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Underground Disposal of Storm Water Runoff

Design Guidelines Manual

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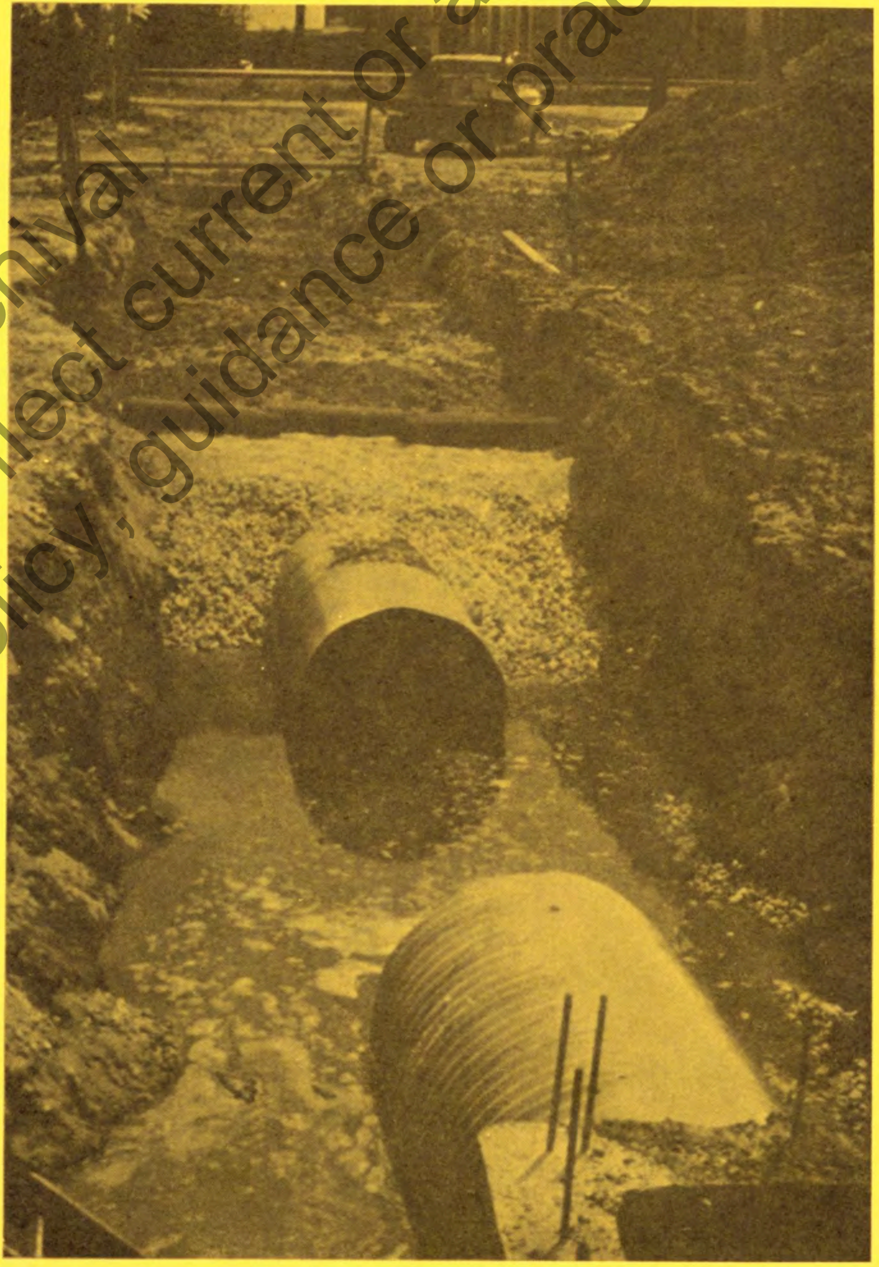

U.S. DEPARTMENT OF TRANSPORTATION
Federal Highway Administration
Offices of Research and Development
Implementation Division (HDV-21)

UNDERGROUND DISPOSAL

OF

STORM WATER RUNOFF

DESIGN GUIDELINES

MANUAL

By

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Prepared By

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Division of Construction
Office of Transportation Laboratory
In Cooperation With
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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS (Continued)

TABLE OF CONTENTS

APPENDIX

TABLE OF CONTENTS (Continued)

INDEX TO FIGURES

INDEX TO FIGURES (Continued)

INDEX TO FIGURES (Continued)

INDEX TO FIGURES (Continued)

INDEX TO TABLES

CONVERSION FACTORS
U.S. Customary to SI (Metric)

To convert	To	Multiply by
inches (in.)	millimeters (mm)	25.40
inches (in.)	centimeters (cm)	2.540
inches (in.)	meters (m)	0.0254
feet (ft)	meters (m)	0.305
miles (miles)	kilometers (km)	1.61
yards (yd)	meters (m)	0.91
square inches (sq in.)	square centimeters (cm^2)	6.45
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
acres (acre)	square meters (m^2)	4047
square miles (sq miles)	square kilometers (km^2)	2.59
cubic inches (cu in.)	cubic centimeters (cm^3)	16.4
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic feet per second (cu/ft/sec)	cubic meters per sec (m^3/sec)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lbs)	kilograms (kg)	0.453
tons (ton)	kilograms (kg)	907.2
pounds per square inch (psi)	Kilonewtons per square meter (KN/m^2)	6.9
gallons (gal)	cubic meters (m^3)	0.0038
acre-feet (acre-ft)	cubic meters (m^3)	1233
gallons per minute (gal/min)	cubic meters per minute (m^3/min)	0.0038

Reference: ASTM E-380-76

UNDERGROUND DISPOSAL OF STORM WATER RUNOFF

I. INTRODUCTION

The rapid growth of urban areas over the past few decades created the need for construction of extensive storm drainage facilities. Runoff collected by the proliferating paved streets and gutters was collected by storm sewer systems and conveyed directly to the nearest practical disposal point. Over the years, however, it has become apparent that the customary exclusive reliance on storm sewers for surface water disposal creates a series of new problems (1). Among the most critical of these are the following:

a. high peak flows in storm sewers and streams which require larger facilities at higher costs;

b. lowering of water tables, with a detrimental effect on existing vegetation; or salt water intrusion in coastal areas;

c. reduction in base flows in receiving streams, affecting aquatic life;

d. excessive erosion of streams and sedimentation in lakes, due to higher discharge velocities;

e. increased pollution of receiving streams and lakes due to industrial fallout on roofs, fertilizers from lawns and debris from streets and paved areas being conveyed directly to the streams;

f. Aggravated damage from flooding due to steadily increasing amounts of runoff.

Nature intended that this water soak back into the earth although present practice prevents it from doing so. In many places the water table has dropped sharply because of insufficient recharging of the ground whereas extensive flooding occurs downstream on a more and more frequent basis. It is obvious that if we continue in this manner, problems will increase to the point where we will be faced with costly damage of great magnitude. The obvious approach would be to design the storm drainage systems that will facilitate nature's process; that is, direct the storm water back into the soil.

New concepts of storm water drainage have developed in recent years. One such concept for disposal of storm water is through use of underground disposal by infiltration drainage. Although this method has not been extensively employed, water resources planners and drainage design engineers are now beginning to consider the infiltration drainage alternative because of the compelling advantages it affords.

The major advantages of using an infiltration system for subsurface disposal of storm water runoff include: 1) the replenishment of groundwater reserves where supplies are being depleted or where overdraft is causing contamination by salt-water intrusion; 2) an economical means of disposing of runoff where conventional methods may require the use of pumping stations or long mains to reach a suitable discharge location; 3) reduction in flow rates by infiltration and storage where the existing outfall is inadequate to carry peak discharges; and 4) a potential for removing pollutants by passage of water through soil. Other benefits include lower costs for surface drainage systems and a reduction in land subsidence. Surface retention prior to infiltration also allows for oxidation of organics and BOD reduction in storm water.

An infiltration drainage system may consist of one or several types of installations. It may be used alone or in combination with conventional systems; serve partially as a detention system and partially as a disposal system. It may be comprised of an open basin; covered disposal trenches utilizing coarse aggregate or pipe with slotted or round perforations; shallow or deep wells; or other components designed to infiltrate the maximum possible volume of runoff into the soil.

The infiltration drainage concept can be incorporated into the design of a transportation facility, commercial development, or subdivision area in many different ways. In the case of the former, little or no additional right-of-way may be required. Side ditches, median areas, unused space within interchanges, small land-locked areas, borrow areas, and space around rest areas are all potential sites. With imaginative planning, infiltration facilities such as the open basins can be terraced and landscaped to offer scenic enhancement and, in some cases, a park-like atmosphere. These systems produce many benefits and cause no negative effects when properly blended with the environment.

Infiltration drainage methods have been used in coastal areas of the United States for groundwater recharge and to solve special drainage problems. They are not limited to coastal areas, however, but may be used in any location where suitable soil conditions exist. Infiltration methods have been used extensively on Long Island, in Florida, parts of Texas, and in California. Research studies by the New York Department of Transportation on recharge basins for highway storm drainage have demonstrated the practicality of the method and, through full scale testing, have validated the design theory. A research study by the California Department of Transportation evaluated infiltration methods in northern California and identified important design considerations as related to highways. Considerable success has been gained in southern Florida with recharge concepts using infiltration trenches. Detention-infiltration systems have also been constructed in Canada. These types of systems may have application in other areas.

This manual has been developed based on experience which was derived from engineering judgment and applied theory. Its purpose is to provide the information necessary to evaluate for feasibility, as well as to plan and design, surface and subsurface infiltration systems or combination systems that can be incorporated into the overall drainage scheme of a particular transportation facility, street system, or commercial development. Basic criteria are presented with examples cited to assist the designer in selecting an appropriate system.

The next two chapters provide introductory and background information on the state-of-the-art utilization of systems for underground disposal of storm water. They provide solutions to problems of groundwater recharge, storm water disposal, and/or prevention of salt-water intrusion. Chapter III, entitled "General Considerations", includes criteria for the evaluation of alternative disposal systems, environmental and legal considerations, and general guidelines for soils exploration and investigation. Chapter IV includes specific design guidelines to enable the designer to plan and develop economical and environmentally feasible designs based on local hydrology and soil infiltration characteristics. Numerous design examples are used to aid the reader.

Chapter V, "Construction Methods and Precautions", and Chapter VI, "Maintenance and Inspection", provide information on the installation and long term performance of various infiltration systems.

The word infiltration, is a general term used throughout this manual to describe the flow of water into the soil. In the discussion of trench systems for subsurface disposal of storm water, the term exfiltration is used to describe the process in which water flows out of the trench or pipe conduit and into the soil.

Reference

1. Theil, P. E., "New Methods of Storm Water Management", Metropolitan Toronto and Region Conservation Authority, Storm Water Management Seminar, November 1977.

II. STATE-OF-THE-ART

Background

Artificial replenishment of groundwaters by surface infiltration has been practiced for many years. As early as 1895, flood waters of San Antonio Creek in southern California were conserved by spreading them on the alluvial fan at the mouth of San Antonio Canyon. After the construction of the City of Fresno's sewerage system in 1891 and until 1907, the city disposed of all of its wastewater on a 40-acre (161,880 m^2) tract. Over the years, Fresno has increased the size of its "Sewer Farm", which uses some surface sprinkling and a large number of infiltration ponds, covering some 1,440 acres (5.8×10^6 m^2) of land in 1972. Although some storm water reaches the site, most of the flows are treated sewage effluent.

Richter and Chun in 1961(1) reported that fifty-four agencies were actively practicing artificial ground water recharge in California, alone, in 1958. Many agencies elsewhere artificially replenish groundwaters. Barksdale and Debuchananne in 1946(2) describe the practice in New Jersey; Boswell in 1954(3) discusses artificial replenishment of groundwater in the London Basin; Brashears in 1946(4) provides information on artificial recharge as practiced on Long Island, New York; Cederstrom and Trainer(5) presented information in 1954 about groundwater recharge in Anchorage, Alaska; Kent(6) reported in 1954 on practices in the Union of South Africa; methods used in southwest Africa were described by Martin in 1954(7); and Sundstrom and Hood in 1952(8) describe the results of artificial recharge of groundwater at El Paso, Texas. An annotated bibliography on artificial recharge of groundwater through 1954 is presented in the U.S. Department of the Interior, Geological Survey, Water Supply Paper 1477(9). For those wishing to review the subject in detail, other published reports are available.

The U.S. Department of Agriculture, in 1970(10), published a summary of the principles of groundwater recharge hydrology which described the more common methods used. These include: basins, ditches or furrows, flooding, natural stream channels, pits and shafts, and injection wells. In the research report, "Infiltration Drainage of Highway Surface Water" (1969), Smith, et al(11) give a summary of the principles of infiltration drainage for highway surface water, and descriptions of the various kinds of systems with numerous references.

During the development of this manual, questionnaires were sent to a number of agencies and engineering consulting firms for the purpose of ascertaining to what extent infiltration systems were being utilized throughout the nation. The results of these inquiries are presented in Appendix A. Although these results represent only a sampling, they seem to indicate extensive utilization of infiltration drainage in localized areas of the country. In other areas, experience with infiltration procedures is almost nonexistent. Environmental and legal restraints are frequently cited as factors prohibiting the use of these systems. These restraints are addressed in Chapter III-A of this manual.

The following sections provide additional state-of-the-art information dealing with facilities constructed for subsurface disposal of storm water. These systems can provide for water conservation by groundwater replenishment and/or prevention of salt-water intrusion; or for disposal of storm water runoff. Basins, trenches, and infiltration well systems are discussed.

The final section of this chapter, "New Products and Methods for Aiding Infiltration", describes recent developments that have been beneficial to the planned infiltration of storm water.

1. Infiltration Basins

Infiltration basins are of natural or excavated open depressions of varying size in the ground surface for storage and infiltration of storm water. Weaver in 1971(12) presented theoretical and experimental work done by the New York State Department of Transportation to develop a procedure for designing infiltration basins. Weaver points out that increasing demands for fresh water and dwindling supplies, together with the advantage of constructing short trunk sewers leading to basins rather than the longer sewers that would have been needed, motivated the use of the infiltration basins on Long Island. More than 2000 infiltration basins are now in use on Long Island, New York.

In a discussion of artificial recharge in water resources management, Dvoracek and Peterson, in 1971(13), point out that maintenance requirements of infiltration basins are usually minimal. They state that, "cleaning the sediments from pits, trenches, and spreading basins is a relatively simple operation, possibly involving nothing more than tillage of these areas. In extreme cases, physical removal of sediment may be necessary." One method to partially offset the need for maintenance in areas of extreme climatic change is to allow the facilities to experience freeze/thaw action. Pit recharge rates have been known to increase sixfold due to freeze/thaw conditions during winter months. A physical breakdown of the surface seal seems to occur, facilitating self-maintenance.

Infiltration basins have been used extensively for many years in California's San Joaquin Valley in areas where immediate discharge of storm water from roadway rights-of-way would normally overtax the adjacent surface drainage systems or where an outfall is not available (11). They serve as storm water retention basins with possible infiltration benefits. However, infiltration is generally a secondary benefit, due to the low permeability of the clayey soils that exist throughout the San Joaquin Valley. In most cases it is considered a safety factor in designing the necessary storage volume of the systems. Other similar experiences are presented in Appendix A.

Many cities and local park districts combine plans for infiltration basin construction with green-belt zoning. This multi-use merging of the two facilities permits developement that is both practical and aesthetically pleasing. An example of a typical detention-infiltration basin in a city park is shown in Figure II-1. Details on the design and construction of these basins can be found in subsequent chapters of this manual. The American Public Works Association Special Report No. 43(14) is also an excellent reference for the location and design of detention systems in urban areas.

2. Infiltration Trenches

Infiltration trenches may be either unsupported open cuts with side slopes, flattened sufficient for stability; or essentially vertical-sided trenches with concrete slab cover, void of both backfill and drainage conduits where side support is not necessary (Figure II-2); or trenches backfilled with coarse aggregate and perforated pipes where side support is necessary (Figure II-3). Dvoracek and Peterson in 1971(13)

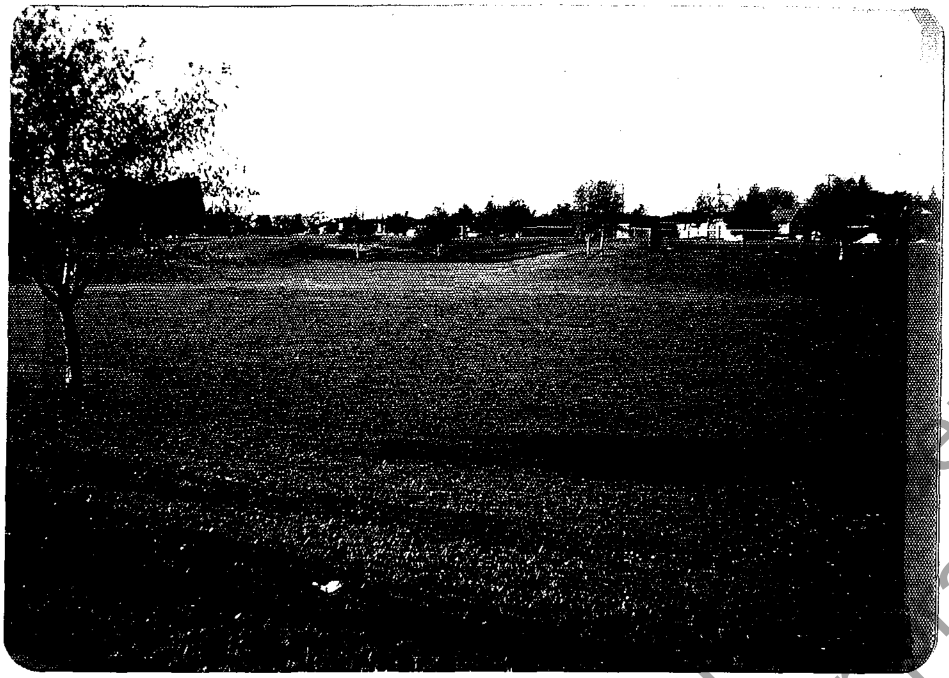


FIGURE II-1 TYPICAL DETENTION-INFILTRATION BASIN IN GREEN BELT AREA (COURTESY OF CALTRANS)

FIGURE II-2 INFILTRATION TRENCH WITH STABLE VERTICAL SIDE WALLS IN NATIVE MATERIAL WITH CONCRETE SLAB COVER (MIAMI AREA) (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA) describe the use of unsupported open recharge trenches as an alternative to pit recharge. "A long narrow trench, with its bottom width less than its depth . . . is utilized rather than the large rectangular pit. Dependent upon the infiltration characteristics of the material into which the trench penetrates and the location of the water table, high rates of recharge are generally expected". Infiltration trenches have been used successfully in southern Florida under high groundwater conditions but have required special engineering considerations. The infiltration trench is a modification of the infiltration basin, discussed in Section 1. Porter in 1976(15) discusses the advantages of covered drainage trenches for "recharge to ground" of storm water runoff. A typical trench cross section is shown in Figure II-4.

The addition of perforated pipe to the infiltration basin concept increases the exfiltration from the trench by more than 100 times that of conventional "French drains" or dry wells which are limited by cross-sectional area. It also serves the function of collecting sediment before it can enter the coarse rock backfill. As collected, sediments are distributed throughout the length of the freeflow area, and clogging is minimized. For example, the sediment-laden water must flow through the cross-section of the conventional French drain to flood the trench and gain access to the trench wall. The perforated pipe distributes the water immediately for its full length, providing immediate access to the trench wall. A French drain 8 ft (2.44 m) deep and 4 ft (1.22 m) wide must exfiltrate through a 32 ft^2 (2.98 m^2) cross-sectional surface. An infiltration trench with 36 inch (0.915 m) diameter pipe running between inlet structures 200 ft (61 m) apart exfiltrates to the coarse backfill rock for the full trench length through an area of $\pi d \times \ell$ = 3.1416 x 3 ft (0.915 m) x 200 ft (61 m) = 1,885 ft^2 (175.3 m^2).

FIGURE II-3 INFILTRATION TRENCH WITH PERFORATED PIPE AND COARSE ROCK BACKFILL. NOTE GROUNDWATER LEVEL IN EXCAVATION (MIAMI AREA) (COURTESY OF DADE COUNTY DEPT. OF PUBLIC WORKS, MIAMI, FLORIDA)

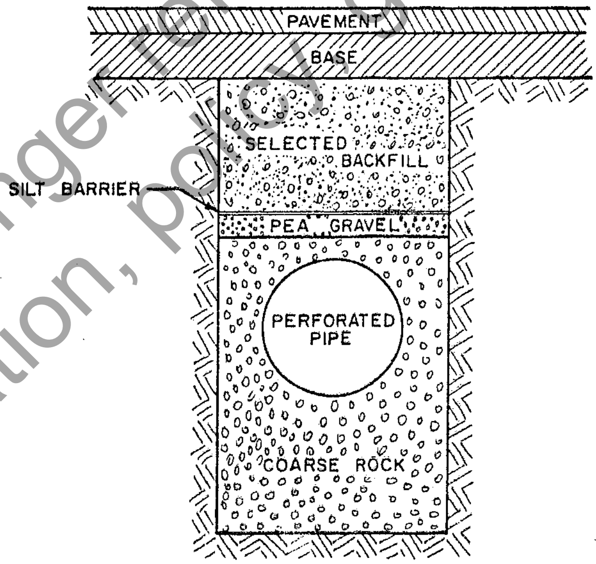


FIGURE II-4 TYPICAL CROSS-SECTION OF INFILTRATION TRENCH (COURTESY OF DADE COUNTY DEPT. OF PUBLIC WORKS, MIAMI, FLORIDA)

Infiltration trenches have been used extensively in Dade County, Florida, and in other areas of the State, as well as in some parts of Canada, as discussed by Porter(15) and Theil(16). A listing of performance information on various installations is provided in Appendix B. Refer to Section II-4 of this Chapter, "New Products and Methods for Aiding Infiltration" for a description of perforated pipe. Examples of these systems are also described and illustrated in detail in other Chapters of this manual.

