Section 4
Runoff Block

Block Description

Introduction

The Runoff Block has been developed to simulate both the quantity and quality runoff phenomena of a drainage basin and the routing of flows and contaminants to the major sewer lines. It represents the basin by an aggregate of idealized subcatchments and gutters or pipes. The program accepts an arbitrary rainfall or snowfall hyetograph and makes a step by step accounting of snowmelt, infiltration losses in pervious areas, surface detention, overland flow, channel flow, and the constituents washed into inlets, leading to the calculation of a number of inlet hydrographs and pollutographs.

The Runoff Block may be run for periods ranging from minutes to years. Simulations less than a few weeks will henceforth be called single event mode and longer simulations will be called continuous mode. With the slight exception of snowmelt, all computations are done identically for the two cases. The distinction between single event and continuous mode is kept mainly for ease of description and interpretation.

The overall catchment may be divided into a maximum of 200 subcatchments and 200 channel/pipes plus inlets. The user can modify these limitations by adjusting the variable NW and NG in the parameter statement of the INCLUDE file “TAPES.INC” and recompiling the program. Inlet flows and pollutographs may be placed on the interfacing file for input to subsequent blocks. However, these blocks have their own limitations on the number of inflow locations they will accept. See Section 2 for details. This section describes the operation of the Runoff Block and provides instructions on data preparation.

Program Operation

The relationships among the subroutines that make up the Runoff Block are shown in Figure 4-1. Subroutine RUNOFF is called by the Executive Block to gain entrance to the Runoff Block. The program prints “ENTRY MADE TO RUNOFF MODEL,” initializes all variables to zero, and

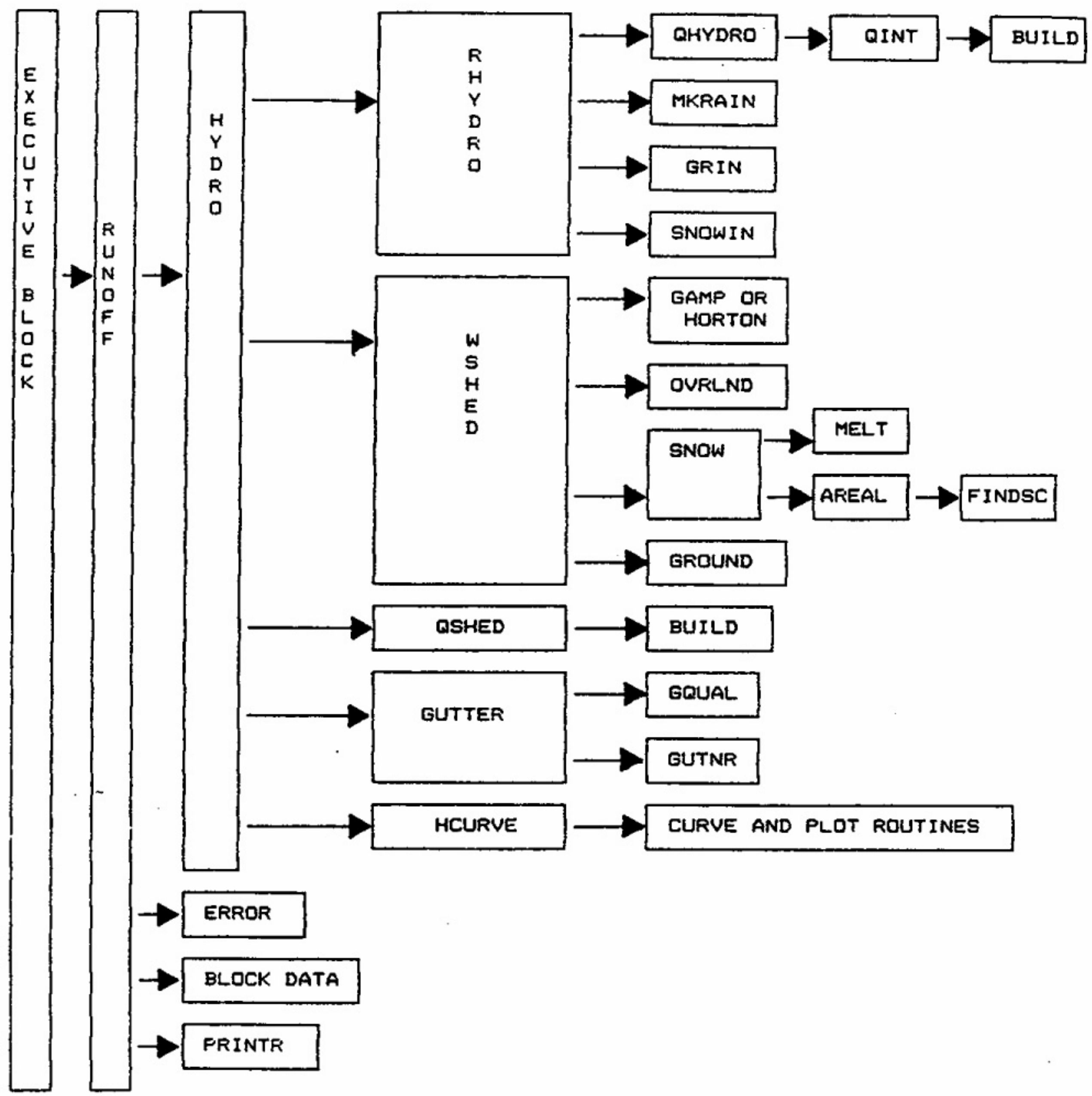


Figure 4-1. Structure of Runoff Block subroutines.

then calls subroutine HYDRO followed by PRINTR. Although BLOCK DATA is not an actual subroutine, it is automatically activated by RUNOFF and initializes some variables. Subroutine PRINTR reads the file headers, and then prints the table headings and results of the quantity and quality simulations.

Subroutine HYDRO computes the hydrograph ordinates and the watershed quality contributions with the assistance of 17 core subroutines, i.e., RHYDRO, GRIN, SNOWIN, QHYDRO, MKRAIN, QINT, QSHED, BUILD, WSHED, OVRLND, HORTON, GAMP, GUTTER, GUTNR, GQUAL, MELT, AREAL, AND FINDSC. RHYDRO reads the information concerning the inlet drainage basins. RHYDRO calls GRIN, MKRAIN, SNOWIN and QHYDRO to read groundwater, precipitation, snow information, and quality information, respectively. QINT and BUILD are then called to initialize the watershed constituent loads if water quality is simulated. HYDRO next sets up an ordering array to sequence the computational order for channel/pipes such that the computations proceed in a downstream direction.

HYDRO then computes the hydrograph ordinate for each time step by calling subroutine WSHED. WSHED calls either GAMP or HORTON to calculate infiltration. If snowmelt is simulated subroutine SNOW is called from WSHED. SNOW calls subroutines AREAL and MELT and subroutine FINDSC is called from AREAL. The runoff from a subcatchment is calculated by subroutine OVRLND and the subsurface flow contribution is calculated by subroutine GROUND. If quality is to be simulated, QSHED and BUILD are called to compute the watershed quality contributions from catchbasins, erosion, dust and dirt, and other sources. GUTTER is then called to compute the instantaneous water depth and flow rate for the channel/pipes and to route the flow. Water flowing into the inlet point is the sum of channel/pipe flow, direct drainage from subcatchments and direct groundwater inflow into the inlet. A continuity check is then made for the disposition of rainfall water in the form of runoff, detention, infiltration, and evaporation losses. The error in continuity is computed and printed as a percentage of precipitation. With the assistance of subroutine HCURVE, HYDRO plots the rainfall hyetograph, total infiltration, and the runoff hydrograph for the total drainage basin.

Interfacing and the Use of Disk Files

The Runoff Block transfers hydrographs and pollutographs for as many as 200 inlets and 10 constituents through an assigned file to other SWMM blocks (see Executive Block description). However, the other blocks may only accept part of this output. These restrictions may be circumvented by making a single run of the Runoff Block and generating a permanent data set (file) that will allow several runs of other blocks utilizing different portions of the output. If this is the first computational block, the title and values for the starting date and time and time step sizes will be used throughout all subsequent blocks.

Blocks, such as Extran and Transport, which may require a smaller time step than that used by the Runoff Block use a linear interpolation technique to generate the required input data from the interface file. Blocks such as the Storage/Treatment Block that may use a longer time step average the interface flows and loads over the longer time step.

Up to five scratch files are required for the single-event mode and as many as seven scratch files are required for the continuous mode; see Table 4-1. In the continuous mode the additional files are used to provide the program with a continuous feed of precipitation data so that there is effectively no limit on the length of the simulation.

Table 4-1. Runoff Off-Line File Allocations

JIN(1)[a]	=	Input unit for precipitation. This data file was created earlier by the RAIN Block or saved in a previous run of the RUNOFF Block (see data group D1 description).
NSCRAT(1)	=	Scratch data file used when precipitation data are input on the E3 data group lines. Not required if precipitation is input on unit JIN(1).
NSCRAT(2)	=	Scratch data file used when precipitation data are input on the E3 data group lines. Not required if precipitation is input on unit JIN(1).
NSCRAT(3)	=	Data file used for storage of processed temperature, evaporation, and wind speed values from the Temp Block.
NSCRAT(4)	=	Scratch data file, always required. Used for temporary storage of output data to be printed.
NSCRAT(5)	=	Scratch data file, required if groundwater is simulated. Stores water table depths, groundwater flows and soil moisture contents for printout.
NSCRAT(6)	=	Scratch data, required if groundwater is simulated. Stores water table depths, groundwater flows and soil moisture contents for graphing by Graph Block.
JOUT(1)	=	Output unit for transfer of Runoff results to subsequent blocks. Required only if subsequent blocks are to be used or plotting is to be done using the Graph Block or statistics performed using the Statistics Block.

[a]Subscript “one” is used if Runoff is the first block run in a SWMM simulation. See explanation of Executive Block (Section 2).

Data Preparation -- General Information

Introduction

Instructions on the use of the Runoff Block are divided into five subsections: general input and control data, meteorological data processing, surface quantity, surface quality and print control. The subsections follow the order of the input data groups shown in Table 4-31 at the end of this section. Many individual parameters are explained in more detail in the footnotes to Table 4-31. For further explanations of methods and techniques, the user should refer to the documentation in the appendices to this report and to the original SWMM documentation (Metcalf and Eddy et al., 1971a).

Basic Runoff Data Sources

Importance of Runoff Block Data

The Runoff Block forms the source of runoff and quality hydrographs and pollutographs for most SWMM applications. Although the other SWMM blocks allow direct input of special hydrographs and pollutographs either bypassing the interfacing file or in addition to it, in most cases these will be generated by the conversion of rainfall/snowmelt into runoff and pollutant loads in the Runoff Block. Hence, the input data for this block are probably the most important in the model.

Key data requirements and sources are mentioned during discussions of individual data groups later. However, the general types of data are discussed briefly at this point.

Meteorological Data

Precipitation data are usually obtained from on-site gages maintained by an agency that has performed rainfall-runoff monitoring such as a local consulting firm, water authority, or city, county, state, provincial or federal agency. In the unfortunate event of a missing rain gage, precipitation data should be obtained from the nearest National Weather Service (NWS for U.S.) or Atmospheric Environment Service (AES for Canada) station. The fundamental data are precipitation hyetographs for the duration of the simulation. (See subsequent discussion for use of synthetic rainfall data.) When snowmelt is simulated, air temperatures and wind speed are needed in addition.

Surface Quantity Data

Flow routing data are usually derived from topographic maps, aerial photos and drainage system plans. These are customarily obtained from the local agency responsible for drainage, usually the city or county. Especially for topographic maps, there is great variation in the quality of such data. Some cities, for instance, have 1:200 scale topographic maps complete with outlines of roads and structures. Slopes are easily derived from the one or two foot contours found on such maps. In some U.S. cities, the only contour information available may be the 1:24000 scale USGS quadrangle maps from which gross parameter estimation is often the only possibility. Seekers of basic quantity data must be prepared to spend several days at the municipal engineer's office to locate needed maps, plans etc. in public files.

A significant problem remains: the reliability of such data sources. Most municipal offices contain design drainage drawings, but recent as-built information is very rare. In older cities, design drawings may date back many decades and only serve as a guide to what actually exists in the field. This most often affects sewer slopes and cross sections (due to deterioration of old sewers). Finally, combined sewer regulators and other hydraulic control locations are often different from design drawings because of deterioration and maintenance. In many instances, hydraulic connections exist that are not included on any plans because of pragmatic action of maintenance crews. In other cases, evident connections have been blocked off. In summary, all such data should be field checked.

Surface Quality Data

Data required to formulate pollutographs are the most controversial of any SWMM input data. Such data and their possible sources are discussed later. At this point it is only re-emphasized that unless actual field sampling of runoff quality has been performed, typically by a government or pollution control agency, the credibility of predicted quality results cannot be established.

Default Parameters

Very few default values for parameters are included in the model. However, the users may insert default parameters directly through the use of "default" and "ratio" options while entering data. The objective is to encourage the user to obtain reasonable values for all parameters on a site-specific basis, rather than to depend upon generalizations. Representative values and guidelines for selection of such parameters are included in this manual.

General and Control Data (Groups A1-B6)

The first four data groups are concerned with a label for the output and general operating parameters. The labels (titles) of group A1 will be placed on the interfacing file for future identification of the output. Most individual parameters are self-explanatory. However, further information on several parameters (e.g., infiltration) may be found in subsequent discussions of those topics. There is no distinction between single event and continuous simulation (except for snowmelt) in SWMM. The discerning user will notice the disappearance of parameter ICRAIN from earlier versions of SWMM. Single event and continuous simulations and modes will still be discussed but this is more of a semantic difference rather than a difference in input and programming. Ten hyetographs can be used for both single event and continuous simulations.

The user has more control over printing in SWMM4 by using the parameters on data group B2. The user can eliminate the printing of most input data by using IPRN(1) (see Table 4-31). IPRN(2) gives the user control over the plotting of rainfall hyetograph(s) and total inlet hydrograph. The user should avoid printing large amounts of unnecessary output and use parameter IPRN(3) on data group B2 judiciously (especially for long simulations). Control data and summary outputs are always printed.

The parameters of data group B3 govern the length of the wet time step (WET), the transitional time step(s) between wet and dry (WETDRY), the dry (DRY) time step, the time units of simulation, and the total simulation length. The exact number of time steps is no longer an input parameter. WET should be less than or equal to the rainfall interval entered on data group D1. It can be longer, but information is lost by averaging the rainfall over a longer time period. A wet time step is a time step with precipitation occurring on *any* subcatchment. A transitional time step has no precipitation input on any subcatchment, but the subcatchment(s) still have water remaining in surface storage. A dry time step has no precipitation input or surface storage. However, it can have groundwater flow. The model is considered either globally wet, in the transitional period, or dry.

The time step should be smaller for periods of rapid change, i.e. during rainfall, and longer during periods of slower change, i.e., during transitional and dry time steps. Runoff computations use the concept of extrapolation to the limit (Appendix V) and can use any time step from 1 second to 1 year. The solution technique is stable and convergent for any length time step.

Typically the WET time step should be a fraction of the rainfall interval. Five minute rainfall should have wet time steps of 1, 2.5 or 5.0 minutes, for example. The rainfall intensity is constant over the wet time step when WET is a fraction of the rainfall interval. A smaller wet time step would be desirable when the subcatchment is small and the time of concentration is a fraction of the rainfall interval. When using one hour rainfall from the NWS wet time steps of 10 minutes, 15 minutes or longer can be used by the model.

The Runoff overland flow routing technique loses water through infiltration, evaporation, and surface water outflow during the transition periods. A subcatchment's surface storage and surface flow always decreases during the transition from a wet condition to a dry condition. A smooth curve or straight line are good models for the shape of the hydrograph. Transport or Extran usually have small time steps and use linear interpolation for input hydrographs with longer time steps. The transition time step, WETDRY, can be substantially longer than WET and generate a good overland flow hydrograph. For example, a WET of 5 minutes can be coupled with a WETDRY of 15 minutes or 30 minutes. When using hourly rainfall input a WET of 15 minutes can be coupled to a WETDRY of 2 hours or 3 hours.

The dry time step should be 1 day to a week. The dry time step is used to update the infiltration parameters, generate groundwater flow, and produce a time step value for the interface file. The dry time step should be day(s) in wet climates and days or week(s) in very dry climates. The synoptic analysis performed by the Rain Block will be of use in selecting the appropriate dry time step. Examine the average storm interevent duration in the storm summary table. The average storm interevent duration ranges from half a week to months depending on station location.

The model can achieve substantial time savings with judicious usage of WET, DRY, and WETDRY for both short and long simulations. As an example consider the time step saving using a WET of 15 minutes, a WETDRY of 2 hours, and a DRY of 1 day versus using a single time step of 1 hour for a year. Using Florida rainfall as input (average annual rainfall between 50 and 60 in. [1250 to 1500 mm]) gives 300 wet hours per year, flow for approximately 60 days per year, and 205 dry days per year. This translates to 1975 time steps. A constant hourly time step for one year requires 8760 time steps. This is greater than a 400 percent savings in time with a better representation of the flow hydrograph due to the 15 minute wet time step.

Data group B4 describes two global parameters pertaining to subcatchments: the rate of infiltration regeneration (REGEN), and percent imperviousness with no depression storage (PCTZER). This data group is optional and need not be entered by the user. These parameters are discussed in more detail later in conjunction with the subcatchment input parameters.

Meteorological Data (Groups C1-F1)

Snowmelt Data

General Parameters

Groups C1 through F1 are used to read all pertinent meteorological data. Groups C1-C5 are concerned with snowmelt, if simulated. Additional snowmelt parameters are found in groups I1-I3. Snowmelt procedures are discussed in detail later.

In group C1, the watershed elevation is used only to compute average atmospheric pressure, which in turn has only a minimal effect on results. Hence, it is not a "sensitive" parameter. The free water holding capacity of a snow pack is the volume of water (as a depth, in inches) within the pack that can be held as liquid melt prior to releasing runoff. In the model it simply acts as an intermediate reservoir; the larger its volume, the greater the delay in the appearance of runoff following the conversion of snow to liquid water. Unfortunately, as is the case for most snowmelt parameters, very few data exist that permit estimation of this parameter in urban areas, let alone make distinctions among three types of snow-covered areas as required in group C1. However, some available information is summarized in Table 4-2.

In natural areas, a surface temperature (SNOTMP) of 34° to 35°F (1-2°C) provides the dividing line between equal probabilities of rain and snow (Eagleson, 1970; Corps of Engineers, 1956). However, parameter SNOTMP in group C1 might need to be somewhat lower in urban areas due to warmer surface temperatures.

Table 4-2. Snowpack Free Water Holding Capacity
(Anderson, 1973; Corps of Engineers, 1956)

Model input (data group C1) is

$$FWFRAC = FW_{max}/WSNOW$$

Where

FWFRAC = free water holding capacity as a fraction of snowpack depth,
FW_{max} = maximum depth of free water stored in pack, inches, and
WSNOW = snowpack depth, inches water equivalent

Snowpack Conditions	FWFRAC
Typical deep pack (WSNOW > 10 in.)	0.02-0.05
Typical shallow early winter pack	0.05-0.25
Typical shallow spring pack or with slush layer	0.20-0.30

FWFRAC increases as pack density increases, pack depth decreases, slush layer increases, ground slope decreases.

The snow gage correction factor accounts for the error in snow gage measurements. The value of SCF is usually greater than 1.0 (the gage tends to underestimate the catch) and increases as a function of wind speed. Representative values are show in Figure 4-2 (Anderson, 1973). In practice, SCF can be used as a calibration factor to account for gains or losses of snow it cannot be determined from available data.

During non-melt periods (i.e., sub-freezing weather) the temperature of the snow pack follows the air temperature, but with a delay, since temperature changes cannot occur instantaneously. Heat exchange and temperature changes during this period are explained in Appendix II, with reference to equations II-15 and II-16. The weighting factor, TIPM, is an indicator of the thickness of the "surface" layer of the snow pack. Values of TIPM <= 0.1 give significant weight to temperatures over the past week or more and would indicate a deeper layer (thus inhibiting heat transfer) than TIPM values greater than about 0.5 which would essentially only give weight to temperatures during the past day. In other words the pack will both warm and cool faster (i.e., track the air temperatures) with higher values of TIPM. Anderson (1973) states that TIPM = 0.5 has given reasonable results in natural catchments, although there is some reason to believe that lower values may be appropriate. No data exist for urban areas.

Heat transfer within the snow pack is less during non-melt periods than during melt periods due to the presence of liquid water in the pack for the latter case. Parameter RNM simply multiplies melt coefficients (described for data groups I1-I3) to produce a lower "negative melt coefficient" for use during non-melt periods. A typical value for natural areas is 0.6, with values for urban areas

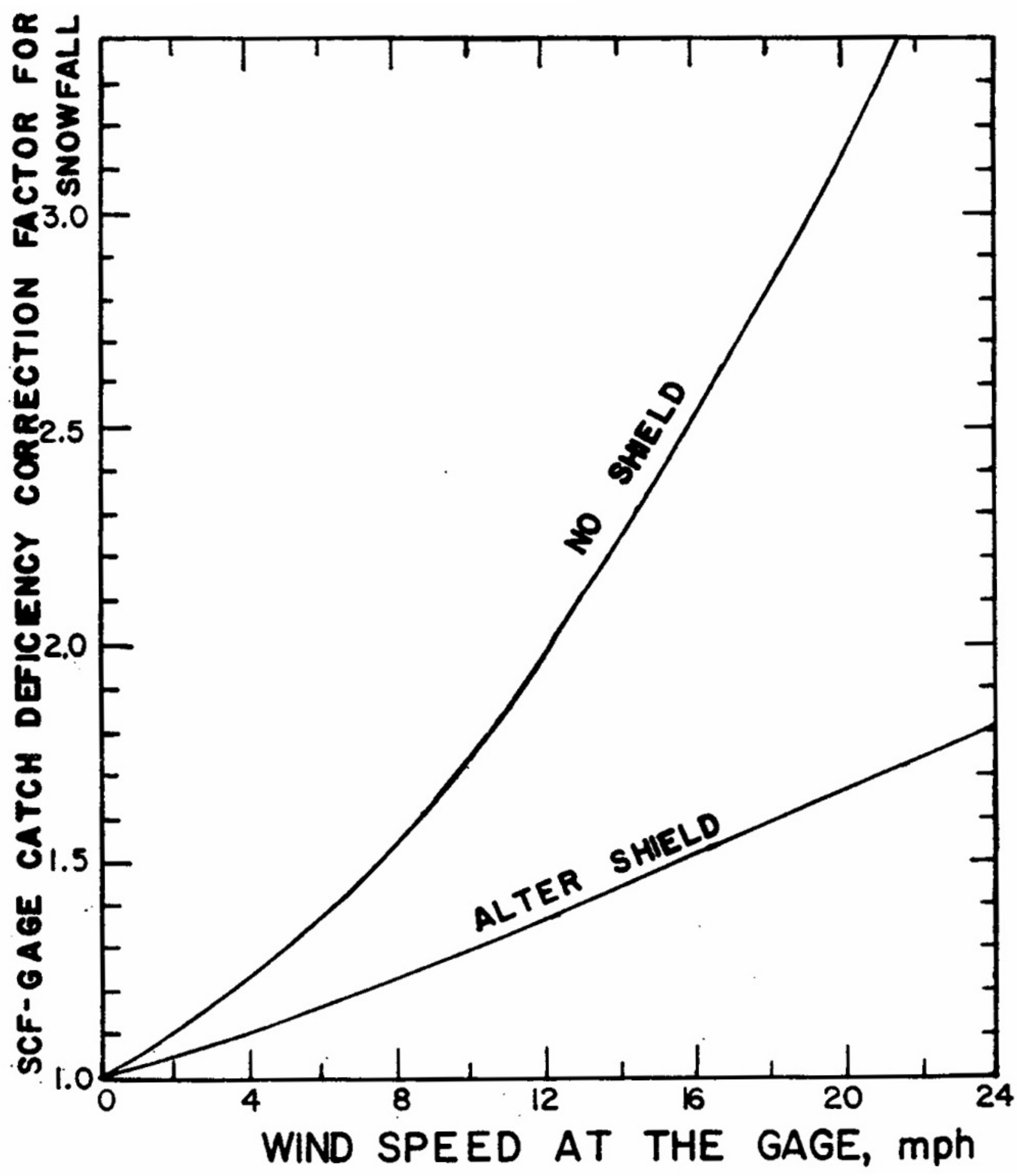


Figure 4-2. Gage catch deficiency factor (SCF) versus wind speed (after Anderson, 1973, p. 5-20).

likely to be somewhat higher because of the higher density of urban packs. The higher the value of RNM, the more rapid is the heat gain or loss of the pack in response to air temperature changes.

The catchment latitude and the longitude correction (described in footnote 8 to Table 4-31) are used only to compute hours of daylight for the catchment. Computations are insensitive to moderate errors in these values.

NWS Temperature Data

Continuous SWMM requires a complete time history of daily maximum and minimum temperatures, from which hourly temperatures are synthesized by sinusoidal interpolation as described later. These max-min temperatures are supplied on the NWS TD-3200, "Surface Land Daily Cooperative Summary of Day." A magnetic tape containing these card images is available for most first-order NWS stations and others within the U.S. from the NOAA National Climatic Data Center in Asheville, NC (phone 704-259-0682). The entire Florida record of 40 years cost $236 in 1987. Such a record, corresponding to the precipitation record, is required for continuous simulation of snowmelt. Values are interpolated for missing dates. The Runoff Block uses the processed data from the Temp Block in its simulation. See Section 11 for more information and instructions in preparing the continuous temperature data file.

Wind Data

Wind speeds, entered in group C2, are used only for melt calculations during periods of rainfall. The higher the values of wind speed, the greater are the convective and condensation melt terms. Of course, if the simulation covers a large city, the wind speeds entered in group C2 can only be considered gross estimates of what are in reality highly variable speeds. Average monthly speeds are often available from climatological summaries (e.g., NOAA, 1974).

An alternate source of wind speed data is TD-3200 from NOAA. The TEMP Block will read wind speed data from TD-3200 alone or in conjunction with temperature and evaporation data and create an interface file. The output of the Temp Block is input to the Runoff Block as file NSCRAT(3). Entering 999 in the first field of the C2 data line will trigger the input of the NOAA wind speed data from NSCRAT(3).

Areal Depletion Curves

Areal depletion curves (ADC) account for the variation in actual snow covered area that occurs following a snowfall. They are explained in detail in Appendix II; a brief description is given here.

In most snowmelt models, it is assumed that there is a depth, SI, above which there will always be 100 percent cover. (Values of SI are input in data group I2.) In some models, the value of SI is adjusted during the simulation; in SWMM it remains constant. The amount of snow present at any time is indicated by the parameter WSNOW, which is the depth (water equivalent) over the snow covered areas of each subcatchment. This depth is non-dimensionalized by SI, called AWESI for use in calculating the fraction of area that is snow covered, ASC; a typical ADC for a natural catchment is shown in Figure 4-3. For values of the ratio AWESI = WSNOW/SI greater than 1.0, ASC = 1.0, that is, the area is 100 percent snow covered.

