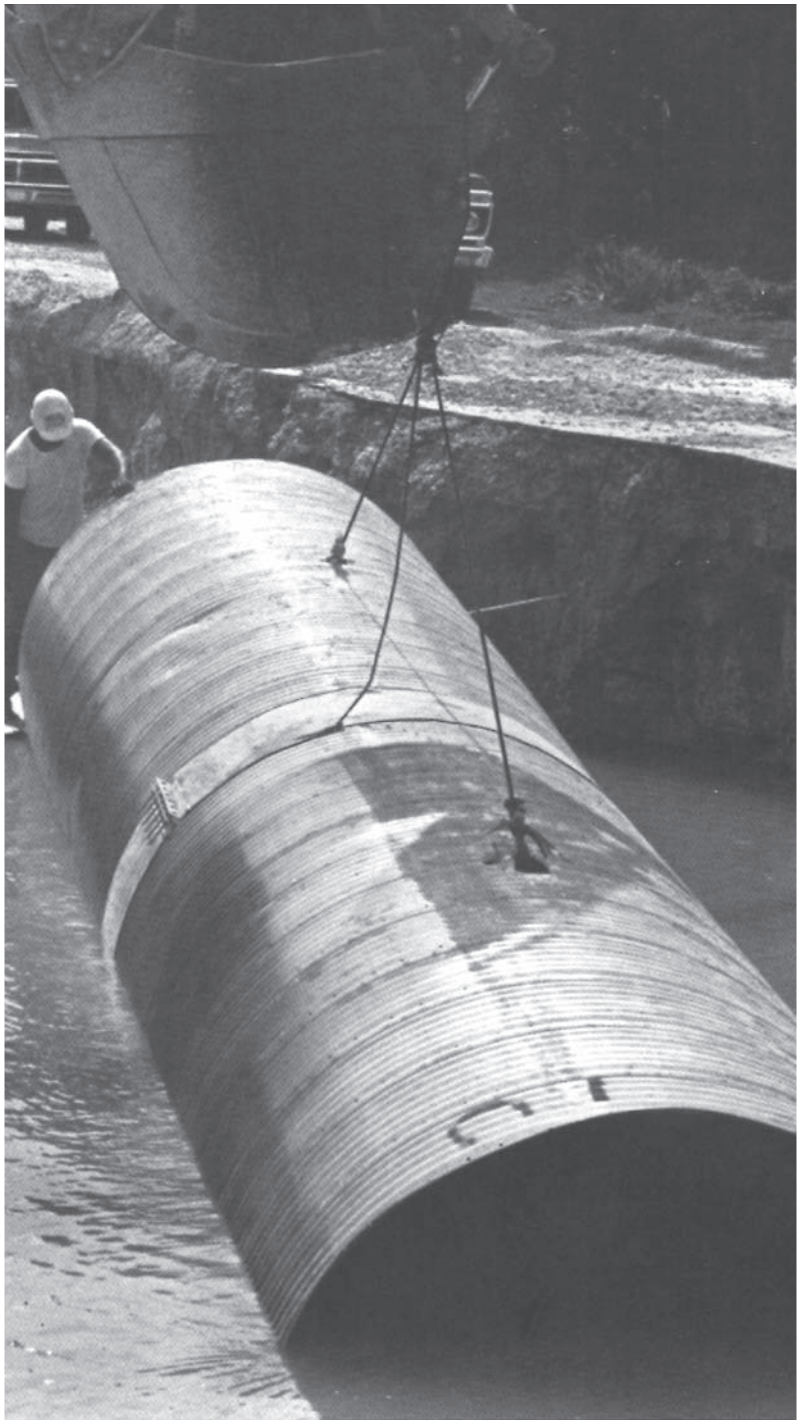


Two 6m joints of fully perforated pipe banded together for ease of installation.

CHAPTER 6 Stormwater Detention & Subsurface Disposal

STORMWATER DETENTION FACILITIES

Detention facilities in new storm drainage systems are increasing in popularity as a means of achieving the urban drainage objectives. Detention facilities may also be incorporated into existing developments where flooding problems due to sewer surcharging are occurring. Each proposed development should be carefully examined in order to determine which method of storm water detention or combination of methods could be best applied. The methods of detention available may be categorized under three classifications: 1) underground, 2) surface, 3) roof top.

Underground Detention

In areas where surface ponds are either not permitted or not feasible, underground detention may be used. Excess storm water will be accommodated in some form of storage tank, either in line or off line, which will discharge at a pre-determined control rate back into either the sewer system or open watercourse. In-line storage incorporates the storage facility directly into the sewer system. Should the capacity of the storage facility be exceeded, it will result in sewer surcharging.

Off-line detention collects storm water runoff before it enters the minor system and then discharges it into either a sewer or open water course at a controlled rate. By making use of the major system and connecting all tributary catch basins to a detention tank, approximately 80% of storm runoff may be prevented from directly entering conventional sewer systems. In areas where roof drains are discharged to the surface, close to 100% of the storm runoff may be controlled. Such facilities are very applicable in areas with a combined sewer system. In such cases catch basins may be sealed where positive overland drainage is assured. Storm water is then collected in underground storage tanks and discharged back to the combined sewer at a controlled rate (See Figure 6.1).

Surface Detention

Surface detention is feasible in developments where open spaces exist. Parking lots provide a very economical method of detaining peak runoff when the rate of runoff reaches a predetermined level. The areas to be ponded should be placed so pedestrians can reach their destinations without walking through the ponded water. Areas used for overflow parking or employee parking are best suited. The maximum depth of ponding would vary with local conditions, but should not be more than 200mm to prevent damage to vehicles. Overflow arrangements must be made to prevent the water depth exceeding the predetermined maximum. Ponds either wet or dry may be located on open spaces or parklands to control runoff. Wet ponds hold water during dry periods, thus they may serve other purposes such as recreational and aesthetic. Trapped storm water might also be reused for lawn watering and irrigation. A detention basin will act as a “cushion” which will have the effect of decreasing the peak runoff, removing sediments and reducing pollutants before discharge to streams and lakes.

Dry ponds are operable during and a short time after a storm event. Since these facilities are designed to drain completely they may serve other functions such as golf courses, parks, playing fields, etc.[1]

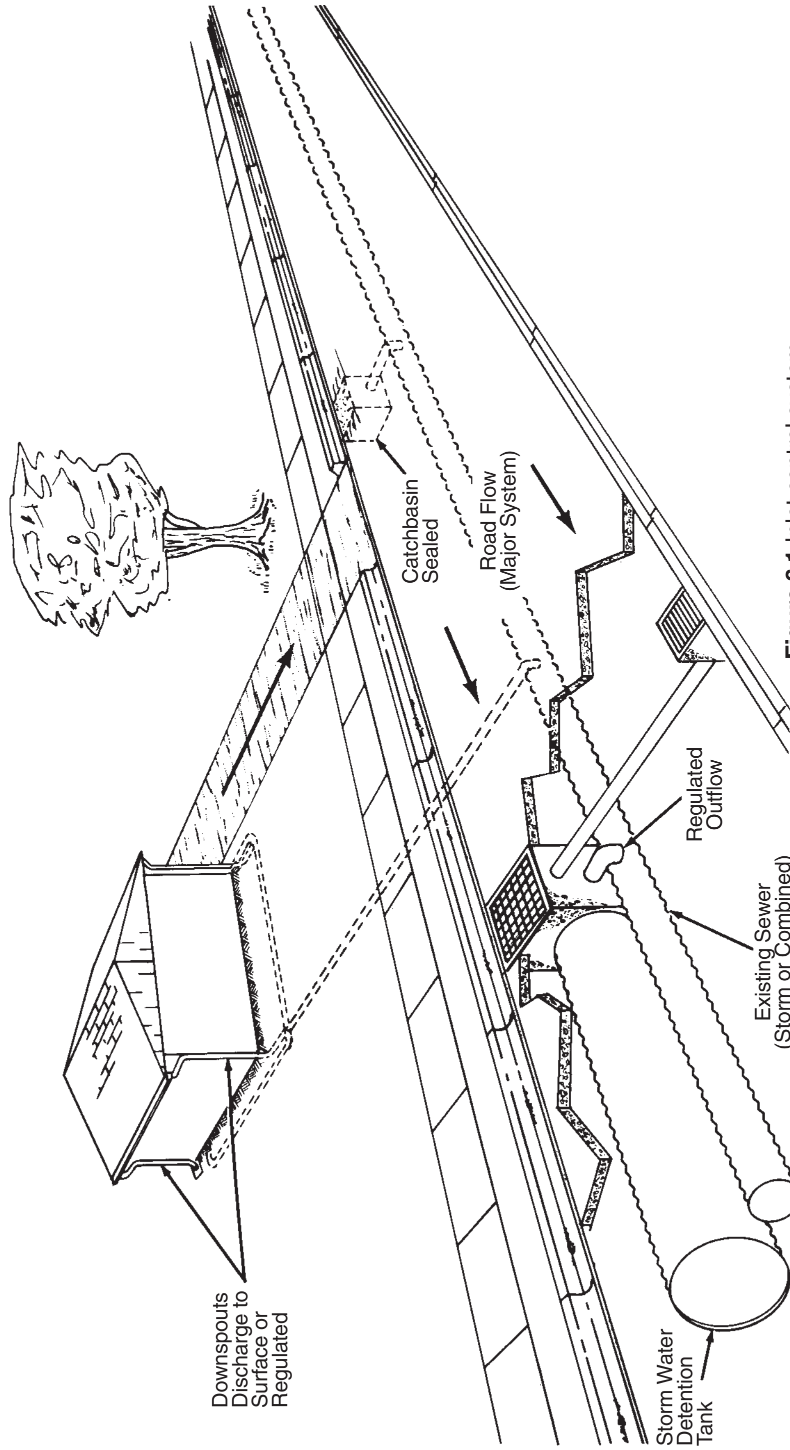


Figure 6.1 Inlet control system

Roof Top Detention

Flat roofs are very common for industrial, commercial and apartment buildings. Since they are often designed for snow load, they will also accommodate an equivalent load of water without any structural changes. A 150mm water depth is equivalent to 150 kilograms per metre squared, less than most snow load requirements in northern United States and Canada.

