STORMWATER MANAGEMENT MANUAL

OCTOBER 1981

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PREFACE

This volume of material is designed to present in a simplified manner some of the basic principles and designs of hydrology, hydraulics, and stormwater impacts as used in stormwater management. These materials are useful as a reference for the designer and analyser. The treatment is not exhaustive but specific to stormwater management within the State of Florida. The design and analysis concepts and procedures were compiled from work completed and monitored within the State. These concepts and procedures are appropriate for the Florida environment. However, other design procedures that are field tested for removal efficiencies (total and dissolved fractions) on appropriate pollution species should be incorporated into future volumes of this manual. In addition, other tested procedures are not included. A bibliography is provided for those interested in a more detailed treatment of the subject materials.

It is understood that the material may not cover all field situations encountered. Also, some of the material may not be used during the design process. However, the authors believe this work will increase the relative effectiveness of stormwater management, thus improving receiving water quality while not discouraging innovative designs.

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STORMWATER IMPACT

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INTRODUCTION

Urban rainfall picks up pollutants from the air, dusty roofs, littered and dirty streets, vehicle-related substances, corrosion products, hazardous spills, fertilizers, herbicides, insecticides, rodenticides, etc.

Characterization of urban stormwater discharges in terms of concentrations and pollutional loads provides useful indications of potential receiving water impacts. A manual of simplified methodology used to assess the impact of urban storm on the quality of receiving waters was developed by Driscoll, E.D. et. al (1979). This methodology is appropriate for use at the planning level where preliminary assessments are made to define the problem, establish the relative significance of contributing sources, assess feasibility of control and determine the need for additional evaluations.

Several studies were conducted nationwide to determine mass loadings due to urbanized stormwater runoff. Similar loadings were obtained from extensive studies conducted on Lake Eola drainage basin as shown in Table 1. From this table, it was obvious that a wide range of values existed for each parameter measured. Loading rate comparisons between measured values, predicted values, and national averages are shown in Table 2. Lake Eola watershed consisted mainly of commercial and residential areas located in downtown Orlando, Florida. It must be realized that these mass loadings will depend on many factors including antecedent dry period, land use, social and economic status, degree of urbanization and volume and type of traffic. Pollutants carried by stormwater runoff may be characterized as organic compounds, suspended solids, bacterial contaminants nutrients, and heavy metals.

Organic Compounds:

Organic compounds in urban stormwater include oxygen consuming material which can be represented by BOD loads, non-biodegradable organics which can be estimated from COD measurements, petroleum hydrocarbons, herbicides, pesticides and others.

TABLE 1

CONCENTRATION AND LOADING*
LAKE RUNOFF SUMMARY, LAKE EOLA, FLORIDA

(Hydrograph Related and Composite Sampling Programs)

Parameter	Number of Storms Sampled	Mass Loading Range (Kg/ha-yr)	Averages** Loadings - Kg/ha-yr	Concentration mg/l
Suspended Solids	14	470 - 2368	991	131
Volatile Suspended	7	234 - 610	538	71
NVSS	7	76 - 587	453	60
BOD_5	8	40 - 315	98	13
COD	6	130 - 1776	711	74
TOC	13	53 - 2572	946	99
TKN	10	10 - 87	32	3.3
Ammonia-N	12	0.2 - 10.4	4.1	0.43
Total Phosphorus	14	1.8 - 16.4	4.8	0.48
Zinc	9	1.2 - 5.5	3.7	0.38
Cadmium	9	0.09- 1.0	0.28	0.03
Arsenic	8	0.17- 1.76	1.02	0.11
Nickel	9	0.06- 0.54	0.28	0.03
Copper	9	0.12- 1.39	0.68	0.07
Magnesium	8	2.58- 31.25	9.86	1.03
Iron	9	2.9 - 16.46	9.52	0.99
Lead	9	1.1 - 9.5	4.26	0.44
Chromium	9	0.07- 0.51	0.25	0.03
Calcium	9	99.7 - 487	308	32.10

** Both Commercial and Residential
* After Wanielista, Yousef and Taylor (1981)

TABLE 2

LOADING RATE COMPARISONS

(Kg/ha-yr)

	SS	BOD_5	TOC	TN	TKN	PO_4	TP
LAKE EOLA							
Commercial	1076	196	1167	32.0	27.8	1.7	3.5
Residential	827	87	757	40.5	36.1	3.1	6.2
+SWMM/Level I							
Commercial	1255	181	-	16.7	-	4.3	-
Residential	922	45	-	7.4	-	1.9	-
++National Averages							
Commercial	941	97	-	14.5	-	3.0	-
Residential	470	39	-	6.6	-	2.0	-

+From (Heaney, 1976)

++From (Wanielista, 1979)

BOD loads will lower the levels of dissolved oxygen (DO) in receiving waterbodies which may destroy sensitive species of fish and aquatic organisms. Also, they may cause anaerobic conditions which produce objectionable end products. Field and Turkeltaub (1980) indicated that from 40 to 80 percent of the total organic loading entering receiving waters from a city is caused by sources other than treatment plants. Based on annual mass balance determinations, it was determined that urban wet weather organic loads are the same order of magnitude as dry-weather loads and 10 times greater through storm periods (Heaney, J. B., et al; 1977). The BOD concentrations in urban stormwater discharges are similar to those of secondary effluent. However, concentrations as high as 885 mg/l have been reported (Huber, et al., 1979; Lager, et al., 1977).

A large fraction of the oxygen consuming material may be associated with settleable and slowly biodegradable solids in stormwater discharges. Therefore the deoxygenation rate tends to be lower and time delayed. In some cases, two different effects on the DO concentrations in streams were observed by Jacobson, and others (1980) an immediate effect caused by degradation of the mainly soluble BOD fraction in the water body and a delayed effect caused by colloidal and suspended particulate matter. Furthermore, concentrations of oxygen consuming materials are usually unevenly distributed within a storm event with greater concentrations often occurring in the first portion of the storm known as the first-flush, which may result in shock effects damaging to aquatic populations.

Keefer, Simons and McQuivey (1980) in their study of stream dissolved oxygen concentrations showed an indication of the potential negative impacts from oxygen demanding material in urban storm runoff. In some cases, DO diurnal cycles disappeared when storm events occurred and flow increased. Also the minimum DO dropped from 1 - 1.5 mg/l below minimum values observed during steady flows and remained constant for periods ranging from 1-5 days. As the flow event subsided, the DO level resumed its cyclic behavior. Levels of 5 mg/l or less were not uncommon. Also, several streams in south Florida showed that the average DO concentrations during wet season are generally lower than the average concentrations during dry months (Yousef, Wanielista, et. al., 1976) as shown in Figures 1 and 2. Additionally, studies on Lake Eola indicate that concentrations of dissolved oxygen, although usually at or above saturation near the surface, drop periodically during the spring and summer months to one milligram per liter or less at depths of four meters or greater as presented in Figure 3. The oxygen depletion is attributed to organic loading associated with stormwater runoff, increased temperature and higher rate of respiration. It is reasonable to expect that deoxygenation and reoxygenration rates are affected by stormwater runoff flows to receiving streams.

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Steady state mathematical models are available and useful for determining the long term average concentrations of dissolved oxygen in the receiving water. However, this information may be insufficient for a complete evaluation since impacts leading to violations of receiving water standards may only occur during or immediately following storm events. A method is needed for estimating the variability of the receiving water response and frequency with which stormwater related problems occur.

Petroleum Hydrocarbons:

Urban runoff contributes a major portion of the petroleum hydrocarbon loads to fresh waters. For instance, urban runoff has been reported to be the major contributor of petroleum hydrocarbon to Lake Washington (Wakeham, 1977). Concentrations ranging from 200 to 7,500 μg/l with a mean value of 1,200 μg/l of total aliphatic hydrocarbons in urban stormwater runoff were reported. Byrne et al, (1980) also reported total aliphatic hydrocarbons of 36 to 5000 μg/l in stormwater runoff from Meginnis Arm, Lake Jackson, Florida. The Lake Washington study detected contaminant concentrations in sediments (Tomlinson et. al., 1980).

Hunter et al. (1979) studied five storm events and the associated urban stormwater runoff from a watershed in Philadelphia, Pennsylvania. The average total hydrocarbon concentration was 3.69 mg/l with 82% of the total associated with particulate matter. His value for the average total disolved hydrocarbon concentration was 400 μg/l; an order of magnitude greater than the values reported by Byrne et. al. 1980. The total particulate hydrocarbon concentrations peaked dramatically at the initial surge of runoff,

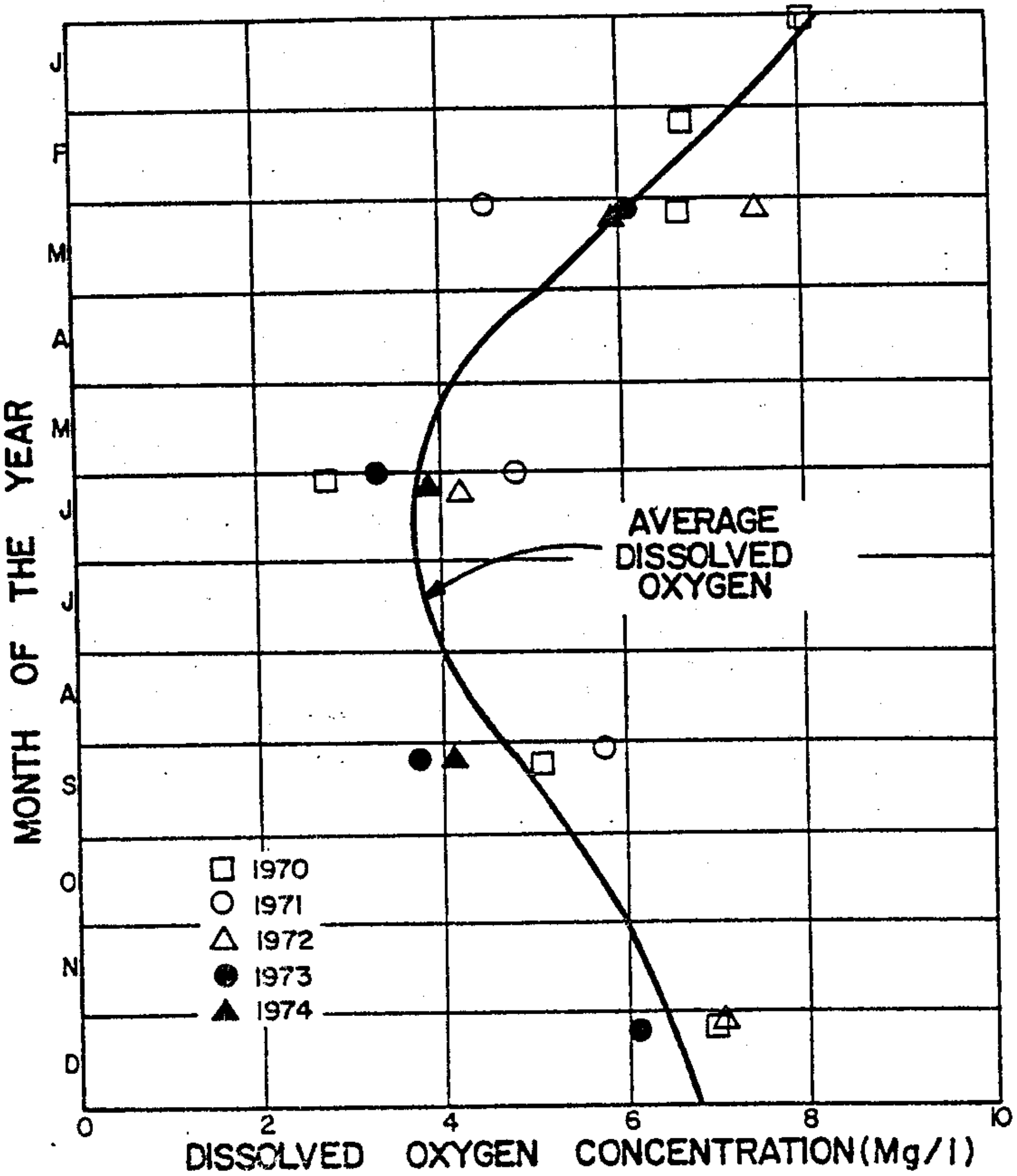

FIGURE 1. CHANGES IN DISSOLVED OXYGEN CONCENTRATION AT STATION WN 14 BOWLEES CREEK IN FLORIDA.

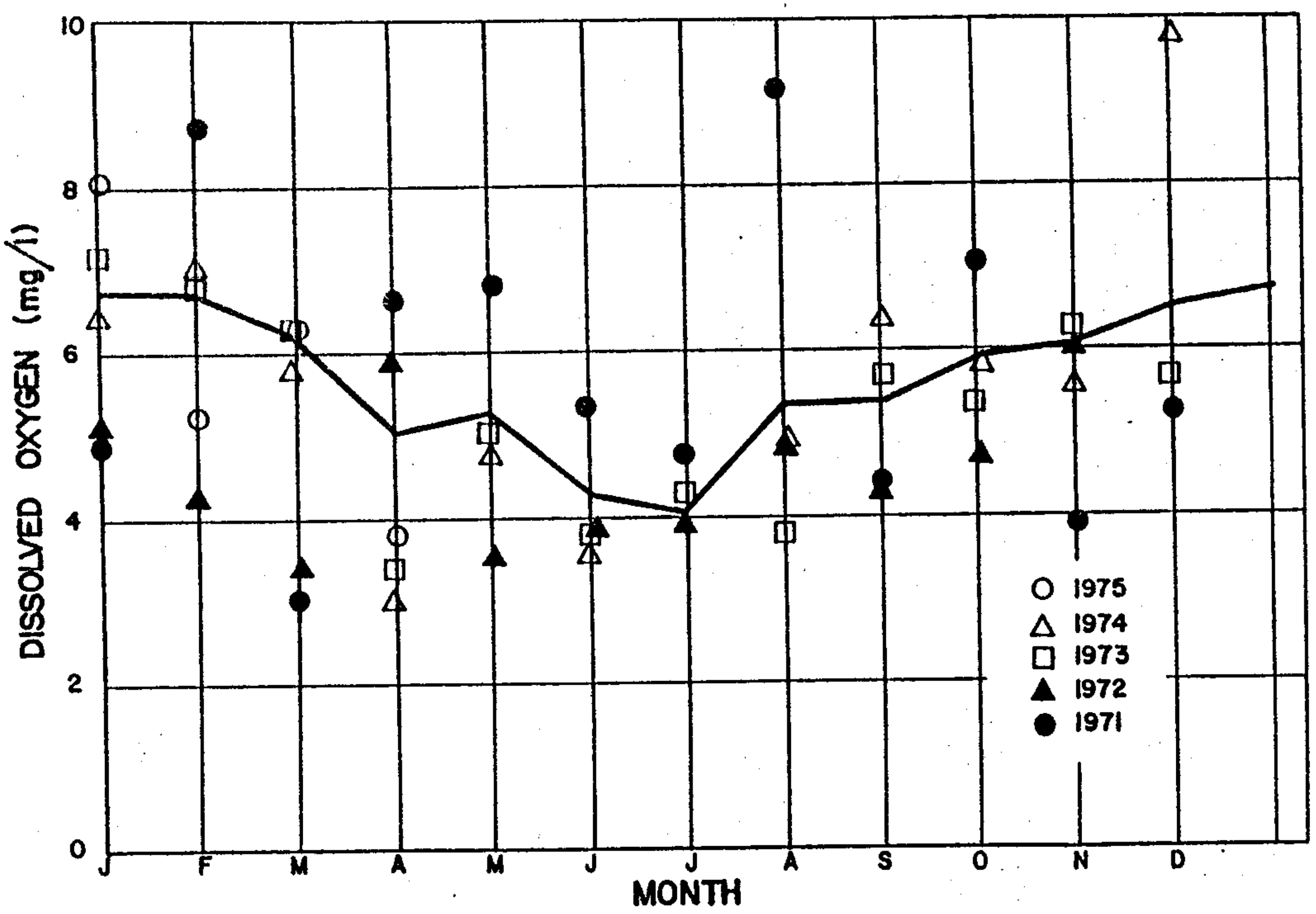

FIGURE 2. CHANGES IN DISSOLVED OXYGEN CONCENTRATION AT STATION 391 ON MANATEE RIVER IN FLORIDA.

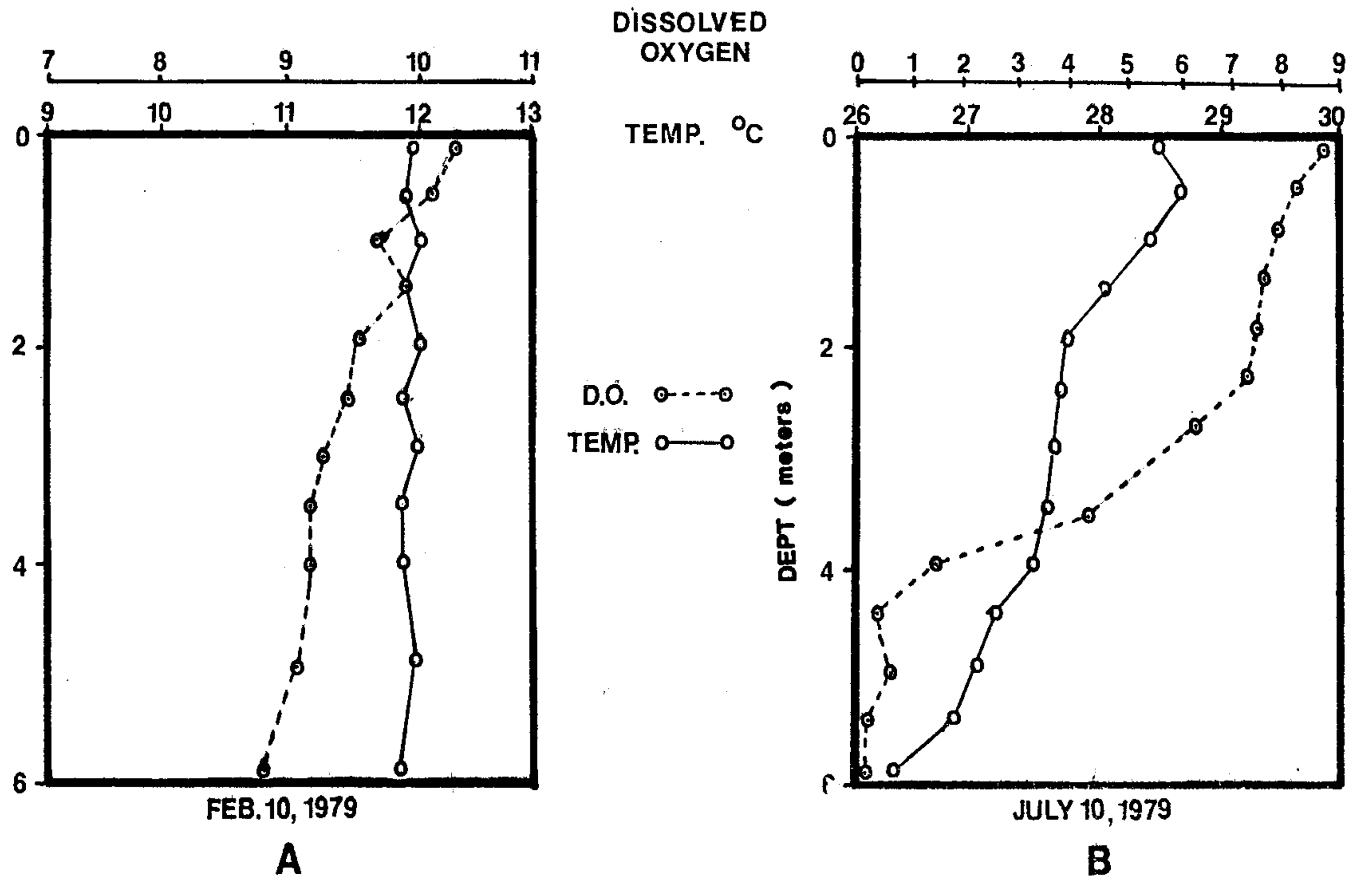

FIGURE 3. TYPICAL TEMPERATURE AND DISSOLVED OXYGEN PROFILES FOR LAKE EOLA, ORLANDO, FL.

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diminished, and peaked a second time later in the storm event. The total dissolved hydrocarbon concentrations were initially low and peaked just before or just at the secondary peaking of the total particulate hydrocarbon.

Petroleum hydrocarbons, particularly the polynuclear aromatic hydrocarbons (PAH's), have been shown to be carcinogenic and mutagenic in mammalian and microbial systems. There is also evidence that PAH's produce cancerous growth in some aquatic invertebrates and vertebrates. Various PAH's are on the EPA priority pollutant list (Ammon and Field, 1980). Hunter et al (1979), showed that aliphatic hydrocarbon represented two thirds of the total petroleum hydrocarbons while the more toxic aromatic represented the remainder.

Nutrients

Nutrients present in urban stormwater runoff, particularly nitrogen and phosphorus, could cause significant water quality deterioration in receiving water bodies. Surface water bodies with long detention times, such as lakes and estuaries tend to concentrate nutrients and other pollutants in both the water column and bottom muds. These pollutants can be resuspended and become available to plant growth when anoxic conditions and favorable environment exist. In Lake Eola, Florida, urban runoff was found to be the sole source of lake degradation. Concentrations and loading rates discharged to Lake Eola as presented in Table 1 indicate an average total Kjeldahl nitrogen (TKN) loading of 32 kg/ha-yr and an average total phosphorus loading of 4.8 kg/ha-yr. Phosphorus concentrations in the runoff were found to significantly increase algal productivity.

Control of flow of limiting nutrient is essential if it is desired to control the process of eutrophication. The limiting nutrient in a water body or a segment of the water body can be determined by measuring the available nitrogen, phosphorus and other elements during the period of maximum phytoplankton biomass. The available nitrogen concentrations are generally the nitrates plus ammonia nitrogen. However, the available phosphorus concentration is generally equivalent to the soluble orthophosphorus plus a fraction (0.2 or 0.3) of the particulate phosphorus content during the period of maximum period of biomass. Lee and Jones (1980) indicated that if the available P concentration is reduced to a few µg/l the phytoplankton growth at the time the samples were collected was most likely limited by P. If the available N concentrations are reduced to about 30 to 50 µg/l or so, N is likely to be the limiting nutrient.

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Several methods are used to determine the limiting nutrients. The ratio of available N to available P can be used to indicate the potential limiting nutrient. Theoretically the uptake weight ratio of these nutrients by algae is 7.5N to 1 P. If N:P is greater than or equal to 10, the limiting nutrient is most likely to be phosphorus. If N:P is less than or equal to 5:1, the limiting nutrient is most likely to be nitrogen. In between these ranges the limiting nutrient can be either one. Also, algal assays are used to estimate the limiting nutrient which is not likely to promote algal growth. Further analysis for available nitrogen and phosphorus during peak biomass production should be conducted along with the bioassay in order to verify that one of these nutrients is actually limiting. If neither phosphorus nor nitrogen concentration is reduced during the period of maximum summer phytoplankton biomass, some other factor such as light or micro-nutrient may limit the algal growth.

Fate of Nutrients in Lake Eola:

Accurate estimates of pollutant loadings should reflect the dynamic nature of the system and require extensive and continuous recording of the water budget and pollutant concentrations. Pollutants are added at different rates during storm events and little, if any, is known about their fate in receiving water bodies. Therefore, simplified models were developed to assses pollutant loadings and effects on the quality of the receiving stream or lake. Kothandaraman and Evans (1979) tested Rand Lake in Illinois for quantity-quality correlations using a method developed by Simmons in 1976 for U.S.G.S. Nitrogen and Phosphorus among other pollutants were estimated using their procedure and verified with a daily sampling scheme. Scheider, et al. (1977) performed a hydraulic and phosphorus budget on Harp Lake in southern Ontario. Combining discrete phosphorus concentrations with continuous stream discharges yielded the best estimate of stream phosphorus imputs.

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Estimates of pollutional loadings based on average concentration in runoff water for several storm events and calculated annual stormwater quantity from Lake Eola drainage basin were calculated by Wanielista, Yousef, and McLellon (1977) and Wanielista, Yousef and Taylor (1980). These estimates did not consider the fate and availability of pollutants to biological communities in the lake. Quantities present in solution and fractions of those in suspension are readily available to plant and animal life. However, quantities retained by the bottom sediments may be locked into it and could become available when the source is depleted from the water body and the environmental conditions are favorable.

A study was conducted for one full year by Yousef, Wanielista, et. al (1981) to determine responses of Lake Eola water to pollutional loadings, particularly phosphorus and nitrogen. Routine water samples from six different locations were collected, and analyzed. Table 3 also showed the ratio of available nitrogen to available phosphorus. Available nitrogen was estimated at 70 percent of total nitrogen and available phosphorus was estimated by the orthophosphorus concentration plus 30 percent of the difference between total phosphorus and orthophosphorus (Cowen and Lee, 1976). The values of available N:P varied between 8.0 and 52.1 and averaged 21.5. These data suggest that Lake Eola is phosphorus limited most of the time. Also, higher N:P ratios existed during wet weather months when compared with dry weather months.

Also Harper, Yousef and Wanielista (1980) concluded from bioassay studies that Lake Eola water is phosphorus limited when concentrations were less than 60 μg-P/l. It appears that Lake Eola is phosphorus limited most of the time.

Physicochemical and biological processes in Lake Eola determine the fate of nutrients associated with stormwater discharged into the lake. Nutrients may remain in solution or suspension, settle to the bottom, chemically interact and precipitate out or adsorb to sediments, plants and other surfaces. Some of the nutrients may be

Table 3. Averages of Total Nitrogen and Phosphorus Concentrations in Lake Eola Water

Sampling Date	Average of Six Samples: Total Nitrogen μg-N/1	Total Phosphorus μg-P/1	Total N:P Ratio	Available N:P Ratio
1/22/80	355.2	72	4.9	9.3
2/12/80	229.1	52	4.4	8.0
4/07/80	716.8	65	11	23.3
5/28/80	774.4	89	8.7	16.8
6/24/80	552	93	5.9	10.6
7/08/80	634.9	69	9.2	15.3
7/29/80	1053.7	61	17.3	32.5
8/12/80	608.2	42	14.5	32.7
8/26/80	534.7	51	10.5	20.3
10/9/80	211.2	19	10.1	20.8
10/30/80	547.0	35	15.6	52.1
11/20/80	351.9	51	5.7	16.3

Table 4. Retention of Nutrients Released to Lake Eola in Stormwater Runoff by Bottom Sediments

Nutrient Specie	Measured Runoff Loadings (Kg)	Runoff ** Volume (1000 cubic meters)	Stormwater* Average Concentration (mg/1)	Estimated Stormwater Mass Loadings (Kg)	R %
Total Phosphorus P	31.1	487	0.48	233.8	86.8
Ortho Phosphorus P	5.4	487	0.24	116.9	93.3
NO_3^{-2}-N	68.8	487	0.65	316.6	85.8
TKN-N	279	487	3.30	1607	77.6

* From Wanielista, Yousef and Taylor (1981)
** Mean Hydraulic Residence Time is Approximately 8 Months.

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released back to solution. Lake Eola was found to be a large retention pond. Most of the nitrogen and phosphorus were retained in the lake and only small fractions were discharged through drainage well. Also, some of the nitrogen may have been lost through the process of denitrification. Assuming the lake was completely mixed, monthly estimates of the changes in the mass of nutrients in solution and suspension within Lake Eola water were made and the annual increase in the mass of various forms of nutrients were based on water quantities and the corresponding average concentrations.

Sediment retention for total phosphorus, orthophosphorus, nitrates, and total kjeldahl nitrogen on an annual basis were estimated to be 86.8, 93.3, 85.8, 77.6 percent, respectively, during 1980, as shown in Table 4. These calculations were based on average concentrations for similar nutrients in stormwater runoff as presented by Wanielista, Yousef and Tyalor (1980). Development of a dynamic model to simulate Lake Eola was beyond the scope of this study.

Trophic State of Lakes

The trophic state of a lake is usually defined in terms of the degree of eutrophication that a lake displays. Lake eutrophication, in turn, is defined as the conditions which are associated with increased productivity. Eutrophication occurs when productivity is accelerated above the rate that would naturally have occurred in the absence of perturbations of the lake system. The most obvious effect of accelerated productivity is the increase of algal photosynthesis per unit area. Bioassay experiments were designed to determine the impact of stormwater runoff on algal productivity in Lake Eola water (Harper, Yousef and Wanielista, 1980) as shown in Figure 4. It appears that mixing lake water with various proportions of untreated stormwater increased algal cell dry weight more than five fold over the control tests. However, when stormwater runoff was coagulated and the phosphorus content was reduced, there was no sizeable increase in algal productivity. Because phosphorus is considered the limiting growth nutrient in Lake Eola, changes in the phosphorus input to the lake are manifested in productivity changes.

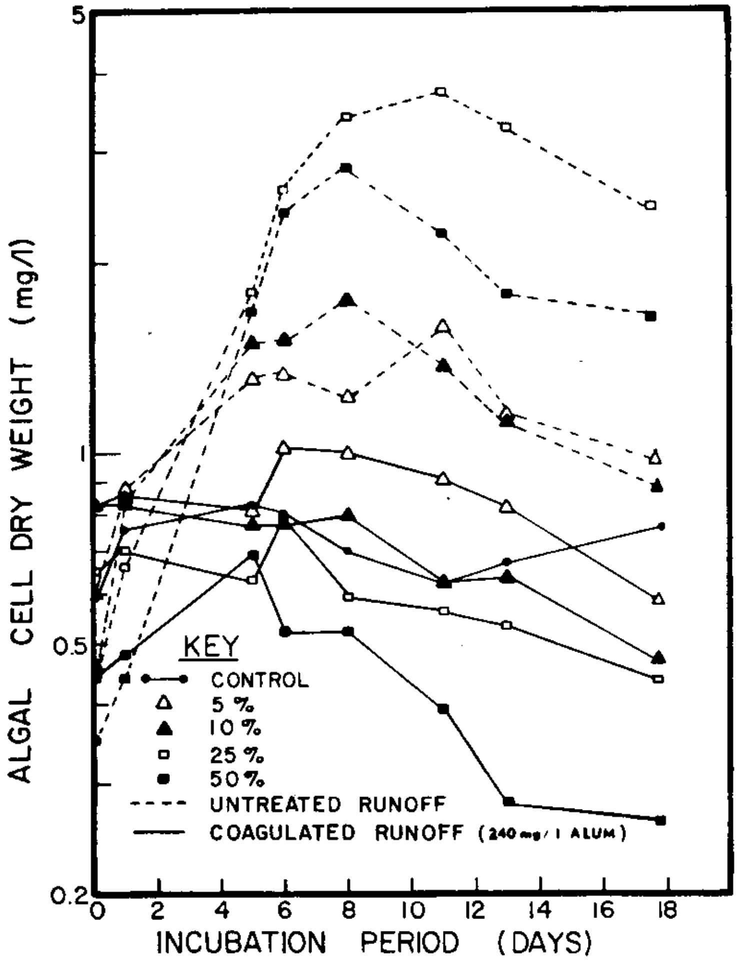

FIGURE 4. . RESPONSES OF INDIGENOUS **ALGAL SPECIES** IN LAKE EOLA TO VARIOUS CONCENTRATIONS OF STORMWATER RUNOFF AND COAGULATED RUNOFF (WATER SAMPLES COLLECTED 4-30-79, STORMWATER COLLECTED 4-28-79).

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Several models to assess the eutrophic state of a lake are based on average annual phosphorus loading, hydraulic residence time and sediment retention. These models are presented here to show the relations that can be developed between phosphorus and trophic state. The models chosen include the Vollenweider model, Dillon model, Larsen-Mercier model, the Shannon-Brezonik Trophic State Index and the Florida Department of Environmental Regulation Model. The first three models - Vollenweider, Dillon and Larsen-Mercier were derived from the same basic conservation of mass balance equation using phosphorus loadings as a parameter. These models were developed using data from northern United States or European Lakes. On the other hand, the Shannon-Brezonik Trophic State Index is a number obtained from substituting measurable values of chemical and biological significance into a multivariate equation developed through regression analysis. Models selected for a geographic area must be calibrated and verified. The trophic state determined (estimated) by various models may not be the same.

Vollenweider (1968) pioneered the use of nutrient loadings to determine the trophic state of a lake. He derived equations in two measurable unknowns to obtain limits of allowable loadings that are the difference between oligotrophic, mesotrophic, and eutrophic states. Vollenweider's parameters are the areal phosphorus loading rates and the ratio of the mean depth to the hydraulic residence time. The hydraulic residence time is defined as the volume of the lake divided by the annual inflow resulting in a dimension of years as shown in Figure 5.

Dillon used the same basic input-output mass balance equation that Vollenweider has used, but introduced a nutrient retention coefficient (R) that is defined as the fraction of incoming phosphorus that does not flow out of the lake. This coefficient is incorporated into a phosphorus loading parameter, $L(1-R)$, with L equal to phosphorus loading in g/sqm/yr and ρ equal to the flushing per year as shown in Figure 6. The flushing per year is simply the reciprocal of the Vollenweider hydraulic retention time. The nutrient retention coefficient, R, has been correlated to the morphological characteristics of a lake and can be estimated.

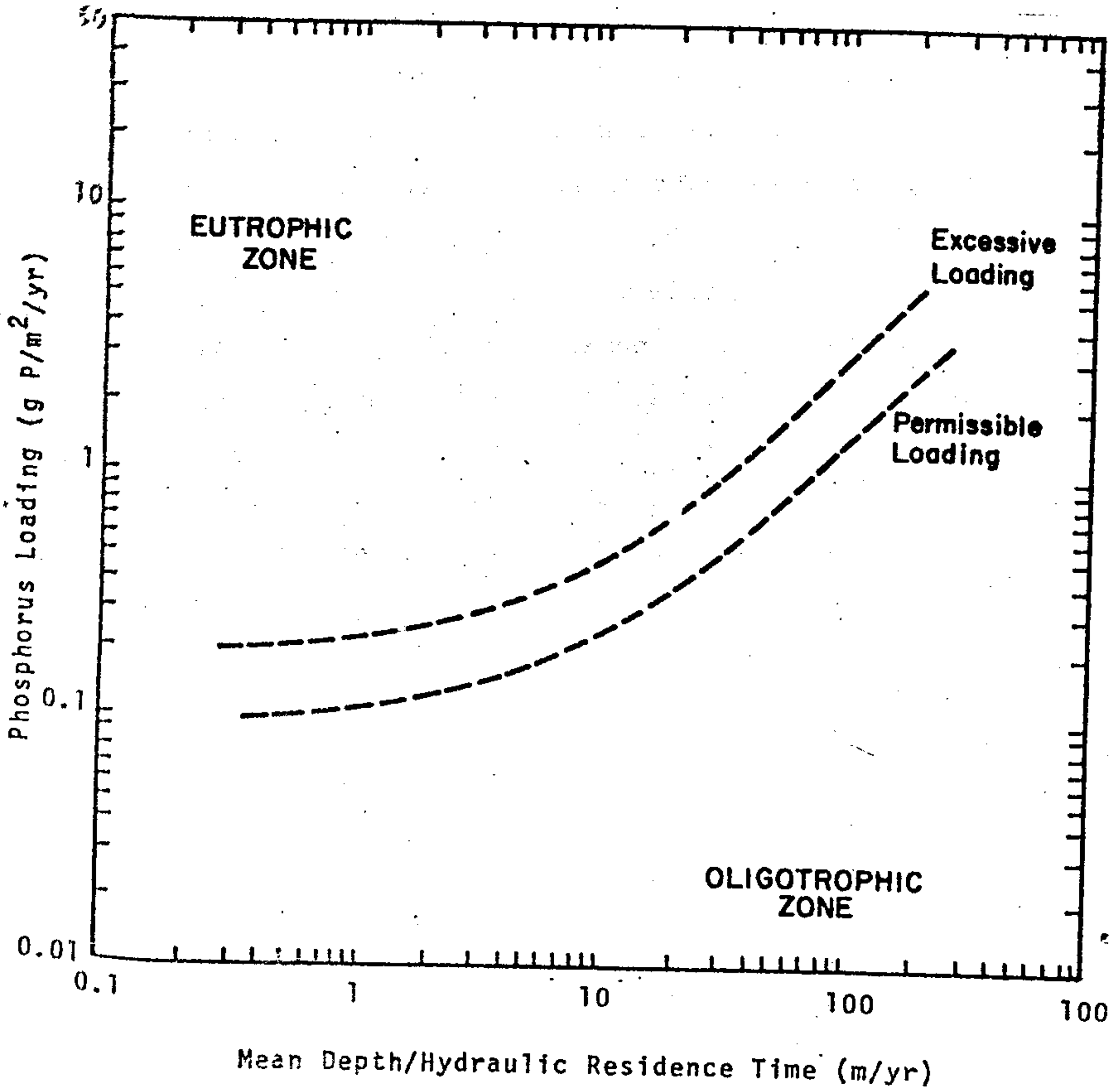

Figure 5. Vollenweider Phosphorus Loading Curve

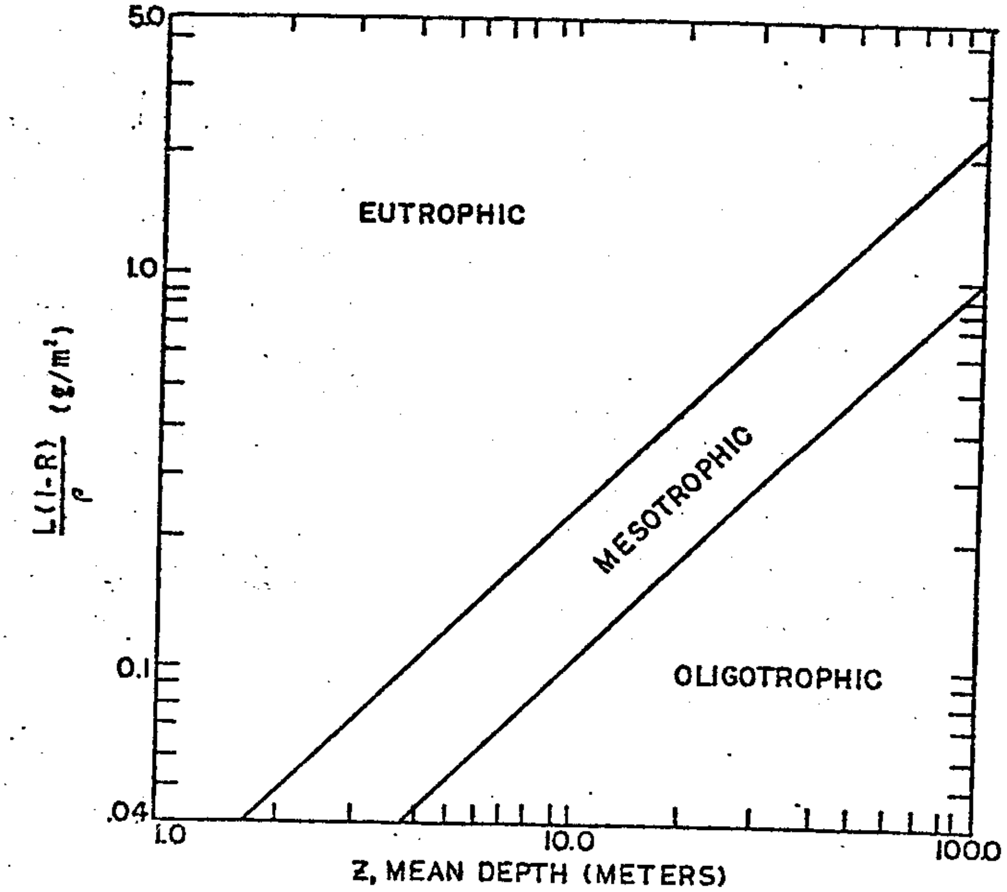

FIGURE 6. TROPHIC STATE INDEX FOR LAKE EOLA. DILLON MODEL

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The Larsen-Mercier model is similar to Dillon and was developed using the same balance equation used by Vollenweider and Dillon. Larsen and Mercier (1975) developed a graphical relationship between average incoming phosphorus concentration and the phosphorus retention coefficent. They used data obtained from 73 lakes to compare their parameters to known trophic states. Their research showed that eutrophic lakes plotted above a line of constant phosphorus concentration equal to 0.02 mg/1 while oliogotrophic lakes fell in the area below 0.01 mg/1 as shown in Figure 7.