Some of the implications of different functional forms of the ADC may be seen in Figure 4-4. Since the program maintains snow quantities, the actual snow depth, WS, and area covered, AS, are related by continuity:

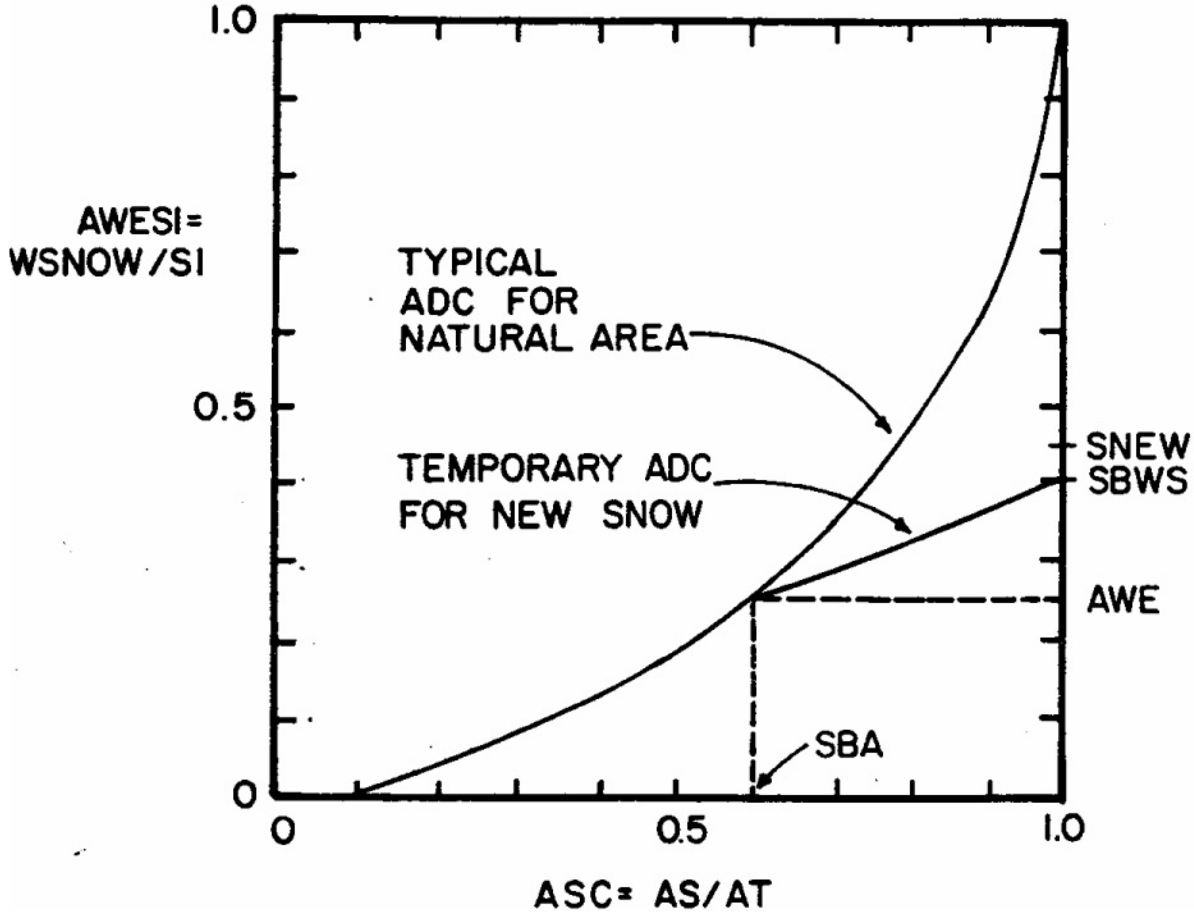


Figure 4-3. Actual areal depletion curve for natural area (after Anderson, 1973, p. 3-15).

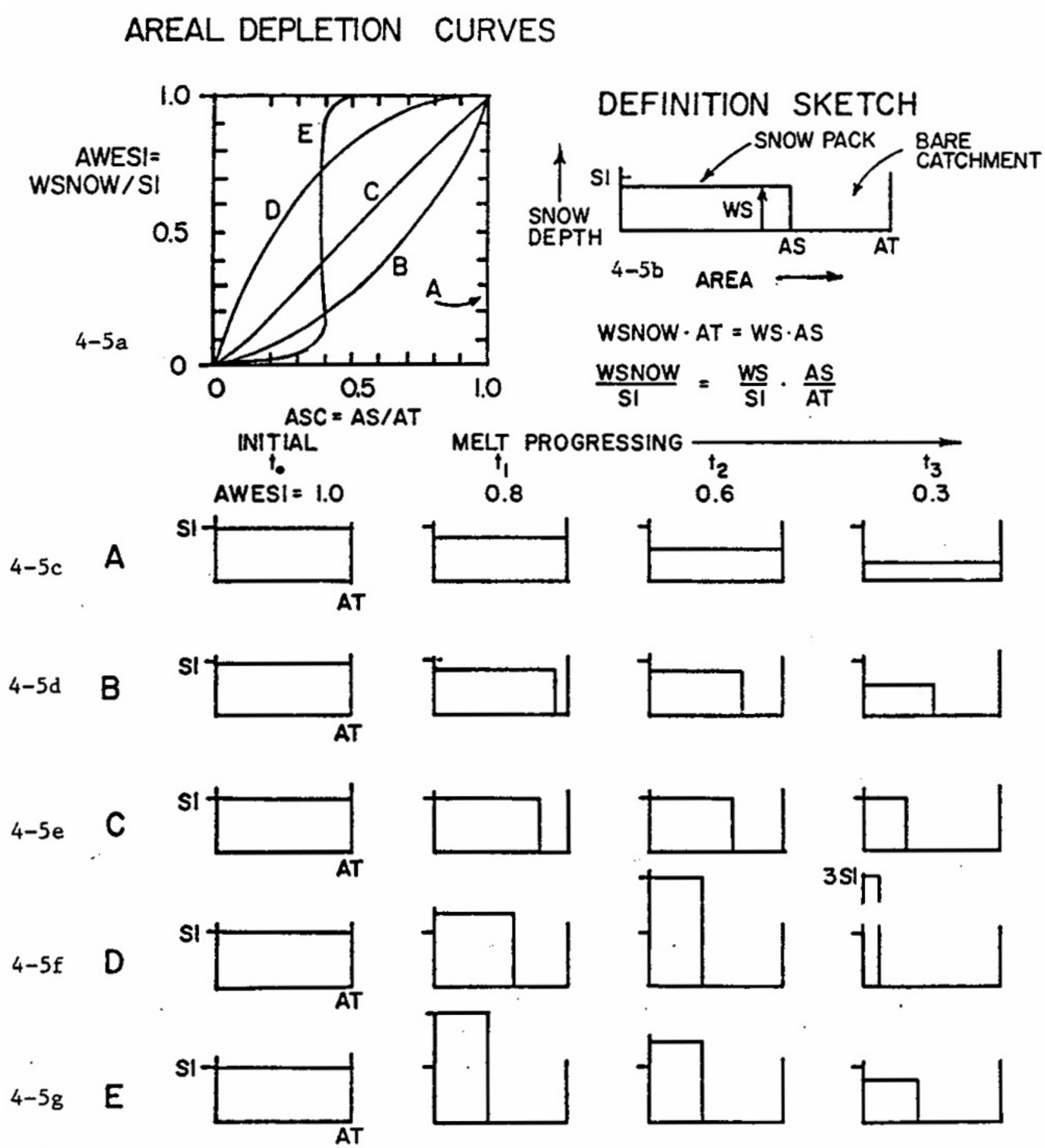


Figure 4-4. Effect of snow cover on areal depletion curves.

$$\text{SNOW} \diamond \text{AT} = \text{WS} \diamond \text{AS} \qquad (\ 4\text{-}1)$$

where

WSNOW	=	depth of snow over total area AT, ft water equivalent,
AT	=	total area, ft^2,
WS	=	actual snow depth, ft water equivalent, and
AS	=	snow covered area, ft^2.

In terms of parameters shown on the ADC, equation 4-1 may be rearranged to read

$$\text{AWESI} = \text{WSNOW/SI} = \text{WS/SI} \diamond \text{AS/AT} = \text{WS/SI} \diamond \text{ASC}$$

(4-2)

Equation 4-2 can be used to compute the actual snow depth, WS, from known ADC parameters, if desired. It is unnecessary to do this in the program, but it is helpful in understanding the curves of Figure 4-4. Thus,

$$\text{WS} = \text{AWESI/ASC} \diamond \text{SI} \qquad (4\text{-}3)$$

Consider the three curves, B, C and D. For case B, AWESI is always less than ASC; hence, WS is always less than SI as shown in Figure 4-4d. For case C, AWESI = ASC, hence WS = SI, as shown in Figure 4-4e. Finally, for case D, AWESI is always greater than ASC; hence WS is always greater than SI, as shown in Figure 4-4f. Constant values of ASC at 100 percent cover and 40 percent cover are illustrated in Figures 4-4c, curve A, and Figure 4-4g, case E, respectively. At a given time (e.g., t_1 in Figure 4-4), the area of each snow depth-area curve is the same and equal to AWESI x SI, (e.g., 0.8 SI for time t_1).

Curve B on Figure 4-4a is the most common type of ADC occurring in nature, as shown in Figure 4-3. The convex curve D requires some mechanism for raising snow levels above the original depth, SI. In nature, drifting might provide such a mechanism; in urban area, plowing and windrowing could cause a similar effect. It is seen that such a convex curve acts to delay melt because of the inhibiting effect on heat transfer of deep snow packs. A complex curve could be generated to represent specific snow removal practices in a city. However, the program utilizes only one ADC curve for all impervious areas and only one ADC curve for all pervious areas. This limitation should not hinder an adequate simulation since the effects of variations in individual areas are averaged out in the city-wide scope of most continuous simulations.

The user must input the two ADC curves for impervious (group C3) and pervious (group C4) areas, as well as values of SI for each subcatchment (group I2). The program does not require the ADC curves to pass through the origin, AWESI = ASC = 0; they may intersect the abscissa at a value of ASC > 0 when ASC = 0.

The preceding paragraphs have centered on the situation where a depth of snow greater than or equal to SI has fallen and is melting. (The ADC curves are not employed until WSNOW becomes less than SI.) The situation when there is new snow is discussed in Appendix II.

Air Temperatures

For a single event snowmelt simulation, air temperatures are input in data group C5. These may be obtained from instrumentation at the catchment or from the nearest NWS (U.S.) or AES (Canada) station. The temperatures are constant over the time interval DTAIR (group C5).

Precipitation Data

Choice of Rainfall Data

Without doubt, rainfall data are the single most important group of hydrologic data required by SWMM. Yet, they are often prepared as an afterthought, without proper consideration of the implications of their choice. The following discussion will briefly describe options for rainfall input and their consequences. Only rainfall is considered since for snow it is the physics of snowmelt rather than snowfall which is important in determining runoff.

SWMM requires a hyetograph of rainfall intensities versus time for the period of simulation. For single event simulation this is usually a single storm, and data for up to ten gages may be entered (if the user is fortunate enough to have multiple gages for the catchment). For continuous simulation, hourly, 15-minute or other continuous data from at least one gage are required; these are usually obtained from the nearest NWS (U.S.) or AES (Canada) station. Thus, for continuous simulation, the options are fewer since a satisfactory generator of, say, a synthetic hourly rainfall sequence is not usually available, and perhaps not even desirable. Hence, a historical rainfall sequence is usually used.

For single event simulation, on the other hand, synthetic design storm sequences are indeed an option in lieu of historical records. However, several pitfalls exist in the use of synthetic hyetographs that may not be obvious at first thought. As a prelude, consider the objectives of hydrologic quantity and quality modeling.

Modeling Objectives

These were treated broadly in Section 1. Models might be used to aid in urban drainage design for protection against flooding for a certain return period (e.g., five or ten years), or to protect against pollution of receiving waters at a certain frequency (e.g., only one combined sewer overflow per year). In these contexts, the frequency or return period needs to be associated with a very specific parameter. That is, for rainfall one may speak of frequency distributions of interevent times, total storm depth, total storm duration or average storm intensity, all of which are different (Eagleson, 1970, pp. 183-190). Traditional urban drainage techniques often utilize frequencies of depths for given durations, taken from intensity-duration-frequency (IDF) curves, which are really conditional frequency distributions. But for the above objectives, and in fact, for almost all urban hydrology work, the frequencies of runoff and quality parameters are required, not those of rainfall at all. Thus, one may speak of the frequencies of maximum flow rate, total runoff volume or duration or of total pollutant loads. These distributions are in no way the same as for similar rainfall parameters, although they may be related through analytical methods (Howard, 1976; Chan and Bras, 1979; Hydroscience, 1979). Finally, for pollution control, the real interest may lie in the frequency of water quality standards violations in the receiving water, which leads to further complications.

Ideally an analyst would develop costs and benefits for designs at several frequencies in preparation for an economic optimization. In practice, it is often difficult to accomplish this for even one case.

However, continuous simulation offers an excellent, if not the only method for obtaining the frequency of events of interest, be they related to quantity or quality. But continuous simulation has the disadvantages of a higher cost and the need for a continuous rainfall record. This has led to the use of a "design storm" or "design rainfall" or "design event" in a single event simulation instead. Of course, this idea long preceded continuous simulation, before the advent of modern computers. However, because of inherent simplifications, the choice of a design event leads to problems.

Design Events

Two methods of obtaining design events are considered: 1) use of a historical sequence and 2) generation of a synthetic sequence. Synthetic sequences are usually constructed by the following steps (Arnell, 1982):

1) A storm duration is chosen, whether on an arbitrary basis or to coincide with the assumed catchment time of concentration, t_c, i.e., equilibrium time at which outflow equals a constant fraction of steady rainfall (or outflow equals rainfall on a catchment without losses). The latter method itself has difficulties because of the dependence of t_c on rainfall intensity and other parameters (Eagleson, 1970).
2) A return period is chosen in order to select the total storm depth for the specified duration from intensity-duration-frequency (IDF) curves.
3) A time history for the storm is assumed, usually on the basis of historical percentage mass curves. If peak intensities occur at the beginning of the storm, the hyetograph takes on the appearance of a decaying exponential curve. If the peak intensities occur near the middle, a "circus tent" hyetograph results (Figure 4-5). The hyetograph is shaped such that depths (or average intensities) for any duration centered about the peak match those from the IDF curve. Several shapes are commonly used (Arnell, 1982); in the U.S., the "Chicago storm" (Keifer and Chu, 1957) and the SCS Type-II distribution (SCS, 1972) are frequently encountered.
4) The continuous hyetograph of Figure 4-5 must then be discretized into a histogram for input to most models.

This procedure was apparently first detailed by Keifer and Chu (1957) and then by Tholin and Keifer (1960) in Chicago. It has since been emulated by many others (Arnell, 1982; Harremoes, 1983).

Many problems with this procedure for construction of synthetic hyetographs (McPherson, 1978; Patry and McPherson, 1979; Arnell, 1982; Harremoes, 1983; Adams and Howard, 1985) and with the underlying rational method and IDF curves on which it is based (McPherson, 1969) have been enumerated. For example:

1) IDF curves themselves may consist of components of several different storms. They in no way represent the time history of a real storm.
2) When frequencies are assigned to total storm depths (independent of duration) they generally do not coincide with the conditional frequencies of depth for the given duration obtained from IDF curves. For instance, the two historical storms shown on Figure 4-5 for comparison with the A5-year@ synthetic storm of 2.28 in. (58 mm) have return periods (based on total depth) of 4.6 and 5.8 years, but total depths of only 1.61 and 1.85 in. (41 and 47 mm), respectively. Thus, IDF curves cannot be used to assign frequencies to storm volumes. If synthetic hyetographs are thence

used for studies of detention storage or pollutant loads, where volumetric considerations are key, no frequency should be assigned to the results.

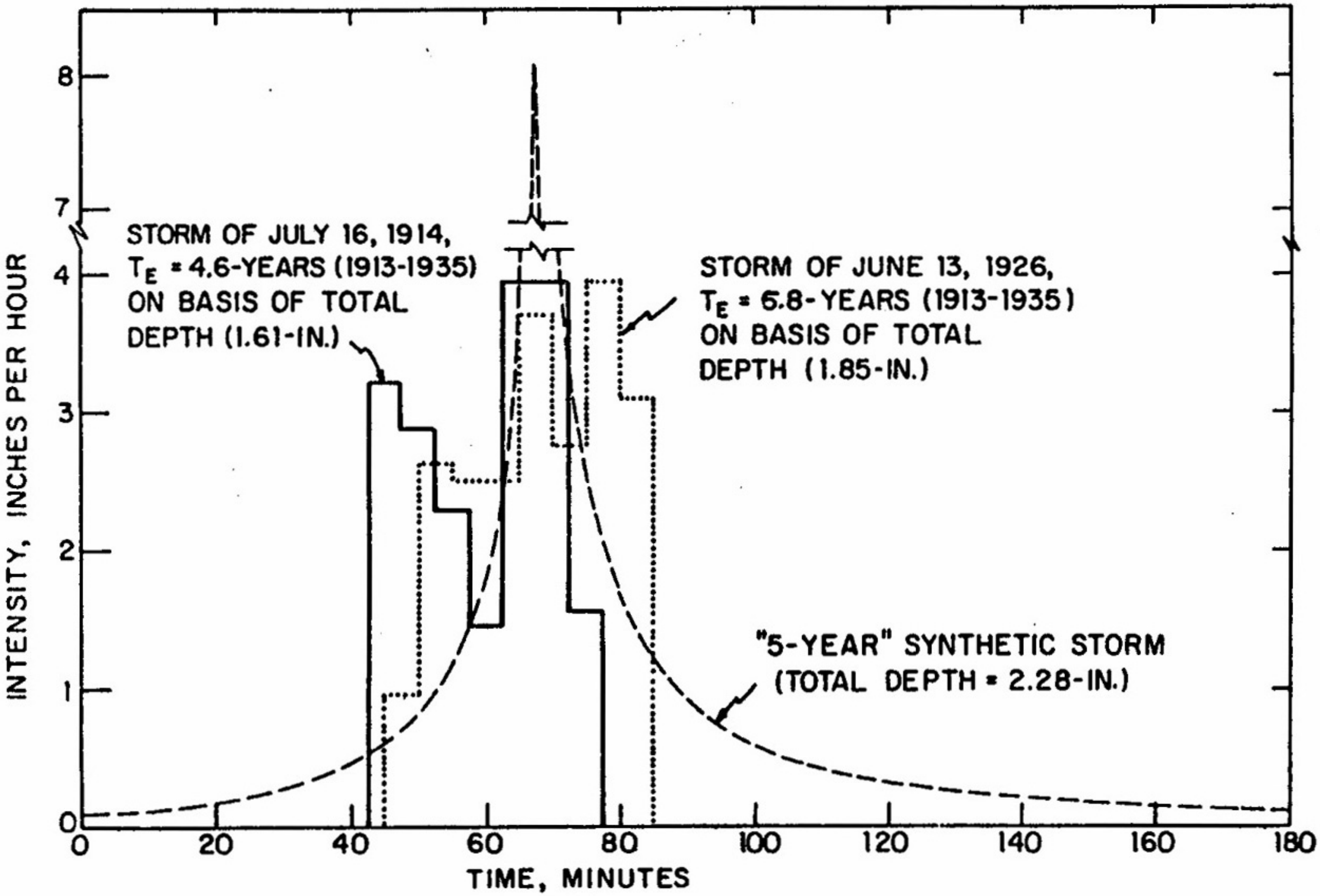


Figure 4-5. Comparison of synthetic versus actual storm patterns, Chicago (after McPherson, 1978, p. 111).

3) Although the time history assigned to a synthetic storm may represent an average of many storms, there is often considerable variability (see P. Bock, Discussion of Tholin and Keifer, 1960). If a frequency could be assigned to a synthetic storm, it would probably be considerably rarer than its nominal frequency, because the joint probability of all time sequences within the storm corresponding to those of an IDF curve is very low. The two historical storms shown on Figure 4-6 certainly do not mimic the synthetic storm.
4) Antecedent conditions must still be chosen arbitrarily when using a design event (either a synthetic or historical storm.) However, historical storms also provide their historical antecedent conditions. That is, a historical storm can be run in a single-event mode using several days of historical antecedent rainfall to generate realistic antecedent moisture conditions in the catchment. This is not possible with synthetic storms.
5) A synthetic design event is one that "never really happened." McPherson (1978) emphasizes the need to design with a real (historical) event to ensure credibility in the eyes of the public.
6) There is evidence that synthetic design events may produce an over-design if the objective is a design for a given return period. Marsalek (1979a,b) has compared continuous simulation results of flood peaks and volumes versus return period with results obtained by single event simulations using the same model with input of n-year synthetic events of the type described earlier. Flood peaks are always higher for the synthetic events. Flood volumes are higher for most synthetic events, depending on the method of generation of the event, because the return periods assigned to the synthetic volumes are incorrect. The verdict is not clear, however. Huber et al. (1986) compared synthetic versus historical storms for simulation of peak flows for a 2000-ac catchment in Tallahassee, Florida. They found that a larger peak was generated by a 22-yr historical storm than by a 25-yr SCS Type-II synthetic storm. In other words, synthetic storms are not always conservative.

Design Event Alternatives

In spite of all of its problems, use of a design event may still be required. Fortunately, there are ways in which this may be accomplished satisfactorily.

Foremost among these is the use of continuous simulation as a screening tool. As stated earlier, continuous simulation for several years of a large catchment with inclusion of spatial detail can be time-consuming. Instead, representative smaller catchments may be simulated from which critical events may be selected for a more detailed, single event simulation. Thus, from a simple long-term continuous simulation, critical subsets may be identified for further analysis. Walesh and Snyder (1979) present ideas along this line, and Robinson and James (1984) and Huber et al. (1986) demonstrate the ideas.

Continuous simulation may also be used to "calibrate" a synthetic design event. That is, the design hyetograph may be adjusted such that it produces flows or volumes that correspond for its return period to those produced by a continuous simulation run. This has been done in studies in Northern Virginia (Shubinski and Fitch, 1976) and Denver (B. Urbonas, personal communication, 1979). Proper adjustment of antecedent conditions can also cause results from synthetic design events to match historical results (Wenzel and Voorhees, 1978).

In any event, several storm events should be processed for design considerations. These may be selected from a continuous simulation run, as suggested above, or chosen from the historical record on another basis. For urban drainage or flood control design, it may be desirable to choose a particular, well-known local rainfall event and make sure that a design will handle that storm.

Calibration of the model remains important for any application. It has been suggested (M. Terstriep, personal communication, 1979) that use of a synthetic design event for analysis of a new system may not be any worse than using historical data in an uncalibrated model.

The question of appropriate rainfall input for models has generated intense interest. Good discussions are given by McPherson (1978), Patry and McPherson (1979), Arnell (1982), James and Robinson (1982), Harremoes (1983) and Huber et al. (1986).

National Weather Service Precipitation Data

Hourly precipitation values (including water equivalent of snowfall depths) are available for 40-year periods for most first-order NWS stations around the U.S. (Similar data are available in Canada from the Atmospheric Environment Service.) Magnetic tapes containing card images of NWS Tape Deck 3240, "Hourly Precipitation Data" are available from the NOAA National Weather Records Center in Asheville, North Carolina (phone (704) 259-0682). The cost for the entire state of Florida was $154 in 1984. Typically, purchasing the entire state record is actually cheaper than purchasing a single station due to extra processing costs for a one station retrieval.

Having obtained these data for a continuous simulation, they are read directly from the tape in the Rain Block. See input details in Section 10, which describes the Rain Block.

Atmospheric Environment Service Data

A special package DATANAL is available from Computational Hydraulics Inc. to convert AES data tapes to the NWS format expected by SWMM (W. James, personal communication, 1987).

Special Input of Precipitation Data

Precipitation input is significantly different in this version of SWMM. Important differences include: (1) free format input, (2) three input types, (3) variable precipitation intervals, (4) precipitation input may be intensity or volume, (5) rainfall print control, (6) fewer input values since zero rainfall need no longer be entered, and (6) the rainfall scratch file can be saved permanently.

For single event simulation, precipitation hyetographs may be input for up to ten gages using data groups D1 through E3. Any one of the ten gages may then be assigned to a subcatchment using parameter JK in the H1 data group.

Subroutine MKRAIN reads the input hyetographs input on data group E3. They are either temporarily or permanently saved on NSCRAT(1). MKRAIN also uses NSCRAT(2) as a temporary work file, making NSCRAT(2) a Runoff Block scratch file requirement. The user has the option of saving NSCRAT(1) as a permanent file for subsequent runs. This might be efficacious when a large amount (> 100 data points) are input in data group E3. The permanent file will save processing time on later calibration runs. There is no limitation on the number of precipitation data points.

The user saves the file permanently by using the @ function (discussed in Section 2) and selecting ROPT=2 on data group D1. The precipitation file has the same format as the precipitation file created by the Rain Block. The interested reader can find a description of a precipitation file in Section 10. The precipitation files are read by Subroutine HYDRO at the beginning of each time step.

Data group E1 defines the type of precipitation (KTYPE), precipitation values or pairs per line (KINC), precipitation print control (KPRINT), variable precipitation intervals (KTHIS), time units (KTIME), precipitation input type (KPREP), number of precipitation values (NHISTO), and the default rainfall interval (THISTO).

Precipitation can be read in three formats as described by parameter KTYPE. KTYPE = 0 is the old SWMM format with a constant precipitation interval (THISTO). The problem with this format is that zeroes must be used to fill the "holes" for dry time steps. For example, the input

E3 1.0 0.0 0.0 0.0 2.0 3.0 1.0 0.4 0.5 0.0 KTYPE = 0 Example Input

means starting at time TZERO the first rainfall interval has an intensity of 1.0 in./hr, the second, third and fourth intervals have intensities of 0.0 in./hr, the fifth has intensity 2.0 in./hr, etc. Each interval is THISTO minutes long unless THISTO is modified by the E2 data group.

Using the other two input formats eliminates the necessity of entering zeroes. The starting time for the interval and the interval precipitation value are the only requirements. For example, the input

E3 0.0 1.0 100.0 0.5 KTYPE = 1 Example Input

means that the rainfall intensity starting at 0 minutes and lasting THISTO minutes is 1.0 in./hr. The rainfall intensity starting at minute 100 and lasting THISTO minutes is 0.5 in./hr. The hyetographs for each raingage are entered consecutively if KTYPE = 0 or 1. If KTYPE is 2, a starting time and a precipitation intensity for each raingage is entered on one line. For example, the input

E3 100.0 1.0 2.0 0.0 KTYPE = 2 Example Input

means starting at minute 100 the rainfall intensity for THISTO minutes is 1.0 in./hr for gage 1, 2.0 in./hr for gage 2, and 0.0 in./hr for gage 3. This format does require the input of zero rainfall at a gage if even one gage has measurable rainfall.

Input parameter KINC is the number of rainfall values per input line (KTYPE=0),or the number of time and precipitation pairs per line (KTYPE=1), and unnecessary for KTYPE=3. KINC equals NRGAG + 1 for KTYPE=3. The user should enter any number in the KINC field for KTYPE=3.

KPRINT controls the echo printing of the rainfall. Select KPRINT=1 to eliminate the echo printing. Only summary statistics by individual raingage will be printed. The summary table lists the total rainfall, maximum and minimum rainfall intensity or volume, and total rainfall duration for each raingage.

The time interval for input of hyetograph intensities, THISTO, (the same for all hyetographs) must be either equal to the wet computation time step, WET (group B3), or an integer multiple or integer fraction (e.g., 1/2, 1/5, etc.) thereof. If THISTO is an integer fraction of WET, the average intensity over time step DELT is used in computations. Realistically, THISTO should be at least equal to the wet time step. Information is lost by averaging over discrete rainfall intensities. The interrelationship between WET and THISTO is discussed later in the section on flow routing parameters.

Parameter KTHIS is the number of variable rainfall intervals input on data group E2. This option allows the user to mix rainfall intervals of differing lengths in a simulation, e.g. 5-minute rainfall between 15-minute or 1-hour rainfall intervals. An input of

E2 100.0 200.0 5.0 1000.0 2000.0 15.0 Variable THISTO input

means between 100 and 200 minutes the rainfall interval is 5 minutes, but between 1000 and 2000 minutes the rainfall interval is 15 minutes. The times are the minutes from the start of simulation. The time periods outside of these two ranges would have THISTO rainfall intervals. THISTO is always the default rainfall interval.

