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Development Of A General Planning Methodology For Storm Water Management In Urban Watersheds

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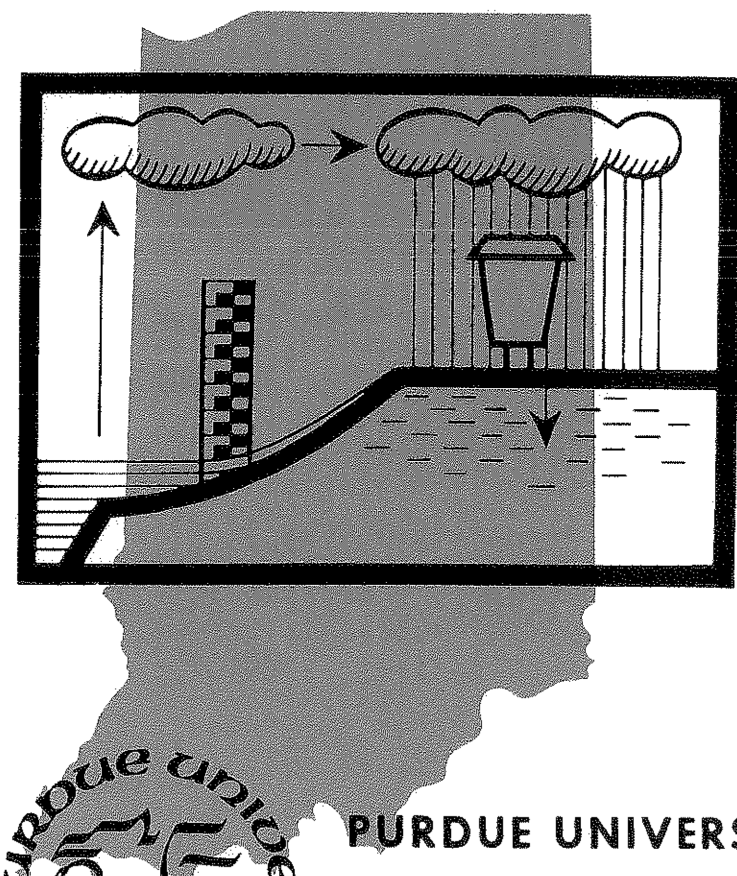

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ABSTRACT

A new methodology is developed for use in the planning of dual purpose detention basins in urban watersheds. The methodology employs continuous simulation, statistical analysis, and a general design heuristic to obtain an integrated system of detention basins. Both water quantity and water quality constraints may be considered.

Several different approaches were considered in the development of the design heuristic. The developed methodology uses a dynamic program to obtain a feasible starting point for a nonlinear search algorithm. The nonlinear search algorithm employs the Complex Method of Box.

The general planning methodology was applied to an actual watershed in Glen Ellyn, Illinois, and to a synthetic watershed that was constructed from average geomorphic data for the the state of Indiana.

I. INTRODUCTION

1.1 PROBLEM DEFINITION

Comprehensive planning for the control of stormwater runoff is becoming an increasingly significant part of overall development objectives in developing urban communities (Zeigler and Lakatos, 1982). The emphasis in urban stormwater runoff abatement traditionally has been either that of conveying large quantities of runoff to a downstream area as quickly as possible, or of attacking a stormwater problem where it is visible. Past trends in urban expansion have been generally to size storm sewers initially for only a modest future growth. A small community, with only a vague idea of its eventual size, usually finds the cost associated with a larger than necessary storm sewer installation to be un-justifiable.

Most rapidly developing urban areas are now finding themselves faced with the almost inevitable problem of storm sewer overloads (Lakatos, 1976). In response to these and other problems, many municipalities are employing detention basins as the primary stormwater management control (Kamedulski and McCuen, 1979). Although detention storage has been shown to be an effective stormwater control, random or unplanned placement can significantly reduce its effectiveness, and in some cases, can actually aggravate potential flood hazards. In addition, designs which fail to consider the long term performance of a basin can result in ineffective management for a wide range of runoff events.

While stormwater detention basins have been used for the control of urban runoff for many years, only recently has there been an interest in examining the impact of detention for the control of water quality loadings in urban runoff (McCuen, 1980). Recent studies, including those of the National Commission on Water Quality (1975) have shown that urban runoff is an important source of pollution and that the national water quality objectives of Public Law (PL) 92-500 are impossible to attain without control of nonpoint sources and urban runoff.

While various policies have been proposed for improving the quality of urban runoff, probably the most effective stormwater management technique is the use of the detention basin (Kamedulski and McCuen, 1979). As a result of this observation, many states are now requiring developers to consider quality control when designing and installing detention basins. This is true despite the fact that very little information is available as to the efficiency of the basins for removal of different kinds of pollutants or as to how detention requirements for both quality and quantity purposes should be integrated (Whipple, 1979).

The widespread use of detention basins is reflected in the results of a recent AWPA survey of 325 public agencies. The results of this survey indicated that over 50 percent of the drainage master plans of the surveyed municipalities included detention basins. Nearly 40 percent of those communities without detention facilities said that facilities are being built, are in the planning stage, or have been considered and are a priority item for the near future (Poertner, 1981). Four of the top eight design objectives, reported by the public agencies responsible for establishing detention facilities, fall in the category of water quality enhancement (Smith et al., 1981).

The relevancy of the general problem of detention basin design and utilization was recently highlighted at the 1982 ASCE specialty conference on detention basins, which was held at Hennicker, New Hampshire. The conference addressed both institutional and design issues. Among the recommendations of the conference was a need to integrate both quality and quantity objectives into the design of individual basins. The need for a better understanding of the quality impact of detention basins was considered. In addition, the need for a better understanding of the interaction of various detention basins within a watershed was identified. The need for general planning methodologies was also addressed.

1.2 OBJECTIVES OF THE REPORT

The primary goal of this study is the development of a general planning methodology which can be used in the design and placement of stormwater detention basins in urban areas so as to minimize local flooding and maximize water quality. The developed methodology is to be applicable in the planning of a watershed detention system in conjunction with the existing major drainage network of the watershed. The general problem involves two different levels of optimization: the optimal design of the individual basins, and the optimal location of the individual basins within the watershed. The optimization problem involves three different objectives: minimization of local flooding, maximization of overall watershed water quality, and minimization of the overall system cost.

1.3 ORGANIZATION OF THE REPORT

The report is divided into two parts. The first part involves the development of a methodology to be used in the optimal design and placement of stormwater detention basins. The second part involves the application of the developed methodology.

The methodology may be applied in two different ways. The first approach involves the application of the methodology in a design mode for a specific site. The second approach involves the use of the methodology to obtain general planning guidelines for a specific region or area. Such an application could involve the use of synthetic watersheds and average design parameters. Based on the results of such an application, an attempt could be made to derive general planning indices for use in the preliminary design of watershed detention systems.

Chapter II consists of a literature review of both water quantity and water quality design considerations as related to detention basin design. The general planning methodology is presented in Chapter III. Chapter IV includes the results of the application of the general methodology to a specific case study. Chapter V contains the results of the application of the general methodology to a synthetic watershed. Finally, Chapter VI consists of a summary of the study with recommendations for further work.

II. LITERATURE REVIEW

2.1 HYDRAULIC DESIGN CONSIDERATIONS

2.1.1 Preliminary Design Procedures

The design procedure for a detention basin will normally involve several trial calculations. This is because the duration of the critical design storm for the chosen design occurrence interval is not known in advance. Thus it is necessary to route design hydrographs for a range of durations through the basin and select the one that yields that worst case (Mein, 1980).

In the past, several authors have presented various preliminary design procedures that avoid repetitive calculations by making various simplifying assumptions. Paintal (1979) has described a procedure that assumes a trapezoidal hydrograph and either a constant outflow rate or orifice control. The critical design storm duration is then determined explicitly by differentiating a derived expression for storage volume. Burton (1980) has also presented a method that assumes a trapezoidal hydrograph and a constant outflow rate. This method also includes an explicit equation for determining the critical storm duration. Additional preliminary design procedures have been presented by Yrjanainen et al. (1973), Baker (1977), and Ordon (1974). Chapter 7 of the Soil Conservation Service Technical Release No. 55 (USDA-SCS, 1975), or TR-55, can also be used in the preliminary design of detention basins. The TR-55 method is based on average storage and routing effects for many structures that were evaluated using more accurate TR-20 methods (USDA-SCS, 1965).

2.1.2 Peak Flow Design Procedures

Urbanization decreases the natural storage in a watershed and thus leads to an increase in both runoff volume and rate. While detention basins do not limit the increase in total runoff volume, they can be used to control the peak runoff rates (McCuen, 1979).

In recent years, many stormwater management policies have been adopted with the intent of limiting peak flow rates from developed areas to that which occurred prior to development (McCuen, 1974). As a result of these policies, several authors: Wycoff and Singh (1976), Bouthiller and Peterson (1978), and Boyd (1981), have presented procedures for estimating the required storage volume for a detention basin as a function of the inflow hydrograph and the desired peak outflow rate. Each of the methods assumes a series of possible idealized hydrograph geometries and basin outflow configurations. Based on these assumptions, equations are derived which directly relate the required basin size to variables representing the standard inflow and outflow hydrographs.

2.1.3 Hydrograph Timing Considerations

In addition to affecting the total runoff volume and peak flow rate for a given watershed, urbanization also affects the timing of the runoff hydrograph. While most stormwater management policies are designed to limit the effect of an increase in the peak runoff rate, they tend to ignore the effects of urbanization on the time characteristics of both the inflow hydrograph and the detention basin outflow hydrograph (Hawley et al. 1981). Most existing laws tend to deal with the problem on an individual site basis as opposed to a regional approach. In fact, many current regulations use the phrase "on-site detention" in describing the required structures (McCuen, 1974).

Despite the widespread use of detention basins for flood control, the possibility exists that certain combinations of stormwater detention basins may actually increase the flooding problems of a given watershed (McCuen, 1974). Detention basins provide for a temporary storage of stormwater runoff for a delayed release. This will always reduce the peak flow immediately downstream of the detention structure. However, if the delay is such that the subsequent release of the runoff water coincides with runoff from more upstream areas, the resulting flooding may be more severe than would have been the case had the water been allowed to run off rapidly prior to the arrival of high flows from upstream areas.

Many investigators have studied this problem and have concluded that the planning of stormwater control systems must be done on a watershed or regional basis as opposed to a sub-area, piecemeal approach. Smiley and Haan (1976), investigated the problem using a synthetic watershed composed of seven subsheds to show that under some circumstances the installation of detention basins may aggravate flooding. Their analysis showed that the installation of detention basins in the lower part of the watershed resulted in higher peaks than would have occurred without the structures.

Similar conclusions, that the unplanned placement of multiple detention reservoirs may aggravate potential flood hazards, have been reached by Lumb et al. (1974) and by Abt and Grigg (1978). Likewise, McCuen (1974) states that stormwater management plans must be evaluated on a regional basis and not just by using on-site control criteria. In a study of a watershed in Montgomery County, Maryland, McCuen (1979) shows where a stormwater management scheme increased peak flows, bedload transport rates and the duration of bankfull flow in the channel downstream of the facility. More recently, Duru (1981) analyzed the Tinkers Creek watershed in Prince George's County, Maryland, and found that detention was not needed in any part of the watershed.

2.1.4 Detention Basin Simulation Models

In order to consider the effect of hydrograph interaction, Mein (1980) has proposed the use of an urban watershed model coupled with a detention basin design subroutine that is run interactively to allow the user to design the basin system on line. Based on the results of the application of the methodology, Mein concluded that a single detention basin is more effective in reducing peak flows at a point, than is a series of basins with the same combined storage capacity. In addition, Mein concluded that the effect of a basin on peak reduction diminishes downstream as more flows contribute.

In a more recent study, Hawley et al. (1981) developed a quick planning method for estimating the potential of adverse downstream effects of a stormwater management basin. The planning method involves estimation of direct runoff hydrographs and basin outflow hydrographs. Discharge rates are estimated using the TR-55 graph method, and empirical timing equations are used for estimating the time coordinates of the hydrographs.

Lakatos (1976) has suggested that the problem be approached using an urban runoff timing analysis computer program such as the Penn State Runoff Model. The Penn State Runoff Model is a simple and concise stormwater 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 combinations of subarea flows provides the basis for evaluation of flood control alternatives for the source of the flooding problem rather than the point of actual flooding.

This model was used in developing a regional stormwater management plan for Allegheny County, Pennsylvania (1980). The plan is being implemented through the use of a release rate percent concept for the control of stormwater runoff (Sprague et al., 1982). The release rate percentage concept utilizes peak flow information to determine a percentage for each subshed which is used by developers to regulate the rate of release from a detention facility. The percentage is determined by analyzing the discharge from the subbasin that contributes to the peak rate for a given problem area and dividing that value by the maximum runoff rate from that subshed. This percentage is then used to compute the allowable release rate from a developer's detention facility.

2.1.5 Optimal Design Models

The optimal design of a stormwater detention basin system involves two levels of optimization. The first level deals with the optimal design of the individual basins while the second level deals with the optimal placement of the detention basins in a watershed. In response to the first level optimization problem, Bondelid and McCuen (1979) have developed a computerized optimization procedure for the hydraulic design of stormwater management basins. The method considers the relationship between inflow, outflow structure configuration, basin storage characteristics, reservoir routing, and outflow. Runoff hydrographs are generated and routed through the reservoir using the TR-20 computer program. For given orifice and riser diameters, the basin side slopes are adjusted so that the outflow peak discharges for the flows of different return periods are approximately equal to the before development peak discharges for the same return period. A large number of combinations of orifice and riser diameters are considered in each computer run. The orifice and riser combination that requires the least storage but still meets the design criteria is determined (Donahue, 1981).

2.1.6 Optimal Design and Placement Models

Very few authors have studied the problem of the optimum placement of detention basins in an urban watershed.

As a first step, Abt and Grigg (1978) developed an approximate method for the sizing and placement detention basins in series. The procedure involves a number of simplifying assumptions and is based upon the application of a storage estimation equation. The equation was applied to a hypothetical watershed and compared to the results of the HEC-1 (USACE, 1973) runoff model which was applied to the same watershed. A comparison of the methods revealed that the reservoir storage volumes generated by the estimation procedure closely approximated the storage generated by HEC-1. However the authors concluded that due to the number of simplifying assumptions the potential application of the storage estimation equation is quite limited.

Mays and Bedient (1982) have applied dynamic programming to the problem, using storage as a state variable while employing simplified routing techniques to transform decision variables between stages. The developed methodology can handle constraints on discharge, storage, and basin area. However, the depth of the basin must be specified and the detention basin is apparently assumed to have vertical sides. In addition, the methodology does not consider water quality constraints. Finally, the model does not calculate the benefits involved in each design and therefore cannot select the best detention basin storage policy. Flores et al. (1982) have applied the dynamic programming algorithm to three synthetic watersheds representative of the Houston area. Based on the results of this study, they concluded that detention storage is usually only needed in the upper part of an urban watershed.

2.1.7 Storm Sewer Models

In the optimization of sewer network systems, both Cheng (1981) and Froise (1978) have developed optimization models based on dynamic programming procedures to determine the minimum cost sewer system design. Although both methodologies included detention storage, the analyses were very simple and failed to consider the timing interactions of the various hydrographs.

2.1.8 Watershed Planning Models

In a separate study, Dendrou and Delleur (1982) present a methodology for the planning of stormwater detention facilities on a watershed wide basis. The methodology employs the runoff simulation model STORM along with a dual level optimization scheme in determining the optimal treatment rate allocation among the various subbasins. Although of value as a general planning model, the model fails to examine explicitly the interaction between detention basin outflow hydrographs, nor does it examine the optimal design of the individual detention basins.

2.1.9 Operation Models

In a somewhat related study, Labadie et al. (1975) have described the development of a general automated control methodology for large scale combined sewer systems. The general methodology was developed using the San Francisco Wastewater Management Plan as a case study. A discussion of the simulation techniques used in the methodology has been presented by Wenzel (1976), while Bradford (1977) has discussed the development and application the required control algorithm. The objectives used in the control algorithm include minimization of overflows and street flooding, and regulation of flow to the sewage treatment plant by utilization of system detention storage. The problem is formulated as a large scale linear programming problem which is then reduced to a multilevel series of smaller problems by application of a disaggregation methodology. Six levels, containing 39 linear programming problems, were required to obtain the control policy for the case study involving the San Francisco Master Plan for Wastewater Management.

In a further examination of the same wastewater management plan, Labadie et al. (1980) have developed a dynamic program which incorporates the full dynamic flow equations and can be used to obtain the gate settings on flow regulation devices. Although convergence to global optima cannot be guaranteed, the algorithm is shown to rapidly determine improved control policies. The authors thus argue that the algorithm may thus be feasible for actual real-time use when there are severe limitations on computational capacity and time for reaching a control decision.

2.2 WATER QUALITY CONSIDERATIONS

2.2.1 Dual Purpose Detention Basins

As a result of growing concerns with regard to the water quality of the urban environment, stormwater management basins are now being used to control water quality in addition to the quantity of runoff. Two of the earliest states to consider the use of dual purpose detention basins were New Jersey and Virginia. Both states have established regulatory programs that require developers to control and manage storm runoff in such a manner as to prevent any adverse environmental consequences resulting from increased development (Kropp, 1982). In a recent APWA survey (1980), four of the top eight objectives reported by the public agencies responsible for establishing detention facilities fall in the category of water quality enhancement (Smith, 1981). Despite the growing use of stormwater management basins to control quality, very little information is available as to the efficiency of these basins for removal of different kinds of pollutants, or as to how detention requirements for both quantity and quality purposes should be integrated (Whipple, 1979).

2.2.2 Pollutant Removal Mechanisms

The mechanisms controlling pollutant removal in detention facilities are complex and numerous. Figure 2.1 (Nix et al., 1981) summarizes the more significant mechanisms. Most of these factors can be related to the concept of detention time. Simply defined, detention time is the time a parcel of water spends in the basin or pond. The mechanisms shown in Figure 2.1 are each affected by or affect detention time. Particle settling is affected by detention time as is biological stabilization. Outlet structures can be designed to achieve various detention times. The inflow rates have a direct bearing on detention times. In short, detention time is the primary indicator of pollution control capability.

The concept of detention time is generally understood, but its computation, especially in stormwater detention, is not always so clear. The basic definition is relatively simple: detention time is the length of time a parcel of water spends in the basin or pond. Detention time is easy to compute under steady state conditions:

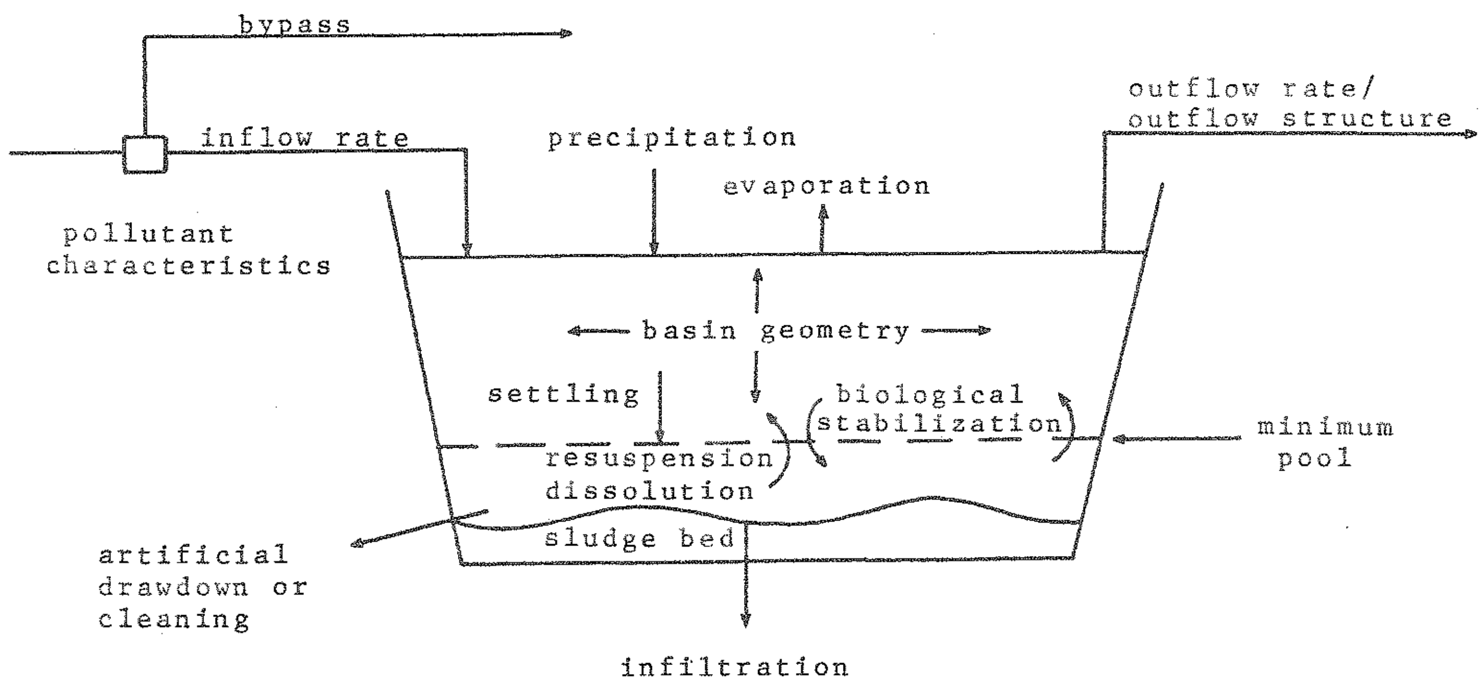


Figure 2.1 Pollutant Removal Mechanisms

$$t_d = S / \bar{q} \qquad (2.1)$$

where t_d = detention time (sec)

S = detention volume (cuft)

$\bar{q}$ = constant flow rate (cuft/sec)

In completely-mixed units, t_d represents the average detention time. In plug-flow units, t_d is the actual time all parcels spend in the detention basin. Unfortunately, a steady state condition is rarely found in a sanitary sewage plant and is certainly improbable in stormwater detention facilities. Therefore, such a computational definition is of limited value (USEPA, 1983).

For stormwater flows, it may be necessary to compute detention time using a computer simulation model because of its predictive value. This is often accomplished by modeling the detention basin or pond as a plug-flow reactor. Such a model simply queues relatively small parcels or plugs (ideally, the parcel is infinitely small) through the basin (Rich, 1973). In other words, the first parcel of water entering the basin is the first parcel to leave. Pollutants entering a basin with a plug are assumed to remain with that plug. The detention time can be calculated for each plug by:

$$t_{d_i} = t_{d_i}(2) - t_{d_i}(1) \qquad (2.2)$$

where t_{d_i} = detention time for plug i

t_{d_i} (1) = point in time that plug i entered the basin

t_{d_i} (2) = point in time that plug i left the basin

Detention facilities may also be viewed as completely-mixed or arbitrary flow reactors. True values of detention time are difficult to calculate under these assumptions. In completely-mixed reactors the inflow parcels and associated pollutants are completely intermixed with all other parcels in the unit and, thus, lose their identity. Arbitrary-flow reactors are a blend of plug-flow and completely-mixed reactors.

2.2.3 Pollutant Removal Studies

It is now becoming more widely accepted that suspended solids are one of the most significant runoff pollutants and can act as a carrier of other important water quality pollutants (Amandes and Bedient, 1980). Pollutants that are believed to have a particularly high affinity for adsorption on suspended particles are phosphorus, heavy metals, and petroleum based organics. Many of the most important pollutants in urban runoff occur in particle form. The association of the pollutants in urban runoff with the suspended solids suggests that the individual pollutant loadings may be reduced by sedimentation (Randall, 1982).

As a result of the above observation, several authors have suggested that the removal of various pollutants can be estimated based on the removal of the suspended sediment load. Such an approach involves the concept of a potency factor which is the ratio of the pollutant's mass to the mass of the accumulated sediment (Sutherland, 1980). For a given set of pollutant potency factors, the removal efficiency of a detention basin for a given set of pollutants can be estimated by determining the sediment trapment efficiency of the basin.

Although the above approach would appear to be quite logical, the assumption that particulate pollutants will be removed proportionally to their concentration in sediment has not been substantially verified. While several studies have been conducted with regard to the settling properties of sediment, very little information is available regarding the actual settling characteristics of urban runoff pollutants. One of the few studies that has been reported in the literature is by Whipple and Hunter (1981). Although the results obtained by Whipple and Hunter indicate a close association between hydrocarbons and suspended solids, the settling characteristics of the remaining pollutants (BOD, Total P, Pb, Cu, Ni, Zn) differed widely.

In a more recent study, Randall et al. (1982) also investigated the settling characteristics of urban runoff pollutants. Seven samples from three different sites were used in studying the settleability of TSS, COD, BOD, total organic carbon, phosphorus forms, nitrogen forms, and six heavy metals. The results showed that substantial reductions of the pollutants in urban runoff can be achieved by sedimentation, and that the particles can be categorized as flocculant particles for analysis and design. The best reductions were obtained for TSS, lead and BOD, with average removals of 90, 86, and 64 percent respectively. Except for lead, pollutant removals did not correlate well with TSS removal although higher TSS concentrations generally produced higher total pollutant removals.

As an alternative to correlating the removal of various pollutants to the removal of sediment, a more accurate approach would be to determine the actual particle size distribution associated with each pollutant. By specifying settling rates for each particle size, an estimation of the removal efficiency of a detention basin for each pollutant may be obtained. Although some information on pollutant particle sizes have been obtained by Sartor and Boyd (1972), the amount of information available on this subject is still very limited.

2.2.4 Predicting Stormwater Detention Basin Performance

There are many methods for estimating the pollution control capability of detention basins and ponds. The majority of the methods may be grouped into one of the following 3 categories:

(1) Trap Efficiency Curves.

(2) Statistical Techniques

(3) Simulation Models

These three techniques are briefly discussed in the following sections.

2.2.4.1 Trap Efficiency Curves

Various trap efficiency curves have been developed by Brown (1943), Brune (1953), and Churchill (1948). Brown's curve is developed from an equation that relates sediment trap efficiency to the detention basin volume and drainage area. Brown based his equation on data collected from over 25 normally-ponded reservoirs. A more refined set of curves has been developed by Brune. These curves were based on data collected from 44 normally-ponded reservoirs located in twenty different states. Rather than basing sediment removal on a volume-drainage ratio, Brune based his curves on the volume-annual inflow ratio. This ratio provides a rough indicator of detention capability but cannot be defined as an average annual residence time. Finally, a method by Churchill relates the percentage of sediment passing through a reservoir to a sediment index for the reservoir. The sediment index is a function of reservoir volume, average runoff event flow rate, and the average cross sectional area of the reservoir.

2.2.4.2 Statistical Techniques

A second approach used in estimating the pollution control capability of detention basins involves the use of statistical techniques. This approach, commonly referred to as the derived distribution approach, is based on the probability distributions of different storm variables such as runoff, overflow, and pollutant load.

One of the first authors to use the derived distribution approach was Howard (1976). He assumed that the runoff volumes from a storm event and the intermittent times between storms are exponentially distributed. These random variables were used in deriving analytical expressions for overflows and related variables. In a separate study, Di Toro and Small (1979) have proposed a derived distribution for stormwater overflows. The flows are assumed to be uniform over the duration of the rainfall event. Flows, duration, and intermittent time are assumed to be gamma distributed. In a more recent study, Smith (1980) takes into consideration the duration of storms. The storm volumes, duration and intermittent time are assumed to be exponentially distributed. The storage level in the detention basin is also assumed to be a random variable.

In another study, Schwarz and Adams (1981) also assume exponential distributions for storm volumes, duration and intermittent time. This paper provides analytical expressions for spill volumes from two detention storage reservoirs in series. Finally, Loganathan and Delleur (1982) have derived distribution functions for overflows and receiving body pollutant concentration level. Storm volume, duration, and intermittent time are assumed to be exponentially distributed. The receiving stream quality is assumed to have a beta distribution while the stream volume is assumed to have a gamma distribution.

2.2.4.3 Simulation Models

A third approach to evaluating the removal efficiency of detention basins is through the use of simulation models. One of the first stormwater models to include the capability to model detention units was the stormwater management model (SWMM) (Huber et al., 1975). Detention basins are considered in the storage/treatment block of the model. Basins may be modeled as completely mixed or plug flow reactors: intermediate (arbitrary) modes are not available. A detailed description of the SWMM storage/treatment block is provided by Huber et al. (1981).

Another early simulation model that considers stormwater treatment is STORM (USACE-HEC, 1976). In using STORM, runoff is routed to storage-treatment facilities where runoff less than or equal to the treatment rate is treated and released. Runoff exceeding the capacity of the treatment plant is stored for treatment at a later time. If storage is exceeded, the untreated excess is wasted through overflow directly into the receiving waters. Pollutant removal may be determined through the use of exponential removal equations.

Ward et al. (1977) have developed a conceptual model which simulates the sedimentation process in reservoirs and sediment detention structures. The DEPOSITS model estimates the trap efficiency of a basin and simulates sediment outflow concentrations as a function of basin geometry, sediment physical properties, inflow hydrograph, inflow sedimentgraph, and basin hydraulic characteristics. In comparing the predicted results of the model with the results of several case studies, the model was shown to be consistently accurate.

Many of the concepts of the DEPOSITS model have incorporated into the USGS DR3M-QUAL model (Smith and Alley, 1982). In addition to determining the efficiency of sediment removal, the DR3M-QUAL model can be used to simulate the basin removal of other pollutants. This is accomplished by assigning a particle size distribution to the influent concentration of each water quality constituent that is entering the basin. Although particle size distributions of the influent should vary throughout a storm event, no data presently exist to quantify this effect accurately, therefore, for a given constituent and detention basin, the particle size distribution of the influent is assumed to remain constant.

A similar, but much simpler model has been presented by Ferrara (1982). This algorithm uses a time variable mathematical model of the conservation of mass equation for individual pollutants. The model requires the particle size distribution for each constituent as well as the settling velocities for each particle size fraction. Based on the results of the model, pollutant removal diagrams can be constructed which may be used in the design of stormwater quality control facilities.

Finally, using a more extensive analytical approach, Medina et al. (1981) have derived several models based on the principle of conservation of mass to represent the movement, decay, storage, and treatment of stormwater pollutants and dry weather flows through natural and engineered transport systems. The developed models have been derived to describe the transient response of storage/treatment systems to highly variable forcing functions of flow and concentration, for completely mixed systems of constant and variable volumes and for one-dimensional advective-dispersive systems. The developed models have been successfully applied to an actual urban watershed in Des Moines, Iowa, and to its receiving water.

In addition, the well-mixed constant volume model was found to describe closely the performance of the Humboldt Avenue detention tank, Milwaukee, Wisconsin when compared to measured effluent BOD mass rates.

2.2.5 Modeling Studies

Current theory is not adequate to formulate models independently to simulate the quality of flow from an urban watershed. In addition, data bases that represent hydrologic and hydraulic conditions, especially systems involving detention structures, are incomplete and not sufficient for formulating reliable empirical models. Kamendulski and McCuen (1979) have suggested that a combination of existing theory and available data would seem to represent the most feasible alternative towards the modeling of a detention basin system. In light of this conclusion, several authors have developed various models to be used in analyzing the quality characteristics of detention basins.

One of the first group of authors to examine the use of dual purpose detention basins was Curtis and McCuen (1977). They studied the performance of a detention basin by using a linked system hydrograph simulation model which included erosion, sedimentation, and detention components. The erosion component is based on a modification of the Universal Soil Loss Equation (Meyer and Wischmeier, 1969) and the sedimentation component is based on Camp's (1945) original work on sedimentation. In applying the model to the Manor Run watershed in Montgomery County, Maryland, Curtis and McCuen showed that detention basin location, basin riser characteristics, and storage volume were important in determining the design efficiency of stormwater management basins.

