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EVALUATION OF THE FLOOD RISK FACTOR IN THE DESIGN OF BOX CULVERTS

Vol. 1. Theoretical Development

G.K. Young, R.S. Taylor, and L.S. Costello

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16. Abstract

This report is an engineering analysis procedure to reduce flood-related damage to highways on a sound probabilistic basis, considering hydrologic hydraulic, and economic factors.

This volume is the first of a two-volume report. The second volume, which concerns documentation of the computer program, is listed below.

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PREFACE

The work presented in this report represents a preliminary effort to integrate economic factors with the physics of highway drainage. Conventional culvert design rests on the selection of a flood peak flow having a particular return period; for example, the flood flow expected to occur once in a hundred-year period. A culvert is designed to pass the selected flood flow. To date, however, the question of the economic and social consequences of conventional culvert design practice has been begged.

A glimpse of insight into this unsettled question is the goal of this report. The general objective is to develop an engineering analysis procedure to reduce flood-related damage to highways on a sound probabilistic basis, considering hydrologic, hydraulic, and economic factors. This objective is implemented using a computerized procedure; the theoretical development, analysis of the procedure, and interpretation of results are the subjects of Volume I, while Volume II describes and documents the computer codes. In brief, construction costs are estimated and balanced against economic losses which are found by the dynamic routing of floods through the stream crossing to estimate damage. Repetitive calculations are performed which include flood routing, estimating embankment failure, and the tallying of resultant losses. A probabilistic approach is employed to determine expected losses.

A few words are appropriate which speak to the emphasized areas of the study. The economic effects of damage to the structure, damage to adjacent upstream area, and traffic-related losses are discussed and presented in detail. Furthermore, a dynamic solution to culvert hydraulics is included which appears to represent an advance in the state-of-the-art of such hydrometric calculations. Finally, the analysis framework is shown operational and thoroughly tested in a sensitivity study which identifies the critical areas for additional research.

A perspective on the economic importance of culverts follows. In 1969, approximately 80,000 miles of road were constructed in the Republic at a cost of $8.33 billion. About two out of every ten miles are influenced to a greater or lesser degree by drainage structures. In fact, approximately 25 to 30 percent of highway construction costs are associated with drainage. Consider the importance of the economics of culvert design by reviewing the effects of hurricane Camille (1969) in Virginia. This storm is known to have produced 27 inches of rain, 90 percent of which fell during an eight-hour period. In Virginia, Camille caused 85 deaths, 72 presumed deaths, 133 destroyed bridges, 25 destroyed miles of primary road and 175 destroyed miles of secondary road. The total highway damage is estimated at $19 million. Other flood-related losses were severe; for example, the flood of the James River at Richmond caused $9 million in damages to business and industry. This study attempts to integrate such flood-related damages into culvert design.

Although the analysis is as comprehensive as possible, the following limitations are noted:

1. High-velocity discharges are not considered in the damage estimates; in fact, a major assumption is that damage to the culvert and road is proportional to the erosion of the downstream fill slope.

2. Seepage and possible pressure losses are not included.

3. A downstream stage-damage function is not considered; the effects of a bursting type of failure with a wave of water proceeding downstream causing damage as it travels are not included.

4. For the purpose of the analysis, it is assumed that the hydrologic data is given; little effort is given in the study to the definition of flood peaks, flood hydrograph shapes, or return-period estimates as these are assumed known.

This report had the cooperation and constructive support of many individuals and agencies. Messrs. Arthur L. Pond, Jr., E. C. Cochran, Jr., L. H. Love, and C. F. Cousins of the Virginia Department of Highways were consulted and furnished example data for the case studies. Data and advice were also obtained from Messrs. C. M. Garza and William W. Smith of the County of Fairfax, Virginia.

The study is indebted to Mr. H. G. Bossy of the Bureau of Public Roads whose knowledge of culvert hydraulics was invaluable in the conduct of the work. In the early phases of the study, Mr. Bossy was the principal government contact for the study team; he coordinated valuable information about drainage design and highway economics received from his colleagues, Mr. Lester Herr and Mr. Robley Winfrey. A draft of Volume I was received and enhanced by a group of Bureau Staff that included Messrs. F. K. Stovicek, H. A. Jongedyk, R. E. Trent, and J. M. Norman.

Three areas of analysis were conducted by consultants to the study team: (i) numerical methods for culvert hydraulics by Professor J. H. Baltrukonis, Catholic University, (ii) erosion failure in cohesive soils by Professors J. Wiggert and D. N. Contractor, Virginia Polytechnic Institute, and (iii) stage-damage economics by Mr. J. J. Hanks, Resources Development Associates. These three studies complemented and filled in the logic required to implement the analysis procedure.

The authors of this report were the principal members of the Water Resources Engineers, Inc. team assigned to the study. The WRE team was stimulated by example and by the continuing encouragement and leadership of G. T. Orlob, our technical director and president. The study also received enthusiastic support from WRE engineers, Messrs. G. F. Tierney, M. R. Childrey, and N. White who reviewed and improved the analysis in the course of their technical contributions to the study. Mrs. Alex Felker provided administrative services and typed the manuscript; in this effort she was assisted by Mrs. Stanton Swafford.

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CHAPTER I INTRODUCTION

The objective of this study is to develop an engineering analysis procedure to reduce flood-related damage to highways on a sound probabilistic basis, considering hydrologic, hydraulic, and economic factors.

Analysis procedures for culverts are presented. The development of the procedures centers on box culverts; however, the same approach, with some modification, could be extended to include bridges as well as culverts of other shapes. The concepts and most of the economic measurements remain essentially the same for culverts and bridges with the major differences being in the hydraulic computations.

Several terms relating to the analysis have particular meanings:

1. *Construction costs* are the costs associated with concrete, steel, fill material, equipment, and labor necessary to build a structure.
2. *Losses* are economic costs associated with a particular flood. Losses include damage to the structure itself, flood damage to the adjacent areas, and traffic-related costs involving delays, accidents, and increased vehicle operations costs. For each flood, there is a probability of occurrence.
3. *Risk* is the sum of the products of the probability of flood occurrence and flood-related economic losses.
4. *Decision variables* define the structural design. Decision variables are manipulated in order to find an economically efficient design. For culverts, fill height, culvert width and depth are decision variables; for bridges, the list is longer.

5. *Data* includes other variables, such as flood hydrographs, unit costs, and accident statistics.

6. *Designs* are specified when decision variables are given values.

The *economic response* (also referred to as *total cost*), is the sum of the annual construction cost and risk. There is a unique economic response for each design. The mathematical objective of this study for a given set of *data* is to *find that design which minimizes the economic response.*

This study presents a procedure which efficiently implements the mathematical objective. Central to the study is the capability to determine the economic response. This capability is based on a computer procedure which implements the risk analysis. The first step in the procedure is to calculate construction costs. Next, floods are routed through the stream crossing structure to determine stream stages which are then used to predict losses. Repetitive calculations are performed which include routing floods, estimating embankment failure, and tallying the resulting losses. The analysis cycle ends with a computation of the economic response and is repeated to search for a design which minimizes this response.

As an example of the results of risk analysis, consider Figure 1-1. This Figure shows the two-dimensional computer output plots of construction costs (C), risk (R), and economic response (T = C + R) versus area of culvert opening for a typical culvert site having fixed fill height (F = 13). The computer plotted symbols (C, R, T) show some scatter and do not define completely smooth curves; this irregularity is due to the fact that waterway area does not capture all the variability of the risk analysis. Costs

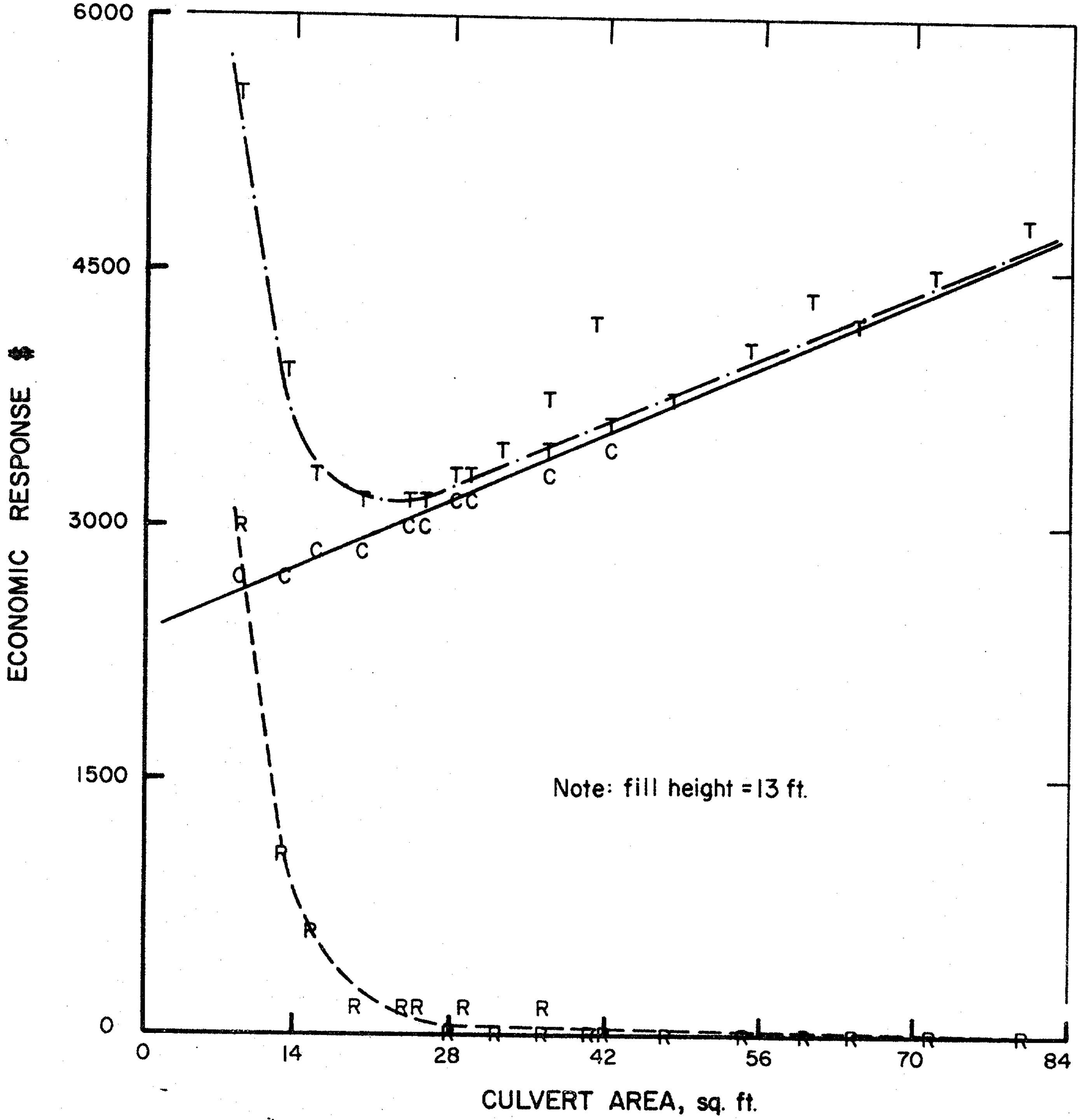


FIGURE 1-1

EXAMPLE RISK ANALYSIS FOR THREE BARRELS

and risks depend on both the breadth and depth of a culvert; however, their trend as a function of area is evident and shows the essence of the analysis.

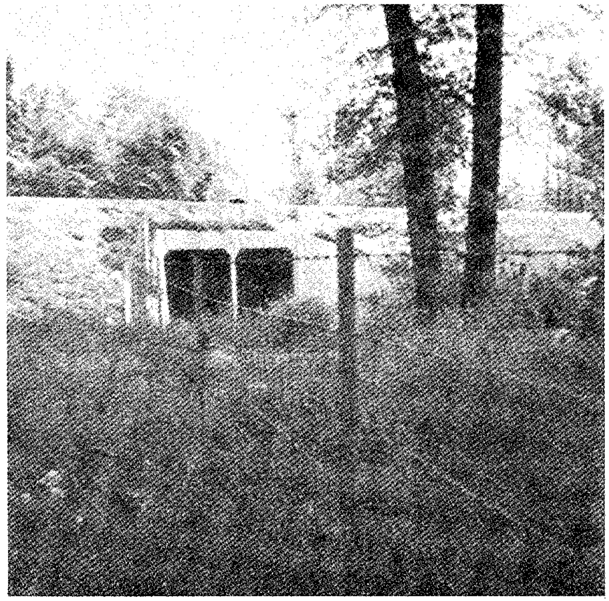
Construction cost varies (approximately) linearly with area; as the area of the opening increases, the costs increase. Risks vary with the inverse of the area; small areas show high risks and as the area increases, the risk decreases. The sum of construction cost plus risk demonstrates a pronounced minimum on Figure 1-1 between 14 and 28 square feet. The search for this low point is the focus of the study; the estimation of R and C requires economic and physical characteristics, the description of which forms the majority of this report.

Two case studies are used to test the techniques developed herein. These cases provide actual data upon which the development effort is based. The strategy of this presentation is to discuss the generalities of the methodology in a given chapter first and then follow with case study information to provide examples; thus, this presentation alternates between the general approach and specific details.

The cases represent widely different situations.

1. Branch of the Great Creek, Brunswick County, Virginia, Interstate 85. This case represents a rural interstate situation.
2. The Glade, Fairfax County, Virginia, Twin Bridges Road. This case represents a secondary road in a suburban residential area.

A summary of data which describes both cases is given in Appendix A. Figures 1-2 and 1-3 are pictures of the crossings. The pictures show the pasture land surroundings of the Interstate 85 site and the wooded, gorge-like terrain of the Twin Bridges Road site.

