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FLOOD PLAIN MANAGEMENT THROUGH ALLOCATION OF LAND USES -- A DYNAMIC PROGRAMMING MODEL

Lewis D. Hopkins
E. Downey Brill, Jr.
Jon C. Liebman
Harry G. Wenzel, Jr.

Institute for Environmental Studies
and
Departments of Landscape Architecture
and Civil Engineering
University of Illinois at Urbana-Champaign

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2535 Hydrosystems Laboratory
Urbana, Illinois 61801

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FOREWORD

One of the first task forces to be established upon the organization of the Institute for Environmental Studies in 1973 was a group to conduct interdisciplinary research on land and water resources planning. The first three authors listed for this report were members of the original task force. As the floodplain studies were undertaken, a person with hydrological expertise was added to the study team.

A research proposal which was submitted to the Center was favorably evaluated and funds were allocated in 1975 to carry out studies which are reported on herein. Further studies are contemplated when funds become available.

This preliminary report illustrates the potential capability of an academic institution to carry on research directed to a relevant societal problem that involves the knowledge of several disciplines. Institutions and agencies involved with land-use planning with special interests in floodplains have found this report useful in their planning activities.

Glenn E. Stout, Director
Water Resources Center

ACKNOWLEDGEMENTS

The hydrology computations, the runs of HEC-1, and many other computational tasks were carried out by Ian Goulter. He also assisted in writing Appendix A. Jerry Schlesinger wrote the initial computer codes for the triangular routing model and the dynamic programming model. Some of the damage data were collected by Steven Nord.

ABSTRACT

FLOOD PLAIN MANAGEMENT THROUGH ALLOCATION OF LAND USES--A DYNAMIC PROGRAMMING MODEL

Despite heroic structural measures, flood damages continue to rise. This research develops a means for identifying more nearly optimal patterns of land use with particular reference to timing, depth, and duration of flooding. The major premise is that flood plain management is best viewed as a problem of allocating land uses to land parcels. A dynamic programming model is developed to determine what combination of downstream uses, which require flood protection, and upstream uses, which may increase runoff or provide protection through longer water retention, should be encouraged. The dynamic programming model and an associated simplified routing technique are demonstrated on a real watershed. Desirable extensions of the model are identified. One major result of the project is the realization of a need to classify watersheds by the degree of effective interdependence among land use decisions so as to determine the most appropriate types of analytical models and public sector interventions for particular cases. Thinking about flood management as a problem of land use allocation is shown to be a fruitful conceptualization for exploring the issues, for developing models, and for identifying appropriate public sector interventions.

Hopkins, Lewis D.; Brill, E. Downey, Jr.; Liebman, Jon C.; and Wenzel, Harry G., Jr.

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KEYWORDS--*flood protection/*land use/economic efficiency/*welfare (economics)/*optimum development plans/*dynamic programming/flood routing/urbanization/optimization/flood plains/non-structural alternatives/land management/economic rent/flood plain zoning/zoning.

1. INTRODUCTION

The problem of flood plain management has previously been conceptualized either as a problem of routing flood water for a given land use pattern (Hall *et al.*, 1968) or as a problem of allocating land uses for a given pattern of flooding (Weisz and Day, 1974a). There is ample evidence that patterns of land use affect levels of flooding, especially in rapidly urbanizing areas (Hollis, 1975; Doehring *et al.*, 1975). The problem should, therefore, be conceptualized as one of allocating uses to subbasins, where a use includes the "hydrologic use" of the subbasin as well as the land use in the more traditional sense. This permits the simultaneous consideration of the effect of floods on land use patterns and the effect of land use on flood patterns. This is a more accurate representation of the problem for the important case of rapidly developing suburban areas, where the land use change affects and is affected by the flooding pattern.

Water from precipitation cannot be eliminated; it can only be allocated with respect to place, volume, duration, timing, and seasonal pattern. A particular temporal pattern of water retention or drainage, ranging from run-off from an impervious surface to storage in a relatively stable reservoir, can be construed as a "hydrologic use" class for a parcel of land. A hydrologic use class can be combined with more traditional land use classes, such as "one dwelling unit per acre single family residential" or "shopping center," to obtain a set of hydrologic-land use classes. The hydrologic characteristics would include the runoff generated while the more traditional designation would indicate the amount of damage for given flood depths and the value of land for the use exclusive of flood damages. The flood management problem is then to allocate these use classes to subbasins of the watershed so as to maximize the total economic rent to land. The use of maximization of total economic rent paid to parcels as the criterion for optimal allocation of land resources has been extensively discussed elsewhere (Alonso, 1964; Weisz and Day, 1974b; Greenberg *et al.*, 1974). Economic rent can be represented in present worth terms as the "bid-price," or price a particular user would be willing to pay to use the parcel of land.

Although flood damage is only one of a multitude of interactions among land uses, this model treats it largely independently of other interactions, such as transportation. Under certain circumstance it dominates other interactions. More importantly, past research suggests that a sequence of modeling structures, each appropriate to particular types of interaction, may be a useful way of dealing with complex land use allocation problems (Hopkins, 1975).

This research is an attempt to determine whether methods can be developed to assist in identifying optimal target allocations of uses at a cost for information collection, solution, and implementation that is less than the aggregate improvement in land value thereby obtained. The necessity of identifying such targets and questions is considered briefly in a theoretical context in Section 2. Section 3 describes a dynamic programming model developed to identify target patterns of use for a watershed. Some preliminary expectations about the cost of data collection and solution are reported for a test problem in Sections 5 and 6.

2. FLOOD DAMAGE AND LAND USE DECISIONS

Decentralized control systems, such as markets, even markets with externality taxes, cannot achieve appropriate use patterns when the effects of upstream land uses on downstream land uses are included. The flooding case requires an extension of a previous argument by Davis and Whinston (1962) to cover the "non-reciprocal" case where one land user imposes an external effect on the other, but not vice versa.

2.1 Interdependence of Land Use Decisions

A simple two parcel example, using Davis and Whinston's cost functions altered so as to create a non-reciprocal case, demonstrates this conclusion.* Each of two owners must decide what density of development, q_i, to construct on his site.

*Davis and Whinston's and our conclusions apply only to non-separable interactions; those in which the output level of the imposer affects the marginal cost for the receiver. Mathematically, non-separability means that $\partial^2 C_1/\partial q_1 q_2 \neq 0$. Assuming the imposer could cease production there is always at least the difference in the external effect from production versus nonproduction by the imposer so that all externalities of interest in a land use location context should be considered non-separable.

Both runoff and value are assumed to increase with density. The cost functions for Owners 1 and 2 are

$$C_1(q_1,q_2) = A_1q_1^{\,r} \tag{1}$$

$$C_2(q_1,q_2) = A_2q_2^{\,n} + B_2q_1^{\,m}q_2. \tag{2}$$

The second term of Owner 2's cost function, $B_2q_1^{\,m}q_2$, is the interaction term. The decision of Owner 1, q_1, changes the runoff and therefore changes the amount of flood damage per unit of density for Owner 2.

The price equals marginal cost criterion for individual profit maximization is then found by taking the partial derivatives of the individual costs with respect to the decisions under the control of the respective owners.

$$P_1 = \frac{\partial C_1}{\partial q_1} = rA_1q_1^{r-1} \tag{3}$$

$$P_2 = \frac{\partial C_2}{\partial q_2} = nA_2q_2^{n-1} + B_2q_1^{m} \tag{4}$$

Owner 2 must assume something about the decision of the upstream owner, because the upstream owner's decision enters the downstream owner's decision criterion. In this non-reciprocal case, however, the upstream owner can make his individual profit maximization decision without knowing the downstream owner's decision. Therefore, a result will be achieved whereby the upstream owner decides; then the downstream owner decides given the upstream owner's decision.