Special roof drains with controlled outlet capacity have been used for many years in order to reduce the size of drainage pipe within an individual building or site. Seldom was this reduction in peak flow recognized in the sizing of the municipal storm sewers, and the total benefit was therefore not achieved. Many flat roofs now also pond significant amounts of storm water; this should also be considered when estimating peak flows. By installing roof drains with controlled outlet capacity, the resultant peak runoff from a roof can be reduced by up to 90 percent, a very significant reduction indeed. In addition to this important advantage, it is obvious that there would be substantial cost savings. For a typical roof drain with controlled outflow, see Figure 6.2.

Overflow mechanisms should be provided so that the structural capacity of the roof is not exceeded. Also special consideration should be given to water tightness when roof top ponding is to be incorporated.

2700mm, diameter, 2mm, 76 x 25 CSP used as an underground detention chamber. The outlet control structure is located at the opposite end and to the right of those pipe.

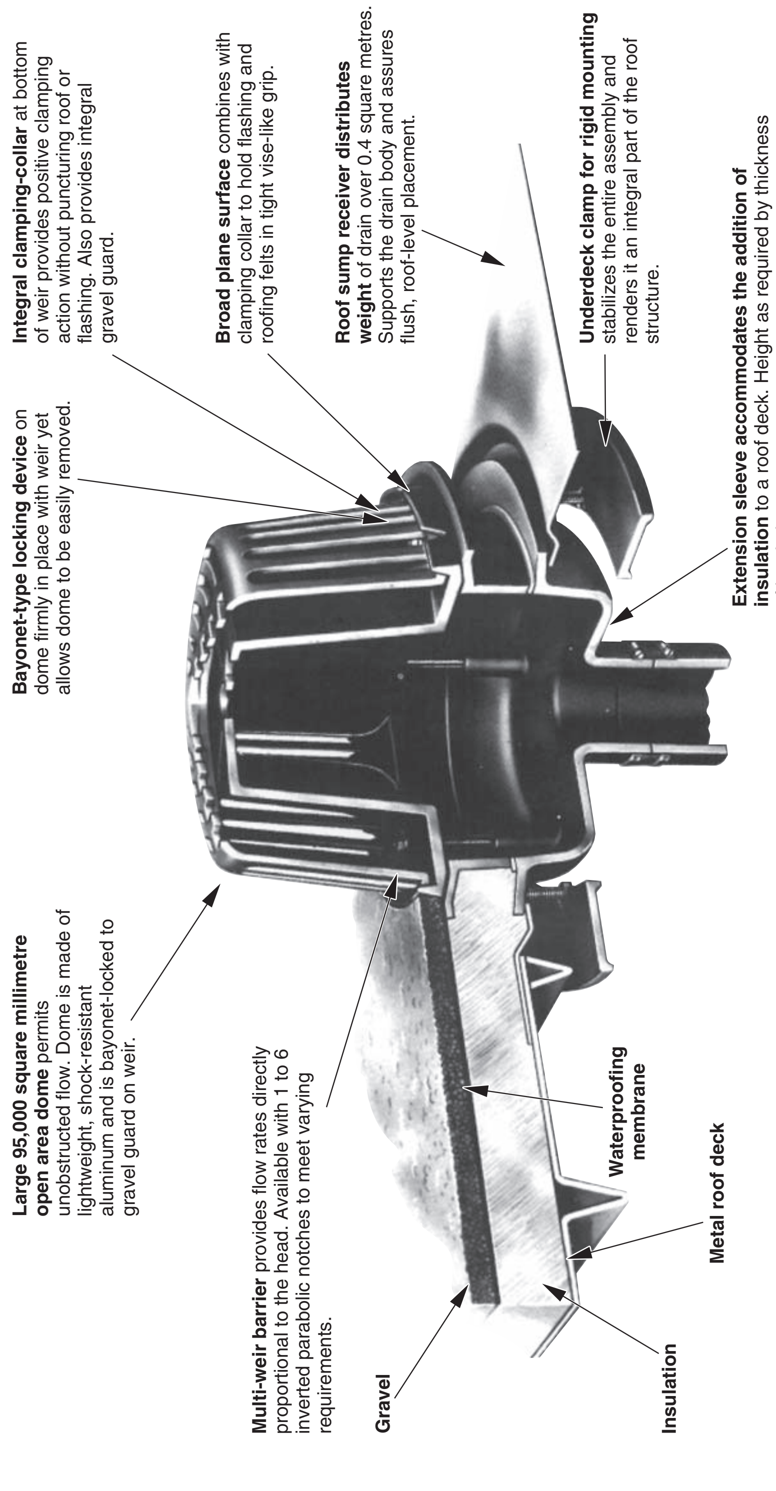


Figure 6.2 Special roof drains with control outlet capacity[2]

DESIGN OF STORM WATER DETENTION FACILITIES

Commonly, in new developments, detention or retention facilities are necessary in order that the storm water management requirements can be met. The requirements for these facilities may be relatively straightforward; for example the objective may be to control the 10-year post-development flow to pre-development rates. Conversely, the requirements may be more complex. The facility may be required to control post-development flows to pre-development levels for a range of storms, or to control the flow rate to a predetermined level for all storm events. Detention facilities may also be used for improving water quality.

The design of the facility generally requires that the following two relationships be established:

a) depth-versus-storage (Figure 6.4)
b) depth-versus-discharge (Figure 6.5)

The depth versus storage relationship may be determined from the proposed grading plan of the facility and the existing topography. The depth-versus-discharge curve is dependent upon the outlet structure.

Many methods may be used for design of the proposed facility. These include both manual and computer aided methods. For the most part the methods used assume that the facility acts as a reservoir.

The storage indication method is widely used for routing flows through reservoirs. The following equation describes the routing process:

$$\bar{I} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2}$$

Where $\bar{I} = (I_1 + I_2)/2$

I_1, I_2 = inflow at beginning and end of time step
O_1, O_2 = outflow at beginning and end of time step
S_1, S_2 = storage at beginning and end of time step
Δt = time step

A working curve of O_2 plotted against $(S_2/\Delta t) + (O_2/2)$ is necessary for solving the equation. An example using the storage indication method is given in "SCS National Engineering Handbook, Section 4, Hydrology."[3]

Hydrograph Method

The design of detention facilities may be determined by knowing the inflow hydrograph and the desired release rate.
Example of Detention Pond Design:
Source: Adapted from Drainage Standards from Fairfax County-Virginia.

Given: 4 ha site to be developed into a commercial shopping center. Design a detention pond in an existing natural drainage course. The natural topography limits the maximum height of the pond to 2.0 m. Allowing 300mm freeboard, the maximum height of water will be 1.7 m.

Given: Pre-development Run-off = 0.43 m^3/s (10-year)
Therefore, the maximum allowable run-off from the detention pond shall be limited to this value.

The post-development 10-year runoff hydrograph from the watershed is given in Figure 6.3.

1. On the hydrograph, plot a straight line from the zero intercept to a point on the hydrograph at the 0.43 m^3/s point. The area between these two curves is the approximate volume of storage required.

   The planimetered area = 9832 mm^2

   Approximate volume = 9832 (0.13) = 1278 m^3

2. Limiting depth of storage is 1.7m, therefore, the required area of the pond is $1278m^3 / 1.7m = 752m^2$. Design the detention pond to be 30m long and 30m wide or 900 m^2.

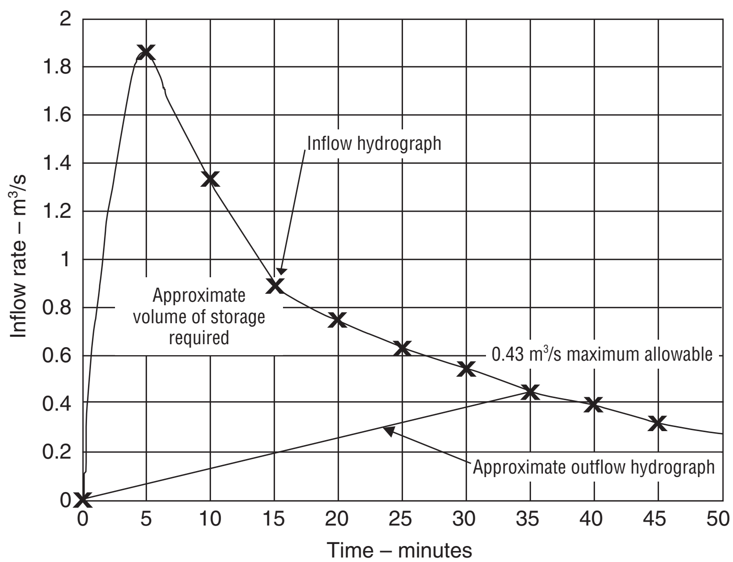


Figure 6.3 Inflow hydrograph

3. Outflow pipe design—use Inlet Control Nomograph

   Assume HW/D = 2.0

   Assume $K_e = 0.5$

   From corrugated steel pipe with inlet control chart at 0.4 m^3/s outflow:

   Diameter of culvert is 400-500mm.

   Assume 500mm outlet pipe

4. Plot volume of storage vs depth of storage curve (Figure 6.4) and depth of storage vs discharge curve (Figure 6.5). The first curve is obtained from topography and grading data and the second curve is obtained from BPR (FHWA) culvert charts for the selected pipe size.

5. Route the 10-year inflow hydrograph through detention facility by using Figures 6.3, 6.4, 6.5. Following is a narrative of the procedure.