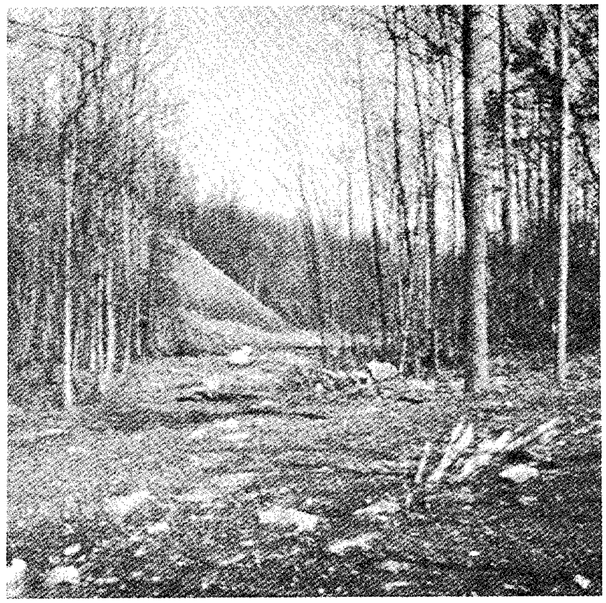


Location of Culvert Crossing

Flood Loss Potential - Agricultural

Figure 1-2

Interstate 85 Site



Location of Culvert Crossing at
Toe of Embankment

Flood Loss Potential - Residential

Figure 1-3
The Glade Site

CHAPTER II CONCLUSIONS AND RECOMMENDATIONS

General results pertaining to the evaluation technique and its application are presented. Specific findings based on the case study data which may have broad implications are listed. Recommendations are based on the strengthening and furthering of the completed work to form the basis of operational procedures for highway design agencies.

General Conclusions

1. An engineering analysis technique which considers flood-related damages to highways as well as construction costs is demonstrated for box culvert design. The procedure has a probabilistic basis and considers hydrologic, hydraulic, and economic factors. Construction costs and economic risks are computed as a function of geometry, hydrology (a set of inflow hydrographs), accident statistics, and the stage-damage information at the site. Probabilities of annual occurrence of runoff hydrographs are applied to the associated losses to determine risks.

2. The technique illustrates the range of tangible economic losses which result from flooding. Conventional design for a particular flow (for example, the one-in-fifty year flood) does not explicitly consider these losses. Therefore, the analysis quantifies, to the maximum extent practicable, the economic consequences to society of culvert design decisions. As the analysis is applied and designers gain in their understanding of the economic-engineering cause and effect phenomenon, design refinements and improvements will result.

3. Automated culvert design for construction cost plus risk is feasible. The automation can be directed to select optimal designs. A designer exerts a strong influence over such automation as he must define the site information and pass on the adequacy of resultant answers.

4. Data requirements for implementing the technique are modest in terms of numbers for computer input. However, considerable engineering is required to process and secure the data for a single application. The additional labor costs over conventional design practices lie primarily in the development of stage-damage curves and other economic data; in addition, more extensive geometric information (such as a stage-storage curve) is required.

5. The analysis in its complete form is probably most applicable to complex design situations involving culverts costing thousands of dollars. Less complex cases, or those having lesser cost considerations, can be handled using simplified or generalized versions of the technique. However, should the technique be applied on a continuing basis by a design agency, file information can be acquired that will facilitate usage.

6. Culvert hydraulic computations are an integral part of the analysis and the techniques developed represent a significant improvement in the state-of-the-art. The technique includes ponding, outflow and headwater prediction as a function of time and analysis of the complete inflow hydrograph. Both inlet and outlet control predictive equations are employed in the procedure depending on the inter-relationships of headwater, tailwater, and culvert geometry.

Case Study Conclusions

1. The optimal three-barrel design for the rural interstate case study (Interstate 85) is: width (B) = 4, depth (D) = 4, total annual cost = $4760, annual construction cost = $4249, and annual risk = $520. The optimal one-barrel design for the suburban secondary road case study (The Glade) is: width (B) = 5, depth (D) = 7, total yearly cost = $13,450, annual construction cost = $11,760, and yearly risk = $1690. Conventional designs for these sites, for the one-in-fifty year flood peak, have greater waterway openings than the optimal designs. To approximate the optimum solution using the present design practice requires the selection of the one-in-five year storm for the I-85 site and the one-in-one year storm for The Glade, as the design flood peak frequencies. The reduced waterway openings of the optimal designs, over the conventional designs, is attributable to the permissibility of ponding and the acceptance of occasional losses in order to reduce construction costs. The total social costs (that is, the sum of construction costs plus the expected losses or risks), are lower when some ponding is allowed. For both case studies, the optimal designs had 9% of their total social costs in the risk category and 91% in construction costs. The conventional practice leads to much lesser levels of risk (0 and 3% of the total social cost) for the cases studied.

2. The incremental cost to society of over design is much less than an equivalent under design. Let:

T_u = total social cost of under design,

T = total social cost of optimal design, and

T_o = total social cost of over design.

The incremental costs to society are:

$$\Delta_u = T_u - T, \text{ for under design, and}$$

$$\Delta_o = T_o - T, \text{ for over design.}$$

For example, the optimal design might be 100 square feet of opening. Then T_u could correspond to 90 and T_o to 110 square feet. The case studies indicate that Δ_o is much less than Δ_u in the vicinity of the optimal design.

3. The factors having the greatest potential for shifting a design to another size in the event of factor estimating error are:

a. interest rate,

b. unit costs of construction materials,

c. stage-damage curves,

d. highway speed,

e. inflow hydrographs, and

f. erosion.

These results derive from sensitivity study of how the total social costs vary with perturbations in individual factors. The list is not ranked and represents the combined results of both case studies.

Recommendations

1. Additional case studies should be performed to make stronger inferences. Two approaches are possible, both of which are recommended. The first is to pursue additional case studies at the Washington, D.C. staff level under the supervision of members of the B.P.R. staff. The second is to conduct the analysis in a state highway department office with testing at the field level. A prime aim is to review and refine the findings

of this report concerning the relationship of optimal designs to the existing or conventional design practice.

2. Further investigation of failure mechanisms should be conducted. In particular, the phenomenon and the economic losses associated with high energy discharges are important in addition to erosion associated with overtopping. This recommendation rises, in part, from the finding that ponding yields significant economic savings; however, ponding implies high velocity discharges which were not explicitly considered in this work.

3. A review of the data requirements which underlie the procedure should be implemented. The objective is to reduce the data to the smallest set of numbers, curves and nomographs that capture the physical and economic measurements, yet provide satisfactory answers. Simplifications and design aids which generalize data of various types are desirable. (For example, the use of an assumed traffic distribution for the general situation.)

4. The methodology (with modifications that reflect the different economic and hydraulic conditions) should be extended to bridge waterways. The risk analysis procedures of culverts provide a framework from which economic criteria can be incorporated into bridge design. Additional work on the hydraulic phenomenon associated with bridge backwater is a necessary component for extending the methodology to bridges.

CHAPTER III ESTIMATION OF COSTS AND RISKS

The risk analysis framework is discussed in this chapter. Details of the various components of the framework are given in subsequent chapters. The major assumptions and overall logic are presented; Figure 3-1 shows the analysis procedure which uses five steps to evaluate each design:

1. calculate annual construction costs,
2. perform flood routing,
3. estimate embankment erosion,
4. calculate losses, and
5. weight losses with flood probabilities to derive risks.

In brief, the analysis calculates the economic response or expected tangible total costs of the stream crossing to society. The definition of economic response is the sum of the annual construction cost and the expected flood-related loss, or risk. The construction costs are computed for each design by the quantity and unit cost method using standard designs.

For each design an estimate is made for structural damage, flood damage to upstream property and traffic-related losses. Structural damage is assumed to be directly related to the extent of embankment erosion caused by overtopping. Flood damage derives from a stage-damage function that is formulated for each crossing site. Traffic-related losses include the cost of lost time, increased running costs, accident losses on the detour, and accident losses due to an unexpected obstacle or barricade placed at the stream crossing site when a failure of the road surface occurs.

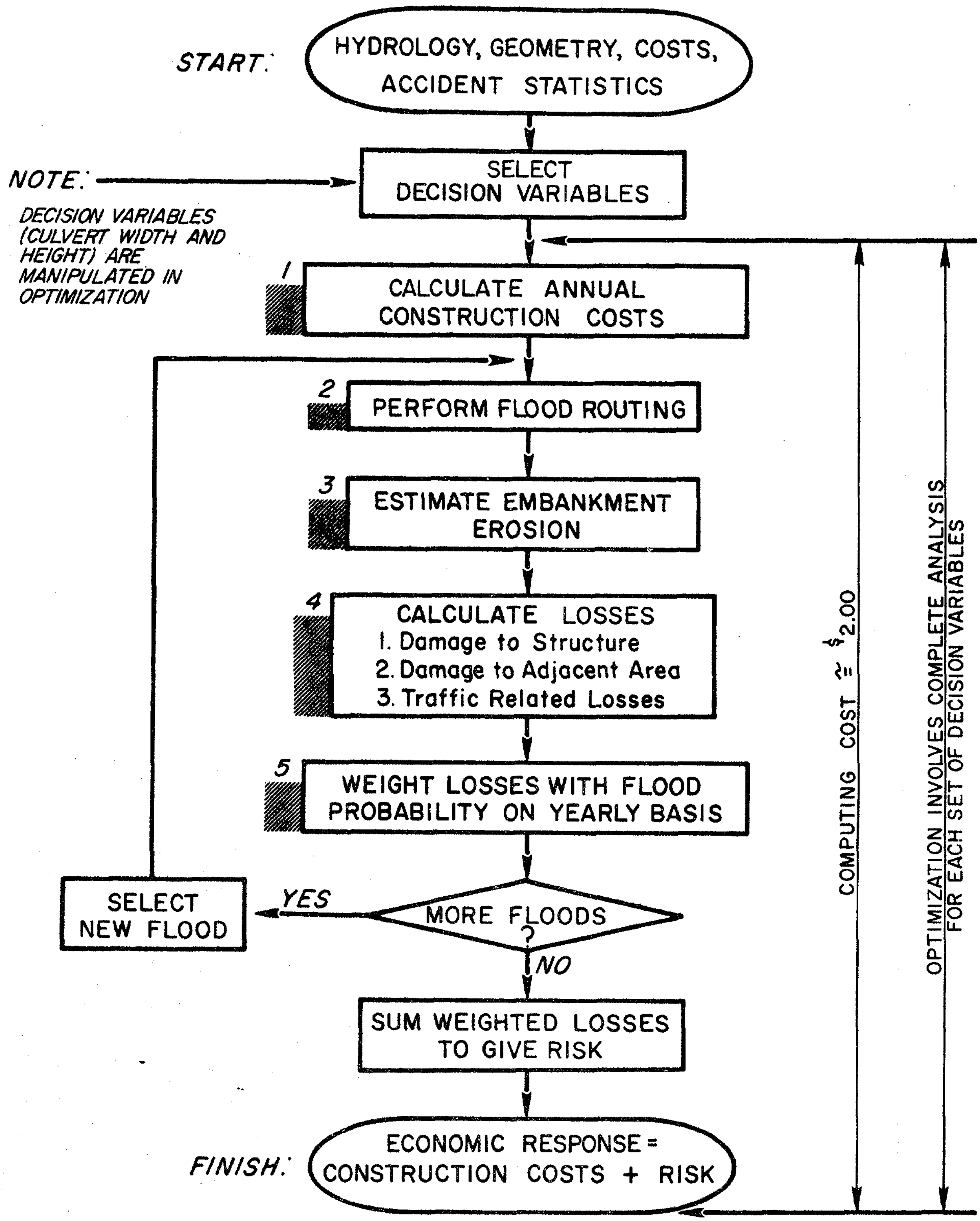


FIGURE 3-1

ANALYSIS PROCEDURE

Estimates of structural damage, flood damage, and traffic-related losses are multiplied by the probability of yearly flood occurrence to obtain the risk for each flood hydrograph; summing the risks over the set of flood hydrographs yields the risk component of the economic response.

1. Calculate Annual Construction Costs

Annual construction costs constitute the cost and contribute to the risk component of the economic response. (That is, annual structural cost is one component of the economic response.) Large floods will cause damage to the stream crossing structure. The estimated cost of repairing the structural damage is weighted by the probability of flood occurrence to determine the expected annual repair cost.

The procedure for cost estimating is the familiar quantity and unit cost method; cost data are based on standard designs. Given grade and road surface elevations along the highway center line, the embankment top width and upstream and downstream slopes, the computation of roadway and embankment quantities is possible. The culvert dimensions determine the cost of the culvert barrel, inlet and outlet structures, and excavation.

The cost of the inlet and outlet portions (e.g., headwalls, wingwalls) of the culvert are calculated from the standards for culvert designs. The depth of excavation for the culvert barrel is assumed to be a function of only the culvert height with no limitations imposed because of geologic or topographic conditions peculiar to the site. The unit cost of roadways includes the cost of the guard rails, shoulders and pavement thereby implying the width of the road.

The total first construction cost equals the sum of the costs of the elements of the design. An interest rate and amortization period are used to compute a capital recovery factor. The annual construction cost is the product of the first cost and the capital recovery factor. The difference in annual maintenance costs for varying culvert sizes are assumed to be negligible and are omitted from the culvert analysis.

2. Perform Flood Routing

For culverts, the influence of storage is used to determine the culvert discharge and overtopping flow. A triangular hydrograph defined by the peak flow, the flood duration, and the time to the flood peak is used. The stage-volume function depends on the topography upstream from the culvert; such a function is usually irregular. The analysis provides for the incorporation of a stage-storage function into the flood routing computations.

The geometry of the culvert, along with the stage, defines the culvert outflow which is estimated using the appropriate inlet or outlet control equations. Should the stage exceed the roadway elevation, overtopping occurs, increasing the total discharge downstream from the culvert. An iterative flood routing technique is used to solve the equation of continuity which relates the change in storage to inflow and outflow. The iterative computations numerically solve the differential equations of flow using a finite difference approximation. The calculations are repeated for all sets of flood hydrographs.

A set of flood hydrographs is used to describe the hydrology at a stream crossing site. In the analysis the relationship between the flood peak and the yearly probability of flood occurrence is specified.

3. Estimate Embankment Erosion

Based on a review of literature related to stream crossing failure, and on-site inspection in the wake of Hurricane Camille, it is assumed that damage and ultimate failure of culvert stream crossings is linked directly to embankment failure -- specifically erosion on the downstream slope due to overtopping. Estimating embankment failure is a preliminary step to calculating repair costs. It is assumed that the fill is undamaged until overtopping occurs and material is eroded from the downstream shoulder. For damage estimation the amount of material carried off is related to the total material in the embankment.

The extent of erosion is related to the velocity on the downstream embankment slope which is determined from the overtopping discharge and the embankment slope and roughness using Manning's Equation. Furthermore, it is assumed that erosion does not begin until a specified threshold velocity is attained after which erosion will be computed independently of the threshold velocity.