However, if we consider social profit maximization, the costs imposed on the downstream owner are charged to the upstream owner. Taking the partial derivative for the sum of the costs with respect to each decision variable, the price equals marginal cost criteria become

$$P_1 = \frac{\partial C_s}{\partial q_1} = rA_1q_1^{r-1} + mB_2q_1^{m-1}q_2 \tag{5}$$

$$P_2 = \frac{\partial C_s}{\partial q_2} = nA_2q_2^{n-1} + B_2q_1^{m} \tag{6}$$

where $C_s = C_1 + C_2$. Under social profit maximization neither of the owners can make a valid decision about density of development without knowing the decision of the other. If the external cost component of the upstream user's criterion is thought of as an externality tax, he does not know what amount of tax to pay. Nor can a controlling agency know what amount to charge. Therefore, even in the simplified flood damage case, where only one owner imposes a nonseparable externality on the other, the attainment of the social optimum cannot be achieved through decentralized pricing. It must depend on some simultaneous decisions---predictions or target plans.

2.2 Need for Target Plans

Davis and Whinston (1962) have shown that, for cases where each of two parties imposes an externality on the other (the "reciprocal case"), the standard externality tax approach breaks down. Neither party can determine its own output decision without knowing the decision of the other. In this case, the decision process actually breaks down even for individual profit maximization before any tax is imposed. Although Davis and Whinston explicitly limit their conclusion to the reciprocal case, an externality tax is equivalent to an externality imposed by the receiver of the physical externality on the imposer of the physical externality. Thus, for the non-reciprocal case, the usual externality tax scheme does not lead to social profit maximization, even though the decision makers can make individual profit maximizing decisions. Although Baumol (1972) accepts the tax on the external effect as theoretically correct for the non-reciprocal case, he appears to assume that either the output of one party is taken as a parameter, or that the optimum densities of development (levels of output) are known in order to determine the amount of the tax. In any case, he concludes that targets must be chosen for any operational application because of the likelihood of multiple optima. Johnson (1967) and Kneese and Bower (1968) argue for a target or standards approach because of institutional and informational requirements.

Given the failure of the externality tax approach, it is necessary to identify target patterns of uses, including both hydrologic and land use, throughout the watershed in order to achieve allocations that maximize total economic rent to land. If the watershed is currently being developed, or undergoing conversion from rural to urban, one cannot take all other uses as fixed and determine the appropriate use of a particular parcel. The changes of all parcels should be considered simultaneously. This could be done by predicting the uses to be located on other parcels in order to make a decision for a particular parcel. However, if decisions are being made for many parcels simultaneously, and therefore many owners are predicting the actions of other owners, each decision maker's tentative decision affects the set of predicted uses for each other owner. The decisions will not be stable. It is therefore useful to solve the decision problem for all owners simultaneously.

Even given the above limitations on decentralized pricing, it is possible, at least conceptually, to use economic incentives to achieve target plans that are known. Heffley (1972) has shown that if the difference in incomes exclusive of interaction costs for different parcels of land is sufficiently large, it will dominate the interaction costs and an optimal arrangement will be sustainable by decentralized prices. Thus, if a target plan is identified, it can be implemented by altering the values of particular parcels for particular uses so that the dominance conditions are met. Rather than altering the land rents, which must be the same for any activity on a given parcel, alter the incomes and therefore the "bid-prices", which can differ among activities even for the same location. This is sensible in that it is the use at the parcel, not the parcel itself that causes the externality. Many possible indirect means for causing such changes in effective incomes from using a particular parcel for a particular use are available (Reuter and Kushner, forthcoming). The problem of choosing the target should really include the costs of achieving it because these costs may vary for public interventions depending on the degree and type of deviations of the target from the results that would occur without intervention (Hopkins, 1974).

3. LAND USE ALLOCATION: DYNAMIC PROGRAMMING FORMULATION

It has been argued above that identification of target patterns of land use, at least implicitly, is essential for cases in which the upstream land use decisions affect downstream flood damages. This is most likely to be the case in areas of rapid conversion from rural to urban development on watersheds of a size that will be almost entirely converted. A dynamic programming model to allocate hydrologic-land use classes was therefore developed with particular concern for this type of problem. The formulation, although crude, permits some tentative exploration of the land use-flood damage tradeoff.

3.1 Basic Formulation

The stream is divided into subbasins. Each subbasin is associated with a reach of the stream. These reaches are the stages of the dynamic program, which is essentially a recursive decomposition of the problem into discrete stages so that each stage can be solved in sequence. The decision variable at each stage is the use class to be located there. For any inflow hydrograph and any use, it is then theoretically possible to determine 1) the outflow hydrograph at the base of the reach by using runoff and routing models, 2) the area flooded and average depth by using "stage-flow" relationships and valley cross sections, and 3) the "bid-price" for the subbasin for the use after subtracting flood damages for the depth and area flooded. This process is diagrammed for a single dynamic programming stage in Figure 1.

Each stage of the dynamic program consists of repeating this set of computations for each combination of inflow peak and land use. For each outflow peak, the combination of inflow peak and land use that yields the highest aggregate bid-price (or economic rent) for all stages up to the present one is saved as the best means of reaching that output level at that reach. When all stages (reaches of the stream) have been processed in this manner, the final output flow

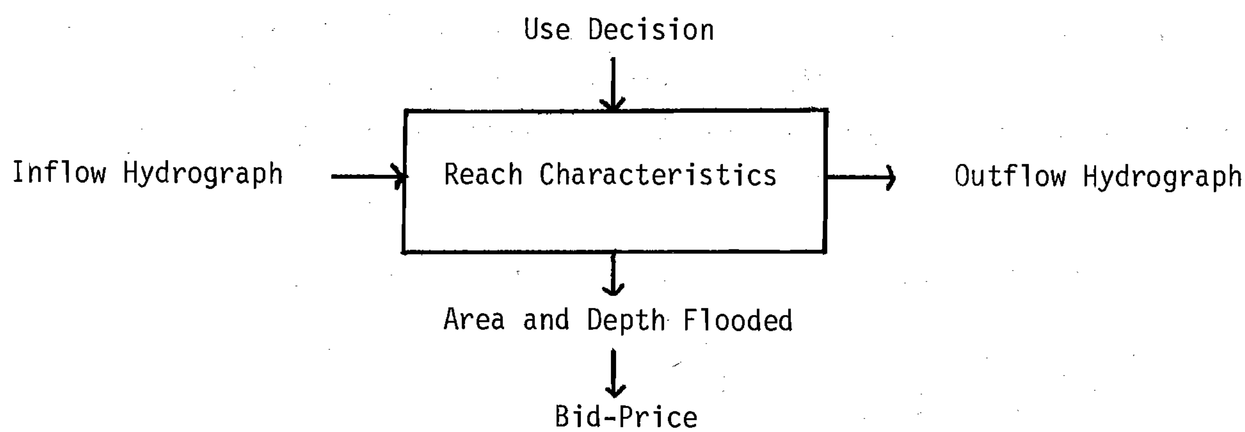


Figure 1 -- Stage in Dynamic Program

with the highest aggregate bid-price is then the starting point to trace back through the set of decisions and associated input flow levels that led to that output flow. In practice, one may wish to constrain the final output below some level. In this case, one would choose the output that has the highest bid-price and meets the constraint. This search procedure yields the optimal allocation of uses to maximize economic rent to land net of flood damage, while specifically considering the impact of upstream development on downstream flood levels and the impact of downstream development on the amount of damage for given flood levels.