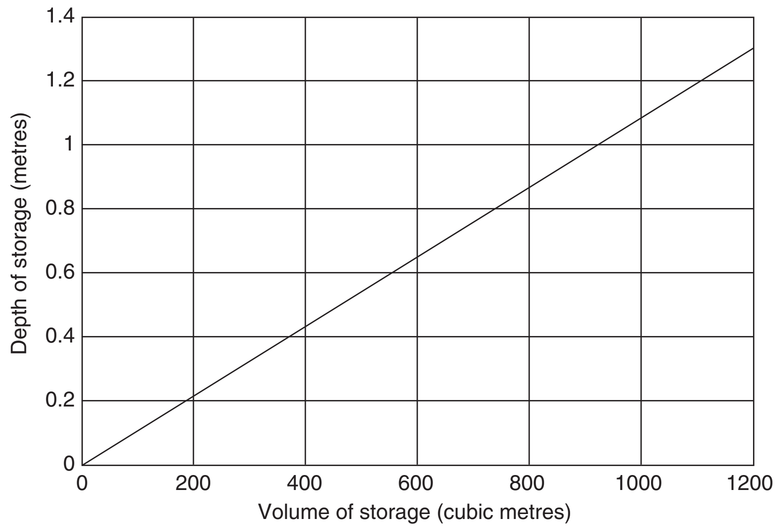


Figure 6.4 Storage depth versus storage volume curve

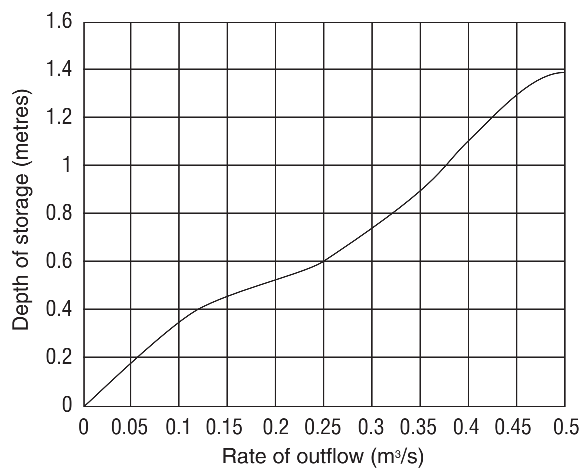


Figure 6.5 Depth of storage versus discharge curve

The basic equation for determining the volume of storage required in a detention facility is that the volume of storage equals the volume of flow into the facility minus the volume of flow released from the facility. For each five minute increment of time, the rate of flow into the facility determined from the inflow hydrograph in Figure 6.3 is averaged with the rate of flow for the previous five minute increment, and this average value (Column 2 of Table 6.1) is multiplied by the time increment of 5 minutes or 300 seconds to obtain the incremental volume in for that particular five minute period (Column 3). This incremental volume is summed with the storage carry over (Column 4) to yield the accumulated storage at the end of the particular time increment (Column 5). A trial water surface elevation is assumed and, from Figure 6.5, the rate of runoff or outflow is determined. These values are placed in Columns 6 and 7, respectively. An average outflow rate is calculated in Column 8, and multiplied by the time increment of 300 seconds to determine the volume released from the detention facility (Column 9). Volume (in) minus volume (out) equals storage (Column 10). From Figure 6.4 a corresponding depth of storage is found (Column 11), and is compared to the trial water surface elevation assumed. If the values differ by more than 30mm, assume a new trial water surface elevation and repeat the process. If the values are less than 30mm difference, the routing for that five minute increment is balanced and the procedure is repeated for the next five minute increment, and so on. The balance in storage is carried over to the next time step (in Column 4). When the balance of runoff in storage (Column 10) begins to decrease in value, the detention pond is beginning to draw down, and the maximum required volume of detention has been reached. The maximum rate of outflow, maximum required storage, and maximum height of storage can be read directly from Table 6.1 or from the curves in Figures 6.4, 6.5. This particular problem yields the following results:

Q (max) = 0.44 m^3/s
Vol. of required storage = 1173 m^3
Max height of storage = 1.3 m

These quantities compare favorably with the assumed design of the pond.

6. The emergency spillway shall be designed to pass the 100 year storm, with 300mm of freeboard maintained.

   The invert of the emergency spillway shall be set at the 10 year detention elevation of 1.3 m. Since the maximum depth of pond is 2.0m and 300mm of freeboard must be maintained, the maximum height of water over the spillway must be kept at 0.4m. The 100-year storm must be routed through the detention pond in a similar method to that used to route the 10-year storm. However, when the depth of storage exceeds 1.3m outflow will occur through the culvert and through the emergency spillway. As an example, assume the outflow culvert pipe is clogged, and the emergency spillway is a weir designed to pass the entire 100-year storm:

Q(100) = 2.5 m^3/s
Q(weir) = $5.92 \times 10^{-8} LH^{1.5}$
H = 400m
$\therefore L = 2.5 / 5.92 \times 10^{-8} (400)^{1.5} = 5{,}280$ mm

The banks of the pond, the overflow weir, and the outlet channel must be adequately protected from erosion, and the capacity of the emergency overflow channel must be sufficient to pass the 100 year storm.

Table 6.1 Example of detention pond design

	1	2	3	4	5	6	7	8	9	10	11	12
Time	Inflow rate (m^3/s)	Average inflow rate (m^3/s)	Volume (m^3)	+ Storage carry-over	Total in (m^3)	Trial W.S. elevation (m)	Outflow rate (m^3/s)	Average outflow rate (m^3/s)	Volume out (m^3)	Balance in storage (m^3)	Corresp. W.S. elevation (m)	
0	0		0				0.00					
5	1.84	0.92	276	–	276	0.27	0.08	0.04	12	264	0.29	ok
10	1.33	1.59	477	264	741	0.76	0.31	0.20	60	681	0.75	ok
15	0.89	1.11	333	681	1014	1.01	0.37	0.34	102	912	0.98	ok
20	0.74	0.82	246	912	1158	1.13	0.41	0.39	117	1041	1.12	ok
25	0.62	0.68	204	1041	1245	1.22	0.43	0.42	126	1119	1.21	ok
30	0.52	0.57	171	1119	1290	1.30	0.44	0.43	129	1161	1.23	ok
35	0.44	0.48	144	1161	1305	1.30	0.44	0.44	132	1173	1.30	ok
40	0.38	0.41	123	1173	1296	1.30	0.44	0.44	132	1164	1.30	ok
45	0.33	0.36	108	1164	1272	1.22	0.43	0.43	129	1143	1.22	ok

Therefore, the maximum storage required is 1173 cubic metres; maximum depth of storage is 1.3 metres; and the maximum rate of runoff is 0.44 m^3/s.

"Blue-Green" Storage

An economical way of detaining surface runoff is the "Blue-Green" approach, where the storage capacity within drainageways is utilized. This technique may be achieved by designing road crossings over drainageways to act as dams, allowing only the regulated outflow rate to be conveyed through the embankments. This technique can be repeated several times along the same drainageway, in effect creating a chain of temporary ponds. In this manner, the dynamic storage characteristics of the greenbelt system will retard the peak flows, yet provide for continuous flow in the drainageway. The culvert(s) through the embankment may be hydraulically designed to permit a range of regulated outflow rates for a series of storm events and their corresponding storage requirements. Should all the storage capacity in the drainageway be utilized, then the overflow may be permitted over the embankment. Overflow depths on minor local streets of 200- 300mm are usually acceptable, with lower values for roads with higher classifications. If the allowable maximum overflow depths are exceeded, then the culvert(s) through the embankment should be increased in size.

The designer must remember to design the roadway embankment as a dam, with erosion protection from the upstream point on the embankment face to below the downstream toe of the embankment.

It is also important to note that since this method is achieved through restrictions in the drainageway, backwater calculations should be performed to establish flood lines.

Flow Regulators

The installation of flow regulators at inlets to storm sewers provides an effective means of preventing unacceptable storm sewer surcharging. The storm water exceeding the capacity of the storm sewer may be temporarily ponded on the road surface, or when this is not feasible, in off-line detention basins or underground tanks. Regulators may also be placed within large sewers as a means of achieving in-line system storage.

Ideally, flow regulators should be self-regulating, with minimum maintenance requirements. The simplest form of a flow regulator is an orifice with an opening sized for a given flow rate for the maximum head available. It is obviously important to avoid openings that could result in frequent clogging. For example, by placing a horizontal orifice directly under a catchbasin grating, the opening can be larger than for an orifice placed at the lower level of the outlet pipe, due to the reduction in head. Where orifice openings become too small, other forms of flow regulators designed to permit larger openings can be used. An example of such a device has been developed in Scandinavia, and has since been successfully applied in a number of installations in North America. This regulator utilizes the static head of stored water to create its own retarding energy, thus maintaining a relatively constant discharge.

It is particularly useful in existing developed areas experiencing basement flooding, such as occur with combined sewers or with separate storm sewers with foundation drains connected, as well as in areas with heavy infiltration into sanitary sewers. In such cases, all that is required is the addition of one or more storage reservoirs, each equipped with a regulator. By placing the regulator between a storage reservoir and a sewer, only the pre-determined rate of flow, which the sewer can handle without excessive surcharging, will be released (see Figure 6.6).

