STORMWATER MANAGEMENT IN COASTAL AREAS - 1982

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INTRODUCTION

The following paragraphs outline the areas that were investigated for this study. The specific results are summarized in reports that are included as appendices. These reports should be read for a description of the methods used and results obtained.

A LITERATURE REVIEW

One objective of this study was to perform a literature review. A literature review was undertaken to identify past work on stormwater management (SWM), with special emphasis on SWM in coastal areas. A detailed literature review was undertaken and summarized in the following report (see Appendix A):

> "Detention/Retention for the Control of the Quantity and Quality of Stormwater Runoff: State-of-the-Art" by R.H. McCuen, S.G. Walesh, and W.J. Rawls

The report is currently being printed by the USDA-SEA-AR. Copies should be available in the near future.

With respect to the objectives of this project, the literature contains only a few articles. Research on stormwater management in coastal areas is almost totally lacking, especially research that discusses the applicability of existing SWM methods to coastal areas and the interaction between SWM and agricultural drainage.

Because of the importance of policy and design and because the literature review revealed very little information, the primary emphasis for the study was placed on SWM policy and design. Because the SCS methods are widely used in MD, a major emphasis was placed on the applicability of these methods in coastal areas.

STORMWATER MANAGEMENT METHODS

There is a legitimate concern about the use of detention storage in low relief areas such as coastal areas and wetlands. Numerous summaries that outline various methods have been compiled. The summary of methods provided in Chapter 7 of TR-55 is fairly comprehensive. Another summary is provided in the following report (see Appendix B):

"Summary of Stormwater Management Methods and Their Applicability in Coastal Wetlands" by R.H. McCuen and S.L. Wong.

It is difficult to make universal statements about their applicability in coastal areas. The effectiveness of any method is highly dependent on site characteristics, as indicated in the report referenced above. It appears that maximum efficiency can be obtained by selecting several methods that appear to be effective for a given site and incorporating them into an integrated design approach. From a hydraulic/hydrologic viewpoint, there is no reason to reject any one method in coastal areas. Site characteristics, including soil type, slope, elevation of the water table, and land use, will determine the effectiveness of any method.

STORMWATER MANAGEMENT POLICY

While analysis of and research on stormwater management design methods is important, stormwater management programs will not be effective if the design methods are not a part of a comprehensive stormwater management policy. Inadequate stormwater management policies can lead to poor designs. Because most governing units (i.e., state, county, and local governments) have had to develop policy statements on their own, there are wide differences in the extent and content of most policies. A survey was undertaken as part of this study to collect SWM policy statements and design standards. An analysis of these documents identified numerous important elements. The following report outlines the aspects of SWM that should be considered in the development of a new policy or modification of an existing policy (see Appendix C):

"Components of a Model Stormwater Management Policy" by M.E. Hawley and R.H. McCuen

It is important to state that this report is intended only as a guide in developing a policy. Because each locality is different, the individual components should be considered in detail for each governmental unit that is developing a policy. For example, agricultural areas may warrant a policy that is different from a policy designed for urban areas. Similarly, a policy for a coastal area may emphasize components that are not important to non-coastal areas. A model SWM policy should not be adopted without proper consideration of each component for the locality for which it is intended.

TEST OF THE SCS METHOD

Because of the widespread use of the SCS TR-55 methods in Maryland, two aspects of design were tested. Firstly, methods of deriving a watershed runoff curve number (CN) estimate were compared. While the data base was not solely for Maryland, most of the watersheds were of low relief. The results are summarized in the following report (see Appendix D):

"Comparison of Methods for Determining Urban Runoff Curve Numbers" by W.J. Rawls, A. Shalaby and R.H. McCuen.

While the watersheds are urbanized, the results of the study are just as applicable for agricultural watersheds.

Secondly, the TR-55 methods were tested using a national data base. While urban watersheds were used in the study (because the title of TR-55 specifies urban watersheds), the results should also be applicable to watersheds with nonurban land uses. The results are summarized in the following report (see Appendix E):

"Evaluation of the SCS Urban Flood Frequency Procedures" by R.H. McCuen, W.J. Rawls, and S.L. Wong.

THE USE OF SCS METHODS FOR DESIGN IN COASTAL AREAS

It is current practice to use the SCS hydrologic design methods (TR-20 and TR-55) for design in Maryland. Because of the concern over the unit hydrograph that is usually used with TR-20 and the tabular method of TR-55, SCS analyzed hydrologic data from coastal areas in Maryland and Delaware. The results of the study indicated that a different unit hydrograph should be used in coastal areas. The commonly used unit hydrograph is characterized by a peak rate factor (D_f) of 484; the SCS study indicated that D_f should be about 284. SCS developed the unit hydrograph for a D_f of 284; a version of the SCS TR-55 tabular method was developed for a D_f of 284; it is available from the MD SCS.

If the SCS methods are to be used for hydrologic design, including stormwater management computations, in coastal areas, then the unit hydrograph is a very important element. Thus, one objective of this study was to examine the peak rate factor, D_f in more detail. The results of this part of the investigation are summarized in the following report (see Appendix F):

"Estimating the SCS Peak Rate Factor" by R.H. McCuen and T.R. Bondelid.

The results indicate that a constant peak rate factor of 284 is reasonable for the coastal areas but it may be somewhat low. A low value of D_f would produce low estimates of the design peak discharge. However, a D_f of 284 is better than the D_f of 484 for coastal areas of MD. The paper indicates that more accurate designs can be obtained by developing the time-storage curve for a watershed and computing the D_f from the time-storage curve. While some additional effort is required, computation of the time-storage curve does not require data beyond that normally compiled for a hydrologic study.

If storm event rainfall/runoff data are available, then the accuracy of designs using the SCS methods can be improved by calibrating the unit hydrograph directly from the rainfall/runoff data. A version of TR-20 that can be used to calibrate the unit hydrograph was developed as part of this study. It is described in detail, including a user's manaual, in the following report (see Appendix G):

"An Automatic-Fitting Version of the SCS TR-20 Hydrologic Model" by T.R. Bondelid and R.H. McCuen.

A copy of the computer program can be obtained from R. H. McCuen.

The TR-20 program can be modified for use in coastal areas by changing the unit hydrographs. The ordinates of the unit hydrograph are contained in the array DIMHYD, which is assigned values by reading the ordinates in through lines 139-185 of subroutine READIN of TR-20. If one wants to permanently modify the TR-20 program to include a dimensionless unit hydrograph other than the standard SCS unit hydrograph, then lines 168-176 of the MAIN program and lines 28-32 of the BLOCK DATA program need to be changed. The array DHX of BLOCK DATA contains the standard dimensionless unit hydrograph. These could be replaced by another dimensionless unit hydrograph, such as the one developed by SCS for coastal areas (D_f = 284).

DOWNSTREAM EFFECTS OF STORMWATER DETENTION BASINS

While stormwater detention basins are widely used for controlling the hydrologic effects of development, there is some concern that these basins may actually create downstream problems. Thus, one objective of this study was to develop a technique that could be easily applied at a site to determine whether or not a more detailed analysis is necessary. The method is presented in the following report (see Appendix H):

"A Planning Method for Evaluating Downstream Effects of Detention Basins" by M.E. Hawley, T.R. Bondelid, and R.H. McCuen.

The method requires very little computational effort; also it is not necessary to collect additional data to use the method. If the method indicates that there may be a downstream problem created by the detention basin, then a comprehensive watershed analysis should be made using TR-20. The effect on channel stability can be made using conventional hydraulic computations, such as the critical velocity method.

INTEGRATED STORMWATER RUNOFF DESIGN

There is a legitimate concern about the use of detention storage in coastal areas. Traditional design methods were often developed using data from non-coastal areas. If these design methods are used in coastal areas, they may require an excessive amount of storage. This is especially critical because of the land that would be required to satisfy the required storage specified by the design method. An integrated approach to stormwater control appears to offer the most efficient design for controlling stormwater runoff. The following reports provide details on the incorporation of various design methods into computation of detention storage requirements using TR-55 (see Appendices I,J, and K):

1. "Integrated Stormwater Management Design" by S.L. Wong and R.H. McCuen.

2. "The Effects of Vegetated Buffer Strips on Runoff Quantity and Quality" by S.L. Wong and R.H. McCuen.

3. "Design of Infiltration Trenches for Control of Stormwater Runoff" by S.L. Wong and R.H. McCuen.

These methods would be especially useful in coastal areas. Buffer strips would be effective for both quantity and quality control; they are especially useful for limiting sediment inflow into streams. If properly maintained, they can reduce runoff volumes significantly. Even the volume of direct runoff from a large storm can be reduced by 5 to 10 percent. Such a reduction can be used to decrease the amount of detention storage required.

RECOMMENDATIONS FOR HYDROLOGIC DESIGN

I recommend the following general policy for hydrologic design:

1. Peak discharges for small drainage areas should be estimated using the TR-55 methods, the chart and graphical methods. The graphical method should be used when an accurate estimate of the time of concentration is available. For low sloped coastal areas, the graphical method based on a peak rate factor of 284 should be used. If such an estimate is not available, then the chart method should be used. When the chart method is used, every attempt should be made to obtain the data necessary to include the corrections and adjustments. Computation sheets for these methods are available in the following text:

   McCuen, R.H., <u>A Guide to Hydrologic Analysis Using SCS Methods</u>, Prentice-Hall, Inc., Englewood Cliffs, NJ, 1982.

2. If the drainage area has a significant channel system and very accurate estimates of the travel times through the channel reaches are not available, the USGS state equation should be used to estimate peak discharges. The USGS equations are given in the following report:

   Walker, P.N., Flow Characteristics of Maryland Streams, Rept. of Investigators, No. 16, Maryland Geological Survey, 1971.

3. If a runoff hydrograph is needed for an inlet area, then the TR-55 tabular method should be used. The expected accuracy of the method will be greatest when the estimates of the times of concentration and travel times for the channel are accurate. The method is especially useful for examining the effects of projected watershed modifications, such as when an area will be cleared of forest land and put into agricultural uses. For low sloped coastal areas, the tabular method based on a peak rate factor of 284 should be used.

4. If a runoff hydrograph is needed and the channel system is significant, the SCS TR-20 program should be used. The TR-20 program should not be used in place of hydraulic computations, such as for the calculation of flow through bridge openings; the Corps of Engineers HEC-2 program should be used for such problems.

5. Where pipe systems represent a major component of the runoff process, a computer model such as INDRA should be used. A Users Manual for the INDRA model is available:

   Donahue, J. and McCuen, R., "User's Manual: Integrated Direct Runoff Algorithm: INDRA," Technical Report, Dept. of Civil Engineering, University of Maryland.

CHANNEL STABILITY

The literature includes very little research concerning the interaction between land use changes and the stability of channels downstream. The method proposed in Appendix H can be used to evaluate the potential for SWM at a site to cause downstream flooding problems. Research to date on the effect of downstream flooding on channel stability is quite theoretical and does not provide a universally accepted means of assessing the interaction. Site specific data requirements are very significant, which prevents this research from being widely used. However, preliminary efforts indicate that SWM does not totally mitigate the effects of land use conversion, especially when channel stability is in question. A method is outlined in report of Appendix L:

> "The Effect of Land Use Change and Stormwater Management on Flow-Duration Curves," by S.L. Wong, R.H. McCuen, and M.E. Hawley.

While estimates made by the method given in the report may not be highly accurate, there is no reason to believe that the relative effects are inaccurate. The flow-duration curve is a better tool for analyzing the effects of land use change and SWM when channel stability is a primary concern.

AGRICULTURAL DRAINAGE

The need for stormwater management can result from the conversion of forested land to agricultural uses, as well as from urbanization. On the Eastern Shore of Maryland the majority of changes in land use involve the establishment of new agricultural areas; there is very little urban development. This region is characterized by very low slopes and high water tables; most of the soils are sandy. Conversion of forested areas to agricultural land can cause hydrologic problems due to both increases in the volume and magnitude of peak runoff and increased erosion and sedimentation.

Cutting and clearing forest vegetation in order to develop agricultural lands will result in increased volume of runoff for some types of storms. The forest land cover intercepts a large percentage of rainfall, both in the tree leaf canopy and in the dead leaf layer at the ground surface. When the forest canopy is removed, this level of interception of rainfall will not occur; even if agricultural crops are planted, the amount of storage in the land cover will generally be less than that of the forest cover. For low-intensity storms, much (if not all) of this formerly-intercepted water will infiltrate into the soil, especially in the sandy soils typical of the Eastern Shore; for longer duration storms, the infiltration capacity may be exceeded and surface runoff will occur.

Because of the generally high water tables and high permeability rates found on the Eastern Shore, even moderate storms are likely to raise the water table to the level of the ground surface in some areas. Usually the low areas in agricultural fields are likely to be covered with standing water after rainstorms. This is sometimes due to surface runoff, but it can also be caused by a rise in the water table.

Standing water on the surface and saturation of the root zone of the soil can be harmful to agricultural crops. The usual approach to solving these problems is to excavate artificial drainage channels that will remove the excess water quickly. While this method will generally solve the problem of increased volumes of water caused by conversion of forests to agricultural land, these artificial drainage canals can adversely affect water quality. Therefore, these drainage channels should be designed with the objectives of solving both the quality and quantity problems.

The most important aspect of water quality in agricultural areas is sediment. Other pollutants, such as herbicides, pesticides, and fertilizers, tend to be absorbed onto sediment particles; therefore, if the flow of sediment through the drainage system is controlled, the movement of these other pollutants can also be controlled to some extent.

There are two potential sources of sediment in these drainage channels. First, erosion may result from the impact of rainfall and the resulting overland flow; thus, sediment may be washed into the channel. Secondly, sediment may be entrained in the channel flow if the flow velocity becomes high enough to scour the bed and sides of the channel. Both of these potential sources of sediment can be controlled if proper design procedures are used in planning the artificial drainage network. Also, provisions can be made for removing sediment before it enters the channel.

The majority of the damage caused by sediment occurs when the sediment-laden water flows out of the artificial drainage channel into tidal waters. Because the velocity of the flow drops as the water leaves the channel and enters the wetlands, the sediment tends to be deposited at the entrance of the wetland that is adjacent to the end of the drainage channel. This excess sediment can bury many types of organisms and the increased turbidity can interfere with photosynthesis by submerged aquatic vegetation. Overall, the effect is a reduction in the biological activity in the wetlands, which can lead to lower productivity of fish and other food organisms. Therefore, it is imperative to prevent the sediment from being carried into the wetland areas or tidal waters into which the agricultural drainage channels usually drain.

Perhaps the easiest way to prevent sediment from entering the drainage channels in surface runoff is to use vegetative buffer strips along the sides of the channels. Design methods for these buffer strips are available (See Appendix J); the strips should be designed for high trap efficiency in serve storms, because the detrimental effects of overland flow are not likely to be significant during small storms because of the high infiltration capacity of the soils. Alternatives to vegetative buffers would be straw bales and/or filter cloth barriers; either of these methods could be used until a buffer strip could be developed. The other alternative would be to leave the forest cover standing along the sides of the drainage channels.

The second potential source of sediment is erosion of the bed and sides of the drainage channel. This will only occur if the velocity of the stream flow exceeds the critical velocity, which is a function of the material lining the channel. Channels can be designed so that scour velocities are only exceeded in extreme storms, and it is usually possible to determine the critical scour velocity from tables published in engineering handbooks. It should be noted that if sediment is allowed to enter the channels from surface runoff, it is likely to be deposited on the beds of the channels somewhere downstream. Therefore, it might be best to assume the critical scour velocity to be the velocity required to entrain this loose sediment, rather than the velocity required to erode the channel itself.

As an alternative to limiting the velocity of flow in the channels, the critical scour velocity of the bed can be raised by lining the channels at critical points with gabion mattresses, filter cloth, or large particles. It must be noted, though, that this will not prevent sediment that enters with overland flow from being carried downstream. Also, these methods of armoring the beds and sides of channels are likely to be expensive.

The final method of preventing sediment from damaging the tidal discharge areas is to design the channel so that there is a sedimentation basin at some point upstream from the tidal outlet. This basin can merely be an area where the channel is considerably wider and deeper, although if the water table is near the ground surface, deepening the channel may not help. The idea is to reduce the velocity of the water so that much of the sediment will settle out. Because a major part of the problem is likely to be caused by suspended sediment, settling may take a considerable amount of time. Also, a sediment trap of this type will require considerable maintenance in order to prevent large storms from reentraining the sediment that has previously settled out.

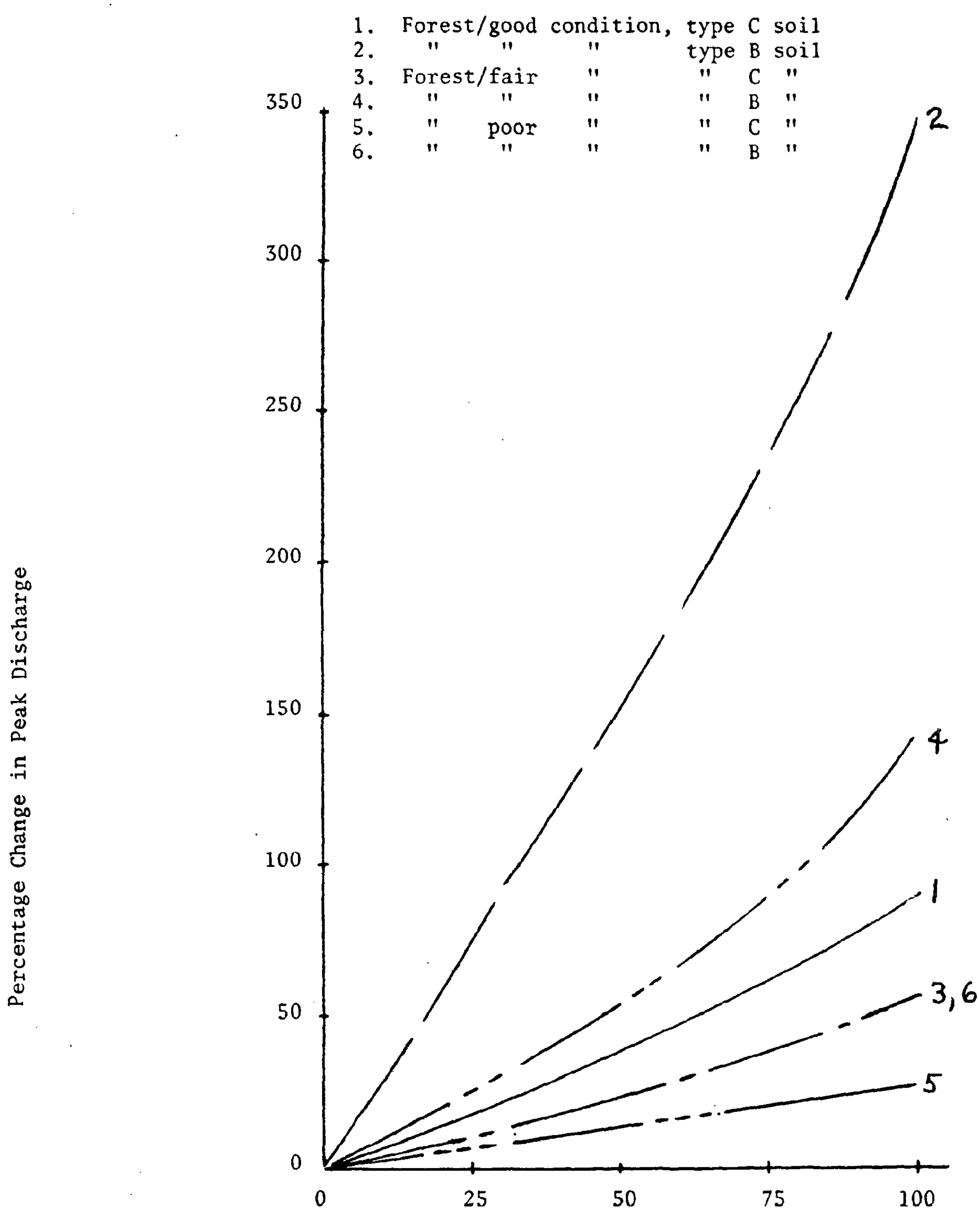
HYDROLOGIC EFFECTS OF THE CONVERSION OF FORESTED LAND TO AGRICULTURAL USES

Given the value of land that is placed into agricultural use, there are economic incentives for converting forested lands to agricultural uses. Such land use conversion, however, has significant hydrologic impacts. Forested land has a very low runoff potential because of the significant amounts of storage. The canopy intercepts significant amounts of rainfall; this amount is especially significant for the more frequent events. The soil below the canopy usually has a relatively high infiltration capacity; thus, the direct runoff volumes and rates are relatively low, and the degree of groundwater recharge is relatively high. While the infiltration and interception potential of a forested area varies with soil type and the condition of the forest canopy, forest lands have a lower runoff potential (i.e., lower SCS runoff curve numbers) than most other land covers.

The conversion of forested land to agricultural use causes very significant changes in the runoff potential of a land parcel. Agricultural lands have significantly less canopy interception; just prior to planting, a canopy may not exist. The loss of interception storage and the removal of camopy litter reduces the potential for water storage; thus, the runoff potential increases significantly. The SCS runoff curve numbers for agricultural land uses are much higher than those of forested lands.

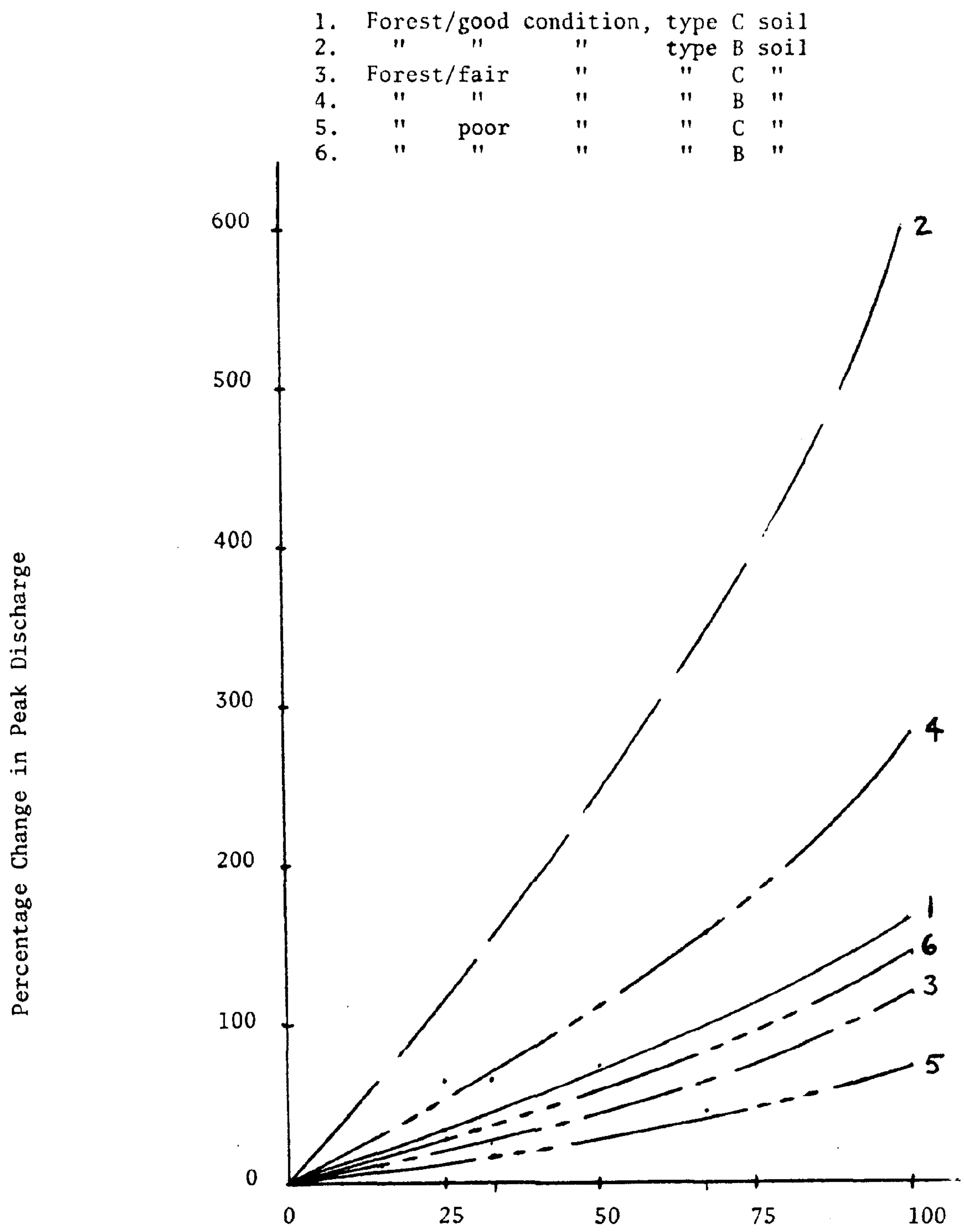
To examine the hydrologic impact of land use conversion, the SCS Chart method was used. The peak discharges for various forested conditions were computed; specifically, forested land in good, fair, and poor condition with types A, B, and C soils were used as the base conditions. Agricultural use was represented by two states: (1) row crops/straight row/poor hydrologic condition and (2) small grain/contoured/good hydrologic condition; these two land covers were selected becasue they represent relatively high and relatively low runoff potentials, respectively. The peak discharges were computed for each condition and for soil types A, B, and C. The results are shown graphically in Figures 1-4 for soil types B and C. These four figures indicate that land cover conversion from forested land to agricultural uses have a very significant effect on the peak discharge. An increase in peak discharge of 100 percent in not uncommon, even for cases where only one-third of the drainage area is converted. For cases where the entire watershed is converted, the change in peak discharge may be as high as 600 percent. The changes for type A soils are not shown because the increases were as much as 5,000 percent, with changes of 1,000 percent not uncommon for type A soils. These extreme changes for type A soils result because of the very low runoff volumes for forested areas on type A soils.

The results of this simple analysis suggests that state and local governments should give serious consideration to the hydrologic impacts of land cover conversions. While stormwater management may be effective for controlling the impact of some land cover conversions, there is reason to be concerned about the conversion of forested land in areas where type A and B soils are predominant.



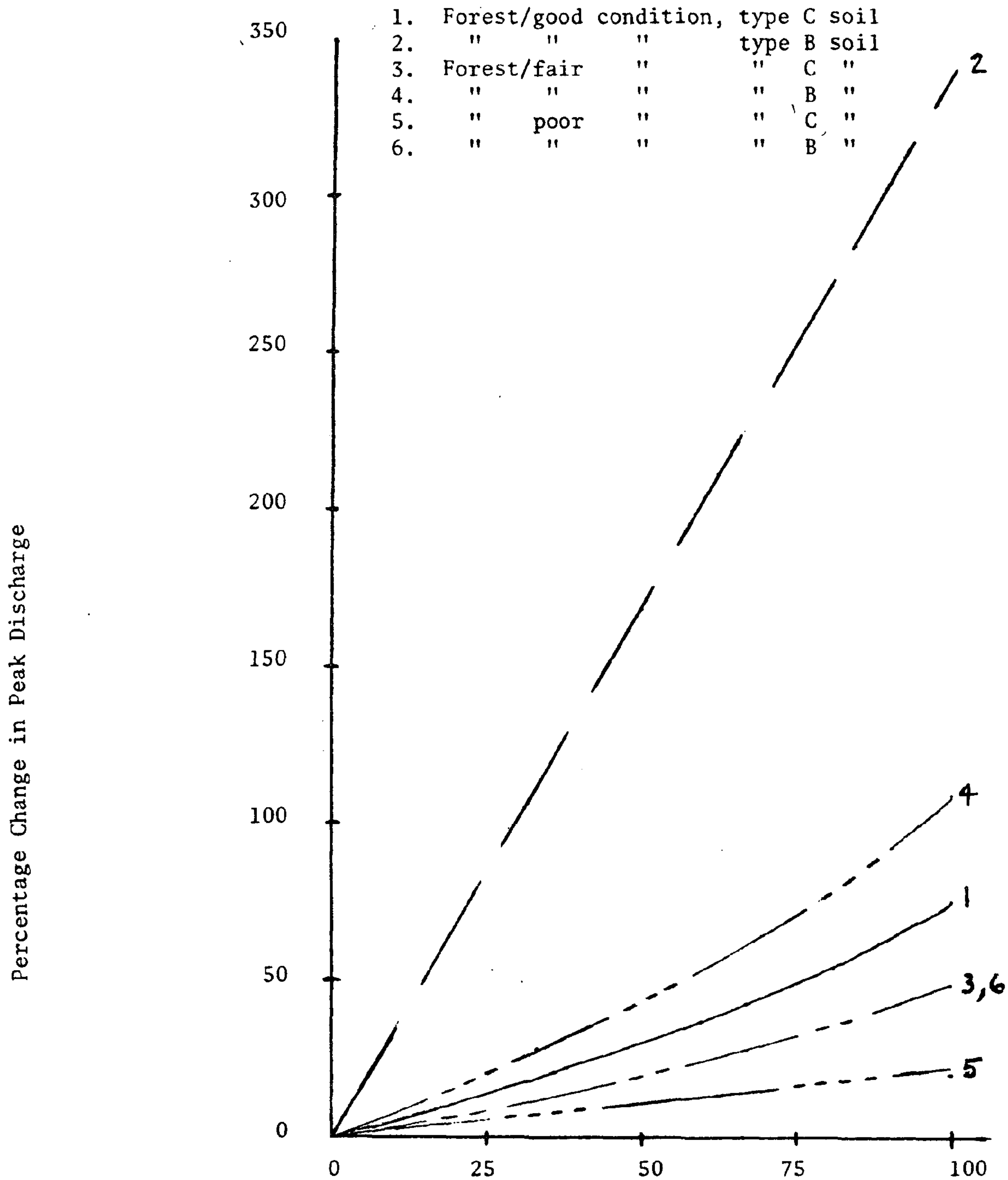
Percentage of Forested Land Cover Converted to Small grain/ Contoured/Good Condition for a Low Sloped Watershed of 20 acres for a 10-year Recurrence Interval Storm Event

FIGURE 1



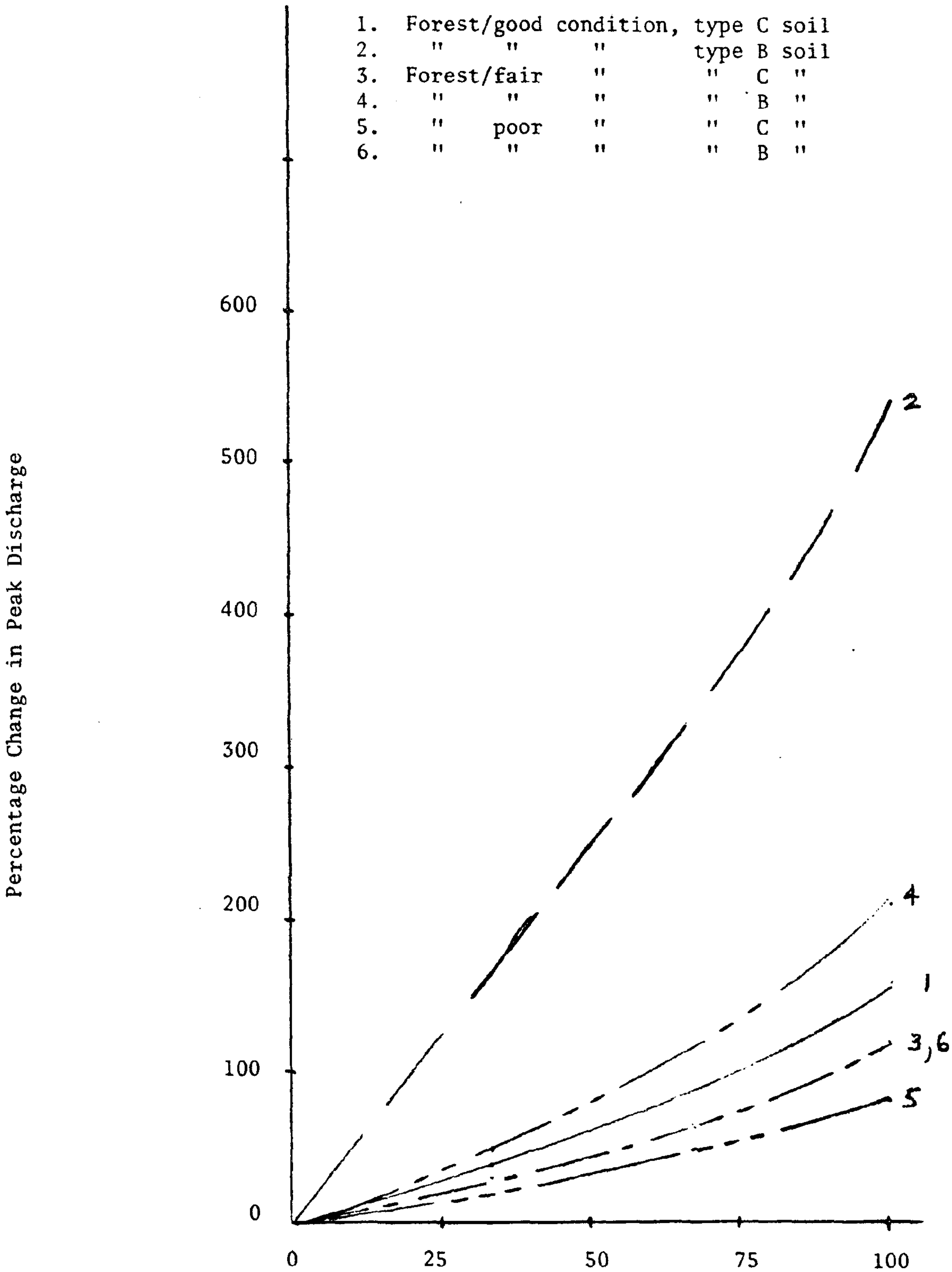
Percentage of Forested Land Cover Converted to Row Crops/ Straight Row/Poor Condition for a Low Sloped Watershed of 20 acres for a 10-year Recurrence Interval Storm Event.

FIGURE 2



Percentage of Forested Land Cover Converted to Small Grain/Contoured/Good Condition for Low Sloped Watershed of 50 acres for a 50-year Recurrence Interval Storm Event.

FIGURE 3



Percentage of Forested Land Cover Converted to Row Crops/Straight Row/ Poor Condition for a Low Sloped Watershed of 50 acres for a 50-year Recurrence Interval Storm Event

FIGURE 4

RESPONSES TO PARTICULAR ISSUES
THAT WERE TO BE ADDRESSED AS PART OF THIS STUDY

Study Product 1: Identification of conditions under which stormwater runoff has a significant effect in coastal areas including a determination of what constitutes a significant effect.

What constitutes a significant effect is a legal question, rather than a hydrologic consideration. From a hydrologic standpoint, any change must be considered as significant; however, at any given site, a large change may not create hydrologic/hydraulic problems. But as a general policy, I believe that conditions in the drainageways, streams, rivers, etc. should not be changed above that which occurs naturally. This is especially important because neither theoretical or empirical studies have adequately addressed the effects of changes. The theoretical bases for the physical systems are not that well understood and empirical studies have been too limited and inconclusive. While a "no in-stream change" policy may appear too restrictive, it is not. Stormwater management control methods have shown to be effective and are continuing to improve; for example, the Soil Conservation Service has recently developed a two-step riser design method. This method would enable stormwater detention basins to be easily designed to control more than just a single point on the frequency curve. The integrated stormwater management design method proposed as part of this study offers the means for using various stormwater management methods as part of a development plan; the use of alternative methods can reduce the need for detention storage requirements. The modification of design methods such as TR-20 for coastal areas should increase the accuracy of designs.

Those involved in establishing development and stormwater management policies for coastal areas must give consideration not just to what constitutes a significant effect, but the criterion variables that effects should be measured on. Flood flow peak discharge is the most commonly used criterion variable.[1] For example, a policy may specify that the peak discharge for a selected excedence probability should not be increased above that which existed prior to the land cover conversion. The use of a single criterion variable may be inadequate. The duration of peak flows is another important criterion variable. The effect of land cover changes on the flow-duration curve of a watershed should be considered in design and addressed in stormwater management policies. While there is concern over sediment loadings, as well as other water quality parameters, it is more difficult to address these parameters because there is little basis for making accurate evaluations of the effect of land cover changes on these parameters. While methods exist for making estimates, it is generally recognized that there is little basis for assessing the accuracy of the estimates in waterways for which extensive testing results are not available. The problem is compounded because there is little data available to indicate exactly what constitutes the "natural" water quality state of all waterways. Thus, measuring changes is difficult and unreliable.

[1]A criterion variable is a quantity that varies from site to site and is used as a criterion for comparison; in the discussion, criterion variables are quantities that are used to judge the effectiveness of stormwater management policies and design methods.

Study Product 2: Analysis of the conditions under which stormwater management methods, both structural and nonstructural, are likely to be effective in coastal areas.

The fact that a watershed lies in a coastal area is not sufficient reason to assume that the effectiveness of a stormwater management method will be different from that for a noncoastal area. The same hydrologic and hydraulic principles apply; however, the relative importance of the input variables may differ somewhat. The effectiveness of stormwater management depends more on the comprehensiveness of the policy and the adequacy of the design method.

This is not to imply that site conditions are unimportant. The slope, soil type, and depth to the water table will influence the relative effectiveness of stormwater management alternatives. If such site variables are not given adequate consideration, then a particular design may not be effective; this is the result of poor design rather than the fault of the stormwater management method. For example, stormwater management methods that rely on infiltration, such as an infiltration bed, will not be effective if a site is characterized by a high water table. This is not a fault of the method but the incompatibility of the method for the site conditions.

A major problem in the application of stormwater management in coastal areas is the failure to properly account for the natural storage in the watershed. Coastal areas with low slopes and sandy soils have the potential for large volumes of natural storage, including noncontributing areas. Failure to account for this storage will lead to overestimation of the required volume of storage through stormwater management. Stormwater management design methods are based on the computation of the volume of runoff and the peak discharge. The time of concentration is an important parameter in estimating the peak discharge, and, thus, the volume of stormwater detention required. If proper care is taken in computing the time of concentration and routing of flows through a watershed, the computed peak discharge and detention volume will be more realistic. Both the peak discharge and the volume of detention may be over-estimated if the time of concentration is underestimated because the natural storage in the watershed is not properly accounted for.

Study Product 3: Analysis of the modifications needed to make TR-20 and TR-55 applicable in coastal areas.

On the basis of their conceptual framework, there is no reason not to use TR-20 and TR-55 in coastal areas. However, because of the characteristics of coastal watersheds several factors should be given special consideration. First, the problem of natural storage discussed above should be accounted for. Specifically, the time of concentration for overland flow paths should account for depressions. Travel times in channels should reflect channel storage, especially through man-made constrictions.

Second, empirical evidence, as well as conceptual rationality, indicates that the unit hydrograph in coastal areas has different distributional characteristics than unit hydrographs for non-coastal areas. The TR-20 program uses a peak rate factor to control the distributional characteristics of the unit hydrographs. While a value of 484 is recommended for noncoastal areas, the analysis of actual runoff hydrographs indicates a significantly lower value should be used in coastal areas. While SCS recommends a value of 284, this value is based on analyses using the HEC-1 program rather than TR-20. Analyses using TR-20 suggest a value nearer to 350 is more appropriate for the coastal areas of Maryland. A modified version of the TR-55 tabular method is available for the peak rate factor of 284.

Study Products 4 and 6: Analysis of the relative importance of variables that affect both stormwater runoff characteristics and the efficiency of stormwater management methods, including a sensitivity analysis of runoff and design characteristics to potential factors.

The dominant factors in predicting the magnitude and frequency of floods are the drainage area, the slope, the land cover and condition, precipitation characteristics, and soil type. Land cover and condition, and the soil type can be represented by the runoff curve number (CN). The time of concentration (t_c) can be computed from the slope, the watershed length (which can be computed from the watershed area), and the land cover and condition. The results in Appendix M indicate the importance of error in estimating either t_c or CN on both the runoff volume and the peak discharge.

The relative importance and sensitivity of design factors of stormwater management methods will depend on the policy used. A policy that requires control of frequent events (e.g., the 2-year event) will result in SWM methods that are relatively more sensitive to the characteristics of the inflow hydrograph than the design characteristics of the SWM method. For SWM methods that are designed to control the larger events, the design characteristics are less sensitive to the characteristics of the inflow hydrograph.

For a stormwater detention basin the magnitude of the outflow will be especially sensitive to the characteristics of the outlet, specifically, the diameter and height of the riser. The timing characteristics of the outflow will be more sensitive to the volume of storage and the distribution of inflow. Detention basins with two stage risers will not show the extreme sensitivity of design characteristics that characterizes basins with one stage risers. The use of the two stage riser will result in outflow hydrographs that more closely approximate the hydrographs of runoff prior to development.

Study Product 5: A determination as to whether stormwater management in coastal areas should address the 100-year event, the 10-year event, the 2-year event, or a combination.

Past research has shown that a SWM policy that specifies only a single storm frequency as the probabilistic basis of design will not control flood magnitudes for other storm frequencies. Therefore, design policies should specify two or more storm frequencies that should be controlled. I would suggest the 2-year and 100-year events. The coastal areas of Maryland are characterized by flood frequency curves with large positive skew. Thus, if the 100-year event is not controlled, stormwater management will be very ineffective during the larger events. Control of the smaller events is very important with respect to limiting channel degradation. For unstable channels, which may result from modification for drainage of agricultural lands, it is important to control the smaller events. Thus, a "two frequency" policy is highly recommended.

It is not unreasonable to require the control of floods of two frequencies. The Soil Conservation Service has just recently completed a publication that outlines the design of two-stage control structures. The method is easy to apply and should not increase the cost or complexity of design. It will, however, improve the effectiveness of stormwater management.

Study Product 7: In conditions under which stormwater management methods are not practical, development of a methodology for determining the level of development that can occur without having an adverse hydrologic impact.

It is difficult to define what is meant by "adverse". The problem is compounded because many criteria can be used to assess the hydrologic impact. For example, peak discharge, volume of runoff, sediment production, bed load transport, or groundwater recharge rates. Even when considering peak discharge, one must be concerned with changes in velocities and the duration of the discharge. Recognizing that it is difficult to define what is meant by "adverse hydrologic impact", I would follow the policy that if stormwater management methods are not practical, then development should not be permitted. The actual hydrologic impact of any proposed development could be assessed using a hydrologic model such as TR-20. However, if development is permitted where SWM is not parctical, then a precedent is set.

<u>Study Product 8</u>: Development of a methodology for assessing the interaction between agricultural drainage practices and stormwater management measures, and identification of the conditions under which such an interaction is likely to have an adverse environmental impact.

If the stormwater management measures are properly designed and located, they should not interact improperly with agricultural drainage practices. However, improper maintenance may result in ineffective stormwater management. Thus, the section of SWM policies concerned with maintenance must specifically state the maintenance requirements and provide for inspection and enforcement. Without such force SWM measures may be ineffective in controlling runoff from agricultural areas. Certainly, SWM methods will not function as intended by the designer if they are not properly maintained.

The interaction between agricultural drainage practices and SWM may be assessed using hydraulic and hydrologic methods commonly used. Where sedimentation and erosion are important elements, there is not widely accepted; procedure this is especially true for evaluating watershed erosion and sedimentation on a storm event basis.

Study Product 9: Development of a methodology for determining whether there is a need for stormwater management when a forested area is cleared and changed to cropland or when cropland is increased through agricultural drainage and identification of the conditions under which such changes are likely to have a significant effect on stormwater runoff.

Again, the determination what constitutes significant effect is not possible without the specification of the criteria to be used for such an assessment. However, the SCS hydrologic methods, TR-20 and TR-55, can be used to assess the hydrologic impact of changes in land cover, such as a change from forested land use to agricultural uses. The effect of changes in the agricultural drainage can be evaluated using the TR-20 model to estimate the surface runoff hydrogrpphs and the Corps of Engineers HEC-2 model to evaluate the water surface profiles in the channels. The TR-20 RESVOR subroutine can be used to evaluate the effect of a stormwater detention basin.

As noted in the discussion on pages 13-15 there is reason to be concerned about the impacts that are likely to occur from the conversion of forested land in areas where type A and B soils are predominant.

Study Product 10: Development of a methodology for assessing the impact of stormwater management on water quality loadings in coastal areas and analysis of whether it is more effective in coastal areas to modify land use practices or install runoff controls to address water quality problems associated with stormwater runoff.

In recent years, stormwater management has evolved into a discipline that considers not only the dissipation, retention and/or elimination of peak wet weather runoffs, but also the impact that these runoffs have on the quality of receiving bodies of water. The growing awareness of the magnitude of the impact that nonpoint source pollutants can have has led to an enormous activity in trying to better understand what happens when precipitation falls upon the earth. Sediment transport caused by rainfall induced erosion has been of concern to those involved in stormwater management for years. Sediment flow into receiving bodies of water has for years been recognized as the largest pollutant in terms of quantity per year that is discharged into these bodies of water. However, it has only been in the last ten or so years that people have been concerned about what these soil particles actually consisted of in terms of specific pollutants.

Sediment flow into bodies of water was initially of concern for two reasons: the sediment could settle to the bottom thereby accumulating and filling in the body of water; and, the sediment could become dispersed in the body of water and inhibit the passage of sunlight thereby inhibiting photosynthetic activity of aquatic plants. As knowledge about sediment-pollutant interaction increased, concern was also expressed for the effects that the sediment associated pollutants had on water quality. Sediments were no longer considered as inert solids with only their physical size and shape of concern. It was recognized that many pollutants could be associated with sediments. These include various organic substances which can lead to a depletion of dissolved oxygen in bodies of water when these organics are oxidized by microorganisms; plant nutrients such as nitrogen and phosphorus compounds; various heavy metals; coliform bacteria; and, a number of pesticides, herbicides, and fungicides.

Nonpoint Source Modeling. Historically, the study of nonpoint source pollutants has been conducted using extensive sampling programs. However, because of the varying nature of nonpoint sources, the cost required to accurately determine annual nonpoint source pollutant loadings has been typically beyond the resources available to most planning agencies. Where data were available, people often developed various pollutant loading factors which could hopefully be related to some particular land use for various hydrologic events (e.g., kilograms of phosphorus per hectare of farm land). However, it was soon realized that not only was it sometimes difficult to properly assess land use, often differences in soil type, differences in application of chemicals to agricultural land, and other differences precluded this method giving reliable values of nonpoint source pollutants for areas other than that for which the loading factor was developed.

A great deal of work has recently been undertaken to combine water quality aspects of surface runoff with various storm runoff predictive models. A number of predictive models have been developed and reported in the literature. They range from complex computer-based models of rainfall/washoff to simple statistical relationships between runoff and areal pollutant yield rates. These mathematical models have been used to assess nonpoint source pollution and evaluate various Best Management Practices. Some of the most widely used models include:

1) the Pesticide Runoff Transport (PRT) model to estimate runoff, erosion, and pesticide losses from field areas (1).
2) the Agricultural Runoff Model (ARM) to estimate runoff, erosion, and pesticide and plant nutrient losses from field areas (2).

3) the Agricultural Chemical Transport Model (ACTMO) to estimate losses from field or basin size areas (3).

4) the ANSWERS models to estimate runoff and erosion and sedimentation from basin sized areas (4).

5) CREAMS, a model for estimating chemicals, runoff, and erosion from field-size agricultural lands (5).

6) the Hydrologic Simulation Program - Fortran (HPSF) model used to simulate watershed hydrology and water quality; this model uses various pollutant runoff models such as the ARM model and the Nonpoint Source (NPS) model (6) as inputs.

7) SPNN, designed to predict the export of sediment, phosphorus, and nitrogen from agricultural basins during individual storm events (7).

8) CMRA, a chemical migration and risk assessment model for pesticides (8).

9) several statistical correlation studies; an example of which is one by Zison (9) in which sediment-pollutant relationships in runoff from selected agricultural, suburban, and urban watersheds are developed.

These models and studies are all a step forward in the quest to truly understand nonpoint source pollution. However, their application in even a planning function is often rather limited. All of the models except CREAMS require data for both calibration and verification. CREAMS requires various parameter and coefficient estimations.

The basic problem with the models is that they are attempts to describe phenomena that are not well understood. Pollutants are transported by storm runoff in two ways: as a soluble fraction in the runoff; or, as something attached in some way to a soil particle. Various control practices can be applied to various land uses (e.g., Best Management Practices) to reduce both the chemicals applied to land and the potential for stormwater runoff and erosion. However, there will usually still be some quantity of surface runoff to either retain or perform some treatment operation if the adverse effects of nonpoint source pollution are to be eliminated or diminished. Even being able to predict how much nonpoint source pollution exists by the use of various models will not always solve the problem of what to do about it.

Sediment Capture. One apparently rational nonpoint source pollution control method might be to capture the sediment in surface runoff. By thus preventing this sediment from reaching various bodies of water, nonpoint source pollution would be reduced. A stormwater management strategy that required for the removal or detention of stormwater induced sediment would be a step in the right direction. However, although it is apparent that sediment capture would reduce nonpoint source pollution due to various sediment-pollutant interactions, the complexities of these interactions make it difficult to quantify the reduction of nonpoint sourse pollution due to sediment capture.

Sediment - Pollutant Interaction. The process of pollutant-soil interaction is quite complex. Soils can retain, modify, decompose, or absorb pollutants. Every year enormous quantities of organic materials, atmospheric pollution, and other liquid and solid wastes are deposted and incorporated into soils and safely decomposed into their basic constituents: carbon dioxide, nitrogen, phosphorus, and other residues of biological decomposition. Because of a high concentration of soil bacteria, the decomposition processes are quite intensive and effective, and represent one of the best natural recycling processes. The processes which participate in the soil decomposition and removal of pollutants include adsorption, filtration, ion exchange, biochemical action of soil microorganisms, particle charge interaction, pH effect, and precipitation. On account of the adsorption and retention processes of the soil, almost all of the phosphorus, heavy metals, many pesticides, and organic chemicals remain near the point of application and move only with eroded soil. The exception is in sandy or peat soils that have little tendency to adsorb pollutants. The residual pollution that passes the soil layers plus that elutriated from the soils enters the groundwater aquifer.

Most sediments fall into one of two broad categories according to origin:

1) organic matter: usually the products of varying degrees of decomposition of animal and plant material ranging in size from colloidal humus (0.6×10^{-6} m diameter) to very large pieces of material.

2) Mineral particles: sediments ranging in size from clay particles of colloidal or near colloidal size, through silts and sands to large bolders.

Colloidal mineral or organic materials are extremely active from an adsorption standpoint, but larger particles are practically inert. However, this is sometimes the result when, for example, sand particles become coated with very active fine organic matter.

These colloids are a concern from a water quality viewpoint; the colloids may then be divided into two groups: clay and organic. Clay particles are usually colloidal or near colloidal in size (0.2×10^{-6} to 4.0×10^{-6} m) and are made up of laminated plates or rods of alumina and silica. Because of their construction, the specific area of particles (ratio of internal and external surface area to mass) is huge. Their structure is such that each surface will attract and adsorb cations present in dry surrounding water.

The structure of organic colloids is more complex and more variable than clays. The nuclei consist of various compounds composed of mostly carbon and hydrogen, which can be oxidized to inorganic material. So, unlike clays, they are not conservative substances. However, they are similar to clays in that the nuclei have a strong negative charge and attract and adsorb cations. In fact, the cation exchange capacity of organic colloids far exceeds that of even the most active clays. Hence, organic colloids will be more active carriers of pollutants.

Adsorbed chemicals are, therefore, usually associated with clay and organic fractions of sediment. Many studies have shown that phosphorus, most heavy metals, ammonia, and some pesticides will undergo interaction with the soil particles and become somewhat immobilized, remaining near the layer of application. Rainfall enduced erosion can, therefore, transport these pollutants away with surface runoff. Nitrate nitrogen is much more mobile and, if excess is present, will either be carried off in soluble form in surface runoff or penetrate downward and possibly contaminate the groundwater. The same phenomena is observed with some pesticides.

Pollutant Capture by Sediment Capture. Although it is known that many pollutants are attached to sediments, it is difficult to generalize concerning the removal percentage of various pollutants even if the removal percentage of sediment is known. Information about the specific soil type would have to be correlated with information concerning the application amount of the various chemicals, the time from chemical application until rainfall event, the time between rainfall periods, various control practices, and other factors to predict pollutant association with sediment without conducting extensive sampling.

However, a stormwater management strategy that contained provisions for sediment capture would result in reducing the nonpoint source pollutant loading to receiving bodies of water. For most effective removal of sediment related pollutants, attempts should be made to capture as many of the colloidal size particles as possible. While this might be somewhat difficult to accomplish in conventional sedimentation basins, the use of aquatic vegetation in sedimentation basins could markedly increase the removal of these colloided particles.

Recommended Policy. At the present time (1982), The state-of-the-art is not sufficient to describe the general effects of stormwater management on water quality. At a specific site, an extensive data collection program can provide the basis for reliable statements about stormwater management and water quality. But there is no evidence to indicate that the results are transferable to adjacant watersheds. While this may cause a legitimate concern by regulatory agencies, it does not mean that attempts should not be made to control the water quality effects of land cover changes.

The water quality models mentioned above can at least be used to measure relative effects of land cover changes. That is, a model could be used to predict the water quality state for the land cover conditions both before and after development. While the individual estimates may not be highly accurate, there is little reason to believe that the relative effect is not accurate. The difference in the water quality states for the two land cover conditions could then be used to assess the expected impact of the land cover conversion. If the change in the water quality state is considered significant, then a stormwater quality management plan could be required.

For purposes of the evaluation of land cover changes on the Eastern Shore, I would recommend the use of the CREAMS model. This model, which was developed by The Agricultural Research Service, does not require calibration. It is based on existing empirical evidence and the experiences of the Department of Agricultural. It is constantly being modified to incorporate new data and experiences into the model framework. Futhermore, the hydrologic component is based on the SCS Methods, which are required for many project types in Maryland. These advantages are significant and make it the most practical choice.

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APPENDIX A

DETENTION/RETENTION FOR THE CONTROL OF THE QUANTITY AND QUALITY OF STORMWATER RUNOFF: STATE-OF-THE-ART

Richard H. McCuen

Stuart G. Walesh

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DETENTION/RETENTION FOR THE CONTROL OF THE QUANTITY AND

QUALITY OF STORMWATER RUNOFF: STATE-OF-THE-ART

Richard H. McCuen[1], Stuart G. Walesh[2], and Walter J. Rawls[3]

INTRODUCTION

During the 1960's professional journals contained numerous technical papers concerning the hydrologic effects of urbanization. Many of these papers showed the increase in peak discharges and volumes, as well as the effect on unit hydrograph characteristics, that resulted as both the percent imperviousness and the degree of channelization and flood plain enroachment increased. Recognizing that urbanization had detrimental flooding impacts hydrologists in the 1970's turned their attention towards reducing the amount of increase and managing the increased stormwater runoff. As part of the environmental movement of the late 1960's and early 1970's the effect of the urbanization process on water quality received much attention in the professional literature. Thus, the professional literature of the 1970's has contained numerous articles concerning the planning, management, and design of stormwater control methods, with articles on the management of both stormwater quantity and quality. During the 1970's some state and local governments developed policies aimed at providing order to the elements involved in the control of stormwater runoff. The stormwater management process, which includes legislation, policy, planning, design, and management, will continue to evolve during the 1980's.

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Recognizing that urbanization has an impact on water quantity and quality characteristics it may be of interest to examine, at least superficially, the urbanization process and the intent of stormwater management practices. Watershed development is a process that involves a change in land use that often involves removing natural vegetation, removing or covering topsoil, grading of the surface, and covering part of the watershed with impervious material. The loss of natural vegetation represents a loss of interception storage that exists with natural vegetation. Grading reduces both infiltration and depression storage. Similarly, the depression storage on impervious surfaces is less than for pervious land uses, and covering a watershed with impervious materials will obviously eliminate infiltration in these areas. The losses of natural storage, i.e., interception, subsurface, and depression, results in the increases in peak discharge rates and runoff volumes that are apparent in urbanized watersheds. The transition from a natural to an urbanized state is an especially troublesome period. The transition period is often characterized by runoff that is especially high in pollution loadings. Even after the land has been stabilized, urban areas are known to have higher pollutant loadings than most natural land uses; this results from the activities that are characteristic of urban watersheds. Another problem that surfaces as urbanization increases is the change in time characteristics of runoff; this change is partly responsible for increased flooding problems.

To mitigate these detrimental effects that occur as a watershed experiences development, a number of different stormwater control alternatives have been proposed and adopted by state and local governments. While these measures may differ in content, the underlying intent should be to offset the effects of urbanization on the hydrologic cycle. Specifically, the stormwater management measures should be designed to minimize hydrologic effects, to replace the natural storage that is lost due to development, and to neutralize the effects of changes in time characteristics. For most parcels of land stormwater management methods, if properly designed and managed, should be able to eliminate the detrimental effects of urbanization.

During the past decade, detention/retention (D/R) facilities have gained increased popularity as major elements in urban stormwater control systems. The original motivation for this evolving use of storage-oriented facilities seems to have been cost savings that are sometimes achieved relative to the traditional conveyance-oriented systems. An apparent second motivation for the growing use of D/R facilities has been their aesthetic, recreational, and other values.

Because of growing concern with control of non-point source pollution in unsewered or separately sewered areas, a potential third reason for considering utilization of D/R facilities has arisen: enhancement of the quality of urban stormwater runoff. However, relatively little performance, planning, design, and operation and maintenance experience is available and that which is available along with the research and development studies that are under way or have been completed, is scattered about varying institutions and literature.

REPORT OBJECTIVES

Because of the diversity of the journals in which technical papers have been published and the variation in technical backgrounds of those interested in the stormwater management process, it should be of interest to all involved in stormwater management, including hydrologists, engineers, and planners as well as legislators and managers, to have a literature summary and a state-of-the-art analysis available. Such a summary should provide a convenient source for identifying a sampling of the literature that is pertinent to the specific aspect of stormwater management that is of interest to the individual. In addition to a state-of-the-art analysis, this report provides abstracts of some articles that are concerned with stormwater management and have appeared in professional publications. Table 1 identifies papers according to selected key words; the references are abstracted in Attachment B. Also, a list of other references on stormwater management, which were not abstracted, are included in Attachment A as another source of information. While it is not always possible to separate the quantity and quality aspects of stormwater management, this report provides a separate state-of-the-art analysis of the literature for the runoff quantity and quality aspects. Part I is devoted to a summary of quantity aspects and Part II provides a detailed analysis of the quality aspects.

Table 1. Literature abstracts referred to in Attachment B.

	Author(s)	Water Quality	Economics	Policy	Design	Planning	Management	Sediment	Data Collection	Coastal Areas	Modeling	Case Study	Urban Land Use	Agricultural Land Use	Non-Structural M[illegible]	Sewer Systems	Porous Pavement
1.	Alley	X							X	X	X	X	X				
2.	Baker				X		X				X		X				
3.	Bedient and Amendes	X	X		X		X		X		X	X	X				
4.	Bedient, Harned, and Characklis	X				X	X		X		X	X	X				
5.	Bouthillier and Peterson				X	X		X			X				X		
6.	Brandt, Conyers, and Ettinger		X				X	X			X	X	X				
7.	Butler and Maher	X	X	X		X	X						X		X		
8.	Calabrese	X	X			X	X			X	X	X	X		X	X	
9.	Cordero				X	X	X	X				X	X				
10.	Curtis and McCuen			X	X	X	X	X			X		X				
11.	Davis, McCuen, and Kamedulski	X		X			X	X			X	X	X	X			
12.	Day and Ho	X							X	X		X		X			
13.	Dendion, Delleur, and Talavage		X			X	X				X	X	X			X	
14.	Diniz (1979)	X				X					X						
15.	Diniz (1980)	X			X						X		X				X
16.	Fruend and Johnson	X					X	X	X		X	X	X			X	
17.	Gbruck and Urban	X			X				X		X	X	X				X
18.	Grigg, et al.		X	X		X							X		X		
19.	Grigg, Duda, and Morris	X		X		X	X			X		X	X		X		
20.	Guy	X	X	X	X	X	X	X					X			X	
21.	Hawkins, Meloy, and Pavon	X		X											X		
22.	Henry and Ahern		X		X	X	X		X		X					X	
23.	Jackson and Ragan				X								X				X
24.	Kamedulski and McCuen (a)			X			X	X	X		X	X	X				
25.	Kamedulski and McCuen (b)			X	X						X		X				
26.	Krishnamurthi and Balzer	X		X	X			X									
27.	Krishnamurthi and Lenocker					X				X	X		X				
28.	Lai				X					X	X	X	X			X	
29.	Lakatos and Wiswell					X					X	X	X			X	
30.	Lockwood				X						X		X		X	X	
31.	Looper			X	X				X	X	X	X	X			X	
32.	Manz			X		X	X			X	X	X	X		X		
33.	Mariles, Bribiesca, and Mora		X		X	X					X		X			X	
34.	Mattraw	X							X		X	X	X			X	
35.	McCuen (1980)	X					X	X	X		X	X	X				
36.	McCuen (1979a)			X		X		X	X		X	X	X				
37.	McCuen (1979b)			X	X			X			X		X				
38.	Moodie, Scholes, and Thompson	X				X		X	X			X	X				
39.	Myers and Ho					X			X	X	X	X					
40.	Nawrocki and Pietrzak		X	X				X					X				
41.	Novitski (1978)	X		X				X		X		X					
42.	Novitski (1977)	X		X				X		X		X					
43.	Patrick	X				X	X	X		X		X		X			
44.	Pennell					X	X			X			X				
45.	Peterson, Bubenzer, and Madison	X				X	X	X		X		X		X	X		

	Author(s)	Water Quality	Economics	Policy	Design	Planning	Management	Sediment	Data Collection	Coastal Areas	Modeling	Case Study	Urban Land Use	Agricultural Land Use	Non-Structural Management	Sewer Systems	Porous Pavement
46.	Pitt	X	X					X	X				X		X	X	
47.	Ports				X			X					X		X		
48.	Putt and Johnson				X		X				X		X				
49.	Reuter and Fox	X	X	X											X		
50.	Schluchter and Teubner					X				X	X	X					
51.	Simons, Li, and Ward	X			X	X		X			X		X				
52.	Slyfield					X				X	X		X				
53.	Smolenyak	X				X		X		X	X	X	X	X			
54.	Ward, Haan, and Barfield				X			X			X		X	X			
55.	Weber and Wilson	X			X			X	X								
56.	Widseth		X		X							X	X	X		X	
57.	Willison, et al.	X	X	X				X							X		
58.	Woodward, Welle, and Moody				X				X	X	X	X		X			
59.	Wu and Ahlert	X				X		X		X	X	X	X				
60.	Wycoff		X		X	X					X		X			X	
61.	Wycoff and Singh				X	X			X		X		X				
62.	Wycoff, Scholl, and Kissoon	X	X	X					X				X				

TERMINOLOGY

The term 'stormwater management' is a term that has evolved because it is recognized that development most often increases the potential for damage from storm runoff, but does not properly mitigate the effects of runoff. A number of management techniques have been used to mitigate the hydrologic effects of increased development, with both structural and non-structural methods currently being used. Structural techniques are construction related while non-structural include legal and administrative procedures. A structure that creates a reservoir is the most popular method of controlling runoff. A number of terms have been applied to these structures, including on-site ponds, detention basins, retention storage, and stormwater management basins. The term "detention" implies a relatively short time period for detaining stormwater runoff while the term "retention" implies a relatively longer time period; however, it is not always clear how these two differ. "On-site ponding" refers to storage methods that control runoff on the site where the runoff is generated; it is a term that can be applied to both detention and retention storage. The term "stormwater management basin," which is the most general, can refer to both surface or subsurface storage. It is frequently used, as it is quite frequently in this report, because it avoids the ambiguities of the other terms and much of the literature that uses the other terms is applicable to all types of structural storage, including rooftop and parking lot storage.

The terms "settling efficiency" and "trap efficiency" and similar terms also have varied and inconsistent meanings and interpretations in erosion-sedimentation literature. Malcolm and New (1975) draw a useful distinction between these terms. Settling efficiency is "the fraction of particles of a certain size that will be trapped in the basin under design conditions at peak outflow." Trap efficiency is "the fraction of material removed from all runoff passing through the reservoir during its life." The definition of trap efficiency is broadened for use herein by basing it on all runoff passing through the reservoir during a period of time. Trap efficiency is expected to significantly exceed settling efficiency for the design conditions because most runoff events will be less severe than the design event.

PART I: WATER QUANTITY

Because of regional variation in hydrologic problems a wide variety of legislation and policy have evolved during the 1970's, the decade of the infancy of small-scale stormwater management. These laws and policies produced a wide variety of design methods. The lack of consistency in the application of design methods has led many, especially the non-engineer, to question the value of stormwater management on small watersheds; this doubt was further intensified by some ambiguous research results, with some research suggesting that stormwater management may actually create problems (McCuen, 1979; Malcolm, 1980). Thus, the area of design methods is a topic of special interest, especially as it relates to the comparison of the different design methods that are available.