The dynamic program can be described mathematically as

$$f_n(X_n) = \max_{D_1,\ldots,D_n} \sum_{n=1}^{N} r_n(X_n,D_n) \tag{7}$$

subject to:

$$X_{n+1} = t_n(X_n,D_n) \tag{8}$$

where $f_N(X_N)$ is the function yielding the highest aggregate bid-price for each final outflow level or stage. D_n is the set of possible uses for stage n; X_n is the set of possible input water flows for stage n. r_n is the return function for each stage, which in this case can be expressed as

$$v_{in}a_n \; - c_{in}^{k} \; f_n \tag{9}$$

where

V_{in} = bid-price per acre of use i in reach n

a_n = number of acres in reach n

C_{in}^{k} = present worth of flood damage per acre for use i in reach n at depth k

f_n = acres flooded to average depth k in reach n

t_n is the transformation function that takes the input flow and decision for stage n and generates the output flow from stage n, which is also the input flow to stage n+1. In this case, the transformation function is a very simple routing model, which is described in Section 4.

This problem can be decomposed and solved using the usual recursion equations of dynamic programming (Nemhauser, 1966).

$$f_n(X_n) = \max_{D_n} r_n(X_n, D_n) + f_{n-1}(X_{n-1}, D_{n-1}) \tag{10}$$

$$= r_n(X_n, D_n) \quad \text{for } n=1 \tag{11}$$

$$= r_n(X_n, D_n) + f_{n-1}(X_{n-1}, D_{n-1}) \quad \text{for } n = 2,N \tag{12}$$

where

$$X_{n+1} = t_n(X_n, D_n) \tag{13}$$

3.2 Elaborations of the Basic Formulation

Beyond this basic formulation there are three elaborations that deserve discussion: 1) a branching stream, 2) a different use in the flood plain from the use in the remainder of the subbasin, and 3) an efficient routing procedure to carry out the transformation between stages. The first two are discussed here; the latter is more involved and is deferred to Section 4. Other elaborations are desirable but have not yet been developed. Most important is the need to handle the probabilistic nature of the flooding events. This can be handled, at least theoretically, within the dynamic programming approach (Nemhauser, 1966). However, the conceptual, computational, and data problems of carrying this out were beyond the scope of this initial study. For the present, the hydrologic data are based on a design storm of given expected frequency; all other flooding is ignored.

Meier and Beightler (1967) have shown that dynamic programming can be applied to the case of branching streams for optimizing reservoir operation. Given the direction of the stage-to-stage transformations in the present case, from the tips of the branches to the trunk, the required modifications of the solution process are straightforward (Nemhauser, 1966). Each branching node is treated as a "decisionless stage" in the dynamic program. Each input flow from the first of the two branches is combined by the routing transformation with each of the input flows from the second branch, yielding a set of output flows. The highest aggregate bid-price, the sum of the bid-prices associated with the respective input flows, is determined for each output. The two input flow states, one from each branch, that yielded the highest summed output for each output state are saved. The only difference between this and other stages is that no decision variable and, therefore, no flooded area, damage, or bid-price is determined. The decisionless branch stages must be differentiated from decision stages; the index of the stages from which the flows are to be combined at each branch must be provided. These operations present little difficulty in carrying out the solution procedure.

The second elaboration attempts to deal with the obvious challenge that the use within the flooded portion of a subbasin should be different from the use in the remainder of the subbasin. This would reduce flood damage without precluding intensive development of non-flooded areas. It could be handled by turning each reach of the stream into two stages of the dynamic program. The first "substage" would determine the non-flood plain use; the second would determine the flood plain use. In such a formulation, the outflow and, therefore, the area flooded in each reach, would be determined by the first substage based on the non-flood plain land use. The second "substage" would then determine the flood plain land use and alter the output flow from the reach accordingly, perhaps even taking into account backwater effects within the reach. Note that some area net of flooded area would have to be assumed for the non-flood plain use in order to determine the flood flow and the flooded area.

Although the two-stage approach has not been rejected for future development, a simpler approach has been implemented at this time. If, in the two-stage approach, the non-flood plain use were initially assumed to cover all of the subbasin, then the difference between the two methods would be that the simpler approach ignores the effect of the flood plain use on the flood flow to the next reach. It assumes that only the non-flood plain use affects the outflow to the next reach.

Given the assumption that output flow from the reach is not affected, the choice of the use for the flooded area depends only on the output flow of the reach, which determines flood damage by determining flooded area and depth. It is, therefore, possible to associate with each of the possible output states for a particular stage of the dynamic programming problem the use that has the highest bid-price net of flood damages. This avoids a flood routing procedure to combine the flows from the flood plain and non-flood plain uses. It adds little to the computational effort or conceptual complexity of the solution procedure.

4. A SIMPLE, EFFICIENT ROUTING PROCEDURE

Up to this point only levels of flow have been distinguished at each stage; it is the peak flow that determines the area flooded, the depth, and therefore, the amount of damage. However, to combine the peak flows from reach to reach, it is necessary to deal with hydrographs that take into account the relative times at which the peaks occur. The rate of change of the flow with time must also be included. In most cases, the peak for the succeeding reach will be the sum of the peak of the channel hydrograph with some other point on the local inflow hydrograph because the peaks of the two hydrographs are not likely to reach the same point in the stream at the same time.

The dynamic programming procedure requires not only the computation of a large number of routings to combine hydrographs, but also the storing of a large number of hydrographs during the computation. The initial input data require a hydrograph for each use in each subbasin. The procedure must calculate between d and dm hydrographs, where d is the number of uses and m is the number of levels of flow or states at each stage of the solution process. Up to m additional hydrographs must be stored and referenced at each stage. A routing procedure must, therefore, be found in which hydrographs can be described with a very small number of values and routed with a small number of computational operations.

4.1 Triangular Hydrographs

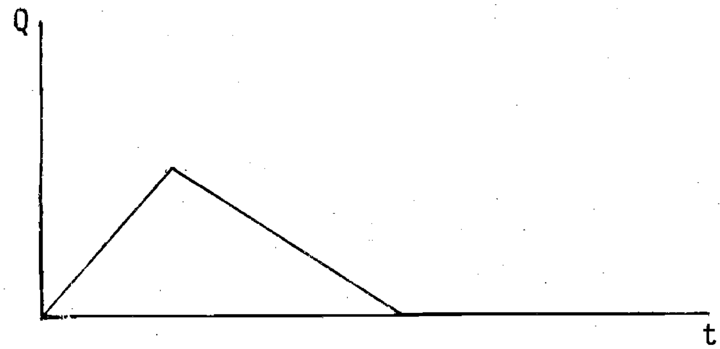
A triangular hydrograph configuration was chosen for the routing procedure because it requires the specification of a minimum number of data elements while at the same time retaining the essential characteristics of the curvilinear hydrographs: the peak and its associated time and the total volume of flow before and after the peak time.

The first step is the generation of local inflow hydrographs for each of the subbasins using HEC-1. Each of these curvilinear hydrographs is then approximated by an equivalent triangular hydrograph as decribed in Appendix A.