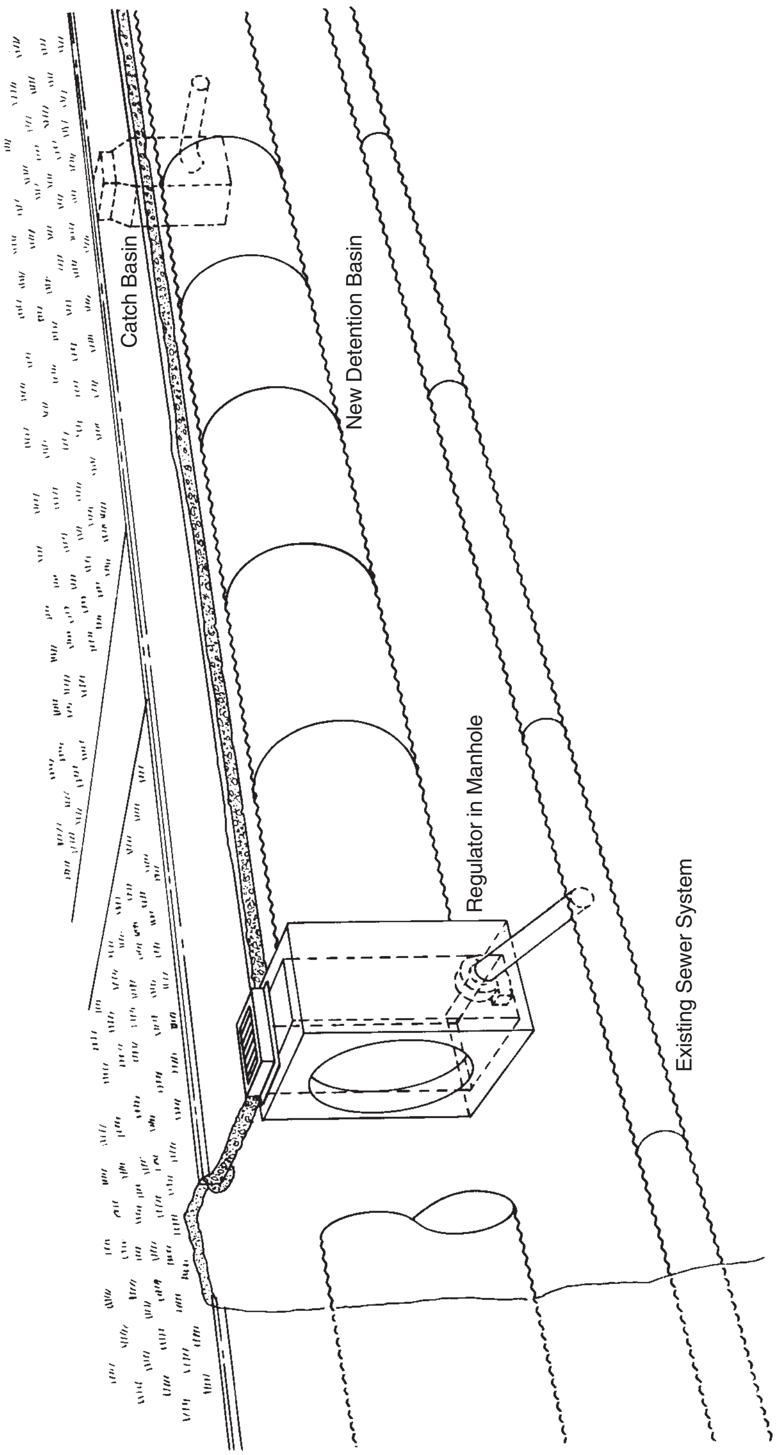


Figure 6.6 Typical installation of regulator for underground storage

SUBSURFACE DISPOSAL OF STORM WATER

Introduction

Increased urbanization has resulted in extensive construction of storm drainage facilities which reduce the natural storage and infiltration characteristics of rural land. The reliance on efficient drainage systems for surface water disposal creates a series of new problems. These include; high peak flows, lowering of the water table, reduction in base flow, excessive erosion, increased flooding and pollution. Nature, through a system of bogs, swamps, forested areas, and undulating terrain, intended that the water soak back into the earth. One approach which would help emulate nature's practices is to direct storm water back into the soil.

In areas where natural well-drained soils exist, subsurface disposal of storm water may be implemented as an effective means of storm water management.

The major advantages of using subsurface disposal of storm runoff are:

1) replenishment of groundwater reserves especially where municipal water is dependent on groundwater sources, or where overdraft of water is causing intrusion of sea water;
2) an economic alternative of disposing of storm runoff without the use of pumping stations, extensive outlet piping or drainage channels;
3) an effective method of reducing runoff rates;
4) a beneficial way to treat storm water by allowing it to percolate through the soil.

Numerous projects involving subsurface disposal of storm water have been constructed and have been proven to be successful. However, whether runoff is being conveyed overland or discharged to underground facilities, careful consideration should be given to any adverse impact that may result. In subsurface disposal this may include the adverse impact of percolated water on the existing quality of the groundwater.

METHODS OF SUBSURFACE DISPOSAL

A variety of methods are currently being employed in practice. The effectiveness and applicability of a given method should be evaluated for each location.[4,5] The basic methods are:

Infiltration Basins

Infiltration basins are depressions of varying size, either natural or excavated, into which storm water is conveyed and then permitted to infiltrate into the underlying material. Such basins may serve dual functions as both infiltration and storage facilities (see Figure 6.7). Infiltration basins may be integrated into park lands and open spaces in urban areas. In highway design they may be located in rights-of-way or in open space within freeway interchange loops.

The negative aspects to basins are their susceptibility to clogging and sedimentation and the considerable surface land area required. Basins also present the problems of security of standing water, and insect breeding (6).

Infiltration Trench

Infiltration trenches may be unsupported open cuts with stable side slope, or vertically sided trenches with a concrete slab cover, void of both backfill or drainage conduits, or trenches backfilled with porous aggregate and with perforated pipes (7) (See Figure 6.8 a & b). The addition of the perforated pipe in the infiltration trench will distribute storm water along the entire trench length, thus providing immediate access to the trench walls. It will also allow for the collection of sediment before it can enter the aggregate backfill. Since trenches may be placed in narrow bands and in complex alignments, they are particularly suited for use in road rights-of-way, parking lots, easements, or any area with limited space. A major concern in the design and the construction of infiltration trenches is the prevention of

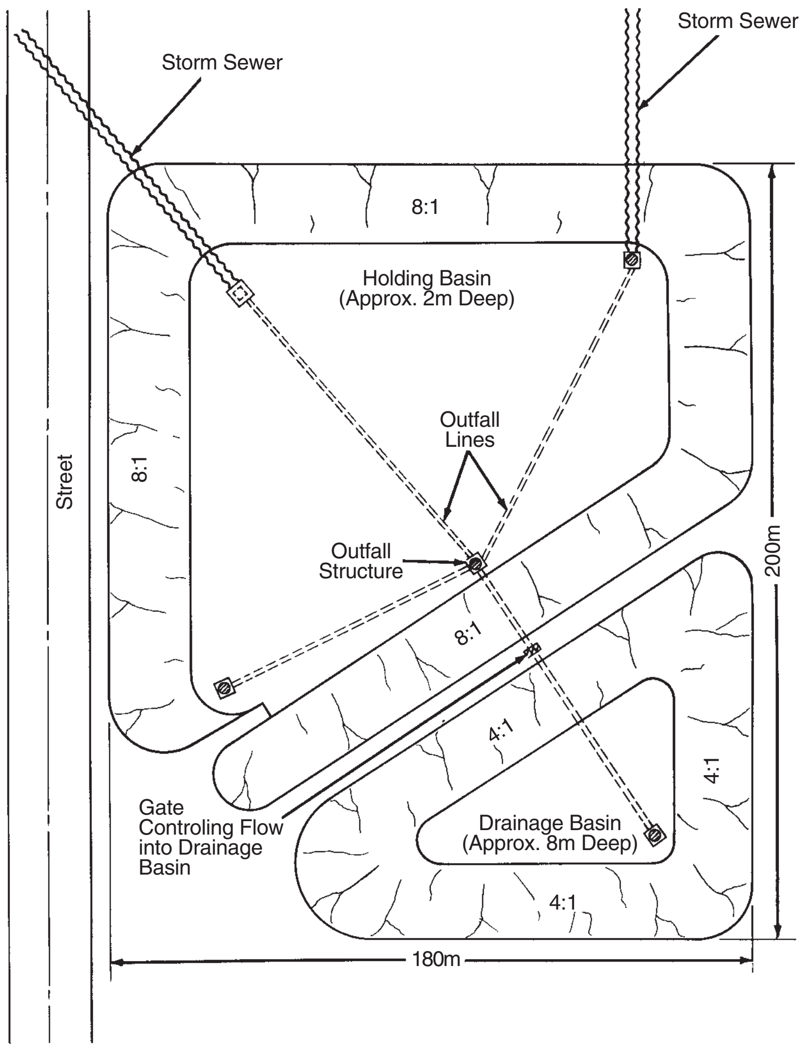


Figure 6.7 Infiltration basin

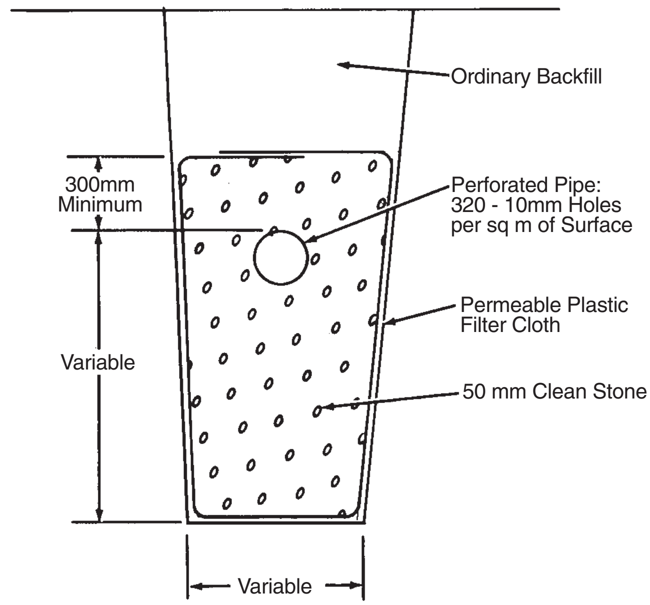


Figure 6.8a Typical trench for perforated storm sewer

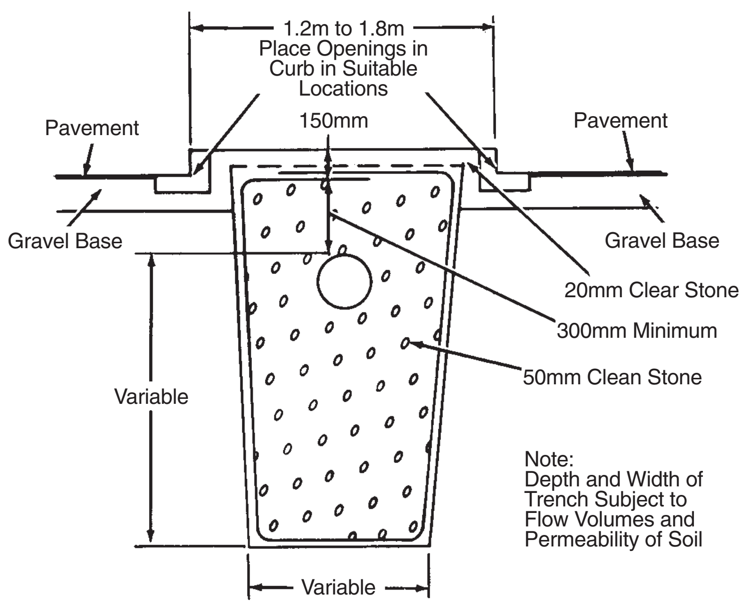


Figure 6.8b Typical trench for parking lot drainage

excessive silt from entering the aggregate backfill thus clogging the system. The use of deep catch basins, sediment traps, filtration manholes, synthetic filter cloths, and the installation of filter bags in catch basins has proven effective.

