a preliminary analysis can be performed to determine the likely critical duration. Very roughly, the duration which is most likely to produce the highest flowrates from a rainfall-runoff model is about 1.5 times the catchment time of concentration. However, with a basin system, the interest is in the outflows from a basin, not the inflows. Since the particular outlet arrangement will affect the outflows, it is not possible to decide on the critical duration in advance. This has to be done by trial and error.

Suppose that the critical duration is really 1 hour. Detailed analysis of outlet arrangement is then carried out using the 1 hour, 50 year ARI event. Once these have been define other 50 year storms (corresponding to the horizontal band in Figure 18.13 can be tested.

At this stage, a storm of another duration may prove to be critical. In fact, in a multiple-basin system, different duration storms may be critical for different parts of the network. Further adjustments to the outlet works may be required. Lastly, the refined system can be tested for different ARIs (corresponding to the vertical band in Figure 18.13). With this approach, it is probably not necessary to consider all combinations of durations and ARIs.

ARI	Duration (hr)							
(year)	0.5	1	1.5	2	3	6	12	24
1	X	**X**	X	X	X	X	X	X
2	X	**X**	X	X	X	X	X	X
5	X	**X**	X	X	X	X	X	X
10	X	**X**	X	X	X	X	X	X
20	X	**X**	X	X	X	X	X	X
50	**X**	**X**	**X**	**X**	**X**	**X**	**X**	**X**
100	X	**X**	X	X	X	X	X	X
PMP	X	**X**	X	X	X	X	X	X

Figure 18.13 Range of Storm Durations and ARIs

Strategies like the above can be adopted to meet various requirements for basins, without having to explore every possibility.

(d) Spillway Operation

It has often been considered that the high-level spillways of detention basins should only come into action during floods greater than the major system ARI. This makes the analysis easier, as it is only necessary to set a very high spillway level and to route major system ARI design floods of various durations through the storage. The spillway crest can then be set at the highest storage level reached.

On reflection, there is no reason why spillways should not be designed to operate more frequently than 100 years ARI. If properly built, they can withstand frequent operation. Such a standard might be appropriate for the entire embankment, if there is confidence that the downstream face could withstand overflows running over it without serious erosion.

The appropriate frequency depends on what is happening downstream and its determination requires a study, which goes beyond the detention storage to consider the total system. This is obviously very difficult, so for convenience, designers have attempted to define more concrete aims or rules which can be used in the majority of cases Chapter 20 provides guidelines on design capacity of spillways.

(e) Extreme Floods

There is a riskof failure of the embankment or wall of a detention storage during a very large flood. If this is rapid, the resulting wave of water, mud and debris may exceed the peak flow, which would occur without the basin in place. If there was loss of life or severe property damage, the operators of the basin would probably be held liable.

On earth embankments used in most basins, the most common failure mechanism is erosion of the downstream face as overflows run over it. If a rill is formed, it may quickly grow, and if enough material is eroded away on the downstream side, water pressure may cause stored water to burst through the weakened embankment.

If there is likely to be danger or high damage in the event of a basin failure, designers should perform an analysis using PMP data. This data can be inputted to a rainfall-runoff model with conservative (i.e. low) loss assumptions, and a probable maximum flood (PMF) calculated at the basin site.

This will surely overtop the basin embankment. Calculations can then be performed to find the depth of overtopping and likely flow velocities. These can be checked to see that they are below safe limits for onset of erosion. If the spillway or basin crest is lengthened, say by curving it, both depth and velocity will be reduced. Another factor, which can be changed, is the slope of the downstream face of the embankment. The flatter this is, the lower the erosion potential. It is therefore usually necessary to locate spillways to the side of a basin embankment, where the wall is low, and the spillway crest can be situated on the natural surface or in cut. Where a spillway is to be located on earth fill, it should be reinforced with rock or concrete. Good grass cover or netting should be provided on the downstream face.

(f) Public Safety

The risk or persons drowning in detention basins has usually been the most important safety consideration. Some drainage authorities have limited depths to 1.2 m or 1.5 m at some average recurrence interval; others have fenced basins to exclude the public; while some have placed no restrictions on ponding depths.

Australian Rainfall and Runoff (AR&R, 1987) sets out some requirements as follows:

- Detention basins are less hazardous than channels, due to slower movement of water, but the public may be caught unawares by the filling of basins normally used for recreation. Preferably the side slopes of basins should not be steeper than 1(V):6(H). Areas with slopes steeper than 1(V):4(H) may require a fence or rail. Special attention should be paid to basin outlets, to ensure that persons are not drawn into these. Rails, fences, crib-walls, anti-vortex devices, and grates should be provided where necessary. Many safe designs are available. Trees and mounds within basins are desirable as refuges.
- These requirements are more important for deeper basins than for ones where the depth during major storms will not exceed 1.2 m. The permissible depth of ponding in basins accessible to the public has been a controversial issue in New South Wales, and the Sydney Division of the Institution of Engineers, Australia (1985) has produced the following settlement:

  "Where suitable land is available, designers should aim to restrict basin depths to 1.2 m at the 20 years ARI level and if possible to a longer recurrence interval. In cases where this is neither practical nor economical and the provision of a detention basin is considered to be better on safety and other grounds than other alternatives, greater depths are acceptable. Suitable safety provisions (such as raised refuge mounds within large basins, fences of various kinds, and warning signs) might be provided in deeper basins."
- Special attention must be given to basin overflow spillways in design and operation to avoid catastrophic failures, which may cause loss of life downstream. PMF studies should be conducted for basins with vulnerable downstream areas and large, multiple-basin

systems to ensure gradual and safe modes of failure in extreme events.