The precipitation input is either in units of intensity, in./hr [mm/hr], or the total rainfall volume over the rainfall interval, in. [mm]. The input type is selected by parameter KPREP. Runoff uses intensity units internally. The unit of time used by data groups E2 and E3 may be either minutes or hours and is selected by parameter KTIME on data group E1.

Temporal Rainfall Variations

The required time detail for rainfall hyetographs is a function of the catchment response to rainfall input. Small, steep, smooth, impervious catchments have fast response times, and vice versa. As a generality, shorter time increment data are preferable to longer time increment data, but for a large (e.g., 10 mi^2 or 26 km^2) subcatchment (coarse schematization), even the hourly inputs usually used for continuous simulation may be appropriate.

The rain gage itself is usually the limiting factor. It is possible to reduce data from 24-hour charts from standard 24-hour, weighing-bucket gages to obtain 7.5-minute or 5-minute increment data, and some USGS float gages produce no better than 5-minute values. Shorter time increment data may usually be obtained only from tipping bucket gage installations.

The rainfall records obtained from a gage may be of mixed quality. It may be possible to define some storms down to 1 to 5 minute rainfall intensities, while other events may be of such poor quality (because of poor reproduction of charts or blurred traces of ink) that only 1-hour increments can be obtained. Variable precipitation intervals can be modeled by using data group E2 (see above). This will allow the interspersing of (for example) 5-minute, 15-minute, and hourly rainfall in a simulation.

Spatial Rainfall Variations

Even for small catchments, runoff and consequent model predictions (and prototype measurements) may be very sensitive to spatial variations of the rainfall. For instance, thunderstorms (convective rainfall) may be very localized, and nearby gages may have very dissimilar readings. For modeling accuracy (or even more specifically, for a successful calibration of SWMM), it is essential that rain gages be located within and adjacent to the catchment, or a storm model such as RAINPAK (James and Scheckenberger, 1983) be used.

SWMM accounts for the spatial variability by the assignment of one of up to ten gages to a particular subcatchment. (Clearly, there is no point in the input of more gage data than there are subcatchments.) If multiple gages are available, this is a much better procedure than is the use of spatially averaged (e.g., Thiessen weighted) data, because averaged data tend to have short-term time variations removed (i.e., rainfall pulses are “lowered” and “spread out”). In general, if the rainfall is uniform spatially, as might be expected from cyclonic (e.g., frontal) systems, these spatial considerations are not as important. In making this judgment, the storm size and speed in relation to the total catchment must be considered. It should be noted that a moving or "kinematic" storm may only be simulated in SWMM by using multiple gages. Storm motion may very significantly affect hydrographs at the catchment outlet (Yen and Chow, 1968; Surkan, 1974; James and Drake, 1980; James and Shtifter, 1981)).

Evaporation Data (Group F1)

An average monthly evaporation rate is required for the month being simulated in the single event mode, or for all months in the continuous mode. This rate is subtracted from rainfall and snowmelt intensities at each time step and is also used to replenish surface depression storage and provide an upper bound for soil moisture and groundwater evaporation. However, it is not used to account for sublimation from snow. Evaporation data may usually be obtained from climatological summaries (NOAA, 1974) or NWS or other pan measurements (e.g., from NWS *Climatological Data* or Farnsworth and Thompson, 1982). Single event simulations are usually insensitive to the evaporation rate, but evaporation can make up a significant component of the water budget during continuous simulation.

Evaporation can be input into the Runoff Block either by using data group F1, or by creating an evaporation time series using the Temp Block. If F1 is used the same monthly estimate for evaporation is used for all simulated years. The time series approach in the Temp Block allows yearly variation in evaporation. Daily, weekly, or monthly evaporation estimates can be read by the Temp Block. The evaporation time series is input into Runoff using NSCRAT(3) as the input file.

Surface Quantity Data (Groups G1-I3)

Runoff Flow Routing Procedures and Options

Data groups G1 through I3 input data used to establish surface and subsurface flow routing and snowmelt parameters for the Runoff Block. Snowmelt and subsurface routing will be discussed subsequently. Surface flow routing is accomplished using four types of elements:

1) subcatchment elements (overland flow),
2) channel elements (trapezoidal or parabolic channel flow),
3) pipe elements (circular channel flow), and
4) control structures (weirs and orifices).

Subcatchment elements receive rainfall and snowmelt, account for losses due to evaporation and infiltration (via Horton's equation or the Green-Ampt equation), and permit surface depression storage to account for losses such as ponding or retention on grass or pavement. "Losses" from infiltration may optionally be routed through a subsurface pathway (quantity simulation only), first into an unsaturated zone storage, then to a saturated zone storage from which baseflow into an inlet or channel/pipe may be generated (see Appendix X). Surface flow from subcatchments is always into channel/pipe elements or inlets. A tree-network of channel/pipes may be used to simulate smaller drainage elements of the sewer system. If they are used, they route hydrographs (and pollutographs) from subcatchments placed on the interfacing file for transmittal to subsequent SWMM blocks. However, the Runoff Block is often used by itself if the more sophisticated routing procedures of the Transport or Extended Transport Blocks are not required (discussed below).

Flow routing for both subcatchments and channel/pipes is accomplished by approximating them as non-linear reservoirs. This is simply a coupling of a spatially lumped continuity equation with Manning's equation. A detailed description is presented in Appendix VI. Should the capacity of a channel/pipe be exceeded, “surcharge” is indicated, and excess water is stored at the upstream end until the channel/pipe can accept it.

Input Data Preparation

Preparation of these input data requires two tasks: 1) discretization of the physical drainage system and 2) estimation of the coefficients necessary to characterize the catchment. These tasks require varying amounts of effort depending on the level of detail desired by the user.

Useful additional information for these tasks is contained in the short course proceedings prepared by the University of Massachusetts (Di Giano et al., 1977). The Runoff Block example is particularly good because of the emphasis on data reduction from typical municipal maps and plans. The SWMM user is encouraged to review these proceedings for alternative explanations and examples. Further useful information is contained in the references.

Discretization of the Catchment

Definition

Discretization is a procedure for the mathematical abstraction of the physical drainage system. For the computation of hydrographs, the drainage basin may be conceptually represented by a network of hydraulic elements, i.e., subcatchments, channels and pipes. Hydraulic properties of each element are then characterized by various parameters, such as size, slope, and roughness coefficient.

Subcatchments represent idealized runoff areas with uniform slope. Parameters such as roughness values, depression storage and infiltration values are taken to be constant for the area and usually represent averages, although pervious and impervious areas have different characteristics within the model. If roofs drain onto pervious areas, such as lawns, they are usually considered part of the pervious area, although conceivably, they could be treated as miniature subcatchments themselves.

Discretization begins with the identification of drainage boundaries using a topographic map, the location of major sewer inlets using a sewer system map, and the selection of those channel/pipes to be included in the Runoff Block system. Note that discretization of the sewer system involves choices that affect elements to be used in either of the subsequent Transport or Extran Blocks (see below). An example will illustrate some of these points.

Example

Two possible discretizations of the Northwood catchment in Baltimore (Tucker, 1968; Huber et al., 1981) are indicated in Figures 4-6 and 4-7 (Metcalf and Eddy et al., 1971a). A “fine” approach was used in Figure 4-6, resulting in 12 subcatchments and 13 pipes leading to one inlet. In Figure 4-7, a “coarse” discretization was used, resulting in five subcatchments and no channels or pipes. "Storm Conduits" shown in Figure 4-7 could either be simulated by the Transport or Extran Block or ignored, feeding all subcatchment flows to the one inlet. The outlet to the creek then represents the downstream point in the simulation. (This could lead, in a larger system, to inlets in the Transport Block.)

A comparison of hydrographs produced by the two methods is shown in Figure 4-8 (Metcalf and Eddy et al., 1971a), in which the differences are relatively minor. Additional calibration effort could bring the two schematizations into better agreement with each other and with the measured hydrograph. Techniques for this purpose are discussed later as are techniques for aggregation of subcatchments.

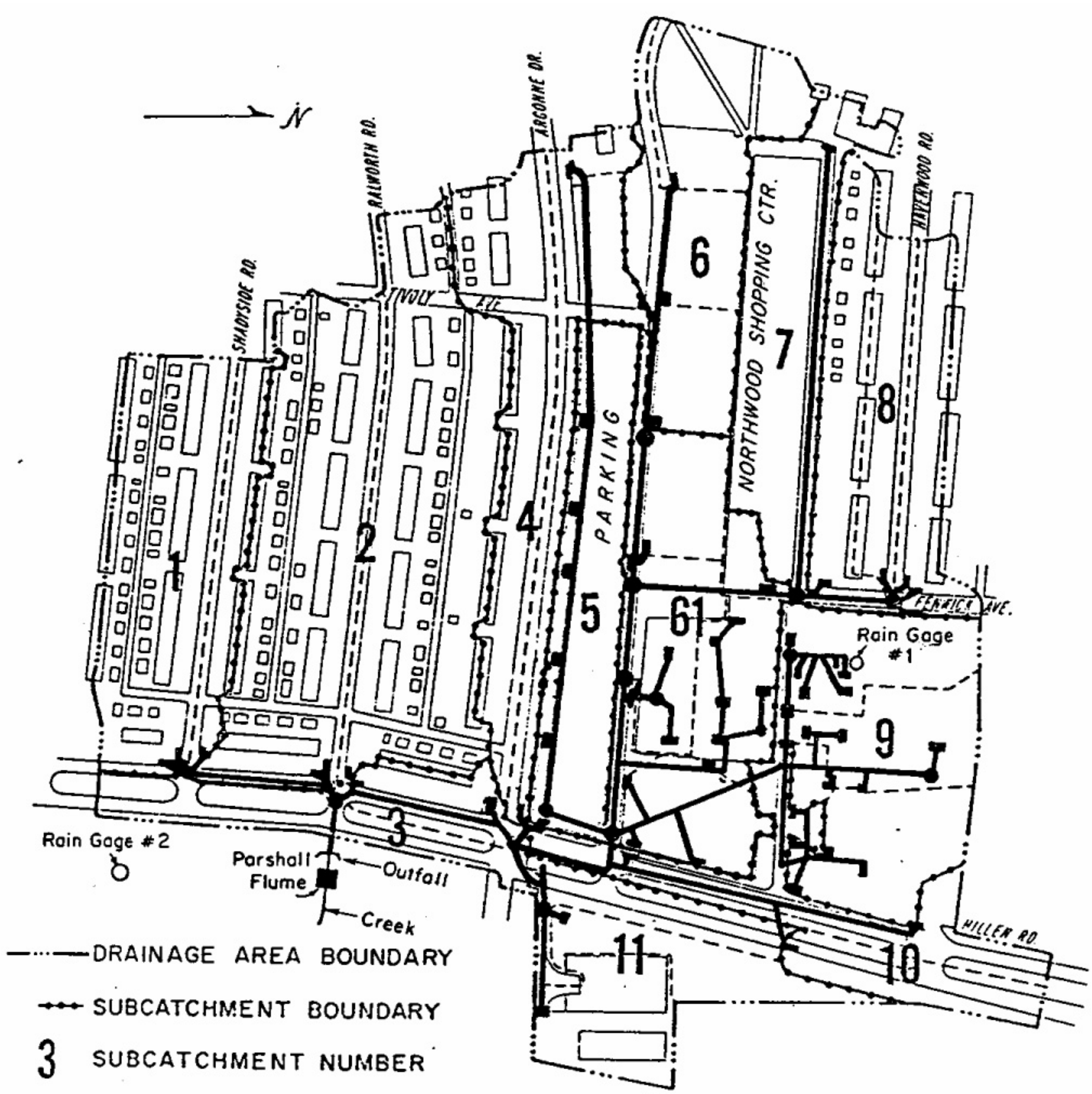


Figure 4-6. Northwood (Baltimore) drainage basin "fine" plan (after Metcalf and Eddy et al., 1971a, p. 50).

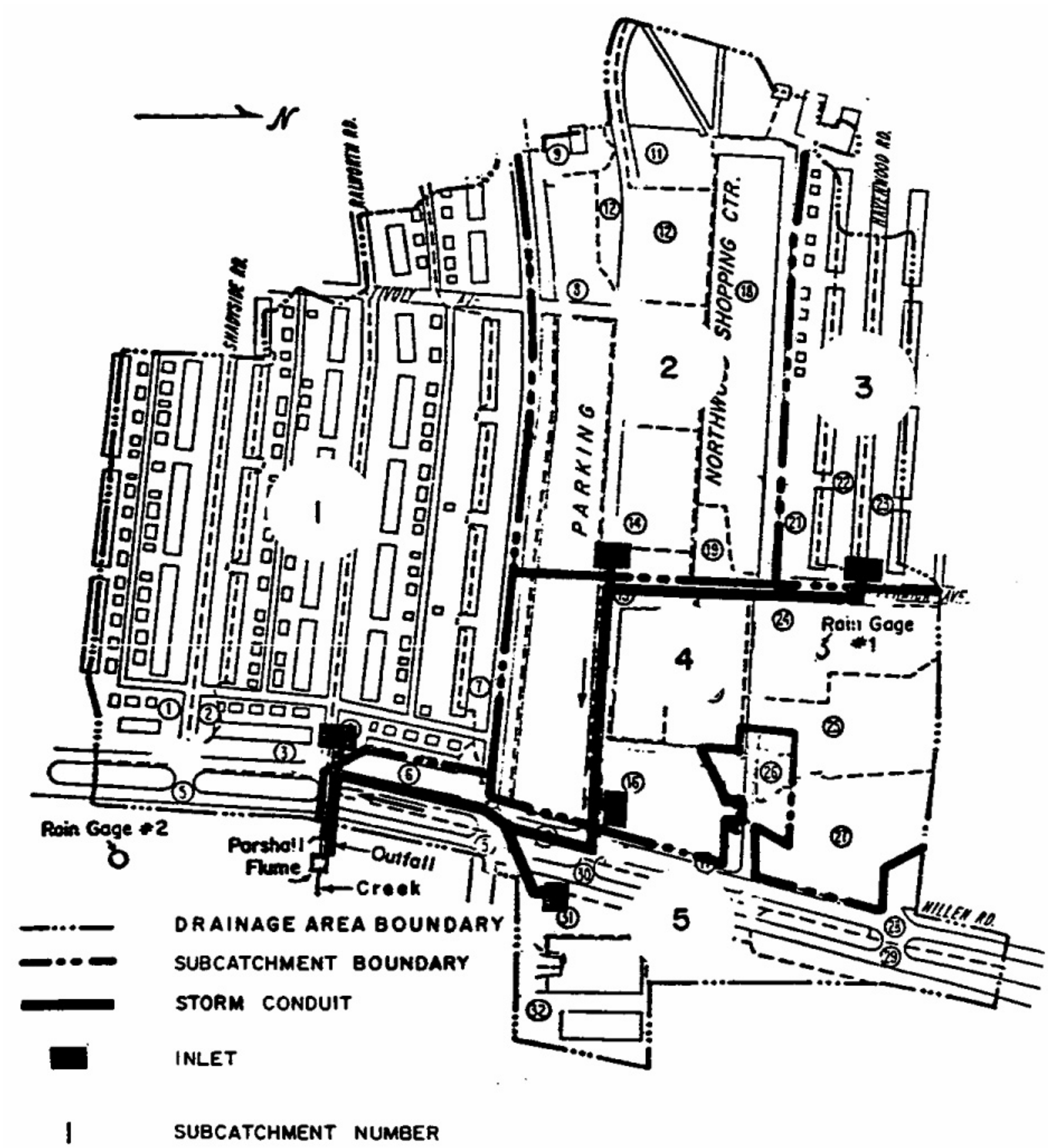


Figure 4-7. Northwood (Baltimore) drainage basin "coarse" plan (after Metcalf and Eddy et al., 1971a, p. 51).

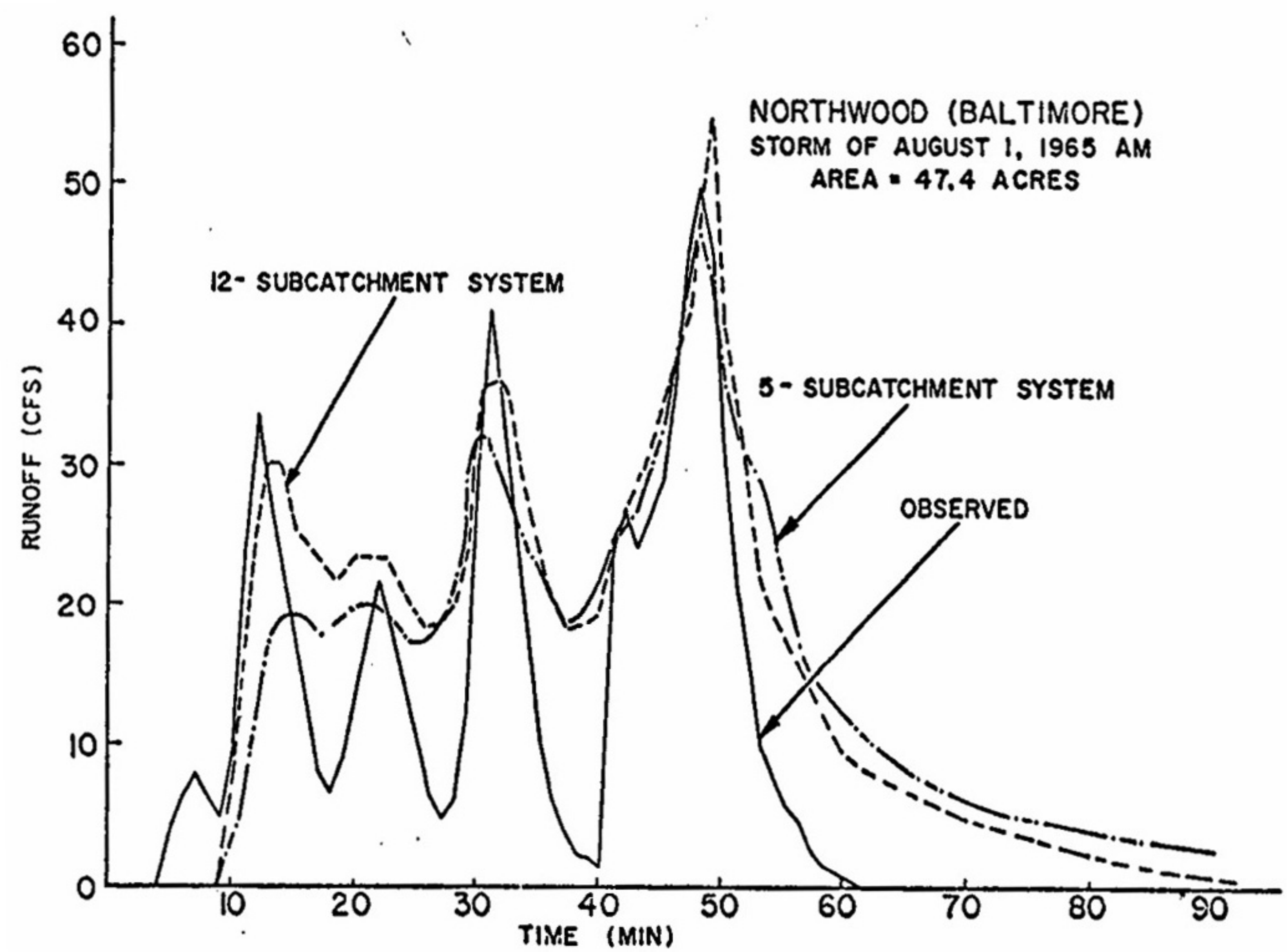


Figure 4-8. Effect of coarse subcatchment system, Northwood (Baltimore) (after Metcalf and Eddy et al., 1971a, p. 74).

Required Amount of Detail

It is anticipated that only a very coarse discretization will be used for continuous simulation. Although up to 200 subcatchments and channel/pipes or inlets are allowed, a typical hourly continuous simulation might include only one subcatchment and no channel/pipes. This economy in the amount of detail simulated is prompted to save computer time and because detail simply is not required for continuous simulation which serves as a screening and planning tool (see Appendix I). Moreover, reasonable agreement is possible between hydrographs produced by coarse and fine schematizations as will be discussed later under "subcatchment aggregation."

Should flow routing be desired during continuous simulation, Runoff Block channel/pipes ordinarily would be used. Although the Transport and Extran Blocks are intended primarily for single event analysis, they may also be employed, at the expense of slightly more interfacing effort. There are no limitations on the number of time steps for any block.

For a single event simulation, the amount of detail should be the minimum consistent with requirements for within-catchment information. Obviously, no information can be obtained about upstream surcharging if the upstream conduits are not simulated and subcatchments are not provided to feed them. In addition, sufficient detail needs to be provided to allow within-system control options to be tried for different areas and land uses. If, however, the primary objective is simply to produce a hydrograph and pollutograph at the outlet, utilizing a single raingage, then one subcatchment will often (but not always) serve as well as many.

A final constraint on the amount of detail is dictated by personnel requirements for data reduction. Once data resources (e.g., maps, plans) are gathered, discretization of the catchment can occupy one to three person-days (a longer time for more subcatchments) with perhaps an additional 15 to 30 minutes per subcatchment for their input parameters. Finally, there is not one "right" way to accomplish the discretization, especially since decisions at this stage can be compensated for during the later calibration phase.

Choice of Sewer System Flow Routing

There are many criteria that influence the choice of the block used for sewer system routing: Runoff, Transport or Extended Transport. Several of these are given in Table 4-3; much more extensive information is contained in this manual and other SWMM documentation (Metcalf and Eddy et al., 1971a; Roesner et al., 1987) pertaining to each block.

Regarding flow routing methods, no backwater effects can be calculated (i.e., in an upstream direction) in the Runoff and Transport Blocks because each conduit element simply provides an inflow to a downstream element with no effect of the latter on the former. Thus, both Runoff and Transport routing act as a "cascade" of elements, each discharging into the next with no other interactions. On the other hand, the solution of the complete St. Venant (gradually varied flow) equations by the Extran Block provides for backwater effects and much more, as indicated in Table 4-3. This is at the cost of considerable extra complexity and computer time.

As a practical matter, the Runoff Block is often used to simulate smaller diameter pipes, e.g., less than 30 in. (762 mm) and either the Transport or Extran Blocks for the larger trunk sewer system. The larger the catchment being simulated, the less important becomes the simulation of small conduits, far upstream. Conduits of less than a 12 in. (305 mm) diameter are rarely simulated. Also, in spite of the fact that in the Runoff Block, trapezoidal conduits could be used to simulate street gutters, it should almost never be necessary to simulate flow in a roadside curb and gutter channel, unless the catchment is extremely small.

Table 4.3. Flow Routing Characteristics of Runoff, Transport and Extended Transport Blocks

		Runoff Block	Transport Block	Extended Transport Block
1.	Flow routing method	Non-linear reservoir, cascade of circuits	Kinematic wave, cascade of conduits	Complete equations, interactive conduit network
2.	Relative computational expense for identical network schematizations	Low	Moderate	High
3.	Attenuation of hydrograph peaks	Yes	Yes	Yes
4.	Time displacement of hydrograph peaks	Weak	Yes	Yes
5.	In-conduit storage	Yes	Yes	Yes
6.	Backwater or downstream control effects	No	No[a]	Yes
7.	Flow reversal	No	No	Yes
8.	Surcharge	Weak	Weak	Yes
9.	Pressure flow	No	No	Yes
10.	Branching tree network	Yes	Yes	Yes
11.	Network with looped connections	No	No	Yes
12.	Number of pre-programmed conduit shapes	3	16	8
13.	Alternative hydraulic elements (e.g., pumps, weirs, regulators)	No	Yes	Yes
14.	Dry weather flow and infiltration generation (base flow)	No	Yes	Yes
15.	Pollutograph routing	Yes	Yes	No
16.	Solids scour/deposition	No	Yes	No
17.	Card input of hydrographs/pollutographs	No	Yes	Yes

[a]Backwater may be simulated as a horizontal water surface behind a storage element.

Numbering Schemes

Subcatchments may be assigned any numbers between 1 and 9999. This is true also for channel/pipes and inlets except that the first number used for printing, and inlet numbers corresponding to Transport Block manholes must be less than or equal to 10,000. Other possible downstream external programs may have their own numbering requirements that should be recognized at this stage. Thus, inflows to such junctions must be numbered accordingly. To be on the safe side, it is often a good idea to reserve relatively low numbers for inlets, etc. that are transferred to subsequent blocks.

Internally, the Runoff Block assigns subscripts (internal numbers) in the order in which the channel/pipes or subcatchment groups are read in. Some error messages use these numbers. It is not necessary to state specifically the inlets to be transferred to subsequent blocks, since all inlets at the downstream end of any subcatchment-channel/pipe flow routing chain are placed in that category and are printed out.

Within the above confines, considerable latitude exists for numbering schemes. Thus, subcatchments may feed channel/pipes with the same number; subcatchments or channel/pipes may be given numbers in a certain range (e.g., 200-299) based on certain characteristics; etc. The Transport Block numbering scheme allows even more latitude since it includes non-conduits (e.g., manholes).

Channel/Pipe Data (Groups G1 and G2)

Routing and Time Step Considerations

The nonlinear reservoir method of channel/pipe flow routing is described in Appendix V, as well as in the original documentation (Metcalf and Eddy et al., 1971a). Since the formulation produces a spatially "lumped" configuration (i.e., there is no dependence upon longitudinal distance for a given channel/pipe element), flows introduced at the "upstream end" of such an element are distributed horizontally over the entire water surface area. The implication is that a concentrated inflow into one "end" of a simulated channel/pipe is a reasonable approximation to the true situation in which channel/pipes receive distributed inflows along their lengths.

At each time step, an iterative (Newton-Raphson) scheme is used to solve the non-linear difference equation used to approximate the differential equation of the non-linear reservoir. The iterative scheme solves for the new channel/pipe depth based on the inflow, initial volume, and the outflow which is based on Manning's equation. The iteration is performed by Subroutine GUTNR and is globally convergent with no restrictions on time step size. The Newton-Raphson method has quadratic convergence and the number of iterations required in GUTNR is usually less than 3.

Three channel shapes are programmed in Runoff: trapezoidal channels, parabolic channels and circular pipes, as shown in Figure 4-9. This translates to five shapes since trapezoidal channels function as three channel shapes: (1) trapezoidal, (2) triangular, and (3) rectangular. A trapezoidal channel has a bottom width, maximum depth, and two side slopes. A triangular channel has no bottom width, maximum depth, and two side slopes. A rectangular or box channel has a bottom width, maximum depth, and side slopes of zero.