Retention Wells

The disposal of storm water directly into the subsurface may be achieved by the use of recharge wells (see Figure 6.9).

The versatility of such installations allows them to be used independently to remove standing water in areas difficult to drain, or in conjunction with infiltration basins to penetrate impermeable strata, or be employed as bottomless catch basins in conventional minor system design.

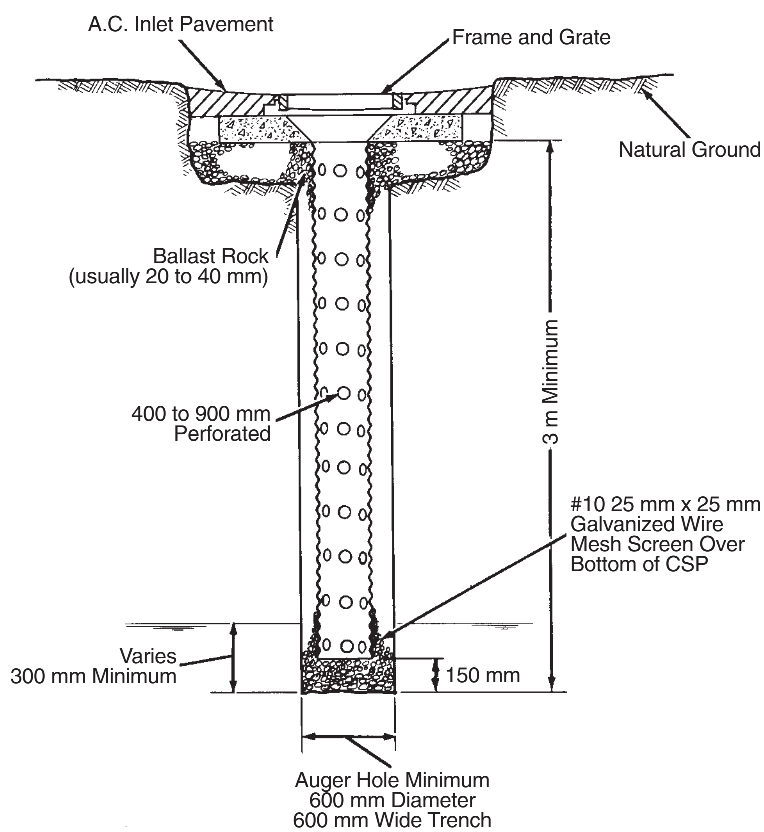


Figure 6.9 Recharge well

SOIL INVESTIGATION AND INFILTRATION TESTS

The rate of percolation (or infiltration) is dependent on many factors, including:

i) type and properties of surface and subsurface soils;

ii) geological conditions;

iii) natural ground slope;

iv) location of the water table.

Several contaminants including dissolved salts, chemical substances, oil, grease, silt, clay, and other suspended materials can clog surfaces reducing the infiltration rate.

The above would strongly suggest that the soil infiltration rate is best determined by carrying out field tests under known hydraulic gradients, water tables, and soil types. Laboratory tests are limited in that the condition within the laboratory may not simulate field conditions and they should only be used to estimate the infiltration rate.

Field investigations should concentrate on the following: (8)

i) The infiltration capability of the soil surfaces through which the water must enter the soil.

ii) The water-conducting capability of the subsoils that allow water to reach the underlying water table.

100 m of 3800 mm diameter, 2.8 mm structural plate pipe with gasketed seams used as an underground detention chamber collecting runoff from a shopping center.

iii) The capability of the subsoils and underlying soils and geological formations to move water away from the site.

iv) Flow from the system under mounding conditions (water table elevation = bottom of infiltration system) at the maximum infiltration rate.

Field Tests

Field tests may be carried out using various methods, including auger holes (cased or uncased), sample trenches, pits, or well pumping tests. The method chosen will depend on the type of facility to be designed and the site location parameters; i.e., presence of underground utilities, number of test sites required, requirements for maintenance of the vehicular and/or pedestrian traffic, type of equipment available to perform the test excavation, and type of soils. For a detailed description of alternative methods and the applicability of each, the reader is referred to a manual entitled "Underground Disposal of Storm Water Runoff," U.S. Department of Transportation.[7]

Laboratory Methods

The permeability of a soil sample may be calculated by laboratory methods. Two methods commonly used are the constant head test for coarse-grained soil, and the falling head test for fine-grained soils. Other laboratory methods for determining permeability are sieve analysis and hydrometer tests. Approximate permeabilities of different soils are listed below.[8]

Table 6.2 Coefficients of permeability

Typical	Value of K (mm/s)	Relative permeability
Coarse gravel	over 5	Very permeable
Sand, fine sand	5 - 0.05	Medium permeability
Silty sand, dirty sand	$0.05 - 5 \times 10^{-4}$	Low permeability
Silt	$5 \times 10^{-4} - 5 \times 10^{-6}$	Very low permeability
Clay	less than 5×10^{-6}	Practically impervious

Laboratory test specimens are mixtures of disturbed materials. The tests may therefore give permeabilities higher or lower than in situ materials. A factor of safety of 2 is commonly used to account for possible differences between laboratory and in situ values.

Darcy's law may be used to estimate the coefficient of permeability. A constant head is maintained during the laboratory test:

$$K = \frac{Q}{A \bullet i}$$

Where: Q = the rate of flow
A = cross sectional areas of soil through which flow takes place
K = coefficient of permeability
i = gradient or head loss over a given flow distance

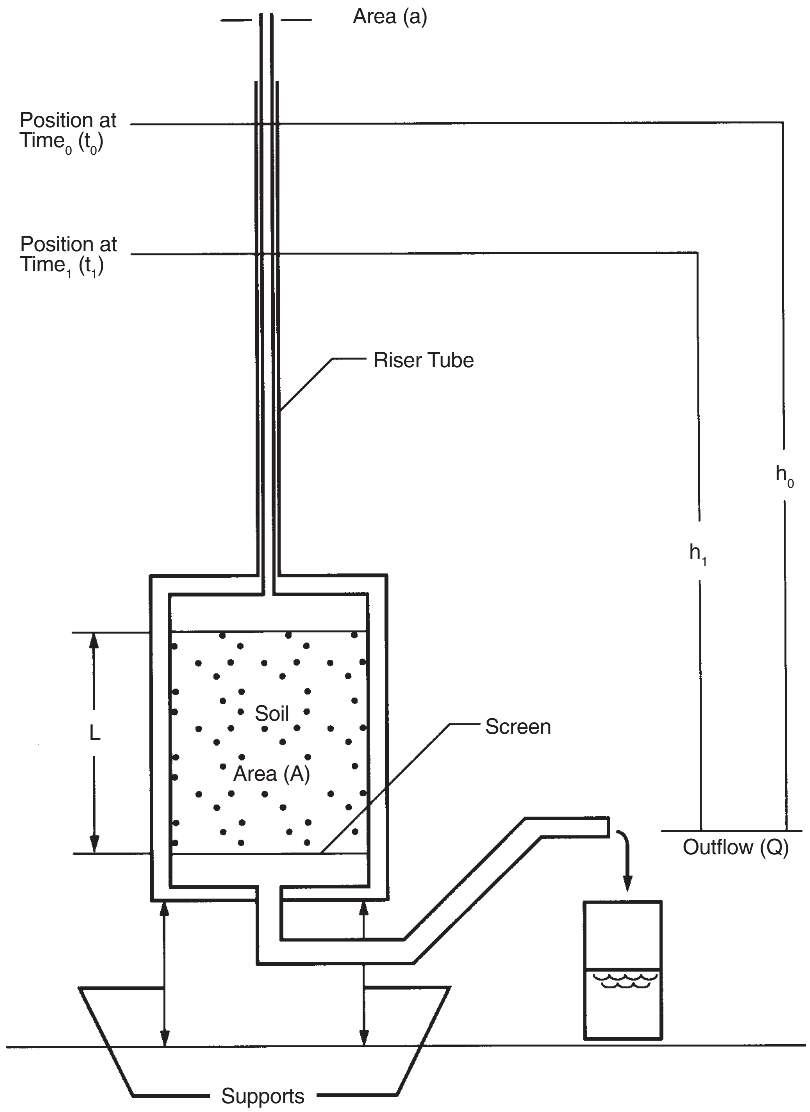


Figure 6.10 Falling head laboratory test

In the falling head laboratory test the head drops from the initial test point to the final test point (Figure 6.10). The following equation may be used to establish the coefficient of permeability:

$$K = \frac{2.3\ a\ L}{A\Delta t}\ \log_{10}\left(\frac{h_0}{h_1}\right)$$

Where: A = cross sectional area of the soil through which flow takes place
K = coefficient of permeability
L = length of the soil specimen
a = cross sectional area of the riser tube
Δt = time interval (t_1 - t_0)
h_0 = initial head
h_1 = final head

Indirect Methods

These methods are used when field or laboratory percolation tests have not been performed.