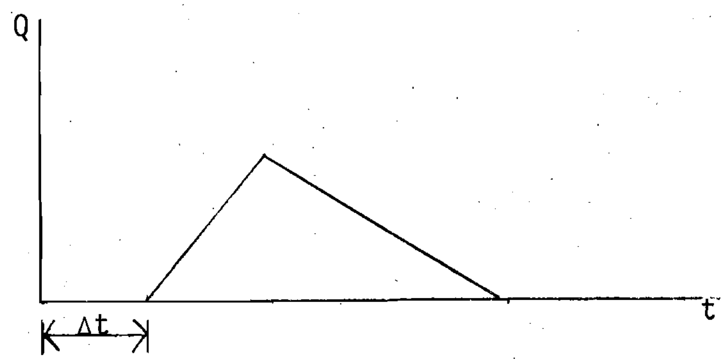
The routing procedure begins at an extreme upstream subbasin where the local inflow hydrograph is taken as the channel hydrograph at the subbasin outlet. This channel hydrograph is then routed using a simple lagging procedure through the next downstream subbasin reach (reach 2 in Fig. 2). The lag time Δt as shown in Fig. 2-b is equal to the time of travel AMSKK described in Appendix A for use in the Muskingum Routing scheme employed in HEC-1.

The routed channel hydrograph is then added to the local triangular inflow hydrograph for this subbasin at the downstream end of reach 2, maintaining a common time scale. The sum of these two triangular hydrographs is a hydrograph consisting of at most five linear segments as shown in Fig. 2-c. It can be shown that the peak of this segmented hydrograph must occur at one of the two peak times, t_{p1} and t_{p2}, of the triangular hydrographs.

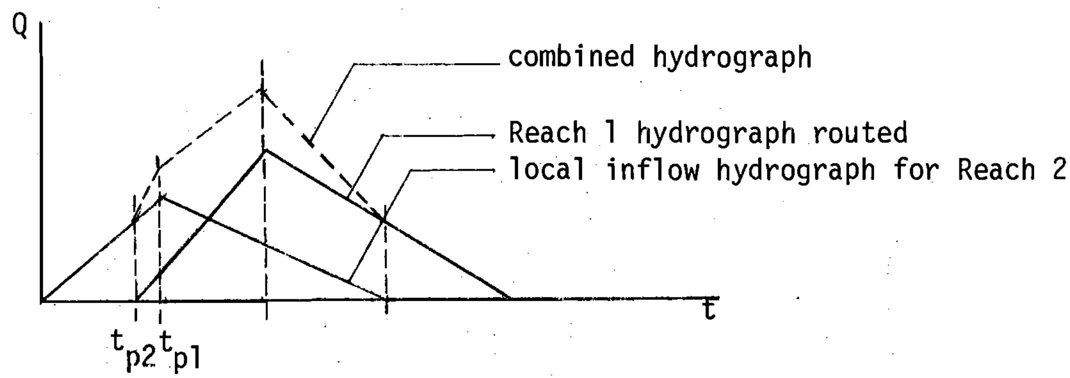
The segmented hydrograph is then in turn approximated by a triangular hydrograph whose peak and peak time are equal to the peak and peak time respectively of the segmented hydrograph as shown in Fig. 2-c. The slope of the rising limb of the new combined triangular hydrograph is determined so that the volume of flow before the new peak time maintains the sum of the volumes of flow in the two triangular inflow hydrographs before the new peak time. The falling limb is determined similarly so as to maintain the sum of inflow volumes after the new peak time. The new combined triangular hydrograph is then taken as the channel hydrograph at the upstream end of the next channel reach and the process is repeated.



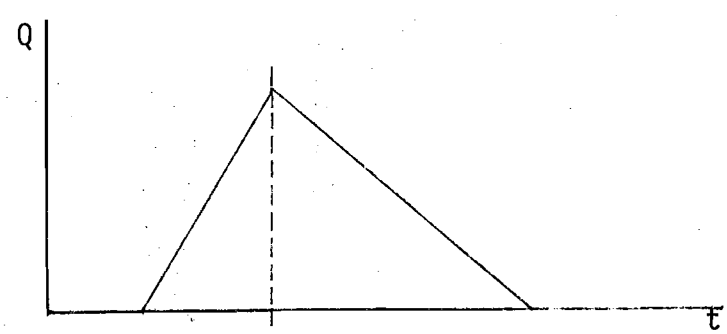
(a) Local inflow hydrograph for Reach 1 (channel hydrograph for Reach 2)



(b) Reach 1 hydrograph routed to downstream end of Reach 2



(c) Formation of combined hydrograph for Reach 2



(d) Combined triangular outflow hydrograph from Reach 2

Figure 2 -- Routing Procedure

All of the subbasins are approximately equal in size. It is, therefore, evident that as the routing proceeds downstream the channel hydrograph will become progressively larger with respect to the local inflow hydrograph and that the peak time of the combined hydrograph will thus be that of the lagged channel hydrograph. The peak of the channel hydrograph will, therefore, occur progressively later in time as the routing proceeds downstream although this trend may be offset somewhat at a junction with a large tributary.

4.2 Validation of the Triangular Routing Procedure

Although this approximation of a hydrograph and the associated routing procedure meet the needs of the dynamic programming procedure, it remains to demonstrate that they provide a valid representation of the hydrologic phenomena being modeled. It was beyond the scope of the present project to provide a thorough validation, but a comparison was made with more sophisticated procedures for the study area used in the demonstration problem described in Section 5. The 'standard of reference' routing procedure was the Muskingum method (U.S. Army Corps of Engineers, 1960; Chow, 1964). The determination of parameters and of the hydrographs for the subbasins is described in Appendix A. Table 1 gives a comparison of the peak flows generated by the triangular routing procedure and by the Muskingum method for the pattern of existing land use. The mean percentage deviation was 23 and the mean absolute deviation was 1,524 cfs. These were computed exclusive of the topmost subbasins, for which no routing was required and which would therefore always be identical. These deviations are quite large relative to the 1,000 or 2,000 cfs intervals used in the dynamic program (see Section 5). For the design storm, the only field observed flow is the discharge near the base of the watershed at the guaging station marked in Fig. 3. This flow of 15,200 cfs is more closely approximated by the triangular routing method than by the Muskingum method. A further comparison of the triangular and Muskingum methods for the results of the first run of the dynamic programming model yielded a mean percentage deviation of 32 and a mean absolute deviation of 2419 cfs. Clearly, this is not encouraging toward a validation of the triangular routing method. However, the match of the triangular method with the observed flow does suggest that a validation study using observed flows for several storms, and preferably several points on the stream, might

TABLE 1

Comparison of Hydrographs Produced by HEC-1
and Triangular Routing Scheme
for Existing Land Uses

Area	HEC-1	Triangular	Error in cfs HEC1-Triangular	Percentage Error Error / HEC-1
1	1562	1562	0	0
2	3069	8484	+415	+14
3	2001	2001	0	0
4	4730	8074	+3346	+71
5	4928	8074	+3146	+64
6	4873	8074	+3201	+66
7	2003	2003	0	0
8	2563	3041	+ 478	+19
9	1495	1495	0	0
10	5340	5998	+ 658	+12
11	2057	2057	0	0
12	6895	8039	+1144	+17
13	2097	2097	0	0
14	7258	8074	+ 816	+11
15	10840	13994	+3154	+32
16	3293	3293	0	0
17	10743	13994	+3251	+30
18	10700	13994	+3294	+31
19	10753	13994	+3241	+30
20	2961	2961	0	0
21	3734	4120	+ 386	+10
22	4010	5022	+1012	+25
23	4161	5022	+ 861	+21
24	4303	5022	+ 719	+17
25	1573	1573	0	0
26	2485	2485	0	0
27	4638	4896	+ 258	+ 6
28	3688	5022	+1334	+36
29	7740	9391	+1651	+21
30	14180	15714	+1534	+11

TABLE 1
(continued)

Area	HEC-1	Triangular	Error in cfs HEC1-Triangular	Percentage Error Error / HEC-1
31	14269	15714	+1445	+10
32	14458	15714	+1256	+ 9
33	14617	15714	+1097	+ 8
34	1292	1292	0	0
34	3368	3636	+ 288	+ 9
36	4206	4480	+ 274	+ 7
37	4289	4480	+ 191	+ 4
38	5735	6338	+ 603	+11
39	6310	7794	+1484	+24
40	6611	8421	+1810	+27
41	6941	9542	+2601	+37
42	18962	16656	+2306	+12
			1524	23

be worthwhile. The allocation model was tested using the triangular method pending validation or development of an alternative method.