The routing calculations yield velocities as a function of time. The amount of sediment water can suspend is related empirically to velocity. Given a threshold velocity above which erosion will occur, a summation over time is performed to calculate the volume of eroded fill.

The individual term in the sum is the concentration of suspended fill material times the water volume. The summation has a lower limit corresponding to the time when the threshold velocity occurs and a final limit equal to the end of overtopping. The summation is conducted in discrete time increments which corresponds to the time increments used in the routing.

4. Calculate Losses

Structural Damage

Structural damage is associated with the volume of material removed from the embankment by overtopping. It is assumed that the downstream shoulder erodes first. As failure progresses, the roadway surface washes away, largely as a result of undermining from the downstream roadway edge. After most of the roadway and embankment is washed away, the culvert itself is subject to damage. The three main structural elements of a culvert stream crossing, embankment, roadway, and culvert are related to embankment erosion; the analysis procedure requires estimates of erosion-repair relationships for all three elements.

Repair Time

In the event of damage to a stream crossing structure, a major portion of the economic loss may be due to a traffic stoppage or a delay caused by an inconvenient detour. The repair time is defined as the time required to restore traffic flow and is related to the degree of damage to the crossing as measured by erosion. For example, five per cent embankment erosion may not stop traffic flow. Minor damage to the roadway, requiring a short traffic stoppage, may occur at 20 per cent embankment erosion. Major erosion at the same crossing may postpone traffic flow for a month. The analysis procedure requires as basic information the estimate of an erosion-repair time relation.

Flood Losses

Flood damage to adjacent property results from the storage of water immediately upstream from the culvert. Flood losses are estimated as a function of flood stage; the extent of flooded areas is determined from a topographic map. Property damage is unique to each stream crossing site and is influenced by the density and type of development.

Traffic-Related Losses

Traffic-related losses are divided into four types:

1. increased running costs,
2. increased time of travel,
3. increased expected accident costs, and
4. increased expected accident costs due to an unexpected obstacle.

After barricades are placed around the site, the first three types of losses are incurred on the detour. The fourth type is the cost of the accidents expected at the stream crossing and is postulated to occur as traffic comes up on the failure immediately after the failure occurs.

Increased *running costs* are the difference between running costs on the detour and the normal route. These costs are computed as a function of average daily traffic, travel distance, duration of detour, design speed, and vehicle distribution. Detour duration is the sum of the overtopping duration plus the repair time. The overtopping duration is computed in the flood routing calculations. Five basic classes of vehicles are assumed and actual traffic distributions are fit, as closely as possible, to the five basic classes. These are:

1. 0.7 ton passenger cars,
2. 1.25 ton commercial delivery vans,
3. 1.55 ton single unit trucks,
4. 2.2 ton gasoline semi-trailer trucks, and
5. 2.75 ton diesel semi-trailer trucks.

Running costs are estimated for passenger cars on zero grades and these costs are adjusted to reflect the other classes of vehicles in the five-class distribution. The passenger car running costs as a function of speed are fit to a parabola as a means for facilitating computer applications. The low point of the parabola, or most economical running speed, occurs between 30 and 40 miles per hour.

Time losses are a function of average daily traffic, duration of detour, travel distance, vehicle occupancy rate, design speed, and the value placed on an individual's time. The occupancy rate and time values are averages which apply to all individuals in the five basic classes of vehicles. Only the difference in time losses between the normal route and the detour are considered.

The death rate is used as the basic unit to compute the increased *accident costs* imposed by the detour. For each death, there are a certain number of personal injuries and a certain number of property damage accidents, each of which may or may not be associated with a death. For example, thirty personal injuries for each traffic death and three hundred property damage accidents might occur for each death. By putting average costs of deaths, personal injuries, and property damage and applying the above rates, the accident losses will be computed. These losses are computed on a vehicle mile basis -- the standard for death rate statistics. *Accident losses* are computed as a function of average daily traffic, length of detour, duration of detour, death rate, ratio of personal injuries to deaths, cost of death, cost of personal injury, and cost of a property damage accident.

The last loss category is the expected accident cost due to the *unexpected obstacle* at the stream crossing site. The higher death rate is defined as the product of the death rate for normal conditions over one mile of roadway, and a death rate factor for unexpected obstacles. This death rate factor is somewhat subjective because of lack of data on this type of accident. Perhaps, a factor of one thousand is appropriate. The engineer also can modify the ratios of personal injuries and property damage accidents to deaths since these ratios may differ from those on the detour.

5. Weight Losses to Derive Risks

An estimate of the economic response for a design requires one complete pass through the analysis shown in Figure 3-1. Construction costs are computed as a function of geometry and unit costs. Losses are computed as a function of geometry, hydrology (a set of inflow hydrographs), accident statistics, and the stage-damage information at the site. Probabilities of annual occurrence of runoff hydrographs are applied to the associated losses to determine risks.

The goal of the culvert designer is to select a design that minimizes the economic response or the sum of the annual construction cost and risk. The construction costs are straightforward to compute; risks involve probabilistic considerations. Each flood hydrograph is routed through a culvert and the expected loss is estimated. For example, if a flood with a 10 per cent chance of occurrence causes losses totaling $100,000, the risk component is 0.1 x $100,000, or $10,000. The analysis considers a set of flood hydrographs having various probabilities, the lower the probability, the higher the loss and vice versa.

When the loss associated with each flood is multiplied by the probability of flood occurrence and the resultant products are summed, the total is the risk. Thus, the risk is the expected value of the loss for a set of flood hydrographs. The economic response is the sum of the annual construction cost and the risk.

The optimization of the design to achieve minimum economic response requires many evaluations of the economic response for various candidate designs. This is feasible with the use of the computer code which implements the analysis presented herein. Details of the various components of the analysis and the sensitivity of designs associated with the two case studies are presented in subsequent chapters.

CHAPTER IV SITE INFORMATION

This chapter presents a specification of the site information used in the analysis. The location and land characteristics of the two case studies are described in detail. Information used to describe the geometry of the crossing is presented in tabular form. A qualitative description of the general data that is needed to define the site characteristics is also presented.

Two case study locations are chosen. One is the crossing of a new section of Interstate 85 over a branch of Great Creek located approximately 65 miles southwest of Richmond, in Brunswick County, Virginia. This site is used to serve as an example of a culvert located on a rural interstate. The second location is a secondary road that serves the suburban community of Reston, Virginia. Specifically, it is the crossing of Twin Bridges Road over The Glade. Reston is located about 20 miles west of Washington, D.C., in Fairfax County, Virginia. Location maps for the case study sites are shown in Figure 4-1.

The Interstate 85 site is located in a farming area. The watershed is mostly pasture land with approximately forty per cent of the area wooded. The drainage basin is about 18,000 feet long by 6,000 feet wide encompassing 2,400 acres. The drainage way has a mild slope of about 0.59 per cent. An estimate of 0.05 is used as the value of Manning's roughness coefficient in the existing stream channel.

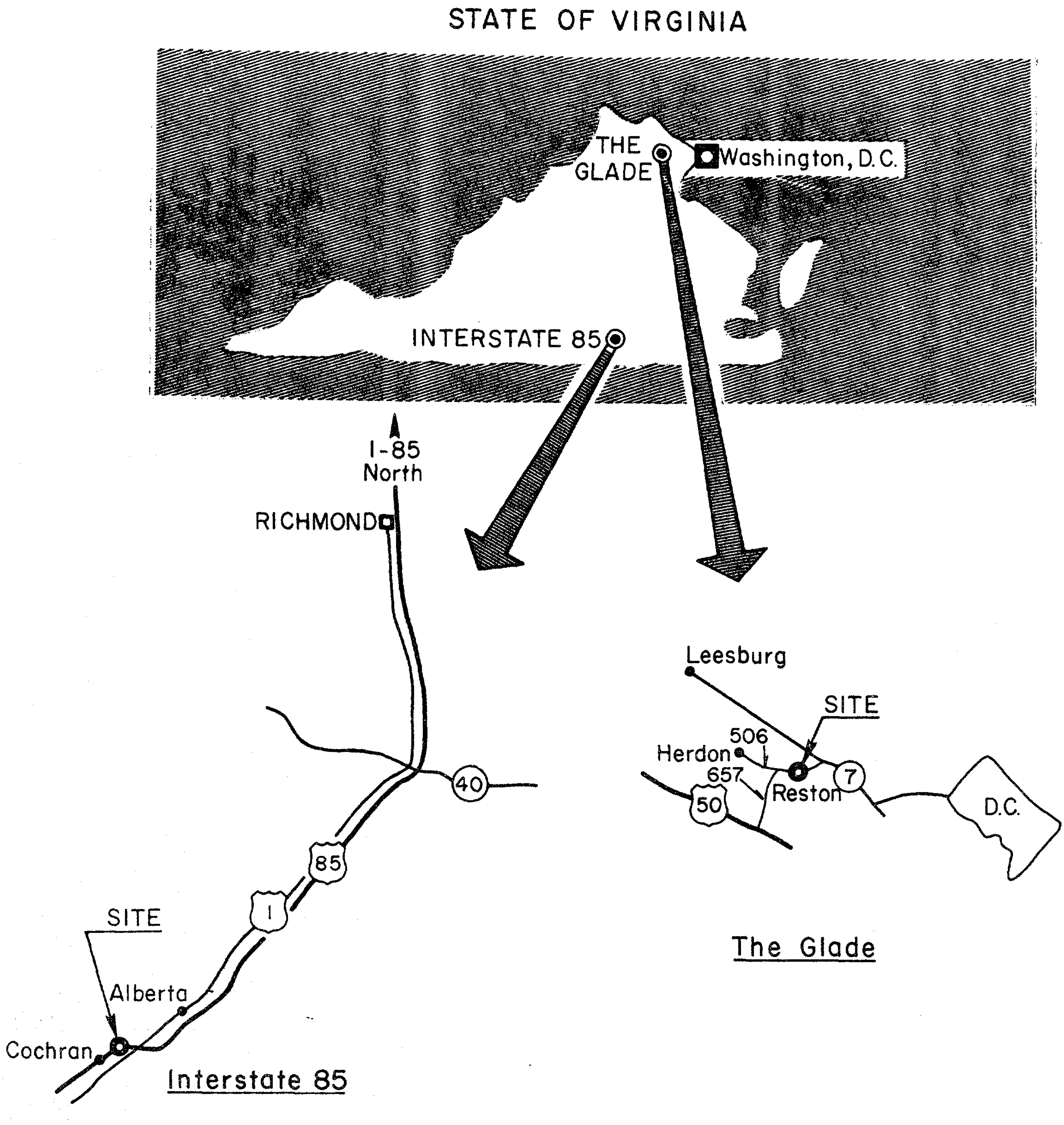


FIGURE 4-1

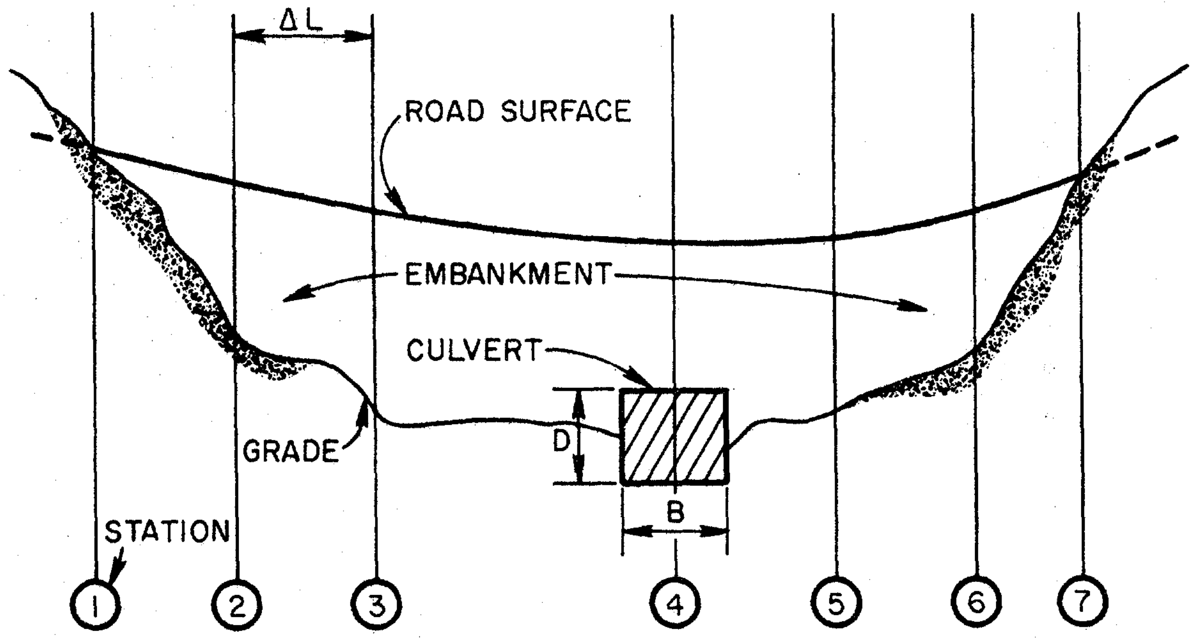
LOCATION MAPS FOR CASE STUDY SITES

The Glade site has a long and narrow drainage area. It is approximately 16,200 feet long and only 2,000 feet wide. Very steep slopes at the drainage divides characterize the watershed for most of its length. At the downstream end the watershed is a gorge. The majority of the watershed is woodlands with heavy undergrowth occupying the flood plain. The main channel with a base flow of approximately one cubic foot per second, winds its way through dense foilage. The value of Manning's roughness coefficient is assumed to be 0.09. The drainage area is about 830 acres with an average stream channel slope of 1.43 per cent.