The simplest of these methods is the use of SCS soil classification maps. Since the maps only give a general idea of the basic soil types occurring in various areas, the soil classification should be verified by field investigation. Such maps will indicate in general the expected drainage characteristics of the soil classified as good, moderate, or poor drainage. This information may aid the designer in preliminary infiltration drainage feasibility studies. Further field permeability testing should be conducted before final design.

The specific surface method of New York State (9) may be used to calculate the saturated coefficient of permeability from an empirical equation relating porosity, specific surface of solids, and permeability. Field permeability tests are recommended before final design.

DESIGN TECHNIQUES

Subsurface disposal techniques have various applications which will result in both environmental and economic benefits. In designing any subsurface disposal system it should be realized that for many applications the rate of runoff is considerably greater than the rate of infiltration. This fact will cause some form of detention to be required for most subsurface disposal facilities. Modifications can also be made to existing systems to take advantage of the infiltration capacity of the soil.

Linear Recharge System

This system is similar to a conventional drainage system making use of catch basins and manholes, but storm runoff is directed to fully perforated pipes in trenches which allow for the exfiltration of the water over a larger area. Thus the zero increase in runoff criteria may be achieved by allowing the volume of water exceeding the pre-development flows to be disposed of into the subsurface stratum. Such systems are applicable to apartment developments, parking lots, or median or ditch drainage in highway construction.

Point Source and Recharge System

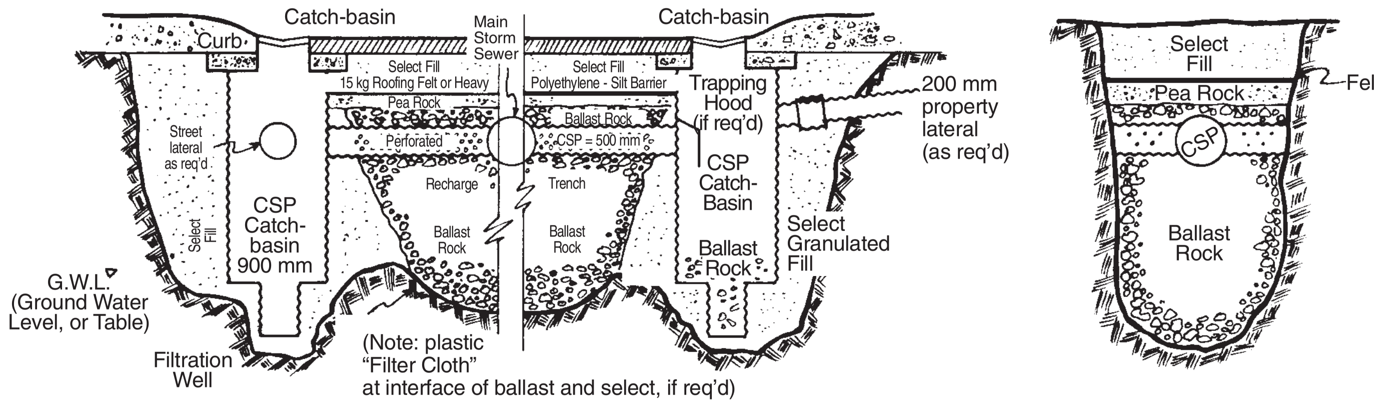
In small areas storm runoff may be collected and disposed of in perforated catch basins or wells. Fully perforated corrugated steel pipe surrounded by a stone filter medium has been found to be very suitable in these applications. In the past such systems were susceptible to silting up relatively quickly. The use of filter cloth surrounding the stone, and filter bags made of filter cloth placed in the catch basins, can virtually eliminate the clogging of the stone media with fines.

Combination System

In large developments fully perforated pipes my be used in place of conventional storm sewers, where soil conditions permit subsurface disposal. The design criteria described previously should be followed to assure that the system operates effectively. Recharge basins, fully perforated catch basins and manholes, detention areas, etc. may all be used as an effective means of stormwater management. Typical installations are shown in Figures 6.11 to 6.14.

An example of a combination underground detention chamber and recharge system. Five lines of 1800 mm diameter corrugated steel pipe with 150 mm slots in the invert.

(May be used in conjunction with longitudinal recharge trench and/or combination positive storm drain system)



LONGITUDINAL SECTION **TRANSVERSE SECTION**

Typical Material Specs:

Note: If "Select Fill" not available, intrusion of fines or sediment into the ballast rock may be prevented by use of new "filter cloth" materials.

Note: After ballast rock has been initially placed to desired elevation, wash down with clear water to allow for initial settlement. If settlement occurs, add additional ballast to required elevation, and complete recharge drain construction.

a. Corrugated Steel Pipe — CAN3-G401-93, latest revision

b. Ballast Rock - shall meet the gradation requirements as specified by AASHTO M-43 for size No. 24 (63mm to 19mm) or No. 4 (37.5mm to 19mm). Materials shall not have more than 45% loss of section as specified by AASHTO M-63, governing the Los Angeles Abrasion Test. It shall not show more than ten percent loss in 10 cycles as specified by the M-63 soundness test. The ballast rock shall be washed, and free of deleterious material.

c. Pea Rock - gradation shall be such that all of the material will pass the 25mm sieve, and not more than 5 percent will pass the 66mm sieve. Crushed lime rock will not be permitted.

d. Select Fill - shall consist of well-graded rock, or rock and coarse sand fill. Fine sand, or fill having a high proportion of sand and/or fines, will not be accepted. Otherwise, filter cloth materials may be specified for the select fill-ballast interface.

SUBSURFACE DISPOSAL OF STORMWATER RUNOFF can be an attractive alternative to present costly storm sewer conveyance systems. With the imposition of zero discharge, or zero increase of runoff regulations on land development in many urban areas, subsurface recharge may become a necessity for the drainage designer.

Figure 6.11 Typical street recharge system (French Drain)

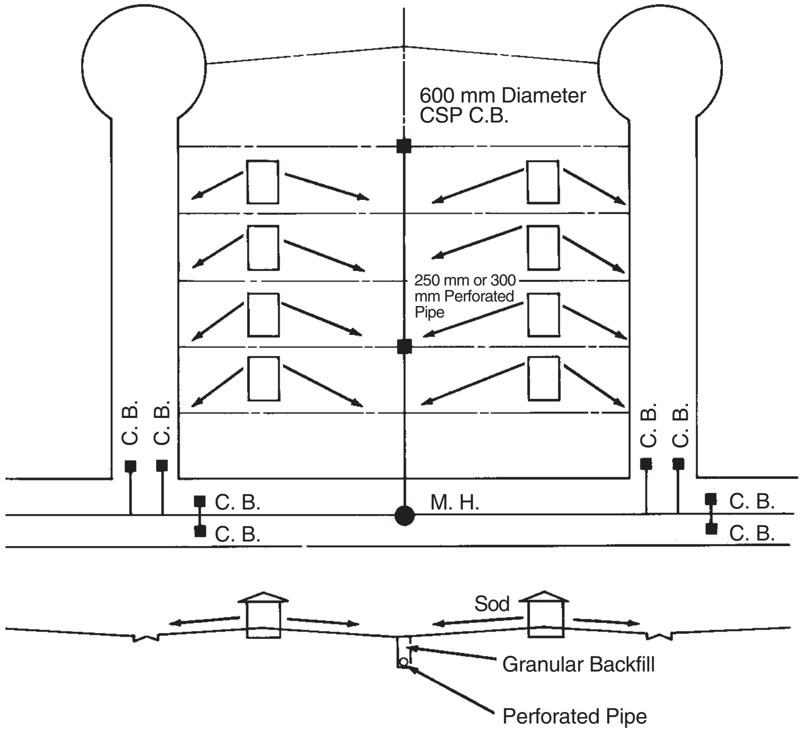
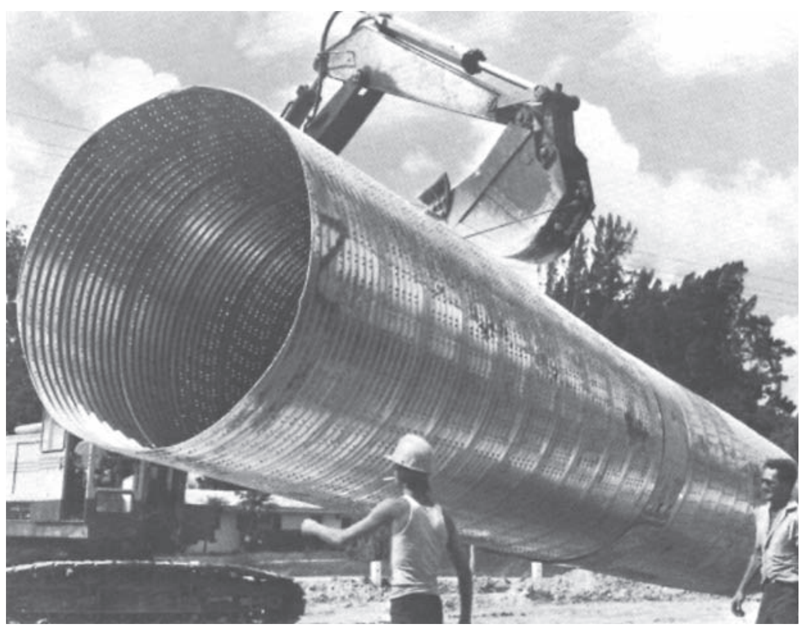


Figure 6.12 Typical plan for "underground disposal of stormwater runoff" for residential development



12 m length of 2100 mm fully perforated pipe banded together for installation.