In a continuation of the previous work, Kamendulski and McCuen (1979) used the linked system hydrograph simulation model to evaluate alternative stormwater management policies for different detention basin configurations. Optimal designs for the different basins were obtained by adjusting the surface area dimensions and the height and diameter of the risers. As a result of this analysis, several conclusions were reached. The inflow runoff volume was shown to have a significant effect on both peak discharge reduction and sediment trap efficiency. In addition, sediment trap efficiency was shown to be dependent on the volume of inflow sediment and the volume of the basin storage.

In a separate study, Davis et al. (1978) analyzed the results of a monitoring program for a sediment basin that served a 45 acre subwatershed in Montgomery County, Maryland. In analyzing the data, two measures of basin efficiency were used: the peak reduction factor, and the trap efficiency of sediment and water quality parameters.

As a result of the analysis, regression relationships were obtained for sixteen water quality parameters as a function of the peak flows into and out of the basin. Davis et al. concluded that basin design criteria should be different for the control of stormwater flow rates and water pollution control. While the riser characteristics are important for stormwater flow rate control, the flow length and detention time were shown to be more important in water pollution control.

In an extension of this study, McCuen (1980) used the developed regression relationships to develop a methodology to predict basin trap efficiencies as a function of the peak reduction factor and the peak flow into the basin. McCuen also examined the effect of storm frequency on trap efficiency and concluded that as the volume of runoff increases, which occurs as either the return period or storm duration increases, the trap efficiency decreases.

In a separate study, Ward (1979) developed regression relationships for trap efficiency for both wet and dry basins using the results from the DEPOSITS model for 9 different ponds and reservoirs. The developed regression equations are a function of the basin capacity, the inflow volume, a weighted average detention time, storm duration, peak outflow rate, peak inflow rate, and the sediment particle size distribution. The developed regression models explained 94% of the variation in basin trap efficiency.

The regression equations were used to predict the efficiencies of two stormwater detention basins with very good results.

2.2.6 The National Urban Runoff Program

Although the previous methodologies have begun to examine the basic processes involved in the treatment efficiency of stormwater management basins, application of the majority of the methods is limited in that they are either site specific or they require data that are not readily available. The unavailability of needed data is partially being overcome through the results of EPA's Nationwide Urban Runoff Program (NURP). NURP was initiated in 1978 and resulted in a total of 28 local projects which had work plans designed to assess the nature, cause, and severity of urban runoff pollution and ways of addressing the problem. Of the 28 local projects involved with NURP, 9 are evaluating detention basins in detail (USEPA, 1982). Data analysis for each of the projects is being performed by the individual project sponsors and by a NURP headquarters team. The basic objective of the analysis of the NURP team is to provide a basis for establishing first order design considerations in terms of receiving water quality effects. Two of the major concerns of the NURP analysis are a determination of a performance function and the application density for a detention basin system. Preliminary conclusions have suggested that, overall, long term average performance is the most appropriate measure of performance to consider (Driscoll, 1982).

Although a summary report of NURP is due in Congress in 1983, preliminary results of several projects have recently been reported in various publications. Preliminary results for 5 major pollutants (COD, TSS, Total P, Pb, and Cu) indicate that runoff concentrations are highly variable. Mean event concentrations have been shown to be log-normally distributed. Analysis of data from NURP monitoring efforts on particle size distributions and settling tests, indicates that while results are quite variable, median settling velocities in the urban runoff range roughly between 0.5 and 20 feet/hour, with the bulk of the data showing medians between about 1 and 5 feet/hour (USEPA, 1982).

Initial results for the NURP Pittsfield Ann Arbor detention basin project have recently been reported by Scherger and Davis (1982). The detention basin drains 66.7 acres of mixed residential (29%), commercial (16%) and parkland (55%) land uses. The reported results were based on 14 storm events with rainfall depths ranging from 0.14 to 0.96 inches and maximum hourly intensities ranging from 0.04 to 0.75 inches/hour. The detention basin was concluded to be very effective in retaining suspended sediment, total phosphorus, and lead. Removal of suspended solids and lead exceeded 70% for rainfall depths ranging from 0.4 to 1.0

inch. Phosphorus generally exceeded 50%, while total Kjeldahl nitrogen removal ranged from 30 to 50%.

Results of another NURP detention project in Lansing Michigan have been reported by Luzkow et al. (1981). This detention basin drains 4872 acres of mixed residential (45%), commercial (15%), industrial (4%), parkland (23%), and agricultural (13%) land uses. The reported results were based on six rainfall events with rainfall depths ranging from 0.25 to 2.25 inches. Removal efficiencies for total suspended solids, total phosphorus, total Kjeldahl nitrogen, total iron and total lead were found to range as follows respectively: 12-85%, 9-82%, 0-30%, 7-72%, and 12-90%.

Results for three additional NURP detention projects near Washington D.C. have been reported by Grizzard et al. (1982). Two of the basins are wet detention basins while the other is a dry detention basin. The results of the detention basin study indicate that wet ponds are much more effective in removing nitrogen and phosphorus than are dry ponds. No conclusion could be made with respect to suspended solids removal, however, because one wet basin achieved 87% TSS removal while the other accomplished only 37%. The dry basin was intermediate with a TSS removal of 77%.

Results from a project in Glen Ellyn, Illinois have recently been presented by Hey (1982). The Glen Ellyn detention project involves the analysis of a small urban lake. Lake Ellyn has an area of 10.2 acres and drains 534 acres of mixed residential (83%), commercial (5%), institutional (5%), and parkland (5%) uses. Results from 95 storm events were reported. Close to 90 percent of the suspended solids entering the lake are retained. Heavy metals are reduced by approximately 80 percent. Other contaminants such as BOD and TOC are reduced by a much smaller percent, in the range of 10 to 25 percent.

III. DETENTION BASIN PLANNING METHODOLOGY

3.1 INTRODUCTION

In deriving a general detention basin planning methodology, there are several design considerations that should be addressed which are usually neglected in the design of an individual basin. These are summarized below.

1. Consideration of the impact of the frequency of the design runoff event on the individual basin design.

2. Consideration of both quality and quantity objectives in the overall design.

3. Consideration of the watershed impact of the placement of the detention basins.

All three of the above design considerations are addressed in the following sections followed by the development of the general planning methodology.

3.2 STOCHASTIC CONSIDERATIONS

The hydrologic input for any deterministic stormwater model is actually stochastic. The random nature of this input may be considered by three different approaches, the design storm approach, continuous simulation, and the derived distribution approach. All three of these approaches are discussed briefly in the following sections.

3.2.1 The Design Storm Approach

A given rainfall event is actually a combination of 2 different random variables, rainfall intensity and duration. Rainfall intensity normally increases with frequency and decreases with increasing storm duration. Point rainfall in a given geographic area may be used along with the above relationship to derive intensity-duration-frequency curves. Such curves are commonly used in various hydrologic design methods such as the Rational Method. For a selected frequency of occurrence and a specific storm duration, the corresponding average intensity may be obtained.

This approach assumes that the rainfall intensity for a given storm remains the same for the duration of the storm. In order to be more realistic, several authors have investigated the intensity distributions of various rainfall events. Based on these investigations, synthetic distributions have been derived. One of the first synthetic distributions to be derived was by Keifer and Chu (1957) for the city of Chicago. The U.S. Department of Agriculture, Soil Conservation Service (1973) has developed 24-hour rainfall distributions and a 6-hour distribution for use in developing runoff hydrographs. Huff (1967) divided recorded storm distribution patterns from small midwestern watersheds into four equal probability groups from the most severe (first quartile) to the mildest (fourth quartile). Thus for a given quartile and frequency, a specified rainfall volume can be distributed based on a selected distribution.

In the past, most hydraulic designs have been based on a design storm approach in which a structure is designed based on a synthetic storm is derived for a specified frequency and storm duration. In constructing the synthetic storm, a uniform or variable distribution may be assumed. In recent years, the design storm approach has been criticized for various reasons. One of the main drawbacks of the approach is that the resulting runoff event is assumed to have the same frequency of occurrence as the selected rainfall event. Another drawback of the design storm approach is that the areal variability of rainfall is typically ignored. Finally, design storms do not yield probability information such as flow duration curves that may be needed for planning purposes. Specifically, for stormwater management procedures which involve the storage and treatment of runoff, the probability distribution of the outflows and overflows becomes a function of the storage capacity and treatment rate. As a result, design storms are not applicable to the determination of nonpoint source pollutant loads and pollutant concentrations in receiving streams (Delleur, 1979).

3.2.2 Continuous Simulation

The main alternative to the use of the design storm approach is continuous simulation. Using historical rainfall data or a synthetic time series model, the response of the watershed over time may be determined. Such an approach is beneficial in that the change and effect of antecedent conditions and the effect of previous storms on storage structure performance can be determined. In using continuous simulation, the actual frequencies of the runoff events can be obtained and the critical storms can be identified. The runoff results from a simulation model could then be input into an optimization program. The optimization results would thus be optimal for the entire design period instead of for a single storm.

The principal limitation of a continuous simulation approach, especially when applied to an optimization problem, is the large number of variables and the resulting cost of long computer runs. As a result, several authors have proposed techniques to improve the design storm approach which consider the statistics of the continuous series. Walesh et al. (1979) present a technique using historical storms in which the major rainfall events are screened from the hydrologic time series. The major storms are then analyzed using an event model to obtain the resulting runoff events. The resulting hydrographs are then used in a discharge-probability and volume-probability relationship. This technique takes advantage of the low cost of the event models while eliminating the need to select a design storm. The results of this type of analysis may then be compared with the results of several design storms to select an appropriate design rainfall event.

In a separate study, Goforth et al. (1981) apply a continuous simulation model to a 26 year precipitation record. The computer program SYNOP (Hydroscience, Inc., 1979) is then applied to the results to obtain statistics for the runoff events. A single year of the record is then selected that has comparable statistics to the entire 26 year period. The single year record is then used in subsequent simulations.

3.2.3 Derived Distribution Approach

A more recent approach to stormwater modeling, especially from a quality standpoint, is the derived distribution approach. This method is based on the statistical distributions of different storm variables. Using hydrologic relationships, distributions are derived for the dependent variables, such as runoff and overflow. Due the hydrologic simplifications involved in most derived distribution approaches, the developed methodologies are generally only applicable on a macro planning level.

3.3 CONSIDERATION OF WATER QUANTITY-QUALITY OBJECTIVES

In order to optimize the design and placement of detention basins in an urban watershed it is necessary to model the hydrologic response of the watershed. The last decade has seen the development of several urban watershed simulation models. In order to consider the interaction of both water quantity and water quality objectives effectively, it is necessary to use a model that simulates both water quantity and water quality processes. Four models that do consider both processes are STORM, DR3M-QUAL, HSPF, and SWMM III. Each model is discussed briefly in the following sections.

3.3.1 STORM

STORM was developed for the U.S. Army, Corp of Engineers, Hydrologic Engineering Center (1976). The program was originally developed to analyze runoff quantity and quality from urban watersheds as part of large scale planning. It is intended to aid in the selection of storage and treatment facilities to control the quantity of stormwater runoff and land surface erosion. Conceptually, the runoff and pollutant washoff from an urbanized watershed may be collected and transported to a treatment facility, conveyed to temporary storage or discharged to receiving waters.

3.3.2 DR3M-QUAL

The second version of the USGS Distributed Routing Rainfall-Runoff Model (DR3M-QUAL) consists of two separate programs, one for rainfall-runoff simulation and the other for runoff quality simulation (Smith and Alley, 1982). The quality program can consider three different sources for water quality constituents: impervious and pervious areas, runoff contributions, and precipitation contributions. A daily accounting of constituent accumulation on impervious areas is maintained between storms. The quality component of the model can be applied on either a lumped or a distributed parameter basis. Pollutant removal can be simulated by use of a particle trapment model similar to DEPOSITS.

3.3.3 HSPF

The Hydrological Simulation Program (HSPF) is the FORTRAN successor of the Stanford watershed model (Crawford and Linsley, 1966). It is a continuous simulation model extended to include water quality constituents. The kinematic wave method is used to obtain subshed flows and to perform channel routing. Empirical equations are used to estimate the runoff quality parameters. HSPF is a modular program which performs deterministic simulations of a variety of hydrologic processes. One of the modules of particular interest in urban hydrology is the nonpoint source (NPS) model. NPS is a continuous simulation model of the generation of pollutants from pervious and impervious land surfaces. NPS simulates the surface and subsurface hydrologic processes, pollutant accumulation, and pollutant transport for any selected period of input meteorologic data.

3.3.4 SWMM III

Version III of the Storm Water Management Model (SWMM) has recently been released by the U.S. Environmental Protection Agency. The new version has been extensively modified and improved (Huber et al. 1981). The principal changes in Version III include continuous simulation, revised storage/treatment routines, revised surface quality generation and an updated scouring-deposition routine in the transport block.

3.3.5 Selection of a Simulation Model

Although all four models can simulate both quantity and quality processes, STORM cannot simulate the storm sewer network of a watershed. DR3M-QUAL is relatively new but is not as comprehensive as SWMM III. SWMM III is generally easier to use than HSPF and has been tested and updated over the last ten years. In addition, SWMM III can use National Weather Service rainfall tapes directly for use in continuous simulation. Because of this fact, and because of the availability of the model, SWMM III was selected as the watershed simulation model to be used in the overall planning methodology.

3.4 DETENTION BASIN DESIGN ALGORITHM

In order to consider the impact of the placement of detention basins on a watershed, some type of design algorithm is needed. Input to the general design algorithm may be provided by a watershed simulation program such as SWMM as discussed previously. While an algorithm that can be shown to yield global optimal solutions is generally desirable for most large scale design or operation problems, there are many instances when such algorithms are not feasible or even desirable. The general detention basin design problem tends to fall into the later category.

Many operational or design problems such as the general detention basin design problem are too complex for the relatively limited integer or mixed-integer optimization algorithms available. In fact, recent studies of computational complexity have suggested that many practical combinatorial problems, termed NP-complete cannot be solved efficiently by exact algorithms (Karp, 1975). Fortunately, however, it is now generally recognized that it is possible to systematically improve decisions without finding optimal solutions. One important way to do this is with heuristic problem solving procedures (Haessler, 1983).

Heuristics are simple procedures that are meant to provided good but not necessarily optimal solutions to difficult problems easily and quickly. There are several instances where the use of a heuristic is desirable and advantageous. One such use is the detention basin design problem. First of all, the data used in analyzing a watershed system may be inexact or limited and thus the resulting model parameters may contain errors much larger than the suboptimality of a good heuristic. Secondly, because of the complexity of the system, some degree of simplification is required to make the problem tractable. Thus, the use of a simplified form of the original problem may make any optimal solution only academic.

In deriving a general design heuristic, the overall problem is first formulated as a general mathematical program. Several optimization techniques are then examined for possible use in the general algorithm. A conceptualization of the general problem is first presented below.

3.4.1 Problem Conceptualization

The general watershed detention problem may be conceptualized as shown in Figure 3.1. The watershed may be thought of as consisting of I watershed segments. Two types of watershed segments are possible; external segments and internal segments. External segments (ex. 2, 4, 5) correspond to subsheds which are located at the outer edges of a watershed. Internal segments (ex. 1, 3) correspond to subsheds which receive drainage and pollutants from external subsheds and or other internal segments. Both rainfall excess E_{it} and K different pollutant loads L_{ikt} may exit each subshed i during time t.

External segments have one potential detention basin location at the outlet of each subshed. Internal segments have an associated channel reach and two potential detention basin locations, one upstream and one downstream. The upstream basin for any segment is assigned a subscript of $j=2$ while the downstream basin for any segment is assigned a subscript of $j=1$. Detention basin variables associated with external segments will thus have a subscript of $j=1$ while detention basin variables associated with internal segments will have subscripts of $j=1$ or $j=2$. Associated with any detention basin location at time t is a basin storage S_{ijt}, basin depth H_{ijt} and a principal spillway pipe diameter D_{ijn} where N different pipe diameters are available. The flowrate Q_{i2t} and pollutant load P_{i2kt} exiting any upstream detention basin i2, during time t, may be routed through the associated internal segment channel i, to produce a new flowrate R_{it} and pollutant load M_{it}. This new flowrate and pollutant load may then routed through the associated downstream basin i1 to produce a new flowrate Q_{i1t} and

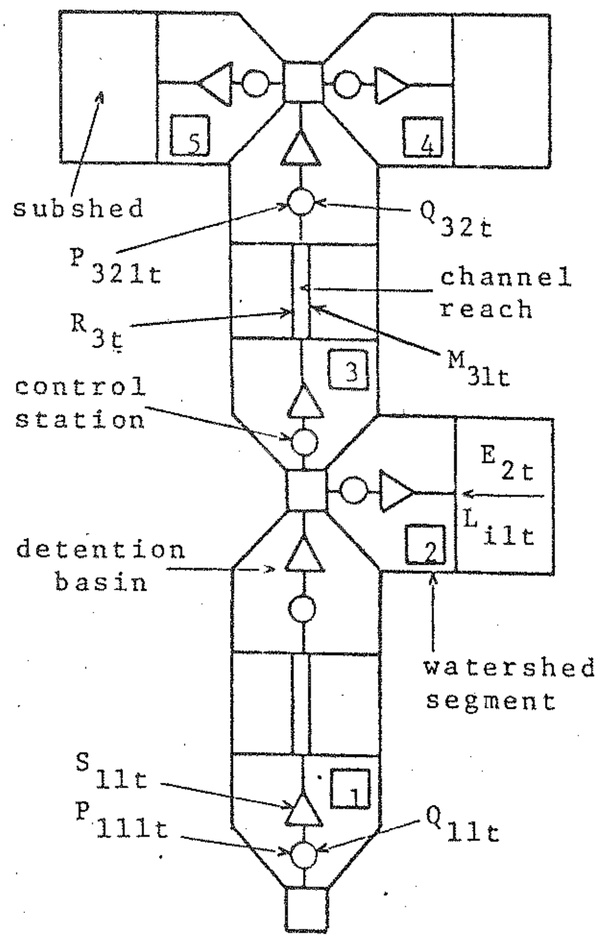


Figure 3.1 Watershed Conceptualization

pollutant load P_{ilt}. Immediately downstream of any detention basin location is a potential control station. These stations represent points where both flowrate $QMAX_{ij}$ and pollutant $PMAX_{ij}$ restrictions may be applied.

3.4.2 Problem Formulation

The basic objective of the optimization problem is to determine the locations and sizes of selected detention basins so as to minimize the overall design cost of the system while satisfying both water quantity (flowrate) and water quality (pollutant load) objectives at specified control stations. The general problem involves two different levels of optimization: the optimal design of the individual basins, and the optimal location of the individual basins within the watershed. The optimization problem also involves three different objectives: minimization of local flooding, maximization of overall water quality, and minimization of the overall system cost. Thus, the proposed problem is a multiple objective problem that may contain linear or nonlinear constraints. Due to the problem of quantifying the costs associated with flood damages and water quality violations, a more tractable approach would be to treat the water quantity and quality objectives as explicit constraints and thus incorporate them into the constraint set of the problem. A formulation of the general optimization problem may be written as follows.

$$\text{Minimize} \quad \sum_{i=1}^{I}\sum_{j=1}^{2} [C_{ij}\alpha_{ij} + f_1(S_{ij}) + \sum_{n=1}^{N} f_2(D_{ijn})\phi_{ijn}] \qquad (3.1)$$

$$\text{Subject To:} \quad S_{ij} = \max(S_{ij1}, S_{ij2}, \ldots, S_{ijT}) \qquad \forall ij \quad (3.2)$$

$$\sum_{n=1}^{N} \phi_{ijn} = \alpha_{ij} \qquad \forall ij \quad (3.3)$$

$$\alpha_{ij} \in (0,1) \qquad \forall ij \quad (3.4)$$

$$\phi_{ijn} \in (0,1) \qquad \forall ijn \quad (3.5)$$

$$S_{ijt} \leq SMAX_{ij}\alpha_{ij} \qquad \forall ijt \quad (3.6)$$

$$H_{ijt} \leq HMAX_{ij}\alpha_{ij} \qquad \forall ijt \quad (3.7)$$

$$D_{ijn} \in (D_{ij1}, D_{ij2}, \ldots, D_{ijN}) \qquad \forall ijn \quad (3.8)$$

$$Q_{ijt} \leq QMAX_{ij} \qquad \forall ijt \quad (3.9)$$

$$P_{ijkt} \leq PMAX_{ij} \qquad \forall ijkt \quad (3.10)$$

[$\forall it$]

$$R_{it} = R(Q_{i2t}, D_{ijn}) \qquad (3.11)$$

$$Q_{i1t} = Q(S_{i1t}, E_{it}, R_{it}, D_{ijn}, H_{ijt}) \qquad (3.12)$$

$$Q_{i2t} = Q(S_{i2t}, Q_{i+11t}, Q_{i+21t}, D_{ijn}, H_{ijt}) \qquad (3.13)$$

[$\forall ikt$]

$$M_{ikt} = M(P_{i2kt}) \qquad (3.14)$$

$$P_{i1kt} = P(L_{ikt}, L_{i2kt}, I_{it}, R_{it}, Q_{i1t}, S_{i1t}, D_{ijn}, H_{ijt}) \qquad (3.15)$$

$$P_{i2kt} = P(P_{(i+1,i+2)1kt}, Q_{(i+1,i+2)1t}, S_{i2t}, D_{ijn}, H_{ijt}) \qquad (3.16)$$

$$S_{ij}, S_{ijt}, D_{ijn}, H_{ijt}, Q_{ijt}, R_{it}, M_{ikt}, P_{ijkt} \geq 0 \qquad (3.17)$$

where I = total number of watershed segments

J = number of detention basins per segment

K = total number of pollutants considered

N = number of spillway pipe diameters available

T = total number of time steps

α_{ij} = 1 if reservoir ij is built, 0 otherwise

ϕ_{ijn} = 1 if diameter n is used, 0 otherwise

C_{ij} = fixed cost for installing basin ij ($)

f_1 = cost function for storage

f_2 = discrete cost function for spillway pipe

S_{ij} = maximum storage required for basin ij (ft^3)

D_{ijn} = diameter n of set N of available spillway pipes for basin ij (ft)

S_{ijt} = storage required at basin ij at time t (ft^3)

$SMAX_{ij}$ = maximum allowable storage for basin ij (ft^3)

Q_{ijt} = flow released from basin ij at time t (cfs)

$QMAX_{ij}$ = maximum allowable discharge for basin ij (cfs)

H_{ijt} = depth of pool in basin ij at time t (ft)

$HMAX_{ij}$ = maximum allowable depth for basin ij (ft)

P_{ijkt} = mass of pollutant k released from basin ij at time t

$PMAX_{ij}$ = maximum allowable pollutant load released from basin ij

R_{it} = flow routed through reach i at time t (cfs)

E_{it} = runoff flowrate for subshed i at time t (cfs)

L_{ikt} = mass of pollutant k washed off subshed i at time t

M_{ikt} = mass of pollutant k routed through channel i at time t

The above formulation involves a nonlinear objective function subject to inequality constraints on both the decision variables (3.6-3.8) and the system variables (3.9-3.10). The decision variables for the problem are basin storage, basin height, and pipe diameter. The system variables for the problem are flowrate and pollutant load. The constraints numbered (3.11) through (3.16) represent the transformation functions for the general problem. These functions are used to obtain the values of the system variables for given values of rainfall excess E_{it}, pollutant load L_{ilt}, and the decision variables. These relationships are discussed in detail in the following sections.

3.4.2.1 Determination of Rainfall Excess

The two major input variables of the detention basin design problem are the subshed hydrographs and the subshed pollutant loadings. Transformation of rainfall into a subshed hydrograph involves two basic processes: determination of excess rainfall or runoff, and the rout ng of the rainfall excess over the subshed. Excess rainfall may be routed over the subshed using unit hydrograph theory or some type of nonlinear model. SWMM generates subshed hydrographs using a nonlinear reservoir model.

The determination of the rainfall excess involves the removal of various hydrologic abstractions from the total rainfall. Two of the most important abstractions are depression storage and infiltration. Depression storage is usually determined using some type of empirical equation. The following equation is used in SWMM.

$$D_s = 0.030\ S^{-0.49} \tag{3.18}$$

where D_s = depression storage (in)

S = catchment slope (percent)

Two of the more widely used infiltration models are Horton's equation and the Green-Ampt equation. The Horton equation has three different parameters and may be written as follows:

$$f_p = f_c + (f_o - f_c)e^{-kt} \tag{3.19}$$

where f_p = infiltration capacity into soil (ft/sec)

f_c = ultimate value of f_p (ft/sec)

f_o = initial value of f_p (ft/sec)

k = decay coefficient (1/sec)

t = time from beginning of storm (sec)

Unlike Horton's equation, the Green-Ampt equation uses physically based parameters which can be predicted a priori. The Green-Ampt equation may be written as follows:

$$f_p = k(h_o + h_c + l_f)/l_f \tag{3.20}$$

where f_p = infiltration capacity into soil (ft/sec)

h_o = depth of ponded water (ft)

h_c = capillary suction head (ft)

l_f = depth to wetting front (ft)

k = hydraulic conductivity (ft/sec)

3.4.2.2 Channel Routing Function

Functional relationship (3.11) represents the channel routing function. The output flowrate R_{it} is a function of the inflow Q_{i2t} and the storage in the channel. For the case of a pipe, the storage is a function of the pipe diameter D_{ijn}.

Flows through the channel may be routed by using either hydrologic routing techniques or hydraulic routing approaches. Hydrologic routing employs the equation of continuity with either an analytic or an assumed relationship between storage and discharge. The equation of continuity may be expressed as follows.

$$I - Q = \frac{ds}{dt} \tag{3.21}$$

where I = inflow rated to the reach (cfs)

Q = the outflow rate from the reach (cfs)

$\frac{ds}{dt}$ = the rate of change of storage (cfs)

Hydraulic routing techniques use both the equation of continuity and the equation of motion. The general form of the partial differential equations may be written as follows.

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{3.22}$$

$$\frac{\partial y}{\partial x} + \frac{\alpha V \partial V}{g \ \partial x} + \frac{1 \partial V}{g \partial t} + Sf - So = 0 \tag{3.23}$$

where

Q = discharge, (cfs)

A = water cross section, (ft^2)

V = velocity of flow, (ft/sec)

α = energy distribution coefficient

g = gravitational constant ($\frac{ft}{sec^2}$)

Sf = friction slope ($\frac{ft}{ft}$)

So = channel bed slope ($\frac{ft}{ft}$)

x = length along channel (ft)

y = water depth (ft)

t = time (sec)

Closed form solutions to the above equations do not exist. Thus the application of these equations requires computer operations to solve them numerically.

The majority of the available large scale stormwater models, such as SWMM, employ a simplification of the above equations. The simplification is obtained by assuming a balance between gravitational and friction forces. The resulting flow is called kinematic. This means that the derivatives in the momentum equation are negligible when compared to the effect of gravity and the effect of friction. Thus the friction gradient can be equated to the channel bed slope. The resulting equations may be written as follows.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3.24}$$

$$Q = a\, y^{b} \tag{3.25}$$

where

A = water cross section, (ft^2)

Q = discharge, (cfs)

a = coefficient

b = coefficient

x = length along channel (ft)

y = water depth (ft)

t = time (sec)

A further simplification of the kinematic equation results in the Muskingum method. This method is based on the following relationship.

$$S = K\,[\,xI + (1-x)Q\,] \tag{3.26}$$

where K = storage time constant

x = weighting factor between 0.0 and 0.5

I = the inflow into the reach (cfs)

Q = the outflow from the reach (cfs)

S = the storage of the reach (ft^3)

Substitution of the Muskingum relationship into the continuity equation yields the following general relationship.

$$Q_t = C_0 I_t + C_1 I_{t-1} + C_2 Q_{t-1} \tag{3.27}$$

where

$$C_0 = \frac{-Kx + 0.5 \Delta T}{K - Kx + 0.5 \Delta T} \tag{3.28}$$

$$C_1 = \frac{Kx + 0.5 \Delta T}{K - Kx + 0.5 \Delta T} \tag{3.29}$$

$$C_2 = \frac{-K - Kx - 0.5 \Delta T}{K - Kx + 0.5 \Delta T} \tag{3.30}$$

thus

$$Q_{i2t} = C_0 R_{i2t} + C_1 R_{i2t-1} + C_2 Q_{i2t-1} \tag{3.31}$$

3.4.2.3 <u>Basin Routing Function</u>

Functional relationships (3.12) and (3.13) are the detention basin routing functions for both upstream and downstream basins in a given watershed segment. The outflow from a basin (Q_{ijt}), is a function of the inflow (E_{it}, R_{it}) or (Q_{i+1jt}, Q_{i+2jt}), basin storage (S_{ijt}), and a stage-discharge relationship. When a principal spillway pipe is employed, the stage-discharge relationship is a function of the depth of the basin (H_{ijt}), and the principal spillway pipe diameter (D_{ijn}).

Although flows can be routed through a basin by using either a hydrologic or hydraulic approach, the most common approach is the hydrologic method. In this case, the continuity equation is used along with a relationship for discharge and storage. The continuity equation may be written in finite difference form as follows.

$$\frac{(I1 + I2)}{2} - \frac{(Q1 + Q2)}{2} = \frac{(S2 - S1)}{\Delta T} \qquad (3.32)$$

Where I is inflow (cfs), Q is outflow (cfs), S is storage (ft^3), and ΔT is the time step (sec). Rearranging the equation with the unknown terms on the left yields.

$$Q_2 + S_2C = I_1 + I_2 - Q_1 + S_1C \qquad (3.33)$$

where $\qquad C=\frac{2}{\Delta t}$

The above equation involves two unknowns, Q_2 and S_2, and can be solved with an additional relationship between Q and S

Discharge from a reservoir is a nonlinear function of the height of the pool above the spillway crest or the total height of the pool above the outlet elevation of a principal spillway pipe. In addition, discharge is also a function of the geometry of the spillway or the diameter of the spillway pipe.