Parabolic channels can be used to approximate "natural channels." Parabolas are often used to characterize the cross sections of small and medium size channels (Chow, 1959). Only a top width (T) and maximum depth (D) are necessary to characterize the symmetric parabolic channel. The equation describing the parabolic channel is:

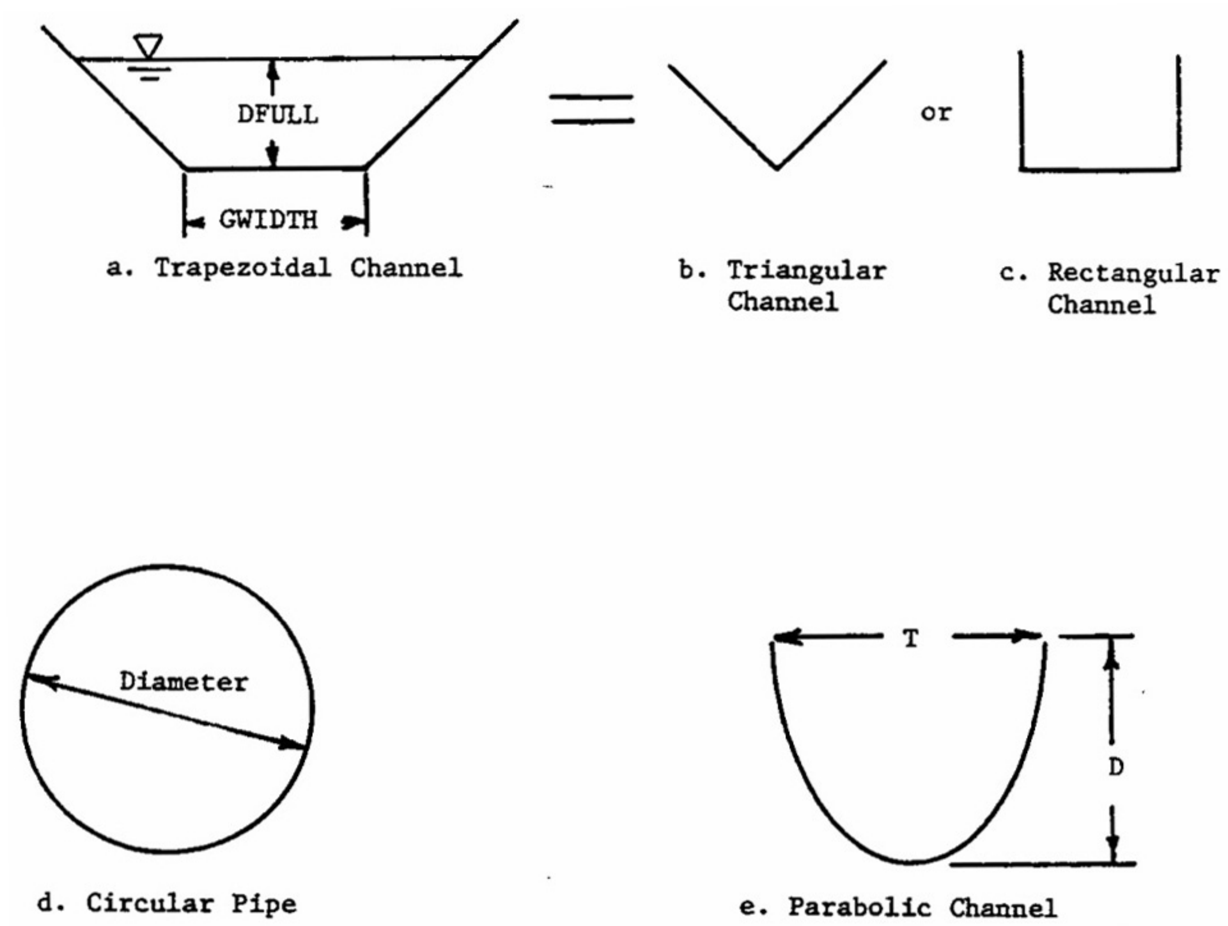


Figure 4-9. Channels and pipe of the Runoff Block.

$$X^2 = k \cdot Y \tag{4-4}$$

where

X	=	horizontal distance from channel center, ft [m],
Y	=	vertical distance from channel invert, ft [m], and
k	=	$T^2/(4 \cdot D)$.

Broad crested, narrow crested, V-notch weirs and orifices can be simulated by linking a trapezoidal channel, parabolic channel or circular pipe with an outflow equation. The volume of the channel is based on either a parabolic, trapezoidal or circular cross section. The weir or orifice outflow equation instead of Manning's equation is used in the non-linear convergence scheme. Data group G2 is used to input the control structure parameters. The weir or orifice is not a separate channel but a modification to the outflow of a modeled channel. Sample configurations are shown in Figure 4-10.

The broad and narrow crested weir equation used in Runoff is the standard weir equation:

$$Q = C \cdot L \cdot (h-h_c)^{1.5} \tag{4-5}$$

where

Q	=	outflow, cfs [m^3/sec],
C	=	weir coefficient, $ft^{1/2}$/sec [$m^{1/2}$/sec],
L	=	weir length, ft [m],
h	=	hydraulic head, ft [m], and
h_c	=	weir crest, ft [m].

The triangular opening of a V-notch weir is assumed to have no upper limit. The equation for V-notched weirs used in Runoff is:

$$Q = C \cdot \tan(a/2) \cdot (h-h_c)^{2.5} \tag{4-6}$$

where,

Q	=	outflow, cfs [m^3/sec],
C	=	weir coefficient, $ft^{1/2}$/sec [$m^{1/2}$/sec],
a	=	angle of notch (angle of opening), degrees,
h	=	hydraulic head, ft [m], and
h_c	=	weir crest (bottom of notch), ft [m].

The orifice is either: (1) a dropout or sump orifice, or (2) a side outlet orifice. A standard orifice equation is used for both types:

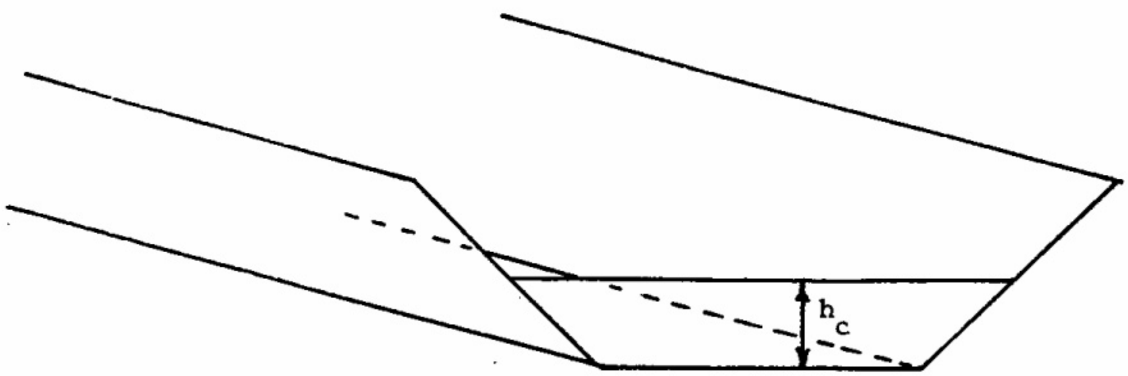


Figure 4-10. Example weir and orifice configurations.

$$Q = C_d ⟐ A ⟐ [2⟐g⟐(h-h_c)]^{0.5} \qquad (4\text{-}7)$$

where

Q	=	outflow, cfs [m^3/sec],
C_d	=	orifice discharge coefficient,
A	=	orifice cross sectional area, ft^2 [m^2],
g	=	gravitational acceleration = 32.2 ft/sec^2 or 9.8 m/sec^2,
h	=	hydraulic head above the orifice, ft [m], and
h_c	=	orifice centerline.

The weir or orifice crest height may be used to store water in a channel or pipe. If groundwater is simulated then the stored channel water affects the groundwater flow via the tailwater flow equation. Trapezoidal and parabolic channels lose water through outflow and evaporation. Pipes lose water only through outflow.

Parameter Selection

Most channel/pipe parameters are self explanatory and little interpretation is needed. The slope and roughness are combined into one parameter for further use in the program, using Manning's equation. Thus,

$$GCON = \frac{KM}{G6} \cdot G3^{1/2} \qquad (4\text{-}8)$$

where

GCON	=	routing parameter,
KM	=	1.49 for units of feet and seconds and equals 1.0 for units of meters and seconds (not required by program),
G6	=	Manning's roughness, n, and
G3	=	invert slope.

Thus, equivalent changes in the routing can be made through changes in either the slope or roughness. An equivalent routing parameter is made for weirs and orifices using the weir coefficient and weir length (or notch angle), or the orifice coefficient and orifice cross sectional area.

Note that U.S. customary units (ft-sec) are used internally in the Runoff Block. When metric units are requested, input and output are converted to and from U.S. customary units to preserve ft-sec units internally. This scheme is also used in the Transport Block. However, metric calculations in the Extran and Storage/Treatment Blocks are used consistently throughout the program, when requested.

The invert slope is usually given on drainage maps or may easily be calculated from invert elevations and conduit lengths. Tables of Manning's roughness coefficient are given in many references; see for instance Chow (1959) or ASCE-WPCF (1969).

Subcatchment Surface Data (Group H1)

Subcatchment Schematization

Many hydrologic models account for spatial variations by subdividing the overall catchment into subcatchments, predicting runoff from the subcatchments on the basis of their individual properties, and combining their outflows using a flow routing scheme. This procedure is followed in SWMM, in which subcatchments are idealized mathematically as spatially lumped, non-linear reservoirs, and their outflows are routed via the channel/pipe (or a subsequent Transport Block) network.

Each subcatchment is schematized as in Figure 4-11, in which three or four subareas (depending on whether snowmelt is simulated) are used to represent different surface properties as enumerated in Table 4-4. The slope of the idealized subcatchment is in the direction perpendicular to the width. Flow from each subarea moves *directly* to a gutter/pipe or inlet and does *not* pass over any other subarea. (Thus, it is not possible to route runoff from roofs over lawn surfaces, for instance).

The width of the pervious subarea, A2, is the entire subcatchment width, whereas the widths of the impervious subareas, A1, A3, A4, are in proportion to the ratio of their area to the total impervious area, as implied in Figure 4-11. Specification of each subarea is through the use of parameters WAREA and WW(3) in Group H1, PCTZER in group B4 and SNN1 in group I1. If desired, any subcatchment may consist entirely of any one (or more) types of subareas.

Of course, real subcatchments seldom exhibit the uniform rectangular geometries shown in Figure 4-11. In terms of the flow routing, all geometrical properties are merely parameters (as explained below) and no inherent "shape" can be assumed in the non-linear reservoir technique. However, in terms of parameter selection, the conceptual geometry of Figure 4-11 is useful because it aids in explaining the flow routing.

Table 4-4. Subcatchment Surface Classification

Subarea	Perviousness	Depression Storage	Snow Cover and Extent	
			Single Event	Continuous
A1	Impervious	Yes	Bare	Normally bare, but may have snow cover over 100% of Subarea A1 plus Subarea A3.
A2	Pervious	Yes	Constant fraction, SNCP, of area is snow covered.	Snow covered subject to areal depletion curve.
A3	Impervious	No	Bare	Same as Subarea A1.
A4	Impervious	Yes	100% covered.	Snow covered subject to areal depletion curve.

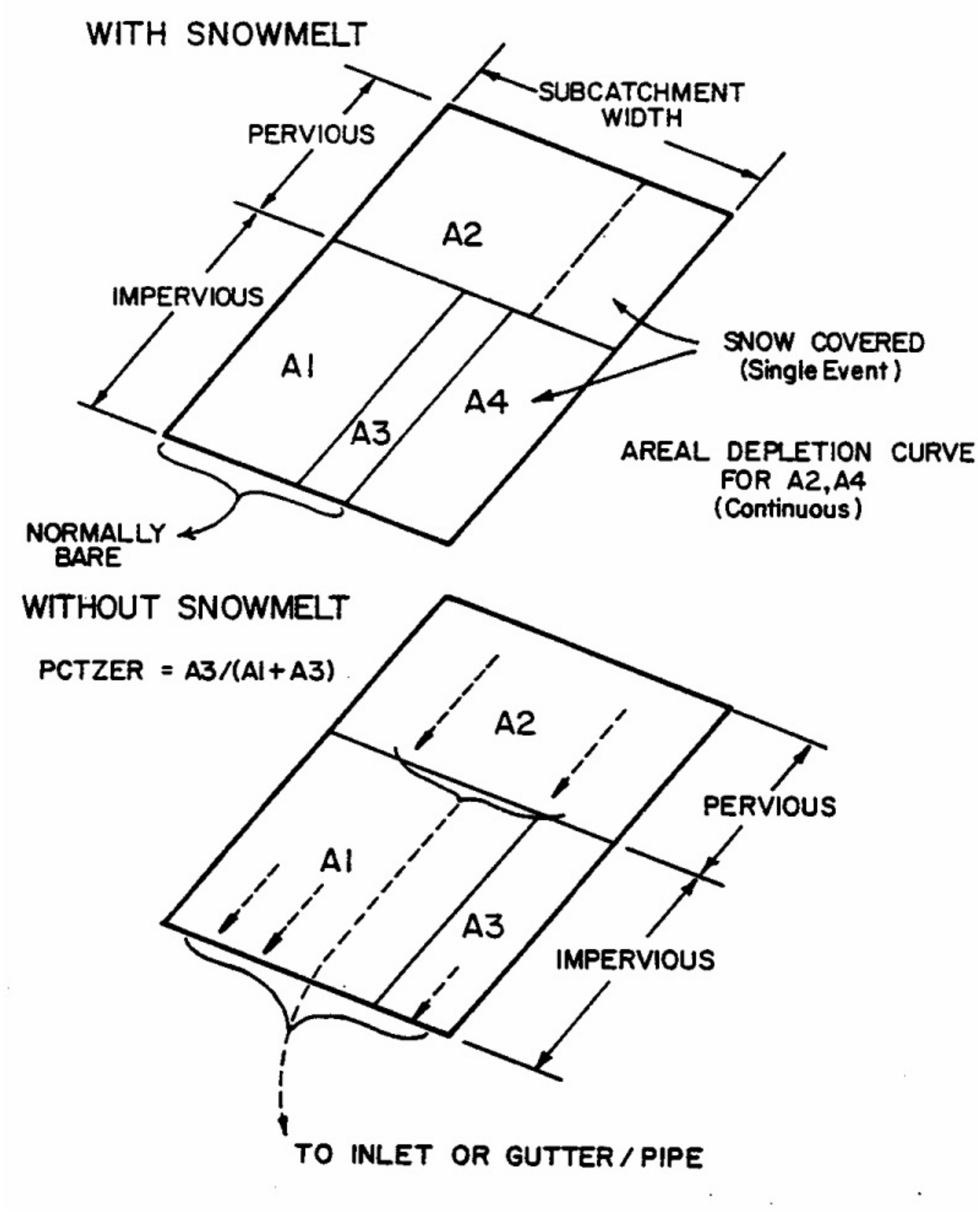


Figure 4-11. Subcatchment schematization. Flows from pervious and total impervious subareas go <u>directly</u> to gutter/pipe or inlet. (E.g., flow from the pervious subarea does <u>not</u> travel over impervious area.)

Routing and Time Step Considerations

The routing and time step discussion given earlier for channel/pipes applies almost identically for subcatchments. A detailed explanation of the non-linear reservoir equations is given in Appendix V. The routing is performed separately for each of the three of four subareas of the subcatchment. Convergence problems are rarely encountered during subcatchment routing because total subcatchment volumes (area times depth) are usually large compared to outflow volumes.

Parameter selection is aided with reference to Figure 4-12 in which the subcatchment "reservoir" is shown in relation to inflows and outflows (or losses). The outflow to channel/pipes and inlets is computed as the product of velocity (from Manning's equation based on the difference between total depth and depression storage), depth and width,

$$Q = W \frac{1.49}{n} \left(d - d_p \right)^{5/3} S^{1/2} \tag{4-9}$$

where

Q	=	WFLOW = subcatchment (or subarea) outflow, cfs,
W	=	WW(1) = subcatchment width, ft,
n	=	WW(5) or WW(6) = Manning's roughness coefficient,
d	=	WDEPTH = water depth, ft,
d_p	=	WSTORE = depth of depression (retention) storage, ft, and
S	=	WSLOPE = slope, ft/ft.

The FORTRAN parameters listed above are the Runoff Block parameters. When combined with the continuity equation (see Appendix V) and divided by the surface area, a new routing parameter is defined for the pervious and total impervious subcatchment areas and used in all subsequent calculations,

$$WCON = - \frac{1.49\, W}{A\, n} S^{1/2} \tag{4-10}$$

where

WCON	=	routing parameter used in subroutine WSHED, ft-sec units, and
A	=	surface area of pervious or total impervious subarea, ft^2.

Note that the width, slope and roughness parameters are combined into one parameter. Thus, equivalent changes may be caused by appropriate alteration of any of the three parameters. Note also that the width and slope are the same for both pervious and impervious areas. Manning's roughness and relative area are the only parameters available to the modeler to characterize the relative contributions of pervious and impervious areas to the outlet hydrograph. (However, see further comments below on the subcatchment width.) Flows computed in the Runoff Block and transferred to subsequent blocks are instantaneous values at the end of a time step.

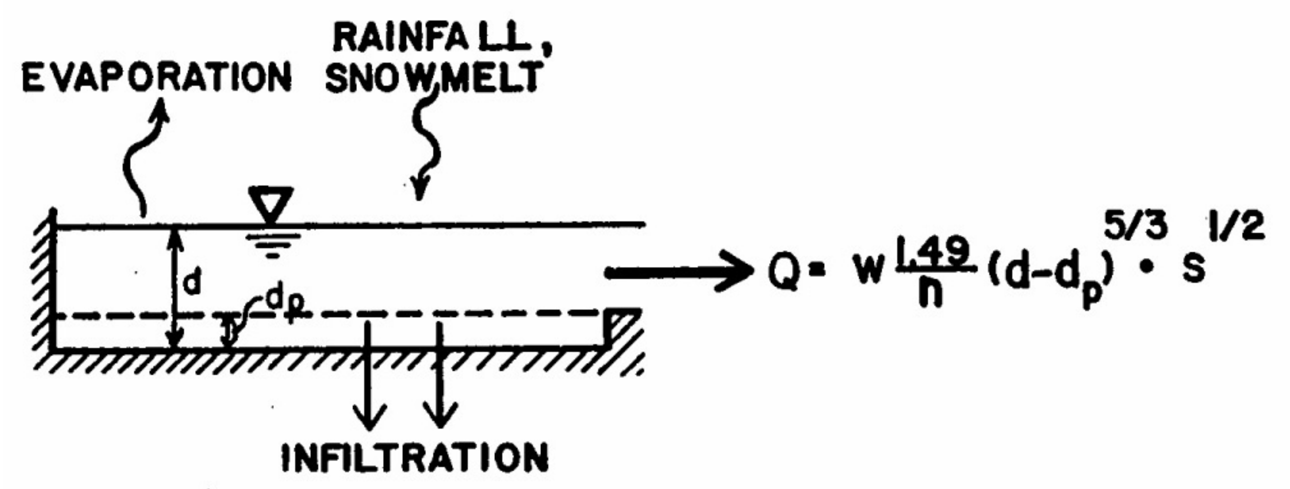


Figure 4-12. Nonlinear reservoir representation of subcatchment.

Subcatchment Width

If overland flow is visualized (Figure 4-11) as running down-slope off an idealized, rectangular catchment, then the width of the subcatchment (data group H1) is the physical width of overland flow. This may be further seen in Figure 4-13 in which the lateral flow per unit width, q_L, is computed and multiplied by the width to obtain the total inflow into the channel. (As mentioned previously, the SWMM channel/pipes can only receive a concentrated inflow, however, and do not receive a distributed inflow in a specific fashion.) Note also in Figure 4-13 that for this idealized case, if the two sides of the subcatchment are symmetrical the total width is twice the length of the drainage channel.

Since real subcatchments will not be rectangular with properties of symmetry and uniformity, it is necessary to adopt other procedures to obtain the width for more general cases. This is of special importance, because if the slope and roughness are fixed (see equation 4-10), the width can be used to alter the hydrograph shape.

For example, consider the five different subcatchment shapes shown on Figure 4-14. Catchment hydraulic properties, routing parameters and time of concentration are also given. The latter is calculated using the kinematic wave formulation (Eagleson, 1970, p. 340),

$$t_c = \left[\frac{L}{a\ i^{*m-1}} \right]^{l/m} \tag{4-11}$$

where

t_c	=	time of concentration, sec,
L	=	subcatchment *length*, ft,
i*	=	rainfall excess (rainfall minus losses), ft/sec, and
a,m	=	kinematic wave parameters.

The kinematic wave formulation assumes that the runoff per unit width (velocity times depth) from the subcatchment is

$$q_L = a\ d^m \tag{4-12}$$

where

q_L	=	flow per unit width, ft^2/sec, and
d	=	depth of flow, ft.

Parameters a and m depend upon the uniform flow equation used for normal flow. For Manning's equation,

$$a = (1.49/n)\ A\ S^{1/2} \tag{4-13}$$

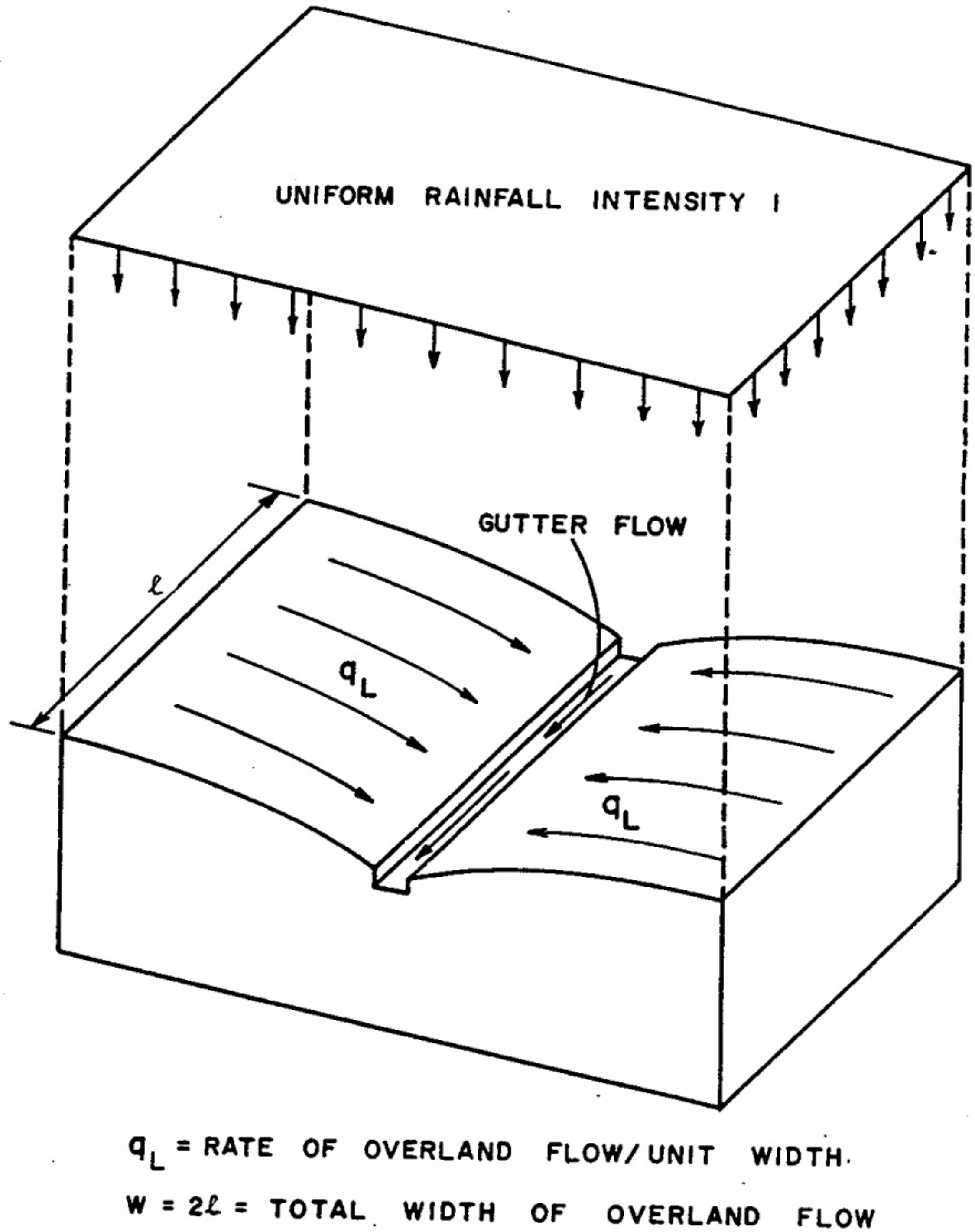
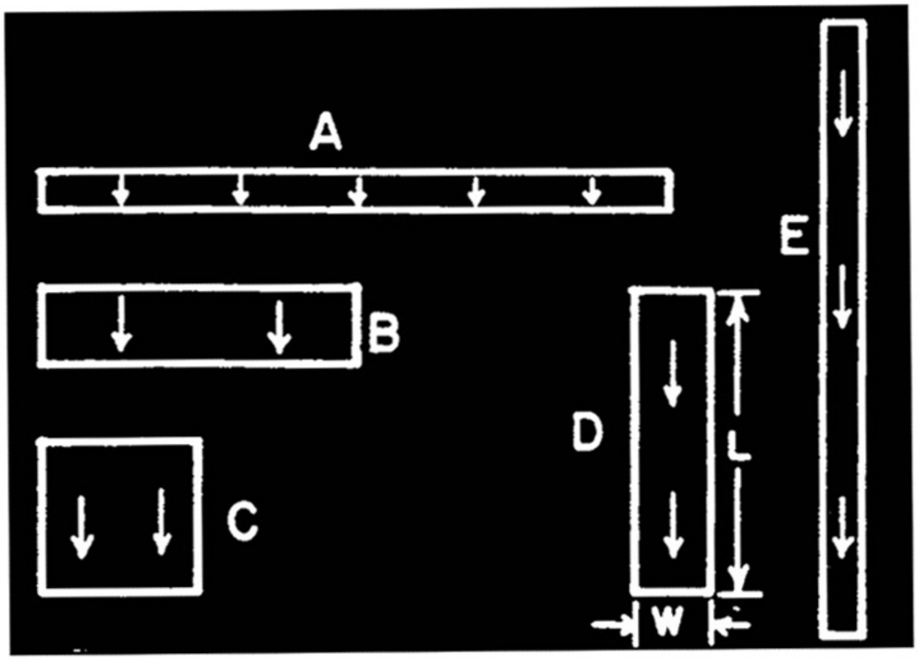


Figure 4-13. Idealized subcatchment-gutter arrangement illustrating the subcatchment width.



Slope = 0.01
Imperviousness = 100%
Depression Storage = 0
n = 0.02
Equilibrium outflow = i*A = 0.926 cfs

DELT = 5 min = 300 sec
i* = Rainfall = 1.0 in./hr = 0.000023148 ft/sec

Shape	A (ft^2)	W (ft)	L (ft)	t_c[a] (min)	WBCON[b] (ft-sec units)
A	40,000	800	50	3.7	-0.149
B	40,000	400	100	5.7	-0.0745
C	40,000	200	200	8.6	-0.03725
D	40,000	100	400	13.0	-0.018625
E	40,000	50	800	19.7	-0.0093125

[a]Equation 4-7
[b]Equation 4-6

Figure 4-14. Different subcatchment shapes to illustrate effect of subcatchment width.

and

$$m = 5/3 \tag{4-14}$$

Note that the units of a depend upon the value of m, and for Manning's equation, feet-second units should be used for all calculations. The subcatchment length may be computed for the assumed rectangular shape simply by dividing the area by the width.

Finally, note the dependence of time of concentration upon the rainfall intensity. As i* increases, t_c decreases. The calculation using equation 4-11 is consistent with the definition of t_c given earlier: t_c is the time to equilibrium, at which inflow equals outflow (for an impervious catchment). Equivalently, t_c is the time taken for the most remote portion of the catchment to contribute to flow at the outlet, which is the time taken by a <u>wave</u> (<u>not</u> a parcel of water) to travel from the remote point to the outlet.

Outflow hydrographs for continuous rainfall and for rainfall of duration 20 min are shown on Figure 4-15. These were computed by the Runoff Block non-linear reservoir equations (Appendix V) using a time step of 5 min. Clearly, as the subcatchment width is narrowed (i.e., the outlet is constricted), the time to equilibrium increases. Thus, it is achieved quite rapidly for cases A and B and more slowly for cases C, D and E. The kinematic wave computation of t_c (Figure 4-14) is not particularly accurate for the non-linear reservoirs for which the asymptotic value of equilibrium outflow is approached exponentially. However, it may be used for guidance.