Shannon-Brezonik (1972) Trophic State Index (TSI) was developed using data obtained from Florida lakes. They derived a function of several critical parameters as shown in the following equation:

$$TSI = 0.18\ T + 0.008\ CD + 1.1\ TN + 4.2\ TP + 0.01\ PP + 0.044\ CL + 0.39\ CR + 0.26$$

Where T = Turbidity JTU

CD = Conductivity, μmho/cm

TN = Total Organic Nitrogen, mg/1-N

TP = Total Phosphorus, mg/1-P

PP = Primary Productivity μg C/1-hr

CL = Chlorophyll a, μg/1

$$CR = \frac{(Ca) + (Mg)}{(Na) + (K)}$$

The assigned values for each trophic classification ranged from 0 to 3.0 for oligotrophic, from 3.0 to 7.0 for mesotrophic, from 7.0 to 10.0 for eutrophic, and greater than 10 for hypereutrophic lakes.

An empirical model was developed by J. Hand (1977) for Florida Department of Environmental Regulation using data from the National Eutrophication Survey. The model was adapted to predict total phosphorus concentration from the nutrient loading rates, morphometric, and hydrological data on the lakes typical of Florida

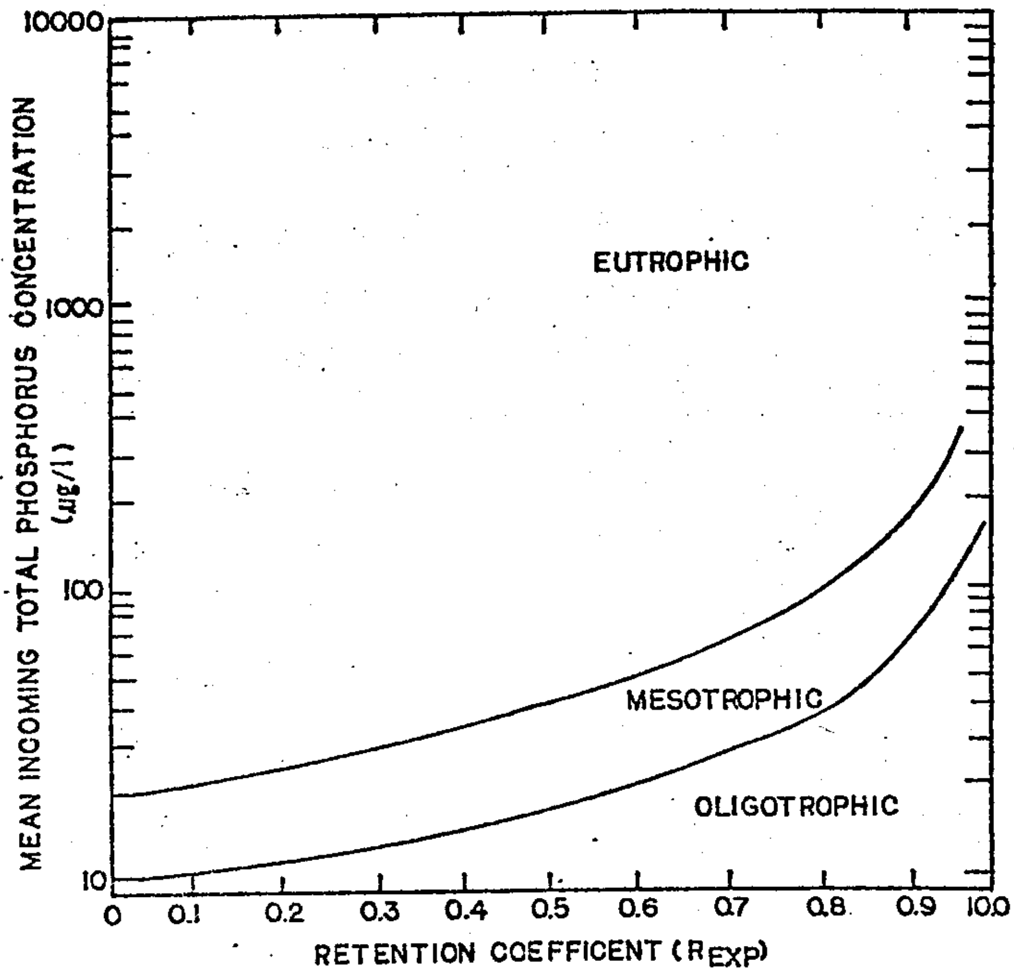

FIGURE 7. TROPHIC STATE INDEX FOR LAKE EOLA
LARSEN MERCIER MODEL

environment. Total phosphorus content is given by $TP = (1-R)\ L_p/Q$ where:

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TP : Predicted phosphorus concentration (mg/1.)

$R = 0.482 - 0.257 \log \frac{Q}{V}$

L_p : Areal phosphorus loading rate (g/m^2 - yr)

Q : Lake outflow (m^3/yr.), V = Lake Volume (m^3)

R : Nutrient retention coefficient

Also, $TN = (1-R) \times L_n/Q$ where TN is total nitrogen

The $CHL\ A = 35.95 \left(\frac{TN}{3} + TP\right) \sqrt{\frac{\text{Shape Factor}}{\text{Depth}}}$

where $\text{Shape Factor} = \frac{L}{W} = \frac{\text{length of the lake}}{\text{width of the lake}}$

This model has been tested by comparing the predicted lake nutrient's concentration with measured samples. The correlation coefficient between measured and predicted nutrients showed a fairly good correlation with $r = 0.7$. The lakes were considered oligotrophic, mesotrophic and eutrophic if the estimated chlorophyll a concentrations were below 10, 10 - 20 and 20 - 30 μg/1 respectively.

Nutrient Load-Eutrophication Response Relationships:

Rast and Lee (1978) developed for the Organization for Economic Cooperation and Development response relationships. Phosphorus P load to a waterbody normalized by the water body's mean depth and hydraulic residence time was related to summer planktonic algal chlorophyll concentrations, planktonic algal-related water clarity as measured by average summer secchi depth, and hypolimnetic oxygen depletion rate for those waterbodies which are thermally stratified, (see Figure 8). This approach is applicable to those waterbodies whose maximum phytoplankton biomass is limited by phosphorus. The OECD organization has sponsored a study of 200 waterbodies (Lake and Impoundment in 22 countries) to determine the relationships between nutrient

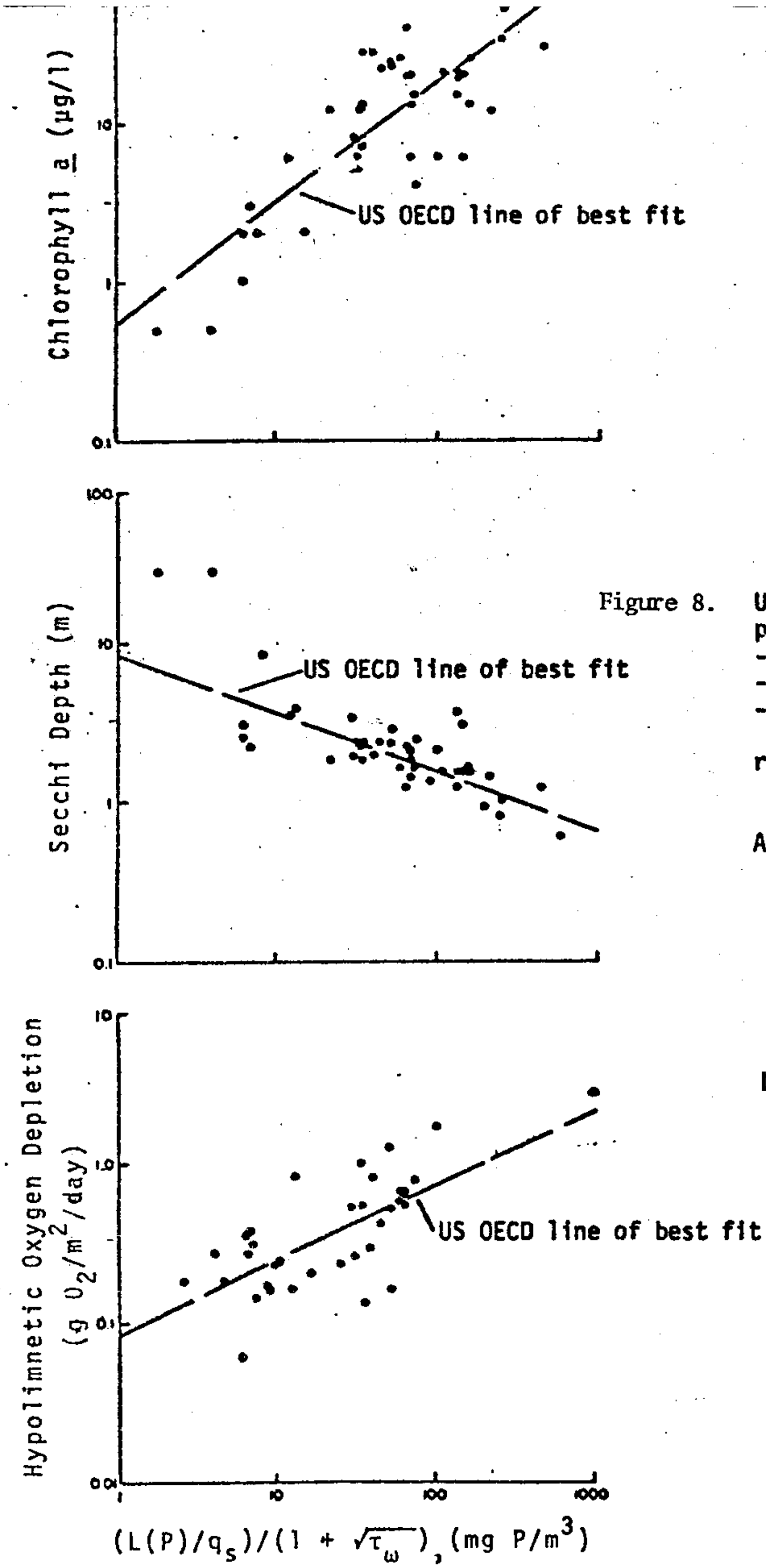

Figure 8. US OECD data applied to phosphorus load
-mean chlorophyll a,
-mean Secchi depth, and
-hypolimnetic oxygen depletion rate relationships

After Lee *et al.* (1978)

KEY

$L(P)$ = areal annual phosphorus load ($mg\ P/m^2/yr$)

q_s = mean depth ÷ hydraulic residence time (m/yr)

τ_ω = hydraulic residence time (yr)

load to a waterbody and the eutrophication response. Lee and his associates (1980) tested 60 waterbodies primarily in the United States & they were found to agree with the relationships developed for OECD.

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Toxicity

Toxicity problems can result from minute discharges of metals, pesticides and persistent organics, which may exhibit a subtle long-term effect on the environment by gradual bioconcentration. A major source of heavy metals in urban stormwater runoff is vehicle related. Other sources of heavy metals includes atmospheric particulate fallout and washout from industrial stacks, soil erosion, chemical spills, pesticides and vegetative material. Of greatest concern are those toxic metals at low concentrations at relatively short time of exposure to a variety of aquatic species or to man. There are 13 heavy metals including: antimony, arsenic, beryllium, cadminum, chromium, copper, lead, mercury, nickel, selenium, silver, thallium and zinc. Some of these metals may be released at toxic levels in urban wet-weather discharges. Comparisons of concentration ranges of these metals for urban stormwater runoff are shown in Figure 9 (Ammon and Field, 1980). Many of these metals are associated with particulate matter; therefore, the bottom accumulation will occur in low velocity regions and impacts will occur in the sediment.

Metal accumulation in the sediments has been investigated by Wanielista, Yousef and Christopher. (1980). Lead concentrations detected in areas of Lake Ivanhoe bottom sediments located beneath bridge scupper drains were much higher than other areas of the lake. Average metal enrichments in sediments in the Saddle River upstream and downstream of Lodi, New Jersey for Cd, Cr, Cu, Pb, Ni and Zn was estimated as 5.2, 5.1, 3.1, 6.7, 2.8 and 3.8 respectively (Ammon and Field, 1980).

Table 1 shows concentration and loading of selected heavy metals from Lake Eola drainage basin. Also, Figure 9 shows concentration ranges for As, Cd, Cr, Cu, Pb, Hg, Ni and Zn in urban stormwater as reported by Ammon and Field (1980). Typical reported average urban

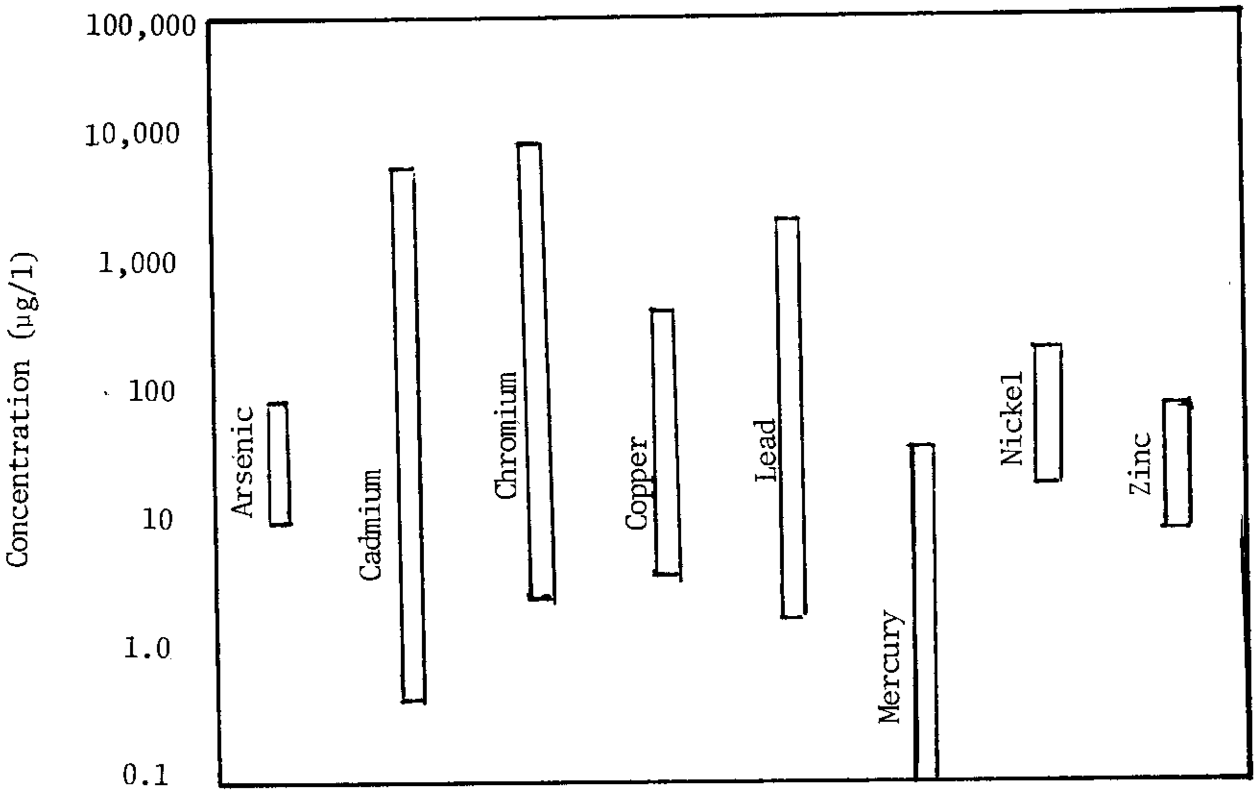

FIGURE 9. Concentrations Ranges of Heavy Metals in Urban Stormwater Discharges

Compiled from information in Ammon and Field (Reference 1).

Compiled from information
in Ammon and Field (Reference 1)

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stormwater concentrations in micrograms per liter for Cd, Cr, Cu, Pb, Hg, Ni, and Zinc are 53, 170, 80, 160, 2.2, 180 and 110 respectively. Bioaccumulation of lead and zinc in the organisms from Coyote Creek was 100 to 500 times greater than water column concentrations. Lead and zinc concentrations in urban samples of algae, crawfish and cattails were two to three times greater than non-urban samples.

Extensive analysis for urban stormwater additional impacts and case studies were presented at a conference held November 26-28, 1979 at Orlando, Florida and proceedings were published by the U. S. Environmental Protection Agency and edited by Yousef, Wanielista, McLellon and Taylor (1980).

Example of Calculation for Trophic State of Lake Eola:

Lake Eola drainage basin consists of 31.6 ha. Commercial, 23.4 ha residential, 4.5 ha Parkland and the lake area is 11 ha (Wanielista, et. al., 1981). Average loading rates from Table 2 were used to determine mass loadings from various sources for selected pollutants as shown in Table 5.

The total estimated phosphorus loading = 291* Kg P/yr

*Notice that calculated mass loading for total phosphorus from Table 4 was 233.8 Kg/yr. However, 1980 was exceptionally dry and rainfall was much lower than the average.

$$\text{Areal phosphorus loading} = \frac{291000}{110000}\ \frac{\text{g P/yr}}{\text{m}^2} = 2.65\ \text{gP/m}^2\ \text{yr}$$

Mean depth of the lake = 3.1 meter, mean hydraulic residence time ≈ 0.67 yr, and the phosphorus retention coefficient ≈ 0.87.

Using Figures 5, 6 and 7 to determine the trophic state of Lake Eola by Vollenweider, Dillon and Larsen-Mercier models, it was found that the lake is classified as eutrophic by all these models tested.

TABLE 5

SOURCE COMPARISONS

USING LAKE EOLA DATA

(Kg/yr)

LAND USE	SS	BOD_5	TOC	TN	TKN	PO_4	TP
31.6 ha Commercial	34,001	6,193	36,877	1,011	878	54	110
23.4 ha Residential	19,351	2,035	17,713	948	849	73	145
+4.5 ha Park Land	600	93	408	*	39	*	8
++11.0 ha Lake	1,634	175	309	*	18	*	6
+++Ducks	*	180	*	*	*	*	21
TOTALS	55,586	8,676	55,307	1,959	1,779	127	291

* No estimate

\+ Parkland overflow into lake assumed to occur 6 times/year or when storm volume exceeds 1.50 inches

++ Rainfall data (Wanielista, 1979)

+++ Assuming a BOD_5 and phosphorus loading of 3.4 and 0.4 g/kg-day, transient duck population is resident 4 months/year, and ducks deposit all waste material directly in the lake. These are liberal assumptions.

Vollenweider model relates the areal phosphorus loading of 2.65 g $P/m^2/yr$ and mean depth divided by hydraulic residence time which is $3.1 \div 0.67 = 4.63$ m/yr.

Dillon model relates phosphorus loading parameters

$$\frac{L(1-R)}{\rho} = \frac{2.65\ (1-0.87)}{1/0.67} = 0.23\ g/m^2 \text{ and}$$

the mean depth 3.1 meters

Larsen Mercier Model relates mean incoming total phosphorus concentration which is 0.48 mg/l or 480 μg/l and retention coefficient of 0.87.

Shannon and Brezonik:

$$TSI = 0.18\ T + 0.008\ CD + 1.1\ TN + 4.2\ TP + 0.01\ PP + 0.044\ CL + 0.39\ CR + 0.26$$

Where T = Turbidity JTU

CD = Conductivity, μmho/cm

TN = Total Organic Nitrogen, mg/l-N

TP = Total Phosphorus, mg/l-P

PP = Primary Productivity μg C/l-hr

CL = Chlorophyll a, μg/l , and

$$CR = \frac{(Ca) + (Mg)}{(Na) + (K)}$$

For Lake Eola

T ≃ 4 JTU

CD ≃ 240 μmho/cm

TN ≃ 0.8 mg/L

TP ≃ 0.06 mg/l

PP ≃ 30 μgC/l-hr

NOTES

NOTES

Ch ≃ 30 μg/L

CR ≃ 2

TSI = 0.18 X 4 + 0.008 X 240

+ 1.1 X 0.8 + 4.2 X 0.06 +

0.01 X 30 + 0.044 X 30 + 0.39 X 2

+ 0.26

= 0.72 + 1.92 + 0.88 + 0.25

+ 0.3 + 1.32 + 0.78 + 0.26 = 6.43

0 - 3 ≃ Oligotrophic, 3 - 7 ≃

Mesotrophic, 7 - 10 Eutrophic

1- ≃ Hyperuetrophic

J. Hand Model Or DER:

$R = 0.482 - 0.257 \text{ Log } \frac{Q}{V}$

$V = 330{,}000 \text{ M}^3$

$Q = 487000 - 330000 = 187{,}080 \text{ M}^3$

$R = 0.482 - 0.257 \text{ Log } \frac{187}{330}$

$= 0.482 + 0.247$

$\simeq 0.729$

$TP = (1 - R)\ {}^{L}P/Q$

$TP = (1 - 0.729)\ \frac{291000 \text{ gm/yr}}{187000 \text{ m}^3}$

$= 0.42 \text{ mg/L}$

$TN = (1 - R)\ L_n/Q$

$TN = (1 - 0.729)\ \frac{1959000 \text{ gm/yr}}{187080 \text{ m}^3}$

$= 2.8 \text{ mg/L}$

NOTES

$$CHLA = 35.95 \left(\frac{TN}{3} + TP \right) \frac{shape}{depth}$$

Where shape $= \frac{L}{W} = 1.45$

$$CHLA = 35.95 \left(\frac{2.8}{3} + 0.42\right) \frac{1.45}{3.10}$$

$= 33$ μg/1 chlorophyll $\underline{a}$

> 20 mg/m^3

<u>Lake is Eutrophic</u>

Below 10 μg/L ≃ oligotrophic

10 - 20 μg/L ≃ Mesotrophic

20 - 30 μg/L ≃ eutrophic

The trophic state of the lake can be improved to oligotrophic if the phosphorus loading was lowered below 0.15 g/m^2 yr as shown from Figure 5, or 94% removal of incoming phosphorus which may delay recovery of the lake for a period of time.

Using the OECD relationship, the Chlorophyll a, Secchi disc and Hypolimnetic oxygen depletion rates can be estimated as follows in Table 6.

The independent parameters $\left(\frac{L(P)}{q_s}\right)\left(\frac{1}{1 + \tau_\omega}\right)$

$$= \frac{L(P)}{3.1 \div 0.67} \times \frac{1}{1 + 0.67} = 0.12\ L(P)$$

TABLE 6. Changes in Water Quality Parameters In Lake Eola As Predicted by OECD Relationships

Parameters	Phosphorus Loading Rate mg P/m^2 yr	
	2650 (Existing)	150 (Predicted)
Chlorophyll *a* μg/l)	30	5
Secchi Depth (meter)	1.0	3
Hypolimnetic O_2 depletion (g/m^2 day)	1.0	0.3

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SECTION 2

HYDROLOGY FOR STORMWATER MANAGEMENT

NOTES

Purpose: The purpose is to develop principles of precipitation, storage, and runoff as related to stormwater management. These principles will be applied to develop methods for estimation of rainfall excess. Rainfall excess is used to size retention and detention facilities.

Hydrologic Cycle. Available freshwater on earth comprises approximately 3% of the total supply. However, 90% is found in the ice caps. In addition, other fresh water is found in remote (distant from population centers) locations or is polluted for human and wildlife use. Water is stored on earth in four basic locations; atmosphere, surface storage, plants, and groundwater (soils). Transition from one storage place to another is common. Precipitation removes water from the atmosphere, evapotranspiration returns water to the atmosphere from plants, surface waters, and soils. Runoff waters transport water from one surface location to other storage areas. Figure 1 illustrates these transformations in a schematic commonly called the Hydrologic Cycle. From these concepts, one realizes that a balance among the storage components results, or for a given watershed.

$$\text{IN} - \text{OUT} = \text{Storage Change} \qquad (1)$$

or $P - R = \Delta S$ (depth or volume)

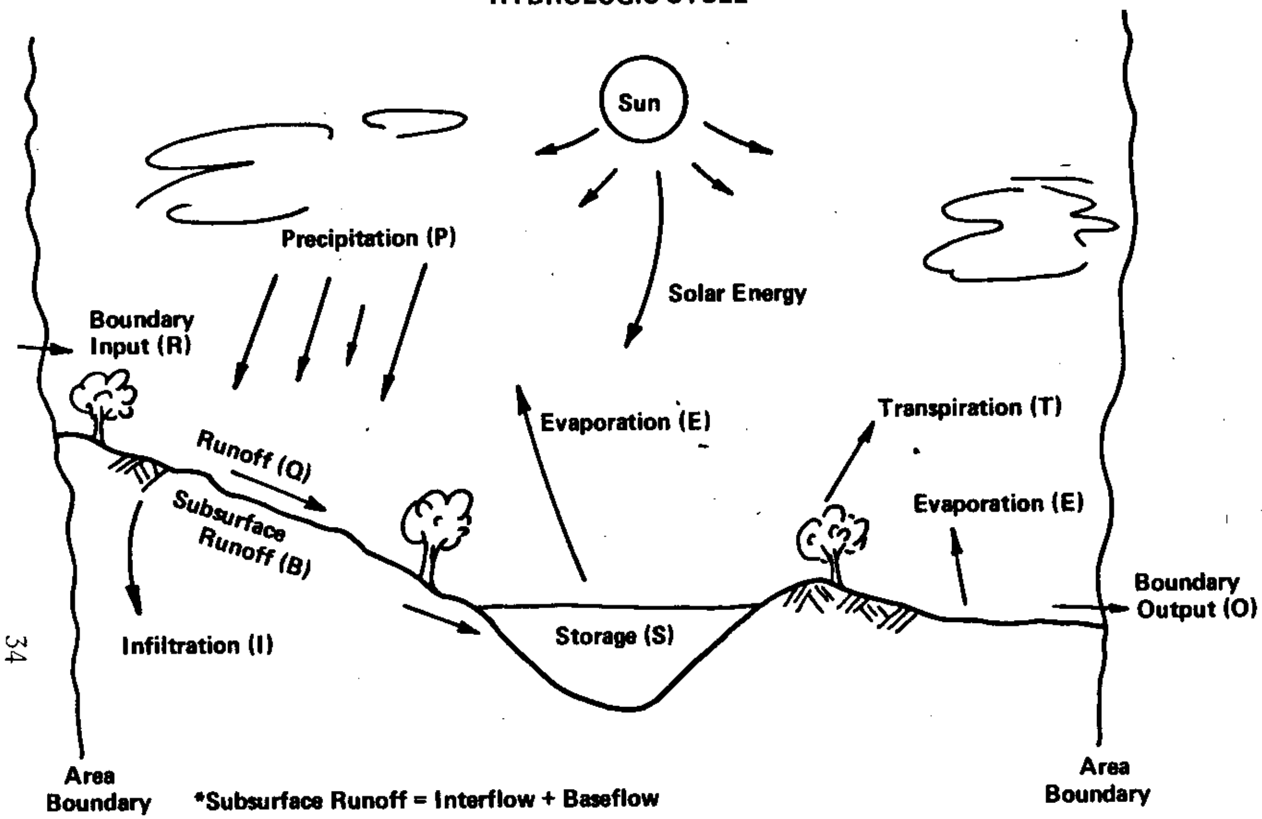
where: P = Precipitation

R = Rainfall Excess

ΔS = Storage change

Rainfall excess is that water available for transport to storage areas within a watershed. The watershed is defined by topographic features in the area of study.

Precipitation: Rainfall is the form of precipitation discussed in this work. It is important to understand the variability of rainfall as related to rainfall excess and storage. Rainfall events occur with different intensities (in/hr.), volumes (inches), and duration (hours). In addition, the geographic distribution is variable from one area to another. This variability on a



Surface Inventory Equation

Beginning Storage

S_o

\+ Inputs

\+ (P + R + B)

\- Outputs

\- (I + T + E + O)

= Ending Storage

S (I - 1)

Where Q (runoff) is considered part of the storage.

FIGURE 1. Hydrologic Cycle

monthly basis is shown in Figure 2. Over a long period of time (years) the variability tends toward a definite volume which occurs at various frequencies. These frequencies are classified in terms of duration and yearly return periods. As an example, see Figure 3 at the end of this section. The rainfall volume of a storm of specific return period and duration produces an estimate for volume of rainfall for a geographic area.

NOTES

Example Problem: For Leon County, Florida, one must calculate the precipitation volume for the one in 2-year storm with duration of one hour. From Figure 3, zone 1 chart, the rainfall is estimated at 2.3 inches (see pages 57-68).

How does this compare with Manatee County? From Figure 3, zone 6 chart the rainfall is estimated at 2.6 inches.

In West Palm Beach, the design specified is 3 years, 1 hour. What is the volume of rainfall? From Figures 3 zone 10 chart, one estimates 3.0 inches

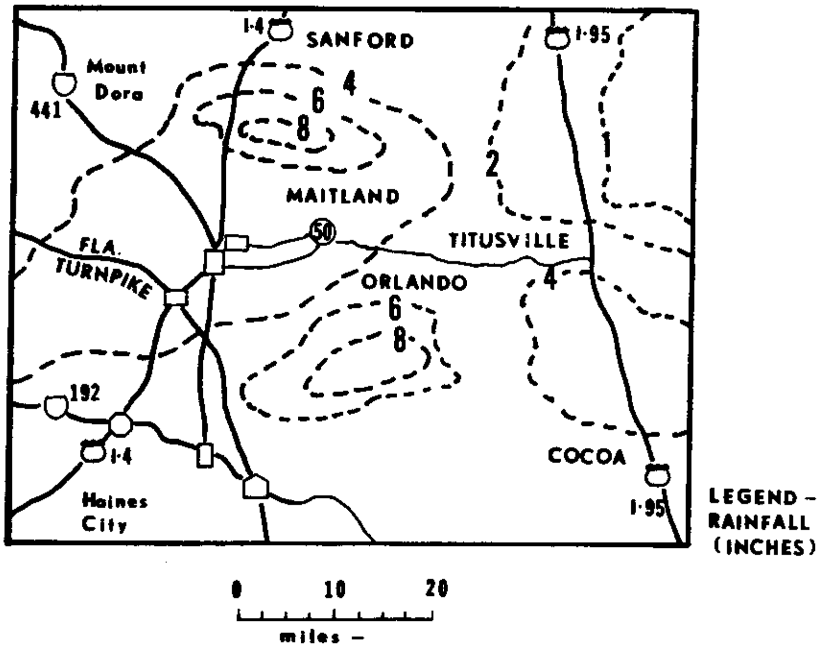


Figure 2. Example Distribution of Monthly Precipitation

NOTES

From the above examples, it is noted that the larger volume of rainfall was chosen when there was doubt as to exact locations. These low frequency, short duration storms are used primarily for calculating pollution control detention and retention facilities. The longer frequency storms (10 years, to 100 years) are used for flood control and or hydraulic sizing of facilities.

Example Problem: As another example compare the results of the 3 year - 1 hour storm for the different regions of Florida

Region	Volume (in.)	Region	Volume (in.)
1	2.6	7	2.7
2	2.5	8	2.8
3	2.5	9	2.8
4	2.4	10	3.0
5	2.6	11	2.5
6	2.6		

From the previous example, it is noted that the maximum storm volume is 3.0 inches. The frequency at which this storm volume is exceeded in any one year is very small or approximately 1 percent. The average number of storms per year in Florida is approximately 121. A storm is defined by rainfall separated by no more than four hour periods of no rainfall. Thus on the average only one storm per year exceeds 3". The mean volume of those storms which exceed 3 inches is approximately 4.5 inches. See Figure 4 for a typical frequency distribution on storm volume for Florida.

The distribution of rainfall volume for a storm duration also is important to determine rainfall excess, the peak rate of discharge (volume per unit time), and other stormwater management procedures. A low intensity storm has more of a chance to percolate into the ground relative to a high intensity storm, thus creating different rainfall excess condition. Rainfall intensity distributions are referred to as hyetographs or a plot of volume per time period over the duration of a storm. Dimensionless graphs as shown in Figure 5 are useful for estimating rainfall volume as a function of time. These dimensionless graphs were calculated from high volume storm events and are thus useful for flood control or hydraulic calculations.

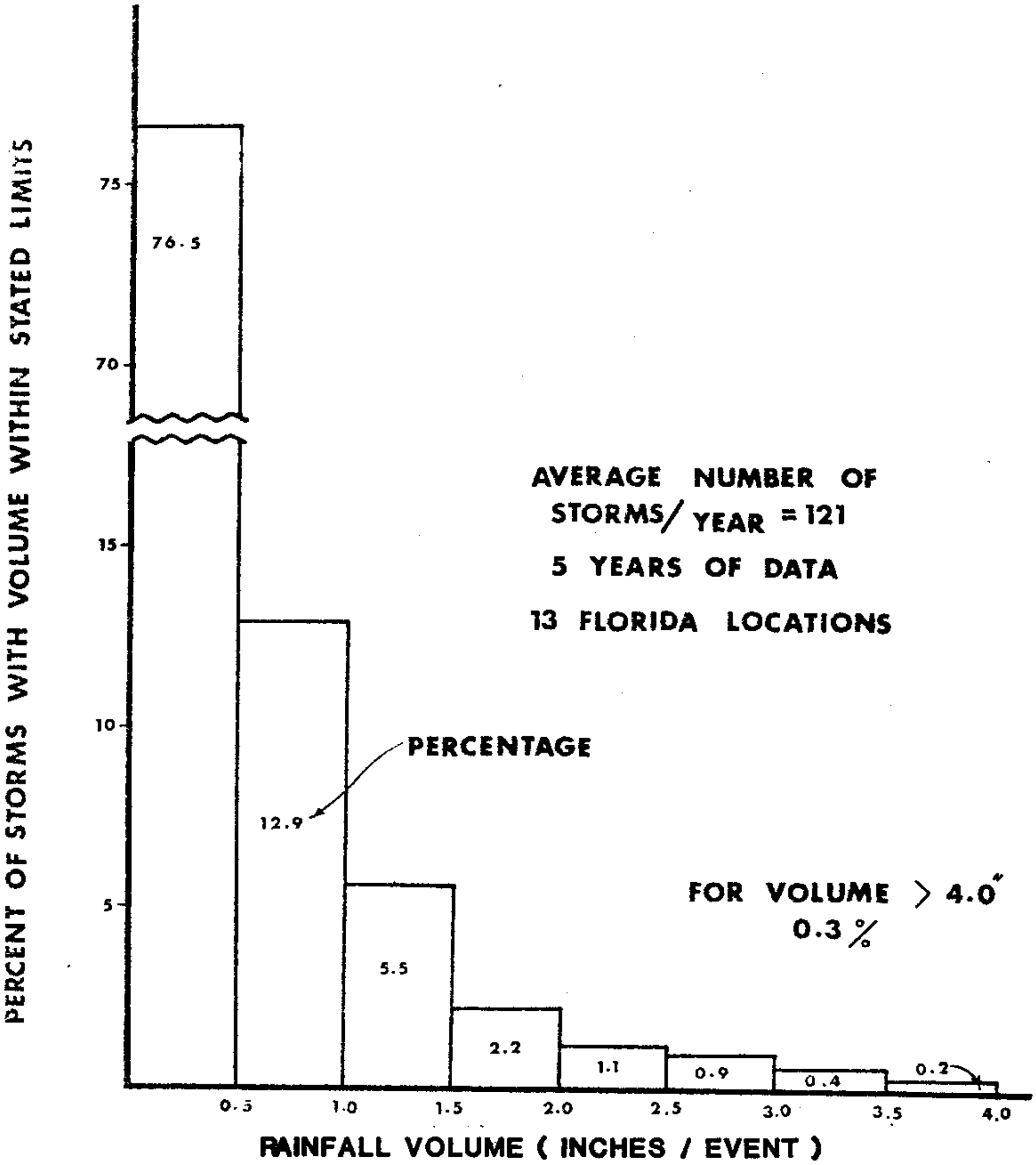
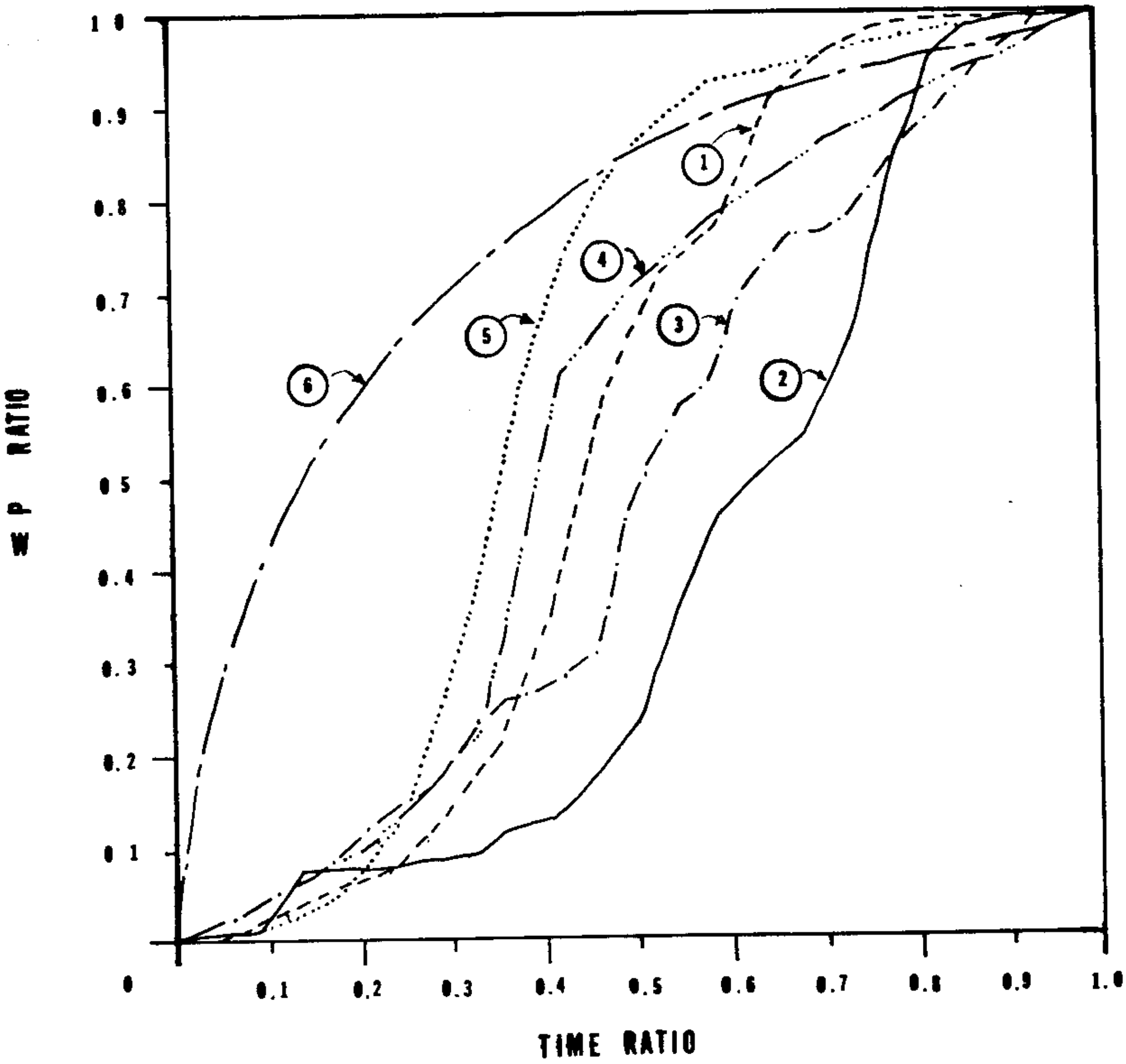


FIGURE 4. DISTRIBUTION OF RAINFALL VOLUME FOR 13 LOCATIONS IN FLORIDA OVER A 5-YEAR PERIOD.