As flow is routed through a reservoir with a principal spillway pipe, the outflow through the pipe may pass through three different flow regimes. The flow in each regime is related to the height of the normal pool by the following relationships;

Weir Flow

$$Q = Cw\ H^{\frac{3}{2}} \qquad (3.34)$$

Orifice Flow

$$Q = Co\ H^{1/2} \qquad (3.35)$$

Full Pipe Flow

$$Q=Cp\ H^{1/2} \qquad (3.36)$$

where Q = discharge (cfs)

C = discharge coefficient

H = effective stage (ft)

These relationships may be illustrated on a plot of Q vs H as shown in Figure 3.2. Because the storage in a reservoir is also usually a nonlinear function of the stage of the pool, discharge is a nonlinear concave function of storage. Because the relationship between storage and discharge is nonlinear, the solution for flowrate and storage at each time step requires an iterative scheme such as the Newton-Raphson method. Using this approach, the continuity equation is written as

$$X(h) = Y \qquad (3.37)$$

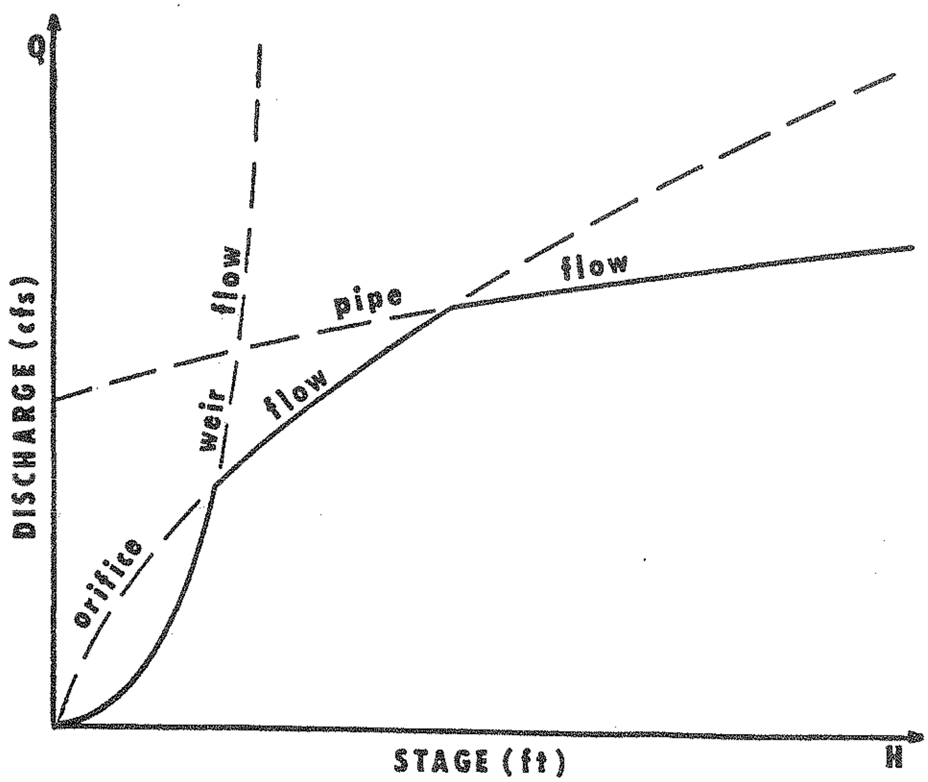


Figure 3.2 Stage-Discharge Relationship

where $$X(h) = Q_2 + S_2C \tag{3.38}$$

$$Y = I_1 + I_2 - Q_1 + S_1C \tag{3.39}$$

If we let $$F(h) = X(h) - Y = 0 \tag{3.40}$$

Then we can solve for the basin depth h by using a iteration scheme with the following relationship:

$$h_{new} = h_{old} + \frac{F(h_{old})}{F(h_{old})} \tag{3.41}$$

where $$F(h) = X(h) - 0 \tag{3.42}$$

Once the basin depth for time 2 has been determined the corresponding flowrate (Q_2) and storage (S_2) can be determined from known relationships of $Q(h)$ and $S(h)$.

One simplified approach for determining the discharge and the storage would be to approximate the continuity equation using a fourth order Runge-Kutta approximation. Using this approach, the continuity equation may be written

$$\frac{ds}{dt} = I(t) - Q(S) \tag{3.43}$$

or in general, $f(S,t) = I(t) - Q(S)$. This equation may be solved by employing the following set of equations.

$$S_{t+\Delta t} = S_t + \frac{\Delta t}{6}[\, k_1 + 2k_2 + 2k_3 + k_4 \,] \tag{3.44}$$

where $k_1 = f(S_t, t)$ (3.45)

$$k_2 = f(S_{t+0.5\Delta t}, t+0.5k_1) \quad (3.46)$$

$$k_3 = f(S_{t+0.5\Delta t}, t+0.5k_2) \quad (3.47)$$

$$k_4 = f(S_{t+\Delta t}, t+k_3) \quad (3.48)$$

The computational procedure involved in routing a hydrograph through a detention basin may be illustrated graphically as shown in Figure 3.3 (Kao, 1975). This figure contains four quadrants which relate to the basic computations involved in the routing. For a given outlet configuration a stage-discharge curve may be constructed as shown in quadrant 1. Likewise, for a given basin geometry, a stage-storage curve may be constructed as shown in quadrant 2. If the walls of the basin are vertical, then the relationship will be linear as shown. Given an inflow hydrograph as shown in quadrant 3, the cumulative amount of storage in the basin (assuming no initial outflow) at time t may be computed and then plotted in quadrant 4. Once the the storage at time t is known, a point on the outflow hydrograph may be obtained by moving around the diagram clockwise. Once this point is obtained, the initial storage estimate is updated and the procedure continued until convergence is achieved. Once convergence has been achieved, the entire process may be repeated until the entire hydrograph has been routed.

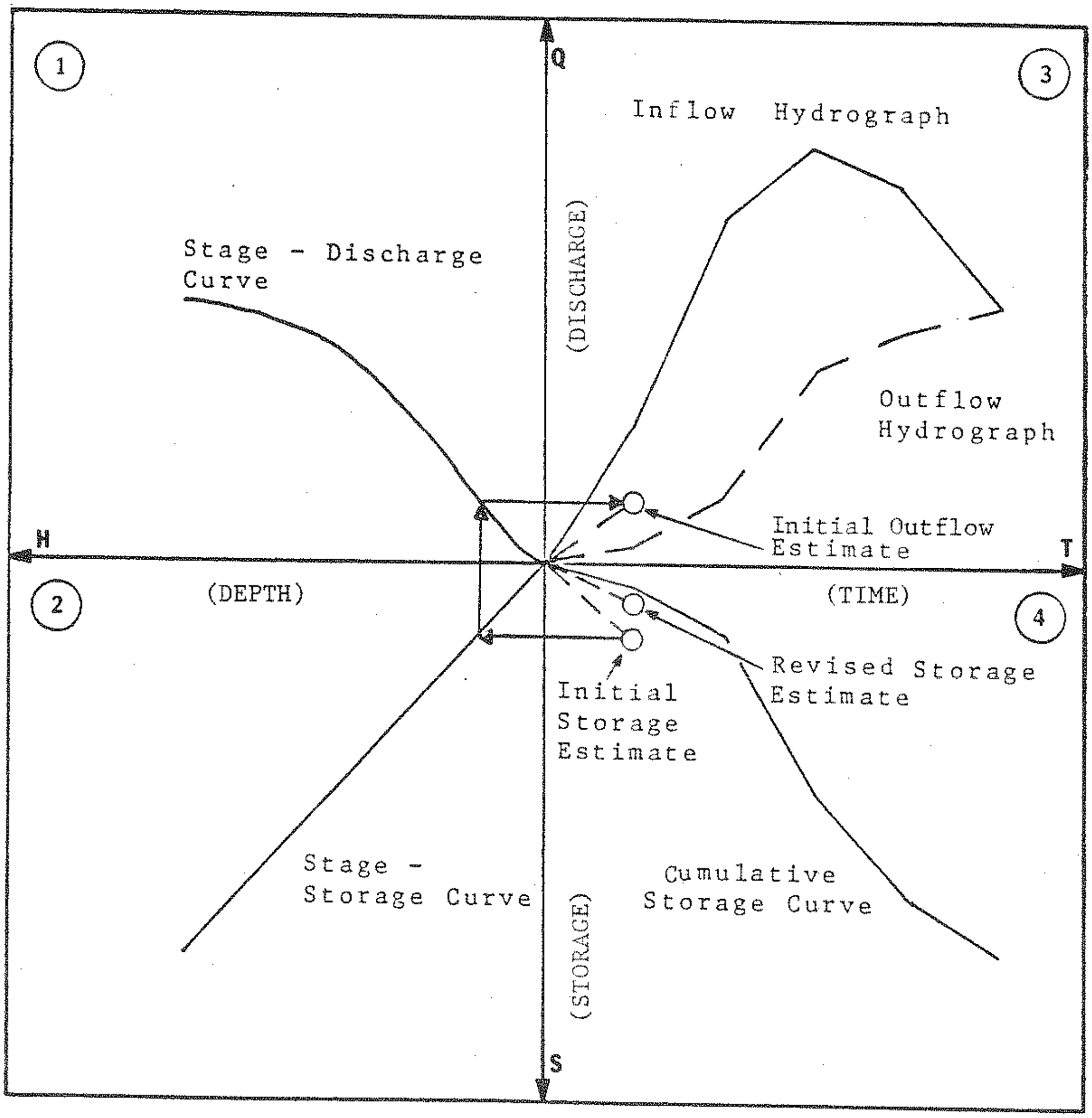


Figure 3.3 Graphical Detention Basin Routing

3.4.2.4 Determination of Pollutant Loadings

Several mechanisms are involved in the genesis of stormwater quality, most notably pollutant buildup and washoff. In an impervious urban area, it is usually assumed that a supply of constituents is built up on the land surface during dry-weather preceding a storm. Such a buildup may be a function of time, traffic flow, dry fallout and street sweeping. When a storm occurs the accumulated material is washed off into the drainage system. The physics of the washoff may involve both erosion and sediment transport mechanisms; however, in general, washoff is modeled using an empirical equation with slight physical justification.

Pollutant buildup may be modeled using linear or nonlinear accumulation equations. The simplest nonlinear accumulation function that has been found to fit street accumulation data is a two parameter model of the following form:

$$L_o = L_{max}(1 - e^{-kt}) \qquad (3.49)$$

where L_o = street accumulation (lbs)

L_{max} = maximum allowable accumulation (lbs)

k = rate coefficient (1/time)

t = antecedent period since rainfall (time)

Pollutant washoff is generally modeled using an exponential washoff equation of the following form:

$$L_t = L_o(1 - e^{-kt}) \qquad (3.50)$$

where L_t = cumulative pollutant washed off (lbs)

L_o = initial load on subshed (lbs)

k = rate coefficient (1/time)

t = time since start of storm (time)

3.4.2.5 Pollutant Routing Function

Relationship (3.14) represents the reach routing function for the pollutant load. If the pollutant load is neither increased nor decreased as the stormwater is routed through the reach, the load may be simply lagged by a time step equal to the average travel time of the reach for the specific pollutant. Thus,

$$M_{ikt} = P_{i2k(t-c_i \Delta t)} \qquad (3.51)$$

where c_i = integer constant for reach i

3.4.2.6 Pollutant Removal Function

The last two functional relationships, (3.15) and (3.16) represent the basin routing (removal) functions for the different pollutants. These relationships will reflect the removal efficiency of the basin for a given pollutant load and time step. Thus

$$P_{i1kt} = \lambda_{1t}(L_{ikt} + M_{ikt}) \quad (3.52)$$

$$P_{i2kt} = \lambda_{2t}(P_{i+11kt} + P_{i+21kt}) \quad (3.53)$$

where λ_{jt} = basin removal efficiency, between 0 and 1

As discussed in Chapter 2, the primary factor in the determination of the pollutant removal efficiency is the detention time. In general, the detention time of a basin is a function of the flow through the basin, the storage in the basin, and the stage-discharge relationship of the basin. Thus

$$\lambda_{1t} = f(\ E_{it},\ R_{it},\ S_{i1t},\ D_{i1n},\ H_{i1t}\ ) \quad (3.54)$$

$$\lambda_{2t} = f(\ Q_{i+11t},\ Q_{i+21t},\ S_{i2t},\ D_{i2n},\ H_{i2t}\ ) \quad (3.55)$$

Basin removal efficiency can be obtained using two different approaches. If the pollutants are characterized only by their magnitude then the removal efficiency may be determined using a removal equation. If an average removal efficiency λ is used, the following general removal equation may be written (Huber et al., 1975).

$$\lambda = \lambda_{max}(1 - e^{-K\ DT}) \quad (3.56)$$

where λ_{max} = maximum removal fraction

DT = detention time (sec)

K = first order decay coefficient (sec^{-1})

If the pollutant is characterized by its particle size, specific gravity or settling velocity then its removal may be expressed as follows.

$$\lambda_q = \frac{V_{sl}}{V_{ij}} \tag{3.57}$$

where V_{sl} = settling velocity of pollutant l

$$V_{ij} = \frac{H_{ij}}{DT_{ij}} \tag{3.58}$$

where H_{ij} = average depth of pool of basin ij

DT_{ij} = average detention time of basin ij

The above equation for λ_q represents the removal efficiency for ideal quiescent conditions. Non-ideal conditions can be approximated through the use of a turbulence factor α ($.01 \leq \alpha \leq 1.0$) and the following equation (Chen, 1975).

$$\lambda = \lambda_q + \ln\frac{\alpha}{4.605} (\lambda_q - \lambda_t) \tag{3.59}$$

where

$$\lambda_t = (1 - e^{-\lambda_q}) \tag{3.60}$$

and

$$\alpha = \frac{V_{sl} \; H_{ij}^{1/6}}{V_t \; n \sqrt{g}} \tag{3.61}$$

where V_t = flow through velocity (ft/sec)

n = basin roughness

g = gravitational constant (32.2 ft/sec^2)

3.4.3 Application of Linear/Mixed Integer Programming

The general detention basin optimization problem may be approached using several optimization techniques such as linear programming, mixed integer programming, nonlinear programming, and dynamic programming. In order to apply linear programming to the detention basin optimization problem, all nonlinear relationships must either be simplified using linear relationships or approximated using linear segments and zero-one variables. Inclusion of zero-one variables in the formulation requires the use of a mixed integer strategy such as branch and bound, cutting planes, or Bender's algorithm. Such algorithms usually employ some type of enumeration scheme in solving a series of individual linear programs. Although mixed integer programming is not the same as linear programming, it still requires the use of linear relationships for the continuous variables. Possible linearizations of the objective function and the transformation constraints are discussed below.

3.4.3.1 Objective Function

The objective function contains three different cost terms. While the first and last term may be directly incorporated into a linear program, the second term, which involves storage, is generally nonlinear and concave. One approach to this problem would be to use a linear cost function for storage. Alternatively, the concave cost function could be approximated using linear segments of the function, but, this would require a zero-one variable for each segment.

3.4.3.2 Channel Routing Function

Hydraulic routing techniques, although more accurate than hydrologic techniques, are highly nonlinear and cannot readily be incorporated into a linear program. An acceptable linear hydrologic routing technique that can be incorporated into a linear program is the Muskingum Method, as discussed previously.

A simplification of the Muskingum Method would be simply to use a constant lag for the entire hydrograph for an associated design option. The design option could be associated with the pipe diameter of the principal spillway pipe. Given the diameter of the pipe, the peak discharge velocity could be determined. Knowing the peak discharge velocity and the length of the channel, a travel time could be determined. The lag associated with a given design option could thus be incorporated into the formulation by simply offsetting the appropriate flow terms.

3.4.3.3 Basin Routing Function

Hydrologic reservoir routing techniques are based on the continuity equation and a relationship between discharge and storage. Although the continuity equation is linear and may be incorporated in a linear program, the relationship between discharge and storage in nonlinear and concave. If the sides of a detention basin are vertical then storage can be related to the height in the basin linearly as follows

$$S = A\ H \qquad (3.62)$$

where A = surface area of basin floor (ft^2)

H = depth of basin (ft)

By multiplying the individual heights H by the basin area A, a curve similar to the one in Figure 3.2 can be derived for Q vs S. By breaking the resulting stage-storage curve into discrete segments, the curve can be linearized as represented below and in Figure 3.4.

where:

$$Q_t = f(S) = \begin{cases} a_1 S_t & S_0 \leq S \leq S_1 \\ a_2 S_t + b_2 \quad \text{for} & S_1 \leq S \leq S_2 \\ a_3 S_t + b_3 & S_2 \leq S \leq S_3 \end{cases} \qquad (3.63)$$

thus $\quad Q_t = a_1\ S_t \quad$ or $\quad Q_t - a_1\ S_t = 0 \quad (3.64)$

$$Q_t = a_2\ S_t + b_2 \quad \text{or} \quad Q_t - a_2\ S_t = b_2 \quad (3.65)$$

$$Q_t = a_3\ S_t + b_3 \quad \text{or} \quad Q_t - a_3\ S_t = b_3 \quad (3.66)$$

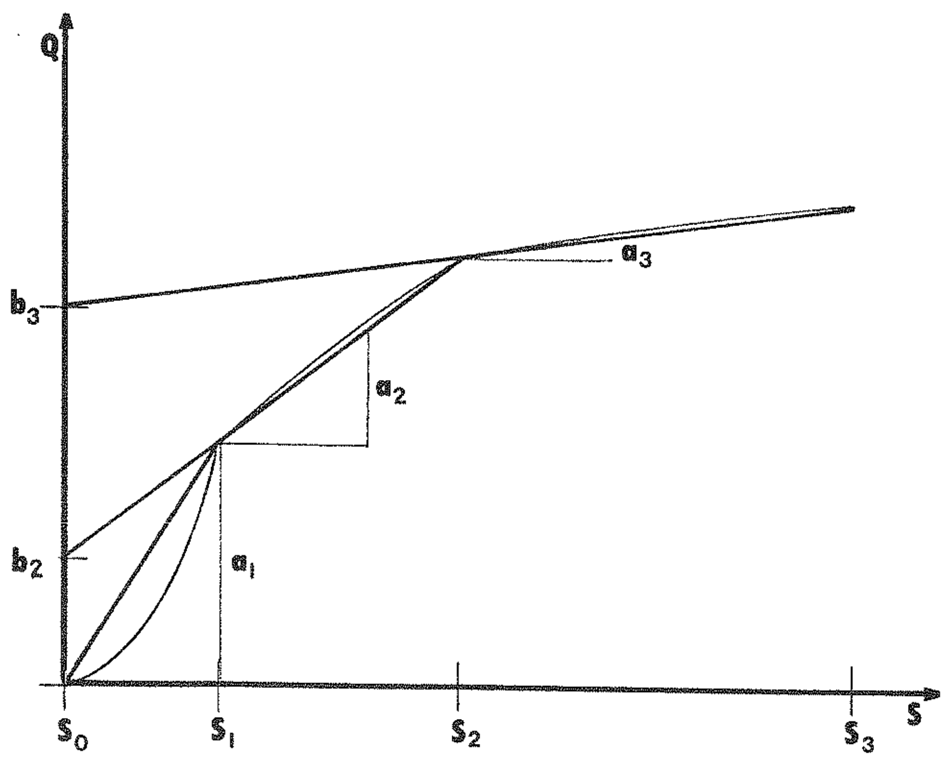


Figure 3.4 Linearized Storage-Discharge Relationship

Thus, the nonlinear nature of the reservoir routing function can be included in a linear formulation by incorporating the above constraint set for each reservoir. This set of equations could be used along with the continuity equation to route the flows through each reservoir. Because the C coefficients in the outflow equations are a function of the pipe geometry and diameter, a different set of constraints for each basin could be included for a range of different pipe diameters.

A partial simplification of the above formulation could be obtained by simulating the various designs of the exterior basins before applying the linear programming algorithm. The results of these simulations could then be incorporated into the formulation as linear constraints of the following form.

$$Q_{i1t} - \alpha_1 Q_{i11t} - \alpha_k Q_{i1kt} = 0 \qquad (3.67)$$

where Q_{i1t} is the outflow from basin i1 at time t, Q_{i1kt} are the outflows from basin i1 at time t for design option k, and α_k is a zero-one variable associated with design option k.

Because of the interaction of the hydrographs, the above simplification cannot be applied to the interior basins. One possible simplification involving the interior basins would be to simulate each basin as a flow retarding device by assigning maximum outlet flowrates to different design options. As long as the inflow does not exceed the design maximum, the flow passes through the basin unaffected. When the inflow does exceed the maximum, only the maximum flowrate is allowed and the excess is retained. This formulation can be written using the following two constraints.

$$I_{ijt} + S_{ijt} - Q_{ijt} - S_{ijt+1} = 0 \qquad (3.68)$$

$$Q_{ijt} - \alpha_1 Q_{ij1} - \dots - \alpha_k Q_{ijk} \leq 0 \qquad (3.69)$$

where I_{ijt} is the inflow into basin ij at time t, S_{ijt} is the storage in basin ij at time t, Q_{ijt} is the outflow from basin ij at time t, Q_{ijk} is the maximum discharge associated with option k, and α_k is a zero-one variable associated with option k.

3.4.4 Application of Nonlinear Programming

Instead of linearizing the problem using zero-one variables, the general formulation could also be approached directly using nonlinear programming. The general problem involves the minimization of a nonlinear objective function subject to nonlinear constraints. While highly efficient methods have been developed for optimizing nonlinear unconstrained functions, less progress has been made in the more practical area of constrained optimization. Most existing approaches to constrained optimization fall into one of the following four categories (Haarhoff and Buys 1969).

(1) Penalty Function Techniques

(2) Constraint Linearization Techniques

(3) The Constrained Fletcher-Powell Method

(4) The Box Complex Method

3.4.4.1 Penalty Function Techniques

Various penalty function techniques have been introduced by Rosenbrock (1960), Kelly (1962), and Lootsma (1967). These techniques deal with constraints through the incorporation of a penalty factor in the objective function. As long as the search remains in the feasible region the penalty factor is set equal to zero. However when a constraint is violated, the penalty factor is assigned a large value which forces the search back into the feasible region. While such methods may work fairly well at times, they all have the disadvantage that the inclusion of a penalty factor in the objective function tends to distort the shape of the response region and thus decrease the efficiency of optimization (Haarhoff and Buys, 1970).

An alternative penalty function formulation has been introduced by Carroll (1961). This method introduces a natural optimum within the feasible region such that the constraints are approached but never violated. This technique has been modified and formalized for minimization of a convex function by Fiacco and McCormick (1968), and in a more general form by Strong (1965). Because the constraints are never violated, the method can be used with an unconstrained optimization technique. This approach has been used in conjunction with the unconstrained method of Fletcher and Powell (1963) to yield fairly good results. However the method is not particularly precise when the optimum lies in a sharp corner.

3.4.4.2 Constraint Linearization Techniques

Various constraint linearization techniques have been introduced by Rosen (1961), Nel (1964), and Glass and Cooper (1965). All three techniques utilize the method of steepest ascent until a constraint is encountered. Once a constraint is encountered, successive linear search directions are chosen in such a way that the nonlinear constraints remain satisfied to a first order approximation. The inherent difficulty behind this approach is that a move which satisfies a linearized form of a constraint does not necessarily result in a move which satisfies the nonlinear inequalities. In dealing with this problem, the various methods employ corrective techniques which insure that a selected point will always be in the feasible region (Beveridge and Schechter, 1969). Kelly (1962) has shown that this process can lead to a decrease in computational efficiency when the constraints cannot be closely approximated by linear functions.

3.4.4.3 The Constrained Fletcher-Powell Method

The constrained Fletcher-Powell method has been described by Haarhoff and Buys (1970). The method incorporates the constraints into a modified, unconstrained objective function which is then optimized by the unconstrained minimization technique of Fletcher and Powell. Derivatives of the objective function are thus required. If the derivatives cannot be obtained analytically then they must be obtained numerically. Inequality constraints are converted to equality constraints by use of slack variables and transformations such that the slack variables will equal zero when the equality constraints are satisfied. While this process may not require much effort for simple mathematical expressions, it can become quite tedious for more complex problems (Kuester and Mize, 1973). Although the method has been shown to be more efficient than the method of Rosenbrock, Haarhoff and Buys failed to show that it was significantly better than the Complex Method of Box (1965).

3.4.4.4 The Box Complex Method

In contrast to many of the above methods, the Box Complex Method is conceptually simple, requires no derivatives, does not distort the region of search, and is directly applicable to problems involving nonlinear inequality constraints. In a comparative study in which all of the four approaches were used, Chu and Bowers (1975) concluded that the Box Complex Method was the most efficient.

The Complex Method of Box (1965) is based on the Simplex method of Spendley, Hext and Himsworth (1962). The method handles constraints by use of a flexible figure, called a complex, which can expand or contract in any or all directions and can extend around corners. The method not only allows for the inclusion of region restrictions of the form $g(X_i) \leq 0$, called implicit constraints, (where $g(X_i)$ is a function of the decision variables X_i), but also includes limits on the decision variables in the form $X_l \leq X_i \leq X_h$, called explicit constraints (Beveridge and Schechter, 1969).

In this method $k \geq n+1$ points are used, where n equals the number of decision variables. In order to generate the initial complex, an initial point must be given or determined that satisfies both the explicit and implicit constraints. The additional (k-1) points required to set up the initial complex are obtained one at a time by the use of pseudo-random numbers and ranges for each of the independent variables which are based on the explicit constraints. A point so selected will thus satisfy the explicit constraints but not necessarily all the implicit constraints. If an implicit constraint is violated, the trial point is moved halfway towards the centroid of those points already selected (where the given initial point is included). Ultimately a satisfactory point will be found. Proceeding in this way, the (k-1) points are found which satisfy all the constraints. Once the initial complex has been formed, further progress is made through expansion or contraction of the complex. These two operations can be visualized as follows.

At each stage of movement the objective function is evaluated at each of the points in the complex, and the vertex of the greatest function value determined. The complex is then expanded away from this worst point, P_h, through the centroid of the remaining points to yield a new point $\dot{P}$. Mathematically this may be written as

$$\dot{P} = (1 + \alpha)\bar{P} - \alpha P_h \qquad (3.70)$$

where α is the expansion coefficient and $\bar{P}$ is the centroid of all points excluding P_h. Thus $\dot{P}$ is on a line joining $\bar{P}$ and P_h, on the far side from P_h with $[\dot{P}\bar{P}] = \alpha[P_h\bar{P}]$. The objective function is then evaluated at this new point $\dot{P}$. If the new point yields a function value which is better than P_h then the point P_h is discarded and replaced by P. In this way the complex moves in the direction of function minimization (see Figure 3.5). If however the value of the new point is worse than P_h then the new point is contracted back toward the centroid and another new point is generated. This can be written as

$$\ddot{P} = \omega\ddot{P} + (1 - \omega)\overline{P} \qquad (3.71)$$

where $\ddot{P}$ is the new point generated and ω is the contraction coefficient (see Figure 3.6).

This dual process of expansion and contraction continues until some constraint is violated or some tolerance level reached. If an independent variable of a new point violates some explicit constraint then that variable is reset to a value just inside the constraint. If the new point violates some implicit constraints, then the point is moved halfway towards the centroid of the remaining points. Assuming the response surface is convex, a permissible point will ultimately be found. The search finally terminates when successive function evaluations have yielded the same result, indicating that the complex has collapsed into the centroid.

Box recommends using a value of 1.3 for the expansion coefficient. The use of an expansion factor greater than 1.0 tends to cause a continual enlargement of the complex

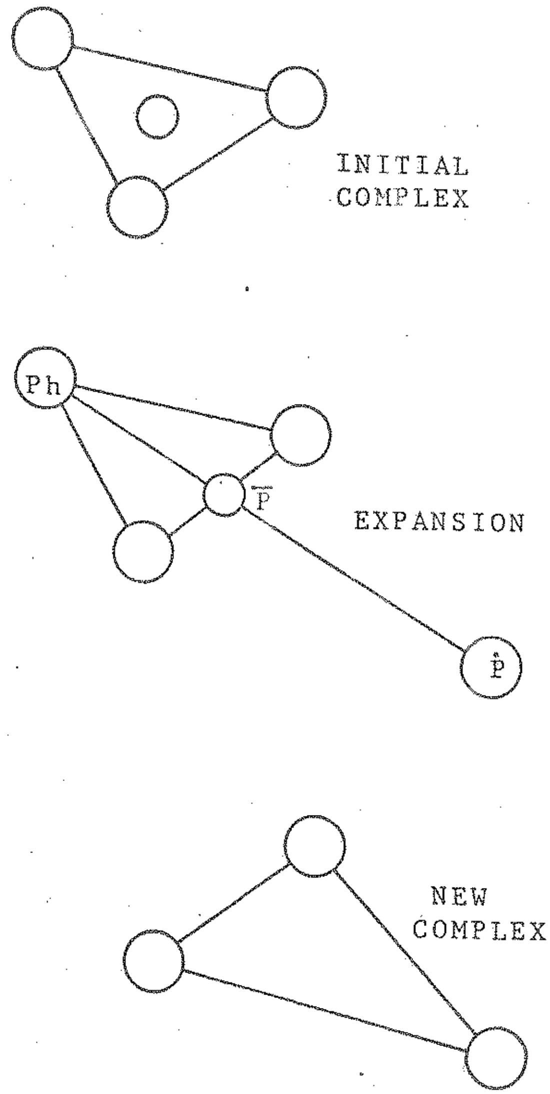


Figure 3.5 Complex Expansion

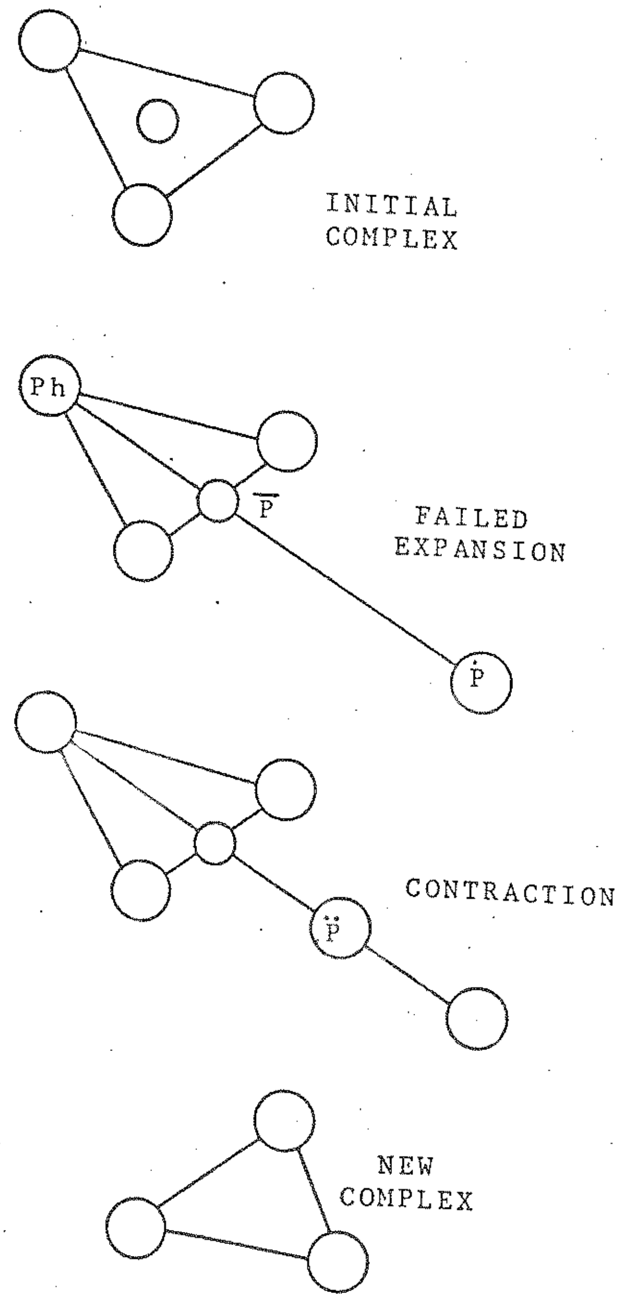


Figure 3.6 Complex Contraction

and thus compensates for moves back toward the centroid. Furthermore, it enables rapid progress to be made when the initial point is remote from the optimum and aids in maintaining the full dimensionality of the complex. The use of $k \geq n+1$ also aids in maintaining the full dimensionality of the complex because with n+1 or fewer points the complex tends to flatten into a subspace (Box, Davies, and Swann, 1969).

While the Complex Method does not require continuity of the problem function, it does implicitly require that the feasible region be convex. This requirement arises in the calculation of the centroid and its use in contracting a failed expansion point. If the region is not convex then the centroid could move into an infeasible region such that continued contraction would fail to produce a feasible point. One possible solution to this problem is to test the centroid for feasibility before making a contraction. If the centroid is feasible then the new vertex is sought between it and the violating point; otherwise the new vertex is sought between the current worst feasible vertex and the centroid (Swann 1974).

One important feature of constrained optimization is the difficulty of showing that a local optimum is in fact the global optimum. With unconstrained problems, a rough check that the global minimum has in fact been found is usually performed by restarting the method from different points, and inferring that if these all lead to the same solution, then this is indeed the global minimum. For constrained optimization, it is not an easy matter to find alternative starting points which satisfy all the constraints, and which differ substantially from each other. In fact, it may even be difficult to obtain an initial feasible point depending on the complexity of the problem.