5. TEST PROBLEM: HICKORY CREEK WATERSHED

The conceptual structure of the model has been described in the previous two sections. The problem described in this section is intended to test this structure in the face of an attempt at application. The problem is based on realistic data and is of a size that would be of interest in practice. The use of realistic data serves to spotlight the likely difficulties in generating and using real data without the tremendous expense and time required to obtain such data. This makes it possible to make a preliminary evaluation of the model without great research investment. It means, however, that the spatial pattern that results should not be construed as having any operational significance for the real problems of the area used as a study site.

It is important to solve a test problem of realistic size to confirm that the model's computational, storage, and data requirements do not become prohibitive. Although it is possible to extrapolate these characteristics deductively from small problems, this is not reliable when the computational effort involved is affected by the pattern of data values, as is the case in this dynamic programming problem. It is also valuable to fully experience the data manipulation and collection implications so as to have greater respect for the difficulties.

5.1 Land Use and Hydrologic Data

The Hickory Creek watershed in Will County, Illinois was chosen for the test problem because of the size of the basin, the expectation that it would undergo substantial development in its upper reaches in the near future, the existing and increasing flood damage problems already occuring in its lower reaches, and the availability of land use and hydrologic data. The watershed encompasses an area of 109.8 square miles. It was divided into 42 subbasins and associated reaches, which are the stages for the dynamic programming model. This number of subbasins is small enough for efficient solution and yields subbasin areas of appropriate size for allocation of general categories of land use. The subbasins ranged in size from 1.08 to 6.44 square miles. This variation is due to the difficulty of delineating equal size subbasins given the topographic conditions that must be followed and to changes required to fit the model structure after the initial data collection. It is likely that a more uniform size could be achieved.

Eight general land use categories, which are listed in Table 2, were chosen for the allocation. These were intended to be appropriate to the suburban nature of the area and to the large size of the subbasins to which uses were allocated. Hindsight would suggest the inclusion of a lower density estate or rural-nonfarm category. It would also be more appropriate to describe the uses as general community types, for example including commercial, industrial, and residential, than as individual types. The runoff and bid-prices used are in some cases realistic for mixes, so that residential, for example, includes neighborhood shopping. Commercial should have included higher density residential. The land use types and their characteristics were derived in part from "The Costs of Sprawl" (Real Estate Research Corporation, 1974). By extending and adapting some of their "community level" analyses it would be possible to develop very good land use classifications for the scale and detail required for a real application of the allocation model.

Hydrographs for each land use type on each subbasin were generated using the Clark method (Clark, 1945) as programmed in the HEC-1 Flood Hydrograph Package (U.S. Army Corps of Engineers, 1973). The data, calculations of parameters, and transformation into triangular hydrographs are described in Appendix A.

The computation of depth ("stage") for given flows was based on the assumptions of uniform flow within the channel. For each reach a number of depths were chosen at each cross section and the corresponding flows determined. A graph of height vs. flow was then plotted for each of the subbasins. The height of flow at points between the cross sections for which the depth-discharge relationship had been determined, was found by linear interpolation between the depths at the immediate upstream and downstream cross sections. The discharge

TABLE 2

Land Use Categories

1. Single family residential: conventional (3 dwelling units/acre)
2. Single family residential: clustered (5 dwelling units/acre)
3. Townhouse clustered (10 dwelling units/acre)
4. Walkup apartments (15 dwelling units/acre)
5. Commercial/industrial
6. Rowcrop agriculture
7. Pasture
8. Recreation

from the entire watershed was available as observed data. The flow at successive upstream points was estimated by reducing the flow at the immediate downstream point by the ratio of the watershed area above the upstream point to the area above the downstream point. The area flooded for a given flow was then determined from the cross sections and topographic maps. A graph of area flooded versus flow was plotted for each subbasin. Both the area and depth curves were extrapolated linearly, in some cases from less than one thousand to several thousand cfs, to cover flows above those for which data were available.

5.2 Flood Damage and Land Value Data

The level of damage per acre for each land use was determined from damage curves for types of structures as a function of depth. These curves were obtained from State and Federal agencies. Grigg and Helweg (1975) have reviewed various potential sources. The data gave percentage damage for value of structures and contents. The values of structures were obtained from the Costs of Sprawl (Real Estate Research Corporation, 1974). A mix of housing types including with and without basements and one and two stories was assumed for each residential type. The value of contents was computed as a percentage of value of structures. An additional percentage was added for indirect costs (Grigg & Helweg, 1975). For the test problem, agricultural and natural lands for recreation were assumed to have zero damage costs. This method was used to generate a table of dollars of damage per acre for depths in one foot increments for each use in each subbasin.

The present worth of expected damages is

$$\sum_{k=1}^{n} \frac{P_k d}{(1 + i)^k} \tag{14}$$

where p_k is the probability of occurrence in time period k, n is the number of time periods, d is the damage if it occurs, and $(1+i)^k$ is the discount factor. Assuming that the design storm used for this study (June 13, 1957) has a recurrence interval of 100 years, p_k=1/100, the formula reduces to a uniform annual series present worth factor as defined in James and Lee (1971).

The objective in this formulation of the flood management problem is to maximize the bid-price for land after subtracting flooding damage. The bid-price data for the model should include all components, other than flooding, that can be determined without knowing the land uses allocated to other sub-basins by the model. For example, the effect on bid-price of location relative to Joliet, Chicago, and interstate highways would be incorporated in bid-price data, but the advantage of being adjacent to a shopping center located by the model could not be included because its location cannot be known. The modeling structure assumes that the costs of these interactions among newly locating uses are dominated by the costs included in the model: costs of interaction with existing activities, costs due to characteristics of the subbasins, and costs from flooding. If this assumption is not valid then an iterative modeling strategy, in which the other important interactions were incorporated in another step, is called for (Hopkins, 1975).

Greenberg *et al.* (1974) have discussed the problems of obtaining bid-price data by alternative approaches, such as used by Weisz and Day (1974b) and Arvanitidis *et al.* (1972). (See also Wheaton, 1972.) The accurate determination of a set of bid-prices is beyond the resources available to this project. For the test problem, bid-price data were constructed from simple assumptions about trip behavior to Joliet and Chicago and adjusted to fit site characteristics and estimates from conversations with realtors. The data used are in no sense real data but do make it possible to interpret and confirm the results of the dynamic programming allocation.

For the test problem run, peak flows were classed into intervals of 2000 cfs. Each stage had eight possible states, that is, eight possible 2000 cfs intervals into which the output flow could be placed. The set was adjusted so as to cover the range of cfs expected at each stage. Flows ranging from 500 to 30,000 cfs were accomodated. The times at which peaks occurred were classed into intervals of one and a half hours and offset in a manner analogous to that used for the classes of peaks. Ten classes of time were used for each stage, making 80 state levels for each stage of the dynamic program. A second test was run using 15 classes of flow of 1000 cfs each and 15 classes of time of one hour each, making 225 state levels.