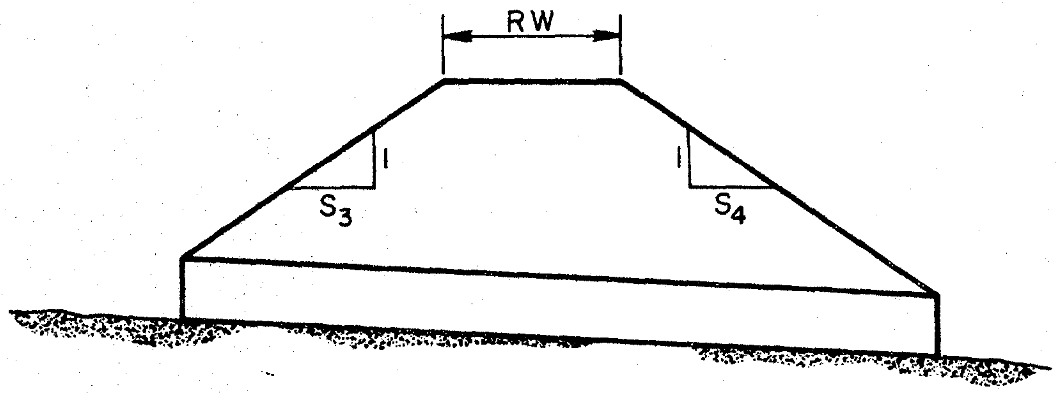
SUMMARY OF BASIN CHARACTERISTICS

Site	Area (acres)	Length (feet)	Average Width (feet)	Slope (ft/ft)	Natural Drainage (n)
Interstate 85	2400	18,000	6,000	0.0059	0.05
The Glade	830	16,250	2,000	0.0143	0.09

Figure 4-2 shows the site geometry that is used in the culvert analysis. The centerline stations are selected at close enough intervals (ΔL) to reflect the topographic conditions at the site. The existing elevations and the completed roadway elevations are taken along the center line. Fill height (F) at each station is the difference between the existing grade and the completed road surface elevation. Culvert width is B, depth is D. Culverts considered are those box culverts listed in the State of Virginia Standards. Standards from another state could also be used to typify designs. The Virginia Standards list designs having from one to four barrels.



Typical Vertical Curve



Typical Cross Section Through Station 4

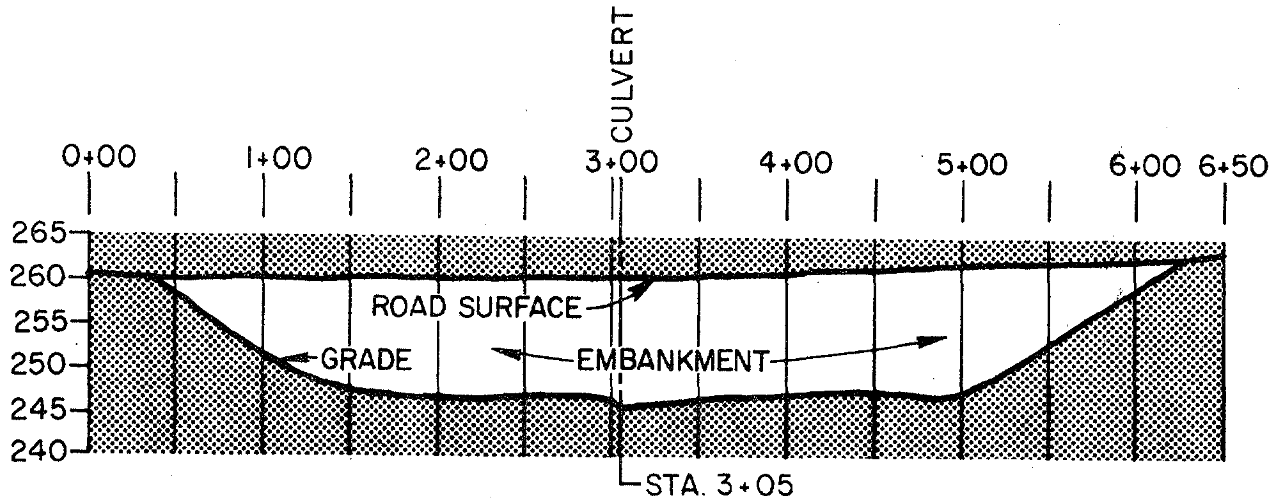
FIGURE 4-2

SITE GEOMETRY

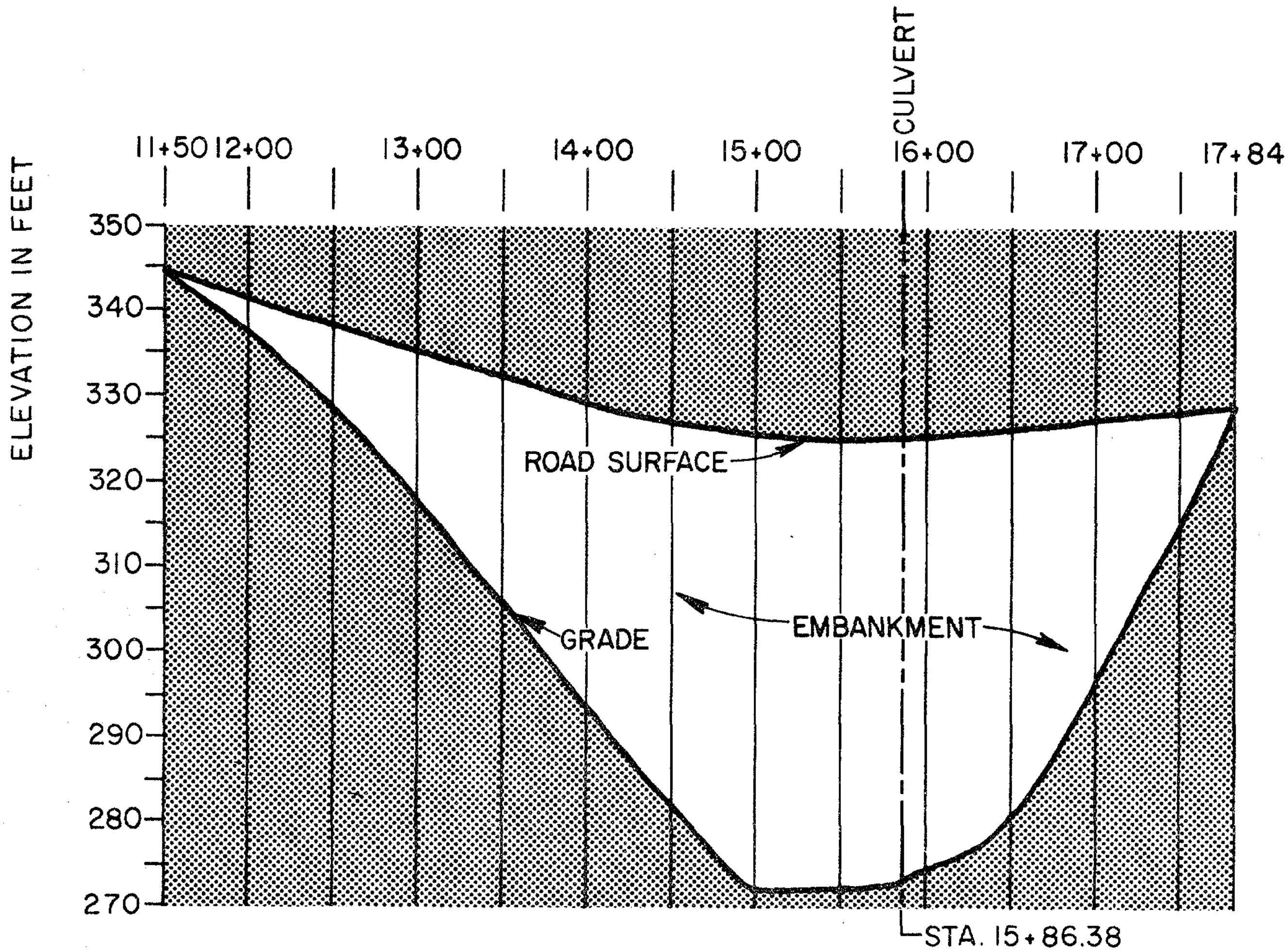
The cross section shown in Figure 4-2 typifies any of the sections taken through the roadway for use in the analysis. RW is the roadway width which is calculated by assuming a single lane is twelve feet wide and the shoulder is fifteen feet. By assuming these values and knowing the number of lanes a value of RW is computed. S_3 is the upstream slope on the roadway embankment which may be different from the downstream slope, S_4. The profiles of the two case studies are shown in Figure 4-3. In summary, the definitions of the geometric variables and their values for the two cases are shown in Table 4-1.

By using the station method to describe the geometry, it is possible to describe the vertical and the horizontal roadway curve. This makes computing the volume of fill and roadway length more accurate than assuming a straight horizontal roadway across the entire drainageway. A value of Manning's roughness coefficient, n, is assumed for the roadway embankment to determine the velocity over the downstream slope for storms of sufficient magnitudes to overtop the roadway. It is assumed that the banks are sodded which implies a roughness coefficient of about 0.03.

The volume of fill required at the crossing site is calculated by the average end area method using center line grade. Figure 4-2 shows a typical culvert site. The volume of fill is computed in sections. At each station the existing grade elevation and the design grade elevation are recorded as in Table 4-1.



Interstate 85



Twin Bridges Over The Glade

FIGURE 4-3

ROADWAY CENTER LINE PROFILES OF CASE STUDIES

TABLE 4-1

EMBANKMENT GEOMETRY

		I-85	The Glade	
F =	Fill Height	14.6	53	(ft)
RW =	Roadway Width	108	54	(ft)
S_3 =	Upstream Embankment Slope	2/1	3/1	(ft/ft)
S_4 =	Downstream Embankment Slope	2/1	2.5/1	(ft/ft)
n =	Value of Embankment Roughness	.03	.03	- - -

Center Line Elevations

I-85			The Glade		
Station	Existing Elevation	Finished Elevation	Station	Existing Elevation	Finished Elevation
0+25	260.10	260.10	11+50	344.25	344.25
0+50	258.30	260.05	12+00	341.25	337.50
1+00	251.60	260.01	12+50	338.25	328.50
1+50	247.20	260.03	13+00	335.25	317.90
2+00	246.80	260.11	13+50	332.25	306.00
2+50	246.50	260.24	14+00	329.35	294.00
3+00	246.50	260.42	14+50	327.15	282.50
3+05*	245.80	260.43	15+00	325.75	272.00
3+50	246.80	260.67	15+50	325.15	272.40
4+00	247.00	260.96	15+86.38*	325.22	273.00
4+50	247.25	261.32	16+00	325.35	274.50
5+00	247.10	261.73	16+50	326.25	281.00
5+50	253.00	262.19	17+00	326.18	296.25
6+00	259.10	262.71	17+50	328.18	315.00
6+25	263.00	263.00	17+84	328.86	328.86

* Culvert Location.

The difference in the elevations represents the fill height at that station. Using this fill height, an area of a cross section is computed at station 'n' by the following equation:

$$A_n = (RW \times F) + 1/2\ S_3F^2 + 1/2\ S_4F^2 \qquad (4\text{-}1)$$

To compute the volume between two stations, the average cross-sectional area of the two sections is multiplied by the distance separating the stations:

$$V_n = 0.5\ (A_n + A_{n+1})\ L_{n,n+1} \qquad (4\text{-}2)$$

The total volume of fill, V, is found by adding the contributions of each of the sections,

$$V = \Sigma V_n \qquad (4\text{-}3)$$

An estimate of the roadway length, R_L, is obtained by using the Pathagorean Theorem, thus

$$R_L = \Sigma\sqrt{\Delta x^2 + \Delta y^2}\ , \qquad (4\text{-}4)$$

where Δx = horizontal distance between stations, and
Δy = vertical difference in the design grade elevations between stations.

The culvert length, L_c, extends from the upstream to the downstream embankment toe;

$$L_c = (\mathrm{Sec}\,\Theta)\ (RW + FS_3 + FS_4) \qquad (4\text{-}5)$$

where Θ is the skew angle.

The structural excavation, E_c, for the culvert barrel is calculated assuming an excavation depth equal to the square root of the culvert height (D), a width two feet greater than the culvert width (W), and a length corresponding to the culvert length (L_c);

$$E_c = (W + 2)\ L_c\sqrt{D} \div 27.0 \qquad (4\text{-}6)$$

The unit quantities of steel and concrete for the culvert barrel(s), headwalls and wingwalls are taken from Standard Designs used by the Virginia Department of Highways. Table 4-2 summarizes structural quantities for optimum culvert designs at Interstate 85 and The Glade.

The other important site parameters such as upstream stage-discharge curve and the tailwater-discharge curve are discussed in a following chapter on hydraulics.

TABLE 4-2

STRUCTURAL QUANTITIES

VARIABLE	I-85	THE GLADE	COMMENTS
Culvert Width (B) (ft)	4	5	Optimum Design
Culvert Height (D) (ft)	4	7	Optimum Design
Number of Barrels	3	1	Optimum Design
Fill Height (F) (ft)	14.63	52.22	At Culvert
Unit Steel (lbs/ft)	153.78	137.88	Virginia Standards
Unit Concrete (cy/ft)	.965	.853	Virginia Standards
Steel (lbs)	26,800	47,045	- - - -
Concrete (cy)	168	291	- - - -
Fill Volume (cy)	32,700	118,644	- - - -

CHAPTER V STRUCTURAL COSTS

The economic response associated with selecting a culvert of given size is composed of two major categories. First is the structural costs associated with the actual construction of the site. The second cost category, which is described in Chapter VIII, is the loss related to flood damage at the site resulting from routing flood hydrographs through the size culvert selected. This Chapter deals with the methods that are employed to determine the first component of the economic response, structural costs.

The total cost of installing a culvert is composed of the following units:

1. cost of excavation,
2. cost of the culvert (steel and concrete),
3. cost of the fill material, and
4. cost of the roadway.

The cost of excavation is the expenditure associated with the cut area required to install the culvert barrel at the correct elevation. This cost is calculated based on the total amount of excavation in cubic yards (see Chapter IV). The volume of excavation is multiplied by a unit cost.

In order to estimate the culvert cost, the structure is divided into three sections: the barrel, headwalls, and wingwalls. The amount of concrete (cubic yards) and steel (pounds) needed in each section is obtained from the Virginia Department of Highway's standards for box culverts. The standards show the amount of materials needed for a certain size barrel, headwall, and wingwall combination at a given fill height. There are three types of wingwalls based on different flare angles and edge conditions around the wall (rounded or square). A sample of the Virginia Standards, giving the amounts of concrete and steel for one culvert barrel is shown in Table 5-1; where there is no standard design, the tabular entry is zero. The total amounts of concrete and steel are the sum of the amounts needed in the barrel (s), headwalls and the wingwalls. The cost is calculated by multiplying the volume of concrete by a unit cost for concrete ($/cy) in place and the weight of steel by a unit cost for steel ($/lb) in place.

The cost of the fill material is based on the total volume of fill required at the site. The method for determining this volume is described in Chapter IV. The total volume is multiplied by a unit cost ($/cy) of the fill in place to obtain the total cost.

The roadway cost (guardrail to guardrail) is calculated by multiplying the roadway length by a cost factor ($/lf) that includes the cost of materials and labor.