1. **Orlando W.B. Airport, March 15, 1960**
2. **Orange City, September 10-11, 1960**
3. **Orlando W.B. Airport, October 15-16, 1956**
4. **SCS Curve**
5. **Orange County**
6. **Weather Bureau**

FIGURE 5. Dimensionless Mass Curves for Rainfall

NOTES

Example Problem: Given a 6 hour, 6 inch storm, what is the distribution of rainfall using the SCS Curve Method. Notice that the "X" axis in Figure 6 is dimensionless and one must determine "real" time. Consider a 15 minute interval with a six hour storm and calculate the time first, such as 3 hours is an 0.5 time ratio. Then read the cumulative precipitation ratio to produce a volume. At 3 hours, using the SCS curve, the cumulative precipitation ratio is 0.7, thus the cumulative rainfall is (0.7) (6.0) = 4.2 inches. The results using 15 minute intervals are shown in Table 1.

Storage: After precipitation, water is either stored in the watershed or becomes available for runoff (rainfall excess). Storage changes with time during a storm event and in general is illustrated in Figure 6. When the watershed surface storage and sub-surface storage is filled, rainfall excess additions are equal to precipitation additions. Various methods have been used to estimate maximum storage. One such method was derived from Figure 6 by the Soil Conservation Service (SCS) assuming that the initial abstraction (surface storage) was equal to 20% of total storage. From Figure 6, a ratio results.

$$\frac{S}{S'} = \frac{R}{P} \qquad (2)$$

and from equation (1) $S = P - R$

Thus: $$\frac{P-R}{S'} = \frac{R}{P}$$

or $$R = \frac{P^2}{P + S'}$$

adjusting for initial abstraction

$$R = \frac{(P-0.2S')^2}{P + 0.8S'} \qquad (3)$$

Where S' is estimated from the soil conditions and the surface cover. Using more than 3,000 soil types, the SCS developed runoff curve numbers (CN) as shown in Table 2.

TABLE 1

DESIGN RAINFALL
25 YEAR FREQUENCY, 6 HOUR DURATION, 15 MINUTE INCREMENTS

Time Minutes	Time Hours	ΣP Inches	ΔP Inches
0	0	0	0
15	.25	.10	.10
30	.50	.21	.11
45	.75	.33	.12
60	1.00	.48	.15
75	1.25	.64	.16
90	1.50	.81	.17
105	1.75	1.08	.27
120	2.00	1.38	.30
135	2.25	2.46	1.08
150	2.50	3.60	1.14
165	2.75	3.90	.30
180	3.00	4.20	.30
195	3.25	4.44	.24
210	3.50	4.68	.24
225	3.75	4.86	.18
240	4.00	5.01	.15
255	4.25	5.16	.15
270	4.50	5.28	.12
285	4.75	5.40	.12
300	5.00	5.52	.12
315	5.25	5.64	.12
330	5.50	5.76	.12
345	5.75	5.88	.12
360	6.00	6.00	.12

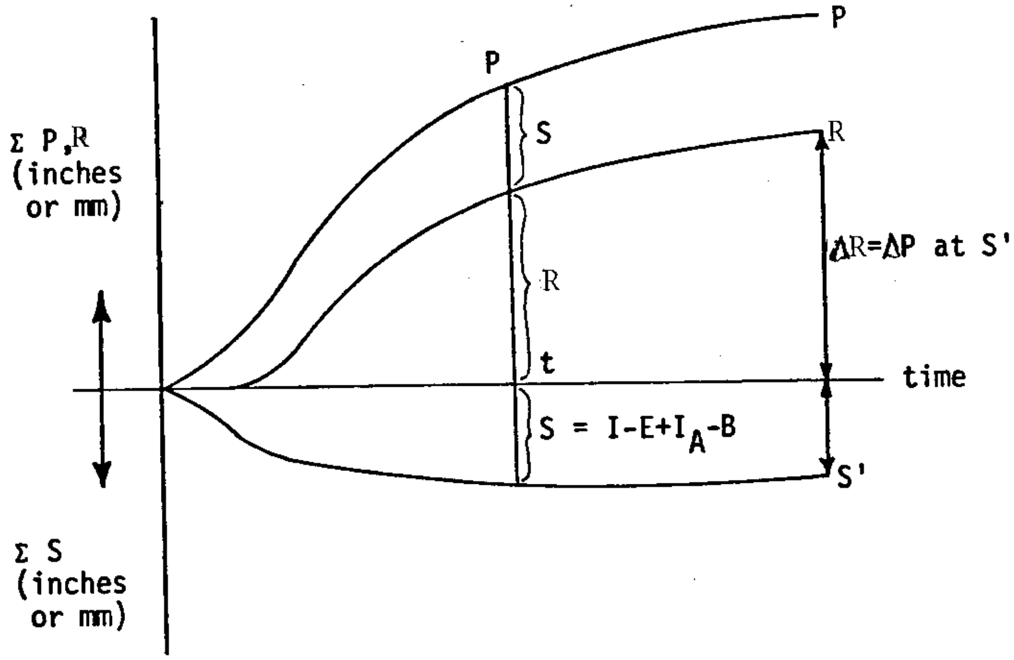


FIGURE 6. TIME VARIABILITY OF RAINFALL EVENTS.

Table 2. Curve Numbers for Selected Land Uses (Antecedent moisture condition 2, and $I_a = 0.2S'$)[f]

Land Use Description	Hydrologic Soil Group A	B	C	D
Cultivated Land[a]				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or Range Land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow				
Good condition	30	58	71	78
Wood or Forest Land				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover[b]	25	55	70	77
Open Spaces, Lawns, Parks, Golf Courses, Cemeteries, etc.				
Good condition, grass cover on 75% or more of the area	39	61	74	80
Fair condition, Grass cover on 50% of the area	49	69	79	84
Commercial and Business Areas (85% impervious)	89	92	94	95
Industrial Districts (72% impervious)	81	88	91	93
Residential[c]				
Average Lot Size (ac) Average % impervious[d]				
≤ 1/8 — 65	77	85	90	92
1/4 — 38	61	75	83	87
1/3 — 30	57	72	81	86
1/2 — 25	54	70	80	85
1 — 20	51	68	79	84
Paved Parking Lots, Roofs, Driveways, Etc[e]	98	98	98	98
Streets and Roads				
Paved with curbs and storm sewers[e]	98	98	98	98
Gravel or paved with swales	76	85	89	91
Dirt	72	82	87	89

[a]For a more detailed description of agricultural land-use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9 (August 1972).
[b]Good cover is protected from grazing and litter and brush cover soil
[c]Curve numbers are computed assuming the runoff from the house and driveway is directed toward the street with a minimum of roof water directed to lawns where additional infiltration could occur.
[d]The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.
[e]In some warmer climates of the country a curve number of 95 may be used.
[f]Kent, K. M. "A Method for Estimating Volume and Rate of Runoff in Small Watersheds", USDA, SCS, TP-149 (April 1973).

NOTES

From these curve numbers, the maximum soil storage can be estimated by:

$$S' = \frac{1000}{CN} - 10 \qquad (4)$$

Example Problem:

Using the SCS Curve Number (CN) procedure estimate the maximum soil storage for a residential area with a CN number of 57.

$$S' = \frac{1000}{57} - 10 = 7.54 \text{ inches}$$

As noted, the CN number is a function of soil type. Soils have been classified as to their runoff potential. This classification is shown below in Table 3.

TABLE 3
SCS Hydrologic Soil Groups*

Soil Group	Potential
A	Lowest Runoff Potential, deep sands and rapidly drained gravel.
B	Moderately Low Runoff Potential. Above average infiltration soils after wetting.
C	Moderately High Runoff Potential, Shallow soils containg clay and collo ids.
D	Highest Runoff Potential, mostly clays, or those with nearly impermeable subhorizons near the surface.

*A/D - Dual classifications refer to drained/ undrained conditions.

Example Problem:

If the residential development of the previous example problem was in A/D type soils, what is the difference in storage capacity if the soil is drained?

D type: $S' = \frac{1000}{86} - 10 = 1.63$ inches

Difference: 7.54 - 1.63 = 5.91 inches

NOTES

Another estimation for subsurface storage was provided by the South Florida Water Management District. This estimate is for sandy soils generally found within the South Florida Water Management District. Depth to the water table is added as a constraint to calculating storage or storage is a function of depth to the water table. In addition, construction activities have been known to compact the soil and thus reduce the storage capacity. Estimates for storage capacity are:

Depth to Water Table (feet)	Non-Compacted Cumulative Water Storage (inches)	Compacted Cumulative Water Storage (inches)
1	0.60	0.45
2	2.50	1.88
3	6.60	4.95
4	10.90	8.18

Also used to estimate soil storage is Horton's equation which is an expression of infiltration rate versus time. In Figure 7 the rate of infiltration decreases with time. Infiltration using a double-ring infiltrometer indicated initial rates of as high as 60 inches/hour and limiting rates as low as 1 inch/hour for "A" type soils of central and south Florida. "D" type soils of Florida indicated initial infiltration rates as high as 5.9 inches/hour and limiting rates of 0.25 inches/hour. Some hard pan (clay) soils have near zero infiltration rates.

Rainfall Excess: Once the storage has been estimated, rainfall excess or that available for runoff can be estimated. Using the SCS-CN method, one can estimate rainfall excess using equation 3, or a simple hydrologic balance as in equation 1.

Example Problem: For a residential area of 30% impervious cover, in A type soil, what is the rainfall excess from a 9" storm event?

Equation 1. From the previous examples, the storage was estimated as 7.54 inches, thus the rainfall excess is:

$$R = P - S'$$

$$R = 9.00 - 7.54 = 1.46 \text{ inches}$$

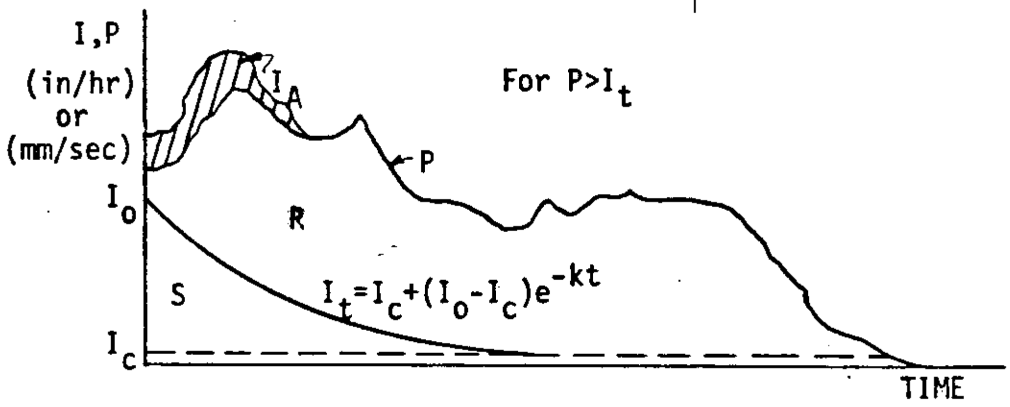


FIGURE 7. INFILTRATION RATE

NOTES

Another procedure for estimating rainfall excess is the use of the rational or runoff coefficients. From field collected data, the quantity of runoff is estimated and related to rainfall quantity, thus

$$R = CP \qquad (5)$$

Where: C = runoff coefficient

Runoff coefficients have been estimated for various land uses and are shown in Table 4. For rainfall conditions with 25 to 100 year return periods, these coefficients are most likely too low and should be increased. Again experience must be used and designs generally use average values for pollution control and larger values for flood control.

Example Problem: What is the rainfall excess for the previous example problem using equation 5?

The area is 30% impervious. Assume most of the rainfall on the impervious area will go into surface storage or runoff (rainfall excess). The runoff coefficient is assumed equal to 0.40 (see Table 4 for average values).

$$R = CP$$

$$R = 0.40\ (9) = 3.6 \text{ inches}$$

For flood control and hydraulic design a larger runoff coefficient, say 0.50 would be used.

For storms of low return periods (5 years or less), the use of the SCS-CN procedures underestimate the rainfall excess for most situations (Wanielista, 1979).

Example Problem: For a 3.0 inch storm volume estimate the rainfall excess from the residential site used in previous examples. Use the rational method and the SCS-CN method, with CN = 57, C = 0.20 (less than 5 year return)

a) SCS-CN $R = (P-0.2S')^2/P+0.8S'$

$$R = (3-0.2(7.54))^2/3+0.8(7.54)$$

$$R = 0.25 \text{ inches}$$

Table 4 Runoff Coefficients[a,b]

Description of Area	Runoff Coefficients	Character of Surface	Runoff Coefficients
Business		Pavement	
Downtown	0.70 to 0.95	Asphalt or concrete	0.70 to 0.95
Neighborhood	0.50 to 0.70	Brick	0.70 to 0.85
Residential		Roofs	0.70 to 0.95
Single family	0.30 to 0.50	Lawns, Sandy Soil	
Multiunits, detached	0.40 to 0.60	Flat, 2%	0.05 to 0.10
Multiunits, attached	0.60 to 0.75	Average, 2-7%	0.10 to 0.15
Residential, suburban	0.25 to 0.40	Steep, 7% or more	0.15 to 0.20
Apartment	0.50 to 0.70	Lawns, Heavy Soil	
Industrial		Flat, 2%	0.13 to 0.17
Light	0.50 to 0.80	Average, 2-7%	0.18 to 0.22
Heavy	0.60 to 0.90	Steep, 7% or more	0.25 to 0.35
Parks, Cemeteries	0.10 to 0.25		
Railroad Yard	0.20 to 0.35		
Unimproved	0.10 to 0.30		

[a]The coefficients in these two tabulations are only applicable for storms of 5- to 10-year return frequencies and were originally developed when many streets were uncurbed and drainage was conveyed in roadside swales.

For recurrence intervals longer than 10 years, the indicated runoff coefficients should be increased, assuming that nearly all of the rainfall in excess of that expected from the 10-year recurrence interval rainfall will become runoff and should be accommodated by an increased runoff coefficient.

The runoff coefficients indicated for different soil conditions reflect runoff behavior shortly after initial construction. With the passage of time, the runoff behavior of sandy soil areas will tend to approach that of heavy soil areas. If the designer's interest is long term, the reduced response indicated for sandy soil areas should be disregarded.

[b]From Design and Construction of Sanitary and Storm Sewers. ACSE Manual of Practice No. 37, 1970. Revised by D. Earl Jones, Jr.

NOTES

b) Rational: $R = CP$

$R = 0.20(3) = 0.60$ inches

By comparison: the SCS-CN method does underestimate rainfall excess for low return period storms, if CN and C values are accurate.

Runoff: The rainfall excess is routed to a discharge point to estimate the rate of flow, both peak rate and time variability. A picture of flow rate with time is called a hydrograph. A typical hydrograph resulting from a uniform hyetograph is shown in Figure 8. Most hydraulic designs are done using an estimation for the peak discharge. In addition, detention systems are designed using the overall shape and peak of the hydrograph. The runoff rate (Q) is calculated from the rainfall excess. Peak discharge (Qp) can be related to rainfall excess.

For small impervious watersheds (<50 acres) the rational formula has been used to estimate the shape and peak of discharges. The estimation formula is:

$$Q_p = CIA \qquad (6)$$

Where Q_p = peak discharge (cfs),

I = rainfall intensity (in hr)

A = area (acres)

For a constant rainfall intensity over the duration of the storm, the resulting hydrograph will be approximately triangular with base equal to 2 times the time of concentration and height equal to peak discharge. This is shown in Figure 9. Time of concentration is generally expressed as the time it takes a particle to travel from the furthest most point in the watershed to the discharge point.

Methods to estimate the time of concentration rely on hydraulic principles and empirical equations. Figures 10 and 11 illustrate two empirical methods for estimating time of travel.

Unit hydrographs, contributing area and the SCS dimensionless curves are other methods for hydrograph estimation. These will be discussed in the Management Practices Section.

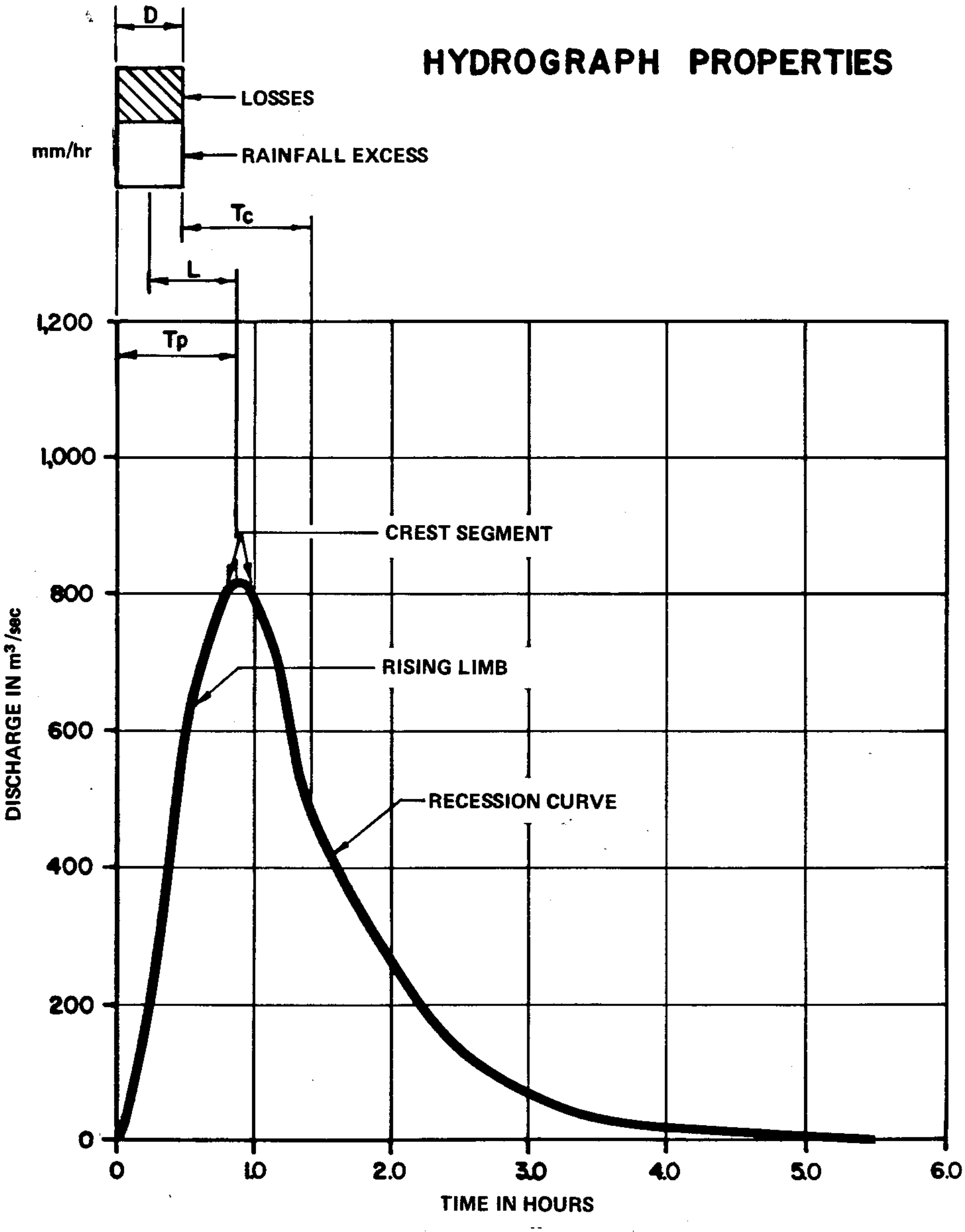


FIGURE 8. Typical Hydrograph

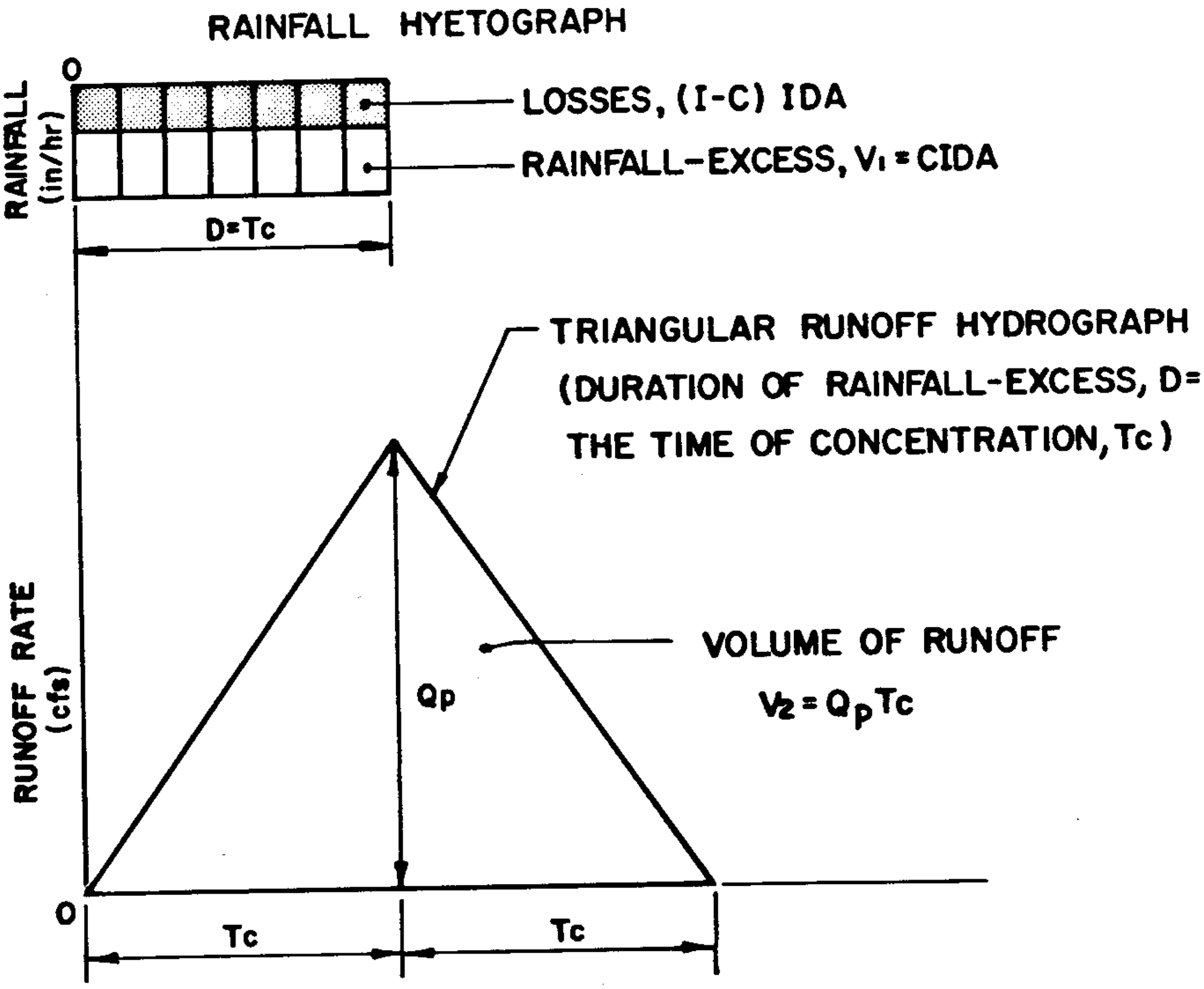


FIGURE 9. TRIANGULAR HYDROGRAPH

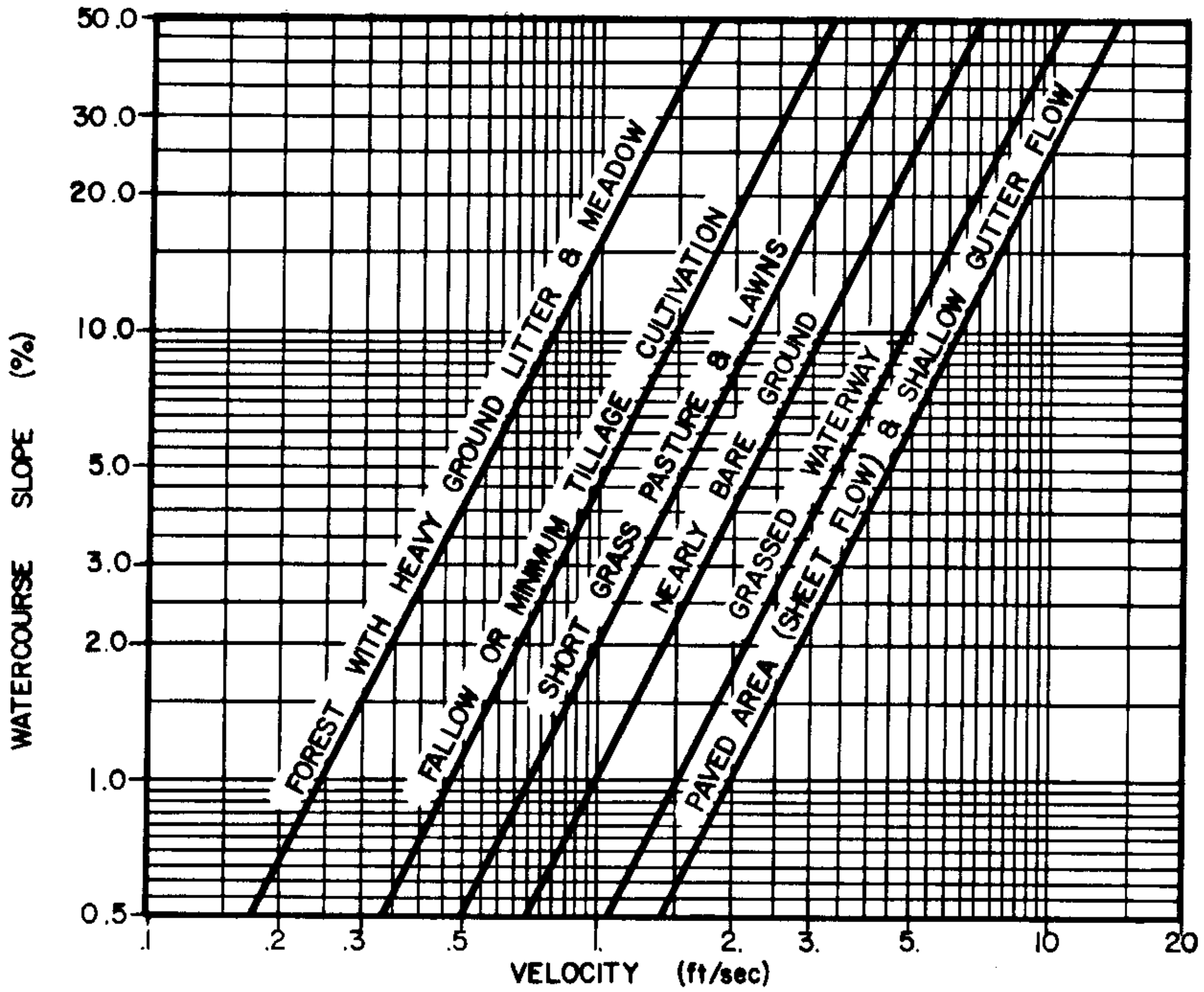


FIGURE 10. AVERAGE VELOCITIES FOR ESTIMATING TRAVEL TIME FOR OVERLAND FLOW. (SCS METHOD)[4]

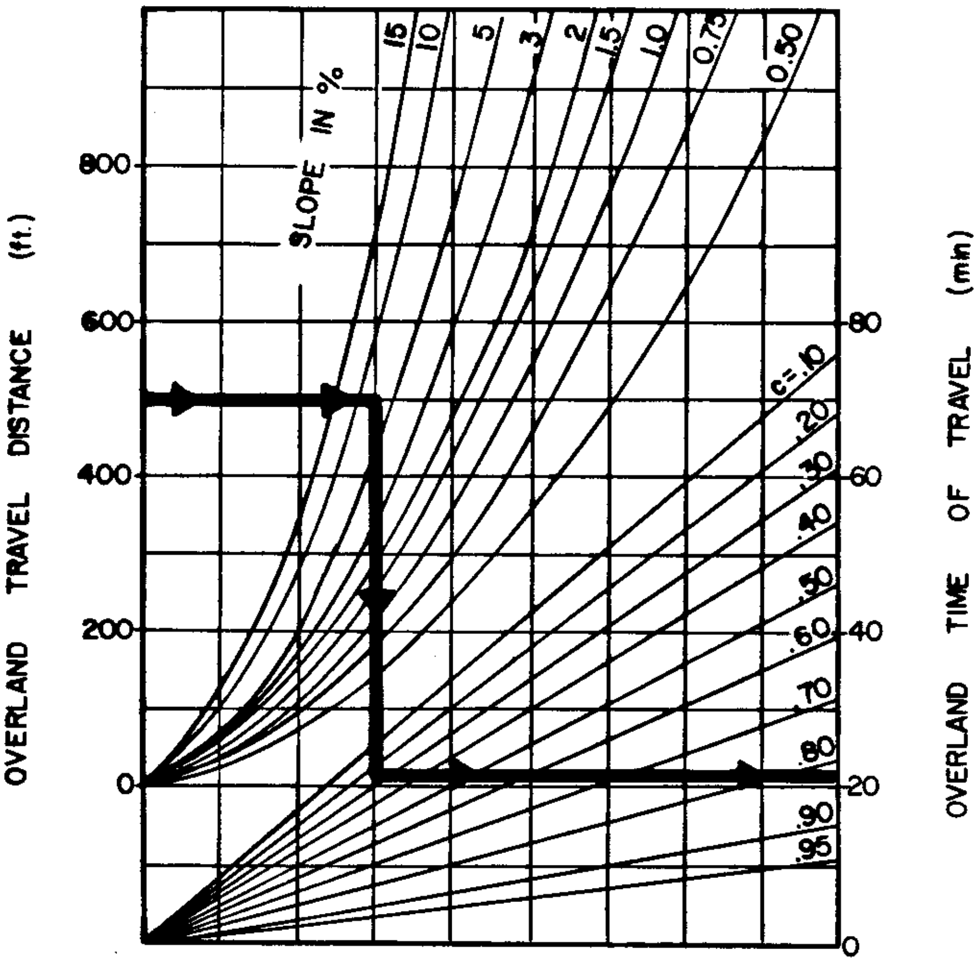


FIGURE 11. RELATION OF OVERLAND TIME OF TRAVEL TO OVERLAND TRAVEL DISTANCE
AVERAGE OVERLAND SLOPE, AND COEFFICIENT C — FOR USE IN RATIONAL METHOD.

NOTES

Example Problem: What is the peak discharge using the rational formula for a 10 acre watershed which has an apartment land use and rainfall intensity = 1"/hr. Use a "C" factor of 0.60, thus:

$$Qp = CIA$$

$$Qp = 0.60\ (1)\ (10) = 6 \text{ cfs.}$$

Example Problem: Calculate the time to peak (Tp) for a watershed if the longest overland travel distance is 500 feet with an average land slope of 7.0% and a C factor of 0.25. Using figure 11, the time of travel is 21 minutes.

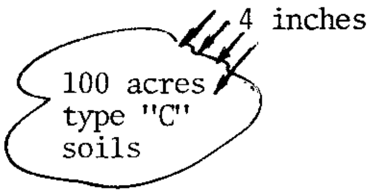
Problems:

1. Assuming average soil moisture conditions on hydrologic soils group C, calculate the run-off volume for a 100-ac suburban development with the following land use if rainfall is 4 in.

Land Use	Percentage of Land
1/4-ac residential lots	40
1/8-ac condominiums	20
Paved streets with curbs	25
Open space. grass cover - 85%	15

Answer:

Sketch



Computation

Land Use	% of Area	CN	Fractional CN
1/4-ac residential	40	83	33.2
1/8-ac condominiums	20	90	18
Paved streets with curbs	25	98	24.5
Open space; grass cover = 85%	15	74	11.1

Total Use: 86.8 - 87

NOTES

From Table II-7 with rainfall equal to 4 inches and CN = 87:

$$R = P - S' = P - (\frac{1000}{CN} - 10) =$$

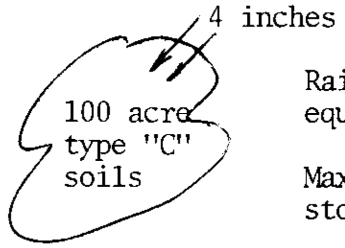
$$4 - (\frac{1000}{87} - 10) = 4 - 1.5 = 2.5 \text{ inches}$$

RAINFALL EXCESS - 2.5 INCHES

2. What is the area of a retention (no outlet) or percolation basin in square feet and acres to store the runoff water (rainfall excess) of Problem #1 if the maximum depth of storage is 5 ft., excluding debris storage and freeboard? Assume a rectangular pond with vertical sides.

Answer:

Sketch



Rainfall excess is equal to 2.5 in.

Maximum depth of storage = 5.0 ft.

Calculation

$$[100 \text{ ac} \times 2.5 \text{ in} \times \frac{1}{12} \text{ ft/in} \times 43{,}560 \text{ ft}^2\text{/ac}]$$

$$/5 \text{ ft} = 181{,}500 \text{ ft}^2 = \underline{\underline{4.17 \text{ ac}}}$$

3. Calculate rainfall excess using 3 different methods if P = 6.00 inches, CN = 80, C = 0.50. Discuss differences.

Answer:

a. Rational Formula

$$R = CP$$

$$R = 3.0 \text{ inches}$$

b. Curve Number

$$S' = \frac{1000}{80} - 10 = 12.5 - 10.0 = 2.5 \text{ in.}$$

$$R = (P - 0.2S')^2/P + 0.8S'$$

$$= [6.0 - 0.2(2.5)]^2/6.0 + 0.8(2.5)$$

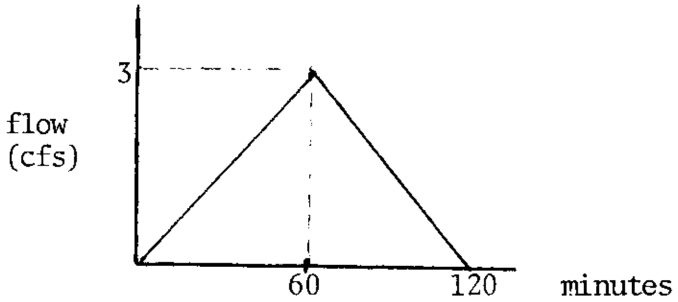
$R = 3.78$ in

NOTES

c. $Q_p = cIA$ Assume $I = 6$ in/hr

$= (0.50)\ (6)\ (1)$

$= 3.0$ cfs $T_c = 60$ minutes



Area = ½(120)(3.0)(60 sec/min)

(1/43,560) (12 in/ft) = 2.98 in.