In spite of the few questions concerning the value of stormwater management, its use has increased exponentially because, in part, studies have almost without exception suggested that there are many economic benefits of stormwater management. Case studies have shown that downstream flood damage may be reduced. Added benefits result because lower peak discharges mean smaller conveyance systems are required. Benefits that are not easily translated into economic terms include positive environmental impacts. In many instances, the economic factors are influential in decisions concerning stormwater management legislation. For this reason it may be useful to examine the literature that specifically addresses the subject of economic impacts of stormwater management.

In recent years developmental pressures have been very great in coastal areas. Thus, there is now a special interest in examining the value of stormwater management in coastal areas. Because of the low slopes and high water tables that exist in coastal areas there is considerable concern that traditional structural methods of control may not be applicable. Thus, coastal area stormwater management is a topic that will receive considerable attention in the 1980's, and a review of existing literature is especially important.

These three important areas of stormwater management, i.e., design methods, economics, and coastal area analysis, will be examined in more detail. Table 1 provides a more complete list of other publications on these three topics, as well as references to other subject areas.

Design for the Control of Flow Rates

Cordero (1972) presented a simplified method for estimating the volume of storage and release rate from a detention structure, with both the volume and release rate being a function of the drainage area. Such methods, while easy to apply, reflected the state-of-the-art in both stormwater design and policy at that time. However, as the policies improved, the design methods became more complex in both the conceptual development and application. Bouthillier and Peterson (1978) provide a method of computing storage requirements; it uses a graphical relationship of the maximum allowable outlet rate/maximum inflow rate vs the required reservoir volume/total inflow volume.

As suggested previously, the storage in detention and retention basins is intended to offset the natural storage that has been lost due to development. Therefore, the next generation of design methods involved hydrograph analysis. Wycoff and Singh (1976) presented a technique that was typical of the simple hydrograph model; these methods reflect the effect of land use changes on both the time and flow rate characteristics of the hydrograph. This generation of design/planning methods did not require routing, which for some design problems may be considered as a limitation.

Baker (1979) presented a method that recognizes that the rate of outflow is dependent upon the depth of impounded water; thus, it uses a linear routing equation to route the flow through the detention basin. The solution procedure attempts to increase the accuracy of estimates by using an iterative procedure in that rainfalls of various durations are used to determine the critical rainfall duration that requires the largest storage volume.

In the design of a detention/retention basin the discharge rate is dependent on the inflow, the outlet structure configuration, and the basin storage characteristics, all of which are dependent on the discharge rate itself. Thus, an iterative procedure that allows for this interdependent relationship should improve the accuracy of estimates of the required volume of storage and the flow rate. Bondelid and McCuen (1979) presented a method that is representative of the latest generation of design methods; this method has the added features of allowing for multi-stage riser designs and being computerized.

While everyone is concerned with the accuracy of prediction, it is difficult to assess the accuracy of any technique because an adequate data base for comparison does not exist. Thus, at the present time it is only possible to compare the various design/planning methods using synthetic data. Donahue, et al. (1981) provided a comparison of various levels of design/planning models, with the results indicating a very significant range of proposed storage volumes and release rates. Thus, in order to provide consistent estimates it may be prudent to select one method for all designs in a region. This policy would result in consistent estimates and eliminate institutional conflicts that arise from policies that are overly flexible. It must be recognized, however, that the true design cannot be known and that, if the method selected is inherently biased in its conceptual development, then all designs will produce inaccurate answers. The latest generation of models are probably sufficiently accurate that in the trade-off between design accuracy and consistency in estimation, it would be best to identify in stormwater management policy statements a single procedure to be used for the design of stormwater management basins.

Stormwater Management in Coastal Areas

As indicated earlier, developmental pressures are currently greater in coastal areas than most other areas. Because of both environmental and economic reasons, governmental bodies are especially concerned about the hydrologic impact of development. Unfortunately, the literature contains very few publications that provide conclusions that can be transferred to other areas. Also, most studies concentrate on the problems created by development but fail to show conclusively that an intensive stormwater management program will eliminate the problems.

Day and Ho (1978) conducted research to determine the ability of natural wetlands in south Louisiana to remove nutrients from agricultural runoff. Grigg, Duda, and Morris (1980) provided an assessment of the magnitude of the stormwater runoff and flooding problem in North Carolina coastal zone, described existing management programs, and provided recommendations for improving stormwater management programs. In another case study, Patrick (1976) examined water quality problems in Louisiana's coastal zone that might be handled using management techniques. Flooding problems in coastal areas are compounded by tidal surges; Myers and Ho (1980) examined an approach to tide frequency assessment, using the Delmarva coast as an example.

Economics of Stormwater Management

Some early studies demonstrated that detention/retention basins could provide economic benefits, such as a reduced cost for downstream conveyance systems (Leach and Kittle, 1966). Guy (1978) indicates that stormwater management, especially when well coordinated with natural drainage, may result in lower initial costs of storm drainage systems; however, he points out that the cost of maintenance may increase. Reuter and Fox (1976) provided a methodology for evaluating the economic costs of nonstructural pollution control techniques; nonstructural alternatives should increase the effectiveness of structural pollution controls while reducing the costs of achieving environmental quality objectives.

Given that a comprehensive methodology for assessing the economic benefits and costs of a storm drainage system that includes one or more stormwater management techniques has not been developed, it is difficult to make generalized statements about costs. Several studies have provided economic comparisons of stormwater management alternatives; such comparisons may provide some help in evaluating the economic impact of stormwater management alternatives. Henry and Ahern (1976) calibrated formulae for estimating sewer pipe costs; the results indicated that sewer system costs for subdivisions were reduced when storage was incorporated. Rawls and McCuen (1978) also provided regression equations for estimating the cost of storm sewer systems; the equations were calibrated from actual projects throughout the United States. They also provided a simple relationship between detention basin cost and project drainage area, which was based on 34 stormwater detention projects in the Washington, D.C. area. Bedient and Amandes (1978) compared costs of a variety of management alternatives to evaluate the relative effectiveness. Calabrese (1980) developed cost-effectiveness data on various management practices and used the method for establishing the least cost combination of management practices for a case study in Florida.

Brendt, et al. (1972) made a comprehensive cost study on the Seneca Creek watershed near Washington, D.C. In this study costs were compared to effectiveness for many erosion and sediment control systems. Drainage costs in streams were also assessed. The results indicated that sediment damages from uncontrolled erosion on urban construction sites could potentially each cost $1,500/acre.

Butler and Maher (1978) also examined the economic impacts for downstream sites. They concluded that the external costs of upstream development require that both flooding and water quality aspects of stormwater runoff be managed basin-wide; site-by-site action will probably not provide adequate control at a reasonable cost.

Grigg, et al. (1976) provided the most comprehensive analysis of the impacts of urban drainage and flood control projects. The measurement of tangible benefits is described; however, they concluded that a direct objective technique for quantifying intangibles was not available. They provided guidelines for selecting the proper discount rate and for estimating flood damages.

PART II: WATER QUALITY

USE OF D/R IN URBAN STORM WATER MANAGEMENT: HISTORIC OVERVIEW

There has been a gradual evolution in the state-of-the-art of storm water management and also in actual practices in recent years (Poertner, 1974). That evolution is briefly described here to demonstrate that the time has arrived to consider the use of D/R for enhancing the quality of storm water runoff in urban and urbanizing areas.

Impact of Urbanization: Water Quantity and Quality

Urbanization usually increases the volume of storm water runoff while decreasing the runoff time. The net effect of these two processes usually is a significant increase in peak runoff rates and stages. Although natural factors, such as land slope and the underlying soil type, are important in determining the hydrologic-hydraulic response, land use superimposed on the soil by man's activities will markedly alter the response.

In addition to increasing the quantity of runoff, often with adverse consequences, the urbanization process can also increase the variety and amount of potential pollutants washed from the land surface and eventually into the receiving waters. This is the result of two factors which tend to have an additive effect. First, urbanization increases the variety and availability of potential pollutants. Examples include soil and other materials that are loosened and exposed as a result of demolition and construction, the introduction of human litter, careless material storage and handling, and the use of street de-icing compounds and sand. Second, and as already discussed, urbanization typically increases the volume and rate of storm water runoff thereby providing a more effective means of transporting the newly exposed potential pollutants from the land surface into the surface waters. Additionally, stream channel erosion was a principal reason that SWM began and is legislated.

Controlling the Quantity of Runoff

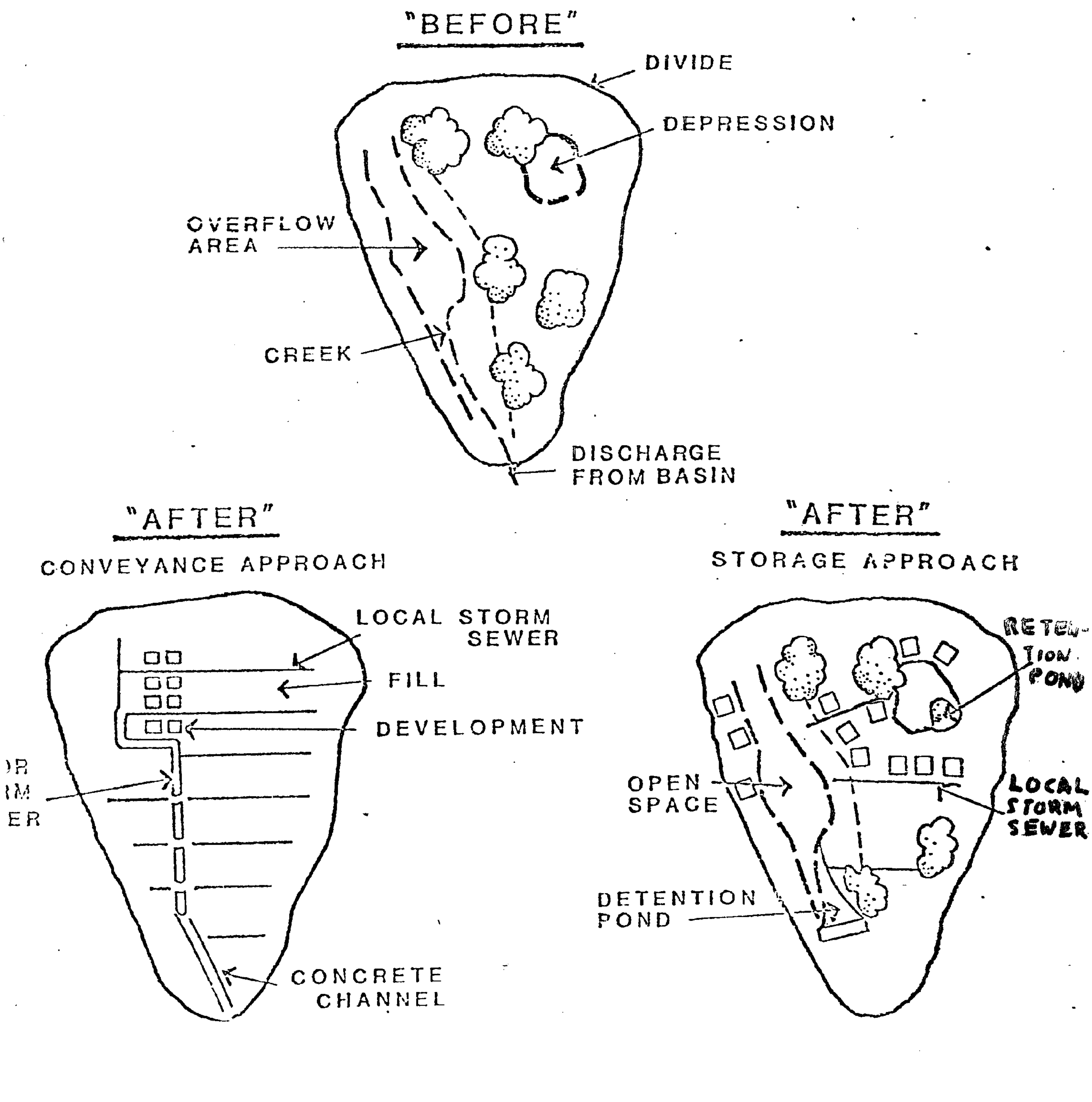
The state-of-the-art of storm water management has developed to the point that there are two fundamentally different approaches to controlling the quantity of storm water runoff. Selected characteristics of these two approaches to storm water management are illustrated in Figure 1.

Conveyance-Oriented Approach

The first of the two approaches is the more traditional "conveyance-oriented" storm water system. Systems designed in accordance with this approach provide for the collection of storm water runoff followed by the immediate and rapid conveyance of the storm water runoff from the area of collection to the point of discharge so as to minimize damage and disruption. The principal components of conveyance-oriented storm water systems are culverts, storm sewers, and channels that are supplemented with storm water inlets and catch basins.

Storage-Oriented Approach

A potentially effective but less common approach to storm water control is the "storage-oriented" system. The function of this type of system is to provide for the temporary storage of storm water in or near the site with subsequent slow release to downstream storm sewers or channels. This approach minimizes damage and disruption both within and downstream of the site. One or more D/R facilities are the principal elements in a storage-oriented system. These principal elements are often supplemented with culverts, storm sewers, inlets, and catch basins.



igure 1. The Two Approaches.

Comparison of Features

Principal advantages of the traditional conveyance-oriented approach to storm water control are: applicability to both existing and newly developing areas, rapid removal of storm water from the service area, minimal operation and maintenance requirements and costs, and accepted analysis and design procedures. Principal advantages of the storage-oriented approach are: possible cost reductions in newly-developing urban areas, the prevention of downstream adverse flooding and pollution associated with storm water runoff, and potential for multi-purpose uses.

Potential Economic Advantage of the Storage-Oriented Approach

The original motivation for use of the newer storage-oriented approach over the traditional conveyance-oriented approach apparently was the realization that the former may offer cost advantages. Poertner (1974) and Donohue & Associates, Inc. (1978, 1979) present cost comparison data for several engineering studies, each of which indicates significant reduction in storm water control costs when D/R is used in lieu of conveyance-oriented systems particularly when the latter utilize storm sewers and concrete-lined channels.

A complete comparison of conveyance-oriented and storage-oriented system must consider other "costs" and "benefits" such as the reduction in available land with the storage-oriented system and increased land values for areas contiguous to D/R facilities. Therefore, cost analyses must be conducted on a case-by-case basis. Nevertheless, there are already enough documented cases of the economic advantage of storage-oriented systems to suggest that D/R facilities should always be considered for controlling the quantity of storm water runoff.

Recreation/Aesthetic Value of D/R Facilities

D/R facilities are being increasingly planned, designed and used as multi-purpose developments. In addition to their primary storm water control function they can provide or be part of a site for recreation activities such as fishing, boating, tennis, jogging, ski-touring, field sports, and sledding. A well planned, designed, and operated D/R facility will also have aesthetic value for contiguous and nearby residential areas.

Mandatory D/R Facilities

Need for Control

In addition to the obvious erosion and sedimentation problems often associated with urbanization, it is becoming increasingly apparent that urban storm water runoff contributes a significant portion of some pollutants to the surface waters. For example, Colston (1974) compared the quality of urban runoff to that of secondary municipal sewage treatment effluent on the basis of weight per unit area per year for Durham, North Carolina. On an annual basis, the urban runoff contributed 91 percent of the chemical oxygen demand, 89 percent of the ultimate biochemical oxygen demand, and 99 percent of the suspended solids. Many Public Law 92-500 "208" studies also concluded that urban storm water runoff was a major contributor of pollutants to surface waters. For example, in southeastern Wisconsin, urban storm water runoff accounts for 54 percent of the suspended solids, 27 percent of the five-day biochemical oxygen demand, and 32 percent of the total phosphorus carried from the land surface to the surface waters (Bauer, 1978).

It appears as though controlling the quality of runoff--at least the first and most significant increment of control--should focus on suspended solids partly because erosion and sedimentation are problems in urbanizing areas. Furthermore, many pollutants such as phosphorus, pesticides, heavy metals, bacteria, and other pollutants are carried by soil particles (e.g., Betz Environmental Engineers, 1976; Whipple, 1979; and Davis, 1979). Therefore, successful erosion and sediment control is also likely to lead to significant control of phosphorus, pesticides, heavy metals, bacteria, and other pollutants.

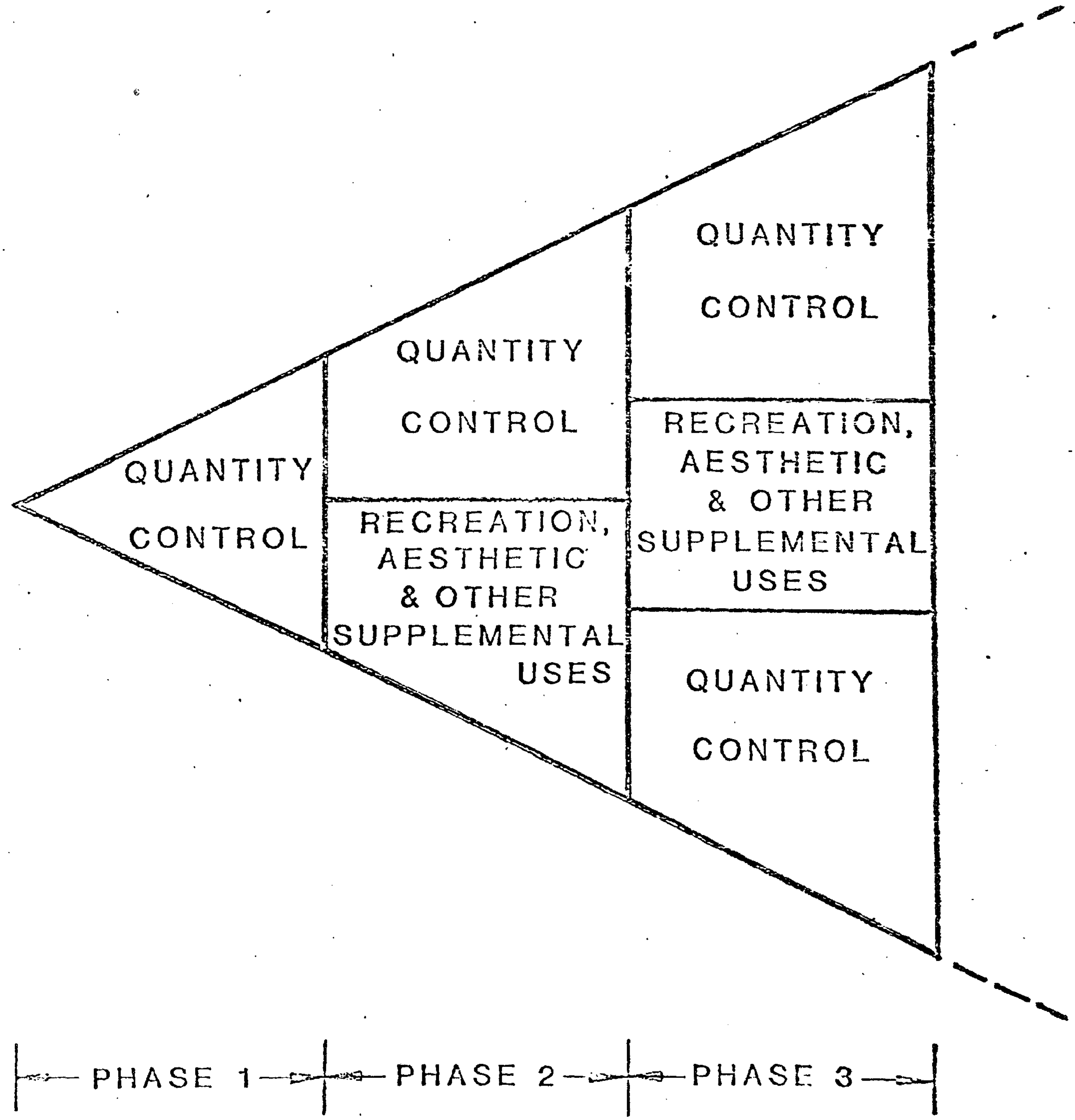
Potential Effectiveness of D/R Facilities

Many measures have been suggested for controlling urban area non-point source pollution in general and erosion-sedimentation in particular. The use of D/R is one of these measures. The state-of-the-art of the use of D/R facilities for control of the quality of urban storm water runoff is the subject of the remainder of this paper. Inasmuch as D/R facilities are finding increased use for controlling the quantity of runoff because of the sometimes cost advantages and recreation/aesthetic values, they should be considered for an important third function: water quality control.

Historic Overview: Summary

The evolution of the use of D/R facilities in urban storm water management is shown graphically in Figure 2. Beginning with the single use of quantity control., D/R facilities have evolved so that they now could serve three compatible functions:

1. Quantity control;
2. Recreation, aesthetic, and other supplemental uses; and
3. Quality control.



;ure 2. Historic Development of the Use of D/R Facilities.

CATEGORIZING THE AVAILABLE LITERATURE

Ideas, information, and data reported in the literature were for purposes of this paper, cited and summarized in these five categories:

1. Water quality performance of D/R facilities based on field studies.
2. Water quality performance of D/R facilities based on laboratory studies.
3. Water quality performance of D/R facilities based on computer modeling studies.
4. Guidelines for planning and designing the water quality features of D/R facilities.
5. Operation and maintenance aspects of D/R facilities.

Literature findings are summarized in the subsequent sections of this paper in accordance with the preceding five categories.

WATER QUALITY PERFORMANCE OF D/R FACILITIES BASED ON FIELD STUDIES

SCS Empirical Trap Efficiency Relationship

The SCS (USDA) has developed graphs of percent sediment trapped versus the ratio of impoundment volume and inflow volume. These trap efficiency curves are based on a study of sedimentation data for relatively large retention reservoirs (Brune, 1953 and Geiger 1963, as cited in Chen, 1975)

For detention, as opposed to retention facilities, five and ten percent reductions in trap efficiency are suggested for incoming suspended sediments that are, respectively, course and fine textured. These adjustments in the trap efficiency curves are intended to account for the increased likelihood of flow directly through a detention facility as opposed to a retention facility.

The trap efficiency curves suggest that D/R facilities, like those finding increased use in urban storm water management, could capture essentially all of the sediment that enters it. There are two qualifications on use of the trap efficiency curves. First, they apply to long-term sediment trapping as opposed to individual runoff events. Second, the curves are based on sediment trap data for impoundments that are generally larger than the typical D/R facility and, therefore, may not actually reflect the behavior of the latter.

Sedimentation Basin - Retention Pond in Montgomery County, MD

Davis (1978) reports on a field monitoring study of the hydrologic and sediment trap performance of the sedimentation basin-retention pond. During the study, the mildly sloped, 45 acre tributary watershed was being developed for a complex of government structures. Watershed soils were reported as "deep, well-drained, and moderately erodible."

The sedimentation basin-retention pond had an area of 1.3 acres at the permanent pool level. Eight acre-feet of water and sediment storage volume--equivalent to 2.1 inches of runoff from the tributary watershed--were available between the bottom of the basin and the crest of the non-perforated circular riser of the riser and barrel service spillway. A four inch diameter port in the vertical riser maintained the normal pool 18 inches below the crest of the riser. This provided up to 2.4 acre-feet of net available storage--equivalent to 0.64 inches of runoff from the tributary area--at the outset of a runoff event.

Inflow and outflow data for ten storms occurring from June through August, 1977 were analyzed. Storm durations ranged from 0.83 to 14.25 hours, rainfall volumes varied from 0.08 to 2.2 inches and average intensities ranged from 0.04 to 0.94 inches per hour. Construction and development proceeded in the tributary watershed during the monitoring as indicated by a general increase in impervious pavement (from 14 to 39 percent of the tributary area) and rooftops (from 8 to 10 percent) and the decrease in exposed soil (from 44 to 3 percent).

Settling efficiency for the ten storms ranged from 88.1 to 99.7 percent. About 92.6 percent of the total sediment that entered the facility during the ten storms was trapped. With the exception of the largest storm--a total of 2.2 inches of rain in 3.9 hours--the peak discharge at the outlet was about 9 percent of the peak discharge entering the facility. In summary, the sediment basin-retention pond was very effective both in trapping sediment and in attenuating peak flows.

The author emphasizes the importance of the amount of storage available between the normal pool level and the crest of the service spillway. A hood fitted to the top of the riser on the riser and barrel service spillway was found to be very effective in preventing floating debris, which is especially likely to be present during construction activities, from escaping from the pond. The author favors non-perforated risers to perforated risers for sediment control based on observed performance of both. Perforated risers permit suspended sediment to pass through the facility and, under some rainfall-runoff situations, results in a "negative" trap efficiency, that is, sediment outflow may exceed sediment inflow, especially during periods of low flow.

According to the author, mitigation of erosion and sedimentation problems in urbanizing areas requires a two-pronged approach--erosion control on disturbed areas, and sedimentation control in the form of sediment basin-retention pond facilities.

Multi-Purpose Retention Facility in Milwaukee

Cherkauer (1977) monitored the quantity and quality of runoff from two adjacent approximately 2.9 square mile urban watersheds in the Milwaukee area for a period of two years. The two watersheds were similar in area, drainage density, total relief, channel gradient, percentage of main channel improved, and percentage of watershed developed.

The most significant difference between the two watersheds is that one contained a 48 acre retention facility normally containing 308 ac-ft. of water and having an average depth (quotient of volume and depth) of 6.5 feet. The retention facility received runoff from the upstream 52 percent of the watershed and was multi-purpose in that it served storm water control, recreational, and aesthetic purposes. Water quality parameters monitored were chloride, sodium, calcium, magnesium, and total dissolved solids, as well as sediment.

The retention facility had the expected effect of significantly reducing peak flows at the watershed outlet while significantly extending the time of hydrograph recession. Chloride, sodium and total dissolved solids monitoring were used as indicators of the effect of winter road salting which was practiced in the watershed for de-icing purposes. The monitoring indicated that the retention facility reduced the transport and concentration of chloride from the watershed during winter runoff events. This was explained as a combination of a dilution effect on the incoming salt water and the tendency of the denser incoming salt water to move to the bottom of the retention facility displacing and forcing out of the retention facility the overlying stored water having a lesser concentration. However, the mean concentrations in runoff from the watershed containing the retention facility were larger. Therefore, the apparent effect of the retention facility was to reduce the annual variability in salt concentrations.

Calcium and magnesium concentrations were monitored to serve as an index to the behavior of substances transported from the soil to the surface water system. The mass transport and concentration of these substances from the watershed containing the retention facility were found to be significantly larger--factor of two or more--during runoff events that occurred during late summer and fall when the quality of baseflow dominated lake quality. The runoff events flushed the stored baseflow from the retention facility and increased downstream concentrations. Although calcium and magnesium are of relatively little practical importance in assessing surface water quality, the behavior of these soil-derived constituents in a drainage system containing a retention facility suggests that the retention facility may store soil-derived pollutants during baseflow periods and cause abrupt and potentially adverse increases in downstream concentrations during an ensuing runoff event. An example of such a pollutant would be certain creosote compounds that have entered the shallow aquifer from a poorly managed creosote operation. In summary, if the soil and, therefore, the baseflow is polluted, the effect of that pollutant on the receiving surface waters may be aggravated if a retention facility is introduced.

Unfortunately, the quantity and quality of lake inflow were not determined coincident with lake overflow characteristics and, therefore, it is not possible to directly demonstrate the effect of the facility on water quality. Also, other water quality parameters of interest such as sediment, dissolved oxygen, fecal coliform, and phosphorus were not included in the analysis.

Underground Detention Facility in Milwaukee

A recent study (Milwaukee and Consoer, Townsend & Associates, 1975) evaluated an underground detention tank as a method for abatement of pollution from combined sewer overflow. The tank received most of the combined sewage from the 570 acre tributary area--relief and interceptor sewers captured some of the combined sewage before it reached the detention tank and transported it past the tank.

Because this facility received combined sewage (a combination of storm water runoff and sanitary sewage) as opposed to only storm water runoff, the results of the study are not directly transferrable to storm water runoff. A study by Dalrymple et al. (1975) suggests that storm water runoff may have better suspended solids settling characteristics than combined sewage. Therefore, the results of this combined sewage detention study for suspended solids may provide a conservative estimate of what could be expected with storm water runoff.

The tank had inside dimensions of 420 x 75 x 16 feet and a capacity of 3.9 million gallons, or 11.97 ac-ft, which is equivalent to 0.25 inches of runoff from the tributary area. Combined sewage flowed by gravity to the tank and entered through a bar screen with 1.5 inch openings. The tank contained several rotary mixers operated only to resuspend solids for pumping out of the tank after a rainfall-runoff event.

Inflow-outflow data were insufficient to directly determine suspended solids and biochemical oxygen demand removal efficiencies. Therefore, a computer model was developed and calibrated using the inflow-outflow data. The sedimentation equation, as discussed by Fair and Geyer (1954), was used in the model to represent suspended solids and biochemical oxygen demand.

Historic hourly rainfall data for three years (wet, normal and dry) were input to the model to test the effect of variable tank size. The size of the storage tank had a significant effect on predicted suspended solids and BOD removal. For example, the modeling indicated that increasing the tank from one million gallons per square mile (3.06 ac-ft/square mile or 0.058 inches of runoff) to seven million gallons per square mile (21.42 ac-ft/square mile or 0.406 inches of runoff) would increase suspended solids removal from about 35 percent to about 90 percent. This is based on the quantity of suspended solids and BOD reaching the tank--not all of it did since some was taken from the drainage area by relief and interceptor sewers. However, interceptor capacity was set at 8 mgd or 12.4 cfs, which was small compared to peak runoff rates during storms.

A model run for 1972 was used to test the effect of pump-out rates on settling efficiency with the conclusion that settling efficiency was relatively insensitive to pump-out time for the range of pump-out time examined. Decreasing the pump-out time from 96 hours to 24 hours for the facility increased overall suspended solids removal efficiency from about 68 to 72 percent and increased overall BOD removal efficiency from about 65 to 70 percent.

Sediment Ponds in Idaho

Bondurant, Brockway, and Brown (1975) discuss the sediment removal efficiency and other characteristics of two sediment ponds in Idaho based on measurements taken over a period of several years. The first and larger basin measured 60 x 500 x 5 feet and served an area of 6,000 acres, or 9.4 square miles. The total volume was 3.44 ac-ft, which is equivalent to only 0.007 inches of runoff from the tributary area. The second basin measured 300 x 40 feet with a depth ranging from 2 to 4 feet and served an area of only 70 acres. The total volume was about 0.8 ac-ft, which is equivalent to 0.15 inches of runoff from the tributary area.

Longitudinal bottom profiles and sediment characteristics (percent clay, silt, sand and density) at the smaller basin revealed a pattern of more sediment and more sandy sediment at inlet end. Periodically determined settling efficiencies varied from 56% to 96% and were greatest during higher flow rates because heavier (more settleable) sediment was carried in. Three years of data collected for the larger basin showed annual trap efficiencies of 65%, 68%, and 74%. High efficiencies did not necessarily mean that less sediment was discharged through either of the ponds.

Ponds in the Woodlands Development, Texas

Davis (1979) monitored low flows and storm water runoff from portions of the Woodlands, a new urban development near Houston, Texas, for two and one-half years. The study area contained two closely spaced artifical lakes in series having a total drainage area of 820 acres and a total volume of 110 ac-ft. The investigations focused on pathogenic and indicator bacteria and included some limited laboratory and field studies of the effect of detention. A few suspended solids and turbidity analyses were also run.

Data presented for five rainfall events which occurred in March (2 events), April, September and October, 1975, indicate large reductions in peak concentrations of suspended solids and turbidity from the upper reaches of the upstream lake to a point in the downstream lake. For example, an approximately four inch rainfall in April produced an upstream peak suspended solids of 2660 mg/l, whereas, the downstream peak was only 356 mg/l--an 87% reduction. Similarly, peak turbidity was reduced from 900 to 210 JTU (Jackson Turbidity Units)--a 77% reduction.

Additional data presented for the above five rainfall events indicated significant reductions in pathogenic bacteria and in fecal coliform and fecal streptococci bacteria. Most reductions in peak bacterial concentrations were in the range of one to three orders of magnitude. Peak concentrations of fecal coliform bacteria in the downstream lake were generally reduced to about 100 to 1000 colonies per 100 ml. The author hypothesized the reductions in pathogen and indicator bacteria were caused by a combination of the processes of dieoff and settling.

Laboratory settling experiments were conducted with 5 gallon samples of storm water runoff. One of a pair of the samples was continuously agitated while the other was not agitated. Samples were periodically drawn from both samples at a depth of 15 cm and analyzed for indicator and pathogen bacteria. Although some high reductions--50% or more--were achieved for indicator bacteria after 30 minutes, the results were inconsistent and, therefore, inconclusive.

Other conclusions were that indicator and pathogen bacteria both exhibit a first flush phenomonen during storm water runoff events and the times of peak bacterial concentration co-relative closely with the peaking times of solids and turbidity. Finally, fecal coliforms as opposed, for example, to total coliforms, appear to be the best indicators of pathogens in storm water runoff.

D/R Basins for Sediment Control at a Maryland Construction Site

Oscanyan (1975) describes the use of two lakes in series (retention facilities) and two sediment basins (detention facilities) into a 65 acre multi-use recreation facility in Montgomery County, Maryland. All four D/R facilities were intended to serve as sediment traps during the construction period.

The upstream lake had a tributary area of 37 acres and a surface area of 0.9 acres. The downstream lake had a tributary area of 75 acres, which included the 37 acres tributary to the upstream lake, and a surface area of 2.0 acres. The larger of the two sediment basins had a tributary area of 15 acres and a surface area of 0.3 acres and the smaller basin had a tributary area of 2.5 acres and a surface area of 0.2 acres. Volumes of the lakes and basins were not given.

A modest sediment inflow-outflow sampling program carried out in the D/R facilities during construction indicated, according to the author, that the two lakes and one of the sediment basins removed over 99 percent of the incoming suspended sediment. However, the second sediment basin trapped only about 30 percent of the sediment during rainfall-runoff events and had a negative efficiency during some rainfall-runoff events. Based on the widely differing sediment trapping performance, the author concluded that factors enhancing sediment trapping include: (1) a high ratio of D/R surface area to stored volume of water at the design storage elevation; (2) a long, narrow shape with flow through the long dimension to prevent short-circuiting; and (3) minimizing turbulence.

Lake in Urban Watershed in Maryland

McCuen (1980) summarizes input-output mass transport quantities and trap efficiencies for 11 parameters. The study focused on runoff from a 148 acre urban watershed in Montgomery County, MD containing a shopping mall, apartment complexes, highways and townhouses under development. Watershed runoff passed through a 5.9 acre lake with outflow controlled by a vertical 24" diameter CMP. The eleven water quality parameters were sampled during "several storm events"--the number and distribution of sampling is not indicated. The monitoring data were used to develop equations for inflow and outflow transport as a function of flow rate. Design storms were then used to calculate inflow and outflow hydrographs for 2, 10 and 100-year recurrence intervals. The hydrographs were in turn combined with the equations to compute mass inflow, mass outflow and settling efficiencies for each of the 11 parameters.

The resulting theoretical settling efficiencies were very high for most potential pollutants. For example, for design storm recurrence intervals of 2, 10, 100-years and storm durations of 0.5, 1.0, 2.0, and 6.0 hours, settling efficiencies ranged from about 20-70 percent for orthophosphate, about 75-90 percent for 5 day BOD and over 95 percent for lead. However, the settling efficiency for total phosphorus was low being between 2 and 22 percent. As expected, settling efficiency tended to decrease with increasing severity of the design storm and with increasing duration within any recurrence interval.

Artificial Marsh/Pond System in Upton, N.Y.

Small (1976) presents input-output concentration data for over 30 parameters collected over a 13 month period (August, 1975 through August, 1976) for an artificial marsh/pond system. This outdoor system, which is located in Upton, NY, received sanitary sewage that had been degritted, screened, and aerated. The marsh/pond system was part of an experiment to develop natural systems for treating sanitary sewage prior to discharging to the groundwater. Although the marsh/pond system received "weak raw sewage or the equivalent of secondary or primary treated sewage", the study is of interest in this paper because of its uniqueness, particularly the use of an artificial marsh and a pond in series.

The upstream, marsh portion of the system covered 0.2 acres and was constructed by excavation, lining with an impermeable membrane, backfilling with 4 to 6 inches of muck, and planting with cattails. Water depth in the artifical marsh was controlled so as not to exceed 2 inches. The downstream pond portion of the system was 5 feet deep and was constructed by excavation, placing a liner, and stocking with carp and other tolerant fish. Because of the carp, algae were the only vegetation supported in the pond. During the 13 month sampling period, the application rate to the artificial marsh varied from 50,000 to 100,000 gpd/acre (1.1 to 2.2 gpd/sq. ft.).

The quality of the flow was significantly enhanced by passage through the marsh system. This may be illustrated by comparing average inflow-outflow concentrations of selected parameters. For example, the average concentration of suspended solids in the inflow was 353 mg/l and the average concentration in the outflow was 43 mg/l--a 88 percent reduction in average values. Inflow concentrations of suspended solids ranged from 50 to 4300 mg/l whereas the range in effluent concentrations was 14 to 100 mg/l. Similarily, the average concentration of biochemical oxygen demand was reduced from 170 mg/l to 19 mg/l--an 89 percent reduction, the average concentration of total phosphorus was reduced from 7.2 mg/l to 2.1 mg/l--a 71 percent reduction, and the geometric mean of fecal coliform bacteria was reduced from 1560 colonies per 100 ml to 50 colonies per 100 ml.

Natural Marsh at Brillion, WI

Inflow to and outflow from a 385 acre natural marsh located at Brillion, WI and having a 19 square mile drainage area were monitored for 15 months (Fetter, et al. 1978). The marsh received surface runoff from the primarily urban watershed and, because of a municipal wastewater treatment plant, secondary effluent accounted for from 2 to 50 percent of the estimated monthly flows into the marsh. The average depth of the marsh was about 1.5 feet and the estimated theoretical detention time, based on mean annual runoff, was 48 days.

A comparison of inflow and outflow concentrations averaged over the study period for various potential pollutants indicates that significant reductions occurred. For example, BOD concentrations were reduced 80 percent, coliform bacteria 86 percent, nitrate 51 percent, COD 44 percent, turbidity 44 percent, suspended solids 29 percent, total phosphorus 13 percent, and orthophosphate about 6 percent. A mass balance analysis indicated a net phosphorus reduction attributed to the marsh of 32 percent for the 15 month study period.

Wayzata Wetland in Minnesota

A study was conducted on the Wayzata wetland in the Minnehaha Creek Watershed District (Wenck, 1977). A goal of the study was to determine the effectiveness of wetlands in removing potential pollutants from incoming streams thus preventing those substances from entering and impairing the quality of lakes. The wetland removed 77 percent of the total phosphorus and 94 percent of the total suspended solids that entered during the study period. Based on a biological assessment, no negative impacts on wildlife or vegetation were noted.

Natural Marsh at Green Lake, WI

This study (Donohue & Associates, 1978) culminated in a management plan for Green Lake, a 7346 acre lake in a preglacial valley about 75 miles northwest of Milwaukee, WI. Green Lake water quality was deteriorating as indicated primarily by increases in nuisance algae and other aquatic plants. The observed water quality degradation and the shift towards a eutrophic status were attributed to an increased influx of nutrients.

One of the recommended management measures is investigation of the feasibility of an artificial marsh on Silver Creek at the point where it enters the east end of Green Lake, to trap incoming suspended sediment and phosphorus.

The recommendations were based, in part, on monitoring of the nutrient trap effect of County Park Marsh through which flow enters the west end of the lake. Grab samples were taken over a period of one year during both wet and dry weather periods. During a monitoring period, 71 percent of the total phosphorus entering the marsh was trapped and prevented from entering the lake.

Artificial Marshes, WI

Studies on several artificial marshes, and a natural marsh at Brillion, WI, suggest significant seasonal patterns (Spangler, et al. 1977). Primary and secondary effluent was passed through the artificial marshes composed of bulrushes and, over an 18 month period, approximately one-third of the total phosphorus in the incoming flow was removed by the marsh.

The variable seasonal phosphorus accumulation pattern was "accumulation over long periods in spring and summer and discharge of phosphorus in sudden spurts either in autumn or winter. ... accumulation in the artificial marshes seems to be associated with high rates of biological activity. Discharge occurs after cold weather kills the flora."

The authors suggest that the water quality enhancement effect of a marsh could be improved by harvesting vegetation and by pumping water from or diverting it around the marsh during expected flushing periods.

PERFORMANCE OF D/R FACILITIES BASED ON LABORATORY STUDIES

Runoff from Durham, North Carolina

Colston (1974) studied a 1.67 square mile (1,070 acre) mixed land use urban basin in Durham, North Carolina. The study area did not have a storm sewer system—drainage was provided by street gutters, swales, small pipes, and culverts. Thirty-six storms were sampled. For most pollutants, concentrations had a standard deviation of 70% to 80%.

Laboratory experiments were conducted on storm water samples using chemical oxygen demand, suspended solids, and turbidity as water quality indicators. The objective was to explore the factors influencing the physical-chemical treatment of storm water runoff. Laboratory sedimentation tests were conducted in 1,500 ml beakers some without pH adjustment and coagulants and others with pH adjustment and various combinations of coagulants.

Fifteen minutes of plain sedimentation under quiescent conditions produced average chemical oxygen demand, suspended solids, and turbidity removals of, respectively 61%, 77%, and 53%. Fifteen minutes of plain sedimentation under quiescent conditions using alum as a coagulant produced higher average chemical oxygen demand, suspended solids and turbidity removals of, respectively, 84%, 97%, and 94%. The author states "within the choices of treatment alternatives, plain sedimentation is a reasonable, relatively inexpensive alternate to chemical treatment of urban land runoff".

After completing the above jar tests, larger batch scale coagulation, flocculation, and sedimentation tests were run on storm water runoff. The test apparatus consisted of a 10 foot tall plexiglass cylinder having a 6.25 inch inner diameter. The results of these tests produced depth versus time relationships for difference percent removal of suspended solids. These relationships were in turn used to construct curves of percent suspended solids removal versus overflow rate. These relationships are for ideal quiescent settling and "would have to be adjusted depending on the relative efficiency of a designed sedimentation basin." Even at an overflow rate of up to 6,000 gpd/sq. ft., over 90% of suspended solids can be removed by sedimentation, according to the settling curves presented in the report.

Unfortunately, the above large scale tests were not run as plain sedimentation, that is, without coagulants or coagulation aids. Based on the results of the jar tests, a smaller maximum allowable overflow rate for a given desired percent removal would be expected.

Runoff from Toronto

Dalrymple and others (1965) reviewed literature dealing with the settling properties of sanitary sewage, combined sewage, and storm water runoff and then performed laboratory suspended solids tests on a few samples of Toronto storm water runoff.

Studies reported in the literature were used to develop a particle size distribution graph for sanitary sewage, combined sewage, and storm water. The graph suggests that particle sizes in potential storm water runoff are significantly larger than in combined sewage. This assumes that storm water runoff will transport all the solids washed off in the street washing experiment. Thus, all other things being equal, better settling should be achieved with storm water runoff than with combined sewage.

Laboratory settling tests were run on storm water runoff collected at an outfall in Toronto. The sample was thought to be higher in clay and silt content relative to most storm water runoff. In one run, one hour of settling reduced suspended solids by 80% from an initial concentration of 337 mg/l. In another run, one hour of settling reduced suspended solids by 87% from an initial concentration of 323 mg/l. Based on their laboratory tests, the authors suggest that the settling characteristics of storm water runoff might be improved by storage prior to settling to provide an opportunity for agglomeration of small particles.

THE WATER QUALITY PERFORMANCE OF D/R FACILITIES BASED ON COMPUTER MODELING STUDIES

Computer Modeling Studies of Hypothetical D/R Facilities

Curtis and McCuen (1977) address the factors that influence the effectiveness of a D/R facility in controlling both flow and sediment. The authors developed a computer model consisting of a soil detachment transport submodel, a detention/retention reservoir sedimentation submodel, and a reservoir routing submodel. All analyses used a 2-year, 6-hour design storm. Sensitivity studies were run leading to the observations that settling efficiency increased with increased proportion of larger (heavier) soil particles, decreased depth of D/R, decreased initial volume of water in D/R, and decreased size of orifices in outlet control structure (a perforated riser pipe). Peak outflow was decreased by all those factors that increased settling efficiency (except for particle size distribution). Because of timing effects, it is possible that multiple D/R in a watershed could increase flow at the watershed outlet.

Kamedulski and McCuen (1979) used a calibrated hydrologic-erosion-sedimentation model to evaluate the consequences of various detention policies. All modeling was based on design storms as opposed, for example, to a series of historic storms. Peak flow reduction through a D/R facility decreased within increased recurrence interval, increased urbanization, and increased storm durations--all because of the increased volume of inflow. Settling efficiencies decreased in similar fashion.

Most D/R policies constrain one point on the discharge-probability relationship which may result in significant over or under design for other possibilities. The general trend suggested by this modeling study was that if a D/R facility is designed for a given recurrence interval, it will be over designed for more severe events and under designed for less severe events. Of interest is the very high--generally 85% or greater--settling efficiencies predicted for all situations. This was explained by a dominance of heavy, large-sized soil particles and long flow lengths.

Settling Efficiency Equation

Haan and Ward (1978) developed a digital computer model to simulate the passage of a hydrograph and sedimentgraph through a D/R basin. A limited amount of data were available to verify the model. Model input includes: inflow hydrograph, inflow sedimentgraph or inflow sediment mass, a stage-discharge-area relationship, sediment particle size distribution and specific gravity. The initial modeling resulted in development of an equation for from the large number of small storms carry most of the pollutants. Therefore, dual purpose D/R facilities should be sized and provided with outlet works that detain as much runoff as possible from small storms and hold it for a long time to provide ample time for settling. Dual purpose D/R facilities should also be designed to safely detain as much runoff from a large storm but release it from the D/R facility as soon as possible to provide room, if needed, for storage of runoff from a subsequent flood. Maintenance will be even more important for dual purpose D/R facilities than for single purpose facilities.

SCS Sediment Basin Concepts and Aids

The Soil Conservation Service (USDA, no date; USDA, July, 1975) present design guidelines for the outlet control structure of a permanent or temporary facility intended to trap sediment. The principal elements of this structure are:

1. a turf-covered earthen embankment with a trapezoidal cross-section,
2. a pipe principal spillway passing beneath the structure provided with a metal riser and trash rack at the upstream end, and
3. an emergency spillway.

This type of hydraulic control structure is frequently modified to serve the dual purpose of controlling large volumes of runoff associated with large infrequent rainfall events while effectively trapping sediment and adsorbed pollutants during smaller but more frequent rainfall and snowmelt events.

settling efficiency as a function of D/R capacity, inflow volume, average detention time, storm duration, peak inflow and outflow rates, and percent of sediment particles smaller than 5 and 20 microns, respectively, at the peak inflow rates.

Potential Adverse Downstream Flooding & Erosion Effects

Malcolm (1978) investigated the effect of a D/R facility receiving flow from a 360 acre watershed on channel bank and bottom erosion downstream of the facility. A hypothetical D/R facility was designed to reduce peak watershed outflow for the selected design storm from 500 to 150 cfs. A digital computer model was used to develop inflow-outflow hydrographs.

In addition to before and after hydrographs for the design storm condition, before and after graphs of tractive force versus time were also constructed. The tractive force graphs indicated that the effect of the storage facility was to reduce the maximum tractive force by about one-half but to double the time of persistence of tractive force above the scouring threshold. This suggests the counter-intuitive conclusion that D/R will not necessarily reduce downstream channel bottom and bank erosion but may actually aggravate it by increasing the duration of tractive forces and by concentrating those forces in and near the channel.

GUIDELINES FOR PLANNING AND DESIGNING THE WATER QUALITY FEATURES OF D/R FACILITIES

Planning and Designing for Control of Quantity and Quality

Whipple (1979) notes that D/R facilities can be planned and designed to control both the quantity and quality of urban runoff. The few large storms that typically occur in a year cause the most flood damage, whereas, runoff

Design Procedures for the Sedimentation Aspects of a D/R Facility

Malcolm and New (1975) present a procedure for designing the sedimentation aspects of a D/R facility. Necessary design criteria include:

1. Selection of a recurrence interval. A 10-year recurrence interval is suggested for the settling performance of the D/R facility with a more severe condition being selected for the hydrologic-hydraulic and structural design of the entire D/R facility.
2. Selection of the size of the smallest particle to be settled with a given settling efficiency at peak outflow. A 40 to 80 micron particle is suggested to be removed at 70 percent efficiency. (Note: 1 micron = 0.001 mm).

The design process, which is described in detail and illustrated with an example, consists of:

1. Sizing a shallow inlet zone to spread the incoming flow across the width of the basin.
2. Sizing the settlng zone based on the design flow and the design particle size and trap efficiency. Hazen's (Fair and Geyer, 1954) theory of sedimentation tank design is used to determine the minimum allowable plan area of the settling zone.
3. Sizing the sediment storage zone which lies beneath the settling zone. The sediment storage zone is sized to contain the total volume of settled sediment that will be trapped during the life of the facility or between sediment removal operations.
4. Sizing the perforated metal riser and the piped spillway which passes beneath the earthen or other dam forming the outlet structure. The riser diameter is based on a weir flow analysis; the number, size and location of perforations in the riser area based on an orifice flow analysis; and the size of the main pipe is based on an inlet control approach.

Oscanyan (1975) also describes and illustrates with an example a procedure for designing the sedimentation aspects of a D/R facility. The procedure is more elaborate than that presented by Rooney and New (1975) and requires more data and, accordingly, may not be as readily applicable. For example, Oscanyan specifies removal of all sediment particles greater than 5 microns in diameter and the corresponding steps in the design procedure requires a grain size distribution curve for the incoming suspended sediment.

The author provides a schematic plan and section drawings of a sedimentation basin. The schematic complements those presented by others (USDA, no date; USDA, July, 1975). Important elements of the basin are: an earthen dam or embankment, a settling zone on top of a sediment storage zone, a metal riser and a pipe spillway which passes beneath the earthern embankment.

The author does not address the design of a combination sedimentation basin-storm water control facility. However, this sedimentation basin design procedure could be integrated with a design procedure for determining the volume of storage and the outlet control works needed to control a large quantity of runoff like that which would be generated by a large storm.

Chen (1975) also presents a design procedure that is similar to the two preceding procedures in that it is based on settling efficiency as determined by settling velocity of particles. An illustrative example is not provided. Useful design aids included in this paper are: a table of dry weights of settled sediment, a table of settling velocities for a range of suspended sediment sizes from fine clay to course sand, and a graph of settling efficiency versus outflow rate (unit overflow rate based on plan area) for the same range in suspended sediment sizes.

OPERATION AND MAINTENANCE ASPECTS OF D/R FACILITIES

A systematic maintenance program is required for the safe operation and proper functioning of a D/R facility is intended to control the quantity or quality of storm water runoff. In areas such as northeastern Illinois where D/R of storm water has been practiced or required for quantity control for a decade or more, there are indications that proper maintenance has not been provided. Inadequate maintenance prevents the D/R facility from functioning as designed and creates safety hazards, flood risks, nuisances, and aesthetic problems.

Madison, WI

Operation and maintenance suggestions based on experience in Madison, Wisconsin are presented by Schoenbeck (1979). The planning and design process should consider operation and maintenance needs by providing ease of access and movement for trucks, front-end loaders, and mowers. Consideration should be given to provision of protective grates on outlet control structures to minimize entrapment of debris and to prevent children from being washed into the outlet control structure.

Madison crews check outlet control structures during and after every runoff event and are experimenting with a weather forecasting service to provide advance information as to the expected occurrence and magnitude of runoff events. Madison has experimented with weirs and other outlet control structures intended to trap sediment.

Highland Park, IL

The City of Highland Park, Illinois (Highland Park, 1977) allows use of a D/R facility as a temporary sediment trap during the construction phase of land development provided that the facility is restored to its design storage capacity. The continued proper functioning of D/R facilities is encouraged by requiring a maintenance procedure for such facilities as part of the plan submitted for land development. Furthermore, the city engineer is required to inspect every D/R facility at least once every five years and to serve notice if repairs or maintenance are required.

Montgomery County, MD

As a result of the Maryland Sediment Control Act of 1970, the Montgomery (County) Soil Conservation District must approve sediment control plans prior to initiation of land development projects. The District's responsibility includes controlling off-site erosion and sedimentation and, therefore, use is made of D/R facilities. Formal maintenance responsibility arrangements must be made prior to construction. If facilities are constructed on public land, the head of the governmental unit must provide a letter indicating responsibility for maintenance. For facilities constructed on private land, the owner must sign a maintenance statement.

DISCUSSION AND CONCLUSIONS

Little is Known

With the exception of a few field and laboratory studies of the effects of D/R facilities on removal of suspended sediment from storm water runoff, few field or laboratory studies have been conducted on the use of D/R for enhancement of storm water runoff. Therefore, the effectiveness of D/R facilities in removing non-point source pollutants--other than sediment--has not been convincingly demonstrated in the field.

A Favorable Prognosis

D/R facilities have, in many instances, been shown to be an effective means for controlling the quantity of storm water runoff while also offering economic, recreation and aesthetic benefits. Suspended solids trap efficiencies in excess of 90 percent have been reported for the few available field studies. Although the state-of-the-art is rudimentary, it appears that with additional effort leading to additional knowledge, D/R facilities could be planned, designed, constructed and operated to also provide for control of the quality of storm water runoff.

Effect on Pollutants Other Than Suspended Solids

Based on the results of very few field and laboratory studies, there are indications that D/R facilities may significantly reduce the concentration of pollutants other than suspended solids. Examples of these pollutants or pollution indicators are biochemical oxygen demand, chemical oxygen demand, pathogenic and indicator bacteria, turbidity and chloride. The solids appear to be a transporter of pollutants such as phosphorus, pesticides, heavy metals, and bacteria. Therefore, suspended solids should be the primary target. The explicit successful control of suspended solids in D/R facilities is likely to result in the removal of other non-point source pollutants.

Factors that Influence Performance

Field, laboratory, and computer modeling studies suggest that D/R facilities can easily achieve suspended solids trap efficiencies of 70% or more. Trap efficiency is improved by or increases with:

1. Volume available for temporary storage of storm water
2. The presence of a restricting outlet control device such as a metal riser
3. Reduction in emptying time between runoff events
4. Increased fraction of larger soil particles

5. Minimizing short circuiting

6. Large surface area for a given storage volume

7. Introduction of a coagulant, such as alum, and coagulant aids

8. An outlet control configuration that minimizes velocity in the D/R facility in the vicinity of the outlet.

The Leveling Effect

Just as D/R facilities reduce the long-term variation in stream flows in downstream reaches, they may also reduce the long-term variation in stream water quality, that is, in the concentration and transport of potential pollutants. Although a leveling of water quality may be generally desirable, there may be adverse effects.

A Cautious Approach to Design

Some design guidelines are available for the sedimentation aspects of D/R facilities for use by practicing engineers. These aids are based on a combination of plain sedimentation basin design procedures drawn from the field of sanitary engineering and empirical sediment trap efficiency studies for large reservoirs. Because incorporation of water quality enhancement features in DR facilities is relatively new, engineers should proceed with caution and make full use of the few aids that are available.

Integrating Quantity and Quality Control Concepts

Preliminary engineering investigations of D/R facilities intended to achieve quantity and quality control with supplemental recreation and aesthetic benefits should draw on experience already gained in the following three areas: hydrologic-hydraulic design of flood control D/R facilities, empirical and theoretical sedimentatiuon design procedures, and use of vegetative filters. One D/R facility concept that integrates these three areas of experience is a series configuration consisting of, in downstream order, a sedimentation basin, a vegetative filter, and a retention reservoir is shown in Figure 3.

Possibility of Increased Downstream Erosion

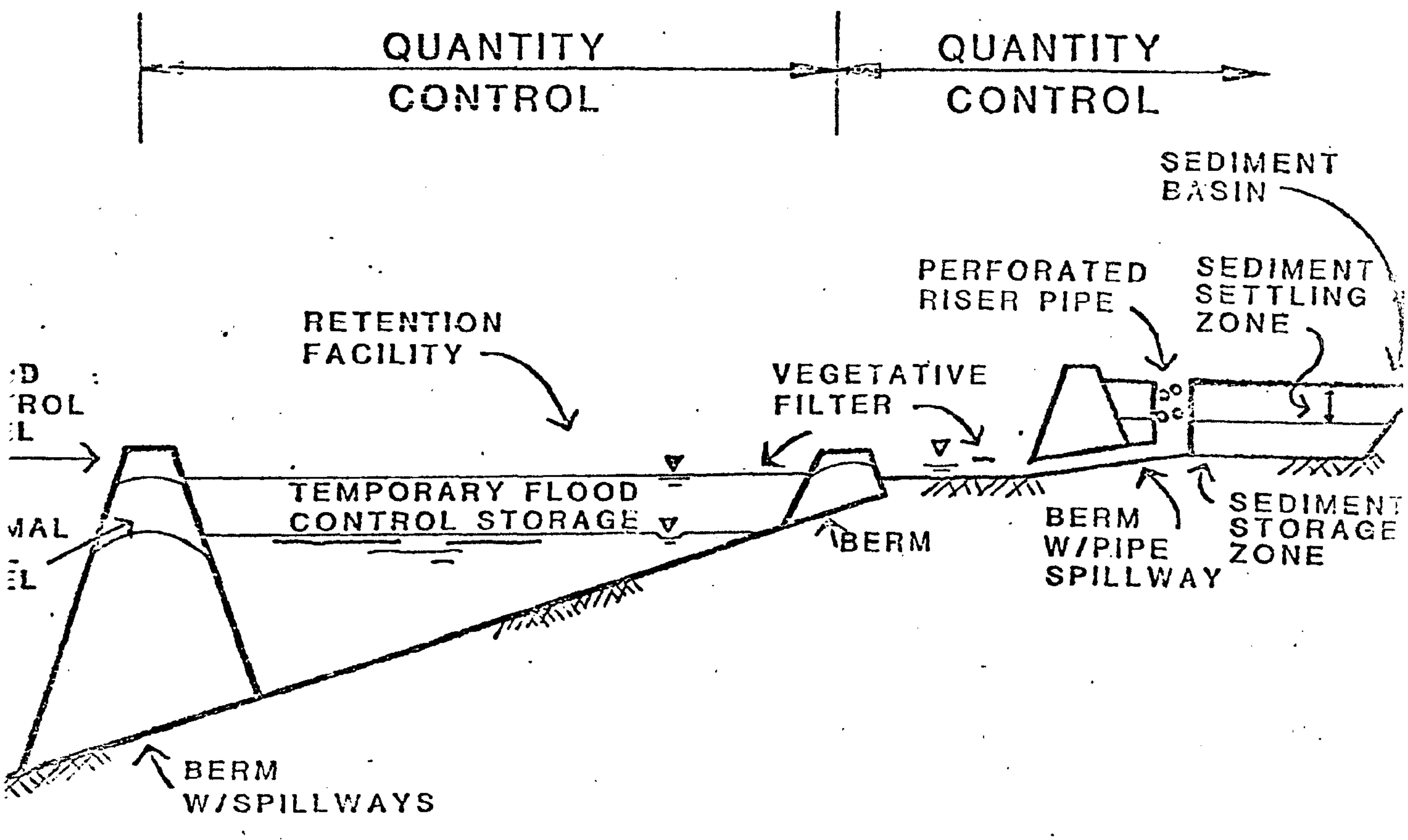
Based on preliminary studies, the construction of a D/R intended to reduce downstream flooding and pollution may under certain conditions actually increase downstream erosion and sedimentation by increasing the duration of erosive forces, by concentrating those forces in or near the channel, and by the reduced sediment load in the D/R discharge. Designers should carefully assess this possibility and its consequences.

The "Little Additional Expense" Idea

In terms of land acquisition and construction costs, relatively little additional expenditures are likely to be required to add effective water quality control components to a planned D/R facility. Storm water D/R facilities intended for control of storm water quantity with supplemental recreation and aesthetic benefits have typically been of such size and configuration that relatively little additional earth work and components would be required.

Potential Inspection and Maintenance Problems

The short experience with operating D/R facilities, most of which were intended to control the quantity--not quality--of runoff, suggests that periodic and storm-related inspection and maintenance is vital to effective operation. One problem that has occurred is accumulation of sediment and debris. Because D/R facilities intended for both quantity and quality control will, by design, accumulate even more sediment and debris, it follows that systematic inspection and maintenance will become even more important.



;ure 3. Intergrating Quantity and Quality Control Concepts.

Inspection and Maintenance Approaches

Proper inspection and maintenance of D/R facilities may be accomplished by municipal staff or be carried out by the owner, with the latter being assured by legal agreement or municipal ordinance. Inspection and maintenance responsibilities should be formalized prior to beginning construction.

One of Several Measures

D/R facilities are one of many structural and non-structural measures potentially available for managing the quantity and quality of storm water runoff. Even if significant advances are made in the state-of-the-art of D/R facility knowledge, other management measures may be more appropriate in a given situation.

"Go Slow" in Mandating D/R Facilities

Given the limited knowledge, local, state and federal governmental units should "go slow" in mandating D/R for controlling the quality of storm water runoff in urban and urbanizing areas. Premature rule-making and regulation is likely to result in "action" but little progress. More is likely to be accomplished in advancing the state-of-the-art and in achieving significant control of non-point source pollutants by (a) funding additional research and development projects and pilot studies and (b) by "encouraging" the control of non-point source pollutants but not dictating the means.

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ATTACHMENT A

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ATTACHMENT B - Papers identified by selected key words presented in Table 1.

1. Alley, W. M. 1980. A simple lumped-parameter runoff quality model. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 236-243.

Abstract: The state-of-the-art of urban runoff modeling is far in advance of urban runoff-quality modeling. Whereas, a physically-based distributed-parameter approach to rainfall-runoff modeling has been successfully performed in many studies, such approaches toward runoff-quality modeling have rarely been a proven success. Recognizing this fact, a single runoff-quality model has been developed. The model is a lumped-parameter model in that no spatial variations in model parameters are accounted for. This model has been linked with a more complex distribution routing rainfall- runoff model. The purpose of this paper is to describe the structure of the runoff-quality model and its application to a small urban drainage basin along the coast of south Florida. A description of the structure and an application of a simple lumped-parameter urban runoff-quality model has been presented. The model utilizes exponential constituent accumulation and washoff equations for runoff-quality simulation and a modified-Rosenbrock direct search algorithm to identify model parameters. An option is included in the model to account for wetfall contributions. The model is limited in its applicability to watersheds and constituents for which runoff loads originate predominantly from effective impervious surfaces and for which constituent concentrations tend to decrease with time from start of storm. The model was calibrated and verified for storm-runoff loads of total nitrogen from a small urbanized watershed in south Florida; standard errors of

approximately 60 percent were found during model verification. The potential importance of accounting for wetfall contribution when determining model parameters for total nitrogén was demonstrated in this application.

2. Baker, W. R. 1977. Stormwater detention basin design for small drainage areas. Public Works, pp 75-79

Abstract: Control and management of stormwater runoff has resulted in associated problems causing it to become an increasing source of concern for public works engineers. One of the most effective methods of reducing the rate of runoff is through the use of a storage or detention basin. The author uses hydrographs to develop a model for uses in determining detention basin design for small drainage areas. A modified version of the rational method was used to generate the inflow hydrograph. This method represents a simple procedure which yields acceptable design results when applied to small drainage areas (i.e., up to 200 acres in area).

3. Bedient, P. B., and C. B. Amandes. 1978. Monitoring, modeling and management of non-point sources in Houston, Texas.. International Symposium on Urban Storm Water Management, C. T. Haan, University of Kentucky, Lexington, pp 151-157.

Abstract: The Department of Environmental Science and Engineering at Rice University has undertaken a coordinated research effort with the City of Houston Health Department on stormwater sampling and management. A comprehensive stormwater quality and low flow sampling program has been designed for the Brays Bayou watershed and tributaries in order to define pollutant loads (TSS, TP, NO_3, NH_3, BOD, DO) and potential impacts. The main objective of this study is to organize the City of Houston sampling effort in order to differentiate the impact of point sources vs. stormwater runoff.

Load-runoff relationships have been developed as a function of land use type and are used in a predictive model (HLOAD) for multiple or individual storm events. Stormwater responses are predicted using a hydrolic formulation such as STORM in concert with the HLOAD methodology. Results from four watersheds have been quite satisfactory for TSS, TP, TKN, and COD. Total annual load calculations indicate 85-90 percent is due to nonpoint source runoff. For a variety of stormwater management practices, including channel modification, detention storage and land use controls, STORM and HLOAD models were used to characterize the effects of storm flows and receiving water quality. Both present and projected land use patterns were imput to investigate the effects of future development in the watershed. Cost data for a variety of management alternatives will be estimated and compared to evaluate relative effectiveness.

4. Bedient, P. B., P. A. Harned, and W. G. Characklis. 1978. Stormwater analysis and prediction in Houston. American Society of Civil Engineers, Journal of Environmental Engineering Division, 104(EE6) 1087-1100.