With the Complex Method, subsequent optimality checks can be readily performed using the same initial point, but different pseudo-random number sequence initiators to set up the initial configuration. The ease with which this can be done should be considered an advantage of the Complex Method. In addition, because the initial configuration is generated so as to roughly span the feasible region, the first few iterations will be even more likely to span the whole of this region. Consequently, it seems reasonable to suppose that if several minima exist, and one of these corresponds to a very much smaller function value than the rest, then this best local minimum (the global minimum) will be found. Conversely, if the global minimum is not found, then there would seem to be a high probability that it would not represent much improvement over the selected minimum (Box, Davies, and Swann, 1969).

3.4.5 Application of Dynamic Programming

Dynamic programming is an efficient enumeration procedure for determining the combinations of decisions that optimize the overall system effectiveness as measured by an objective function. In order to apply dynamic programming, the problem must be separable into sequential stages which may represent a point in time or space. Each stage has a finite number of states which describe the condition of the system at that stage. Associated with each state may be a vector of state variables. Each state variable in turn may have a vector of discrete values. The basic concept of dynamic programming is based on Bellman's (1957) principle of optimality: "An optimal policy has the property that whatever the initial decisions are, the remaining decisions must constitute an optimal policy with regard to the state resulting from the first decision."

In applying dynamic programming to the detention basin optimization problem, the following scheme is suggested. First, let each stage correspond to the distance from the watershed outlet as measured by the number of detention basins along any reach. Associated with each stage is a set of detention basins. Associated with any detention basin may be several different state variables, such as storage, height, spillway diameter, etc. Associated with any state variable may be a set of values that the variable may assume. This scheme may be illustrated as in Figure 3.7.

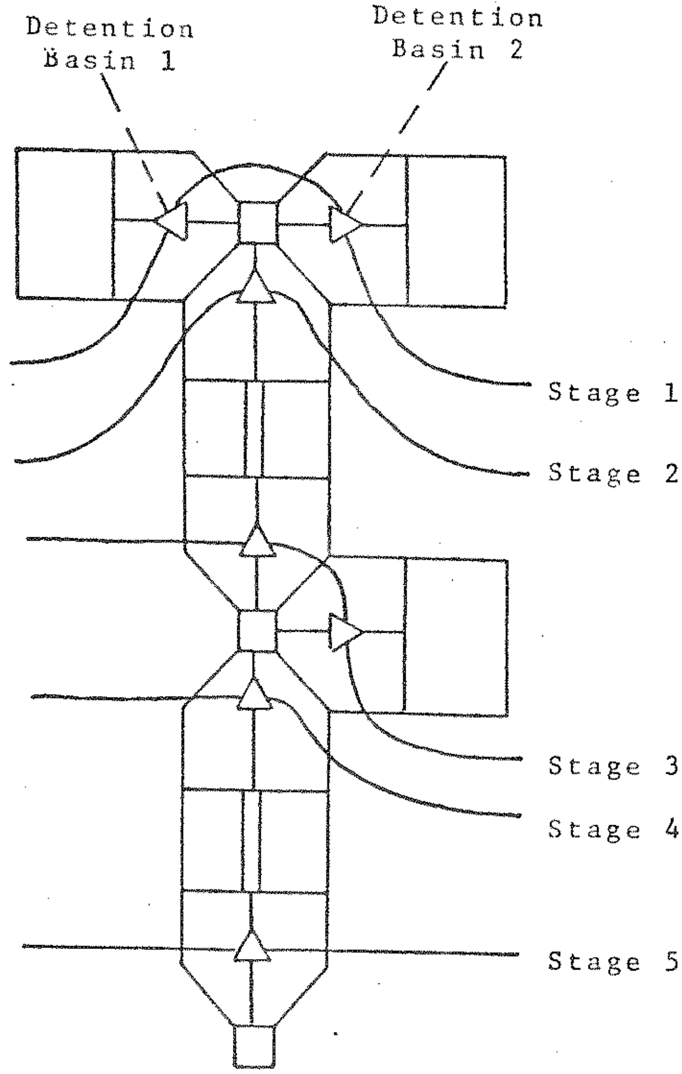


Figure 3.7 Stage-State Conceptualization

3.4.5.1 The Direct Formulation

There are two basic approaches that can be used in applying dynamic programming to the detention basin optimization problem. The first approach is to consider basin storage, basin height, and spillway diameter as state variables. These three variables may be used to derive both a stage-discharge relationship and a stage-storage relationship. Alternatively, the spillway pipe diameter could be replaced with a continuous orifice diameter such that the outlet pipe would then be determined based on the peak discharge from the orifice. If each basin is assumed to have vertical sides, then the basin storage and height may be combined into a single state variable of basin area.

During the evaluation of each state, different state variable vectors (representing different combinations of A, and D) may be evaluated and the resulting outflow hydrograph generated. Each vector that yields a violation of either a quantity or quality constraint is eliminated from the set of possible state vectors for a given state. The associated costs of the set of remaining vectors are then evaluated and added to the cost of the optimal path associated with each feasible state vector. At the end of the computations, the optimal downstream state is determined and the rest of the optimal state variables are determined by backtracking through the various stages. This formulation may be illustrated graphically as shown in figure 3.8.

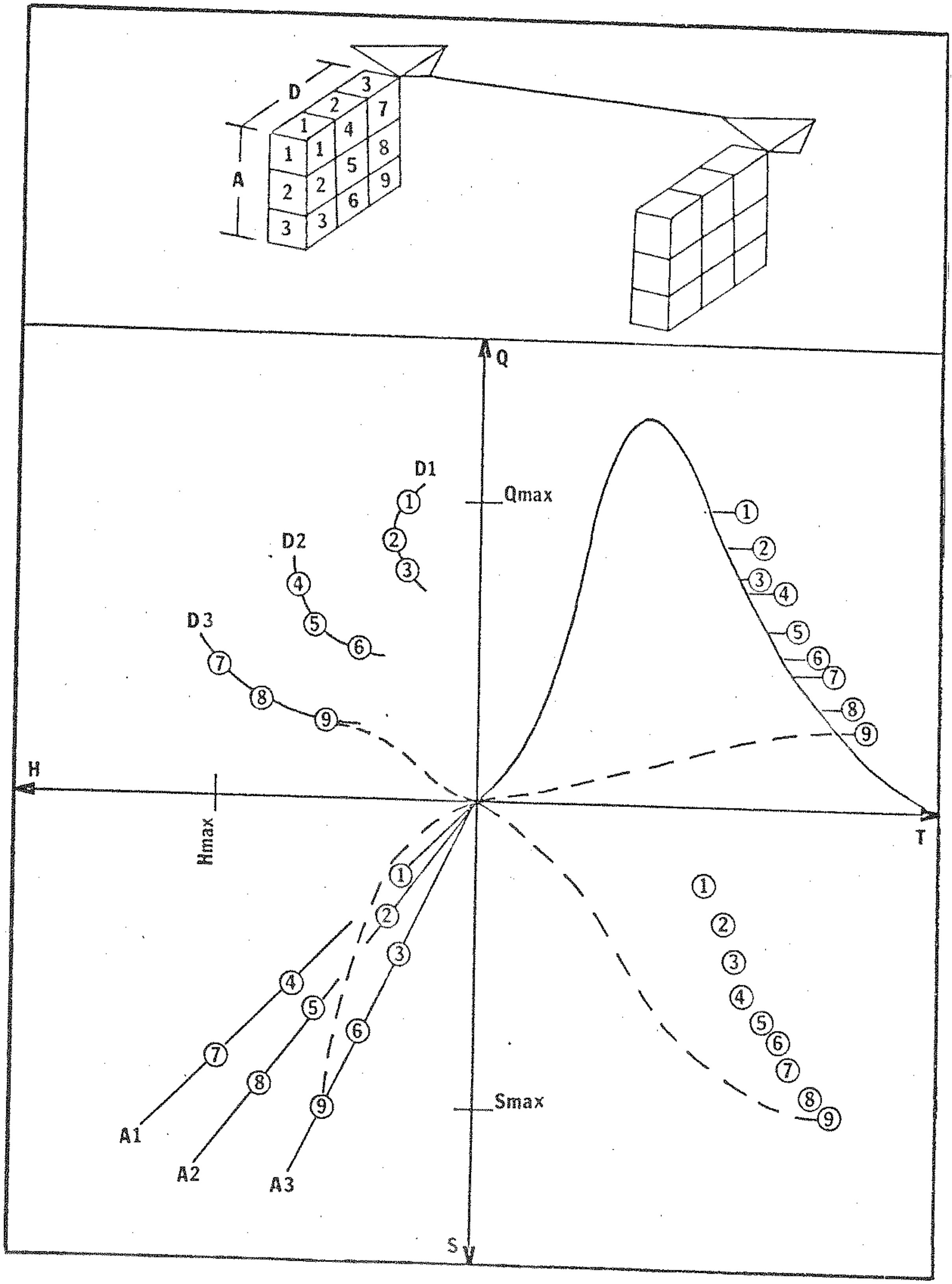


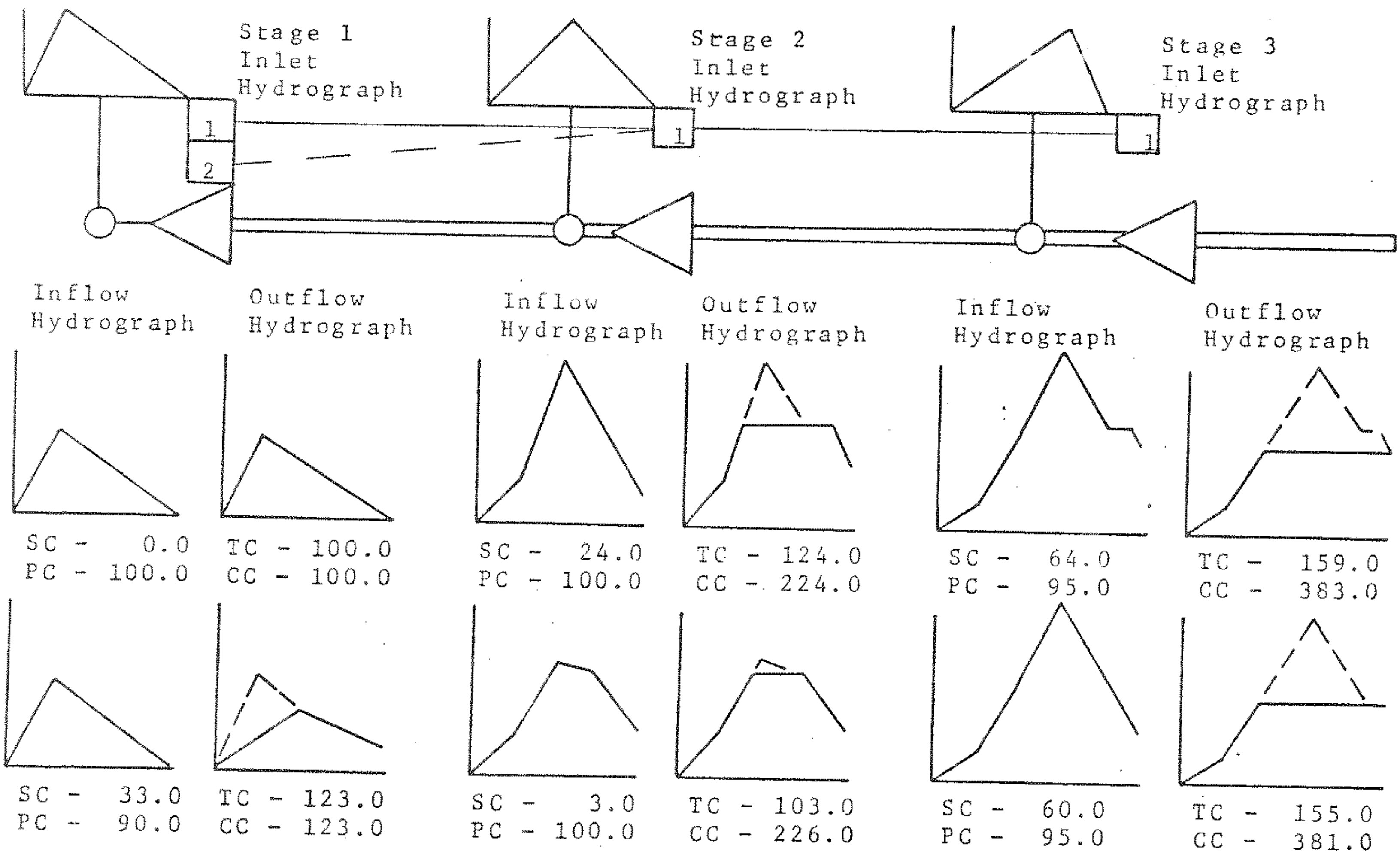
Figure 3.8 Direct DP Formulation

A close examination of the direct formulation reveals that there may be instances when Bellman's principle of optimality could be violated. Due to the interaction of the outflow hydrographs, it is possible that the selection of a sub-optimal upstream state vector could result in an outflow hydrograph that when combined with a downstream hydrograph could lead to a global solution that is better than the solution obtained using the upstream optimal state vector. This problem may be illustrated in Figure 3.9.

Although the dynamic program associated with the direct formulation can possibly yield a suboptimal solution, it is still a valuable hueristic for use in obtaining a feasible design. Because optimality is at least guaranteed between the stages of the problem, some improvement in the design is assured assuming that feasible solutions do indeed exist.

3.4.5.2 <u>The Indirect Formulation</u>

A second approach that may be used in applying dynamic programming to the general problem is to consider both the outflow hydrograph and total pollutant load as state variables. The outflow hydrograph may be characterized by a functional relationship between flowrate and time. If a desired outflow hydrograph shape is preselected, then the hydrograph may be characterized by two state variables: the hydrograph peak, and the time to peak. If the time to peak is pre-selected based on some hydrologic criteria, such as



SC - Storage Cost
PC - Pipe Cost

TC - Total Cost
CC - Cumulative Cost

Figure 3.9 Illustration of Stage Inseparability Problem setting the time to peak equal to the maximum time to peak of the inflow hydrographs, then the hydrograph may be characterized by a single state variable: the peak discharge.

Once a desired outflow hydrograph has been set, a cumulative storage curve may be obtained. Once this curve has been obtained the maximum required storage may be determined. For a given pollutant removal efficiency, a required basin depth can be determined. Once the basin depth has been determined it can be used with the peak discharge to determine the necessary outlet dimension. Once the required outlet dimension has been obtained, a stage-discharge curve can be obtain. Once this curve has been determined, the stage-storage curve required to produce the selected outflow hydrograph may be derived

In this formulation, feasibility checks may be made in relation to the maximum required storage, the maximum basin height, the required outlet dimension, and the feasibility of the required stage-storage curve. If a constraint is violated, then that particular state vector is deleted from the state space. Because the upper limits of the state variables (peak discharge and pollutant load) are now defined for a given state, a set of state vectors can be generated such that each vector satisfies both quantity and quality constraints. If the shape of the rising limb of the outflow hydrograph is specified then the basin storage, basin height, and spillway diameter are defined. Thus the cost associated with any state vector can be readily determined. This formulation is illustrated in Figure 3.10.

By defining the state variables as the outflow hydrograph and the pollutant load, the potential inseparability problem associated with the first approach can be basically overcome. However, the basic inseparability problem can reappear when a do nothing state is introduced into the formulation. By inclusion of a do nothing state, the possibility again arises that a sub-optimal upstream state could produce a hydrograph that when passed through the downstream do nothing state yields a better global optimal solution. In effect, the possibility exists that a decision at an upstream state could affect a state further downstream than the immediate downstream state and this would thus violate Bellman's principle of optimality. There are two possible solutions to this problem. One solution would be to eliminate the do nothing decision from consideration. This of course would limit the decision space and thus possibly lead to a sub-optimal decision.

A second possible solution would be to enumerate completely the states associated with the do nothing decision. By doing this, the state space would grow at each stage. However, because only one state is being enumerated the process would still be much more efficient than total

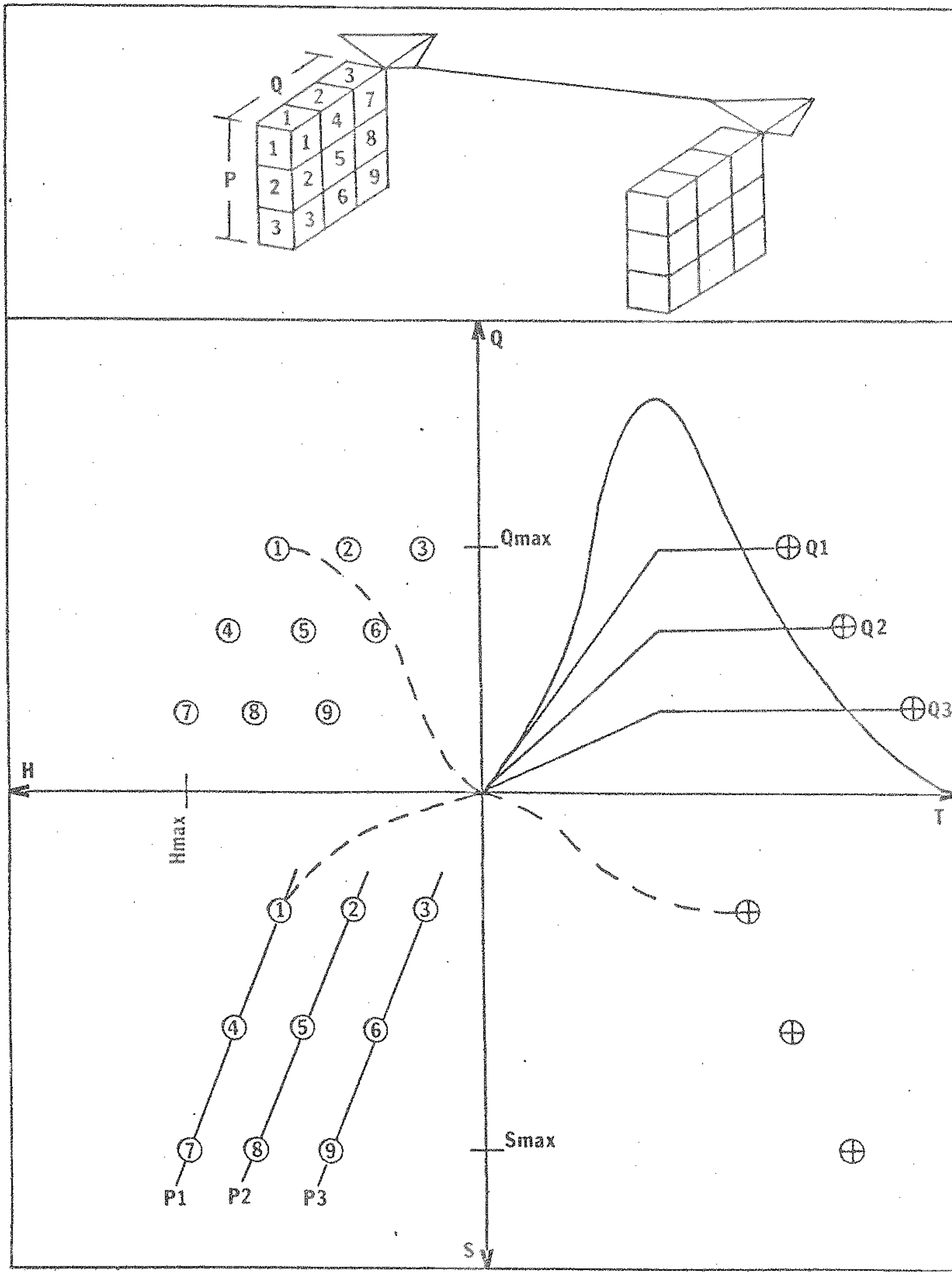


Figure 3.10 Indirect DP Formulation

enumeration. In addition, because the do nothing state passes both unattenuated hydrographs and untreated pollutant loads it is quite possible that the majority of these paths will be eliminated due to flowrate and pollutant load constraints. Thus the state space has the possibility of collapsing to the dimensions of the regular dynamic program.

3.4.6 Construction of a Design Heuristic

In order to apply linear or mixed integer programming to the detention basin optimization problem, all nonlinear relationships must be either simplified using linear relationships or approximated using linear segments and zero-one variables. Although these modifications will permit the application of linear or mixed integer programming to the problem, and thus guarantee a global optimal solution, there remains a question of what relevance this result has in relation to the original nonlinear problem.

Linear programming has the advantage that a global optimal solution will always be found if the problem is feasible. Inclusion of zero-one variables into the formulation requires the use of a mixed integer strategy such as branch and bound, cutting planes, or Bender's algorithm. Although global optimality can again be insured, the computational complexity of the problem may be greatly increased, and in some cases can lead to a total enumeration of the discrete variables.

A particular problem with the application of linear or mixed integer programming to the detention basin problem concerns the evaluation of functional constraints. Because the functional constraints must be included in the overall problem formulation, an entire new set of variables and constraints is required for each additional time step that is considered. Because of the large number of functional constraints required at any time step, the overall formulation can become very large with only a small number of time steps. Thus, from a computational point of view, the formulation may be severely limited in the total number of time steps that may be considered.

The nonlinear programming approach to the detention basin optimization problem has several advantages over the linear programming approach. First, the nonlinear programming approach may use nonlinear relationships in both the objective function and the constraint set. Thus nonlinear cost functions for storage may be used directly. In using nonlinear programming, the transformation of the variables between different stages of the problem, as represented by the functional equations in the original formulation, may be accomplished external to the optimization problem using nonlinear relationships or even mathematical models. Thus, the simplifications required in the linear programming formulation are no longer required. As a result, more accurate relationships may be used. The main problem with nonlinear programming is that a global optimal solution cannot be guaranteed. In addition, given the complexity of the problem, a feasible starting point is not always directly available.

Dynamic programming possesses many of the same advantages as nonlinear programming. Dynamic programming may also use nonlinear relationships because the feasibility of the state variables may be determined external to the optimization routine using simulation. In addition, as long as Bellman's principle of optimality is satisfied, a global optimal solution is guaranteed for the particular degree of discretization of the problem. One problem with dynamic programming is the degree of discretization that is needed to define the solution space effectively. The problem can become particularly acute with problems involving more than one state variable. This problem is generally known as the "curse of dimensionality".

In applying dynamic programming to the detention basin optimization problem, two different formulations were examined. Both approaches were found to be potentially inseparable. However, the problem with the second approach can be overcome by totally enumerating the do nothing state. Although this formulation is feasible and could be applied to the general problem, several approximations are required in order to define the outflow hydrograph states. These approximations introduce an added complexity to the programming and raise again the question of the transferability of the final result to the original problem.

Because of the highly nonlinear nature of the problem, and the unavailability of a suitable mixed integer algorithm, neither a linear programming nor a mixed integer approach was employed. Both the nonlinear and dynamic programming approaches incorporate the nonlinear nature of the problem. One of the main problems with the norlinear approach is the need of an initial feasible starting point. Given the complexity of the problem, such a point is rot always easy to obtain. Such a point can be readily obtained however by using dynamic programming. Thus dynamic programming can be combined with a nonlinear algorithm to produce a dual level planning hueristic. By combining the two approaches, the nonlinear algorithm can be used to both check the dynamic program and/or provided some refinement to the design when a large discretization scheme is used with the dynamic program.

Given the complexity of the overall problem, neither the direct or indirect dynamic program formulations can be shown to consistently produce the best results. Although the indirect formulation would appear to have some advantage in relation to separability, the direct formulation is more straight forward and does not require as many assumptions.

Because of this fact and because of programming ease, the direct formulation was selected for use in the general design algorithm. Thus, the direct formulation was used with the Complex Method of Box to produce a general detention basin design heuristic. The overall algorithm is illustrated in the flowchart in Figure 3.11.

3.4.7 Description of the Design Heuristic

The general algorithm uses the direct DP formulation approach to obtain an initial feasible solution. This solution may or may not correspond to an optimal solution of the complete nonlinear problem. Four different state variables may be considered at each detention site: basin length, basin width, basin side slope, and the orifice outlet dimension. For the purpose of this study, square orifices were assumed. In addition, three different costs may be considered: storage cost, area cost, and orifice cost. The cost of the required downstream pipe or channel may also be included in the problem if desired. The overall program is very flexible and may include the storm sewer network in the overall design problem. For the purpose of this study, the slope of any designed pipe is assumed be equal to a specified ground slope.

Once an initial feasible solution has been obtained from the dynamic program, the algorithm continues, using the Complex Method of Box. Using the initial feasible solution,

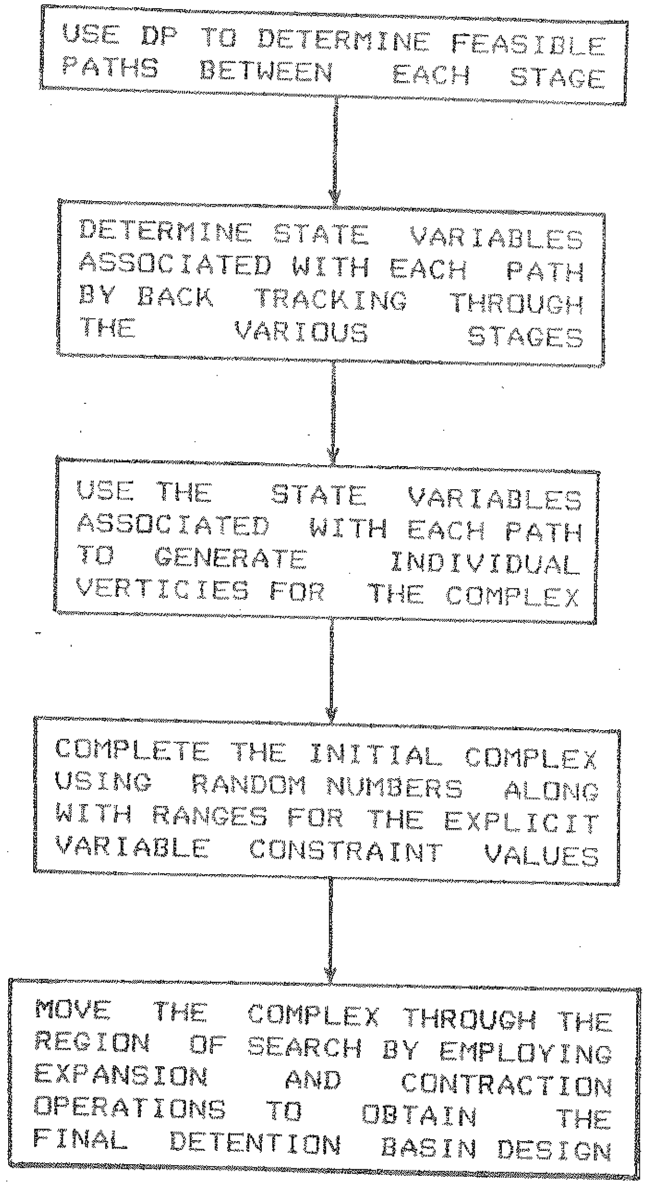


Figure 3.11 Flowchart of the Design Heuristic

an initial complex is generated. Each vertex of the complex corresponds to a vector of variables for the entire problem. The complex moves through the region of search by employing both expansion and contraction operations. After each expansion, the explicit constraints on the decision variables are evaluated. If a constraint is violated, the violating decision variable is reset just inside the constraint boundary.

Once the explicit constraints have been evaluated, the implicit constraints are evaluated. This operation requires a complete simulation of the trial design. If an implicit constraint (flowrate, pollutant load) is violated, then the trial vertex is contracted and the simulation repeated. If the implicit constraints are not violated, then the cost of the trial design is determined and compared with the worst design in the complex. If the trial design is better than the current worst design, then the worst design is replaced with the new design. If the trial design is worse than the current worst design, then the trial vertex is contracted until an acceptable design is obtained. This process of expansion and contraction is continued until a convergence criterion is satisfied or until a specified number of trials is exceeded.

One additional advantage of using dynamic programming in conjunction with the Complex Method is that dynamic programming can be used to obtain all the feasible paths through the specified state space in addition to the optimal path. These additional feasible solutions could be used to generate the initial complex and thus the additional computations required to set up the initial complex could be avoided. In using the additional feasible paths care must be taken to insure that each feasible path is unique. If this requirement is not observed, redundant solutions would be introduced into the complex and the complex would tend to collapse prematurely.

Embedded in both programs is a simulation program that routes the inlet hydrographs and pollutant loads through the watershed. Hydrographs are routed through the channels using a simple time lag approach. Hydrographs are routed through the detention basins using the Newton-Raphson iteration technique. In determining pollutant removal levels, pollutants are characterized by a total load approach in which the pollutant removal is determined based on the ratio of the average settling velocity of the pollutant to the critical settling velocity of the basin. Non-ideal flow conditions are approximated through the use of a turbulence factor α.

3.4.8 Cost Data

Although the general design algorithm can consider four different costs (storage, area, orifice, and pipe), only storage and pipe costs were considered in the present study. Storage costs may be obtained using the costs in Table 3.1 or by using the following relationship (Benjes et al., 1975)

$$SC = (0.025*ENRCCI/2500) * S^{0.73} \qquad (3.72)$$

where SC = storage cost ($\$ \times 10^6$)

ENRCCI = Engineering News Record cost index

S = storage (mil gal)

This relationship was derived for a typical earthen detention basin assuming 2.5:1 side slopes with an average depth of 18 feet. Table 3.1 was developed from data by Zoller and Rolf (1977) and was updated to 1983 values by using Engineering News Record cost indexes.

An estimate of pipe costs may be obtained using the unit costs in Table 3.2 or the following relationship.

$$TPC = PC*PL + (ENRCCI/3376)*(1.93D+1.688H-12.6)*PL \qquad (3.73)$$

where TPC = total pipe cost ($)

ENRCCI = Engineering News Record Cost Index

PC = unit pipe cost ($/ft)

PL = pipe length (ft)

D = pipe diameter (in)

H = invert depth (ft)

Table 3.1 Storage Excavation Costs

Volume (Cu. Yards)	Cost ($/Cu. Yard)
0 - 1999	5.39
2000 - 4999	7.55
5000 - 9999	4.38
10000 - 24999	3.60
25000 - 49999	4.33
50000 - 99999	2.82
Over 100000	1.99

Table 3.2 Concrete Pipe Costs

Pipe Dia (in)	Pipe Cost ($/linear foot)
12	50.0
18	70.0
24	93.0
30	140.0
36	190.0
42	220.0
48	245.0
54	275.0

This relationship was derived by Han et al. (1980) as a result of an analysis of several different available cost relationships. The relationship is valid for pipes with diameters less than or equal to 36 inches and invert depths less than or equal to 20 feet. Table 3.2 was derived as a result of a review of bids for several storm sewer projects in the state of Kentucky during 1980. These costs include the total installation costs of the pipes and are valid for trench depths up to 18 feet. The listed costs have been updated to 1983 prices using Engineering News Record cost indices. The Engineering News Record cost index for 1983 is 4208.

3.5 METHODOLOGY DEVELOPMENT

The proposed planning methodology uses a continuous simulation model (SWMM) to obtain a time series of runoff events. Based on a statistical analysis of the resulting runoff series a set of individual critical design runoff events are selected. These runoff events are then used as input to the design heuristic. Once a design for a specific design event has been obtained, it should be evaluated by applying the remaining design events of the same return frequency. If the design fails to perform adequately, it may be necessary to use another design event of the same return frequency. Once a design has been obtained for a given return frequency, the design should be fixed and the next series of design storms applied. This process is then repeated until the final design is obtained. The major steps of the proposed methodology are summarized below. The overall methodology is illustrated in Figure 3.12.

1. Select the upper limit of the critical design period for the overall system design (ex. 25 years).

2. Simulate the critical design period using SWMM.

3. Determine the frequency of occurrence of the various critical design parameters (i.e. runoff volume, peak flowrate, pollutant load, etc.) for various return frequencies (ex. 2, 5, 10, 25 years).