Two routing effects may be observed. A <u>storage effect</u> is very noticeable, especially when comparing hydrographs A and E for a duration of 20 min. The subcatchment thus behaves in the familiar manner of a reservoir. For case E, the outflow is constricted (narrow); hence, for the same amount of inflow (rainfall) more water is stored and less released. For case A, on the other hand, water is released rapidly and little is stored. Thus case A has both the fastest rising and recession limbs of the hydrographs.

A <u>shape effect</u> is also evident. Theoretically, all the hydrographs peak simultaneously (at the cessation of rainfall). However, a large width (e.g., case A) will cause equilibrium outflow to be achieved rapidly, producing a flat-topped hydrograph for the remainder of the (constant) rainfall. Thus, for a catchment schematized with several subcatchments and subject to variable rainfall, increasing the widths tends to cause peak flows to occur sooner. In general, however, shifting hydrograph peaks in time is difficult to achieve through adjustment of Runoff Block flow routing parameters. The time distribution of runoff is far and away the most sensitive to the time distribution of rainfall. Further discussion of the effect of subcatchment width on hydrograph shapes will be given below under "Subcatchment Aggregation and Lumping."

What is the best estimate of subcatchment width? If the subcatchment has the appearance of Figure 4-13, then the width is approximately twice the length of the main drainage channel through the catchment. However, if the drainage channel is on the side of the catchment as in Figure 4-14, the width is just equal to the length of the channel. A good estimate for the width can be obtained by first determining the maximum <u>length</u> of overland flow and dividing the area by this length.

Most real subcatchments will be irregular in shape and have a drainage channel which is off center, as in Figure 4-16. This is especially true of rural or undeveloped catchments. A simple way of handling this case is given by DiGiano et al. (1977). A skew factor is computed,

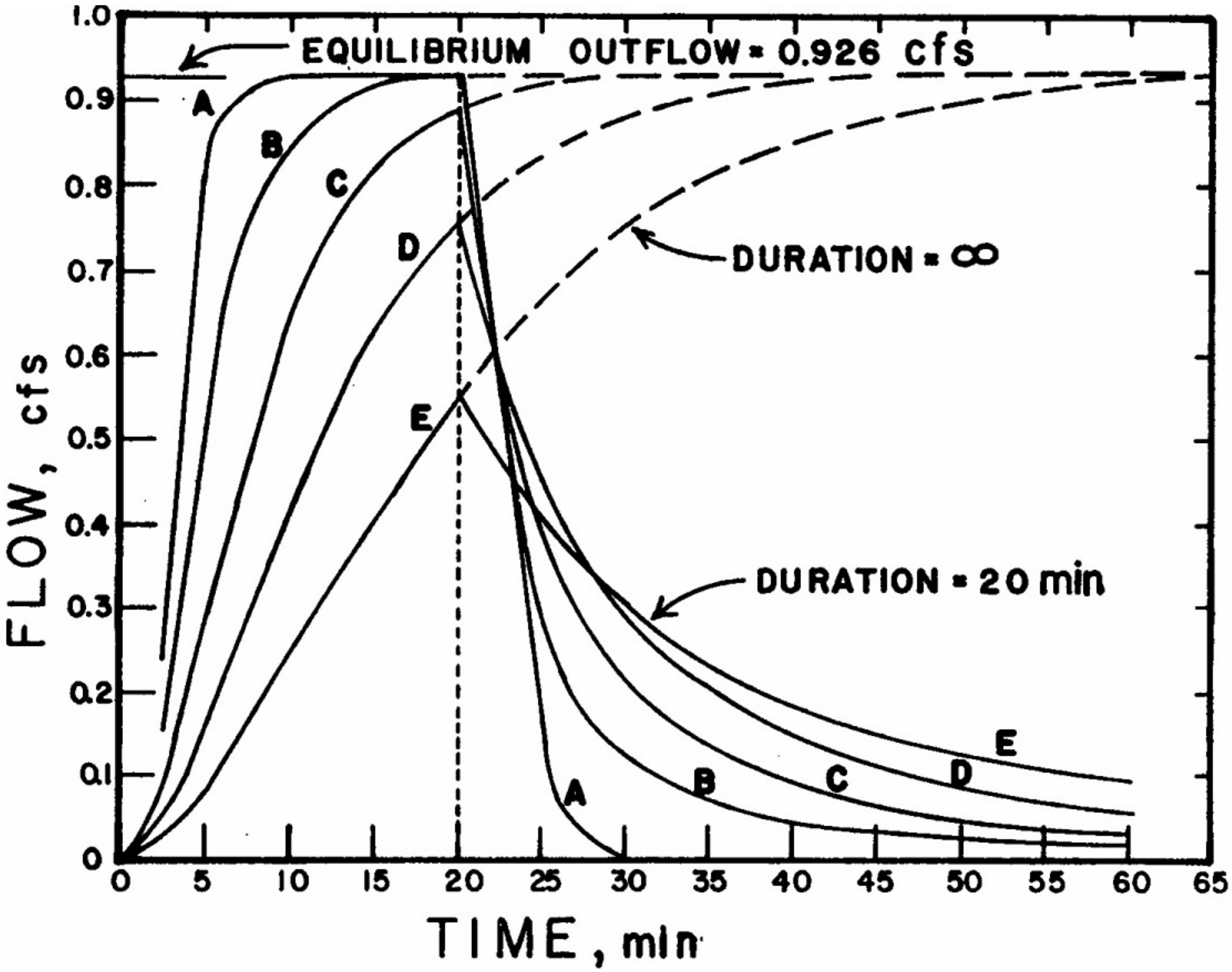


Figure 4-15. Subcatchment hydrographs for different shapes of Figure 4-14.

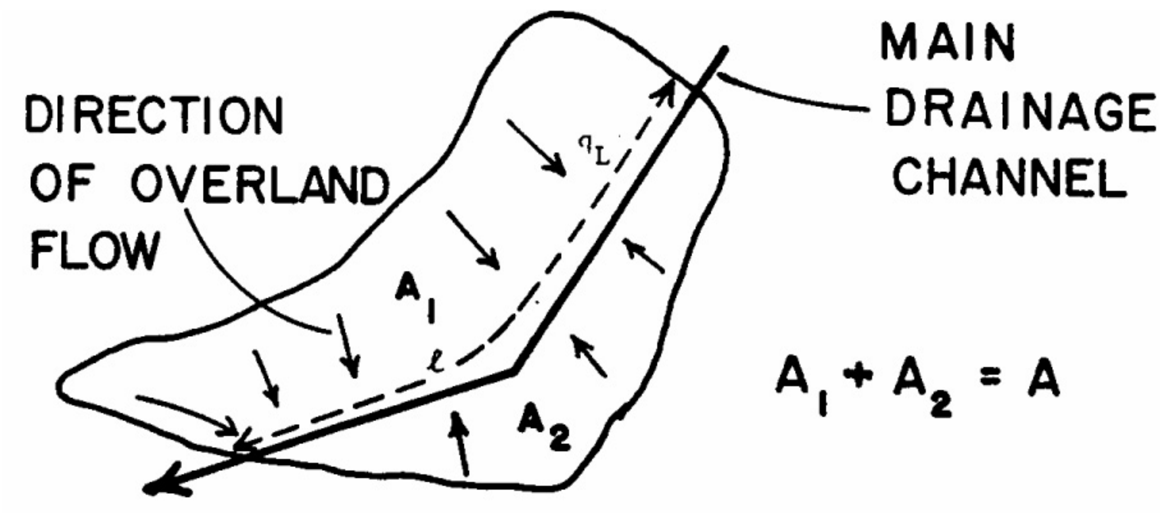


Figure 4-16. Irregular subcatchment shape for width calculation (after DiGiano et al., 1977, p. 165).

$$S_k = (A_2 - A_1)/A \tag{4-15}$$

where

S_k	=	skew factor, $0 \le S_k \le 1$,
A_1	=	area to one side of channel,
A_2	=	area to other side of channel, and
A	=	total area.

The width is simply weighted between the two limits of *l* and 2*l* as

$$W = (2 - S_k) \cdot l \tag{4-16}$$

where

W	=	subcatchment width, and
l	=	length of main drainage channel.

To reiterate, changing the subcatchment width changes the routing parameter WCON of equation 4-10. Thus, identical effects to those discussed above may be created by appropriate variation of the roughness and/or slope.

Subcatchment Area

In principle, the catchment and subcatchment area can be defined by constructing drainage divides on topographic maps. In practice, this may or may not be easy because of the lack of detailed contour information and the presence of unknown inflows and outflows. This may be most noticeably brought to the modeler's attention when the measured runoff volume exceeds the measured rainfall volume, if the latter is correct. Actually storm rainfall is seldom accurately measured over all subcatchments.

From the modeling standpoint, there are no upper or lower bounds on subcatchment area (other than to avoid convergence problems, as discussed earlier). Subcatchments are usually chosen to coincide with different land uses, with drainage divides, and to ease parameter estimation, i.e. homogeneous slopes, soils, etc. Further guidance is given later under subcatchment aggregation.

Imperviousness

The percent imperviousness of a subcatchment is another parameter that can, in principle, be measured accurately from aerial photos or land use maps. In practice, such work tends to be tedious, and it is common to make careful measurements for only a few representative areas and extrapolate to the rest.

Care must be taken to ensure that impervious areas are hydraulically (directly) connected to the drainage system. For instance, if rooftops drain onto adjacent pervious areas, they should not be treated as a hydraulically effective impervious area in the Runoff Block. Such areas are noneffective impervious areas (Doyle and Miller, 1980). On the other hand, if a driveway drains to a street and thence to a stormwater inlet, the driveway would be considered to be hydraulically connected. Rooftops with downspouts connected directly to a sewer are definitely hydraulically connected.

Should rooftops be treated as "pervious," the real surrounding pervious area is subject to more incoming water than rainfall alone and thus might produce runoff sooner than if rainfall alone were considered. In the unlikely event that this effect is important (a judgment based on infiltration parameters) it could be modeled by altering the infiltration parameters or by treating such pervious areas as separate subcatchments, and increasing their rainfall by the ratio of roof area plus pervious to pervious alone. Since the roof areas would then not be simulated, continuity would be maintained.

Another method of estimating the effective impervious area given measured data is to plot the runoff (in. or mm) vs. rainfall (in. or mm) for small storms. The slope of the regression line is a good estimate of the effective impervious area (Doyle and Miller, 1980). Further information on the concept of hydraulically connected (or "hydraulically effective") impervious areas is contained in USGS studies (Jennings and Doyle, 1978) and documentation of the ILLUDAS model (Terstriep and Stall, 1974).

For continuous simulation in which very large subcatchments are being used, even spot calculations of imperviousness may be impractical. Instead, regression formulations have been developed in several studies (Graham et al., 1974; Stankowski, 1974; Manning et al., 1977; Sullivan et al., 1978). These typically relate percent imperviousness to population density, and are compared in Figure 4-17 (Heaney et al., 1977). The New Jersey equation (Stankowski, 1974) is perhaps the most representative:

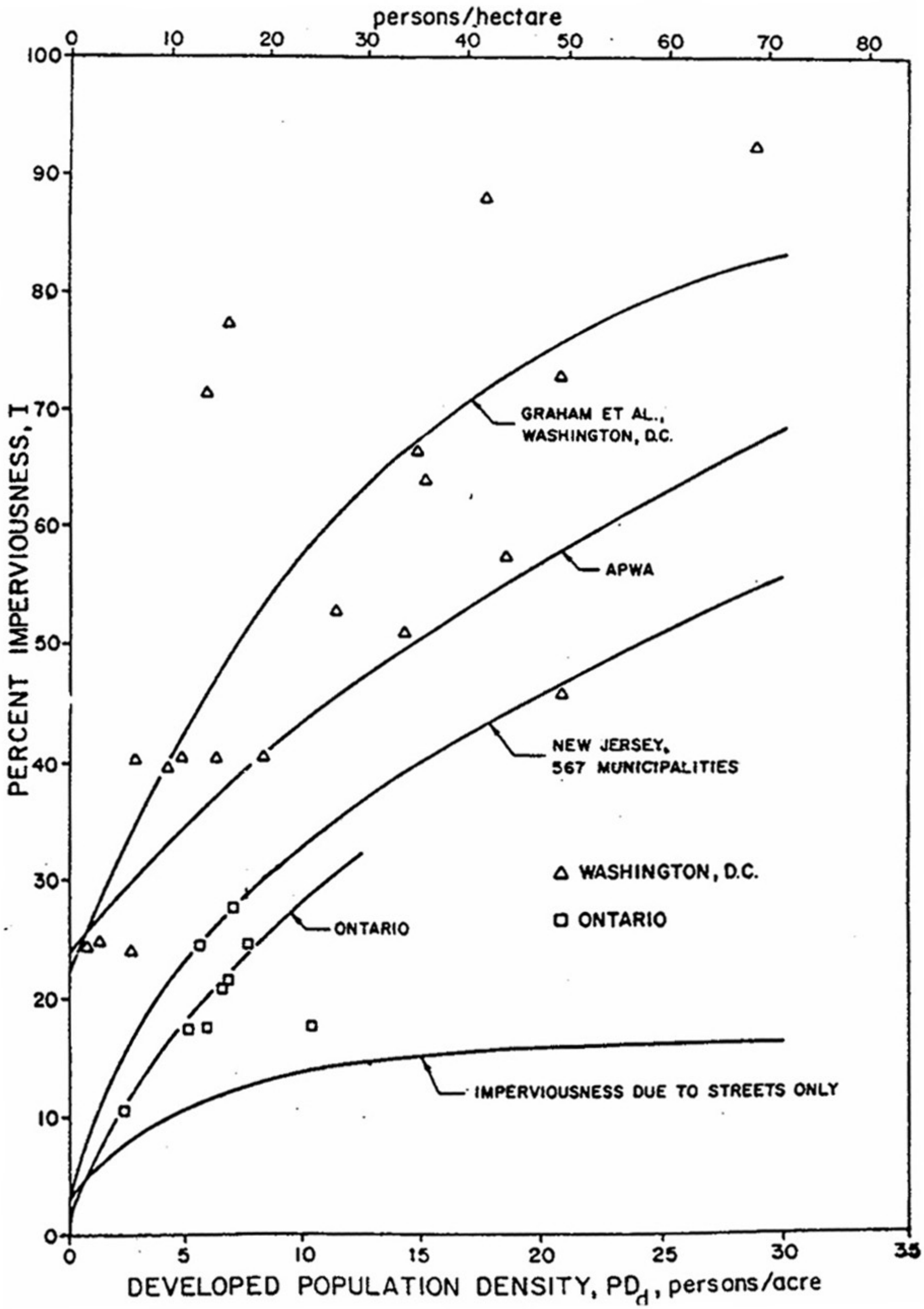


Figure 4-17. Percent imperviousness versus developed population density for large urban areas (after Heaney, et al., 1977, p. 105).

$$I = 9.6\, PD_d^{\left(0.53 - 0.0391 \log^{10} PD_d\right)} \tag{4-16}$$

where

I	=	WW(3) = imperviousness, percent, and
PD_d	=	population density in developed portion of the urbanized area, person per acre.

The "developed portion" excludes large segments of undeveloped (i.e., natural or agricultural) lands that may lie within the area being simulated. Also note that the relationships shown in Figure 4-17 were all developed for large (city-wide) urban areas as a whole. Their use may be tenuous for smaller sub-basins.

Slope

The subcatchment slope should reflect the average along the pathway of overland flow to inlet locations. For a simple geometry (e.g., Figures 4-13 and 4-14) the calculation is simply the elevation difference divided by the length of flow. For more complex geometries, several overland flow pathways may be delineated, their slopes determined, and a weighted slope computed using a path-length weighted average. Such a procedure is described by DiGiano et al., 1977, pp. 101-102). Alternatively it may be sufficient to simulate what the user considers to be the hydrologically dominant slope for the conditions being simulated. Choose the appropriate overland flow length, slope, and roughness for this equivalent plane.

Manning's Roughness Coefficient, n

Values of Manning's roughness coefficient, n, are not as well known for overland flow as for channel flow because of the considerable variability in ground cover for the former, transitions between laminar and turbulent flow, very small depths, etc. Most studies indicate that for a given surface cover, n varies inversely in proportion to depth, discharge or Reynold's number. Such studies may be consulted for guidance (e.g., Petryk and Bosmajian, 1975; Chen, 1976; Christensen, 1976; Graf and Chun, 1976; Turner et al., 1978; Emmett, 1978), or generalized values used (e.g., Chow, 1959; Crawford and Linsley, 1966; Huggins and Burney, 1982; Engman, 1986). Roughness values used in the Stanford Watershed Model (Crawford and Linsley, 1966) are given in Table 4-5 along with more recent values from Engman (1986). Engman also provides values for other agricultural land uses and a good literature review.

Depression Storage

Depression (retention) storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas (see Figure 4-12); a good discussion is presented by Viessman et al. (1977). It represents a loss or "initial abstraction" caused by such phenomena as surface ponding, surface wetting, interception and evaporation. In some models, "depression storage" also includes infiltration in pervious areas. In the Runoff Block, water stored as depression storage on pervious areas is subject to infiltration (and evaporation), so that it is continuously and rapidly replenished. Water stored in depression storage on impervious areas is depleted only by evaporation. Hence, replenishment typically takes much longer.

Table 4-5. Estimates of Manning's Roughness Coefficients for Overland Flow

Source	Ground Cover	n	Range
Crawford and Linsley (1966)[a]	Smooth asphalt	0.01	
	Asphalt of concrete paving	0.014	
	Packed clay	0.03	
	Light turf	0.20	
	Dense turf	0.35	
	Dense shrubbery and forest litter	0.4	
Engman (1986)[b]	Concrete or asphalt	0.011	0.0-0.013
	Bare sand	0.01	0.01-0.016
	Graveled surface	0.02	0.012-0.03
	Bare clay-loam (eroded)	0.02	0.012-0.033
	Range (natural)	0.13	0.01-0.32
	Bluegrass sod	0.45	0.39-0.63
	Short grass prairie	0.15	0.10-0.20
	Bermuda grass	0.41	0.30-0.48

[a]Obtained by calibration of Stanford Watershed Model.
[b]Computed by Engman (1986) by kinematic wave and storage analysis of measured rainfall-runoff data.

As described earlier (e.g., Table 4-4), a percent "PCTZER" (data group B4) of the impervious area is assigned zero depression storage in order to promote immediate runoff. This percentage is the same for all subcatchments. Should variation among subcatchments be desired, PCTZER may be set to zero, and zero values for WSTORE entered in data group H1 as needed.

Depression storage may be derived from rainfall runoff data for impervious areas by plotting runoff volume (depth) as the ordinate against rainfall volume as the abscissa for several storms. The rainfall intercept at zero runoff is the depression storage. Data obtained in this manner from 18 urban European catchments (Falk and Niemczynowicz, 1978, Kidd, 1978a, Van den Berg, 1978) are summarized in Table 4-6. The very small catchments (e.g., less than 1 ac or 0.40 ha) were primarily roadway tributaries to stormwater inlets and catchbasins.

Table 4-6. Recent European Depression Storage Data (Kidd, 1978b)

Catchment Name	Country	Area (ac)	Paved Area (ac)	Imperviousness (%)	Slope (%)	Depression Storage (in.)	No. of Events	Reference
Lelystad Housing Area	Netherlands	4.94	2.17	44	0.5	0.059	10	Van den Berg, 1978
Leylstad Parking Lot[a]	Netherlands	1.73	1.73	100	0.5	0.035	10	Van den Berg, 1978
Ennerdale Two	U.K.	0.088	0.079	89	3.1	0.020	6	Kidd, 1978a
Ennerdale Three	U.K.	0.022	0.022	100	3.0	0.016	9	Kidd, 1978a
Bishopdale Two	U.K.	0.146	0.111	76	2.4	0.018	11	Kidd, 1978a
Hyde Green One	U.K.	0.120	0.085	71	2.2	0.019	7	Kidd, 1978a
Hyde Green Two	U.K.	0.209	0.103	49	2.0	0.020	8	Kidd, 1978a
School Close One	U.K.	0.113	0.070	62	1.7	0.009	11	Kidd, 1978a
School Close Two	U.K.	0.177	0.097	55	0.9	0.026	11	Kidd, 1978a
Lund 1:75	Sweden	0.072	0.072	100	2.1	0.005	11	Falk and Niemczynowicz, 1978
Klostergarden 1:76	Sweden	0.081	0.081	100	0.9	0.041	11	Falk and Niemczynowicz, 1978
Klostergarden 1:77	Sweden	0.083	0.083	100	2.3	0.020	13	Falk and Niemczynowicz, 1978
Klostergarden 2:76	Sweden	0.020	0.020	100	3.3	0.019	11	Falk and Niemczynowicz, 1978
Klostergarden 2:77	Sweden	0.019	0.019	100	4.1	0.013	12	Falk and Niemczynowicz, 1978
Klostergarden 3:76	Sweden	0.076	0.076	100	3.1	0.022	11	Falk and Niemczynowicz, 1978
Klostergarden 3:77	Sweden	0.102	0.102	100	2.3	0.022	13	Falk and Niemczynowicz, 1978
Klostergarden 4:76	Sweden	0.068	0.068	100	1.6	0.020	10	Falk and Niemczynowicz, 1978
Klostergarden 4:77	Sweden	0.069	0.069	100	1.9	0.022	13	Falk and Niemczynowicz, 1978

[a]55% brick pavement. Other catchments have primarily asphalt pavement

The data were aggregated and a regression of depression storage versus slope performed as part of a workshop (Kidd, 1978b). The data are plotted in Figure 4-18 along with the relationship developed by the workshop,

$$d_p = 0.0303 \cdot S^{-0.49}, \quad (r = -0.85) \tag{4-17}$$

where

d_p	=	WSTORE = depression storage, in., and
S	=	WSLOPE = catchment slope, percent.

Viessman et al. (1977, p. 69) illustrate a similar but linear plot, a portion of which is shown in Figure 4-18, in which depression storage values for "four small impervious areas" near Baltimore, Maryland, range from 0.06 to 0.11 in. (1.5 to 2.8 mm), considerably higher than the European values shown in Figure 4-18. The reason for this discrepancy is not known, but it appears that the recent European data may be better suited to provide depression storage estimates, mainly because of their extent.

Separate values of depression storage for pervious and impervious areas are required for input in data group H1. Representative values for the latter can probably be obtained from the European data just discussed. Pervious area measurements are lacking; most reported values are derived from successful simulation of measured runoff hydrographs. Although pervious area values are expected to exceed those for impervious areas, it must be remembered that the infiltration loss, often included as an initial abstraction in simpler models, is computed explicitly in SWMM. Hence, pervious area depression storage might best be represented as an interception loss, based on the type of surface vegetation. Many interception estimates are available for natural and agricultural areas (Viessman et al., 1977, Linsley et al., 1949). For grassed urban surfaces a value of 0.10 in. (2.5 mm) may be appropriate.

As mentioned earlier, several studies have determined depression storage values in order to achieve successful modeling results. For instance, Hicks (1944) in Los Angeles used values of 0.20, 0.15 and 0.10 in. (5.1, 3.8, 2.5 mm) for sand, loam and clay soils, respectively, in the urban area. Tholin and Keifer (1960) used values of 0.25 and 0.0625 in. (6.4 and 1.6 mm) for pervious and impervious areas, respectively, for their Chicago hydrograph method. Brater (1968) found a value of 0.2 in. (5.1 mm) for three basins in metropolitan Detroit. Miller and Viessman (1972) give an initial abstraction (depression storage) of between 0.10 and 0.15 in. (2.5 and 3.8 mm) for four composite urban catchments.

In SWMM, depression storage may be treated as a calibration parameter, particularly to adjust runoff volumes. If so, extensive preliminary work to obtain an accurate *a priori* value may be pointless since the value will be changed during calibration anyway.

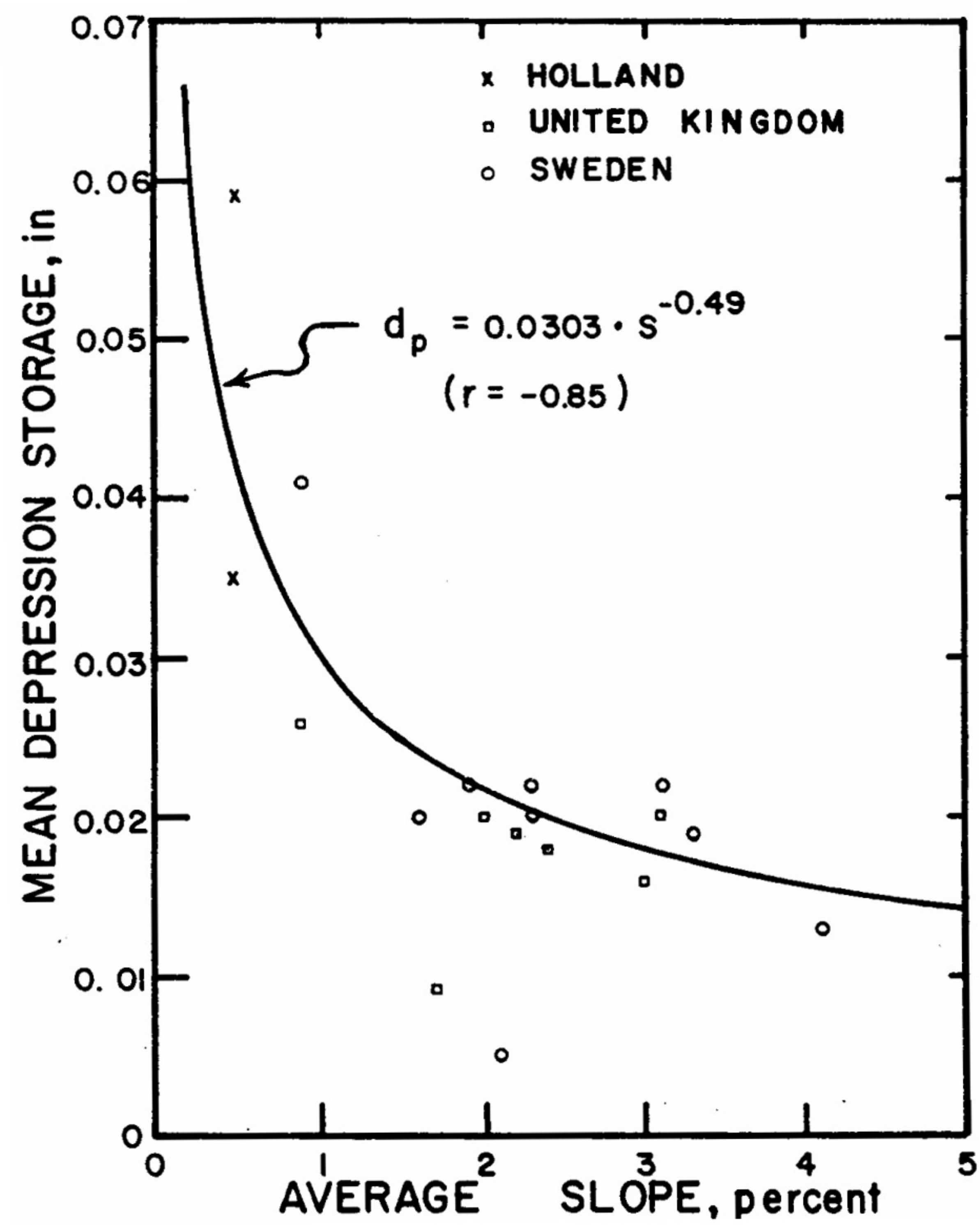


Figure 4-18. Depression storage vs. catchment slope (after Kidd, 1978b). See Table 4-6 for catchment data.

Infiltration*

Options. Infiltration from pervious areas may be computed by either the Horton (1933, 1940) or Green-Ampt (1911) equations described below. A complete description of the theoretical background and programming details for both is given in Appendix V. In SWMM, the method to be used for all subcatchments is determined by the input parameter INFILM (group B1). Parameters required by the two methods are quite different.