6. RESULTS OF TEST PROBLEM

The solutions from the two tests are described in Table 3. Land use 1 is the non-flood plain land use; land use 2 is the flood plain land use; greatest value use is the land use with the highest value on that parcel; same indicates whether the two tests yielded the same land use. The solution from the first test is also mapped in Fig. 3. The run of the first test required 27 seconds of computer time and 250k bytes of storage on an IBM 360/75. The run of the second test required 41 seconds and 474k bytes. Several trial runs were required for each test to establish the ranges of times and peaks for each stage of the dynamic program. This could be avoided with a more complex coding of the computer program. These times and storage requirements are well within the range for reasonable implementation of the model, taking into account the need to make many runs for sensitivity studies in any real application.

The solutions from the two runs for the test problem were identical except for parcels 3, 21, and 29, as indicated in Table 3. The relative stability of solutions suggests that the somewhat artificial discretization of the flood levels and times is acceptable. The solutions were not very sensitive to the size of the interval for the discrete classes.

The approach for handling the flood plain and non-flood plain land uses was not satisfactory for this case because parcels 1, 2, 4, and 41 were 100 percent flooded according to the model. This is counter to the assumption that the flooded area would constitute only a small portion of the total parcel. This problem could be overcome by using the two stage approach suggested in Section 3.2.

The most important result is the lack of interdependence between upstream and downstream land use decisions in the test problems. If the interdependence existed, then the non-flood plain part of a subbasin would not necessarily be

Table 3

RESULTS OF TEST PROBLEM RUNS

	Run #1: NQMAX = 8		Run #2: NQMAX = 12			
	Land Use #1	Land Use #2	Land Use #1	Land Use #2	Greatest Value Use	Same
1	None	6	None	6	1,6	√
2	None	6	None	6	1,6	√
3	6 *	1	1	1	1	
4	None	1	None	1	1	√
5	6	6	6	6	1,6	√
6	6	6	6	6	1,6	√
7	6	6	6	6	6	√
8	6	6	6	6	6	√
9	6	6	6	6	6	√
10	6	6	6	6	6	√
11	1	6	1	6	1	√
12	1	6	1	6	1	√
13	6	6	6	6	6	√
14	1	8	1	8	1	√
15	1	8	1	8	1	√
16	1	6	1	6	1	√
17	8	8	8	8	1,8	√
18	1	8	1	8	1	√
19	1	8	1	8	1	√
20	1	6	1	6	1	√
21	1	6	6 *	6	1	
22	8	8	8	8	8	√
23	8	8	8	8	8	√
24	8	8	8	8	1,8	√
25	1	6	1	6	1	√
26	1	6	1	6	1	√
27	1	8	1	8	1	√
28	1	8	1	8	1	√
29	8	8	1	8	1,8	
30	1	8	1	8	1	√

RESULTS OF TEST PROBLEM RUNS
(Continued)

	Run #1: NQMAX = 8		Run #2: NQMAX = 12			
	Land Use #1	Land Use #2	Land Use #1	Land Use #2	Greatest Value Use	Same
31	8	8	8	8	1,8	√
32	1	1	1	1	1	√
33	5	4	5	4	4,5	√
34	1	6	1	6	1	√
35	1	6	1	6	1	√
36	8	8	8	8	8	√
37	8	8	8	8	8	√
38	1	6	1	6	1	√
39	1	6	1	6	1	√
40	1	6	1	6	1	√
41	None	3	None	3	4,5	√
42	5	4	5	4	4,5	√

Land Use #1 = Non-flood plain land use

Land Use #2 = Flood plain land use

* = Land uses different from greatest value use

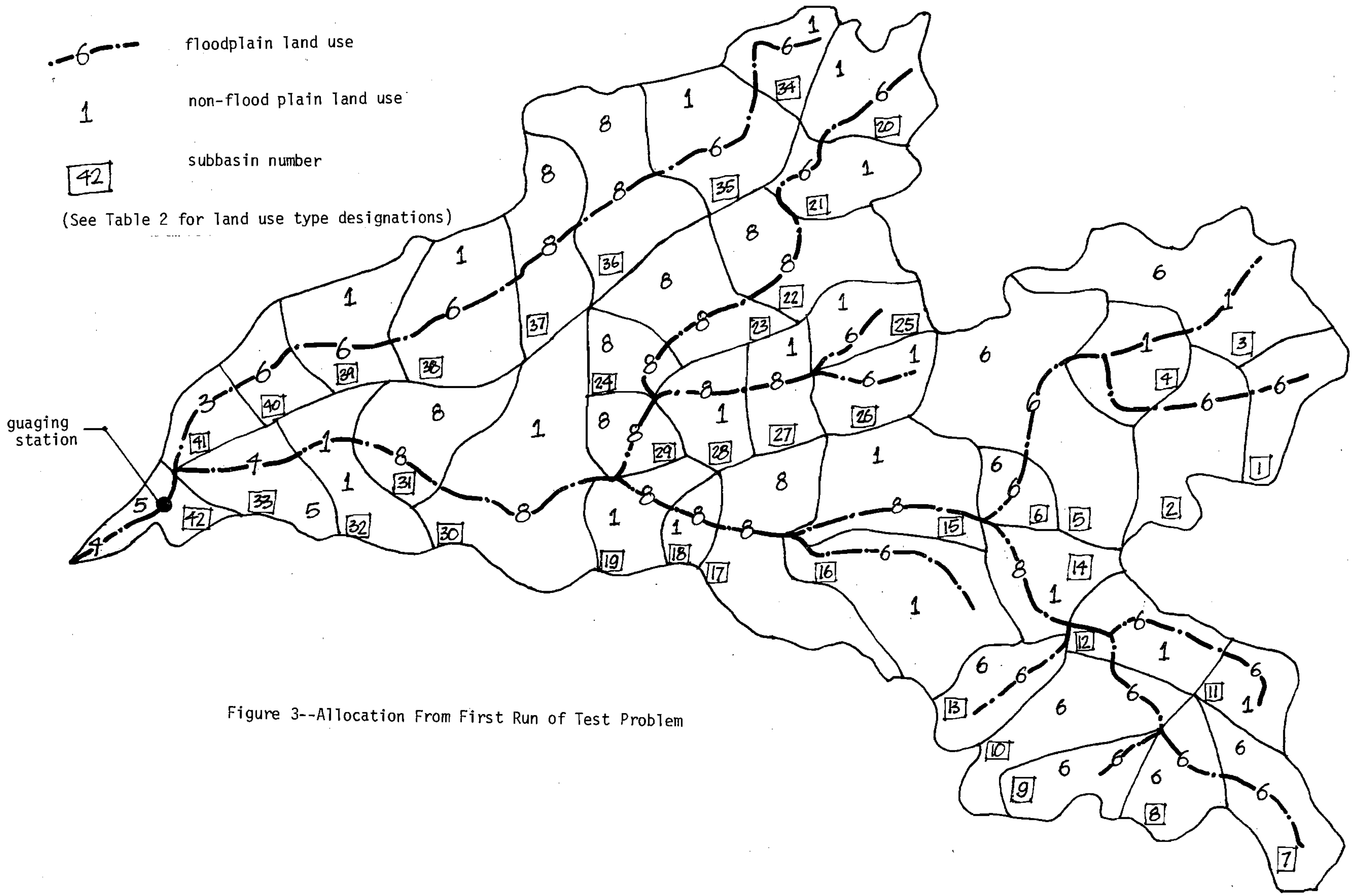


Figure 3--Allocation From First Run of Test Problem

assigned the use with the highest value for that subbasin. As shown in Table 3, only one subbasin in each run, 3 and 21 respectively, was assigned a use with a lower value in order to preclude higher downstream damages. If this interdependence is indeed so slight, it is pointless to develop the dynamic programming model to simultaneously locate upstream and downstream land uses. This result also has important implications for the possible means of achieving optimal patterns in practice (see Sec. 2.).