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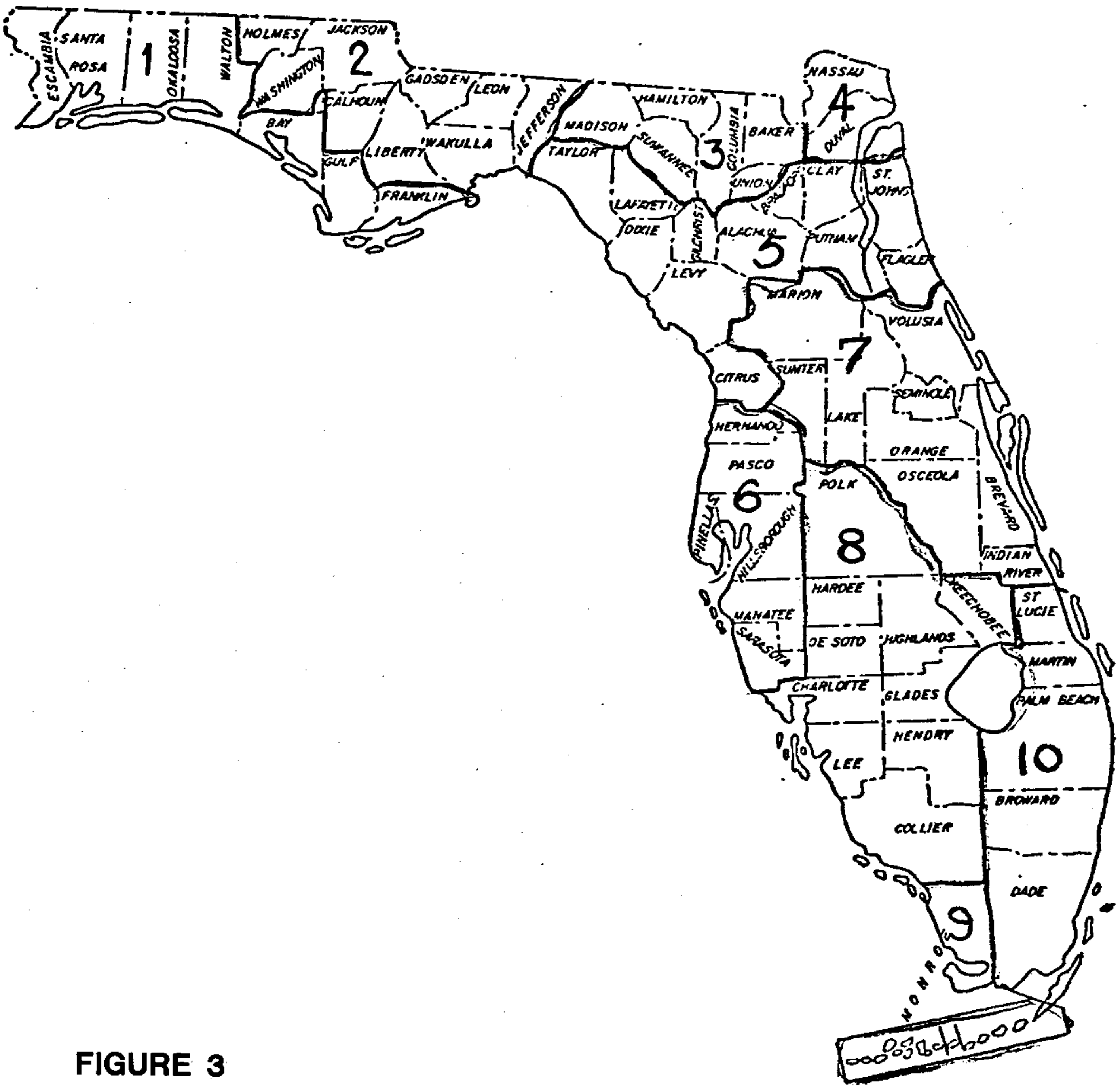
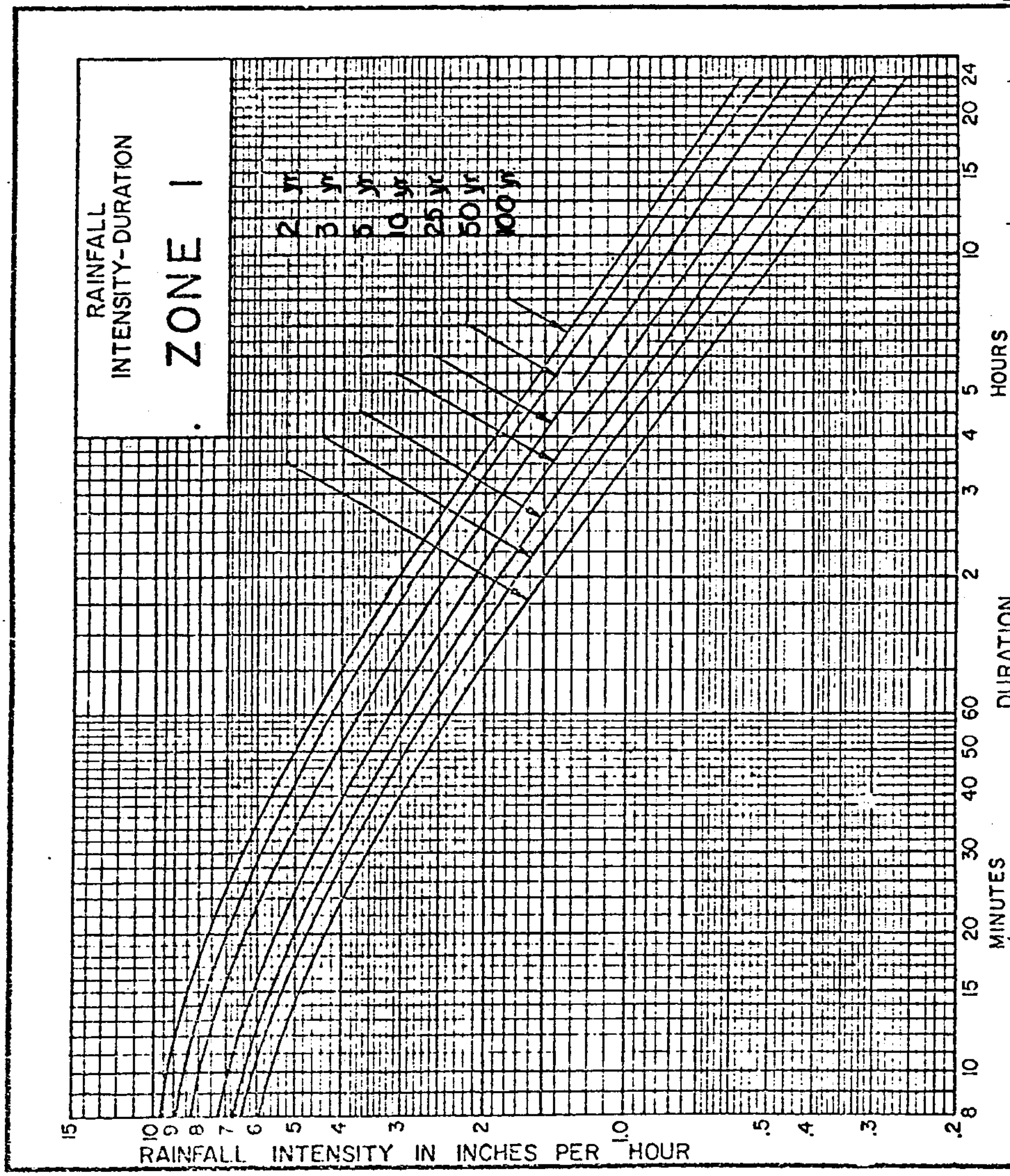
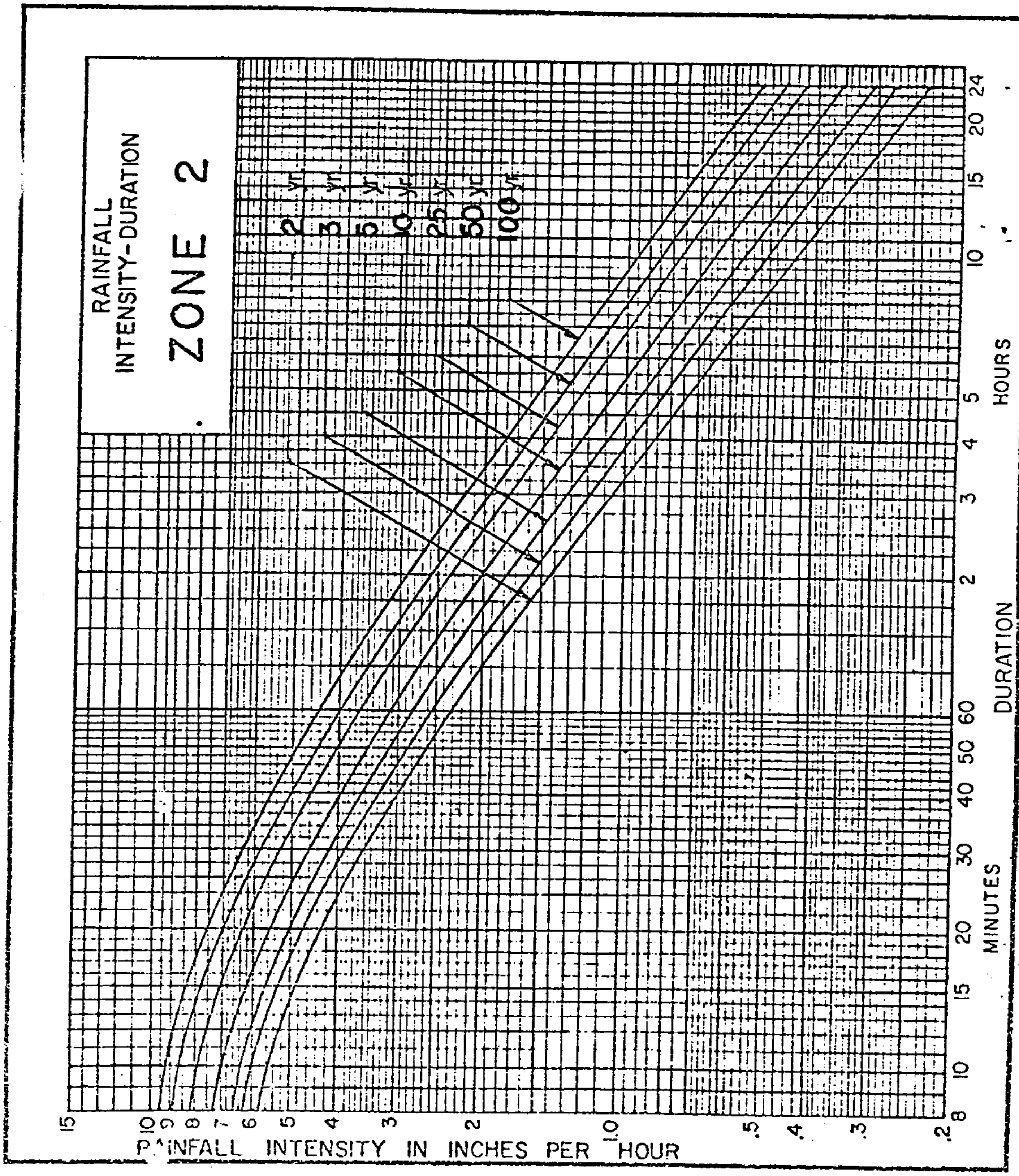
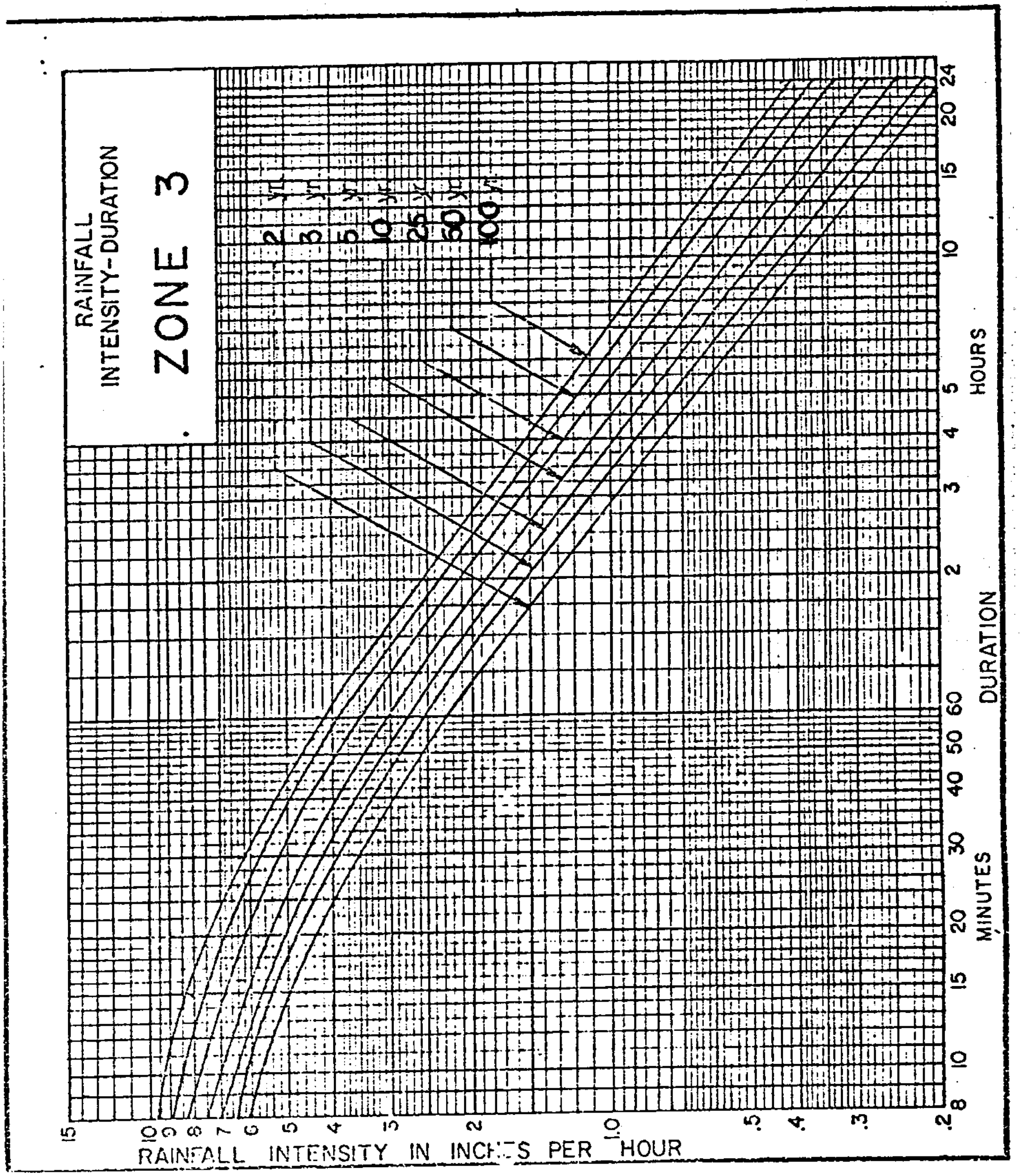


FIGURE 3

RAINFALL INTENSITY DURATION FREQUENCY
(FROM FLORIDA DOT DRAINAGE MANUAL STATE DOT TALLAHASSEE, FL, LATEST EDD EDITT EDITION)


RAINFALL
INTENSITY-DURATION
ZONE I
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
15
10
9
8
7
6
5
4
3
2
1.0
.5
.4
.3
.2
RAINFALL INTENSITY IN INCHES PER HOUR
8
10
15
20
30
40
50
60
2
3
4
5
10
15
20
24
MINUTES
DURATION
HOURS


RAINFALL
INTENSITY-DURATION
ZONE 2
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
15 10 9 8 7 6 5 4 3 2 1.0 .5 .4 .3 .2
INFALL INTENSITY IN INCHES PER HOUR
8 10 15 20 30 40 50 60
MINUTES
DURATION
2 3 4 5 10 15 20 24
HOURS


RAINFALL
INTENSITY-DURATION
ZONE 3
2
3
5
10
25
50
100 yr
DURATION
MINUTES
HOURS
RAINFALL INTENSITY IN INCHES PER HOUR
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10
9
8
7
6
5
4
3
2
1.0
.9
.8
.7
.6
.5
.4
.3
.2
8
10
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20
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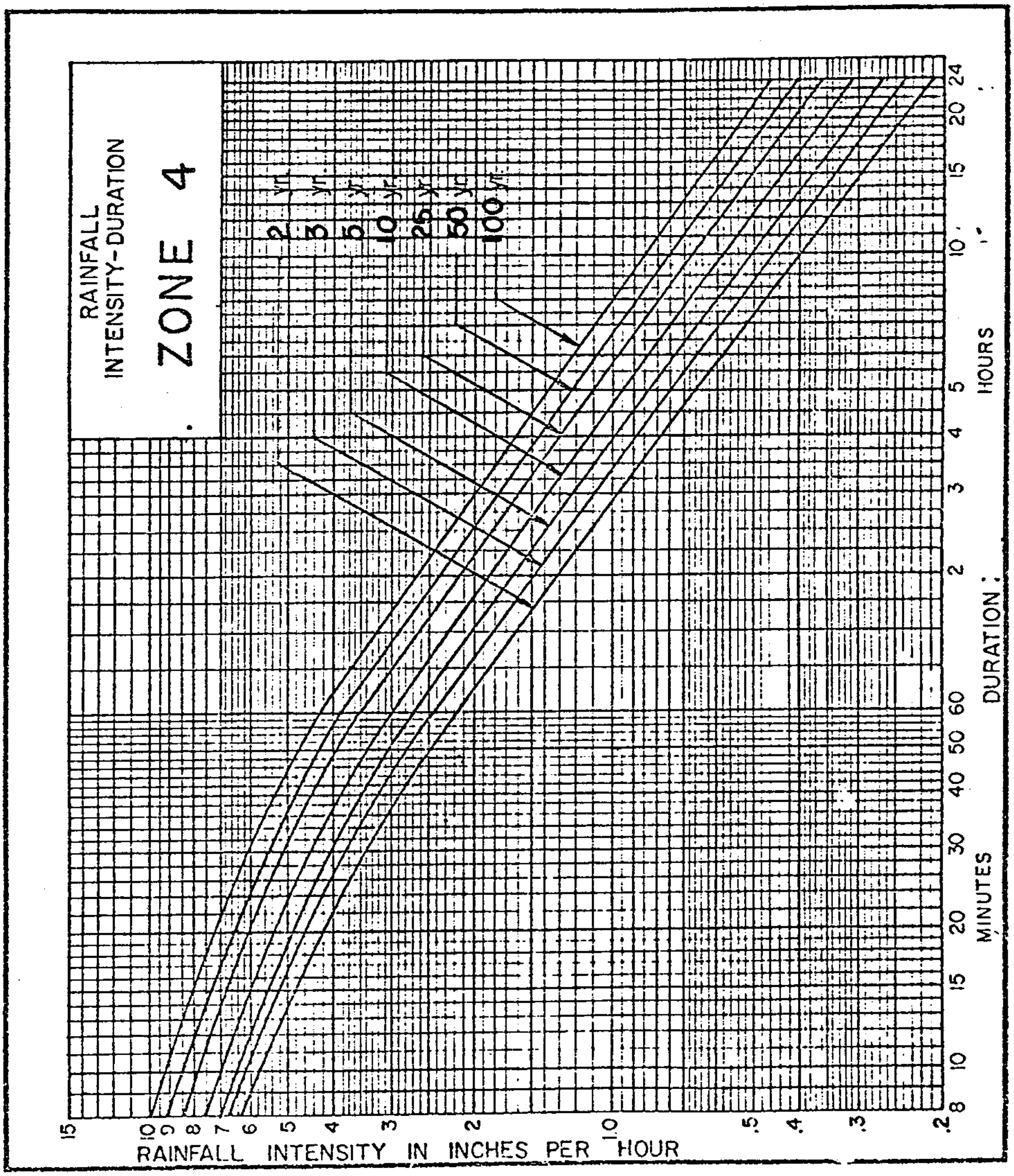
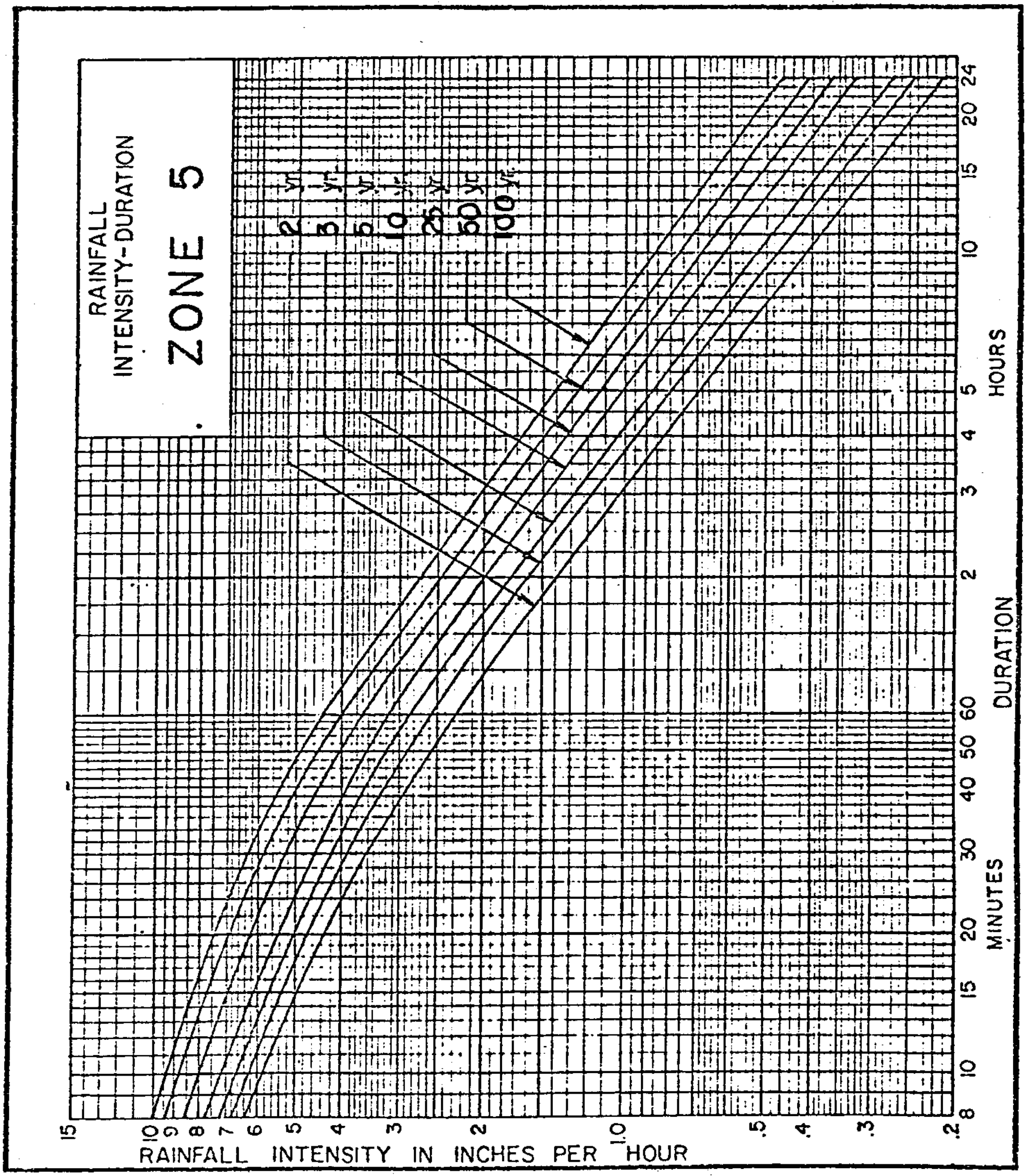
10
15
20
24


RAINFALL
INTENSITY-DURATION
ZONE 4
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
RAINFALL INTENSITY IN INCHES PER HOUR
DURATION:
MINUTES
HOURS


RAINFALL
INTENSITY-DURATION
ZONE 5
2 yr.
3 yr.
5 yr.
10 yr.
25 yr.
50 yr.
100 yr.
DURATION
MINUTES
HOURS
8
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40
50
60
2
3
4
5
10
15
20
24
15
10
9
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6
5
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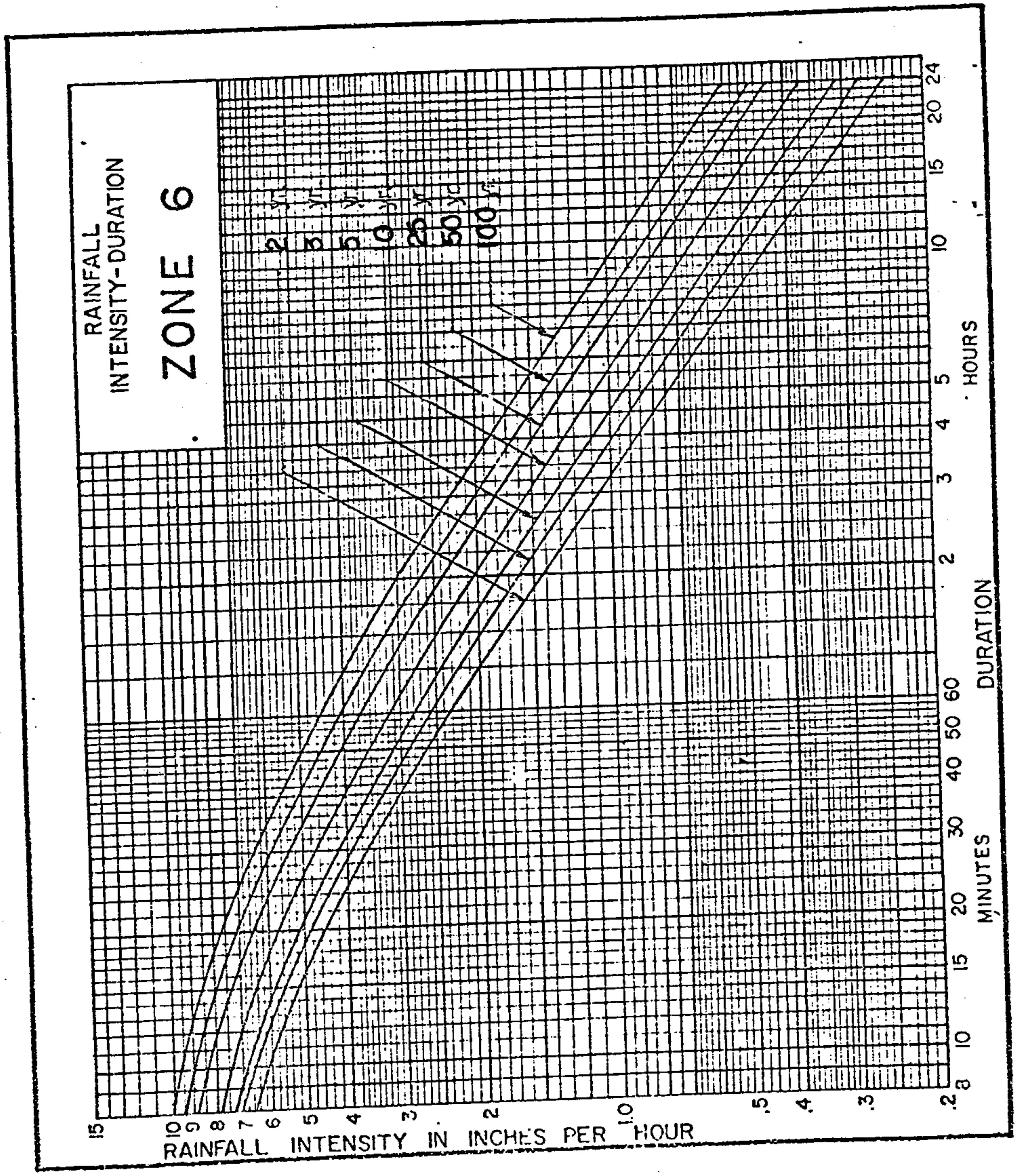
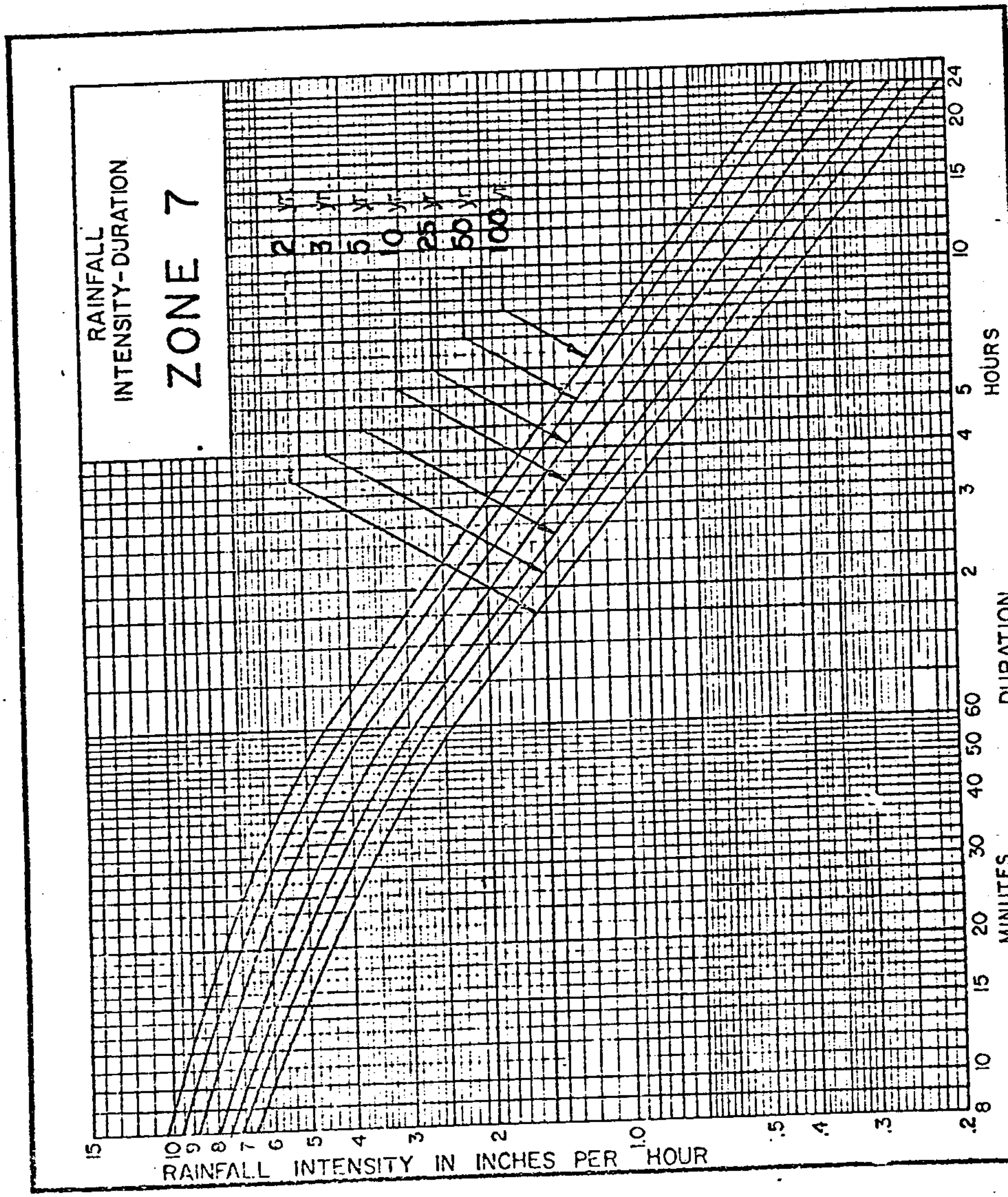
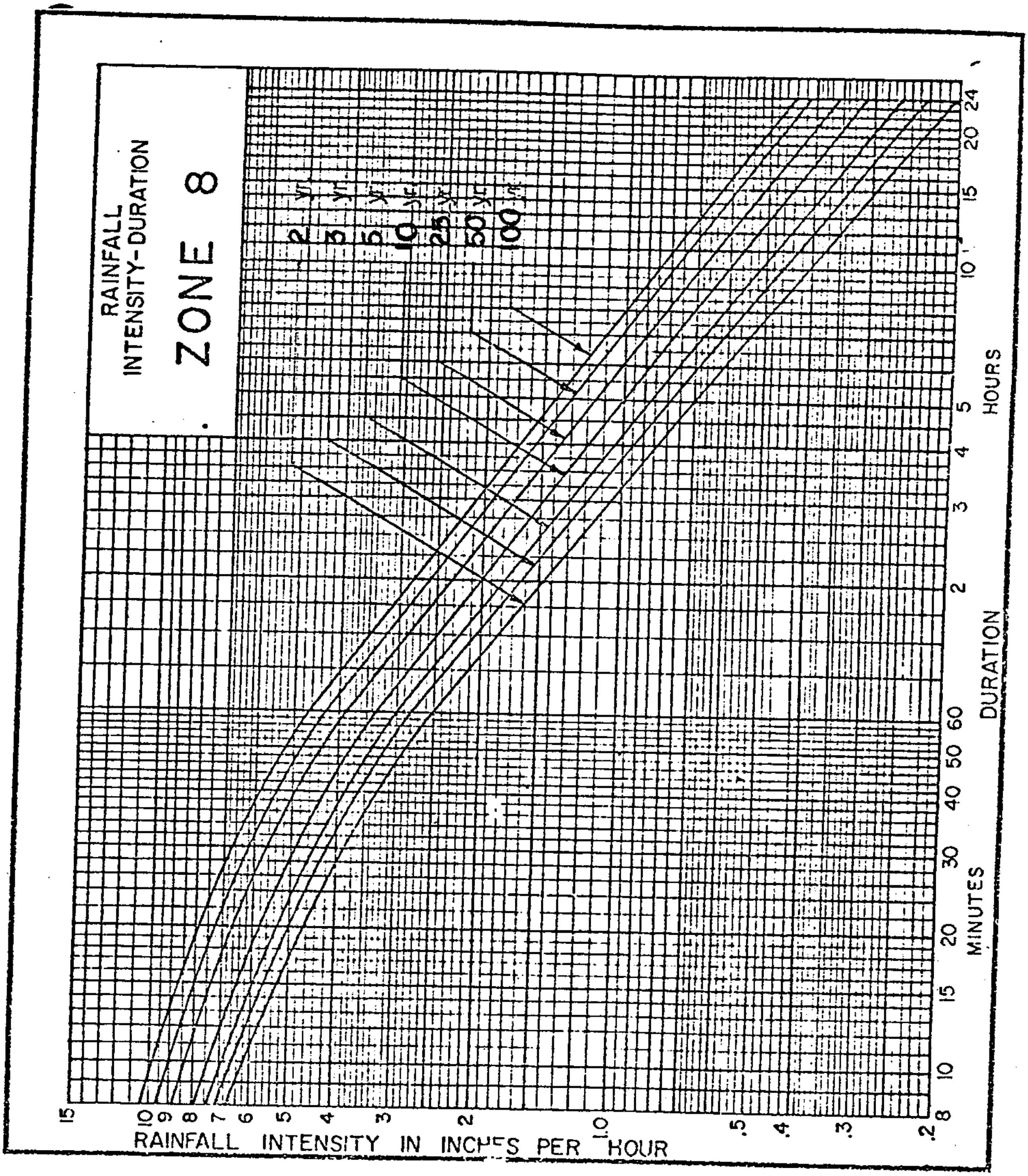
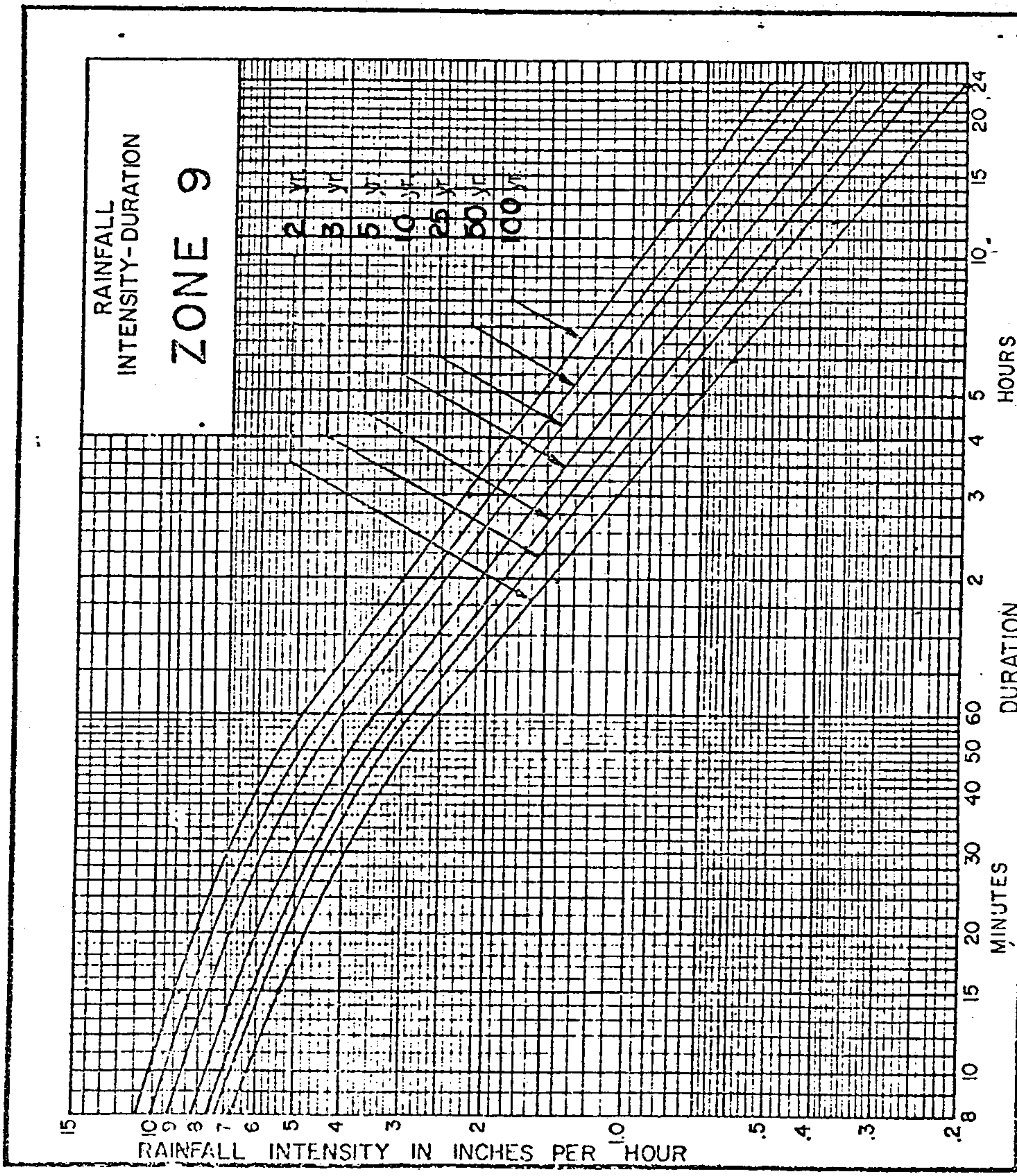
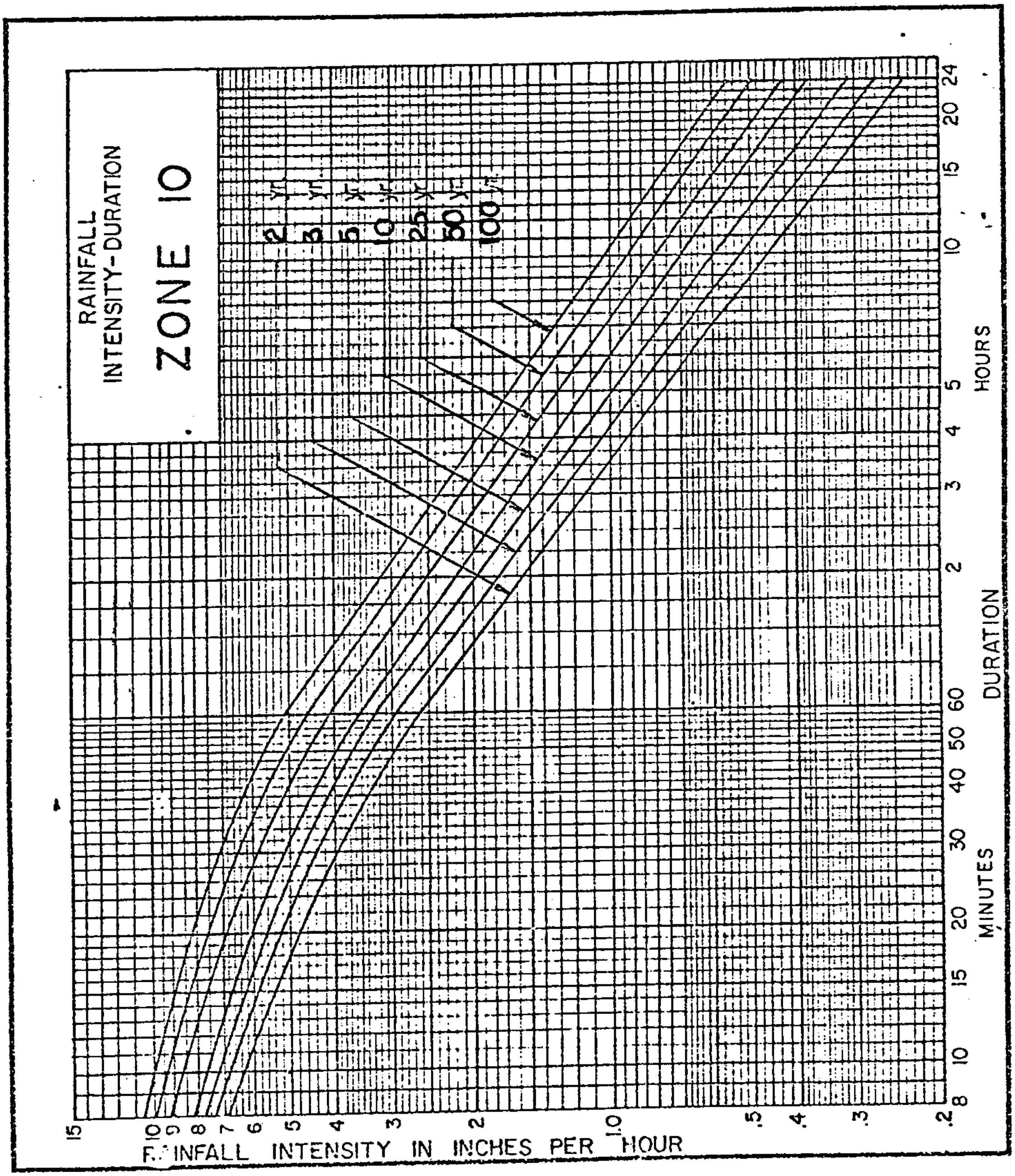
3
2
1.0
.5
.4
.3
.2
RAINFALL INTENSITY IN INCHES PER HOUR


RAINFALL
INTENSITY-DURATION
ZONE 6
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
MINUTES
DURATION
HOURS
RAINFALL INTENSITY IN INCHES PER HOUR


RAINFALL
INTENSITY-DURATION
ZONE 7
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
RAINFALL INTENSITY IN INCHES PER HOUR
HOURS


RAINFALL
INTENSITY-DURATION
ZONE 8
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
MINUTES
DURATION
HOURS
RAINFALL INTENSITY IN INCHES PER HOUR


RAINFALL
INTENSITY-DURATION
ZONE 9
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
RAINFALL INTENSITY IN INCHES PER HOUR
15 10 9 8 7 6 5 4 3 2 1.0 .5 .4 .3 .2
MINUTES
8 10 15 20 30 40 50 60
HOURS
2 3 4 5 10 15 20 24
DURATION


RAINFALL
INTENSITY-DURATION
ZONE 10
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
15
10
9
8
7
6
5
4
3
2
1.0
.5
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.3
.2
RAINFALL INTENSITY IN INCHES PER HOUR
8
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40
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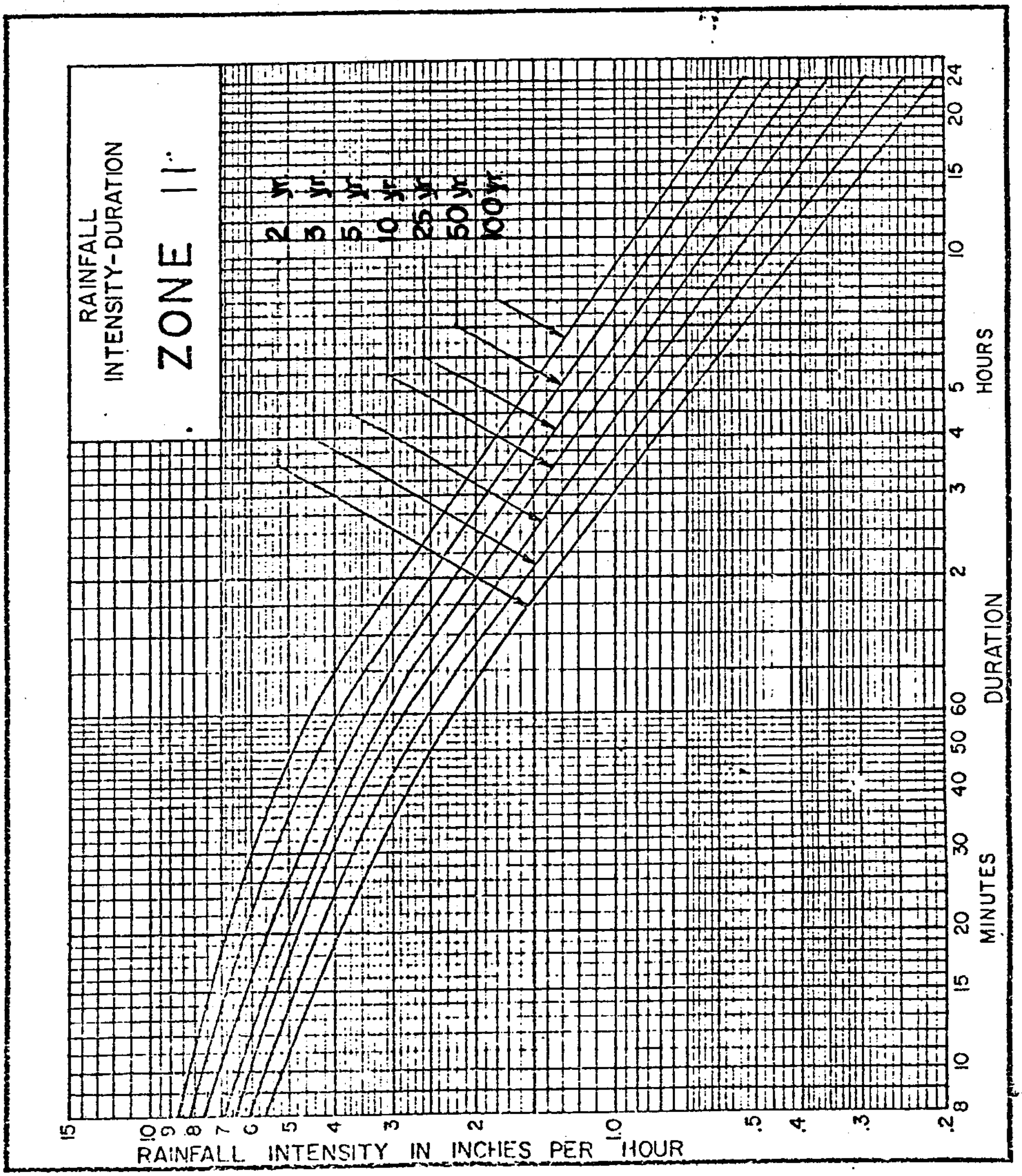
20
24
MINUTES
DURATION
HOURS


RAINFALL
INTENSITY-DURATION
ZONE II
2 yr
3 yr
5 yr
10 yr
25 yr
50 yr
100 yr
RAINFALL INTENSITY IN INCHES PER HOUR
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8
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1.0
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DURATION
MINUTES
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HOURS
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24

SECTION 3

NOTES

BASIC HYDRAULICS FOR STORMWATER MANAGEMENT

Methodologies for estimating runoff hydrographs before and after development were presented in the previous sections. Various methods of calculation can be used to predict runoff response characteristics from given rainfall information and site drainage conditions. The important runoff factors which must be defined include peak discharge, time to peak and total runoff volume. Controlling the peak discharge for a given design runoff event to the peak rate corresponding to pre-development conditions is a common runoff control method. This is accomplished by sizing the outlet to discharge the specified peak flow after detaining the required storage capacity.