3. Infiltration Wells

Recharge or infiltration wells have been used for many decades for conducting water into the ground. Perhaps the oldest kind is the "dry well", which is a small-diameter hole or pit dug into the ground for the disposal of water that has no natural drainage. A dry well is usually filled with pea gravel, coarse sand, or other aggregate, or contains a slotted or perforated pipe, backfilled with materials which allow water to penetrate and soak into the ground, while preventing collapse of the walls. Frequently, a layer of filter sand is placed in the top few inches (0.1 m±) of a well and mounded up slightly over the well, to trap silt and other sediment that might clog the well. The sand can be periodically, removed and cleaned, or replaced. An enlarged version of the dry well is the "seepage pit" used for disposal of sewage from septic tanks. These are discussed in detail by the U.S. Department of Health, Education, and Welfare's Public Health Service Publication No. 526(17). In some States, seepage pits are permitted when absorption fields are impracticable, and/or where the top 3 or 4 feet (0.9 or 1.2 m) of soil is underlain with porous sand or fine gravel and the subsurface conditions are otherwise suitable for pit installations.

Abandoned wells, or wells specifically designed for artificial recharge, have been used for many years to inject water into the ground. The U.S. Department of Agriculture publication 1970(10), states: "The use of injection wells is confined largely to areas where surface spreading is not feasible because extensive and thick impermeable clay layers overlie the principal waterbearing deposits. They may also be economically used in metropolitan areas where land values are too high to use the more common basin, flooding, and ditch-and-furrow methods."

This publication also points out: "Many attempts to recharge groundwater through injection wells have been disappointing. Difficulties in maintaining adequate recharge rates have been attributed to silting, bacterial and algae growths, air entrainment, rearrangement of soil particles, and flocculation caused by reaction of high-sodium water with soil particles."

Cased, gravel-packed wells have been used for injecting good quality water to provide a barrier to salt water intrusion. Bruington and Seares(18) in 1965 reported "The control of intrusion of coastal groundwater basins by sea water has become of economic importance in groundwater basin management."

Many researchers have contributed to the body of knowledge on flows to and from wells. Muskat(19) in 1937 developed theories for steady-state seepage toward a single well, small groups of wells, and infinite sets of wells in one-, two-, and three-line arrays. His work provides the background for many refinements in seepage theory that have been developed in recent years. Hantush(20) (1963), Glover(21) (1966), Leonards(22) (1962), Peterson(23) (1961), Harr(24) (1962), and Todd(25) (1959) are just a few references on well theory.

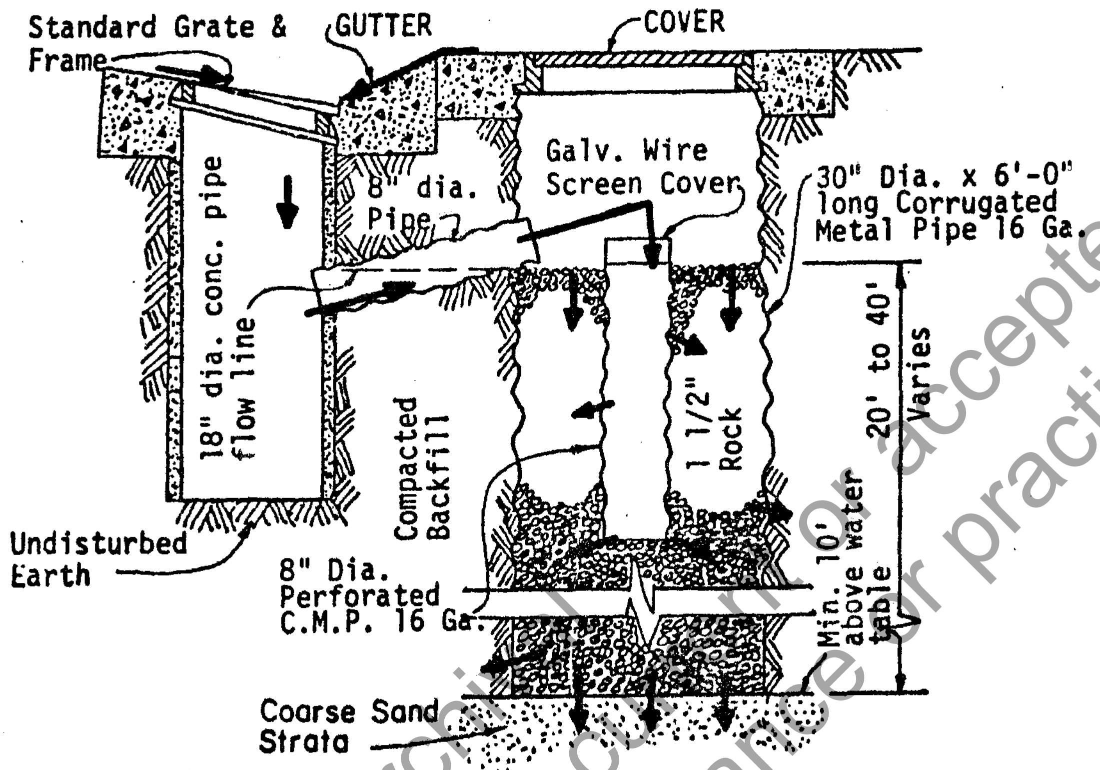
Kashef(26) in 1976 reported the results of a theoretical study of the effect of injection into batteries of wells on salt-water intrusion. His report presents charts that may be useful to those managing salt-water intrusion systems using injection wells.

Even though well theories can be useful to those designing water injection or recharge wells, numerous practical considerations ultimately determine their effectiveness. For example, Reference(10) from the U.S. Department of Agriculture contains the following statement, ". . . the Los Angeles County Flood Control District in California has successfully operated injection wells as part of a large-scale field experiment to ascertain the feasibility of creating and maintaining a fresh-water ridge to halt sea-water intrusion in the Manhattan-Redondo Beach area in Los Angeles County. In general, it has found that gravel-packed wells operate more efficiently and require less maintenance than non gravel-packed wells. At Manhattan Beach, California, a 24-inch (0.61 meter) gravel-packed well with an 8-inch (0.203 m) casing was found more desirable for recharging purposes. On Long Island, New York, where cooling water is returned to the ground-water basin, a minimum casing size of 8-inches (0.203 m) and a minimum packing of 2-inches (0.05 m) have been recommended."

The Transportation Laboratory of the California Department of Transportation in a 1969 report(11) discussed recharge or "drainage" wells as follows: "Drainage wells are basically water supply wells operating in reverse, although, in practice, they have many unique features and problems. There are also several types, ranging from simple gravel-filled shafts to highly sophisticated pump injection wells. Like basins, they have both good and bad features. Wells require a minimum of space and may be designed with very little unsightly surface structure. They can be extended through impervious soils down to permeable sand or gravel, and will drain a small area fairly rapidly when surface runoff is of satisfactory quality.

"Unfortunately, wells clog up very easily when the water contains silt or sediment, and cleaning or restoration can be difficult. Drainage wells are readily capable of polluting groundwater supplies and health departments have strict regulations regarding them. Capacity for drainage is difficult to predict: one well may have a good rate of infiltration, while another 50 feet (15.3 m) away will drain very poorly. The cost of well construction and maintenance makes well drainage a fairly expensive method of disposal. Basins are much more economical in terms of cost per unit volume of water drained. Normally, a drain well should be considered for disposal of small quantities of water, or as a supplement to recharge basins or some other type of disposal system."

The City of Modesto, California, with an average annual rainfall of 12 inches (305 mm), makes extensive use of drain or rock wells to serve seventy percent of the city area. Their experience with over 6,500 individual installations has varied. Some wells, considered as marginal, have resulted in ponding on streets following severe rainstorms. These facilities have required continuing maintenance. Figure II-5 shows a typical cross-section of the standard "rock well" used in the City of Modesto for street drainage.



Note: Arrows depict flow of storm water

FIGURE II-5 CROSS-SECTION OF STANDARD ROCK WELL (DRAIN WELL) INSTALLATION FOR STREET DRAINAGE (COURTESY OF MODESTO, CALIF. DEPT. OF PUBLIC WORKS)

Infiltration wells or "diffusion" wells, as used by the New York Department of Transportation on Long Island are large, often very deep, concrete-lined pits. Weaver(12) states: "As used by this Department on Long Island, these have customarily been large vertical shafts constructed of reinforced concrete precast sections. The sections are 6-feet (1.8 m) high with a 16-inch (0.406 m) wall and an inner diameter of 10 to 16 feet (3.1 to 4.9 m). A diffusion (infiltration) well is constructed in the same manner as a drop shaft or open caisson. The shell sinks under its own weight as the soil at the bottom is excavated, and additional sections are added from the surface. By means of rectangular openings through the wall, each 10-foot (3.1 m) inside diameter section provides approximately 9.1 square feet (0.85 m^2) of effective lateral drainage area. When the shaft is completed, a heavy reinforced concrete cover is placed over the top. The cover contains an open grating about 8 square feet (0.74 m^2) in size. Over the cover at the floor of the basin, a graded filter is placed to prevent silt from entering the well." Weaver points out that most of these wells have been carried at least 6 feet (1.83 m) below the water table and often to depths between 100 and 200 feet (30.5 to 61 m). He indicates these shafts or wells have most often been used as a remedial measure to correct the results of inadequate design and/or inadequate maintenance of existing infiltration basins. Because of their high cost, there is a question as to whether this type of recharge well is justified on the basis of hydraulic conductivity. Weaver emphasizes that his department makes use of seepage analyses methods to estimate their inflow capacities even though the "design of a diffusion well is a multi-component, highly complex task." He also states: "Owing to their high cost and low efficiency, they are the least desirable method of disposing of highway drainage. Also, because of their low efficiency, a rather large infiltration basin is necessary merely to hold the storm inflow for eventual disposal by the diffusion well, so that wells are <u>not alternates</u> to basins--they are an <u>extra cost</u> added to the basin cost."

Various patented dry well systems are available for subsurface disposal of stormwater. These systems are very similar to those previously discussed.

4. New Products and Methods for Aiding Infiltration

a. Synthetic Filter Fabrics

Many kinds of engineering and agricultural drainage systems make use of graded filters or multiple-layer drains for the safe removal of water from soil formations. When aggregates are used, their gradations are usually established with the well-known "Terzaghi" or "Bertram" filter criteria. These are discussed in Chapter IV-C, "Design of Storm Water Collection and Disposal Systems." Good quality mineral aggregates are virtually indestructible, and until recently have been economical and available in many geographical locations. However, as the supplies of dependable aggregates has diminished and the cost of placing more than one kind of aggregate (in trenches, for example) has increased, there has been an impetus to make use of the synthetic fabrics either to act as separators to keep fine erodible soils out of porous drains, or to work as filters to allow free flow of water while preventing the movement of the erodible soils. Barrett(27) (1966), Calhoun(28) (1972), Dunham and Barrett(29) (1974), the U.S. Army(30) (1975), Carroll(31,32) (1975, 1976), Rosen and Marks(33) (1975), Seemel(34) (1976), and many others worked with fabrics and developed standards and specifications for their use.

Polyvinylidene chloride, polypropylene, and other synthetic resins used in making filter fabrics are inert materials not subject to rot, mildew, or insect and rodent attack. They are, however, very sensitive to long term exposure to ultraviolet components of sunlight. Also, some are affected by alkalies, acidic material, components of asphalt, or fuel oils. If a fabric is substituted for an aggregate filter, care should be taken to prevent tearing or puncture of the fabric. Adjacent sheets should be overlapped and secured to prevent openings from developing.

To insure the required performance for the life expectancy of the project, synthetic fabrics (either woven or non-woven) for infiltration systems or any other long-term application, must be carefully selected, based on the properties required. As with aggregate filters, fabric filters must provide two very important functions: (1) they must be able to prevent clogging of the drain by erodible soil or other material, which could also result in erosion, piping, or other problems with the facility being protected; and (2), they must not inhibit the free flow of water. In situations where the fabrics work only as separators, and there is no significant flow of water, they need only satisfy the first requirement.

b. Precast Concrete or Formed-in-Place Perforated Slabs

The current emphasis on storm water management has resulted in new drainage concepts aimed at reducing the flow of storm water from developed areas. Smith(35) in 1974 described the use of porous precast paving slabs with perforations as a means to induce water to soak in and not flow off large parking areas, while these areas support grass in keeping with the "green belt" concept. This concept involves the use of proprietary formers and patented processes to produce reinforced concrete with holes that allow water to soak in and grass to grow. These materials produce grassy looking parking areas that are self draining, mud-free, and attractive in appearance. In essence they produce a load-bearing lawn which can absorb a good deal of rain thereby reducing surface runoff.

c. Porous Pavements

Porous pavements have been suggested in recent years to recharge groundwater supplies and reduce storm water runoff(36,37,38). These pavements allow storm water to infiltrate through the pavement surface and be stored in the structural section for eventual percolation through the underlying native soil. This idea may have merit for parking lots but is not recommended for pavements that are subjected to large numbers of repetitions of heavy wheel loads which could increase replacement and maintenance costs.

Porous pavements are designed based on the load-bearing capacity of a saturated subgrade for an expected number of wheel load repetitions. The porous structural section is designed with sufficient reservoir capacity to handle the design rainfall. To function properly and provide vertical drainage, the native subgrade soil should have high permeability. Figure II-6 illustrates a structural section for a typical porous asphalt concrete parking lot pavement. The pavement provides storage for 4.20 inches (107 mm) of rainfall assuming 15 percent voids in the surfacing and 30 percent voids in the aggregate base.

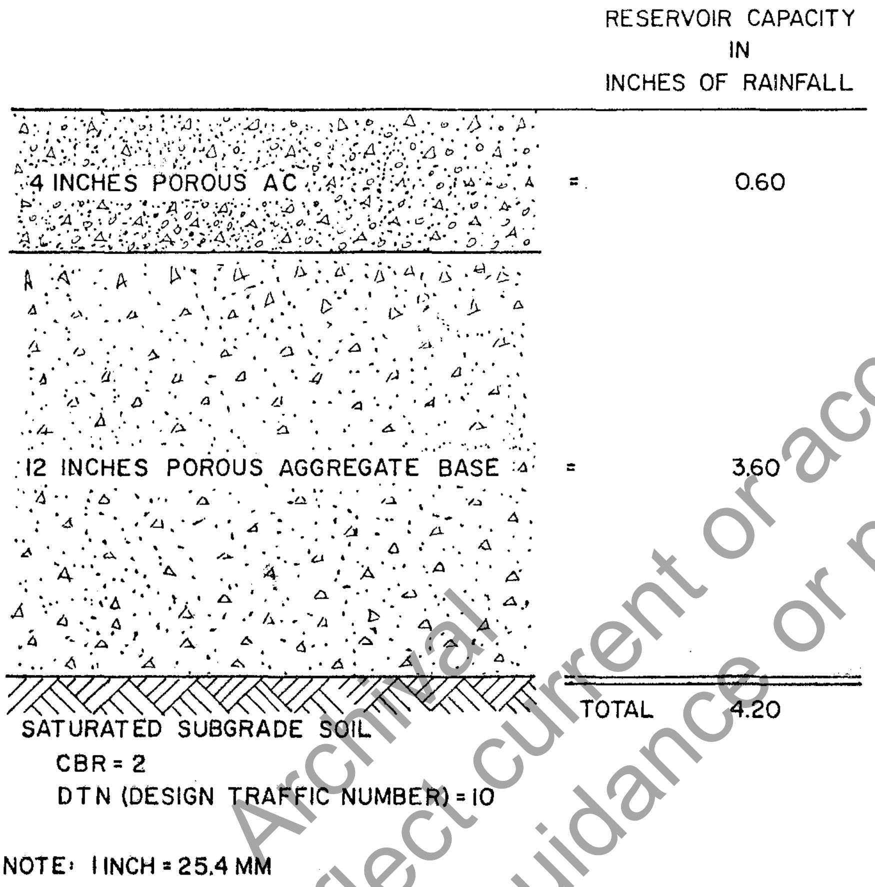


FIGURE II-6 TYPICAL POROUS ASPHALT CONCRETE PARKING LOT PAVEMENT [AFTER (36)]

For design of pavements refer to the Design Manuals of the Asphalt Institute, Cement and Concrete Association or other references on the subject.

d. Perforated or Slotted Pipe

The Corrugated Steel Pipe Institute "Drainage Technology Newsletter", November, 1976(15), describes a new type of fully perforated pipe for use in trench drains of the kind used by Dade County, Florida, for temporary storage and subsurface disposal of storm water. Pipes manufactured of aluminum, concrete, and other materials are also available for this application.

For perforated corrugated metal pipes [CMP 3/8 inch (9.5 mm)] diameter perforations uniformly spaced around the full periphery of a pipe are desirable. Not less than 30 perforations per square foot (0.093 m^2) of pipe surface should be provided. Perforations not less than 5/16 inch (8.0 mm) in diameter or slots can be used if they provide an opening area not less than 3.31 square inches (2135 mm^2) per square foot (0.093 m^2) of pipe surface. The photo in Figure II-7 shows the inside of a metal pipe with perforations around the full periphery.

The liberal number of holes are to insure free and rapid flow in and out of the pipe. The purpose of the large-sized pipes is to add to the total storage volume for storm water and to reduce the quantity of expensive rock backfill.

Coarse gravel or other aggregate is used for backfilling the trench around, below, and above the pipe so that part of the storm water is temporarily stored in the voids of the backfill. The photo in Figure II-8 shows the typical coarse rock used for infiltration trench backfill. Experiments made by the Dade County Department of Public Works have indicated that 3/4 inch x 1 1/2 inch (19 mm x 38 mm) coarse gravel backfill with pipe systems having 3.31 square inches per square foot (23×10^3 mm^2/m^2) of perforations will

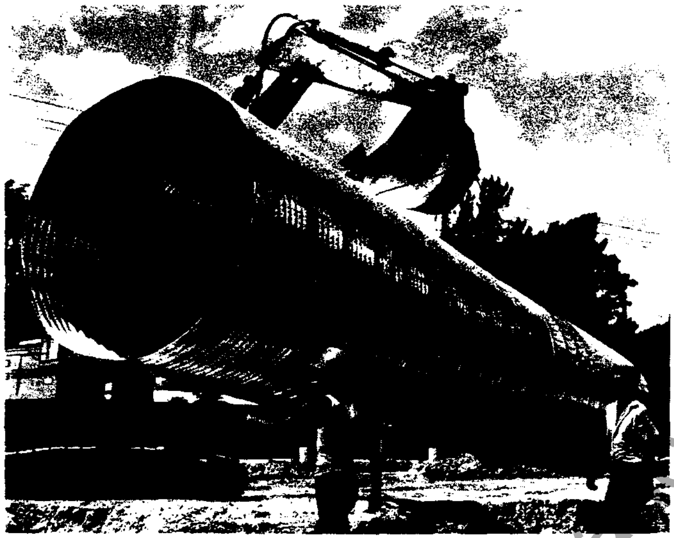


FIGURE II-7 TYPICAL PERFORATED PIPE FOR INFILTRATION TRENCH CONSTRUCTION (COURTESY OF SYRACUSE TANK & MANUFACTURING CO., WEST PALM BEACH, FLORIDA)

FIGURE II-8 TYPICAL COARSE ROCK FOR INFILTRATION TRENCH BACKFILL (COURTESY OF DADE COUNTY DEPT. OF PUBLIC WORKS, MIAMI, FLORIDA) provide pipe exfiltration rates which exceed the best infiltration rates of soils normally encountered in the field. Refer to Appendix C for information on experimental development tests by Dade County.

In addition to utilizing fully perforated CMP, the Florida Department of Transportation has utilized slotted concrete pipe on several south Florida installations. Pipe meeting the general requirements of ASTM C-76 is modified to provide 3/8 inch (9.5 mm) wide slots. The slots are either saw cut after casting or formed in the fresh concrete during casting. The slots are either centered about the springline and staggered on both sides of the pipe barrel by saw cuts (Alternate A) or cast above and below the springline (Alternate B). No significant reduction in strength has been observed using the standardized details shown in Figure II-9. The design provides sufficient pipe exfiltration rates. Additional slots could be provided when soils with extremely high infiltration rates are encountered. Pipe diameters between 18 inches (0.458 m) and 48 inches (1.22 m) have been used, depending on flow and storage requirements. Although the installations have not been test verified, one 48 inch (1.22 m) diameter slotted concrete pipe in a coarse rock trench in a high permeable clean sand apparently exfiltrated runoff from a severe storm without significant discharge from the positive relief drain. The storm deposited approximately 11 inches (0.28 m) of rainfall within a 10 hour period. Controlled field tests using pipe with precast slots have recently verified the performance of this alternate slot detail.