4. Based on a statistical analysis of the simulation results, select critical design storm events for the desired design frequencies (ex. 2, 5, 10, 25 years).

5. Obtain an initial design for a given design frequency by applying the planning heuristic to a selected runoff event.

6. Evaluate the performance of the initial design by application of the runoff events of the same return frequency. If the design fails to meet the selected performance criteria, then select another design event and repeat steps 5 and 6.

7. Once an optimal design has been obtained for a given return frequency, fix that design and repeat steps 5 and 6 with the next level of design events.

8. Once the final design has been obtained, it may be desirable to test the overall performance of the design by rerunning the entire continuous simulation with the final design in place.

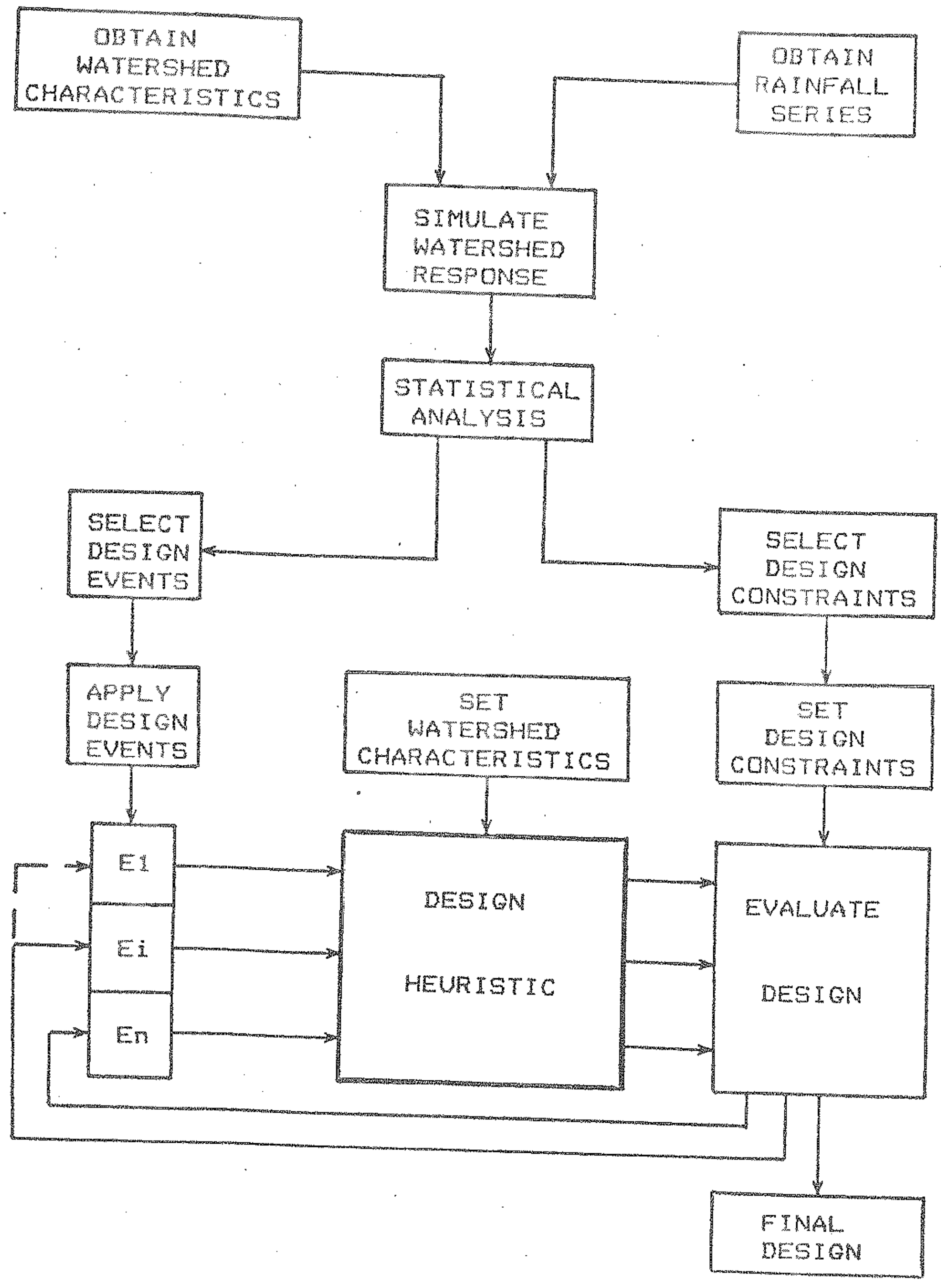


Figure 3.12 Flowchart of the Planning Methodology

IV. GLEN ELLYN WATERSHED APPLICATION

4.1 INTRODUCTION

In order to test the general planning methodology, it was applied to a watershed in Glen Ellyn, Illinois. Glen Ellyn is located in Dupage County, Illinois, just west of Chicago. The Glen Ellyn watershed encompasses 534 acres of moderately sloped land. The watershed is composed of two major subsheds that drain into a small lake at the outlet of the watershed.

Lake Ellyn is one of 9 detention facilities currently being investigated as part of the National Urban Runoff Program. Approximately 18 months of data have been collected with regard to the Lake Ellyn study. These data include 5 minute rainfall and flow data as well as data for 47 different quality constituents. A map of the Glen Ellyn watershed is provided in Figure 4.1 (Cowan, 1982). A summary of the physiographic, land use and hydrologic characteristics of the Glen Ellyn watershed is provided in Table 4.1 (NIPC, 1980).

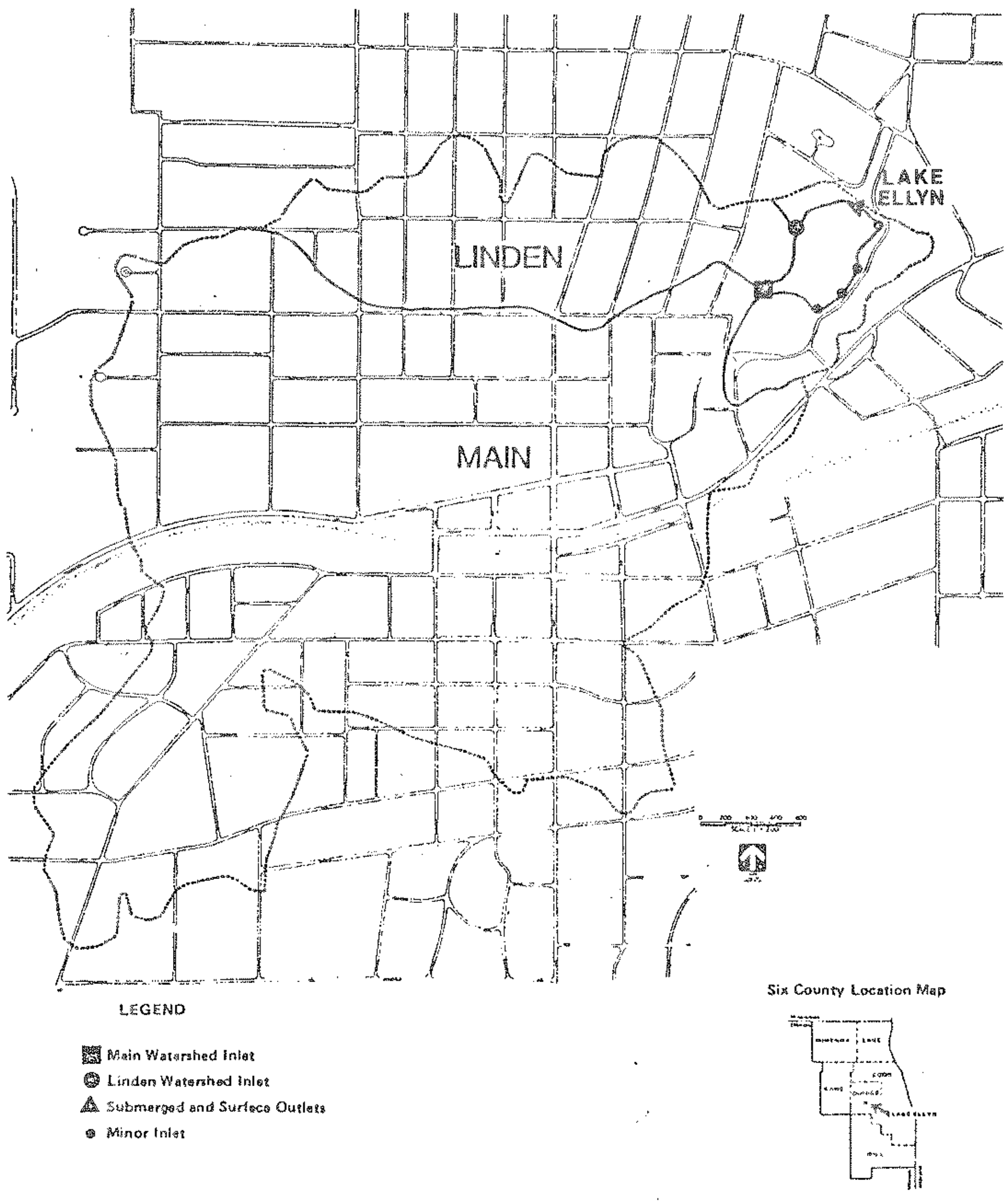


Figure 4.1 Glen Ellyn Watershed

Table 4.1 Physiographic and Hydrologic Characteristics of Glen Ellyn Watershed

Total Drainage Area	534 acres
Impervious Area	182 acres
Effective Impervious Area	91 acres
Land Use	
Low Density Residential	427 acres
High Density Residential	16 acres
Institutional	53 acres
Commercial	27 acres
Wetland	11 acres
Pollutant Loading (TSS - lbs/curb mi)	
Low Density Residential	711 lbs
High Density Residential	267 lbs
Institutional	460 lbs
Commercial	611 lbs
Average Hydrologic Soil Group	C
Main conveyance slope	49 ft/mi
Average basin slope	220 ft/mi
Population Density	5000 pn/mi
Street Density	21.6 mi/sm

4.2 CALIBRATION OF SWMM

Before applying the general planning methodology to the Glen Ellyn watershed, SWMM was first calibrated. In calibrating the model, the main subshed was broken down into two smaller subsheds as shown in Figure 4.2. A summary of the physiographic and hydrologic characteristics of the resulting three subsheds is presented in Table 4.2. Infiltration was modeled using the Green-Ampt equation. Initial infiltration parameters were selected based on a hydrologic soil group of C. Pollutant buildup was modeled using a linear buildup relationship. Initial pollutant loadings for each subshed were obtained from Table 4.2. Pollutant washoff was modeled using an exponential washoff equation. The decay coefficient k was initially set equal to 1.5.

Three discrete storms were selected from the 18 months of record and used to calibrate SWMM. Final selected parameter values were obtained from a average of the various calibration parameters. Both flowrate and water quality parameters were adjusted in calibrating the model. A comparison of the predicted results with the measured data for the three events is presented in Figures 4.3 - 4.8.

After the model was calibrated, it was used in a continuous simulation of the 18 month period of record. The

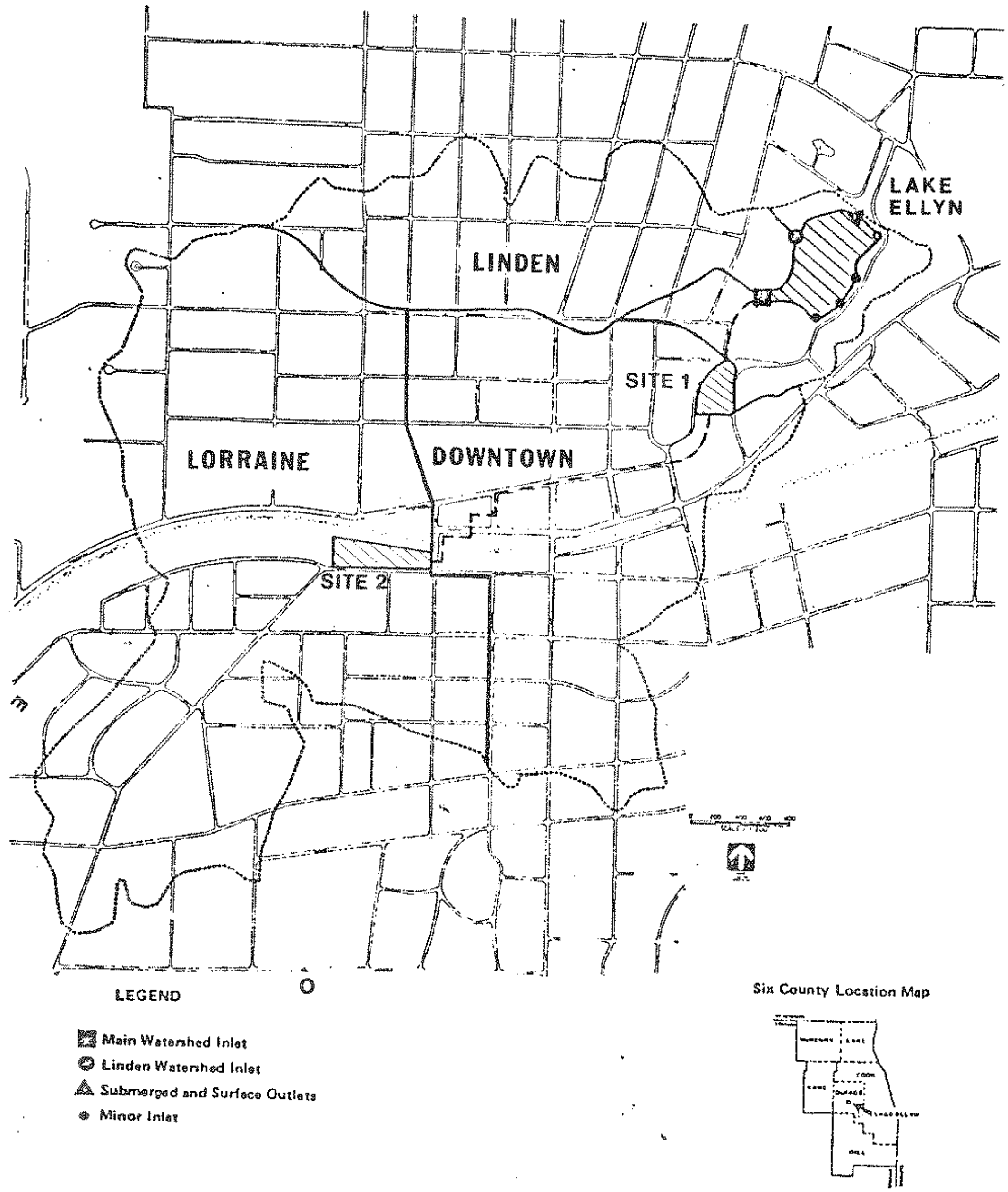


Figure 4.2 Watershed Discretization

Table 4.2 Physiographic and Hydrologic Characteristics of Glen Ellyn Subsheds

Linden Subshed

Subshed Area	117 acres
Eff Imp Area	11 acres
Subshed Slope	100 ft/mi
Subshed Length	4373 feet
Solids Loading	961 lb/dy

Downtown Subshed

Subshed Area	162 acres
Eff Imp Area	50 acres
Subshed Slope	53 ft/mi
Subshed Length	4920 feet
Solids Loading	4360 lb/dy

Lorraine Subshed

Subshed Area	255 acres
Eff Imp Area	30 acres
Subshed Slope	64 ft/mi
Subshed Length	4373 feet
Solids Loading	3562 lb/dy

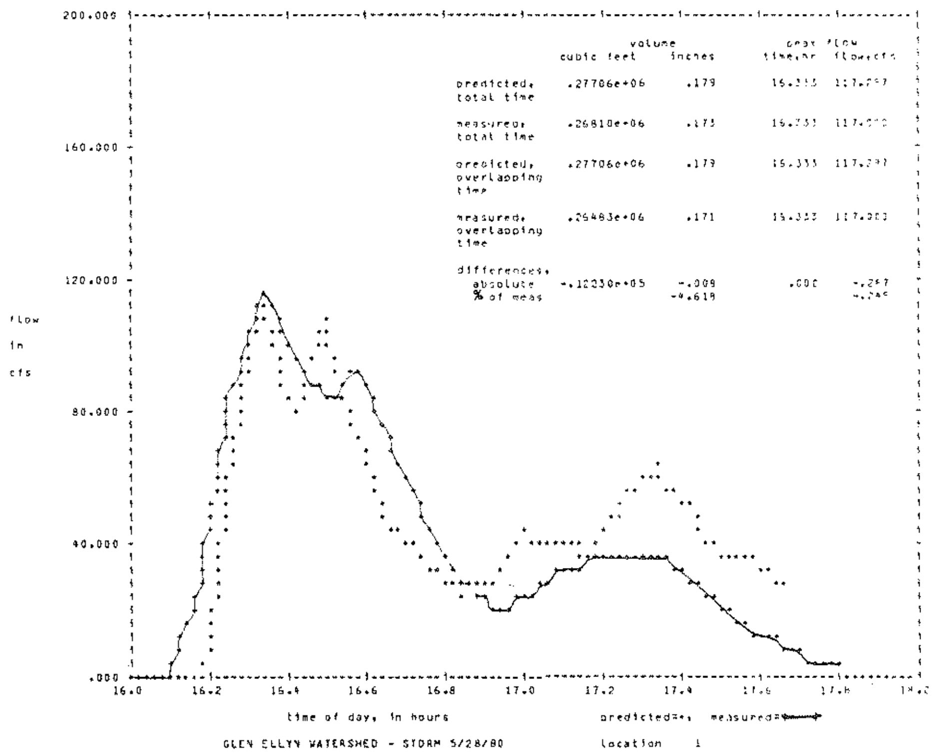


Figure 4.3 Measured and Predicted Hydrographs for Event 5/28/80

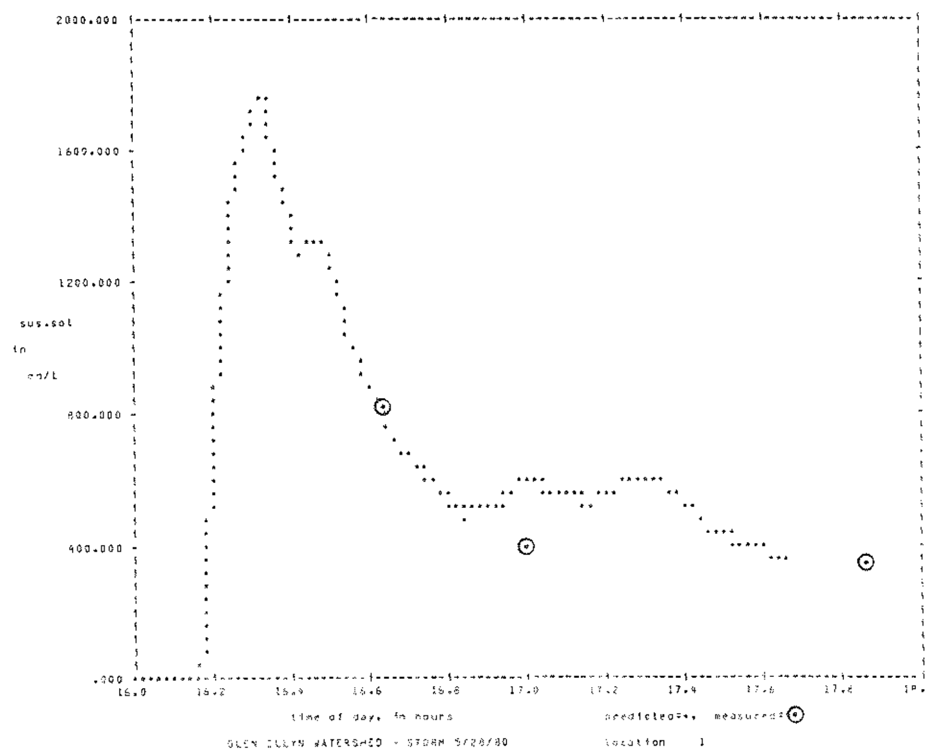


Figure 4.4 Measured and Predicted Pollutographs for Event 5/28/80

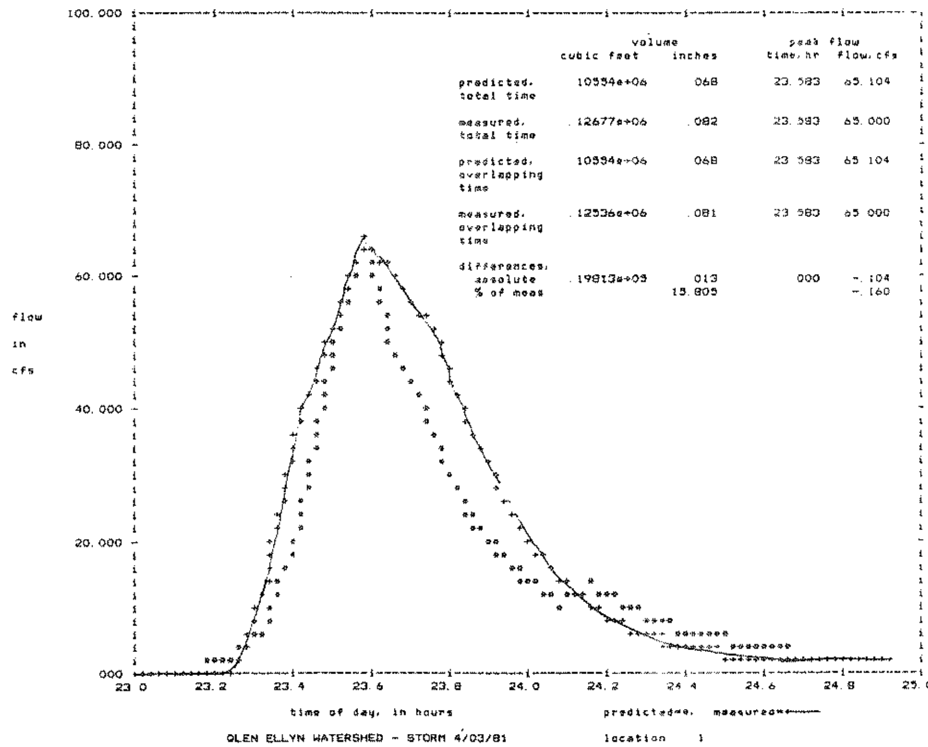


Figure 4.5 Measured and Predicted Hydrographs for Event 4/03/81

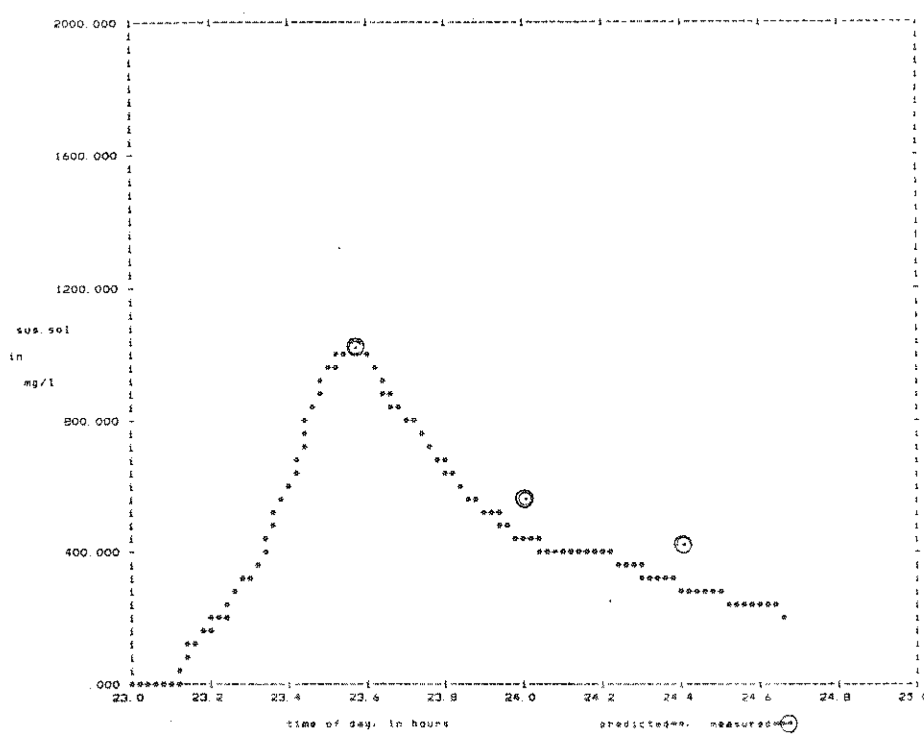


Figure 4.6 Measured and Predicted Pollutographs for Event 4/03/81

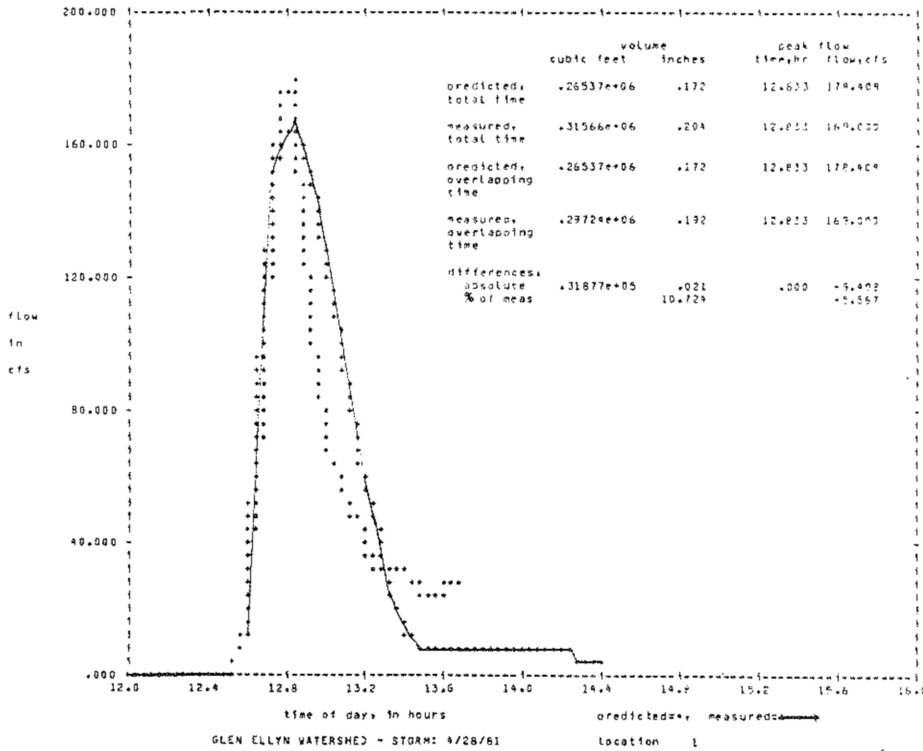


Figure 4.7 Measured and Predicted Hydrographs for Event 4/28/81

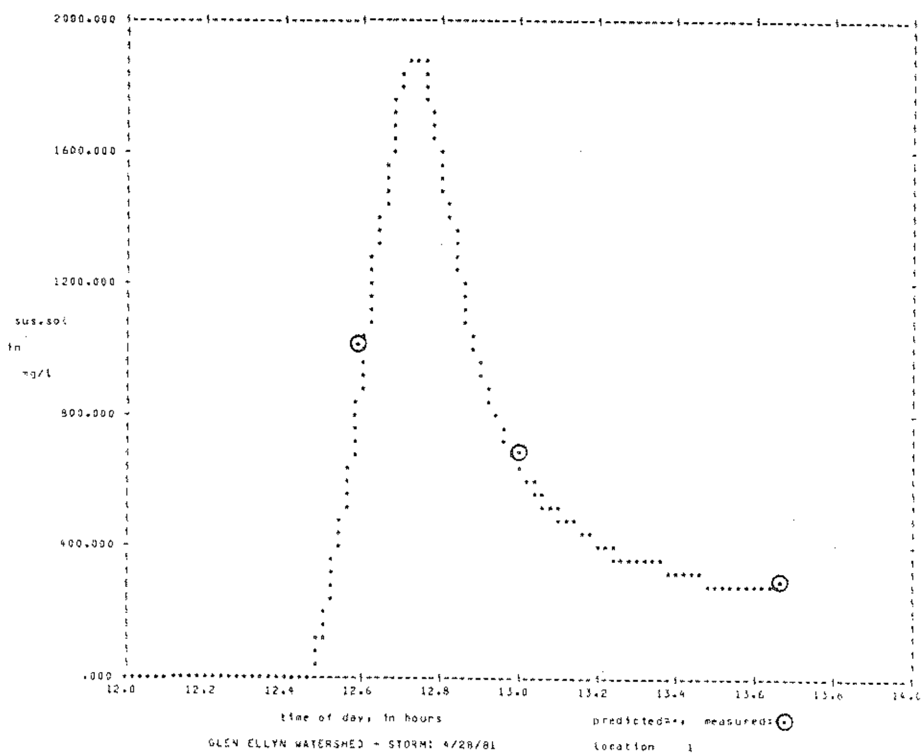


Figure 4.8 Measured and Predicted Pollutographs for Event 4/28/81

monthly totals from this simulation are presented in Table 4.3. In comparing the results of this simulation with the measured results, the total runoff volume was underpredicted by 10 percent while the total suspended solids load was overpredicted by only 2 percent.

4.3 METHODOLOGY APPLICATION

The objective of applying the general planning methodology to the Glen Ellyn watershed was to test both the simulation program and the design heuristic with a real data base. In applying the methodology to the Glen Ellyn watershed, two additional detention basin sites were investigated. These two sites are shown in Figure 4.2.

4.3.1 Watershed Simulation

The first step in the general planning methodology is the simulation of the watershed. This was accomplished using the calibrated SWMM model along with the 18 month rainfall series. For the continuous simulation, a one hour time step was employed. In addition to flowrate, total suspended solids loading and washoff were also simulated.

Table 4.3 Monthly Summaries for Continuous Simulation

Results for 1980

month	inlt	rain inch	flow inch	sus.sol pounds
apri	1	1.16	.18	2.598e+03
may	1	3.17	.92	2.052e+04
june	1	5.37	1.14	3.983e+04
july	1	3.43	.75	2.341e+04
augu	1	3.63	.62	2.117e+04
sept	1	5.46	1.27	3.147e+04
octo	1	3.29	.59	1.808e+04
nove	1	.68	.10	2.368e+03
dece	1	2.49	.41	1.049e+04
year	1	28.68	5.99	1.70e+05

Results for 1981

month	inlt	rain inch	flow inch	sus.sol pounds
janu	1	.00	.00	0.000e+00
febr	1	1.82	.29	7.008e+03
marc	1	.38	.05	1.022e+03
apri	1	4.87	.90	3.010e+04
may	1	3.61	.75	2.245e+04
june	1	2.24	.37	1.038e+04
july	1	3.16	.61	2.438e+04
augu	1	4.28	.73	2.596e+04
augu	1	1.37	.22	6.232e+03
year	1	21.73	3.93	1.27e+05

4.3.2 Statistical Analysis

The second major step in the general planning methodology involves a statistical analysis of the simulation results. The results of this analysis are then used in the selection of a set of design events and design constraints. A ranking of the top five events for nine different hydrologic statistics is provided in Table 4.4.

Normally, if hourly rainfall data was the only precipitation data available, the results of Table 4.4 would be used to select a range of design events for different frequencies of design. In this particular case, 5 minute rainfall data was also available. In order to improve the overall design of the detention system, this data was used along with the calibrated SWMM model to obtain a new ranking of the runoff events. Instead of conducting a new continuous simulation using 5 minute time intervals, 15 discrete simulations were used. The events corresponding to the 15 discrete simulations were selected based on the results of the continuous simulation. The continuous simulation was thus used as a screening tool to obtain a smaller set of runoff events that could then be examined in more detail using 5 minute time interval simulations. A ranking of the runoff events based on the results of these simulations is provided in Table 4.5.