__Abstract__: Analyses of available stormwater hydrologic and water quality data for several Houston area watersheds yielded direct linear relationships between pollutant mass loading rates and cumulative storm runoff volume. These relationships were then used to rank the watersheds on a pollutant load per unit runoff basis, and to calculate total annual pollutant loads for each area. Further, a plot of dimensionless cumulative flow versus the dimensionless cumulative flux provides a quantifiable means to evaluate the first flush, allowing comparison of this effect between watersheds. Finally, the developed load-runoff relationships provide the basis for a simple, yet effective method of predicting pollutant mass flows for individual or sequential storm events.

5. Bouthillier, P. H., and A. W. Peterson. 1978. Storage requirements for peak runoff control. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 13-18.
Abstract: Storage of storm runoff is becoming a common part of the design of land developments because most environmental control authorities have imposed limits on the peak rate of runoff. The objective of such regulations is to reduce erosion by flows which would exceed those which occurred before development. A method of computing storage requirements for storm runoff is presented. A plot of maximum allowable outlet rate/maximum inflow rate vs required reservoir volume/storage. A bottom outlet is used, the rate of outlet varying as the square root of the head on the outlet. Curves are provided for triangular and for cusp shaped hydrographs. Computations are based on total runoff (coefficient of runoff x rainfall) and a chosen base length of hydrograph. Allowable peak outlet rates may be assumed or set by regulatory agencies. Examples are given using Edmonton rainfall data. The range of options provided, viz; return storm period, base length of hydrograph, and runoff coefficients should enable wide applicability of this method. Considering the uncertainties of rainfall intensity and patterns it is believed that the method is sufficiently accurate for most applications.
6. Brandt, G. H., E. S. Conyers, M. B. Ettinger, F. J. Lowes, J. W. Mighton, and J. W. Pollack. 1972. An economic analysis of erosion and sediment control methods for watersheds undergoing urbanization. The Dow Chemical Company. Midland, Michigan.
Abstract: Economic benefits are expected from controlling erosion and sediment during urban construction, but control costs for urban erosion have not been previously related to benefits. In this study, costs are compared to effectiveness for many erosion and sediment control systems. Potential soil

losses were calculated with the universal soil loss equation. Damage costs in streams were assigned to specific construction sites by equations incorporating estimated cost constants and sediment delivery ratios for various "legs" along a drainage path. Specific damage values were determined for numerous downstream sediment problems. The Seneca Creek watershed, located 20 miles northwest of Washington, D.C., was used as a model watershed for the study. Control practices presently being used in the watershed include sediment basins, diversion berms, level spreaders, grade stabilization structure, sodded ditches, seeding, and straw mulch tacked with asphalt or disked. This conventional system is estimated to cost $1,125 per acre and to control 91% of the potential erosion. Control systems incorporating large sediment basins can boost control to 96% at less total cost. Multipurpose impoundments design with sediment forebays for chemical flocculation can boost urban sediment control to 99%. In addition, such structures contribute very significantly to controlling sediment from agricultural and idle land. Sediment from these non-urban sources must be controlled to significantly reduce the sediment pollution problem. Urban construction sites can contribute large quantities of sediment per unit area, but constitute a minor portion of the total sediment pollution problem because total acreage is generally small compared to other land that contributes sediment. Sediment damages from uncontrolled erosion on urban construction sites in the Seneca watershed could potentially reach $1,500/acre. This estimated damage value applies to specific conditions prevailing in the watershed where 300 tons of sediment could erode per acre of uncontrolled development during an 18-month construction period. The estimated maximum soil erosion rate approaches 200 ton/acre/year or 128,000 ton/mile^2/year. However, downstream damages would be caused by only a

portion of this total potential sediment. Sediment delivery and transport ratios determine damage distribution along a stream and are a major source of uncertainty in this economic analysis.

7. Butler, R. V., and M. D. Maher. 1978. The economics of urban stormwater management. Center for Economic Theory and Econometrics, Rice University, Houston, Texas.

Abstract: Recent studies have shown that urban flooding and water quality problems are due in part to the effects of upstream urbanization. Economic analysis suggests that upstream development will continue and the problem will worsen because upstream residents have no incentive to alter their behavior that causes stormwater problems. The consequences of the increased runoff they generate are borne by others - by affected downstream residents and by the general taxpayer if traditional government protection programs are used to lessen damage downstream. The major policy implication of this analysis is that treatment and mitigation efforts should focus on the source of the problem as well as the site of damage. The external costs of upstream development requires that both flooding and water quality aspects of stormwater runoff be managed basin-wide, since uncoordinated community-by-community action cannot simultaneously implement controls upstream, upland and downstream. In addition, the efficient management strategy must consider both flooding and water quality simultaneously. Since the pollution and flooding problems are produced jointly, it will typically make sense to plan for joint treatment. Several policy instruments for basin-wide control are discussed.

8. Calabrese, M. M. 1980. Optimization of stormwater management practices. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 183-194.

Abstract: In recent years, stormwater has been found to be a major source of pollution to receiving waters. Major research efforts have been directed in this area, primarily as a result of the water pollution acts and amemdments. Yet, there remains a need for more data in this field of stormwater management, especially cost-performance data, and the planning methodology to optimally select management practices based on the cost-performance data. The objectives of this research were to develop cost-effective data on various management practices, and use these data to find optimal combinations of stormwater management practices for storm-sewer systems. The optimal combination of structural and non-structural management practices for wet-weather pollution control was determined using linear programming methods. The selection is a most critical element of stormwater management research because stormwater removals must be related to lake productivity and the best combination of management practices must be specified to achieve desired water quality improvement. Combinations of computer analyses and mathematical programming were used to analyze the data. The research culminating in the computer program "MANAGE" described in the body of this paper reveals a methodology for the choice of an optimal combination of stormwater management practices. However, more work is needed to estimate localized cost-efficiencies for stormwater management practices. Incorporating mathematical programming methods to this work saves time and money since it eliminates guesswork and allows one to select a combination of practices which will remove a maximum amount of pollutants at least cost. The program was beneficial in establishing the least cost combination of management practices in the Lake Eola watershed, Orlando, Florida.

9. Cordero, C. T. 1972. Management of runoff from minor storms in Montgomery County, Maryland. American Society of Agricultural Engineers, St. Joseph, Michigan. (preprint)

Abstract: The purpose if this paper is to present briefly the action taken by and in the County to relieve some of the problems connected with storm water runoff in a rapidly urbanizing area. First will be presented the procedure by which the criteria was developed, and a synopsis of the criteria itself. Then several examples of how the criteria was applied will be briefly described. Finally, an assessment of how well the program is succeeding will be offered to give others the benefit of the experience to date.

10. Curtis, P. C., and R. H. McCuen. 1977. Design efficiency of stormwater detention basins. Journal of American Society of Civil Engineers, Water Resources Planning and Management Division, 103(WR1): 125-140.

Abstract: In addition to increased flood runoff, urban development has caused a significant increase in sediment loads in streams. While many means of sediment and runoff control have been proposed, stormwater detention has been shown to be one of the more cost efficient means. Because detention facilities have not been used extensively in the past, a data base is not available for determining the effect of design factors on sediment trap efficiency and runoff control characteristics. A mathematical model, which includes erosion, sedimentation, and detention facility components, was developed from principles of hydraulics and classical settling mechanics. The model was used to examine the effect of: (1) Detention basin location; (2) soil particle size distribution; (3) basin depth; (4) initial storage, and (5) orifice diameter. An understanding of the relative importance of these factors may lead to better design of stormwater detention facilities.

11. Davis, W. J., R. H. McCuen, and G. E. Kamedulski. 1978. The effect of storm water detention on water quality. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 211-218.

Abstract: Recent studies have indicated that non-point sources of pollution are often a major contributor to the degradation of stream quality. While storm water detention basins have proven to be an effective means of controlling both runoff rates and sediment loads, the effectiveness of detention facilities has not been documented. Montgomery County, Maryland, has completed a Section 208 research study of the effectiveness of a functional sedimentation control basin and a storm water management basin, as examples of local non-point source pollution control structures. These structures are recommended for strategic use in a plan for area-wide waste treatment management developed by the Metropolitan Washington Council of Governments for the Washington, D.C, area. The study included monitoring of rainfall and water quality parameters in basin inflow and outflow, as well as detention basin and land use characteristics. Analysis of measured data provided relationships of inflow sediment and pollutant loadings to land use and rainfall. Trap efficiencies for sediment and pollutant loads were evaluated and related to rainfall, land use, and basin design characteristics. Using the measured data, a hydraulic/hydrologic model was calibrated and used to examine the effect of storm water detention on sediment and other pollutant loads. The model was used to examine the effect that variation of basin design characteristics would have on efficiency of the structures. Monitoring installations and data collection are described, data analysis techniques and results are presented, and the model, its calibration, and operation are discussed.

12. Day, J. W., and C. L. Ho. 1978. Assimilation of Agricultural runoff by a swamp forest wetland. Agricultural and Mechanical College, Center for Wetlands Resources, Louisiana State University, Baton Rouge.

Abstract: The general objective of this research is to determine the ability of natural wetlands in south Louisiana to remove nutrients from agricultural runoff. We are specifically interested in an area of swamp forest near Lac des Allemands. This lake is now highly eutrophic, principally because of agricultural runoff. Under natural hydrologic conditions, runoff from higher areas would percolate through the swamp. Now, because of an extensive network of drainage canals, nutrient laden runoff flows directly into waterbodies causing the eutrophication problem. This problem affects both fresh and estuarine waters and is beginning to threaten commercial fisheries. Earlier studies in Louisiana have shown that when fishery wastes were applied to wetlands, the wastes were assimilated and productivity of wetland vegetation increased by as much as 50%. If the research proposed herein is successful, it will show the utility of natural treatment for non-point source runoff. Thus, a widespread cause of deterioration of water quality in south Louisiana may have an economical solution.

13. Dendrou, S. A., J. W. Delleur, and J. J. Talavage. 1978. Systematic planning of urban storm-drainage utilities. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp. 229-234.

Abstract: A computer program package is developed that integrates and interfaces an urban growth simulation model, LANDUSE, and an urban hydrology model, a modified version of STORM. Alternate growth scenarios can thus be directly related to the corresponding storm-drainage systems. If these systmes are

signed to achieve specified standards of performance, then a useful comparision among several possible urban growth patterns can be perforned. While an urban area encompasses several natural watersheds, the hydrologic models simulate one watershed at a time. The different watersheds that partition an urban agglomeration create a tree-like or dendriform configuration. The planning of a global storm drainage system for such a conglomerate of basins can be efficiently accomplished by a coordination of the interactions among the different basins. A model for these interactions is developed. The planning variables are the drainage network capacity, the placement and size of the storage facilities, and the size of a central treatment facility. An example of application is shown for a medium size community in Indiana.

14. Diniz, E. V. 1979. Water quality prediction for urban runoff, an alternative approach. Stormwater Management Model Users' Group Meeting Proceedings, Project Officer, H. C. Torno, Washington. D. C.

Abstract: The accurate prediction of storm water quality resulting from non-point sources of pollution has recently become a significant area of interest. In general, storm water quality is a function of the total pollutant accumulation on all surfaces exposed to rainfall. However, the rainfall intensity and wash-off capacity of the pollutants are also important factors. After pollutants have been dislodged from exposed surfaces, overland flow and channel hydraulics determine the pollutant concentrations evidenced at a downstream observation of sampling site. Recent studies have indicated that pollution accumulation rates are affected by several factors including land use, population density, impervious area, traffic intensity, condition of surface area, total overland flow length, time from last rain, street sweeping

frequency, climate, and season. A water quality prediction strategy, as normally formulated, requires the compilation of a data base, calibration of a predictive model, simulation of existing and future conditions, analysis of these predictions, and use of the results in planning decisions. Consequently, the data base provides a foundation on which both the analysis and subsequent decisions will be carried out. Because of the importance of valid data to the overall strategy, numerous investigators have attempted to compile data which could be used to develop predictive correlations between land use, watershed characteristics, runoff, and water quality. The result of these efforts is a very comprehensive data base for areas throughout the United States. Unfortunately, the data collection and analysis procedures utilized by each investigator were rarely uniform and consequently different sets of data have to be carefully processed to be comparable.

15. Diniz, E. V. 1980. Quantifying the effects of porous pavements on urban runoff. National Symposium on Urban Hydrology, Hydraulics, and Sediment Control, Lexington, KY., pp 63-70.

Abstract: Porous pavements have been suggested as a means to reduce volume of runoff as well as peak flows resulting from urbanization. The use of porous pavements allows for infiltration into the ground from paved areas which would otherwise be impermeable. Porous pavements can also be used to reduce the overload on existing storm sewers. The effects on runoff quantity and quality from porous pavements have been quantified by a modeling scheme which considers the pavement and subgrade as two hydraulically connected control volumes forming a single system. Inflow to the system are direct rainfall and an overland flow hydrograph from contributing impervious areas. Outflows from the system include vertical seepage, horizontal drainage, evaporation, and

surface flow in the case of a surcharged porous pavement. Model input requirements include physical dimensions of the porous pavement, rainfall intensities, and impermeabilities of the pavement, subgrade, and natural ground. A depth-storage function for the pavement and subgrade is also necessary. A comprehensive temporal accounting of flow and storage in each element of the system is output from the model.

16. Freund, A. P., and C. D. Johnson. 1980. Comparison and relationships of stormwater quality and basin characteristics: Madison, Wisconsin. International Symposium on Urban Storm Runoff, C. T. Haan, Editor, University of Kentucky, Lexington.

Abstract: The physical and hydrologic characteristics of three large and diverse urban catchments in Madison, Wisconsin are examined and relationships are developed between rainfall, discharge, sediment and nutrients using regression analysis. The developed relationships provide a basis for comparing the behavior of each basin as related to its physical characteristics. The results of a street debris sampling program undertaken in each of the monitored urban basins are reported, and implications for stormwater management are briefly explored.

17. Gburek, W. J., and J. B. Urban. 1980. Storm water detention and groundwater recharge using porous asphalt. Initial Results, International Symposium on Urban Storm Runoff, Lexington, KY.

Abstract: Data collected and observations made during the first 2 years of operation of the Willow Grove facility allow us to assess the potential of porous asphalt pavement for storm water detention and groundwater recharge. To date, the porous asphalt plot has produced no surface runoff from either high-intensity or long-duration rainstorms to which it has been subjected;

7.0 in/hr for 6 min (25-yr return period), and 0.37 in/hr for about 8 hr (5-yr return period), respectively. Generally, 70 to 90% of the rainfall appears as percolate below the plot on both the monthly and the individual storm basis, although commonly no percolate appears during individual events of up to about 0.3 in. Groundwater beneath the porous asphalt plot responds relatively rapidly to rainfall, usually within about 6 hours, and at the center of the plot it rises approximately 5 ft. per inch of rainfall. A very localized groundwater mound is formed by every storm that causes percolate to occur, but this mound forms and dissipates rapidly. Concentrations of both the critical inorganic and organic water quality parameters within the percolate leaving the porous asphalt plot are well below acceptable drinking water standards; the percolate seems to pose no groundwater contamination threat. Field testing of the strength of the porous asphalt plot showed that the plot as constructed is able to support light to moderate traffic. Observations during severe weather conditions indicate that the porous asphalt layer does not seem to be affected by freeze/thaw conditions, and remains relatively skid resistant during both wet and freezing weather. Finally, the initial results clearly show that groundwater is recharged under the porous asphalt plot throughout the year, whereas that under the adjacent grass cover plot is recharged little or not at all during the growing season.

18. Grigg, N. S., L. H. Botham, L. Rice, W. J. Shoemaker, and L. S. Tucker. 1976. Urban drainage and flood control projects: Economic, Legal and financial aspects. Hydrology Paper No. 85, Colorado State University, Fort Collins.

Abstract: Techniques for evaluating minor and major Urban Drainage and Flood Control (UDFC) Projects are described. Economic, political, engineering, financial and legal problems must be faced prior to implementation of proper levels of these projects. The measurement of tangible benefits is described while a literature review revealed no direct objective techniques for quantifying intangibles. However, some methods for establishing the relative rankings of intangible contributions show promise for improvement of evaluation techniques. The legal problem of establishing benefits is described and a copy of recent enacted Colorado legislation is included. Information on the estimation of flood damages and the selection of discount rates is presented for use by the analyst. Careful coordintion of land use and drainage control measures is stressed. Related recent legislation and regulations are included.

19. Grigg, N. S., A. M. Duda, and J. Morris. 1980. Stormwater management in coastal North Carolina. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 33-34.

Abstract: The objective of this paper is to provide an assessment of the magnitude of the stormwater runoff and flooding problem in the North Carolina coastal zone, to describe existing management programs, and to provide recommendations for improving stormwater management programs. The discussion will be based on the practical aspects of a fragmented and pluralistic approach to stormwater management in the coastal zone, including: three levels of government, several categories of problems, and the multiple objective approach to problem solutions. The State of North Carolina approaches the stormwater management problem realistically using the approach of seeking multiple objectives through multiple means. We cannot expect that new comprehensive stormwater management legislation or extensive new management programs will be approved by the General Assembly in the near future. The only altenative is to make mid-course corrections in our various existing programs at the local, state, and federal level to ensure that they work together and work more effectively.

20. Guy, H. P. 1978. Sediment management concepts in urban storm water system design. International Conference on Urban Storm Drainage. P. P. Helliwell, Editor, University of Southampton, England.

Abstract: Storm drainage systems can be designed which will greatly reduce peak rates of runoff and the amount of sediment and pollutants normally transported from urbanizing and urban areas to receiving water bodies. Reduction in peak flow rates reduces the potential for serious channel enlargement and additional sediment problems downstream from the development. Optimum design can be achieved through good land-use planning that is well coordinated with natural drainage. This in turn, will make it possible to minimize excavation and soil exposure during construction, and provide a maximum of individual and (or) community onsite storm water detention storage. The resulting storm drainage system would usually have a lower initial cost and result in a more esthetically pleasing neighborhood than generally exists with conventional designs; but, may cause loss of convenience and be more costly to maintain.

21. Hawkins, R. A., B. F. Maloy, and J. L. Pavon. 1979. Procedure for the establishment of statewide wasteland allocations. Stormwater Management Model Users' Group Meeting Proceedings, Project Officer, H. C. Torno, Washington, D.C.

Abstract: State regulatory agencies are faced with the problem of developing effluent limitations for industrial, semi-public and municipal dischargers which assure that water quality standards are met and that the economic burden of environmental protection is shared as equitably as possible among all dischargers to a stream segment. In the past, emphasis has been placed on the development of sound technical water quality modeling procedures and the calibration and verification of the water quality models. The purpose of this paper is to present the minimum treatment requirements which have been established for dischargers by the United States Environmental Protection Agency and a procedure which allows regulatory agencies to decide where limited resources for water quality sampling and field investigation should be placed in developing defensible waste load allocations.

22. Henry, J. G., and P. A. Ahern. 1976. The effect of storage on storm and combined sewers. Canada-Ontario Agceeement Research Program, Ottawa, Ontario, Canada.

Abstract: The effect of storage on storm and combined sewers has been investigated for a residential subdivision of approximately 100 acres area, under development in Southern Ontario. The storage methods examined were: (1) ponding of rainfall on flat roofs; (2) on-site storage of roof runoff; (3) storage of flows from street gutters at catch basins; and (4) holding reservoirs as part of the sewer network. A hydrograph model for estimating stormwater runoff from residential areas has been developed and incorporated in a computer program. The model was used to determine the outflow hydrograph at the storm sewer outfall from the subdivision for a synthetic two-year design storm. An alternate design, replacing the separate sanitary and storm sewers by combined sewers, was carried out and the outflow hydrograph from a combined sewer network was determined. The effects of different storage methods on the outflow hydrograph were tested for both the separate and combined systems. Substantial reductions in peak flow were found to occur as

the level of storage was increased, with holding reservoirs providing the greatest reductions. Cost data from the original separate system design were used to calibrate formulae for estimating sewer pipe costs. The total costs of systems incorporating various storage options were compared. A combined sewer system was found to be slightly less expensive than the separate system and small savings in subdivision sewer costs resulted when storage was incorporated.

23. Jackson, Thomas J., and Robert M. Ragan. "Hydrology of Porous Pavement Parking Lots,", Journal of the Hydraulics Division, ASCE, Vol. 200, No HY12, Proc. Paper 11010, December, 1974, pp. 1739-1752.

Abstract: Numerical solutions of the Boussinesq equation were used to examine the hydrologic behavior or porous pavement systems incorporating subdrains in open graded base courses placed on impermeable membranes. A series of numerical experiments showed that substantial control of the runoff hydrograph from parking lots could be obtained through the use of porous pavements. The numerical experiments conducted with synthetic design storms were used to develop equations and graphs for use by engineers designing porous pavement systems for runoff control.

24. Kamedulski, G. E., and R. H. McCuen. 1978. The effect of maintenance on storm water detention basin efficiency. Water Resources Bulletin, 14(4):1146-1152.

Abstract: Storm water detention is an effective and popular method for controlling the effects of increased urbanization and development. Detention basins are used to control both increases in flow rates and sedimentation. While numerous storm water management policies have been proposed, they most often fail to give adequate consideration to maintenance of the basin.

Sediment accumulation with time and the growth of grass and weeds in the emergency spillway are two maintenance problems. A model that was calibrated with data from a storm water detention basin in Montgomery County, Maryland, is used to evaluate the effect of maintenance on the efficiency of the detention basin. Sediment accumulation in the basin caused the peak reduction factor to decrease while it increased as vegetation growth in the emergency spillway increased. Thus, the detention basin will not function as intented in the design when the basin is not properly maintained. Thus, maintenance of detention basins should be one component of a comprehensive storm water management policy.

25. Kamedulski, G. E., and R. H. McCuen. 1979. Evaluation of alternative stormwater detention policies. American Society of Civil Engineers, Journal of Water Resources Planning and Management Division, 105(WR2): 171-186.

Abstract: Mathematical modeling is a quick, inexpensive and effective means of evaluating existing stormwater detention policies and examining alternative policies. The model was then calibrated with data collected from a watershed located in Montgomery County, Maryland. The data base included measured precipitation, and both sediment and flow into and from the detention basin. Model studies indicate that many existing SWM policies do not meet the intent of SWM because they ignore the volume-duration frequency concept that has long been used in hydrologic design. SWM policies often fail to specify a duration for detention basin design and limit the design frequency to a single return period. A detention that is designed using a policy based on a single return period will not limit the entire flood frequency curve to that of the before-development conditions, and thus the policies do not satisfy the intent of a SWM. Also, less costly designs are possible if both basin volume and outlet characteristics are considered simultaneously.

26. Krishnamurthi, N., and J. L. Balzer. 1978. Design-storm for sedimentation ponds. Environmental Quality Department Utah International Inc., San Francisco, California.

Abstract: The office of Surface Mining and Reclamation has adopted a 10-year, 24-hour precipitation event as the design storm for sedimentation ponds to limit sediment concentrations in effluents. The authors feel that the design-storm approach using the frequency analysis is seldom warranted often leading to overdesign and undue costs. This paper discussed the physical process of precipitation that causes runoff and how it should be considered in developing the criteria for the design of sedimentation ponds. The authors recommend that the Office of Surface Mining revise the design principles of sedimentation ponds both for environmental and economical reasons and that they increase emphasis on limiting the sediment volume flowing past a mining facility rather than limiting its concentration in the effluent. It is their hope that these recommendations could influence EPA effluent regulations in the future.

27. Krishnamurthi, N., and W. T. Lenocker. 1980. Comparison of HEC 1 and TR 20 Programs. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, N. Y., pp 92-98.

Abstract: Determination of peak flows in an ungaged watershed has been of interest to the practicing engineers due to the increased interest on flood plain management programs at the local, state and federal levels. For the past two decades, research institutions and academic universities have developed many rainfall-runoff simulation models for the ungaged watersheds

and some of the notable and commonly used models are: (1) Stanford Watershed Model, (2) HEC 1, (3) U.S.G.S.-Dawdy Model, (4) Tr 20, (5) USDAHL, and (6) ILLUDAS. While these models are well documented relative to the purpose of programs, identification of variables and explanation of input and output procedures, very little work is done on the comparision of these models. The objective of this paper is to compare two of the above-mentioned models, namely HEC 1 and TR 20, with respect to their accuracy and resources needed. HEC 1 program was developed by the Hydrologic Engineering Center of U.S. Army Corps of Engineers to handle flood hydrograph computations associated with precipitation and runoff on a complex, multisub-basin, multichannel river basin (1). TR 20 program was developed by the Soil Conservation of U.S. Department of Agriculture to compute surface runoff resulting from any synthetic or natural rainstorm (2). Detailed descriptions of these programs and of imput parameters are explained in their user's manuals (1, 2). Based on preliminary testing, the authors feel that if rainfall/runoff data are available, HEC 1 program is best suited to calibrate the input parameters of a rainfall/runoff simulation and to predict the peak flow of floods. However, Snyder and Clarks' coefficients for HEC 1 program are not readily available for many ungaged watersheds. For such watersheds TR 20 programs can be best utilized through the use of soil survey maps.

28. Lai, C. 1980. Urban storm sewer flows in coastal areas. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 244-254.

Abstract: Analyses and management of urban stormwater in coastal areas have become increasingly important subjects in recent years. Urban basins are generally small in size and generally consist of a large percentage of

impervious surface. Many sewers in coastal areas are characterized by small conduit slopes and by tide-affected flows. Additional features, such as rapid change of discharge due to subtropical rainstorms, are common to the sewers of southern coastal areas. Storm sewers in the Miami, Florida area exhibit a number of these flow characteristics typical of urbanized southern coastal areas. Accurate simulation of this class of flows remains a difficult subject, and general, rational and reliable deterministic methods for solving such flow problems are needed. The purpose of this papaer is to examine and explore methods for analyzing the transient flows of urban storm sewers in southern coastal areas. The study considers three general cases: a) full-pipe or closed conduit flow, b) free-surface or open-channel flow, and c) two-phase or mixed flow. For ease of formulating a workable approach, storm sewer configurations in Southeast Florida are used to develop the numerical solution technique. Numerical schemes for simulating these three types of low have been investigated, and a computer model for full-pipe flow has been applied to a storm sewer flow in South Miami, Florida. The simulated discharge hydrograph generally agrees well with the measured one; however, some refinement in low discharge analysis seems to be necessary. The use of rigid water column theory for full-pipe flow is adequate for most sewers. "Minor losses" contribute significantly to hydraulic lost in the flow simulation. A U-shaped flow control device may be used advantageously to obtain a point discharge in a sewer pipe for the open-channel flow regime. In fact, some U-shaped constrictions have been installed for this purpose in south Florida sewers by the U.S. Geological Survey.

29. Lakatos, D. F., and K. C. Wiswell. 1978. Computer simulation of storm drainage system: A case study based on the use of the Penn State runoff model. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 1-12.

Abstract: Successful management of urban water resources depends on the ability of urban planners and managers to predict accurately the effects that increased urban development will have on storm water runoff. The lack of simple methods for prediction of watershed response to storm events is a major factor contributing to increased urban flooding. The Penn State Runoff Model is a simple and concise storm water simulation program, developed for the purpose of analyzing the timing of subarea flow combinations and their effect on downstream flow rates. Information on the manner of combination of subarea flows provides the basis for evaluation of flood-control altenatives for the source of the flooding problem rather than the point of actual flooding. The Peak Flow Presentation Table, a major output of the model, lists the subwatershed flows that combine to form a flood flow downstream. The timing of peak flows from individual subareas determines the magnitude of aggregate flow downstream, which in turn determines the extent of flooding. The results of a storm drainage study illustrates the practicality of the Penn State Runoff Model in developing realistic control alternatives for storm-related flooding problems. This study involved development of detailed schematics of the storm drainage system and accumulation of flow and rainfall data required for use of the model. Evaluation of the various runoff-control alternatives revealed some situations where conventional control measures are required to alleviate the immediate problems. The planning and analysis had to be sufficiently comprehensive to insure that the control program developed for the Town did not place an unwarranted runoff burden on downstream properties.

30. Lockwood, G. 1980. New approach to storm drainage pipe design. Civil Engineering, pp. 59-61.

Abstract: Using a recently developed computer program, drainage swales can be designed to provide short-term storage of stormwater. Benefits of this approach to stormwater management include reduced sewer pipe size requirements, minimal pollution of streams and lakes from run-off, reduced flooding during peak storm periods and, if desired, a recharged aquifer.

31. Looper, R. D. 1980. Calibration and application of a watershed planning model. National Symposium on Urban Stormwater Management in Coastal Areas. C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 152-161.

Abstract: Accurate determination of the storm water runoff process is critical to the overall effectiveness of a stormwater drainage masterplan. For a stormwater drainage masterplan consisting of sixteen basins in Pinellas County, Florida, The Soil Conservation's TR-20 computer model was selected. This model was formulated to improve the quality of watershed project planning by providing a large degree of flexibility in developing input parameters and by quickly evaluating alternative drainage systems. However, in order to apply this model to a coastal area, calibration of various input parameters had to be accomplished. The TR-20 model utilizes synthetic unit hydrograph theory which use input parameters based on the physical characteristics of a watershed. Several of these parameters, such as rainfall, infiltration, soil moisture capacity and time of concentration required special attention due to the topography and hydrology of coastal areas. Verification of TR-20 parameters was accomplished through use of the continuous event model, STORM, and the single event model, HEC-1. This paper will present the calibration and application of the TR-20 watershed planning model to Alligator Creek Basin

in Pinellas County, Florida. Problems encountered with the HEC-1 model pertain mainly to the loss rate parameter relationship with STORM and TR-20. By reconstituting observed runoff events, HEC-1 must optimize loss rate parameters given estimates of starting values for input variables. These loss rate parameters are not directly comparable to those used in the SCS curve number runoff analysis. The principal difficulty in using the TR-20 model is encountered when trying to apply the model to develop basings. Although TR-20 provides a routing procedure for reservoirs and channels, closed conduit systems are not provided for. This limitation excludes TR-20 from being used in the analysis of s ered areas.

32. Manz, P. E. 1980. Development of a dynamic stormwater management system. National Symposium on Urban Stormwater Management in Coastal Areas. C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 163-172.

Abstract: Conventional stormwater drainage masterplans are generally developed to provide both a solution to existing flood problems and to provide adequate drainage for anticipated future development in a watershed. The evolution and selection of drainage improvements are based on Ultimate Land Use (ULS) conditions, as envisioned at the present time. Any changes to the ULU conditions of a drainage basin or revision of the recommended improvement to a channel in the basin can make the stormwater masterplan obsolete or inadequate. Extensive revisions and lengthy computations are required to update the masterplan and maintain it as an effective tool in solving drainage problems. A systems approach was used to develop dynamic stormwater drainage masterplans for Pinellas County, Florida. Computer programs were obtained or developed to digitize the basin, calibrate and develop the hydrology and model the hydraulic characteristics of the basin. The strength of a dynamic

masterplan lies in its flexibility after the final masterplan is submitted. The client is provided with program decks on the computer models used and copies of all input files. As land use patterns change, the masterplan can be updated and checked to see if the alternatives are still adequate. If the alternatives prove inadequate or other changes are proposed, they can be quickly evaluated and incorporated into the masterplan. In this way the client always has a current masterplan that can be an effective tool for use in urban planning. The advantages of dynamic stormwater masterplans include: 1. The ability to maintain a current masterplan by incorporating changes in the land use pattern. 2. The ability to quickly evaluate proposed changes to the masterplan at a specific location. 3. The ability to quickly evaluate downstream responses to changes in the masterplan. 4. The ability to obtain water quality indications of the proposed changes.

33. Mariles, O. A., J. L. S. Bribiesca, and R. D. Mora. 1978. A numerical procedure to design drainage networks based on the hydraulic and storage capacities of the pipes. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 333-346.

Abstract: A numerical procedure based on finite difference computations has been developed by the authors to calculate, at any time and at any site of a given network, the hydraulic parameters, i. e., velocity and depth. By this way and taking into account the storage at registers and manholes, it is possible to simulate the whole work of the network and to know the volume of storm water that is not possible to manage inside the pipes. Since such a volume will produce a flood and cause a certain damage, it is possible to associate the damage for several storms with different return periods with the expected damage for a certain design. In such way the expected damage

associated to each design can be known. If the benefit of a design is assumed to be the expected avoided damage at actual conditions and with a certain design, two graphs can be drawn; the first one will be the cost of the design against its capacity and the second will be the benefit associated against its capacity. The selection of the right design becomes obvious.

34. Mattraw, H. C. Jr., J. Hardee, and R. A. Miller. 1978. Urban stormwater runoff data for a residential area, Pompana Beach, Florida. U.S. Geological Survey Open-file Report 78-324, Tallahassee, Florida, 108 pp.

Abstract: The U.S. Geological Survey has measured rainfall, runoff, and runoff quality for three urban sewered basins in Broward County, Florida. Approximately 2 years of records have been collected for a 47-acre single-family residential area; a 58-acre, 3,000 foot secondary divided-highway segment; and a 28-acre commercial shopping center. The three homogeneous areas were gaged with an automatic, integrated instrumentation package or urban hydrology monitor. The urban hydrology monitor collects and synchronously records 36-second interval rainfall and water-level information The collection time of 24 water-quality samples is also recorded. Discharge is computed from the difference in stage through a U-shaped construction (Venturi flume) placed in a sewer pipe. Approximately 100 rainfall-runoff periods per site were digitized from analog records and stored in an urban data management system. Sets of nutrient and heavy-metal water-quality data were collected for 30 or more storms at each of the three areas and stored in the system. Loads computed for the three areas indicate the importance of the hydraulic interconnection between impervious areas. Factors which affect storm-water runoff loads include landuse, proportion of hydraulically interconnected impervious area, seasonal distribution of rainfall, and the

antecedent dry period. The variability within a basin is great, indicating the need for systematic collection of runoff-quality information for a variety of conditions prior to any satisfactory calibration of currently available models.

35. McCuen, R. H. 1980. Water quality trap efficiency of stormwater management basins. Water Resources Bulletin, 16(1):15-21.

Abstract: While the quality of rivers has received much attention, the degradation of small streams in upland areas of watersheds has only recently been recognized as a major problem. A major cause of the problem is increases in nonpoint source pollution that accompany urban expansion. A case study is used to examine the potential for storm water detention as a means of controlling water quality in streams frequently used to control increases in discharge rates, can also be used to reduce the level of pollutants in inflow to receiving streams. Data collected on a 148-acre site in Maryland shows that a detention basin can trap as much as 98 percent of the pollutant in the inflow. For the 11 water quality parameters, most showed reductions of at least 60 percent, depending on storm characteristics.

36. McCuen, R. H. 1979. Downstream effects of stormwater management basins. American Society of Civil Engineers, Journal of Hydraulics Division, 105(HY11): pp 1343-1356.

Abstract: Urbanization decreases the natural storage of a watershed, which changes the timing characteristics of the runoff. Stormwater management (SWM) basins are an attempt to put the storage lost through development back into the runoff process. While SWM basins provide the proper volume of storage, they fail to return the timing characteristics to those that existed prior to development. The changes in storage and timing characteristics may have adverse effects on flow rates and bedload transport in downstream channel reaches. A study of a 2.12-sq mile watershed in Montgomery County, Md., showed that a SWM basin increased both peak flows and bedload transport rates in the channel downstream from the facility. This results from both the change in timing characteristics and the increased duration of bankfull flow. These results indicate that SWM policies must require evaluation of SWM plans on a regional basis and not just using on-site control criteria. Also, methods that can evaluate storage and timing changes must be used in the design of SWM facilities.

37. McCuen, R. H. 1979. Stormwater management policy and design. Journal of Civil Engineering Design, 1(1): 21-42.

<u>Abstract</u>: Stormwater management is recognized as a requisite to controlling storm runoff from developing areas. To best meet societal needs, numerous policies have been adopted to meet the intent of stormwater management. Unfortunately, many policies are deficient in their failure both to lead to designs that meet the intent of stormwater management and to provide the proper guidelines for translating policy into a design that provides the maximum benefit to society. Specific deficiencies of many policies include: 1) the use of a single frequency; 2) neglect of storm duration; 3) inadequate consideration of maintenance; 4) insensitivity to the importance of soil characteristics; 5) lack of recognition of differences between water quality and quality control; and 6) lack of consideration of downstream effects of detention storage. The effect of these policy deficiencies were evaluated. Different hydrologic methods are being used to translate policy into design, including graphical, empirical, unit hydrograph, and conceptual models. These methods provide different designs and differ on other important criteria,

including accuracy, training requirements, design cost, total cost, and their sensitivity to policy components. The different hydrologic methods are compared using these criteria.

38. Moodie, A. R., J. D. Scholes, and D. G. Thompson. 1978. Design of a stormwater quality and quantity monitoring system for an urban catchment - An Australian case study. International Symposium on Urban Stormwater Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 135-142.

Abstract: The results of a preliminary investigation into stormwater runoff quality from an urban catchment in Melbourne, Australia, are presented and discussed. Similarities with the results obtained by workers in the USA are noted. The results of the preliminary investigation are shown to be useful in designing and operating a stormwater quality and quantity monitoring system for the catchment. Aspects of the design of this system, such as the selection of sub-catchments for instrumentation and the design of a device for constant proportional depth sampling in a large Parshall flume, are also outlined.

39. Myers, V. A., and F. P. Ho. 1980. Coastal tide frequency, Virginia to Delaware. National Symposium on Urban Stormwater Management in Coastal Areas. C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 74-82.

Abstract: More than 2000 miles of United States coast are at a risk from temporary inudations by storm-driven sea water. Judicious decision on the part of the coastal zone that shall be occupied and on rational use of this occupied part, on lessening risk to life and property by structural means, on routes and refuges that will be needed for evacuation when a major storm threatens, and on flood damage insurance rates, requires knowledge--in probability terms--of the elevation and lateral extent of the space that

storm-driven sea water is likely to occupy. The behavior of coastal storms is ultimately assessed from past experience. The longer the return period of flood heights to be determined, the larger the experience sample required. Local experience with hurricanes does not necessarily reveal the true risk. In this study, "experience" is enhanced by applying all the data of a region to each coastal point, by deriving data needed (storm tides) from data available (meteorlogical parameters), by computation with tested models, and by developing hypothetical storms that collectively represent the full range of possibilities. This summary paper illustrates this approach to coastal tide frequency assessment, using the coast from the Virginia-North Carolina border to Delaware Bay ("Delmarva" coast) as the example.

40. Nawrocki, M., and J. Pietrzak. 1976. Methods to control fine-grained sediments resulting from construction activity. Hittman Associates, Inc., Columbia, Maryland.

Abstract: This publication is the third in a series issued under Section 304(e)(2)(C) of Public Law 92-500 concerning the control of water pollution from construction activity. This document was prepared for use by planners, engineers, resource managers, and others who may become involved in programs to effectively provide for sediment control. Standard erosion and sediment control measures are usually effective for preventing the runoff of the total sediment load. The effectiveness of these standard techniques, however, has been found to be relatively poor with regard to preventing the runoff of the fine-grained fractions, such as the silts and clays. The objective of this study was to research practical, cost-effective methods that would help to reduce specifically the fine-grained sediment pollution derived from construction activities. The prime consideration during this study was the use or adaptation of existing technology, as described in the current literature or data, to the fine-grained sediment pollution problem.

41. Novitzki, R. P. 1978. Hydrology of Wetlands in Wisconsin. U.S. Department of the Interior Geological Survey, Water Resources Division, Madison, Wisconsin.

Abstract: Interest in the role of wetlands in the hydrologic system has intensified in response to increased environmental concern. However, little information is available concerning those hydrologic factors that influence wetlands and conversely, the effects of wetlands on the hydrologic system. To manage wetlands as a significant element in the total environment will require a better understanding of their function in the hydrologic system and their effect on the quality of water moving through them. The initial purpose will be to classify wetlands according to their function in the hydrologic system. In particular, the effect of the wetland on runoff, sedimentation, and chemical characteristics of lakes or streams and the wetland's relation to water levels and water quality in the ground-water system will be defined. Several wetland areas, each representative of one wetland category, will be studied in detail. The hydrologic system near each wetland will be defined to determine direction of water movement, both vertical and horizontal, to determine areas of recharge and discharge, to determine the function of the wetland in the local system, and to define the influence of the wetland on flood flows and low flows and sedimentation in associated streams in the local system. The chemical nature of water entering, within, and leaving the wetlands will be examined to determine the effect of the wetland on water quality.

42. Novitzki, R. P. 1977. Hydrology of the Nevin fresh-meadow wetlands. U.S. Department of the Interior, Geological Survey, Madison, Wisconsin.

Abstract: Wetlands have been vanishing from the Wisconsin landscape at a rapid rate. Approximately half of the original 5,000,000 acres have been drained or affected by drainage. The fresh-meadow wetlands have been vanishing most rapidly because of agricultural and urban pressures. To preserve wetlands in the future, the Wisconsin Department of Natural Resources needs the ability to evaluate the tangible values of wetland areas. With such an evaluation, they (DNR) may be able to predict the type and magnitude of disturbance that will be caused by proposed wetland changes. This project will attempt to define the hydrologic system that presently maintains the nevin wetland and to monitor and explain changes which may occur. The water quantity and quality entering and leaving the area will be evaluated with respect to the influence of land-use changes, drainage, nutrients, sediment, municipal and local pumpage, and the effects of the hatchery. Data collections will include spring flows into wetlands, streamflow leaving wetlands, water level, precipitation and evaporation. Water samples for analysis of sediment, pesticides, and total organics will be collected from water entering and leaving welands and from ground water in shallow aquifers in the study area. Inventory of hydrology and geology influencing wetlands, and analysis of quantity and quality of water moving through wetlands and its related drainage basin will be made. Changes in quantity and quality detected by monitoring will be useful in evaluating those factors which may be affecting the wetlands.

43. Patrick, W. H. 1981. Effect of man's alterations on wetland and estuarine chemistry. Agricultural and Mechanical College, School of Agriculture, Louisiana State University, Baton Rouge.

Abstract: Objectives are (1) to determine if specific lakes of Barataria Basin are a source or sink for nitrogen and phosphorus; (2) to characterize sediment profiles and sediment water interfaces in shallow bays, lakes, and channels; (3) to determine the significance of sedimentation throughout Barataria Basin in relation to nitrogen, phosphorus, and carbon chemistry; (4) to determine the importance of mineral components of organic marsh soils in neutralizaing plant toxins such as sulfide; and (5) to continue studying nitrogen reactions in wetland systems; nitrogen fixation in water-sediment and plant-soil systems using labelled nitrogen, and denitrification in natural systems. The information obtained from this study will be used by public agencies concerned with management of coastal lands and resources, in problems dealing with eutrophication, agricultural runoff, wastewater management, water quality, primary production, etc., of Louisiana's coastal zone. Identified benefits to date are (1) a trophic state index based on cluster analysis and factor analysis of several aquatic parameters has been developed; (2) a study has examined the relationship between chemical and physical properties of marsh soils and the intriguing variations in plant growth; (3) demonstrated the significance of sediment transport to marsh accretion and nutrient cycling; and (4) develop a better understanding of the nitrogen cycle in a Spartina alterniflora salt marsh.

44. Pennel, A. B. 1980. Retention/detention basins in coastal areas. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 299-302.

Abstract: Stormwater management is an integral part of urban planning and development. Urban growth translates directly to an increase in the precent of impervious surfaces that generate higher runoff rates and columes over a shorter time period than previously generated from the undeveloped condition. The traditional role of stormwater management was to remove the additional water from the developed site as rapidly and efficiently as possible to minimize the flood hazard and inconvenience. Continued growth and development, completing with a renewed awareness of environment issues and concerns, has led to a re-evaluation of stormwater management practices. Present stormwater management practices are moving to multiple objective concepts. These generally include the traditional flood protection along with the preservation or maintenance of the existing flow quality and quanitity. Use of the retention/detention basin system serves as one of the basic tools in achieving these objectives. The practicality of retention/detention basins in coastal areas is a function of the optimization of design parameters relative to the value served. The design parameters of particular influence in this regard are groundwater table elevations and tailwater stages. Utilization of a model that accurately accounts for these parameters is an economical and timely manner is paramount to fulfilling the obligations and goals of stormwater management in coastal areas.

45. Peterson, J. O., G. Bubenzer and F. Madison. 1977. Evaluation of the implementation of the White Clay Lake protection and management plan. School of Agriculture and Life Sciences, University of Wisconsin, Madison.

Abstract: Implementation of the plan to provide protection of the water quality of White Clay Lake presents an opportunity to assess the effectiveness of protecting water quality by applying simple sediment and nutrient control

measures in an agricultural watershed. Techniques such as clear-water diversion, terracing, contour stripping, better manure handling systems, and stream bank protection are useful soil and water conservation practices, but institution of regulations to require application of such techniques on a broad basis should be preceded by an evaluation of their effectiveness for water quality protection. Major changes in water quality resulting from implementation of conservation practices will be most easily noted in the incoming streams. Continuous monitoring of two streams which drain about 45% of the watershed provides two years of background data on water flows, nitrogen, phosphorus, chloride and sediment transport. Evaluation of the management plan implementation involves a continuation of current lake and watershed monitoring activities plus some additional work using sediment cores to delineate the history of lake infilling. Additionally, evaluation of the effect which marsh areas have on the quality of water entering the lake will help guide judgments about the preservation of these wetlands for protection of lake water quality.

46. Pitt, R. 1978. The potential of street cleaning in reducing nonpoint pollution. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 289-301.

<u>Abstract</u>: This paper briefly describes the conclusions available at this time from an EPA-sponsored demonstration study of nonpoint pollution abatement through improved street cleaning practices. An important aspect of the study was development of sampling procedures to test street cleaning equipment performance in real-world conditions. These procedures and the experiment design are described in detail in the interim report. This paper summarizes accumulation rate characreristics of street dirt in the area studied. The

results of performance tests for street cleaning equipment and the factors that are thought to affect this performance are also summarized. These data are used to draw preliminary conclusions about elements that must be considered in designing an effective street cleaning program. The study of urban runoff yielded information on overall flow characteristics, concentrations and total mass yields of monitored pollutants in the runoff, and street dirt removal capabilities and effects on deposition in the sewerage for various kinds of storms. These data are summarized here, and urban runoff water quality is compared with recommended water quality criteria and the quality of sanitary wastewater effluent. Costs and labor effectiveness of street cleaning, runoff treatment, and combined runoff and wastewater treatment are also compared, and preliminary results from a study of airborne dust losses from street surfaces are summarized.

47. Potts, M. A. 1978. Erosion and sediment control design. Watershed Permits Section, Maryland Water Resources Administration, Annapolis, Maryland.

Abstract: Since soil erosion and sedimentation by water are complex processes, a better understanding of them provides a sound basis for developing improved prediction and control methods for urban areas. The Universal Soil Loss Equation is an important technique for evaluating urban erosion rates, and various conservation practices are effective for controlling urban erosion and sedimentation. Design of control plans requires the selection of appropriate control practices. These topics are discussed and illustrated as they relate to solving urban problems in erosion and sedimentation.

48. Pitt, R. A., and R. L. Johnson. 1978. On-site stormwater retention. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 91-94.

Abstract: Many urban areas have tried various stormwater detention and controlled release schemes to decrease peak flow rates for many reasons, including lower costs for smaller storm sewers and decreased downstream flooding. However, detention schemes have some disadvantages, which include land requirements and safety problems. An alternate method for reducing storm sewer requirements is to use retention, and reroute part of the flow that would ordinarily be urban runoff into the soil for subsequent percolation into the ground water supply. Onsite retention methods can provide the same advantages of peak flow reduction as detention without the extra land requirements. Most stormwater runoff in developed areas is from impervious cover. Reduction of runoff, both volume and rate, can be accomplished by retaining a portion of the runoff from roofs using porous media "french drains." Modifications were made to the Runoff Block of the Storm Water Management Model (SWMM) from the Environmental Protection Agency. The computer program was changed to allow a fraction of the runoff from impervious surface to be immediately directed to onside retention units of specified volume. Overflow from the retention units during extreme storm periods was onto adjacent pervious surfaces. Using a hypothetical residential drainage area, retention volumes of 500 ft^3/acre and 1000 ft^3/acre were studied. The fraction of impervious area was set at 0.30 and one-half of this area drained into the retention storage. The modified SWMM computer program showed that for a 5-year, 1-hour storm, the total runoff volume with no retention was 363 ft^3 which was reduced to 256 ft^3 and 218 ft for 500 ft^3/acre and 1000 ft^3/acre of onsite retention, respectively. For the same storm event, the sum of the peak runoff rates of 132 cfs with no retention was reducted to 95 cfs with 500 ft^3/acre and 79 cfs with 1000 ft^3/acre. Reduction of this magnitude could have a large impact on stormwater management strategies.

49. Reiter, F., and C. Fox, Jr. 1976. Evaluation of the cost-effectiveness of non-structural pollution controls: A manual for water quality management planning. CONSAD Research Corporation, Pittsburgh, Pennsylvania.

Abstract: The report presents a methodology for evaluating the economic costs of nonstructural pollution control techniques such as land use control. Used instead or as a complement to structural pollution controls, nonstructural alternatives can increase the effectiveness and/or decrease the costs of achieving environmental quality objectives. Included in the report are chapters on legal feasibility, social impacts, and alternative compensatory mechanisms for nonstructural controls.

50. Schluchter, S., and M. Teubner. 1980. Coastal flooding of tidal estuaries. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 275-278.

Abstract: At the present time, several different two-dimensional surge models are being used by Federal and private agencies to predict the magnitude of hurricane surges along the open coast; however, surge levels in estuaries can vary significantly from open coast values. Flood levels in these estuaries can be accurately and economically determined using a one-dimensional unsteady flow model. This paper considers such a model which includes both dynamic and storage effects associated with the channel flow and the tidal flats, as well as including the effects of multiple-connected river junctions. An area along the Texas coast has been selected to demonstrate the model's capabilities.

In this paper, a one-dimensional unsteady flow model has been briefly described. Some of the features of the model include:

(1) its capability of being able to handle a large number of rivers, tributaries and canals at one time;

(2) its adaptation to using more accurately determined input data involving cross-sectional parameters;

(3) its ability to use predetermined elevation hydrographs, either from an associated two-dimensional model or from measured data, as input elevations.

Unfortunately, both the original one-dimensional model and the combined model suffer from one main problem: insufficient surge data for calibration and verification. It is relatively easy to recreate existing tidal conditions, but to accurately determine storm surge elevations, more detailed data is required.

51. Simons, D. B., R. M. Li, and T. J. Ward. 1978. Estimation of sediment yield for a proposed urban roadway design. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 309-315.

Abstract: Roadways are one of the primary sources of sediment in urban environments. Soil erosion and the resultant sediment yield are important problems during construction and post-construction periods associated with the development of transportation road surfaces. Increased soil erosion and sediment yield often result from scarification of the natural terrain and changes in drainage patterns caused by urban road construction. The increased sediment loading to streams and drainages from roadway construction affects water quality, riparian and aquatic ecosystems, and flooding potential due to

aggradation of channels. Techniques have been developed for estimating erosion and sediment yield from roadways. One approach used complex computer simulation of physical processes that determine erosion and sediment yield. Although a useful technique, complex computer simulation is often difficult to comprehend for many field users. To avoid problems associated with the complex approach, a simpler and more readily understood physical process computer simulation model has been developed. This simplified model is based on the same physical processes as the complex model but integrates and averages the processes to develop a physically realistic solution. In this paper, the simplified version of a roadway sediment yield model is presented along with applications to selected roadway designs. Using this model and available roadway data, a user can quickly estimate sediment yield from a proposed roadway design. Roadway components considered in the model include the road surface, cut and fill slopes, drainage ditches and culverts. Impact of design alternatives such as a roadway cross section, road gradient, road surfacing, and size and spacing of cross drains can be determined by the model. Use of the simplified model in road design can alert the urban planners and roadway design engineers to potential erosion and sediment problems and help the designers avoid such problems.

52. Slyfield, R. E. 1978. Management of urban runoff by remote sensing in south Florida. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 235-244.

Abstract: A continuous urban area 100 miles long by 10 miles wide borders the south east coast of Florida. Because of the unique character of this area - very flat topography, average elevation of about 5 feet above sea level, and very intensive tropical rainfall - immediate water management decisions are

required. Consequently, the South Florida Water Management District which operates the areas's primary water management facilities is constructing a remote sensing and control system. This system transmits vital hydrometeorological data via a terrestial telemetry system to a control center where management decisions are made. Operational directions are then transmitted over the system to control facilities and executed remotely. Simultaneously these actions are monitored at the control center. Because the system's main function is to perform during periods of high stress, design requirements have been adapted to assure that it will be operational when needed most. Various innovative features are described which increase reliability. The system consists of a microwave backbone in a loop configuration and capable of fully duplex transmission under computer control. Remote acquisition and control units (RACU's) contain environmental sensors and control elements which operate water control facilites. RRACU's also contain transceivers powered either by commercial or solar power. Many sensors are of unique design, and high accuracy requirements have been verified.

53. Smolenyak, K. J. 1980. Stormwater runoff modeling of a planned community. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York, pp 378-387.

<u>Abstract</u>: The Tampa Palms development, which is located approximately eleven miles northeast of downtown Tampa, is a residentially oriented master planned community totaling 5,400 acres in size with an ultimate density of 13,000 units. Stormwater discharges from this property will enter the Hillsborough River, a potable source of water for the City of Tampa. In order to assess the effects of the development of the Tampa Palms Property on the quantity and

quality of stormwater runoff entering the receiving systems, the Storage Treatment Overflow Runoff Model (STORM) was employed. STORM, a widely used and proven hydrologic model (continuous simulation), was initially applied to the project in its current undeveloped condition, using water quantity and quality data collected from several monitoring stations strategically located at surface runoff sites throughout the property for model calibration. The proposed changes in land use and drainage characteristics due to development were then incorporated into the model to predict future stormwater runoff quantity and quality conditions and to analyze potential impacts to receiving systems. The net impacts of stormwater discharges from the Tampa Palms property on the Hillsborough River should be minimal, because significant treatment of stormwater runoff will be achieved in the Tampa Palms drainage system and the pollutant loads (mass) from the developed property will only constitute a small percentage of the loads carried by the river. For all the pollutants modeled, total loads for the post-development basins represent a minor to very minor percentage of the total loads for the Hillsborough River. In addition, stormwater discharges from the developed drainage system will be attenuated (via controlled release) which will increase the ability of the Hillsborough River to assimilate the loads.

54. Ward, A. J., C. T. Haan, and B. J. Barfield. 1977. Simulation of the sedimentology of sediment detention-basins. University of Kentucky Water Resources Research Institute, Lexington.

Abstract: Sediment detention basins are a widely used means of controlling downstream sediment pollution resulting from stripmining and construction activities. A mathematical model for decribing the sedimentation characteristics of detention basins has been developed. This model requires as inputs the inflow hydrograph, inflow sediment graph, sediment particle size distribution, detention basin stage-area relationship and detention basin stage-discharge relationship. Based on this information the model routes the water and sediment through the basin. In this routing process the outflow sediment concentration graph, the pattern of sediment deposition in the basin and the sediment trapping efficiency are estimated. Comparison of predicted results with measured sediment basin performance indicates the model accurately represents the sedimentation process in detention basins. This report details the model, illustrates its use in design, explains how to process the model on a digital computer and presents a program listing of the model.

55. Weber, W. G. Jr., and C. Wilson. 1976. Evaluation of sediment control dams. Pennsylvania Department of Transportation, Bureau of Materials, Testing and Research, Harrisburg, Pennsylvania.

Abstract: Small dams in waterways have been extensively used by PennDOT in erosion control. The evaluation of the performance of these small dams was conducted to determine how well they performed in removing sediment from flowing water. Also the various methods of constructing these dams were compared. The results indicate that only minor amounts of suspended sediment are removed from the flowing water. However, the bottom transport is trapped by these dams. The rock dams generally were stable.

56. Widseth, R. A. 1978. Utilizing a storm sewer outlet. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 317-322.

Abstract: Crookston is located in Northwestern Minnesota. The University of Minnesota operates a college and an agricultural experiment station north of the City. In 1970 the University installed an 11,000 foot long storm sewer

outlet to drain the campus area. This outlet crossed undeveloped University-owned farmland along its route, but was not sized to serve this property because a need was not anticipated. Now 100 acres of this land has been sold for development and requires storm sewer service. By the use of a storm water detention basin and a controlled outlet this development can be drained by the existing storm sewer for approximately one-seventh the cost of constructing a new outlet.

57. Willison, J., et al. 1977. Preventive approaches to stormwater management. U. S. Environmental Protection Agency, NPS Branch (WH-554), Washington, D. C.

Abstract: This document was prepared for use by local agency administrators, and others who may be involved in programs to abate pollution from urban runoff. The concept of source control (BMP) has been discussed in the past and this report lists many techniques that would be included in a Best Management Practice. The problems associated with implementing these practices, legal, financial, and institutional are also discussed. The objective of this study was to provide a basic understanding to local administrators on what BMP is and those techniques which would comprise a BMP.

58. Woodward, D. E., P. I. Welle, and H. F. Moody. 1980. Coastal plains unit hydrograph studies. National Symposium on Urban Stormwater Management in Coastal Areas, C. Y. Kuo, Editor, American Society of Civil Engineers, New York pp 99-107.

Abstract: A hydrologic study initiated in 1975 indicated that the Soil Conservation Service (SCS) was not able to simulate recorded flood hydrographs or regional peakflow frequency curves on the Delmarva Peninsula in Delaware, Maryland, and Virginia. The standard SCS unit hydrograph was used in these detailed hydrologic studies. The peak rate equation for this standard SCS unit hydrograph can be expressed as $q_p = 484QA/T_p$ where 484 is a shape and units conversion factor, q_p is the peak discharge in cfs, Q is the volume of runoff in inches, A is the drainage area in square miles, and T_p is the time to peak in hours.(3) The 484 factor is referred to in this paper as a dimensionless unit hydrograph peak factor (DUPE). This paper describes the studies made to develop a dimensionless unit hydrograph for the Delmarva Peninsula. The standard SCS unit hydrograph was developed from small agricultural watersheds primarily in the Midwest. These watersheds are generally characterized by local relief of 50 to 100 feet with little or no natural storage. Since physical characteristics in the Midwest differ from those in the Delmarva Peninsula, it was felt necessary to develop a unit hydrograph specifically for the study area. The basic shape of the new hydrograph appears rational because when compared to the standard SCS unit hydrograph, the recession limb contains more volume and the peak rate is lower. Analysis of the August 3, 1967 storm on Murderkill River was used as a check on the reasonableness of the selected unit hydrograph.

59. Wu, J. S., and R. C. Ahlert. 1978. Prediction and analysis of stormwater pollution. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 183-188.

Abstract: In recent years, pollution from nonpoint sources has become an increasingly important consideration in water quality planning and management. Stormwater pollution can be characterized, in magnitude and in concentration of pollutants, as intermittent and impulse-type discharges into receiving waters, causing shock-loading problems to the ecosystems of these water bodies. The classical approach, using critical low flow as the design criterion for

water quality management schemes, must include the effect of storm runoff. This paper aims to summarize state-of-the-art methodologies for predicting storm runoff loads. Four categories of prediction methods are presented; these include zero-order, rational, statistical and descriptive methods, for achieving different levels of prediction, i.e. (i) average annual storm load, (ii) storm load per event and (iii) storm load distributions within events. Also, it is intended to present the authors' ongoing efforts to develop an analytical method for assessing stormwater impacts, with regard to biochemical oxygen demand (BOD) and sediment, on receiving water quality. The receiving water body is represented by a completely mixed reactor and a settling basin, during storm runoff periods. Surface runoff and pollutant loads enter the receiving stream at an average uniform rate over each time interval, i.e. a step-wise, steady-state approach to input variables. Deposition and/or resuspension of sediment are limited by the sediment transport capacity of the stream. Model equations and analytical solutions will be presented, with the Assunpink Basin of New Jersey as the illustrative example.

60. Wycoff, R. L. 1978. Computer-aided hydraulic design of storm drains. International Symposium on Urban Storm Water Management, C. T. Haan, Editor, University of Kentucky, Lexington, pp 325-321.

Abstract: Conventional hydraulic design of closed conduit storm drains usually consists of selecting conduit sizes based on open channel flow computations. Conventional design of highway culverts, on the other hand, makes use of the available static energy head at the culvert site. A computer-aided design procedure is presented whereby the hydraulic equations of culvert flow are utilized to size branching, closed conduit, storm drainage networks. By this method, the available static energy head is utilized as

efficiently as possible. The design algorithm selects a pipe size array which will carry the design flows without overtopping, while minimizing the largest conduit size selected. The pipe size array obtained by application of the computer-aided design procedure to an example network is compared to the pipe size array obtained by open channel flow design procedures. In general, the computer-selected sizes are smaller than the open channel sizes, resulting in construction cost savings.

61. Wycoff, R. L., and V. P. Singh. 1976. Preliminary hydrologic design of small flood detention reservoirs. Water Resources Bulletin, 12(2): 337-349.

Abstract: Flood detention reservoir design is a common problem encountered by engineers and others involved with water resources problems. This paper presents a method by which the volume of flood storage required for a single reservoir or a series of reservoirs may be estimated without using numeric flood routing techniques. An idealized model based on triangular hydrographs was developed to define the major relationship among the variables. A generalized equation was then obtained by multiple linear regression analysis of computed flood-routing data. The effect of storage distribution on flood peak reduction efficiency was then investigated by means of a computer model study of equal-sized reservoirs located in series. This resulted in a relationship between number of reservoirs and peak reduction efficiency and, together with the generalized storage volume equation, form a proposed procedure for the preliminary hydrologic design of small flood detention reservoirs.

62. Wycoff, R. L., J. E. Scholl, and S. Kissoon. 1979. 1978 needs survey - Cost methodology for control of combined sewer overflow and stormwater discharges. U.S. Environmental Protection Agency, Municipal Construction Division, Office of Water Program Operations, Washington, D.C.
Abstract: The major objective of this project is to develop updated nation-wide cost estimates for control of pollution from combined sewer overflow and urban stormwater runoff on a State-by-State basis. A secondary objective is to establish the National Combined Sewer System Data File. This file contains information on every known combined sewer system in the nation, including location, sewer system and receiving water characteristics, and the status of current Combined Sewer Overflow (CSO) planning.

APPENDIX B

SUMMARY OF STORMWATER MANAGEMENT METHODS AND THEIR APPLICABILITY IN COASTAL WETLANDS

Richard H. McCuen

Stanley L. Wong

SUMMARY OF STORMWATER MANAGEMENT METHODS AND THEIR APPLICABILITY IN COASTAL WETLANDS

Engineering projects, if not properly designed, can have serious physical impacts on wetlands. Changes in the characteristics of surface and subsurface flows may cause major biological changes and changes in the characteristics of the existing ecosystem. Projects may also interfer with the existing characteristics of tidal flows, which would have physical, as well as biochemical effects. In addition to projects located outside of wetlands, but affecting wetlands, engineering projects within wetlands may change the physical, chemical, and biological characteristics of wetlands. Physical impacts of specific interest are:

- changes in the mean water level of the wetland
- changes in local water table levels
- changes in periodic variations of both surface flows and tidal flows
- changes in circulation and salinity patterns

Because of the potential damage to wetlands that may result from these changes every engineering project should include an assessment of the physical impacts of construction and land use change and the identification of methods for mitigating these changes.

Changes in Mean Water Level

Engineering projects may involve damming effects and increases in drainageways. If water is retained outside of a wetland area, the mean water level within the wetland may decrease. Damming within the wetland may increase the mean water level. Increased drainage within the wetland area may decrease the mean water level. Changes in the water level indicate changes in the storage of the wetland.

Changes of the mean water level may alter the spatial arrangement of plant species. Additionally, changes in the size of the wetland, and therefore the storage, may affect the ability of the wetland to provide downstream flood protection. Also, the reproduction characteristics of aquatic species may change as the water level and storage change.

Assessment of the impact of engineering projects would require water level information both before, after, and during construction. Measurements should be made over a sufficient time frame so that an accurate estimate of mean water level can be made.

Changes in Local Water Table Levels

Wetlands are characterized by high water tables, which intercept surface water levels where dictated by local topography. Wetland flora may be quite sensitive to permanent changes in local water table levels. Therefore, the piezometric surface should be identified prior to construction and should be monitored both during and after construction. Changes in the water table can be minimized through proper drainage.

Changes in Periodic Variations in Water Levels

In addition to the mean water level of both surface and subsurface waters, the variation in water levels will affect the flora and fauna of wetlands. Periodic variations may result from both daily and seasonal causes. Tidal patterns will require measurements throughout the day to measure differences in flood and ebb tide stages, as well as measurements over the lunar month to measure spring and neap tide stages. In addition to tidal fluctuations in coastal wetlands, seasonal periodicites must be assessed. These periodic variations in water levels affect the primary productivity of the wetland and the reproduction cycles of flora and fauna. Many wetland plants depend on periods of inundation as well as periods for drying out; these periods are instrumental in maintaining the supply of necessary nutrients. Measurements may be taken using either stage recorders or continuous measurement devices.