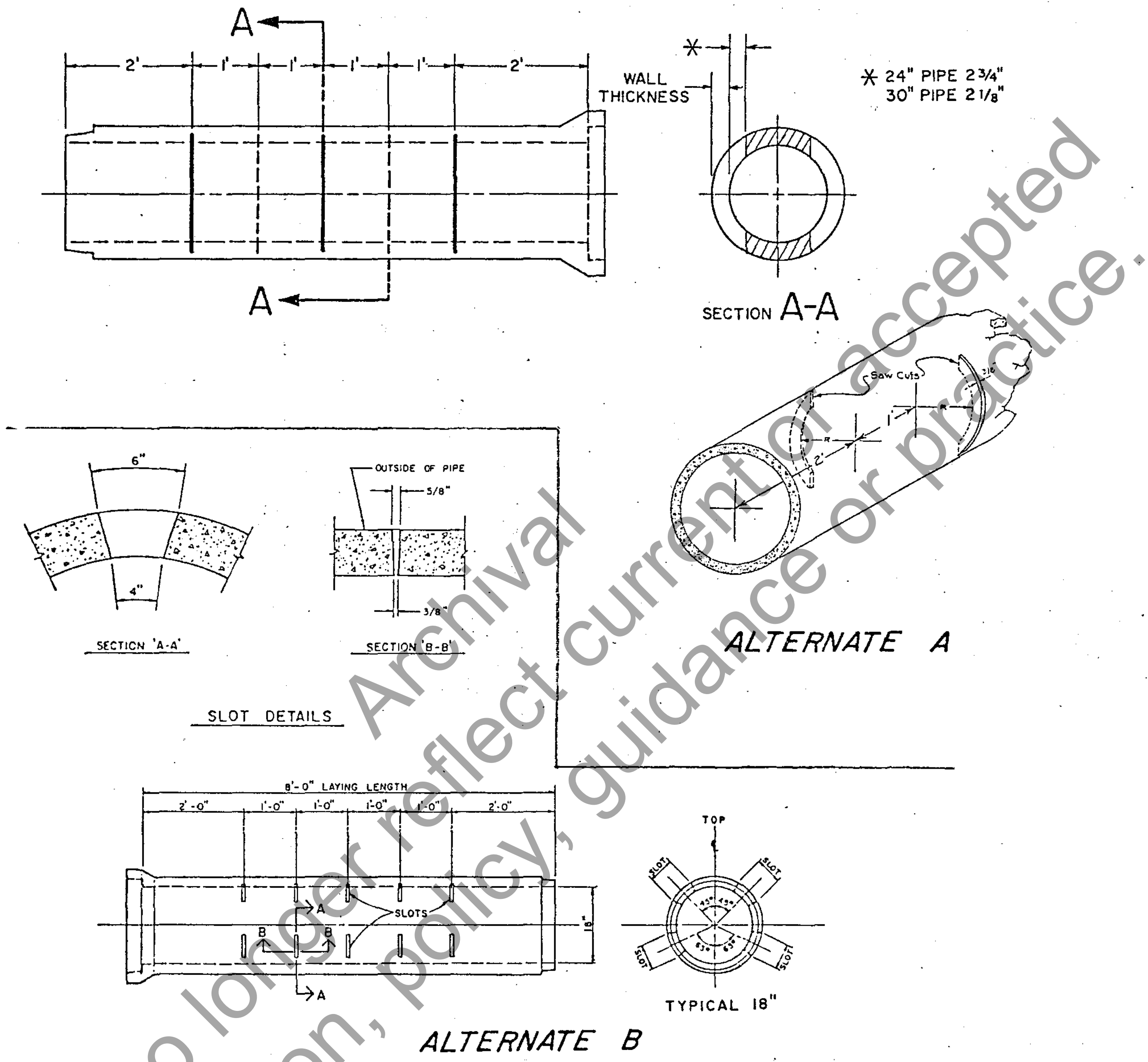


FIGURE II-9 DETAILS OF SLOTTED CONCRETE PIPE
(COURTESY OF FLORIDA DOT)

Determining the size of pipe and trench needed requires an estimate of the surface runoff and a storage volume sufficient to retain this amount of water until it can seep into the adjacent soil, or be released to a conventional storm sewer. The final quantity would be reduced by any detention-exfiltration into the soil that might occur during that interval. Infiltration systems can also be incorporated as part of a positive outfall or combination system to exfiltrate storm water as needed to recharge ground water at various locations along the alignment. Flow can be confined to the conventional storm drain system in areas of the alignment where recharge is restricted by local ordinance. The design of these and other types of subsurface storm water disposal/detention systems are discussed in detail in Chapter IV-C.

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III. GENERAL CONSIDERATIONS

Preliminary Information

The disposal of storm water by infiltration can provide a practical and attractive alternative to the more conventional and often costlier storm water conveyance systems. Recent legislative mandates lend impetus to consideration of this alternative. The imposition of requirements for zero discharge (zero increase of runoff) within urban areas coupled with regulations on land developments provides an increased emphasis on the infiltration alternative for disposition of storm water.

Infiltration systems provide the designer an additional degree of flexibility in the development of new facilities that avoid additional flow to existing storm drains, or streams and rivers. These systems afford a dual potential in that they are often less costly than conventional systems and they serve to replenish depleted groundwater supplies and increase groundwater levels, preventing undesirable intrusion into aquifers. However, the legal and environmental regulations and soil conditions should be investigated for a particular locality before designing a given system. Governmental agencies should be consulted concerning the amount of aquifer clearance required.

Important sources for information to be considered when determining the feasibility of a particular system are:

- environmental and legal constraints (Chapter III-A)
- groundwater data (Chapter III-A)
- local Soil Conservation Service (SCS) maps (Chapter III-B)

- aerial photos (Chapter III-B)
- soil boring logs (Chapter III-B)
- soil properties data (Chapter III-B)
- rainfall data (Chapter IV-B)

A good source of information is the U.S. Geological Survey-operated National Water Data Exchange (NAWDEX), a cooperative clearing house for water data, including groundwater quality information. NAWDEX assists users of water data to identify, locate, and acquire needed data. Refer to "Status of the National Water Data Exchange (NAWDEX) - September 1977" by M. D. Edwards, U.S. Geological Survey Open-File Report 78-154, 1978.

The following sub-chapters of this manual (III-A, B and C) provide guidelines for selection and evaluation of alternate storm water disposal systems.

A. ENVIRONMENTAL AND LEGAL CONSIDERATIONS

1. Introduction

This section discusses the various environmental and legal constraints that should be given consideration in planning and designing underground disposal systems for storm water runoff.

Studies sponsored by the U.S. Environmental Protection Agency, Federal Highway Administration, U.S. Geological Survey, and others, have identified constituents of paved roadways and parking facilities in runoff waters. Assessment of the impact of runoff-conveyed pollutants on receiving waters is continuing(1,2,3,4). Few studies are concentrated on the impact of pollutants in roadway runoff on the groundwater system. Perspective on the possible environmental aspects of subsurface disposal of storm water runoff can be gained from information available on the land treatment of municipal wastewater. Design guidelines for the use of these systems are defined in detail in the "Process Design Manual for Land Treatment of Municipal Wastewater", published jointly by the U.S. Environmental Protection Agency, U.S. Army Corps of Engineers, and U.S. Department of Agriculture(5). In the cover letter to that manual, Jorling and Graves make the following very meaningful statement.

> "Wastewater treatment is a problem that has plagued man ever since he discovered that discharging his wastes into surface waters can lead to many additional environmental problems. Today, a wide variety of treatment technologies are available for use in our efforts to restore and maintain the chemical, physical, and biological integrity of the nation's waters.

> "Land treatment systems involve the use of plants and the soil to remove previously unwanted contaminants from wastewaters. Land treatment is capable of achieving removal levels comparable to the best available advanced wastewater treatment technologies while achieving additional benefits. The recovery and beneficial reuse of wastewater and its nutrient resources through crop production, as well as wastewater treatment and reclamation, allow land treatment systems to accomplish far more than conventional treatment and discharge alternatives.
>
> "Land treatment processes should be preferentially considered as an alternative wastewater management technology. While it is recognized that acceptance is not universal, the utilization of land treatment systems has the potential for saving billions of dollars. This will benefit not only the nationwide water pollution control program, but will also provide an additional mechanism for the recovery and recycling of wastewater as a resource."

Land treatment of wastewater can provide an alternative to discharge of conventionally treated wastewater. However, careful consideration of any adverse impact of percolated wastewater on the quality of the groundwater is an essential prerequisite for all such projects. It has been demonstrated in numerous reported case histories(5) that a system of disposal which includes filtration through soil can be successful.

The response to a questionnaire circulated for the purpose of eliciting state-of-the-art information for this manual suggests reasons why these systems have not had widespread use. It was indicated that many agencies refrain from using infiltration or subsurface methods for the disposal of storm water to avoid possible adverse impact on groundwater. On the other hand, emphasis is being given in many areas to reduction or elimination of discharge of storm water into surface waters to avert possible pollution, particularly the initial half-inch (13 mm) of runoff, which comprises the "first flush" and carries the highest concentration of surface pollutants(6). This quantity of runoff, however, may vary depending upon development of new information and should not be specified arbitrarily since runoff in excess of one-half inch (13 mm) may be required to "flush off" surface pollutants. Subsurface disposal provides an alternative method of handling these storm water contaminants.

Like land treatment of wastewater, subsurface disposal of storm water is an attractive, cost-effective alternative to conventional discharge into surface waters. Consideration of the impact of subsurface disposal of infiltrated storm water on the quality of the groundwater is essential. The quality of groundwater should be determined and compared to established standards for its current or intended use and monitored for change in quality with time.

Proposed U.S. Environmental Protection Agency Proposed (EPA) requirements in the Federal Register, dated April 20, 1979(7), establish the technical criteria and standards to be used in implementing underground injection control programs within individual states. The proposed requirements prevent the use of systems that endanger underground drinking water sources. These regulations establish programs which prohibit any underground injection by either gravity or pressure injection not authorized by State permit. However, some general State rules are allowed without case-by-case permits. "Well injection", as defined under these proposed requirements, is "subsurface emplacement of fluids through a bored, drilled, or driven well; or through a dug well where the depth is greater than the largest surface dimension and a principal function of the well is the subsurface emplacement of fluids".

These systems are classified under the proposed requirements as Type V wells which includes storm water disposed wells, salt-water intrusion barrier wells, and subsidence control wells. Underground sources of drinking water as defined by EPA include, "All aquifers or their portions which are currently providing drinking water and, as a general rule, all aquifers or their portions with fewer than 10,000 parts per million of total dissolved solids [ppm or mg/l of TDS]".

Before any system is developed for infiltrating water or making any other change in natural runoff, designers should also make sure that the system will not create legal liabilities for the owners. Legal problems cannot always be averted, but developers should be aware of the water laws and codes of practice of their locality.

2. Environmental Considerations of Runoff Waters

The principal motivation for elimination of storm sewer discharge into surface waters stems from concern over the impact on public health and the aquatic ecosystem. As combined sanitary-storm sewer systems have been identified and direct discharges reduced, attention has focused on the quality of storm water.

Under Section 208 of Public Law 92-500 (Water Pollution Control Act Amendments of 1972) states are developing areawide water quality management plans to identify and mitigate both point and non-point sources of water pollution. Non-point sources of pollution include land development activities, construction, mining, logging, agricultural and silvicultural activities. The nature of the land surfaces over which storm waters flow, i.e., the use to which they are subjected, is widely recognized as one of the key factors of the quality of storm water(8).

Various approaches to the evaluation of storm water quality and its potential impacts are being considered in the development of the 208 plans. Valuable information should be gained by this effort and consideration of subsurface disposal of storm water will undoubtedly be addressed in the various study plans.

As the permit process for discharge of storm water to surface water becomes more stringent in response to Section 208 evaluations, the subsurface disposal of storm water will attract attention as a possible disposal alternative.

The general references for groundwater quality are drinking water standards since many near-surface or water table aquifers constitute the main source of public water supplies. For areas affected by salt-water intrusion or locations with naturally poor-quality groundwater, disposal of poor quality surficial storm water is not a serious concern. The EPA proposed drinking water standards are listed in Table III-A-1.

TABLE III-A-1 EPA-PROPOSED REGULATIONS ON INTERIM PRIMARY DRINKING WATER STANDARDS, 1975(9)

Constituent Characteristic	Value	Reason for Standard
Physical		
Turbidity, units	1[a]	Aesthetic
Chemical, mg/L		
Arsenic	0.05	Health
Barium	1.0	Health
Cadmium	0.01	Health
Chromium	0.05	Health
Fluoride	1.4-2.4[b]	Health
Lead	0.05	Health
Mercury	0.002	Health
Nitrates as N	10	Health
Selenium	0.01	Health
Silver	0.05	Cosmetic
Bacteriological		
Total coliform, per 100 mℓ	1	Disease
Pesticides, mg/L		
Endrin	0.0002	Health
Lindane	0.004	Health
Methoxychlor	0.1	Health
Toxaphene	0.005	Health
2,4-D	0.1	Health
2,4,5-TP	0.01	Health

The latest revision to the constituents and concentration should be used.

[a] Five mg/L of suspended solids may be substituted if it can be demonstrated that it does not interfere with disinfection.

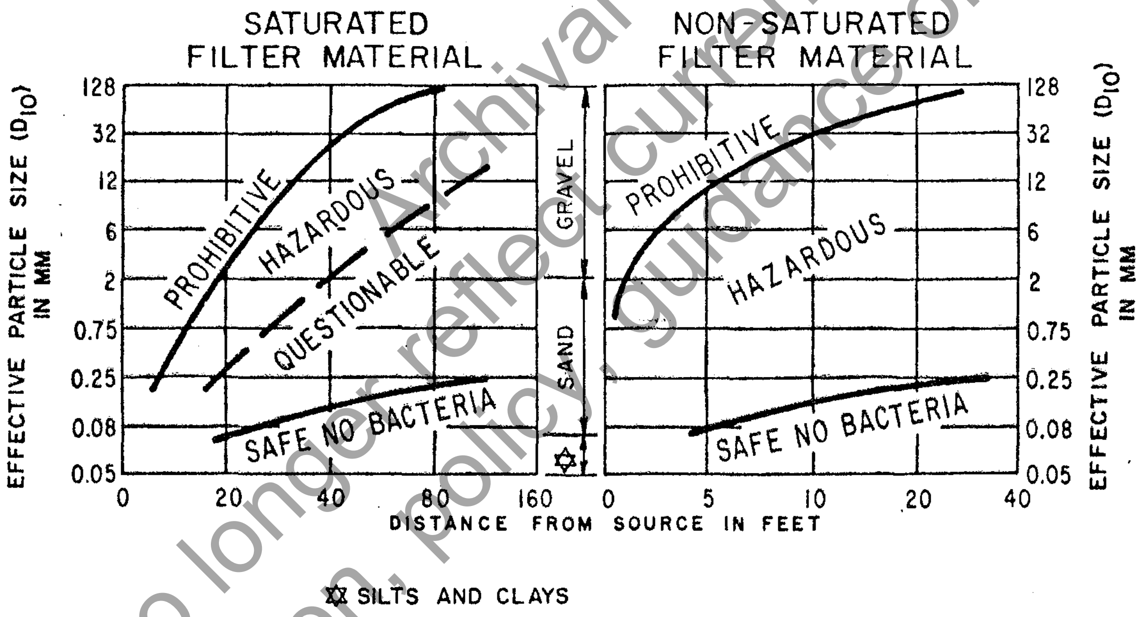
[b] Dependent on temperature; higher limits for lower temperatures.

If groundwater contaminants are substantially higher in the area of concern than any of the current listed standards for drinking water quality, future use as a public water supply is doubtful and the subsurface disposal permit process should be greatly simplified.

Most State Health Departments prohibit direct discharge of storm water runoff into underground aquifers. Recharge systems are not utilized in some states because these requirements place restrictions on storm water infiltration systems. Water pollution law in Ohio, for example, can charge offenders with polluting groundwater but those charges must be made and proven in a court of law(10).

Some northern states use large quantities of road de-icing salts during winter months. These states have tended to refrain from use of storm water recharge systems fearing possible contamination of groundwater. To prevent groundwater pollution, some agencies in California require a 10-foot (3.1 m) aquifer clearance for infiltration well construction(11). Infiltration wells are readily capable of polluting groundwater supplies and local regulatory agencies should be consulted concerning the amount of aquifer clearance required for a specific project.

Guidelines are not currently available for aquifer separation distance for infiltration of storm water. However, there are guidelines for sewage effluent from septic tank leach fields. The graphs in Figure III-A-1 suggest the purification mechanism of soil in terms of distance that effluent must move through various soils for complete removal of bacteria. These graphs indicate that bacteria removal is a function of particle size and groundwater location with reference to filter media. These graphs have been used for several years by the State of California for assessing the soil media below dry wells, septic tanks, and leach fields and are based on research conducted by Colorado State University(12,13,14). The graphs are provided in this manual as a guide for establishing separation distances between the bottom elevation of infiltration systems and groundwater level. However, the condition of the storm water entering an infiltration system will probably require less filter media thickness in most cases. Questionable installations should be monitored to identify changes in groundwater quality as discussed herein.



NOTE: THE EFFECTIVE GRAIN SIZE (D_{10}) CORRESPONDS TO THE GRAIN SIZE DIAMETER WHERE 10% OF THE PARTICLES ARE FINER AND 90% COARSER THAN THE EFFECTIVE SIZE BY WEIGHT.

FIGURE III-A-1 SIZES OF FILTER MATERIAL PARTICLES THAT ARE EFFECTIVE OR INEFFECTIVE IN TREATING SEPTIC TANK EFFLUENT IN A LEACH LINE SYSTEM [MODIFIED FROM (14)]

a. Groundwater Quality Processes

Chemical analyses of water commonly report constituent concentrations as "total". This designation implies that nitrogen for example, is a total of dissolved and particulate phases. The principle dissolved nitrogen species are ammonia, soluble organic nitrogen, nitrite, and nitrate. The particulate phase can be either adsorbed nitrogen, organic matter containing nitrogen, or insoluble mineralogic phases with nitrogen in the lattice.

The particulate phases of the various elements are also represented in the suspended sediments. The distinction is sometimes important as soils and interstitial areas of some aquifers can filter out particulate or suspended solids thereby reducing the impact of the various pollutants on the groundwater. This is particularly important in the case of bacteria.

The natural filtration of runoff water by the soil removes most harmful substances before they can reach the water-bearing aquifer. Nearly all pathogenic bacteria and many chemicals are filtered within 3 to 10 feet (0.9 to 3.1 m) during vertical percolation, and within 50 to 200 feet (15.3 to 61 m) of lateral water movement in some soil formations(15).

Tests made by the U.S. Department of Agriculture for the Fresno Metropolitan Flood Control District, indicated heavy metals such as lead, zinc, and copper present in the upper few centimeters of storm water infiltration basin floors. Generally after 10 to 15 years of storm water collection, this layer may require removal or other treatment where a build-up of concentrations of these elements has occurred. The particular locations tested by the U.S. Department of Agriculture had soils with a relatively high clay content(10). Layers of fine sands, silts, and other moderately permeable soils also very definitely improve the quality of storm water. This concept underlies the practice of disposing of domestic sewage in septic tanks with leach lines or pits and the land disposal techniques.

One of the major traffic-related contaminants is lead. Although lead is primarily emitted as particulate matter, it is fairly soluble. Lead in its ionic form, tends to precipitate in the soil as lead sulfate and remains relatively immobile due to low solubility(16). Lead can also be tied up by soil microorganisms, precipitate with other anions, ion exchange with clay minerals, or be absorbed by organic matter or uptake by plants. Once ionic lead reaches the groundwater table by precipitation, ion exchange, or adsorption the available lead can still be reduced. Surface and groundwater quality samples collected near a major highway interchange in Miami, Florida, revealed that lead concentrations were very low(17). The interaction of lead with the high bicarbonate in this particular location probably caused precipitation in the surface water borrow pond near the highway. Lead concentrations in the bottom sediments of these ponds were found to be relatively high.

If impure water is allowed to enter directly into coarse gravel or open joints in rocks, the impurities may enter into and contaminate adjacent groundwaters. Sites that are underlain with highly permeable strata or cracked and jointed rocks have the best capabilities for rapid disposal of surface waters. Unless adequate arrangements are made to treat contaminated water, or to filter impurities, infiltration systems may degrade the groundwater quality. Faults and intrusions, should always be evaluated for their effect on groundwater occurrence, influence on quality, and direction of movement. If the underlying rock strata is fractured or crevassed like limestone, storm water may be diverted directly to the groundwater, thereby receiving less treatment than percolation through soil layers.