Horton Infiltration. Infiltration capacity as a function of time is given by Horton (1933, 1940) as

$$f_p = f_c + (f_o - f_c)\, e^{-kt} \tag{4-18}$$

where

f_p	=	infiltration capacity into soil, ft/sec,
f_c	=	minimum or ultimate value of f_p (WLMIN, ft/sec,
f_o	=	maximum or initial value of f_p (WLMAX), ft/sec,
t	=	time from beginning of storm, sec, and
k	=	decay coefficient (DECAY), sec^{-1}.

This equation describes the familiar exponential decay of infiltration capacity evident during heavy storms. However, the program does not use equation 4-18 directly; rather, the integrated form is used in order to avoid an unwarranted reduction in f_p during periods of light rainfall. Details are given in Appendix V.

Required parameters for data group H1 are f_o (WLMAX), f_c (WLMIN) and k (DECAY). In addition a parameter used to regenerate infiltration capacity (REGEN, group B2) is required for continuous simulation. Although the Horton infiltration equation is probably the best-known of the several infiltration equations available, there is little to help the user select values of parameters f_o and k for a particular application. (Fortunately, some guidance can be found for the value of f_c.) Since the actual values of f_o and k (and often f_m) depend on the soil, vegetation, and initial moisture content, ideally these parameters should be estimated using results from field infiltrometer tests for a number of sites of the watershed and for a number of antecedent wetness conditions. If it is not possible to use field data to find estimates of f_o, f_c, k and for each subcatchment, the following guidelines should be helpful.

The U.S. Soil Conservation Service (SCS) has classified most soils into Hydrologic Soil Groups, A, B, C, and D, dependent on their limiting infiltration capacities, f_c. (Well drained, sandy soils are “A”; poorly drained, clayey soils are “D.”) A listing of the groupings for more than 4000 soil types can be found in the SCS Hydrology Handbook (1972, pp. 7.6-7.26); a similar listing is also given in the *Handbook of Applied Hydrology* (Ogrosky and Mockus, 1964, pp. 21.12-21.25), but the former reference also gives alternative groupings for some soil types depending on the degree of

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*The infiltration section was prepared by Dr. Russell G. Mein, Monash University, Clayton, Victoria, Australia.

drainage of the subsoil. The soil type itself may be found in the U.S. from county SCS Soil Survey maps.

The best source of information about a particular soil type is a publication entitled "Soil Survey Interpretations" available from a local SCS office in the U.S. Information on the soil profile, the soil properties, its suitability for a variety of uses, its erosion and crop yield potential, and other data is included on the sheet provided. A copy of the listing for Conestoga silt loam is shown in Figure 4-19. Parameter f_c is essentially equal to the saturated hydraulic conductivity, K_s, which is called "permeability" on the soil survey interpretation sheet. For Conestoga Silt Loam, a range of 0.63-2.0 in./hr (16-51 mm/hr) is shown.

Alternatively, values for f_c according to Musgrave (1955) are given in Table 4-7. To help select a value within the range given for each soil group, the user should consider the texture of the layer of least hydraulic conductivity in the profile. Depending on whether that layer is sand, loam, or clay, the f_c value should be chosen near the top, middle, and bottom of the range respectively. For example, the data sheet for Conestoga silt loam identifies it as being in Hydrology Group B which puts the estimate of f_c into the range of 0.15-0.30 in./hr (3.8-7.6 mm/hr), much lower than the K_s value discussed above. Examination of the texture of the layers in the soil profile indicates that they are silty in nature, suggesting that the estimate of the f_c value should be in the low end of the range, say 0.15-0.20 in./hr (3.8-5.1 mm/hr). A sensitivity test on the f_c value will indicate the importance of this parameter to the overall result.

Table 4-7. Values of f_c for Hydrologic Soil Groups (Musgrave, 1955)

Hydrologic Soil Group	f_c (in./hr)		
A	0.45	-	0.30
B	0.30	-	0.15
C	0.15	-	0.05
D	0.05	-	0

Caution should be used in applying values from Table 4-7 to sandy soils (group A) since reported K_s values are often much higher. For instance, sandy soils in Florida have K_s values from 7 to 18 in./hr (180-450 mm/hr) (Carlisle et al., 1981). Unless the water table rises to the surface, ultimate infiltration capacity will be very high, and rainfall rates will almost always be less than f_c, leading to little or no overland flow from such soils.

For any field infiltration test the rate of decrease (or "decay") of infiltration capacity, k, from the initial value, f_o, depends on the initial moisture content. Thus the k value determined for the same soil will vary from test to test.

PA-SOILS 5
REVISED 6-71
FILE CODE SOILS 12
CECA CECI
CECB CECJ
CECH CECE
CECC CECD

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
HARRISBURG, PENNSYLVANIA

SOIL SURVEY INTERPRETATIONS

Pennsylvania Date 7/28/71 Subject to Change SOIL: CONESTOGA silt loam

MLRA 148

MAP SYMBOLS: See Pag ... [illegible]

BRIEF DESCRIPTION Deep, well drained upland soils formed from weathered micaceous or shaly limestone and calcareous schist and phylite. They have a silt loam surface layer, a silty clay loam subsoil. Bedrock occurs at about 75 inches.

Hydrology Group B Irrigation Group 1 Drainage Group: WA

ESTIMATED PHYSICAL AND CHEMICAL PROPERTIES

Depth To Bedrock (ft) 5+ Depth To Seasonal High Water Table (ft): 4+ Flood Hazard: None

Depth in Inches From Surface	Classification: USDA Texture	Classification: Unified	Classification: AASHO	Coarse Fraction - Than 3 In	Percent Passing Sieve: No. 4 (4.7 mm)	No. 10 (2.0 mm)	No. 40 (.42 mm)	No. 200 (.074 mm)	Range in Permeability Inches/Hr	Available Moisture Capacity In/in of soil	Reaction pH	Shrink – Swell Potential
0-15	SIL	ML	A-4	-	90-100	90-100	80-100	60-90	0.63-2.0	.16-.20	4.5-6.5	low
15-43	SICL	ML,CL MH,CH	A-4, 5,6 7	0-5	90-100	90-100	80-100	60-90	0.63-2.0	.12-.16	4.5-5.5	low
43-75	SH SIL	GM,ML CL,MH CH	A-2 A-5 A-7	0-15	35-90	35-85	35-85	30-80	0.63-2.0	.06-.10	5.6-7.8	low
75+	- - -	- - -	- - -	- - -	MICACEOUS LIMESTONE			- - -	- - -	- - -	- - -	- - -

SUITABILITY OF SOIL AS A SOURCE OF ---

TOPSOIL	SAND AND GRAVEL	ROADFILL
GOOD to 15 inches	UNSUITABLE	FAIR to POOR; A-4, 5, 6, 7

SOIL FEATURES AFFECTING SPECIFIED ENGINEERING USES

Use	Major Soil Feature Affecting Use
Highway and Road Location	Moderate potential frost action; cuts and fills needed
Ponds-Reservoir Area	Moderate perm.
Ponds-Embankments	Fair to poor stability and compaction; fair to poor resistance to piping
Drainage	NA
Sprinkler Irrigation	Moderate intake rate; moderate perm.; high available moisture capacity
Terraces or Diversions	Fair to poor stability
Grassed Waterways	High available moisture capacity; moderate fertility
Winter Grading	Fair trafficability
Pipeline Construction and Maintenance	All features are favorable

SOIL LIMITATIONS FOR COMMUNITY DEVELOPMENT

Use	Phase	Degree of Limitation	Major Soil Feature Affecting Use
Septic Tank Filter Fields	0-8% 8-15% 15-35%	SLIGHT MODERATE SEVERE	HWC Slope; HWC Slope
Sewage Lagoons	0-3% 3-8% 8-35%	MODERATE MODERATE SEVERE	Moderate perm. Slope; moderate perm. Slope
Low Buildings With Basements	0-8% 8-15% 15-35%	SLIGHT MODERATE SEVERE	 Slope Slope
Lawns and Landscaping	0-8% 8-15% 15-35%	SLIGHT MODERATE SEVERE	 Slope Slope
Parking Lots and Streets in Subdivisions	0-3% 3-8% 8-35%	SLIGHT MODERATE SEVERE	 Slope Slope
Sanitary Land Fills	0-8% 8-15% 15-35%	SLIGHT MODERATE SEVERE	 Slope Slope

Figure 4-19. Soil Conservation Service Soil Survey Interpretation for Conestoga silt loam (found near Lancaster, PA).

MLRA 148 — SOIL: CONESTOGA silt loam

U.S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE, HARRISBURG, PENNSYLVANIA

SOIL LIMITATIONS FOR RECREATIONAL USES

Use	Phase	Degree of Limitation	Major Soil Features Affecting Use
Campsites - Tents	0-6%	SLIGHT	
	6-12%	MODERATE	Slope
	12-35%	SEVERE	Slope
Campsites Trailers	0-3%	SLIGHT	
	3-6%	MODERATE	Slope
	6-35%	SEVERE	Slope
Low Buildings Without Basements	0-6%	SLIGHT	
	6-12%	MODERATE	Slope
	12-35%	SEVERE	Slope
Paths and Trails	0-15%	SLIGHT	
	15-25%	MODERATE	Slope
	25-35%	SEVERE	Slope
Picnic and Play Areas	0-6%	SLIGHT	
	6-12%	MODERATE	Slope
	12-35%	SEVERE	Slope
Athletic Fields	0-3%	SLIGHT	
	3-6%	MODERATE	Slope
	6-35%	SEVERE	Slope
Golf Fairways	0-6%	SLIGHT	(moderate on eroded phase)
	6-12%	MODERATE	Slope (severe on eroded phase)
	12-35%	SEVERE	Slope

LAND CAPABILITY, SOIL LOSS FACTORS, AND ESTIMATED "B" MANAGEMENT YIELDS

Soil Phase	Capability	Soil Loss Factors K	T	T/K	Corn bu.	Oats bu.	Wheat bu.	Soybeans				Alfalfa Hay T.	Clover-Grass Hay T.	Pasture: Blue-Grass c.a.d.	Pasture: Tall Grass-Legume c.a.d.
0-3%	I	.43	4	9.3	135	80	50	45				5.5	3.5	160	315
3-6%	IIe	.43	4	9.3	135	80	50	45				5.5	3.5	160	315
3-6% sev.	IIIe	.43	3	7.0	125	75	45	35				5.0	3.5	160	285
6-12%	IIIe	.43	4	9.3	125	75	45	35				5.0	3.5	160	285
6-12% sev.	IVe	.43	3	7.0	110	65	40	-				4.5	3.0	135	255
12-25%	IVe	.43	4	9.3	110	65	40	-				4.5	3.0	135	125
12-25% sev.	VIe	.43	3	7.0	-	-	-	-				-	-	115	-
25-35%	VIe	.43	4	9.3	-	-	-	-				-	-	115	-

WOODLAND

1-Slight 2-Moderate 3-Severe

Soil Phase	Erosion Hazard	Equip. Restrict.	Seedling Mort.	Plant Compet. C	Plant Compet. H	Wind Throw Hazard	Species To Favor in -- Natural	Species To Favor in -- Plantation	Species and Site Index	Ord. Group
0-6%	1	1	1	3	2	1	RO	YP	Ex. 85+	
6-12%	2	1	1	3	2	1	A	BW	RO	
12-35%	3	2	1	3	2	1	SM	L	A	
							YP	NSp	SM	
							BW	WP	Ex. 95+	
									YP	

WILDLIFE

Soil Phase	Grain and Seed Crops	Grass and Legumes	Wild Herb. Upland Plants	Hardwood Trees, Shrubs, Vines	Coniferous Woody Plants	Wild Herb. Wetland Plants	Shallow Water Dev.	Shallow Excavated Ponds	Openland Wildlife Habitat	Woodland Wildlife Habitat	Wetland Wildlife Habitat
0-3%	1	1	1	1	1	4	4	4	1	1	4
3-6%	2	1	1	1	1	4	4	4	1	1	4
6-12%	2	1	1	1	1	4	4	4	1	1	4
12-25%	3	2	1	1	1	4	4	4	2	1	4
25-35%	4	2	1	1	1	4	4	4	2	1	4

(Wildlife Habitat Elements: columns Grain and Seed Crops through Shallow Excavated Ponds; Kinds of Wildlife Habitat: Openland, Woodland, Wetland)

1 Good 2 Fair 3 Poor 4 Very Poor

Figure 4-19. Continued.

It is postulated here that, if f_o is always specified in relation to a particular soil moisture condition (e.g., dry) and for moisture contents other than this the time scale is changed accordingly (i.e., time “zero” is adjusted to correspond with the constant f_o), then k can be considered a constant for the soil independent of initial moisture content. Put another way, this means that infiltration curves for the same soil, but different antecedent conditions, can be made coincident if they are moved along the time axis. Butler (1957) makes a similar assumption.

Values of k found in the literature (Viessman et al., 1977; Linsley et al., 1975; Overton and Meadows, 1976; Wanielista, 1978) range from 0.67 to 49 hr^{-1}. Nevertheless most of the values cited appear to be in the range 3-6 hr^{-1} (0.00083-0.00167 sec^{-1}). The evidence is not clear as to whether there is any relationship between soil texture and the k value although several published curves seem to indicate a lower value for sandy soils. If no field data are available, an estimate of 0.00115 sec^{-1} (4.14 hr^{-1}) could be used. Use of such an estimate implies that, under ponded conditions, the infiltration capacity will fall 98 percent of the way towards its minimum value in the first hour, a not uncommon observation. Table 4-8 shows the rate of decay of infiltration for several values of k.

Table 4-8. Rate of Decay of Infiltration Capacity for Different Values of k

k value hr^{-1} (sec^{-1})	Percent of decline of infiltration capacity towards limiting value f_c after 1 hour
2 (0.00056)	76
3 (0.00083)	95
4 (0.00115)	98
5 (0.00139)	99

The initial infiltration capacity, f_o depends primarily on soil type, initial moisture content, and surface vegetation conditions. For example, Linsley et al. (1982) present data which show, for a sandy loam soil, a 60 to 70 percent reduction in the f_o value due to wet initial conditions. They also show that lower f_o values apply for a loam soil than for a sandy loam soil. As to the effect of vegetation, Jens and McPherson (1964, pp. 20.20-20.38) list data which show that dense grass vegetation nearly doubles the infiltration capacities measured for bare soil surfaces.

For the assumption to hold that the decay coefficient k is independent of initial moisture content, f_o must be specified for the dry soil condition. The continuous version of SWMM automatically calculates the f_o value applicable for wetter conditions as part of the moisture accounting routine. However, for single-event simulation, the user must specify the f_o value for the storm in question, which may be less than the value for dry soil conditions.

Published values of f_o vary depending on the soil, moisture, and vegetation conditions for the particular test measurement. The f_o values listed in Table 4-9 can be used as a rough guide. Interpolation between the values may be required.

Table 4-9. Representative Values for f_o.

A.	DRY soils (with little or no vegetation):	
	⑨ Sandy soils:	5 in./hr
	⑨ Loam soils:	3 in./hr
	⑨ Clay soils:	1 in./hr
B.	DRY soils (with dense vegetation):	
	⑨ Multiply values given in A by 2 (after Jens and McPherson, 1964)	
C.	MOIST soils (change from dry f_o value required for single event simulation only):	
	⑨ Soils which have drained but not dried out (i.e., field capacity): divide values from A and B by 3	
	⑨ Soils close to saturation: Choose value close to f_c value.	
	⑨ Soils which have partially dried out: divide values from A and B by 1.5-2.5.	

For continuous simulation, infiltration capacity will be regenerated (recovered) during dry weather. SWMM performs this function whenever there are dry time steps -- no precipitation or surface water -- according to the following equation (see Figure V-3, Appendix V).

$$f_p = f_o - \left(f_o - f_c\right)e^{-k_d (t-t_w)} \tag{4-19}$$

where

k_d	=	decay coefficient for the recovery curve, sec^{-1}, and
t_w	=	hypothetical projected time at which $f_p = f_c$ on the recovery curve, sec.

In the absence of better knowledge of k_d, it is taken to be a constant fraction or multiple of k,

$$k_d = R\,k \tag{4-20}$$

where R = constant ratio, probably << 1.0, (implying a “longer” drying curve than wetting curve). The parameter R is represented in the program by REGEN, group (B2).

On well-drained porous soils (e.g., medium to coarse sands), recovery of infiltration capacity is quite rapid and could well be complete in a couple of days. For heavier soils, the recovery rate is likely to be slower, say 7 to 14 days. The choice of the value can also be related to the interval between a heavy storm and wilting of vegetation. The value of k_d is then,

$$k_d = 0.02/D \qquad (4\text{-}21)$$

where

k_d	=	R k = recovery curve decay coefficient, day^{-1}, and
D	=	number of days required for the soil to dry out (recover).

The factor of 0.02 in equation 4-21 assumes 98 percent recovery of infiltration capacity (i.e., $e^{-0.02} = 0.98$). The value of R may then be calculated from equation 4-20. For example, for k = 4.14 day^{-1} and drying times of 3, 7 and 14 days, values of R are 1.61 ❸ 10^{-3}, 6.9 ❸ 10^{-4} and 3.45 ❸ 10^{-4}, respectively.

Green-Ampt Infiltration. The second infiltration option is the Green-Ampt equation (1911) that, although not as well known as the Horton equation, has the advantage of physically based parameters that, in principle, can be predicted a priori. The Mein-Larson (1973) formulation of the Green-Ampt equation is a two-stage model. The first step predicts the volume of water, F_s, which will infiltrate before the surface becomes saturated. From this point onward, infiltration capacity, f_p, is predicted directly by the Green-Ampt equation. Thus,

For $F < F_s$:

$$f = i \text{ and } F_s = \frac{S_u \, IMD}{i/Ks - 1} \text{ for } i > K_s ; \qquad (4\text{-}22)$$

No calculation of F_s for i ⦸ K_s

For F ☚ F_s:

$$f = f_p \text{ and } f_p = K_s \left(1 + \frac{S_u \, IMD}{F} \right) \qquad (4\text{-}23)$$

where

f	=	infiltration rate, ft/sec,
f_p	=	infiltration capacity, ft/sec,
i	=	rainfall intensity, ft/sec,
F	=	cumulative infiltration volume, this event, ft,
F_s	=	cumulative infiltration volume required to cause surface saturation, ft,
S_u	=	average capillary suction at the wetting front (SUCT), ft water,
IMD	=	initial moisture deficit for this event (SMDMAX), ft/ft, and
K_s	=	saturated hydraulic conductivity of soil, (HYDCON) ft/sec.

Infiltration is thus related to the volume of water infiltrated as well as to the moisture conditions in the surface soil zone. Full computational details are given in Appendix V.

Like the Horton equation, the Green-Ampt infiltration equation has three parameters to be specified, S_u (SUCT), K_s (HYDCON) and IMD (SMDMAX). Again, estimates based on any available field data should take precedence over the following guidelines. No default values are provided.

The "Soil Survey Interpretation" sheet (see Figure 4-19) available for most soils from the SCS shows values of "permeability" (hydraulic conductivity) for the soil, K_s. However, these values are taken from data for disturbed samples and tend to be highly variable. For example, for Conestoga silt loam the values range from 0.63 to 2.0 in./hr (16 to 51 mm/hr). A better guide for the K_s values is as given for parameter f_c for the Horton equation; theoretically these parameters (i.e., f_c and K_s) should be equal for the same soil. Note that, in general, the range of K_s values encountered will be of the order of a few tenths of an inch per hour.

The moisture deficit, IMD, is defined as the fraction difference between soil porosity and actual moisture content. Sandy soils tend to have lower porosities than clay soils, but drain to lower moisture contents between storms because the water is not held so strongly in the soil pores. Consequently, values of IMD for dry antecedent conditions tend to be higher for sandy soils than for clay soils. This parameter is the most sensitive of the three parameters for estimates of runoff from pervious areas (Brakensiek and Onstad, 1977); hence, some care should be taken in determining the best IMD value to use. Table 4-10, derived from Clapp and Hornberger (1973), gives typical values of IMD for various soil types.

Table 4-10. Typical Values of IMD (SMDMAX) for Various Soil Types

Soil Texture	Typical IMD at Soil Wilting Point
Sand	0.34
Sandy Loam	0.33
Silt Loam	0.32
Loam	0.31
Sandy Clay Loam	0.26
Clay Loam	0.24
Clay	0.21

These IMD values would be suitable for input to continuous SWMM; the soil type selected should correspond to the surface layer for the particular subcatchment. For single event SWMM the values of Table 4-10 would apply only to very dry antecedent conditions. For moist or wet antecedent conditions lower values of IMD should be used. When estimating the particular value it should be borne in mind that sandy soils drain more quickly than clayey soils, i.e., for the same time since the previous event, the IMD value for a sandy soil will be closer in value to that of Table 4-10 than it would be for a clayey soil.

The average capillary suction, S_u, is perhaps the most difficult parameter to measure. It can be derived from soil moisture - conductivity data (Mein and Larsen, 1973) but such data are rare for most soils. Chu (1978) gives average values of the product of S_u◇IMD for a range of soils, but these are not based on measurements. Fortunately the results obtained are not highly sensitive to the estimate of S_u (Brakensiek and Onstad, 1977). The approximate values which follow result from a survey of the literature (Mein and Larsen, 1973; Brakensiek and Onstad, 1977; Clapp and Hornberger, 1978; Chu, 1978). Published values vary considerably and conflict; however, a range of 2 to 15 in. (50 to 380 mm) covers virtually all soil textures. Table 4-11 summarizes the published values. An excellent local data source can often be found in Soil Science departments at state universities. Tests are run on a variety of soils found within the state, including soil moisture versus soil tension data, with which to derive S_u. For example, Carlisle et al. (1981) provide such data for Florida soils along with information on K_s, bulk density and other physical and chemical properties.

Table 4-11. Typical Values of S_u (SUCT) for Various Soil Types

Soil Texture	Typical Values for S_u (inches)
Sand	4
Sandy Loam	8
Silt Loam	12
Loam	8
Clay Loam	10
Clay	7

Generalized Green-Ampt parameters for U.S. locations are tabulated by Rawls et al. (1983). It is very difficult to give satisfactory estimates of infiltration equation parameters that will apply to all soils encountered. Which ever infiltration equation is used, the user should be prepared to adjust preliminary estimates in the light of any available data such as infiltrometer tests, measurements of runoff volume, or local experience.

Subcatchment Aggregation and Lumping

As discussed earlier, it is desirable to represent the total catchment by as few subcatchments as possible, consistent with the needs for hydraulic detail within the catchment. That is, if the only interest is in hydrographs and pollutographs at the catchment outlet, as is likely for continuous simulation, then one subcatchment should suffice for the simulation (although up to 30 can be used for continuous simulation). For a single event, detailed simulation, the number of subcatchments needed is a function of the amount of hydraulic detail (e.g., backwater, surcharging, routing, storage) that must be modeled. In addition, enough detail must be simulated to allow non-point source controls to be evaluated (e.g., detention, street sweeping). Finally multiple subcatchments are the only means by which a moving (kinematic) storm may be simulated. Coupled spatial and temporal variations in rainfall can significantly alter predicted hydrographs (Yen and Chow, 1968; Surkan, 1974; James and Drake, 1980; James and Shtifter, 1981).

Clearly, the required volume of input data (and personal time) decreases as the number of subcatchments decreases. How then, can subcatchments be aggregated or "lumped" to provide hydrographs and pollutographs that are equivalent to more detailed simulations?

The most complete study of this question is contained in the Canadian SWMM report (Proctor and Redfern and J.F. MacLaren, 1976a) in which the effect of lumping is compared on real and hypothetical catchments. Similar work has been performed independently by Smith (1975). In both studies it is shown that a single equivalent lumped catchment can be formulated by proper adjustment of the subcatchment width.

In SWMM, Runoff and Transport simulation of the drainage network (i.e., conduits and channels) adds storage to the system and thus attenuates and somewhat delays the hydrograph peaks. When the drainage network is removed from the simulation, subcatchment runoff feeds "instantaneously" into inlets, with consequent higher and earlier hydrograph peaks. The key to aggregation of subcatchments is thus the replacement of the lost storage. This is best accomplished through variation of the subcatchment width, although the same effect could be achieved through variation of the slope or roughness (see discussion of equation 4-9). However, it is assumed that reasonable average values of the latter two parameters for the total catchment may be obtained by weighing individual subcatchment values by their respective areas. (For the roughness an area-weighted harmonic mean may be used, although it is probably an unnecessary refinement.) Hence, the subcatchment width is a more logical parameter to be adjusted.

It was shown in the discussion of the subcatchment width, that reducing its value increases storage on the subcatchment. Hence, as subcatchments are aggregated and drainage network storage lost, the total catchments width, i.e., the sum of the subcatchment widths, must be reduced accordingly. This may be seen in Figure 4-20 for a very schematized drainage network in which the subcatchment widths are nominally twice the length of the drainage conduits (Smith, 1975). The lumped catchment could be represented by a single subcatchment, as in the bottom sketch of Figure 4-20, in which the width is approximately twice the length of the main drainage channel. Experience indicates (Smith, 1975; Proctor and Redfern and J.F. MacLaren, 1976a) that good results can be obtained with <u>no</u> channel/conduit network. However, the Canadian study (Proctor and Redfern and J.F. MacLaren, 1976a) does illustrate the routing effect of an "equivalent pipe" in the Transport Block. Note that if the storm duration is long compared to the catchment time of concentration, and if the rainfall intensity is constant, the peak flows obtained for either a lumped or detailed simulation will be about the same, since equilibrium outflow must ultimately result (see the discussion of Figure 4-15).

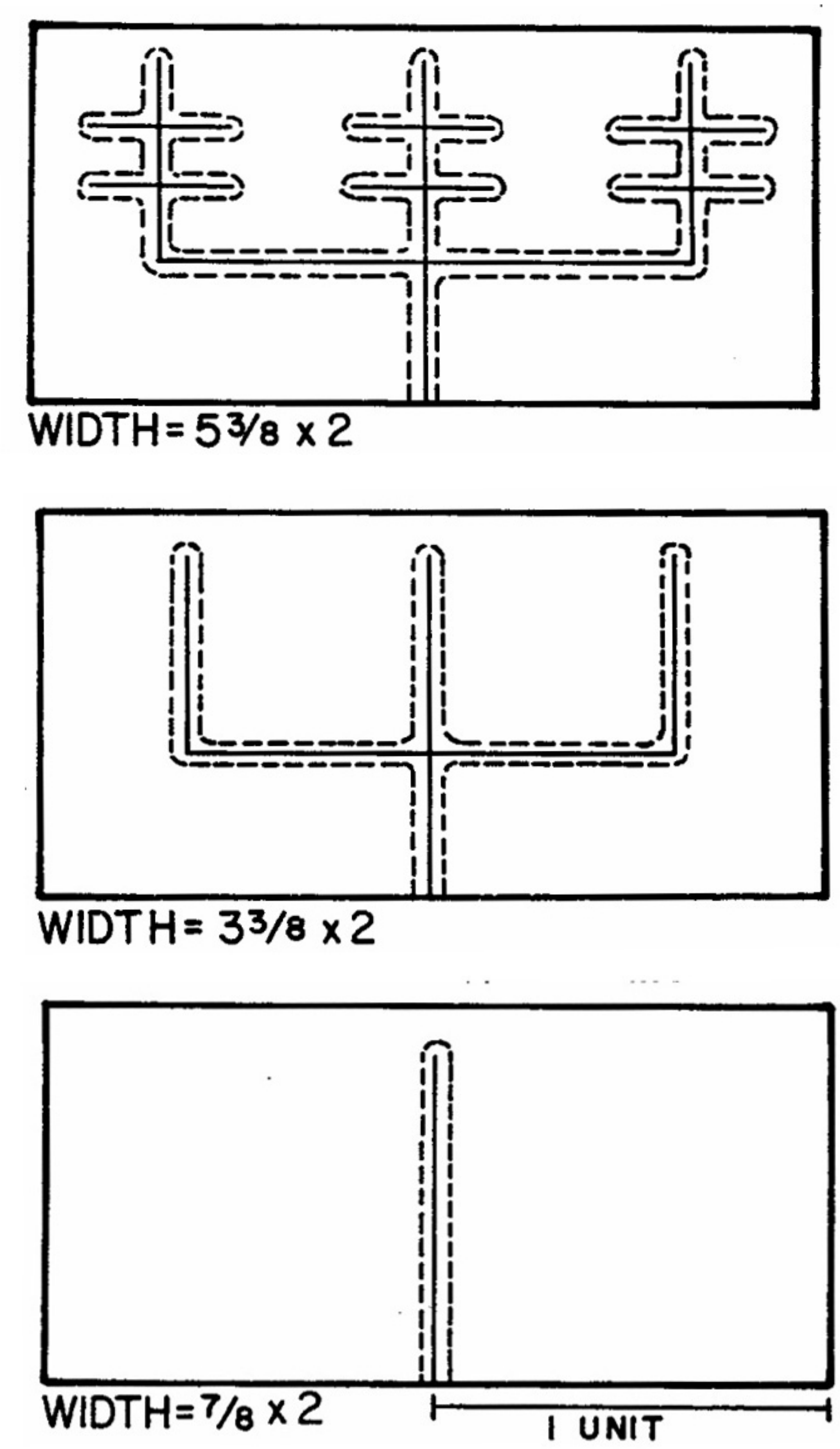

Figure 4-20. Effect of changing the level of discretization on the width of overland flow (after Smith, 1975, p. 57).