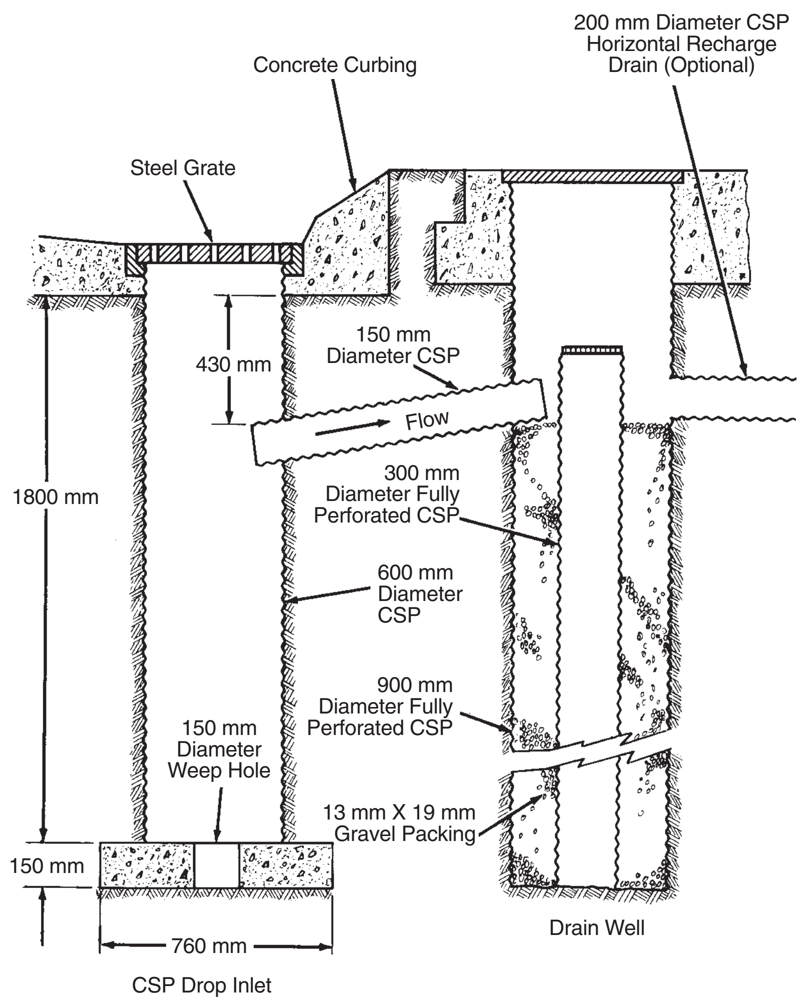
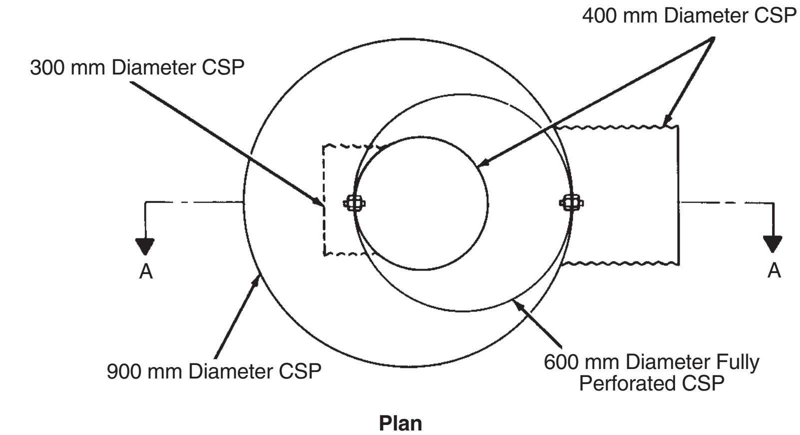


Figure 6.13 Typical design for combination catch basin for sand and sediment and recharge well. Catch basin would be periodically cleaned, and recharge well jetted through lower pipe to flush silt, and restore permeability



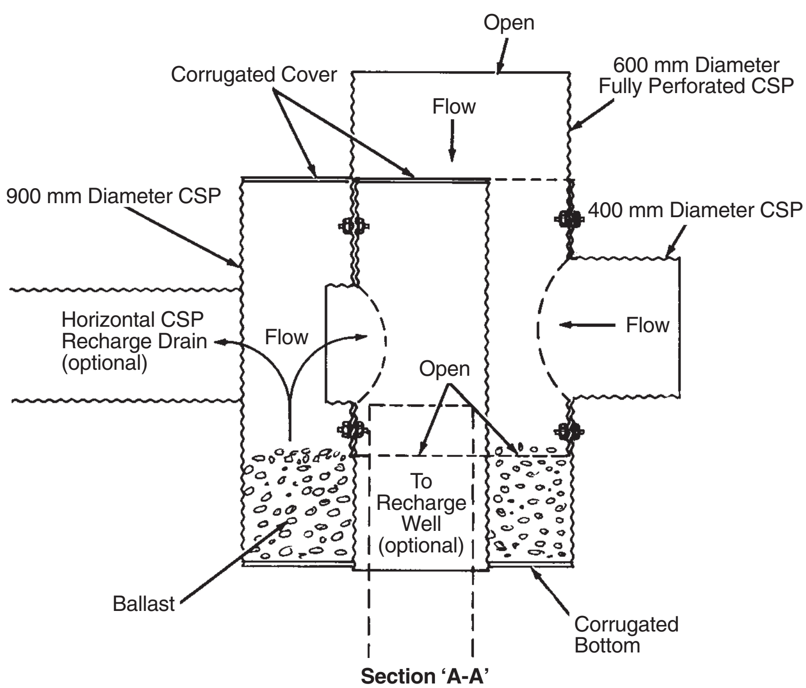


Figure 6.14 Typical CSP "Filter Manhole"

DESIGN PROCEDURE

The main steps to be followed when designing a stormwater subsurface disposal system are summarized as follows:

- Determine "Q," or storm runoff.
- Determine soil profile and groundwater levels.
- Determine infiltration rate.
- Design subsurface disposal systems.

The design runoff may be regulated by using the techniques discussed in Chapter 3, Hydrology.

Soil characteristics may be determined from subsurface soil investigations.

The potential infiltration rate or permeability may be estimated either from field or laboratory tests.

DESIGN EXAMPLE

Several design examples for infiltration basins, infiltration trenches, and retention wells are given in "Underground Disposal of Storm Water Runoff"[10]. A relatively straightforward example, using an estimated coefficient of permeability, Darcy's law, and a simplified hydrological method is given below.

An apartment development is proposed on a 0.55 ha site. The municipality requires a zero increase in runoff for a five-year storm. A combination system will be designed utilizing a regulator to discharge the pre-development outflow rate, with the excess storm being detained in an infiltration perforated pipe facility.

Determination of Pre-Development Peak Runoff

$A = 0.55$ ha $k = 0.00278$, constant factor

$T_c = 20$ Minutes

$I = 69$mm/hr (5 year storm)

$C = .2$ (pre-development)

$$Q = kCIA$$
$$= 0.00278\ (0.2)(69)(0.55)$$
$$= 0.021\ m^3/s$$

Exfiltration Analysis

Soils investigations indicate a relatively pervious sub-soil, with an estimated coefficient of permeability of $K = 6.68 \times 10^{-1}$mm/s. It is recommended that a factor of safety of 2 be applied to this figure when calculating exfiltration.

Exfiltration Calculations

A 900mm perforated pipe surrounded by 50mm clean stone will be used (Figure 6.15). The average trench surface area exposed for infiltration is 2m + 2m = 4m (trench walls only considered).

Surface area of trench for exfiltration = 4 m^2/m length

Length of trench = 12m

Area of exfiltration = 12m x 4 m^2/m = $48m^2$

The soil investigation has shown that the pervious subsoil extends 9m from the bottom of the trench to the ground water table. The hydraulic gradient (i) may now be estimated.

$$i = \frac{h}{l}$$

where: h = average available head

l = flow distance

$$i = \frac{1.0 + 9.0}{9.0} = 1.1$$

A hydraulic gradient of 1 will be used in the design.

Exfiltration from trench: Q = A•K•i √ safety factor.

$$= \frac{48m^2 \times 6.68 \times 10^{-4}\ m/s \times 1}{2.0}$$

$$= 1.61 \times 10^{-2}\ m^3/s$$

Time Min.	Accumulated[1] runoff vol. (m^3)	Allowable[2] release (m^3)	Exfil.[3] vol. (m^3)	Total outflow (m^3)	Storage requirements (m^3)
5	29.1	6.3	4.8	11.1	18.0
10	46.2	12.6	9.7	22.3	23.9
15	58.1	18.9	14.5	33.4	24.7[5]
20	66.6	25.2	19.3	44.5	22.1
30	81.6	37.8	29.0	66.8	14.8
40	89.7	50.4	38.6	89.0	0.7
60	105.0	47.0	58.0	105.0[4]	–
80	121.0	43.7	77.3	121.0	–
100	127.4	30.8	96.6	127.4	–
120	130.5	14.6	115.9	130.5	–
180	149.9	–	149.9	149.9	–

[1] Determined from mass outflow calculations using 5-year Intensity Duration Frequency Curve and post-development runoff factors.

[2] Rate of .021 (m^3/s) (5-year pre-development).

[3] Rate of 1.61 x 10^{-2} (m^3/s) (exfiltration rate).

[4] Once runoff volume becomes less than allowable release plus exfiltration volume then inflow equals outflow.

[5] Maximum storage required.

Storage requirement for a 5-year storm is 24.7 m^3.