Changes in Circulation and Salinity Patterns

Circulatory patterns within a wetland are an important factor in determining the distribution of nutrients and dissolved gases. The salinity gradient in coastal wetlands is affected by the circulatory patterns. It is important to chart the circulatory patterns within a wetland because the flora and fauna have different requirements for nutrients and tolerances to pollutants.

Changes in the existing mixing of fresh and salt water can cause significant changes in flora and fauna in a coastal wetland. Nontoxic dyes and periodic measurements can be used to identify circulatory patterns. Seasonal variations in the patterns should be assessed before construction and monitoring should take place during construction. Deviation from normal patterns should be minimized through stormwater management.

Compilation of Site Information

The first step in runoff and sediment control is to prepare a control plan, which should include the identification of local requirements, the evaluation of on-site and off-site information, and the development of a control strategy. In developing a control strategy it may be necessary to identify various alternatives and select the one, or combination, that is considered to be the most effective. The development of a control strategy should be followed by a field review to examine the technical feasibility of the control strategy. The field review may identify necessary revisions in the finalization of the control plan.

Assuming that the engineer is familiar with the legal requirements, the engineer should proceed with the evaluation of site information. It is important to make a thorough study of the site because the site information may dictate the control methods adopted as part of the control strategy; it will also help identify the most cost-effective strategy. Site information includes topography, hydrologic response characteristics, climatic characteristics, soil characteristics, land use, and geologic information.

Topographic maps are available from the U.S. Geological Survey and local planning agencies. These maps provide useful information about relief and slope gradients, drainage basin divides, road locations, and stream systems.

Soil maps and surveys are available from the U.S. Soil Conservation Service or the local office of the Soil Conservation District. The surveys provide information about the soil types and extents, the engineering properties of the soils, the erodibility of soils, and general textural characteristics of the soils.

While the topographic maps and soil surveys often identify land use, it is worthwhile obtaining a recent low level aerial photograph of the study site and downstream locations. For large projects Landsat imagery are available and can be of value. Local land use and zoning maps may provide information about projected land uses that may be useful in control plan formulation.

Geologic maps are available from the U.S. Geological Survey and local offices of the state Geological Survey. These maps identify the geologic trends in the area, types of strata, and the location of geologic hazards. It may also be important to identify locations with high-water tables, especially in coastal areas near wetlands.

The information obtained for the site is necessary in the selection of mitigating measures in the planning, construction, and completion stages of a project. Methods for runoff and sediment control can be categorized into three classes: off-site conveyance, nonstructural on-site control, and structural on-site control. Off-site conveyance techniques include storm drains, open channels and ditches, and channelization; the purpose of these methods is to remove the runoff from the site to a downstream point where the runoff can be controlled without causing damage. Nonstructural on-site control methods include swale and vegetative strips, porous pavement, and infiltration beds; the intent of these methods is to provide the maximum possible opportunity for runoff to infiltrate and, therefore, use natural storage. Structural on-site methods include detention, rooftop, and parking lot storage; the purpose of these methods is to use on-site, man-made storage to replace the natural storage lost during development. While it may appear that all of these methods are intended primarily for runoff control, it is important to recognize that the control of runoff quality can not be separated from the control of runoff quantity. The design factors that are important in controlling runoff rates are also important in quality control. Furthermore, the methods should not be viewed as independent methods; quite often, a combination of the methods may lead to the most efficient control of stormwater runoff quantity and quality. For example, an infiltration bed should be separated from the runoff source by a vegetative strip; this will provide for some particle removal, which will help prevent the infiltration bed from becoming clogged.

Methods for Mitigating the Physical Impacts of Engineering Projects

Before examining specific runoff, erosion, and sediment control measures, it may be worthwhile identifying the general objectives of mitigating measures:

- control runoff volumes into the wetland to maintain water levels and the extent of the wetland
- control peak runoff rates into wetlands to maintain periodicities and variations in both the surface water level and the water table level
- control the volume of storage on upstream (adjacent) lands so as to maintain the extent of the wetland.
- control sediment volumes into the wetland to prevent deposition within the wetland, which may have physical impacts on both the bottom taxa and the circulation patterns
- control sediment concentrations of runoff into the wetland to prevent increases in turbidity
- maintain natural drainageways within the wetland to prevent both damming of flow paths or the creation of channelized flow where none existed prior to construction.

While these general objectives concentrate on runoff and sediment problems there are other pollutants that result from construction activities. Construction activities may result in the discharge of petroleum products and other chemicals into wetlands. In some cases, the measures used to control runoff and sediment will reduce the detrimental effects of these other pollutants. In some cases, it may be necessary to include other control measures into the control strategy to ensure the mitigation of pollutants other than sediment.

Strategies for controlling runoff, sediment, and erosion have been instrumental in protecting the environment from the adverse effects of urbanization and development. Without proper controls the changes in the land use and physical features of the landscape (e.g., drainage paths) cause the response of a watershed to differ significantly from the original predevelopment conditions. The increased flow rates and volumes of runoff often overload the capabilities of the natural drainage. The transition period when land is being cleared for development often produces unusually high volumes of sediment and other pollutants.

The following is a summary of the advantages, disadvantages, and design factors for the more common techniques used in controlling the increased amount of runoff and sediment from developing areas. The design factors should be considered when attempting to implement the proposed method into a control plan. The summary will aid in the selection of the appropriate approach, or combination of approaches, that are available for mitigating the effects of development in or near wetlands. For small construction sites many other methods exist. The selection of these other methods will depend on the information obtained in the site survey. A brief outline of these other methods is given at the end of the section.

ROUTING OF FLOW

Storm Drain Construction

Storm drainage systems have a primary purpose of conveying surface runoff from developing sites. They are especially valuable for routing flow around environmentally sensitive areas. An underground pipe system transports the increased volume of water to receiving streams, retention ponds, or other storage structures that are capable of accommodating larger quantities of water without degradation of the local environment. The construction of storm drainage systems helps to alleviate existing runoff problems and can also serve as a preventive measure against potentially damaging (i.e., intense) rainfall events.

Advantages. Storm drainage systems have a major advantage in the small amount of surface land that is required. Since the extensive conduit layout is underground, the aesthetic value is acceptable to the public. Storm drain systems have additional benefits. The flexibility of including other stormwater management techniques as well as expanding the conduit system allows for further development with little disruption to existing areas. The drainage system is relatively maintenance free and may have a service life up to 50 years.

Disadvantages. A significant drawback of storm drainage systems is the changes that occur in the hydrologic response of a watershed. The effect of increasing the conveyance of runoff decreases the times of concentration, hence the peak discharge may be greater and occur sooner. The overall consequence may be a transference of the problem to another location that is downstream of the development.

Because of their impact on the hydrologic response of a watershed, the use of storm drain systems may decrease surface and subsurface water levels in wetlands when the flow is routed around the area. If pipe systems are directed into a wetland, the variations in flow, with increased peak rates of flow and decreased base flow, will increase; furthermore, pipe systems will eliminate the natural cleansing action of a watershed and thus increases in pollution loadings may result.

Design Considerations. The designing of a storm drainage pipe network requires hydrologic and hydraulic knowledge of the watershed and the manner in which the flow travels through a conduct. The sizing of the components in the pipe system will depend on the predicted runoff rates. The peak discharge is usually of primary concern in the determination of pipe sizes. Also, the invert slope of the pipe and the Manning's roughness coefficient, n, of the pipe are important considerations that affect the velocity of flow.

In addition to design factors one must also consider site factors. Pipe systems may not be affective in extremely low sloped areas. The construction of a system should not interfere with other utilities or natural environmental factors that can not withstand the construction disturbance. Outlets from pipe systems represent a danger to the local environment unless they are protected from scour that results from high, concentrated flow rates. Such scour in wetlands can upset the balance within the bottom areas.

Open Channels and Ditches

Open channels and ditches are typically found in areas of low density, which have sufficient surface land that is not required for other purposes. Open channels may have a continuous flow (i.e., base flow). Ditches located along roadsides are not intended to carry the larger quantities of water that open channels are capable of handling. Ditches adjacent to construction sites must be properly designed to control erosion.

Advantages. Open channels and ditches may be designed with different linings such as concrete, rip-rap, or grass. The infiltration of flow in unlined channels and ditches is also a beneficial result because it serves to recharge groundwater.

Concrete or rip-rap linings improve the stability of the channel bed in the event of high velocity flows. The erodibility of the soils is, therefore, reduced. Erosion can also be prevented by using check dams and drop box culverts.

Disadvantages. Open channels may have adverse consequences if not properly designed. If the drainage area in the channel bed is not large enough, flooding may occur. There should be an overflow area for those occasions where a major storm could fill the available drainage area. The amount of required land is thereby increased. Maintenance is frequently needed should the channel become clogged with debris or become filled with siltation.

Channels and ditches constructed in wetlands may increase drainage from the wetland; this may result in decreased storage and water table levels and thus a decrease in the extent of the wetland. Channels and ditches constructed adjacent to wetlands may cause significant changes in the circulation patterns within wetlands.

Design Considerations. The factors that affect the design of open channels and ditches are similar to those that are involved with the design of storm drainage systems. The size and shape of a channel is dependent on the quantity and velocity of the runoff. The velocity of the flow determines the type of lining that is needed in the channel bed. The velocity varies directly with the square root of the bed slope and is inversely proportional to Manning's roughness coefficient n. The values of n have been derived for various surfaces and are available in most hydraulic textbooks.

To control erosion of the bed, the tractive force must be balanced with the oppositely acting force of gravity. The most important factors are the slope of the bed and side slopes, and the shape, size, and weight of the soil particles. Bed slopes can be reduced when check dams and drop inlet culverts are used.

Channelization

Channelization, which is used to prevent flooding in susceptible areas by increasing the conveyance of water, requires the straightening or dredging of a stream and the clearing of the banks. Channelization allows a larger amount of water to travel a shorter distance so that the flood flow passes rapidly. Channelization of a stream may occur naturally when flood waters force the flow to travel in a straight path.

Advantages. Channelization offers the greatest advantage in the protection of life and property from dangers that result from flooding. Communities that have already built in the flood plain may benefit from the flood relief. Channelization also provides a proper conveyance for boating. The aesthetics of urban streams may improve as the banks are cleared of debris.

Disadvantages. The process of channelizing a stream may have adverse effects if not properly implemented. The soil should be sufficiently fertile to support vegetative growth to prevent erosion. In the event of high velocities, artificial surfaces should be used to line the stream bed. Concrete or rip-rap lining is commonly used in these cases.

The effectiveness of channelization as a stormwater management alternative is not always favorable. The increase in the flood passage may transfer the problem to another point causing increased peak flows downstream. Scour may occur behind the channelized sideslopes if proper maintenance is not provided. In many cases, the public find a natural environment to be aesthetically favorable when compared with a concrete lined channel.

Channelization within a wetland reduces the surface area available for aquatic biota; the increased velocities may also be unfavorable to the biota. Channelization in areas adjacent to wetlands can cause changes in circulation patterns within wetlands.

Design Considerations. The hydraulic principles that control the stability of channel are well documented. In general, one must consider the slope of the channel, the soil characteristics, and the cross-sectional properties in designing a stable channel. The sediment discharge is proportional to the square of the discharge and the square of the slope and inversely proportional to the 1.5 power of the mean particle size of the soil. If a channel is to be modified, the engineer should take the necessary procedures to ensure stability of the bed and side slopes. If a vegetated surface is not possible, the channel may have to be lined.

GROUNDWATER RECHARGE SYSTEMS

Natural Infiltration Technique

Engineering projects often increase the percentage of impervious area, thus, decreasing the available natural storage. As the watershed is developed the amount of surface runoff is increased by the excess water that is no longer infiltrating or stored in depressions. The timing of the runoff from the watershed is also affected, causing peak flows to occur sooner. The increase in surface runoff is accompanied by decreases in base flow.

Infiltration of the runoff will decrease the initial volume of the overland flow. Areas that are covered with good vegetative growth have higher infiltration rates than those that are bare. Water flows over vegetation at a lower velocity, enhancing the opportunity to infiltrate into the groundwater system. Hence, by routing overland flow over grassy areas, the runoff is detained, allowed to infiltrate, and decrease in amount.

Advantages. The advantages of utilizing natural infiltration to decrease runoff is evident. The natural drainage system is maintained, and the use of construction methods to prevent erosion by keeping existing vegetation intact is a good conservation practice. The recharge of aquifers is also accomplished when maximizing infiltration.

Disadvantages. Areas that have little or no vegetative cover are excluded from using this technique. Also, vegetation may require maintenance.

Design Considerations. The effectiveness of natural infiltration depends on the hydraulic flow length, the density and type of vegetation, the infiltration capacity of the soil, and the slope. The length of the hydraulic flow path should be maximized. For paths characterized by high slopes, the vegetation should be selected so that high velocity runoff does not cause erosion. An increase in depression storage, when acceptable, will increase infiltration and decrease runoff volumes.

Porous Pavement

Porous pavements reduce the total volume of direct runoff from parking lots, highways, and paved areas. The development of porous pavements as a stormwater management technique is a relatively new concept. While still in the experimental stage, porous pavements are being evaluated as a method to limit runoff by allowing water to percolate into the ground. The practicality of the method depends on its ability to carry loads, withstand freeze-thaw cycles, and be cleaned, should the pores become clogged.

Advantages. Porous pavements have many advantages that make it a desirable technique. Because pavements are normally impervious to water, the effect of having an absorbent pavement may reduce the peaks of local floods and reduce the need for storm drainage systems. Moreover, it increases groundwater recharge by utilizing the natural drainage system. While not believed to be applicable to roadways, it may be used on shoulder areas that do not receive heavy traffic loads.

Disadvantages. The effectiveness of porous pavements has not been widely assessed under actual circumstances. More testing should be conducted to evaluate the feasibility of usage. The problem of maintaining clogged pores has not yet been resolved and applicability is subject to the characteristics of the soil beneath the pavement.

Design Considerations. The effectiveness of porous pavements will depend on the slope and roughness of the pavement, the hydraulic conductivity of both the pavement and the underlying soil, and the degree to which the pavement is clogged. Porous pavement can be used in areas subjected to heavy erosion.

Infiltration Beds

Infiltration beds are designed to collect runoff and store it until it infiltrates into the ground. The quantity of runoff is thereby reduced allowing the natural storage to be maintained. The bed is excavated and filled with a highly permeable material such as gravel or rocks. These facilities are useful around commercial buildings or parking lots.

Advantages. The properly designed infiltration bed will serve as a means to maximize infiltration. The local flooding potential is reduced and the recharge to groundwater supplies are enhanced. The required capacity of the downstream storm drainage system is reduced because of the lower volumes of direct runoff. Infiltration beds located on construction sites near wetlands can decrease peak runoff rates and sediment concentrations in flows into wetlands.

Disadvantages. The primary disadvantage of infiltration beds is the frequent maintenance that is required. Measures should be taken to screen leaves and trash before runoff enters the bed. If surface runoff that is directed into an infiltration bed contains significant volumes of sediment, the runoff should be intercepted by a sediment trap or a vegetation strip prior to discharge into the infiltration bed; otherwise, the sediment will reduce the infiltration and storage capacity of the infiltration bed. The method also requires an overflow area, which may increase the amount of land required for the facility. Scour at the inlet must be prevented. Safety is also a necessary consideration.

Design Considerations. The effectiveness of the beds is highly dependent on the soil characteristics of the bed, the volume of storage, the location of the bed within the hydraulic flow path, and the location of the water table. An infiltration bed should be sited within the flow path in order to have the greatest effect on runoff rates near the time of the peak discharge. For areas having a high water table, the effective storage will be highly dependent on the surface area allocated to the bed.

STORAGE OF EXCESS RUNOFF

On-Site Detention Ponds

On-site detention ponds are effective in reducing the peak flow both at the site of development and in downstream areas that are sensitive to frequent flooding. The runoff is detained in these ponds and released at a rate that is similar to the predevelopment runoff rate. The size of a detention pond depends on the volume of water that must be stored to prevent increases in peak flow rates and velocities. Construction of detention ponds requires excavation and fill operations, which must be properly controlled to prevent adverse environmental impacts.

Advantages. Detention ponds serve as a multipurpose facility. One of the primary purposes, which initiated the usage of detention ponds, is the control of sedimentation from the erosion of soil during construction. The larger ponds may also be used for recreation activities such as boating or fishing. As a result of controlling the maximum runoff, detention ponds also effectively reduce the size of the storm drain system required. If properly sited and designed, ponds can be aesthetically pleasing. Large ponds can also be used for recreation. They will also help to maintain base flow into wetlands. Detention ponds can help maintain water levels in wetlands and reduce the changes in variations of flow rates that result from construction activities.

Disadvantages. The implementation of detention ponds requires the allocation of a significant amount of land; such land then may not be used for development. Another problem of implementing the facility is in attempting to convince local communities that such stormwater management measures are worthwhile when the downstream neighbors reap the benefits. The responsibility of maintenance, providing safety measures, and the legal water rights should also be considered. If not properly designed, detention ponds may create flooding problems in wetland areas downstream of the construction area.

Design Considerations. In order to maximize the efficiency of a pond for controlling runoff rates, sediment rates, and other pollutants, the volume of storage must be sufficient to provide adequate detention time. This will allow for sufficient setting of pollutants and helping flow rates at a minimum during periods when uncontrolled flow have maximum runoff rates. The loss of storage due to inadequate maintenance should be considered in design. In wetland areas with high water tables, the expected location of the water table must be considered when determining the spatial area/depth ratio of the pond. The capacity of the outlet facility must be determined with consideration of both allowable downstream flow rates and control of the time of detention within the basin. The dead storage/total storage ratio must be considered. Baffles can be used to increase flow lengths and detention times and thus settling rates of pollutants.

Rooftop Ponding

Rooftop ponding provides storage for stormwater runoff by limiting the flow to the down spout. The technique is used primarily in highly urbanized areas that include a significant number of multi-family housing units; it will have a significant effect on reducing the runoff if a large percentage of roofs contribute to runoff. The structural integrity of each roof should be evaluated before incorporating rooftop storage. The provision of an emergency overflow must be made to prevent structural damage or leakage. Runoff should overflow before the maximum load is attained.

Advantages. Rooftop storage facilities can provide many benefits. There is no inconveniences to the public since the storage structure is not visible from the ground. The technique utilizes existing structures in urban areas where there is usually no available land for other types of controlled storage.

Disadvantages. This method of storage requires planning when the structure is designed; that is, it is often not possible with existing buildings. The location of the housing units within the tract of land can have a significant impact on the effectiveness of rooftop storage. Rooftop ponding requires maintenance including the repairing of leaks, removal of debris and ice, and maintaining the downspout and emergency overflow facility.

Design Considerations. In addition to the design of the structure, it is necessary to consider the design characteristics of the outlet facility; these should be designed to provide optimum control of the timing of the runoff.

Parking Lot Ponding

Parking lot storage reduces the runoff rates and pollution loadings on storm drainage systems. The storage is temporary and used in paved areas around commercial buildings, offices, and shopping centers. The stormwater is collected and released at a regulated rate.

Advantages. Parking lot ponding is easily implemented in areas that are already developed. Water can be ponded around the least used portions of the parking lot, perhaps channeling the runoff to infiltration beds. Maintenance is easily performed using mechanical street cheaning machines.

Disadvantages. Ponding in parking areas may cause an inconvenience to the public during storms. The problem is minimized by designating storage areas in spaces least used.

Design Considerations. The slope and roughness of the area will control the rate of inflow to these areas of the parking lot used for storage of stormwater runoff. The capacity of the outlet will determine the effectiveness of the storage. Because dead storage is not usually provided, unless the effluent is directed to an infiltration bed, parking lot storage is usually not effective for pollutant control.

Additional Control Measures

Engineering projects require measures to limit physical impacts on wetlands; four general impacts were identified: (1) changes in the mean water level of the wetland, (2) changes in the local water table level, (3) changes in periodic variations of both surface flows and tidal flows, and (4) changes in circulation and salinity patterns. In order to minimize these impacts on wetlands, runoff and pollution characteristics (i.e., volumes, peaks) must be controlled. While the control methods discussed in detail are the primary control measures, other effective measures are available.

The following control measures are effective for controlling peak runoff rates into a wetland:

(1) temporary barriers made of straw bales can impede surface runoff and induce infiltration;

(2) diversion ditches can increase flow paths and are especially useful for diverting surface flow around cleared areas;
(3) the omission of grubbing increases the surface roughness and impedes flow;
(4) level spreaders can be used to convert channel or pipe flow to sheet flow, which results in reduced rates of runoff;
(5) a roughened surface, which is made by disking or ploughing, reduces runoff rates and increases infiltration.

The following measures are effective for controlling increases in surface runoff volumes:

(1) benches constructed on the steeper slopes serve to increase infiltration;
(2) spray irrigation of surface runoff can be used to spatially distribute the water on pervious surfaces;
(3) level spreaders can be used to divert water to flow over pervious surfaces;
(4) serrated cuts will increase infiltration and reduce erosion potential.

The following measures are effective for controlling increases in sediment volumes:

(1) check dams prevent erosion and lengthen runoff times, thus increasing settling of suspended solids;
(2) chemical stabilization reduces surface erosion potential;
(3) compaction of cleared areas can decrease erosion rates;
(4) fertilization can increase the growth rate of vegetated covers, thus increasing erosion resistance;
(5) filter fences placed in drainageways and at slope toes can be effective in limiting the passage of sediment;
(6) gabions can dissipate the energy of surface runoff in channels and along ditches;
(7) jute and plastic matting is used as a surface and drainageway protection;
(8) pipe outlet protection can eliminate scour at outlet;

(9) rip-rap can dissipate energy and reduce channel erosion;

(10) sediment traps are especially useful where flows are concentrated and contain significant volumes of sediment;

(11) seeding, followed by fertilization or chemical stabilization, can increase the vegetative cover and reduce the exposed surface area;

(12) vegetative buffer strips, which are especially useful in low sloped areas, serve as filters to limit sedimentation.

APPENDIX C

ELEMENTS OF A COMPREHENSIVE STORMWATER MANAGEMENT POLICY

Richard H. McCuen

Mark E. Hawley

ELEMENTS OF A COMPREHENSIVE STORMWATER MANAGEMENT POLICY

In developing a SWM policy it is important to keep the goals of SWM in mind. These goals will not be met if the policy is ambiguous or incomplete. The elements discussed below should be considered for inclusion in any SWM policy that is being formulated at the level of government that has the primary SWM responsibility. In most cases, this level of government is the municipal or county level, although regional councils and planning agencies may have the primary responsibility for SWM in some areas. In any case, the goals of SWM will be more easily achieved if the SWM policy contains these elements because the policy will be clearer, more specific, and more comprehensive.

Statement of Authority to Regulate

A comprehensive policy should always include a statement explaining the source of the authority under which the policy is promulgated. This statement is helpful to developers, regulators, and the general public because the limits of the regulators' authority are defined. Regulators may not know the basis and extent of their powers if this statement is not included; this may lead to misunderstandings and less than optimum performance of their duties. Furthermore, an explicit statement of the authority to regulate may eliminate unnecessary lawsuits and disagreements caused by misinterpretation on the part of the developer. Institutional conflicts may also be avoided because the limits of each government authority are specified in the policy.

The statement of authority to regulate must not only contain the legal basis for regulations, but must also assign responsibilities. In many areas, more than one agency may have jurisdiction over development; therefore, the statement of authority should specify the responsibilities of each agency so that conflicts are eliminated. For instance, a technical agency such as the State Soil Conservation Service might be given the responsibility for developing a suitable set of SWM design criteria, while responsibility for issuing permits and enforcing the SWM regulations might be given to another government body, such as a Department of Public Works. This division of responsibilities is dependent on the nature of the laws under which the authority to regulate is established. All responsibilities should be specified in the regulations so that developers and administrators will know the limits of the authority and responsibilities of each government agency.

Goals and Purpose of the Policy

Every SWM policy should include a statement of the goals and purposes of the policy. This is actually the most important part of the entire policy because all the other parts are merely the ways of ensuring that these goals are met. The statement of goals and purpose should be comprehensive; both water quality and water quantity goals should be specified. The primary purpose of most policies is to ensure the public's health and safety; the specific goals are the standards that must be met in order to achieve this primary purpose.

Definitions of Terms

Much of the vocabulary used in SWM policies and regulations is technical, and many of the people that will have to interpret the regulations do not have a technical background; therefore, it is important to include a section in which all terms are defined. Inclusion of such a section in a set of SWM regulations will make the regluations less ambiguous and may lessen the possibility of legal difficulties caused by misinterpretation of the regulations.

Previous Relevant Legislation

The enactment of specific SWM legislation and regulations may cause confusion due to the existence of previously enacted legislation concerning flood plain management, sediment control, and other related topics. Therefore, the relationship between the previous regulations and the newly enacted SWM regulations should be described precisely in the SWM legislation. In many cases it is simpler to enact a new, comprehensive set of regulations that concern SWM and all related topics at once; this new set of regulations can incorporate or replace the old floodplain and sediment control laws. This approach is preferred because there is no possibility of conflicts with existing regulations if the comprehensive set of rules is properly formulated. Conflicts with existing laws can lead to confusion on the part of the developers and regulators, and should therefore be avoided if possible.

Sediment and Grading Control

Many governments that have no SWM regulations already have sediment control laws in effect; because one of the goals of SWM is the control of erosion and sedimentation, sediment control should not be considered as a separate subject. The most reasonable and efficient way of achieving all the goals of floodplain management, sediment and grading control, and SWM is to formulate one comprehensive set of regulations covering these and any other related topics such as regional planning. This is the only feasible way of eliminating conflicts that can be detrimental to the public's well-being.

Floodplain Management

The subjects of stormwater management and floodplain management cannot be completely separated. SWM involves controlling the rates and volumes of flow through stream channels, while floodplain management involves controlling the land use in the channel areas. Because the rates and volumes of flow determine the size of the floodplain, these subjects should be considered together rather than as two separate problems. In writing SWM regulations, this connection should be considered because one of the goals of SWM is also the goal of floodplain management, namely, to minimize flood damage. If possible, the floodplain management regulations should be incorporated into the SWM regulations so that potential conflicts are avoided.

Cooperation with Other Local Governments

In many cases, storm runoff from one county or municipality drains into streams and channels that pass through downstream jurisdictions. The upstream jurisdictions have a responsibility, morally if not legally, to control their runoff to avoid adverse effects in the downstream reaches. Many waterways also serve as political boundaries; the banks of a stream may lie in two or more separate jurisdictions. In order to avoid conflicts between upstream and downstream counties and between counties on opposite banks of a waterway, all local SWM regulations should require cooperation between the various local governments. Each government should be required to consider the effects of its actions on downstream areas and to regulate the activity of developers so that downstream areas will be protected. This type of cooperation is probably best ensured by the creation of regional planning bodies.

Elimination of Conflicts Between Local, State, and Federal Laws

The Federal government and state governments have jurisdiction over many of the larger rivers, lakes and streams in the U.S. Confusion can arise as to which government actually controls the use of a particular body of water. Because the Federal, state, and local government regulations often differ, it is important for the developer to know which set of regulations applies to the area being developed. In most cases, state authority supersedes that of local government and Federal authority supersedes that of the state; but because the limits of state and Federal authority are often ambiguous, it is not always obvious where the authority to regulate lies. Therefore, a comprehensive set of local regulations should include a statement describing the limits of the local authority and requiring compliance with other authorities where they have jurisdiction. In this way, conflicts between the various levels of government can be avoided, and the design process can be expedited.

Exemptions for Government Projects

The SWM regulations of many jurisdictions contain exemptions for government projects, although the reasons for these exemptions are seldom explained. In cases where the exemption is from the application and permit process only, these exemptions make sense because it is often important to the public welfare that government projects be completed as quickly as possible. The project is usually subject to the same design requirements as a private project would be, but the approval of the design is expedited by having the government engineers design the facilities themselves. Some SWM regulations contain an exemption from the inspection clause for government projects, but because many government projects are actually constructed by private contractors, exemptions from inspection requirements may not always be reasonable. The critical factor in determining whether government projects should be exempt from any particular regulation should be the impact of the exemption on the public welfare. If an exemption is not in the public's interest, it should not be included in the regulations. Any exemptions for government projects that are deemed to be in the public's interest should be specified in the SWM regulations to avoid ambiguity.

Design Criteria

The heart of any SWM policy is the set of design criteria with which the developer must comply. These criteria should be developed at the local level if possible because different environments require different SWM procedures. In most parts of the U.S., each county has a local branch of the state Soil Conservation Service (SCS); these local branches are familiar with the terrain and soils in their areas and are probably the people best qualified to formulate the design criteria for SWM. However, in most areas the local SCS branches do not have the authority to institute SWM regulations; that authority is either not exercised by any government, exercised by the state government, or delegated by the state government to the local governments. Because of the diversity of human and hydrologic environments encountered in many states, the most sensible approach is usually for the state government to delegate SWM authority to local governments and regional commissions and then play an advisory role. In this case, the local Department of Public Works (DPW) is usually charged with developing and enforcing the SWM design criteria. Alternately, the local SCS can develop the criteria and the DPW can enforce the policy. Usually the agency that is charged with developing the design criteria can receive help and advice from the state and Federal governments.

Regardless of the means of administrating a SWM policy, a comprehensive set of design criteria is necessary if the goals of the policy are to be met. Incomplete policies are ambiguous and lead to misunderstandings. The following is a list of design elements that should be in a complete set of criteria; under some circumstances, other elements should also be included.

Computational Methods. Many different hydrologic models are available for evaluating the hydrologic effects of land use changes. Because different models will often give different results when applied to the same situation, it is important for the reviewing agency to adopt one computational method (model) for use in evaluating the developer's SWM plan. This standardization of methods will help to eliminate technical disagreements because the design engineer and the personnel with the authority to approve the plans will both be using the same computational methods.

Design Limits on Slopes. Steep slopes are much more susceptible to erosion than flatlands and, therefore, special limitations should be specified for areas containing steeply sloping land. This is not much of a problem in areas where the topography is generally very flat, but the problem of erosion on slopes may still occur along the banks of channels or in areas where the local topography is modified by man.

Design Storm Characteristics. Most SWM regulations require that certain design storms be controlled. The question of which return frequency to select should be determined locally, based on economics (there is no point in spending thousands of dollars to prevent flooding that will only cause hundreds of dollars of damage). Many jurisdictions only specify the return period of the design storm; this is a major failure on the part of the policymakers, because the hydrologic effects of a particular design storm are not only a function of total rainfall but also of storm duration, storm pattern, and antecedent conditions. Therefore, the characteristics of the design storm that must be specified are: 1) the return period, 2) the storm duration, 3) the antecedent conditions, and 4) the pattern of rainfall intensity. If references for determining the return period and all the listed characteristics of the design storm are specified, then designers and evaluators should have no disagreements over the characteristics of the design storm.

Emergency Spillways. Many SWM plans include structures designed to impound water behind them; these structures are designed to control the rate of flow under a certain range of conditions. When the runoff from a 50-year storm encounters a structure designed to control a 10-year storm, the top of the structure will usually be inundated. This inundation may damage or destroy the structure unless an emergency spillway is provided to ensure that the excess water can pass through the structure. Therefore, all impounding structures should be required to include a spillway capable of handling runoff from extreme storm events. Otherwise, sudden failures of SWM structures may cause more damage than would have occurred with no structure in place; also, the costs of maintaining the structures will usually be higher if no spillway is provided.

Fencing of Ponds. Stormwater management ponds are often considered to be a hazard to public health due to their attractiveness to children. Many jurisdictions require that all ponds be fenced; some others require fences but issue waivers of this requirement in rural areas. In some cases, nearby homeowners will insist on fencing to protect their children, while others will think that fencing is an unnecessary eyesore. Actually, fencing is usually only a deterrent to small children; only a very high barbed-wire topped fence is likely to stop teenagers. Some states do not recognize liability cases based on "attractive nuisance," so theoretically the government is not liable for damages due to pond-connected accidents regardless of whether or not the pond is fenced. If fences are required, it should be because it is the will of the local populace.

Location of SWM Facilities. One of the most important elements of a set of SWM design criteria is a regulation that controls the location of SWM facilities. Many jurisdictions require that all SWM structures be located on the site of the proposed development and that the postdevelopment runoff rates do not exceed predevelopment rates for specific storms. Alternately, off-site and multi-site control facilities may be allowed.

When on-site control of runoff is required, each developer will plan his own system independently. No coordination of the discharges from various developments will occur. If the postdevelopment runoff rate is regulated to be no greater than the corresponding predevelopment rate, the postdevelopment duration of peak flow must be larger than the predevelopment duration because the volume of runoff is greater in the postdevelopment condition. Thus, each development in the watershed will be releasing storm runoff at the predevelopment peak rate for a longer period of time than in the predevelopment situation; these hydrographs with increased durations of peak flows may combine in the channel and cause increased flooding downstream due to the constructive interference of the runoff hydrographs. This phenomenon is often referred to as peak overlap. Peak overlap can usually be avoided by planning on a larger scale. If each developer makes his own plans, there is no reason for him to consider the possibility of peak overlap; but if a regional authority participates in the planning process, SWM systems can be designed to eliminate the possibility of peak overlap.

Watershed-wide planning is a much more sensible approach to SWM than simply requiring each developer to provide on-site controls. Besides eliminating peak overlap, planning can also lead to the use of off-site and multi-site structures that may provide greater benefits to the public at lower cost to the developers. Economies of scale may be realized in both construction and maintenance if one large structure serving a number of development sites can be substituted for a series of smaller structures. In some jurisdictions a developer may be issued a waiver of on-site control requirements if he cooperates with other developers in planning multi-site facilities. In other areas the local government or regional planning body may require the developer to contribute funds or land to be used for an off-site control structure in lieu of on-site control facilities.

One of the most important problems associated with changes in land use is the increase in erosion that results from the increases in runoff velocities and durations that accompany development. After development, most areas erode more quickly than they did in the undeveloped state; therefore, the amount of sediment delivered to the drainage channels is increased. Detention basins, either on-site or off-site, are commonly used to control the increased rates of runoff from developed areas; as the silt-laden runoff from a developed area flows into a detention basin, it loses velocity and consequently the sediment carrying capacity of the flow is lessened. This results in sedimentation in the detention basin. As the impounded water is released into the downstream channel, the velocity increases, and creates the potential for erosion of the channel immediately downstream. Also, because of the increased volume of run-off, water flows through the downstream channel at near-peak rates for a much longer time than in the predevelopment state; this can lead to enlargement of the channel by erosion of the stream banks. Some of this erosion can be controlled by use of outfall and channel protection methods, but a typical detention structure is often collecting sediment on the upstream side and causing increased channel erosion on the downstream side. Because the presence or absence of sediment in the stream is one of the most important aspects of water quality, these erosion problems should be considered in locating detention basins. In some cases, on-site structures may be the only practical means of SWM, while in other cases off-site basins may be necessary if both the quantity and quality of storm runoff are to be controlled. Requirements should be developed on a watershed-by-watershed basis, rather than mandating on-site control for all situations.

Maintenance Considerations. Maintenance is an important factor in the development of a SWM policy due to the erosion and siltation problems associated with development. Usually either the local government or the landowners are assigned the responsibility for maintaining the SWM system; in many cases, maintenance consists primarily of mowing grassy areas and removing the collected sediment from detention structures. Regardless of whether the local government or the landowners have maintenance responsibility, SWM systems should be designed so as to require as little maintenance as possible and to facilitate the maintenance that is required. Consideration of maintenance in the design of the SWM system will result in lower costs for the responsible parties and will also lessen the amount of damage caused by neglect in most cases. Therefore, consideration of maintenance in design of SWM systems is beneficial to the general public as well as to the party responsible for maintenance.

Methods of SWM. A complete policy should include a list of the types of SWM facilities that a developer may use because some types of facilities are not appropriate under certain conditions. For instance, in areas where water tables are high or soils are very shallow, infiltration facilities may not perform very well. If the allowable types of facilities are listed, the design engineer will know what options are available. A statement should also be included that allows the use of types of facilities that are not included in the list if their effectiveness in a particular situation can be satisfactorily demonstrated. The decision as to the types of facilities that are permitted should be based on the computational methods used in evaluating proposed SWM site plans; if the model will not account for the effects of a particular type of facility, there is no means of judging the facility's effectiveness. The model should be chosen with this problem in mind so that an engineer designing a SWM system will be free to propose innovative solutions to SWM problems if he desires to do so.

Reference List. A list of references to be used in evaluating SWM plans should be included in the policy so that the design engineer knows where to look for any required information. This will eliminate ambiguity and disagreements over the methods used to evaluate the plan, lowering the cost to both the developer and the government.

Sensitive Areas. In many localities, the government may find that there are areas within its jurisdiction that require special consideration. These are often called sensitive areas; designation as a sensitive area may be due to land use (historic districts or landfills), geomorphological features (highly erodible soils or slopes, scenic areas), or biological considerations (water supply reservoirs and trout streams). In any case, it may be necessary to impose SWM requirements of a more stringent nature in some localities in order to protect these sensitive areas. The localities subject to these special regulations should be described in the SWM regulations; in this way, the public's health and welfare will be protected.

Inspections

In order to ensure that the goals of the SWM policy are met, approved projects must be inspected both during and after construction. Many of the dimensions of detention basins are critical; if these structures are to perform as expected, the elevations of the top of the riser and the emergency spillway must be as specified in the design. Any part of a SWM system that is located underground should be inspected before it is covered over to ensure that it has been constructed as designed. Inspections are necessary after construction has been completed to ensure adequate maintenance and proper functioning of the facilities. Inspections are usually the responsibility of the DPW or a separate government inspection agency. The following list of elements should be considered in developing inspection clauses for SWM policies.

Access to Site. Free access to the site for government inspectors must be guaranteed both during and after construction. This will help to prevent an unscrupulous developer from deviating from the approved design in order to cut costs and will also help to ensure adequate maintenance. Only by having guaranteed access can the government inspectors protect the interests of the public.

Inspection During Construction. An inspector should be on hand at all critical times during construction of the system to ensure that SWM facilities are built as specified. It is common practice to require the developer to give notification at least 24 hours in advance of performing such critical operations as setting the riser and spillway crest elevations in the construction of detention basins. Also, any construction that will be covered when the system is completed should be inspected before backfilling.

Final Certification. A final inspection of the completed SWM system should be required, specially in cases where the government will assume the responsibility for operation and maintenance. If the system passes the final inspection, the local government should certify that the system has been built as designed. This will limit the contractor's liability for failure of the system or its components.

File of Inspection Reports. Many jurisdictions require that the agency responsible for inspections keep an up to date file of the inspection reports for each project in a place accessible to the general public as well as to the contractor. This allows concerned citizens to ensure that the inspections are being carried out as required and protects the integrity of the inspection department. It is important that the contractor have access to the inspection reports so that they may be reviewed for accuracy; many SWM policies require the inspection department to provide the contractor with a copy of each report.

Maintenance Inspections. Inspections to ensure proper maintenance of all SWM facilities are necessary if the system is to function as expected. Usually, inspections are made to ensure that pipes and channels are not clogged with natural or man-made debris or sediment and that vegetation, mulch, and other sediment-controlling land covers are in good condition. Inspections for maintenance may be required either at set time intervals (every spring, for instance) or after certain flood events (such as any time the emergency spillway is inundated). Underground facilities can present special problems for inspectors because not all of them are easily accessible; where possible, access structures such as manholes should be incorporated into the design. If this is not possible, it may be necessary to observe the operation of such facilities during actual storm events to gage their performance.

Notification of Violations. It is wise to specify the method of notification of violations in the inspection clause. Also, the time period allowed for correction of violations should be stated. In this way, both the developers and the inspection agency will know how to proceed in cases where violations occur.

Maintenance

Maintenance is one of the most important factors in a SWM policy because even a properly designed and constructed facility will not perform as intended by the designer if it is not properly maintained. The usual maintenance problems involve man-made and natural debris or sediment clogging pipes and infiltration surfaces. Another problem is the performance of erosion-resistant land covers and mulches. Good maintenance consists of regular cleaning of pipes and basins and the mowing and repair of land covers.

and to collect the funds required for this purpose from the private owners.

Bonds for Maintenance. It is common practice to require the developer and subsequent landowners to bear the financial burden of maintaining the SWM system that serves their land. Many governments require that a maintenance bond be posted to ensure that the landowners will meet this obligation; the amount of bond is usually the estimated cost of maintaining the system for a specified time period, such as five years. This bond will ensure that the general public does not have to pay to maintain a system that benefits some far more than others. Bonds may be required for publicly owned, as well as privately owned, systems if the SWM policy requires that the developer and subsequent landowners pay for maintaining the system.

Responsibility for Maintenance. The responsibility for maintaining a SWM system can be subdivided into two components; these are the financial responsibility and the administrative responsibility. Some jurisdictions require that the landowners in the developed area assume both parts of this responsibility. Some require only that the landowners assume the financial burden, and some local governments assume all of the responsibility themselves. Assignment of the administrative responsibility to the landowners is often avoided because of the complications that arise if the landowners fail to perform the necessary maintenance. In most cases, the most equitable policy seems to be one requiring that the landowners pay for maintaining the system because they derive the primary benefit from it (that is, the use of their land). This question of benefits should be considered in deciding who shall bear the responsibility for maintenance. It is important to note that any policy that assigns any maintenance responsibility to the landowners must include a provision stating that this responsibility is transferable with the title to the land.

There are two distinct approaches to assigning maintenance responsibilities that are commonly used. In the first approach, all completed SWM systems are dedicated to a government agency and the government assumes all maintenance responsibilities. The other approach is to assign the maintenance responsibility to the landowners and require that maintenance be performed whenever inspection reveals that the system is not functioning properly. Variations and combinations of these two approaches are also used. Some jurisdictions assume control of systems in residential areas but require that systems in commercial and industrial areas be maintained by the owners. Other policies accept the system for government maintenance but levy special taxes on the property owners in the developed area to pay for maintenance of the system. Any policy that assigns maintenance responsibility to the landowners must include a statement making that responsibility transferable with ownership. Factors to be considered in developing a maintenance policy for SWM systems are listed below.

Acceptance of Systems by the Local Government. Often the local government feels that the only way of assuring proper maintenance of SWM systems is to assume that responsibility itself. Standards for acceptance should be developed in order to avoid the construction of systems that require excessive maintenance. As noted above, maintenance should be considered in the design of SWM systems, and designs that minimize maintenance requirements should be chosen whenever possible when the public is responsible for maintenance.

Authority to Perform. A statement giving the local government the authority to perform needed maintenance on privately owned systems will allow the government to protect the public interest. Sometimes the owners of a system will fail to perform necessary maintenance, even after being notified of violations. In such cases, the local government must have the authority to perform the maintenance

Stormwater Management Taxes. A stormwater management tax is often included in policies that assign financial responsibility for maintenance to the landowners. This tax is an additional property tax and is usually collected in the same manner with the same consequences for failure to pay. Such fees may be charged regardless of whether the administrative maintenance responsibility is assigned to the landowners, as in private systems, or assumed by the local government. The inclusion of a stormwater management tax is the easiest way for many jurisdictions to force the landowners to bear the financial burden of maintenance.

Multi-Site SWM Systems

Regional planning is essential if multi-site facilities are to be utilized extensively and effectively. In many situations, multi-site systems are advantageous for both technical and economic reasons. These facilities can be designed to correct existing stormwater problems, as well as those that may be caused by proposed development, if the planning agency has the authority to participate in the design and financing of SWM systems. To maximize the efficiency of SWM systems, the agency responsible for SWM should have the powers described in the following paragraphs. Unless a comprehensive regional planning effort is to be made, multi-site systems are often not feasible because of the time and effort required to evaluate plans for these systems.

Government Participation in SWM Systems. If the government agency responsible for SWM has the authority to participate in the financing and planning of SWM systems, then existing runoff problems can often be corrected and potential problems can be avoided. When a developer submits a preliminary SWM plan for a project, he may be required to construct a system capable of controlling runoff from other areas as well; the government usually finances the increased cost of the larger system. With comprehensive planning and government participation, the number and cost of SWM facilities can be minimized; this increased efficiency is expected to result in lower maintenance costs.

Runoff from areas that were developed before SWM systems were required causes problems in many localities; such problems can often be corrected by requiring a downstream developer to construct a system that will correct the existing problem while also controlling the runoff from the new site. If the comprehensive plan allows for development of additional upstream areas, the downstream developer may be required to build a system large enough to control anticipated future conditions; this is usually economically feasible only if the upstream areas are to be developed within a few years. If a downstream developer is required to build an oversize SWM system in order to control runoff from other areas, the government should provide financing and design assistance. In the case where the oversize system is needed to control anticipated future upstream development, the government can recover the costs of the larger system from the upstream landowners when they apply for permission to develop their land. Where the oversize system is needed to control an existing runoff problem, the government generally absorbs the cost because the correction benefits the general public. By participation in the financing and planning of facilities the government can develop an efficient SWM system for its entire jurisdiction.

Planning Responsibility for Multi-site Systems. The government is often the only party with the knowledge, authority, and staff required to design multi-site systems, but in some cases multi-site systems are proposed and designed by developers. A single developer may have two or more sites in one locality and his engineers may propose a multi-site system; alternately, two or more developers with adjoining property may decide to cooperate in building a multi-site system. Therefore, while the government should bear the primary responsibility for design of multi-site systems, developers should be encouraged to initiate multi-site planning where they find it to be feasible. However, the government must review all of the developers' plans to ensure that the goals of SWM are met before giving construction authorization.

Waivers of On-Site SWM Requirements. In areas where multi-site systems are used, some developers are not required to provide complete on-site SWM systems because the runoff from their sites is controlled elsewhere. In these cases, on-site requirements are waived and donations of money or land are usually required. The funds collected from these developers are used to finance the construction of multi-site systems. Donations of land may be required in order to accommodate larger SWM systems or for other public uses. In multi-site areas, the government must have the authority to waive on-site requirements in favor of donations.

Permits and SWM Plans

Most local jurisdictions have the authority and responsibility to require permits for construction and development. Permits are usually issued only after a review of the design plans submitted by the developer. Once the reviewing agency has evaluated the plans and determined that the goals of the SWM regulations will be achieved, building and land use modification permits can be issued. Some jurisdictions require permits for any land-disturbing activity while others only require permits for construction; in most populated areas, any change in land use can result in storm runoff problems, so permits should be required for all such activities. Specific elements of a comprehensive SWM policy that concern the permit process are discussed below.

Evaluation of Plans. In order to obtain a permit, the developer must provide a set of plans of the SWM facilities he intends to use to control any potential runoff problem. A government technical agency such as the DPW, planning board, or local soil conservation district must be assigned the responsibility for evaluating SWM plans. A specific set of design criteria must be developed and specified in the SWM regulations so that there is no ambiguity concerning the requirements. Permits for development should only be issued after an evaluation to ensure that the system will function as designed.

Exemptions from Permit Requirements. Projects of less than a specified size should be exempt from the permit requirements because the effects of land use changes on a small scale is usually negligible. The exact circumstances that warrant an exemption should be a function of the degree of imperviousness proposed for the future land use as well as the size of the area. The construction of a single house does not usually require a SWM plan, but a small parking lot may have the hydrologic impact of many detached houses.

Expiration and Renewal of Permits. Permits should have an expiration date to ensure speedy completion of projects; this feature will decrease the potential for erosion because the area will be permanently stabilized faster. In addition, temporary stabilization with straw, mulch, or vegetation should be used whenever possible during the construction period. Renewal of permits should not be automatic. Renewals should only be approved when they will benefit the public, but appeals of denials to an authority specified in the permit regulations should be allowed.

Fees for Review of Plans. Many governments charge the developer a fee for reviewing and evaluating SWM plans. Usually this fee is intended to offset the actual costs to the government. These fees are imposed to ensure that the general public is not forced to subsidize the costs of development. The amount of the fee may be a function of the size of the area being developed and of the proposed land use, but some jurisdictions have one set fee for all plans.

Liability Insurance. Developers are sometimes required to obtain liability insurance before a permit can be issued; this requirement is intended to reduce the possibility of the developer failing to complete development due to financial problems; thus, this requirement protects the public. Construction sites are often hazardous, and there have been cases where developers have been bankrupted by suits for damages. In these cases, the project is not likely to be completed in a timely manner and stormwater erosion on the disturbed area can become a problem. Therefore, liability insurance may be required to ensure the financial well-being of the developer, which will in turn help to ensure speedy completion of the project.

Modifications to Plans. Either the developer or the government may find it desirable to modify development plans at any time during the review and construction period. At the review and evaluation stage of the permit application process, the reviewing authority may demand changes in order to meet the goals of on-site SWM or to provide for multi-site systems. During the construction period, conditions are often found to be different from those assumed in preparing and reviewing the plans; depths of soil layers, availability of materials, and water table levels may cause problems with the construction based on approved plans. In cases where conditions are not as expected, either the developer or the government may modify the plans. The developer should be required to secure permission for any necessary modifications; permission should be obtained before construction of the proposed change is made.

In many jurisdictions the permit application process requires submission of both preliminary and final design plans. The preliminary plan is reviewed by the government authorities to ensure compliance with all pertinent regulations; then the plans are returned to the developer with the comments from the reviewing agency. Only then can the developer prepare his final plans, which must be submitted for a re-evaluation before permits may be issued.

Performance Bonds. To ensure that projects that are started are also completed, a performance bond is required before permits may be issued in many jurisdictions. The amount of this bond is usually the estimated cost of completing the construction of the required SWM facilities. Such bonds are forfeited to the government if the system is not completed within the period of time specified on the permit unless an extension or renewal is granted. The money collected in such cases of forfeiture is used to complete the system. Performance bonds thus help to ensure that the public will be protected from the adverse effects of the failure of a developer to complete his project. In many jurisdictions, provisions are made for the release of portions of the performance bond upon partial completion of the required SWM system.

Responsibility for Compliance with Permit Conditions. The responsibility for ensuring that the project is constructed as specified in the design plans should be assigned to the developer. While the government may inspect the construction and contractors may perform the actual work, the permit should be issued to the developer and compliance must be his responsibility. In cases where a performance bond is required, this assignment of responsibility is implicit; in other cases, a clause specifying the developer's responsibility should be included in the SWM policy.

Prevocation and Suspension of Permits. The government may find it necessary to revoke or suspend development permits if the construction is not proceeding in accordance with the approved plans. Regular inspections should be performed during the construction period, and if the conditions of the permit are not being met, then the developer should be notified. If violations are not corrected within a reasonable period of time, stop-work orders may be used to prevent the construction from proceeding. These stop-work orders are equivalent to temporary permit suspensions; if the developer fails to demonstrate his intent to correct the violations and complete the project as designed, his permit should be revoked. The power to suspend and revoke permits is one of the necessary tools for enforcement of SWM regulations.

Situations That Require Permits. A complete SWM policy must state exactly which situations require that permits be obtained. This statement will result in a less ambiguous policy and minimize the number of misunderstandings. A common way of explaining which situations require permits is to state that permits are necessary for all land use alterations affecting areas greater than a certain minimum size. Exemptions to these guidelines can then be listed separately. Exemptions are commonly given for changes in agricultural use or for projects involving a single private residence.

Specifications for Plans. In order for the reviewing agency to accurately evaluate the effects of a given design, the plans must include certain features. Complete descriptions of facilities, including material specifications, are required; the use of a standardized format will also expedite the review. Many jurisdictions require that plans be of a certain size, with blocks for the approval authorities to indicate their findings and comments on the plans. Also, many jurisdictions require that the plans be certified by a registered professional engineer before being submitted. The reason for these requirements is to facilitate the plan evaluation and thus help to ensure that the development will not adversely affect the general public.

Waivers and Appeals. In cases where preliminary investigation shows that a proposed change in land use will not have detrimental results, there is no reason to require a permit. Waivers from the permit requirements should be issued in such cases in order to ease the burden on both the developer and the permit agency. The most common way of providing for these waivers is to require that the developer demonstrate to the permit agency that the effects of his proposed project will either not be detrimental or will be insignificant. If the developer can demonstrate this to the satisfaction of the government, a waiver may be granted. It is wise to specify the procedure for appeal when waiver requests are denied in the SWM regulations, because the situation in which a developer is denied a waiver that he feels he deserves is almost certain to arise.

Zoning Approval. Before issuing permits for development, a consultation with the local zoning authority should be mandatory. This consultation should ensure that no zoning regulations will be violated by the proposed development so that the goals of the zoning statutes are met.

Grandfather Clause

Most SWM policies include a so-called grandfather clause in order to avoid legal complications. A grandfather clause merely provides exemptions to the provisions of the policy for projects that have already reached a certain stage of completion by the date the SWM regulations become effective. A developer who has already received approval for a project under the old regulations is likely to protest if informed that the project must be reevaluated in order to comply with a new set of regulations. In addition, a grandfather clause is often included because the reviewing agency is likely to be overloaded with work if required to review all the previously approved projects that have not yet been completed.

Severability of Provisions

A clause stating that each element of the SWM legislation is an independent statute is often included in such legislation. These clauses state the principle of severability of the provisions, meaning that if one provision is found to be unenforceable (due to lack of authority, unconstitutionality, or other reasons) the other provisions remain in force. This clause strengthens the legislation by ensuring that minor faults will not result in the entire ordinance being invalidated.

APPENDIX D

EVALUATION OF METHODS OF DETERMINING URBAN RUNOFF CURVE NUMBERS

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Evaluation of Methods for Determining Urban Runoff Curve Numbers

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ABSTRACT

THE runoff curve number, which integrates land cover, soil type, and antecedent soil moisture conditions, is an important input parameter to the Soil Conversation Service (SCS) hydrologic procedures. The determination of curve numbers is a time consuming procedure requiring detailed data on land cover and soils. Data from 175 urban watersheds in various sections of the United States were used to evaluate whether runoff curve numbers developed by measuring the fractions of land use and soils separately and integrating them using a weighted average scheme (the contingency table approach) or using land use maps developed by the U.S. Geological Survey (USGS) were comparable to the curve numbers developed by the conventional SCS methods. The results indicated that estimates of runoff curve numbers are sensitive to the land use classification system and not very sensitive to the method of integrating soils and land cover data for watersheds of 0.1 to 472 km². Also, from the data set, a set of curve numbers was developed for the four hydrologic soil groups for level II USGS land use categories.

INTRODUCTION

Many of the hydrologic models used in the design and planning of urban water resources projects require input parameters that are defined in terms of land cover so that alternative forms of development can be planned and future changes assessed. The series of hydrologic models developed by the Soil Conservation Service (SCS, 1969, 1972, 1975) are among the most widely used models in water resources planning and design. Originally, they were developed for agricultural areas; however, they have been extended for use in urban areas (SCS, 1975). These models require a value of the runoff curve number (CN), which is an indication of the flood runoff potential of a watershed. The CN is a function of the land cover, the soil type and the antecedent soil moisture conditions. Four hydrologic soil groups (A, B, C, D) are used to reflect variation in runoff potential for different soil types. The National Engineering Handbook (NEH) Section 4 (SCS, 1972) lists the many soil types and their corresponding hydrologic soil group. The soil type can be identified from soil surveys which are available from local SCS offices. Land cover is separated into the categories of Table 1, which also shows the CNs for average soil moisture conditions (antecedent soil moisture condition II).

A watershed is rarely composed of both homogeneous land cover and soils of the same hydrologic soil group. Therefore, it is necessary to integrate the soils data with the land use information to derive a value of the runoff CN for the watershed or subarea. The determination of CNs is often a time consuming procedure because of the way that land cover and soils data are integrated.

Several investigators (Ragan and Jackson, 1980; Slack and Welch, 1980; Bondelid et al., 1980) have investigated the use of Landsat data for estimating curve numbers. Jackson and Ragan (1977) showed that land cover data can be obtained in a cost effective manner using Landsat. The past studies using Landsat were performed with watersheds in Georgia, Maryland and Pennsylvania and included a range of land covers such as forest, agriculture, urban, strip mine and wetlands. The investigators found that Landsat data could not be used to identify land cover at the level of detail described in Table 1. However, Ragan and Jackson (1980) found that by using a less detailed land cover classification system developed for Landsat data, the curve number could be estimated with an accuracy of two curve numbers.

Bondelid et al. (1980) investigated the use of the land cover maps developed by the USGS for three rural watersheds in Pennsylvania. They studied the USGS land use maps because these maps are available for about 80 percent of the United States. Bondelid et al. (1980) estimated curve numbers for the USGS land use classification system and found that for the rural study area the data produced curve numbers comparable to those produced by conventional methods.

The objectives of our study were to evaluate whether urban runoff curve numbers developed by integrating land use and soils separately using a weighted average scheme (contingency table approach) or using USGS land use maps were comparable to the runoff curve numbers developed using the conventional SCS approaches.

METHODS FOR INTEGRATING LAND USE AND SOILS DATA

The SCS hydrologic methods are very sensitive to the CN (Hawkins, 1975). Therefore, it is important to obtain estimates of the CN that accurately reflect the runoff potential of a watershed. Three critical aspects of estimating the CN are the determination of soils, the

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TABLE 1. RUNOFF CURVE NUMBERS FOR SELECTED AGRICULTURAL, SUBURBAN, AND URBAN LAND USE. (ANTECEDENT MOISTURE CONDITION II, and $I_a = .2S$) (FROM SCS TR-55 1975)

Land use description		Hydrologic soil group A	B	C	D
Cultivated land*: Without conservation treatment		72	81	88	91
With conservation treatment		62	71	78	81
Pasture or range land: poor condition		68	79	86	89
good condition		39	61	74	80
Meadow: good condition		30	58	71	78
Wood or forest land: thin sstand, poor cover, no mulch		45	66	77	83
good cover†		25	55	70	77
Open space (lawns, parks, golf courses, cemeteries, etc)					
good condition: grass cover on 75% or more of the area		39	61	74	80
fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business area (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential‡:					
Average lot size	Average % impervious §				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lots, roofs, driveways, etc.‖		98	98	98	98
Streets and roads:					
paved with curbs and storm sewers‖		98	98	98	98
gravel		76	85	89	91
dirt		72	82	87	89

* For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972.

† Good cover is protected from grazing and litter brush cover soil.

‡ Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

§ The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

‖ In some warmer climates of the country, a curve number of 95 may be used.

determination of land cover and the method used for integrating the land cover data with the soils data.

The Conventional Approach

The conventional method of determining curve numbers involves low altitude aerial photographs, land use maps and onsite investigations. To integrate land cover and soils data, the hydrologic soils maps are overlaid on aerial photographs or quad sheets that delineate the land cover. The area for each land cover-soil type complex is planimetered, and a curve number for the entire watershed or a subwatershed is estimated by weighting the CNs for each unique land cover-soil type complex.

The Contingency Table Approach

We hypothesized that the effort required to estimate watershed CNs could be reduced, without an appreciable loss of accuracy, by replacing the overlaying of soils and land use delineation with a separate analysis of soils and land use data; this would make it possible to use computerized data bases of land use and soils data. Specifically, the percentage of each soil group would be determined independently from the percentages of the different land covers. A weighted mean CN would then be determined using the product of three values: the percentage of land use, the percentage of a soil group and the CN associated with the soil and land use:

$$CN_W = \sum_{i=1}^{n} \sum_{j=1}^{4} L_i S_j CN_{ij} \qquad [1]$$

in which CN_w = the weighted CN for a watershed, n = the number of different land uses within the watershed, L_i = the fraction of land use i, S_j = the fraction of soil group j (i.e., A, B, C or D) and CN_{ij} = the curve number associated with L_i and S_j. This method will be identified as the contingency table approach.

ESTIMATING LAND USE WITH THE USGS LAND USE SYSTEM

The USGS is currently producing land use and land cover maps for the United States with a base of 1:250,000. The data are being digitized and will be made available in both graphic and digital form. This Geographic Information Retrieval Analysis System (GIRAS) is designed to manipulate, analyze and output the land use and land cover data (Mitchell et al., 1977). The maps currently are available for level II classification. A minimum mapping unit of about 4 ha (10 acres) is used for all urban and water covers and a few other categories. For all other cover types a minimum unit of 16 ha (40 acres) is used. Each spatial mapping unit must be assigned to a single category. Because these maps are readily available for a major portion of the United States and are frequently updated, they provide a convenient alternative to the task of obtaining land use data from individual sites by collection and interpretation of low level aerial photographs.

DATA BASE

Land cover and soils data from 175 urban watersheds were used to investigate the study objectives. The distribution of watersheds over the United States is given in Fig. 1. The size of the watersheds ranged from 0.1 to 472 km² with a mean size of 41.4 km². The land cover, soils data and conventionally determined curve numbers were compiled by SCS state hydrologists for the particular

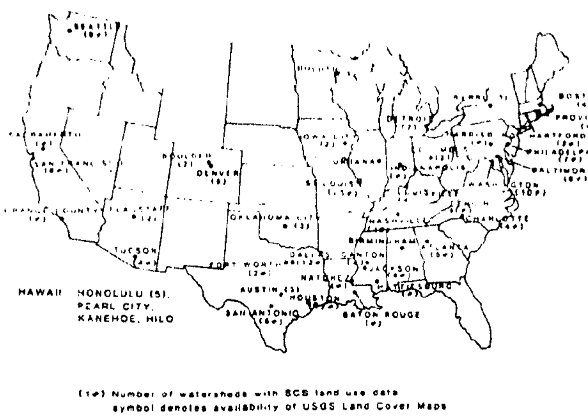

FIG. 1 Metropolitan areas included in the land use study.

state in which the watershed was located. The SCS normally uses the curve numbers shown in Table 1; however, local CN values that differed from those in Table 1 were used for ten of the 175 watersheds. Five of the ten watersheds are located in the state of Washington. Differences in CNs covered the spectrum of land cover types, including pasture, forest, residential and commercial. The use of different curve numbers appears to be location dependent because of differences in runoff potential for similar land covers in different parts of the country.

USGS land cover data were available for 120 of the 175 watersheds. These 120 watersheds are identified separately in Fig. 1. The USGS data were compiled by hydrologists of the USGS through a cooperative project with the Federal Highway Administration (Sauer et al., 1980). The size of the watersheds ranged from 0.8 to 472 km² with a mean size of 47.0 km². The frequency distribution of the SCS hydrologic soil groups is given in Table 2 for both the 175 and 120 watershed groupings.

TABLE 2. FREQUENCY DISTRIBUTION OF SCS HYDROLOGIC SOIL GROUPS FOR THE WATERSHEDS

Hydrologic soil group	Based on 175 watersheds		Based on 120 watersheds	
	Mean, %	Standard deviation, %	Mean, %	Standard deviation, %
A	1.91	6.93	1.46	5.52
B	36.66	33.51	32.04	31.72
C	39.84	35.04	42.43	34.06
D	21.59	32.55	24.07	36.08

Land Use Distributions

The frequency distribution for the SCS land use data of the 175 watersheds is given in Table 3. Approximately 44 percent of the land use is classified as residential. Other significant land use types are pasture, forest, open space and commercial.

The USGS developed a land use classification system that includes several levels of detail (Anderson et al., 1976). Table 4 shows the level II land use categories used in the study. The data base included 120 watersheds for which USGS land use data were available. Table 5 shows the frequency distribution of land use in the 120 watersheds on the basis of USGS land use classification. Approximately 51 percent of the land use is classified as residential. Other significant land use types are commercial, urban and forest.

EVALUATION OF CURVE NUMBERS ESTIMATED USING THE CONTINGENCY TABLE APPROACH

The conventional SCS approach for estimating curve numbers was used as the true value for determining the accuracy of CNs estimated using the contingency table approach, although CN values obtained using the conventional approach are also estimates and, thus, are subject to error. Use of the CNs estimated by the conventional approach as a standard was the most feasible criterion for estimating the accuracy of CNs derived using the contingency table approach. The residuals (difference between the conventional and contingency table approach curve numbers) were computed for each of the 175 watersheds; the distribution of the residuals is given in Fig. 2. The residuals had a mean of −0.064,

TABLE 3. FREQUENCY DISTRIBUTION AND CORRELATION COEFFICIENTS OF THE RESIDUALS OF THE SCS LAND USE FOR THE 175 WATERSHEDS.

Land use description	Mean, %	Standard deviation, %	Correlation Coefficient
Cultivated land: Without conservation treatment	1.50	6.64	0.00
Cultivated land: With conservation treatment	1.51	5.68	0.14
Pasture or range land: poor condition	2.94	11.28	0.00
good condition	6.29	14.01	0.09
Meadow: good condition	2.14	6.69	−0.01
Wood or forest land: thin stand, poor cover, no mulch	5.42	12.93	−0.02
Wood or forest land: good cover	10.47	18.99	−0.08
Open space (lawns, parks, golf courses, cemeteries, etc)			
good condition: grass cover on 75% or more of the area	6.23	9.93	−0.18
fair condition: grass cover on 50% to 75% of the area	3.10	8.03	−0.03
Commercial and business area (85% impervious)	6.84	9.03	0.03
Industrial districts (72% impervious)	4.91	9.77	0.02
Residential:			
Average lot size — Average % impervious			
1/8 acre or less — 65	10.17	17.78	0.03
1/4 acre — 38	16.71	23.53	0.02
1/3 acre — 30	5.66	12.82	0.02
1/2 acre — 25	6.43	13.71	0.00
1 acre — 20	5.06	13.59	−0.01
Paved parking lots, roofs, driveways, etc.	1.71	3.29	0.03
Streets and roads:	2.89	4.29	0.02
paved with curbs and storm sewers			
gravel			
dirt			

TABLE 4. USGS LEVEL II LAND COVER CURVE NUMBERS.