Breeding and Dawson(18) describe a system of 127 recharge wells used by the City of Roanoke, Virginia, to dispose of storm runoff from newly developing industrial and residential areas. Several major faults exist in the underlying bedrock. These faults play a significant role in the effectiveness of the drainage wells, and also in the movement of groundwater. The authors also indicate that these direct conduits to groundwater have caused quality degradation in one area; however, "groundwater users in adjacent Roanoke County have not experienced quality problems that could be connected to this means of storm water disposal."

The case cited illustrates the possibility of groundwater contamination in areas where fractured and highly permeable rock layers exist, providing conduits for widespread movement of contaminants. It is, therefore, important in the planning stages of a large subsurface storm water disposal project to identify the underlying soil strata in terms of its hydraulic, physical, and chemical characteristics. Pertinent physical characteristics include: texture, structure, and soil depth. Important hydraulic characteristics are: infiltration rate and permeability. Chemical characteristics that may be important include pH, cation-exchange capacity, organic content, and the absorption and filtration capabilities for various inorganic ions. If detailed groundwater quality analyses are available, it is possible to compute the solution-mineral equilibrium(19). This approach does not guarantee that an anticipated chemical reaction will occur but does indicate how many ionic species should behave.

The items referring to physical and hydraulic characteristics are addressed to some extent in other chapters of this manual. Further discussion of the chemical characteristics of soils is beyond the scope of this manual. Definitive information on this subject can be obtained by consulting appropriate references, i.e., Grim(20), or other references on the subject. The importance of proper identification of the hydraulic characteristics of the rock strata has been illustrated above.

b. Groundwater Monitoring

Environmental laws and regulations now in force require monitoring groundwater where adverse effects to its quality may result from disposal and storage of solid and liquid wastes(21). Monitoring systems have not as yet been required for groundwater recharge utilizing storm water. However, consideration of such monitoring systems should be incorporated in the design of subsurface drainage systems that discharge storm water directly into groundwater.

Proposed EPA requirements for Type V wells (gravity or injection), which discharge directly into surficial aquifers, call for immediate action with respect to injection that poses a significant risk to human health. An assessment is required of the contamination potential, available corrective alternatives, and their environmental and economical consequences(7).

When properly installed, a groundwater monitoring system should provide sufficient data for determining the extent of contamination buildup with time, as well as concentration and distribution of the contaminants.

Geologic analysis of the area can provide vital information for developing the monitoring system. Factors to be considered include: depth and type of subsurface soils, depth to bedrock, relative permeabilities, depth to groundwater, and relative groundwater gradients. Proper layout of monitoring wells cannot be accomplished until information relative to such factors has been obtained and evaluated. Wells must be sufficiently close to the potential source of contamination to detect any degradation of groundwater quality at an early stage. Where monitoring wells are used as an early warning system, it is imperative that the preproject quality of on-site groundwaters be established, and, thereafter employed as a standard for comparison with groundwater samples taken subsequent to initiation of the proposed subsurface drainage system. Sufficient samples of groundwater should be obtained over a time period adequate to establish the "ambient" groundwater conditions prior to storm water disposal. The number and location of monitoring wells will be governed by the magnitude of the project and careful consideration of information developed by the aforementioned site geology analysis.

An appropriate monitoring well should be so designed as to provide the quantity and quality of sample required at the lowest cost. Small diameter (1 1/2 inch [38 mm]) PVC riser pipe, with either plastic well screens or slotted plastic pipe, will usually prove adequate in developing a sampling well. Slotted pipe is the least expensive and most convenient material for developing a suitable well screen(21). Materials used in the construction of the sampling well should be chosen so that they do not influence the characteristics of the sample.

To prevent the migration of fines into the sampling well, all well screens or slotted sections should be installed with a backfill of clean filter sand. Precautions should be taken to prevent the migration of fines into the wells. The top portion of the well pipe should be backfilled with concrete or cement grout to provide a seal which prevents contamination by surface waters. The well seal should comply with State and local requirements.

A shallow well groundwater quality monitoring system has been developed in southern Florida which will be installed routinely as a contract item on infiltration trench projects in Dade County. Details of this system are similar to the cross-section and plan shown on Figures III-A-2 and III-A-3.

3. Legal Considerations

a. Introduction

Before any system is developed for infiltering water or making any other change in natural runoff, designers should make sure that the system will not create legal liabilities for the owners. Major construction projects can change the natural runoff patterns, reducing flows in some areas and increasing it in others. Areas that had no known record of flooding before the construction of a major work may subsequently develop drainage problems. Often the increased discharges can be attributed to "improvement" of the natural

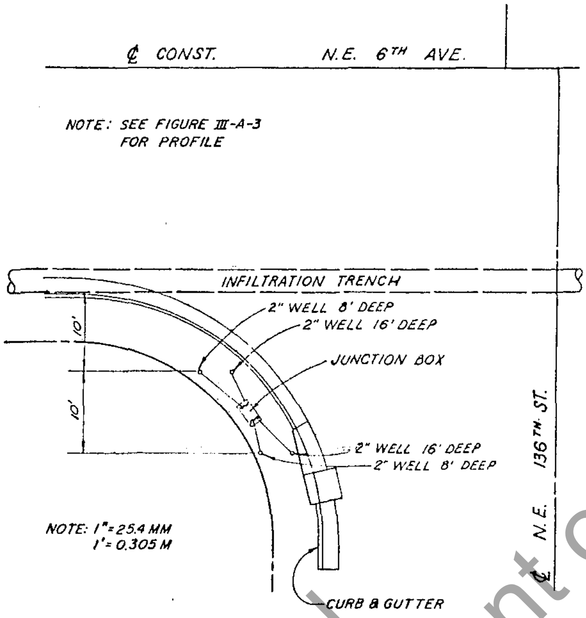


FIGURE III-A-2 PLAN VIEW OF GROUNDWATER MONITORING SYSTEM FOR INFILTRATION TRENCH CONSTRUCTION (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA)

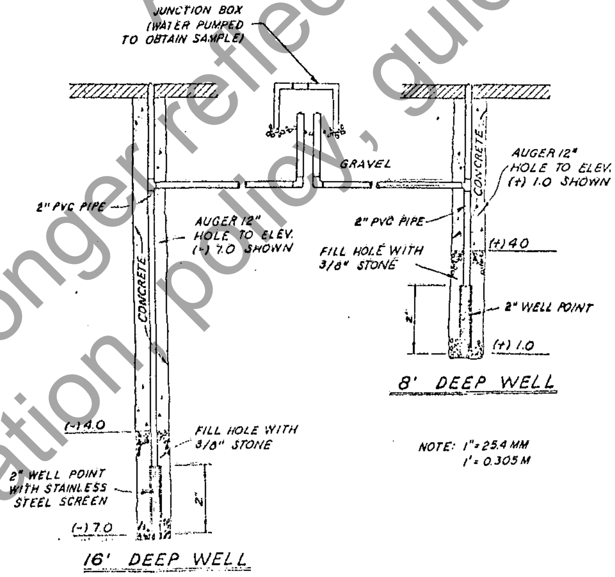


FIGURE III-A-3 TYPICAL GROUNDWATER MONITORING WELLS FOR INFILTRATION TRENCH CONSTRUCTION (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA) delivery system, rather than diversions. In other areas, farmers or others who have depended on natural flows in streams for their livestock or crop production, see the available supplies sharply reduced. In semi-arid areas, the construction of detention ponds, seepage pits or wells, catch basins, reservoirs, etc., for "water harvesting" has reduced the flows to downstream landowners. Such changes can lead to litigation. Legal problems cannot all be averted. Developers of systems should contact appropriate local or state agencies regarding compliance with laws or local codes of practice.

Drainage of surplus storm water from changing land use and development may cause increased erosion with resultant pollution in natural waterways. Relatively new political constraints have been imposed because of this and burgeoning public sensitivity to further environmental degradation. Levels for various constituent concentrations in discharge or receiving waters may be specified in permits to maintain water quality objectives. Legislation specifying zero discharge and zero increase in discharge has been enacted in some cases without provision for exceptions, despite their merits, environmental or otherwise.

Zero increase in discharge may be a difficult legal concept. It attempts to recognize the need for runoff and provide for engineering flexibility. However, legal problems will arise from interpretation of runoff coefficients. A coefficient by definition is a ratio or, as commonly expressed, a percentage figure. Even in a natural watershed, with excellent rainfall and runoff (discharge) records, the runoff coefficient has been shown to vary with the rainfall frequency, rainfall intensity or rate, period of antecedent dry conditions (soil moisture content), and the seasonally dependent vegetation. When comparing areas, the infiltration rate (percolation) of the soils, the size of the area, the degree of imperviousness (roads and roofs, etc.), the slope, and vegetation type, become important. It would be difficult to anticipate a runoff coefficient with a high degree of confidence for an area that is to be altered with respect to these variables.

b. Water Rights

When subsurface drainage systems are to be employed, consideration must be given to their effect on water rights downstream, or senior claims to the water, as the source of flow will be diminished when the runoff is diverted from its normal or historic drainage channel(22). If the concept of "zero" increase in runoff is pursued, no interference in downstream rights would be anticipated.

The Process Design Manual for Land Treatment of Municipal Wastewater(5) points out that water rights problems tend to arise in either water-deficient areas or those areas fully allocated.

Most riparian (land ownership) rights are in effect east of the Mississippi River, while most appropriation (permit system) rights are in effect west of the Mississippi River.

Legal distinctions are made between discharges to a receiving water in a well-defined channel or basin (natural watercourse), superficial waters not in a channel or basin (surface waters), and underground waters not in a well-defined channel or basin (percolating or groundwaters)(23).

Transportation-related aspects of water rights are discussed in "AASHTO Guidelines for the Legal Aspect of Highway Drainage"(24).

Possible water rights problems related to complex drainage systems may require consultation with water masters or water rights engineers at the State or local level. An excellent reference is the National Water Commission publication, "A Summary-Digest of State Water Laws"(25). Similar case histories can be found in references(22,23, 24,25). The assistance of an attorney versed in water law is often helpful.

4. Summary and Conclusions

a. Since the character and concentration of pollutants generated from paved surfaces vary considerably depending upon the type of development, location, population, and dilution by storm water runoff, no attempt is made in this manual to define these constituents and evaluate their effects on the environment. Various studies are underway at the present time which address this problem.

b. Land treatment of storm water by infiltration through soil is capable of removing pollutants at levels comparable to the best available advanced wastewater treatment technologies. This capability will vary with the hydraulic, physical, and chemical characteristics of the receiving soil strata and the character and concentration of the pollutants carried by the storm water.

c. A monitoring program may be required to determine the quality of groundwater and compare it to established standards for current or intended use, and to evaluate any potential for degradation with time. It is, therefore, advisable to consult state and local regulatory agencies in regard to environmental and legal questions relative to subsurface disposal systems for storm water.

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B. SOILS EXPLORATION

1. Considerations for Determining Subsurface Soil and Groundwater Conditions

A key element in any design analysis of soil infiltration capacity undertaken for a subsurface storm water disposal system is a comprehensive soils investigation program, supervised by a Soils Engineer qualified to plan and implement the program and interpret the results. Valuable professional assistance or guidance may be available from governmental agencies such as the Soil Conservation Service, U.S. Department of Agriculture. A hydrogeologist knowledgeable with the local geohydrology could also provide valuable information. The details of subsurface exploration programs related to this subject are beyond the scope of this manual. However, any soils exploration program should be oriented to the following objectives:

a.) Define the subsurface profile within the infiltration basin or well area; or along the length of the proposed system.

1.) Identify soil and rock strata, 2.) locate the otatic water table, and 3.) anticipate its seasonal fluctuations.

b.) Provide representative samples from the explorations for laboratory testing purposes.

c.) Provide for field permeability tests to be performed at the site as necessary. For suggested methods refer to Chapter IV-A of this manual.

d.) Review data on historic and existing groundwater conditions to provide information on possible mounding effects of the proposed system.

2. Preliminary Activities

This phase of an investigation can be categorized as a reconnaissance study since a great deal of subsurface information is frequently available from various sources. Available data can often be acquired at little or no cost. Its acquisition can provide insight into existing conditions and aid in determining the extent of the subsurface explorations program needed for final design.

a.) Possible Sources of Existing Subsurface Data

1.) Soil surveys prepared by the Soil Conservation Service, U.S. Department of Agriculture, are available for all states, as well as Puerto Rico and the Virgin Islands. The soil surveys published subsequent to 1957 contain interpretations of the mapped soil deposits useful for engineering purposes, including soil suitability for drainage and irrigation. Soil surveys prior to 1958 require more engineering interpretation. Copies of these surveys can also be inspected in Soil Conservation District or County Agricultural Extension Offices.

2.) Geologic reports and groundwater resource reports prepared by the U.S. Geological Survey in Cooperation with state agencies are frequently available. These reference sources can be quite informative.

3.) Subsurface data obtained previously in the area for other projects should not be overlooked. Such data may have been obtained in connection with utility company projects; or private ventures such as commercial developments; or earlier public agency projects.

4.) When available, aerial photographs can also be of value when properly interpreted by trained personnel in defining soil type categories and qualitative soil-moisture conditions.

b.) Preliminary Site Inspection

A great deal can be learned about sites in non-developed areas through examination of the terrain and its surface features. Types of vegetation and lake levels may give some preliminary indication of groundwater levels. The natural terrain is indicative of land forms, which in turn imply the types of soil categories that exist. Soil maps and bulletins of the U.S. Department of Agriculture's Soil Conservation Service are most helpful in the interpretation of information derived from such on-site examinations. Commercial or residential construction records can also provide information on soil and groundwater conditions. Inspection of existing wells may yield general groundwater data.

3. General Guidelines for Explorations Programs

The subsurface explorations and field testing program should be established after a review of the data obtained in the reconnaissance phase. The storm water disposal system might be only part of a larger project that has its own explorations requirements. Explorations should be made to serve dual purposes whenever possible. Those required specifically for storm water disposal can be planned after due consideration and evaluation of existing data and, where applicable, considering explorations requirements for other project design features.

A preliminary program of limited scope can help establish groundwater and soil types and aid in verifying the information obtained in the reconnaissance stage, or serve, itself, as the reconnaissance stage where other data is not available. A preliminary program is advisable whenever possible since it might well indicate at an early stage whether a subsurface storm water disposal system is feasible or not. In addition, the magnitude of explorations for final design would be more apparent following a preliminary program.

Explorations can be implemented in various ways, such as machine-cased borings, test pits, trenches, or auger holes. Penetration resistance is not considered an applicable exploration method.

Although an in-depth discussion of types of exploration and methods is not within the scope of this manual, some comments on selecting exploration types and procedures are appropriate. Test pits or trenches are often the best methods since they expose soil and water conditions. Pits or trenches may also be the most economical method depending on the equipment and manpower available to the designer. Where cased borings are used, they should be made to the maximum depth possible without the use of water to facilitate determination of natural groundwater depths. In addition, cased borings, or auger holes, should provide continuous samples to some depth below the final bottom elevation of the proposed infiltration trench, well, or basin. This is necessary in order to establish a continuous definition of soil types through which the storm water will percolate and to aid in determining groundwater depth through differences in soil moisture. The recommended minimum depth of exploration is 10 feet (3.1 m) below the bottom of the seepage discharge level, or to the static water table, whichever occurs first.

An example of a typical subsurface exploration program for basin design and for trench design are shown on Figures III-B-1 and III-B-2, respectively. The finished grade for the basin example in Figure III-B-1 and for the trench example in Figure III-B-2 are less than 4 ft (1.22 m) above the static water table in sand and gravel materials. An adjustment of these grades may be desirable to satisfy local environmental considerations. Mounding conditions above the water table should be anticipated in both cases with some reduction in infiltration capacity.

Long term readings are essential to evaluate seasonal groundwater fluctuations, particularly where the groundwater level may be within 10 feet (3.1 m) of the seepage discharge level. This information can be obtained by inserting perforated or slotted tubing or pipe in the boring and taking periodic water level readings.

An area with a high groundwater table and/or soils having a high percentage of silt and clay size material will not normally accommodate subsurface storm water disposal. In such areas a storage-retention type of system should be considered as an alternative. There are exceptions to this, since, in some areas, a high water table with pervious soil conditions may not be detrimental to the use of subsurface disposal systems.

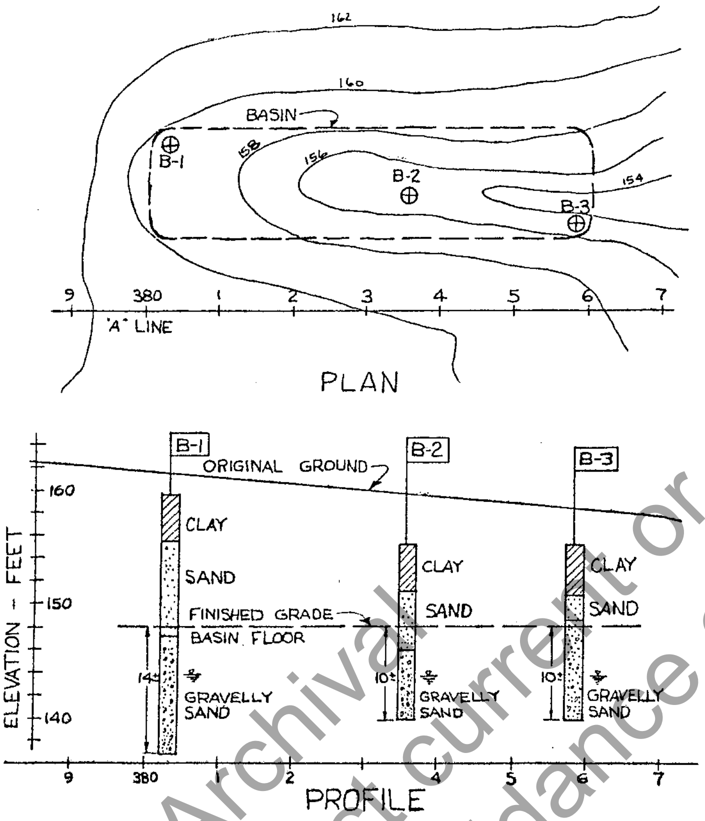

FIGURE III-B-1 TYPICAL EXPLORATION PROGRAM FOR INFILTRATION BASIN DESIGN

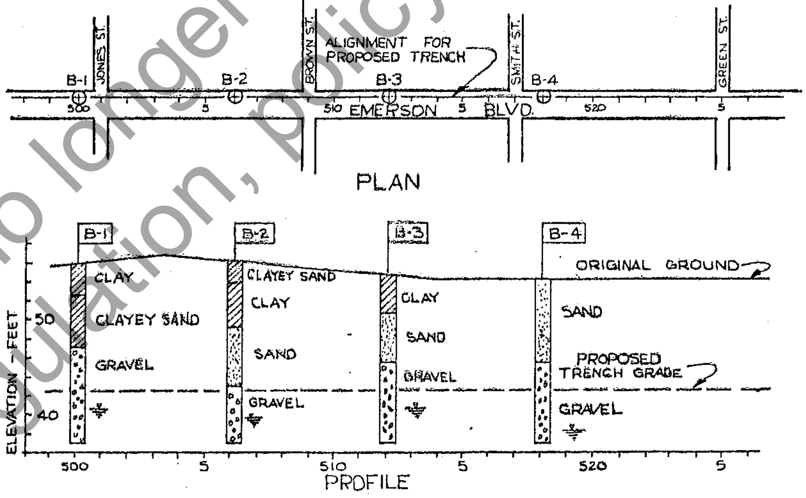

FIGURE III-B-2 TYPICAL EXPLORATION PROGRAM FOR INFILTRATION TRENCH DESIGN

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C. EVALUATION OF ALTERNATIVE DISPOSAL SYSTEMS

1. Alternatives to Positive Discharge

a. Basins

When ample space is available and other criteria are satisfied, the use of infiltration basins can provide a relatively inexpensive solution to storm water disposal in terms of cost per unit volume of water drained. As mentioned earlier, space is often available within areas of the right-of-way, such as highway interchanges; or on non-used portions of residential or commercial developments.

In some communities infiltration basins have been integrated with attractive parks and/or recreation areas. This dual role of the basin benefits both the facility and the public.

Among the negative aspects of infiltration basins are their susceptability to early clogging and sedimentation, and the considerable surface land areas required for their construction. Basins also present a security problem due to exposed standing water, and a potential for insect breeding. These problems are discussed at length in Chapter VI of this manual.

b. Wells and Pits

Wells and pits are often used to handle drainage problems in small areas where an outfall is not available. They are also used in conjunction with infiltration basins to penetrate impermeable strata overlaying pervious soil layers. Infiltration wells and pits can be installed quickly and inexpensively to remove standing water in areas difficult to drain. A disadvantage is the tendency of filter media to clog with silt or sediment, requiring considerable maintenance. Also, their capacity for drainage is difficult to predict. One well may induce a good rate of infiltration; while another, a very short distance away, will drain very poorly.

c. Trenches

Infiltration trenches are a viable solution for long-term underground storm water disposal at locations having soils or rocks capable of absorbing large quantities of water. Trenches are ideally suited to urban development; e.g., under lot lines, within easements, under road right-of-way, beneath parking lots and in landscaped areas.