There are several possible explanations for this result, which was unexpected given the data on the effects of urbanization on flooding. The difference in land value for different land uses may be greater than the flood damages prevented by the changes in runoff thereby achieved. This might change if land use categories based on meeting stringent runoff performance requirements, which have been proposed or implemented in some jurisdictions, were included. Then the change in land value associated with a change in runoff would be the cost of meeting the runoff requirements, rather than the difference between land values for urban and rural land uses. If the costs for providing retention basins and the like were small relative to the differences in land values between high density residential and agriculture, then the apparent interdependence would increase. Determining these relative tradeoffs would provide important information for policy decisions. The extreme case would be construction of a reservoir, which could be treated as a land use category by the model.

The model currently includes only the expected damage from the 100 year flood. This understates the interdependence because the expected damage should also include damages from lesser and greater floods. The particular design storm used in the test problem consisted of a major storm following a small storm. This meant that the differences in runoff characteristics for the different land use types were at least partially nullified because the initial storm was sufficient to saturate soils and thus equalize loss rates. The apparently slight interdependence could also be the result of errors in the data, inaccuracy of the triangular routing model, or characteristics of the particular watershed or land use mix used in the test problem.

7. CONCLUSIONS

The results of this initial experiment suggest the following approach for dealing with a particular watershed. First, determine whether the interdependence between upstream and downstream land use decisions is sufficiently strong to justify dealing with the effects of land use decisions on flooding and the effects of flooding on land use decisions simultaneously. This could be determined by running an accurate routing model for a set of land use patterns that ranged from a high degree of upstream sacrifice in land value for downstream protection to a high degree of downstream sacrifice in land value for upstream increases in runoff. If the highest total values among these patterns result from combinations of sacrifices both upstream and downstream, then the dynamic programming model developed here would be useful for finding good land use patterns. The ramifications for implementation schemes discussed in Sec. 2 would also prevail.

If no sacrifice in land value by upstream users seems justified, then upstream land uses could be determined by models or markets that do not consider flood damage. The resulting flood pattern could be taken as given for determining appropriate flood plain uses. If, on the other hand, no sacrifice by downstream owners seems justified, then the downstream uses determined by models or markets and the assumed flood patterns could be taken as given. The appropriate upstream land uses could then be determined. In either case a linear programming model is implied. A model for the first case has been suggested by Weisz and Day (1974a). The ramifications for implementation schemes for this case have not been explored specifically in the present study, but it is likely that much simpler measures would be feasible.

If the interdependence is demonstrated for at least some watersheds of interest, which seems likely if the changes discussed in Sec. 6 are carried out, then several improvements in the dynamic programming model should be pursued. The capabilities of the model should be extended to incorporate probabilistic expectations of floods of different magnitudes, rather than simply the 100 year design storm used here. The triangular routing procedure should be tested by comparison with observed hydrographs on a larger sample of storms and streams.

If it proves unreliable, then it might be modified to suppress the peaks, given that all but one of the estimated peaks were too high when compared to the Muskingum model. It may be necessary to find another efficient routing scheme. In view of the relatively short computer time required for the model runs, it may be possible to use slightly less efficient, but more accurate routing methods.

The land use types should be defined more appropriately for the sizes of the subbasins by using more general categories involving mixes of activities. Land uses defined so as to meet stringent runoff control regulations and equivalent land uses without these requirements should be included to determine the worth of such regulations. More accurate bid-price data should be generated. More accurate estimates of the area flooded for given flows could be obtained through already developed digital terrain analysis techniques.

The proposed "dry reservoir" on Hickory Creek could be included as a land use. Routing through it would be relatively straightforward because it would be expected to be empty prior to a storm. This would extend the model toward the full range of the concept of allocating hydrologic-land use classes.

Sensitivity analysis should be conducted of both the structuring of the data for the model and of the values of the data. Variations in the data values, including the hydrologic, bid-price, and damage data should be considered using some probabilistic error framework such as those used by Mercer and Morgan (1976) and the Institute of Ecology of the University of Georgia (1971). The approach best suited to the dynamic programming model would have to be determined by further research.

If the interdependence between upstream and downstream land use decisions is not strong for any watersheds of interest, then work on models that assume given flood patterns and allocate land uses should be pursued. Improved data, sensitivity analyses, and perhaps improved model structures are still called for. In either case the value of conceptualizing the flood plain management problem as one of allocating hydrologic-land use classes to parcels of land provides a useful framework for developing models and implementation procedures.

APPENDIX A

HYDROLOGY COMPUTATIONS

Most of the hydrology computations were carried out using programs included in HEC-1 Flood Hydrograph Package developed by the Hydrologic Engineering Center of the U. S. Army Corps of Engineers. Section A.1 describes the computation of the hydrographs for each use on each subbasin. Section A.2 describes the computations for the Muskingum routing method, which was used as a standard of comparison for the triangul r routing method and to obtain more accurate hydrologic response data for land use patterns found by the dynamic programming model.

A.1 Calculation of Unit Hydrographs

The unit hydrographs were computed using the Clark method (Clark, 1945) as programmed in the HEC-1 package. The HEC-1 programs perform "lumped parameter modeling of the precipitation runoff process," which means that spatial and temporal average values of the input parameters are used. The synthetic dimensionless time-area curve provided in the HEC-1 package was used. As described in the HEC-1 User's Manual, this is

$$AI = T^{1.5}/0.707 \text{ for } 0<T<0.5 \tag{A1}$$

$$1 - AI = (1-T)^{1.5}/0.707 \text{ for } 0.5<T<1 \tag{A2}$$

where

AI = area as a ratio of total basin area

T = time as ratio of time from beginning of runoff to time of concentration

This dimensionless curve is first transformed into a dimensional curve using the basin area and time of concentration. It is then converted to a time-area runoff curve, I, with a total of one inch of precipitation excess. The unit hydrograph is this runoff curve routed through storage at the outlet of the subbasin using the Muskingum routing procedure with the Muskingum X coefficient set equal to zero and the storage coefficient, R, set at the appropriate rate for the subbasin. This is descirbed in the HEC-1 User's Manual as

$$Q_2 = (CA \cdot I) + (CB \cdot Q_1) \qquad (A3)$$

$$QUNGR = 0.5(Q_1 + Q_2) \qquad (A4)$$

$$CA = TRHR/(R + 0.5 \cdot TRHR) \qquad (A5)$$

$$CB = 1-CA \qquad (A6)$$

where

Q_2 = instantaneous flow at end of period

Q_1 = instantaneous flow at start of period

I = incremental area during period (converted to cfs per inch of rainfall)

QUNGR = unit hydrograph ordinate

TRHR = tabulation interval in hours

R = basin storage coefficient in hours

The HEC-1 program terminates its computation of unit hydrograph ordinates when the hydrograph volume exceeds .995 inches or when 100 ordinates have been computed. An exponential recession of the flow from preceding runoff is used to describe base flow.

$$Q_2 = Q_1 / RTIOR^{0.1} \qquad (A7)$$

where

RTIOR = ratio at recession flow to that 10 intervals later

Fitting of parameters for the hydrologic models was hampered because there was only one stream gauging station with a sufficiently long record and this was at the base of Hickory Creek, in Joliet. A flood hydrograph was constructed from the flow records by plotting the peak flow and the time at which it occurred and the average daily flows for the preceding and succeeding days, each as of noon of the day they occurred. These points were then joined to form a flood hydrograph. The storm of April 28, 1959 was used to fit the parameters because the resulting hydrograph was relatively smooth and could therefore be readily manipulated. The HEC-1 program for fitting parameters was used to find the loss rate parameters that minimized the weighted squared deviations between the observed hydrograph and the hydrograph generated using the given rainfall data. This was done for the Hickory Creek basin as a whole because only one gauging station provided useful observed flows. The sensitivity of the peak flow to variations in these parameters enabled the most significant parameters to be isolated so that more emphasis could be placed on estimating them.