The construction cost factors for the two case studies are shown in Table 5-2. These are supplied as approximate figures for use in the analysis, courtesy of the Virginia Department of Highways. The unit prices for The Glade are generally higher than the Interstate 85 values. This reflects higher costs associated with smaller construction projects. The Glade site is a secondary road in a small subdivision; therefore, the contractor's unit costs are higher than the Interstate 85 site which is part of a much larger project. The one cost factor that appears to be higher at the Interstate 85 is the roadway. However, this higher cost

TABLE 5-1

PARTIAL LIST OF

UNIT QUANTITIES FOR A SINGLE CULVERT BARREL

CONCRETE (CY/LF)

Maximum Fill Height (ft)	Culvert Span (ft)	Culvert Height (ft) 1	2	3	4	5	6
Under 12	1	0.000	0.000	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.000	0.000	0.264	0.300	0.000	0.000
	4	0.000	0.000	0.305	0.341	0.378	0.423
	5	0.000	0.000	0.394	0.431	0.467	0.504
	6	0.000	0.000	0.000	0.525	0.562	0.599
	7	0.000	0.000	0.000	0.687	0.801	0.801
	8	0.000	0.000	0.000	0.806	0.916	0.916
	9	0.000	0.000	0.000	0.981	1.104	0.104
	10	0.000	0.000	0.000	1.172	1.309	1.309
12-25	1	0.000	0.000	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.000	0.000	0.274	0.311	0.000	0.000
	4	0.000	0.000	0.366	0.402	0.439	0.498
	5	0.000	0.000	0.521	0.571	0.620	0.645
	6	0.000	0.000	0.000	0.720	0.793	0.873
	7	0.000	0.000	0.000	0.929	1.089	1.089
	8	0.000	0.000	0.000	1.142	1.317	1.317
	9	0.000	0.000	0.000	1.320	1.568	1.568
	10	0.000	0.000	0.000	1.615	1.850	1.850
25-35	1	0.000	0.000	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.000	0.000	0.305	0.341	0.000	0.000
	4	0.000	0.000	0.431	0.474	0.517	0.559
	5	0.000	0.000	0.559	0.648	0.697	0.747
	6	0.000	0.000	0.000	0.848	0.910	0.964
	7	0.000	0.000	0.000	1.112	1.259	1.259
	8	0.000	0.000	0.000	1.338	1.587	1.587
	9	0.000	0.000	0.000	1.600	1.837	1.837
	10	0.000	0.000	0.000	1.910	1.998	1.998
35-50	1	0.000	0.000	0.000	0.000	0.000	0.000
	2	0.000	0.000	0.000	0.000	0.000	0.000
	3	0.000	0.000	0.350	0.393	0.000	0.000
	4	0.000	0.000	0.485	0.528	0.572	0.598
	5	0.000	0.000	0.666	0.714	0.773	0.813
	6	0.000	0.000	0.000	1.032	1.125	1.225
	7	0.000	0.000	0.000	1.332	1.518	1.518
	8	0.000	0.000	0.000	1.621	1.888	1.888
	9	0.000	0.000	0.000	1.942	2.237	2.237
	10	0.000	0.000	0.000	2.334	2.691	2.691

TABLE 5-1 (continued)

PARTIAL LIST OF
UNIT QUANTITIES FOR A SINGLE CULVERT BARREL

STEEL (LB/LF)

Maximum Fill Height (ft)	Culvert Span (ft)	Culvert Height (ft) 1	2	3	4	5	6
0-12	1	00.00	00.00	00.00	00.00	00.00	00.00
	2	00.00	00.00	00.00	00.00	00.00	00.00
	3	00.00	00.00	32.02	35.03	00.00	00.00
	4	00.00	00.00	43.31	46.34	49.38	52.49
	5	00.00	00.00	54.82	57.85	60.88	63.94
	6	00.00	00.00	00.00	76.26	79.30	82.33
	7	00.00	00.00	00.00	83.89	91.56	91.56
	8	00.00	00.00	00.00	108.51	115.98	115.98
	9	00.00	00.00	00.00	130.18	136.51	136.51
	10	00.00	00.00	00.00	158.83	150.88	150.88
12-25	1	00.00	00.00	00.00	00.00	00.00	00.00
	2	00.00	00.00	00.00	00.00	00.00	00.00
	3	00.00	00.00	51.85	53.74	00.00	00.00
	4	00.00	00.00	58.30	61.62	64.94	75.55
	5	00.00	00.00	90.28	94.20	98.13	101.46
	6	00.00	00.00	00.00	112.09	117.56	120.02
	7	00.00	00.00	00.00	131.44	142.91	142.91
	8	00.00	00.00	00.00	163.09	173.81	173.81
	9	00.00	00.00	00.00	204.68	198.22	198.22
	10	00.00	00.00	00.00	228.88	233.13	233.13
25-35	1	00.00	00.00	00.00	00.00	00.00	00.00
	2	00.00	00.00	00.00	00.00	00.00	00.00
	3	00.00	00.00	56.76	60.58	00.00	00.00
	4	00.00	00.00	77.73	81.67	84.05	86.65
	5	00.00	00.00	95.66	102.34	106.40	107.89
	6	00.00	00.00	00.00	133.88	134.68	142.40
	7	00.00	00.00	00.00	158.68	175.41	175.41
	8	00.00	00.00	00.00	188.69	202.68	202.68
	9	00.00	00.00	00.00	226.06	240.48	240.48
	10	00.00	00.00	00.00	265.34	294.71	294.71
35-50	1	00.00	00.00	00.00	00.00	00.00	00.00
	2	00.00	00.00	00.00	00.00	00.00	00.00
	3	00.00	00.00	68.70	72.91	00.00	00.00
	4	00.00	00.00	86.72	90.74	94.89	101.02
	5	00.00	00.00	122.83	127.16	128.17	129.43
	6	00.00	00.00	00.00	159.96	165.68	167.56
	7	00.00	00.00	00.00	199.22	214.83	214.83
	8	00.00	00.00	00.00	239.81	247.91	247.91
	9	00.00	00.00	00.00	279.23	309.59	309.59
	10	00.00	00.00	00.00	346.25	354.62	354.62

TABLE 5-2

CONSTRUCTION COST ESTIMATES

INTERSTATE - 85

Concrete	57.70	$/cy
Steel	0.15	$/lb
Fill	0.47	$/cy
Structural Excavation	2.66	$/cy
Roadway, Guardrail to Guardrail (108 foot roadway)	57.72	$/lf
Amortization Period	100	years
Interest Rate	6.5	per cent/year

THE GLADE

Concrete	125.00	$/cy
Steel	0.18	$/lb
Fill	1.00	$/cy
Structural Excavation	8.00	$/cy
Roadway, Guardrail to Guardrail (54 foot roadway)	17.00	$/lf
Amortization Period	100	years
Interest Rate	6.5	per cent/year

is explained by the requirement for a wider roadway (108 feet as compared to 54 feet) with a higher strength pavement design.

The total economic response is computed on an annual cost basis. To obtain an annual cost for the construction, it is necessary to get the total initial construction cost and spread it over the life of the structure (amortization period). The total initial cost is obtained by summing the excavation, fill, roadway, and culvert costs. In order to determine the annual cost of the project, the initial cost is multiplied by the capital recovery factor for the amortization period at the appropriate interest rate. Given the amortization period and the interest rate, the capital recovery factor is:

$$CRF = \frac{1}{\frac{(1+i)^n - 1}{i}} + i \tag{5-1}$$

where

i = annual interest rate

n = number of periods (amortization period in years)

For both case studies, the interest rate and amortization period are 6.5 per cent and 100 years, respectively. Substituting these values into Equation 5-1 yields a capital recovery factor equal to .0651 for the case studies.

Table 5-3 shows typical values for the structural costs for the case studies. As expected, all values of cost are higher at The Glade than at

TABLE 5-3

STRUCTURAL COSTS FOR CASE STUDIES

ITEM	SITE: INTERSTATE 85	SITE: THE GLADE
Roadway	$34,600	$ 10,800
Embankment	15,400	118,600
Culvert and Structural Excavation	15,100	51,100
Total	$65,100	$180,500

Interstate 85, except the roadway cost. The major portion of the large difference in total cost is explained by the high fill cost at The Glade. The maximum fill height at The Glade is over three times that at Interstate 85. This, coupled with the higher unit cost ($1.00 as compared to $0.47 per cubic yard) causes the higher fill costs at The Glade. The fill cost at The Glade accounts for approximately two-thirds of the total cost, making it the dominant portion of the total cost. At the Interstate 85 location, the roadway cost dominates, accounting for 57 per cent of the total.

CHAPTER VI HYDROLOGY

The hydrologic data represents one of the major sets of information that is needed in the culvert design model. It is necessary to carefully analyze the hydrologic information available to generate the runoff hydrographs. The runoff hydrographs are assumed to be triangular in shape. To describe an unique triangular hydrograph, three basic parameters must be defined. They are the time to peak (T_p), flood duration (T_b), and the peak flow (Q_p). A general representation of the assumed runoff hydrograph is shown in Figure 6-1.

T_p is estimated using the drainage basin characteristics incorporated into a design method suggested by the Bureau of Reclamation. It is assumed that T_p can be approximated by the time of concentration (T_c). T_c is defined as the travel time of the runoff from the hydraulically most distant point to the point of interest, in this case the culvert site. Two approaches are presented by the Bureau of Reclamation for estimating the value of T_c.

The first method is based on

$$T_c = L/V, \tag{6-1}$$

where, T_c = time of concentration,
L = length of longest watercourse, and
V = average stream velocity.

Tables published by the Bureau of Reclamation relate average stream slope and ground cover to average stream velocity.

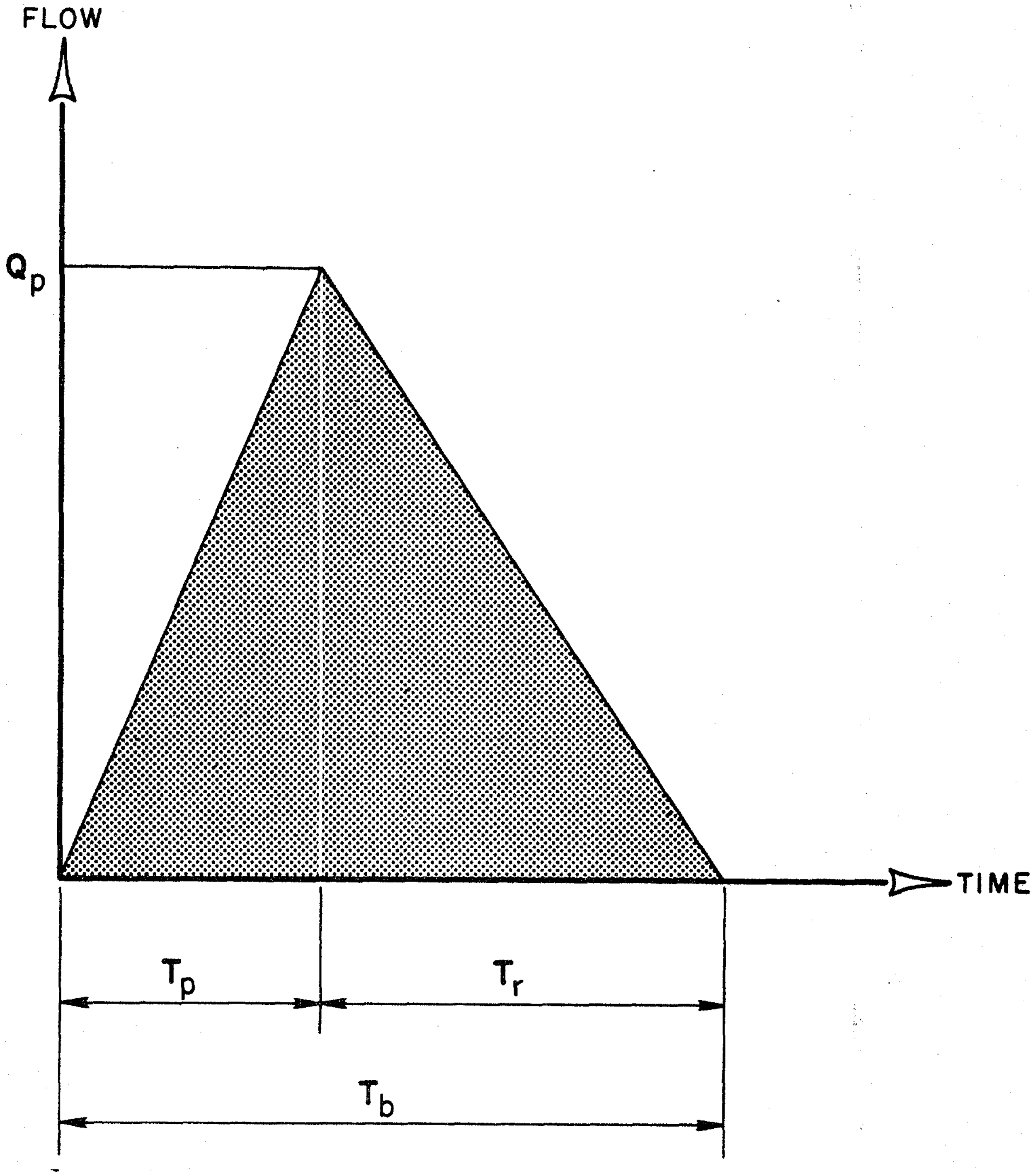


FIGURE 6-1
ASSUMED FLOOD HYDROGRAPH CHARACTERISTICS

The second method depends on the use of an empirical equation suggested by the Soil Conservation Service as a guide for determining T_c. The empirical equation is,

$$T_c = \left(\frac{11.9L^3}{H}\right)^{0.385} \tag{6-2}$$

where, T_c = time of concentration

L = length of longest watercourse in miles, and

H = elevation difference in feet.

To develop a value of T_c for the case studies, an average value of the two methods is computed.

The Bureau of Reclamation guidelines are also followed in order to determine a value for T_r, the length of time between the peak flow and the end of the hydrograph. For a watershed the relationship between T_r and T_p is approximately,

$$T_r/T_p = \text{Constant.} \tag{6-3}$$

The value of the constant for a particular stream may be computed from recorded hydrographs. Analyses by the Soil Conservation Service show 1.67 as a general average value for the constant for ungaged watersheds. This constant when substituted into Equation 6-3 yields the predictive equation:

$$T_r = 1.67\ T_p \tag{6-4}$$

The values of T_p and T_r for the two case studies, determined by the preceding methods, are shown in the following table.