Slab-covered trenches and trenches with perforated or slotted pipe backfilled with coarse aggregate provide economical alternatives to surface disposal. The slab-covered trench is feasible where rock strata will support the slab and trench walls and still provide necessary infiltration. Such conditions are found in certain areas of Florida although this particular design may have limited application elsewhere.

Perforated or slotted pipe backfilled with coarse rock, installed in trenches, can provide a long-term solution to underground storm water disposal. The capacity of this system is controlled by the native soil permeability characteristics. The pipe provides storage and also serves as a continuous catchment for silt. Clogging of perforations or slots and coarse aggregate is thus minimized. Catch basins which are points of entry of storm water also provide silt catchment and easy access for cleanout.

d. Combination Systems

These systems can incorporate retention storage with subsequent infiltration and discharge of residual flow through a positive outfall system. Individual systems can be designed to infiltrate storm water along the entire alignment of the drainage system or to infiltrate water only in selected areas.

e. Economic Considerations

Excavation materials from infiltration basin areas or trenches can provide a savings by their utilization in the construction of embankments. Considerable savings are also possible by reducing or eliminating costly outfall facilities. Local drainage problems can also be solved in some areas by installing sumps and drilling dry wells to take advantage of the infiltration characteristics of the soil and reduce storm drain requirements. In order to evaluate the economic feasibility of a given design, in addition to initial cost, the long term maintenance requirements are an essential consideration.

2. Site Evaluation and Selection of Alternative Infiltration Systems

The following is a check list of the steps which should be included in a feasibility evaluation:

a. Potential Benefits

1.) Economic benefits compared to direct discharge (positive system).

    - a.) Reduced outflow requirements.
    - b.) Reduced need for treatment of storm water.
  - 2.) Groundwater Recharge
  - 3.) Reduced or zero increase in discharge
  - 4.) Reduced subsidence due to groundwater withdrawal
  - 5.) Reduction or prevention of salt-water intrusion
- b. Evaluate alternate systems based on constraints.
  - 1.) Environmental
    - a.) Local impacts
    - b.) Groundwater quality
  - 2.) Legal
  - 3.) Physical site
- c. Evaluate site characteristics
  - 1.) Soil (surface and subsurface)
    - a.) Type and depth of soil
    - b.) Infiltration characteristics
    - c.) Location of groundwater table
  - 2.) Hydrologic
- d. Select most feasible system based on:
  - 1.) Economic evaluation
  - 2.) Construction evaluation
  - 3.) Maintenance evaluation
  - 4.) Potential benefits (2.a. above)
  - 5.) Constraints (2.b. above)
  - 6.) Site characteristics (2.c. above)
- e. Design system

IV. DESIGN

A. DETERMINATION OF INFILTRATION RATE

1. Factors Affecting Infiltration Rate

The capabilities of sites to accept surface water and distribute it into groundwater systems depend on a great many factors. Among the most important are: natural ground slope, type and properties of surface and subsurface soils, geologic conditions, and subsurface hydrologic conditions. The amount of water to be distributed and the kinds and amounts of contaminants and dissolved matter in the water have a profound influence on the capacities of systems to accept and distribute water on a long-term basis. Dissolved salts and other chemical substances, oil, grease, silt, clay, and other suspended matter can clog the surfaces through which water must enter a system. Such materials will greatly reduce infiltration rates if they are not intercepted by catchment basins or frequently removed by appropriate maintenance methods. The depth of the water table, and its natural slope, as well as the unsaturated and saturated horizontal and vertical permeabilities of soil formations, have important influences on rates of inflow and the rate of buildup of saturation mounds under infiltration systems.

For simplicity, the words: permeability, infiltration, and percolation, are used interchangeably in this manual in describing the ability of soil to absorb water. Specific definitions are included in the Glossary Section of this manual.

Investigations for the design of infiltration systems should concentrate on the following vital aspects of infiltration and dissipation of water: (1) the infiltration capabilities of the soil surfaces through which water must enter the soil, (2) the water-conducting capabilities of the subsoils that allow water to reach underlying water table, (3) the capabilities of the subsoils and underlying soils and geologic formations to move water away from the site, and (4) flow from the system under mounding conditions at maximum infiltration rates.

The rate of infiltration is greatly affected by the permeability of the soil formations. The infiltration rate for the first application of water in an infiltration test is generally greater than after longer application of water. As water application continues and the uppermost sediments become saturated, the infiltration rate gradually decreases and reaches a nearly constant rate, usually within a few hours. If all of the sediments are uniform or the deeper sediments are more permeable than those near the surface, and the water table is at considerable depth, the infiltration rate is controlled by the sediments near the surface. However, when the deeper formations are less permeable than the shallower ones, the shallow sediments soon become saturated and the resultant infiltration is controlled by the less permeable sediments at greater depth and groundwater gradients under these mounding conditions.

The principles of infiltration have been studied by many investigators, some of whom are referenced at the end of this chapter. One of the most complete studies of the waterflow patterns below infiltrometers is that of Aronovici in 1955(1), who illustrated the significance of surface and subsurface conditions on observed infiltration rates. His study suggested also that pressure head is the dominant factor involved in filtration rates in initially dry or damp soils, and emphasized the influence of the differential hydraulic head in causing a decrease in infiltration rate with time.

Compaction of the exposed surface of a test area reduces the infiltration rate. Wisler and Brater in 1949(2) pointed out that rain beats down on an unprotected soil, compacts it, washes fine debris into the pores, and thereby reduces the permeability.

Musgrave and Free in 1937(3) found that even slight water turbidity caused a considerable decrease in infiltration rate. According to the U.S. Salinity Laboratory in 1954(4), water having the same quality as that to be used later in actual infiltration should be used for the infiltration test.

Weaver(5) indicates that "Infiltration into pervious unsaturated or dry materials is predominantly controlled by capillary suction similar to the process of capillary rise except that gravity assists rather than impedes downward flow." He adds: "While it was long assumed that Darcy's law was valid to deal with these problems of unsaturated flow, this was not proved until 1950, by Childs and Collis-George(6). The difference from the usual applications of Darcy's law is that neither the conductivity term nor the driving potential term are constants; both are functions of water content." He notes that infiltration basin efficiency being directly proportional to the operating head, there is a definite advantage in designing for operation at relatively high heads. For an infiltration analysis, values of four soil properties must be obtained with depth in the profile. Weaver lists these properties as: (1) capillary suction, (2) transmission zone water content, (3) saturated permeability, and (4) soil porosity.

In describing New York's method of analysis of infiltration from basins Weaver states that it is necessary to "Plot and summarize all subsurface and laboratory test data in the same general fashion as for all other types of foundation design problems. These data must then be studied in terms of significance with respect to infiltration theory, to deduce the value of the soil properties controlling the infiltration rate at the site. As a general rule, the control zone for the infiltration rate will be in the first 10 feet (3.1 m) of the uppermost soil layer. The soil properties outside this area may exert a secondary control only when the soil is markedly less permeable and the profile is such that lateral spread of the wet front is prevented if its vertical advance is impeded by this layer. In other words, the surface control zone -- where the primary transmission zone is established -- will control infiltration under any condition where the water transmitted through it has someplace to go, either vertically or laterally."

No soil layer that has a hydraulic conductivity less than the soil within the lower limits of the infiltration facility should be overlooked as a possible zone which would result in groundwater mounding. Mounding over these zones could drastically reduce the infiltration rate of a proposed facility under perched or high groundwater conditions.

Since many factors affect infiltration rates, considerable judgment and experience are needed for selection of the proper test procedure to obtain reliable results from which to design an infiltration system. To interpret infiltration data properly the investigator must know the hydrology of the deep as well as the shallow formations. Adequate subsurface explorations as discussed in Chapter III-B of this manual should always accompany infiltration tests.

Some typical infiltration (permeability) rates for the various soil groups of the unified soil classification system are given in Table IV-A-1, for saturated and compacted laboratory specimens. Since laboratory test specimens are mixtures of *disturbed* materials, the tests may give permeabilities lower or higher than those of the in-place materials. If the in-place materials are dense, uniform deposits, and the laboratory specimens are less dense, the laboratory permeabilities could be *too high*. But if the natural deposits are stratified (sorted) formations, the laboratory permeabilities can be *too low*. The wide ranges in permeability values in Table IV-A-1 (even for relatively similar materials) emphasizes the need for good subsurface explorations and field permeability and infiltration tests.

2. Methods for Determining Soil Permeability

a. General Discussion

Those working with infiltration systems often make use of permeability tests, which are intended to measure a soil's ability to infiltrate water. These tests should simulate as closely as possible, the conditions that will develop in an infiltration system, and presume that each square foot of basin, trench, etc., will infiltrate the rate determined by the test. The value of these tests, therefore, depends on the degree to which they simulate the real conditions.

TABLE IV-A-1

PERMEABILITY RATES FOR DIFFERENT SOIL GROUPS FOR SATURATED AND COMPACTED LABORATORY SPECIMENS(7)

MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES OF SOIL GROUPS	Unit Dry Weight lb per cu ft		Permeability (K)* and Percolation Characteristics when Compacted and Saturated	
				Std. AASHTO	Mod. AASHTO	cm per sec	ft per day
COARSE-GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels Gravel-sand mixtures little or no fines.	125-135	125-140	10^{-1} to 10^{-4} Pervious	300 to 0.3
		GP	Poorly graded gravels or gravel-sand mixtures little or no fines.	110-125	110-140	10 to 10^{-2} Very pervious	3 x 10^4 to 30
		GM	Silty gravels, gravel-sand-silt mixtures.	115-135	115-145	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to 3 x 10^{-3}
		GC	Clayey gravels, gravel-sand-clay mixtures.	115-130	120-145	10^{-6} to 10^{-8} Impervious	3 x 10^{-3} to 3 x 10^{-5}
	SANDS AND SANDY SOILS	SW	Well-graded sands gravelly sands, little or no fines.	105-120	110-130	10^{-2} to 10^{-4} Pervious	30 to 0.3
		SP	Poorly graded sands or gravelly sands, little or no fines	100-120	105-135	10^{-1} to 10^{-3} Pervious	300 to 3
		SM	Silty sand, sand-silt mixtures	100-125	100-135	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to 3 x 10^{-3}
		SC	Clayey sands, sand-clay mixtures.	105-125	110-135	10^{-6} to 10^{-8} Impervious	3 x 10^{-3} to 3 x 10^{-5}
FINE-GRAINED SOILS	SILTS AND CLAYS LL IS LESS THAN 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	85-115	90-125	10^{-3} to 10^{-6} Semi-pervious to impervious	3 to 3 x 10^{-3}
		CL	Inorganic clays of low to medium plasticity gravelly clays, sandy clays, silty clays, lean clays	90-120	90-130	10^{-6} to 10^{-8} Impervious	3 x 10^{-3} to 3 x 10^{-5}
		OL	Organic silts and organic silty clays of low plasticity.	80-100	90-105	10^{-4} to 10^{-6} Semi-pervious to impervious	0.3 to 3 x 10^{-3}
	SILTS AND CLAYS LL IS GREATER THAN 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	70-95	80-105	10^{-5} to 10^{-7} Semi-pervious to impervious	0.03 to 3 x 10^{-4}
		CH	Inorganic clays of high plasticity, fat clays.	75-105	85-115	10^{-6} to 10^{-9} Impervious	3 x 10^{-3} to 3 x 10^{-6}
		OH	Organic clays of medium to high plasticity, organic silts.	65-100	75-110	10^{-6} to 10^{-8} Impervious	3 x 10^{-3} to 3 x 10^{-5}

*Permeability values as modified by H. R. Cedergren

Soil permeability or infiltration rate is best determined by actual field tests under known hydraulic gradients and known seepage areas. The value of laboratory tests is limited to the degree to which the specimens tested actually represent the soil mass in the field. One of the more important factors influencing the permeability of a soil of a given grain size distribution is the porosity and structural arrangement of the grain particles. In laboratory test specimens both properties are likely to be disturbed during sampling or during test preparation. Also, soil formations are stratified to a greater or lesser degree, and are variable within a formation; hence it is best to determine permeabilities in the field since a large zone of influence can be tested with less error.

Because of the possibilities of error introduced by laboratory permeability testing, as noted above, it is suggested that such tests be used only as a guide for preliminary evaluation of proposed infiltration drainage sites.

Field methods should be used to simulate conditions that most nearly predict the drainage capability of the proposed drainage system. This can be accomplished by auger holes, (cased or uncased), and sample trenches or pits; or other field procedures. The method chosen will depend on the type of facility to be designed and on the site location parameters; i.e., presence of underground utilities, number of test sites required, requirements for maintenance of vehicular and/or pedestrian traffic, type of equipment available to perform the test excavation, and type of subsoil.

The size of the test excavation should be large enough to aid in visual inspection when possible and to provide sufficient surface area to distribute the water, either laterally or vertically, depending on the type of test performed. Normally 12-inch to 24-inch (0.3 to 0.6 m) width or diameter is sufficient to accomplish this with testing equipment available. Longer test areas or excavations may be required for basin or pit testing.

The number of test sites is somewhat dependent on existing soil conditions and the drainage system layout.

For a basin, or a subsurface system for a paved parking lot area 300 ft x 300 ft (92 m x 92 m), two or three tests would normally be sufficient. On a continuous linear trench system of 1/2 mile (800 m) or more, 500 foot (150 m) intervals between test locations is sufficient, provided soil is uniform in composition.

Tests should be performed at each distinct change in soil strata and should continue downward to the approximate bottom elevation of the drainage facility being designed. If test results indicate low infiltration rates, excavation and testing should be continued to a depth that would provide satisfactory infiltration and yet still be economical for construction of the drainage facilities and within compliance of local and state regulations as defined in Chapter III-A, "Environmental and Legal Considerations".

An adequate supply of water should be available to both presoak the sides of test excavation or auger hole and perform testing. This can be supplied by either truck, hose, or fire hydrant. Excavation equipment may be either auger, backhoe, or trenching machine. A timing device with a second hand is needed for performing the test. Backfill material should be available to cover the excavation when testing is completed.

Test data should be recorded in a form that can be easily analyzed in the field to determine if the results are satisfactory to accommodate the design drainage facility.

b. Indirect Methods

1.) SCS Soil Classification Maps

These are maps that give the SCS classification of surface soils in many parts of the United States. They are published in National Cooperative Soil Survey Reports published by the Soil Conservation Service, U.S. Department of Agriculture, in cooperation with other agencies. Soil survey information is available on a county by county basis. A portion of a typical map is shown in Appendix D-1. These maps cannot possibly cover variations occurring in short distances; they give only a general idea of the basic types of soils occurring in various areas. Any use of these maps to catalogue soil type for estimating permeability should be verified by actual field inspection and classification of soils in the study area. Such maps can indicate in a general way whether soils might be expected to have good drainage, moderate drainage, or very poor drainage. Therefore, they may be utilized to some extent in preliminary infiltration drainage feasibility studies. Before any system is designed, more specific information based on field permeability testing should be obtained for a given site.

2.) Specific Surface Method of New York State(8)

This method (Appendix D-2) is used only for cohesionless granular material that is uniform and non-stratified. The saturated coefficient of permeability is calculated with a formula developed empirically which relates porosity, specific surface of solids, and permeability. Its principal advantage is simplicity. It requires only a small number of samples of material to obtain a standard gradation, the shape characteristics of the grains contained in each sieve size interval, and calculation of the specific surface based on the data obtained from the grain size analysis and physical examination and an estimated in-place porosity. As with other indirect methods, it does not allow for variations in soil structure or stratification which often control permeability. Field permeability tests are, therefore, recommended in conjunction with this procedure.

c. Laboratory Methods

The laboratory constant head permeability test (ASTM Test Method No. D2434) is normally performed on moderate to highly permeable soils and filter materials, while the falling head test using the consolidometer (ASTM Test Method No. D2435) is performed on materials with low permeability. Both tests measure permeability under saturated conditions. For information concerning laboratory methods refer to Appendix D-3.

d. Field Methods for Design of Basins

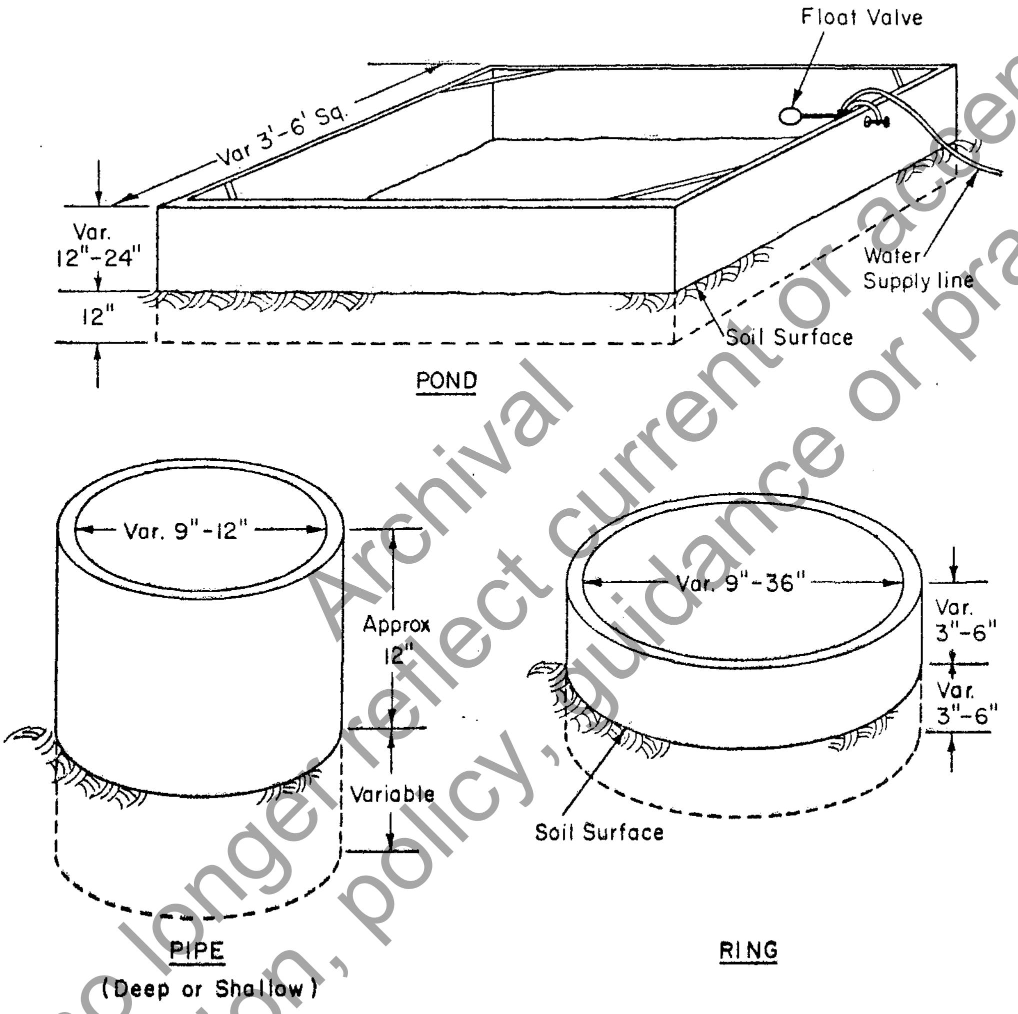
(1) Single Ring (Contra Costa County, California)

This test is applicable for infiltration basins in areas with low water table.

A 12-inch (305 mm) diameter or larger steel pipe is driven into the ground a minimum distance of 12-inches (305 mm), with the ground elevation at the time of the test not more than one foot (0.3 m) from the final profile of the bottom of the spreading basin. Water is kept in the test ring for a sufficient period of time to provide calculated saturated infiltration rates under falling head conditions that do not vary by more than 5%. A minimum of three infiltration tests should be made for each basin. For additional test details refer to Appendix D-4.

2.) Double Concentric Rings

This test is applicable for infiltration basin sites with a low water table. If the permeabilities of the soils under a proposed infiltration basin site vary with depth, tests should be made at sufficient depths to establish the effect of depth on permeability and to aid in determining the required depth of the basin. An infiltrometer is essentially a small model basin consisting of a section of pipe or a bottomless box set to the desired depth in the soil (Figure IV-A-1). The basin is filled to a given depth with water and maintained at a constant head with a float valve for a period of at least a week to measure long-term infiltration rates under saturated conditions. The rate of loss of water in ft/day, in/hour, or cm/day, is defined as the "infiltration rate". If soils are stratified,



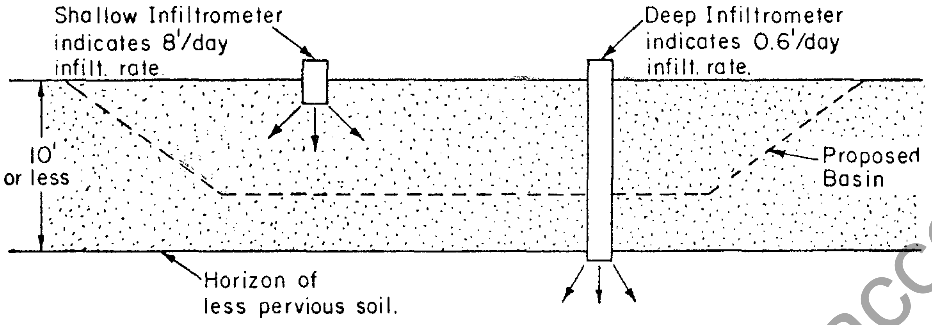
Note: Float valves used on both pipe and ring infiltrometers to regulate water level.