Table 4.4 Continuous Simulation Results

RAINFALL DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	5/29/80 (1.14)	7/13/81 (.710)	9/16/80 (2.05)
2	6/07/80 (0.87)	5/29/80 (.413)	6/07/80 (1.77)
3	4/28/81 (0.79)	7/25/81 (.340)	5/28/80 (1.69)
4	7/20/80 (0.78)	7/20/80 (.298)	7/20/80 (1.49)
5	5/28/80 (0.74)	6/07/80 (.295)	5/29/80 (1.24)

FLOWRATE DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	7 20/80 (.193)	5/29/80 (.061)	9/16/80 (.730)
2	5/29/80 (.181)	6/07/80 (.054)	6/07/80 (.643)
3	6/07/80 (.152)	5/28/80 (.052)	5/28/80 (.576)
4	9/16/80 (.151)	9/16/81 (.049)	7/20/80 (.525)
5	4/28/81 (.137)	6/28/80 (.045)	5/29/80 (.487)

POLLUTANT DATA

Rank	Peak (mg/l)	Avg (mg/l)	Total (lbs*E3)
1	5/28/80 (1051)	6/15/80 (756.)	9/16/80 (35.6)
2	6/28/80 (1033)	7/12/81 (742.)	6/07/80 (34.6)
3	6/15/80 (1016)	7/13/81 (728.)	5/28/80 (32.9)
4	4/28/81 (1015)	7/16/81 (722.)	7/20/80 (32.4)
5	7/25/80 (1006)	4/04/81 (710.)	4/28/81 (26.4)

Table 4.5 Discrete Simulation Results

RAINFALL DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	5/29/80 (5.16)	7/13/81 (.710)	9/16/80 (2.05)
2	6/07/80 (5.16)	5/29/80 (.413)	6/07/80 (1.77)
3	4/08/81 (3.72)	7/25/81 (.340)	5/28/80 (1.69)
4	4/28/80 (3.24)	7/20/80 (.298)	7/20/80 (1.49)
5	7/20/80 (3.12)	6/07/80 (.295)	5/29/80 (1.24)

FLOWRATE DATA

Rank	Peak (in/hr)	Avg (in/hr)	Vol (in)
1	5/29/80 (.605)	5/29/80 (.125)	6/07/80 (.380)
2	6/07/80 (.566)	7/13/81 (.096)	9/16/80 (.378)
3	4/08/80 (.406)	6/07/80 (.065)	5/28/80 (.323)
4	4/28/81 (.384)	7/25/81 (.054)	5/29/80 (.312)
5	6/28/80 (.263)	6/28/80 (.048)	5/29/81 (.232)

POLLUTANT DATA

Rank	Peak (mg/l)	Avg (mg/l)	Total (lbs*E3)
1	4/08/80 (2432)	5/24/81 (1243)	5/28/80 (26.2)
2	4/28/81 (2231)	7/20/80 (1062)	9/16/80 (24.2)
3	5/24/81 (2225)	6/28/80 (948)	4/28/81 (23.4)
4	6/28/80 (2150)	4/08/80 (926)	4/08/80 (20.3)
5	7/20/80 (2061)	4/28/81 (916)	7/25/81 (19.7)

4.3.3 Design Event Selection

Once a statistical analysis of the simulation results has been performed, the results of this analysis can be used in the selection of specific design events for different design frequencies. For the Glen Ellyn application, only one design frequency was considered: a frequency of 18 months. Even when only a single design frequency is considered, as in this case, the selection of an appropriate design event is not always a straightforward process. Examination of Tables 4.4 and 4.5 reveals that the various events do not have the same rankings for the different selected parameters (ie, peak, average, total). Thus, for a given system variable such as flowrate, one must decide whether to select the design event based on a peak ranking, average ranking, or a total ranking. When pollution parameters are considered in addition to flowrate, the selection process can become very difficult.

In general, for a detention basin design, peak statistics are more important in the selection of design constraints while total statistics are more important in the selection of design events. However, the selection of a final design event will still require an examination of both statistics. For example, an event with a medium volume and a very high peak will probably be more severe than an event with a very large volume and much lower peak. In general, the final selection will tend to involve a certain degree of engineering judgement.

The selection of the most appropriate pollutant statistic will tend to depend on the specific pollutant objectives of the overall design. In this study, the total load statistic was considered to be the most important statistic since the design heuristic determines removal efficiencies based on total load.

Although a single design event for a given frequency may be obtained based on an examination of simulation results, a more appropriate approach would be to select a set of design events. By using a set of events, the design corresponding to a particular event can be tested by applying the remaining events. Ideally, one of the individual designs will be satisfactory for all of the events. If no single satisfactory design can be found, then some manual adjustment must be made. Such adjustments could result in designs that correspond to larger return frequencies for a particular hydrologic statistic. However, such an approach will result in designs that satisfy the design frequencies of all of the hydrologic parameters and not just one or two. In applying the general planning methodology to the Glen Ellyn watershed, a set of 4 different design events was selected. A listing of the selected events is provided in Table 4.6.

Table 4.6 Set of Design Events for
Glen Ellyn Application

1	5/28/80
2	5/29/80
3	6/07/80
4	9/16/80

4.3.4 Design Constraint Selection

Once the appropriate design events have been selected, the next step is to select a set of design constraints for the watershed. These constraints on flowrate and pollutant load may be set at the outlet of the watershed or at various points within the watershed.

Flowrate constraints for a developed watershed are usually obtained by limiting the peak flowrates to those which occurred prior to development. Thus once the design event has been obtained, the corresponding rainfall pattern could be reapplied to the watershed in an undeveloped state to obtain the flowrate constraints for the developed condition. When a set of design events is utilized as opposed to a single event, it will be necessary to examine the predevelopment flowrates corresponding to all of the design events. Selection of the final constraints would then depend on a certain amount of engineering judgement.

For the Glen Ellyn application, an alternative approach was used. In order to examine the response of the design heuristic, a range of flowrate constraints were used. These flowrate constraints were based on percentages of the flowrate corresponding to the design event which had the largest peak flowrate. For the purpose of this study, percentages of 25, 50 and 75 percent were used.

Although flowrate constraints for a developed watershed can usually be readily obtained, the selection of pollutant constraints is usually not as straightforward. Part of this problem stems from the lack of overall guidelines in relation to stormwater pollutant removal. Part of the difficulty in establishing such guidelines is the stochastic nature of the impact of the pollutants on a receiving body.

For this study, total load constraints were selected based on percentages of the total load corresponding to the design event which had the largest load. As with the flowrate constraints, percentages of 25, 50, and 75 percent were used. A summary of the flowrate and pollutant constraints associated with each subshed is provided in Table 4.7

Associated with each potential detention basin site, including Lake Ellyn, is a set of variable constraints. A summary of the constraints associated with each detention site if provided in Table 4.8.

Table 4.7 Subshed Constraints

Linden Subshed

Flowrate Constraint (100%)	326 cfs
Flowrate Constraint (75%)	244 cfs
Flowrate Constraint (50%)	163 cfs
Flowrate Constraint (25%)	82 cfs
Pollutant Constraint (100%)	26235 lbs
Pollutant Constraint (75%)	19680 lbs
Pollutant Constraint (50%)	13120 lbs
Pollutant Constraint (25%)	6560 lbs

Downtown Subshed

Flowrate Constraint (100%)	272 cfs
Flowrate Constraint (75%)	197 cfs
Flowrate Constraint (50%)	134 cfs
Flowrate Constraint (25%)	68 cfs
Pollutant Constraint (100%)	22018 lbs
Pollutant Constraint (75%)	16514 lbs
Pollutant Constraint (50%)	11009 lbs
Pollutant Constraint (25%)	5505 lbs

Lorraine Subshed

Flowrate Constraint (100%)	133 cfs
Flowrate Constraint (75%)	100 cfs
Flowrate Constraint (50%)	67 cfs
Flowrate Constraint (25%)	33 cfs
Pollutant Constraint (100%)	6093 lbs
Pollutant Constraint (75%)	4570 lbs
Pollutant Constraint (50%)	3047 lbs
Pollutant Constraint (25%)	1523 lbs

Table 4.8 Basin Constraints

Glen Ellyn Site

Maximum Weir Length	5.0 feet
Maximum Basin Width	530 feet
Maximum Basin Length	720 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Pipe Diameter	5.0 feet
Maximum Basin Depth	5.0 feet
Maximum Basin Area	435600 sqft
Maximum Basin Storage	1951488 cuft

Detention Site 1

Maximum Weir Length	5.0 feet
Maximum Basin Width	230 feet
Maximum Basin Length	620 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Pipe Diameter	4.5 feet
Maximum Basin Depth	5.0 feet
Maximum Basin Area	160000 sqft
Maximum Basin Storage	8000000 cuft

Detention Site 2

Maximum Weir Length	5.0 feet
Maximum Basin Width	280 feet
Maximum Basin Length	680 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Pipe Diameter	4.0 feet
Maximum Basin Depth	5.0 feet
Maximum Basin Area	210000 sqft
Maximum Basin Storage	1050000 cuft

4.3.5 Application of the Design Heuristic

The watershed detention system may be designed using interior flow constraints along with constraints at the watershed outlet, or using constraints at the outlet only. In addition, the system may be designed considering the connecting pipe or channel network, or designed assuming that an existing network is already in place. In applying the general design heuristic to the Glen Ellyn watershed, all four of the above possible design considerations were examined. This resulted in four different case studies. In addition, three different flowrate constraints were considered in combination with four different pollutant constraints (including a null constraint). This resulted in a total of twelve different possible designs for each case study. A description of each case study is provided in Table 4.9.

Table 4.9 Description of Case Studies

Case Study	External Constraints	Internal Constraints	Storage Costs	Pipe Costs
1	X		X	
2	X	X	X	
3	X		X	X
4	X	X	X	X

In order to obtain each individual design, the design heuristic was applied using the first design event (5/28/80). This design was then tested using DBSP and the remaining 3 design events. If the design did not satisfy one of the following events, it was eliminated from further consideration. After the first event was used the second design event was evaluated. This process was continued until all of the design events were considered. If more than one acceptable design was obtained, the least cost design was selected. When no acceptable designs were obtained, the best design was modified to produce an acceptable design.

After several simulations, it was determined that the flowrate and pollutant objectives were so conflicting that no single design could be obtained that would satisfy all of the remaining events. As a result of this observation, a composite event was constructed from the peak runoff event (5/29/80) and the peak load event (5/28/80). The resulting composite event did produce designs which did satisfy all of the design events. The hydrograph and pollutograph used in constructing the composite runoff event are presented in Figures 4.9-4.10.

The use of a composite design event greatly increases the probability the resulting design will not be a global optimal design. Given the complexity and the number of design events being considered, it is quite possible that a

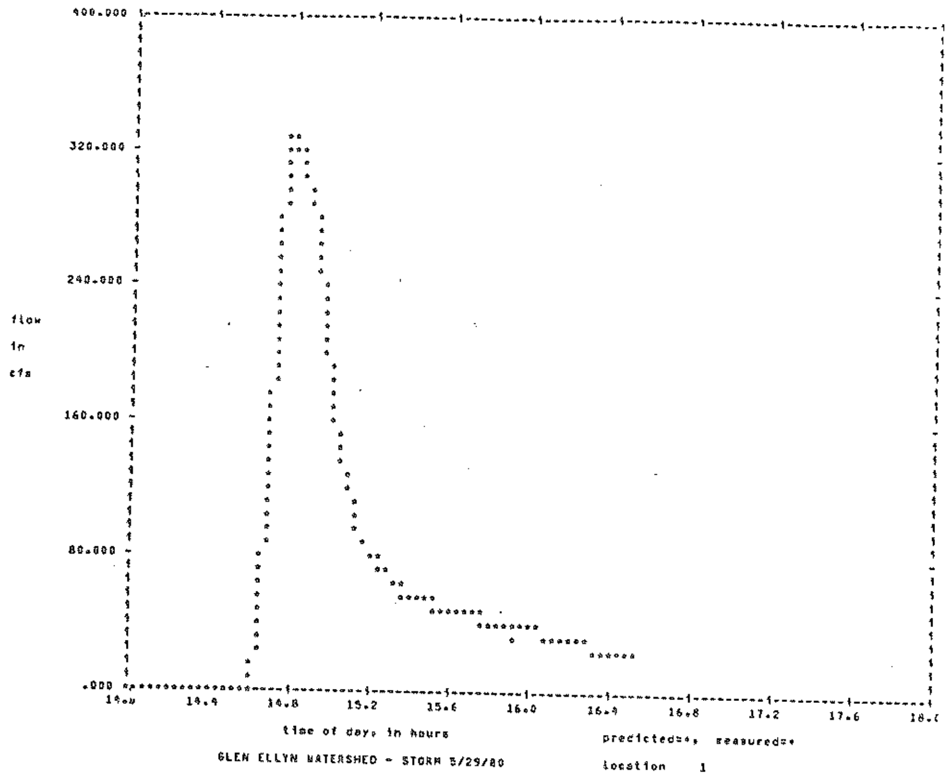


Figure 4.9 Composite Design Hydrograph

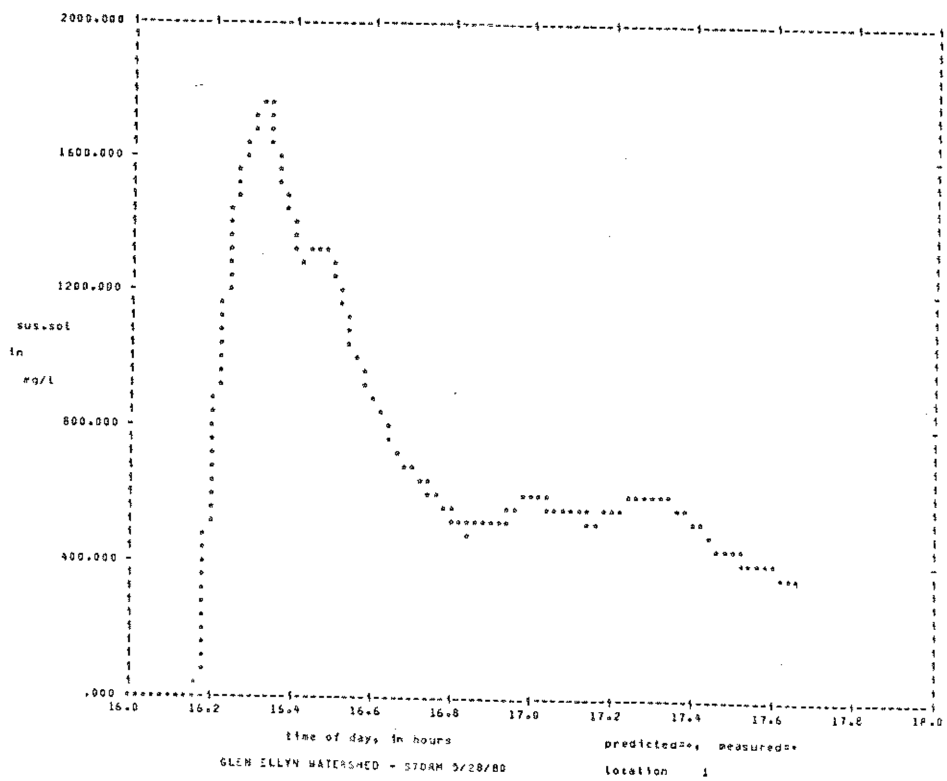


Figure 4.10 Composite Design Pollutograph

better composite design event could be derived. The proposed method, however, is fairly straightforward and does lead to designs which satisfy all of the design constraints. Although it is true that the resulting design may correspond to frequencies greater than the selected design frequency for individual hydrologic statistics, (i.e. peak, total) the objective of the proposed methodology is to yield designs which satisfy the design frequencies of all of the hydrologic parameters and not just one or two.

4.3.6 Discussion of the Results

The results of the application of the general design heuristic are presented in Figures 4.11-4.14 and Tables 4.10-4.17. Figures 4.11-4.14 illustrate the costs of each individual design for a given case study. Tables 4.10-4.17 contain the values of the design parameters and the resulting system variables associated with each design. A brief discussion of the results for each case study is provided below.

4.3.6.1 Case 1

For case 1, flowrate and pollutant load constraints were imposed only at the watershed outlet. In addition, only storage costs were used in the overall cost analysis. As can be seen from Figure 4.11, different cost curves were constructed corresponding to the four different levels of

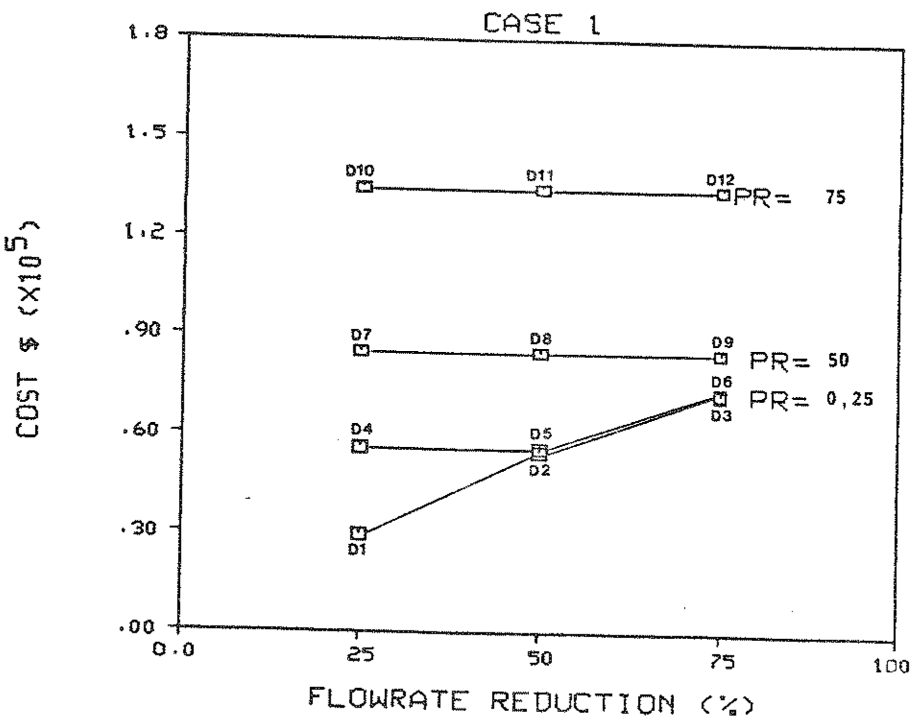


Figure 4.11 Summary of Results for Case 1
(PR = Pollutant Removal %)

Table 4.10 Design Results for Case 1 (Designs D1-D6)

BN # —	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D1	Flow Reduction = 25%						Pollutant Removal = 0%					0.30
LE				238	238	244	25.	25.				
B1	4.4	5.0		204	196		21.	21.		6.8	21.	0.11
B2	2.8	4.8		133	93		6.1	4.7		20.	44.	0.19
D2	Flow Reduction = 50%						Pollutant Removal = 0%					0.55
LE				163	163	163	21.	21.				
B1	4.9	3.6		303	141		22.	17.		45.	192.	0.55
B2				133	133		6.1	6.1				
D3	Flow Reduction = 75%						Pollutant Removal = 0%					0.72
LE				82	82	82	15.	15.				
B1	2.3	4.7		303	71		22.	10.		125.	281.	0.72
B2				133	133		6.1	6.1				
D4	Flow Reduction = 25%						Pollutant Removal = 25%					0.55
LE				172	172	244	19.	19.	20.			
B1	3.8	4.8		303	153		22.	15.		55.0	192.	0.55
B2				133	133		6.1	6.1				
D5	Flow Reduction = 50%						Pollutant Removal = 25%					0.56
LE				163	163	163	20.	20.	20.			
B1	4.0	4.2		303	144		22.	15.		55.4	200.	0.56
B2				133	133		6.1	6.1				
D6	Flow Reduction = 75%						Pollutant Removal = 25%					0.73
LE				82	82	82	15.	15.	20.			
B1	2.5	4.1		303	70		22.	11.	116	282.	0.73	
B2				133	133		6.1	6.1				

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

Table 4.11 Design Results for Case 1 (Designs D7-D12)

BN # --	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D7	Flow Reduction = 25%						Pollutant Removal = 50%					0.85
LE	2.8	3.6		71	71	244	14.	13.	13.	7.82	17.9	0.10
B1	2.0	4.8		302	60		22.	9.1		150.	297.	0.75
B2				133	133		6.1	6.1				
D8	Flow Reduction = 50%						Pollutant Removal = 50%					0.85
LE	2.8	3.6		71	71	163	14.	13.	13.	7.82	17.9	0.10
B1	2.0	4.8		303	60		22.	9.2		150.	297.	0.75
B2				133	133		6.1	6.1				
D9	Flow Reduction = 75%						Pollutant Removal = 50%					0.85
LE	2.8	3.6		71	71	82	14.	13.	13.	7.82	17.9	0.10
B1	2.0	4.8		303	60		22.	9.2		150.	297.	0.75
B2				133	133		6.1	6.1				
10	Flow Reduction = 25%						Pollutant Removal = 75%					1.35
LE	1.7	5.0		74	46	244	14.	6.6	6.6	133.	218.	0.60
B1	2.1	5.0		302	61		22.	9.8		144.	292.	0.75
B2				133	133		6.1	6.1				
11	Flow Reduction = 50%						Pollutant Removal = 75%					1.35
LE	1.7	5.0		74	46	163	14.	6.6	6.6	133.	218.	0.60
B1	2.1	5.0		302	61		22.	9.8		144.	292.	0.75
B2				133	133		6.1	6.1				
12	Flow Reduction = 75%						Pollutant Removal = 75%					1.35
LE	1.7	5.0		74	46	82	14.	6.6	6.6	133.	218.	0.60
B1	2.0	5.0		302	61		22.	9.8		144.	292.	0.75
B2				133	133		6.1	6.1				

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

pollutant removal. While the curve corresponding to the null constraint rises with increased flow reduction, the remaining curves are either totally or partially flat, indicating no change in the cost of the design. The flat cost curves result from the fact that the pollutant constraints are governing the solution space. For example, in order to obtain pollutant removal levels of 50 and 75 percent, designs are required that result in flowrate reductions of more than 75 percent. These designs also satisfy the flowrate reduction constraints of 50 and 25 percent and thus the cost of the design remains the same.

In contrast to the pollutant constraints, control of the solution space by the flowrate constraint is indicated by cases where the cost of a given design does not increase with an increase in pollutant removal. This condition is illustrated in Figure 4.11 with designs D2 and D3. As can be seen from the figure, the cost of designs D2 and D3 do not substantially increase when a 25 percent pollutant removal constraint is enforced. This indicates that the designs corresponding to the flowrate reductions of 50 and 75 percent already provide pollutant removal levels of at least 25 percent.

Designs that are not dominated by either a pollutant or flowrate constraint are indicated by designs D1 and D5. As can be seen from Figure 4.11, the cost of these designs increase when going to the next pollutant removal level or the next flowrate reduction level.

An examination of Tables 4.10 and 4.11 reveals that for low pollutant removal levels (i.e. when flowrate constraints tend to be dominate) detention basins are placed in the upper part of the watershed. However, when pollutant constraints are controlling, detention basins are placed in the lower part of the watershed. In addition, as flowrate reduction is increased, the incremental costs of the resulting designs tend to remain the same, while as pollutant removal is increased the incremental costs of the resulting designs tend to increase.

4.3.6.2 Case 2

For case 2, flowrate reduction and pollutant removal constraints were imposed throughout the watershed. As can be seen from Figure 4.12 the results of case 2 are very similar to case 1. For case 2, the 75 percent pollutant removal level was not attainable and thus no cost curve is illustrated.

In general, the costs of the designs of case 2 are higher than the costs of the corresponding designs of case 1. This result would tend to be expected given the increased constraints on the overall problem. However, unlike case 1, all of the designs for case 2 consist of

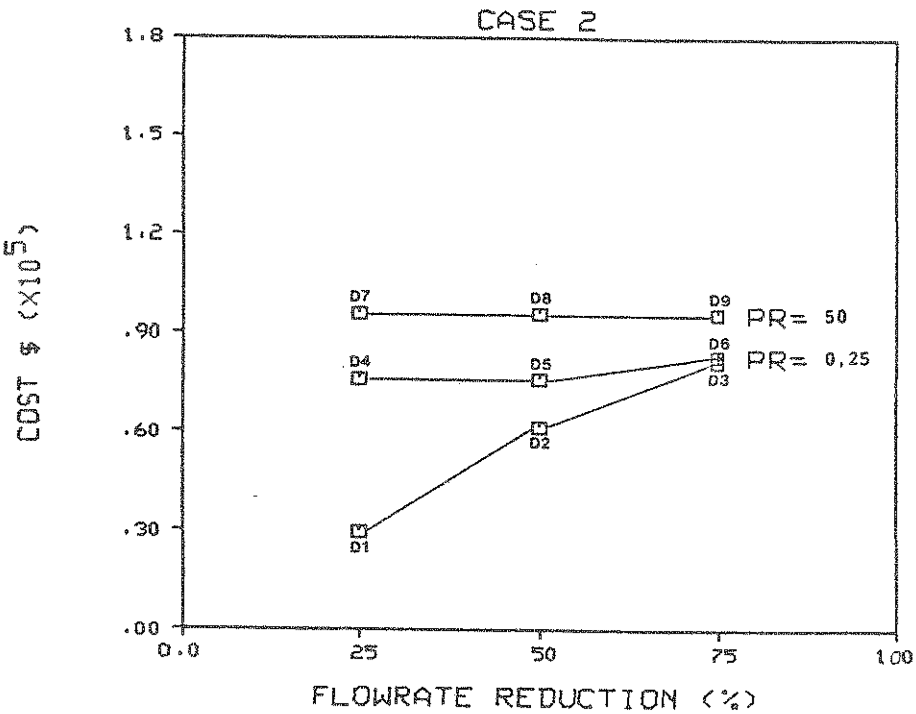


Figure 4.12 Summary of Results for Case 2
(PR = Pollutant Removal %)

Table 4.12 Design Results for Case 2 (Designs D1-D6)

BN #	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D1	Flow Reduction = 25%						Pollutant Removal = 0%					0.30
LE				238	238	244	25.	25.				
B1	4.4	5.0		204	196	197	21.	21.		6.8	21.	0.11
B2	2.8	4.8		133	93	100	6.1	4.7		20.	44.	0.19
D2	Flow Reduction = 50%						Pollutant Removal = 0%					0.61
LE				156	156	163	21.	21.				
B1	4.3	3.6		204	127	134	20.	17.		27.	99.	0.34
B2	2.8	3.1		133	63	67	6.1	3.8		29.	74.	0.27
D3	Flow Reduction = 75%						Pollutant Removal = 0%					0.81
LE				76	76	82	16.	16.				
B1	2.2	4.5		204	64	68	18.	12.		70.	149.	0.45
B2	1.7	3.2		133	32	33	6.1	2.1		70.	106.	0.36
D4	Flow Reduction = 25%						Pollutant Removal = 25%					0.68
LE				95	95	244	20.	20.	20.			
B1	3.3	4.8	0.0	297	124	197	20.	15.	17.	40.6	124.	0.40
B2	2.6	3.4	0.0	133	60	100	6.1	3.3	4.6	32.7	78.2	0.28
D5	Flow Reduction = 50%						Pollutant Removal = 25%					0.68
LE				95	95	163	20.	20.	20.			
B1	3.3	4.8	0.0	297	124	134	20.	15.	17.	40.6	124.	0.40
B2	2.6	3.4	0.0	133	60	67	6.1	3.3	4.6	32.7	78.2	0.28
D6	Flow Constraint = 75%						Pollutant Removal = 25%					0.80
LE				82	82	82	16.	16.	20.			
B1	2.2	3.7	0.0	204	68	68	18.	12.	17.	69.9	143.	0.45
B2	1.7	3.4	0.0	133	33	33	6.1	2.3	4.6	63.4	105.	0.35

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

Table 4.13 Design Results for Case 2 (Designs D7-D12)

BN	DPTH	ORF	DIA	QIN	QOT	QMX	PIN	POT	PMX	AREA	STOR	COST
#	ft	ft	ft	cfs	cfs	cfs	lbs	lbs	lbs	sqft	cuft	$
--	+E0	+E0	+E0	+E0	+E0	+E0	+E3	+E3	+E3	+E3	+E3	+E5
D7	Flow Reduction = 25%						Pollutant Removal = 50%					0.96
LE				55	55	244	13.	13.	13.			
B1	1.4	5.0	0.0	204	34	197	18.	7.9	11.	148.	198.	0.56
B2	1.1	4.1	0.0	133	20	100	6.1	2.1	3.0	119.	126.	0.40
D8	Flow Reduction = 50%						Pollutant Removal = 50%					0.96
LE				55	55	163	13.	13.	13.			
B1	1.4	5.0	0.0	204	34	134	18.	7.9	11.	148.	198.	0.56
B2	1.1	4.1	0.0	133	20	67	6.1	2.1	3.0	119.	126.	0.40
D9	Flow Reduction = 25%						Pollutant Removal = 50%					0.96
LE				55	55	82	13.	13.	13.			
B1	1.4	5.0	0.0	204	34	68	18.	7.9	11.	148.	198.	0.56
B2	1.1	4.1	0.0	133	20	33	6.1	2.1	3.0	119.	126.	0.40
10	Flow Reduction = 25%						Pollutant Removal = 75%					INF
LE						244			6.6			
B1						197			5.5			
B2						100			1.5			
11	Flow Reduction = 50%						Pollutant Removal = 75%					INF
LE						163			6.6			
B2						134			5.5			
B1						67			1.5			
12	Flow Reduction = 75%						Pollutant Removal = 75%					INF
LE						82			6.6			
B1						68			5.5			
B2						33			1.5			

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol

detention basins in the upper part of the watershed (i.e. the Lake Ellyn Site was never selected).

4.3.6.3 Case 3

For case 3, flowrate reduction and pollutant removal constraints were again imposed only at the outlet. However in this case, pipe costs were also included in the total cost analysis.

Inclusion of the pipe cost in the study produced an interesting result. As can be seen from Figure 4.13, the designs for pollutant removal levels of 0, 25, and 50 percent are all the same for each level of flowrate reduction. This result is due to the fact the incremental pipe cost is greater than the incremental storage cost and thus the pipe cost is the controlling factor in the design. In order to decrease the cost of the required downstream pipe the upstream storage is increased so that the resulting outflow from each basin is decreased. For this case study, the trade-off between storage and pipe cost yields a design which satisfies the pollutant removal constraint up to a level of 50 percent for a flowrate reduction constraint up to a level of 75 percent. This design does not satisfy the 75 percent pollutant removal level and so therefore a new design is required. As can be seen from Figure 4.13, this new design is controlled by the pollutant constraint and thus the corresponding cost curve is horizontal.