Land use description*	Hydrologic soil group A	B	C	D
Residential	61	70	83	87
Commercial and services	87	89	91	93
Industrial	81	88	91	93
Transportation, communications, utilities	84	87	90	92
Industrial and commercial	83	85	89	91
Mixed urban or built-up land	55	71	83	88
Other urban or built-up land	72	78	84	88
Crop or pasture	63	69	77	82
Orchards, groves, nurseries, and ornamental horticultural areas	23	52	69	75
Deciduous forest land	55	63	71	75
Evergreen forest land	35	60	73	79
Mixed forest land	43	64	75	81
Lakes	100	100	100	100
Reservoirs	100	100	100	100
Forested wetland	45	66	77	83
Strip mines, quarries, and gravel pits	74	82	87	89
Transitional areas	63	74	81	84
Shrubs and brush	49	69	79	84
Other agricultural	59	74	82	86

*Only the level II categories used in the study are included.

TABLE 5. FREQUENCY DISTRIBUTION FOR USGS LAND USE DATA FOR THE 120 WATERSHEDS

Land use description	Mean, %	Standard deviation, %
Residential	50.90	22.75
Commercial and services	10.63	7.87
Industrial	1.74	3.23
Transportation, communications, utilities	2.32	3.82
Industrial and commercial	0.89	2.50
Mixed urban or built-up land	0.93	2.93
Other urban or built-up land	5.34	5.72
Crop or pasture	12.83	20.18
Orchards, groves, nurseries, and ornamental horticultural areas	0.18	1.17
Deciduous forest land	5.81	12.84
Evergreen forest land	1.15	4.54
Mixed forest land	4.88	12.14
Lakes	0.04	0.26
Reservoirs	0.09	0.25
Forested wetland	0.04	0.45
Strip mines, quarries, and gravel pits	0.29	1.38
Transitional areas	1.02	2.52
Shrubs and brush	0.87	4.83
Other agricultural	0.0075	0.04

which indicated the absence of a significant bias, and a standard deviation of 1.50, which indicated a small level of imprecision.

In an attempt to identify the cause of the imprecision, we first analyzed the 12 most extreme residuals to determine if the error variation was due to a bias in drainage area size, soil type or land cover. No bias was found due to drainage area size, soil type or land use.

The second analysis involved correlation analyses between the residuals and drainage area and land use. The correlation coefficient for the 175 residuals and the drainage areas of the corresponding watersheds was −0.062, which is not significant. However, Fig. 2, a graph of the residuals versus drainage area size, shows an apparent trend toward wider variation for the smaller drainage areas, especially those smaller than about 26 km². The correlation coefficients were also computed between the residuals and the percentages of each of the SCS land covers; the resulting values are given in Table 3. The two largest correlation coefficients are 0.18 and 0.14, with 14 of the 18 values less than or equal to 0.03 indicating also land cover did not appear to cause the imprecision.

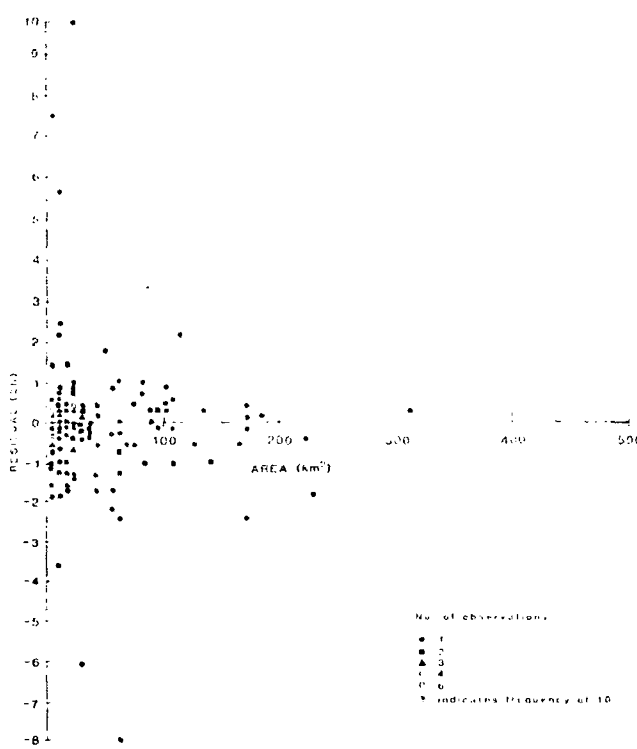

FIG. 2 Plot of residual curve numbers vs. the drainage area size for the 175 watersheds.

EVALUATION OF CURVE NUMBERS ESTIMATED USING USGS LAND COVER

The data for the 120 watersheds were used to define the CNs for the level II land use categories shown in Table 4. Using the description of the SCS and USGS land use categories, the SCS categories were grouped to correspond to the USGS categories and curve numbers were derived maintaining internal consistency between soil groups. When these curve numbers were used on the 120 watersheds, they were found to have a positive bias of approximately one curve number and an imprecision of approximately 3.5 curve numbers when compared with the curve numbers that were estimated using the conventional approach. Thus, the initial estimates of the curve numbers using USGS land use data were adjusted until the bias was reduced to near zero and the imprecision was made as small as possible. The best-fit curve numbers are shown in Table 4.

Because the same data base that was used to fit the CNs for the USGS land use classification system was used to estimate the bias and precision, an unbiased measure of the accuracy of the curve numbers is not available. However, because the data base included such diversity of geographic location, degree of urbanization and soil type, the CN values of Table 4 may be considered reasonably accurate for making runoff computations in the future; that is, the estimates of bias and precision are good estimates of the accuracy obtainable when using the values of Table 4. In comparing the CNs derived using the USGS land use system with the CNs derived conventionally, the bias was near zero and the imprecision was approximately 3 curve numbers, which

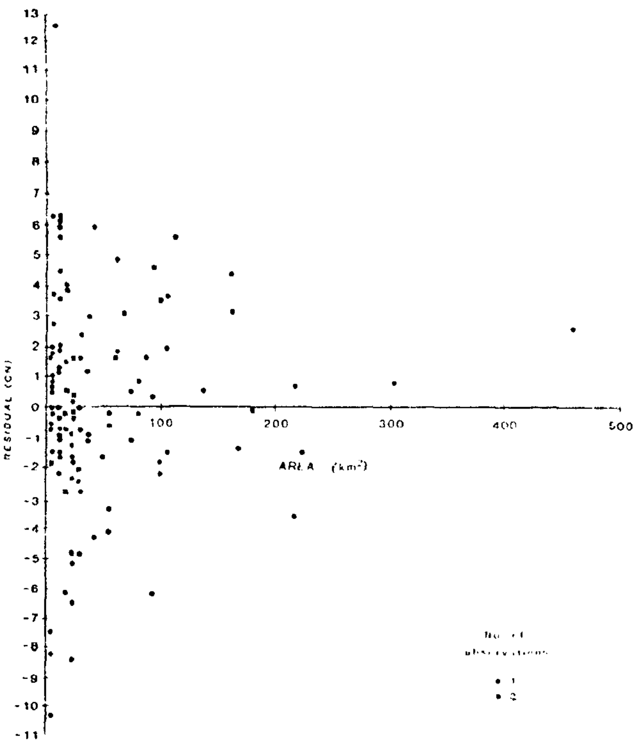

FIG. 3 Plot of residual curve numbers vs. the drainage area size for the 120 watersheds.

must be considered significant, since it can cause up to about a 15 percent error in runoff predictions. The distribution of the residuals is shown in Fig. 3 for the 120 watersheds. Approximately 91 percent of the residuals were within 6 curve numbers of the conventionally derived values.

Analysis of the 12 most extreme residuals indicated a tendency for the watersheds corresponding to the largest residuals to contain less residential and industrial land and corresponding more than average of forest land. An attempt at decreasing the level of imprecision by further adjusting the CN values of Table 4 was made with no success. Although the cause of the imprecision could not be identified conclusively, the limited separation of residential land use seemed to be a major factor. Separation of the USGS residential land use into categories according to lot size in a manner similar to the SCS residential land use separation would, perhaps, reduce the imprecision to the USGS land use CNs.

A correlation analysis between the 120 residuals and either drainage area or the USGS land use categories was also performed in an effort to reduce the imprecision. This analysis indicated that the size of the drainage area or the USGS land use categories did not cause the imprecision. However, Fig. 3, a graph of the residuals versus drainage area size, shows an apparent trend toward wider variation for smaller drainage areas, especially those smaller than about 26 km².

SUMMARY AND CONCLUSIONS

The objectives of this study were centered around two problems: reducing the effort required to integrate land use and soils data in the estimation of SCS runoff curve numbers and simplifying the method of obtaining land use data.

The conventional approach to CN estimation involves overlaying land use and soils data. Because planimetering areas having homogeneous land use and soils is very time consuming, we investigated the accuracy of CNs estimated by integrating independently the estimates of the percentages of each soil group and the land use. The resulting estimates of CNs were accurate, indicating that the detailed conventional approach may not be necessary. However, the results suggested that the conventional approach should be used for watersheds of less than about 26 km². This new method of integrating the land use and soils data is important because the independent estimation of the two inputs would make possible the use of computerized data bases in determining curve numbers.

Land use maps developed by the USGS are readily available and frequently updated, thus providing a convenient alternative to the task of obtaining land use data from individual site collection and/or low level aerial photograph interpretation. Reasonable estimates of the watershed CN, especially for watersheds larger than about 26 km², were made using a set of curve numbers developed for the level II USGS land use categories. While the CNs were not in total agreement with those estimated from conventional field inspection, the level of imprecision was satisfactory for most types of hydrologic analysis.

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APPENDIX E

COMPARISON OF URBAN FLOOD FREQUENCY PROCEDURES

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EVALUATION OF THE SCS URBAN PEAK FLOW METHODS[1/]

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ABSTRACT

The peak flow hydrologic methods in the SCS TR-55 are widely used for planning and design on small urban watersheds. The urbanization factors of the TR-55 methods (i.e., the lag factors for the graphical method and the peak factors for the chart method) were apparently not directly based on analyses of measured data and have not previously been systematically tested. Therefore, the objective of this study was to test the TR-55 methods and recalibrate the urbanization factors if the methods were found to be biased. A data base was compiled for 51 small urban watersheds under 4,000 acres (1,600 ha) in the United States. The annual maximum flood series were used to develop Log Pearson Type III frequency curves, from which peak discharge estimates were estimated for the following return periods: 2-year, 5-year, 10-year, 5-year, 50-year, and 100-year. The bias and accuracy of the five methods were evaluated using the Log Pearson Type III estimate as the expected true value. The results indicate that no urbanization correction factors were needed for the graphical method when time of concentration was determined using the velocity method. Modifications to the urban adjustment factors used in the chart method were proposed.

Key Words: Hydrology, urban floods, frequency analysis, runoff, drainage, stormwater, peak flow.

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EVALUATION OF THE SCS URBAN PEAK FLOW METHODS[1/]

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INTRODUCTION

Man's impact through urbanization dramatically affects the flood response of a watershed. The conversion from pervious to impervious surfaces typically inhibits infiltration and groundwater recharge and reduces surface roughness, surface retention, and depression storage. Thus, rainfall losses are reduced and direct storm runoff is increased. The alterations and improvements made to the existing drainage networks cause a more rapid runoff response because of increased flow velocities over smooth surfaces to drainage inlets and then by pipe to improved natural channels. Thus, the impervious surfaces and improved drainage systems increase the amount of runoff and reduce flow travel times, which produces a flood hydrograph of increased magnitude with a shortened time-to-peak and recession limb.

With urban growth and development, there is an ever-increasing need for flood information and estimating techniques for use in areas where little or no data exist (6). Therefore, design flood estimation procedures capable of

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accounting for urbanization are required at two levels (1). First, simple procedures are needed both to design drainage facilities on small basins and to evaluate alternative developmemt schemes. Second, more detailed models are needed to design major drainage facilities. The simple procedures should produce a flood discharge of a given frequency, while the complex models should compute and route the design flood hydrograph through the drainage system. A recent survey (6) of Federal agencies, State highway departments, and the private sector indicated that about 80 percent of hydrologic design problems are currently evaluated using "simplified" hydrologic methods.

The Soil Conservation Service TR-55 (4) presents two methods for estimating peak discharge rates in urbanizing areas. One method, identified as the graphical method, is described in Chapter 5 of TR-55. The second method, identified as the chart method, is described in Chapter 4 and appendices D and E of TR-55. These methods are widely used because of their national applicability and computational simplicity (1). The SCS TR-55 procedures are based on well known hydrologic techniques (4), with the addition of adjustment factors to account for increased imperviousness and modifications to the hydraulic length of channels resulting from urbanization. Although the SCS hydrologic procedures have been extensively used, the validity of the urbanization adjustments have not been verified. Hence, the objective of this study was to evaluate the SCS TR-55 urban peak discharge methods using data from urban locations throughout the United States, and if the methods are found to be inaccurate or biased, then they will be recalibrated.

SCS TR-55 PROCEDURES

Graphical Method

Input requirements for the Graphical method are the time of concentration (hours), the drainage area (sq. mi.), and the depth of precipitation (inches) for the 24-hour SCS type II storm distribution. A graph is provided (Fig. 5.2 of TR-55) relating the unit peak discharge (cubic feet per second per square mile per inch of runoff) to the time of concentration for the SCS type II, 24-hour storm distribution. The volume of runoff is computed using the SCS rainfall runoff relationship.

The SCS recommends a velocity method for estimating the time of concentration. This method is based on the velocity of flow and the travel length and requires as input the hydraulic length, slope, and land use type. The average flow velocity should be determined for each segment of the flow path using the average slope and land use. The time of concentration for each flow segment equals the ratio of the hydraulic flow length to the velocity. The watershed time of concentration equals the sum of times of the concentration of the flow segments. TR 55 does not specify any limitations on the drainage area when using the velocity method to calculate the time of concentration for use with the graphical method.

The volume of direct runoff required for the graphical method is computed using the depth of the 24 hour precipitation for a given recurrence interval and the runoff curve number (CN). The curve number is a function of the land use, the hydrologic soil group, and the antecedent soil moisture condition. An estimated peak discharge is computed as the product of the unit peak discharge, the drainage area, and the depth of runoff. The graphical method is applicable for watersheds where: 1) valley routing is not required, and 2) land use, soil group, and cover are reasonably uniformly distributed throughout the watershed (4).

Chart Method

The input requirements for the SCS Chart method are the land use, the SCS hydrologic soil groups, the hydraulic length (feet), the drainage area (acres), the percentage and general watershed location of pond and swampy areas (%), the 24-hour precipitation for the given recurrence interval (inches), the percent of imperviousness (%), the percent of the hydraulic length modified (%), and the watershed slope (%). Charts are provided in appendix D of TR-55 for determining the unit peak discharge (cfs per inch of runoff) for nonurbanized conditions. This discharge is based on the drainage area, the watershed slope, and the curve number. Adjustments are provided in appendix E of TR-55 for conditions where pond and swampy areas exist and for conditions where either the average watershed slope or the watershed shape vary significantly from those used in the initial calculations.

The depth of direct runoff is computed using the depth of the 24-hour precipitation for the given recurrence interval and a runoff curve number. With the adjusted unit peak discharge and runoff depth, a nonurbanized peak flow can be computed. The peak flow is then adjusted for the impact of urbanization on travel time using adjustment factors (Figure 2) given in Chapter 4 of TR-55. The adjustment factors are based on the percent imperviousness and the percent modification of the hydraulic length. This method is applicable for watersheds where: 1) the drainage area is less than 2,000 acres; 2) valley routing is not required; 3) the land use, soil group, and cover are reasonably uniformly distributed throughout the watershed; and 4) where accurate estimates of the time of concentration are not available.

ACCURACY ASSESSMENT

In testing a method for predicting the peak discharge, it is necessary to establish criteria for assessing the quality of prediction. The criteria should reflect both systematic and random variation about the true, or accepted, value of the peak discharge. In general, one wants a prediction method that, on the average, closely approximates the true value; this is a measure of the bias. As a general rule, the bias is the average difference between the predicted and true values and, therefore, is a measure of the systematic variation of the computed estimates from the true value. When comparing peak discharge estimates, it is necessary to modify the general rule for computing bias because the true value is different on each watershed. In terms of a plot of the predicted and true peak discharges, the bias can be measured by the difference between the slope of the line that provides the "best" fit to the data points (i.e., in a least squares sense) and the slope of the line equaling the predicted and true values. Therefore, the bias of the model is:

$$\text{Bias} = \left(\frac{\sum_{i=1}^{n} Q_p - \hat{Q}_p}{\sum_{i=1}^{n} Q_p^2} \right) - 1.0 \tag{1}$$

in which $\hat{Q}_p$ is the predicted peak discharge, Q_p is the true peak discharge, and n is the number of watersheds. A positive bias value indicates over-prediction, while a negative bias value indicates underprediction.

In addition to the systematic variation, one also expects nonsystematic, or random, variation of estimated peak discharges about the true value; this random variation results from factors affecting runoff rates that are not accounted for by the model. The accuracy is a measure of the closeness of a predicted peak flow to the true peak flow. Representing the accuracy in the form of a standard error, it is computed by:

$$\text{Accuracy} = \left(\frac{1}{n-1} \sum_{i=1}^{n} \left(\frac{\hat{Q}_p - Q_p}{Q_p} \right)^2 \right)^{0.5} \qquad (2)$$

The accuracy, as given by Eq. 2, measures the deviation of the predicted discharge from the true discharge; it is standardized by dividing by Q_p to reduce the effect of variation in the size of the events. That is, if Eq. 2 were not standardized, then errors for a few of the larger events might dominate the computed value of the accuracy.

DATA BASE

Data used in this study were obtained from the U.S. Geological Survey National Urban Frequency Study, which was reported on by Sauer, et al. (3). This data base includes a comprehensive list of topographic, climatic, land use variables, including indices of urbanization, and urban and rural flood frequency estimates for 269 watersheds located throughout the United States. The watersheds range in size from 76 acres to 73,000 acres (30 to 29,000 ha) and have exhibited a relatively constant degree of urbanization over the period of record. Data from a total of 56 metropolitan areas located in 31 states are included in this data base. For this study, 68 watersheds were used with drainage areas less than 4,000 acres (1,600 ha) and no channel storage. The location and distribution of the watersheds are shown in Figure 1. Watershed characteristics are summarized in Table 1. The impervious area averaged 24 percent of the total watershed area, and ranged from 3 to 98 percent. The percentage of the hydraulic length modified averaged 31 percent and ranged from 0 to 100 percent. Of the 68 watersheds, 51 had measured times of concentration. These time of concentration values were determined using the velocity method. Watersheds in California, and Oregon were excluded from this testing because the SCS procedures in TR-55 are primarily for areas where a SCS type II storm is applicable. Also, since it has been shown (2, 7) that the procedures do not perform well on watersheds exhibiting low slope (one-half percent and less), the Texas watersheds were excluded, reducing the data base to 40 watersheds having detailed channel information and measured time of concentration values.

Flood frequency estimates for peak flows obtained from the data base were derived from analyses of annual peak flow series. Log Pearson Type III procedures, as recommended by the Water Resources Council Bulletin 17A, (5), were used to fit each frequency curve to the data. A peak discharge was estimated for each watershed for recurrence intervals of 2-, 5-, 10-, 25-, 50-, and 100-years. While these are actually predicted values themselves, for these analyses they were assumed to be the true values. The record length of record for the 68 watersheds of concern ranged from 9 to 18 years, averaging about 13 years.

RESULTS

Graphical Method

The Graphical method was applied to the 40 watersheds. The resulting values of the bias (Eq. 1), accuracy (Eq. 2), and the mean and standard deviation of the errors are shown in Table 2. Standardized bias values indicate that the graphical method is essentially unbiased over all return periods (bias values are not statistically different from zero at the 1 percent level). The mean of the errors shows a positive bias ranging from 210 cfs (5.9 m^3/s) for the 2-year to 440 cfs (12.5 m^3/s) for the 100-year return periods. The residual errors were examined to determine if an adjustment for the time of concentration measurements could be developed, based on watershed characteristics such as area, percent impervious area, etc., that would more accurately reflect urban conditions and, in turn, improve the bias and accuracy of the procedure. It was determined that the errors were not correlated with watershed characteristics and adjustments could not be developed that would significantly improve the accuracy of the model.

Chart Method

Because the TR-55 Chart method is based on the SCS lag formula, its use is limited to 2,000 acres (800 ha). This limitation was believed to be arbitrary and, therefore, was tested using a straight-line extrapolation of the TR-55 Appendix D charts. Because the data base used in this analysis contained a significant break in watershed size at about 4,000 acres (1,600 ha), it was decided to use those watersheds for which the other limitations of the method were satisfied, to test the applicability of the method for watersheds up to 4,000 acres (1,600 ha). Results of this analysis for the six return periods are summarized in Table 3. Comparing the statistics resulting from the analyses of the 45 watersheds less than 2,000 acres with those resulting from the analyses of the 68 watersheds less than 4,000 acres (1,600 ha), suggests that the drainage basin size limit can be extended to drainage areas up to at least 4,000 acres (1,600 ha) without a change in the bias or accuracy. Based on these results, all further analyses were performed using watersheds having drainage areas up to 4,000 acres (1,600 ha).

The Chart method of TR-55 provides for a correction of the percentage of ponds and swamps in the watershed; the adjustment depends on the location of the ponded areas, with different correction factors provided for ponding in the lower, middle, and upper portions of the watershed. Because the data base used for testing did not indicate the location of the ponds, the bias and accuracy statistics were evaluated for each of the three options to determine the effect of the location of the ponding. Statistical testing of the resulting values showed that there was no significant difference; therefore, for our analysis we assumed all ponding to be located in the middle of the watersheds. Lack of a significant difference resulted because there was very little ponding (i.e., an average of less than 1 percent) in the watersheds included in the test.

The Chart method was applied to the 40 urban watersheds. Resulting values of the bias (Eq. 1), accuracy (Eq. 2) and the mean and standard deviation of the errors are shown in Table 4. The statistics for the Chart method (Table 4) are similar to those for the Graphical method (Table 2). The Chart method incorporates urban factors based on the percentage of imperviousness and the percentage of the hydraulic length modified to adjust the rural peak discharge for urban conditions. Because these urban factors are essentially an addition to the rural SCS procedures to account for urbanization and are apparently based on limited data, our efforts to improve the Chart method were directed to these urban correction factors. The data base was subdivided according to whether or not channel modifications were made. Analysis of the residual errors for the 9 watersheds with no channel modifications indicated that the adjustment factor for the percentage of imperviousness (Figure 2) could not be improved significantly. Most of the watersheds in the data base had impervious areas from 0 to 30 percent; thus, the results were considered to be a verification of the impervious area adjustment factor for values less than 30 percent.

In testing the adjustment factor for hydraulic length modified relationship (Figure 2) we thought it would be more reproducible and hydrologically rational if this factor reflected only channel modifications rather than modifications to the total time of concentration hydraulic length path, which is the present SCS interpretation. Statistics reflecting the change in the hydraulic length definition are summarized in Table 4. Comparison of these statistics for the two definitions of hydraulic length indicates that there is no significant difference. Analysis of the residual errors using the new definition of hydraulic length indicated that the hydraulic length modification factor need not be a function of curve number and that adjustment factor could be best represented by the curve for a curve number of 90 (Figure 2). A comparison of the statistics in Table 4 indicates that the one curve is slightly better primarily because of the significant decrease in the mean error.

CONCLUSIONS

Evaluation of the SCS urban peak flow methods on 40 urban watersheds located around the United States indicated:

(1) A time of concentration adjustment to reflect urban conditions for use in the SCS Graphical method is not necessary when time of concentration is determined by the velocity method.

(2) The SCS Chart method can be used for areas up to 4,000 acres (1,600 ha) without a loss of accuracy.

(3) The adjustment factor for the percentage of imperviousness (Figure 2) that is used with the Chart method was found to be adequate.

(4) The adjustment factor for the hydraulic length modified that is used with the Chart method should be limited to modification of the channel only, and the hydraulic length modification adjustment relationship is not a function of curve number. The curve for a curve number of 90 (Figure 2) provides the most accurate prediction.

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Table 1.--Watershed Characteristics

	Mean	Standard deviation	Range Minimum	Range Maximum
Drainage area (mi^2)	2.6	1.5	0.12	6.17
Watershed length (mi^2)	2.8	1.1	0.5	5.3
Watershed slope (%)	4.2	3.5	0.5	20.0
Pond and swampy area (%)	0.8	1.6	0.0	8.0
SCS runoff curve number	80.	5.7	65.	91.
Impervious area (%)	24.	16.4	3.	98.
SCS Hydrologic soil type	C	---	A	D
Hydraulic length modified (%)	31.	19.	0.	100.

Table 2.--Results of Testing the SCS Graphical Method (n = 40)

Statistic	Return period (years)					
	2	5	10	25	50	100
Bias	0.09	0.06	0.5	-0.02	-0.4	-0.07
Accuracy	0.96	0.81	0.97	1.02	1.11	1.17
Mean Error (cfs)1/	210	312	383	380	438	440
Standard deviation of errors (cfs)	640	992	1,259	1,616	1,903	2,267

1/ To convert cfs to m^3/s multiply by 0.0283

Table 3.--Evaluation of the Effects of Watershed Size on the Bias and Accuracy of the SCS Chart Method

Method	Limit on area (acres)[1/]	Return period: 2 years		5 years		10 years		25 years		50 years		100 years	
		Bias	Accuracy	Bias	Accuracy	Bias	Accuracy	Bias	Accuracy	Bias	Accuracy	Bias	Accuracy
Chart	2,000 (800 ha)	-0.46	1.11	-0.42	1.23	-0.38	1.26	-0.37	1.59	-0.36	1.55	-0.33	2.48
Chart	4,000 (1,600 ha)	-0.28	1.08	-0.29	1.16	-0.28	1.21.	-0.32	1.46	-0.34	1.47	-0.34	2.17

[1/]There were 45 watersheds less than 2,000 acres (800 ha) and 68 less than 4,000 acres (1,600 ha).

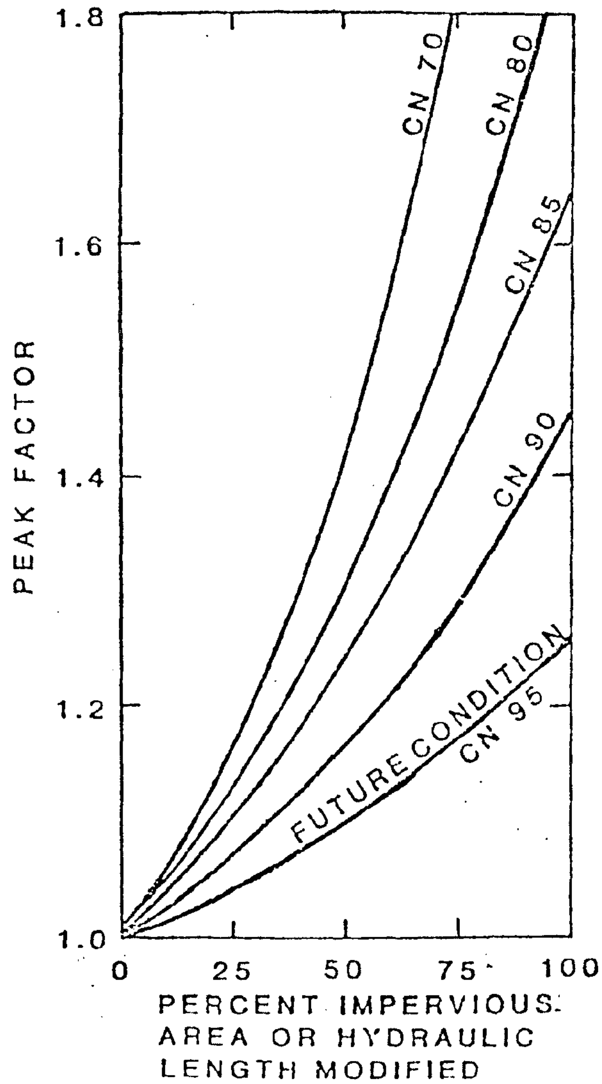
Table 4.--Results of Testing the SCS Chart Method1/

Modifications to Chart Method	Statistics	Return period					
		2 years	5 years	10 years	25 years	50 years	100 years
None 2/3/	Bias	0.03	0.02	0.01	-0.04	-0.05	-0.08
	Accuracy	1.09	0.92	0.96	0.93	1.01	1.05
	Mean error (cfs)4/	165	220	267	227	261	240
	Standard deviation of errors (cfs)	551	769	945	1,155	1,351	1,587
Recalibrated impervious area and hydraulic length adjustment factors	Bias	-0.02	-0.03	-0.04	-0.09	-0.10	-0.13
	Accuracy	1.13	1.02	0.99	0.95	1.02	1.06
	Mean error (cfs)	133	174	208	157	174	143
	Standard deviation of errors (cfs)	554	790	974	1,212	1,418	1,679
Hydraulic length defined as channel2/3/	Bias	0.05	0.04	0.04	-'.02	-0.03	-0.06
	Accuracy	1.09	0.91	0.93	0.89	0.95	0.99
	Mean error (cfs)	166	219	264	222	254	232
	Standard deviation of errors (cfs)	576	799	975	1,181	1,376	1,606
Hydraulic length defined as channel (use CN 90 curve)3/	Bias	-0.11	-0.12	-0.12	-0.17	-0.17	-0.20
	Accuracy	1.02	0.81	0.82	0.79	0.85	0.89
	Mean error (cfs)	57	60	63	-21	-34	-96
	Standard deviation of errors (cfs)	537	759	932	1,159	1,355	1,601

1/ 40 watersheds
2/ Use Figure 2 hydraulic length adjustments
3/ Use Figure 2 impervious area adjustments
4/ To convert cfs to m^3/s multiply by 0.0283


LOCATION OF WATERSHEDS
SEATTLE, WA (1)
SAN FRANCISCO, CA (2)
BOULDER, CO (1)
DENVER, CO (5)
TUCSON, AZ (3)
OKLAHOMA CITY, OK (1)
DALLAS, TX (2)
FT. WORTH, TX (1)
AUSTIN, TX (2)
SAN ANTONIO, TX (2)
HOUSTON, TX (5)
IOWA CITY, IA (1)
ST. LOUIS, MO (8)
CHICAGO, IL (5)
INDIANAPOLIS, IN (1)
COLUMBUS, OH (2)
NASHVILLE, TN (5)
CANTON, MS (1)
NATCHEZ, MS (1)
BATON ROUGE, LA (1)
ATLANTA, GA (1)
WASHINGTON, DC (4)
BALTIMORE, MD (4)
PHILADELPHIA, PA (3)
PATERSON, NJ (3)
HARTFORD, CT (5)
MONOLULU, HI (1)
ALASKA
LEGEND
( ) Number of watersheds

Figure 1



1.8
1.6
1.4
1.2
1.0
PEAK FACTOR
CN 70
CN 80
CN 85
CN 90
FUTURE CONDITION
CN 95
0
25
50
75
100
PERCENT IMPERVIOUS
AREA OR HYDRAULIC
LENGTH MODIFIED

Figure 2

APPENDIX F

ESTIMATING THE SCS PEAK RATE FACTOR

Richard H. McCuen

Timothy R. Bondeled

ESTIMATING THE SCS PEAK RATE FACTOR

ABSTRACT

The Soil Conservation Service (SCS) dimensionless unit hydrograph model TR-20 is widely used for hydrologic design. The model uses a dimensionless unit hydrograph based on a peak rate parameter of 484. While vague guidelines have been provided for modifying the value of the parameter, almost all designs are made using a value of 484. A method is presented that provides a systematic means of obtaining a value of the peak rate factor that is unique to a watershed. The method only requires a runoff curve number map, which should be developed to use the TR-20 model anyway. Application of the method to six watersheds, including three coastal basins, indicates that the method increases the accuracy of prediction; the peak rate factors computed using the procedure outlined compared very favorably with the values obtained by optimizing on measured rainfall hyetographs and runoff hydrographs.

ESTIMATING THE SCS PEAK RATE FACTOR

Richard H. McCuen and Timothy R. Bondelid[1]

INTRODUCTION

A recent nationwide study (WRC, 1981) indicated that the Soil Conservation Service (SCS) hydrologic methods were some of the most widely used methods in hydrologic analyses. The SCS TR-20 program (SCS, 1969) provides the means for developing runoff hydrographs, routing hydrographs through both channel reaches and structures, and combining hydrographs. Manual hydrologic computations can be performed using the methods detailed in TR-55 (SCS, 1975); these methods can be used to obtain either a peak discharge or a runoff hydrograph. The methods that are presented in TR-55 are based on the concepts behind the development of the TR-20 program.

A unit hydrograph is used in the TR-20 program for developing runoff hydrographs (SCS, 1972). The unit hydrograph is dimensionless with axes of q/q_p and t/t_p, in which q is the discharge at any time t and q_p is the peak discharge at time t_p; thus, the peak of the dimensionless unit hydrograph equals 1.0 and t/t_p equals 1.0 at time t_p. The dimensionless unit hydrograph is dimensionalized by multiplying the time values by the estimated time-to-peak and the discharge ordinates by the peak discharge, which is given by:

$$q_p = \frac{D_f A Q}{t_p} \tag{1}$$

in which q_p is the peak discharge in cubic feet per second, A is the drainage area in square miles, Q is the runoff volume in area-inches, and D_f is called

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the peak rate factor. For the dimensions specified for A, Q, t_p, and q_p and the assumption that 37.5 percent of the total volume of runoff occurs prior to the t_p, D_f will equal 484; most application of TR-20 assume this value of D_f. SCS (1972) indicates that D_f can vary from 300 to 600, with a value of 300 in very flat swampy country and a value of 600 in steep terrain. However, an accurate, systematic method for selecting the appropriate value of D_f for a watershed has not been proposed.

Hydrologists who must use either TR-20 or TR-55 in coastal areas recognize that the standard value of 484 for D_f does not reflect the flood runoff characteristics of coastal watersheds. Since policies in Maryland require the use of SCS techniques for various hydrologic computations there was considerable interest in an examination of the accuracy of the standard peak rate factor in coastal areas. A study by Woodward, et al., (1980) suggested a value of 284 for the Delmarva peninsula. This created some question on the applicability of the 284 value for low sloped areas on the western shores of Maryland. This concern emphasizes the need for an accurate, systematic method for determining the peak rate factor and dimensionless unit hydrograph for ungaged watersheds where the design engineer believes the standard SCS dimensionless unit hydrograph to be inappropriate.

Because runoff hydrographs computed using TR-20 and TR-55 are very sensitive to the value selected for D_f, it is important to have an accurate estimate of its value. While regional estimates could be derived from an analysis of measured storm hydrographs, this may not be either practical or possible for most hydrologic designs. Furthermore, if the watershed under investigation is not similar to those used to obtain the regional estimate, then the estimate of D_f may be inaccurate.

The objective of this study is to present a method of estimating the dimensionless unit hydrograph and the peak rate factor for ungaged watersheds.

THE PEAK RATE FACTOR AND HYDROGRAPH CHARACTERISTICS

The value of 484 for the peak rate factor reflects both the units in which q_p, A, Q, and t_p are given and the area under the rising limb of the hydrograph. The shape and magnitude of the rising limb of a hydrograph is a function of the storage characteristics of a watershed and the precipitation characteristics (Linsley, et al., 1958). If the distribution of precipitation is assumed constant for the incremental duration of the unit hydrograph, then one can assume that the rising limb of the hydrograph depends primarily on the watershed storage characteristics. For a watershed in which storage characteristics are relatively homogeneous, the time-area curve can be used to represent the volume of detention storage within the watershed. The time-area curve shows the distribution of the drainage area that contributes to runoff during various time increments. Using the hydraulic features of the "bank-full" channel system, the travel times to the watershed outlet are estimated for a number of points in the drainage basin and time contour lines with equal time intervals are drawn (Kraijenhoff van de Leur, 1966). If the storage characteristics are not relatively homogeneous throughout the watershed, then a time-storage curve could be developed instead of the time-area curve. If a time-storage curve defines the volume under the rising limb of a hydrograph, then the SCS peak rate factor (i.e., D_f of Eq. 1) can be estimated from the time-storage curve analysis.

PROCEDURE FOR ESTIMATING THE PEAK RATE FACTOR

The following procedure is recommended for obtaining the SCS peak rate factor:

1. Develop a runoff curve number (CN) map by integrating the soils data and the land use data;
2. develop a slope map from a topographic map;

3. compute runoff velocities (using the CN and slope maps) for a large number of points throughout the watershed and form a velocity map;
4. use the runoff velocity map to get iso-travel time lines, with approximately 15 to 20 intervals recommended;
5. find the fraction of the total area of the watershed contributing during each time interval, and form a plot of the fraction of the watershed within each iso-travel time lines versus time;
6. form a time-storage plot by dimensionalizing the plot by dividing the ordinates by the largest fraction, and find the proportion of the area under the rising limb of the time-storage plot; and
7. compute the peak rate factor by:

$$D_f = 1290.67\ p \tag{2}$$

in which p is the proportion of the area under the rising limb of the time-storage curve.

It is important to note that the development of the time-storage curve,and computation of the peak rate factor,does not require any information beyond that required to develop the input required for a typical TR-20 evaluation. Obtaining the soils, land use, and topographic information is usually a first step in the watershed analysis process. Computing the velocity map and time-storage curve is a simple process and uses only the curve number and topographic maps, in addition to a formula for estimating the velocities as a function of CN and slope.

APPLICATION OF THE TIME-STORAGE PROCEDURE

The procedure outlined previously was applied to six watersheds, including two of the coastal watersheds used by Woodward, et al. (1980). The Powells Creek watershed W-1 is a small upland watershed near Blacksburg, VA; it is an undeveloped wooded and agricultural area of moderate slopes (about 2 percent). The rainfall, runoff, and basin characteristics were obtained from SEA-AR experimental watersheds as reported in ARS publications (e.g., ARS, 1972). The Powells Creek watershed has a drainage area of 182 acres and a length of approximately 4,300 feet. Soils include Enon, Wilkes, Bremo, and Lloyd loam. The watershed is comprised of 16 percent hardwood forest, 60 percent native grass pasture, 6 percent row crops (mostly, corn and tobacco), 7 percent small grain crops, 5 percent alfalfa and other hay crops, 4 percent idle land, and 2 percent roads. These data were used to develop a runoff curve number map, which was necessary to develop the time-storage curve. The dimensionless unit hydrograph (DUH) resulting from the time-storage curve is shown in Fig. 1. Since approximately 38 percent of the volume of the DUH lies under the rising limb, Eq. 2 provides a value of 490 for the peak rate factor; this is in very close agreement with the standard value of 484 suggested by the SCS.

The three coastal watersheds lie on the Delmarva peninsula. The Murderkill River watershed near Felton, MD, has an area of 8,704 acres and is predominately cultivated and forested, with over 60 percent of the watershed in crops and approximately one-fourth wooded; the remaining land includes marshland, farmsteads, and roadways. The main drainage channels lie within narrow wooded floodplains, which provide for increased watershed storage. Fig. 1 shows the time-storage curve for the watershed. A numerical integration of the volume under the rising limb indicates that p equals 0.307, which yields a value for D_f of 396 (Eq. 2).

The Faulkner Branch watershed near Federalsburg, MD, drains approximately 4,544 acres. Except for farmsteads and roads, the drainage basin is predominately cropland and woodland. The main drainage channels lie within narrow wooded floodplains. The soils consist of sassafras (SCS group B) and Woodstown (SCS group C) sandy loams. The time-storage curve is shown in Fig. 1. Approximately 27.3 percent of the volume lies under the rising limb, which yields a value of 354 for the peak rate factor.

The Morgan Creek watershed is located near Kennedyville, MD, and has a drainage area of 12.7 square miles. The watershed is primarily cropland, although there are woodlands, marshy areas, and farmsteads throughout the watershed. The main channels lie within narrow wooded floodplains or open marshy areas. Channel slopes are less than one percent, while land slopes are mostly less than two percent. The dimensionless unit hydrograph was obtained from the time-storage curve; a peak rate factor of 435 was computed.[1]

Pony Mountain Branch Watershed W-1 is located in Culpeper County, VA. The 192 acre watershed is roughly trapezoidal with an average elevation of 2,400 feet. Approximately 66 percent of the basin has a slope range less than 4 percent, but the remaining portion of the watershed has slopes from 12 to 25 percent. The soils are of the Penn (SCS Soil Group C) and Bucks (Soil Group C) series. The watershed has a mixed cover, with about 50 percent in farm woods, predominantly hardwood,and 30 percent native grass; the remaining portion of the watershed consists of a mixture of orchard grass, clover, alfalfa, small grain, and road surfaces. The dimensionless unit hydrograph, which was obtained from a time-area curve, yielded a peak rate factor of 360.

The Coshoction W-177 Watershed is an ARS experimental watershed in Coshocton, Ohio. The 75.6 acre watershed is predominantly cropland, with small portions in woodland, farmsteads, and roads. While the average slope is about 9 percent, the slopes range from 5 to 20 percent. The dimensionless unit hydrograph was computed from a time-area curve, and the peak rate factor of 495 was obtained from Eq. 2.

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[1] Data on the Maryland watersheds were provided by Dr. Helen F. Moody, College Park, MD, and Mr. Donald Woodward, USDA-SCS, Broomall, PA; the authors are especially grateful for their effort

TEST OF THE TIME-STORAGE METHOD

The proposed procedure for estimating a value of the peak rate factor for an ungaged watershed was tested using measured rainfall and runoff data. Data for twenty-two storm events were available for the six watersheds. The rainfall and runoff data were measured at two minute intervals for the Powells Creek, Pony Mountain, and Coshocton watersheds; a one-hour interval was used for the data from the three coastal watersheds, Morgan Creek, Faulkner Branch, and the Murderkill River.

A version of the SCS TR-20 program that provides for automatic fitting to observed hydrographs (Bondelid and McCuen, 1981) was used to identify the best-fit relationship between D_f and t_c for each of the 22 storm events. The program was used to develop a response surface that shows the value of the objective function (i.e., a weighted least squares fit of the hydrograph) as a function of t_c and D_f. The response surface for the May 31, 1962, storm event on the Powells Creek watershed is shown in Fig. 2. Each response surface was characterized by a linear valley having a positive slope. While the fitting precision changes rapidly as the combination of t_c and D_f moves perpendicular to the valley, the response surface indicates that there are an array of combinations of values of t_c and D_f that provide almost an identical fit of the computed runoff hydrograph to the measured hydrograph.

A straight line relationship was obtained for each storm event. A linear relationship would be expected, as evident from a transformation of Eq. 1:

$$t_p = \left(\frac{AQ}{q_p}\right) D_f \qquad (3)$$

Using the SCS relationship between t_p and t_c yields:

$$t_c = \left(\frac{1.5AQ}{q_p}\right)D_f \tag{4a}$$

$$= KD_f \tag{4b}$$

in which K is the peak rate slope coefficient. Table 1 gives the values of K derived from the measured values of A, Q, and q_p and the corresponding values obtained from fitting the hyetographs and hydrographs. For storm events on which the volume of runoff exceeded one inch, the two estimates of K showed good agreement; for the remaining events the degree of agreement between the two estimates varied. For storm events based on small volumes of runoff, the value of K from Eq. 4 would depend on the representativeness of the peak discharge.

The accuracy of the procedure described herein for estimating the peak rate factor was tested using the twenty-two storm events. The peak rate factor ($D_{f|min}$) and time of concentration ($t_{c|min}$) that provided the best fit between the measured and predicted hydrographs are shown in Table 1 for each storm event. The mean value of D_f for the storm events were computed for each watershed and are given in Table 2; the average error for the six watersheds was 4.8 percent. The mean peak rate factor computed from the storm event data showed good agreement with the values of D_f obtained from time-storage analyses for four of the six watersheds. The two values of D_f differed by 39 and 50 on the Murderkill River and Powells Creek watersheds, respectively. Five of the seven storm events from Powells Creek had runoff volumes less than 0.3 inches and are less than the 1-year event. For this reason, the storm response may not be indicative of the runoff potential of the watershed as indicated by both the peak rate factor and the dimensionless unit hydrograph. For the two larger storm-events, with runoff volumes of 0.9 and 2.0 inches, the computed peak rate factor showed perfect agreement with the value obtained from the time-storage curve. The two storm events for the Murderkill River watershed were the two largest runoff volume producing events included in the study. The difference of 39 in the hydrograph computed and watershed derived peak rate factors is not considered significant because there was little difference in the value of the objective function between D_f values of 350 and 400; that is, the valleys on the response surfaces were very long and not steeply sloped along the valley. The fit of the observed hydrograph for a D_f of 396 was almost identical to that obtained at the optimun values of 355 and 360 for the two storm events.

The mean values of the best-fit and watershed times of concentration are also given in Table 2. The percentage error ranged from 5 percent for the Morgan Creek and Murderkill River watersheds to 29 percent for the Pony Mountain Branch watershed. The mean value for Pony Mountain Branch watershed was based on three storm events all having runoff volumes less than 0.05 inches. Thus, the fitted times of concentration for the six watersheds are in good agreement with the times of concentration obtained by conventional means. The weighted average error was 10.7 percent.

CONCLUSIONS

The SCS methods (i.e., TR-20 and TR-55) are some of the most widely used hydrologic design methods. They are generally applied using the standard SCS dimensionless unit hydrograph with a peak rate factor of 484. While these are probably valid for many applications, it is recognized that the design accuracy can be improved if the dimensionless unit hydrograph is calibrated. The regionalization provided by Woodward, et al., for the Delmarva peninsula resulted in a more rational dimensionless unit hydrograph for those coastal areas. However, past experience has suggested that unit hydrographs are not constant for all watersheds in a region.

The hydrologic effects of land cover conversion from forested to agriculture land use is of special concern to those involved in hydrologic analysis on the Delmarva peninsula. If channel modifications are to be made for the purpose of improving the drainage efficiency within the watershed, it is important to accurately evaluate the hydrologic response of the direct runoff. Furthermore, in coastal areas where stormwater detention must be provided to control the effects of land use conversion, the accuracy of the shape of the hydrograph is an important determinant of the volume of required storage. Given the economic value of land, farmers in coastal areas are very concerned about the surface area requirements of detention facilities. Hydrologic computations of detention storage requirements could be highly inflated if the unit hydrograph is more peaked than the true hydrologic response of the watershed. Thus, agricultural drainage in coastal areas represents a case where standard unit hydrograph shapes are of special concern.

A procedure was outlined herein that can be used to derive a dimensionless unit hydrograph for use with the SCS hydrologic design methods. The method does not require any data that is not normally used in the application of the SCS design methods. The method was tested on six watersheds, including three coastal watersheds; the method provided good agreement with the analysis of the measured data. Use of the procedure should increase the accuracy of designs.

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TABLE I. Comparison of Best Fit and Storm Event Values

Watershed	Date of Storm	$\frac{1.5AQ}{q_p}$	$\frac{\partial t_c}{\partial D_f}\Big\|_{min}$	Storm Event CN	Storm Event Q	$D_f\|_{min}$	$t_c\|_{min}$	$t_c\|_{D_f}$
Powells Creek	7/10/59	0.00132	0.01440	68.1	0.047	350	0.305	0.496
	10/ 8/59	0.00132	0.00114	71.3	0.223	550	0.802	0.601
	5/31/62	0.00155	0.01728	84.2	0.892	490	0.605	0.605
	7/29/63	0.01324	0.00140	83.5	0.290	350	0.360	0.571
	7/11/65	0.00198	0.00221	83.4	1.980	485	0.729	0.725
	9/20/66	0.00215	0.00093	79.6	0.106	425	0.488	0.554
	8/23/67	0.00264	0.00555	83.5	0.102	430	1.070	1.390
Coshocton W-177	6/18/40	0.00104	0.00147	90.6	0.342	495	0.482	0.490
	6/12/57	0.00103	0.00133	79.1	1.374	490	0.318	0.335
	4/25/61	0.00238	0.00123	93.6	1.035	425	0.391	0.484
	5/13/64	0.00099	0.01620	78.2	0.127	495	0.551	0.554
Pony Mtn. Branch	5/26/62	0.00263	0.00667	64.0	0.028	350	0.180	0.184
	6/14/67	0.00276	0.00174	72.3	0.054	300	0.239	0.370
	6/17/68	0.00368	0.00455	81.8	0.049	425	1.384	1.085
Morgan Creek	1955	0.0371	0.0360	52.1	1.211	350	10.40	12.61
	1958	0.0311	0.0260	75.0	1.341	490	6.35	5.02
	1960	0.0287	0.0342	79.5	2.317	500	11.89	11.05
Murderkill River	1960	0.0670	0.0835	59.0	2.642	360	17.7	20.5
	1967	0.0660	0.0551	85.7	6.726	355	10.1	12.7
Faulkner Branch	1955	0.0470	0.0280	51.1	1.901	320	7.20	8.20
	1958	0.0580	0.0128	66.5	2.291	355	4.86	4.85
	1960	0.0416	0.0400	61.7	2.538	370	11.75	11.05

TABLE 2. Summary of Means for Best-Fit and Storm Event Values

		Mean Peak Rate Slope		Peak Rate Factor		Time of Concentration	
Watershed	n	Storm Event	Fitted	Watershed	Fitted	Watershed	Fitted
Powells Creek	7	0.00346	0.00613	490	440	0.670	0.623
Coshocton	4	0.00136	0.00506	495	476	0.500	0.436
Pony Mtn. Branch	3	0.00302	0.00432	360	358	0.467	0.601
Morgan Creek	3	0.0323	0.0321	435	449	9.12	9.55
Murderkill River	2	0.0665	0.0693	396	357	13.3	13.9
Faulkner Branch	3	0.0489	0.0269	352	348	8.70	7.94

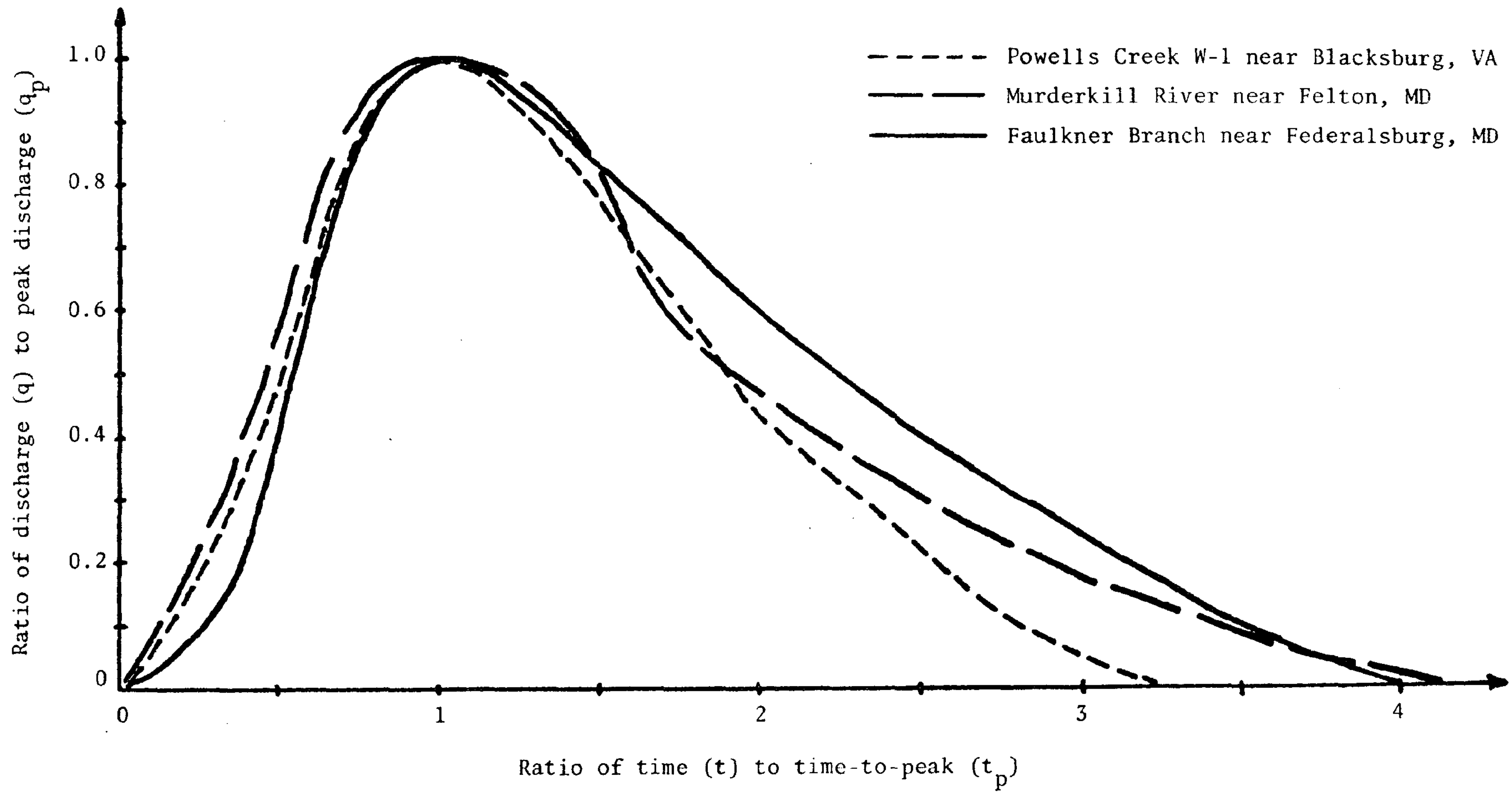

FIGURE 1. Dimensionless Unit Hydrographs Derived from Time-Storage Curves for Three Watersheds

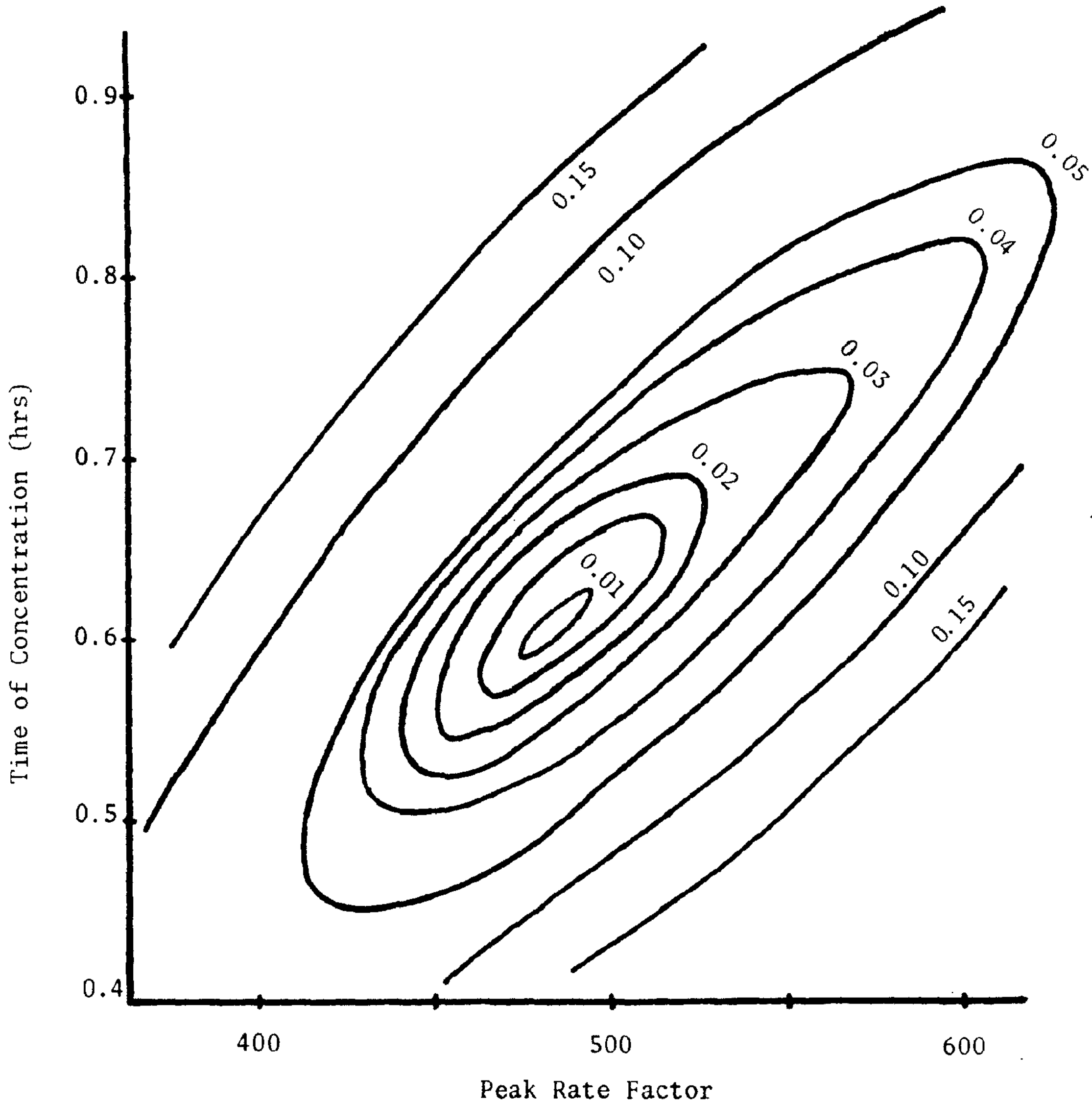

FIGURE 2. Peak Rate Factor Response Surface for 5/31/62 Storm Event on Powells Creek Watershed

APPENDIX G

AN AUTOMATIC FITTING VERSION OF THE SCS TR-20 HYDROLOGIC MODEL

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An Automatic Fitting Version of the SCS TR-20 Hydrologic Model

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INTRODUCTION

While peak discharge formulas, such as the rational method, are still widely used, hydrologic methods that produce an entire runoff hydrograph are being used more frequently in hydrologic design. This has resulted from the need to consider storage in design, such as detention storage in urban design problems. The hydrologic methods developed by the Soil Conservation Service (SCS) are some of the most widely used methods for hydrologic analyses (WRC, 1981). The SCS TR-20 computer program (SCS, 1969) can be used for hydrologic analyses where it is necessary to generate flood hydrographs, route them through either channels or structures, or both, and combine hydrographs. The SCS TR-55 (1975) tabular method, which is the result of numerous TR-20 runs, also can be used to develop flood hydrographs.

As with most flood hydrograph generating methods, the flood hydrograph algorithm of TR-20 is based on the concepts of a unit hydrograph and convolution. The SCS dimensionless unit hydrograph (DUH) is synthetic. A unit hydrograph is generated from the DUH using the depth of runoff, which is a function of the 24-hour precipitation depth and the runoff curve number (CN), the drainage area, the time of concentration, and a peak rate factor, which is a parameter that indicates the proportion of the volume that is under the rising limb of the unit hydrograph. A value of 484 is most frequently used for the peak rate factor.

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It is believed that the value of this parameter will change for different regimes, with a value of 300 recommended for coastal areas and a value of 600 for mountainous watersheds (SCS, 1972). Woodward, et al. (1980) used measured rainfall and runoff to develop unit hydrographs for watersheds on the Delmarva peninsula; from these unit hydrographs they estimated the peak rate factor to be 284. Other studies have shown that urbanization also changes the shape and distribution of a unit hydrograph. McCuen and Bondelid (1981) provided a procedure for obtaining the dimensionless unit hydrograph and peak rate factor for ungaged watersheds. The value of the parameter is supposed to reflect the design accuracy. Therefore, it is deemed important to be able to estimate the correct value of the parameter for ungaged watersheds where the value of 484 is not considered to be the optimal value.

In most regions hydrologic data are available that could be used to obtain a regional estimate of the unit hydrograph parameter. However, this rainfall/runoff data cannot be used with the TR-20 program because the current program form does not provide for data analysis. Thus, a version of TR-20 that could calibrate the dimensionless unit hydrograph and the peak rate factor using gaged data would be of value in obtaining a regional estimate ; this should increase the accuracy of designs made with the SCS hydrologic methods.

The purpose of this study was to develop a computerized automatic calibration version of TR-20 that could be used in analyzing storm event rainfall and runoff data to estimate the best-fit dimensionless unit hydrograph. While the method proposed by McCuen and Bondelid (1981) should increase the design accuracy over that obtained by using a peak rate factor of 484, the accuracy could be further improved by calibrating the parameters of the TR-20 model using measured rainfall and runoff data.

AN AUTOMATIC OPTIMIZATION STRATEGY

Optimization, or fitting, is a term used to indicate the process in which a model is fitted to data and "best" estimates of model coefficients are obtained. Optimization is most efficient when a systematic strategy is used. The actual fitting requires four components: (1) a model, (2) an objective function, which is the criteria used to define "best" fit; (3) a data set; and (4) a set of constraints, if applicable. For the case of fitting the SCS hydrologic method, the dimensionless unit hydrograph was assumed to be the model. The strategy provides several options for obtaining a best fit, with the options depending on the available input data. Two objective functions are available for fitting, with one based on the accuracy of the peak discharge and the other based on a weighted least squares fitting of the entire runoff hydrograph. The optimization program recognizes that the available input data and problem objective will vary; thus, several options are available for specifying the input data structure.

Objective Function

A common technique in curve fitting is to minimize the sum of the squares of differences between the observed and computed data points. As usually applied, the least squares principle assumes the observations to be independent and, therefore, gives each observation equal weight; this is not the case in hydrograph analyses. In fitting flood hydrographs, there is, in general, more concern in being closer on the larger discharges, especially the peak of the hydrograph; that is, it is more important for flood hydrograph prediction to be accurate in predicting the high flows than in predicting the lower flows, which occur at the beginning and end of the runoff event.

An objective function that combines the least squares concept and weighting of the higher discharges is given by:

$$\min F = \frac{\sum[(\hat{Q}_i - Q_i)^2 Q_i]}{\sum Q_i^3} \tag{1}$$

in which the summations are taken over the observed discharge values, Q_i is the observed discharge, $\hat{Q}_i$ is the computed discharge occurring at the same time as Q_i, and F is the value of the objective function. The numerator in Eq. 1 represents a weighted least squares and the denominator makes the F value dimensionless. The weighting by Q_i ensures that the larger flows, especially the peak discharge, will be given a greater influence on the values of the fitting parameters. The goal of the optimization procedures for this objective function will be to minimize the right hand side of Eq. 1.

As an alternative to Eq. 1, the program includes a second objective function. Specifically, if the peak discharge is of principle interest and the timing of minor importance, then values for the fitting parameters can be obtained such that the absolute value of the difference between the computed and measured peak discharges is a minimum:

$$\min F = | \hat{Q}_p - Q_p | \tag{2}$$

in which $\hat{Q}_p$ and Q_p are the computed and measured peak discharge, respectively.

Dimensionless Hydrograph Generation

The structure of the SCS dimensionless unit hydrograph is not specified by a known probability density function. While it seemed desirable to have a unit hydrograph that would closely approximate the shape of the SCS unit hydrograph, it also seemed reasonable to select a known probability density function so that its characteristics would be known. In addition to selecting a known probability density function, there are several constraints on the shape of the dimensionless unit hydrograph. The dimensionless unit hydrograph must peak at a time of 1.0 and have a peak discharge value of one; also, the DUH must be unimodal and have the general shape of a typical runoff hydrograph. Additionally, the unit hydrograph must be able to be varied in a consistent and stable manner by varying specific parameters.

The gamma density function satisfies these requirements. Because it has a flexible shape, the gamma function can take the extreme forms of a decaying exponential function or almost an equilateral triangle; in general, it takes the form of a typical unit hydrograph. The gamma function has been widely used in hydrologic analysis; Dooge (1973) described its use for predicting agricultural runoff, and Sarma, et al., (1969) used it for evaluating the effect of urbanization on small watersheds. The gamma density function is given by:

$$f(\chi) = \left(\frac{\chi}{b}\right)^{c-1} \frac{\exp(-\chi/b)}{b\Gamma(c)} \qquad (3)$$

in which χ is the random variable defined on the abscissa, $f(\chi)$ is the ordinate, b is a scale parameter, c is a shape parameter, and $\Gamma(c)$ is the gamma function with argument c. For $c>1.0$, $f(\chi)$ has a shape similar to typical runoff hydrographs. The mode of $f(\chi)$ is given by $b(c-1)$. The gamma density function can, thus, be easily scaled so that the peak occurs at $\chi=1$, with $f(\chi)=1$. Also, b and c can be readily varied to produce a wide range of unit hydrograph shapes.

Sensitivities of the Gamma Parameters. The parameters of Eq. 3 are parameters that define the dimensionless unit hydrograph ordinates. However, the parameter of real interest in TR-20 is the dimensionless hydrograph factor, or peak rate factor, D_f. Therefore, a logical strategy is to first evaluate the parameters of the gamma function (c and b) in relation to D_f and then evaluate the sensitivity of TR-20 to D_f.