Some of the parameters determined for the whole basin are not applicable to the subbasins used for the dynamic program; time of concentration and storage coefficient were determined on a subbasin level. From the characteristics of the eight land uses and the parameter fitting just described the loss rates and runoff parameters for each subbasin and land use were determined. These are shown in Table A1.

For rural land uses the time of concentration for each of the subbasins was determined by measuring the longest flow path to the downstream boundary of the subbasin and its average slope. These were taken from United State Geological Survey topographic maps. Time of concentration was then computed using the Kirpich formula.

$$t_i = \left(\frac{L}{\sqrt{5}} \right)^{0.77} \qquad \text{(A8)}$$

where

L = length of channel in feet

s = channel slope in feet/foot

t_i = time of concentration in hours

For urban land uses a drainage system was assumed to have been installed. A grid with block width of 500 feet was superimposed on each subbasin and the longest flow path for that system determined. A typical flow velocity of 5 feet per second in gutters and sewers was applied to this flow length and the effective time of concentration computed as

$$t_c = \frac{L}{v} \tag{A9}$$

where

L = longest flow length in feet
v = flow velocity in feet/second
t_c = time of concentration in seconds

The storage coefficient in hours for the Clark unit hydrograph method could not be obtained directly. However, investigation of the sensitivity of the peak to variations in this parameter showed that the peak flow was not particularly sensitive to these changes. A value of .3, which is typical for areas of the size of the subbasins in this project was assumed for all 42 subbasins. The ratio of the recession flow to that ten intervals later, a required input parameter, was taken from the hydrograph generated at the gauging station in Joliet.

The proportion of the subbasin that would be impervious for the various land uses was taken from tables in SCS National Engineering Handbook (Soil Conservation Service, 1969), "Urban Storm Drainage Criteria Manual" (Wright-McLaughlin Engineers, 1969), and The Costs of Sprawl (Real Estate Research Corporation, 1974). The initial rainfall loss and constant loss rate after this initial rainfall loss had been fulfilled were taken from the first two references.

These data were then used in the Clark method hydrograph generation program of the HEC-1 package to generate and plot hydrographs for each land use on each subbasin. The triangular hydrographs for use in the dynamic programming model were fitted to the plotted hydrographs by eye. The peak of the original hydrograph was taken as the peak of the triangle. The rising limb, which was very close to a straight line in most cases, was approximated by a straight line from the point of initial rise to the peak. The recession limb was approximated

TABLE A.1

Summary of Coefficients Used for Each of the Eight Land Uses

RTIMP = Proportion at basin that is impervious
STRTL = Initial rainfall loss in inches
CNSTL = Uniform rainfall loss in inches/hour

Land Use	STRTL	CNSTL	RTIMP
1	0.22	0.5	0.34
2	0.21	0.5	0.40
3	0.20	0.5	0.45
4	0.20	0.5	0.46
5	0.1	0.5	0.95
6	0.35	0.5	-
7	0.4	0.5	-
8	0.5	0.5	-

by a straight line such that the total area under the line was equal to the total area under the recession limb of the original hydrograph.

A.2 Calculations for Muskingum Routing Method

The Muskingum routing method (Chow, 1964; Rockwood, 1964) as programmed in the HEC-1 package was used to provide more accurate hydrologic response from the land use patterns found by the dynamic programming model than could be generated by the triangular routing method used within the dynamic program. In the Muskingum method storage in the reach is a function of both inflow and outflow, where the relative importance of these flows is given by the coefficient X.

$$Q_2 = (CA - CB)\ I, + (1-CA) \cdot Q_1 + CB \cdot I_2 \qquad \text{(A10)}$$

$$CA = 2 \cdot TRHR/(2 \cdot AMSKK \cdot (1-X) + TRHR) \qquad \text{(A11)}$$

$$CB = (TRHR - 2\ AMSKK \cdot X)/(2 \cdot AMSKK \cdot AMSKK \cdot (1-X) + TRHR) \qquad \text{(A12)}$$

where

X = Muskingum weighting coefficient
AMSKK = subreach travel time (Muskingum 'K coefficient')
I_1 = inflow at beginning of period
I_2 = inflow at end of period

As there were no flow records within the watershed except for the gauging station at Joliet, it was not possible to use the HEC-1 program package to obtain initial estimates of the required parameters. The Muskingum method was used because the parameters required could be more easily estimated. Cross sections were available for subbasins on the Spring Creek and Marley Creek watersheds within the Hickory Creek watershed. Other channel cross sections had to be estimated, though field work is currently in progress at the Illinois State Water Survey. A basic shape as shown in Figure A.1 was assumed with variations in D, d and W as the cross section were taken further upstream.

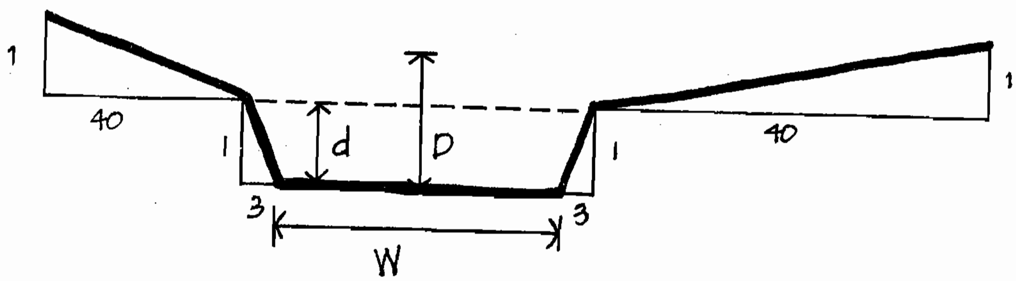


Figure A.1 - Cross Section

Given the cross section for each reach and an average slope for each reach, computed from a topographic map, a flood flow for each reach was calculated. It was assumed that the peak flow at the guage at Joliet could be used as a base and that the magnitude of flows at each upstream reach was directly proportional to the total area upstream of the reach; the flow at each reach is the same portion of the flow at Joliet as the area of above the reach is of the total area above Joliet.

The time of travel for each reach was calculated under the assumptions of uniform flow in the cross section of that reach. Manning's formula was used to compute velocity in the cross section.

$$V = \frac{1.49}{n} \cdot R^{2/3} \cdot S^{1/2}$$

where

R = hydraulic radius = wetted perimeter/flow area
n = Manning's n
S = channel slope in feet/foot
V = velocity in feet/second

This velocity was assumed to be the average velocity in that reach, which yields the travel time in the reach as

$$AMSKK = \frac{L}{V} \tag{A14}$$

where

L = length of reach channel in feet

The assumption of uniform flow ignores backwater effects and non-steady and non-uniform flow; these assumptions are consistent with the assumptions of the dynamic programming formulation.

The Muskingum X coefficient was set at a typical value of .3 for all reaches because it was not possible to estimate values from the available data.

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