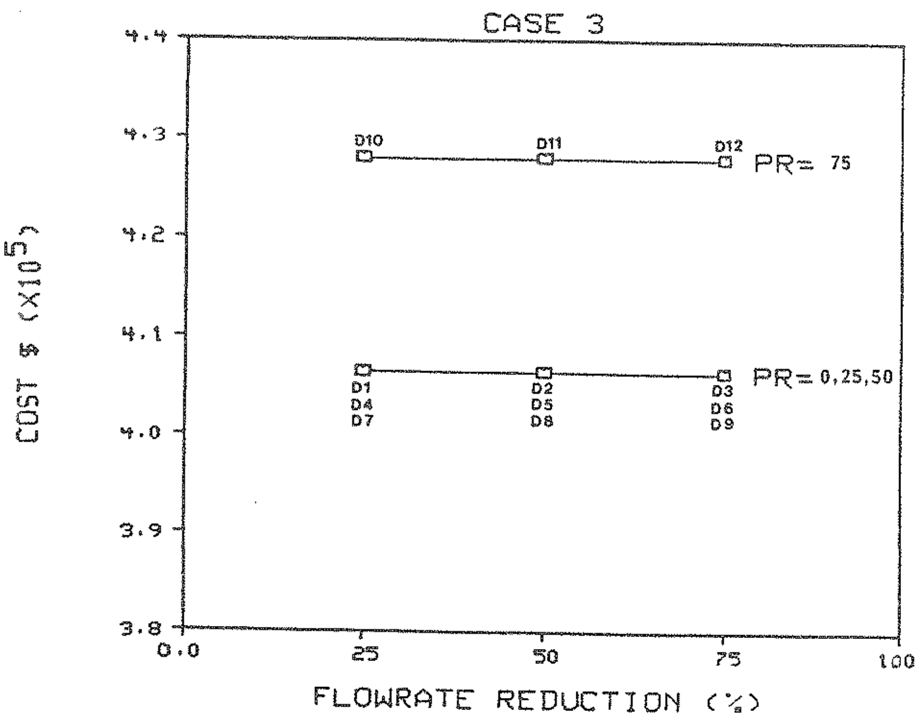


Figure 4.13 Summary of Results for Case 3
(PR = Pollutant Removal %)

Table 4.14 Design Results for Case 3 (Designs D1-D6)

BN # --	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D1	Flow Reduction = 25%						Pollutant Removal = 0%					4.07
LE				57	57	244	13.	13.				
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D2	Flow Reduction = 50%						Pollutant Removal = 0%					4.07
LE				57	57	163	13.	13.				
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D3	Flow Reduction = 75%						Pollutant Removal = 0%					4.07
LE				57	57	82	13.	13.				
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D4	Flow Reduction = 25%						Pollutant Removal = 25%					4.07
LE				57	57	244	13.	13.	20.			4.07
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D5	Flow Reduction = 50%						Pollutant Removal = 25%					4.07
LE				57	57	163	13.	13.	20.			
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D6	Flow Reduction = 75%						Pollutant Removal = 25%					4.07
LE				57	57	82	13.	13.	20.			
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

Table 4.15 Design Results for Case 3 (Designs D7-D12)

BN # --	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D7	Flow Reduction = 25%						Pollutant Removal = 50%					4.07
LE				57	57	244	13.	13.	13.			
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
D8	Flow Reduction = 50%						Pollutant Removal = 50%					4.07
LE				57	57	163	13.	13.	13.			
B1	3.0	1.0	18	204	10		18.	8.1	11.	99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9	3.0	170.	194.	2.94
D9	Flow Reduction = 75%						Pollutant Removal = 50%					4.07
LE				57	57	82	13.	13.	13.			
B1	3.0	1.0	18	204	10		18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5		6.1	1.9		170.	194.	2.94
10	Flow Reduction = 25%						Pollutant Removal = 75%					4.28
LE	0.9	4.9		56	16	244	13.	6.1	6.6	62.	51.	0.21
B1	2.7	1.0	18	204	10		18.	8.0		121.	312.	1.20
B2	0.9	2.9	18	133	10		6.1	2.0		190.	165.	2.87
11	Flow Reduction = 50%						Pollutant Removal = 75%					4.28
LE	0.9	4.9		56	16	163	13.	6.1	6.6	62.	51.	0.21
B1	2.7	1.0	18	204	10		18.	8.0		121.	312.	1.20
B2	0.9	2.9	18	133	10		6.1	2.0		190.	165.	2.87
12	Flow Reduction = 75%						Pollutant Removal = 75%					4.28
LE	0.9	4.9		56	16	82	13.	6.1	6.6	62.	51.	0.21
B1	2.7	1.0	18	204	10		18.	8.0		121.	312.	1.20
B2	0.9	2.9	18	133	10		6.1	2.0		190.	165.	2.87

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

Similar to case 1, the designs for pollution removal levels of 0 and 25 percent contain detention basins only in the upper part of the watershed. For the 75 percent removal level, only the Lake Ellyn site is used.

4.3.6.4 Case 4

For case 4, flowrate and pollutant constraints were imposed throughout the watershed. As in case 3, pipe costs were also included in the overall cost analysis. Similar to case 2, the 75 percent pollutant removal level was not attainable. Similar to case 3, the pipe costs were again the controlling factor in the overall design. This led to a design which satisfies the pollutant removal constraint up to a level of 50 percent for a flowrate reduction constraint up to a level of 75 percent. In fact, the design for case 4 is the same design as for case 3. Thus the design resulting from the trade-off between the storage and pipe costs satisfies the pollutant removal constraint up to a level of 50 percent for a flowrate reduction constraint up to a level of 75 percent for the entire watershed and not just at the watershed outlet.

4.4 Summary and Conclusions

The general planning methodology was applied to the Glen Ellyn watershed which is located in Glen Ellyn, Illinois. Four different case studies were construct based

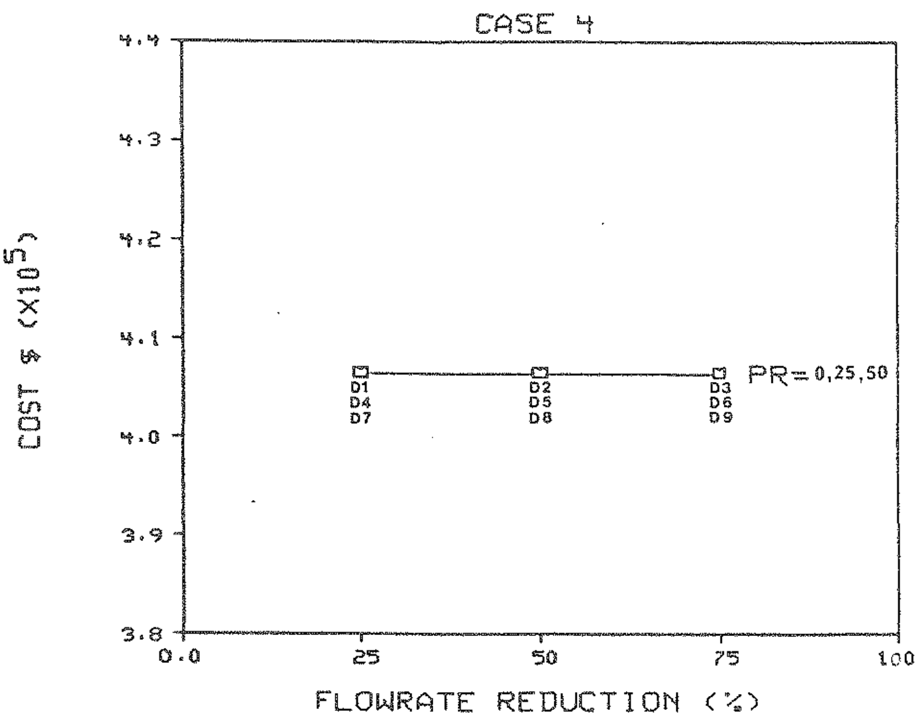


Figure 4.14 Summary of Results for Case 4
(PR = Pollutant Removal %)

Table 4.16 Design Results for Case 4 (Designs D1-D6)

BN #	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D1	Flow Reduction = 25%						Pollutant Removal = 0%					4.07
LE				57	57	244	13.	13.				
B1	3.0	1.0	18	204	10	197	18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5	100	6.1	1.9		170.	194.	2.94
D2	Flow Reduction = 50%						Pollutant Removal = 0%					4.07
LE				57	57	163	13.	13.				
B1	3.0	1.0	18	204	10	134	18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5	67	6.1	1.9		170.	194.	2.94
D3	Flow Reduction = 75%						Pollutant Removal = 0%					4.07
LE				57	57	82	13.	13.				
B1	3.0	1.0	18	204	10	68	18.	8.1		99.	276.	1.13
B2	1.2	1.0	18	133	5	33	6.1	1.9		170.	194.	2.94
D4	Flow Reduction = 25%						Pollutant Removal = 25%					4.07
LE				57	57	244	13.	13.	20			
B1	3.0	1.0	18	204	10	197	18.	8.1	17.	99.	276.	1.13
B2	1.2	1.0	18	133	5	100	6.1	1.9	4.6	170.	194.	2.94
D5	Flow Reduction = 50%						Pollutant Removal = 25%					4.07
LE				57	57	163	13.	13.	20.			
B1	3.0	1.0	18	204	10	134	18.	8.1	17.	99.	276.	1.13
B2	1.2	1.0	18	133	5	67	6.1	1.9	4.6	170.	194.	2.94
D6	Flow Reduction = 75%						Pollutant Removal = 25%					4.07
LE				57	57	82	13.	13.	20.			
B1	3.0	1.0	18	204	10	68	18.	8.1	17.	99.	276.	1.13
B2	1.2	1.0	18	133	5	33	6.1	1.9	4.6	170.	194.	2.94

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

Table 4.17 Design Results for Case 4 (Designs D7-D12)

BN # --	DPTH ft +E0	ORF ft +E0	DIA ft +E0	QIN cfs +E0	QOT cfs +E0	QMX cfs +E0	PIN lbs +E3	POT lbs +E3	PMX lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D7	Flow Reduction = 25%						Pollutant Removal = 50%					4.07
LE				57	57	244	13.	13.	13.			
B1	3.0	1.0	18	204	10	197	18.	8.1	11.	99.	276.	1.13
B2	1.2	1.0	18	133	5	100	6.1	1.9	3.0	170.	194.	2.94
D8	Flow Reduction = 50%						Pollutant Removal = 50%					4.07
LE				57	57	163	13.	13.	13.			
B1	3.0	1.0	18	204	10	134	18.	8.1	11.	99.	276.	1.13
B2	1.2	1.0	18	133	5	67	6.1	1.9	3.0	170.	194.	2.94
D9	Flow Reduction = 75%						Pollutant Removal = 50%					4.07
LE				57	57	82	13.	13.	13.			
B1	3.0	1.0	18	204	10	68	18.	8.1	11.	99.	276.	1.13
B2	1.2	1.0	18	133	5	33	6.1	1.9	3.0	170.	194.	2.94
10	Flow Reduction = 25%						Pollutant Removal = 75%					INF
LE						244			6.6			
B1						197			5.5			
B2						100			1.5			
11	Flow Reduction = 50%						Pollutant Removal = 75%					INF
LE						163			6.6			
B1						134			5.5			
B2						67			1.5			
12	Flow Reduction = 75%						Pollutant Removal = 75%					INF
LE						82			6.6			
B1						68			5.5			
B2						33			1.5			

BN = Basin Number DPTH = Basin Depth ORF = Orifice Width DIA = Pipe Diameter
AREA = Basin Area QIN = Flow In QOT = Flow Out QMX = Maximum Flow
STOR = Basin Storage PIN = Pollutant In POT = Pollutant Out PMX = Maximum Pol.

on different cost and constraint selections. Associated with each case study were 16 different designs which were derived based on different levels of pollutant and flow constraints. The resulting designs were obtained by application of the general design heuristic and a composite design event. The composite design event was obtained as a result of a statistical analysis of 18 months of simulated runoff and pollutant loadings.

The design of a detention basin system has been shown to involve a trade-off between storage and pipe costs. In this particular study, the pipe costs were the controlling factor in the overall design. In the absence of pipe costs, the overall design may be dominated by either the flowrate constraint or the pollutant constraint. In general, designs corresponding to high pollutant removal levels and low flowrate levels tend to be dominated by the pollutant constraint. Likewise, designs corresponding to high flowrate reduction levels and low pollutant levels tend to be dominated by the flowrate constraint. In addition to these two regions, there tends to be a middle region where neither constraint is dominant. The degree of control of the solution space by either constraint will depend on the specific case study being investigated. For case studies 1 and 2, the pollutant constraint tended to be more dominant than the flowrate constraint. In addition, the incremental cost of the pollution removal tended to be higher than the incremental cost of flowrate reduction.

V. SYNTHETIC WATERSHED APPLICATION

5.1 INTRODUCTION

In addition to applying the planning methodology to a specific watershed, the methodology could also be used to obtain general planning guidelines for a specific region or area. Such an application could involve the use of synthetic watersheds and average design parameters. Based on the results of such an application, an attempt could be made to derive general planning indices for use in the preliminary design of watershed detention systems.

This chapter provides an illustration of the possible application of the methodology to a synthetic watershed. The synthetic watershed is derived based on average values of watershed parameters obtained for the state of Indiana. The necessary data required to construct and analyze the synthetic watersheds include geomorphic data and hydrologic data. A brief discussion of both geomorphic considerations and hydrologic considerations is provided in the following sections.

5.2 GEOMORPHOLOGY CONSIDERATIONS

A famous paper by Horton (1945) laid the foundation for much of the subsequent work in quantitative geomorphology of drainage basins. In particular, Horton made two major contributions to the study of stream patterns. First, he devised a system of stream classification or ordering, which proved to be very useful in the quantitative discussion of drainage composition. Second, he developed two laws for stream numbers and stream lengths. Additional laws were later developed for both basin area and basin slope.

A major criticism of Hortons's work is that the stream ordering scheme is very insensitive to variations in structure and lithology. Bifurcation ratios were found to be remarkably stable from one area to another, and generally cluster in the range of 3.5 to 4.0. In an attempt to generate a more sensitive ordering scheme and a model devoid of Horton's inconsistencies, Shreve (1966) proposed a random model based solely upon combinatorial properties. From this initial formulation, Shreve (1969) and Smart (1968) have proceeded to derive laws of stream lengths and areas based largely upon the postulates of the random topology model. A brief review of Horton's laws as well as the random topology model is presented below.

5.2.1 Horton's Stream Classification

For all practical purposes, the quantitative study of channel networks began with Horton's method of classifying channels by order. Later on, Strahler (1952) proposed a modification of Horton's ordering scheme. Strahler's method is now generally preferred because of its simplicity and greater freedom from subjective decisions (Smart, 1972).

If stream channels are idealized as single lines, the resulting diagram is known in geomorphic literature as a channel network. Sources are the points farthest upstream in a channel network, and the outlet is the point farthest downstream. The point at which two channels combine to form one is called a junction.

The Strahler ordering procedure may be described as follows: (1) channels that originate at a source are defined to be first order streams; (2) when two streams of order ω join, a stream of order $\omega + 1$ is created; (3) when two streams of different order join, the channel segment immediately downstream has the higher of the orders of the two combining streams. An example of a the Strahler ordering scheme is presented in Figure 5.1.

5.2.2 Horton's Laws

Using his stream ordering procedure, Horton was able to develop several basic laws of drainage composition. The law of stream numbers states that:

$$R_b = \frac{N_\omega}{N_{\omega+1}} \tag{5.1}$$

where the ratio of the number of segments of a given order N_ω to the number of segments of the higher order $N_{\omega+1}$ is termed the bifurcation ratio R_b. Observations on natural networks indicate that when using the Strahler ordering scheme, the bifurcation ratio is usually between 3 and 5.

The second of Hortons's laws is the law of stream lengths which states that:

$$R_l = \frac{\bar{L}_\omega}{\bar{L}_{\omega-1}} \tag{5.2}$$

The length ratio R_l (which is the ratio of the mean length $\bar{L}_\omega$ of segments of order ω to mean length of segments of the next lower order $\bar{L}_{\omega-1}$) tends to be constant throughout the successive orders of a watershed. When using the Strahler ordering scheme, the stream length ratio usually ranges between 1.5 and 3.5.

Horton also suggested that there should be an analogous relationship for areas, and one was later stated explicitly by Schumm (1956). This relationship may be stated as :

$$R_a = \frac{\bar{A}_\omega}{\bar{A}_{\omega-1}} \tag{5.3}$$

where $\bar{A}_\omega$ is the mean area drained by streams of order ω (including their tributaries of lower order) and R_A is the basin area ratio. The Strahler basin area ratio, R_A, normally falls in a range between 3 and 6.

A fourth law of drainage composition is the law of stream slopes. Horton and many others since have found empirically that, in general

$$R_s = \frac{\bar{S}_\omega}{\bar{S}_{\omega-1}} \tag{5.4}$$

From measurements on three geologically mature streams in a humid climate, Horton (1945) found $R_s = 0.55$, while for a younger stream in a semiarid climate Broscoe measurements (1959) give $R_s = 0.57$.

A final geomorphic relationship concerns the relation between basin area and basin length. Hack (1957) reported that his measurements on 90 drainage basins in Virginia and Maryland and measurements by Langbein et al. (1947) on about 400 basins in the northeastern United States could be well represented by the following relationship.

$$L = 1.4\ A^{0.6} \tag{5.5}$$

where L is stream length in miles measured to a point on the drainage divide, and A is area in square miles. Later work by Gray (1961) gave essentially the same results. In a study of 14 watersheds in Indiana, Lee and Delleur (1972) obtained the following relationship.

$$L = 1.64 A^{0.55} \tag{5.6}$$

5.2.3 Shreve Network Classification

In the Shreve network classification scheme, it is assumed that multiple junctions do not occur. An exterior link is a segment of channel network between a source and the first junction downstream; an interior link is a segment of channel network between two successive junctions or between the outlet and the first junction upstream. A channel network with n sources has n exterior links, n-1 interior links, and n-1 junctions. The magnitude μ of a link is the number of sources upstream; thus an exterior link has magnitude unity and an interior link has a magnitude that is the sum of the magnitudes of the two links joining at its upstream end. The magnitude of a channel network is that of its outlet link. An example of the Shreve network ordering scheme is shown in Figure 5.2.

5.2.4 Development of The Random Topology Model

The properties of a random topology stream network model were first introduced by Shreve (1966). Shreve noted that channel networks with equal numbers of sources are comparable in topological complexity because they also have

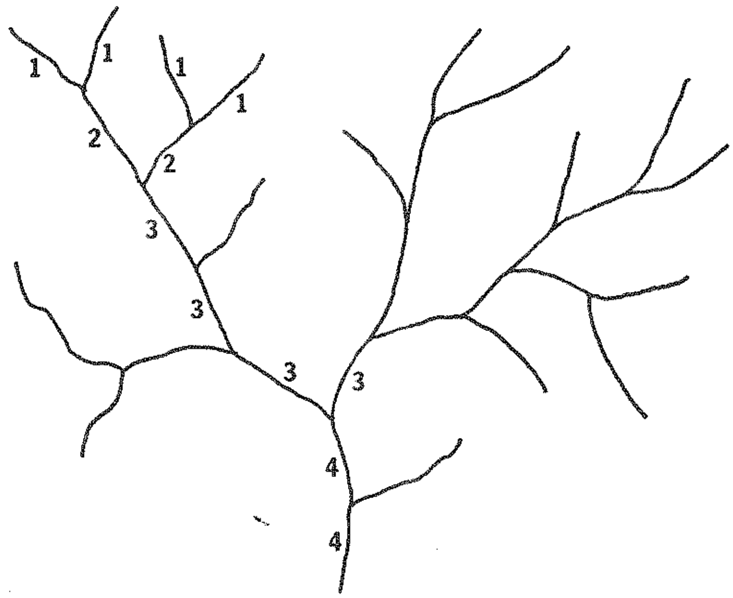


Figure 5.1 Strahler Network Ordering Scheme

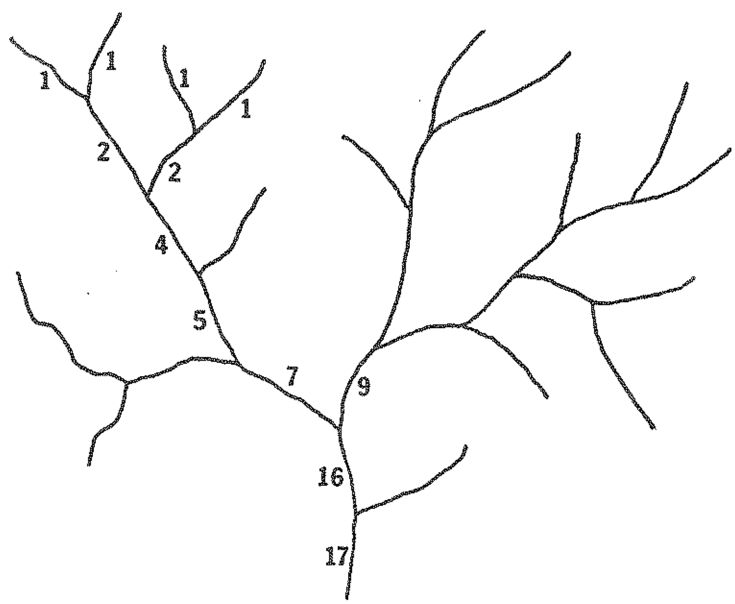


Figure 5.2 Shreve Network Ordering Scheme

equal numbers of links, junctions, and first-order Strahler streams. If a stream network possesses μ sources (i.e., first order streams), there must exist $N(\mu)$ topologically distinct channel networks (TDCN's) as a result of the following relationship.

$$N(\mu) = \frac{1}{2\mu-1}\binom{2\mu-1}{\mu} \tag{5.7}$$

A TDCN is defined as a network that when it is projected on a plane surface, cannot be continuously deformed or rotated with that surface such that it becomes congruent with any other TDCN of the same number of sources. The 14 possible TCDN for μ=5 are shown in Figure 5.3.

Because of the large values of $N(\mu)$ for even relatively small μ, some method of grouping TCDN into classes is required before much quantitative investigation can be done. Smart (1973) has suggested that TCDN of the same magnitude be grouped according to ambilateral classes. Two channel networks belong to the same ambilateral class if and only if they can be made topologically identical by reversals of the right-left order at one or more junctions. Magnitude 5 networks, for example, have three ambilateral classes, corresponding to the first eight, the next four, and the last two networks as illustrated in Figure 5.3. Smart argued that although hydrologic variables such as discharge and sediment load might depend on network topology, they should be essentially independent of the right-left order or

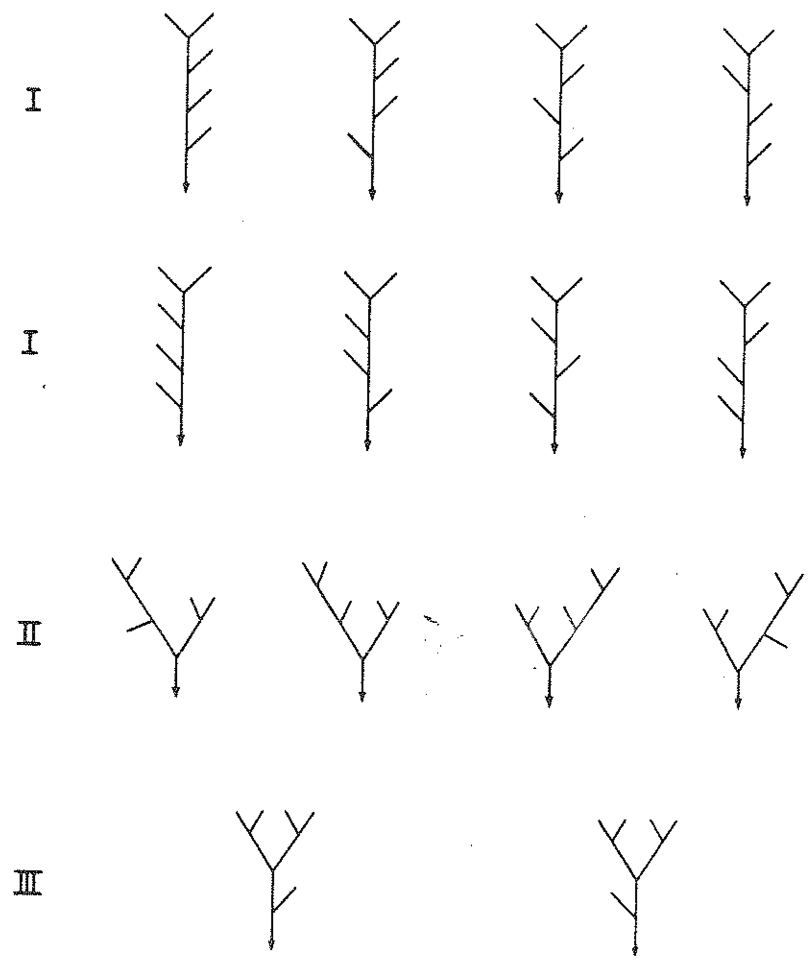


Figure 5.3 Topologically Distinct Channel Networks ($\mu = 5$)

subnetworks at the junctions. Because reversing the right-left arrangement leaves the link magnitudes unchanged, all networks of the same ambilateral class have the same set of magnitudes.

5.2.5 Postulates of the Random Topology Model

The first postulate of the the random model is that in an area uniform in lithology and free from structural controls each TCDN will occur with equal frequency. Thus in the absence of geologic controls, channel networks are topologically random (Shreve, 1966).

The second postulate of the random model was proposed by Smart (1968). For drainage basins under comparable environmental conditions, Smart suggested that the exterior and interior link lengths are independent random variables with a single common distribution for each type. In an investigation of the lengths of exterior and interior links in 12 disparate areas, Abrahams and Miller (1982) derived a mixed gamma density for link lengths by assuming that both the component link length distribution for each relatively homogeneous part of the landscape and the mixing distribution of weights assigned to the various component distributions can be represented by gamma distributions.

The first two assumptions of the random model are analogous to the first two Horton laws in the sense that the first deals with purely topologic properties and the second deals with length properties. Although some attempts have been made to develop a general postulate relating to areal properties, the results have been inconclusive. A relationship has been proposed, however, relating the basin length to the basin area. This relationship may be expressed as follows.

$$K = \frac{L^2}{(2\mu-1)A} \qquad (5.8)$$

where L is the total channel length, A is the total drainage area, and μ is the magnitude of the network. As a third basic postulate of the random model, Smart (1973) has proposed that K is equal to 1.

Finally, in an extension of the random model, Flint (1976) formulated a general model representing the distribution and expectation of interior link slopes for an entire channel network. In a study of 11 drainage basins in the Appalachian Plateau, Flint found that the average log link slope $\hat{s}_\mu$ varied with the number of links present in the sub-network of magnitude μ following a relation of the form

$$\hat{s}_\mu = k + r\log(2\mu - 1) \qquad (5.9)$$

where k is equal to the stream gradient for $\mu = 1$ and r is the rate of change in the stream gradient with magnitude.

5.2.6 The WATER Computer Program

In order to determine the various geomorphic parameters associated with a given watershed, large amounts of data must be collected and then analyzed. This can be a very time consuming process, especially for large watersheds. In order to facilitate the application of advanced fluvial analysis to actual problems of water management and river control, Coffman et al. (1971) developed WATER, the Water and Terrain Evaluation Research program. Using digitized data from topographic maps, WATER can determine 28 different statistics for a given watershed. Statistics may be obtained based on either the Strahler stream ordering method or the Shreve stream ordering scheme. The output from the program can thus be used to obtain regional parameters for various groups of watersheds.

Lee and Delleur (1972) have applied the WATER program to a data base of geomorphic data for the state of Indiana. The total data base contains network data for 34 watersheds and topographic data for 38 watersheds. For this study, 16 watersheds were analyzed. A summary of the results of the analysis is provided in Table 5.1. A map showing the location of the 16 watersheds is provided in Figure 5.4.

Figure 5.4 Indiana Map of Selected Watersheds

Table 5.1 Summary of Indiana Geomorphic Data

Stream Link	Number Obs.	Mean Stream Length (mi)	Number Obs.	Mean Stream Slope ft/ft
1	16916	.1320	3258	.0143
2	4063	.1182	748	.0125
3	1981	.1266	391	.0139
4	1251	.1256	259	.0146
5	884	.1332	173	.0172
6	665	.1286	128	.0135
7	538	.1287	106	.0152
8	439	.1272	72	.0181
9	365	.1289	69	.0169
10	311	.1248	56	.0197
11	271	.1180	48	.0184
12	213	.1247	41	.0185
13	205	.1625	32	.0164
14	187	.1384	27	.0131
15	160	.1315	21	.0317
16	153	.1499	24	.0297
17	132	.1217	21	.0345
18	132	.1307	21	.0171
19	116	.1377	17	.0115
20	98	.1230	15	.0367

5.2.7 Urban Stream Network Topology

Very little research has been conducted in the area of urban stream network topology. One of the few authors who has investigated the subject is Graf (1977). Graf's research was based on data derived from a small instrumented drainage basin near Iowa City, Iowa. As a result of urbanization, Graf found that the mean length of external stream links decreased while the mean length of internal stream links remained the same. In addition, Graf determined that internal links become more significant than external links in terms of length and drainage area. Graf also found that the shape of a subbasin tends to become more rectangular with increased urbanization. Using the data obtained from the instrumented drainage basin, Graf derived several regression equations relating geomorphic parameter values to the degree of urbanization of a watershed. Equations for both exterior and interior link lengths are provided below.

$$Le_u = Le_n (1 + 3.195 P_u) \tag{5.10}$$

$$Li_u = Li_n (1 + 2.217 P_u) \tag{5.11}$$

where Le_u = Total length of urbanized exterior links

Le_n = Total length of natural exterior links

Li_u = Total length of urbanized interior links

Li_n = Total length of natural interior links

P_u = Percent of urbanization

5.3 HYDROLOGIC CONSIDERATIONS

The hydrologic data necessary to construct and analyze a synthetic watershed include land use data, pollutant loading data, and precipitation data. A brief discussion of each type of data used in the development of the synthetic watershed is presented below.

5.3.1 Land Use Data

Average land uses for major cities in the state of Indiana may be obtained from Table 5.2 (Heaney, 1977). The percent imperviousness associated with a given land use may be approximated using Table 5.3 (USDA-SCS, 1975).

5.3.2 Pollutant Loading Data

Initial pollutant loadings for different land uses may be obtained using Table 5.4 (Manning et al., 1977, and APWA 1969). These pollutant loadings may be related to land use areas by using Table 5.5 (Heaney et al., 1977).

5.3.3 Precipitation Data

Rainfall data for various stations in the state of Indiana may be obtained from NOAA. The standard data format is to record hourly rainfall values in hundredths of an inch on days when there is rain. Days without rain are not recorded on the tape. Hourly data for the first day of each

Table 5.2 Land Use Percentages for Major Cities in Indiana

City	UNDV	INST	RESD	INDL	COMM
Anderson	27.3	13.2	42.4	10.8	6.2
Chicago Metro	41.1	10.7	34.4	8.7	5.1
Evansville	39.8	11.0	35.2	8.9	5.2
Fort Wayne	42.1	10.5	33.8	8.6	5.0
Indianapolis	56.5	7.9	25.4	6.4	3.7
Lafayette	33.1	12.2	39.0	9.9	5.7
Muncie	38.4	11.2	36.0	9.1	5.3
South Bend	47.6	9.5	30.6	7.8	4.5
Terra Haute	51.1	8.9	28.6	7.2	4.2
Avg for State	47.1	9.6	30.9	7.8	4.6

Table 5.3 Percent Impervious Associated With a Specified Land Use

Land Use	Percent Impervious
Undeveloped	5.0
Institutional	20.0
Residential	50.0
Industrial	72.0
Commercial	85.0

Table 5.4 Pollutant Loadings vs Land Uses

Land Use	Pollutant Loadings lb/curb - mi/day				
	TSS	BOD_5	COD	PO_4	Tot-N
Undeveloped	79.0	.396	1.584	.0008	.0039
Institutional	79.0	.396	1.584	.0008	.0039
Residential	88.0	.378	3.520	.0044	.0480
Industrial	319.0	.729	12.76	.0096	.1372
Commercial	116.0	1.340	6.786	.0122	.0713

Table 5.5 Curb Miles/Acre vs Land Uses

Land Use	Curb Mile/Acre
Undeveloped	0.023
Institutional	0.030
Residential	0.059
Industrial	0.034
Commercial	0.070

month are recorded regardless of whether it rained or not. Currently, many stations have over 25 years of data.