The purpose of this section is to provide a brief overview of hydraulic principles as they relate to stormwater collection, management and impact on receiving water bodies. It is intended to discuss flow through open channels such as pipes or ditches, flow measurements and control structures.

Flow Through Ditches, Canals and Partially Filled Pipes: Normal flow in open channels is computed by the Chezy-Manning formula

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \qquad (1)$$

Q = Channel flow in cfs.

n = Manning's coef. of roughness

A = Cross section area of flow in sq. ft.

R = Hydraulic radius in feet

$$= \frac{\text{cross section Area}}{\text{wetted perimeter}}$$

S = Channel slope

If metric units are used,

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \qquad (2)$$

Also continuity equation specifies:

$$Q = AV \qquad (3)$$

Where V = water velocity ft/sec

NOTES

Typical values of Manning's coefficients n are listed below:

Typical values of "n"

Channel Surface	n
Vitrified Sewer Pipe	0.010 - 0.017
Cast Iron	0.011 - 0.015
Concrete, Precast	0.011 - 0.015
Clay Drainage Tile	0.011 - 0.017
Brick with Cement Mortar	0.012 - 0.017
Riveted Steel	0.017 - 0.020
Cement Rubble Surfaces	0.017 - 0.030
Corrugated Metal Storm Drain	0.020 - 0.024
Earth Excavation	0.022
Ditches	0.028
Natural Stream	0.030
Channels Not Maintained	0.050 - 0.100
Well Finished Gutters	0.016

Manning's Equations as represented in (1) and (2)

are used for:

(1) Pipes Flowing Full

(2) Partially Filled Pipes

(3) Open channels of various sections

There are graphic solutions to equations (1) and (2) for pipes as shown in Figure 1.

Stormwater is collected through stormwater drains, ditches, swales, etc. Examples of flow calculations will follow

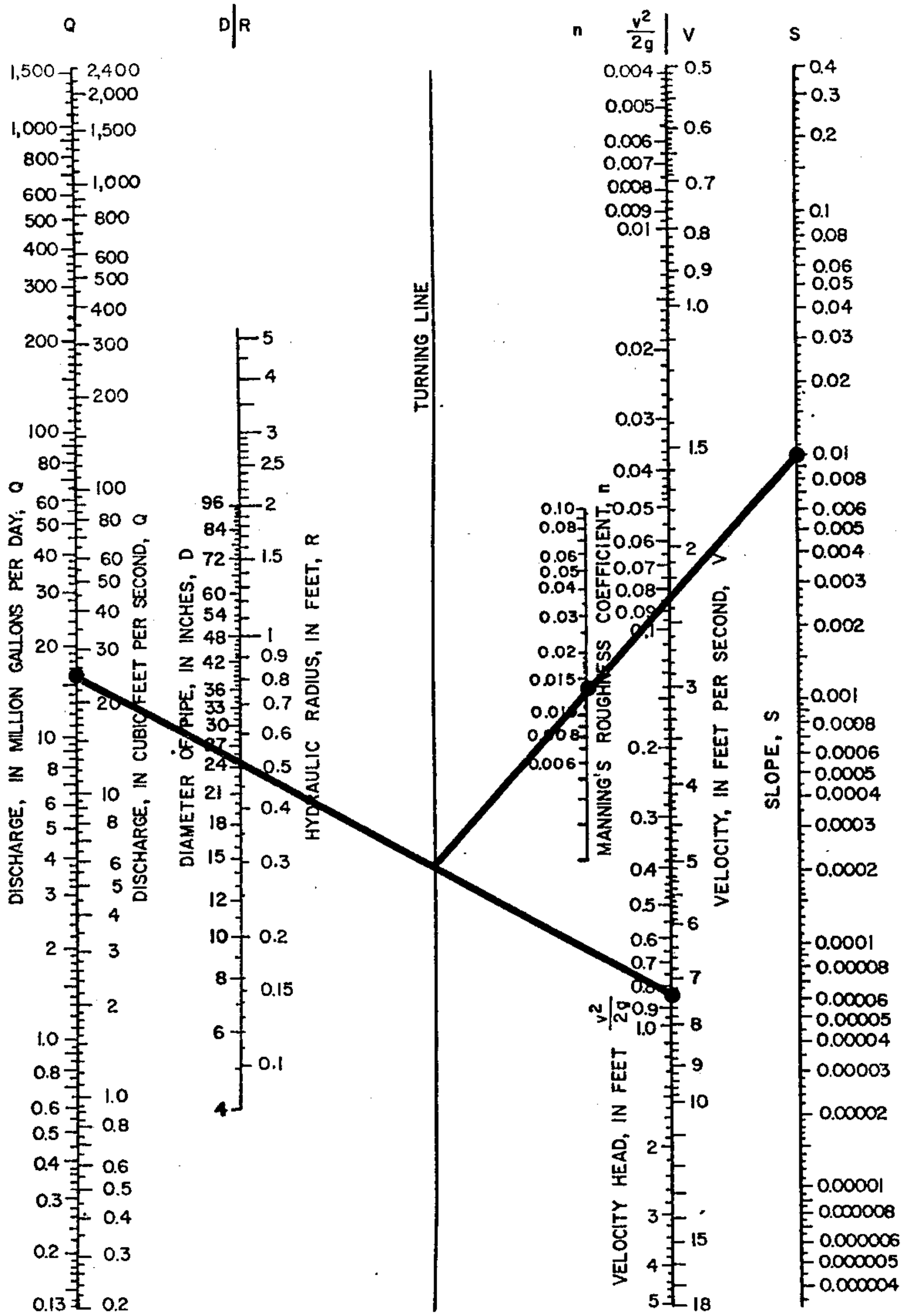
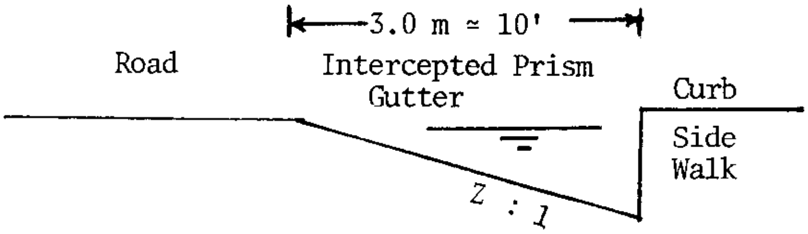


Figure 1. Manning's Nomograph for Open Channel Flow

NOTES

Gutters:

Manning's equation may be used. For well finished gutters n = 0.016. For gutters with broken pavement or unpaved, much higher values of n should be used.

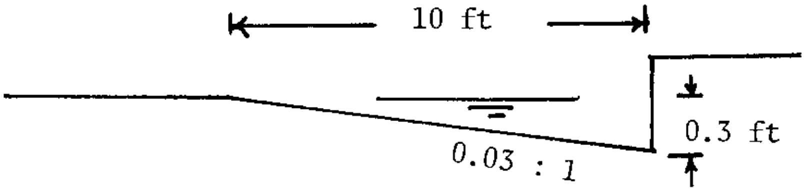


Transverse slope is generally 1:20, curb height ≃15 cm ≃6", width of flow ≃3M ≃10 ft
Recommended slope ≃0.01 in the longitudonal direction.

Flat slopes will result in much higher errors between calculated and actual flows.

Gutter flow is intercepted and directed to burried sewers by drop inlets.

Example-1



If n = 0.016

slope = $0.05 \frac{ft}{ft}$

Area $= \frac{0.3}{0.03} \times 0.3 \times 1/2$ = Area of Triangle

Length Depth

= 10 X 0.3 X 0.5 = 1.5 sq. ft.

Wetted perimeter = $0.3 + \sqrt{0.3^2 + 10^2}$

= 10.3 ft

$$Q = \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$= \frac{1.486}{0.016} \left(\frac{1.5}{10.3}\right)^{2/3} (0.05)^{1/2} \text{ X Area}$$

$$= (9.288) \quad (0.277) \quad (0.224) \quad A$$

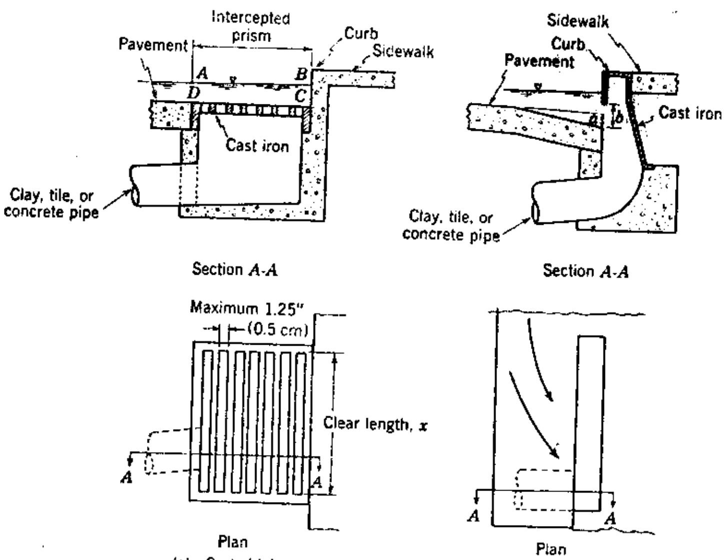
$$= 5.76 \frac{ft}{sec} \text{ X } 1.5 \text{ sq. ft.}$$

$$= 8.64 \frac{cu\ ft}{sec}$$

NOTES

INLETS

(a) Grated Inlet
(b) Curb Opening Inlet



(a) Grated inlet
(b) Curb-opening inlet

Storm Drain Criteria:

(1) Pipes to flow full under steady uniform flow

(2) Min. diam. 10 or 12 inches. 8" pipes are used in some cities

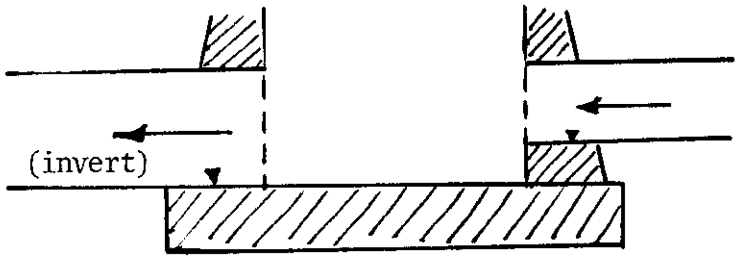
(3) Min. velocity flowing full at least $2.5 \frac{ft}{sec}$ or 0.75 meters/sec.

NOTES

(4) Pipe sizes should not decrease in the downstream sections

(5) Pipe slopes conform to ground slopes

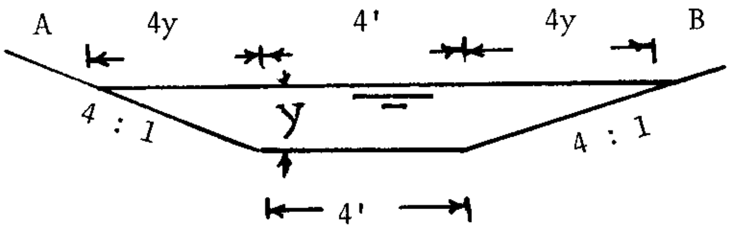
(6) The invert of larger pipes should be lower than that of smaller pipe



Example 2

A trapezoidal swale with a bottom width of 4 feet and side slopes 4:1 (four horizontal to one vertical) carries a maximum water flow rate of 50 $\frac{ft^3}{sec}$ calculate the depth of water if n = 0.03 and bottom slope is 0.0004

Solution:



The width of water surface AB = 4.0 + 8 y

The cross sectional area = $\frac{(4) + (4 + 8y)}{2}$ y

$= 4\ y + 4\ y^2$

The wetted perimeter $= 4 + 2^y \sqrt{4^2 + 1^2}$

$= 4 + 2^y \sqrt{17}$

$= 4 + 8.25y$

NOTES

$$\text{Hydraulic Radius} = \frac{A}{P} = \frac{4y + 4y^2}{4 + 8.24y}$$

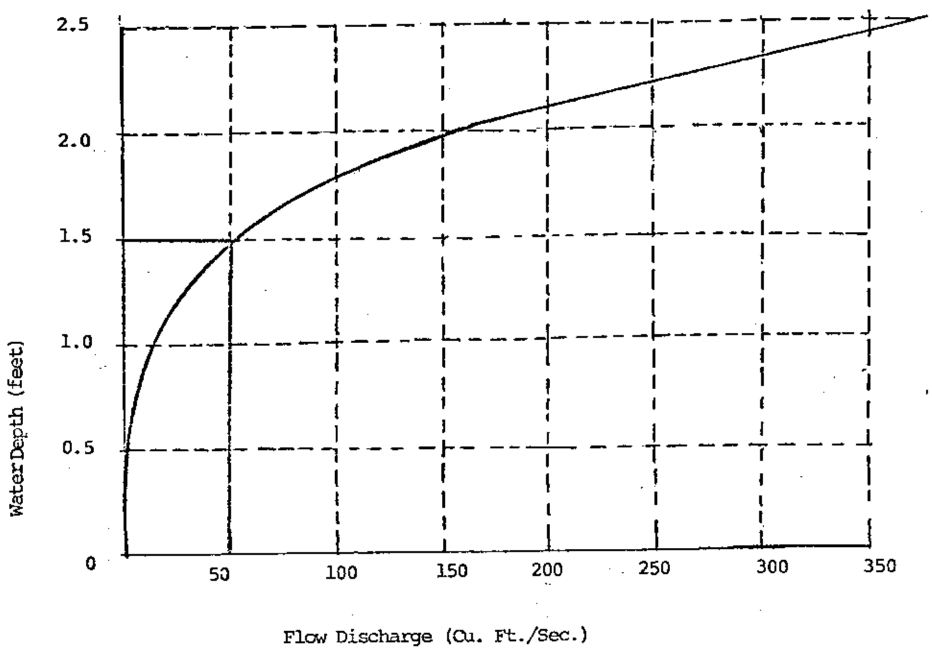
$$= \frac{y + y^2}{1 + 2.06y}$$

$$50\frac{ft^2}{sec} = \frac{1.486}{0.03}\left(\frac{y + (y)^2}{1 + 2.06y}\right)^{2/3} (0.0004)^{1/2} \; (4y + 4y^2)$$

$$= \frac{1.486}{0.03} \quad X \; 4 \; X \; 0.02 \quad \frac{(y + y^2)^{5/3}}{(1 + 2.06y)^{2/3}}$$

$$50 = 3.96 \quad \frac{(y + y^2)^{5/3}}{(1 + 2.06y)^{2/3}},$$

Y (feet)	Q (Cu.Ft/Sec)
1	11.92
1.2	22.99
1.4	40.59
1.5	52.56
1.6	67.08
1.8	105.26
2.0	158.45
2.5	383.53



For a flow discharge of 50 Cu. Ft./Sec. the water depth in the canal is 1.5 ft. The above curve shows the increase in water depth with increasing of flow discharges. For a flow of 50 Cu. Ft./Sec., The water velocity can be calculated as $V = \frac{Q}{A} = \frac{50}{(4)\ (1.5) + 4\ (1.5)^2} = 3.33$ Ft./Sec.

Acceptable velocities vary between 2 and 8 Ft./Sec. according to channel stability.

NOTES

Flow Through Hydraulic Structures: For purposes of stormwater management design, the simplest types of flow control devices are orifices and weirs. The theoretical flow characteristics of selected control devices will be presented as follows:

(1) Circular Orifices:

$$Q = CA\sqrt{2gh}$$

where

Q = Orifice discharge in cfs

C = Coefficient of discharge

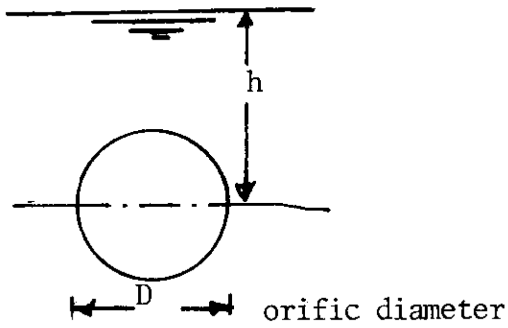
= 0.6

A = Orifice cross sectional area in sq. ft.

g = Gravitational acceleration

= 32.2 ft/sec^2

h = Hydraulic head above the center of the orifice

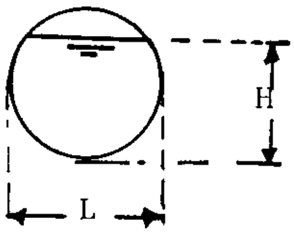


$$Q = CA\sqrt{2gh}$$

$$C = 0.6, \quad A = \frac{\pi D^2}{4}$$

NOTES

Note: When water surface behind orifice falls below the top of the orifice opening, the orifice is treated as a weir:



$Q = CLH^{3/2}$

$C = 3$

L = diameter of orifice pond

H = Hydraulic head above bottom of weir opening in feet

(2) Flow Under Gates: Flow under a vertical gate can be defined as a square orifice. Two flow conditions exist depending on submergence of the gate opening.

(a) For free outflow or downstream water level is not influencing the flow as shown in Figure 2. Flow under a vertical gate can be rewritten as:

$$Q = b\ a\ c\sqrt{2g}\left(\frac{Ho}{\sqrt{Ho + H_1}}\right)$$

Where Q = flow through the gate in cfs.

b = Width of gate in feet,

a = Gate opening height in feet,

c = Discharge coefficient,

g = Gravitational acceleration, 32.2 ft/sec^2

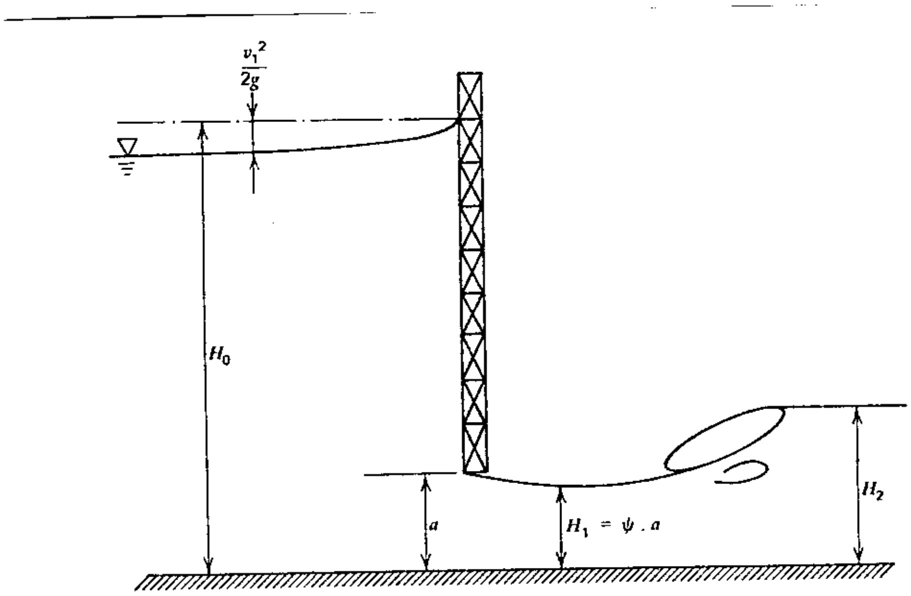


FIGURE 2. Notations for flow under gates.

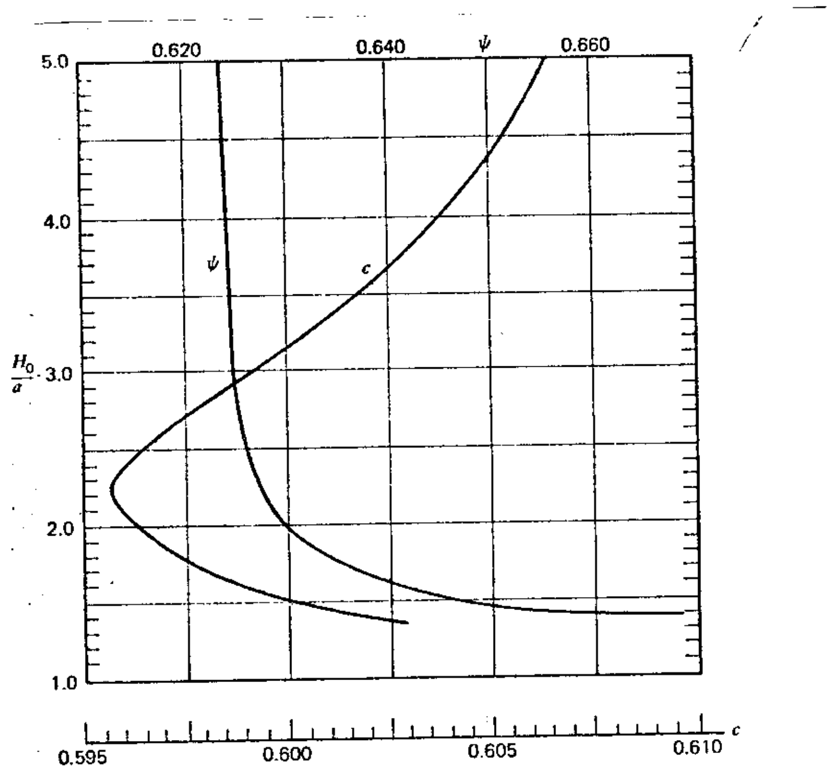


FIGURE 3. Discharge coefficient for flow under gates.

Ho = Upstream water depth in ft., and

H_1 = Downstream water depth

$= \Psi a < a$

Experimentally determined values for c and Ψ for various values of $\frac{Ho}{a}$ are shown in Figure 3.

(b) For outflow influenced by downstream water level,

$Q' = KQ$

where K is a coefficient found in Figure 4.

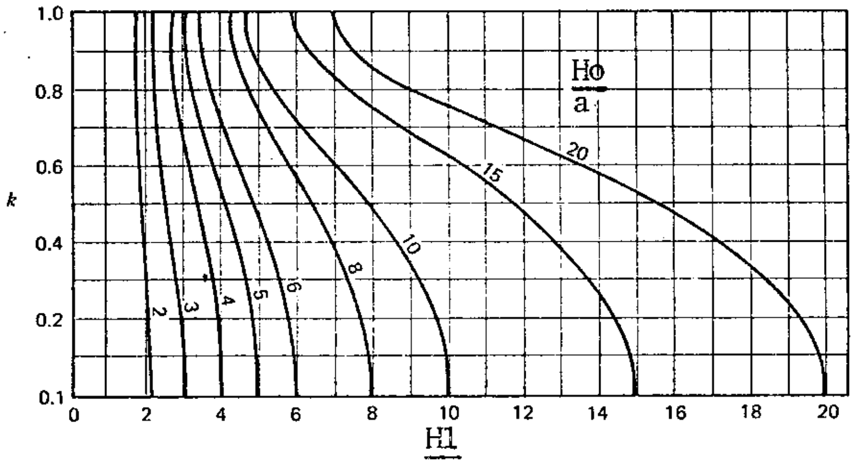


FIGURE 4. Absolute downstream control of flow under gates.

Example 3.

A) A 8.0 ft wide vertical gate on the top of a spillway withholds a 5.0 ft deep water. Determine the discharge under the gate if it is raised by 1.0 ft.

b = 8.0 ft, a = 1.0 ft, Ho = 5.0 ft.

$$Q = bac\sqrt{2g}\ \frac{Ho}{\sqrt{Ho + \Psi a}}$$

NOTES

NOTES

The coefficients C and Ψ may be obtained from Figure 3:

for $\frac{Ho}{a} = \frac{5}{1} = 5$, $\Psi = 0.623$ and $C = 0.607$

$$Q = 8\ (1)\ (0.607)\ \sqrt{64.4}\ \frac{5.0}{\sqrt{5.0 + (0.623)\ \ (1)}}$$

$= \underline{82.17 \text{ cfs ANS.}}$

B) Also you can determine the required opening of the gate for selected rate of discharge

C) If the above gate discharges into a pool in which the water level is 5 feet and the upstream water level is 6.0 feet and the opening is 1.25 ft. Determine the discharge through the gate.

$\frac{H1}{a} = \frac{5.0}{1.25} = 4.0 \qquad \frac{Ho}{a} = \frac{6.0}{1.25} = 5.0$

It is noticed that overflow is influenced by the dowstream. Partially retarded discharge

$Q' = KQ$ where $K = 0.52$ from Figure 4.

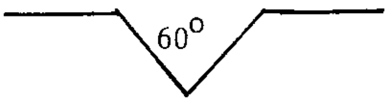
$$Q = 8\ (1.25)\ (0.607)\ \sqrt{64.4}\ \frac{6.0}{\sqrt{6.0 + 0.623\ (1.25)}}$$

$= 112.13$ cfs

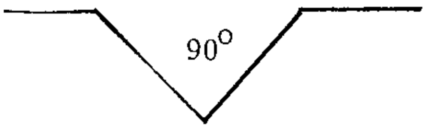
$Q' = 0.52 \times 112.13 = \underline{58.3 \text{ cfs ans.}}$

(3) Weirs:

Discharge Equations for Common Weir Shapes



60^{o} V-notch $Q=1.43H^{2.5}$



90^{o} V-notch $Q=2.49H^{2.48}$
(See Figure 5)

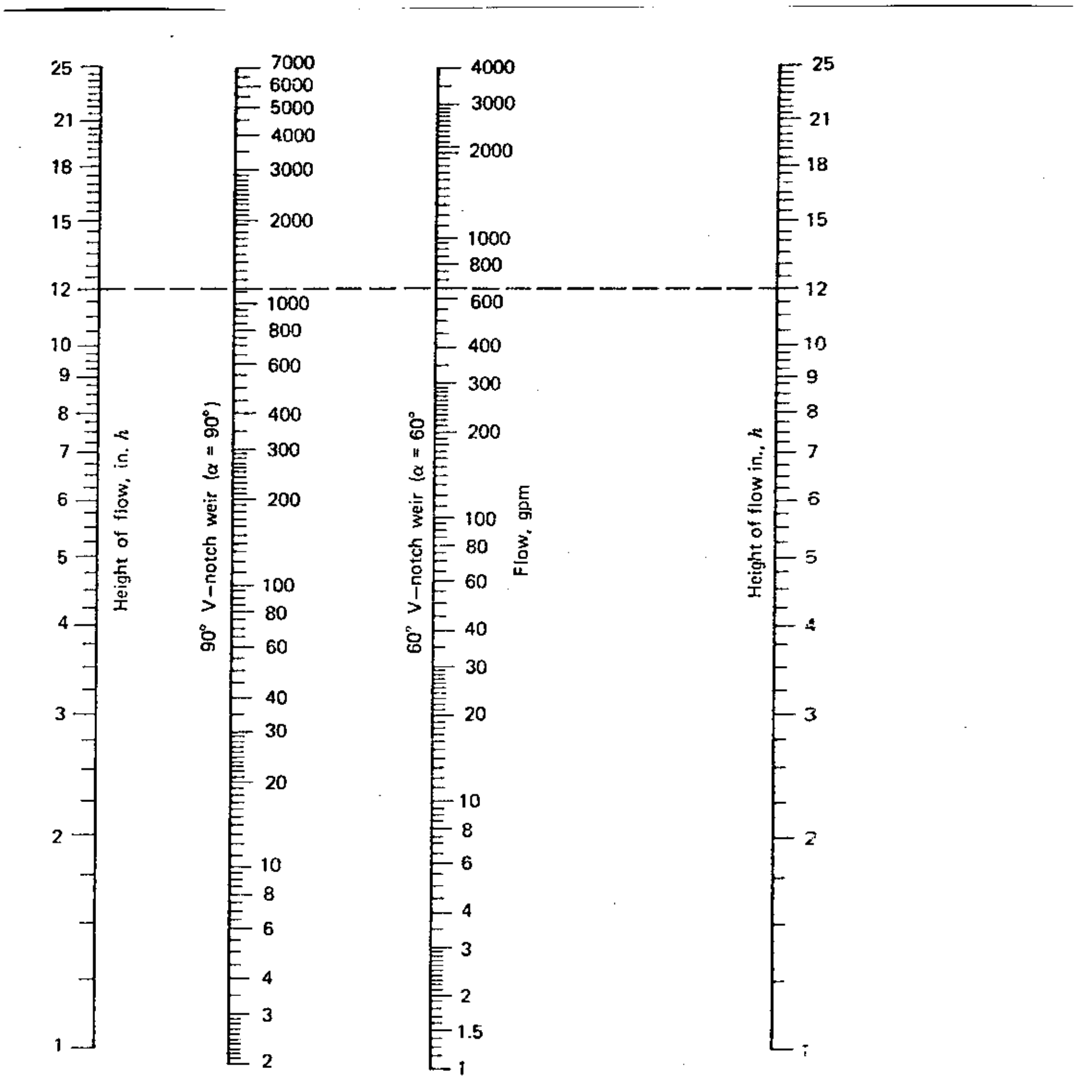
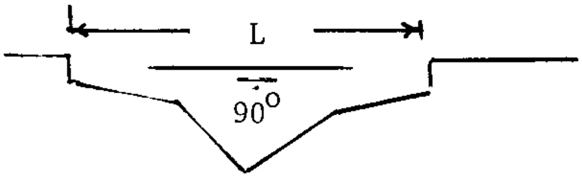
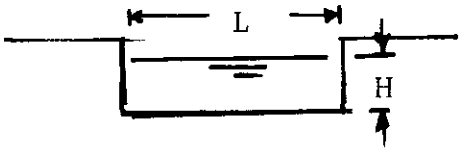


FIGURE 5. Nomograph for V-notch weir discharge. (Courtesy of Public Works Magazine.)

NOTES



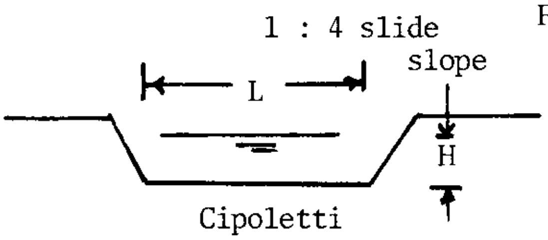
$$Q = 3.9H^{1.72}-1.5 + 3.3\ LH^{1.5}$$



Rectangular

$$Q = 3.33H^{1.5}(L-0.2H)$$

Fully controlled ends



Cipoletti

$$Q = 3.367LH^{1.5}$$

(See Figure 6)

Example 4

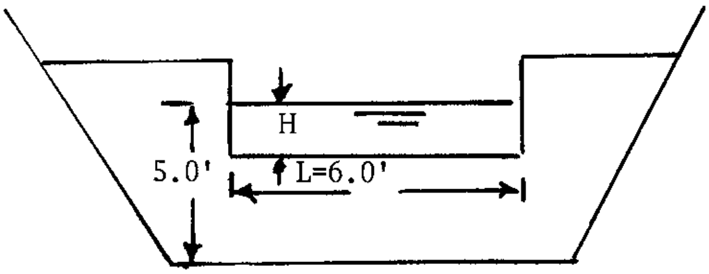
A rectrangular sharp-crest weir with end contractions has a crest 6.0 ft long. How high should it be placed in a channel to maintain an upstream depth of 5.0 feet for a discharge of 3.0 ft^3/sec.

Solution:

The contracted horizontal weir has a crest that is shorter than the width of the channel, so that water must be contracted both horizontally and vertically in order to flow over the weir.

$$Q = C_d\ (L - \frac{nH}{10})\ H^{3/2}$$

where n = 2 for contraction of both ends.



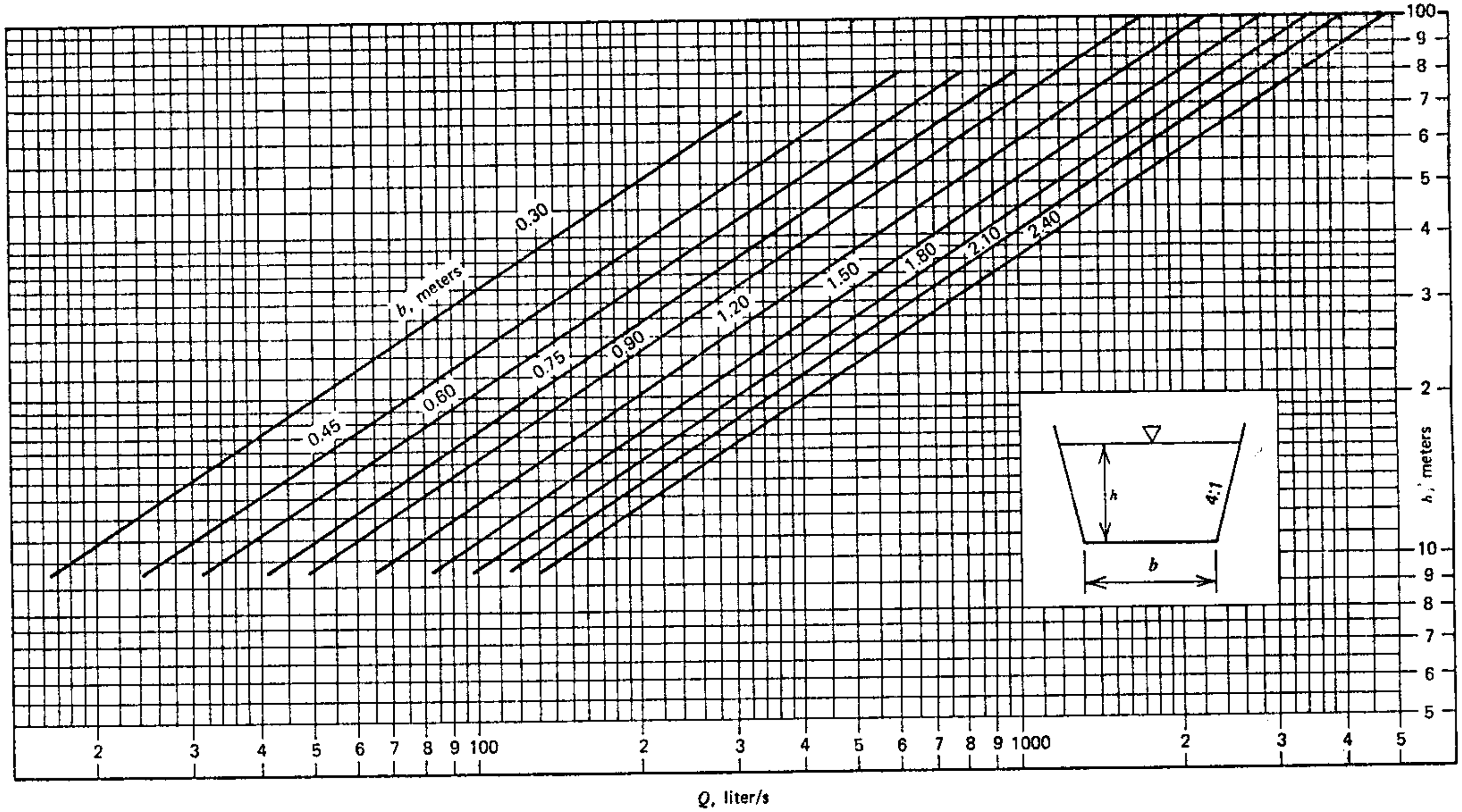


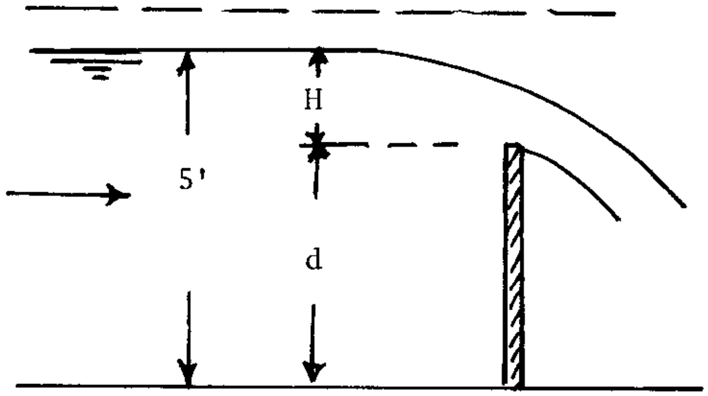
FIGURE 6. Discharge nomograph for Cipoletti weirs. (Courtesy of Public Works Magazine.)

NOTES

The standard contracted horizontal weir is one whose crest and sides are so far removed from the bottom and sides of the channel that full contraction is developed.