SITE	T_p	T_r
Interstate 85	2.50	4.00
The Glade	2.00	3.34

The final parameter required to describe the triangular shape of the hydrographs is the peak flow. The range of peak flows associated with different recurrence intervals are determined by fitting runoff data on the sites to a Gumbel Plot. A Gumbel Plot is a linearized graph of relative flood peak magnitude versus average recurrence interval (years). The axes are divided in a manner such that the storm frequency distributions plot as approximately a straight line. Figures 6-2 and 6-3 show Gumbel Plots for the two case studies. The plots of the provided runoff data from the two sites are taken as straight lines. The peak flow of a storm of any recurrence interval for either site can be determined from the plots.

In the culvert analysis, it is necessary to consider a wide range of hydrologic possibilities. Any flood has a probability of occurrence in a single year. A large number of floods, each having its associated probability, is analyzed to assess the risks. The probability of a storm, p, occurring in any one year is the reciprocal of its recurrence interval, R,

$$p = 1/R \tag{6-5}$$

For example, a storm with a recurrence interval of ten years (e.g., so-called ten-year storm) has a ten per cent chance of occurring every year, or a yearly probability of .10. Using this type of calculation, it is possible to construct Tables 6-1 and 6-2. These tables show peak flood magnitudes and the yearly cumulative probabilities associated with them.

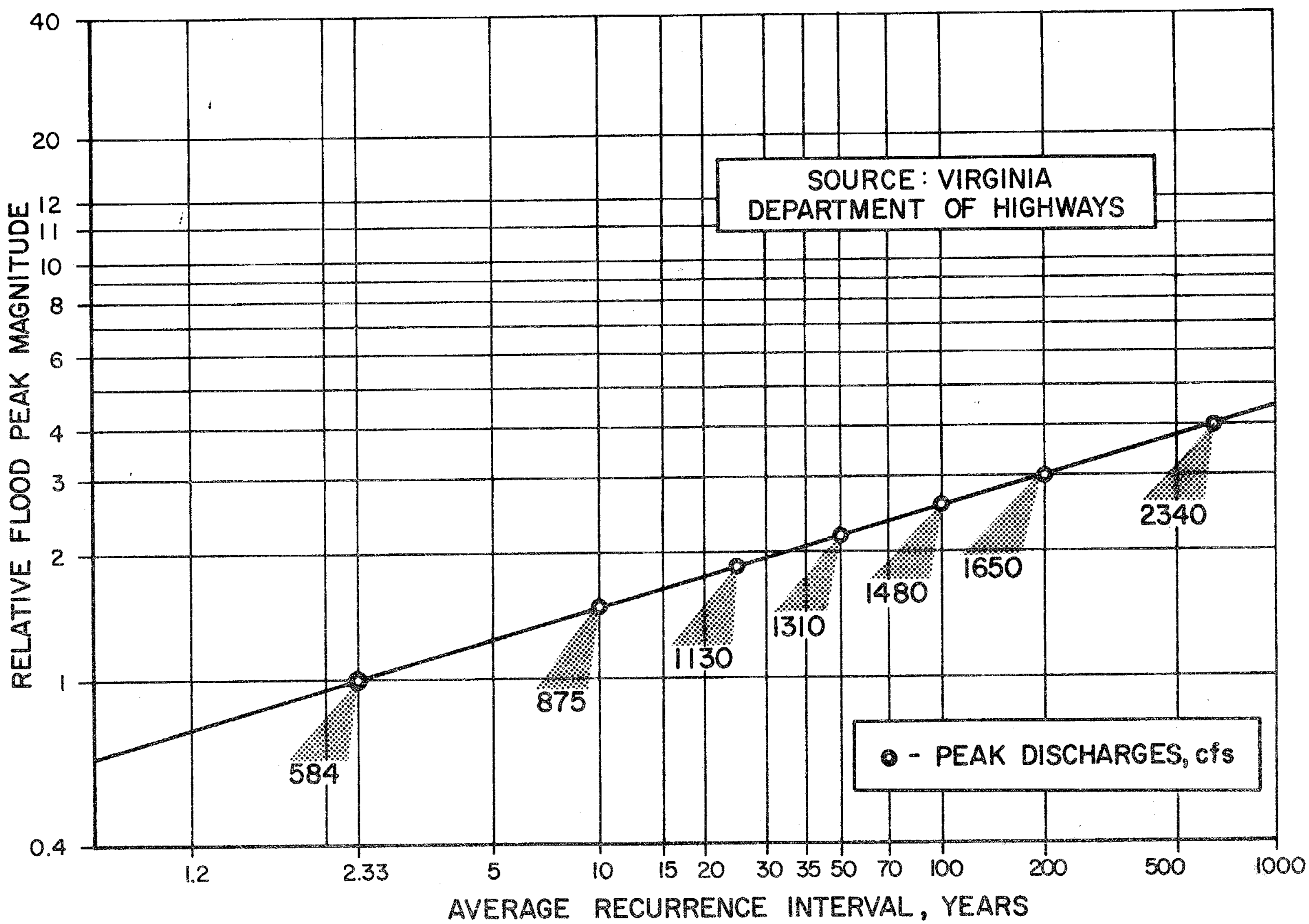


FIGURE 6-2

ESTIMATED FLOOD PEAK RECURRENCE AT I-85

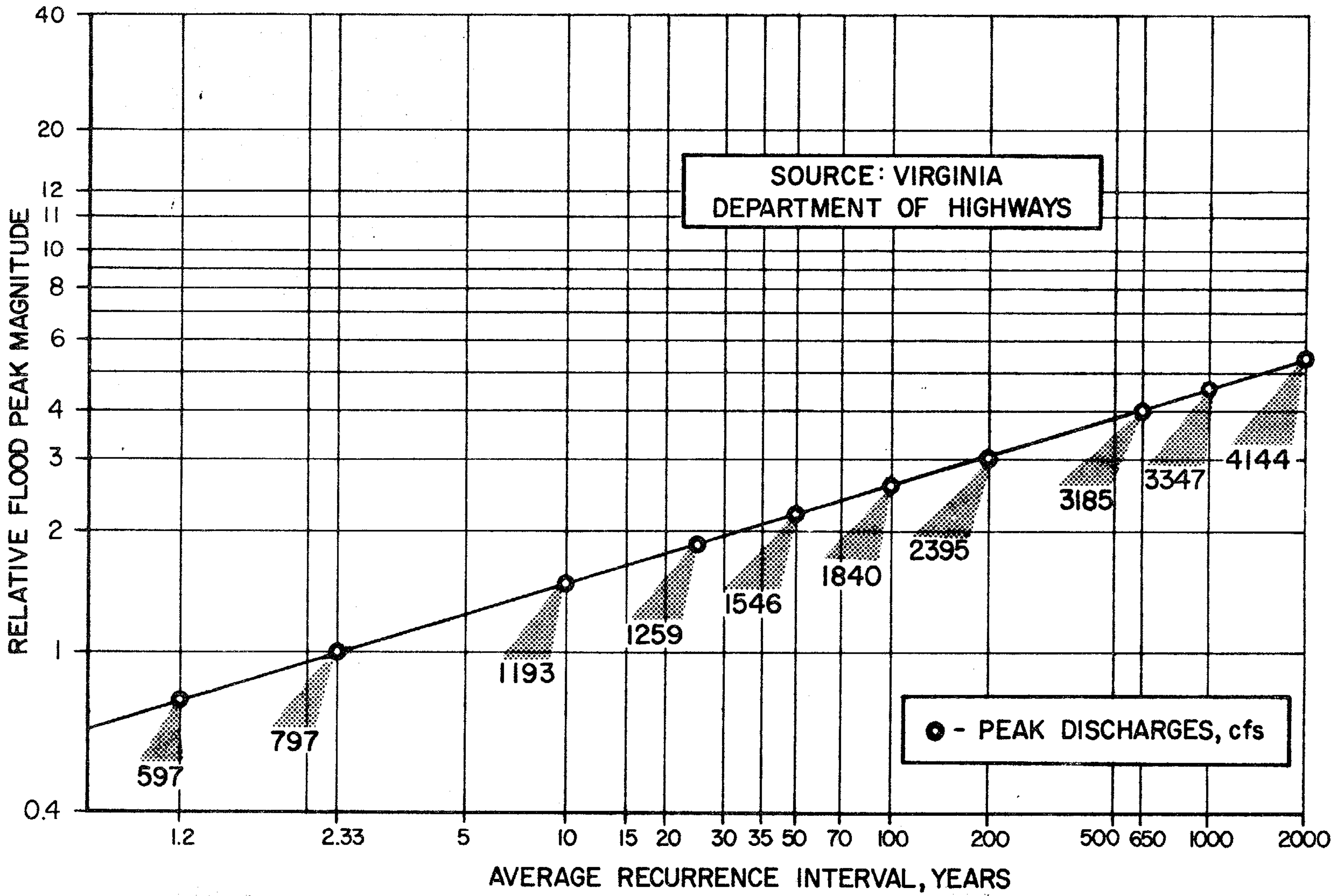


FIGURE 6-3

ESTIMATED FLOOD PEAK RECURRENCE AT THE GLADE

TABLE 6-1

INTERSTATE 85

FLOOD PROBABILITY ANALYSIS

1 Q cfs Class Interval	2 Return Period yrs	3 Class Midpt. cfs	4 Σp (1÷by Col. 2)	5 Probability of Col. 3 Values
293	1.11		0.9100	
		439		0.4800
584	2.33		0.4300	
		730		0.3300
875	10		0.1000	
		1003		0.0600
1130	25		0.0400	
		1220		0.0200
1310	50		0.0200	
		1395		0.0100
1480	100		0.0100	
		1565		0.0050
1650	200		0.0050	
		1995		0.0035
2340	650		0.0015	
				Σ = 0.9085

TABLE 6-2

THE GLADE

FLOOD PROBABILITY ANALYSIS

1	2	3	4	5
Q cfs Class Interval	Return Period yrs	Class Midpt. cfs	Σp (1÷by Col.2)	Probability of Col. 3 Values
597	1.2		0.8350	
		697		0.4050
797	2.33		0.4300	
		995		0.3300
1193	10		0.1000	
		1226		0.0600
1259	25		0.0400	
		1403		0.0200
1546	50		0.0200	
		1693		0.0100
1840	100		0.0100	
		2118		0.0050
2395	200		0.0050	
		2790		0.0035
3185	650		0.00155	
		3266		0.00055
3347	1000		0.00100	
		3746		0.00050
4144	2000		0.00050	
				Σ = 0.83455

For the purpose of the case studies, the peak flows are divided into classes and the midpoint of the class is taken as the representative flood flow for the class. The probability of a representative flow occurring in its class is the difference in the cumulative probabilities of the class boundary values.

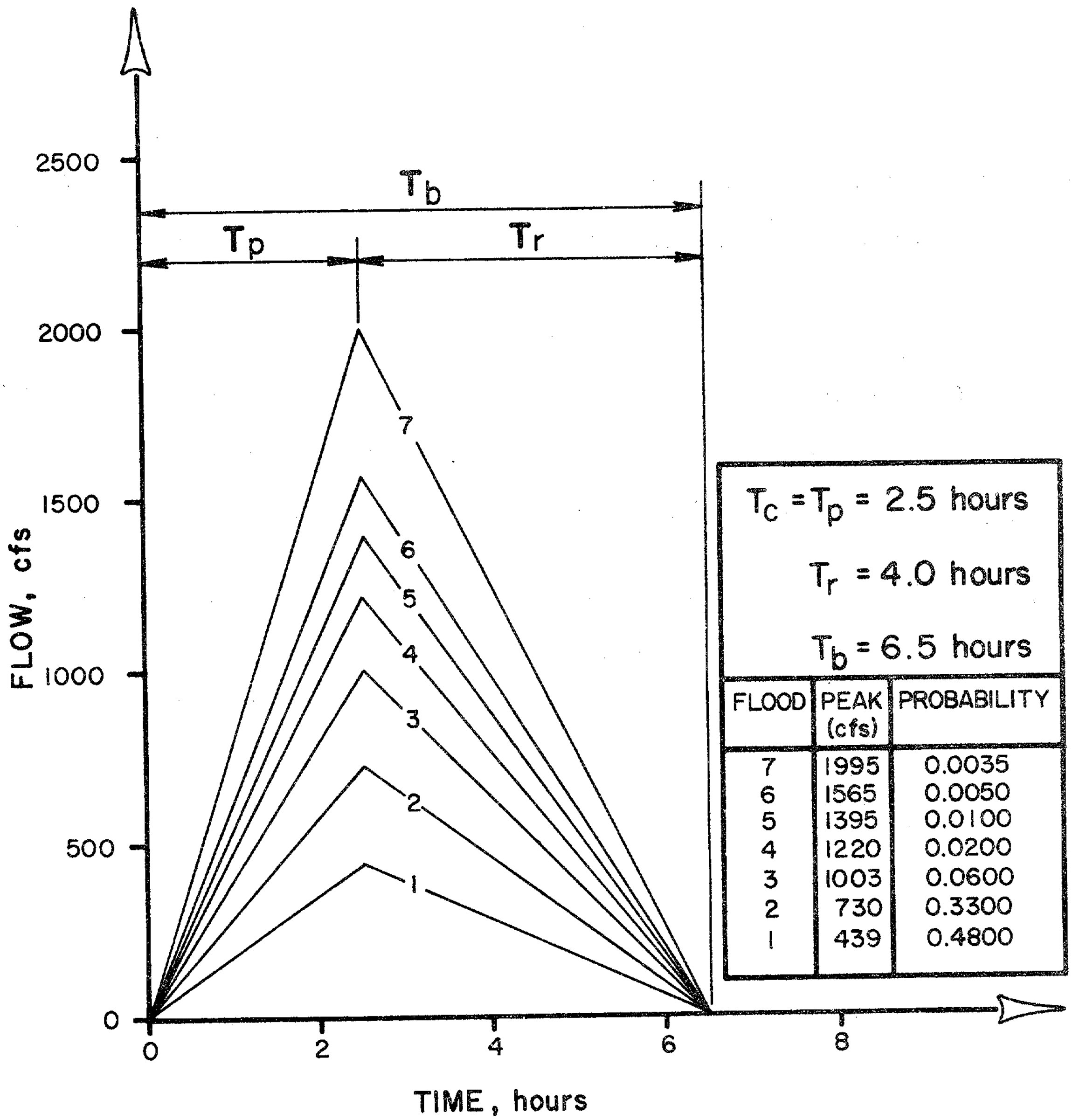
Table 6-1 presents data on flood magnitude and frequency for the crossing site at Interstate 85. The range of peak flows considered vary from a recurrence interval of 1.1 years to 650 years. Columns (1) and (2) are determined from runoff data and use of Figure 6-2. The data in the remaining columns are determined from Columns (1) and (2). Column (3) tabulates the midpoint of successive flood peaks in Column (1) and is the representative value of its class. Column (4) is the reciprocal of the return period in Column (2). Column (5) is the difference in successive cumulative probabilities shown in Column (4). The sum of the probabilities in Column (5) is 0.9135 indicating that the full flood peak spectrum, or population, is not considered. An inspection of Column (4) shows that the upper and lower end of the flood peak population are omitted. The justification for excluding extreme events from the analysis is that small floods are of insignificant economic importance and the large floods have a very small probability of occurrence which makes the product of probability times economic loss (risk) negligible.