Check storage capacity of pipe and trench.

$$\text{Pipe} = \frac{12 \times \neq(.9)^2}{4} = 7.63\ m^3$$

$$\text{Trench (43\% voids)} = (2 \times 2 \times 12 - \frac{\neq(.9)^2}{4} \times 12).43 = 17.36\ m^3$$

Total Volume Available = 24.99 m^3

∴ enough storage provided for excess water

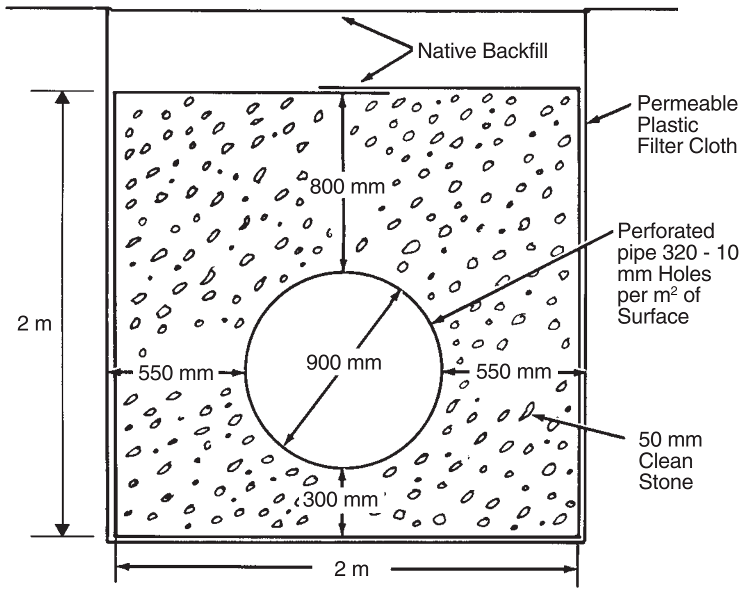
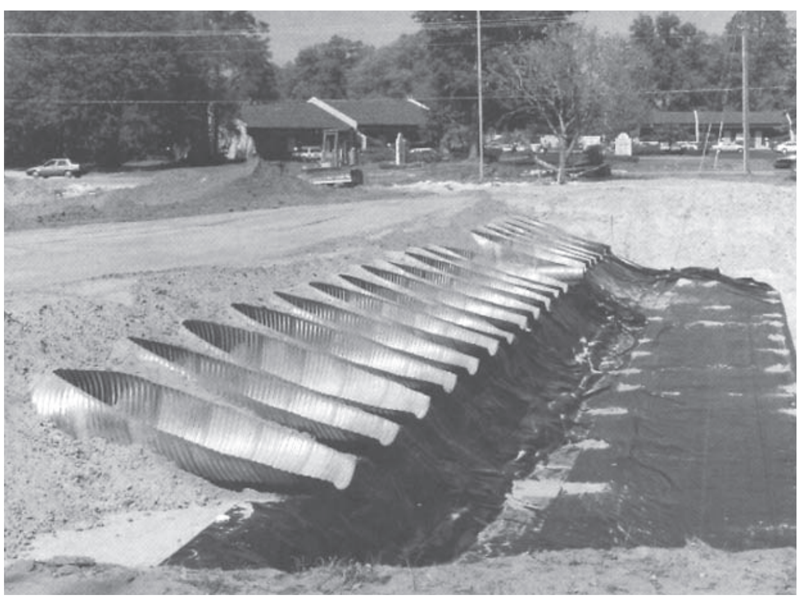


Figure 6.15 Infiltration trench cross-section



This construction site shows a detention basin, a 400 mm diameter corrugated steel storm drain pipe and 18 lines of 1200 mm diameter fully perforated corrugated steel pipe used for a recharge system.

CONSTRUCTION OF RECHARGE TRENCHES

Trench and ballast construction can be categorized under two soil conditions:

Trench in Permeable Rock and/or Stable Soil

A recharge trench of permeable soil or rock which will support its own walls without the need for protective shoring or cages is the least difficult to construct. Unless the sidewalls are heavy in silt or fines, there is rarely a need to line the trench walls with filter cloth to deter backflow of "fines" into the ballast rock filter.

Trench depth is not critical. The recharge CSP drain should be below the frost line, but there appears to be no problem in placing the trench bottom below normal groundwater level.

A bedding of ballast rock 25 - 50mm in size is laid prior to pipe placement, usually not less than 600mm deep. The perforated pipe is placed on the bedding, and covered a minimum of 300mm on sides and top, or up to the 6mm "pea gravel" level shown in the cross-section drawing. (Figure 6.11). A minimum of 150mm of the 6mm rock is laid over the ballast, and this in turn is covered with two layers of 15kg construction quality felt, or two layers of construction polyethylene sheeting. This barrier is most important in preventing the vertical infiltration of silts or sediments into the ballast rock, resulting in clogging of the recharge system. The sequence is finalized with earth or base course.

The construction sequence, as shown on the following pages, is carried forward as an "assembly line" process, with the entire sequence in close proximity. It is important that care be taken not to excavate any more trench than can be completed in the working period. If too much of the trench is excavated and the walls collapse, the trench will have to be re-excavated, and the fallen wall area replaced by ballast rock. Also, any rainfall may lead to an influx of sediments into the excavated area, resulting in clogging of the pervious layers in the trench wall.

Trench in Non-Cohesive Soil or Sand

Trench in non-cohesive soil or sand will result in a wider trench, and possibly the need for considerably more of the expensive ballast rock. A high percent of fines of either silt or sand may also suggest the advisability of a filter cloth between the ballast rock and native material.

A field-constructed "slip-form" of plywood can maintain the narrow width of ballast in the trench, and expedite the placement of the filter cloth envelope around the ballast rock. After excavation, the plywood form is set in place, the filter cloth is loosely tacked from the top and stretched down the sides of the form.

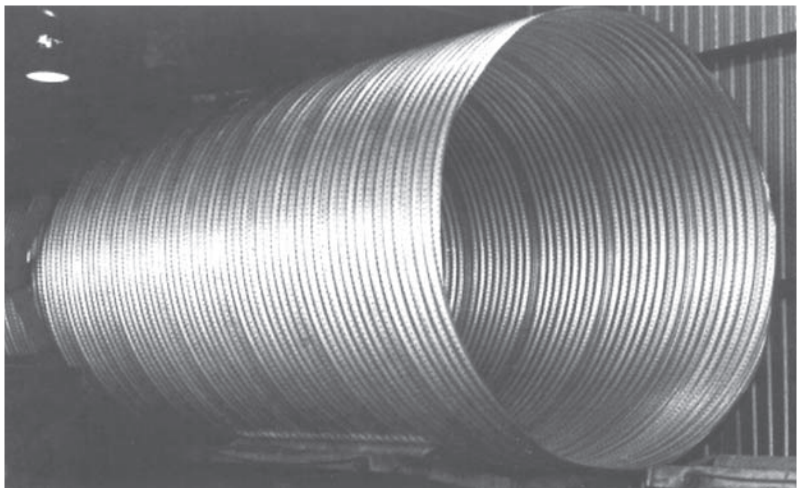
As the sequence of bedding, pipe-laying, ballast and side fill proceeds, the tacks are pulled, and the form slowly lifted. This allows the fill to hold the rock in place instead of the form, with the filter cloth in between. The sequence is continued until the ballast rock is to desired grade. The filter cloth is then lapped over the top of the ballast rock to finish the trench.

Perforated Pipe

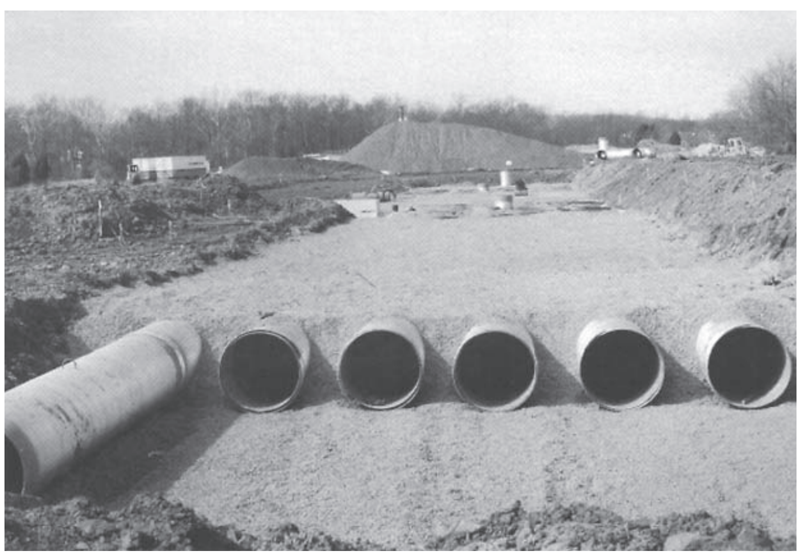
Fully perforated pipes are shown on page 190. Such pipes, when used in conjunction with an infiltration trench, allow for the entire concept of subsurface disposal of storm water.

Common recharge trench installation showing relative placement of perforated pipe ballast rock, gravel, and asphalt impregnated building paper.

Perforations of l0mm diameter uniformly spaced around the full periphery of the pipe are desirable, with not less than 320 perforations per square metre. Perforations of not less than 3mm may be used provided that an opening area of not less than 23,000mm^2 per m^2 of pipe surface is achieved. At manhole, junction, or other structures, the perforated pipe should be attached to a 1200mm stub of unperforated pipe attached to the structure. This will prevent piping at the structure with subsequent soil settlement.



3000 mm diameter 765 x 25 mm fully perforated pipe being fabricated on helical pipe mill.



One of two corrugated steel pipe detention chambers constructed on this industrial tract, each consisting of 730 m of 500 mm diameter pipe.

Synthetic Filter Fabrics

Multi-layered graded aggregate filters have been commonly used for the prevention of soil migration through the filter median. The diminishing supply of dependable aggregates and increasing prices has resulted in the increased use of synthetic filter fabrics. These fabrics are inert materials not susceptible to rot, mildew and insect and rodent attack.

Fabric filters must provide two important functions:

1) They must prevent the migration of fines to the aggregate material.
2) They must not inhibit the free flow of water. In situations where the fabric is to act as a separator, condition 1 need only be met.

Pipe Backfill

The aggregate material should provide sufficient void space to allow the free flow of water, and pass the fine sands, silts, silty clay and other fine material found in storm water without clogging. The void space will also provide additional storage within the trench.

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