Initial analyses indicated that D_f is much less sensitive to the scale parameter (b) than to the shape parameter (c); that is, D_f is almost entirely determined by the shape parameter of the gamma function. The lack of sensitivity of the b parameter was verified on all of the test watershed events; the gradient of the b parameter with respect to the objective function F of Eq. 1 was always at least an order of magnitude less than the gradient of the scale parameter with respect to F. In all subsequent analyses, the scale parameter was set so that the mode of the gamma distribution occurred at a value of 1.0. The mode can be set at 1.0 by using the equation:

$$b = \frac{1}{c-1} \tag{4}$$

The accuracy of fitting would not significantly improve if both parameters were optimized simultaneously because there is high correlation between the sensitivity functions of the two parameters (McCuen, 1973).

A Limitation of the Gamma Function. Use of the gamma distribution function for generating dimensionless unit hydrographs has one slight limitation. Specifically, the maximum value of D_f that can be produced using the gamma distribution is 645. The value of 645 corresponds to one-half of the area of the dimensionless unit hydrograph being under the rising limb; the gamma distribution can not have more than half of the area before the mode of the distribution function. As D_f approaches 645, the dimensionless hydrograph approaches a "spike" shape.

In spite of this constraint, the gamma function is still a very reasonable dimensionless unit hydrograph. For most natural watersheds, one would not expect more than 50% of the area under the unit hydrograph to occur before the peak. Even on urban watersheds this might only occur if the drainage patterns were altered significantly and the storage characteristics were drastically modified. In fact, the gamma function is still more flexible than the standard SCS dimensionless unit hydrograph; this is necessary if the peak rate factor is to be optimized.

Optimum CN

A desirable goal in hydrograph prediction is to have the simulated runoff volume equal to the observed runoff volume. The observed runoff volume can be readily computed by integration. Because the total rainfall depth P is also known, the SCS rainfall-runoff formula can be used to estimate the retention:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \tag{5}$$

in which P is the rainfall depth (in.), S is the watershed retention (in.), and Q is the runoff volume (in.). The retention S is related to the CN by:

$$S = (1000/CN) - 10 \tag{6}$$

By selecting the program option that uses the CN computed from Eq. 4, the simulated and ovserved runoff volumes can be made equal. The program also provides the option that the CN obtained by conventional means can be part of the data base for input.

Estimation of the Time of Concentration

Most formulas for estimating the time of concentration are based on very limited data bases; however, most hydrologic design methods, including the SCS methods, are very sensitive to the time of concentration. Therefore, an option was included in the automatic fitting program to use measured rainfall and runoff data to find the optimum storm event time of concentration. The precision in the estimated t_c is controlled by an input parameter. The time of concentration can be fitted using a two-phased search procedure, which is part of the optimization strategy.

The Optimization Strategy

If the user elects to find the optimum value of either t_c or D_f, a two-phased strategy is used to ensure that the optimum is obtained. The first phase of the search is a systematic search of the response surface (i.e., the value of the objective function versus the unknown, D_f or t_c); this phase of the search requires the extent of the search, (i.e., the upper and lower values of t_c or D_f), and the number of intervals into which the search region is divided. The second phase of the fitting process uses the point obtained in the first phase that has the minimum value of the objective function as the initial point. A pattern search algorithm is used to find the minimum value of the objective function. The degree of precision is controlled by the number of iterations, which is specified by the user.

PROGRAM DESCRIPTION

The objective of the automatic fitting version of the SCS TR-20 hydrologic model is twofold. The first objective is to provide a straightforward procedure for calibrating the TR-20 model parameters to observed rainfall and runoff data. The second objective is to provide a flexible tool that can be used for future research. Therefore, a wide variety of user-selectable options have been incorporated into the program.

The computer program consists of a main routine and sixteen (16) subroutines. The program structure is modular in that most of the primary operations are contained in separate subroutines; this allows for flexibility in updating and re-using the program. The program has been written in FORTRAN IV and is,therefore, essentially machine independent.

The program analyses are rainfall/runoff event per execution. The TR-20 reservoir and channel routing routines have not been incorporated into the program. Inclusion of the routing options would greatly increase the complexity of the calibration procedures. Therefore, the program utilizes only the TR-20 RUNOFF subroutine for hydrograph generation.

Numerous program options are available to the user. These options include: (1) either specifying the CN or having the program compute the optimum CN value; (2) optimize an either t_c or D_f; (3) inputting a SCS dimensionless hydrograph or have the program generate dimensionless hydrographs using the gamma function; (4) utilize either of the two objective functions specified by Eqs. 1 and 2; (5) input an observed hydrograph or have the program generate hypothetical observed hydrographs; (6) separate baseflow from the observed hydrograph; and (7) produce graphical output in two plot sizes.

The program also performs a hydrograph analysis including computation of runoff volume and centers of mass of runoff, rainfall, and rainfall excess. The graphical output consists of three plots. The first plot shows the observed and optimum hydrographs. The second plot shows both the optimum dimensionless hydrograph and the standard SCS dimensionless hydrograph with $D_f = 484$. The third plot shows the rainfall and runoff hyetographs. These plots are very useful for judging the adequacy of the optimization procedure.

Input Description

The input data consists of essentially three (3) groups of data: (1) control data for specifying the program options to be used; (2) the observed rainfall in the form of a TR-20 format rainfall table; and, (3) the observed discharge in cfs. Table 1 presents the formats, variable names, and variable descriptions for the input data. A maximum of nine (9) card types are required. A description of each of the input data elements is described below.

Title. This is an 80-column title used for watershed and event description.

AREA. This is the measured watershed area in sq. mi.

CN. The user can specify the CN to be used in the optimization by specifying a non-zero CN. If the input CN is 0.0, then the CN computed from the observed rainfall and runoff volumes is used.

TC. This is the time of concentration in hours to be used when the peak rate factor, D_f, is the optimization parameter. If t_c is the optimization parameter, then a value of zero should be input.

DMHFAC. The peak rate factor, D_f, to be used when the optimization parameter is t_c. An input value less than or equal to 0.0 specifies that a TR-20 format dimensionless hydrograph table is to be read. If the optimization parameter is D_f, then a value of zero is input.

VOLMIN. The gamma function is unbounded in the right tail of the distribution; therefore, it is necessary to specify a bound in order for the gamma function to be used as a unit hydrograph. The parameter VOLMIN specifies

TABLE 1. Input Data for the Automatic Fitting Version of TR-20

Card Type	Variable Name	Columns	Variable Description
Title card	-	1-80	Title card
Primary Control	AREA	1-7	Area (sq. miles)
	CN	8-14	Runoff curve number (if zero, then the CN is computed from observed rainfall and runoff)
	TC	15-21	Time of concentration, hours (if zero, then the value is optimized)
	DMHFAC	22-28	Peak rate factor (if zero, then the value is optimized)
	VOLMIN	29-35	Minimum fraction of the gamma function unit hydrograph; used to set limit on right tail
	TZ	36-42	Time of start of rainfall distribution (hours)
	RDEPTH	50-56	TR-20 Rainfall depth (set equal to 1.0 when an actual event is used)
	RAINDU	57-63	TR-20 Rainfall table duration (set equal to 1.0 when an actual event is used)
	DELTM	64-70	TR-20 Computation interval (hours)
	NTER	71-74	Maximum number of iterations in a step for a phase II search of t_c or D_f
	NSTEP	75-78	Number of steps in a phase II search
	ITELL	79	Graphical output option (0, 1, or 2)
	IOPT	80	Specifies objective function and type of hydrograph input (1, 2, 3, or 4)

Card Type	Variable Name	Columns	Variable Description
Parameter Optimization	NINCTC	1-7	Number of increments in the phase I search for t_c (NINCTC = 0; if optimizing D_f)
	NINDMH	8-14	Number of increments in the phase I search for D_f (NINDMH = 0, if optimizing t_c)
	TCMIN	15-21	Lower value of t_c for phase I search } = 0 for D_f optimization
	TCMAX	22-28	Upper value of t_c for phase I search } = 0 for D_f optimization
	DMMIN	29-35	Lower value of D_f for phase I search } = 0 for t_c optimization
	DMMAX	36-45	Upper value of D_f for phase I search } = 0 for t_c optimization
Dimensionless Hydrograph Table	-	-	Some format as the SCS dimensionless hydrograph table (DIMHYD and ENDTBL cards required); input only if DMHFAC $\leqslant$ 0 on card 2
Rainfall Table	-	-	Same format as the SCS rainfall table (RAINFL and ENDTBL cards required);
Baseflow Specification	TB1	1-7	Times (hrs) for first baseflow point (relative to input hydrograph)
	QB1	8-14	Discharge (cfs) at TB1
	TB2	15-21	Time (hrs) for end of baseflow point
	QB2	22-28	Discharge (cfs) at TB2
Format	FORM	1-80	Format for observed hydrograph (used if IOPT = 1 or 2)
Hydrograph Length	NPT	1-5	Number of points on the observed hydrograph (used if IOPT = 1 or 2)
Hydrograph Data	HYDR	FORM	Observations on observed hydrograph

the minimum volume allowed under the gamma unit hydrograph generating function. VOLMIN must be less than 1.0 and is used to avoid excessive "chopping off" of the tail of the unit hydrograph. A value of 0.99 is recommended.

TZ. Specifies the time in hours of the first entry in the TR-20 rainfall table.

RDEPTH. This is the TR-20 rainfall depth in inches. If the rainfall table contains actual depths,then RDEPTH should be set at 1.0. If the rainfall table is scaled from 0.0 to 1.0, as in the 24-hour type II rainfall distribution, then RDEPTH should be set equal to the actual rainfall depth.

RAINDN. This parameter specifies the TR-20 rainfall table duration and is normally set at a value of 1.0 for the analysis of actual storm events.

DELTM. DELTM is the TR-20 computation interval, in hours. This value should be selected on the basis of the general watershed runoff characteristics. For instance, if the event being analyzed is on a small watershed and the event lasts for only a few hours, then a DELTM of 0.1 hour should be used; if the event is for a large watershed lasting on the order of 24 hours or more, then a DELTM of 1.0 may be appropriate.

NTER. The maximum number of iterations per step in a phase II search. This will depend on the uncertainty in the bounds of phase I search. A value of 25 may be adequate for small watersheds; a value of 50 to 100 would be more appropriate for a large watershed in which there is uncertainty in t_c or D_f.

NSTEP. NSTEP is the maximum number of steps in a phase II search. The maximum possible number of iterations is NSTEP times NTER.

ITEL. The variable ITEL specifies the plot size for the graphical output. If ITEL = 0, then no graphical output is produced. If ITEL = 1, then plots of 80 columns by 40 lines are produced; this size is suitable for 8½ x 11 in. paper.

If ITEL = 2, then plots of 100 columns by 50 lines are produced; this size provides the greatest detail and uses a full page of computer paper.

IOPT. The variable IOPT specifies both the type of objective function to be used and the type of hydrograph to be used for calibration. Four values of IOPT are possible:

IOPT Value	Program Options
1	Use observed runoff hydrograph and the weighted least squares objective function (Eq. 1).
2	Use observed runoff hydrograph and the difference in peak discharges as the objective function (Eq. 2).
3	Use a TR-20 generated hydrograph as the input TC and D_f = 484 as the "observed" hydrograph and the weighted least squares objective function (Eq. 1).
4	Same as IOPT = 3, except use the difference in peak discharges as the objective function (Eq. 2).

The IOPT values of 3 and 4 will not normally be used in actual calibration; the options are provided for possible research into TR-20 parameter interactions.

DIMHYD Table. This is a complete TR-20 dimensionless hydrograph table in TR-20 format and must include the DIMHYD and ENDTBL cards. This table is read only if DMHFAC is less than or equal to 0.0. If DMHFAC is greater than 0.0, then do not include the DIMHYD table in the input deck. This option provides an alternative to the gamma function; it could be used, for example, where the gamma function may not be appropriate, such as for watersheds with a D_f greater than 645.

RAINFL Table. This is a complete TR-20 rainfall table in TR-20 format and must include the RAINFL and ENDTBL cards. This table specifies the observed rainfall distribution for calibration.

NINCTC. The variable NINCTC is the number of increments in t_c in a phase I search when optimizing t_c. NINCTC must be set at zero when the optimization is on D_f.

NINDMH. This is the number of increments in D_f in a phase I search when optimizing on D_f. NINDMH must be set at zero when the optimization is on t_c. Also, NINCTC and NINDMH cannot both be set at zero in any one program execution.

TCMIN and TCMAX. These variables are the lower and upper bounds, respectively, of t_c in a phase I search. If NINCTC = 0, then these values are ignored.

DMMIN and DMMAX. These variables are the lower and upper bounds, respectively, of D_f in a phase I search. If NINDMH = 0, then these values are ignored.

TB1 and QB1. TB1 and QB1 are the time (hrs.) and discharge (cfs), respectively, of the first baseflow separation point.

TB2 and QB2. These variables are the time (hrs.) and discharge (cfs), respectively, of the last baseflow separation point. TB1, QB1, TB2, and QB2 specify two points on the runoff hydrograph; during program execution the runoff is understood to be the hydrograph above a straight line connecting these points. When baseflow is not present, these values should all be set to 0.0.

FORM. The FORTRAN format specifications, up to 80 columns long,of the observed hydrograph cards. The format should include the left and right parentheses and specify reading of REAL variables (F or E format), e.g., (10F8.2). The left parenthesis should be placed in column 1.

NPT. This variable specifies the number of points on the observed input hydrograph.

Input Hydrograph. Each point on the input hydrograph requires two (2) values, the time (hrs.) and discharge (cfs). The values are read in pairs so that the time and discharge for the first point is read, then the time and discharge for the second point, etc. The format specified by FORM is used so that, for instance, if the format is (10F8.0), then each card can contain five (5) points on the hydrograph.

TESTING OF THE FITTING METHOD

Data consisting of observed rainfall/runoff data for six watersheds were selected to test the automatic fitting procedure. The watersheds were selected to provide a wide range of drainage areas and slopes, including coastal watersheds. Three coastal watersheds were included in the study: the Faulkner, Morgan, and Murderkill basins; these watersheds are rural and are located on the eastern shore of Maryland. The watersheds range in area from 3,072 to 8,704 acres and the slopes range form 0.5 to 5 percent. Two or three rainfall events for each watershed were used. The approximate return periods of the rainfall events ranged from less than 5 years to greater than 100 years.

The three remaining watersheds that were used for testing the program were the Coshocton, Powells Creek, and Pony Mountain Branch experimental agricultural watersheds. The Powells Creek and Pony Mountain Branch Watersheds are located in southeastern Virginia near Blacksburg. The Coshocton watershed is located in central Ohio. The watershed areas range from 75.6 to 192 acres, and the average slopes range from 1.9 to 10.0 percent. Four to seven rainfall events on each watershed were selected. The approximate return periods of the rainfall events ranged from less than 2 years to about 100 years.

The automatic fitting program was used to compute the optimum time of concentration for each of the twenty-two storm events. The mean value of the optimum times of concentration, which are shown in Table 2, were computed for

TABLE 2. Watershed and Fitted Times of Concentration (hours)

Watershed	Number of Storm Events	Watershed t_c	Mean Storm Event t_c	Percent Error
Morgan Creek	3	9.12	9.55	4.5
Murderkill River	2	13.3	13.9	4.3
Faulkner Branch	3	8.70	7.94	9.6
Powells Creek	7	0.670	0.623	7.5
Coshocton W-177	4	0.500	0.436	14.7
Pony Mountain	3	0.467	0.601	22.3

each of the six watersheds. The time of concentration was also computed by conventional means (i.e., the velocity method) using the land use, slope, and hydraulic length as input; these watershed t_c values are shown in Table 2. The percentage of error for four of the 6 watersheds was less than 10 percent, with the largest error being 22.3 percent. The largest percentage error, which was for the Pony Mountain Branch, resulted from three storm events, each have less than 0.06 inches of runoff. Thus, there appears to be good agreement between the time of concentration optimized by the automatic fitting version of TR-20 and times of concentration computed conventionally from the land use, slope, and hydraulic lengths of the watershed.

CONCLUSIONS

There has been increased use of unit hydrograph methods in hydrologic design because of the need to incorporate storage computations in the design. The SCS TR-20 program and the SCS TR-55 tabular method, which is based on the TR-20 program, are unit hydrograph based methods that have seen increased usage. It is recognized that the accuracy of a unit hydrograph method can be improved significantly if it is calibrated to data measured either on the watershed where the design point is located or within the region. McCuen and Bondelid (1981) provided a method for obtaining the SCS peak rate factor and the dimensionless hydrograph using the time-storage curve of the watershed. While this should increase the accuracy of designs based on the TR-20 program, the accuracy could be improved further if the paramaters of the TR-20 program were calibrated to measured rainfall and runoff data.

An automatic fitting version of the TR-20 program was developed and described herein. The program can use measured rainfall hyetographs and runoff hydrographs to compute optimum values of various parameters. Depending on the fitting options selected, the calibration paramaters include the runoff curve number (CN), the time of concentration (t_c), and the dimensionless unit hydrograph ordinates. The calibration procedure uses automatic optimization to obtain a predicted flood hydrograph that most closely agrees with the observed flood hydrograph. The method was tested on 22 storm events for six watersheds. The time of concentration was used as the fitting parameter. The mean optimum time of concentration showed reasonable agreement with the watershed time of concentration; the weighted average error was 10.4 percent, and four of the 6 watersheds showed errors less than 10 percent. Simultaneously, the calibration process provided dimensionless unit hydrographs for each watershed; the optimum peak rate factors varied from 352 to 495, with four of the six watersheds having values that were significantly different from the 484 value that is used with the standard SCS dimensionless unit hydrograph. Runoff hydrographs computed with a peak rate factor of 484 were not as close to the observed runoff hydrograph as the hydrographs computed with the correct time of concentration and peak rate factor. These results support the hypothesis that the automatic fitting program can increase the accuracy of flood hydrograph predictions.

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APPENDIX H

A PLANNING METHOD FOR EVALUATING DOWNSTREAM EFFECTS OF DETENTION BASINS

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A PLANNING METHOD FOR EVALUATING DOWNSTREAM EFFECTS OF DETENTION BASINS[1]

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ABSTRACT: While storm water detention basins are widely used for controlling increases in peak discharges that result from urbanization, recent research has indicated that under certain circumstances detention storage can actually cause increases in peak discharge rates. Because of the potential for detrimental downstream effects, storm water management policies often require downstream effects to be evaluated. Such evaluation requires the design engineer to collect additional topographic and land use data and make costly hydrologic analyses. Thus, a method, which is easy to apply and which would indicate whether or not a detailed hydrologic analysis of downstream impacts is necessary, should decrease the average cost of storm water management designs. A planning method that does not require either a large data base or a computer is presented. The time coordinates of runoff hydrographs are estimated using the time-of-concentration and the SCS runoff curve number; the discharge coordinates are estimated using a simple peak discharge equation. While the planning method does not require a detailed design of the detention basin, it does provide a reasonably accurate procedure for evaluating whether or not the installation of a detention basin will cause adverse downstream flooding.
(KEY TERMS: detention; flood control; hydrology; planning; reservoirs; storm water management.)

INTRODUCTION

Storm water management (SWM) basins are the most widely used means of controlling the rate of runoff from developed areas. In many jurisdictions, SWM policies have been established with the intent of limiting the detrimental effects of urbanization on downstream areas. Usually, the policy requires that the peak runoff rate from the urbanized area be controlled so that the peak discharge rate after development does not exceed the predevelopment peak discharge rate. Unfortunately, limiting the peak discharge rate in this manner does not necessarily ensure that the detrimental effects at downstream points are minimized (McCuen, 1974, 1979).

Urbanization decreases the natural storage capacity of a watershed; therefore, the volume of direct runoff from a given depth of rainfall increases with development. Urbanization also has the effect of decreasing the time required for water to flow from remote points to the watershed outlet; that is, the time of concentration is inversely proportional to the degree of urbanization. Thus, urbanization affects both the volume and the timing of the runoff hydrograph. While SWM policies are designed to limit the effect of increases in peak discharge rate, they tend to ignore the effects of urbanization on the time characteristics of both direct runoff and flow through the SWM basin.

McCuen (1979) has shown that in some circumstances the change in timing caused by a SWM basin can result in increased downstream flooding. This type of situation is illustrated in Figure 1. In the predevelopment condition, the peak discharge rate from area 2 passes point A before the peak from area 1 arrives. After development, the peaks arrive at point A at nearly the same time; this causes constructive interference and results in a larger total peak discharge rate at point A, even though the peak flow from area 2 has not been increased over that which occurred prior to urbanization. Because of this possibility, the timing of the SWM basin outflow hydrograph should be considered when a SWM basin is to be used for controlling runoff from an area that is to be developed.

If SWM policies required all SWM basin analyses to include measuring the downstream effects, the cost of SWM design would increase substantially. In addition to the computer costs, it would be necessary to compile hydrologic, land use, and soil data bases for the downstream area. Because SWM will not always have a detrimental downstream effect, the average project costs would be decreased if a simplified method that would indicate whether or not a SWM basin would have a detrimental downstream effect were available; that is, a method is needed that can be used in the planning stage of site development to indicate whether or not downstream effects should be incorporated into the design of a SWM basin. The intent of this paper is to present such a method and illustrate its application.

DEVELOPMENT OF THE PLANNING METHOD

Analysis of Planning Requirements

The best way to determine the downstream effects of a SWM basin is to compare the predevelopment and postdevelopment

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hydrographs at downstream points. In the planning stage it is not necessary to have exact hydrographs; reasonably accurate estimates of the hydrographs should be sufficient. The hydrograph at a downstream point, such as point A in Figure 1, is the sum of the hydrographs from the upstream areas 1 and 2. The predevelopment hydrograph at A is the sum of the predevelopment direct runoff hydrographs from areas 1 and 2, while the postdevelopment hydrograph at A is the sum of the direct runoff hydrograph from area 1 and the SWM basin outflow hydrograph from area 2. Thus, the planning method consists of nothing more than a means of quickly estimating three hydrographs, the predevelopment direct runoff hydrograph from area 2, the direct runoff hydrograph from area 1, and the SWM basin outflow hydrograph representing the postdevelopment hydrograph from area 2.

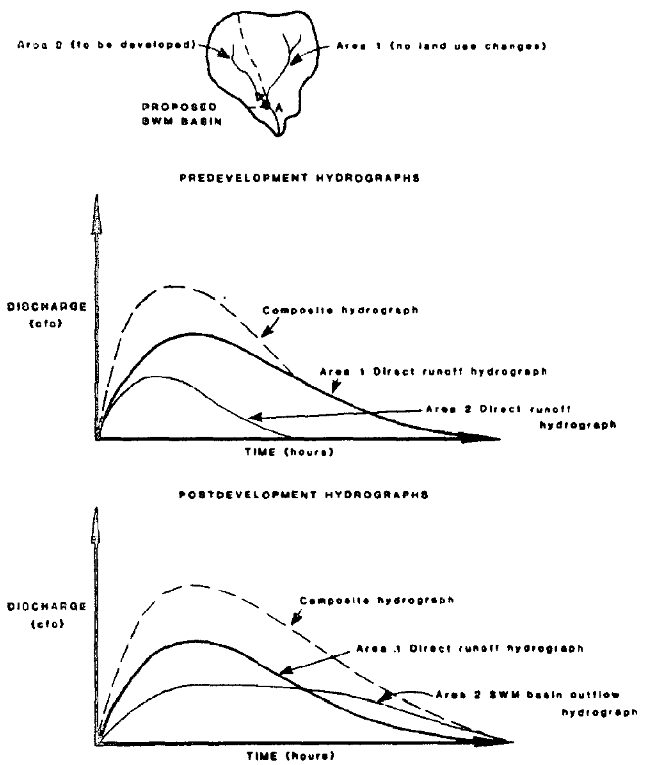

Figure 1. Illustration of the Possibility of Increased Peak Discharges Due to SWM Basins.

In order to simulate these hydrographs quickly, estimates of both the timing coordinates and the discharge coordinates are required. Two models that provide these estimates are the SCS TR-20 model and the TR-55 tabular model. The TR-20 model requires a considerable data base and a computer, and is therefore not practical for preliminary planning purposes. The TR-55 tabular method is less complex than TR-20 and can be performed by hand, but it is not capable of estimating SWM basin outflow hydrographs. The SCS methods were designed to simulate natural storage conditions rather than man-made SWM basins. Examination of a few SWM basin hydrographs showed that the characteristic shape of a SWM basin outflow hydrograph is considerably different from the characteristic shape of a direct runoff hydrograph. Examples of these two hydrograph shapes are shown in Figure 1. A SWM basin outflow hydrograph generally has a smaller slope on the rising limb and an extended period during which flows are at or near the peak rate; the shape of the basin outflow hydrograph is more nearly trapezoidal than the triangular shape of a direct runoff hydrograph. For this reason, separate models are required for estimating the SWM basin outflow and direct runoff hydrographs.

Storm water management policies often require SWM basins to control the postdevelopment runoff so that the peak discharge is no greater than the predevelopment peak discharge. Therefore, the predevelopment and postdevelopment peaks can be considered equal; the TR-55 graph method is a fast and widely used method of estimating peak discharge, although it does not provide any estimates of the timing characteristics of the runoff. To be consistent, it is best to use the same techniques for estimating both direct runoff and SWM basin outflow hydrographs; therefore, the logical method is to use the TR-55 graph method for estimating the peak discharge rates and to develop empirical equations for estimating the time coordinates of all the hydrographs. Because of the fundamental differences between the two types of hydrographs, it was not possible to develop one set of timing equations that would work for both types; therefore, separate sets of timing equations were developed for the runoff and SWM basin outflow hydrographs.

Data Generation

In order to develop equations for predicting time coordinates of hydrographs, a data base encompassing a wide range of watershed conditions was required. Collection of a set of measured hydrologic data that included the proper mix of watershed conditions was not possible; therefore, a synthetic data base was generated. Because of the fundamental differences between the characteristics of the direct runoff hydrographs and the SWM basin outflow hydrographs, different methods were used for synthesizing these two types of hydrographs.

Synthesizing the Direct Runoff Hydrographs. The data base for the direct runoff hydrographs was generated using the SCS TR-20 runoff model with a variety of watershed conditions. The usual data requirements for TR-20 include the area of the watershed, the average slope, the curve number, the hydraulic length, the time of concentration, and the depth of rainfall. The type II storm distribution with half-hour rainfall increments was used for all analyses in this study. The average slope and the hydraulic length are used only in computing the time of concentration; therefore, if the time of concentration is specified, then these data need not be included. The area does not affect the timing of the runoff hydrograph if the time of concentration is specified. Because the TR-55 graph method results in peak discharge in cfs per unit area rather than in cfs, it was not necessary to vary the watershed area in generating the data base. The remaining watershed variables are the curve number, the rainfall depth, and the time of concentration. Values of the curve number were 60, 70, 80, 90, and 95; values of the rainfall depth were 2.8, 4.3, and 7.0 inches, which correspond to the 2-year, 10-year, and 100-year storms in the state of Maryland; values of the time of concentration ranged from 0.25 to 2.0 hours in quarter-hour increments. Thus, 120 combinations of values were used to generate 120 direct runoff hydrographs; these hydrographs form the data base used in developing the timing equations for the direct runoff hydrographs.

Generating the SWM Basin Outflow Hydrographs. The SWM basin outflow hydrographs were generated using the Bondelid method (Bondelid and McCuen, 1980). Tests showed that the timing of the SWM basin outflow hydrograph is determined almost entirely by the depth of rainfall and the watershed conditions. The peak outflow rate at any one site may be limited to the predevelopment peak by using many different SWM basin configurations, but all of these configurations will result in nearly identical outflow hydrographs. This is rational because the peak flow rate and the volume of basin storage required are not functions of the SWM basin design parameters (i.e., size and number of riser pipes, depth, side slope); rather, the SWM basin parameters are dictated by the peak flow rate and the volume of basin storage that is required. The peak flow rate is determined by the predevelopment watershed conditions, and the volume of basin storage is determined by the differences between the predevelopment and postdevelopment watershed conditions. Therefore, the SWM basin design parameters have very little effect on the basin outflow hydrograph as long as the set of basin design parameters constitutes a feasible solution. Table 1 illustrates this point by showing the timing of the outflow hydrograph for each of several different feasible SWM basin designs. The outflow hydrographs of the various basin designs exhibit almost no variation in timing.

Because the outflow hydrograph is not sensitive to the basin parameters, the Bondelid program for generating feasible SWM basin designs was modified to give only one feasible solution for each combination of watershed conditions. The five watershed conditions that were varied in creating this data set are the watershed area, average slope, rainfall depth, predevelopment curve number, and postdevelopment curve number. The predevelopment and postdevelopment times of concentration were calculated using the SCS lag formula:

$$t_c = \frac{5}{3} \times \frac{L^{0.8}(S+1)^{0.7}}{1900\, Y^{0.5}} \tag{1}$$

in which t_c is the time of concentration (hrs), L is the computed hydraulic length (ft), Y is the average slope (percent), and S is related to the curve number by the function:

$$S = \frac{1000}{(\text{curve number})} - 10 \tag{2}$$

The computed hydraulic length, L, is derived from the watershed area using the function:

$$L = 209 \times (\text{area})^{0.6} \tag{3}$$

All of these equations are designed for use with SCS hydrograph models (SCS, 1972) and are compatible with the Bondelid method of designing SWM basins. The values of the watershed area that were used are 20, 60, and 100 acres; values of the predevelopment curve number were 60, 70, and 80; postdevelopment curve numbers ranged from 65 to 90 in increments of 5 and were always greater than the predevelopment curve number; values of the average slope were 1, 3, 5, and 7 percent; the rainfall depths were 2.8, 4.3, and 7.0 inches. Substituting the various combinations of these values into the

TABLE 1. Timing of SWM Basin Outflow Hydrographs From Various Basin Designs.

Design	Riser Diameter (ft)	Volume of Storage (acre-feet)	Surface Area (acres)	Depth* (ft)	Time (hrs) From Start of Precipitation				
					T_{50R}	T_{75R}	T_p	T_{75F}	T_{50F}
1	3.50	3.048	9.599	0.758	12.0	12.2	12.8	14.4	15.7
2	3.25	3.167	9.077	0.802	12.0	12.2	12.8	14.5	15.8
3	3.00	3.187	8.305	0.867	12.0	12.2	12.8	14.5	15.8
4	2.75	3.199	7.437	0.952	12.0	12.2	12.8	14.5	15.8
5	2.50	3.149	6.353	1.085	12.0	12.2	12.8	14.5	15.7
6	2.25	3.080	4.971	1.294	12.0	12.2	12.8	14.6	15.7

Watershed Conditions:

Watershed Area = 100 acres.
Predevelopment Curve Number = 70.
Postdevelopment Curve Number = 80.
Predevelopment Time of Concentration = 1.0 hours.
Postdevelopment Time of Concentration = 0.5 hours.
Rainfall Depth = 2.8 inches.

*Depth is maximum height of water surface above top of riser.

Bondelid program resulted in a data set of 324 SWM basin outflow hydrographs.

Developing the Timing Equations

An examination of the direct runoff hydrographs showed that they could be approximated with the required degree of accuracy by a triangular hydrograph; three points on the hydrograph had to be located, then joined by straight lines. The SWM basin outflow hydrographs are much flatter than the direct runoff hydrographs; therefore, five points were used in estimating the basin outflow hydrographs. Typical estimated hydrographs of both types are shown in Figure 2.

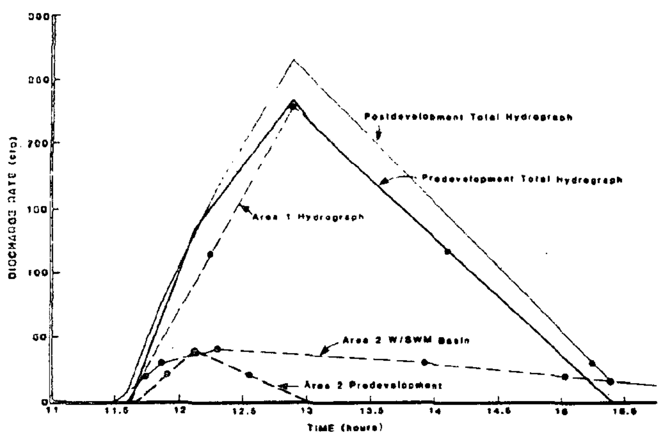

Figure 2. Estimated Hydrographs for Example of Planning Method.

Peak discharge rates for the estimated hydrographs were estimated using the TR-55 method; equations that can be used to estimate the timing coordinates of the hydrographs were developed. These equations were developed by using regression techniques with the generated hydrograph data base and are referred to as the timing equations.

Timing Equations for Direct Runoff Hydrographs. The first step in deriving the timing equations for the direct runoff hydrographs was to separate the synthesized data into three groups on the basis of the depth of rainfall. The hydrographs were synthesized for design storms of 2.8, 4.3, and 7.0 inches because these are the depths of the 2-year, 10-year, and 100-year design storms in Maryland. Separate equations were developed for each rainfall depth in order to determine whether one set of equations could be used for all return periods. A stepwise regression was then performed with each data set. The criterion variables were the time to peak (T_p), the time to 50 percent of peak on the rising limb (T_{50R}), and the time to 50 percent of peak on the falling limb (T_{50F}). The predictor variables were the curve number (CN), the time of concentration (t_c), the depth of runoff (RO), and the peak flow rate (Q_p). Correlation matrices of the predictor and criterion variables were calculated separately for each of the 2-year, 10-year, and 100-year design storms.

Analysis of the results showed that the correlations between predictor and criterion variables were very similar for all three design storms. In each regression, the first variable to enter the equation was the time of concentration; the curve number was always the second variable to enter. Because the differences in the rainfall depths of the three design storms did not make a significant difference in either the correlation between the variables or the regression coefficients, the data set was recombined and all 120 observations were used with the stepwise regression program. The resulting timing equations for the direct runoff hydrographs are shown in Table 2, along with goodness-of-fit statistics; these equations are applicable to 24-hour type II design storms of any return period. The very high correlation coefficients and low standard errors of estimate indicate that these three points (T_p, T_{50R}, and T_{50F}) can be predicted very accurately within the range of the calibration data.

Timing Equations for the SWM Basin Outflow Hydrographs. Equations were developed for estimating the times to five discharge stages of the basin outflow hydrographs. These five equations estimate the times to reach 50 and 75 percent of peak discharge on the rising limb (T_{50R} and T_{75R}, respectively), and time to peak (T_p), and the times to reach 75 and 50 percent of the peak discharge on the falling limb (T_{75F} and T_{50F}, respectively). The equations were developed by using stepwise regression techniques with the data set generated by the Bondelid method. The predictor variables that were available were: 1) the area, the average slope, and the computed hydraulic length of the watershed; 2) the predevelopment curve number, time of concentration, runoff depth, and peak discharge rate; 3) the postdevelopment curve number, time of concentration, runoff depth, and peak discharge rate that would occur if no SWM basin were installed; and 4) the change in the curve number, the rainfall depth, the volume of basin storage required, and the ratios of the predevelopment to postdevelopment runoff depths and the predevelopment to postdevelopment discharge rates. All 324 SWM basin outflow hydrographs were used as the data base; no distinction was made on the basis of rainfall depths because one set of timing equations was desired that would be applicable to any type II design storm depth. The timing equations that resulted from this analysis are shown in Table 3, along with goodness-of-fit statistics and the standardized partial regression coefficients.

The standardized partial regression coefficients in Table 3 are indicative of the relative importance of the predictor variables. On the rising limb of the hydrograph, the postdevelopment time of concentration is very important, while the runoff depth is considerably less important; both of these variables reflect the watershed conditions rather than the SWM basin design, so it seems that the timing of the outflow hydrograph on the rising side is almost entirely controlled by watershed characteristics. At the peak discharge and on the falling side of the hydrograph, the ratio of the postdevelopment peak discharge rate that would occur in the absence of a SWM basin to the predevelopment peak discharge rate is more important as a predictor variable than the depth of runoff. This ratio of

TABLE 2. Timing Equations and Goodness-of-Fit Statistics for the Direct Runoff Hydrographs.

Criterion		Equation	Correlation Coefficient	Standard Error of Estimate (hours)	Standard Deviation of Criterion Variable (hours)
T_{50R}	=	$12.20278 + \underset{(.8641)}{0.30759} * t_c - \underset{(-0.4089)}{0.00651} * CN$	0.9559	0.06	0.20
T_p	=	$12.32515 + \underset{(.9491)}{0.65588} * t_c - \underset{(-0.2012)}{0.00622} * CN$	0.9702	0.10	0.40
T_{50F}	=	$13.16962 + \underset{(.9136)}{1.36070} * t_c - \underset{(-0.2421)}{0.01613} * CN$	0.9452	0.28	0.86

NOTES:

t_c = time of concentration (hours).
CN = curve number.
T_{50R} = time to reach 50 percent of peak flow on rising limb (hours).
T_p = time to reach peak flow (hours).
T_{50F} = time to reach 50 percent of peak flow on falling limb (hours).
Numbers inside parentheses are standardized partial regression coefficients.

TABLE 3. Timing Equations for SWM Basin Outflow Hydrograph Estimation.

Criterion		Equation	Correlation Coefficient	Standard Error of Estimate (hours)	Standard Deviation of Criterion (hours)
T_{50R}	=	$11.79903 + \underset{(0.8548)}{0.53128} * t_{cpost} - \underset{(-0.3063)}{0.04274} * Q_{post}$	0.9640	0.06	0.23
T_{75R}	=	$11.90993 + \underset{(0.8679)}{0.70941} * t_{cpost} - \underset{(-0.2575)}{0.04727} * Q_{post}$	0.9533	0.09	0.31
T_p	=	$11.39838 + \underset{(0.6848)}{1.51487} * t_{cpost} + \underset{(0.7337)}{0.18329} * q_{post}/q_{pre}$	0.9387	0.29	0.83
T_{75F}	=	$9.95355 + \underset{(0.3613)}{3.12555} * t_{cpost} + \underset{(0.9349)}{0.91341} * q_{post}/q_{pre}$	0.9591	0.92	3.24
T_{50F}	=	$9.1829 + \underset{(0.3306)}{4.18501} * t_{cpost} + \underset{(0.9453)}{1.35179} * q_{post}/q_{pre}$	0.9632	1.28	4.74

NOTES:

t_{cpost} = postdevelopment time of concentration (hours).
Q_{post} = postdevelopment depth of runoff (inches).
q_{post} = postdevelopment peak discharge rate without SWM basin (cfs).
q_{pre} = predevelopment peak discharge rate (cfs).
Numbers inside parentheses are standardized partial regression coefficients.

peak flow rates is indicative of the degree to which the basin must control the runoff hydrograph. Thus, the importance of this ratio can be considered as an indicator of the importance of the SWM basin characteristics in determining the timing of the SWM basin outflow hydrograph. The standardized partial regression coefficients show that as the rate of discharge decreases the SWM basin design becomes more important and the postdevelopment time of concentration becomes less important to the timing of the hydrograph.

The equations in Table 3 are suitable for use for any 24-hour, type II design storm depth because the effect of variation in rainfall depth is included in each equation. In the first two equations the effect of rainfall depth is reflected in the postdevelopment depth of runoff. In the last three equations, the effect of rainfall depth is inherent in the ratio of the peak discharge rates, because the depth of rainfall is very important in determining the ratio. The change in the ratio with rainfall depth can be explained by reference to Figure 10-1 of Section 4 of the SCS National Engineering Handbook (SCS, 1972). For any pair of predevelopment and postdevelopment curve numbers, the ratio of the postdevelopment to the predevelopment runoff depth is much greater for small rainfall depths than for larger depths. This effect is due to the greater relative importance of the initial abstraction in smaller storms. In the data base used in this study, the average value of the ratio of peak discharge rates was about 5 for a 2.8 inch design storm and about 2 for a 7.0 inch design storm. This indicates that smaller storms require more control if the SWM basin outflow peak discharge rate is to be no greater than the predevelopment direct runoff peak rate.

PLANNING PROCEDURE

The planning procedure consists of estimation and comparison of the predevelopment and postdevelopment hydrographs at a critical downstream point. The critical point is usually the point at which the runoff from the area to be urbanized (area 2 in Figure 1) joins the runoff from the undeveloped area (area 1 in Figure 1). This junction point (point A in Figure 1) is critical because channel flow from this junction tends to smooth out the hydrograph and decrease the peak discharge rate; therefore, if constructive interference between the area 1 direct runoff hydrograph and the area 2 SWM basin outflow hydrograph causes the postdevelopment peak discharge rate to be greater than the predevelopment peak, the effect will be most apparent at this junction point.

In estimating the predevelopment and postdevelopment hydrographs at the junction point the discharge rates are estimated using TR-55, while the timing coordinates are estimated using the equations in Tables 2 and 3. An important part of the planning process is the selection of a design storm; in many jurisdictions, legislation or established policy dictates this choice. Selection of a design storm is specific to each situation and will not be discussed in this paper. Once the design storm has been selected, which establishes the depth of rainfall, the planning procedure proceeds as described in the following paragraphs.

Estimation of Direct Runoff Hydrographs

In using the TR-55 graph method for estimating peak discharge rates, the required data are the watershed area, the rainfall depth, the curve number, and the time of concentration. A number of methods for estimating time of concentration are described in the TR-55 manual; all of these methods are compatible with the timing equations. Once the data have been collected, TR-55 can be used to estimate the peak flow rates. Then the equations in Table 2 can be used to estimate the time coordinates. The three points on the hydrograph may be located by the coordinate pairs (T_{50R}, $Q_p/2$), (T_p,Q_p), and (T_{50F}, $Q_p/2$). Connecting these points with straight lines, as in Figure 2, completes the estimation of the direct runoff hydrographs for the undeveloped area (area 1 in Figure 1) and for the predevelopment conditions on the area being developed (area 2 in Figure 1).

Estimation of SWM Basin Outflow Hydrograph

The estimation of the SWM basin outflow hydrograph requires knowledge of the postdevelopment curve number and time of concentration as well as the watershed area and rainfall depth. These values are used with TR-55 to estimate the postdevelopment peak discharge rate from area 2 that would occur in the absence of SWM basin (q_{post}). Once q_{post} has been determined, the ratio q_{post}/q_{pre} can be calculated. This ratio, the postdevelopment depth of runoff (Q_{post}), and the postdevelopment time of concentration (t_{cpost}) are substituted into the equations in Table 3 to estimate the time of coordinates of the SWM basin outflow hydrograph. Because the SWM basin peak discharge rate is to be controlled so that it is no greater than the predevelopment peak discharge from area 2 (q_{pre}), q_{pre} is used for the flow rate coordinates. The five points on the SWM basin outflow hydrograph have coordinates (T_{50R}, $q_{pre}/2$), (T_{75R}, $3q_{pre}/4$), (T_p, q_{pre}), (t_{75F}, $3q_{pre}/4$), and (T_{50F}, $q_{pre}/2$); the lines connecting these points represent the estimated SWM basin outflow hydrograph.

Estimation of the Downstream Hydrographs

The important hydrographs are not those for the individual areas, but rather the downstream hydrographs formed by the addition of the individual area hydrographs. Figure 2 shows how the downstream hydrographs are generated. The predevelopment downstream hydrograph is simply the sum of the predevelopment area 2 direct runoff hydrograph and the area 1 direct runoff hydrograph. The postdevelopment downstream hydrograph is the sum of the SWM basin outflow hydrograph and the area 1 direct runoff hydrograph. In Figure 2 the installation of a SWM basin appears to increase the peak flow at downstream points significantly.

Example of the Planning Method

To illustrate its use, the planning method was applied to the Crabbs Branch watershed in Montgomery County, Maryland. Figure 1 contains a schematic diagram of the watershed, the data and calculations appear in Table 4, and the estimated hydrographs are shown in Figure 2. The hydrographs in Figure 2 indicate that the proposed development of area 2 and the installation of a SWM basin will result in an increased peak discharge rate at point A. The predevelopment peak rate is about 230 cfs and the postdevelopment peak is about 265 cfs, indicating an increase of roughly 15 percent. In order to substantiate these results, the example was recomputed using TR-20 to generate the direct runoff hydrographs and the Bondelid method (Bondelid and McCuen, 1980) to generate the SWM basin outflow hydrograph. The results of this simulation indicated an increase in peak discharge only slightly lower than that indicated by the planning method.

CONCLUSIONS

On some watersheds the use of SWM basins to control runoff from developing areas can increase discharge rates at downstream points rather than limiting discharges to those which occurred prior to development. In this paper, a quick planning method for estimating the potential for adverse downstream effects of a SWM basin has been developed. The planning method involves estimation of direct runoff hydrographs and SWM basin outflow hydrographs. The TR-55 graph method

TABLE 4. Worksheet for Planning Method.

Design Storm: Return Period = 10 years   P (depth of rainfall) = 4.3 inches

Variable		Area 1*	Area 2** Predevelopment	Area 2** Postdevelopment
A	= watershed area (square miles)	A_1 = 0.80	A_2 = 0.07	A_3 = 0.07
CN	= curve number	CN_1 = 65	CN_2 = 58	CN_3 = 81
t_c	time of concentration (hours)	t_{c1} = 1.5	t_{c2} = 0.25	t_{c3} = 0.10
S	= retention = (1000/CN)–10	S_1 = 5.385	S_2 = 7.241	S_3 = 2.346
Q	= runoff volume = $(P-0.2S)^2/(P+0.8S)$ (inches)	Q_1 = 1.21	Q_2 = 0.806	Q_3 = 2.38
q_u	= unit peak discharge from Fig. 5-2, TR-55 manual (CSW/inch)	q_{u1} = 238	q_{u2} = 735	q_{u3} = 1000
q_p	= peak discharge = A * q_u * Q (cfs)	q_{p1} = 230	q_{p2} = 41.5	q_{p3} = 167
a	= q_{p3}/q_{p2}	–	–	a = 4.02
T_{50R}	= time-to-rise to 50 percent of q_p (hours)	Eq. 1 12.24	Eq. 4 11.90	Eq. 7 11.75
T_{75R}	= time-to-rise to 75 percent of q_p (hours)	–	–	Eq. 8 11.87
T_p	= time-to-rise to q_p (hours)	Eq. 2 12.91	Eq. 5 12.13	Eq. 9 12.29
T_{75F}	= time-to-fall to 75 percent of q_p (hours)	–	–	Eq. 10 13.94
T_{50F}	= time-to-fall to 50 percent of q_p (hours)	Eq. 3 14.16	Eq. 6 12.57	Eq. 11 15.04

Eq. No.	Timing Equations for Area 1
1.	$T_{50R} = 12.20278 + 0.30759\ t_{c1} - 0.00651\ CN_1$
2.	$T_p = 12.32515 + 0.65588\ t_{c1} - 0.00622\ CN_1$
3.	$T_{50F} = 13.16962 + 1.36070\ t_{c1} - 0.01613\ CN_1$

Eq. No.	Timing Eqs. for Area 2: Predevelopment
4.	$T_{50R} = 12.20278 + 0.30759\ t_{c2} - 0.00651\ CN_2$
5.	$T_p = 12.32515 + 0.65588\ t_{c2} - 0.00622\ CN_2$
6.	$T_{50F} = 13.16962 + 1.36070\ t_{c2} - 0.01613\ CN_2$

Eq. No.	Timing Eqs. for Area 3: Postdevelopment
7.	$T_{50R} = 11.79903 + 0.53128\ t_{c3} - 0.04274\ Q_3$
8.	$T_{75R} = 11.90993 + 0.70941\ t_{c3} - 0.04727\ Q_3$
9.	$T_p = 11.39838 + 1.51487\ t_{c3} + 0.18329\ a$
10.	$T_{75F} = 9.95355 + 3.12555\ t_{c3} + 0.91341\ a$
11.	$T_{50F} = 9.18290 + 4.18501\ t_{c3} + 1.35179\ \epsilon$

*Area 1 is the area upstream from the area being developed.
**Area 2 is the area being developed.

is used for estimating discharge rates, and empirical timing equations are used for estimating the time coordinates of the hydrographs. The method was calibrated using simulated hydrographs developed with the TR-20 model. The calibration data included times of concentration from 0.25 to 2.0 hours, and testing indicates that the method is reasonably accurate for times of concentration as low as 0.10 hours. Once the allowable peak discharge rate from a SWM basin has been set, variations in the SWM basin design parameters do not significantly affect the SWM basin outflow hydrograph; therefore, the planning method does not require that a SWM basin be designed. The planning method is a fast and reasonably accurate way to determine whether or not installation of a SWM basin will increase flooding at downstream points.

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APPENDIX I

INTEGRATED STORMWATER MANAGEMENT DESIGN

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INTEGRATED STORMWATER MANAGEMENT DESIGN

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INTRODUCTION

The changes that occur during the development of a watershed have a direct influence on the runoff response of a watershed to a precipitation event. The impact of land development is often assessed by the extent of land use changes that accompany land development. Land use changes, especially in urban areas, usually cause increases in both the peak runoff rate and the runoff volume. The degree of pervious cover is reduced, thereby changing the timing characteristics of a watershed and decreasing the travel time. Less natural storage is available during the initial period of runoff, which causes the volume of the runoff hydrograph to increase.

When the extent of land cover change is significant, the detrimental consequences of such change, including downstream flooding, erosion, and poor water quality, must be prevented with appropriate stormwater management (SWM) techniques. Many of the adverse consequences may occur concurrently at different locations and may create additional problems if not properly mitigated. For example, the runoff response of construction sites that are left devoid of vegetative cover will result in larger direct runoff volumes within a shorter time span, increases in the velocity of overland flow, and a significant increase in erosion and sediment transport. Therefore, a variety of criteria must be examined in measuring the effectiveness of a SWM program. These efficiency criteria may include: 1) flood peak control; 2) flood volume control; 3) erosion control;

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4) sedimentation capacity; 5) reduction in overland flow velocity; 6) induced infiltration and groundwater maintenance; and 7) various water quality enhancements. The selection of appropriate SWM control measures should reflect the effect of the program on each of these efficiency criteria.

Various SWM measures have different efficiencies with respect to each of the above criteria. Thus, single method control plans do not achieve maximum efficiency. The use of more than one SWM method should take advantage of the strength provided by each measure with respect to all of the criteria listed. Hence, an integrated approach to SWM control can maximize efficiency by accounting for the interaction and interdependent effect of the individual SWM methods. The implementation of a combination of SWM controls depends on policy requirements, as well as hydrologic and hydraulic considerations. The objectives are to assess the efficiency criteria obtainable by various SWM control techniques and to propose an integrated SWM design method that will maximize the efficiency of a SWM program.

EFFICIENCY CRITERIA FOR STORMWATER MANAGEMENT METHODS

The goal of a SWM program for controlling storm runoff volumes and rates, erosion, and sediment transport is to provide the controls that are necessary to ensure that there is little change in the response of a watershed when compared to that resulting from the original predevelopment conditions. The changes in land cover and drainage patterns may cause the natural drainage capabilities to be exceeded, and thus, require implementation of SWM controls.

The following is a summary of various efficiency criteria that should be used to assess the effectiveness of SWM control programs. The adaptability and feasibility of a SWM control technique is reflected by the extent of fulfillment to each of these criteria. The summary should aid in the selection of the appropriate approach or combination of approaches to mitigate the adverse effects of development.

Peak Runoff Rate

One of the primary effects of land cover changes is the significant increase in peak runoff rates when compared with the predevelopment peak rates. The amount of increase will depend upon the extent of development and the distribution of impervious cover. Reductions in the peak discharge are often the only criteria that is required in the design of SWM facilities. Detention basins, for example, are commonly accepted as being a sufficient SWM control because the structure is capable of limiting the peak outflow release rate to predevelopment rates. However, the time-to-peak is often changed significantly and the duration of the peak rate is extended (McCuen, 1979). This results in an extended period of downstream bankfull flows.

SWM controls that reduce peak runoff rates are effective in limiting downstream flow rates. However, detention basin designs most often are limited to the control of a single return period; this may lead to inadequate control of storms for other return periods (Kamedulski and McCuen, 1979). Evaluation of risks associated with peak flows often constitutes the major concerns of SWM strategies when a watershed undergoes extensive urbanization.

Volume of Runoff

The decrease in the natural available storage of a watershed causes a corresponding increase in the volume of direct runoff. Hence, various SWM measures that provide for either man-made storage or natural storage attempt to replace the natural storage that was lost in development. The downstream flow capacity requirements may, therefore, be alleviated. Man-made storage methods (e.g., holding ponds, rooftop ponding, and parking lot storage) are most often implemented to substitute for losses in natural storage. The effectiveness in controlling runoff volumes depends on the available storage within the structure and the location of the storage control facility within the watershed.

Erosion Control

Construction activities are generally known to generate substantial amounts of sediment. Prevention of erosion from exposed soil surfaces is a necessary conservation practice that should be included in planning phases. Methods to stabilize the surface to resist erosion are more effective in controlling sediment than measures that attempt to mitigate effects on flows that are heavily ladened with sediment. Hence, efforts to prevent erosion should be investigated, including the lining of ditches.

Sedimentation Controls

Erosion occurs despite the careful implementation of preventive measures. Hence, additional attempts should be made to avoid sediment transport to receiving streams. Both structural and nonstructural methods are available, including: 1) check dams that increase the settling of solids; 2) vegetative covers that act as filters to "strain" sediment; 3) filter fences; 4) energy dissipating devices to reduce both scour at outfalls and channel erosion; and 5) sediment traps. Site conditions will often dictate the overall effectiveness of the alternative methods.

Reduction in Overland Flow Velocities

Increases in runoff rates are associated with increases in overland flow velocities. High flow rates can cause serious damage to surface conditions, drainage patterns, and the integrity of channels. Urbanization results in velocities of sufficient magnitude such that extensive storm drain construction is required to convey flows around or through environmentally sensitive areas. Delaying of flows in storage structures, increasing surface retardance, and temporary barriers are alternate methods of reducing flow velocities.

Induced Infiltration

Increases in the percentage of impervious area will decrease the natural infiltration and depression storage causing changes in the timing of runoff. Inducement of infiltration enables the maintenance of the natural drainage system and replenishment of groundwater supplies. Volumes of runoff may be effectively removed from runoff hydrographs if sufficient vegetative buffer zones are maintained. Overland flow is detained and decreased by that amount which is infiltrated. Porous pavements and infiltration beds also utilize natural infiltration to decrease runoff.

Water Quality Enhancement

The runoff quality criteria is not independent of water quantity control criteria. The factors that are important in controlling runoff rates are not separate from those that are important in water quality control. For example, the control of sedimentation is considered to be water quality criteria because many pollutants are adsorbed on sediment surfaces and transported along with suspended solids. Hence, many water quality parameters are linked to water quantity criteria.

AN INTEGRATED APPROACH TO SWM DESIGN

The hydrologic model used as the basic element of an integrated approach is the Soil Conservation Service (SCS) tabular method (SCS, 1975). The tabular method provides a means of estimating the runoff hydrograph from small watersheds undergoing development. The tabular method is adapted because of the versatility in evaluating SWM designs. The method is designed to develop composite hydrographs at any point within the watershed for one or more subwatershed areas. Additionally, the tabular method permits the assessment of effects that a combination of SWM controls have on the composite hydrograph.

Tabular hydrograph discharge values are computed for subareas using a time of concentration (t_c) and a total travel time (t_t) to the design point as input. The peak discharge is determined from the equation:

$$q = q_p \quad A \; Q \tag{1}$$

where q is the estimated peak discharge in cubic feet per second, q_p is the tabular unit peak discharge rate in cubic feet per second per square mile per inch of runoff, A is the drainage area in square miles, and Q is the direct runoff in inches. The hydrograph coordinates at the design point due to each subarea is determined using the values of t_c, t_t, A, Q, and q_p for each subarea. The total composite hydrograph is computed by summing the subarea hydrographs.

The effects of development can be determined by computing the discharge hydrographs for present and future conditions. Land cover changes alter both the peak discharge and timing characteristics of the runoff. Additionally, the direct runoff volume, Q, is often greater after development due to increases in the percentage of impervious area.

To offset the effects of development, SWM control measures are implemented. The effectiveness of a SWM method will vary depending on both the hydrologic and hydraulic conditions of the site, the importance of the efficiency criteria discussed previously, and the extent of development. Control of the peak discharge rate and volume of runoff is managed through reductions in the direct runoff, Q. The values of direct runoff may be sufficiently reduced depending on the type and spatial configuration of the SWM controls employed. When SWM controls are employed extensively, greater reductions in direct runoff are achieved. The composite hydrograph resulting from the addition of alternate SWM controls is computed by:

$$q = q_p \quad A \; Q' \tag{2}$$

in which Q' is an adjusted direct runoff depth that represents the future conditions with the addition of alternate SWM control methods. The value of Q' will be assessed in the same way as the value of Q except that the value will be reduced by the amount of storage allocated to the storage losses for SWM control in the subarea; that is, Q' is the difference between Q and the losses due to SWM methods within the subarea.

A method of estimating the required volume of storage within detention basins and holding ponds is outlined by SCS (1975). These methods provide design procedures to estimate the storage required in order to limit the outflow to a predetermined flow rate. The methods are based on average storage and routing effects for many single-stage structures, specifically for weir flow and pipe flow structures.

Recognizing that detention facilities do not provide the most efficient solution to all of the problems created by development, a SWM solution may provide for incorporating various SWM measures in an integrated program. Also, the implementation of many alternate SWM measures may effectively delay and reduce the runoff volume and discharge rates that enter detention basins. Thus, the total amount of storage volume required in detention basins can be reduced when including the use of alternate SWM methods. The performance efficiency of the SWM methods can be evaluated in terms of the criteria previously discussed.

STORMWATER MANAGEMENT CONTROL METHODS

The control of stormwater runoff may be managed through a variety of alternative methods. The selection of particular control methods will vary depending on the conditions of development, physical land features, and hydrogeologic conditions. Regardless of the SWM method selected, proper planning and design procedures must be maintained to ensure an efficient control strategy.

The design procedures of SWM practices are important because efficiency criteria are not applicable for all development conditions. Limitations exist for some SWM methods where conditions are not favorable. For example, the use of infiltration trenches may not be practical in areas with high water tables. Therefore, design parameters should be investigated to determine the feasibility of implementing alternate SWM controls. A brief description of the design considerations and efficiency criteria applicable to selected SWM methods are discussed.

Vegetative Buffer Strips

Vegetative buffer strips provide for infiltration and natural storage of surface runoff. Buffer strips are also effective for controlling sediment loadings. The vegetation should be a deep rooted, dense grass that is placed perpendicular to surface flows. Design parameters include: 1) soil type; 2) surface slopes; 3) type of vegetation; and 4) overland flow velocity (Wong and McCuen, 1981a). Placement of vegetative buffer strips will depend on the area available and funds allotted for installation.

Among the efficiency criteria satisfied are: reductions in the runoff volume, erosion control, sediment removal, reductions in flow velocities, and water quality enhancement. Provisions for infiltration are made available by delaying surface flows to promote natural storage and thus, replenishing of groundwater supplies. The vegetative cover serves as a filter to remove both suspended solids and pollutants that are attached to the surface of sediment particles. Also, vegetative covers stabilize the soil surface, reducing the erosive forces of precipitation and overland flow. While vegetative buffer strips satisfy the majority of efficiency criteria, they are seldom effective when applied without a series of other SWM alternatives because of their limited effect on flow volumes.

Infiltration Trenches

Infiltration trenches decrease the volume of runoff by providing subsurface storage. Trenches are excavated and filled with crushed stone where runoff is stored within available pore space. Filter clothes and topsoil backfill are placed over the stones to filter runoff, prevent the clogging of voids, and increase the design life (Wong and McCuen, 1981b).

Design considerations are important to achieve maximum efficiency. Topsoil backfill and filter cloths overlaying stones need to have high permeability rates to allow surface flows to be transmitted into storage. High groundwater elevations are to be avoided so that depletion of storage is permitted. Additional vegetative buffer strips placed upstream will further filter sediment particles. Maintenance is required when void spaces are clogged.

Rooftop Ponding

Rooftop ponding can be very effective for controlling runoff volumes, especially in highly urbanized commercial, industrial, and multi-family residential developments. Controlled flow roof drains are commercially available and provide the means of reducing both flow rates and the volume of immediate runoff. Since rooftops must be designed to withstand snow loadings, the roofs can also be used for detention storage and provide as much as five area inches of detention without structural modifications. Scuppers should be installed to prevent the depth of ponding from exceeding five inches.

Design requires estimation of the depth-discharge relationship for the rooftop outlet structure (McCuen and Piper, 1975). The volume of precipitation retained (as opposed to detained) can be used as the volume for reducing Q in Eq. 1; it is especially important to convert this volume to an area-depth amount that is compatible with the subwatershed subarea. The volume retained will be reflected in the lower portion of the depth-discharge relationship; it is a function of the outflow characteristics of the drain.

Parking Lot Storage

Parking lot storage can be implemented to control runoff from large acres of paved areas, including shopping centers and industrial districts. Because paved areas have very high runoff potentials in terms of both volumes and peak rates, parking lot storage may reduce runoff rates by providing temporary storage where flows are collected and released at a regulated rate. Water can be ponded around portions of the parking lot that are least used to minimize inconveniences to the public.

The volume of storage available is dependent on site conditions. The slope and roughness of the area will govern the rate of inflow, and the depth of ponding should not exceed eight inches for public safety reasons. The rate of release may be regulated at the outlet with control devices, such as constricting pipes. Proper design should prevent flooding with provisions for overflow during extreme storm occurrences. Permanent water storage is not usually provided and, therefore, parking lot storage is not effective in pollution control. Maintenance is easily managed using mechanical street cleaning machines.

The depth of runoff (i.e., Q in Eq. 1) can be reduced by the volume stored only if the outlet facility is designed as a retention storage basin. That is, the outlet facility should consist of one pipe or opening that releases water for a duration of approximately 24 hours and a second larger pipe or opening to handle runoff that is in excess of the available storage. Only the storage volume released through the smaller pipe should be considered in adjusting the value of Q in Eq. 1.

Porous Pavement

Porous pavements are a relatively new concept in SWM. Development of porous pavements to infiltrate runoff from traditionally impervious surfaces is still in the evaluation phase; however, many porous pavement sites do exist today. Porous pavements will become more widely accepted as a viable SWM control measure as performance criteria are further assessed.

Porous pavements are advantageous in reducing runoff volumes by allowing water to absorb into the pavement, thus utilizing the natural storage in the ground below the pavement. Hence, groundwater recharge is enhanced. The effects of creating a pervious area that would normally be impervious to water (e.g., parking lots and roadways not receiving heavy loads) can reduce the flow capacity required for storm drainage systems and reduce peak discharge rates.

Many design considerations need to be addressed to ensure the proper performance of porous pavement systems during operation. The physical feasibility is often limited to areas where the hydraulic conductivity of the underlying soil is sufficiently high to transmit the downward movement of water; subdrains may be installed at regular intervals to expediate the drainage of saturated layers. The drainability requirement of the porous pavement surface and base courses are satisfied with open graded aggregate mixtures. Both the durability and the load-bearing strength of the pavement are also important design components. Additional design requirements include problems of withstanding freeze-thaw cycles and the maintenance of clogged pores.

INTEGRATED DESIGN IN COASTAL AREAS

Currently, there is special concern over the stresses placed on coastal environments, including wetlands, because of increased development, both urban and agricultural, in coastal areas. Many states, county, and local governments are developing stormwater management policies to control the adverse impact of land development. Because of the proven effectiveness of on-site detention storage, many policies that require or favor on-site detention exist. While on-site may be a very effective control measure, there is some concern about its applicability in coastal areas because of the large surface area that would be necessary to meet the volume requirements.

The integrated design approach has significant potential for use in coastal areas because the use of a variety of stormwater management methods will reduce the volume of direct runoff and, thus, the volume of storage required. For urban development in coastal areas, surface storage methods, such as rooftop and parking lot detention, should be encouraged. For areas subject to agricultural development pressures, vegetative buffer strips and infiltration trenches amy be used to decrease the volume of surface runoff and, thus, the required on-site detention storage. In areas where water tables are high, every attempt should be made to increase the length of drainage paths and other surface storage.

CONCLUSIONS

In summary, the interaction of the various efficiency criteria are apparent. The physical impact of land development may create a variety of problems that need to be identified with respect to each of the many criteria outlined herein. Once these problems are recognized, then adequate SWM controls may be implemented, with each SWM control measure exhibiting its strengths.

The selection of specific SWM control measures to be implemented into an integrated design approach should be based on the effect of the control measure on the efficiency criteria outlined. Such an integrated approach can maximize performance efficiency with respect to all of the criteria by accounting for the interdependent effects of the SWM control measure. A composite of SWM controls becomes effective because individual controls are inadequate in satisfying all the efficiency criteria individually.

The integrated design approach recommended herein uses the SCS methods to estimate the runoff hydrographs and the storage-volume estimation techniques for estimating the volume of detention storage. The effects of combining SWM controls into a comprehensive design scheme can be evaluated using these procedures. Such hydrologic modeling of watersheds will permit the evaluation of design techniques to ensure that the intended SWM goals are achieved.

Watersheds that undergo development are subject to many adverse consequences that, if not corrected, may cause both structural and environmental damage. Unplanned growth should be avoided because potential problems are not determined. Proper planning should stem from policies that specify the intended goals of SWM and adequate design techniques to translate policy into realized benefits.