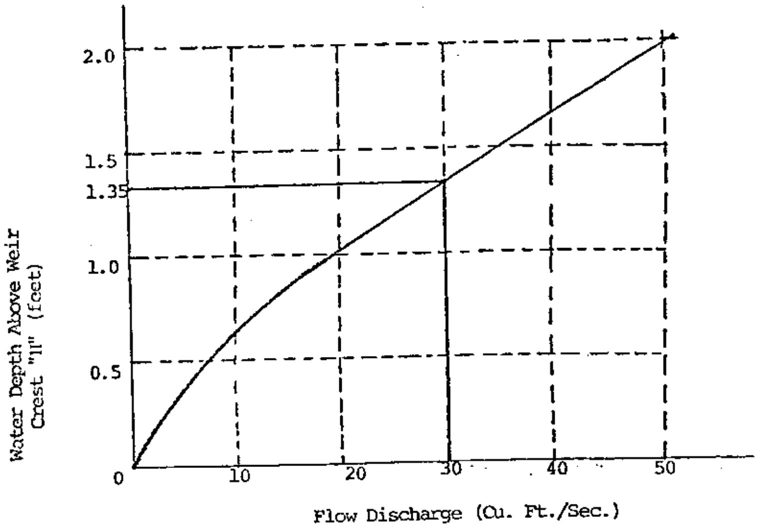
$$Q = 3.33\ (L - 0.2\ H)\ H^{3/2}$$

$$30 = 3.33\ (6.0 - 0.2H)\ H^{3/2}$$



By trial and error the following curve can be constructed as follows:

H (feet)	Q (Cu. Ft./Sec.)
0.25	2.48
0.5	6.95
1	19.31
1.2	25.21
1.3	28.33
1.35	29.94
1.4	31.57
1.5	34.87
1.6	38.28
1.8	45.36
2.0	50.86



H is equal to 1.35 feet on the crest which should be placed at a distance = 5 - 1.35 = 3.65 ft. above the bottom of the channel.

4. Culverts: Culverts are built at the lowest valley to pass water across embankments of highway. Inlet structures are built to protect embankments from erosion and improve the hydraulic conditions of culverts. Outlet structures are designed to protect outlets from scouring. Various modes of hydraulic operation of culverts are shown in Figure 7.

NOTES

(a) If the outlet is submerged as shown in Figure a, the culvert discharge is determined primarily by the tail water elevation and the head loss h_L, regardless of the culvert slope.

$$h_L = h_{ent} + h_f + \frac{V^2}{2g}$$

= entrance head + friction head + velocity head.

$$= K_{ent} \left(\frac{V^2}{2g}\right) + \frac{n^2V^2L}{2.21R^{4/3}} + \frac{V^2}{2g}$$

$$= \left[K_{ent} + \frac{n^2L}{2.21R^{4/3}} (2g) + 1\right] \frac{V^2}{2g}$$

$$= \left[K_{ent} + \frac{n^2L}{2.21R^{4/3}} (2g) + 1\right] \frac{8\,Q^2}{\pi^2\, gD^4}$$

$K_{ent} \simeq 0.5$ for a square edged entrance

$\simeq 0.1$ for a well rounded entrance

$n \simeq 0.012$ for concrete pipe

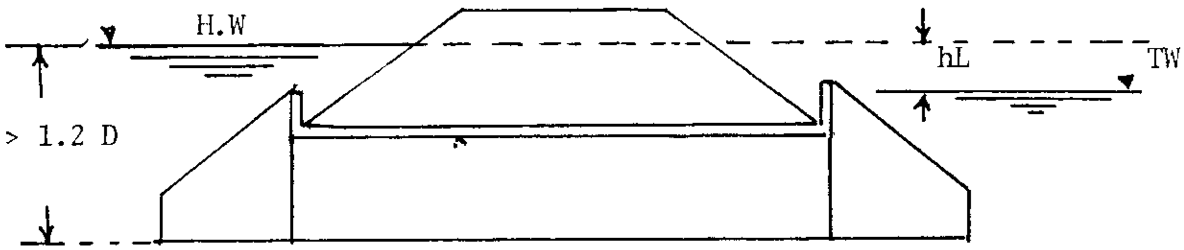
$\simeq 0.024$ for corrugated steel pipe

$R = \frac{D}{4}$ = hydraulic radius, feet

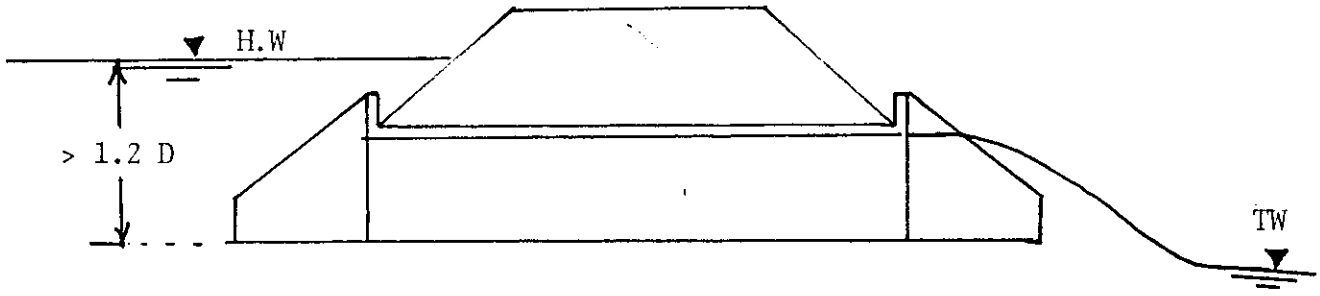
Q = flow, Cu ft/sec

D = diameter, feet.

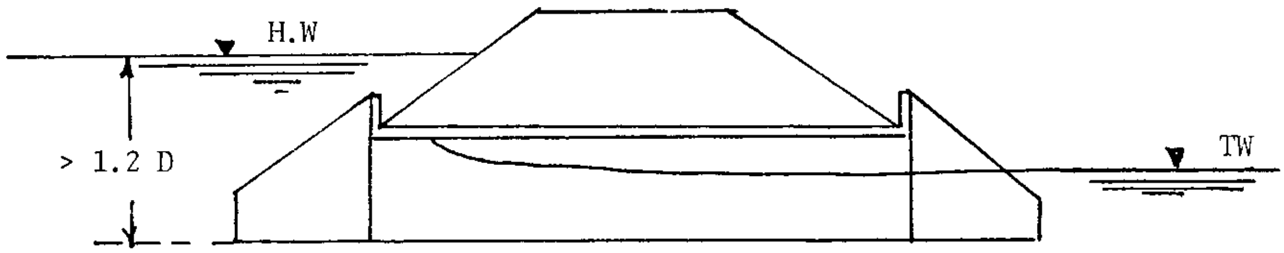
g = 32.2 ft/sec.^{2}



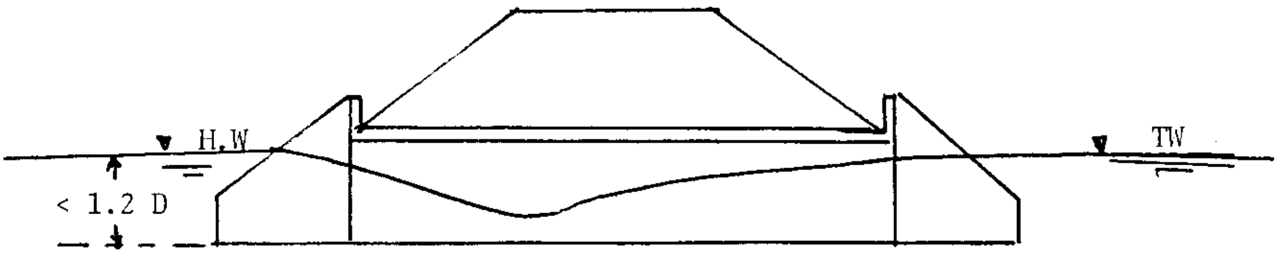
(a) Submerged inlet and outlet



(b) Full pipe flow with free outlet



(c) Partially full culvert



(d) Unsubmerged inlet and outlet

Figure 7. Hydraulic Operation of Culverts

NOTES

In the metric system:

$$h_L = [K_{ent} + \frac{n^2 L}{R^{4/3}} (2g) + 1] \frac{8Q^2}{\pi^2 g D^4}$$

(b) If the discharge carried in a culvert has a normal depth larger than the barrel height, the culvert will flow full, even if the tail water drops below that of the outlet as shown in Figure b.

(c) For partially full pipe, Figure C, the culvert discharge is controlled by the entrance conditions and is said to be under entrance control

$$Q = C_d A \sqrt{2 gh}$$

where C_d = 0.62 for square-edged entrance

= 1.0 for well-rounded entrance

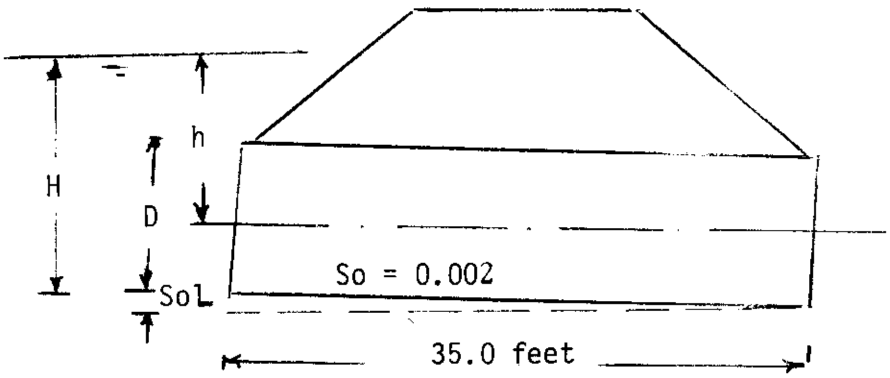
A = cross sectional area

h = hydrostatic head above all the center of the orifice

(d) When the hydrostatic heed is less than 1.2D, the culvert will flow under no pressure as an open channel

Example 5:

A corrugated steel pipe is used as a culvert that must carry a flow rate of 5.3 ft^3/sec and discharge into the air. At the entrance, the maximum available water head is 2.5 feet above the bottom. The culvert is 35 feet long and has a square-edged entrance and slope of 0.002. Determine the diameter of the pipe.



(a) Allowing full flow

$$h_L = H - D + S_oL$$

$$= 2.5 - D + 0.002 \times 35$$

$$= 2.57 - D$$

Also $h_L = \left(K_{ent} + \frac{n^2 L}{2.21 R^{4/3}} (2g) + 1\right) \frac{8\,Q^2}{\pi^2\, g\, D^4}$

$$= \left(1.5 + \frac{(0.024)^2 \times 35 \times 64.4}{(D/4)^{4/3} \times 2.21}\right) \frac{8\,(5.3)^2}{\pi^2\, 32.2\, D^4}$$

Combining $2.57 - D = \left(1.5 + \frac{8.24}{2.21^{4/3}}\right) \left[\frac{0.708}{D^4}\right] =$

$$\left[1.5 + \frac{3.73}{D^{4/3}}\right] \left[\frac{0.708}{D^4}\right]$$

By trial and error D = <u>1.25 feet ANS.</u>

(b) If the pipe flow is partially full, then the discharge is controlled by the entrance conditions only - In this case the head h is measured above the center line of the pipe.

$$h + \frac{D}{2} = 2.5 \qquad h = 2.5 - \frac{D}{2}$$

$$Q = 5.3 = C_d\ A \sqrt{2gh}$$

$$= 0.62\ \frac{\pi D^2}{4} \sqrt{2 \times 32.2\left(2.5 - \frac{D}{2}\right)}$$

D = <u>1.00 ft. ANS.</u>

Also a typical nomograph for culverts under outlet control is shown in Figure 8. The following example will illustrate its application.

NOTES

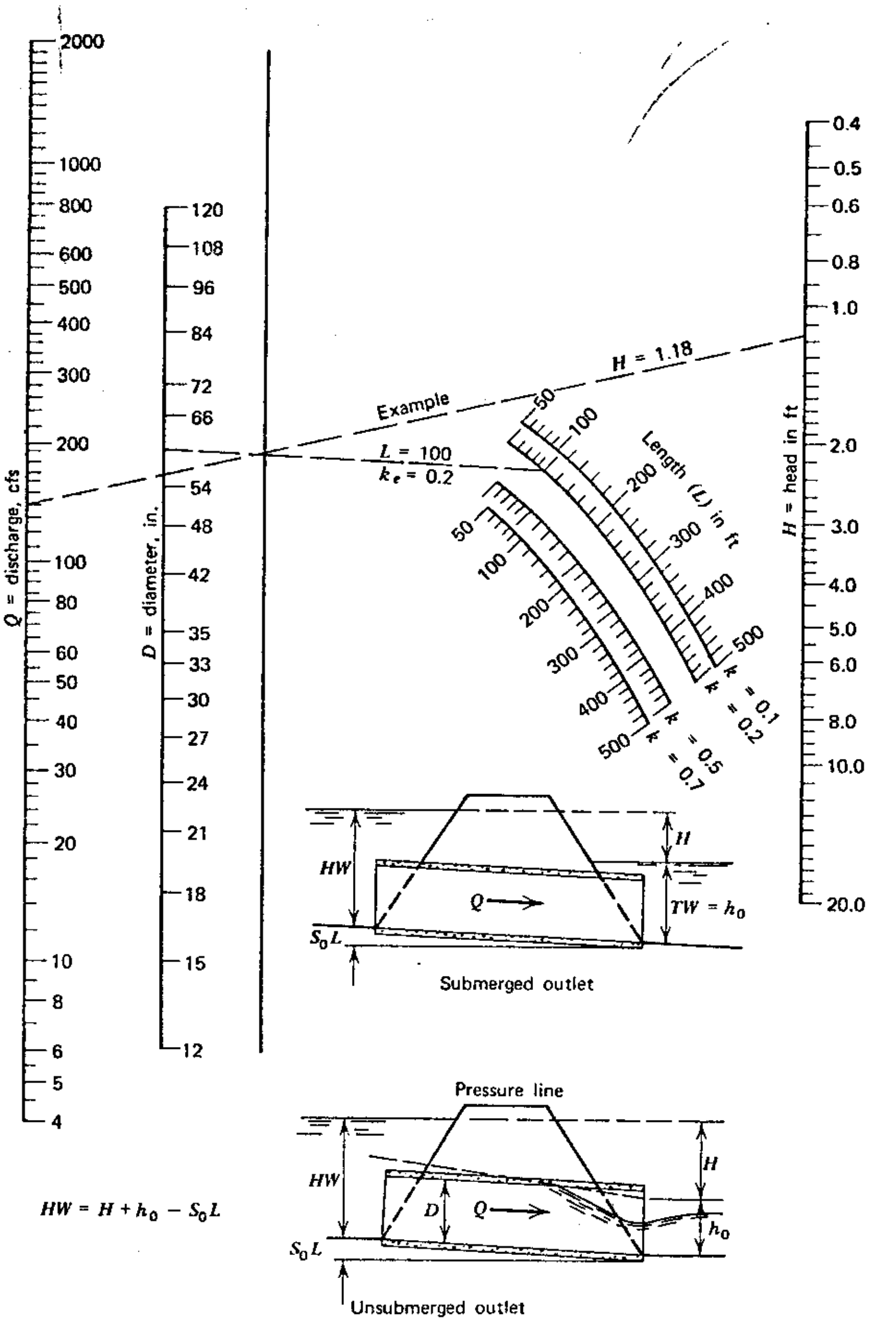


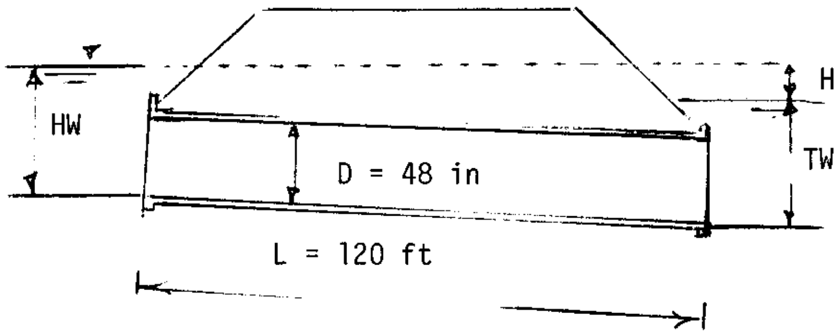
FIGURE 8 Typical nomograph for culverts under outlet control. (From *Handbook of Concrete Culvert Pipe Hydraulics*, Portland Cement Association, 1964.) Head for concrete pipe culverts flowing full; $n = 0.012$. Adapted from Bureau of Public Roads chart 1051.1.

$$\text{Equation: } H = \left[\frac{2.5204(1+k_e)}{D^4} + \frac{466.18n^2L}{D^{16/3}}\right]\left(\frac{Q}{10}\right)^2$$

H = head in ft.
k_e = entrance loss coefficient
D = diameter of pipe in ft.
n = Kutter's roughness coefficient
L = length of culvert in ft.
Q = design discharge in cfs.

Example 6:

A 48 in. diameter concrete culvert is 120 ft. long and its entrance loss coefficent if $K_e = 0.2$. Determine the head loss in the culvert if the discharge is 100 cfs and the pipe is flowing such that both ends are submerged.



NOTES

Solution:

The culvert is flowing full. Assume the concrete roughness n = 0.012:

$K_e = 0.2$

$L = 120$ ft.

$Q = 100$ cfs

$D = \frac{48}{12} = 4$ ft.

From the formula shown in Figure 8

$$H = \left[\frac{2.5204(1+k_e)}{D^4} + \frac{466.18n^2L}{D^{163}}\right](Q/10)^2$$

$$= \left[\frac{2.5204(1.2)}{4^4} + \frac{466.18(0.012)^2 \text{ X } 120}{4^{16/3}}\right]\left(\frac{100}{10}\right)^2$$

$$= (0.0118 + 0.0050)100$$

$$= 1.676 \text{ ft}$$

or from nomograph Figure 8 , we find H = 1.7 ft.

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SECTION 4
STORMWATER MANAGEMENT PRACTICES

NOTES

Purpose: Design procedures for stormwater management practices will be presented. Specific details for the designs of retention, exfiltration, swales and detention systems are developed. But first, a summary of some of the more commonly used technologies for control of stormwater is presented.

Introduction: A measure of effectiveness for management practices is the removal efficiency achieved for various operating conditions. Pollution removal efficiencies are variable among storm events and frequently are difficult to estimate. However, average yearly removal efficiencies can be estimated with a high degree of accuracy. Thus the efficiencies presented in the literature are in general based on a one year period of time.

There are at least three general methods for stormwater management: (1) permit, (2) structural, and (3) nonstructural. Permit to discharge implies in-depth design and operation considerations. This is usually done for relatively unknown removal efficiencies and impacts. Other methods meet common design and operating criteria that in general produce desired removal efficiencies. Structural methods are those which require some type of physical modifications within the watershed while non-structural do not require watershed modifications. Typical structural and non-structural methods are shown in Table 1.

Table 1. Some Management Practices

Structural	Non-Structural
Retention	Regulations
Detention	Public Awareness
Swales	Street Cleaning
Exfiltration Trench	Street Flushing
Underdrains	Catch Basin Flushing
Detention/Filtration	Gasoline Additives
Filtration	Lawn Chemicals
Porous Pavement	Erosion Control
Swirl Concentrator	
Fabric Filters	
Chemical Precipitation	
Watershed Storage	

NOTES

Some measures already have gained in popularity. In low water table areas, methods to increase percolation before surface water discharge are popular. Such methods are: retention (off-line by diversion), swales which are on-line, porous pavements for parking lots, exfiltration trench and watershed storage with discharge at a slow rate for percolation. In high water table areas, percolation is not possible and other methods are popular. Such methods are: detention (on-line), underdrains, detention with effluent filtration, and storage with some form of treatment.

To clarify the differences between detention and retention, the following descriptions are offered. Detention reduces flow rates and releases water for surface discharges while providing some evaporation and percolation over a period of several days. Retention does not release water for surface discharge but allows for percolation and evaporation. Some of the confusion results when detention facilities percolate a relatively small volume of water. The major difference between the two are in direct surface discharge; detention facilities directly discharge retention facilities do not.

Retention Designs: The use of a retention facility is often related directly to some sort of diversion system, usually controlled by hydraulic techniques, thus requiring no energy consumption. Because retention depends primarily on soil percolation rates, the soils in the vicinity of the pond must be tested to determine infiltration rates and to locate the water table depth.

From field research and computer simulations completed in the State of Florida, retention by diversion of the first flush of stormwater was shown to be 80-90% efficient at a relatively low cost up to approximately a diversion depth of 1/2 inch. The diversion depth is the depth of runoff water from the total watershed. The efficiency diversion depth curves for 2 watersheds are shown in Figure 1. These curves were developed from hydrograph related storm data and a computer simulation of 20 years of rainfall. The

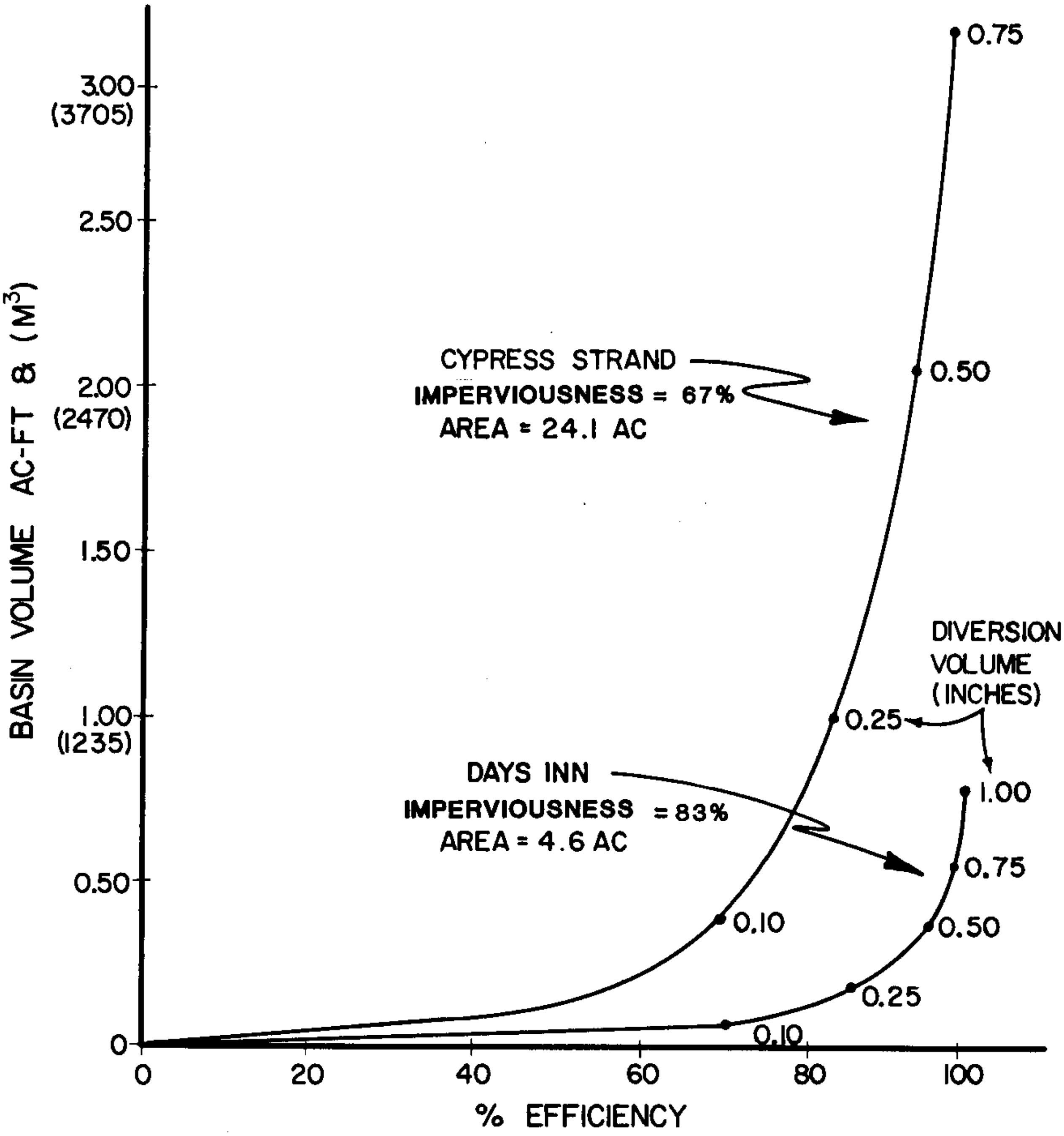


FIGURE 1. VOLUME OF PERCOLATION BASIN WITH AREA FOR BEST PERCOLATION CONDITION

curves incorporate the time variability of rainfall, diversion depth, and size of watershed. Note that the size of basin volume (percolation pond) increases rapidly beyond the 0.25/0.50 inch diversion volume.

NOTES

Example Problem: What is the runoff volume in acre-feet for a watershed of 24 acres if the diversion depth is 1/2 inch?

The Volume is:

$$V = \frac{(1/2)\ (24)}{12}$$

V = 1 Acre-feet

Surface and below ground percolation ponds are common retention systems. A common surface percolation pond with diversion is shown in Figure 2. A typical section of an underground system designed for the Lake Eola Watershed in Orlando is shown in Figure 3. The underground perforated pipe accepts runoff waters from a street and a parking lot. Another design of an underground percolation system for diverted stormwater sewer flows is shown in Figure 4. The existing storm sewer is a 24" square culvert. A concrete diversion wall is placed in a manhole to divert the first 1/4 - 1/2 inch of runoff waters for percolation.

Designs for retention systems are based on an estimate of rainfall excess and a specified diversion volume. Based on extensive field investigations and simulations using 20 years of rainfall data, average yearly efficiencies were estimated for fixed diversion volumes and are presented in Table 2 for on-site (small) watersheds.

TABLE 2.
PERCENT OF EFFICIENCIES/RETENTION

% Eff/Ret	% Diversion Volume (inches)
99	1.25
97	1.00
95	0.75
90	0.50
80	0.25

Assuming that a percolation basin will percolate stormwater before the next storm event, a volume of storage can be calculated using:

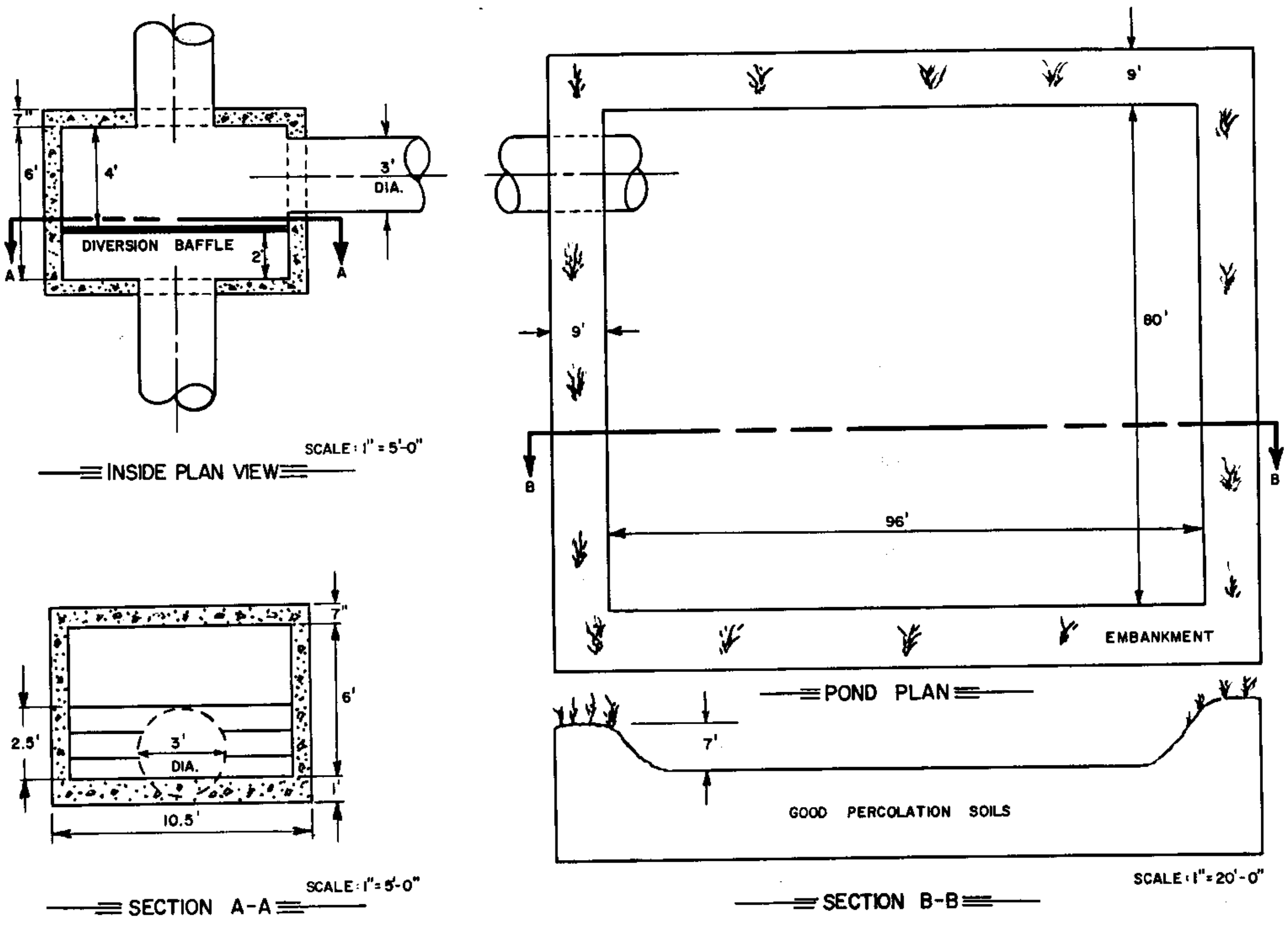


FIGURE 2. DIVERSION STRUCTURE / PERCOLATION POND,

NOTES

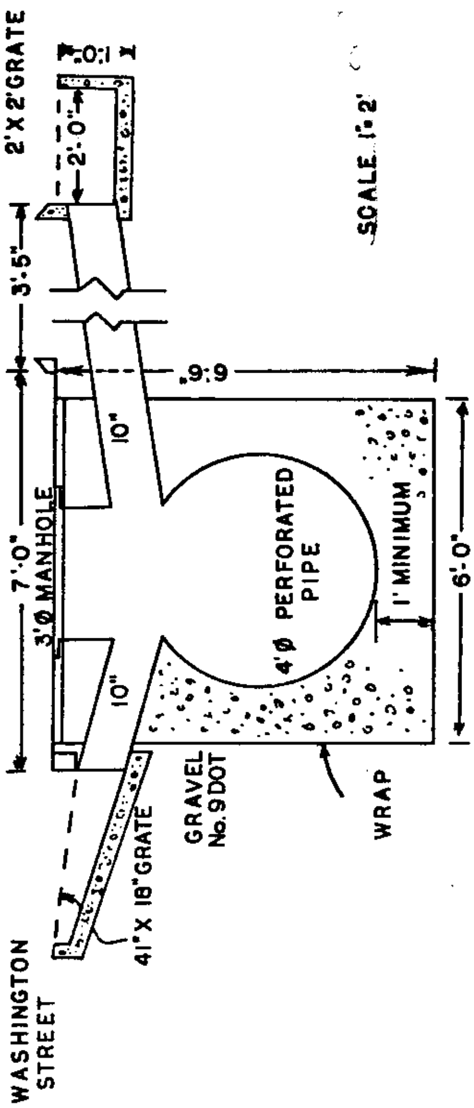


FIG 3. TYPICAL SECTION OF UNDERGROUND PERCOLATION SYSTEM

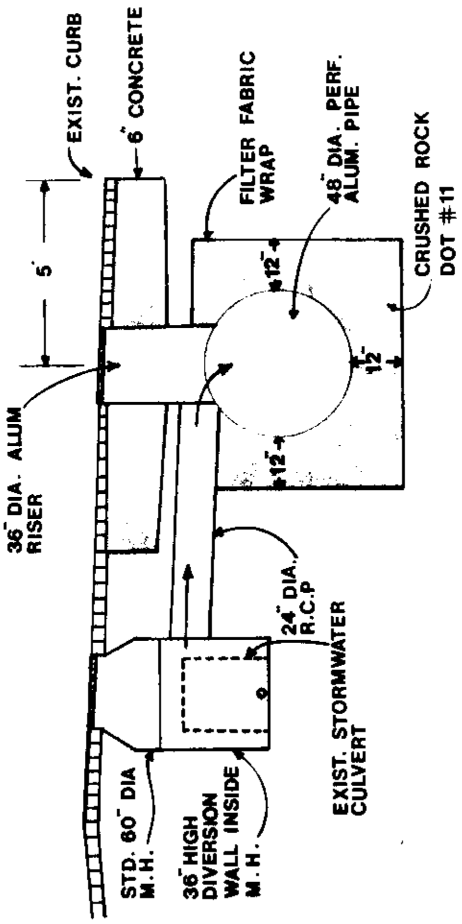


FIG. 4. UNDERGROUND PERCOLATION SYSTEM SECTION

NOTES

NOTES

$$V_m = \frac{A \times DI}{12}$$

Where: V_m = minimum basin volume (acre-feet)
A = contributing watershed area (acre)
DI = diversion volume (inches)
12 = conversion factor (in/ft)

Note that this is the minimum volume and must be increased to allow for the time variability of rainfall events, infiltration rates, size of watersheds, and basin depth (deep basins requiring a longer time to drain). The following design equations were developed for shallow basins and two removal efficiencies.

TYPE A SOILS

80% efficiency $V_I = 0.016\ A^{1.28}$ for impervious watershed

$V_5 = V_I(0.59 + 0.37\ CN/100)$ for composite land use

90% efficiency $V_I = 0.046\ A^{1.18}$ for impervious watershed

$$V_D = V_m + \left(\frac{V_5 - V_m}{4}\right)(D-1),\ 1 \le D \le 5$$

$$V_D = V_m,\ D < 1$$

Where: V_I = basin volume at 5 foot depth for impervious areas (acre-feet)
V_D = basin volume at depth D (acre-feet)
V_m = minimum basin volume (acre-feet)
V_5 = basin volume at 5 foot depth (acre-feet)
D = basin water depth (feet)

CN = average curve number (soil moisture II condition).

The rational formula has long been used to calculate peak discharges. Modifications of the formula can be used to also predict volume discharges. As an alternative to the above equations, one can use the rational formula to estimate retention volume for watersheds of up to approximately 100 acres in size. The Wanielista Design equations can be used for watershed sizes up to 500 acres in size. Beyond 500 acres a linear relationship can be used which relates size of retention in multiples of those calculated for 500 acres.

NOTES

The Wanielista design formulas and the rational formula do not consider the shape or the time of concentration of a watershed. The spatial variation of time of concentration of a watershed is an important parameter in the routings of stormwater runoff. This concept was first described by Sherman (1932), and named the Unit-Graph Method (Walsh, 1981).

With gaged rainfall/runoff data, the unit hydrograph is easily calibrated. Several synthetic unit-hydrographs (SCS-TR-20, SBUH, etc.) have been developed to predict hydrographs from rainfall events in the absence of gaged data. They have met with varying degrees of success. For developed conditions, the unit-hydrograph can easily be derived from the drainage plans.

As the first flush of stormwater pollution from the upper reaches of a watershed mixes with runoff from the longer reaches, it is generally diluted. This is because the first flush from the lower reaches has already passed by and the runoff from these lower reaches is now relatively "cleaner". Therefore, if a site specific unit hydrograph can be developed, the routing of the first flush from all areas of a watershed and its dilution can be predicted from an on-site loadograph utilizing mass balance since the unit hydrograph simply describes the spatial variation of time of concentration of the runoff water. Since the same runoff water carries these pollutants, should not the same spatial variation apply to the pollutant time of concentrations?

The shape of the unit-hydrograph has significant impact on the off-site or delivered loadograph. The following hypothetical examples will be used to illustrate this. Four SBUH "K" values (0.45, 0.3, 0.2 & 0.1) which are hydrograph shape factors, were used as the unit hydrographs for four separate watersheds. The on-site loadograph from Wanielista (1979) was used to describe the on-site first flush for all four watersheds. (See Figure 5). Note that 98% (almost complete) removal is obtained for 1.25 inches of runoff which will be used for comparative purposes.

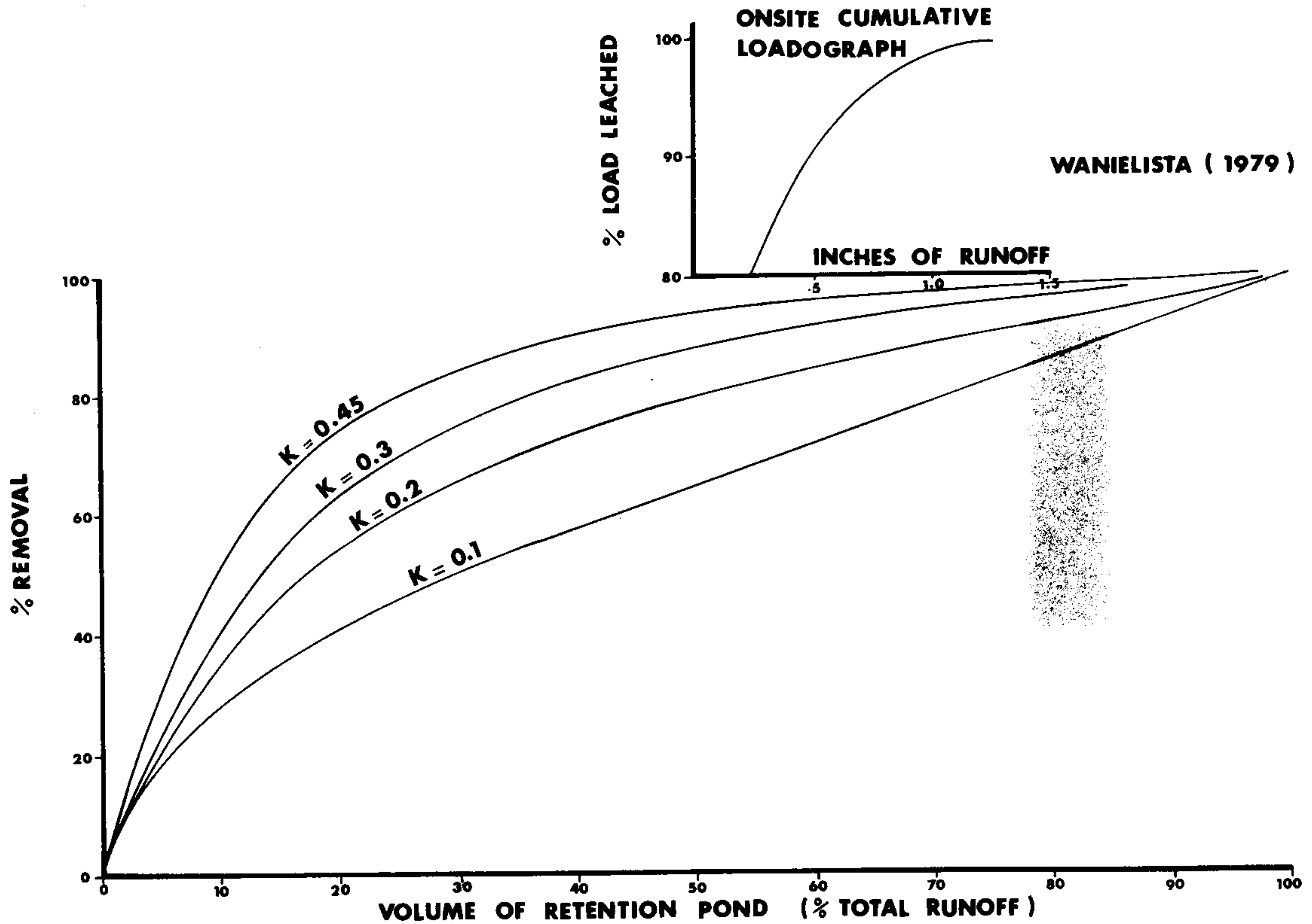


FIGURE 5. OFF-SITE CUMULATIVE LOADOGRAPH FOR VARIOUS "K" VALUES.