Several examples of lumping using real rainfall data on real catchments are shown by Proctor and Redfern and James F. MacLaren (1976a) and Smith (1975). An instructive example for the 2330 ac (943 ha) West Toronto area is taken from the former reference and shown in Figure 4-21. A Runoff-Transport simulation using 45 subcatchments and including the drainage network is compared with three Runoff-only simulations with no drainage network. The best agreement, in terms of matching of peak flows, between the detailed and lumped simulations occurred for a single subcatchment width of 60,000 ft (18,000 m) which is about 1.7 times the length of the main trunk conduit in the actual system. Even if a factor of two had been used (i.e., a width of 70,600 ft or 21,500 m) as a first guess, agreement would not be bad. The timing of the peaks for the single subcatchment representation is somewhat early, but adequate for most purposes. Recall that it is difficult to change the timing of subcatchment hydrograph peaks by changing only the width.

It is assumed that when subcatchments are aggregated, other parameters required in group H1 are simply areally weighted. When this is done, very little difference in runoff volume occurs between the aggregate and detailed representations. Differences that do result are usually from water that remains in storage and has not yet drained off of the lumped catchment, or from very slightly increased infiltration on the lumped catchment, again due to the longer presence of standing water on pervious areas (because of the reduced width).

To summarize, many subcatchments may be aggregated into a single lumped or equivalent subcatchment by using areally weighted subcatchment parameters and by adjustment of the subcatchment width. The lumped subcatchment width should be approximately twice the length of the main drainage channel (e.g., the trunk sewer) through the catchment in order to match hydrograph peaks. The effect on runoff volume should be minimal.

Runoff quality predictions are affected by aggregation of subcatchments to the extent that hydrographs and surface loadings are changed. When areal weighted averages of the latter are used for a lumped catchment, total storm loads are essentially the same as for a detailed simulation. Pollutographs of concentration versus time then vary only because of hydrograph variations.

Subsurface Flow Routing (Data Groups H2-H4)

Introduction

Routing of flows only (no quality) is performed by Subroutine GROUND and other subroutines. Full details are provided in Appendix X, but briefly, infiltration by either the Horton or Green-Ampt methods may be routed through an unsaturated zone lumped storage, followed by routing through a saturated zone lumped storage. Outflow may occur from the saturated zone to channel/pipes or may be “lost” (from the simulation) to deep groundwater. Evapotranspiration (ET) from both the upper and lower zone may also be simulated, and the groundwater table is dynamic: if it rises to the surface, the upper zone disappears and infiltration will be stopped; if it drops below the elevation of the bottom of the effluent channel/pipe, groundwater outflow will cease. The processes are illustrated schematically in Appendix X, Figure X-1. If quality simulation is included, any water routed through the subsurface zones will be “clean” and act to dilute concentrations in downstream channel/pipes.

Data needs closely reflect soil properties. That is, data for subsurface flow routing involves parameters such as porosity, field capacity, hydraulic conductivity, water table elevation, etc. These data must be obtained from SCS or other sources. As mentioned previously in regard to infiltration parameters, state university Soil Science departments can often provide such information, e.g., Carlisle et al. (1981).

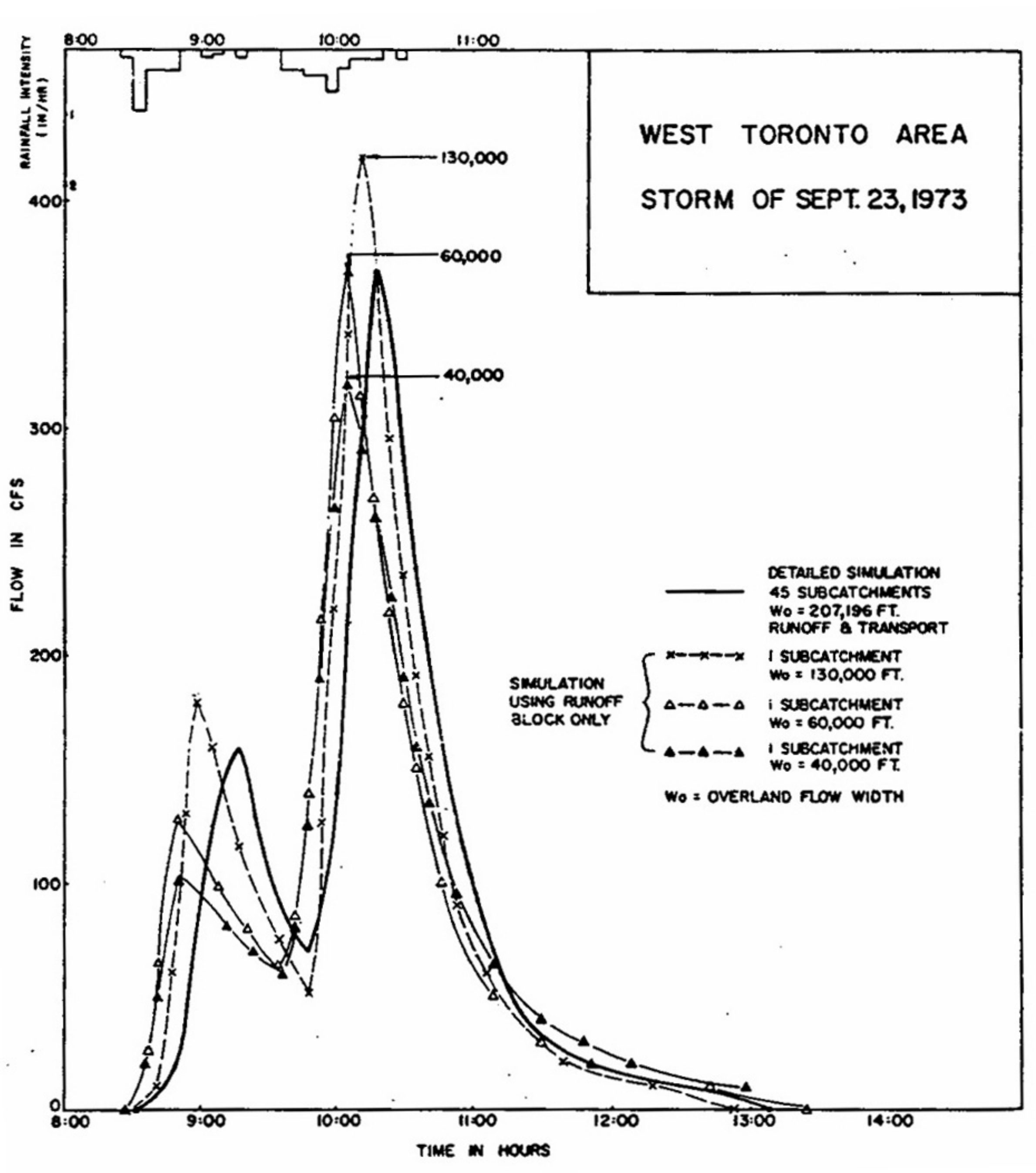

Figure 4-21. Effect on hydrographs of changing subcatchment width for West Toronto area (after Proctor and Redfern and J.F. MacLaren, 1976a, p. 216).

Subsurface routing for an individual subcatchment is indicated by the presence of an H2, H3 and H4 data group immediately following an H1 data group. Subsurface routing for up to any 100 subcatchments (herein called “subsurface subcatchments") can be simulated. The subcatchment number NMSUB on data line H2 must match the subcatchment number NAMEW on data line H1 immediately preceding it. Data groups H2-H4 for subsurface subcatchments may be interspersed among the surface subcatchments; not all surface subcatchments are required to have a corresponding subsurface subcatchment.

Groundwater outflow may be routed to a channel/pipe; if this option is used, the channel/pipe number is denoted by NGWGW. If no channel/pipe number is indicated, groundwater outflow will be “lost” from the simulation, although it will be accounted for in the continuity check. The channel/pipe number does not have to be the same as for the surface runoff.

General Input and Elevations (Data Group H2)

The variables ISFPF and ISFGF are flags that tell the program to save the data from that particular subsurface subcatchment for printouts and graphs, respectively. If ISFPF is 1, then NSCRAT(5) must be defined (greater than zero in the Executive Block); if ISFGF is 1, then NSCRAT(6) must be defined. An undefined scratch file will result in an error message followed by termination of execution.

Elevation variables (BELEV, GRELEV, STG, BC, and TW -- Figure X-1) can be referenced to some known benchmark, such as mean sea level, or they can be referenced to the bottom of the lower zone by setting BELEV equal to zero. The option of referencing the elevation variables to some known benchmark was added so that, among other reasons, the user could easily compare his or her predicted stage data to measured stage data.

Groundwater Flow and Soil Parameters (Data Group H3)

Groundwater outflow parameters A1, A2, A3, B1 and B2 are defined by equations X-24 and X-25. Because of the general nature of the two equations, a variety of functional forms can be approximated. For example, a linear reservoir can be selected by setting B1 equal to one and A2 and A3 equal to zero. The Dupuit approximation can be selected, within its usual limitations (Todd, 1980), as illustrated in the example in Appendix X.

Two methods can be used to simulate the effect of channel/pipe tailwater elevations on groundwater outflow. The first method involves setting TW greater than BC (data group H2) and A2, B2 greater than zero (data group H3). It is also desirable to route the flow to a channel or pipe (otherwise, water will be lost from the simulation but accounted for in the continuity check), but this is not absolutely necessary. When this method is chosen, the user is actually supplying the average channel flow influence over the entire run; that is, parameter TW in equation X-25 will be constant for all time steps. This method is most applicable when the depth of the water in the channel is thought to remain fairly constant for the length of the run.

For the second method, TW must be less than zero, which makes it simply an indicator parameter. In addition, A2 and B2 must be greater than zero, and the groundwater flow must be routed to a previously defined trapezoidal channel or circular pipe. If this method is chosen, the program will use the elevation of the water surface in the channel or pipe at the end of the previous time step as the current time-step value for the variable TW (elevation of water surface in channel or pipe). The invert elevation of the channel/pipe is assumed to equal BC. Because of the fact that flow routing in the subsurface zone is not coupled with the channel/pipe routing, oscillations can occur in the groundwater flow as elevation D1 hovers near the variable tailwater elevation TW. This can usually be cured by reducing the simulation time step, WET or WETDRY. See the discussion in Appendix X.

Under-drains can be simulated as shown in the example in Appendix X. However, since groundwater flow from each subsurface subcatchment can only be routed to one pipe, a network of under-drain pipes must be replaced by one equivalent pipe for simulation purposes.

One very important rule to remember, regardless of the functional form chosen, is that groundwater flow should never be allowed to be negative. Although negative flow may be true for the actual system (i.e., bank recharge), it should not be allowed to happen in the model because there is currently no means of subtracting flow from the channel (since the channel flow routing is not coupled to the groundwater flow routing). An easy way to assure that groundwater flow remains positive is by making A1 greater than or equal to A2 and B1 greater than or equal to B2.

Saturated hydraulic conductivity, porosity, wilting point, and field capacity (HKSAT, POR, WP, and FC) are all measurable but difficult-to-obtain values. A discussion regarding saturated hydraulic conductivity was presented previously for the infiltration parameters (parameters HKSAT on data group H3 and HYDCON on data group H1 are treated separately in the program and need not be the same number). WP and FC are usually related to specific suction pressures. Table X-1 contains typical values for wilting point and field capacity (Linsley et al., 1982). SCS and university sources, especially agricultural extension offices in the U.S., often have these data.

For purposes of these groundwater routines, actual porosity and apparent porosity are considered to be equal, since no mechanism exists for adjusting for entrapped air and the difference is usually minor. Porosity is critical to this formulation because of its role in determining moisture storage. WP and FC are less important because they act only as threshold values at which processes change.

Percolation and Evapotranspiration Parameters (Data Group H3)

Water “percolates” from the unsaturated upper zone to the saturated lower zone. Parameters HCO and PCO are defined by equations X-21 and X-22, respectively. HCO can be estimated from an exponential fit of hydraulic conductivity to soil moisture, assuming such data are available. See Figure X-5 for example fits. Fitted or not, HCO is a sensitive calibration parameter for movement of unsaturated zone water into the saturated zone. PCO is the slope, PSI/TH, of the soil tension versus moisture content curve. An average value can be used from data of the form of Figures X-2, X-3 and X-4. It can also be used for calibration, although it is likely that a better estimate of PCO can be obtained than for HCO.

The model includes a deep percolation term which is intended to account for losses through a confining layer, if they could be quantified. Parameter DP is defined by equation X-23. The functional form provides for a first order decay, typical of water table recession curves. Because of the uncertainty associated with this term, it is reasonable to use it for other saturated zone losses that can be quantified by calibration but less than adequately explained physically.

As explained in Appendix X, potential evaporation available for subsurface water loss is the difference between monthly (or other time interval) evaporation input to the Runoff Block (data group F1) and evaporation used by the surface routing. Upper zone ET is a fraction CET of this difference, by equation X-9. Lower zone ET removes the remaining fraction linearly as a function of depth to the water table according to equation X-12. Parameter DET is the maximum depth to the water table for which ET can occur. Subsurface ET can be "turned off" by setting CET = 0 and DET = 0.

Calibration

Example runs are shown in Appendix X. Calibration is aided by examination of the output times series of water table elevation, soil moisture content and groundwater outflow hydrograph. These series are tabulated if parameter ISFPF = 1 in data group H2 and saved for plotting if parameter ISFGF = 1. Groundwater outflow can be routed to any channel/pipe, not necessarily the same one that receives the surface runoff for the subcatchment. In this way, the surface and subsurface flows can be routed separately, if desired.

Snowmelt (Data Groups I1-I3)

Overview of Procedures

SWMM snowmelt routines are based on earlier work done on the Canadian SWMM study by Proctor and Redfern and James F. MacLaren (1976a, 1976b, 1977). Since snowmelt computations are explained in detail in Appendix II, only an outline is given here. Most techniques are drawn from Anderson's (1973) work for the National Weather Service (NWS). For continuous simulation, daily max-min temperatures from the NWS (see Section 11) are converted to hourly values to sinusoidal interpolation, as explained earlier.

Urban snow removal practices may be simulated through "redistribution fractions" input for each subcatchment (discussed below), through alteration of the melt coefficients and base temperatures for the regions of each subcatchment, and through the areal depletion curves used for continuous simulation. Anderson's temperature-index and heat balance melt equations are used for melt computations during dry and rainy periods, respectively. For continuous simulation, the "cold content" of the pack is maintained in order to "ripen" the snow before melting. Routing of melt water through the snow pack is performed as a simple reservoir routing procedure, as in the Canadian study.

The presence of a snow pack is assumed to have no effect on overland flow processes beneath it. Melt is routed in the same manner as rainfall.

Subcatchment Schematization

When snowmelt is simulated, a fourth subarea is added to each subcatchment as illustrated in Figure 4-11. The properties of each subarea are described in Table 4-4. The main purpose of the fourth subarea is to permit part of the impervious area (subarea A4) to be continuously snow covered (e.g., due to windrowing or dumping) and part (subareas A1 plus A3) to be "normally bare" (e.g., streets and sidewalks that are plowed). However, during continuous simulation, the normally bare portion can also have snow cover up to an amount WEPLOW (group I2) inches water equivalent (in. w.e.). (All snow depths and calculations are in terms of the equivalent depth of liquid water.) The snow covered and normally bare impervious areas are determined from fraction SNN1 (group I1). During single event simulation, subarea A4 retains 100 percent snow cover until it has all melted. During continuous simulation, an areal depletion curve, discussed earlier, is used.

Similarly, for single event simulation, a fraction SNN2 (group I1) of the pervious area remains 100 percent snow covered. During continuous simulation, the whole pervious area is subject to areal depletion curve.

Initialization

Initial snow depths (inches water equivalent) may be entered using parameters SNN3, SNN4 (group I1) and SNN7 (group I2). This is likely to be the only source of snow for a single event simulation although snowfall values may be entered as negative precipitation in data group E2. During continuous simulation, the effect of initial conditions will die out, given a simulation of a few months.

No liquid runoff will leave the snow pack until its free water holding capacity (due to its porosity) has been exceeded. The available volume is a constant fraction, FWFRAC (group C1) of the snow depth, WSNOW. Hence, initial values of free water, FW, should maintain the inequality:

$$FW \le FWFRAC \cdot WSNOW \tag{4-24}$$

Melt Equations

During periods of no rainfall, snowmelt is computed by a degree-day or temperature index equation:

$$SMELT = DHM \cdot (TA - TBASE) \tag{4-25}$$

where

SMELT	=	snowmelt rate, in. w.e./hr,
DHM	=	melt coefficient, in w.e./hr-°F,
TA	=	air temperature, °F, and
TBASE	=	snowmelt base temperature, °F.

There is no melt when TA ≤ TBASE. For single event simulation, the melt coefficient, DHM, remains constant. For continuous simulation it is allowed to vary sinusoidally from a minimum value on December 21 to a maximum value on June 21 (see Figure 4-22) in order to reflect seasonal changes.

Melt coefficients and base melt temperatures may be determined both theoretically and experimentally. Considering the former, it is possible to first write a snowmelt equation from a heat budget formulation that includes all relevant terms: change in snow pack heat storage, net short wave radiation entering pack, conduction of heat to the pack from underlying ground, net (incoming minus outgoing) longwave radiation entering pack, convective transport of sensible heat from air to pack, release of latent heat of vaporization by condensation of atmospheric water vapor, and advection of heat to snow pack by rain. (It is assumed here that the pack is "ripe", i.e., just at the melting point, so that rain will not freeze and release its latent heat of fusion.) The equation may then be linearized about a reference air temperature and reduced to the form of equation 4-22. Exactly this procedure is followed in a detailed example presented in Appendix III.

Alternatively, observed melt, in inches per time interval, may be plotted against temperature for that time interval, and a linear relationship developed of the form of equation 4-22. An often-cited such development for natural areas is illustrated in Figure 4-23 taken from the Corps of

Engineers (1956). Viessman et al. (1977) also present a good discussion of degree-day equations.
In the highly

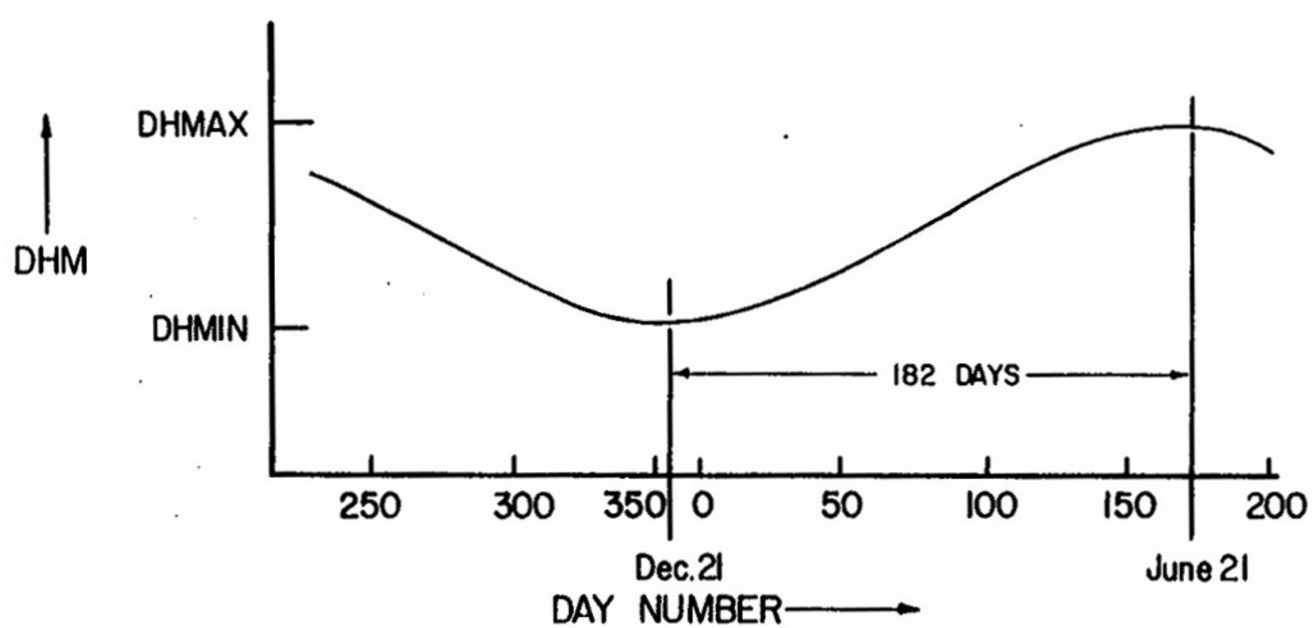

Figure 4.22. Seasonal variation of melt coefficients for continuous simulation.

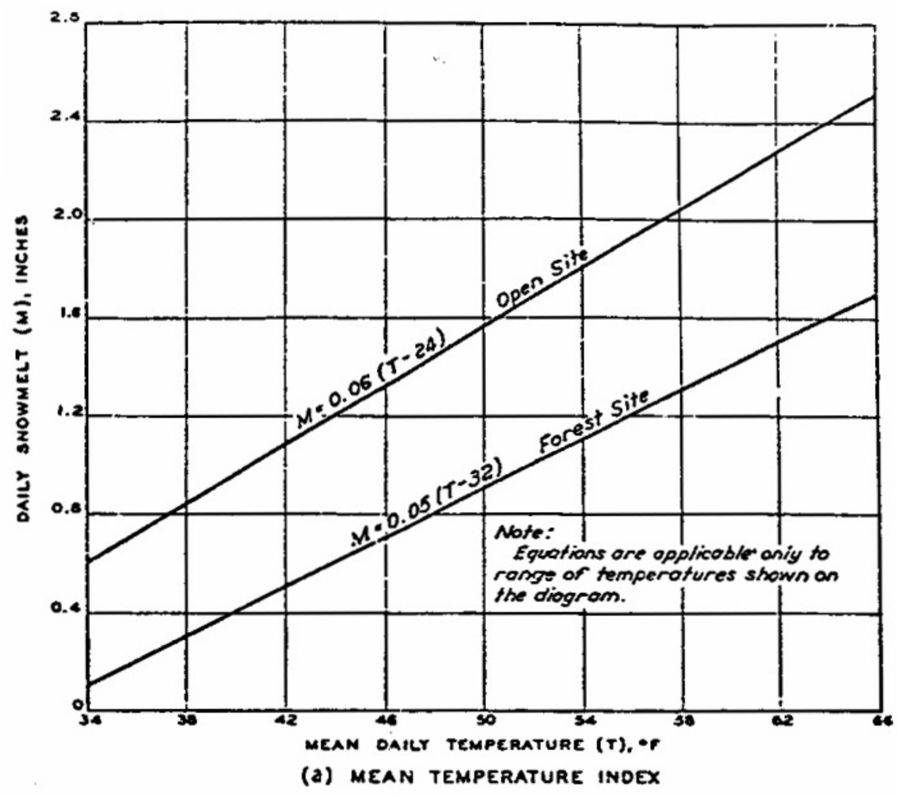

Figure 4-23. Degree-day equations for snow melt (after Corps of Engineers, 1956, plate 6-4).

desirable but unlikely event that snowmelt data are available, the experimental procedure of Figure 4-23 is probably best for urban areas due to the considerable variation of snow pack and meteorological conditions that will be encountered, making reasonable theoretical assumptions more difficult.

For natural areas, considerable range in melt coefficients exits, on the order of 0.0006 to 0.008 in/hr-EF (0.03 to 0.4 mm/hr-°C). Although base melt temperatures are nominally near the freezing point (i.e., 32�F or 0�C) they may be considerably lower depending on the exposure of the site and meteorological conditions. For instance, for the linearization performed in Appendix III a base melt temperature of 9�F (-13�C) was computed, which is valid only over the range of air temperatures used in the linearization (approximately 30 to 40�F or -1 to 5�C).

If the effects of snow removal practices (e.g., street salting) and land surface factors are known, different melt coefficients and base melt temperatures may be entered for the different snow covered subareas of a subcatchment. For instance, street salting lowers the freezing point in proportion to the concentration of the chemical. Handbook values (Chemical Rubber Co., 1976, pp. D218-D267) for freezing point depression are plotted versus concentration in Figure 4-24 for several common roadway salting chemicals. Thus, the base melt temperature computed for pure water might be lowered by an amount taken from Figure 4-24 if an idea is know about the likely concentration on the roadway. The concentration will depend upon the amount of chemical applied and the amount of snowfall and might not be easily computed. An interesting alternative would be to let SWMM predict it!

During periods (i.e., time steps) with rainfall, good assumptions can be made about relevant meteorological parameters for the complete heat balance melt equation. It then replaces the degree-day equation for “wet” time steps. Melt during these time steps is linearly proportional to air temperature and wind speed.

Areal Depletion Parameters

In the earlier discussion of areal depletion curves it was noted that there would be 100 percent cover above a depth of SI inches water equivalent. Values of SI for impervious and pervious areas are read in group I2.

For natural areas, Anderson (1973) recommends that a distinction be made on the basis of areal homogeneity. For a very heterogeneous area there are likely to be areas that receive little snow, or else it will quickly melt. The value of SI for such areas might be about the maximum depth anticipated. For homogeneous areas a much lower value would be appropriate.

No specific information is available for urban areas; however, they are likely to be quite heterogeneous, especially if large, aggregated subcatchments are being used for the continuous simulation. Hence, a high value is probably indicated. Whichever values are used, they should be consistent with the form of the areal depletion curves entered in groups C3 and C4. In general (depending somewhat on the areal depletion curve), the higher the value of SI, the more “stacked up” on a catchment is the snow, and snowmelt will occur at a lower rate over a longer time.