FIGURE IV-A-1 INFILTROMETER TYPES (COURTESY OF CALTRANS) there is a tendency for the infiltrated water to spread laterally. This will have more effect on infiltration from small basins than from large basins. To compensate for spreading tendencies, a larger outer ring or box is also kept filled with water to form a "buffer zone" to confine the primary flow from the inner test cylinder. Only the flow from the inner ring is used in calculating the "infiltration rate". Refer to Figure IV-A-2, ASTM Test Method D3385 and Appendix D-5.

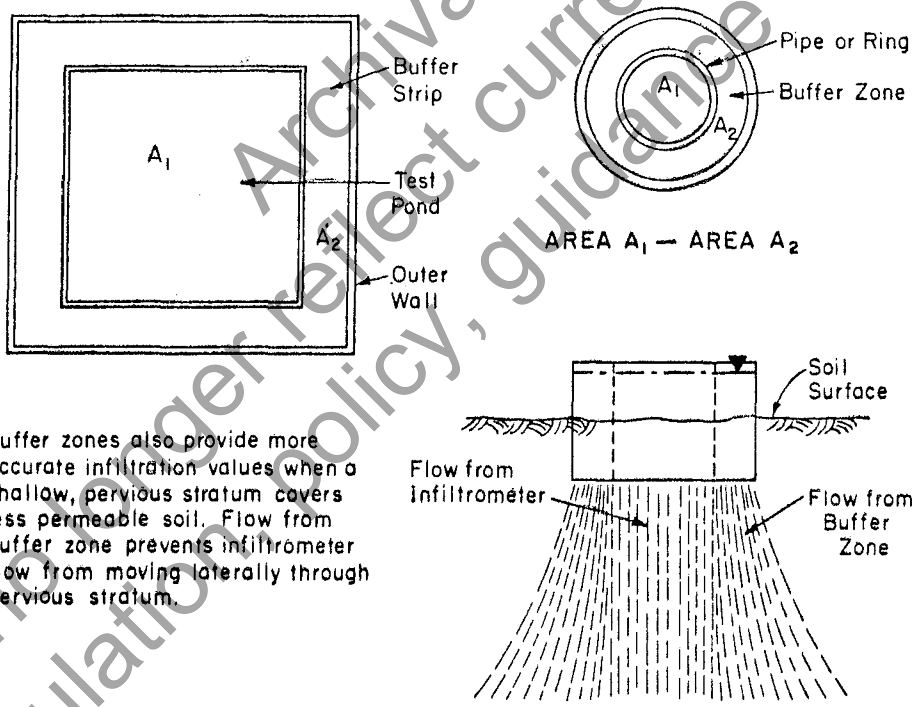
Judgment, based largely on experience, is an important requirement in evaluating infiltration rate data especially where conditions are non uniform. Robinson and Rohwer in 1957(9) studied infiltration in relation to canal seepage and used a variety of equipment installed in the field. They concluded that large-diameter test rings using 6-feet (1.83 m) for an interior ring and 18 feet (5.49 m) for an outer ring provided more accurate measurements than the more commonly used 1 to 2-foot (0.3 to 0.6 m) rings.

3.) Auger Hole Permeability Tests

When water tables are well below the planned bottom elevation of the basin floor, falling head or constant head permeability tests can be performed in auger holes. Numerous procedures are in use for making and interpreting such tests. Methods used by the U.S. Navy are described in Appendix D-6-1. When the U.S. Department of Health, Education, and Welfare method is used for percolation(10), a test hole is kept filled with water for a number of hours, preferably overnight to pre-wet the soil and allow expansive soils to swell at least 24 hours (see Appendix D-6-2 for this procedure). During the test, the drop in water level that occurs in 30 minutes is used as the percolation rate. In sandy or other permeable soils,



A. Type of infiltrometer test employed depends on proposed basin depth and thickness of pervious strata. A deep infiltrometer test provides more realistic values when a shallow permeable layer overlays a less pervious one.



B. Buffer zones also provide more accurate infiltration values when a shallow, pervious stratum covers less permeable soil. Flow from buffer zone prevents infiltrometer flow from moving laterally through pervious stratum.

FIGURE IV-A-2 INFILTROMETER DEPTH AND BUFFER ZONES (COURTESY OF CALTRANS) the time interval between measurements is taken as 10 minutes and the drop that occurs in the final 10 minutes of a 60-minute run is taken as the percolation rate. Basin dimensions can be determined using empirical factors relating basin infiltration to auger hole infiltration, or percolation testing. For design details refer to Section C of Chapter IV, "Design".

e. Field Methods for Design of Infiltration Trenches

1.) Falling Head Percolation Tests In Auger Holes (Dade County, Florida)

This test has application for infiltration trenches in areas of high water table. At points located along the centerline of a proposed infiltration trench, holes 9-inches (229 mm) in diameter or larger are bored to at least 2-feet (0.6 m) below the low-water elevation expected at the site, or to the anticipated elevation of the trench bottom. The portion of the hole below the water table must be kept open during a test. This can be accomplished using a special casing developed specifically for testing in sandy soil as shown in Figure IV-A-3. The special casing is lowered into the auger hole as shown in Figure IV-A-4. The surface elevation, depth to water table, and depth to bottom of casing are recorded. Water is then introduced through the casing until water surface elevation is equal to the design elevation of the top of the proposed drain field which is normally 3 feet (0.915 m) below final ground level. The time is recorded as the water drops in the test hole in 6-inch (152 mm) increments as determined by a float device similar to that shown in Figure IV-A-5.

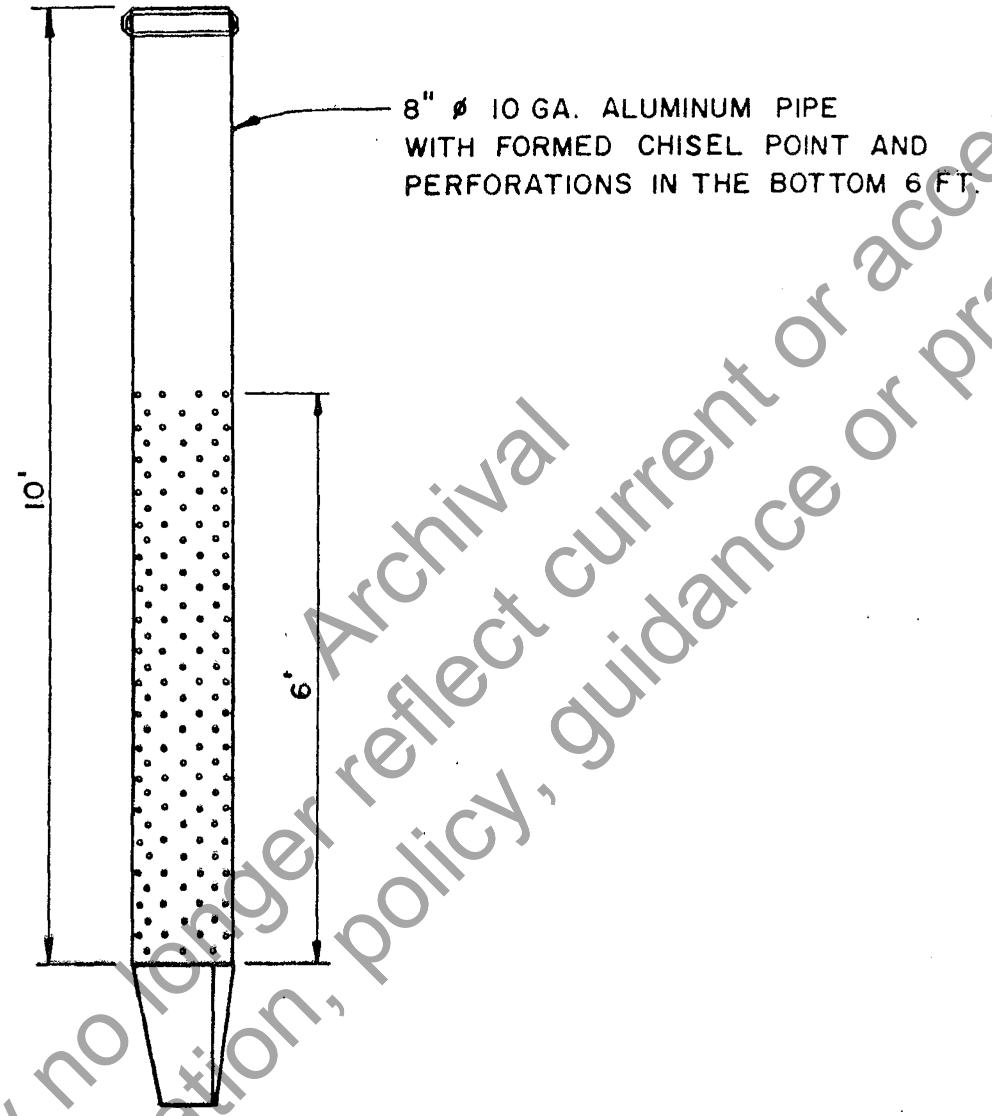


FIGURE IV-A-3 CASING FOR INFILTRATION TEST IN SANDY SOIL (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA)

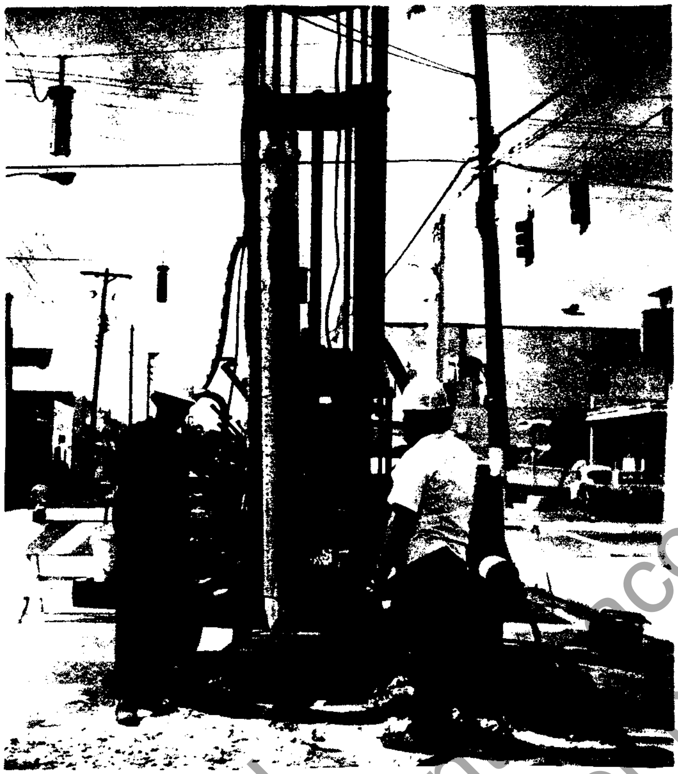


FIGURE IV-A-4 SPECIAL CASING FOR AUGER HOLE PERMEABILITY TESTING (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA)

FIGURE IV-A-5 SPECIAL FLOAT DEVICE FOR MEASURING WATER LEVEL CHANGE IN AUGER HOLES (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA)

The volume of water in a specific 6-inch (152 mm) increment of the test hole divided by the time recorded to drop that 6-inch (152 mm) increment results in a rate of infiltration for that specific 6-inch (152 mm) increment:

Q = Infiltration Rate

$Q = V/\Delta t$

where: V = volume of test hole for increment Δh

Δt = time interval for water to fall increment depth (Δh) as shown in Figure IV-A-6.

The rate of infiltration for a specific 6-inch (152 mm) incremental drop divided by the circumference of the test hole gives an infiltration rate for that specific increment per linear foot (0.305 m) of wall area of the test hole as per the following expression:

$Q_{L.F.}$ = Infiltration Rate per lineal foot (0.305 m) of wall

$$Q_{L.F.} = \frac{Q}{C} = \frac{V}{\Delta t x C}$$

Where: V and Δt are defined above and

C = Circumference of test hole

Since the proposed infiltration trench has two sides, a factor of 2 is applied to give the total exfiltration rate (Q_t) per lineal foot (0.305 m) of trench for a particular 6-inch (152 mm) increment of test hole.

The bottom of the test hole is not considered in the design since it has minimal influence on overall exfiltration rate. In design the bottom of the trench is also ignored as an exfiltration area and provides an added safety factor. The permeability in the lateral direction is usually significantly larger than that in the vertical direction.

Let: Q_t = exfiltration rate per linear foot (0.305 m) of trench (cfs or m^3/sec)

$$Q_t = 2Q_{L.F.} = \frac{2V}{\Delta t x c}$$

Figure IV-A-7 illustrates the exfiltration rate per linear foot (0.305 m) of trench based on percolation tests at 6-inch (152 mm) increments of test hole. The design rate is based on the highest practical elevation of hydraulic head that can be obtained.

2.) Constant Head Percolation Tests In Auger Holes (Dade County, Florida)

The initial preparation for this test is the same as for the falling head test. However, water is discharged into the test hole at a rate to allow a constant head to be held in intervals of one or more feet (0.3 m or more) depending on depth of hole. This is done to determine if localized soil strata affects infiltration. Water is continually added until the top elevation of the drain field is reached. A constant head should be held for at least 5 minutes at each interval; however, a longer period would provide more accurate infiltration rates.

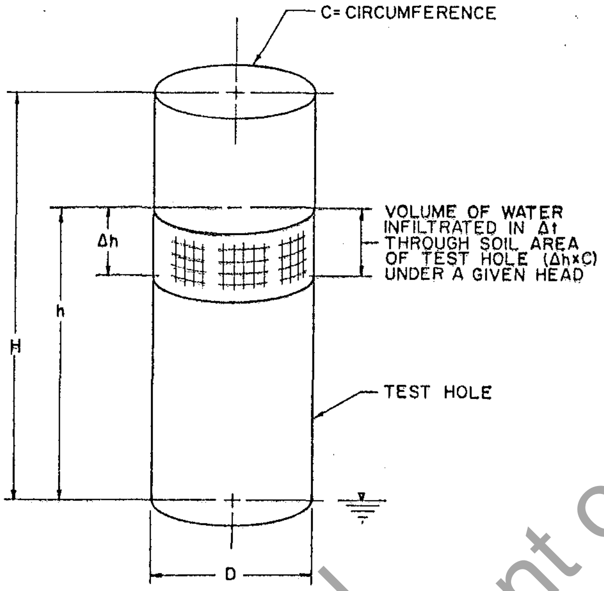


FIGURE IV-A-6 TEST HOLE

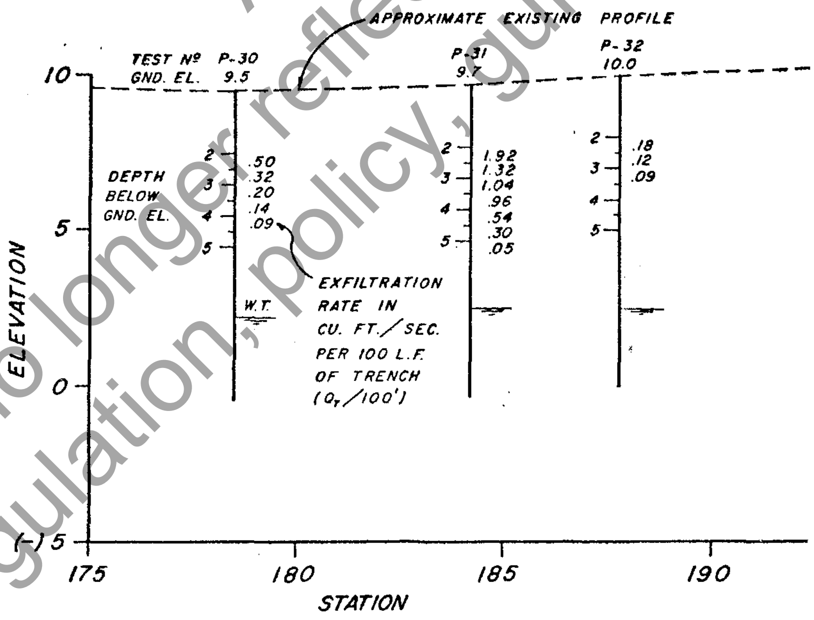


FIGURE IV-A-7 EXFILTRATION RATES DETERMINED FROM FALLING HEAD PERCOLATION TESTS (COURTESY OF BRISTOL, CHILDS & ASSOCIATES, CORAL GABLES, FLORIDA)

The infiltration rate per linear foot (0.305 m) of wall area of test hole can be determined for a given constant head using the inflow, Q in cfs or m^3/sec, required to maintain the constant head and relating the flow to the circumference, C, of the test hole, i.e.:

$$Q_{L.F.} = \frac{Q}{C}$$

The exfiltration rate per linear foot of trench is:

$$Q_t = 2\ Q_{L.F.}$$

The design rate is based on the highest practical elevation of hydraulic head that can be obtained.

For actual trench design refer to Chapter IV-C.

3.) Auger Hole Permeability Tests

When water tables are below the planned seepage trenches, falling head (or constant head) permeability tests are frequently made in auger holes drilled to the planned depth of the trenches. Numerous procedures similar to the method described in Sections e-(1) and e-(2) are in use for making and interpreting such tests. The tests described in Section d-(3) and in Appendix D-6 can also be utilized for trench design. The trench dimensions are determined using empirical factors relating trench flow with auger hole flow. For details refer to Chapter IV-C.

f. Field Methods for Design of Wells and Pits

1.) Well Pumping Test

In situations where the flow will be below an existing water table under saturated conditions, well pumping tests provide one of the best methods for estimating in-place permeability. Since the flow to wells is predominately in a horizontal direction (see Appendix D-7), well pumping tests are, in essence, measuring horizontal permeability, which determines the capabilities of underlying soils to discharge seepage laterally. The "well" is pumped while the amount of drawdown is measured in one or more arrays of observation wells. Permeability is calculated as defined in Appendix D-7.

Usually a number of calculations of permeability are made using various combinations of drawdown in pairs of wells and the average is used as representing the permeability of the soil tested.

2.) Auger Hole Permeability Test

Tests similar to those described in Sections d-(3), e-(1), e-(2) and e-(3) and Appendix D-6 can be utilized to design shallow dry wells and seepage pits.

3. Theoretical Methods For Estimating Infiltration Rates

The Darcy coefficient of permeability (k) is defined either as the discharge velocity ($v_d = \underline{ki}$) under a hydraulic gradient (i) of 1.0, or as the quantity of seepage per unit area under a hydraulic gradient of 1.0. For a given soil under a given state of compaction, etc., k has a specific value that can be used for calculating seepage velocities and seepage quantities under any hydraulic gradient selected for analysis.

In order to apply Darcy's law, or flow nets and other calculation methods using seepage fundamentals, it is necessary to know the Darcy coefficients of permeabilities of the soil formations in which water is flowing. While Darcy's law was originally conceived for saturated flow, it can also be used for unsaturated flow when care is taken to use appropriate coefficients of permeability. The general procedures for using Darcy's law for various cases are presented in Appendix D-8.

Various theoretical methods have been developed for analyzing flow in both saturated and unsaturated soils. A method described by Weaver(<u>5</u>) was developed by the New York Department of Transportation for estimating infiltration rates for unsaturated flow [bottom of basin or trench more than a few feet (1 m ±) above the groundwater level or an impervious stratum]. The method is used for infiltration basins with a large ratio of surface area to perimeter, assuming all outflow is downwards. It provides conservative results for point and line sources (catch basins and trenches) where a large portion of the flow will move laterally through the sides.

Where the bottom of a infiltration basin or trench is below the groundwater table, the infiltration rate should be estimated on the basis of saturated flow. The same is true if the bottom of the basin is only slightly above the groundwater level on an impervious stratum, and the groundwater can be expected to mound up to the bottom of the basin or a perched groundwater table can develop under a basin or trench.

Theoretical considerations in the gravity flow of water out of ditches are given by Muskat in 1937(11) and by Harr in 1962(12). Numerous books and reports contain formulas for estimating flow into wells or slots. By making appropriate conversions, these formulas can be adapted to the case of outflow from wells or slots(13).

Approximate two-dimensional methods for estimating flow to large excavations or sumps were given by Cedergren in 1977(14). These methods can also be adapted to the outflow case.

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B. HYDROLOGY

1. General

The hydrologic input required for the design of any infiltration drainage system is the time-related inflow distribution. This input is usually in the form of a hydrograph or a mass inflow curve. The appropriate hydrologic method used to define this relationship can best be determined by the designer based on consideration of the physical and hydrologic characteristics of the drainage area, the data available, and the degree of sophistication warranted in the design. The designer must be aware of the various methods available to estimate runoff and particularly the limitations of these methods.

It is not the intent of this manual to discuss hydrology in detail nor to recommend a method for estimating runoff. The purpose is rather to discuss data sources, and briefly describe the more commonly utilized runoff estimating procedures and their limitations.