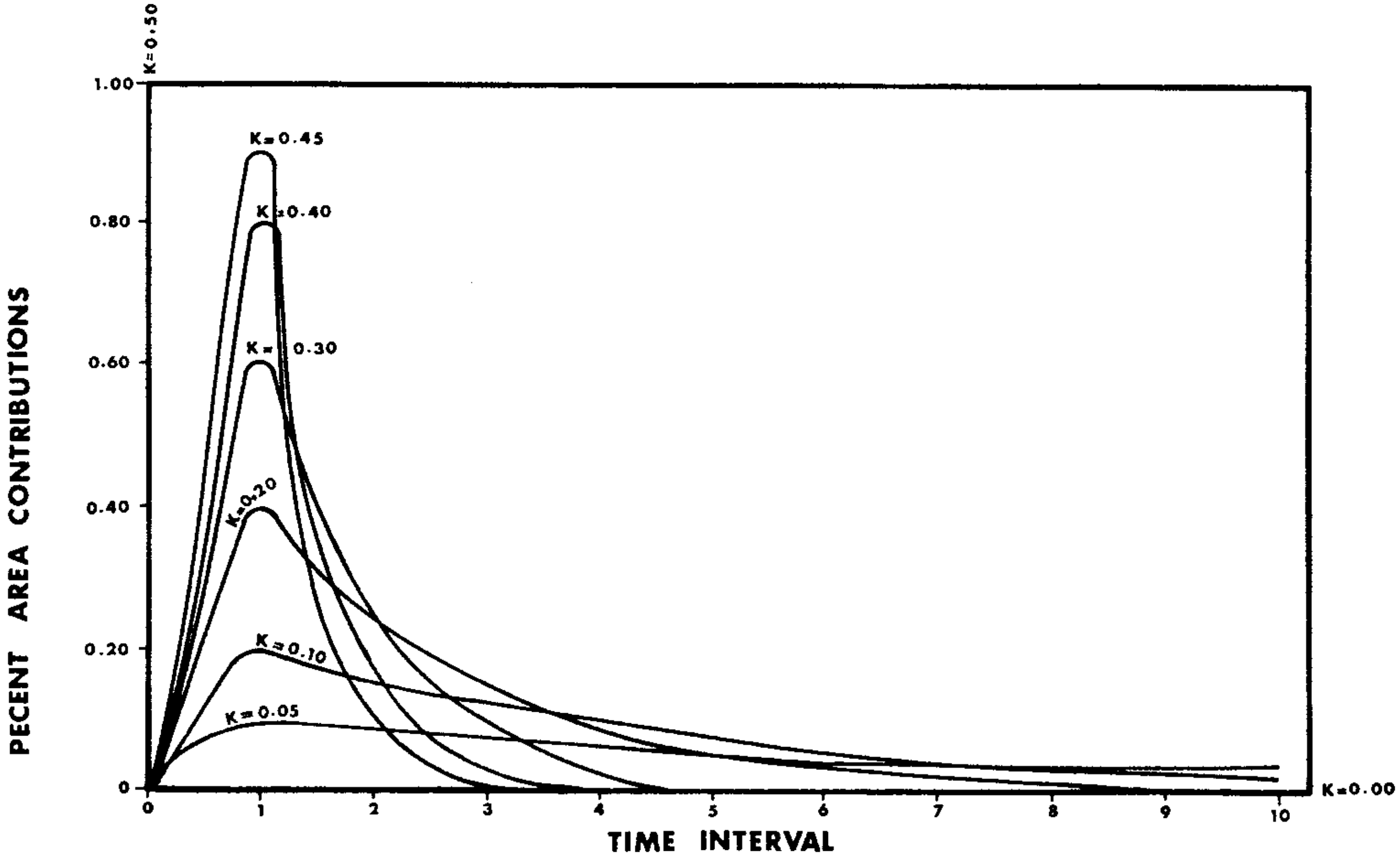


FIGURE 6. NORMALIZED SANTA BARBARA UNIT HYDOGRAPHS.

Figure 5 depicts the offsite loadographs for these hypothetical watersheds. The hypothetical hydrograph shapes are shown in Figure 6. From this figure, it can be seen that the shape of the unit hydrograph, indeed, has significant impact on the size of retention ponds. If, for example, it is determined that 80% of the pollutant mass must be retained, 75% of the first 1.25 inches of runoff (0.94 inches) must be retained for the watershed with the unit hydrograph described by SBUH "K-0.1" However, for the watershed with a unit hydrograph described by SBUH K=0.1 only 25% of the runoff (0.31 inches) must be retained to yield the same pollutant removal efficiency. Note that the percolation rate and the stochastic effects of storms are not considered. However, these factors can be considered in design.

Therefore, the design of a drainage system should take into account its implications as to the routing of the first flush phenomenon to the retention pond. This methodology is also beneficial in the site selection process of the pond itself.

Again, as is the use of the Rational Formula, Wanielista Design Equations, or the Unit-Graph Method, the designer must estimate the rational coefficient, composite CN, or the routing coefficient "K" respectively. The designer must be prepared to make and support the estimates.

NOTES

Example Problem

A watershed has 30 acres of watershed area and a composite CN number of 80. A regulation specifies treatment of the first 1/2" of runoff. Using the Wanielista Design equations, first half inch and the rational formula with C=0.70, calculate the size of a 5 foot deep retention pond in acre-feet. Comment on the results.

80%: $V_I = 0.016\ (30)^{1.28} = 1.24$ acre-feet

90%: $V_I = 0.046\ (30)^{1.18} = 2.55$ acre-feet

80%: $V_5 = 1.24\ (0.49 + 0.37\ (80/100)) = 1.10$ Ac-Ft

90%: $V_5 = 2.55\ (0.59 + 0.37\ (80/100) = 2.26$ Ac-Ft

and: V = (first half-inch) area/12

$V = (0.5)\ (30/12) = 1.25$ Ac-Ft

NOTES

and: V = C (1 inch) (area)/12

$$V = (0.7)\ (1)\ (30)/12 = 1.75 \text{ Ac-Ft}$$

Since the rational coefficient and the curve number are estimates for calculating rainfall excess, it would appear that the above variability in the estimates are as expected. The first half inch formula produces a pond which will operate to remove approximately 80% of the yearly pollutants. The pond volume estimated from the runoff from the first inch of rainfall (rational formula) will operate to remove between 80-90% of the average yearly pollutants. Thus, for a watershed of 30 acres, the rational formula produces acceptable results.

Swales

A swale is a drainage area that permits the percolation of stormwaters. Thus, a swale does not have "standing" water and has a cover crop (grass is usual) that must be harvested. To be consistent with retention pond efficiency, swales must be designed to remove at least 80% of the yearly runoff waters. Estimation of the percolation rate is critical. A double ring infiltrometer as shown in Figure 7 is generally used to estimate infiltration.

Field installation for estimating infiltration is done until the infiltration rate becomes constant. This constant rate is the limiting rate and is expressed in units of depth per unit time period.

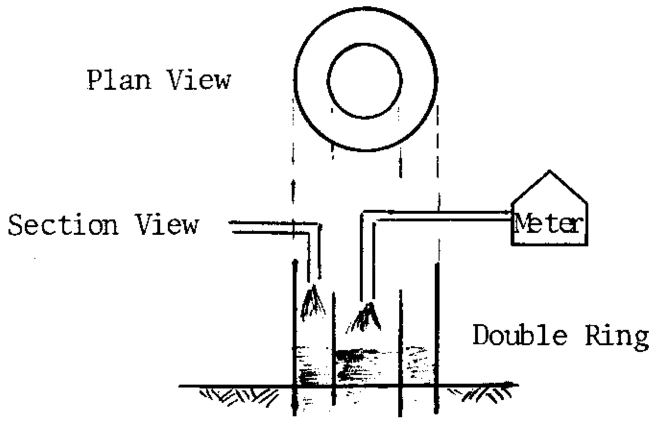


FIGURE 7. DOUBLE RING INFILTROMETER

Example Problem: for a watershed area of 8 acres and a runoff coefficient of 0.8, a stormwater management plan is submitted in which swales are used. The infiltration rate was estimated at 0.1 ft/min using a double ring infiltrometer. The swale area is 7200 square feet. Using a 2.5 in/hour storm can 80 percent of runoff waters be percolated?

NOTES

The rate of percolation is:

Volume infiltration rate = 0.1 x 7200

V_I = 720 Ft^3/min.

or V_I = 720/60(sec/min)

V_I = 12 CFS

The peak discharge for a 2.5 in/hr storm is:

Q_p = CIA

Q_p = 0.8 (2.5) (8)

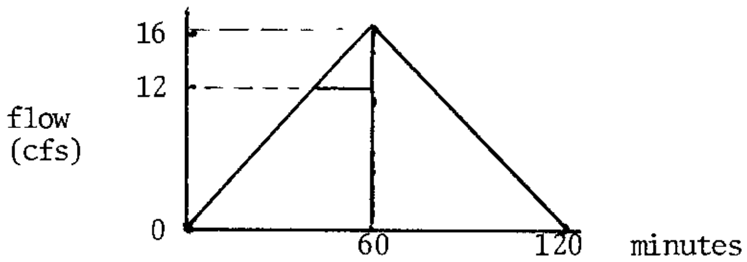
Q_p = 16 CFS

Assuming the rational formula with a time of concentration equal to 60 min, the resulting hydrograph would have a triangular shape with base equal to 120 minutes.

Does 80% of runoff waters from the storm percolate?

Infiltration will occur for percolation rates up to 12 CFS. Then the swale will overflow.

By similar triangles:



$$\frac{60}{16} = \frac{X}{4}$$

$X = \frac{1}{4}(60) = 15$ minutes

The infiltration volume is:

$\frac{1}{2}$ [b h] - $\frac{1}{2}$ [30 x 4]

$\frac{1}{2}$ [120(16)] - $\frac{1}{2}$ [30(4)]

$\frac{1}{2}$ [1920 - 120] =

$\frac{1}{2}$ [1800] = 900 CFS min.

900 CFS-min X 60 sec = <u>54,000 CF</u>

The total volume is:

$\frac{1}{2}$ [(120)(16)] 60 = <u>57,600 CF</u>

Assuming concentration of pollutants is proportional to flow, the percent removal is (54,000/57,600) 100 or 93.75%. If first flush (higher mass at start of runoff relative to latter runoff) were present, the removal rates would be higher.

This is a simplified design procedure and as such it most likely can be modified. Other methods for design may be substituted.

Exfiltration

Exfiltration systems are underground retention areas. When land cost is very expensive, it may be more economical to place the retention system underground. Design equations are derived considering a mass balance.

Rainfall excess = volume stored + volume exfiltrated

Volume stored is the summation of water within the pipe and within the void space of rock fill around the pipe

$$Vs = \rho L\ (WH - Ap) + ApL$$

Where Vs = Storage Volume, Ft^3

ρ = Porosity

W = Trench width, Ft

H = Height of trench, Ft - must be above water table

NOTES

NOTES

L = Length of trench, Ft

A_p = Area of pipe

If the porosity of the area above the water table is 0.25 and the pipe area is 1/3 (W) (H),

$$V_s = 1/4 \ (WHL - 1/3 \ WHL) + 1/3 \ WHL$$

Thus: $V_s = 1/4 \ WHL - 1/12 \ WHL + 1/3 \ WHL$

or $V_s = 1/2 \ WHL$

The volume exfiltrated is estimated from field studies and involves excavation of rechargeable pilot trench to the depth at which the proposed trench will be resident. The trench is illustrated as shown in Figure 8. The ends of the trench may be blocked (impervious) but it is not necessary

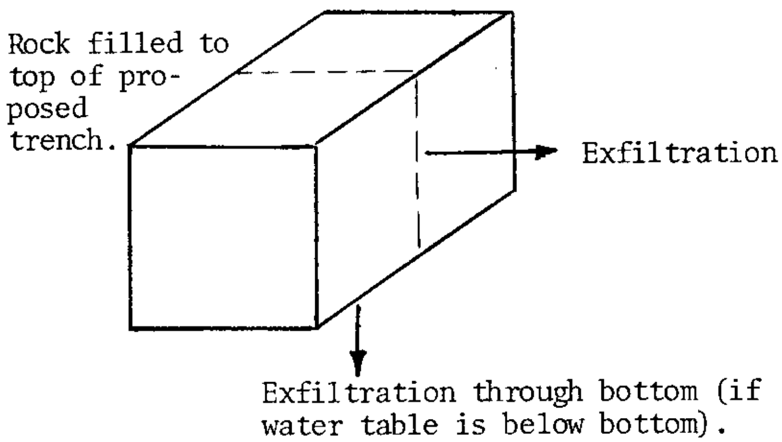


FIGURE 8. EXFILTRATION RATE SET UP

The infiltration rate is estimated in units of (depth/unit time) over the entire side and bottom wall areas. Thus, exfiltration rates are estimated as inch of water per minute per exfiltration area of pilot trench. The exfiltrated volume is:

$$V_E = h \ (\text{in/min} - \text{Ft}^2) \ (A_{sides} + A_{bottom} + A_{ends})$$

$$V_E = h \ (2HL + WL + 2WH)$$

Trenches are generally constructed in the shape of a square, so that the trench height is equal to the width and the length is much longer than the width or height, thus, the above equation can be simplified using:

$$W = H$$

and $W \cdot H$ is negligible

or $V_E = h \left(\frac{in}{min}\right)\left(\frac{1}{12}\frac{Ft}{in}\right)\left(60\ \frac{min}{hr}\right)\left(2\ WL + WL\right)$

and: $V_E = 15\ h\ WL$ (for one hour)

Example Problem

Design an exfiltration trench for a 20 acre area with a rational coefficient of 0.5 and a design storm of 2.5 inches. The trench length must be specified for a width and depth of 4 feet. Assume the exfiltrated volume is 1/3 of that calculated by formula because of suspected line blockage. Exfiltration rate was estimated as 0.1 inch/minute. (After dividing by 3)

Excess = Storage + Exfiltrate

$$\text{Excess} = C\ (A\ \text{acres})\ (R\ \text{in})\frac{1}{12}\ \frac{Ft}{in}\ \ 43{,}560\frac{Ft^2}{Acre}$$

Excess = CAR (3630)

Thus 3630 CAR = 0.5 WHL + 5 hWL

or 3630 CAR = 0.5 WL (H + 10h)

and $L = \frac{7260CAR}{W\ (H + 10h)}$

For R = 2.5 in $\quad L = \frac{18{,}150CA}{W(H + 10h)}$

For C = 0.5, A = 20 $\quad L = \frac{18{,}150(0.5)(20)}{4(4+1.0)}$
W = H = 4
h = .1

L = 9075 feet of trench

It should be noted that many simplifying assumptions have been made to develop the answer to this example problem. It would be advisable to return to the basic mass balance and check the assumptions of the equation development before designing.

NOTES

Detention is primarily used for hydrograph attenuation (peak flow reduction). Removal efficiencies are difficult to estimate and depend on size of facility and the quantity of pollutants in settleable form. In general, for the same watershed conditions, as volume of storage and/or percent settleable solids increase, storage efficiencies increase. Average yearly removal efficiencies for suspended solids have been reported in the range of 20 - 60%. Detention pond designs for pollution control have not been successful in removing 80 percent of the average yearly mass. However, longer detention times and enhancement of littoral vegetation zones may help. This remains to be proven.

For hydrograph attenuation, runoff hydrographs must be estimated. The rational formula, SCS-dimensionless curves, Santa Barbara Urban hydrograph, unit-hydrographs, and others with routing and hydraulic techniques can be used.

The duration of the storm is important for the calculation of required detention storage. Using the rational formula and four different durations for the same watershed, Figure 9 is constructed. It is noted by example that the required detention volume is not always largest for a particular storm event. More than one storm event should be used for calculating the volume.

Example hydrographs for pre- and post-development conditions are illustrated in Figure 10. The designer can now estimate the size of detention pond as shown in Figure 10. However, this assumes that this storm duration produces the worst results.

Detention with Filtration

Detention facilities of typical current design store approximately one inch or more of runoff waters before discharging. After the storm events, the stored water is released without any additional treatment. The release time is slow & varies between a few hours to three days. Removal efficiencies for detention systems were estimated from laboratory column studies. The runoff waters were from the Lake Eola watershed,

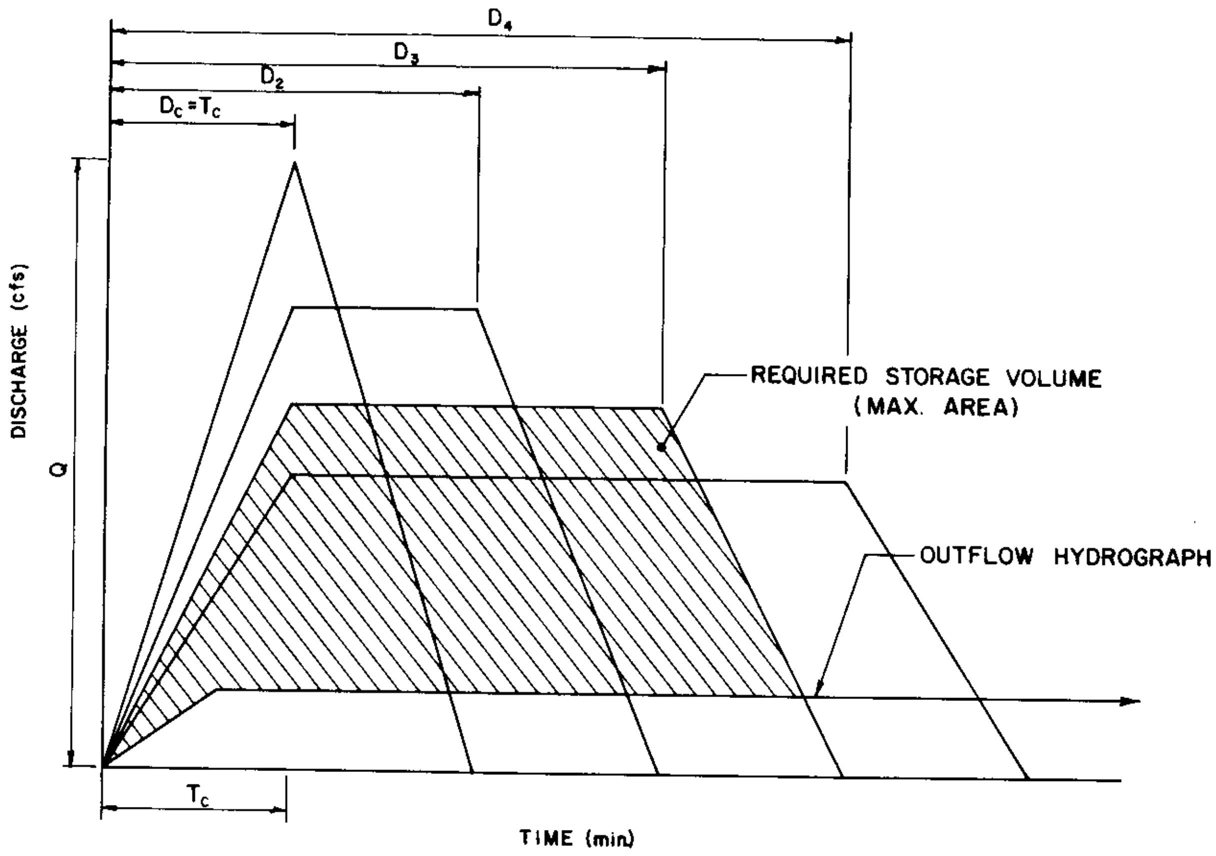


FIGURE 9: Detention Volume and Storm Duration

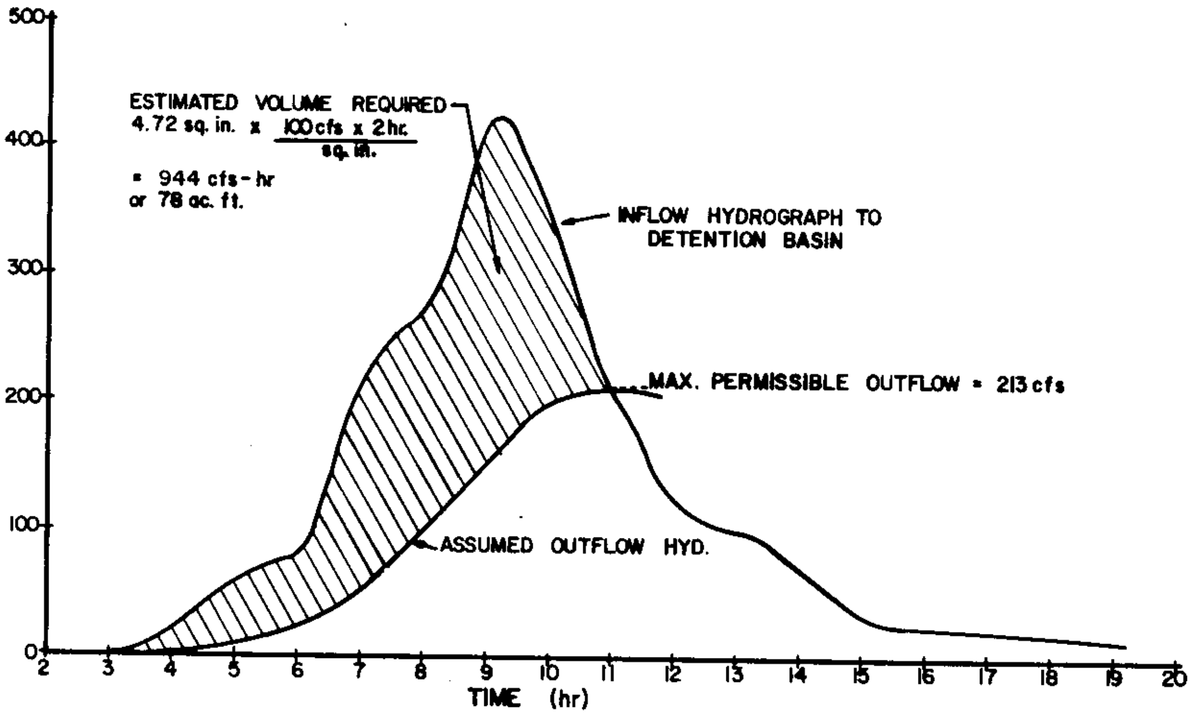


FIGURE 10. REQUIRED STORAGE VOLUME

Orlando, Florida. Because a significant quantity of the pollutants in the stormwater (over 50%) were in the dissolved or colloidal state, detention of at least 3 hours reduced the pollutants load by a maximum of only 50%. An overall average yearly reduction of 30-40% was estimated for these particular influent conditions. Of course, this level of efficiency is not equivalent to those obtained with retention facilities.

NOTES

Presently, if a detention system is designed for hydrograph attenuation and water quality control, storage volumes necessary must be increased somewhat to provide the extra treatment. However, by using a filtration system in conjunction with a detention pond, it may be possible to significantly increase the yearly average removal of pollutants. A conceptual increase in detention volume is shown in Figure 11. Given a design storm and the resulting hydrograph, the runoff water is stored and released to a filtration system. In Figure 11, it was assumed that the filtration rate is lower than the detention system release rate, thus additional detention volume was added. If the runoff hydrograph exhibits higher rates and/or volume for a fixed design, pollutant materials will flow over or around the infiltration system. For most areas of Florida pollutant removal can be enhanced if the first flush or the runoff from the first inch of rainfall can be both detained and filtered before discharge to surface waters. Typical filtration systems after detention are: (1) multi-media filtration,(2) bank and/or bottom infiltration through sandy soils, (3) natural vegetation, and (4) land-spreading.

Performance curves in terms of removal efficiencies and cost reflect the relative investments to achieve an equal efficiency. In a general way, Figure 12 reflects the cost of achieving a stated removal. Removal efficiencies will vary and are dependent upon local conditions of hydrologic and pollution characteristics (especially dissolved fraction and pH). Postulated in Figure 12 are effluent filtration of detention pond releases that will reduce the cost of achieving a stated removal or will remove more pollutants for a given cost.

Filtration of the detained waters is generally done with bank or bottom filtration. This will improve the removal efficiencies relative to

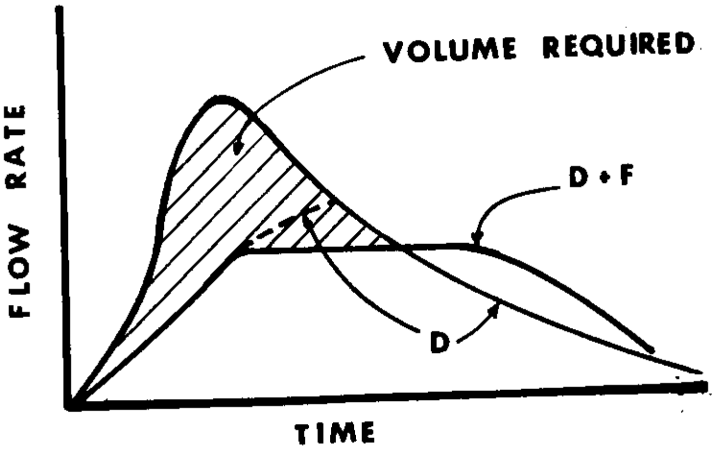


FIG.1 1 GENERALIZED DETENTION PLUS FILTRATION VOLUME REQUIRED

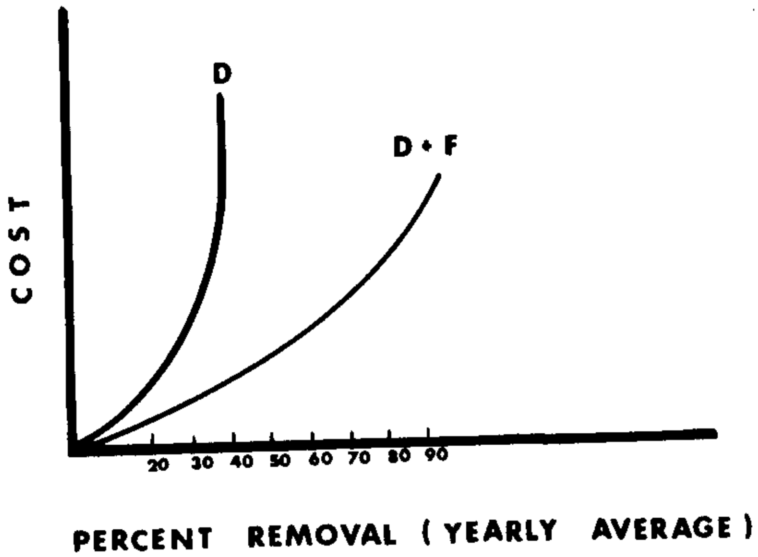


FIG.1 2 GENERALIZED COST VS AVERAGE YEARLY REMOVAL

D = DETENTION
D + F = DETENTION PLUS FILTRATION

NOTES

a detention alone design. Using a drainage rate of 10 inches/hour, the spacing of perforated pipe was estimated for one and two foot burial depths and different hydraulic conductivities. This is done using Hooghoudt's equation. The results are shown in Figure 13. A 4" diameter drain pipe was used.

An effective procedure for providing peak flow attenuation and at the same time providing treatment of the storm water runoff is by a detention, effluent filtration (D/EF) basin or lake. In this type of system, the first runoff from the first inch of runoff is made to pass through a perforated or slotted pipe underdrain located in the sides of the lakes or bottom of the basins as shown in Figures 14 and 15. The underdrains are encased in a sand/gravel mixture which filters the runoff as it passes through.

As an example, assume the runoff from the first is inch of rainfall is equal to 0.5 feet of depth in the basin or lake. Therefore, if the crest of the flood control structure from the basin is set at this distance above the normal low level of the lake or the bottom of the basin, outflow of this segment of runoff is necessarily through the perforated pipe drains. When located in the sides of a lake, the invert of the perforated or slotted pipe underdrain is set at or just below the Normal Low Water level as shown on Figure 14. Drain pipe size must be sufficient to remove the runoff from the first inch of rainfall.

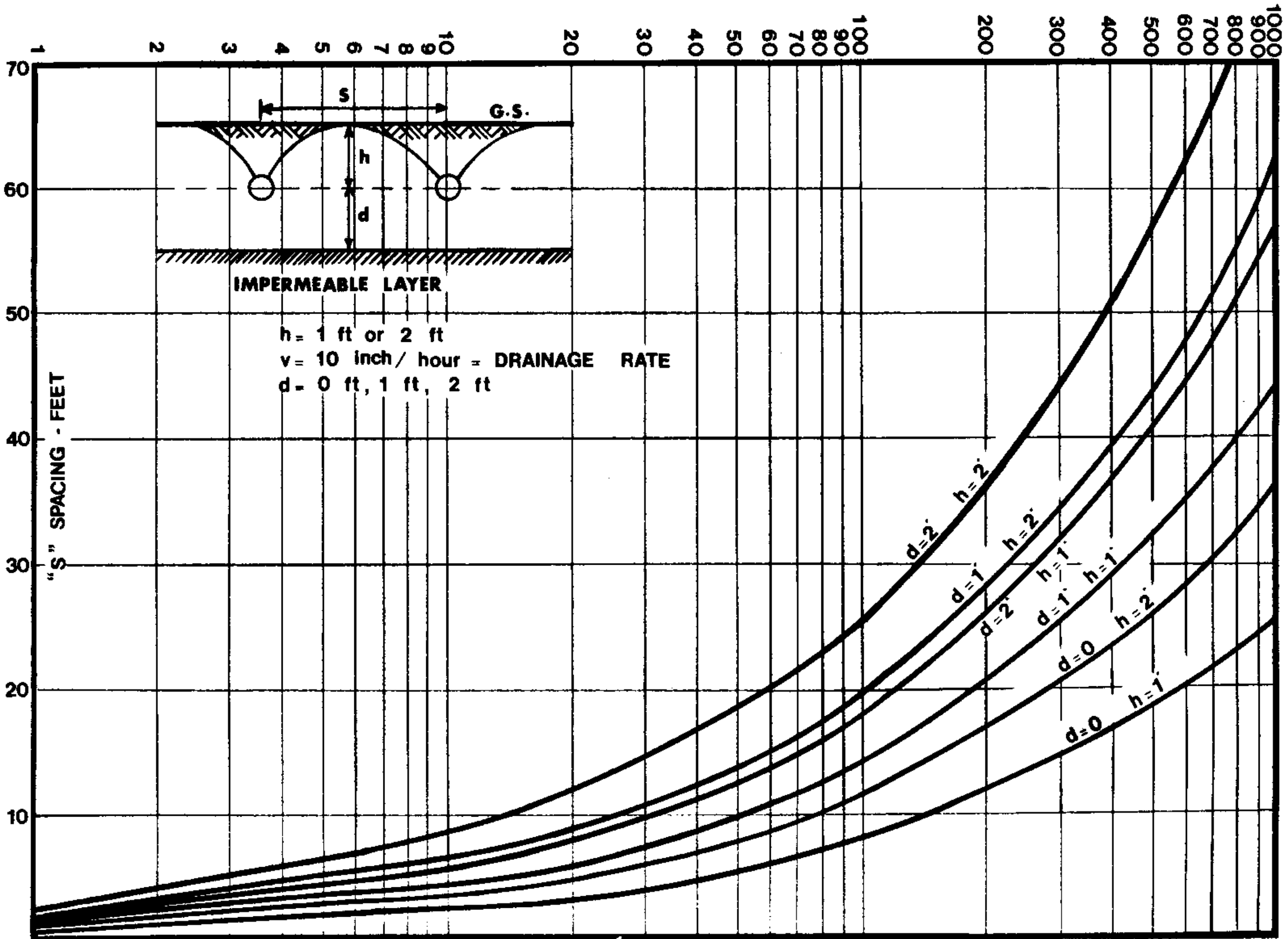
In the preparation of the subsequent sample problems presented in this section of the Manual, certain general hydrologic and hydraulic design criteria, methods and procedures were used and general design assumptions made. In order to better understand the sample problems, the storm water management system design criteria used in their preparation is subsequently presented. However, the information contained herein (particularly in the hydrologic field) may not be in accordance with the requirements of the local agency having jurisdiction which information is used solely for illustrative purposes.

Example:

Stormwater Management System Design Criteria

A. System Configuration

FIGURE 13. UNDERDRAIN SPACING AID



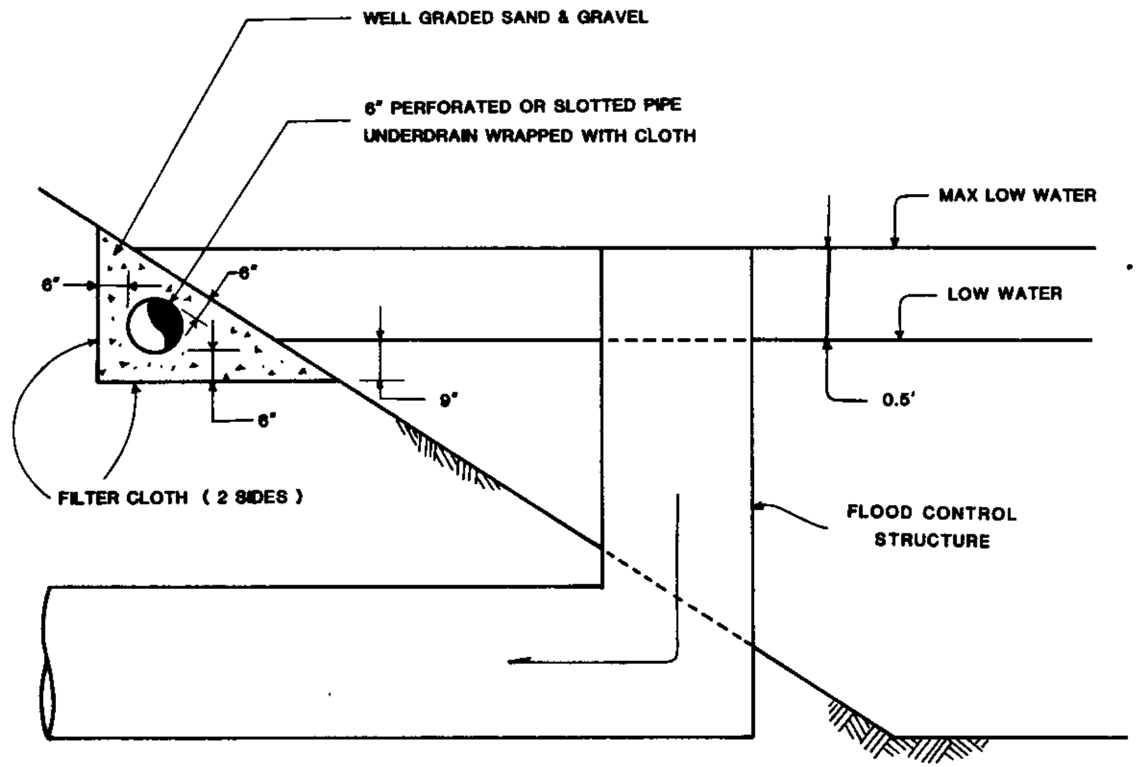


FIGURE 14. DETENTION / EFFLUENT FILTRATION.
(ON TOP OF LAKE)

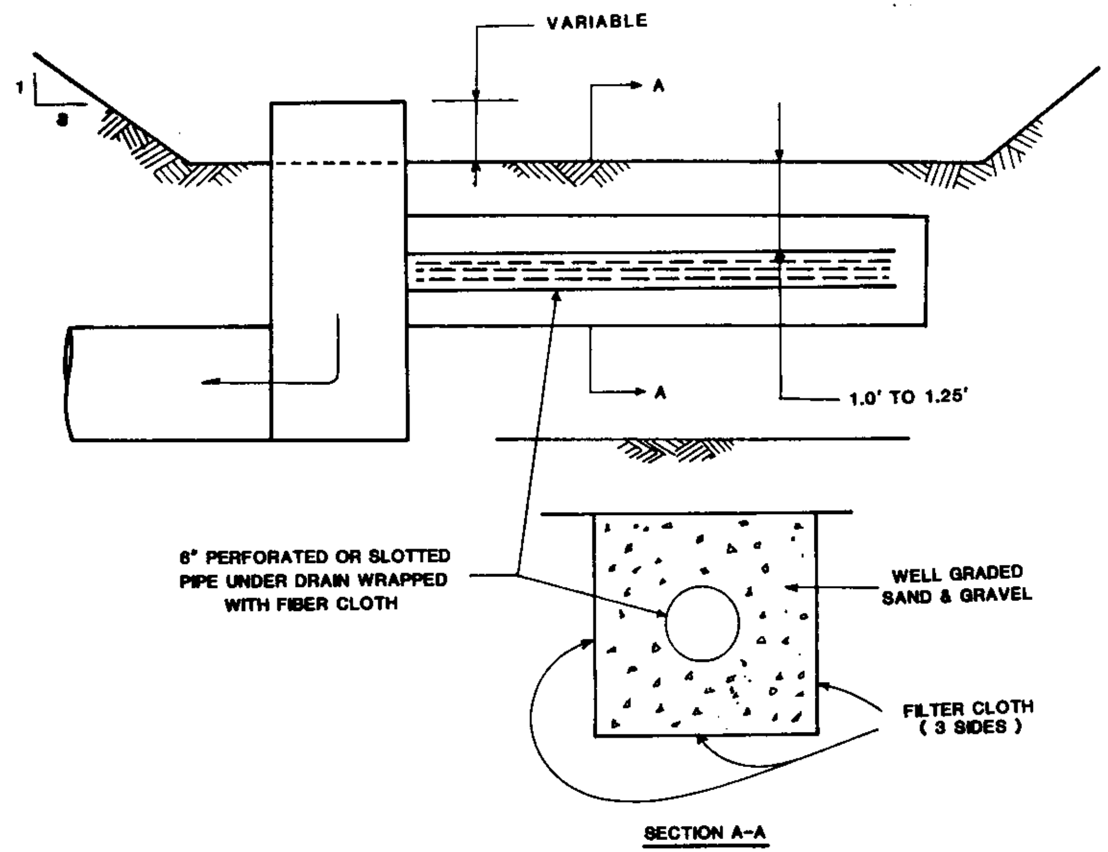


FIGURE 15. DETENTION / EFFLUENT FILTRATION.
(NORMALLY DRY)

NOTES

1) The overall storm water management system (plan) for the development shall consist of detention/effluent filtration (D/EF) lakes or basins, interconnected or separate, each D/EF lake or basin receiving runoff from a definative interior basin prior to the runoff entering the receiving waterway. The D/EF lakes or basins would act to reduce peak outflow from the development and provide treatment for the first one-half inch of runoff prior to entering the receiving waterway.

   The proposed D/EF lakes or basins will be controlled by drop structures (drop inlet spillway) or culverts which would limit overflow therefrom.

2) Treatment of the first 1/2 inch of runoff shall be accomplished by allowing it to pass through the side slopes of the D/EF lakes or the bottom of the basins to a perforated (slotted) pipe underdrain buried therein; the first one-half inch of runoff from a typical drainage basin being approximately equal to the first 0.5 to 1.0 foot of depth in the lake or basin above normal low water or the bottom of the basin (and below the crest of the outlet spillway or weir). Outflow of this last 0.5 foot of depth in the lake or basin would pass through the side slopes or bottom of the D/EF lakes or basins which action would "filter" the runoff, to the underdrain, and then out through the culvert. The spillway surcharge storage above the crest of the drop inlet spillway and the 0.5 foot below this crest will serve the three important functions of 1) providing the necessary reservoir storage so that peak runoff from the development will not be greater than peak runoff from the same area prior to development, 2) retaining the first one-half inch of runoff, and 3) provide long term detention storage which will settle out suspended solids and by doing so the pollutants attached thereto.

3) The D/EF lakes or basins and their drop inlet spillway/outlet pipe/underdrain infrastructures shall be so proportioned that a freeboard of ½-1 foot is maintained

above the maximum water surface in the lakes resulting from a 25 Year Frequency-24 Hour rainfall over the entire drainage basin.

NOTES

B. Pipe Sizes and Types, Side Slopes, etc.

1) All large culvert pipes shall extend out to the toe of slope - no headwalls shall be used. However, culvert pipe lengths shall be reduced as much as possible by the construction of special approach channels to the pipe culverts and drop inlet spillways.