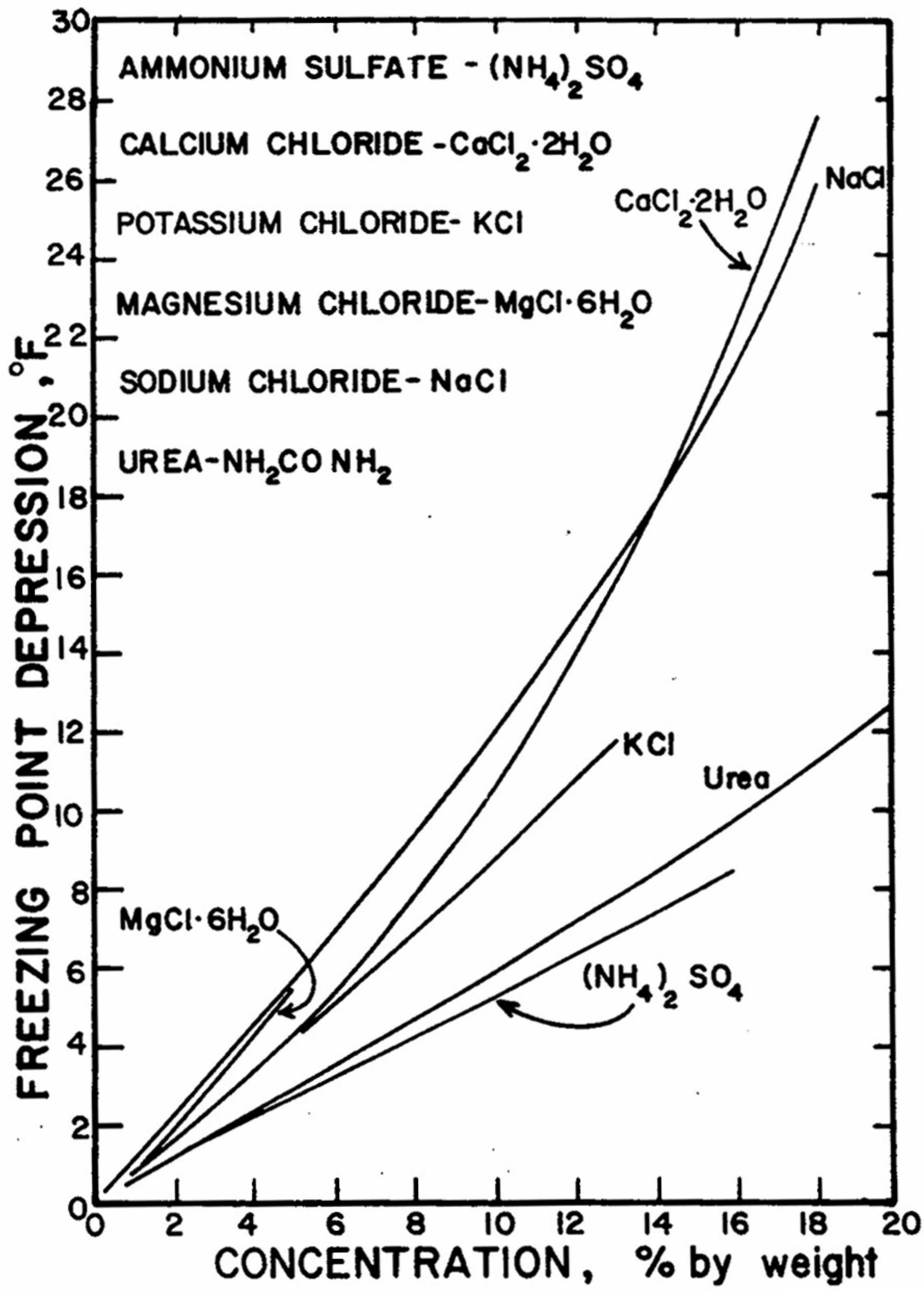

Figure 4-24. Freezing point depression versus roadway salting chemical concentration. Compiled from data from CRC (1976).

Snow Redistribution

The program allows (during continuous simulation) snow that falls on the normally bare impervious areas to be redistributed according to the fractions given as SFRAC in group I2. This is intended to simulate plowing and other snow removal practices in urban areas. Snow depths above WEPLOW inches water equivalent are thus redistributed according to Figure 4-25.

The value of WEPLOW depends upon the level of service given the particular impervious area. That is, at what snow depth do removal practices start? Some guidelines are provided by Richardson et al. (1974) in Table 4-12.

The five fractions SFRAC, should sum to 1.0 and are defined on the basis of the ultimate fate of the removed snow. For instance, if snow is plowed from a street onto an adjacent impervious or pervious area, fractions SFRAC(1) or SFRAC(2) would be appropriate. It may also be transferred to the last subcatchment (e.g., a dumping ground) or removed from the simulation (i.e., removed from the total catchment) altogether. Finally, it may be converted to immediate melt. Should variations in snow removal practices need to be simulated, different subcatchments can be established for different purposes and the fractions varied accordingly.

Surface Quality Data (Groups J1-L1)

Preface to Quality Simulation

Simulation of urban runoff quality is a very inexact science if it can even be called such. Very large uncertainties arise both in the representation of the physical, chemical and biological processes and in the acquisition of data and parameters for model algorithms. For instance, subsequent sections will discuss the concept of “buildup” of pollutants on land surfaces and “washoff” during storm events. The true mechanisms of buildup involve factors such as wind, traffic, atmospheric fallout, land surface activities, erosion, street cleaning and other imponderables. Although efforts have been made to include such factors in physically-based equations (James and Boregowda, 1985), it is unrealistic to assume that they can be represented with enough accuracy to determine a priori the amount of pollutants on the surface at the beginning of the storm. Equally naive is the idea that empirical washoff equations truly represent the complex hydrodynamic (and chemical and biological)
processes that occur while overland flow moves in random patterns over the land surface. The many difficulties of simulation of urban runoff quality are discussed by Huber (1985, 1986).

Such uncertainties can be dealt with in two ways. The first option is to collect enough calibration and verification data to be able to calibrate the model equations used for quality simulation. Given sufficient data, the equations used in SWMM can usually be manipulated to reproduce measured concentrations and loads. This is essentially the option discussed at length in the following sections. The second option is to abandon the notion of detailed quality simulation altogether and either use a constant concentration applied to quantity predictions (i.e., obtain storm loads by multiplying predicted volumes by an assumed concentration) (Johansen et al., 1984) or use a statistical method (Hydroscience, 1979; Driscoll and Assoc., 1981; EPA, 1983b; DiToro, 1984). Two ways in which constant concentrations can be simulated in SWMM are by using a rating curve (equation 4-41) with an exponent of 1.0 or by assigning a concentration to rainfall. Statistical methods are based in part upon strong evidence that storm event mean concentrations (EMCs) are lognormally distributed (Driscoll, 1986). The statistical methods recognize the frustrations of physically-based modeling and move directly to a stochastic result (e.g., a frequency distribution of

EMCs), but they are even more dependent on available data than methods such as those found in SWMM. That is, statistical parameters such as mean, median and variance must be available from

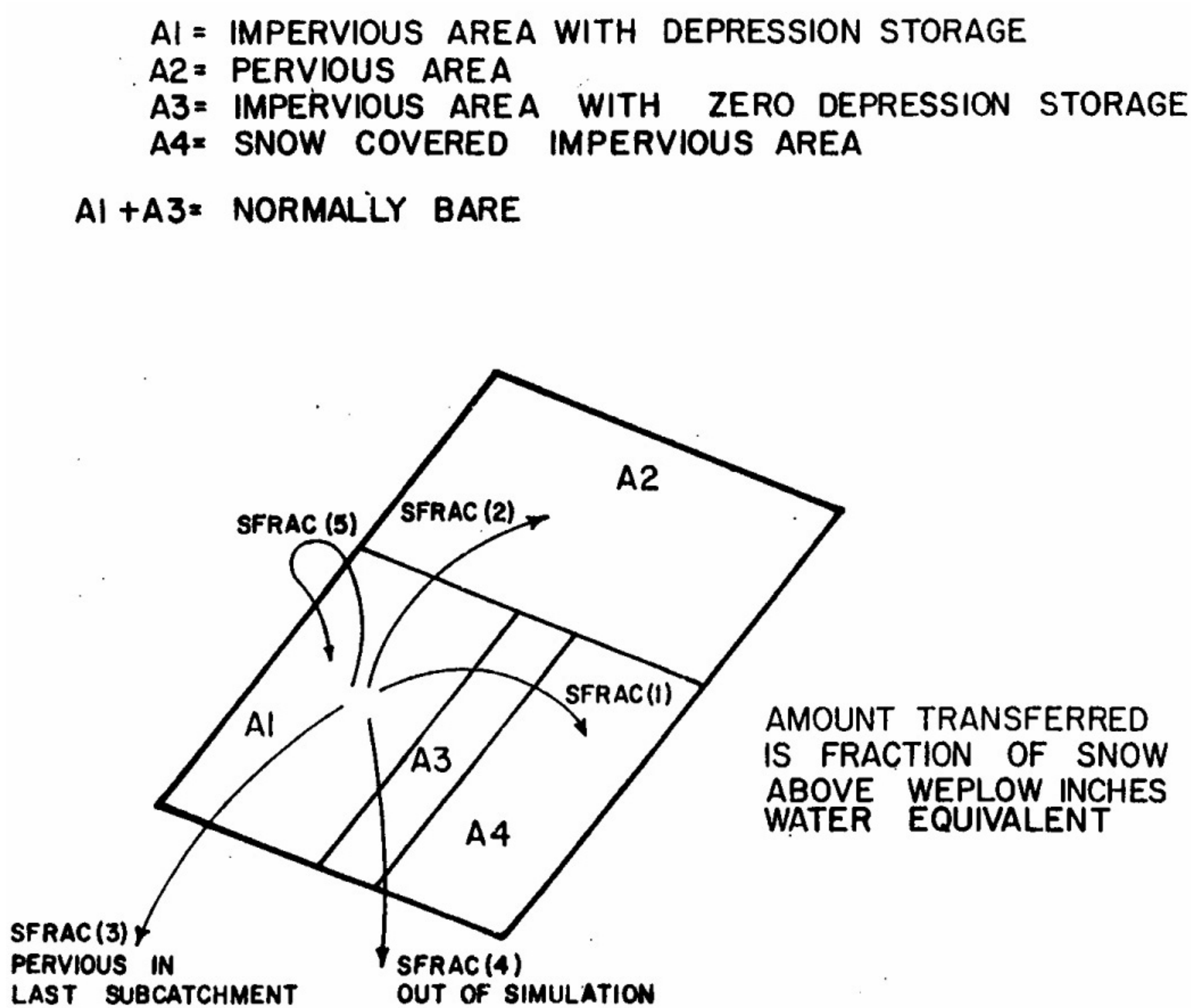

Figure 4-25. Illustration of snow redistribution fractions.

Table 4-12. Guidelines for Levels of Service in Snow and Ice Control (Richardson et al., 1974)

Road Classification	Level of Service	Snow Depth to Start Plowing (inches)	Maximum Snow Depth on Pavement (inches)	Full Pavement Clear of Snow After Storm (hours)	Full Pavement Clear of Ice After Storm (hours)
1. Low-Speed Multilane Urban Expressway	• Roadway routinely patrolled during storms • All traffic lanes treated with chemicals • All lanes (including breakdown lanes) operable at all times but at reduced speeds • Occasional patches of well-sanded snow pack • Roadway repeatedly cleared by echelons of plows to minimize traffic disruption • Clear pavement obtained as soon as possible	0.5 to 1	1	1	12
2. High-Speed 4-Lane Divided Highways Interstate System ADT greater than 10,000	• Roadway routinely patrolled during storms • Driving and passing lanes treated with chemicals • Driving lane operable at all times at reduced speeds • Passing lane operable depending on equipment availability • Clear pavement obtained as soon as possible	1	2	1.5	12
3. Primary Highways Undivided 2 and 3 Lanes ADT 500-5000	• Roadway is routinely patrolled during storms • Mostly clear pavement after storm stops • Hazardous areas receive treatment of chemicals or abrasive • Remaining snow and ice removed when thawing occurs	1	2.5	2	24
4. Secondary Roads ADT less than 500	• Roadway is patrolled at least once during a storm • Bare left-wheel track with intermittent snow cover • Hazardous areas are plowed and treated with chemicals or abrasives as a first order of work • Full width of road is cleared as equipment becomes available	2	3	3	48

other studies in order to use the statistical methods. Furthermore, it is harder to study the effect of controls and catchment modifications using statistical methods.

The main point is that there are alternatives to the approaches used in SWMM; the latter can involve extensive effort at parameter estimation and model calibration to produce quality predictions that may vary greatly from an unknown "reality." Before delving into the arcane methods incorporated in SWMM and other urban runoff quality simulation models, the user should try to determine whether or not the effort will be worth it in view of the uncertainties of the process and whether or not simpler alternative methods might suffice. The discussions that follow provide a comprehensive view of the options available in SWMM, which are more than in almost any other comparable model, but the extent of the discussion should not be interpreted as a guarantee of success in applying the methods.

Overview of Quality Procedures

For most SWMM applications, the Runoff Block is the origin of water quality constituents. Although effects of dry-weather flow and scour and deposition may be included in the Transport Block, (dry-weather flow quality may also be included in the Storage/Treatment Block), the generation of quality constituents (e.g., pollutants) in the storm water itself can only be included in the Runoff Block.

Several mechanisms constitute the genesis of stormwater quality, most notably <u>buildup</u> and <u>washoff</u>. In an impervious urban area, it is usually assumed that a supply of constituents is built up on the land surface during dry weather preceding a storm. Such a buildup may or may not be a function of time and factors such as traffic flow, dry fallout and street sweeping (James and Boregowda, 1985). With the storm the material is then washed off into the drainage system. The physics of the washoff may involve rainfall energy, as in some erosion calculations, or may be a function of bottom shear stress in the flow as in sediment transport theory. Most often, however, washoff is treated by an empirical equation with slight physical justification. Methods for prediction of urban runoff quality constituents are reviewed extensively by Huber (1985, 1986).

As an alternative to the use of a buildup-washoff formulation, quality loads (i.e., mass/time) may be generated by a rating curve approach in which loads are proportional to flow raised to some power. Such an approach may also be justified physically and is often easier to calibrate using available data.

Another quality source is catchbasins. These are treated in SWMM as a reservoir of constituents in each subcatchment available to be flushed out during the storm.

Erosion of "solids" may be simulated directly be the Universal Soil Loss Equation (USLE). Since it was developed for long term predictions (e.g., seasonal or annual loads), its use during a storm event in SWMM is questionable. But it is convenient since many data are available to support it.

A final source of constituents is in the precipitation itself. Much more monitoring exists of precipitation quality at present than in the past, and precipitation can contain surprisingly high concentrations of many parameters. This is treated in SWMM by permitting a constant concentration of constituents in precipitation.

Many constituents can appear in either dissolved or solid forms (e.g., BOD, nitrogen, phosphorus) and may be adsorbed onto other constituents (e.g., pesticides onto "solids") and thus be generated as a portion of such other constituents. To treat this situation, any constituent may be computed as a fraction ("potency factor") of another. For instance, five percent of the suspended solids load could be added to the (soluble) BOD load. Or several particle size - specific gravity ranges could be generated, with other constituents consisting of fractions of each.

Up to ten quality constituents may be simulated in the Runoff Block. All are user supplied, with appropriate parameters for each. All are transferred to the interface file for transmittal to subsequent SWMM blocks, but not all may be used by the blocks; see the documentation for each block.

Up to five user supplied land uses may be entered to characterize different subcatchments. Street sweeping is a function of land use, and individual constituents. Constituent buildup may be a function of land use or else fixed for each constituent. Considerable flexibility thus exists.

When channel/pipes are included, quality constituents are routed through them assuming complete mixing within each gutter/pipe at each time step. No scour, deposition or decay-interaction during routing is simulated in the Runoff Block.

Output consists of pollutographs (concentrations versus time) at desired locations along with total loads, and flow-weighted concentration means and standard deviations. The pollutographs may be plotted using the Graph Block. In addition, summaries are printed for each constituent describing its overall mass balance for the simulation for the total catchment, i.e., sources, removals, etc. These summaries are the most useful output for continuous simulation runs.

In the following material, the processes described above are discussed in more detail. The various parameters are related to individual data groups as appropriate.

Quality Simulation Credibility

Although the conceptualization of the quality processes is not difficult, the reliability and credibility of quality parameter simulation is very difficult to establish. In fact, quality predictions by SWMM or almost any other surface runoff model are almost useless without local data for the catchment being simulated to use for calibration and validation. If such data are lacking, results may still be used to compare <u>relative</u> effects of changes, but parameter magnitudes (i.e., actual values of predicted concentrations) will forever be in doubt. This is in marked contrast to quantity prediction for which reasonable estimates of hydrographs may be made in advance of calibration.

Moreover, there is disagreement in the literature as to what are the important and appropriate physical and chemical mechanisms that should be included in a model to generate surface runoff quality. The objective in the Runoff Block has been to provide flexibility in mechanisms and the opportunity for calibration. But this places a considerable burden on the user to obtain adequate data for model usage and to be familiar with quality mechanisms that may apply to the catchment being studied. This burden is all too often ignored, leading ultimately to model results being discredited.

In the end then, there is no substitute for local data, that is, rain, flow and concentration measurements, with which to calibrate and verify the quality predictions. <u>Without such data, little reliability can be placed in the predicted magnitudes of quality parameters</u>.

Required Degree of Temporal Detail

Early quality modeling efforts with SWMM emphasized generation of detailed pollutographs, in which concentrations versus time were generated for short time increments during a storm event (e.g., Metcalf and Eddy et al., 1971b). In most applications, such detail is entirely unnecessary because the receiving waters cannot respond to such rapid changes in concentration or loads. Instead, only the total storm event load is necessary for most studies of receiving water quality. Time scales for the response of various receiving waters are presented in Table 4-13 (Driscoll, 1979;

Table 4-13. Required Temporal Detail for Receiving Water Analysis (after Driscoll, 1979 and Hydroscience, 1979)

Type of Receiving Water	Key Constituents	Response Time
Lakes, Bays	Nutrients	Weeks - Years
Estuaries	Nutrients, DO(?)	Days - Weeks
Large Rivers	DO, Nitrogen	Days
Streams	DO, Nitrogen Bacteria	Hours - Days Hours
Ponds	DO, Nutrients	Hours - Weeks
Beaches	Bacteria	Hours

Hydroscience, 1979). Concentration transients occurring within a storm event are unlikely to affect any common quality parameter within the receiving water, with the possible exception of bacteria. The only time that detailed temporal concentration variations might be needed within a storm event is when they will affect control alternatives. For example, a storage device may need to trap the "first flush" of pollutants.

The significant point is that calibration and verification ordinarily need only be performed on total storm event loads, or on event mean concentrations. This is a much easier task than trying to match detailed concentration transients within a storm event.

Quality Constituents

The number and choice of constituents to be simulated must reflect the user's needs, potential for treatment and receiving water impacts, etc. Almost any constituent measured by common laboratory or field tests can be included, up to a total of ten. The name and concentration units are entered in data group J3. These will be passed to subsequent blocks and are used as column headings for tabular output of concentrations, as illustrated in Figure 4-26. This heading style is used in both the Runoff and Transport Blocks.

Options for concentration units are reasonably broad and broken into three categories, indicated by parameter NDIM in data group J3. Most constituents are measurable in units of milligrams per liter, mg/l. Although parameters such as metals, phosphorus or trace organics are often given as micrograms per liter, ug/l, the output of concentrations for NDIM = 0 is F10.3 (allowing for three decimal places), and it is expected to be compatible with reported values of such parameters. Thus, the use of mg/l should suffice for all parameters for which the "quantity" of the parameter is measured as a mass (e.g., mg).

A notable exception to the use of mass units is for bacteria, for which constituents such as coliforms, fecal strep etc. are given as a number or count per volume, e.g., MPN/l. Setting NDIM = 1 accounts for these units (or any other type of "quantity" per liter, including mass if desired). Concentration output for these constituents is given an E9.3 format.

A third category covers parameters with specialized concentration-type units such as pH, conductivity (umho), turbidity (JTU), color (PCU), temperature (°C), etc. These are simulated using NDIM = 2. For these parameters, interpretation of concentration results is straightforward, but "total mass" or "buildup" is mostly conceptual. Since loads (e.g., mass/time) are transmitted in terms of

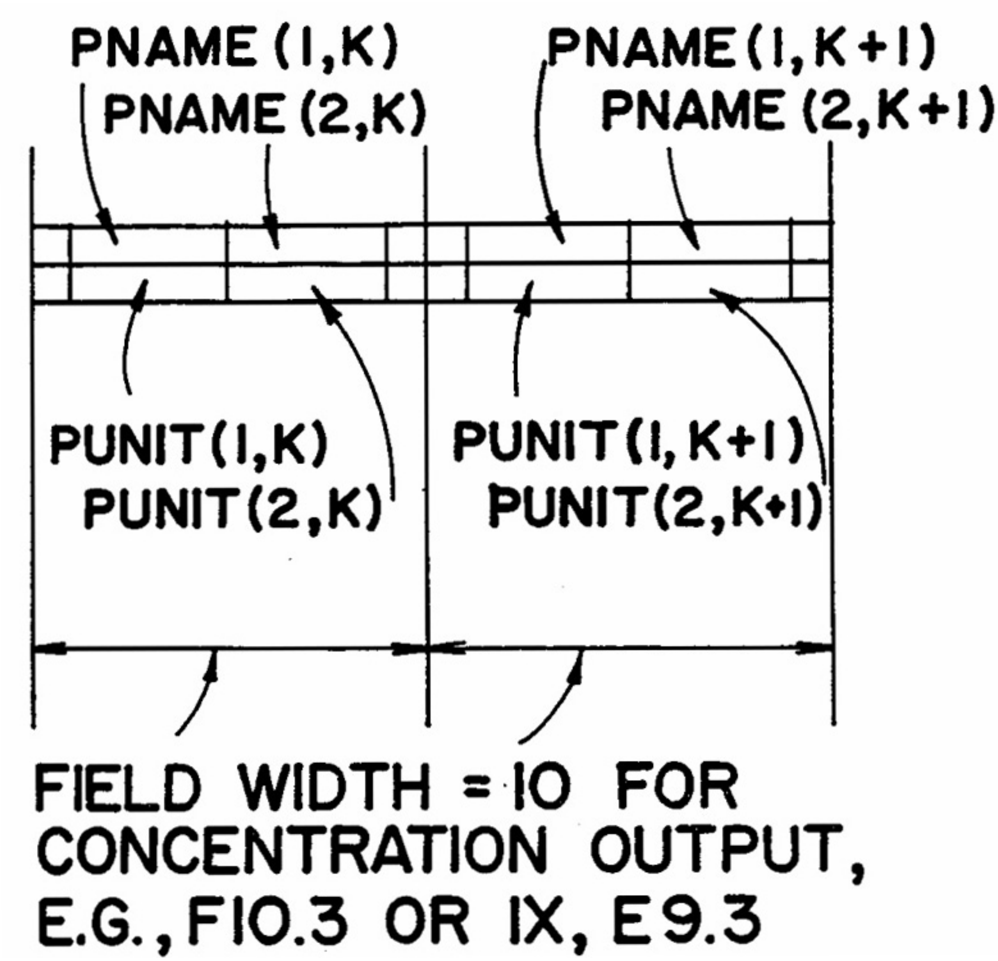

Figure 4-26. Layout of quality constituent headings. Parameters PNAME and PUNIT are entered in card group J3, Table 4-31.

concentration times flow rate, whichever concentration units are used, proper continuity of parameters is readily maintained. Of course, simulation of a parameter such as temperature could only be done to the zeroeth approximation in any event since all Runoff Block constituents are conservative.

Land Use Data (Group J2)

Each subcatchment must be assigned only one of up to five user supplied land uses. The number of the land use is used as a program subscript, so at least one land use data must be entered. Street sweeping is a function of land use and constituent (discussed subsequently). Constituent buildup may be a function of land use depending on the type of buildup calculation specified for each in group J3. The buildup parameters DDLIM, DDPOW, and DDFACT in group J2 are used only when constituent buildup will be a function of “dust and dirt” buildup. This is discussed in detail below.

The land use name, LNAME, will be printed in the output using eight columns. The land use types are completely arbitrary, but they could reflect those for which data are available and, of course, those found in the catchment, or an aggregate thereof.

Buildup

Background

One of the most influential of the early studies of stormwater pollution was conducted in Chicago by the American Public Works Association (1969). As part of this project, street surface accumulation of “dust and dirt” (DD) (anything passing through a quarter inch mesh screen) was measured by sweeping with brooms and vacuum cleaners. The accumulations were measured for different land uses and curb length, and the data were normalized in terms of pounds of dust and dirt per dry day per 100 ft of curb or gutter. These well known results are shown in Table 4-14 and imply that dust and dirt buildup is a linear function of time. The dust and dirt samples were analyzed chemically, and the fraction of sample consisting of various constituents for each of four land uses was determined, leading to the results shown in Table 4-15.

From the values shown in Tables 4-14 and 4-15, the buildup of each constituent (also linear with time) can be computed simply by multiplying dust and dirt by the appropriate fraction. Since the APWA study was published during the original SWMM project (1968-1971), it represented the

Table 4-14. Measured Dust and Dirt (DD) Accumulation in Chicago by the APWA in 1969 (APWA, 1969).

Type	Land Use	Pounds DD/dry day per 100 ft-curb
1	Single Family Residential	0.7
2	Multi-Family Residential	2.3
3	Commercial	3.3
4	Industrial	4.6
5	Undeveloped or Park	1.5

Table 4-15. Milligrams of Pollutant Per Gram of Dust and Dirt (Parts Per Thousand by Mass) for Four Chicago Land Uses from 1969 APWA Study (APWA, 1969)

	Land Use Type			
Parameter	Single Family Residential	Multi-Family Residential	Commercial	Industrial
BOD5	5.0	3.6	7.7	3.0
COD	40.0	40.0	39.0	40.0
Total Coliforms[a]	1.3×10^6	2.7×10^6	1.7×10^6	1.0×10^6
Total N	0.48	0.61	0.41	0.43
Total PO4 (as PO4)	0.05	0.05	0.07	0.03

[a]Units for coliforms are MPN/gram.

state of the art at the time and was used extensively in the development of the surface quality routines (Metcalf and Eddy et al., 1971a, Section 11). In fact, the formulation and data may still be used in SWMM should the user wish to rely upon highly site specific results for Chicago. Needless to say, unless the application is in Chicago this is not recommended. Several useful studies have been conducted since the pioneering APWA work which permit much more selectivity.

Of course, the whole buildup idea essentially ignores the physics of generation of pollutants from sources such as street pavement, vehicles, atmospheric fallout, vegetation, land surfaces, litter, spills, anti-skid compounds and chemicals, construction, and drainage networks. Lager et al. (1977a) and James and Boregowda (1985) consider each source in turn and give guidance on buildup rates. But the rates that are (optionally) entered into the Runoff Block only reflect the aggregate of all sources.

Available Studies

The 1969 APWA study (APWA, 1969) was followed by several more efforts, notably AVCO (1970) reporting extensive data from Tulsa, Sartor and Boyd (1972) reporting a cross section of data from ten US cities, and Shaheen (1975) reporting data for highways in the Washington, DC, area. Pitt and Amy (1973) followed the Sartor and Boyd (1972) study with an analysis of heavy metals on street surfaces from the same ten US cities. More recently, Pitt (1979) reports on extensive data gathered both on the street surface and in runoff for San Jose. A drawback of the earlier studies is that it is difficult to draw conclusions from them on the relationship between street surface accumulation and stormwater concentrations since the two were seldom measured simultaneously.

Amy et al. (1975) provide a summary of data available in 1974 while Lager et al. (1977a) provide a similar function as of 1977 without the extensive data tabulations given by Amy et al. Perhaps the most comprehensive summary of surface accumulation and pollutant fraction data is provided by Manning et al. (1977) in which the many problems and facets of sampling and measurements are also discussed. For instance, some data are obtained by sweeping, others by flushing; the particle size characteristics and degree of removal from the street surface differ for each method. Some results of Manning et al. (1977) will be illustrated later. Surface accumulation data