- Where a basin is considered unsafe for public use, it should be fenced, though this will inhibit dual use of the land. Even in fenced basins, authorities have a duty of care to persons, particularly children, who may enter.

(g) Environmental Considerations

Where water quality and water conservation objectives are to be met, additional studies are required through inputs from biologists, planners, and landscape architects.

18.4.3 Retention

The storage volume of a retention facility is over the infiltrating surface or pore volume of the stone filling. These facilities are emptied either to underlying soils or through specially designed underdrains. They can also be equipped with an overflow outlet to drain off the excess water.

For proper design generally the surface soils and geohydrologic conditions at the site have to be known and understood. This include data on soil permeability and porosity, groundwater level and its fluctuation with seasons, soil profiles etc.

Except for the most permeable soils, the inflow rate to the infiltration system will exceed the outflow rate. It is therefore necessary to store the water on-site and to allow time for it to soak away slowly through infiltration (Figure 18.14). Provision of sufficient storage capacity is essential for an infiltration system to perform properly. If the infiltration system is incorrectly designed, the outflow rate may not be enough to allow the system to empty sufficiently before the next rainfall event. The infiltration system will then overflow and the scheme will be deemed a failure. The design guidelines contained in this manual are intended to enable infiltration systems to be designed to acceptable factors of safety.

For a soil to be suitable for accepting enhanced infiltration it must, in particular, be (a) permeable and (b) unsaturated. In addition, it must be of sufficient thickness and extent to disperse the water effectively.

The capacity of a soil to infiltrate water can be described by using an infiltration coefficient. This is the discharge infiltrating into the soil divided by the area of infiltration. The infiltration coefficient of the soil is related to its permeability. This will be high for coarse grained soils such as sands and gravels and low for fine soils such as silts and clays.

These figures may provide a useful first indicator of the magnitude of the infiltration capacity but the high ranges reported illustrate the importance of factors such as soil packing, soil structure, swelling clay content and the presence of fissures in rock which will significantly affect the infiltration capacity. It is necessary, therefore, to conduct a percolation test on-site to demonstrate the ability of the ground to accept infiltration in situ (Appendix of Chapter 21). It is possible to dispose of water at a point beneath the water table but this practice, usually known as artificial recharge (Figure 18.14), requires special considerations (Chapter 22). Infiltration systems require an unsaturated soil to disperse the stormwater effectively and remove contaminants from it.

If the infiltration system is founded too close to the water table, a rise in water table during wet conditions could cause groundwater to enter the infiltration system, reducing the available storage.

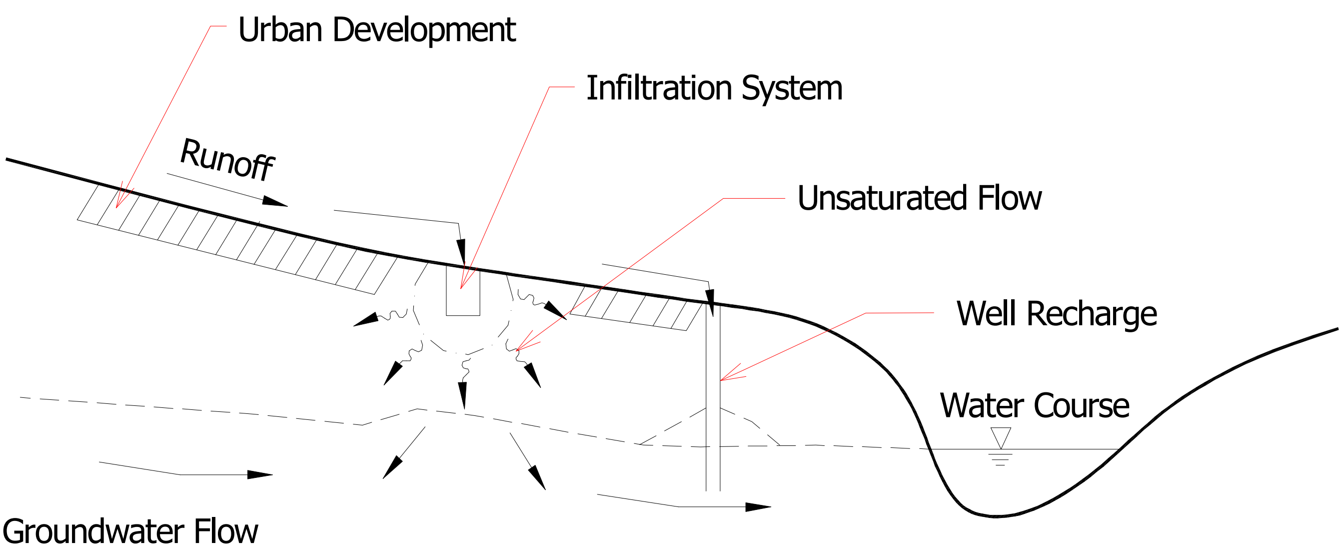

Figure 18.14 Infiltration and Artificial Recharge System used for Stormwater Disposal (CIRIA, 1996)

Table 18.1 Approximate Values of Soil Porosity (CIRIA, 1996)

Type of Soil	Percent Effective Porosity
Crushed rock	30
Gravel and macadam	40
Gravel (2 to 20 mm)	30
Sand	25
Pit run natural gravel	15-25
Till (boulder clay)	5-10
Dry crust clay	2-5
Clay and silt (below surface)	0

Table 18.2 Permeability of Various Soils (CIRIA, 1996)

Type of Soil	Range in Permeability (m/year)
Gravel	30,00 to 3,000,000
Sand	30 to 300,000
Silt	0.03 to 300
Boulder clay	0.003 to 30
Clay	Less than 0.03

With the issues previously considered in mind, the next step is to estimate the size of system required (Chapter 21).