2) All drop inlet spillway towers shall be placed sufficiently far into its coutributory lake so as to insure that the full periphery of the circular weir contributes.

3) All major culverts and drop inlet spillways shall be laid at zero grade - slope = 0.00 ft/ft.

4) The side slopes of lakes and canals shall be 2.5:1 or flatter.

C. Hydrology:

1) The D/EF lakes and basins and their drop inlet spillway/outlet pipe infrastructures shall be analyzed by the computation of composite design (inflow) hydrographs from the contributory drainage basins to each lake and then determining the affect of spillway surcharge storage by flood routing the composite design hydrographs through storage in the lake or basin.

2) The proposed overall storm water management system for the area to be developed (the proposed lakes and basins and their drop inlet spillway/outlet pipe infrastructures) shall be designed to pass runoff from a 25 Year Frequency-24 Hour Duration Design Rainfall of 9.0".

3) The 25 Year Frequency-24 Hour Duration Design Rainfall shall be assumed to fall in conformance with the SCS Standard Type II Distribution (Modified) and fall uniformly over the entire basin. The derived hyetograph for this design rainfall required for design flood hydrograph computation is shown

on Figure 16. One-half hour increments and summations for this design rainfall are listed on Table 3.

NOTES

4) Design flood (inflow) hydrographs shall be computed by the HNV-Santa Barbara Urban Hydrograph Method (HNV-SBUH Method), or other acceptable method. The actual selected procedure for hydrograph computation shall reflect the impact of the directly connected impervious area of the basin.

5) The time of concentration used for computing the design inflow hydrographs to each lake or basin shall be the largest travel time of any of the interior drainage basins contributing thereto.

6) The total impervious area and directly connected impervious area of each basin shall be determined by a sampling and weighting process.

7) In the computation of Design Hydrographs by the selected design procedure, a wet but not saturated antecedent moisture condition shall be assumed.

D. Hydraulics:

1) Each of the proposed D/EF lakes or basins and its drop inlet spillway/outlet pipe infrastructure shall be designed in accordance with the accepted hydraulic procedures and practices, formulas and equations.

2) Outflow hydrographs from the lakes and basins shall be determined by flood routing through reservoir storage in the lake or basin utilizing the modified Puls method with drop inlet spillway (outflow structure) sizes dependent on the maximum allowable water surface elevation in each lake or basin.

3) Drop inlet spillway sizes shall be determined such that the resultant headwater in the culvert portion of the drop inlet spillways will not drown weir flow over the crests of the circular towers - rating curves for drop inlet spillways shall be dependent on weir action alone.

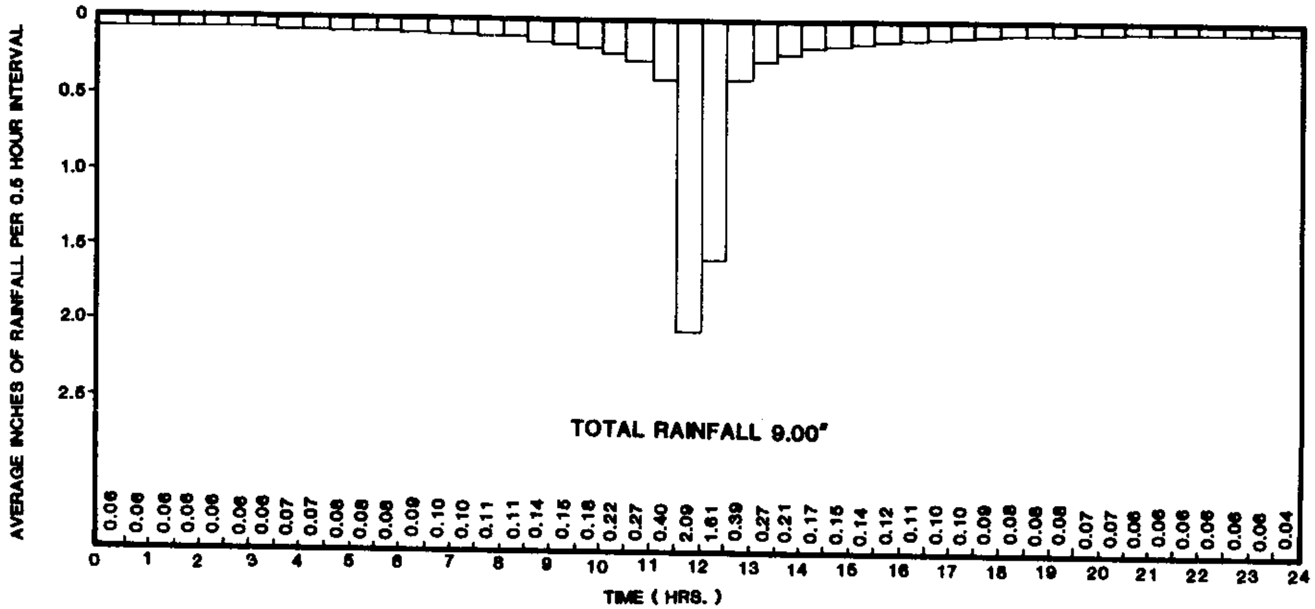


FIGURE 16. 25 YR-24 HR. DURATION RAINFALL - 30 MINUTE INCREMENTS.

TABLE 3

25 YEAR FREQUENCY
24 HOUR DURATION RAINFALL
ΣP = 9.0" - SCS TYPE II DIST (MOD)
30 MINUTE INCREMENTS

Time (Hrs)	ΣP (in)	ΔP (in)	Time (Hrs)	ΣP (in)	ΔP (in)
0					
0.5	0.05	0.06	12.5	6.37	1.61
1.0	0.11	0.06	13.0	6.76	0.39
1.5	0.17	0.06	13.5	7.03	0.27
2.0	0.23	0.06	14.0	7.24	0.21
2.5	0.29	0.06	14.5	7.41	0.17
3.0	0.35	0.06	15.0	7.56	0.15
3.5	0.41	0.06	15.5	7.70	0.14
4.0	0.48	0.07	16.0	7.82	0.12
4.5	0.56	0.07	16.5	7.93	0.11
5.0	0.64	0.08	17.0	8.03	0.10
5.5	0.72	0.08	17.5	8.13	0.10
6.0	0.80	0.08	18.0	8.22	0.09
6.5	0.89	0.09	18.5	8.30	0.08
7.0	0.99	0.10	19.0	8.38	0.08
7.5	1.09	0.10	19.5	8.46	0.08
8.0	1.20	0.11	20.0	8.53	0.07
8.5	1.31	0.11	20.5	8.60	0.07
9.0	1.45	0.14	21.0	8.66	0.06
9.5	1.60	0.15	21.5	8.72	0.06
10.0	1.78	0.18	22.0	8.78	0.06
10.5	2.00	0.22	22.5	8.84	0.06
11.0	2.27	0.27	23.0	8.90	0.06
11.5	2.67	0.40	23.5	8.96	0.06
12.0	4.76	2.09	24.0	9.00	0.04

Example Problem #1: | NOTES

General:

A 47.6 acre flat, sandy area (drainage basin) will be developed as a medium density residential area. As part of the amenities of the development, and for the purposes of obtaining fill, a 4.5 acre lake, approximately six feet in depth, will be constructed in the drainage basin, the upper portion of which lake will also act as a D/EF basin to attenuate peak flows and retain the first one-half inch of runoff (=0.5' of vertical depth).

$$\underline{0.5} \text{ in} \times \frac{1/12 \text{ ft/in} \times 47.6 \text{ acres}}{4.5 \text{ acres}} = 0.44 \text{ ft} \quad \text{say } 0.5 \text{ ft.}$$

Outflow from the proposed D/EF basin (lake) would be by a drop inlet spillway (outlet pipe/underdrain infrastructure) as shown on Figure 17[1].

Land Use, Soils, etc.:

Based on a study of the preliminary land use plan for the development and type of proposed buildings, roadway network and widths, drainage system, etc., it was determined by sampling and weighting that 35 percent of the total basin area would ultimately be impervious (I_t=0.35) and 30% of the total basin area would constitute directly connected impervious area (DCIA=0.30).

Based on a study of the SCS County Soils Maps, it was further determined that the soil in the basin was essentially classified as Group B/D; Group D being that condition in the underdeveloped basin (predevelopment condition) when the groundwater table was high (August-September). However, it was felt that urbanization of the area would lower and keep the groundwater table sufficiently down so that a Group B condition could be assumed for design purposes (post-development condition).

Based on a preliminary layout of the proposed interior drainage system, the time of concentration of this basin was computed to be 39 minutes (=0.65 hours).

[1]The final actual configuration of the outflow structure shown on the figures was the result of several alternative trials of various sizes and types.

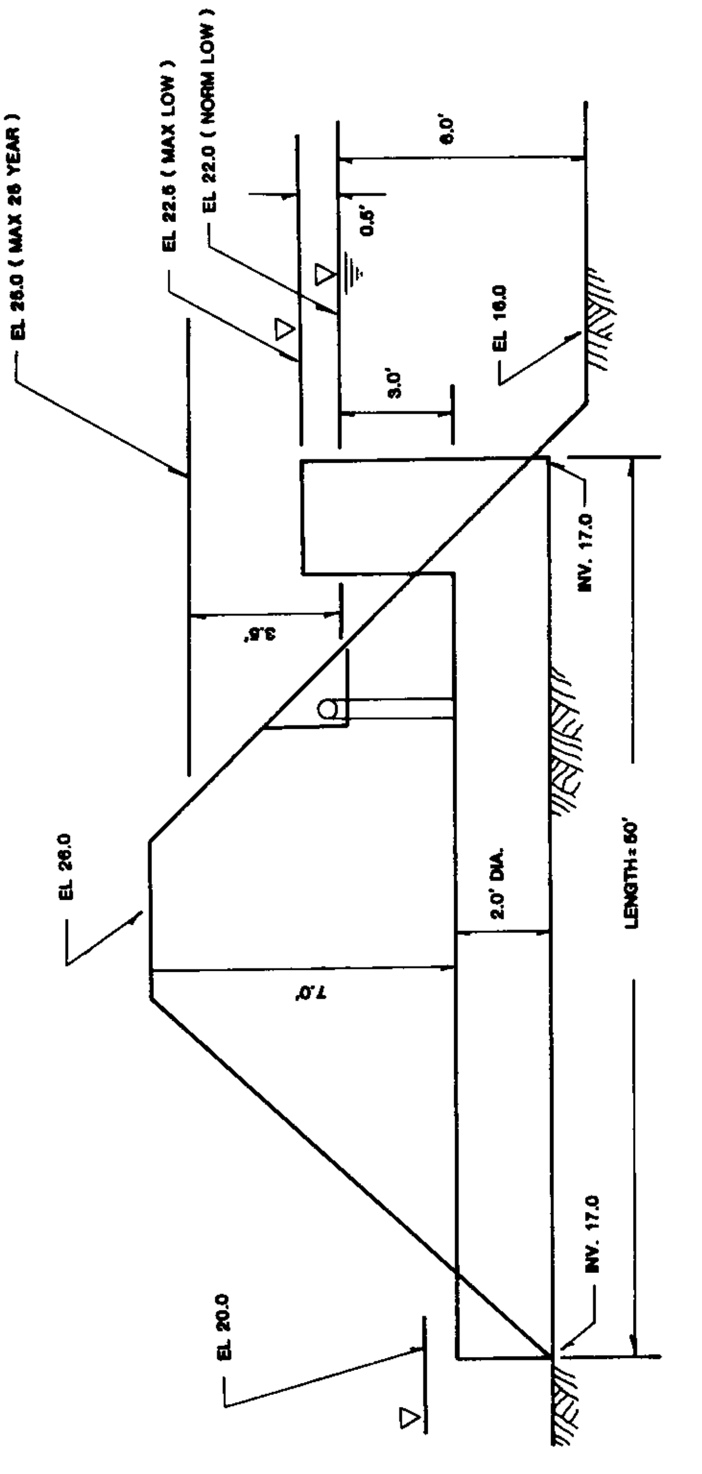


FIGURE 17. SAMPLE PROBLEM D/EF BASIN.

NOTES

Computation of Runoff Hydrograph:

In conformance with the Storm Water Management System Design Criteria, the runoff hydrograph from the 47.6 acre basin from the 25 Year Frequency-24 Hour Duration Rainfall was computed (by the HNV-SBUH Method) utilizing the 30 minute rainfall increments listed on Table 3 of the Criteria.

A plot of this inflow hydrograph is shown on Figure 18. In the computation of this hydrograph a "Rather Wet (Condition 3)" Antecedent Moisture Condition for a Group B soil was assumed.

The following parameters were input to the computer program used in the actual computation of the hydrograph by the HNV-SBUH Method[1].

Basin Area - Acres	= 47.6
Total Impervious Area - Decimal	= 0.35
Directly Connected Imp. Area - Decimal	= 0.30
Time Increment - Hrs.	= 0.5
Initial Infiltration Rate - In/Hr	= 8.0
Final Infiltration Rate - In/Hr	= 0.5
Accumulated Infiltration - Inches	= 2.8

Storage- Elevation Values:

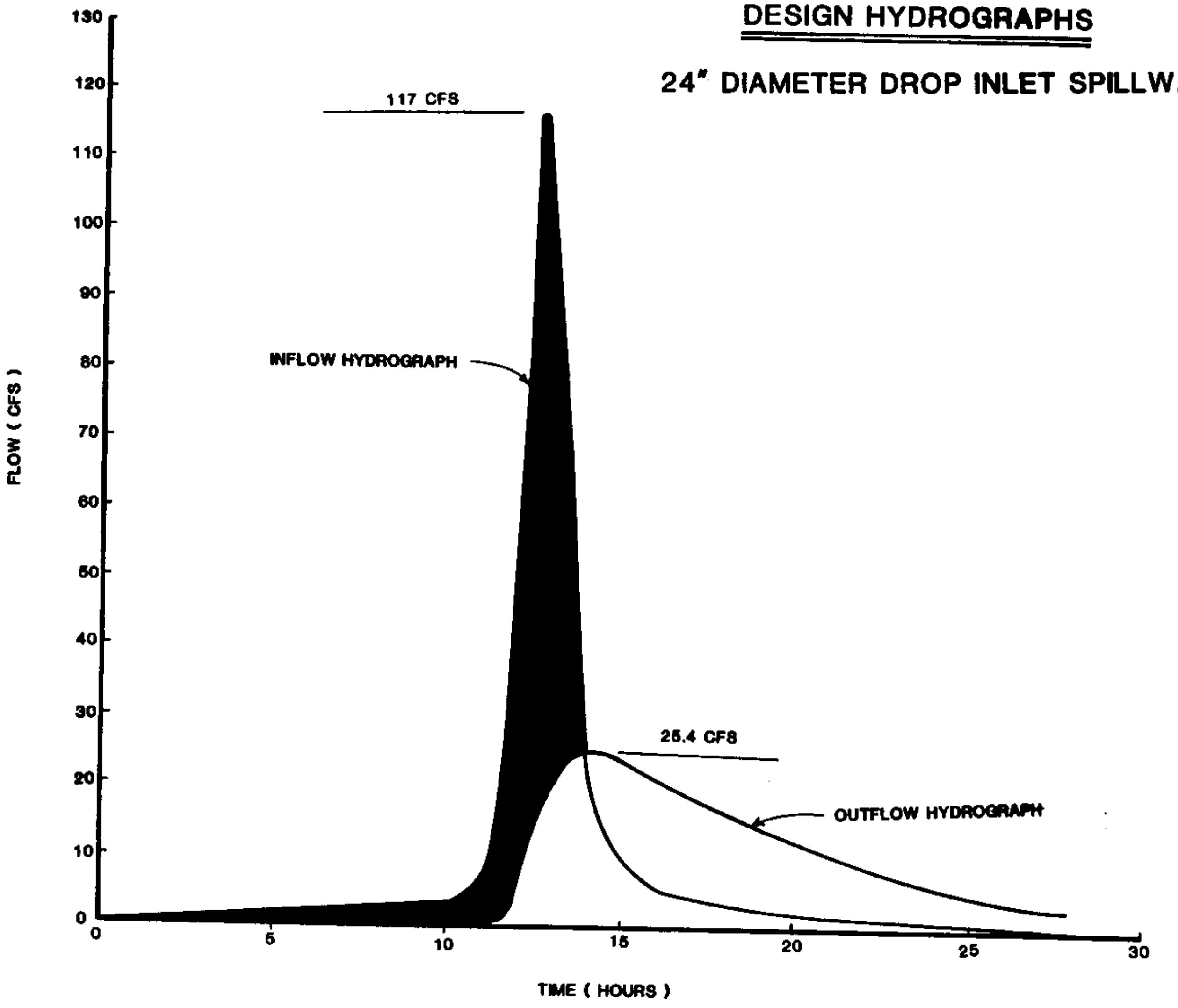
For ultimate purposes of flood routing through reservoir storage in the proposed D/EF basin (lake), the storage in acre feet at various depths above normal low water at 0.5 foot intervals of elevation were computed and are listed as follows. In this case, the lake was assumed to have vertical banks.

[1]Computer programs for the HNV-SBUH Method, flood routing, culvert design, water surface profiles, etc. are listed and explained in the book "Practical Basic Language Programs in Hydrology & Hydraulics Engineering" available from MICROCOMP, Civil Engineering Software Systems, 125 E. Ball Road, Anaheim, California 92605.

FIGURE 18.

DESIGN HYDROGRAPHS

24" DIAMETER DROP INLET SPILLWAY



NOTES

Elevation	Storage (Acre Ft.)
22.0	0
22.5	2.25
23.0	4.5
23.5	6.75
24.0	9.0
24.5	11.25
25.0	13.50
25.5	15.75
26.0	18.0

Elevation - Discharge Values:

The elevation-discharge (rating curve) values for the outlet structure were computed assuming weir control over the riser of the drop inlet spillway existed and that total outflow from the D/EF lake would be the sum of flow over the circular weir (top) and flow through the walls of the lake to and out via the perforated pipe underdrain; flow from both dropping down to the horizontal culvert outlet pipe. The horizontal culvert outlet pipe would be subsequently designed so that the head on the inlet of the culvert would not drown out the assumed weir flow condition.

The computation of the elevation-discharge (rating curve) values for combined outflow from the DE/F basin (over the weir and through the perforated pipe underdrain) are shown on Table 4. For this computation, it was assumed that the perforated pipe underdrain in the wall of the detention basin (out through which flows the first one-half inch of runoff =0.5 foot of lake depth) was 100 feet in length.

On Table 4, the capacity of the circular weir was computed by the Equation:

$$Q = Co(\pi D)H^{3/2}$$

where

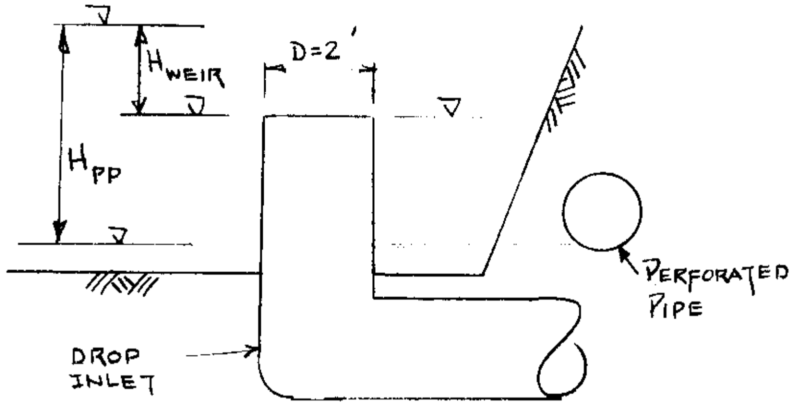
Q = the discharge over the weir - cfs

NOTES

TABLE 4

ELEVATION - DISCHARGE VALUE

Elev.	H (ft) Perf Pipe	H (ft) Drop Inlet	2.0' Drop Inlet H/R	2.0' Drop Inlet C_o	2.0' Drop Inlet Q (cts)	Q_{PP}	Q_{TOT}
22.0	0	0	0	0	0	0	0
22.5	0.5	0	0	0	0	0.5	0.5
23.0	1.0	0.5	0.50	3.35	7.44	1.0	8.44
23.5	1.5	1.0	1.00	2.03	12.75	1.5	14.25
24.0	2.0	1.5	1.50	1.37	15.81	2.0	17.81
24.5	2.5	2.0	2.00	1.00	17.77	2.5	20.27
25.0	3.0	2.5	2.50	0.9±	22.35	3.0	25.35
25.5	3.5	3.0	3.00	0.8±	26.12	3.5	29.62
26.0	4.0	3.5	3.50	0.7±	28.80	4.0	32.80



NOTES

πD = the weir length - ft

H = the total head over the weir - ft

C_o = the coefficient of discharge

The values for C_o were taken from Figure 283 on page 417 of the book "Design of Small Dams". (Bureau of Reclamation, U.S. Dept. of the Interior, Supt. of Documents, Washington, D. C., 1973.)

In computing the capacity of the 100 feet long perforated pipe underdrain, it was assumed that the underdrain would intercept 5 gpm (=0.01 cfs per foot of head per running foot of pipe.

Discharge-Storage Values (rating curve):

Discharge-storage values, required for flood routing through storage in the proposed D/EF lake, as calculated previously for the same elevation are listed below:

Table 5
Rating Curve

Discharge (cfs)	Elevation (ft)	Storage (acre-ft)
0	22.0	0
.5	22.5	2.25
8.4	23.0	4.50
14.3	23.5	6.75
17.8	24.0	9.00
20.3	24.5	11.25
25.4	25.0	13.50
29.6	25.5	15.75
32.8	26.0	18.00

Flood Routing:

Flood routing through storage in the D/EF basin (upper portion of the proposed lake above normal low water) was done by a computer utilizing the modified Puls method. The resultant routed outflow hydrograph is also shown on Figure 18.

The maximum volume of runoff stored in the lake was 13.50 acre feet which occurred at Hour 14.0 of the design storm at which time 15.4 cfs was being discharged by the outlet structure. This time (Hour 14.0) is the time at which the routed outflow hydrograph crosses the inflow hydrograph (=peak of the routed outflow hydrograph); the maximum volume stored in the lake being the shaded area between the inflow and routed outflow hydrographs.

NOTES

From the discharge-storage (rating curve) values for the drop inlet spillway, it can be immediately determined that the maximum water level in the lake during the design storm would be at Elevation 25.0.

Culvert Outlet Pipe:

For the purposes of the adequacy of a proposed 24" culvert outlet pipe, it was assumed that the tailwater at the outlet end of the culvert was at Elevation 20.0, 3 feet above its proposed invert at Elevation 17.0. As this would result in a full flow condition, the head loss H was computed by the equation on Figure 8 page 89 of this Manual. Since the culvert slope So=0, the Headwater HW on the culvert is computed by the equation:

HW = H + TW

With a total culvert length of 50 feet (length of riser neglected), an entrance loss coefficient of 0.5 and a Manning's Coefficient of 0.012 (for concrete pipe), the headwater HW on the 24" culvert was computed for various flows as shown below.

Q (cfs)	H (ft)	TW (Elev.)	HW (ft.)	HW (Elev.)
0	0	20.0	3.0	20.0
5	0.08	20.0	3.1	20.1
10	0.32	20.0	3.3	20.3
15	0.72	20.0	3.7	20.7
20	1.28	20.0	4.3	21.3
25	2.00	20.0	5.3	22.0
30	2.88	20.0	5.9	22.9

With a maximum peak outflow of 25.4 cfs at Hour 14.0, the maximum head on the culvert outlet pipe entrance is just over 5.3 feet in depth which does not drown out the assumed weir flow condition over the crest of the drop inlet spillway.

Example Problem #2

NOTES

General:

An 8.3 acre flat, sandy area (drainage basin) will be developed as a relatively high density residential area. Since the available development area is limited and no fill is required, storm water runoff will be placed into a shallow, normally dry 0.5 acre irregularly shaped detention/effluent filtration basin located in the adjacent green belt area which dry basin will provide reservoir storage to attenuate peak flows and "retain" the first one-half inch of runoff (=0.7 of vertical depth).

$$\frac{0.5 \text{ in} \times 1/12 \text{ ft/in} \times 8.3 \text{ acres}}{0.5 \text{ acres}} = 0.69 \text{ ft} \quad \textbf{Use 1.0 feet}$$

Outflow from the proposed dry D/EF basin will be by a drop inlet/outlet pipe/underdrain infrastructure consisting of a Florida DOT Standard Type D ditch bottom inlet to a 24" diameter reinforced concrete pipe which will convey flow to the receiving waters as shown on Figure 19.

Land Use, Soils, etc:

Based on a study of the preliminary land use plan for the area, roadway network, interior drainage system, etc., it was determined by sampling and weighting that 50% of the total basin area would ultimately be impervious (I_t=0.50) and that 40% of the total basin area would constitute directly connected impervious area (DCIA=0.40).

As in the previous sample problem, the soil in the basin was essentially Group B/D and again it was assumed that urbanization of the area would lower and keep the groundwater table sufficiently down so that a Group B condition could be assumed for design purposes.

Based on a preliminary layout of the proposed interior drainage system, the time of concentration of this basin was computed to be 18 minutes (0.30 hours).

Computation of Runoff Hydrograph:

In conformance with the Storm Water Management System Design Criteria, the runoff hydrograph from the 8.3 acre basin resultant from the 25 Year Frequency-24 Hour Duration Design Rainfall was computed (by the HNV-SBUH Method) utilizing the 30 minute

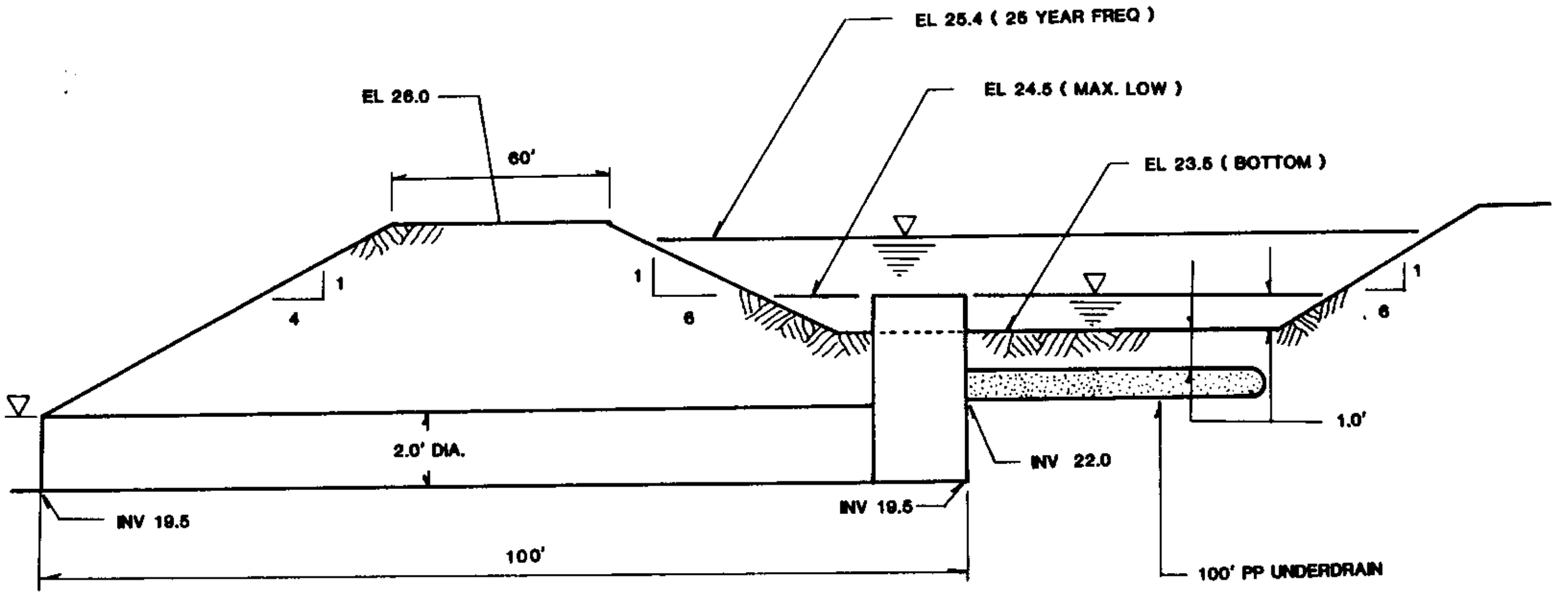


FIGURE 19. DETENTION / EFFLUENT FILTRATION BASIN OUTLET STRUCTURE

rainfall increments listed in Table 3. A plot of the computed inflow hydrograph to the D/EF basin as computed by the HNV-SBUH Method is shown on Figure 20. In the computation of this hydrograph as in the first sample problem, a "Rather Wet (condition 3) Antecedent Moisture Condition for a Group B soil was assumed.

NOTES

The following parameters were input to the computer program used in the actual computation of this design hydrograph by the HNV-SBUH Method:

Basin Area - Acres	= 8.3
Total Impervious Area - Decimal	= 0.5
Directly Connected Imp. Area - Decimal	= 0.4
Time Increment - Hrs.	= 0.5
Initial Infiltration Rate - In/Hr.	= 8.0
Final Infiltration Rate - In/Hr.	= 0.5
Accumulated Infiltration - Inches	= 2.8

Storage Elevation Values:

For ultimate purposes of flood routing through reservoir storage in the proposed D/EF basin, the total available storage in acre feet at 0.25 feet intervals above the basin bottom were computed and are listed as follows:

Table 6
Storage/Elevation

Elevation	Storage (Acre-Ft.)
23.50	0
23.75	0.74
24.00	0.30
24.25	0.48
24.50	0.68
24.75	0.90
25.00	1.14
25.25	1.40
25.50	1.68

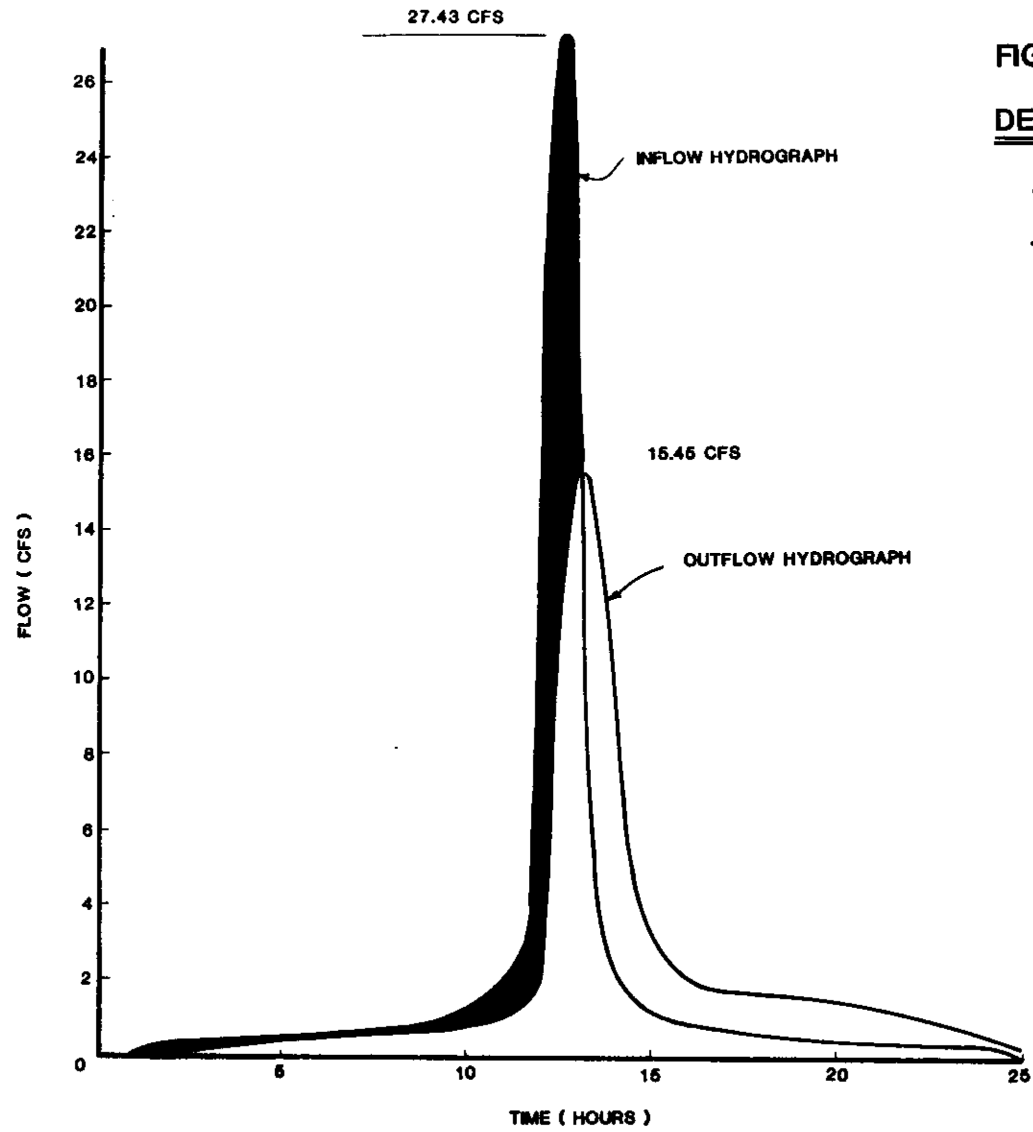


FIGURE 20.

DESIGN HYDROGRAPH

SUBBASIN NO. 12
24" DIA. RCP WITH
TYPE D INLET

Elevation-Discharge Values:

NOTES

The elevation-discharge (rating curve) values for the outlet infrastructure were computed as shown on Table 7. In the computation of the values, it was assumed that weir/orifice control through the top of the Type D drop inlet existed and that total outflow from the D/EF basin would be the sum of the flows through the top of the drop inlet and through the perforated pipe underdrain to be located in the bottom of the basin; both passing out to and through the pipe outlet as shown on Figure 6. The horizontal culvert outlet pipe would be subsequently designed so that weir/orifice flow through the grate of the drop inlet would not be drowned out at high flows.

In the computation of the Elevation-discharge values shown on Table 7, it was assumed that the 100 feet long perforated pipe underdrain would convey 5 gpm (=0.01 cfs) per running foot of pipe per foot of head on its invert.

Table 7
Elevation-Discharge

WS	Head (Feet)		Q (cfs)		
Elev.	PP Underdrain	Grate	PP Underdrain	Grate	Total
23.50	1.00	0	1.00	0	1.00
23.75	1.25	0	1.25	0	1.25
24.00	1.50	0	1.50	0	1.50
24.25	1.75	0	1.75	0	1.75
24.50	2.00	0	2.00	0	2.00
24.75	2.25	0.25	2.25	2.65	4.90
25.00	2.50	0.50	2.50	7.60	10.10
25.25	2.75	0.75	2.75	13.75	16.50
25.25	3.00	1.00	3.00	21.50	24.50

Flow through the grate was computed using the procedures outlined on pages 12-26 through 12-28 of the publication "Drainage of Highway Pavements". (Hydraulic Engineering Circular No. 12, U.S. Dept. of Transportation Federal Highway Administration, Supt. of Documents, Washington, D.C., March 1969).

NOTES

Discharge-Storage Values:

Discharge-storage values required for flood routing were taken from Tables 6 and 7 for the same elevation and are listed on Table 8 below:

Table 8
Storage - Discharge

Discharge (cfs)	Storage (Acre-Ft.)
1.00[1]	0
1.25	0.14
1.50	0.30
1.75	0.48
2.00	0.68
4.90	0.90
10.10	1.14
16.50	1.40
24.50	1.68

[1]Assumed equal to zero for purposes of flood routing.

Flood Routing:

Flood routing through reservoir storage in the basin was done as previously described for the previous sample problem. The resultant routed outflow hydrograph is shown on Figure 20. The maximum volume of runoff stored in the lake was 1.54 acre feet which occurred at Hour 13.0 of the design storm at which time 15.5 cfs was being discharged by the outlet infrastructure.

From the storage-elevation curve for the basin, it can be immediately determined (by interpolation) that the maximum water level in the basin would be at Elevation 25.4.

Culvert Outlet Pipe:

For purposes of investigating the adequacy of a proposed 24" culvert outlet pipe, it was assumed that the tailwater at the outlet end of the culvert was at Elevation 21.5 at Hour 14.0 of the design storm, 2 feet above its proposed invert at Elevation 19.5, at the crown of the pipe. As this would result in a full flow condition, the equation on Figure 8 pg. 89 was used to compute H. The equation HW=H+TW (So=0) was used to compute head loss HW through the structure.

NOTES

With a total culvert length of 100 feet, an entrance loss coefficient of 0.5 and a manning's Coefficient of 0.012 (for concrete pipe), the headwater HW on the culvert was computer for various flows as shown below illustrating correct assumptions.

Q (cfs)	H (ft)	TW (Elev.)	HW (ft.)	HW (Elev.)
0	0	21.5	2.0	21.5
5	0.1	21.5	2.1	21.6
10	0.4	21.5	2.4	21.9
15	0.91	21.5	2.9	22.4
20	1.61	21.5	3.6	23.1

Final Comparison

A final comparison of management practices using overall efficiencies and average capital costs is shown in Table 9. The efficiencies and costwere obtained from field measured runoff studies and actual installation costs. It is apparent that the higher efficiencies are associated with percolation/exfiltration systems.

Problems:

1. Estimate the retention volume for a 192 acre watershed in a residential area with 30% impervious area in "A" type soil. The water table is 5 feet below the surface. A retention pond 3 feet deep can be built, and 80% efficiency is specified.

   Answer:
   Sketch

   Assume: CN=57
   C = 0.4
   192 ac
   32% Imp.

   a. $V_M = \frac{A \times DI}{12} = \frac{(192)\ (.25)}{12} = 4$ ac-ft

   b. $V_I = 0.016\ (192)^{1.28} = 13.39$ ac-ft.

Table 9. Comparative Data Per Impervious Acre, Land-Cost Not Included

Management Practice	Impervious Area (% of Total	Overall[a] (%) Efficiency	Average ($/ac/% removal) Capital
Diversion/ Percolation[b]	70	99	25.00
Percolation Pond[c]	42	99+	36.30
Swales with Percolation[d]	23	92	28.40
Residential Swales[d]	20	80	26.08
Sedimentation[e]	50	50	19.20
Fabric Bag[f]	30	25	1.00
Advanced Sweeping[g]	70	68	30.40

[a]Yearly average of BOD_5, N, P and SS not discharged to surface waters.
[b]Designed 1-in. runoff diversion.
[c]Designed for 4-in. runoff diversion.
[d]80% of the rainwater percolates.
[e]Designed for 0.65 in. of runoff water from a watershed with runoff coefficient = 0.30
[f]Fabric bag replacement every two years.
[g]Assumed 60% nitrogen in particulate form.

$$V_5 = 13.39(0.59 + 0.37 \ \frac{57}{100}) = 10.72$$

NOTES

$$V_3 = V_M + \frac{V_5 - V_M}{4} \ (D\text{-}1)$$

$$V_3 = 4 + \frac{(10.72 - 4)}{4} \quad (3\text{-}1)$$

$$V_3 = 4 + 3.36 = \underline{7.36 \text{ ac-ft}}$$

c. $R = cP$ where $P = 1''$

$R = 0.40(1)$

$R = 0.40$ in

$$\text{Volume} = \frac{0.40 \text{ in x } 192}{12} = \underline{6.4 \text{ ac-ft}}$$

2. What is the maximum infiltration volume of a swaled area 10,000 feet long and four feet wide, if the rated infiltration is 0.2 ft/min? Consider a 60 minute infiltration time over the entire swaled area.

Answer:

Area of swale: 10,000 feet x 4 feet = 40,000 sq. ft.

Infiltrated volume rate= 0.2 ft/min.

Volume infiltrate = (40,000) (0.2 ft/min.) (60 minutes)

$$\underline{V_I = 480,000 \text{ ft}^3}$$

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