5.5 SYNTHETIC WATERSHED CONSTRUCTION

Using the data from Table 5.1, a typical watershed representative of watersheds in Indiana was constructed for use in the application of the planning methodology. For the purpose of this study, average parameter values for the entire state were used. Using mean link lengths and slopes, a simple network configuration was constructed. The subshed areas associated with the various channel lengths were then obtained by application of the mainstream length to area relationship described previously. The constructed watershed is shown in Figure 5.5. A conceptualization of the watershed is shown in Figure 5.6.

The synthetic watershed was simulated in both a developed and undeveloped condition. For the natural state, infiltration parameters were selected assuming a hydrologic soil group of C. A listing of the assumed geomorphic and hydrologic parameters for the natural watershed is provided in Table 5.6.

For the developed condition, the original watershed was modified using the regression relationships developed by Graf (1977). Since these relationships were developed from

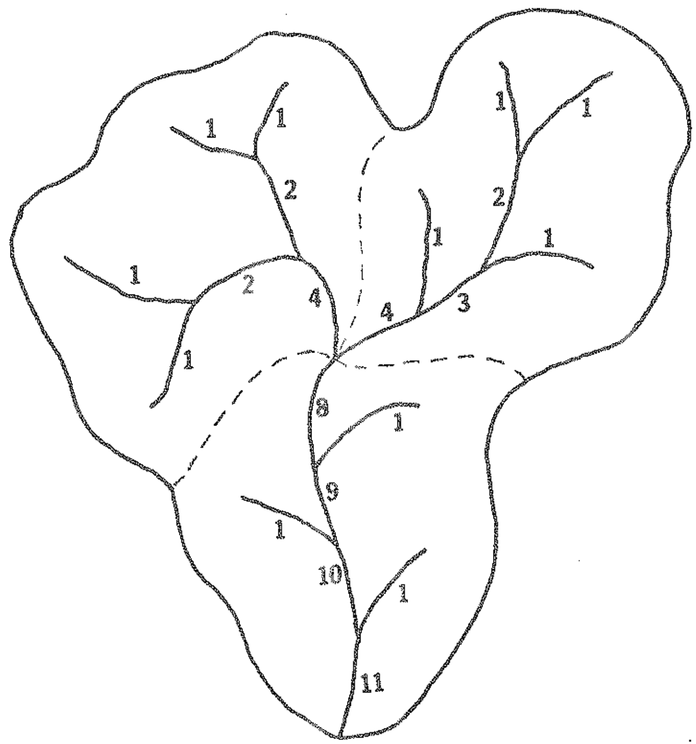


Figure 5.5 Map of Synthetic Waterhsed

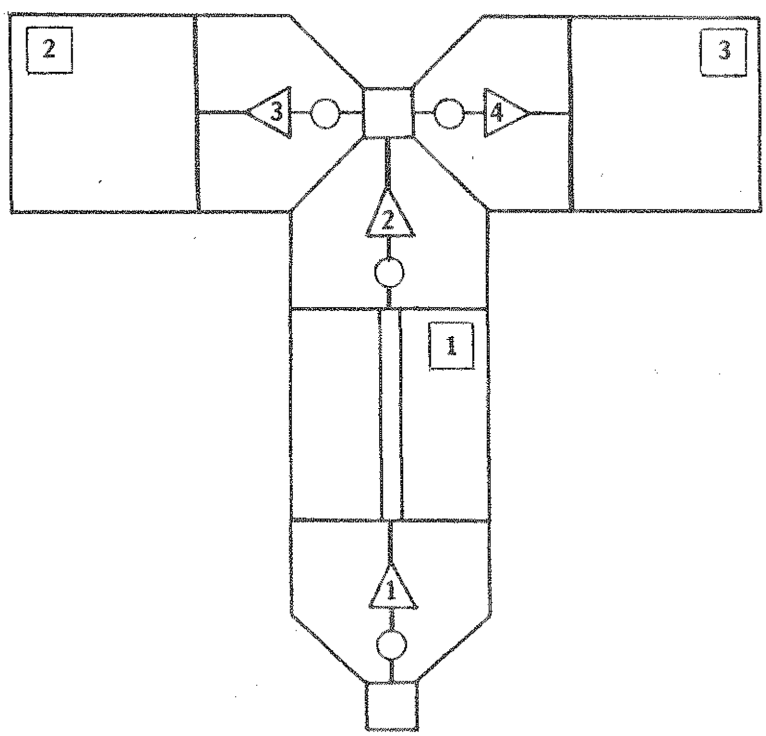


Figure 5.6 Watershed Conceptualization

a single watershed in Iowa, their use in this study is for illustrative purposes only.

General land use conditions were then determined using Table 5.2. For the purpose of this study, average land use values for the entire state were used. The percent of imperviousness associated with each land use was approximated using Table 5.3. Only one pollutant, total suspended solids (TSS), was modeled in this study. Pollutant buildup was assumed to be linear. The initial loadings for TSS were obtained using the average state land use values and Tables 5.4 and 5.5. An exponential washoff equation was used to generate the pollutant loadings during each storm. For this study, an exponential decay coefficient of 1.5 was assumed. For pervious areas, infiltration parameters were selected assuming a hydrologic soil group of C. A summary of the assumed parameter values for the developed watershed are listed in Table 5.7.

5.6 METHODOLOGY APPLICATION

Using the constructed synthetic watershed, the detention basin planning methodology was applied for a design period of 20 years. A 20 year rainfall record for West Lafayette, Indiana was obtained from NOAA for use in the analysis. Design frequencies of 5, 10 and 20 years were selected. For this particular application, the channel network was assumed to be able to carry the 20 year

Table 5.6 Assumed Parameters for Synthetic Undeveloped Watershed

Subshed 1

Subshed Area	95 acres
Eff Imp Area	5 acres
Subshed Slope	97 ft/mi
Subshed Length	2634 feet
Effective Width	9451 feet

Subshed 2

Subshed Area	71 acres
Eff Imp Area	4 acres
Subshed Slope	73 ft/mi
Subshed Length	1985 feet
Effective Width	9400 feet

Subshed 3

Subshed Area	95 acres
Eff Imp Area	5 acres
Subshed Slope	73 ft/mi
Subshed Length	2653 feet
Effective Width	9488 feet

Table 5.7 Assumed Parameters for Synthetic Developed Watershed

Subshed 1

Subshed Area	95 acres
Eff Imp Area	28 acres
Subshed Slope	97 ft/mi
Subshed Length	2634 feet
Effective Width	22724 feet
Solids Loading	366 lb/dy

Subshed 2

Subshed Area	71 acres
Eff Imp Area	21 acres
Subshed Slope	73 ft/mi
Subshed Length	1985 feet
Effective Width	23336 feet
Solids Loading	278 lb/dy

Subshed 3

Subshed Area	95 acres
Eff Imp Area	28 acres
Subshed Slope	73 ft/mi
Subshed Length	2653 feet
Effective Width	23510 feet
Solids Loading	360 lb/dy

predevelopment flow and was not considered in the overall design.

5.6.1 Watershed Simulation

The synthetic watershed was simulated for both developed and undeveloped conditions. In addition to flowrate, suspended solids loading and washoff were also simulated. A 20 year continuous simulation was conducted for both conditions using a 1 hour time step.

5.6.2 Statistical Analysis

After the continuous simulations were completed, a statistical analysis was performed for both simulation runs. A listing of events based on their frequency of occurrence for both the developed and undeveloped conditions is provided in Table 5.8.

5.6.3 Design Event Selection

For the purpose of this study, a composite design event was derived for each selected design frequency. The composite event for a given frequency was constructed using the hydrograph associated with the peak flowrate and the pollutant load associated with the peak pollutant load. The hydrographs of the three composite design events are presented in Figure 5.7.

Table 5.8 Event Statistics for Continuous Simulation of the Synthetic Watershed

RAINFALL DATA

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	12/31/65 (2.80)	12/31/65 (2.80)	5/15/68 (4.00)
10.0	8/01/61 (2.64)	7/14/58 (2.56)	9/14/56 (3.90)
6.7	7/11/58 (2.56)	6/13/58 (2.01)	7/02/62 (3.49)
5.0	8/12/56 (2.04)	1/29/59 (1.70)	8/01/61 (3.14)
4.0	6/10/58 (2.01)	8/02/67 (1.40)	1/26/66 (3.10)

FLOWRATE DATA
(Undeveloped Condition)

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	8/01/61 (1.27)	12/31/65 (.726)	9/14/65 (3.10)
10.0	7/11/58 (1.26)	8/01/61 (.598)	5/15/68 (2.75)
6.7	12/31/65 (1.16)	9/14/56 (.517)	7/11/58 (2.43)
5.0	9/14/65 (1.07)	6/10/58 (.504)	8/01/61 (2.39)
4.0	12/31/65 (.902)	7/08/71 (.443)	12/31/65 (2.18)

FLOWRATE DATA
(Developed Condition)

Return Period	Peak (in/hr)	Avg (in/hr)	Vol (in)
20.0	12/31/65 (1.75)	12/31/65 (1.25)	9/14/56 (3.43)
10.0	7/11/58 (1.65)	6/10/58 (.900)	5/15/68 (3.10)
6.7	8/01/61 (1.62)	7/08/71 (.839)	8/01/68 (2.72)
5.0	8/12/56 (1.27)	1/20/59 (.701)	7/11/58 (2.70)
4.0	6/09/58 (1.20)	9/14/65 (.686)	12/31/65 (2.51)

POLLUTANT DATA

Return Period	Peak (mg/l) (+E03)	Avg (mg/l) (+E03)	Total (lbs) (+E6)
20.0	9/15/60 (11.93)	9/19/60 (10.6)	5/10/57 (.241)
10.0	8/10/61 (11.92)	5/10/67 (10.3)	4/18/70 (.238)
6.7	5/15/58 (11.88)	6/15/57 (10.2)	7/29/70 (.236)
5.0	6/12/73 (11.86)	12/12/65 (10.1)	1/26/54 (.230)
4.0	5/26/65 (11.86)	9/29/70 (10.1)	5/09/55 (.228)

5.6.4 Design Constraint Selection

For this particular application, flowrate constraints and pollutant load constraints were set only at the watershed outlet. Flowrate constraints were based on the undeveloped simulation results for the associated return period. Pollutant load constraints were based on 50 percent of the maximum total load associated with the selected design frequency. A summary of the constraints for each design frequency is provided in Table 5.9.

In addition to the system constraints, different variable constraints were imposed on each detention site for each design frequency. A listing of the variable constraints for each detention site and design frequency is provided in Tables 5.10-5.12.

5.6.5 Application of the Design Heuristic

In applying the design heuristic to the synthetic watershed, an attempt was made to derive a single overall design which would meet the pollutant and flowrate constraints of all three design frequencies. In deriving the final design, two different design strategies were investigated.

The first strategy (Case 1) involves a sequential design process. In this case a design is first obtained for the lowest design frequency (ex. 5 years). Once a design

Table 5.9 Watershed Constraints

Return Period	Flowrate (cfs)	Pollutant (lbs)
20 yrs	335.00	120500
10 yrs	332.00	119000
5 yrs	281.00	115000

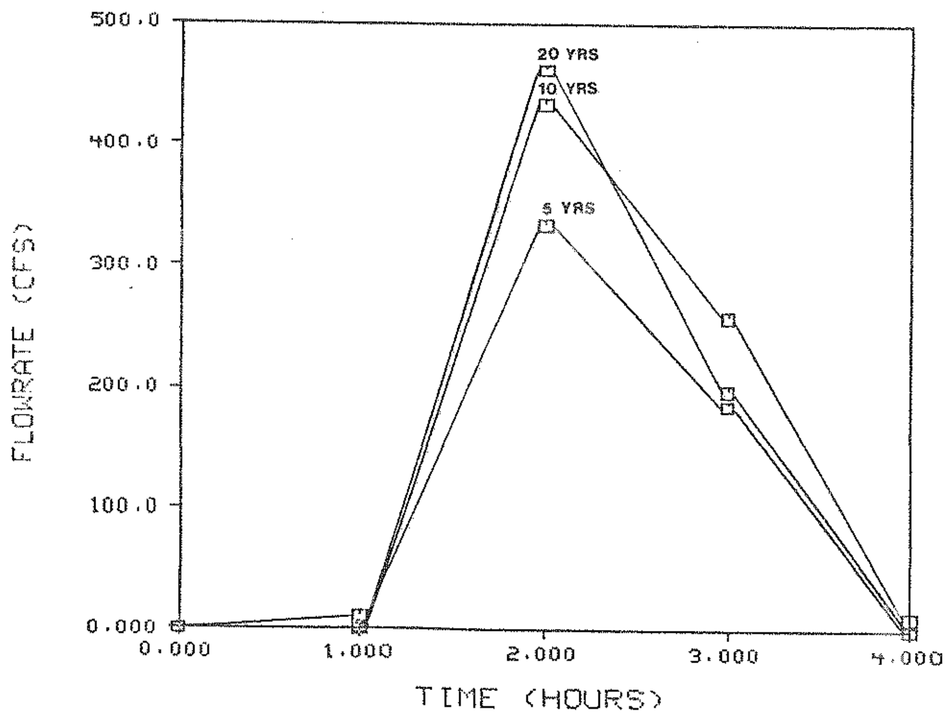


Figure 5.7 Composite Design Hydrographs

Table 5.10 Basin Constraints
(5 year frequency)

Detention Site 1

Maximum Weir Length	5 feet
Maximum Basin Width	600 feet
Maximum Basin Length	700 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	500000 sqft
Maximum Basin Storage	3000000 cuft

Detention Site 2

Maximum Weir Length	5 feet
Maximum Basin Width	500 feet
Maximum Basin Length	600 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	350000 sqft
Maximum Basin Storage	2100000 cuft

Detention Site 3,4

Maximum Weir Length	5 feet
Maximum Basin Width	400 feet
Maximum Basin Length	500 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	4 feet
Maximum Basin Area	250000 sqft
Maximum Basin Storage	1000000 cuft

Table 5.11 Basin Constraints
(10 year frequency)

Detention Site 1

Maximum Weir Length	5 feet
Maximum Basin Width	650 feet
Maximum Basin Length	750 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	550000 sqft
Maximum Basin Storage	3300000 cuft

Detention Site 2

Maximum Weir Length	5 feet
Maximum Basin Width	600 feet
Maximum Basin Length	700 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	500000 sqft
Maximum Basin Storage	3000000 cuft

Detention Site 3,4

Maximum Weir Length	5 feet
Maximum Basin Width	550 feet
Maximum Basin Length	650 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	400000 sqft
Maximum Basin Storage	2400000 cuft

Table 5.12 Basin Constraints
(20 year frequency)

Detention Site 1

Maximum Weir Length	10 feet
Maximum Basin Width	700 feet
Maximum Basin Length	750 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	8 feet
Maximum Basin Area	550000 sqft
Maximum Basin Storage	4500000 cuft

Detention Site 2

Maximum Weir Length.	5 feet
Maximum Basin Width	650 feet
Maximum Basin Length	700 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	550000 sqft
Maximum Basin Storage	3300000 cuft

Detention Site 3,4

Maximum Weir Length.	5 feet
Maximum Basin Width	600 feet
Maximum Basin Length	650 feet
Minimum Basin Side Slope	2.5 ft/ft
Maximum Basin Depth	6 feet
Maximum Basin Area	500000 sqft
Maximum Basin Storage	2500000 cuft

has been derived for the intial frequency, that design is then fixed and the design event corresponding to the next higher frequency is applied (ex. 10 years). This event is then used to obtain a design that satisfies the constraints of that particular frequency, and also the constraints of all the lower frequencies. Once this design has been obtained, the process is then repeated for the next design frequency until a final design is obtained. Such a strategy will thus insure that the final design will satisfy the constraints of all the selected design frequencies.

The second strategy (Case 2) involves a single design approach. In this case the largest design frequency (ex. 20 years) was used to obtain a single design. The performance of this design is then tested via simulation for all of the lower design frequencies (ex. 5, 10 years). If the derived design satisfies all of the lower frequency design constraints, then a final design is obtained. If the derived design violates a lower frequency design constraint, then the design must be modified in some way until an acceptable design is obtained.

In applying the general design heuristic to the synthetic watershed, both design strategies were employed. In addition, two different constraint conditions were examined for each strategy. This resulted in a total of four different case studies. A description of each case study is provided in Table 5.13.

Table 5.13 Description of Case Studies

Case Study	Single Strategy	Sequential Strategy	Flowrate Constraint	Pollutant Constraint
1A		X	X	
1B		X	X	X
2A	X		X	
2B	X		X	X

5.6.6 Discussion of the Results

The results of the application of the design heuristic are presented in Figure 5.8 and Tables 5.14-5.19. Figure 5.8 provides a summary of the cost of each design for each case study. Tables 5.14-5.19 contain the values of the design parameters and the resulting system variables associated with each design. A brief discussion of the results of each case study is presented below.

As can be seen from Figure 5.8, the costs of the designs corresponding to case 2 are less than the costs of the designs associated with case 1. This result is due to the fact that the designs associated with case 1 are more constrained than the designs associated with case 2. In general, the designs associated with case 2 (those derived using the single design strategy) did satisfy both the pollutant and the flowrate constraints of the lower design frequencies. The only exception to this trend was the 20

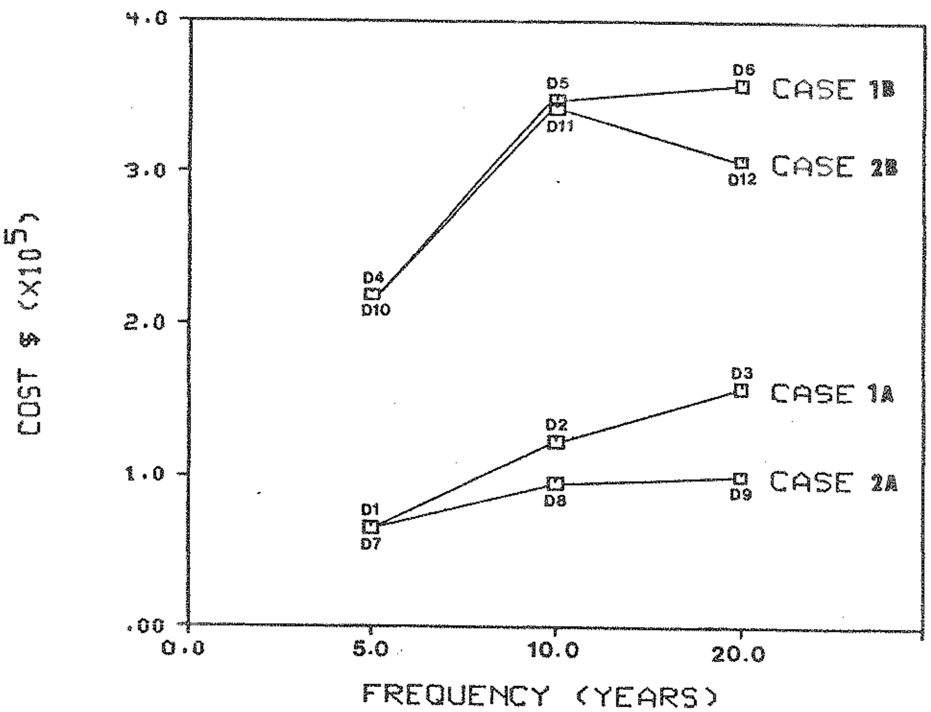


Figure 5.8 Summary of Results

Table 5.14 Design Results for Case 1.A (Designs D1-D3)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D1	DESIGN = 5 yrs				SIMULATION = 5 yrs			0.65
B1		275	281	210	120			
B2	4.2	215	182	140	120	64.	245.	0.65
B3		94	94	50	50			
B4		122	122	90	90			
D2	DESIGN = 5,10 yrs				SIMULATION = 10 yrs			1.23
B1		325	332	208	208			
B2	4.6	229	214	131	115	76.	271.	0.71
B3	3.1	121	93	52	38	63.	180.	0.52
B4		159	159	93	93			
D3	DESIGN = 5,10,20 yrs				SIMULATION = 20 yrs			1.58
B1		335	335	199	199			
B2	4.7	244	224	120	104	80.	280.	0.72
B3	3.1	129	96	53	36	79.	183.	0.53
B4	4.7	169	153	94	83	26.	96.	

QIN = Flow In　　PT = Design Point　　PIN = Pollutant In
QOT = Flow Out　　BN = Basin Number　　POT = Pollutant Out
AREA = Basin Area　　STOR = Basin Storage

Table 5.15 Design Results for Case 1.B (Designs D4-D6)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D4	DESIGN = 5 yrs				SIMULATION = 5 yrs			2.19
B1	3.1	330	117	230	115	423.	1282	2.19
B2		215	215	140	140			
B3		94	94	50	50			
B4		122	122	90	90			
D5	DESIGN = 5,10 yrs				SIMULATION = 10 yrs			3.48
B1	3.6	334	148	202	118	470.	1485	2.44
B2		204	204	109	109			
B3	3.2	121	113	52	46	22.	64.	0.25
B4	3.3	159	108	93	63	102.	320.	0.80
D6	DESIGN = 5,10,20 yrs				SIMULATION = 20 yrs			3.58
B1	3.4	343	138	201	120	468.	1414	2.35
B2	4.9	212	211	110	106	12.	41.	0.18
B3	3.3	129	119	53	47	30.	66.	0.25
B4	3.3	169	109	94	63	121.	323.	0.80

QIN = Flow In PT = Design Point PIN = Pollutant In
QOT = Flow Out BN = Basin Number POT = Pollutant Out
AREA = Basin Area STOR = Basin Storage

Table 5.16 Design Results for Case 2.A (Designs D7-D9)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
D7	DESIGN = 5 yrs			SIMULATION = 5 yrs				0.65
B1		275	281	210	120			
B2	4.2	215	182	140	120	64.	245.	0.65
B3		94	94	50	50			
B4		122	122	90	90			
D8	DESIGN = 10 yrs			SIMULATION = 10 yrs				0.96
B1		332	332	215	215			
B2	5.4	280	209	145	122	84.	413.	0.96
B3		121	121	52	52			
B4		159	159	93	93			
D9	DESIGN = 20 yrs			SIMULATION = 20 yrs				1.00
B1		332	332	215	215			
B2	5.9	298	206	147	120	83.	440.	1.00
B3		129	129	53	53			
B4		169	169	94	94			

QIN = Flow In PT = Design Point PIN = Pollutant In
QOT = Flow Out BN = Basin Number POT = Pollutant Out
AREA = Basin Area STOR = Basin Storage

Table 5.17 Simulation Results for Case 2.A (Designs D7-D9)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
S7	DESIGN = 10 yrs		SIMULATION = 5 yrs					0.81
B1		262	262	204	204			
B2	4.4	215	166	140	114	81.	325.	0.81
B3		94	94	50	50			
B4		122	122	90	90			
S8	DESIGN = 20 yrs		SIMULATION = 5 yrs					0.82
B1		258	258	204	204			
B2	4.5	215	163	140	115	79.	331.	0.82
B3		94	94	50	50			
B4		122	122	90	90			
S9	DESIGN = 20 yrs		SIMULATION = 10 yrs					0.99
B1		324	324	213	213			
B2	5.8	280	203	145	120	83.	430.	0.99
B3		121	121	52	52			
B4		159	159	93	93			

QIN = Flow In PT = Design Point PIN = Pollutant In
QOT = Flow Out BN = Basin Number POT = Pollutant Out
AREA = Basin Area STOR = Basin Storage

Table 5.18 Design Results for Case 2.B (Designs D10-D12)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
10	DESIGN = 5 yrs			SIMULATION = 5 yrs				2.19
B1	3.1	330	117	230	115	423.	1282	2.19
B2		215	215	140	140			
B3		94	94	50	50			
B4		122	122	90	90			
11	DESIGN = 10 yrs			SIMULATION = 10 yrs				3.43
B1	4.1	181	129	156	119	180.	698.	1.41
B2	2.7	274	96	142	63	397.	1054	1.90
B3	3.2	121	119	52	49	9.	25.	0.12
B4		159	159	93	93			
12	DESIGN = 20 yrs			SIMULATION = 20 yrs				3.08
B1	3.5	179	134	156	119	173.	577.	1.22
B2	2.6	298	89	147	62	393.	1017	1.85
B3		129	129	53	53			
B4		169	169	94	94			

QIN = Flow In
QOT = Flow Out
AREA = Basin Area
PT = Design Point
BN = Basin Number
PIN = Pollutant In
POT = Pollutant Out
STOR = Basin Storage

Table 5.19 Simulation Results for Case 2.B (Designs D10-12)

PT / BN	DPTH ft +E0	QIN cfs +E0	QOT cfs +E0	PIN lbs +E3	POT lbs +E3	AREA sqft +E3	STOR cuft +E3	COST $ +E5
10	DESIGN = 10 yrs		SIMULATION = 5 yrs					3.03
B1	3.3	135	90	138	95	186.	591.	1.25
B2	2.2	211	72	140	48	373.	821.	1.58
B3	2.8	94	91	50	50	9.	22.	0.11
B4	3.4	122	120	90	90	6.	15.	0.08
11	DESIGN = 20 yrs		SIMULATION = 5 yrs					2.69
B1	2.9	133	103	143	99	170.	480.	1.07
B2	2.2	215	68	140	53	390.	847.	1.62
B3		94	94	50	50			
B4		122	122	90	90			
12	DESIGN = 20 yrs		SIMULATION = 10 yrs					3.18
B1	3.6	181	143	159	126	174.	604.	1.27
B2	2.8	280	95	145	66	394.	1063	1.91
B3		121	121	52	52			
B4		159	159	93	93			

QIN = Flow In   PT = Design Point   PIN = Pollutant In
QOT = Flow Out   BN = Basin Number   POT = Pollutant Out
AREA = Basin Area   STOR = Basin Storage

year design for case 2B (D12) which failed to satisfy the pollutant constraint for the 10 year design event. Thus for a pollutant removal level of 50 percent, the 20 year design associated with case 1B would be preferred.

A very interesting result occurred in relation to the costs of designs D11 and D12. In this particular case, the 10 year design event yielded a design more costly than the 20 year event. At first glance, such a result would appear to be counter-intuitive. The reason for such a result is revealed through an examination of Table 5.8. For this particular study, the composite design events were selected based on the peak flow statistic. Examination of Table 5.8 reveals that although the selected 20 year design event has a higher peak value than the 10 year event, the 10 year event has a higher total volume than the 20 year event. Thus although the peak statistic was dominate for those designs without any pollutant removal (Cases 1A and 2A), the volume statistic was dominate for those designs involving pollutant removal.

5.7 SUMMARY AND CONCLUSIONS

The general planning methodology was applied to a synthetic watershed constructed from average geomorphic parameters for the state of Indiana. The synthetic watershed was analyzed for both undeveloped and developed conditions using 20 years of hydrologic data. Based on a statistical analysis of the continuous simulation, different design events were constructed for 5, 10 and 20 year design frequencies. The derived design events were then used in examining two different design strategies. The results of this study indicate that neither strategy can be guaranteed to always be better. While the single design strategy should always produce the least cost design, the sequential strategy will generally always produce a feasible design.

Instead of considering the single and sequential design strategies separately, a more appropriate approach would be to combine both strategies into a single design methodology. In using such a methodology, the single design strategy would be employed first. If this strategy yields an acceptable design for all other design frequencies then this design should be selected. If the single design violates constraints of the lower frequency designs then the sequential design strategy should be used.

VI. SUMMARY AND CONCLUSIONS

6.1 SUMMARY OF THE REPORT

A new methodology has been developed for use in the planning and design of dual purpose detention basins in urban watersheds. The methodology employs continuous simulation, statistical analysis, and a general design heuristic to obtain an integrated system of detention basins.

The design of any detention basin system involves several different factors and or design considerations. The new planning methodology addresses many design considerations that are usually neglected in the design of a single basin or even a multiple basin system.

In the past, most detention basins have been designed based on a subshed or piecemeal approach. The need for regional planning of stormwater control systems has been identified by several authors. In response to this need, the new planning methodology uses a general design heuristic to obtain an overall design which considers the interaction of the various basins in a watershed.

While stormwater detention basins have been used for the control of urban runoff for many years, only recently has there been an interest in examining the impact of detention for the control of water quality loading in urban runoff. One constraint to the effective design of dual purpose detention basins has been the lack of a general design methodolgy for use in designing such basins. The new planning methodology has been developed as a first step toward eliminating such a constraint. The new methodology can be used to evaluate the pollutant removal efficiencies of various basin designs. In addition, the general design heuristic can be used to evaluate the interaction of both flowrate and pollutant load constraints in relation to the overall system design.

In the past, most detention basins have usually been designed using a design storm approach. This approach assumes that the resulting runoff event has the same frequency of occurrence as the selected rainfall event. This study has shown that this assumption is not always valid. Indeed, different hydrologic parameters of the same runoff event may have different frequencies of occurrence.

In order to incorporate the effect of the frequency of the design runoff event on the detention system design, the new methodology uses continuous simulation along with a statistical analysis of the results to derive design runoff events. Different sets of design runoff events can then be evaluated using the general design heuristic to obtain an integrated detention basin system.

Another stochastic element that is usually ignored in the design of a detention system is the response of a single design to different frequencies of runoff events. Currently, many detention basins are designed for a single design frequency without regard to the effect of other runoff events. This study has proposed the use of the general design heuristic along with a two step design strategy to obtain an overall design that will meet the design constraints of several different design frequencies.

6.2 GENERAL CONCLUSIONS

The current study has demonstrated the need for a general planning methodology for use in the design of dual purpose detention basins. Such a methodology has been developed and tested. The general interaction of both storage and pipe costs and flowrate and pollutant constraints in relation to the overall system design has been illustrated. The effect of various design frequencies constraints on the overall system design has also been investigated. Although some initial results have been obtained in relation to the above considerations, any general conclusions should be delayed until more case studies have been investigated. It is quite possible, given the complexity of the system, that any general conclusion obtainable might be site specific.

The general planning methodology has been applied using several design constraints to yield a wide range of designs. Although there can be no guarantee that the resultant designs are globally optimal, the designs do tend to follow a consistent pattern. Thus, although no formal proof has been presented to guarantee the optimality of the algorithm (if such a proof is even possible) the heuristic does yield improved designs which do correspond to the expected results for given constraint sets.

The new detention basin planning methodology should prove to be a valuable tool in the analysis and design of dual purpose detention systems. The new methodology can be used to obtain an individual system design or used in a sensitivity analysis of a given system. Such an analysis can be used to construct cost graphs as a function of different flowrate reduction and pollutant removal levels. By deriving such graphs, information can be obtained concerning the region of control of each constraint. This information could then be used in the selection of a design that provides the best trade-off between pollutant and flowrate objectives for a selected level of flowrate reduction or pollutant removal.

6.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The development of a general detention basin planning methodology has provided a new tool for use in the analysis of detention basin systems. Such a tool can be used to analyze existing systems or to examine the sensitivity of various design parameters. The new methodology can be used to investigate the possibility of deriving general planning guidelines as a result of various applications of the general methodology.

Another area of further research concerns the simulation of pollutant removal. The current methodology determines pollutant removal using a total load approach. One possible extension of the current methodology would be to evaluate pollutant removal using a concentration approach. Such an approach would thus involve the determination and transformation of subshed pollutographs.

In addition to the above area, further research is needed in relation to the settling characteristics of various pollutants. Although some work has been done in this area, much more research is needed before detention systems can be designed that are effective for a wide range of pollutants. The critical settling characteristics associated with different pollutants need to be identified so that this information can be used in the development of improved design procedures.

The current research has demonstrated the complexity and importance of the stochastic nature of hydrologic design. Although an attempt has been made to incorporate the stochastic element of the detention basin design problem in a general planning methodology, further research in this area seems warranted.

A final area of further research concerns the general design heuristic. Although the existing heuristic has been found to perform very well, it is quite possible that the algorithm could be improved. One area that might be further investigated is the indirect dynamic programming formulation. Alternatively, by further use of the existing algorithm, some general trends might be observed that could be used to develop a much simpler heuristic.

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