2. Hydrologic Information

The National Weather Service (NOAA) collects precipitation data and publishes the results in various documents, as listed in Table IV-B-1(1). The information is presented as isohyetal lines on geographic maps of the United States, Puerto Rico, and the Virgin Islands. The technical publications listed under subheadings A and B in Table IV-B-1 give the precipitations to be expected within certain durations and return periods.

TABLE IV-B-1(1)

NATIONAL WEATHER SERVICE PUBLICATIONS* - PRECIPITATION DATA

A. Durations to 1 day and return periods to 100 years

NOAA Technical Memorandum NWS Hydro-35 "5 to 60-Minute Precipitation Frequency for Eastern and Central United States", 1977

Technical Paper 40. 48 contiguous states (1961)
(Use for 37 contiguous states east of the 105th meridian for durations of 2 to 24 hours. Use NOAA NWS HYDRO-35 for durations of 1 hour or less.)

Technical Paper 42. Puerto Rico and Virgin Islands (1961)
Technical Paper 43. Hawaii (1962)
Technical Paper 47. Alaska (1963)

NOAA Atlas 2. Precipitation Atlas of the Western United States (1973)

Vol. I, Montana
Vol. II, Wyoming
Vol. III, Colorado
Vol. IV, New Mexico
Vol. V, Idaho
Vol. VI, Utah
Vol. VII, Nevada
Vol. VIII, Arizona
Vol. IX, Washington
Vol. X, Oregon
Vol. XI, California

B. Durations from 2 to 10 days and return periods to 100 years

Technical Paper 49. 48 contiguous states (1964)
(Use SCS West Technical Service Center Technical Note - Hydrology - PO-6 Rev. 1973, for states covered by NOAA Atlas 2.)

Technical Paper 51. Hawaii (1965)
Technical Paper 52. Alaska (1965)
Technical Paper 53. Puerto Rico and Virgin Islands (1965)

C. Probable maximum precipitation

Hydrometeorological Report 33. States east of the 105th (1956)
(Use Fig. 4-12, NWS map for 6-hour PMP (1975). This map replaces ES-1020 and PMP maps in TP-40** which are based on HM Report 33 and TP-38.)

Hydrometeorological Report 36. California (1961)

Hydrometeorological Report 39. Hawaii (1963)
(PMP maps in TP-43** are based on HM Report 39)

Hydrometeorological Report 43. Northwest States (1966)

Technical Paper 38. States west of the 105th meridian (1960)
Technical Paper 42** Puerto Rico and Virgin Islands (1961)
Technical Paper 47** Alaska (1963)

Unpublished Reports:
***Thunderstorms, Southwest States (1972)
Upper Rio Grande Basin, New Mexico, Colorado (1967)

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*National Weather Service (NWS), National Oceanic and Atmospheric Administration (NOAA), U. S. Department of Commerce, formerly U. S. Weather Bureau.

**Technical papers listed in both A and C

Being replaced by Hydrometeorological Report No. 51 "Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 20,000 Square Miles and Durations from 6 to 72 Hours", available end of 1977.

***Being replaced by Hydrometeorological Report No. 49 "Probable Maximum Precipitation, Colorado and Great Basin Drainages".

Technical Publication No. 40, listed under "A" in Table IV-B-1, is a valuable tool in urban drainage studies, since it give rainfall for various durations and frequencies of recurrence. Other federal agencies such as the USGS and the Corps of Engineers are also good sources of rainfall information. In addition, records are maintained by State Highway or Transportation Departments, State Water Resources Agencies, Cities, Counties, local drainage districts, and utility companies. For rainfall intensity-duration-frequency data for Canada, refer to reference (2) at the end of this chapter.

a. Rainfall Intensity - Duration Curves

Rainfall intensity - duration-frequency (I.D.F.) curves are derived from the statistical analysis of rainfall records compiled over a number of years. Each curve represents the intensity-time relationship for a storm of a certain return frequency (3). Refer to Figure IV-B-1a.

The intensity, or the rate of rainfall, is usually expressed in a depth per unit time, with the highest intensities occurring over short time intervals and progressively decreasing as the time intervals increase. The highest intensity for a specific duration for n years of record is called the n year storm, with a frequency of once in n years.

It should be noted that the I.D.F. curves do not represent a rainfall pattern, but are the distribution of the highest intensities over time durations for a storm of n frequency. The rainfall intensity-duration curves are readily available from governmental agencies, and are widely used in

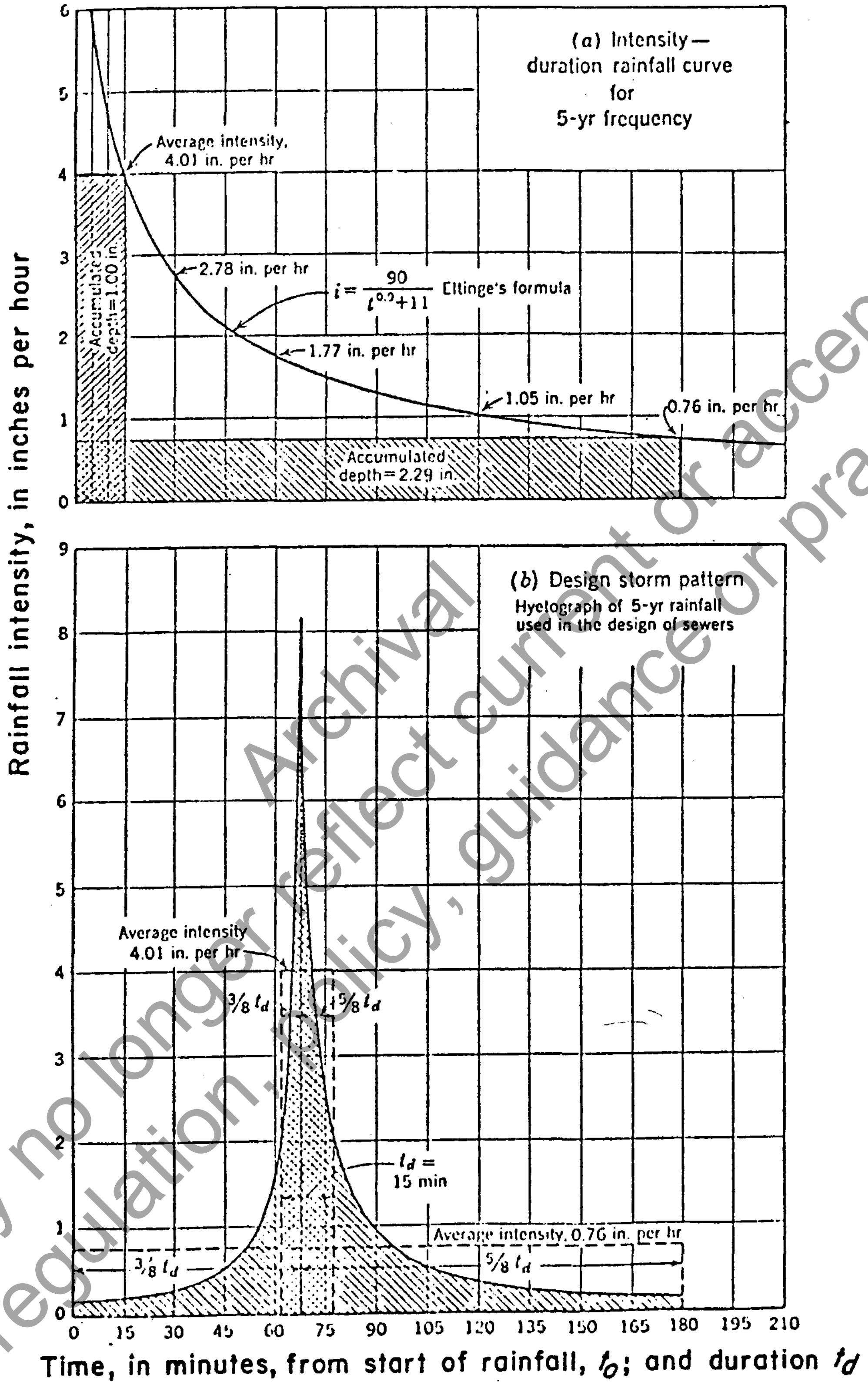

FIGURE IV-B-1 INTENSITY-DURATION CURVE AND CONCOMITANT STORM PATTERN CHICAGO, ILL. (5)&(6)

the designing of storm drainage facilities and flood flow analysis.

b. Rainfall Hyetographs

Rainfall hyetographs are a graphical representation of rainfall over time. Synthetic design hyetographs may be derived from the I.D.F. curve, using the Chicago Method (4).

Briefly stated, this method consists of selecting an allowable storm frequency for the proposed storm drain and determining from rainfall statistics the intensity-duration curve (Figure IV-B-1a) for the selected storm frequency. The chronological storm pattern or hyetograph (Figure IV-B-1b) is then determined for storms which are most likely to cause excessive runoff. The design storm pattern or hyetograph is computed to conform at all points of the intensity-duration curve.

The average rate of rainfall during the maximum 15-minute period of the hyetograph equals the rate shown for 15-minutes duration on the intensity-duration curve, and similary for all other durations (5).

More recently, (1977), the development and use of the non-dimensional triangular hyetograph has been reported by Yen and Chow (6). They report that

> "An analysis of 9,869 rainstorms at four locations indicates that for a given season the non-dimensional triangular hyetographs for heavy rainstorms are nearly identical, having only secondary effects from the duration of rainfall, measurement accuracies of standard U.S. National Weather Service precipitation data, and insignificant effect of geographic locations."

Simple procedures of how to use the nondimensional triangular hyetograph to produce the design hyetograph are outlined below:

Notation:

D = Depth of rainfall
t_d = Duration of rainfall
T_R = Return period
a = Time to peak = $t_d a°$
$0.33 < a° < 0.50$
h = Peak rainfall intensity

Procedure:

1. Determine D from NOAA ATLAS For the desired storm duration t_d
2. then $h = 2D/t_d$
3. Plot rainfall hyetograph with these parameters (Figure IV-B-2).

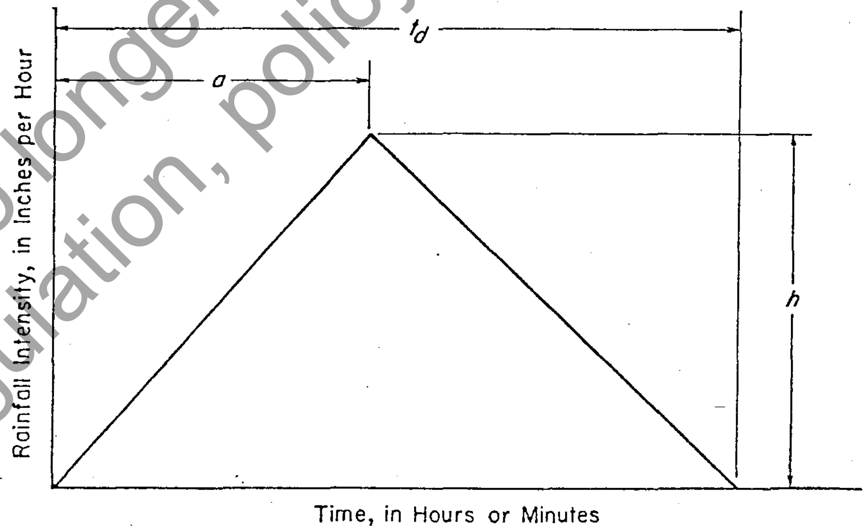

FIGURE IV-B-2 SIMPLIFIED TRIANGULAR HYETOGRAPH

A simplified hydrograph procedure based on the assumed triangular hyetograph is described in Section 3C(3).

3. Methods for Estimating Runoff

There are numerous methods available today for estimating runoff, ranging from the Rational Method developed in 1889 (7) to sophisticated computer simulation models.

The selection of any method must be based on the degree of accuracy required, recognizing the scope and limitations of each method. Except in the rare cases where the infiltration rate of the soils meet or exceed the peak rate of runoff, a graph showing runoff distribution with time must be developed to design an infiltration system. This can be in the form of a hydrograph or a mass inflow curve.

a. Rational Method

The Rational Method is widely used to determine peak flows in positive drainage systems by the equation

$$Q = CIA$$

Where Q = Design peak flow (runoff), in cubic feet per second.

C = Coefficient of runoff

I = Average rainfall intensity, in inches per hour for a given frequency and for the duration usually equal to the time of concentration.

A = Drainage area, in acres.

When using the Rational Method, the following assumptions are made:

1. The rainfall intensity is uniform over the entire watershed during the entire storm duration,

2. the maximum runoff rate occurs when the rainfall lasts as long or longer than the time of concentration, and

3. the time of concentration is the time required for the runoff from the most remote part of the watershed to reach the point under design.

1.) Coefficient of Runoff, C

The only manipulative factor in the Rational Formula is the runoff coefficient C. Judgment should be used in selecting this value, as it must incorporate most of the hydrological abstractions, soil types, antecedent conditions, etc. Typical values for coefficient of runoff are shown in Table IV-B-2 for various types of land use and surface conditions. These coefficients are applicable for storms of 5 to 10-year frequencies. Less frequent higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller affect on runoff (5). It is common practice to select average coefficients and assume that the coefficients will not vary through the duration of the storm. However, it is generally agreed that these coefficients of runoff for any given surface will vary with respect to prior wetting.

TABLE IV-B-2

TYPICAL RUNOFF COEFFICIENTS FOR VARIOUS TYPES OF LAND USE AND SURFACE CONDITIONS(5)

LAND USE	RUNOFF COEFFICIENTS (C)
Business:	
Downtown areas	0.70 to 0.95
Neighborhood areas	0.50 to 0.70
Residential:	
Single-family areas	0.30 to 0.50
Multi units, detached	0.40 to 0.60
Multi units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment dwelling areas	0.50 to 0.70
Industrial:	
Light areas	0.50 to 0.80
Heavy areas	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard areas	0.20 to 0.40
Unimproved areas	0.10 to 0.30
SURFACE CONDITIONS	
Streets:	
Asphaltic	0.70 to 0.95
Concrete	0.80 to 0.95
Brick	0.70 to 0.85
Drives and walks	0.75 to 0.85
Roofs	0.75 to 0.95
Lawns; Sandy Soil:	
Flat, 2%	0.05 to 0.10
Average, 2 to 7%	0.10 to 0.15
Steep, 7%	0.15 to 0.20
Lawns; Heavy Soil:	
Flat, 2%	0.13 to 0.17
Average, 2 to 7%	0.18 to 0.22
Steep, 7%	0.25 to 0.35

> "Usually a substantial period of rainfall will have occurred before the beginning of the time of concentration and consequently, the low coefficients indicated at the beginning of rainfall are in no way representative of storm conditions when the average design intensity occurs." (1)

2.) Rainfall Intensity, I

The rainfall intensity to be used in the Rational Method for determining peak flow should be for the design frequency, and of a duration equal to the time of concentration. This information is developed as previously discussed.

3.) Time of Concentration, t_c

The time of concentration (t_c) is the time required for runoff to arrive at the point of concentration (such as the inlet to an infiltration system) from the most remote point of the drainage area. Time of concentration is generally developed relative to the initial point of concentration. Drainage system calculations also require the addition of time of flow in the system between the inlet and the point of control. Inlet times generally used in urban drainage design vary from 5 to 20 minutes with the channel flow time being determined from pipe flow equations.

4.) Limitations of Rational Method

The Rational Method does have limitations and should only be applied to relatively small drainage areas. The maximum acceptable size of the watershed varies from 200 to 500 acres (0.90 to 1.422 Km^2) depending upon the degree of urbanization. The APWA Special Report No. 43 (8) recommends that urban drainage areas should be limited to less than 20 acres (0.284 Km^2) in size, such as rooftops and parking lots. As drainage areas become larger and more complex, the C coefficient cannot account for the many natural hydrological abstractions, surface routing, and antecedent moisture conditions.

b. Modified Rational Method for Development of Mass Inflow Curves

The Rational Method has been used to calculate the total cumulative volume of rainfall runoff versus time (mass flow) by modifying the formula to read V = CIAT.

Where V = Volume of runoff in cubic feet
C = Coefficient of runoff
I = Average rainfall intensity, in inches per hour for a given frequency and for selected durations of time in increments sufficient to plot a curve showing total cumulative volume of rainfall runoff versus time
A = Drainage area, in acres
T = Time in seconds which corresponds to the selected durations of rainfall.

The following is an example calculation for the mass flow curve for a 3-year frequency design storm, using hourly intensities from Figure IV-B-3 and the Modified Rational Equation:

Assume A = 1.0 acre (4,047 m^2) drainage area and C = 0.9. Using the Modified Rational Formula, V = CIAT, CA = (0.9) (1.0) = 0.9. The following cumulative volumes of flow are developed:

Time Minutes	CA	x	I Inches/hr.	x	Time Seconds		Volume Cu. Ft.
10	0.9	x	5.60	x	600	=	3,024
15	0.9	x	4.90	x	900	=	3,969
20	0.9	x	4.40	x	1,200	=	4,752
30	0.9	x	3.75	x	1,800	=	6,075
60	0.9	x	2.65	x	3,600	=	8,586
90	0.9	x	2.10	x	5,400	=	10,206
120	0.9	x	1.75	x	7,200	=	11,340
150	0.9	x	1.50	x	9,000	=	12,150
180	0.9	x	1.35	x	10,800	=	13,122
240	0.9	x	1.10	x	14,400	=	14,256
360	0.9	x	0.83	x	21,600	=	16,135

The resulting inflow curve is shown in Figure IV-B-4. Specific applications are discussed under "Design of Storm Water Collection and Disposal Systems", in Chapter IV-C.

It should be recognized that most mass inflow curves constructed using the above procedure do not truly reflect the expected accumulated runoff as a function of time, since the probable storm pattern and the storage affects of the watershed are not considered. However, the results in sizing underground disposal systems using this procedure should be conservative in most instances. The simplicity of the method makes it attractive where more detailed studies may not be warranted.

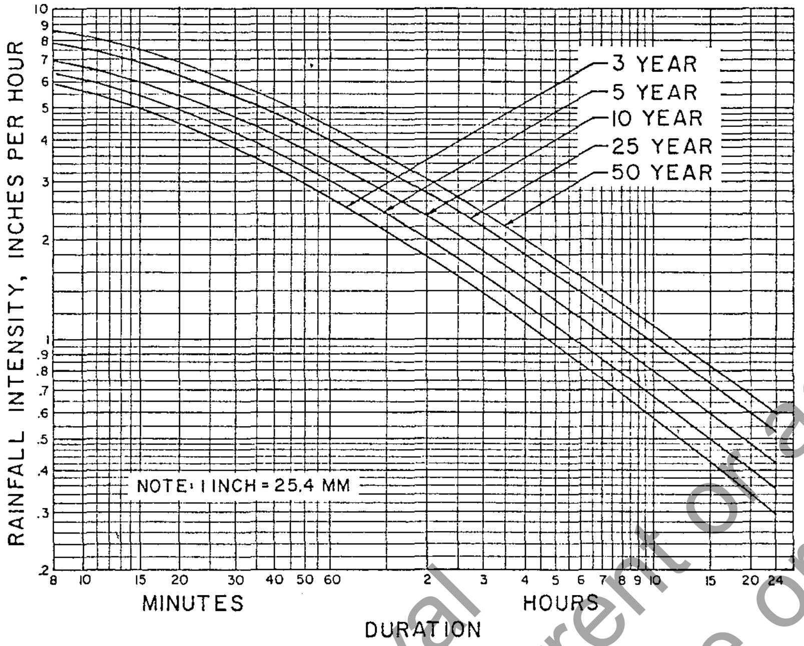

FIGURE IV-B-3 RAINFALL INTENSITY-DURATION-FREQUENCY CURVES FOR ZONE 5, MIAMI (FROM FLORIDA DOT)

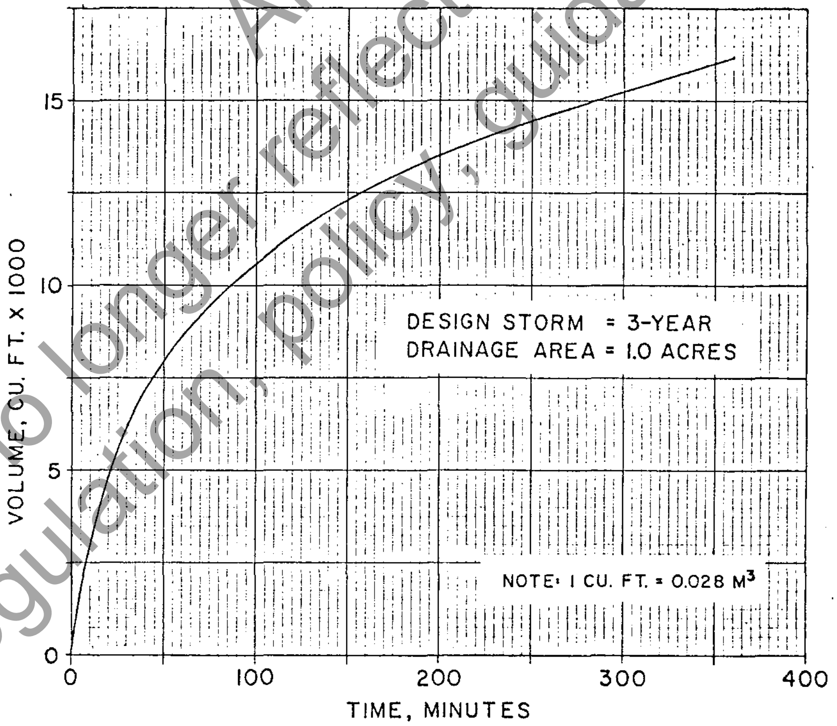

FIGURE IV-B-4 MASS INFLOW CURVE (COURTESY OF BRISTOL, CHILDS & ASSOC., CORAL GABLES, FLORIDA)

c. Hydrograph Methods

As previously indicated in this chapter, hydrograph methods relate runoff rates to time during a design storm, and are generally more applicable to larger watersheds, though used also with small watersheds, particularly where storage is considered.