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APPENDIX J

THE DESIGN OF VEGETATIVE BUFFER STRIPS FOR RUNOFF AND SEDIMENT CONTROL

Stanley L. Wong

Richard H. McCuen

THE DESIGN OF VEGETATIVE BUFFER STRIPS FOR RUNOFF AND SEDIMENT CONTROL

Stanley L. Wong and Richard H. McCuen[1]

INTRODUCTION

Structural measures, such as detention basins, are often used for the management of stormwater runoff; they are used to reduce both the volume and the rate of runoff that enters receiving streams. While detention basins provide storage of runoff, they are costly, require special design techniques, and land must be designated for the single-purpose usage of ponding water. A more economical method of managing runoff is with vegetative controls, which usually consists of dense turf judiciously placed across the path of surface runoff. Vegetative buffer strips have other inherently favorable aspects; in addition to being ecominical, they are a multi-purpose control method. When properly used, they serve as a protective cover in reducing the potential of erosion from exposed soils while promoting conservation practices. This multi-purpose control method has aesthetic and recreational value. The savings in both cost and time during both the design and construction phases are additional incentives.

Buffer strips have not received the attention they deserve as a means of controlling stormwater runoff because they have not been viewed as a viable alternative to the detention basin. Design methods have not been developed that can account for the effects of buffer strips on runoff volumes and sediment loadings. However, it seems reasonable that the total volume of detention storage that is required to mitigate the effects of development could be reduced if properly designed and located buffer strips were used to reduce runoff volumes and sediment loadings into the detention facility.

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The design of a buffer strip for controlling runoff and sediment will be a function of both stormwater management policy considerations and physical characteristics of the site. For example, the required length of the buffer strip will depend on the desired sediment trap efficiency, which is usually established by policy. The soil-cover complex at the site will control the infiltration and thus the ability of the buffer strip to control runoff rates. The objective of this study was to develop a method for sizing buffer strips and computing the reduction in the volume of direct runoff.

LITERATURE REVIEW

A number of studies have been undertaken to examine the effectiveness of buffer strips in removing sediment and other pollutants. However, those methods that presented results of field studies often did not attempt to develop design criteria.

The size distribution of soil particles is important in determining the amount of sediment that is deposited within the buffer strip and the portion of material that remains in the runoff (Meyer, 1976). Neibling and Alberts (1979) measured the amount and the particle size distribution of naturally eroded sediment retained by various length of vegetative filters under broad sheet flow conditions. The sod buffer strips achieved reductions of 56, 70, 94, and 95 percent for lengths of 2, 4, 8, and 16 feet, respectively, for particle sizes that are greater than 0.02 mm. The results suggest an effective removal of medium silt sized sediment when the ratio of the slope length of the contributing area to the buffer strip length is 10 percent.

Wilson (1967) studied sediment removal from runoff using grass filtration. The development of an economical and efficient method of improving the quality of water required for artificial recharge stimulated the research. Several field experiments were conducted using various lengths of Bermuda grasses. A maximum percentage of sand, silt, and clays were trapped at buffer lengths of 10, 50, and 400 feet, respectively. The Bermuda grasses (coastal and common) were found to be the more efficient variety of vegetative filter as a result of a higher roughness coefficient. A sediment trap efficiency of 95 percent for a 700 foot buffer strip with very flat slopes and a flow rate of 0.011 cfs per foot of width of buffer strip. The suggested criteria for selecting the grass filter media are: 1) a deep root system, 2) dense, well branched top growth, 3) resistance to flooding and droughts, and 4) an ability to recover growth after sediment inundation.

Vegetative filters may also enhance the water quality of streams with the treatment of runoff from livestock feedlots, which are characteristically high in both sediment concentrations and nutrients. Vanderholm and Dickey (1978) observed reductions of nutrients, solids, and oxygen-demanding materials on the order of 80 percent on a concentration basis and 95 percent on a mass-balance basis. The results were obtained by monitoring four field systems over a period of two years. Based on the recommendation for a minimum contact time of 2 hours, the design criteria for overland-flow filter lengths ranged from 300 feet for slope of 0.5 percent to 860 feet for a slope of 4 percent. The contact time is a function of the flow length, velocity of flow, slope, and type of vegetation.

Young et al. (1979) reported an average reduction of 92 percent of the sediment and 80 percent of the volume of runoff from vegetative buffer strips 80 feet in length. The experimentation was conducted using a rainulator to simulate rainfall over an active feedlot and to induce runoff and erosion. The authors concluded that vegetative filters provide sufficient treatment of feedlot runoff.

Considerable research has also been conducted in the area of controlling erosion from highway construction activities. Many structural and vegetative control methods are available to limit erosion and sedimentation during and after construction. Vegetative controls are used quite extensively as a soil stabilizer and as a sediment filter. Ohlander (1976) used field data to develop a regression equation to estimate the amount of sediment trapped by a vegetative buffer located below a road drainage outlet. The standard buffer length capable of trapping 1 ton of sediment per year was given by:

$$L = (16.16 + 17.69\ K + 1.34\ S)\ \left(\frac{31\ CN}{6900 - 69\ CN}\right) \tag{1}$$

in which L is the standard buffer length as a slope distance in meters, K is the soil erodibility index from the Universal Soil Loss Equation, S is the slope in percent, and CN is the SCS runoff curve number. The ratio of the actual buffer length to the standard buffer length times 1 ton yields the annual amount of sediment trapped. Eq. 1 has not been experimentally verified; however, it does provide a means of assessing alternative planning and management schemes.

Recognizing that field studies could not be totally controlled experiments, Tollner, et al. (1976) conducted experiments using a laboratory flume to simulate the flow of sediment ladened water through an artificial rigid grass media. Under laboratory conditions, a range of values for the independent variables were obtained by changing experimental conditions. The dependent variable of interest was the outflow sediment concentration. A model was developed to estimate the fraction of sediment trapped using transformations and performing a regression analysis. The probability of a particle being trapped was related to both the level of turbulence and the potential number of times in which the particle came into contact with the bottom while being transported. The fraction of sediment trapped was given by:

$$TE = \exp\left[-1.05 \times 10^{-3} \left(\frac{V_m R_s}{\nu}\right)^{0.82} \left(\frac{v_s L_T}{V_m d_f}\right)^{-0.91}\right] \tag{2}$$

in which TE is the trap efficiency, V_m is the mean flow velocity in feet per second, R_s is the spacing hydraulic radius in feet, ν is the kinematic viscosity in square feet per second, v_s is the setting velocity in feet per second, L_T is the section length in feet, and d_f is the depth of flow in feet. A correlation coefficient of 0.87 was reported for the data set. The authors found the mean velocity to be the most influential parameter that effects the sediment trapped. Eq. 2 relates the important physical parameters in a quantitative manner and can serve as a useful design tool when all of the variables can be quantified.

DESIGN OF VEGETATIVE BUFFER STRIPS FOR SEDIMENT CONTROL

The sediment yield from a watershed is a nonpoint source pollution problem that is not easily remedied. Improper management of highway construction or land development, cultivation of agricultural fields, and strip mining, for example, contribute to potential erosion problems by exposing the soil to the weathering processes of nature. Eroded soil that result from these disturbances often disrupt the delicate ecological balances of receiving waters through the deposition of sediment.

The mathematical model of Eq. 2 includes variables that reflect the important design factors for vegetative buffer strips, including soil characteristics, cover complex characteristics, and runoff characteristics. The velocity of runoff depends on the roughness of the vegetal cover and can be computed using a modified Manning's equation:

$$V_m = \frac{1.49}{n} R_s^{2/3} S^{1/2} \tag{3}$$

in which V_m is the mean flow velocity in feet per second, n is Manning's roughness coefficient, R_s is the spacing hydraulic radius in feet, and S is the buffer strip slope in feet/feet. Recommended values of n are 0.20, 0.35, and 0.80 for light turf, dense turf, and conifer/deciduous forest with dense grass understory respectively; the values assume the grasses are in good hydrologic condition.

The characteristics of the grass media may be described by the spacing hydraulic radius, R_s. This parameter reflects the arrangement of the grass media spacing in combination with the flow depth and is given as (Tollner, et al. 1976):

$$R_s = \frac{S_s d_f}{2d_f + S_s} \tag{4}$$

in which R_s is the spacing hydraulic radius in feet, S_s is the grass media spacing in feet, and d_f is the flow depth in feet. The flow depth of runoff should be restricted to a height less than the top of the grass media since the development of Eq. 2 applies to the case of nonsubmerged flow. This restriction does not limit the applicability of Eq. 2 since little sediment is trapped if the buffer strip were allowed to become flooded.

The trap efficiency is related to the mean settling velocity of the suspended sediment particles. The sediment particles settle at a rate such that the weight of the particle exceeds or equals the resistance of its downward movement. The settling velocity of spherical grains under laminar conditions may be computed from Stokes law:

$$v_s = (s-1) \frac{g}{18\nu} D^2 \tag{5}$$

in which v_s is the mean settling velocity in feet per second, s is the specific gravity of the sediment particle with respect to the fluid, g is the acceleration of gravity or 32.2 feet per sec^2, ν is the kinematic viscosity of water in $feet^2$ per second, and D is the particle size in feet.

The solution of Eq. 2 for computing the sediment trap efficiency of a vegetative buffer strip can be represented graphically. Figure 1 shows the relationship between trap efficiency and the length and slope of the buffer strip, as well as the roughness coefficient of the vegetation. The required length of a buffer strip is very sensitive to variation in the trap efficiency as it approaches

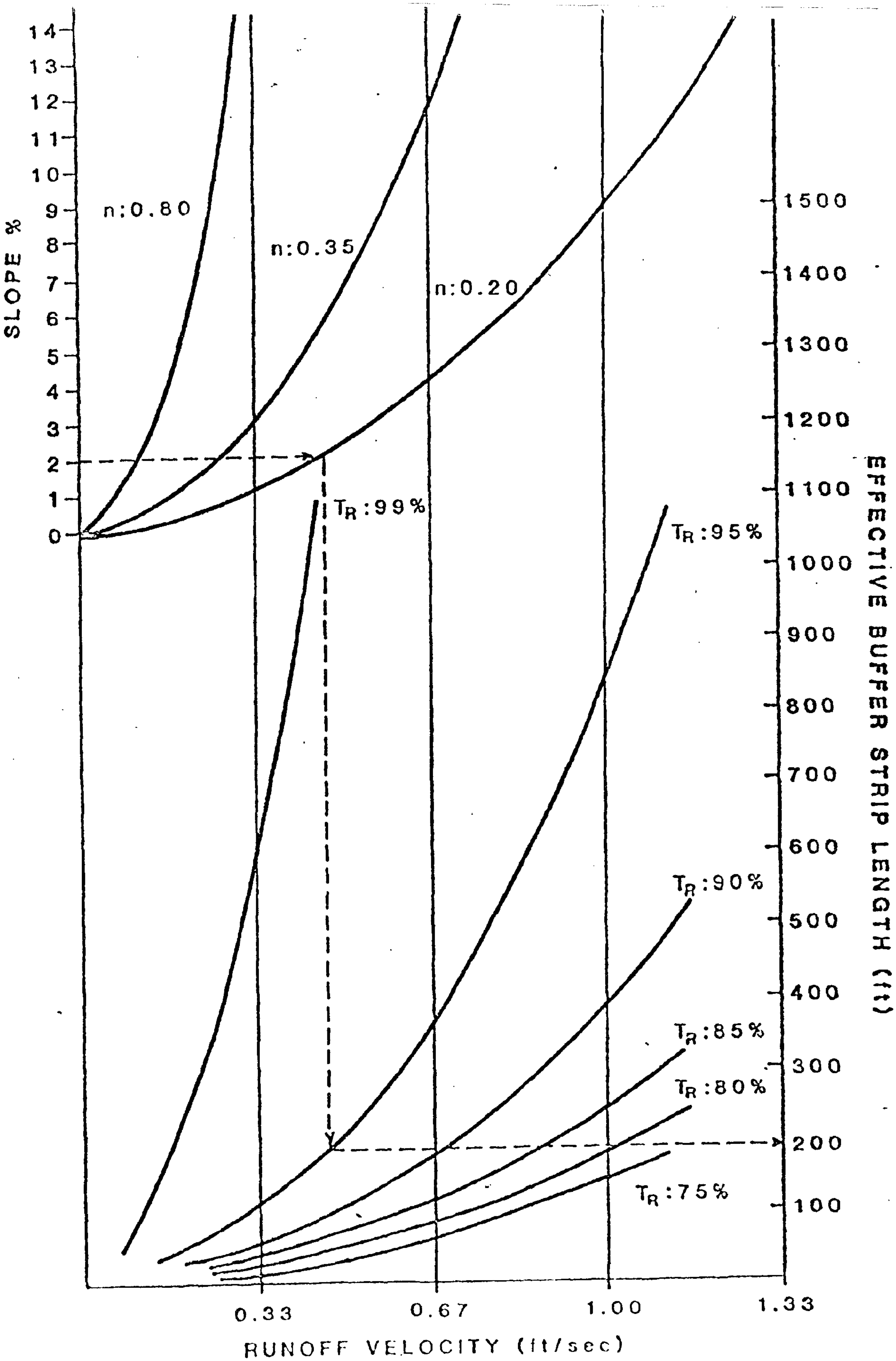

FIGURE 1. Effective Buffer Length Determination For Trap Efficiencies (T_R) of 75, 95, and 99 percent

100 percent, indicating that a small incremental increase in the trap efficiency requires a considerable addition in the buffer length. The curves also suggest that a significant trap efficiency (up to 75 percent) may be achieved at relatively short buffer lengths. Figure 1 assumes a coarse silt with a mean settling velocity of 0.002 feet per second through a buffer strip with an average spacing hydraulic radius of 0.010 feet. Curves similar to Fig. 1 are easily derived for other soil textures.

The trap efficiency for other soil textures may also be determined using Fig. 1. The settling velocity of sediment particles manifests the appropriate trap efficiencies that are attainable using buffer strips for a particular particle size. In general, the greater the settling velocity, the higher the trap efficiency per length of buffer strip. For example, the ratio of the settling velocities for a coarse silt and a fine silt is 4.9, the buffer strip length obtained from Fig. 1 should be multiplied by this ratio to obtain the buffer strip length for a fine silt. This would provide the same trap efficiency indicated on Fig. 1. The settling velocity ratio of coarse silt to medium silt, fine sands, and medium sands are 1.3, 0.02, and 0.005, respectively.

THE EFFECT OF VEGETATIVE BUFFER STRIPS ON RUNOFF QUANTITY

While vegetative buffer strips are designed primarily to control sediment, they also reduce the volume of runoff that results from excess rainfall. A reduction in the runoff volume occurs as the vegetation impedes and retards the flow of water, allowing a portion of it to infiltrate into the soil. The rate of infiltration is a function of: 1) the condition of the vegetative cover, 2) the properties of the underlying soil, 3) the rainfall intensity, and 4) antecedent soil conditions. These factors act in an interrelated manner to influence the amount of water that infiltrates into a buffer strip.

The infiltration of surface water is a complex process that is time variant. The rate at which a soil may absorb water will vary with time, decreasing in an exponential manner until a final infiltration rate is achieved. Most methods of estimating infiltration are based on empirical formulas that represent the results of field observation. Other methods are based on theoretical solutions of equations for porous media flow; however, many approximations have been developed.

In addition to soil characteristics, the volume of water that infiltrates is also a function of the physical characteristics of the buffer strip. The ability of the vegetation to retard flow has the effect of decreasing the velocity of the runoff, which increases the detention time of the overland flow. The increased detention time increases the opportunity for infiltration. Differences in flow velocities can be significant for different types of vegetative cover (i.e., different values of n).

Recognizing that the infiltration losses may be small, the estimated reduction of the surface runoff from a vegetative buffer strip may be computed using the simplified model:

$$V_L = f_c t_c L \tag{6}$$

in which V_L is the volume of infiltrated water per width of buffer strip in cubic feet per foot, f_c is the equilibrium or minimum infiltration rate in feet per hour, t_c is the overland flow travel time over buffer strip in hours, and L is the length of buffer strip in feet. Values of the minimum infiltration rate are given in Table 1 for various texture classes and soil types, where the soil types are classified using the SCS hydrologic soil groups A, B, C, and D; the part of Table 1 used to estimate f_c will depend on the type of soil data that is available. The solution to Eq. 6 is provided in Figs. 2, 3, and 4 for the vegetative covers of light grasses, dense grasses, and conifer/deciduous forest with dense grass understory respectively. The reduction in

TABLE 1. Minimum Infiltration Rates (f_c) for Nine Soil Texture Classes and Four SCS Soil Groups

Soil Texture	f_c (in/hr)	SCS Soil Group	f_c (in/hr)
Sand	4.64	A	0.43
Loamy Sand	1.18	B	0.26
Sandy Loam	0.43	C	0.13
Silt Loam	0.26	D	0.03
Loam	0.13		
Sandy Clay Loam	0.06		
Clay Loam	0.04		
Silty Clay Loam	0.04		
Sandy Clay	0.03		
Silty Clay	0.02		
Clay	0.01		

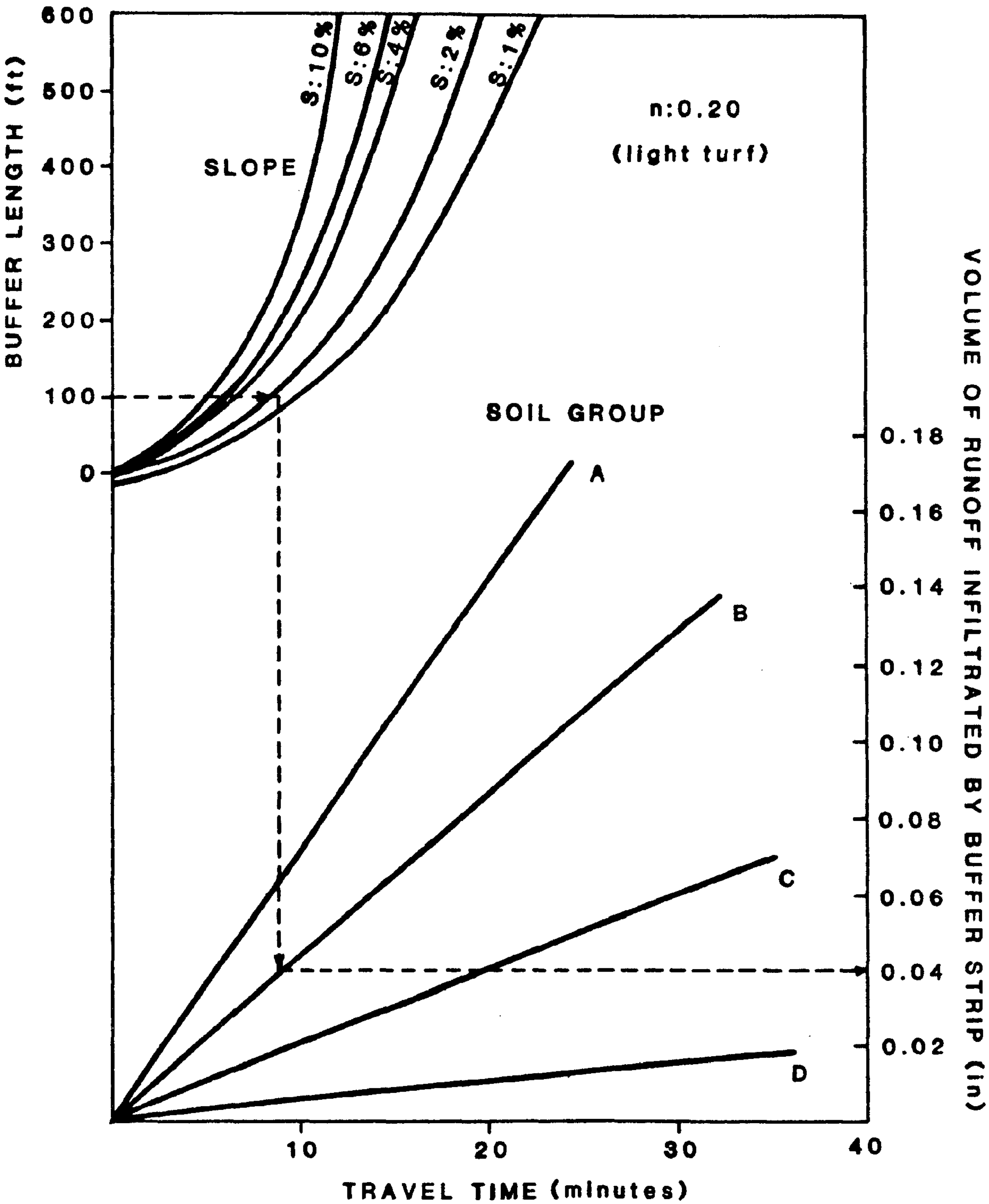

FIGURE 2. Determination of Volume of Infiltrated Runoff From Vegetative Buffer Strip (Light Turf)

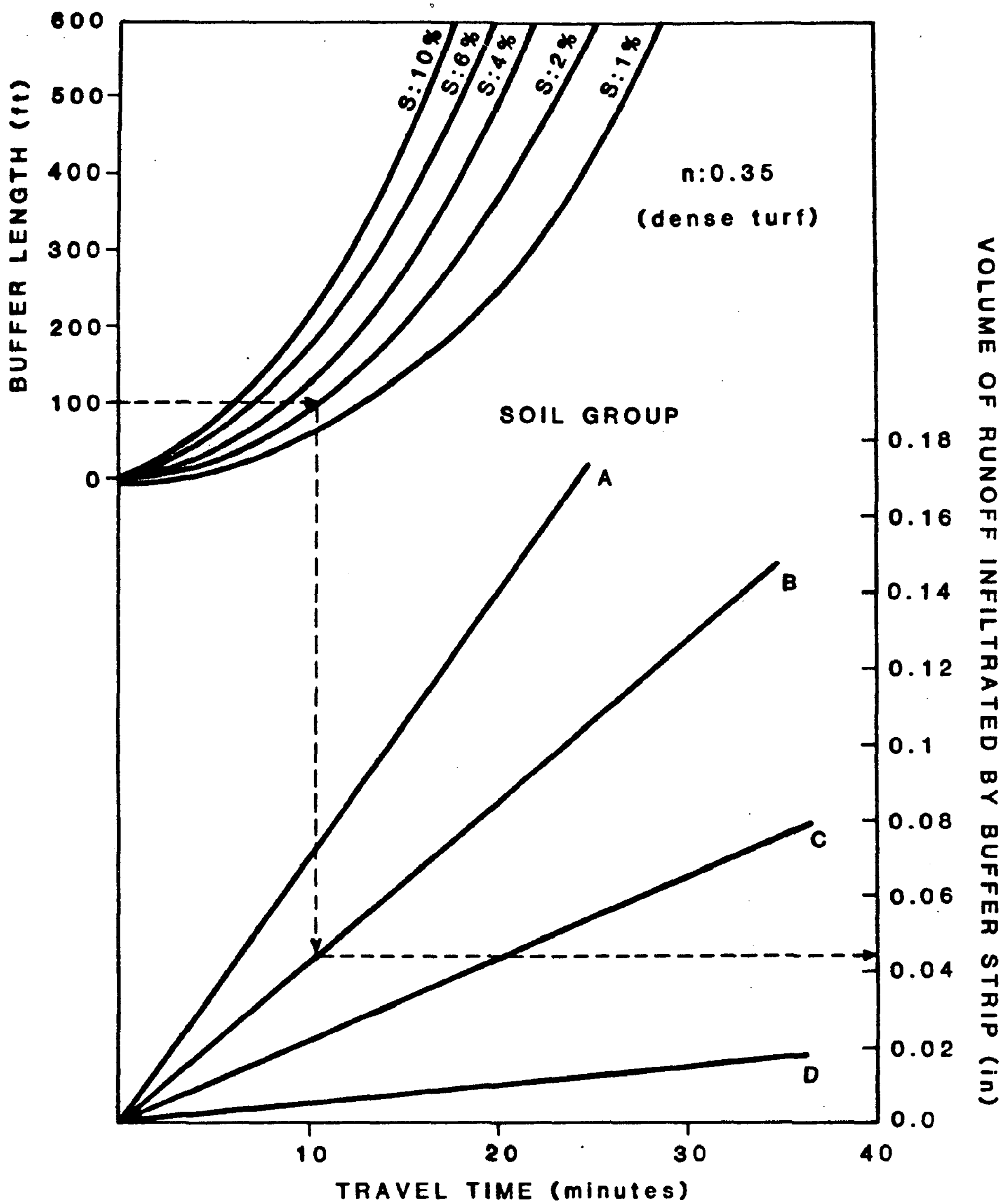

FIGURE 3. Determination of Volume of Infiltrated Runoff From Vegetation Buffer Strip (Dense Turf)

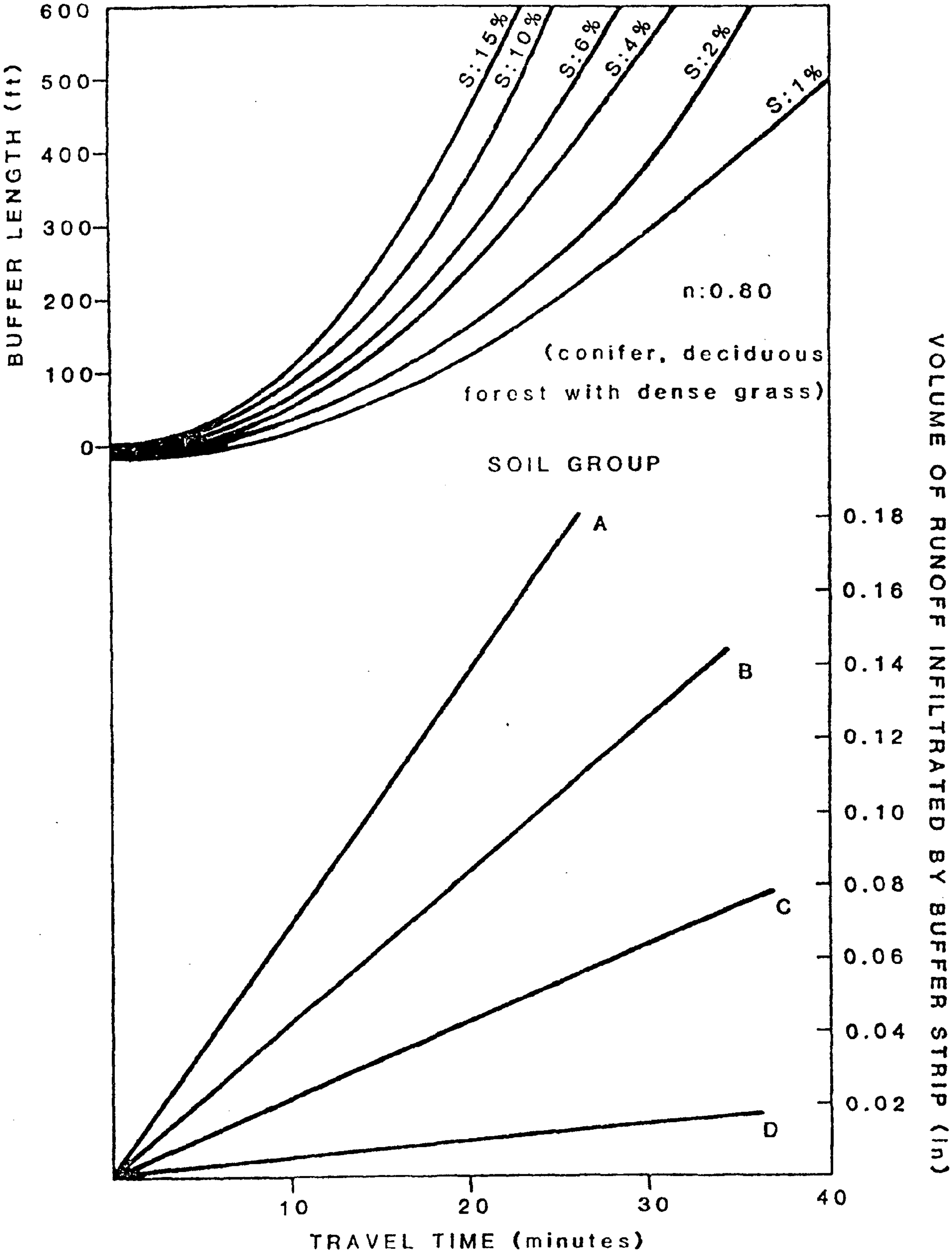
600
500
400
300
200
100
0
BUFFER LENGTH (ft)
S:15%
S:10%
S:6%
S:4%
S:2%
S:1%
n:0.80
(conifer, deciduous
forest with dense grass)
SOIL GROUP
A
B
C
D
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
VOLUME OF RUNOFF INFILTRATED BY BUFFER STRIP (in)
10
20
30
40
TRAVEL TIME (minutes)

FIGURE 4

the volume of runoff is expressed as inches of infiltrated water per unit area of buffer strip.

The use of Figs. 2, 3, and 4 for computing losses from vegetative buffer strips provides a means of assessing its effectiveness as a stormwater management control measure in reducing the runoff volume. The procedure is a conservative approach because the condition of minimum infiltration rate is assumed.

APPLICATION OF THE METHOD

The area of study in which the design procedures are illustrated is a 47.7 acre watershed in Montgomery County, Maryland. The watershed area has five distinct subwatersheds; a detention basin is located at the lower end of the watershed. Fig. 5 depicts a schematic of the watershed and summarizes the characteristics of the subareas. The land use consists of grass, roadways, rooftops, and exposed soils. The soils are predominantly Chester silt loams that are well drained and moderately erodible.

The average annual watershed erosion is estimated using the Universal Soil Loss Equation. The average watershed erosion may be determined from the empirical formula:

$$A = R \cdot K \cdot LS \cdot VM \tag{7}$$

in which A is the soil loss in tons per acre per year, R is the rainfall factor, K is the soil erodibility factor, LS is the slope-length factor, and VM is the vegetatibe-erosion control factor. The computation of the average annual watershed erosion is shown in Table 2. The total estimated annual erosion is 49.14 tons/year.

The area adjacent to the upstream segment of the detention basin may be utilized to establish a buffer strip, which under other circumstances might be either bare soil or grass in poor condition. Fig. 3 was used to obtain the

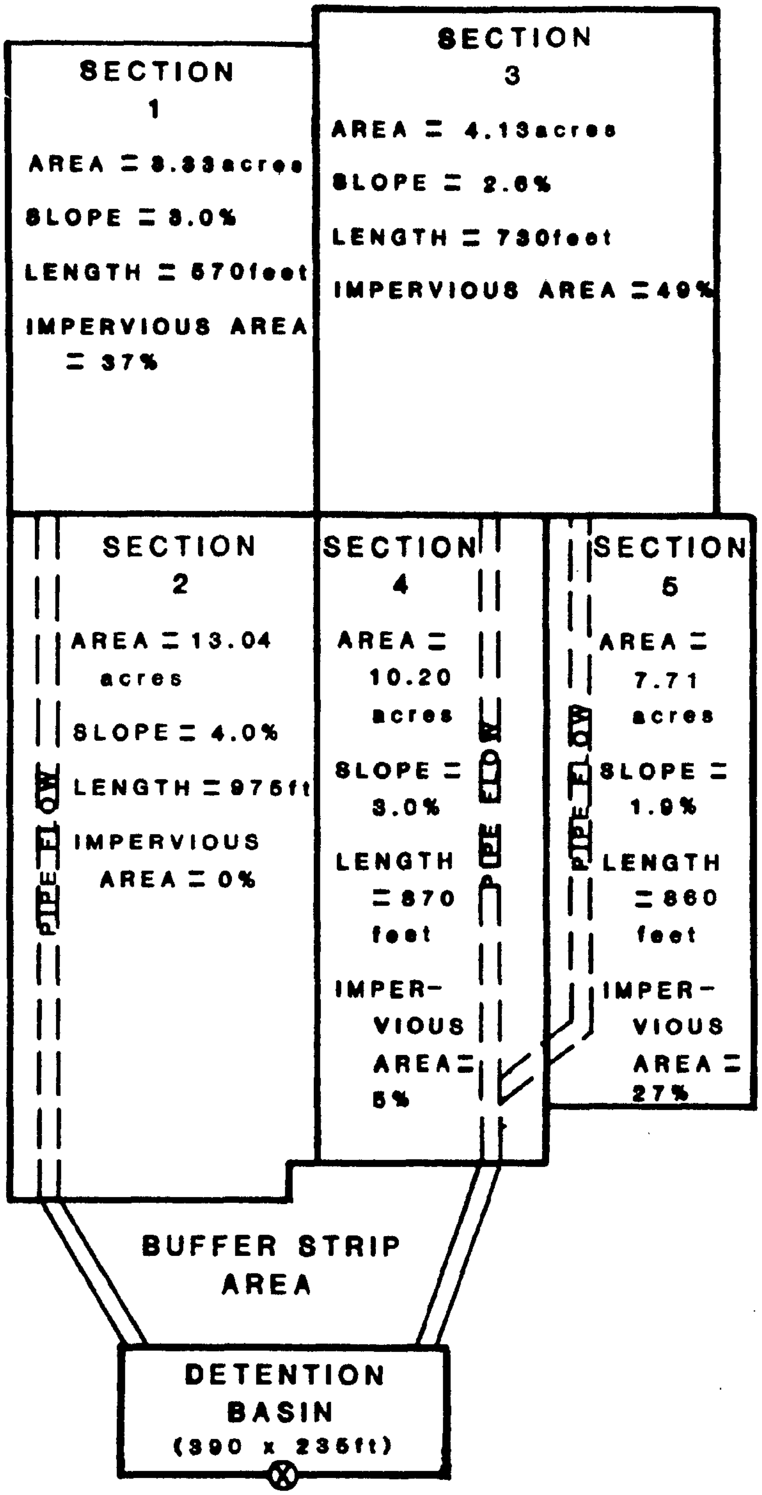

FIGURE 5. Conceptualization of Study Watershed and Subarea Characteristics

TABLE 2. Estimation of the Average Annual Erosion From the Study Watershed*

Section No.	Area (acres)	Slope (ft/ft)	Length (ft)	Percent of Previous Area Covered With Grass	VM Vegetative Factor	LS Factor	A (tons/acre/yr)	Total Soil Loss (tons/yr)
1	3.33	0.030	570	85	0.03	0.48	0.81	2.68
2	13.04	0.040	975	75	0.04	0.97	2.17	28.33
3	4.13	0.026	730	80	0.04	0.45	1.01	4.16
4	10.20	0.030	870	85	0.03	0.55	0.92	9.43
5	7.71	0.019	860	85	0.03	0.35	0.59	4.53
						Average Annual Soil Loss		= 49.14 tons/yr
								= 1.03 ton/acre/yr

*R = 175; K = 0.32

trap efficiency for various lengths of buffer strips that could be located immediately upstream from the detention facility. Table 3 lists the potential trap efficiencies of various vegetative buffer length when implemented within the available area. The corresponding reduction in the estimated average annual sediment loading is also given in Table 3. Table 4 is a summary of observed sediment volumes at the inlet of the existing detention basin. The 90 percent sediment trap efficiency from a 150 foot length of buffer strip will effectively reduce the observed volume of sediment entering the detention basin by the amount recorded in Table 4.

The disparity between the total estimated annual and the observed sediment volumes is due to the degree of development that the watershed was undergoing when the observed sediment volumes were measured. During the period of the ten storm events, the SCS curve numbers ranged from 77 to 90, 70 to 81, 79 to 88, 79 to 89, and 85 to 94 for sections 1 through 5, respectively. The computed soil loss, using Eq. 7, does not reflect the transition period of the watershed during the development stages. The computations in Table 2 represent the erosion in the post-development stage.

A reduction in the volume of runoff depends upon the overland flow travel time, the minimum infiltration rate, and the length of available buffer strip. The volume of runoff infiltrated by a buffer strip may be determined from Figs. 2 and 3, which is given in inches of losses per area of buffer strip. The available buffer strip located upstream of the detention basin has dimensions of a 150 foot length and a 400 foot width. The infiltration losses over a buffer strip having 3 percent slope and B type soil will be 0.041 inches, from Fig. 2. This reduction in runoff results as the flow travels the distance of 150 feet within a 9 minute travel time. Thus, the total volume of infiltrated flow for the duration of the runoff is equal to the product of the area of buffer strip and the duration divided by the travel time over the buffer strip. A volume of 205 cubic feet of runoff is infiltrated over the buffer strip in 9

TABLE 3. Potential Trap Efficiency Attainable and Reduction in Estimated Sediment Using Vegetative Buffer Strips

Available Buffer Strip Length (ft)	Potential Trap Efficiency (%)	Reduction in Sediment Using Buffer Strip* (tons/year)
50	75	36.85
100	82	40.29
150	90	44.22
200	93	45.69
250	94	46.18
300	95	46.67

*a slope of 3.0%, light turf (n = 0.20)

TABLE 4. Summary of Measured Sediment Volumes for Ten Storm Events and the Potential Sediment Reduction from a Vegetative Buffer Strip

Date	Measured Sediment Volume (tons)	Watershed Sediment Loading (tons/acre)	Potential Sediment Reduction from 150 foot Buffer Strip (tons)
6/28/77	24.83	0.52	22.35
7/12/77	8.09	0.17	7.28
7/17/77	79.24	1.66	71.32
7/20/77	0.20	0.01	0.18
8/1/77	21.23	0.45	19.11
8/5/77	0.26	0.01	0.23
8/8/77	2.85	0.06	2.57
8/10/77	1.74	0.04	1.57
8/14/77	31.45	0.66	28.31

minutes. Table 5 summarizes the reductions in volumes that would have occurred for the ten actual storm events for which data were measured. The percent reduction of runoff into the detention basin is greatest for the smallest storm event and, in general, the total volume loss will increase when the duration of the runoff is longer.

CONCLUSIONS

Vegetative buffer strips can be used to control both the quality and quantity of surface runoff. Because the quantity of runoff intercepted by the buffer strips will be minimal, the design is most often based on quality control criteria, specifically the sediment trap efficiency. In actual practice, the sediment trap efficiency would be set by a stormwater management policy or regulation. Thus, the designer would determine the buffer strip length that is required to trap the required fraction of the sediment in the inflow. Once the length of the buffer strip has been determined for quality control, the effect on the runoff volume can be determined. The depth given in area-inches by Figs. 2 and 3 can be converted to a total volume by multiplying by the area of the buffer strip.

For most design problems, buffer strips will not be capable of controlling sediment and runoff volumes by themselves. Thus, they will have to be used as one component of an integrated stormwater control system; that is, they may be used in either series or parallel with other control methods to achieve a desired level of control at a site. In order to be effective, the characteristics of buffer strips must be considered in the site location plan. The design curves provided herein assume nonsubmerged flow. Therefore, flow rates may have to be controlled by other means to ensure that acceptable flow rates are maintained. Additionally, if sediment volumes in the inflow are excessive, considerable volumes of sediment may be deposited at the upper end of the buffer strip. With time, a berm may form, and the berm may be washed-out when an extreme rainfall

TABLE 5. Summary of Observed Volumes and the Potential Reduction of Flow Entering Detention Basin

Date	Observed Flow Volume (ft^3)	Duration of Runoff (min)	Total Volume of Flow Infiltrated (ft^3)	Percent Reduction (%)*
6/28/77	33,890	240	5,460	16.1
7/12/77	24,492	180	4,100	16.7
7/17/77	77,441	47.5	1,082	1.4
7/20/77	7,896	240	5,467	69.2
8/1/77	42,373	117.5	2,676	6.3
8/5/77	320	150	320	100.0
8/8/77	13,485	180	4,100	30.4
8/10/77	6,831	120	2,733	40.0
8/14/77	60,847	117.5	2,676	4.4

*for a Buffer Strip having dimensions of 150 ft x 400 ft, 3% slope, n = 0.2, Hydrologic Soil Group B.

event occurs. Thus, it is important either to prevent the berm from forming or to provide proper maintenance. It is possible to prevent the berm from forming by trapping extremely large volumes of sediment by other methods.

The processes of filtration and deposition within a grass buffer strip will also lessen the loadings of any nutrients and pesticides that are carried with the runoff by trapping those sediment particles which transport them. Additional quality control will result if significant volumes of water are encouraged to infiltrate. Infiltration of water containing soluable pollutants will reduce the loadings of such pollutants that reach the stream with the direct runoff. Infiltration can be encouraged by reducing the slopes of the area allocated to buffer strips or increasing the surface roughness. If the natural soil is not highly porous, the buffer strip may be placed over an infiltration bed that uses a soil that has a high infiltration rate. Significantly larger volumes of runoff, and thus soluable pollutants, will be removed from the direct runoff.

The design of effective vegetative buffer strips to properly manage runoff requires a strategy of evaluating performance criteria. A design method was provided herein which can be used to evaluate both the sediment trap efficiency and the reduction of runoff volumes due to the infiltration of overland flow from vegetative buffer strips. The design method uses soil characteristics, site characteristics, and policy information, all of which are important in design. Vegetative buffer strips are recommended for those areas that would otherwise be exposed soil.

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APPENDIX K

DESIGN OF INFILTRATION TRENCHES FOR CONTROL OF STORMWATER RUNOFF

Stanley L. Wong

Richard H. McCuen

DESIGN OF INFILTRATION TRENCHES FOR CONTROL OF STORMWATER RUNOFF

Stanley L. Wong and Richard H. McCuen[1]

INTRODUCTION

In the past, urbanization has usually begun with the installation of storm sewers for controlling the hydrologic effects of increased imperviousness in developing communities. After considerable development has taken place, often with little or no advanced planning, it was recognized that storm sewer systems are inadequate by themselves. Thus, it became necessary to incorporate other structural measures, such as detention ponds, to control the increased volume and rate of runoff. While the stormwater detention provided control of runoff rates, its effect on the volume and duration of storm runoff has been shown to be less than acceptable in many cases (McCuen, 1979). Additionally, these structural control methods fail to reverse the trend towards reduced groundwater recharge that accompanies urbanization and they require expensive land. Thus, there is considerable interest in stormwater management methods that provide for more "natural" control, especially vegetative and nonstructural control methods. While structural controls can probably not be eliminated entirely, the use of these measures may often be improved by combining them with methods that utilize the process of induced infiltration.

Infiltration trenches are also applicable in agricultural areas. Although infiltration rates of cultivated land are often thought to be high, a combination of various agricultural practices may result in large quantities of runoff. For example, the seasonal transitions of the crop growth cycle effects the volume

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of flows, with larger volumes of runoff occurring during periods of fallow land-use. Poor conservation practices coupled with soil textures having a high run-off potential frequently create runoff problems that require control measures. The control measures that reduce the volume of runoff from agricultural fields have an additional benefit of reducing pollutant loadings that often accompany flows to receiving streams.

An infiltration trench is a control method that enhances the infiltration of stormwater into groundwater flow systems. Past research has not provided a procedure for designing infiltration trenches. This method of inducing infiltration is analyzed herein, with an emphasis upon the design criteria for estimating the dimensions of a trench that could control a portion of the volume of excess direct runoff.

CONSIDERATIONS IN DESIGN AND CONSTRUCTION

An infiltration trench may be described as a structural device for inducing infiltration into the subsurface soils, thus replenishing groundwater supplies. However, the primary goal of such a stormwater management device is more often used to aid in the control of surface flows, rather than to serve principally as an artificial recharge device. Trenches are excavated and filled with crushed stone, which stabilizes the side walls and provides a significant increase in the storage capacity of the subsurface area; the trench can also serve to filter sediment from the runoff. The volume of void spaces tends to clog with time; hence, a filter cloth can be placed over the stone and beneath a topsoil backfill. The filter cloth should prevent clogging and, thus, increase the design life of the infiltration trench. The infiltration characteristics of either the filter cloth or the topsoil backfill can be the limiting design factor; thus, it is important to consider their characteristics in design. The topsoil overburden will prevent damage to the filter cloth and will also serve to filter pollutants included in the stormwater runoff; such pollutants could clog the void spaces in the bottom and sidewalls of the trench. The configuration of an infiltration trench is shown in Fig. 1.

Sizing of Trench Dimensions

Runoff from both impervious surfaces and surfaces with low infiltration capacities may be intercepted by infiltration trenches, which are preferably located in low sloped areas. The temporary storage and eventual percolation of storm runoff into the soil is the primary purpose of an infiltration trench. The desired dimensions of an infiltration trench will depend upon the volume of direct runoff that requires control and the characteristics of the watershed and soils. In general, the design of a trench must consider the limiting effects of the storage volume and the infiltration characteristics of both the topsoil backfill and the soil surrounding the infiltration trench. Quite often, the design assumes that the infiltration capacity of the surrounding soil is very small; this is a reasonable assumption because it would not be practical to install infiltration trenches in areas with highly porous soils.

The volume of available storage within an infiltration trench, which is the primary design variable, is a direct function of the porosity of the crushed stone or gravel fill material. The porosity, defined as the ratio of the volume of voids to the total volume of the infiltration trench, may be expressed as a percentage of the total volume; it is also called the percent voids. The void spaces are filled with water where available storage exists to detain surface flows. The maximum volume of storage occurs when the crushed stone is saturated, so that all available voids are occupied.

Various combinations of depths and widths of infiltration trenches may be used to achieve the necessary cross-sectional area of storage. The total volume of storage is computed by:

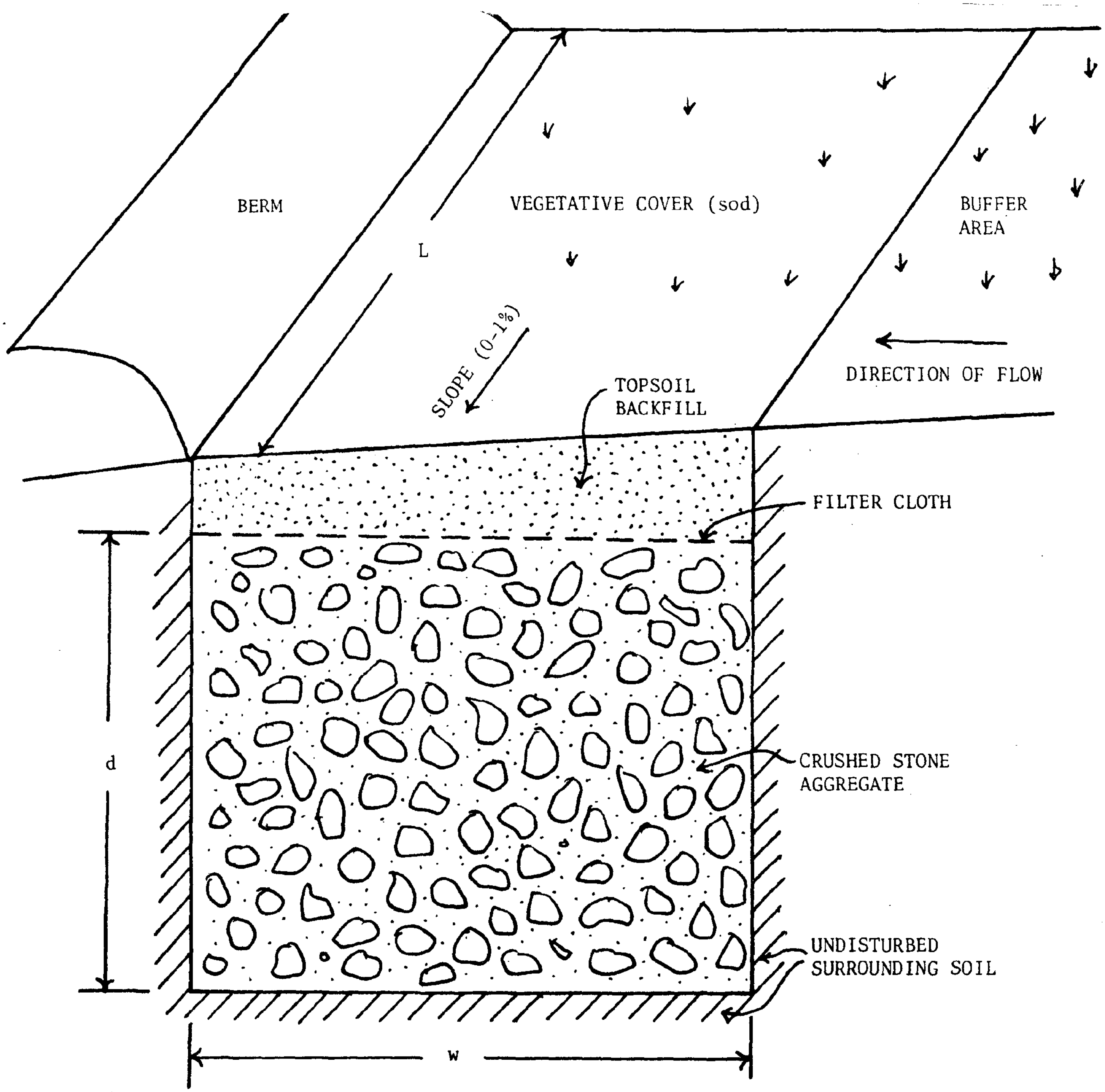

FIGURE 1. Infiltration Trench Configuration for Controlling Storm Runoff

$$V_s = ndwL \tag{1}$$

in which V_s is the volume of available storage in ft^3, d is the depth of gravel fill in ft., w and L are the width and length of the trench in ft., respectively, and n is the porosity of gravel fill in dimensionless units. The dimensions of the trench are shown in Fig. 1.

The gravel backfill material is typically coarse aggregate from 3/8-inch to 1½-inch in size. The percent voids range in value from about 30 to 40 percent at dense compaction and loose compaction, respectively, for a gravel mixture from the Mid-Atlantic coastal plain (Wills, 1967). The void content of coarse aggregates may differ between gravels obtained from various regions throughout the country. Particular specifications of gravels are available upon request when purchasing fill material.

Storage of the direct surface runoff within the infiltration trench is accomplished by sizing the dimensions to accommodate the volume of water infiltrated from the surface flows. By simply equating the volume of storage with the volume of available surface runoff, the dimensions of the trench may be determined. The product of the depth of direct runoff and the upland area are equated with Eq. 1 to determine the ratio of the upland area and the length of the infiltration trench:

$$\frac{A}{L} = \frac{ndw}{Q} \tag{2}$$

in which A is the upland drainage area in sq. ft., and Q is the depth of the direct surface runoff in feet. The required length of the infiltration trench is computed from either Eq. 2 or Fig. 2 for a given upland area. Fig. 2 is a graphical solution of Eq. 2, with curves that represent different trench cross-sectional areas and a void content of 35 percent for the gravel fill. The procedure of computing the volume of direct runoff is often dictated by the regulatory policy for the state of local jurisdiction.

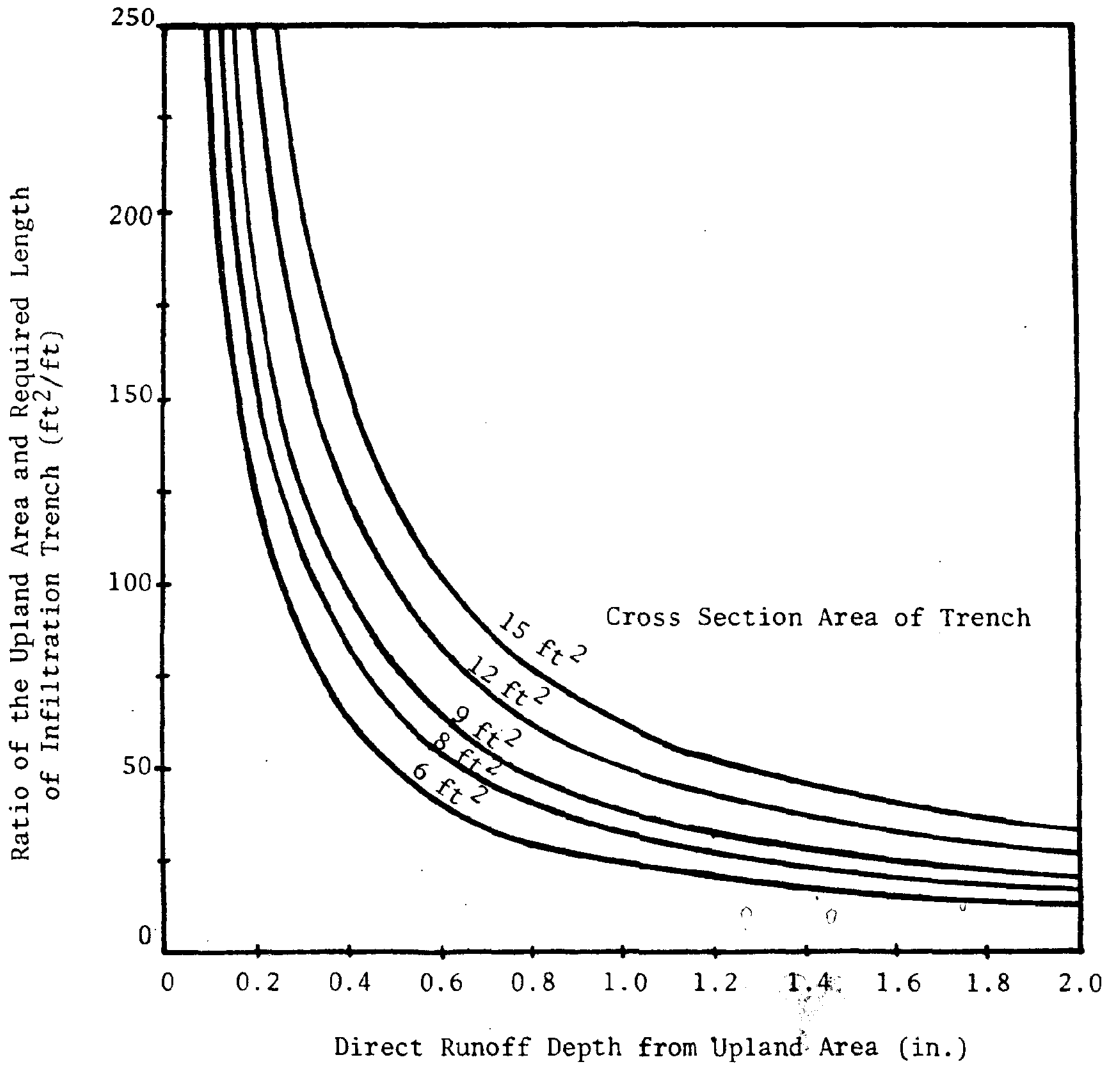

FIGURE 2. Sizing of Infiltration Trenches to Control Direct Runoff from Upland Areas (coarse aggregate fill porosity equal to 0.35)

Infiltration of Surface Flow as a Design Constraint

While the volume of void space within the infiltration trench is a limiting factor, the infiltration rate through the topsoil overburden and filter cloth must also be considered in the design of the infiltration trench dimensions. The stormwater must infiltrate through the topsoil overburden and into the trench prior to percolation into the surrounding soils. The total volume that enters an infiltration trench will depend on the infiltration capacity of the overlying topsoil backfill. The topsoil material should be selected such that the infiltration rate is high so storm runoff is more easily infiltrated into the trench. A sandy textured soil is recommended to fulfill the requirement of a high infiltration rate; a sandy soil is also capable of sustaining vegetative growth. A dense turf cover over the infiltration trench provides a leaf canopy that protects against the sealing of pores by sediments created upon rainfall impact. Also, vegetative root systems penetrate the soil and increase the infiltrating capacities by furnishing small channels between soil pores.

The volume of stormwater entering the infiltration trench may be estimated if the minimum infiltration capacity of the topsoil overburden is known. The volume of infiltrated water, V_I (inches), may be estimated by:

$$V_I = f_c L w t_D \tag{3}$$

in which f_c is the minimum infiltration rate of the topsoil overburden in inches per hour, L is the length of the infiltration trench in ft. (determined from Eq. 2), w is the width of the infiltration trench in ft., and t_D is the duration of the direct runoff in hours. The duration of direct runoff may be approximated as twice the time of concentration of the flow from the upland contributing area. Solutions to Eq. 3 are given graphically in Fig. 3. The design curve chosen will depend upon the type of soil information that is available; curves are provided in Fig. 3 for different soil textures and for the four SCS soil groups.

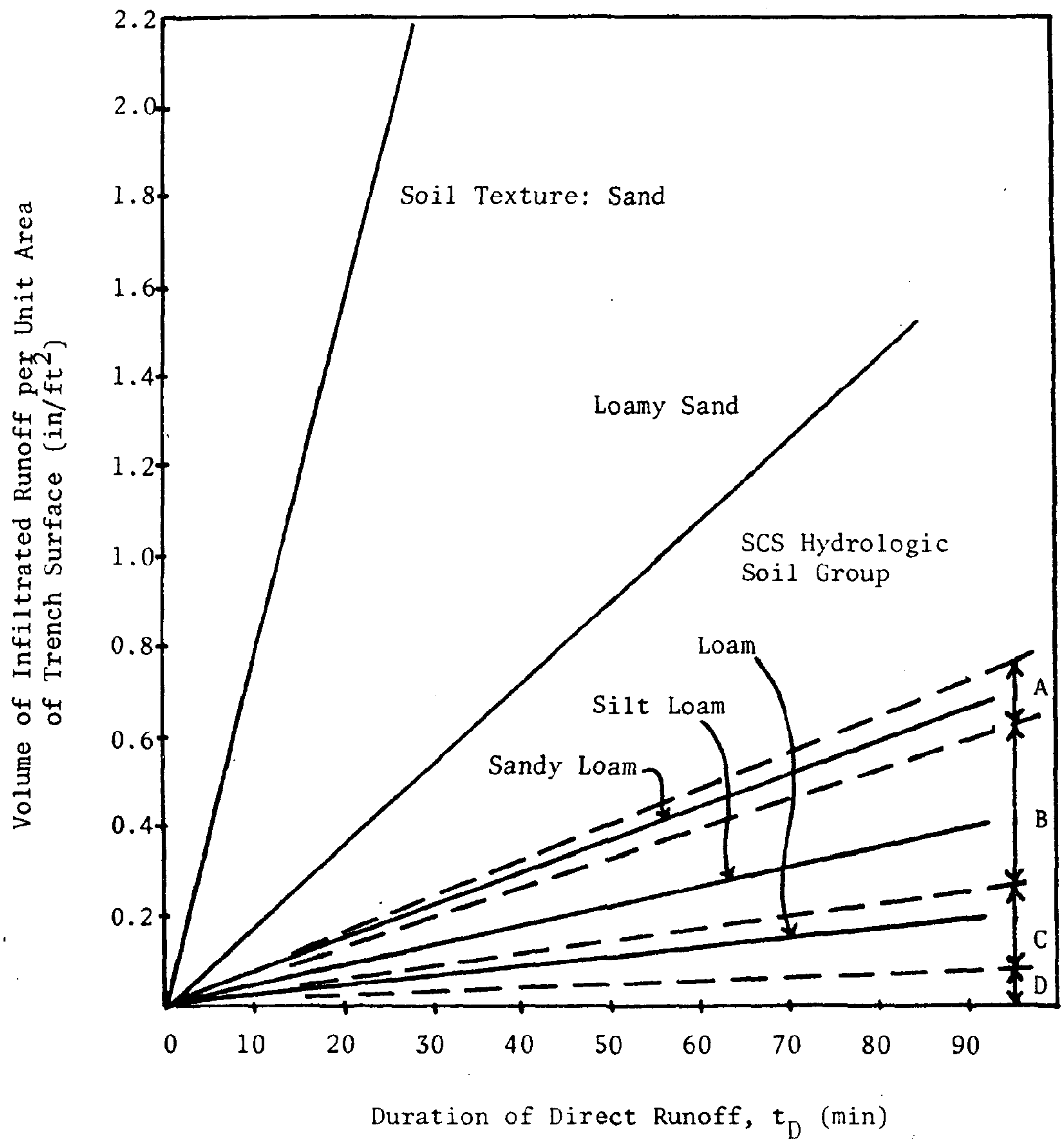

FIGURE 3. Volume of Infiltrated Runoff into Infiltration Trench for Various Soil Textures and SCS Hydrologic Soil Groups

The minimum, or equilibrium, infiltration capacity was selected for Eq. 3; this will result in a conservative design. Estimates of f_c are based upon the final constant rate of infiltration within the upper soil horizon after prolonged wetting of the soil. The value of f_c will depend on the soil type, antecedent soil moisture, and the duration of the rainfall event. Hence, the minimum infiltration rate represents a long-term steady-state condition that neglects the early decay portion of infiltration capacity curve and should correspond to the saturated hydraulic conductivity of the soil. Additionally, the saturation of the topsoil overburden is assumed to have occurred prior to the interception of direct runoff by the infiltration trench.

Strategies for Increasing the Infiltration Potential

The total volume of surface runoff intercepted by an infiltration trench is constrained by the volume of voids within the trench, the infiltration rate of the topsoil overburden, and the surface area above the trench. The total volume of infiltrated runoff also depends on the duration of the direct runoff; the duration of the direct runoff from impervious surfaces is frequently short due to the low resistance associated with the relatively smooth surfaces that are associated with developed areas. Because of all of these potential limiting factors, the fraction of the total surface runoff volume that is intercepted by a trench is often small. Therefore, a means of inducing the infiltration would enhance the efficiency of the trench.

A number of different strategies may be used to effectively increase the volume of runoff entering the infiltration trench. To be effective, a strategy must overcome one of the constraints identified. For example, a vegetative buffer strip that is placed between the runoff generating surface and the site of the infiltration trench would reduce the flow velocity and thus increase the duration of flow across the surface area above the trench. Similarly, grading of the site so that the surface slope of the trench contact area is minimized should increase the contact time, and thus, increase the volume of water infiltrated. Also, a similar effect will be attained by increasing the surface roughness of the trench contact area. An additional advantage of installing a vegetative buffer strip, increasing the surface roughness, or decreasing the slope is the filtration and removal of sediment particles from the surface flow (Wong and McCuen, 1981).

The temporary ponding of water with various structural devices may be employed to prolong the contact time of the runoff. Retaining a portion of the surface runoff on the surface above the trench will reduce the volume of runoff to downstream areas, while allowing for increased infiltration of water into the trench. A berm may be placed along the edge of the trench when the overland flow is in the direction across the width of the trench, as shown in Fig. 1. Berms should be constructed with soils that are not easily eroded and seeded to prevent washing back into the trench. The required height of the berm will vary with slope. The volume of storage retained by the berm (V_b) will depend on the height of the berm, the surface slope, and the surface area behind the berm. The volume of infiltration given by Eq. 3 can be modified to include the volume retained by the berm:

$$V_I = f_c L w t_D + V_b \qquad (4)$$

Check dams may be installed to detain the water when an infiltration trench is placed beneath a swale and the flow is directed along the length of the trench. Check dams may consist of either graded stones, sheet pilings, or staked straw bales. Both the berm and the check dam serve to reduce the flow velocity, lengthen the detention time, and thus, increase the volume of infiltrated water.

Siting of an Infiltration Trench

In addition to the factors previously identified, the effectiveness of an infiltration trench will depend on siting factors. The selection of a satisfactory location to implement an infiltration trench as a control measure will necessitate a site investigation; thus, information obtained pertaining to the inherent soil characteristics may reveal the suitability of the environment to accommodate groundwater storage and recharge. Hydrologic, topographic, and geologic investigations will provide the basis for a decision.

Much of the necessary information may be obtained from existing sources of data. County soil surveys that identify the soils of the particular county are available from the Soil Conservation Service. The permeability rate is of particular interest because it reflects the rate at which the water moves in the soil. The permeability rate may be measured on-site by various methods (e.g., percolation test and well pumping) and should be used whenever possible. The individual soil horizons between the ground surface and the aquifer should not have any impermeable layers that restrict the downward movement of water.

The depth between the water level in the saturated aquifer and the soil surface should be determined to ensure that the trench will not become rapidly inundated due to a rise in the water table elevation. Areas with water table elevations that are seasonally high may require consideration of additional design requirements. Coastal areas, for example, are frequently faced with groundwater tables that rise rapidly, which limit the ability of the soil to adequately drain the stormwater runoff. Circumstances such as this may cause infiltration trenches to become ineffective in both the storage of stormwater and the recharge of groundwater. Furthermore, the inundation of infiltration trenches by a rapidly rising water table may lead to the intrusion of soil particles into the trench from the surrounding soil, which clog available void space. Placement of filter cloths along the bottom and sidewalls may be necessary to avert soil particle intrusion. However, high costs may prohibit such a requisite. Water table elevations should be several feet below the trench bottom to allow for adequate clearance as water levels rise. The actual distance will depend on soil characteristics.

Considerations in Selecting the Filter Cloth

The implementation of filter cloths as a separating layer between the topsoil backfill and coarse aggregate fill should not be slighted. These fabrics are critical for the continued effective functioning of the infiltration trench. If the installation of a filter cloth is neglected, the topsoil overburden has no means of being held in place. Hence, the void space in coarse aggregate fill would quickly clog, with the particles rendering the trench useless. Filter cloths are available with various specifications for different criteria. The selection of filter cloths should be based upon: 1) permeability (i.e., size of pore openings), 2) strength of fabric, and 3) percent of open area. Filter cloths are being used more frequently to stabilize drain systems and care should be taken to ensure that it is not the limiting permeable material.