Natural hydrographs are those obtained directly from the flow records of a gaged stream channel or conduit. Synthetic hydrographs are developed using watershed parameters and storm characteristics to simulate a natural hydrograph. A unit hydrograph is defined as a hydrograph of a direct runoff resulting from 1 inch (25.4 mm) of effective rainfall generated uniformly over the watershed area during a specified period of time or duration. The unit hydrograph can be used to develop the hydrograph of runoff for any quantity of effective rainfall.

The unit hydrograph theory, assumptions, and limitations are discussed in detail in references (9) and (10).

1.) Synthetic Unit Hydrographs

In most drainage basins rainfall runoff data from which unit hydrographs can be derived is unavailable, thus a synthetic unit hydrograph must be derived. The U.S. Soil Conservation Service (SCS) has developed a method of hydrograph synthesis which is now being widely used.

The development of the SCS unit hydrograph technique is well documented (11). Studies by the U.S. Soil Conservation Service over the last 30 to 35 years have resulted in empirical relationships between rainfall runoff and the associate land use which are used in conjunction with the SCS Unit Hydrograph Method. Each particular land use is assigned a corresponding runoff curve number (CN), which is an indication of the runoff potential. The value is based on a combination of hydrological soil group, treatment class and antecedent conditions.

The following are limitations of SCS Unit Hydrographs:

1. The drainage area should be limited to 20 square miles (51.8 Km^2). If the total watershed is very large, it should be broken down into uniformly shaped divisions with a maximum of 20 square miles (51.8 Km^2) each.
2. The drainage areas should have a constant CN value.
3. There should be a homogeneous drainage pattern within the drainage area.
4. Care should be taken in determining the representative CN value as it will have a direct effect in the hydrograph peak.

2.) SCS Tabular Hydrograph Method

This method provides a tabular approach to estimating peak concentration and travel time. It also develops hydrographs for each sub-drainage area and then routes them through the watershed area resulting in a composite hydrograph at the outfall. This method can readily predict the increase in peak flow when all or a portion of the watershed is to be developed. The SCS tabular method is described along with examples of applications in SCS Technical Release No. 55 (12).

3.) Simplified Equivalent Triangular Hydrograph

A normal curvilinear hydrograph can usually be represented by an equivalent triangle as shown in Figure IV-B-5. Both graphs represent the same amount of runoff and the same time to peak; therefore, for practical purposes the triangle is an adequate representation of the curvilinear graph.

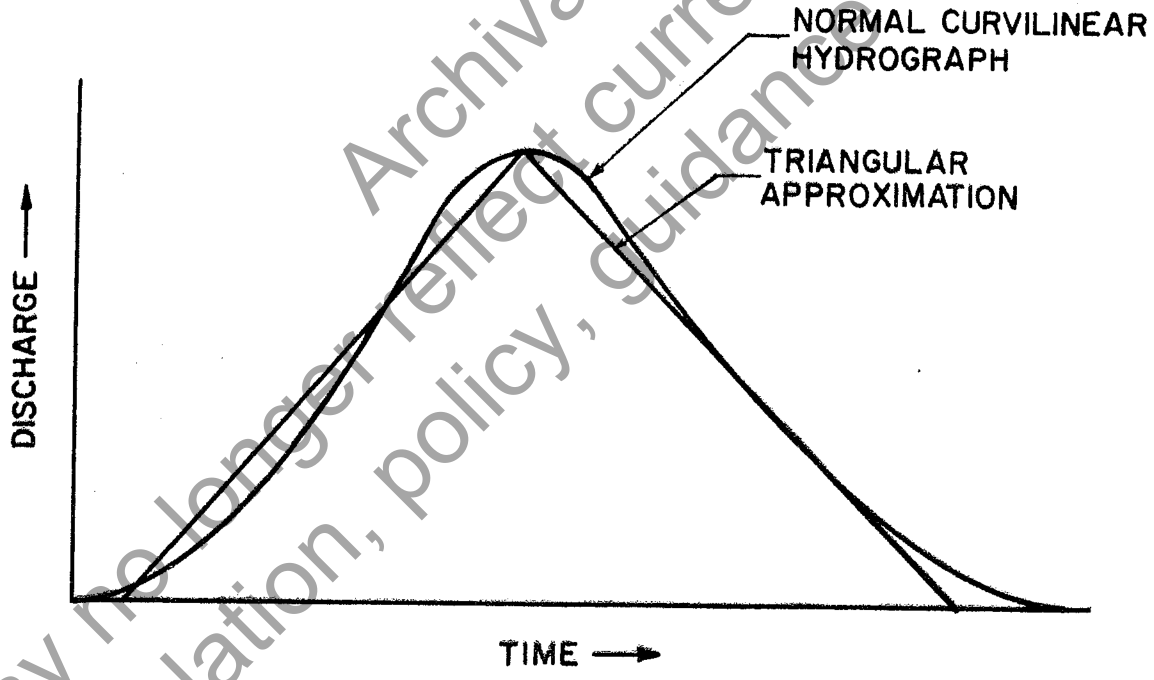

FIGURE IV-B-5 TRIANGULAR APPROXIMATION OF RUNOFF HYDROGRAPH

The simplified hydrograph procedure is based on an assumed triangular hyetograph as previously described in this chapter. Abstractions are applied from information using SCS curve numbers or guidance provided by local experience.

Assuming a linear watershed response (i.e., the area contributing to runoff increases more or less uniformly up to the time of concentration), a triangular distribution of excess rainfall may be converted to an approximate triangular runoff hydrograph as shown in Figure IV-B-6. The peak runoff is equal to the maximum average effective rainfall intensity over the time of concentration and is shifted to the right $(1 - a^{\circ})\ t_c$ units.

The peak runoff in cfs (m^3/sec) is determined from the equation:

$$Q_p = I_p \left(1 - \frac{t_c}{2b}\right) A$$

Where I_p = Maximum effective rainfall intensity in inches/ hour (mm/hr.)

A = Area of the drainage basin in acres

t_c = Time of concentration in minutes, and

b = Duration of effective rainfall in minutes ($b \geq t_c$).

The lengthening of the time base, Δ, is given by

$$\Delta = \frac{t_c}{(2b - t_c)}\ b$$

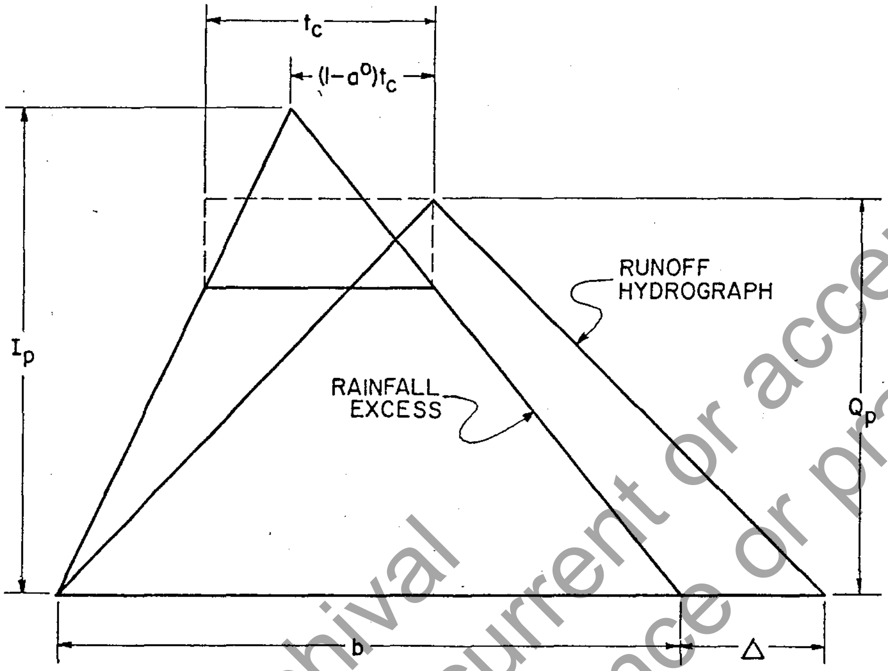

FIGURE IV-B-6 RELATIONSHIP BETWEEN SIMPLIFIED TRIANGULAR PLOT OF RAINFALL EXCESS AND TRIANGULAR RUNOFF HYDROGRAPH

Example

Assume the following:

Drainage area = 6 acres (0.156 Km^2)
Design Storm Frequency (Return Period) = 10 years
Duration of Storm = 2 hours
$a^\circ = 0.33$
Rainfall Depth, D = 3.63 inches (92.2mm)
Time of Concentration, t_c = 30 minutes
Infiltration rate of drainage area:
Initial = 1 inch/hr. (25.4mm/hr.)
Final = 1/4 inch/hr. (6.4mm/hr.)

<u>Step 1</u> Derive and plot the triangular hyetograph as previously described (Figure IV-B-2).

$$h = \frac{2D}{t_d} = \frac{2(3.63)}{2} = 3.63 \text{ inches/hr. } (92.2 \text{ mm/hr.})$$

$$a = t_d\, a° = 2 \text{ hr. } (60 \text{ min./hr.})\ (0.33) = 40 \text{ minutes}$$

<u>Step 2</u> Deduct losses as shown in Figure IV-B-7. (The resulting shape must be approximately a triangle.)

<u>Step 3</u> Scale the new time base and the maximum effective rainfall intensity.

$b = 105$ minutes

$I_p = 2.9$ inches/hr. (73.7 mm/hr.)

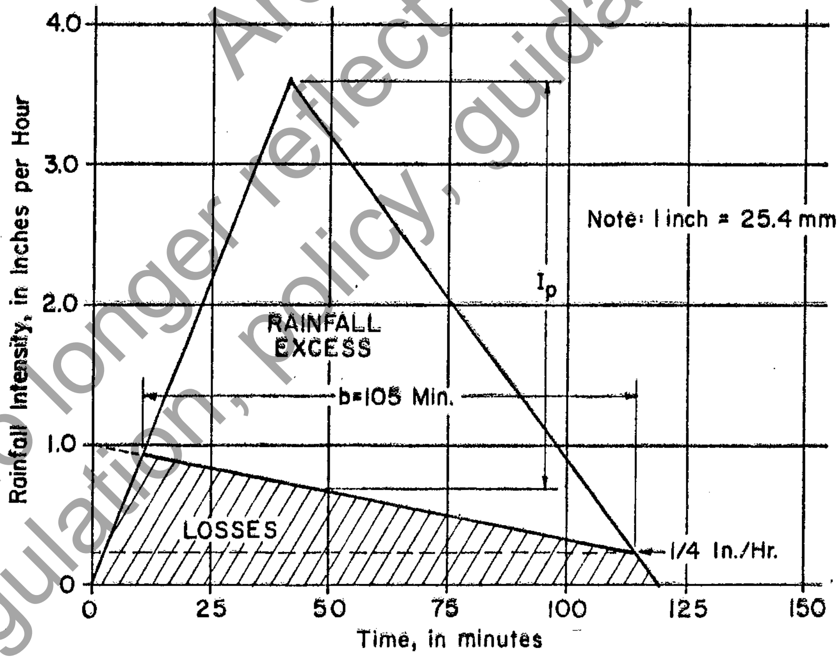

FIGURE IV-B-7 TRIANGULAR HYETOGRAPH SHOWING PEAK RAINFALL INTENSITY

Step 4 Compute Q_p, and the time to peak.

$$Q_p = I_p \left(1 - \frac{t_c}{2b}\right) A$$

$$= 2.9 \left(1 - \frac{30}{210}\right) 6 = 14.9 \text{ cfs } (0.417 \text{ m}^3/\text{sec})$$

$$= \left(\frac{t_c}{2b-t_c}\right) b = \left(\frac{30}{210-30}\right) 105 = 17.5 \text{ minutes}$$

$$\text{Time to peak} = 40 + (1-0.33)(30)$$
$$= 60 \text{ minutes}$$

Step 5 Plot the triangular runoff hydrograph using these parameters (Figure IV-B-8a).

Step 6 The cumulative runoff curve is determined by summing the area under the triangular hydrograph from left to right and plotting the results as a function of time (Figure IV-B-8b).

Runoff hydrographs will differ depending on the storm duration chosen. The designer may need to investigate various types of storms in sizing an underground disposal system.

d. Computer Modelling

In recent years computer models have been developed to aid the designer in his analysis of the hydrological and hydraulic analyses of drainage systems. Of the numerous models available today, the ones listed below are believed to be most applicable in the generation of runoff for the design of subsurface disposal facilities:

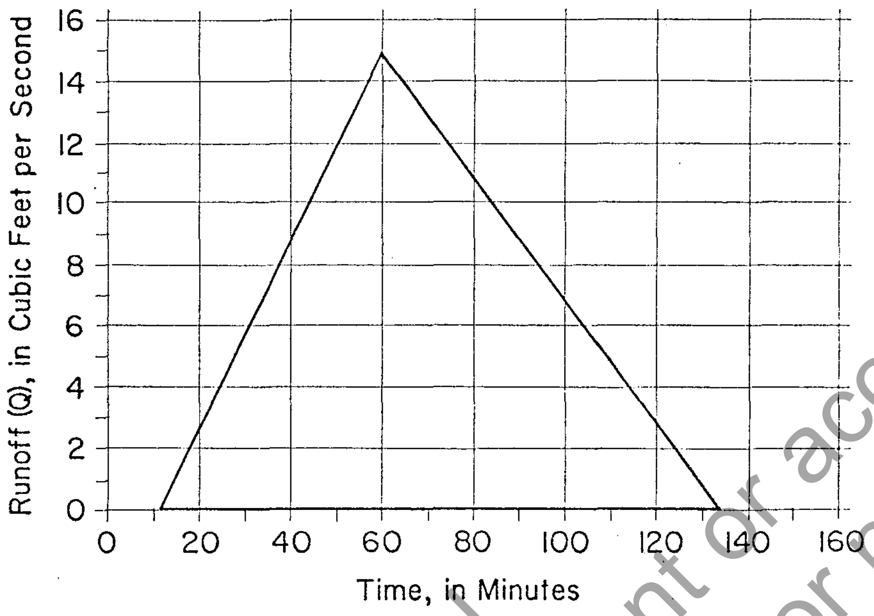

FIGURE IV-B-8a TRIANGULAR RUNOFF HYDROGRAPH

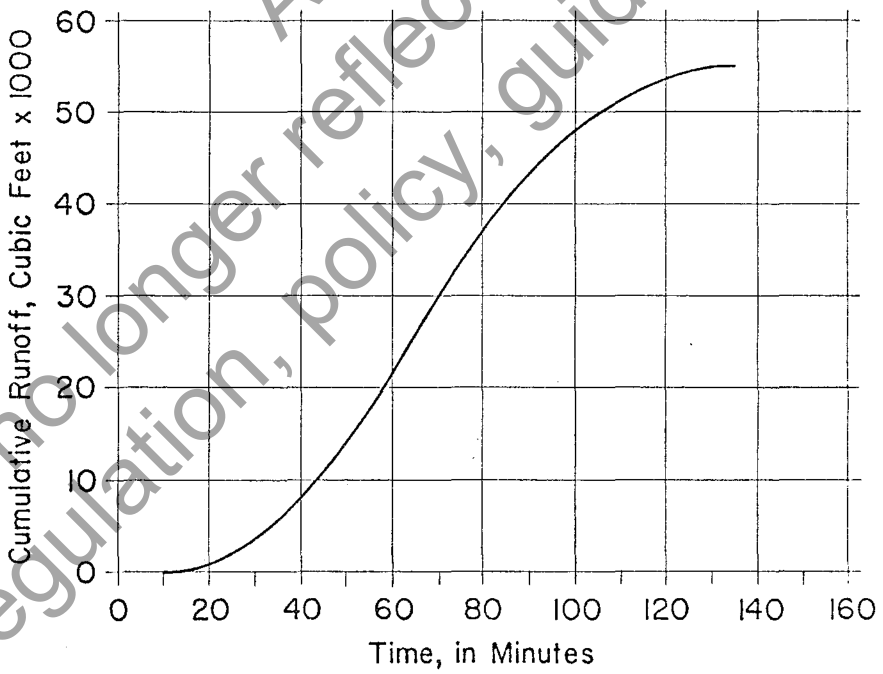

FIGURE IV-B-8b CUMULATIVE RUNOFF CURVE

SWMM: A sophisticated hydrologic and hydraulic simulation model used primarily for complex urban drainage systems.

ILLUDAS: A simulation model with the capacity of accurately simulating the runoff from urban areas, but continuing a relatively simple routing procedure for pipe flow.

HYMO: A model well-suited for generating runoff from rural or undeveloped lands (may also be used in urban areas) based on the SCS CN runoff parameters, but with a modified unit hydrograph procedure.

4. Summary

This chapter has provided a brief overview on the hydrollogy involved in estimating storm water runoff for underground disposal systems. A reference list is provided at the end of this section to allow the designer to obtain additional information on methods and techniques which he feels are applicable to his study area. Particular design applications are contained in Chapter IV-C.

Table IV-B-3 summarizes the characteristics and application of the methods covered in this chapter, to assist the designer in the selection of the appropriate method.

TABLE IV-B-3

RUNOFF MODELS

METHOD	DRAINAGE AREA	REQUIRED INFORMATION	VARIABLES	OUTPUT	APPLICATIONS
RATIONAL METHOD	≤ 20 acres (APWA Spec. Rep. No. 43) <500 acres (FHWA)	Land Cover Time of Concentration IDF Curves	Runoff Coefficient (C)	Peak Flows	Minor and Major Storm System Design
UNIT HYDROGRAPH	≤1000 sq. mi. (only daily rainfall and average daily discharge) Up to 5000 sq. mi. (extensive records)	Rainfall and Streamflow Records		Hydrograph	Flood Flows Major Storm System Storage Volumes
SCS UNIT HYDROGRAPH	Up to 20 sq. mi. if large watershed, break down to 20 sq. mi. sections	Soil Type Rainfall Hyetograph Time of Concentration	Runoff Curve No. (CN) Runoff (Q) inches ≥1.5 CN ≥60	Hydrograph	Flood Flow Minor and Major Storm Systems Storage Volumes
SCS TABULAR METHOD	Up to 20 sq. mi. if large watershed, break down to 20 sq. mi. sections	Soil Type 24 Hr. Cumulative Rainfall Time of Concentration	Runoff Curve No. (CN) Accounts for Hydrological Abstrations	Hydrograph	Flood Flows Major Storm System Storage Volumes
SCS GRAPHICAL METHOD	≤20 sq. mi.	Soil Type Cumulative Rainfall	Runoff Curve No. (CN) Runoff (Q) inches ≥1.5 CN ≥60	Peak Flow	Flood Peaks Minor and Major Storm Systems
COMPUTER MODELLING	Dependent on capacity of program	See Users Manual	See Users Manual	Hydrographs	Trouble Shooting Design of Minor and Major Storm Systems Storage Volumes
RATIONAL MASS INFLOW	≤20 Acres Spec. Rep. #43	Landcover IDF Curves	Runoff Coefficient (C)	Storage Volume	Detention and Infiltration Facility Design
SIMPLIFIED EQUIVALENT TRIANGULAR HYDROGRAPH	≤200 Acres	Soil Type Rainfall Hyetograph Time of Concentration	Runoff Curve No. (CN) Runoff (Q) inches ≥1.5 CN≥60	Hydrograph	Small Storage and Infiltration Facility Design

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C. DESIGN OF STORM WATER COLLECTION AND DISPOSAL SYSTEMS

1. Methods of Collecting Storm Water

Surface runoff can be collected at either a point or along a linear collector. The point collector can consist of a catch basin, inlet, small pond, or basin. A linear collector can be a swale, ditch, curb and gutter, or perforated or slotted pipe.

2. Methods of Disposal of Collected Storm Water

a. Positive Systems

Any system that conveys accumulated runoff directly to a stream, canal, river, lake, sea, or ocean is considered a positive system. These would include normal outfall systems such as underground pipes, box culverts, and open or covered trenches or ditches. Pipe sizing and design specifics of such systems are not within the scope of this manual. However, for detailed information refer to a hydraulics textbook or agency publication on storm drainage. A few of many references available on the subject are listed at the end of Chapter IV-B.

b. Infiltration Systems

There are three basic types of infiltration systems: basins, vertical wells or pits, and trenches. Each has a particular "best area of use", dependent upon situation and conditions.