Recognizing that filter cloth restricts the movement of the topsoil material into the coarse aggregate fill, particles will tend to clog the pore openings within the filter cloth and reduce the permeability rate. This causes soil particles to form a mud-caked layer on top of the filter cloth, which requires periodic maintenance. Depending upon the depth of topsoil backfill, the accessibility of the filter cloth will vary. Maintenance schedules should be considered in design when considerations are made for the implementation of infiltration trenches. Furthermore, the removal of the aggregate fill to be washed or replaced with new aggregate material may be required; however, proper maintenance of the filter cloth should make this an infrequent maintenance requirement.

DISCUSSION AND CONCLUSIONS

While surface detention storage appears to be an effective measure for controlling the increased runoff rates and volumes that accompany development there is some concern about the need for groundwater recharge and the value of land used for detention storage. Infiltration trenches provide a means of simultaneously decreasing surface runoff volumes and increasing groundwater recharge rates. The effectiveness of infiltration trenches was shown to be a function of the volume of storage within the trench, the infiltration capacity of the topsoil overburden, the surface detention time, the proper selection of filter cloth, and various siting characteristics. In addition to a method for determining the appropriate dimensions of a trench, various strategies were identified herein for increasing the effectiveness of the trench.

Integration of Stormwater Management Methods

In most applications, infiltration trenches will not be capable by themselves of controlling increases in runoff volumes that accompany urbanization and land cover conversions in agricultural areas. Therefore, they will probably be most effective when combined with other stormwater control methods, such as detention ponds, parking lot and rooftop storage, and vegetative buffer strips. The design procedure outlined herein can be used to incorporate infiltration trenches into an integrated stormwater control program.

The use of methods such as infiltration trenches and buffer strips can reduce the total storage volume required for detention basins. The usual design procedure involves estimating the required volume of detention storage. However, with an integrated design program, design curves such as those provided herein for infiltration trenches could be used to reduce the volume of required storage. The reduced volume of detention storage that would be required should increase the flexibility of the stormwater management plan; that is, a smaller required storage volume may make it possible to locate the detention basin at a site having characteristics less desirable for either urban or agricultural land uses. This could increase the return on the development of the land.

Policy Considerations

Stormwater management policies and regulations play a critical role in the effectiveness of control methods, including infiltration trenches. Policies and regulations must provide control over the design, installation, and maintenance of the trenches. Failure at any of these aspects may lead to an ineffective infiltration trench. In order for a policy to be effective it must provide for assurrances that the design will properly coordinate the storage volume with both soil and site characteristics, as well as the characteristics of the filter cloth. Regulations must be sufficiently specific to limit the installation of infiltration trench to a period during which upland areas are not exposed; otherwise, eroded soil may clog the storage space within the infiltration trench. For maximum effectiveness, the infiltration trench should be installed after most of the land development has been completed. It should also be recognized that a poorly maintained infiltration trench will not function as the designer intended. Therefore, policies must provide control of maintenance, and regulations must provide for inspection and enforcement of the policies. Only when properly designed, installed, and maintained will an infiltration trench contribute to the control of stormwater runoff.

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APPENDIX L

THE EFFECT OF LAND USE CHANGE AND STORMWATER MANAGEMENT ON FLOW-DURATION CURVES

Stanley L. Wong
Richard H. McCuen
Mark E. Hawley

THE EFFECT OF LAND USE CHANGE AND STORMWATER MANAGEMENT ON FLOW-DURATION CURVES

by

Stanley L. Wong, Richard H. McCuen, and Mark E. Hawley[1]

INTRODUCTION

Hydrologists have recognized the effect of land use changes on peak discharge rates for many years, but the effects on other runoff characteristics have received much less attention. While the effect on peak discharge is important, urbanization can also cause significant changes in the velocity of runoff, the duration of near-peak flows, and the sediment transport capabilities of the runoff. Changes in these characteristics usually lead to changes in stream morphology due to increased erosion of the stream banks and deposition of eroded material at points downstream.

While studies have shown the effects of urbanization and stormwater management on the magnitude and duration of flooding independently, the interaction of the two watershed modifications has not been examined. The interaction between the magnitude and duration of flood flows can be represented by a flow-duration curve. Flow-duration curves for urbanized and non-urbanized watersheds were computed for Long Island, New York (USGS, 1981). The effects of urbanization on the runoff characteristics of a watershed can be evaluated by developing flow-duration curves for various design storm events. These curves are graphs of the duration of the period during which the discharge rate equals or exceeds a certain magnitude versus the magnitude of flow. Flow-duration curves are an alternative to flood frequency curves; however, they are preferable to the flood frequency curve because they provide information about both the duration and frequency of runoff.

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The objective of this study is to examine the effects of urbanization and stormwater management on flow-duration curves and to discuss the method of incorporating flow-duration curves into the stormwater management design process. Alterations in the time characteristics of direct runoff may be readily assessed by the analysis of flow-duration curves for various watershed conditions. Comparison of flow-duration curves before development and after development may serve to illustrate the changes in the hydrologic response of the watershed. Flow-duration curves may also be employed to evaluate the effectiveness of SWM measures in controlling the increased runoff volume and peak discharge rates. The extent of channel degradation and changes in stream morphology due to increased duration of bankfull flows is also investigated where detention basins and other SWM controls are to be constructed.

FLOW-DURATION CURVES AND STORMWATER MANAGEMENT DECISIONMAKING

Flow-duration curves are not commonly used in evaluating the effects of urbanization due to deficiencies in the existing policies for stormwater management (SWM). Most current SWM policies require only control of the peak discharge rate for a selected recurrence interval, such as the 10-year event. These policies resulted from the belief that most problems are the result of peak flow rate and that the peak discharge rate was related to the other runoff characteristics; from these beliefs it then followed that, if the peak discharge were controlled, then the problems associated with the other stormwater runoff characteristics would also be solved. Therefore, most drainage and stormwater management policies developed in the last ten years have required that the peak discharge rates be controlled so that the post development peak is no greater than the pre-development peak for a specified exceedence probability, or return period. Peak discharge rates are commonly estimated using the rational or SCS design methods, and detention basins are designed to control the discharge rate.

Although detention basins are quite effective in controlling peak drainage, they also affect the duration characteristics of the runoff. Detention basins provide artificial storage of runoff to compensate for the loss of natural storage and increased volume and duration of runoff caused by urbanization. Previous studies (Kamedulski and McCuen, 1979) have demonstrated that a basin designed to control the peak discharge rate from a certain design storm may not adequately control the runoff from other storms. Other studies have shown that current SWM policies that are concerned only with controlling peak discharge can result in significant increases in the duration of peak and near-peak flows (McCuen, 1979). Field inspections have demonstrated that these policies are not effective in controlling sediment problems and can, therefore, lead to significant changes in stream morphology.

Concern over the detrimental effects of urbanization on infiltration and stream characteristics has led to development of alternative methods of SWM. Porous and modular pavements, infiltration trenches, and vegetative buffer strips are examples of alternative measures that encourage the infiltration of stormwater on site. These methods provide a more natural type of control because they cause some of the stormwater volume to enter the soil, rather than flowing directly into the stream. Thus, the increase in volume of runoff due to urbanization is lessened.

On some watersheds, soil types and subsurface conditions, such as impermeable layers or high water tables, may limit the potential rate of infiltration, while on intensively developed watersheds the value of the land may be so great as to make those methods economically unsuitable. In those cases, alternative means of providing temporary storage may be useful; commonly used methods of providing this storage are rooftop storage, detention basins, and parking lot ponding structures. The volume of storage provided by each of these methods serves to regulate the rate of discharge, but these methods do not lessen the total volume of runoff to any significant extent. Thus, they do not give adequate consideration to the duration characteristics of the runoff.

Although these structural methods of controlling discharge rates are useful in meeting the requirements of SWM policies, they often fail to control the adverse downstream effects of urbanization. SWM policies that are concerned only with peak flow rates are deficient because they ignore the effects of urbanization on the time characteristics of both direct runoff and discharge from SWM structures (Hawley, et al., 1981). The erosive capacity of streams can increase tremendously due to changes in the timing characteristics of runoff, leading to rapid changes in downstream morphology.

Stormwater management decisionmaking should not be limited to the control of a peak discharge rate for a single exceedence probability. While the control of two or more points of the before development flood frequency curve is certainly an improvement, stormwater management decisionmaking will be most effective when the policy requires control of the flow duration curve for two or more exceedence probabilities. Such a policy is not unreasonable given the current state-of-the-art of stormwater management design. Recent advances have provided the design basis for integrated stormwater management design (Wong and McCuen, 1982) and two stage riser design (Soil Conservation Service, 1982). The integrated design approach permits the use of control methods other than the detention basin; the integrated approach will also decrease the required storage volume of a detention basin. The following analysis will demonstrate how the integrated design approach and the flow duration curve can be used to properly control the effects of urbanization.

STUDY STIE DESCRIPTION

The study site if a 47-acre subwatershed of Crabbs Branch, a tributary of Upper Rock Creek in Montgomery County, MD. The subwatershed includes part of the County Service Park, a complex of County government warehouses and maintenance depots. Land uses on the study site include portions of a railroad, highway, and a Department of Liquor Control warehouse with associated offices and parking lots. A Department of Transportation materials and equipment storage and maintenance depot and a main access road were under development during the study. The percentage of cleared area ranged from 34 to 44 percent. Slopes of less than 5 percent existed before total grading in preparation for the development of the largely impervious complex of large, flat-roofed buildings and parking lots. The predominant native soils are Chester silt loams, which are characterized as deep, well-drained, and moderately erodible. These soils are typical of the upland soils in Montgomery County.

At the foot of the slope, adjacent to the valley stream channel, is a broad rectangular sediment basin, measuring 400 ft x 140 ft and having a permanent pool surface area of 1.29 acres. The storage volume and riser characteristics were designed using a policy based on a 10-yr.return period. The intent of the SWM basin is, therefore, to limit the peak runoff rate from a 10-yr storm to that which would have occurred prior to development. During the development period, the primary basin outlet was a nonperforated corrugated metal pipe (CMP) riser and barrel. The 6-ft high, 48-in diameter riser was topped with a closed-lit, circumferential CMP antivortex box, or hood. The riser also had a shielded 4-inch port, which maintained a normal pool level 18-in below the crest of the riser. A 32-ft wide grassed overflow channel was provided as an emergency spillway. The basin was altered and retained for permanent use as a SWM basin following completion of the upslope development site.

FLOW-DURATION CURVES AS INDICATORS OF THE EFFECTS OF URBANIZATION

The effects of urban development on the magnitude and duration of flooding are reflected in the hydrograph ordinates at downstream points. The downstream direct runofff hydrographs may be synthesized for various watershed states, such as before development, after development, and after development with SWM control measures. To examine the effects of each condition, it is necessary to estimate the runoff hydrographs using a hydrologic design model. The SCS TR-20 hydrologic model (SCS, 1965) is used to account for the changes in land use and the storage provided by SWM. The hydrologic model is based on Soil Conservation Service techniques (SCS, 1972). Flow-druation curves may be completed for the hydrgraph discharge rates of each watershed condition. A flow-duration curve is constructed by plotting the magnitude of flow versus the duration of the magnitude.

The data from Crabbs Branch subwatershed were used in evaluating the changes in the storage and timing characteristics brought about by urbanization. The data were used as input parameters for the TR-20 computer program to examine the hydrologic response for selected return periods using the SCS Type II rainfall distribution. The direct runoff hydrographs were computed at the downstream outlet for the 2-, 10-, and 100-yr storm events for various watershed conditions. The watershed conditions include: (1) before development, (2) after development without SWM, (3) after development with a detention basin, (4) after development with rooftop ponding and infiltration trenches, and (5) after development with a detention basin, rooftop detention, and infiltration trenches; the latter watershed condition will be referred as the integrated SWM control system.

The computed hydrographs for each storm event were used in developing the flow-duration curves at the downstream outlet for all five watershed conditions, shown in Figs. 1, 2 and 3. The impact of urbanization for all storm events are readily apparent by the increased magnitude of the peak discharge rates, when comparing the after development conditions with that of the before development conditions. It is evident from the figures that urbanization causes a more substantial increase for the 2-yr peak rate (286 percent) than for both the 10-yr (126 percent) or the 100-yr (50 percent) storms. Additionally, the effect of urbanization on the runoff volume is also evident from Figs. 1, 2, and 3; the increases in the runoff volume are represented by the portion of the curve above the predevelopment flow-duration curve.

Thus, the impact of future development on the magnitude of peak discharge rates for all storm frequencies is recognized. Clearly, urbanization will affect

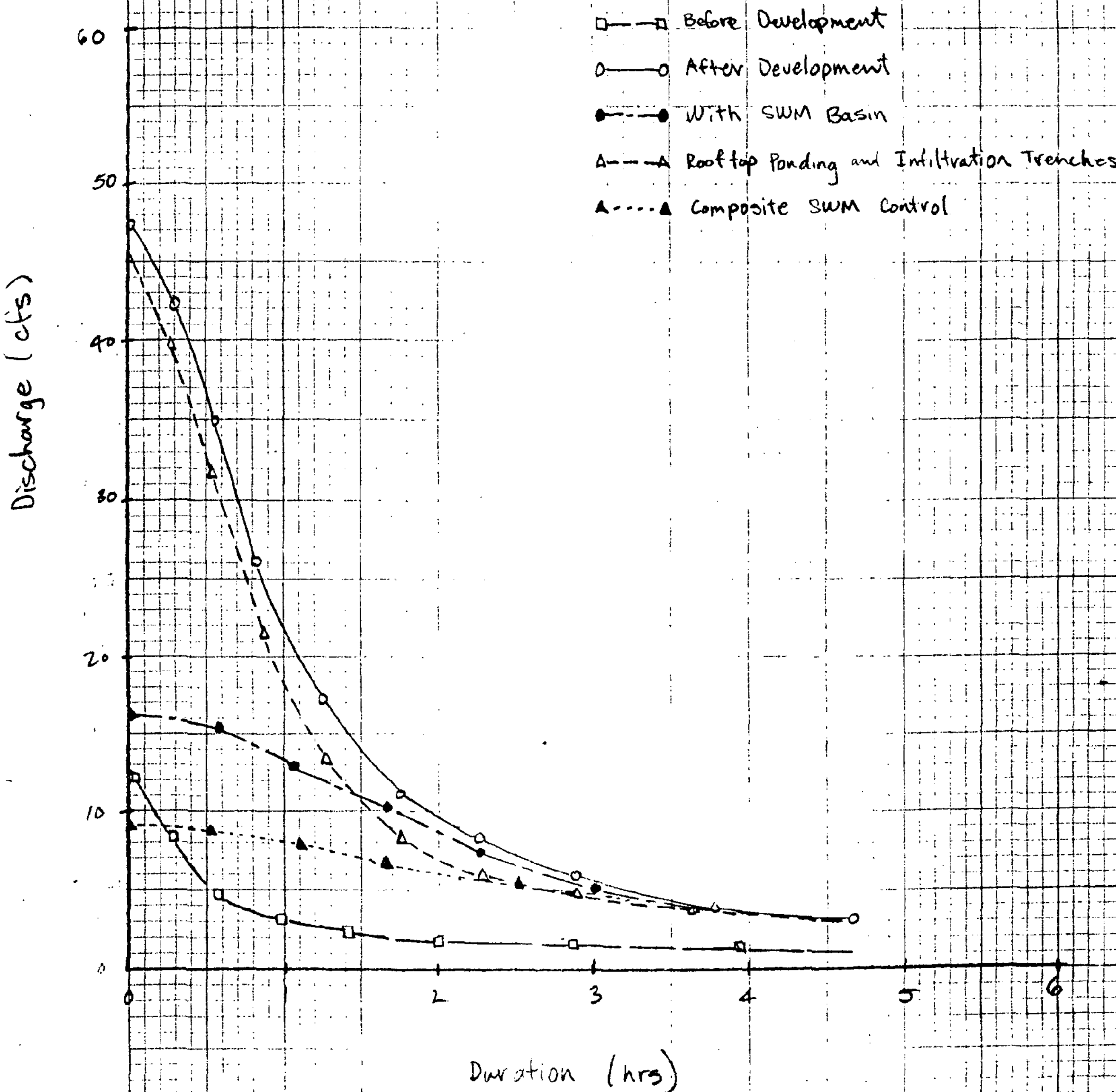

FIG. 1. - Effects of Urbanization and SWM Measures on Flow-Duration Curves (2-yr Storm)

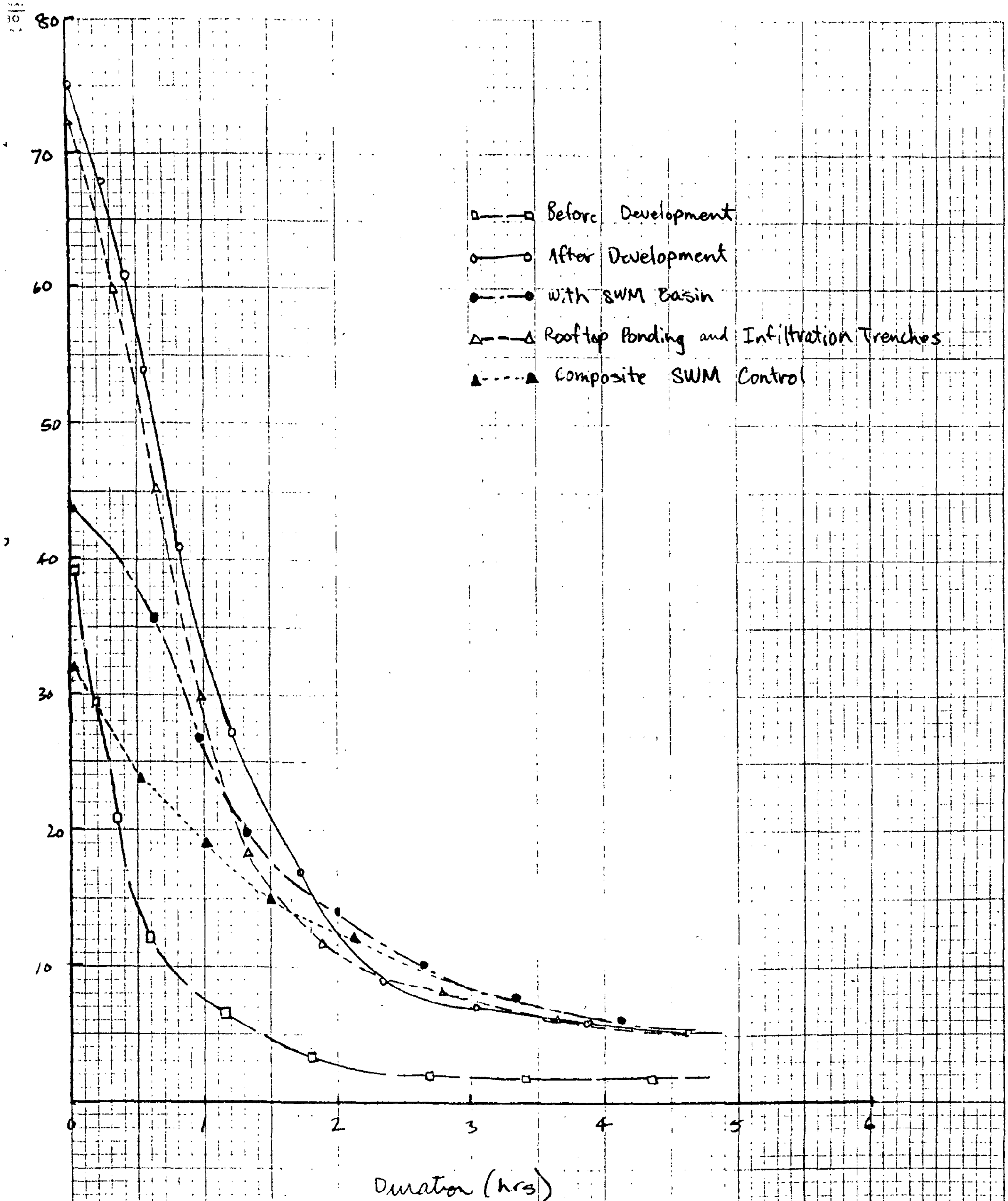

FIG. 2. - Effects of Urbanization and SWM Measures on Flow - Duration Curves (10-yr Storm)

Squares to the Inch

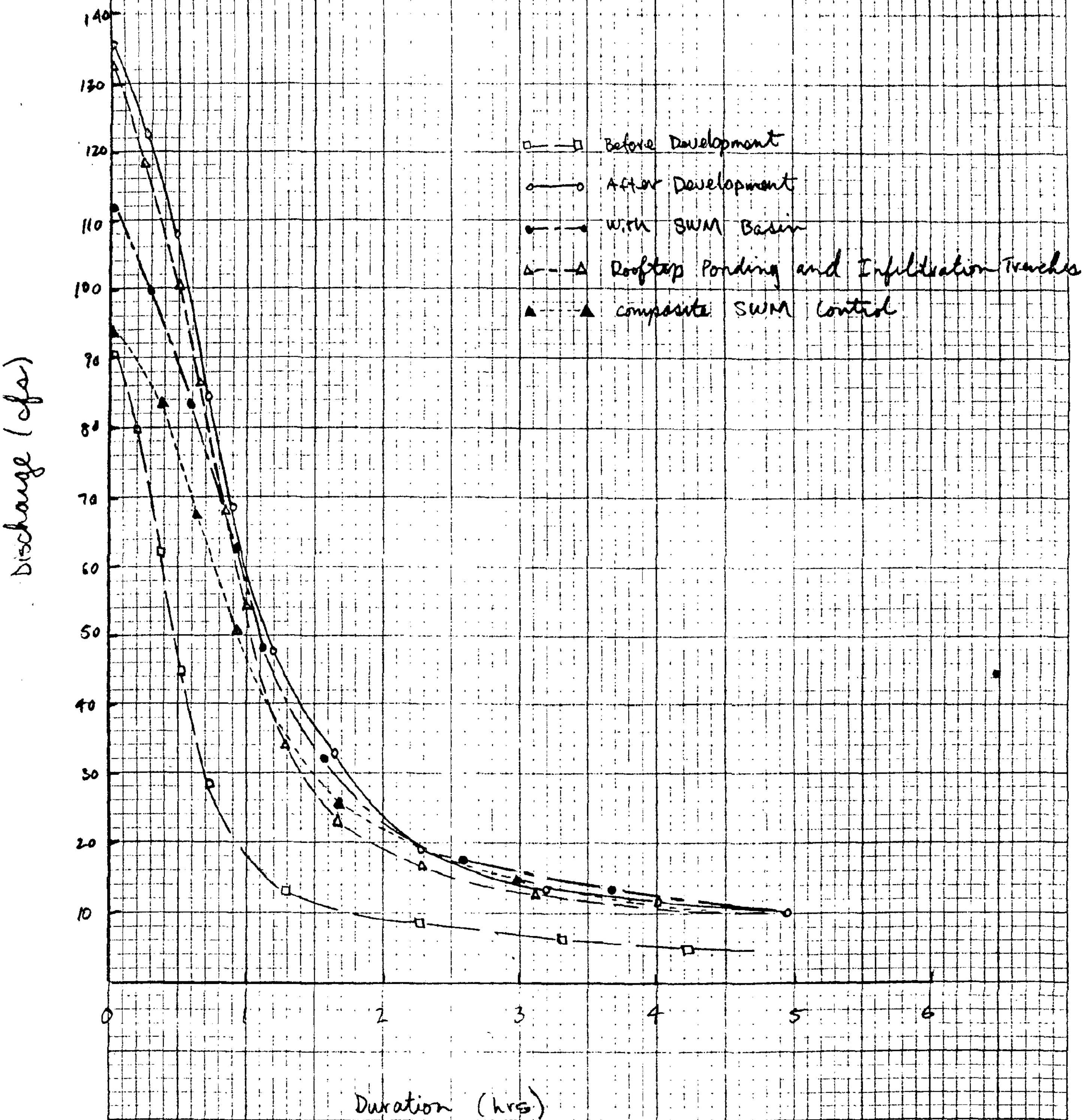

FIG. 3. Effects of Urbanization and SWM Measures on Flow-Duration Curves. (100-yr Storm)

10 Squares to the Inch

both the magnitude and frequency of flooding as well as causing a change in the timing characteristics of the runoff hydrograph. The development of flow-duration curves for urbainizing areas serves to illustrate the changes in the original durations of flow. Hence, remedial SWM control methods may be further evaluated on additional criteria, other than that of controlling peak rates, with the analysis of flow-duration curves.

The flow-duration curves of the study watershed are simulated for the implementation of SWM measures. Fig. 1 reveals that when the single SWM detention basin is used, a reduction is acheived in the peak discharge rate after development to levels that existed prior to development; however, the duration of the peak rate has been extended from an instantaneous occurrence to a flow of 1.25 hours. Similarly, the 10-yr and 100-yr predevelopment peak flow rates have increased in duration by 0.45 hours and 0.50 hours, respectively. In addition to the peak rates, the flow rates throughout the runoff hydrograph also increase in duration. Thus, the manner in which SWM basins manage urban flows is to detain the runoff volumes to be released at a regulated rate with the intent of limiting the peak, while increasing the duration of the residual flows.

In comparison with detention basins, the combination of rooftop ponding and infiltration trenches have a lesser effect in reducing both the magnitude and the duration of the after development flows, especially nearer to the peak rate. This may be attributed to the greater available storage within the detention basin. Figs. 1, 2, and 3 show that rooftop ponding and infiltration trenches are more beneficial in reducing the flow-duration of runoff rates that are considerably lower than the peak rate; although, the reductions do not differ significantly from the uncontrolled, developed state of the study area.

The integrated SWM control system presents the most efficient means of controlling the after development peak to the predevelopment level. The reductions in runoff volume due to rooftop detention and infiltration trenches during the initial period of the storm event provide an additional amount of storage within the detention basin; subsequently a greater reduction in the peak runoff rate results. While the integration SWM control system provides further reductions in both the magnitude of discharge rates and flow-durations, the increased volume of direct runoff due to urbanization will continue to cause the effects of prolonging the flow-durations for the predevelopment watershed discharge rates.

FLOW DURATION CURVES AND DOWNSTREAM BEDLOAD TRANSPORT

Urbanization causes increases in the volume of direct runoff. While SWM controls are effective in limiting peak flow rates, their effect on volumes of flow are minimal. The flow-duration curves show that the SWM controls do little to limit the volume of flow. In streams with the potential for bedload transport, the significant increases in the duration of bankfull flows can cause additional bedload movement. Consequently, changes in the stream morphology of unstable, undeveloped streams are likely to occur more rapidly. Deposited sediments will be scoured and washed from stream bottoms to be transported to larger water courses downstream.

The transport of sediment occurs as a result of scour from banks. The actual bedload discharge is difficult to determine due to the complex nature of the many factors involved. Accurate field measurements are difficult to obtain, because most existing equipment tends to disturb the natural movement of sediment which leads to discrepancies in data. Thus, many bedload formulas are based on the statistical theory of random bedload movement or on the analysis of the equilibrium of forces acting on a sediment particle.

The predicition of bedload movement has been empirically developed by Schoklitsch (Shulits and Hill, 1976). This method is used herein as a means of assessing the impact of urbanization and SWM controls on the stream morphology. The bedload formula gives the peak sediment discharge per unit width of channel, G_1, in lbs/sec/ft, as a function of the grain diameter, D, in ft, the slope, S, the critical discharge, Q_{01}, and the stream discharge, Q_1, in cfs/ft:

$$G_1 = 25 \frac{S^{3/2}}{D^{1/2}} (Q_1 - Q_{01}) \tag{1}$$

The critical discharge is computed for a sediment specific gravity of 2.65 as:

$$Q_{01} = \frac{0.0638 \cdot D}{S^{4/3}} \tag{2}$$

While the quantities of sediment computed with Eqs. 1 and 2 may not agree closely with actual field results, Schoklitsch's equations are useful in evaluating the various watershed conditions that reflect urbanization and SWM.

The bedload discharge rates were computed for the 2-, 10- and 100-yr storm events and each of the watershed conditions. The weighted average value of 0.0015 ft was computed as the effective grain particle size using the particle size diameters given by McCuen (1979). The mean slope at the downstream outlet is 0.006 ft/ft. Thus, the effects of the watershed conditions on bedload movement and total sediment volume may be computed; the results are shown in Table 1. The peak sediment discharge rates after development increase by factors of 3.9, 1.5, and 0.4 for the 2-, 10-, and 100-yr storm events, respectively. However, the total volume of sediment movement after development increases by factors of 6.5, 3.1, and 1.5 for the 2-, 10-, and 100-yr storms, respectively, during the major portion of the event. This illustrates that, while the peak sediment discharge rate will increase as a function of the peak flow rate, the total volume of sediment movement will increase by a greater percentage than the peak sediment discharge when the stream flow rates increase in duration and magnitude.

Schoklitsch's bedload equations are particularly useful in comparing the effects of various SWM methods on stream morphology. The concern of sediment transport in natural streams is receiving more attention as the imapct of SWM methods on stream erosion becomes more evident. The results of Table 1 reveal that detention basins create changes in the pattern of the stream stability, which is reflected by increases in both the sediment peak discharge rates and total volumes of bedload movement. The installation of a detention basin does, however, reduce the impact of the development flow rates on stream morphology. Additionally, rooftop ponding and infiltration trenches are somewhat effective, although a significant reduction in stream erosion is not expected when examining the flow-duration curves.

The integrated SWM control system limits the peak sediment discharge rate as well as the peak runoff rate to predevelopment levels. The parallel between the sediment and runoff peaks is related directly by the bedload formula. Similarly, the flow-duration may be correlated with the total volume of sediment movement. That is, as the effects of flow-duration increases the stream discharge, the total volume of eroded sediment increases. Thus,

TABLE 1 Effect of Watershed Conditions on Bedload Movement and Total Sediment Volume

Watershed Condition	Return Period, 2-yr			Return Period, 10-yr			Return Period 100-yr		
	G_1	V_T	F_I	G_1	V_T	F_I	G_1	V_T	F_I
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Before Development	2.8	1.9	---	8.6	5.4	---	26.3	17.7	----
After Development without SWM	13.8	14.1	6.5	21.8	21.9	3.1	36.8	44.0	1.5
with Detention Basin	4.8	9.3	4.0	12.7	18.0	2.3	32.7	41.7	1.4
with Rooftop Ponding and Infiltration Trenches	10.3	13.3	6.1	20.6	22.6	3.2	38.4	41.8	1.4
with Detention Basin, Rooftop Ponding, and Infiltration Trenches	2.5	6.3	2.4	8.0	14.1	1.6	27.1	37.3	1.1

Note: G_1 = peak bedload, in lbs per second; V_T = total sediment volume for storm, in cubic yards; and F_I = factor of increase in V_T over before development volume.

the total volume of bedload increases by factors of 2, 4, 1.6, and 1,1 during the 2-, 10-, and 100-yr storm events, respectively, for the integrated SWM system. The percentage of increase is greatest for the 2-yr event and becomes less significant as the probability of exceedence decreases, due to the greater relative consequence of the flow-duration on the smaller storm of greater frequency.

CONCLUSIONS

The effectiveness of SWM methods in controlling the impact of development is often assessed by the degree in which the peak flow rate is controlled. The goal is often to limit the downstream peak discharge to that of a pre-development rate in order to lessen the adverse impact of urbanization. However, when the extent of the landcover change is significant, the increases in the volume of runoff over the duration of the storm is more difficult to manage. The development and analysis of flow-duration curves illustrates that various SWM methods handle the increased volume of direct runoff in different ways, with detention basins having the single most significant effect. The integration of infiltration trenches and rooftop ponding controls provide an additional degree of control; the integrated SWM control system will limit the peak flow rate more effectively. However, the increased volume of runoff after development prolongs the duration of the residual flows, which is evident by the flow-duration curves.

The analysis of the flow-duration curves also addresses the importance of the timing characteristics on the stream morphology. The prolonged bankfull flow of streams is observed to cause the total volume of bedload movement to increase by more than two fold for the 2-, 10-, and 100-yr storm events. Therefore, the impact of stream stability is an important consideration in evaluating the effects of development and SWM methods. The changes in the upstream watershed characteristics can significantly alter the flow-duration curves and change the streamflow characteristics.

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APPENDIX M

SENSITIVITY OF SCS MODELS TO CURVE NUMBER VARIATION

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SENSITIVITY OF SCS MODELS TO CURVE NUMBER VARIATION

by

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INTRODUCTION

The Soil Conservation Service (SCS) models, including the TR-20 computer program (SCS, 1969) and the simplified methods in TR-55 (SCS, 1975), are widely used in hydrologic design. Applications include small urban watershed analyses and stormwater detention basin design, as well as designs that require either a peak discharge estimate or an entire flood hydrograph. In fact, numerous government agencies recommend that the SCS models be used for hydrologic design. Examples include the states of Maryland and Michigan.

The SCS methods are replacing methods such as the rational formula, which has been and still is one of the most widely used methods. Application of SCS methods is increasing rapidly for numerous reasons. First, the inherent deficiencies of methods such as the rational formula are becoming more important as the cost of inaccurate designs increases. Second, recent studies (Ragan and Jackson, 1979; Bondelid, et al., 1980) have shown that land use or vegetation data for the SCS methods can be obtained using remotely sensed data, which can greatly reduce the cost of data collection. Third, the SCS methods provide a

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a high degree of flexibility in the type of hydrologic analyses that can be performed using the same type of input data, varying from peak discharge estimates for small urban inlet areas to analyses on large watersheds that involve structures. Existing methods such as the rational formula are valid only on small watersheds where a peak discharge is required; for other types of problems it is necessary to collect other types of input data.

One of the reasons for the popularity of the SCS models is that the primary input parameter, the runoff curve number (CN), is defined in terms of land use treatments, hydrologic condition, antecedent soil moisture, and soil type. The models can, therefore, be readily applied to ungaged watersheds. The accuracy of models that use the CN for estimating design peak discharge and runoff volume are especially sensitive to the estimated CN values.

The objective of this study is to evaluate the sensitivity of the SCS models to errors in CN estimates. Specifically, the objective is to quantify in as straightforward a manner as possible the changes in runoff volume and peak discharge computations as a function of changes in CN estimates.

OVERVIEW OF SCS MODELS

The SCS hydrologic models were developed from field data on a moderate number of small watersheds (Rallison, 1980). Several versions of the SCS models are described in the National Engineering Handbook, Section 4: Hydrology (NEH-4) (Soil Conservation Service, 1972). These versions range from simple to quite complex and have been adapted for urban areas (SCS, 1975). All versions use the CN to estimate runoff volume, with the basic expression:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \quad \text{for } P>0.2S \tag{1}$$

in which Q is the runoff volume, P is the precipitation, and S is a retention parameter. All of these variables are expressed in inches. In practice, the retention parameter, S, is related to the runoff CN through the following transformation:

$$S = (1000/CN) - 10 \qquad (2)$$

The SCS curve numbers are based on the assumption that the initial abstraction equals 0.2S.

The runoff curve number, which integrates land cover, soil type, and antecedent soil moisture conditions, is an important input parameter to the SCS procedures. The determination of curve numbers is a time consuming procedure requiring detailed data on land cover and soils. While there are numerous methods for obtaining the necessary input data, Rawls, et al., (1980) showed that estimates of runoff curve numbers are not very sensitive to the method of integrating soils and land cover data; however, the land use classification system significantly affects the accuracy of runoff curve numbers.

Two of the most commonly used SCS models are TR-20 (SCS, 1972) and TR-55 (SCS, 1975). TR-20 is a computerized single event model that incorporates the surface runoff hydrograph development and includes streamflow and reservoir routing procedures.

TR-55 is a distillation of the results of a large number of TR-20 runs. The results are expressed in TR-55 as a series of tables and graphs. TR-55 utilizes the standard SCS 24-hour type II rainfall distribution. This design storm is applicable for use in all of the continental United States except for locations on the Western coast. It has been shown that the TR-55 methods provide peak discharge estimates that are not significantly different from estimates obtained from TR-20 as long as the initial abstraction is not greater than 25% of the total rainfall (Bondelid, 1980; McCuen, 1981). The advantage of using TR-55 in the analyses of peak discharge estimates instead of using TR-20 is that the analyses can be performed analytically. The results from analyses with TR-55 methods are valid to TR-20 as long as the conditions under which TR-55 applies are not violated.

TR-55 (SCS, 1975) includes several methods for peak discharge and runoff hydrograph estimation. The graphical methods provides a relationship between the unit peak discharge and the time of concentration on a log-log plot. The unit peak discharge (q_u) is expressed in cubic feet per second per sq. mi. (CSM) per inch of runoff (Q); the time of concentration is in hours. Because the graphical relationship does not plot as a straight line on log-log paper the curve can be expressed as a piecewise linear curve.

$$q_u = \begin{cases} 476.51\ t_c^{-0.32192} & \text{for } t_c < 0.2 \text{ hours} \quad (3a) \\ 379.14\ t_c^{-0.46394} & \text{for } 0.2 \leq t_c < 0.4 \text{ hours} \quad (3b) \\ 341.90\ t_c^{-0.57678} & \text{for } 0.4 \leq t_c < 0.7 \text{ hours} \quad (3c) \\ 321.48\ t_c^{-0.74946} & \text{for } 0.7 \leq t_c \leq 5.0 \text{ hours} \quad (3d) \end{cases}$$

The peak discharge (q_p) in cfs equals the product of the unit peak discharge, the drainage area (A) in square miles, and the runoff volume (Q) in inches (i.e., $q_p = q_u AQ$). Eqs. 1, 2, and 3 will be used for determining the magnitude of errors in Q and q_p as functions of variation in estimated CN's.

SENSITIVITY ANALYSIS OF THE TR-55 GRAPHICAL METHOD

Hawkins (1975) used equations 1 and 2 to perform a sensitivity analysis of runoff volume to error in precipitation and the curve number; a figure showing the errors in runoff resulting from 10 percent errors in either CN or P was provided, and a figure that showed the domain of greatest absolute error and error contours in calculating Q assuming a 10 percent in either P or CN. Hawkins concluded: (1) for a considerable range of P, accurate values of CN are more important than accurate estimates of P; (2) errors in estimating Q are especially dangerous near the threshold of runoff. Hawkins did not examine the sensitivity of peak discharges to errors in P and CN.

Sensitivity Definitions

The variation in Q and q_p as functions of variation in estimated CN's can be expressed in terms of the sensitivity of Q and q_p to change in CN. In general, the absolute sensitivity of a variable y to changes in a variable x is given by:

$$S = \frac{dy}{dx} \tag{4}$$

where S is the sensitivity of y to changes in x. Another useful measure is the relative sensitivity (R_s) of y to changes in x (McCuen, 1973); R_s is defined by:

$$R_s = \frac{dy/y}{dx/x} = \frac{dy}{dx} \cdot \frac{x}{y} \tag{5}$$

R_s measures the proportional change in x for a proportional change in y. A principal advantage of using the relative versus absolute sensitivity is that R_s is non-dimensional, so that sensitivities of different variables can be compared.

A modification of Eq. 5 is desirable for expressing the sensitivities of Q and q_p to changes in CN. The modification is to express proportional changes in Q and q_p per unit change in CN. These proportions can be expressed by:

$$R_Q = \frac{dQ/Q}{dCN} = \frac{dQ}{dCN} \cdot \frac{1}{Q} \tag{6}$$

and

$$R_q = \frac{dq_p/q_p}{dCN} = \frac{dq_p}{dCN} \cdot \frac{1}{q_p} \tag{7}$$

where R_Q and R_q are the proportional changes in Q and q_p, respectively, per unit change in CN.

The use of Eqs. 6 and 7 rather than Eq. 5 is desirable because proportional changes in CN are not as meaningful as unit changes in CN for several reasons. The CN is nondimensional, so there is no dimensional advantage to CN proportions. Also, variations in CN result from misclassifying either land cover, treatment, hydrologic conditions, and/or soil type, so that the magnitude of the CN deviation depends on both the size of the area misclassified and the type of misclassification; the proportional change in CN is not determined only by the proportion of measurement variation.

Analysis of Derivatives

Eqs. 6 and 7 can be expressed in terms of P, CN, and t_c by application of Eqs. 1, 2, and 3. The design rainfall depth, P, is a constant in terms of CN, but the t_c varies with the CN. A commonly used formula developed by the SCS for computing t_c is:

$$t_c = \frac{5}{3} \left[\frac{\ell^{0.8} (S+1)^{0.7}}{1900\ Y^{0.5}} \right] \tag{8}$$

where ℓ is the hydraulic length in feet, Y is the average watershed slope in percent, S is given by Eq. 2, and the t_c is in hours. Eq. 8 will be used in developing an expression for R_q as a function of P and CN. A time of concentration should be modified using the adjustment coefficients provided in Chapter 3 of TR-55 when there is a significant percentage of impervious area or the hydraulic length has been modified.

Analysis of R_Q. Eq. 6 can be rewritten as:

$$R_Q = \frac{\partial Q}{\partial S} \cdot \frac{dS}{dCN} \ /Q \tag{9}$$

By application of Eqs. 1 and 2, and rearranging terms yields:

$$R_Q = \left[\frac{0.4}{(P-0.2S)} + \frac{0.8}{(P+0.8S)} \right] \frac{1000}{CN^2} \tag{10}$$

Eq. 10 can be directly applied to determine the proportional change in Q that results from a unit change in CN for a particular design rainfall depth.

Analysis of R_q. Eq. 7 can be rewritten as:

$$R_q = \left[\frac{\partial q_p}{\partial Q} \cdot \frac{dQ}{dCN} + \frac{\partial q_p}{\partial t_c} \cdot \frac{dt_c}{dCN}\right] / q_p \tag{11}$$

Differentiation of Eq. 3 with respect to Q yields:

$$\frac{\partial q_p}{\partial Q} = a\ t_c^{-b} \tag{12}$$

in which a and b are the coefficients of Eq. 3. Differentiation of Eq. 3 with respect to t_c yields:

$$\frac{\partial q_p}{\partial t_c} = -Qabt_c^{-(b+1)} \tag{13}$$

Application of Eqs. 1 and 2 yields:

$$\frac{dQ}{dCN} = \left[\frac{-0.4(P+0.8S)(P-0.2S)-0.8(P-0.2S)}{(P+0.8S)^2}\right]\left[\frac{-1000}{CN^2}\right] \tag{14}$$

By Eq. 8 and rearranging terms:

$$\frac{dt_c}{dCN} = \frac{\partial t_c}{\partial S} \cdot \frac{dS}{dCN} = \frac{-700\ t_c}{(S+1)(CN^2)} \tag{15}$$

After rearranging, the combining of Eqs. 3, 11, 12, 13, 14, and 15 yields:

$$R_q = \left[\frac{0.4}{(P-0.2S)} + \frac{0.8}{(P+0.8S)}\right] \frac{1000}{CN^2} + \frac{700b}{(S+1)(CN^2)} \tag{16}$$

It is important to notice that R_q is not a function of t_c. The coefficient b is dependent on t_c, but b is constant over various ranges of t_c. Therefore, for t_c between 0.7 and 5.0 hours, R_q is dependent only on P and CN.

Separation of Error in R_q. Eqs. 10 and 16 can be combined to yield:

$$R_q = R_Q + \frac{700b}{(S+1)(CN^2)} \tag{17}$$

The second term on the right-hand side of Eq. 17 represents the proportional change in q_p due to the variation in estimating t_c that may result from deviation in CN, ℓ, and Y. Therefore, the variation in q_p is equal to the sum of the deviation in runoff volume plus the deviation associated with estimating t_c.

EVALUATION OF SENSITIVITIES

R_Q is a function of P and CN as observed in Eq. 10. Fig. 1 presents curves of R_Q versus P for CN's ranging from 50 to 95. Fig. 1 shows that R_Q decreases as P and CN increase. This is reasonable because the CN determines the initial abstraction and the infiltration rate. As P and CN increase, the proportion of the rainfall that goes into the initial abstraction and infiltration decreases, so the proportional error also decreases.

Fig. 1 can be used directly to determine the percent error in Q associated with a unit change in CN for a particular design rainfall depth. Fig. 1 can also be used for CN variation other than unity. For instance, for a design rainfall depth of 6 inches and a CN of 70, the value or R_Q is approximately 0.033. A CN error of ± 5 will, therefore, produce an error of about 0.003 x 5, or 16.5 percent.

Fig. 2 presents curves of R_q versus P for CN ranging from 50 to 95. A value for b of 0.75 is used, which is associated with t_c between 0.7 and 5.0 hours. Fig. 3 is analogous to Fig. 1 and can be used for estimating proportional errors in q_p in the same manner as Fig. 1 can be used.

Fig. 2 shows that R_q decreases as P increases for a particular CN. However, R_q does not necessarily decrease as the CN increases. This behavior is in contrast to that shown in Fig. 1, in which R_Q always decreases as the CN increases.

The difference in the behavior of R_q as a function of CN as compared to the behavior of R_Q can be explained by examination of Eq. 17. The second term on the right-hand side is the error associated with the error in estimating t_c. This term is independent of P, so for a given value of CN, R_q is equal to R_Q plus a constant. As P increases, R_Q decreases, so the effects of the t_c term on R_q become more dominant. The effects of the t_c term on R_q cause the difference in the behavior of R_q as compared to R_Q.

The effects of the t_c term in Eq. 17 are illustrated in Fig. 3, which shows the proportion of the error in R_q that is caused by the error in t_c. The effects of the error in t_c increase as both P and CN increase. The effects can be significant; at P=8 in. and CN=95, the error in t_c accounts for 75% of the total error in q_p.

APPLICATIONS OF THE SENSITIVITY ANALYSIS

The sensitivity analysis as summarized in Figs. 1 and 2 can be used directly for evaluating the hydrologic effects of differences in estimated CN's. Differences in CN estimates can occur for several reasons. One reason can be the use of different methods for estimating CN's. Another reason is random variation; random variation can be caused by both human judgment and data base errors. Finally, CN's are at best approximations of the true runoff potential; the sensitivity analysis can serve as a bridge between the degree of inaccuracy in the CN tables and the hydrologic effects of the inaccuracy.

Effects of Different CN Estimation Methods

An example of direct use of the sensitivity analysis is to evaluate the hydrologic effects due to the use of two different methods for estimating CN's. For instance, a study of three watersheds in southeastern Pennsylvania (Bondelid, et al., 1980) compared CN's estimated by conventional methods vs. using Landsat data. The sensitivity analysis can be readily applied to this study to determine the hydrologic effects of using Landsat vs. conventionally derived CN's.

The study by Bondelid et al., (1980) was conducted by estimating the conventional and Landsat CN's on three watersheds. These watersheds are the Quittapahilla, Chickies Creek, and Little Mahanoy basins. Tables 1, 2, and 3 summarize the percent differences in Q and q_p for each subwatershed in the Quittapahilla, Chickies Creek, and Little Mahanoy watersheds, respectively, for the 100-year, 24-hour rainfall of 6.3 inches. The values were determined by computing R_Q and R_q from Eqs. 10 and 17, multiplying by the residual, and then multiplying by 100 to convert to percent. The residuals were computed by subtracting the Landsat CN from the conventional CN. If the conventional CN's are taken as the standard by which other CN estimation procedures are evaluated, then Table 1, 2, and 3 give the percent error in computed values of Q and q_p that can occur as a result of using Landsat.

Tables 1, 2, and 3 show that the differences in q_p are approximately twice as large as the differences in Q for the 100-yr. storm. The average differences for the 53 subwatersheds in computed values of Q and q_p are -4.5 and -6.3 percent, respectively. Given the accuracy of the models and the random error associated with CN estimation, the overall differences are probably not significant. This example illustrates how the sensitivity analysis can be applied in evaluating alternative CN estimation methods. The differences in the methods can be readily evaluated in terms of the resulting hydrologic differences.

Effects of Input Variation

A recent study, which is not published at the present (1981), involved the estimation of peak discharges on approximately 70 watersheds, with estimates made independently by 5 hydrologists on each watershed. The study involved the comparison of nine hydrologic methods, including the TR-55 graphical method. Thus, the data base included a large number of curve number estimates, including independent estimates for the same watershed. Curve number estimation requires the synthesizing of land use, hydrologic condition and treatment, and soils data. Thus, any variation in the delineation of areas of homogeneous land use, hydrologic conditions, or soil type will be reflected in the variation of CN estimates. For watersheds in the midwestern United States, where the 100-year, 24-hour precipitation is on the order of 6.5 inches, the five CN estimates often varied by 3 curve numbers. For a CN of 70, Figs. 1 and 2 show that the relative error in runoff volume and peak discharge would be 0.035 and 0.05, respectively. Thus, for a variation of 3 curve numbers one could expect 10 percent and 15 percent differences in runoff volume and peak discharge, respectively, as a result of variation solely in the delineation of land use and soil type. For watersheds that are characterized by wide variation in land use and soil type, one can expect larger variations. This is not the fault of the CN concept, but results from failure of the hydrologist to take the proper care in gathering the input data for CN estimation.

CONCLUSIONS

A straightforward technique for evaluating the hydrologic effects of CN variation has been developed in this study. These effects are evaluated in terms of the proportional changes in total runoff and peak discharge for a unit change in CN. The results are summarized in a series of curves in which the proportional changes in total runoff and peak discharge are shown as functions of design rainfall depth. The curves show that the effects of variation in CN decrease as design rainfall depth increases. This supports Hawkins (1975) conclusion that errors are especially dangerous near the threshold of runoff.

Examples of the applicability of the analysis are presented. These examples include the evaluation of the hydrologic effects of Landsat vs. conventionally derived CN's. Another example is evaluation of the effects of human judgment in estimating CN's. The curves of Figs. 1, 2, and 3 should be especially useful for those who do not apply SCS methods on a regular basis.

Figs. 1, 2, and 3 were developed using the TR-55 graphical method. The graphical method was developed from numerous TR-20 computer runs. Therefore, Figs. 1, 2, and 3 should provide good estimates of the sensitivity of volumes and peak discharges estimated with TR-20 when the conditions specified in TR-55 for using the graphical method are not violated.

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TABLE 1. HYDROLOGIC EFFECTS AT LANDSAT CN'S IN QUITTAPAHILLA

Subarea	Conv. CN	Landsat CN	Residual	%Change in Q	%Change in QP
1	84.	81.	-3.	-7.410	-14.824
2	84.	81.	-3.	-7.410	-14.824
3	83.	80.	-3.	-7.542	-14.794
4	70.	72.	2.	6.212	10.308
5	77.	75.	-2.	-5.582	-9.954
6	72.	72.	0.	.000	.000
7	81.	79.	-2.	-5.167	-9.855
8	85.	82.	-3.	-7.283	-14.873
9	82.	80.	-2.	-5.074	-9.857
10	72.	70.	-2.	-6.212	-10.308
11	85.	83.	-2.	-4.814	-9.937
12	82.	80.	-2.	-5.074	-9.857
13	72.	73.	1.	3.004	5.088
14	85.	84.	-1.	-2.387	-4.981
15	81.	78.	-3.	-7.823	-14.787
16	84.	76.	-8.	-20.669	-39.419
17	78.	78.	0.	.000	.000
18	81.	79.	-2.	-5.167	-9.955
19	70.	69.	-1.	-3.215	-5.232
20	75.	74.	-1.	-2.878	-5.017
21	81.	75.	-6.	-16.098	-29.650
22	77.	76.	-1.	-2.763	-4.966
23	81.	77.	-4.	-10.529	-19.727
24	73.	74.	1.	2.940	5.050
25	84.	82.	-2.	-4.897	-9.897
26	87.	86.	-1.	-2.309	-5.049
Average:				-4.929	-9.508

TABLE 2. HYDROLOGIC EFFECTS OF LANDSAT CN's IN CHICKIES CREEK

Subarea	Conv. CN	Landsat CN	Residual	%Change in Q	%Change in QP
1	68.	69.	1.	3.293	5.291
2	59.	58.	-1.	-4.347	-6.242
3	59.	59.	0.	.000	.000
4	74.	74.	0.	.000	.000
5	70.	72.	2.	6.212	10.308
6	78.	74.	-4.	-11.164	-19.908
7	79.	81.	2.	5.167	9.855
8	79.	79.	0.	.000	.000
9	79.	79.	0.	.000	.000
10	75.	74.	-1.	2.878	-5.017
11	76.	77.	1.	-2.763	4.966
12	78.	77.	-1.	-2.709	4.949
13	79.	79.	0.	.000	.000
14	79.	80.	1.	-2.608	4.929
15	77.	76.	-1.	-2.763	-4.966
16	77.	76.	-1.	-2.763	4.966
Average:				-4.11	-.669

TABLE 3. HYDROLOGIC EFFECTS OF LANDSAT CN'S IN LITTLE MAHANOY

Subarea	Conv. CN	Landsat CN	Residual	%Change in Q	%Change in QP
1	62.	60.	-2.	-8.049	-11.865
2	63.	64.	1.	3.747	5.676
3	67.	67.	0.	.000	.000
4	74.	72.	-2.	-5.943	-10.136
5	74.	71.	-3.	-9.011	-15.263
6	71.	70.	-1.	-3.142	-5.179
7	73.	70.	-3.	-9.213	-15.392
8	73.	73.	0.	.000	.000
9	75.	73.	-2.	-5.817	-10.065
10	84.	82.	-2.	-4.897	-9.897
11	73.	72.	-1.	-3.004	-5.088
Average:				-4.121	-7.019

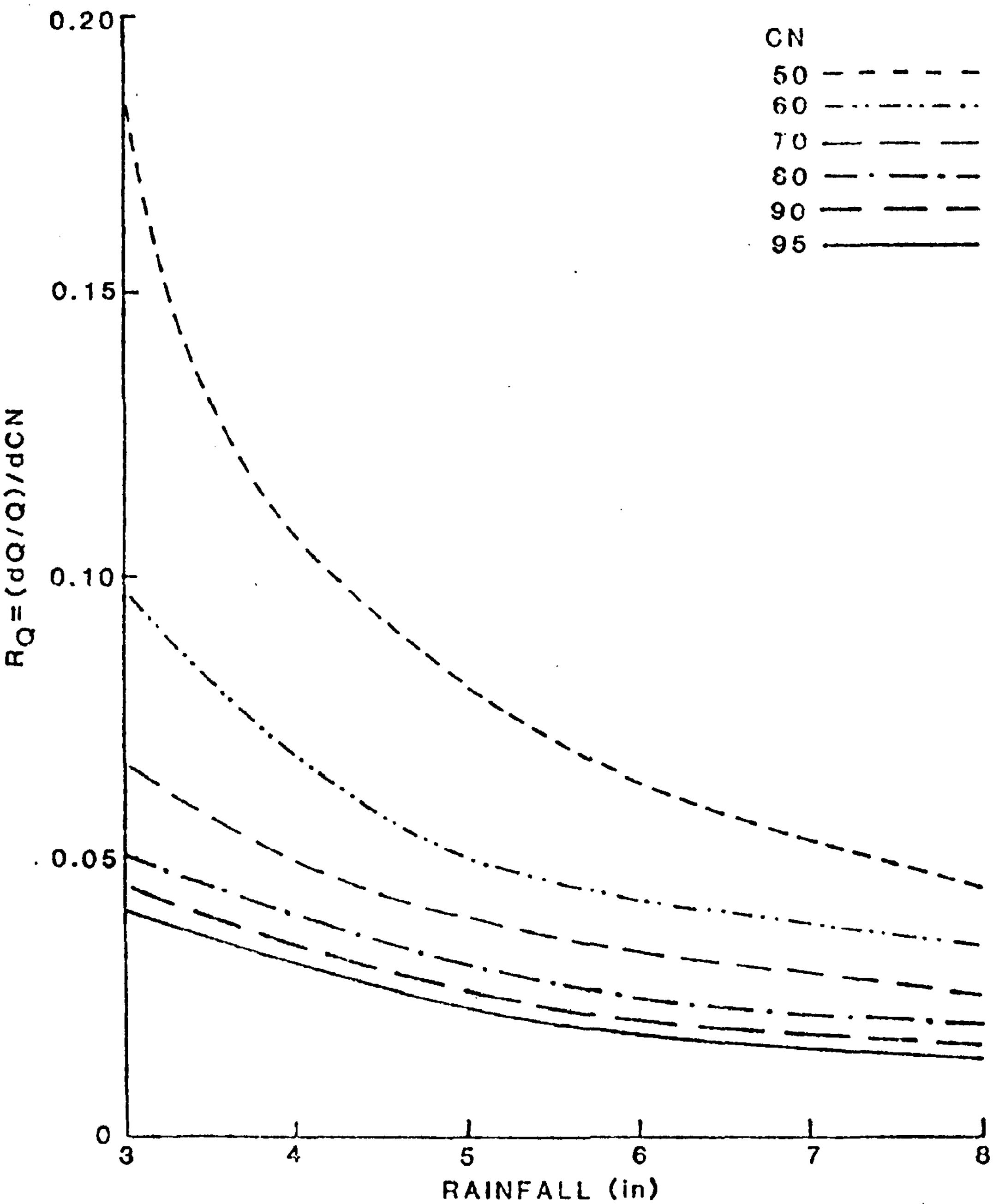

FIGURE 1. RELATIVE SENSITIVITY OF Q TO CN
($0.7 \leq t_c \leq 5.0$ hours)

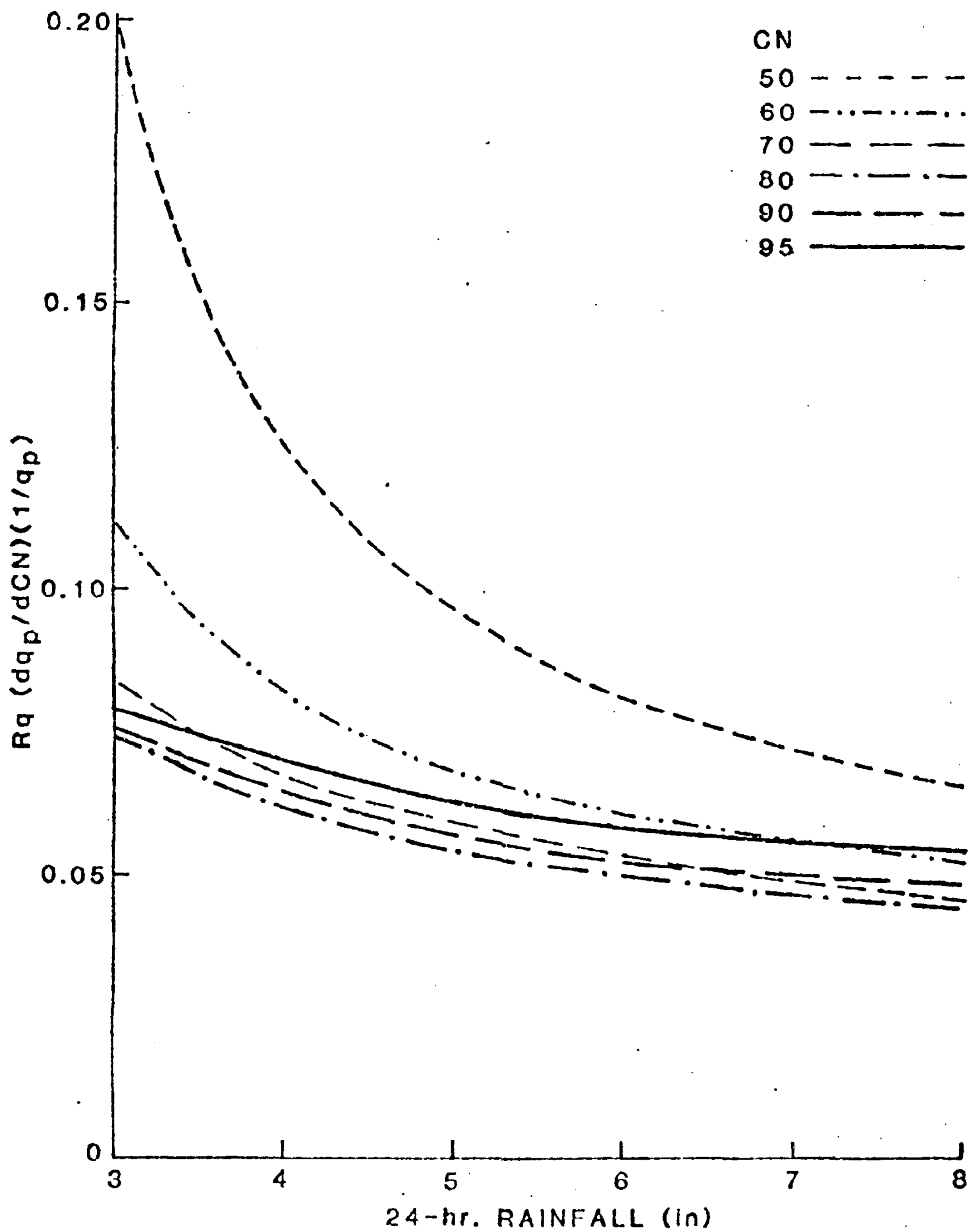

FIGURE 2. RELATIVE SENSITIVITY OF q_p TO CN
($0.7 \leq t_c \leq 5.0$ hours)

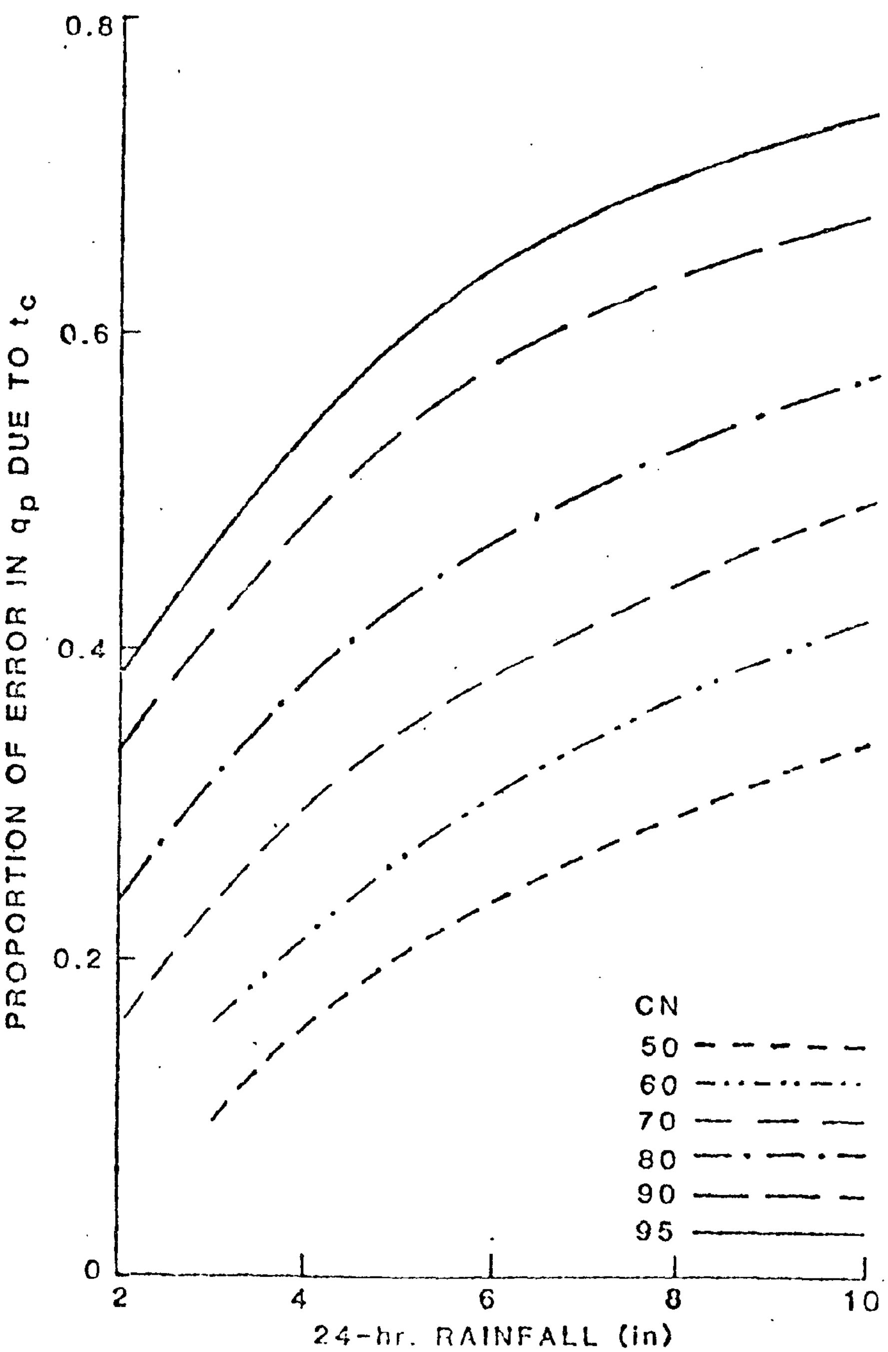

FIGURE 3. PROPORTION OF ERROR DUE TO t_c

($0.7 \leq t_c \leq 5.0$ hours)