Table 6-2 parallels Table 6-1 and presents data on flood magnitude and frequency for the second case study, The Glade. The range of peak flows considered varies from a recurrence interval of 1.2 years to

2,000 years. The data in the columns are obtained in the same manner as described in the preceding paragraph.

The hydrographs shown in Figures 6-4 and 6-5 represent the hydrologic data used in the model for the two case studies. The time to peak and flood duration are considered constant within each case study. Probabilities of the peak flows are determined directly from Tables 6-1 and 6-2.

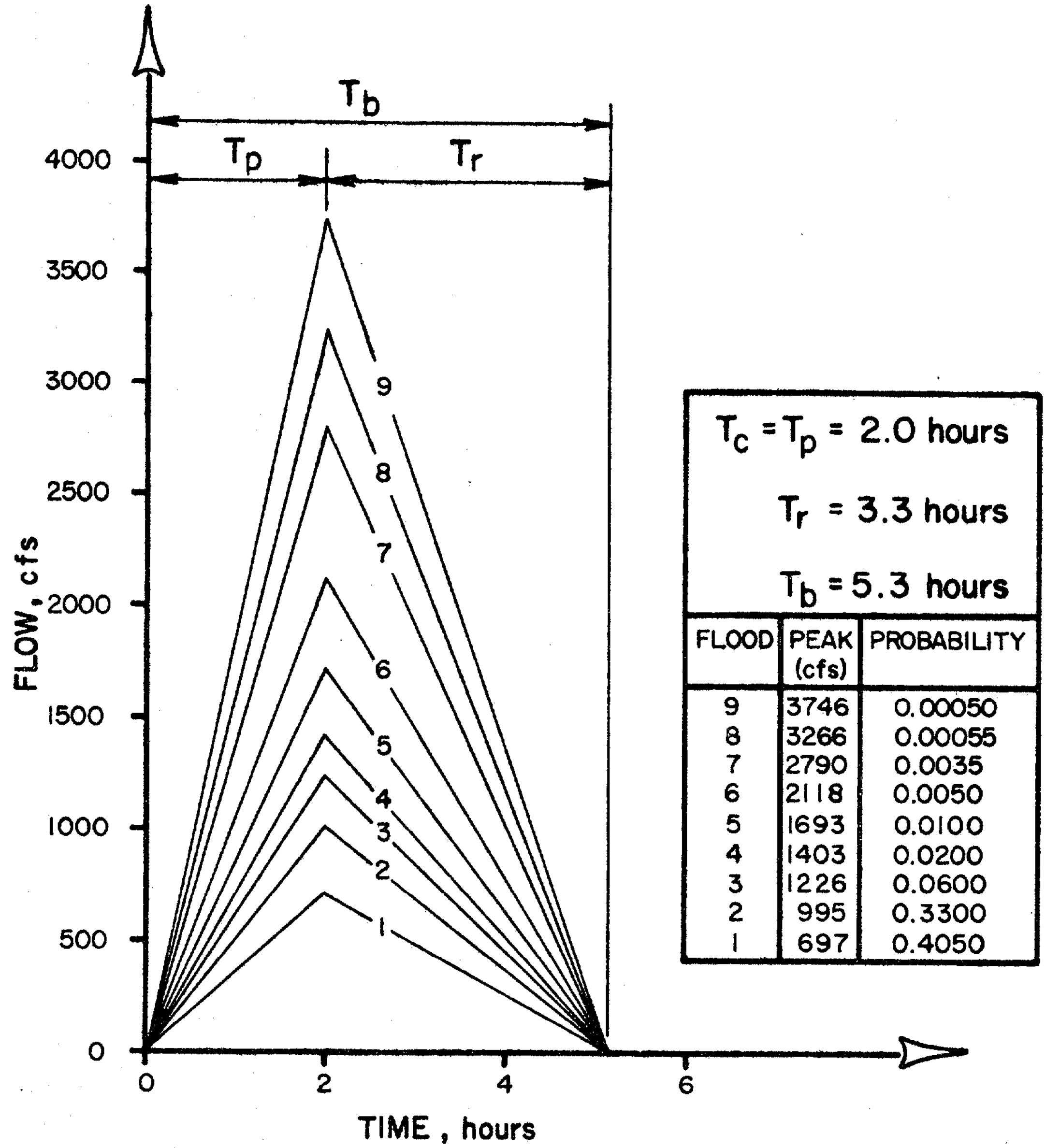
A complete summary of the hydrologic data for both case studies can be found in Appendix A, Summary of Data.



FLOOD	PEAK (cfs)	PROBABILITY
7	1995	0.0035
6	1565	0.0050
5	1395	0.0100
4	1220	0.0200
3	1003	0.0600
2	730	0.3300
1	439	0.4800

FIGURE 6-4

FLOOD HYDROGRAPHS FOR I-85



FLOOD	PEAK (cfs)	PROBABILITY
9	3746	0.00050
8	3266	0.00055
7	2790	0.0035
6	2118	0.0050
5	1693	0.0100
4	1403	0.0200
3	1226	0.0600
2	995	0.3300
1	697	0.4050

FIGURE 6-5

FLOOD HYDROGRAPHS FOR THE GLADE

CHAPTER VII HYDRAULICS

This chapter describes the technique for analyzing the dynamic hydraulic response of a box culvert under flood conditions. The hydraulic response obtained from routing a flood through the culvert site forms the physical basis for the economic analysis. Economic losses involving the surrounding area, the culvert site, and the highway travelers are determined by using the information obtained from the flood response. The purpose of the flood routing calculation is to determine the basis for estimating the economic losses.

Topics in this chapter include the description of the flood routing methodology, the culvert discharge-headwater relationships, including inlet and outlet control, the method for computing overtopping, and the results of the case studies. The hydrologic data that are needed in the culvert hydraulics analysis are introduced in Chapter VI. A detailed explanation of the flood-related losses is contained in Chapter VIII.

Dynamic Solution Process

Figure 7-1 outlines the procedure followed in the flood routing analysis. The technique routes a set of flood hydrographs through a culvert site using the following procedure (keyed to Figure 7-1):

1. Start with the first flood to be routed.
2. Initialize all variables.
3. Compute the average inflow from the flood hydrograph.
4. Solve for discharge and the headwater depth using inlet control criteria.

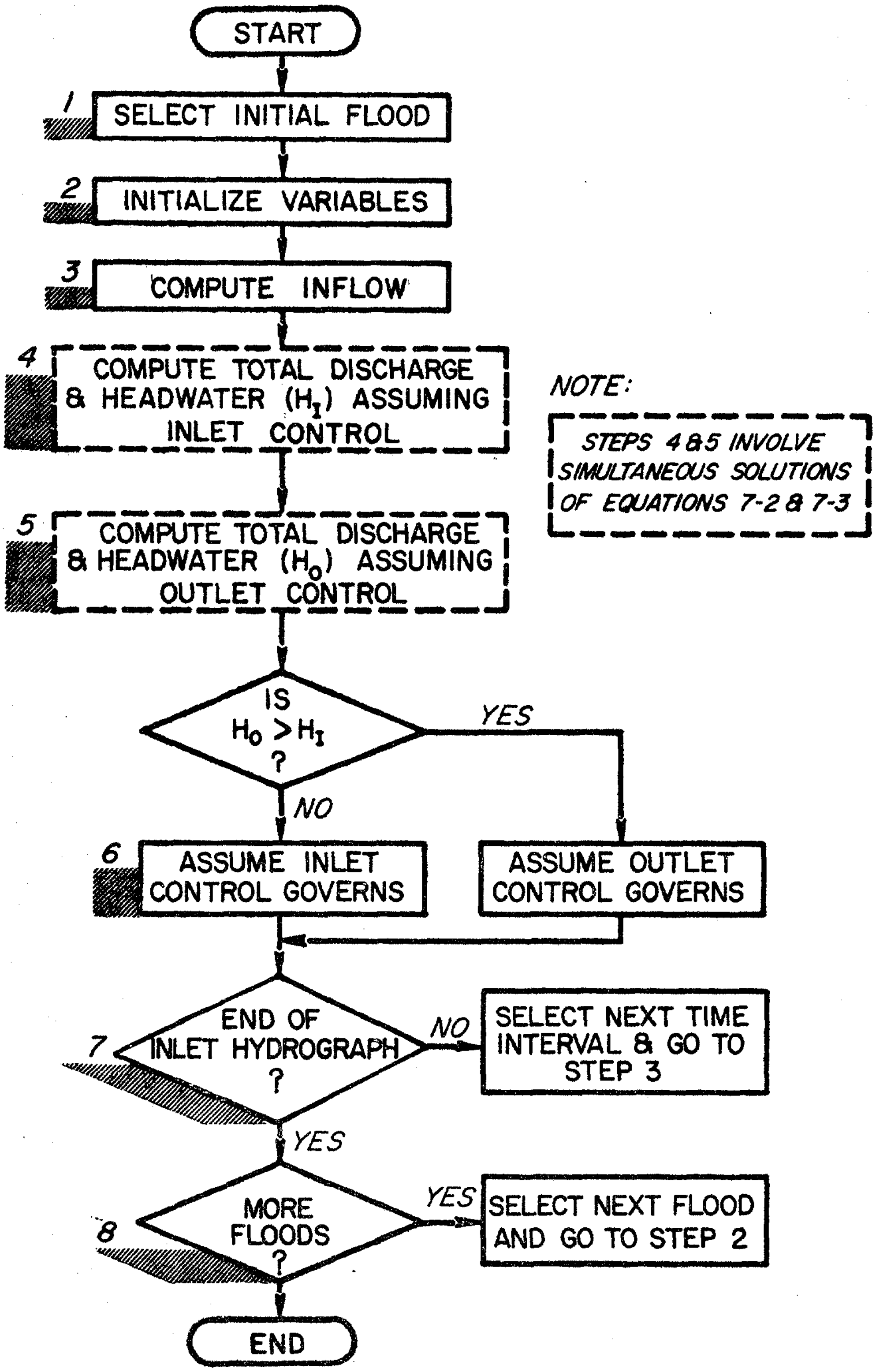


FIGURE 7-1

FLOOD ROUTING LOGIC

5. Solve for discharge and the headwater depth using outlet control criteria.

6. Select type of control with the greater headwater depth. (This is the criteria used by the Virginia Department of Highways on recommendation from the Bureau of Public Roads.)

7. Increment the time interval and repeat Steps 3 to 7 until the entire inflow hydrograph is routed.

8. Select the next flood and repeat Steps 2 to 8 until all flood hydrographs are routed through the site.

For each inflow hydrograph, the flood routing procedure yields a time description of the upstream ponding, amount and duration of overtopping, and the downstream flow conditions; this information is used to estimate the associated losses. The procedure of routing a flood hydrograph through a culvert is similar to directly routing a flood through a reservoir. Figure 7-2 illustrates the features of a culvert stream crossing. The highway embankment acts as a dam which creates an upstream pond whenever the inflow (I) to the pond exceeds the culvert discharge capacity. Downstream from the culvert, the tailwater depth (TW) is estimated by the stage-discharge characteristics of the natural streambed using Manning's formula and a normal depth assumption. If the headwater depth (HW) during periods of flooding exceeds the minimum roadway elevation, overtopping of the highway embankment occurs.

Flood routing is based on the mass balance equation which establishes an equality between the initial storage and average inflow and the final storage and average outflow for a finite time interval. The equation is usually presented in the form:

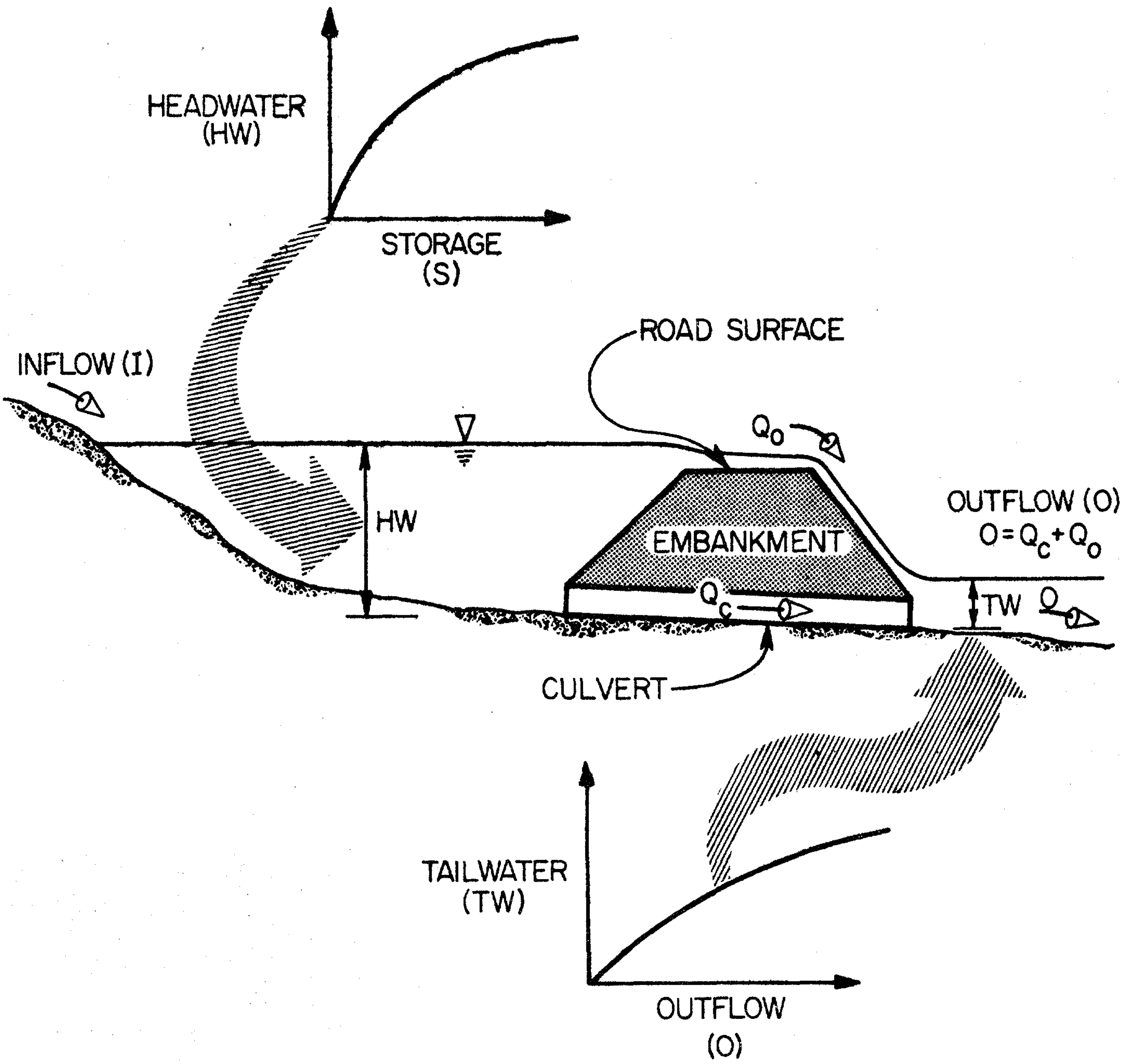


FIGURE 7-2

BASIC SITE CHARACTERISTICS FOR FLOOD ROUTING CALCULATIONS

$$S_1 + 0.5\ (I_1 + I_2)\ \Delta t + S_2 + 0\Delta t \qquad (7\text{-}1)$$

Where, i = 1 = beginning of time interval

i = 2 = ending of time interval

S_i = storage (L^3)

I_i = inflow (L^3/T)

0 = average outflow (L^3/T)

Δt = time interval (T)

Solving Equation 7-1 for S_2, in terms of 0, yields,

$$S_2 = S_1 + 0.5\ (I_1 + I_2) - 0\Delta t \qquad (7\text{-}2)$$

At a particular point in the routing, S_2 and 0 are unknown. Values of S_2 and 0 that satisfy Equation 7-2 derive from the relationships between headwater depth, culvert discharge, and highway overtopping. Outflow is governed in part by headwater. Headwater depth is determined by the amount of water in storage. The outflow hydraulics are formulated for inlet control, outlet control, and broad-crested weir overflow. The relationship,

$$0 = Q_{culvert} + Q_{overtopping} = f\ (\text{Average Storage}) \qquad (7\text{-}3)$$

is used in conjunction with Equation 7-2 to route the floods. For a given headwater depth and culvert size, culvert discharge is estimated, together with the overtopping flow, to yield Equation 7-3. This equation is solved simultaneously with Equation 7-2 to yield a final storage at the end of a finite time interval.

Solving these equations for a succession of time intervals yields a time history of the culvert flow overtopping, flow, and storage duration; this response is saved and used to estimate losses.

The method of simultaneous solution of Equations 7-2 and 7-3 is shown in Figure 7-3. At a particular coordinate on the inflow hydrograph, in the course of solving for storage and outflow, the following items are given:

I_1 = inflow hydrograph coordinate,

I_2 = inflow at the point t units to the right of I_1,

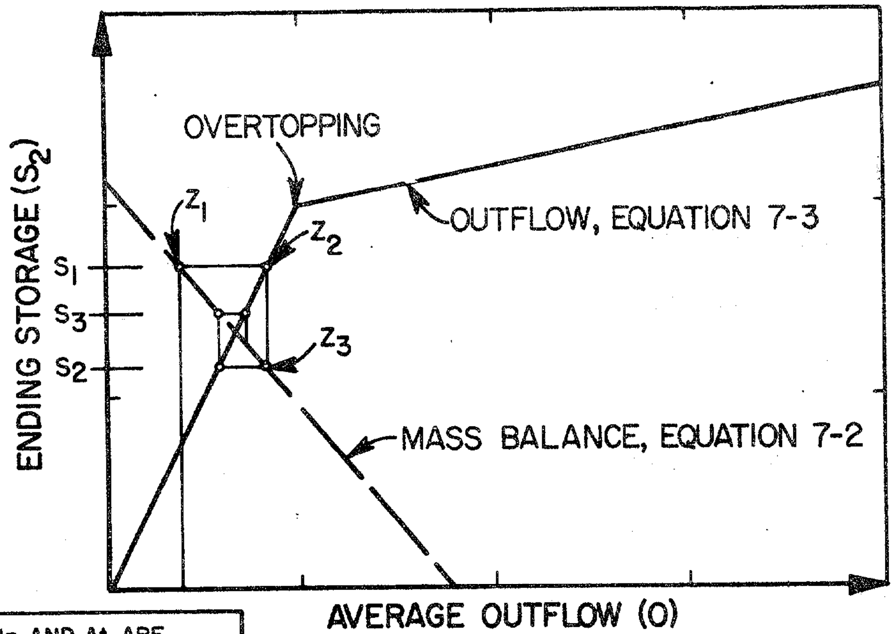
Δt = time interval, and

S_1 = volume in storage at the start of the time interval.

These quantities determine the plotting positions of Equation 7-3; it is desired to find the intersection of these two curves which define the value for average outflow (O) and ending storage (S_2) that correspond to the time interval (Δt).

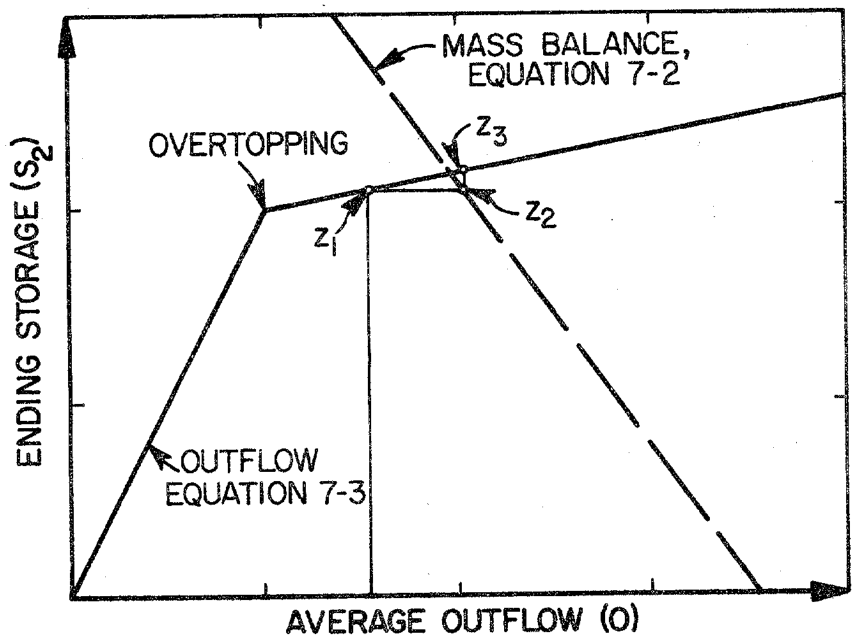
Consider the mass balance curve (the dotted line in Figure 7-3). When S_2 is plotted as a function of O, the resultant line has an intercept of $S_1 + 1/2\ (I_1+I_2)\Delta t$ and a slope of $-\Delta t$; thus the intercept varies with I_1, I_2, Δt, and S_1 and the slope varies with Δt.

On the other hand, the outflow curve (the solid line in Figure 7-3) consists of two segments which intersect at a break point. The break point corresponds to the point where overtopping of the road commences. To the left of the break point all the discharge is through the culvert; to the right the discharge is the sum of culvert flow plus broad-crested



NOTE: I_1, I_2, I_3, AND Δt ARE GIVEN AND DETERMINE THE LOCUS OF POINTS DESCRIBING EQUATIONS 7-2 AND 7-3. SUCCESIVE SOLUTIONS ARE z_1, z_2, z_3, ETC.

A. Without Overtopping



B. With Overtopping

FIGURE 7-3

MASS BALANCE AND OUTFLOW EQUATIONS FOR FLOOD ROUTING CALCULATIONS weir flow. As with the mass balance curve, the outflow curve position on the graph of S_2 versus O depends upon I_1, I_2, Δt and S_1; in addition, the geometry of the culvert influences the outflow curve.

The simultaneous solution of Equations 7-2 and 7-3 is accomplished by assuming a discharge, computing a value for S_2 using one of the equations, substituting S_2 into the other equation and solving for a new value of discharge, and then repeating the tandem usage of the equations until successive values of discharge and S_2 are tolerably close. This succession of solutions is shown by the arrows in Figure 7-3 where the first three successive solutions are z_1, z_2, and z_3. There are two conditions (A&B) which determine the order of solution.

Condition A is defined as culvert flow only; in this case z_1 is determined using Equation 7-2 and the solution proceeds from z_1. Condition B is defined as the situation where the culvert and broad-crested weir are both flowing; in this case z_1 is determined using Equation 7-3 and the solution proceeds from z_1. This conditional order of solution is imposed on the calculations to take advantage of the relative slopes of Equation 7-2 and 7-3; the objective is to rapidly achieve the ending condition of having successive values of outflow and storage tolerably close.

Consider the ratio of absolute values of successive solutions for S_2 (see Condition B, Figure 7-3),

$$\frac{|S_3 - S_2|}{|S_2 - S_1|} = r \tag{7-4}$$

If r is greater than one, successive solutions are getting farther apart; the solution process diverges. This is confirmed by inspection of Figure 7-3 which has r values of less than one for both conditions A and B. The ordering of solutions (that is, computing z_1 using Equation 7-2 for condition A and using Equation 7-3 for condition B) is done as one means of keeping r less than one. In other words, the ordering is done to keep successive solutions on the path to convergence.

The ratio, r, also depends on the value selected for Δt. This is due to the fact that the individual solutions, s_1, s_2, and s_3, which together define r in Equation 7-4, are dependent upon Δt (recall that $-\Delta t$ is the slope of the mass balance, Equation 7-2). Therefore, judicious selection of Δt can help to keep r at values less than one. Investigation of simplified cases and computational experience indicates that by keeping

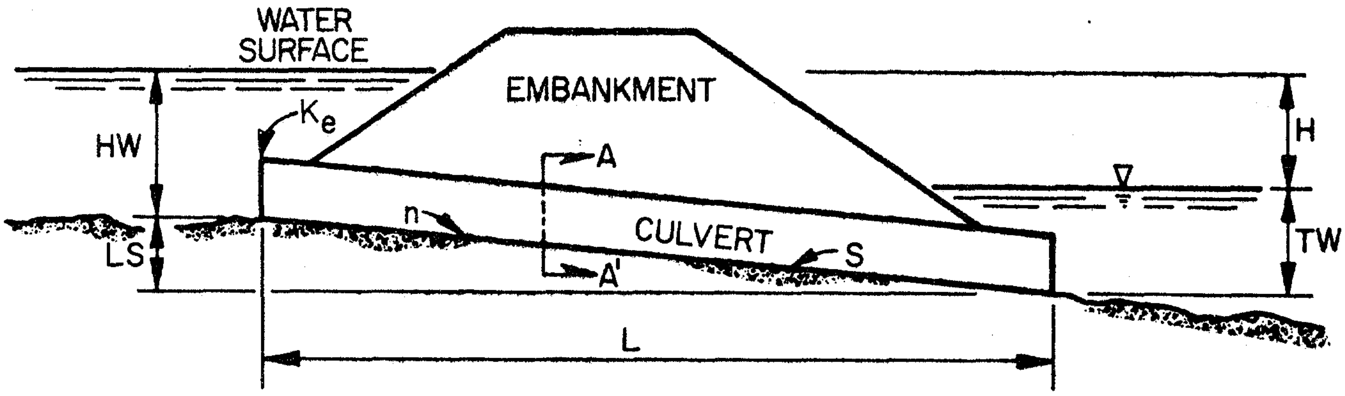
$$\Delta t \lesssim 5 \text{ minutes} \tag{7-5}$$

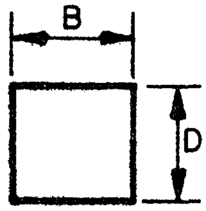
the value of r should remain less than unity. The time intervals used in the computations presented in this report varied from two to six minutes.

Discharge Relationships

General

Figure 7-4 defines the primary physical and hydraulic variables associated with a typical culvert. Headwater (HW) is measured from the culvert invert at the entrance. Tailwater (TW) is measured from the culvert invert at the exit. The head (H) across the culvert is the difference between the headwater and tailwater elevations. The height (D), width (B),





Culvert Cross Section A-A'

FIGURE 7-4

CULVERT GEOMETRY

length (L), slope (S), entrance loss coefficient (K_e), barrel roughness (n) are the physical parameters needed to describe the hydraulic condition of the culvert.

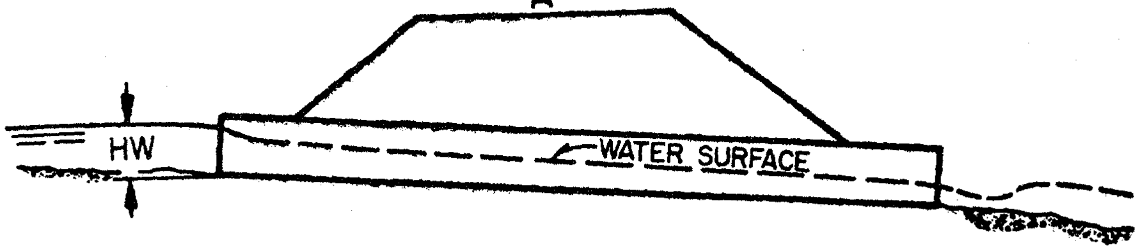
The usual factor controlling the discharge estimating procedure is whether the flow is governed by inlet or outlet hydraulic conditions. Figure 7-5 illustrates examples of inlet control flow conditions in which the discharge is governed by the entrance or inlet geometry. Figure 7-6 shows outlet control conditions in which the discharge is limited by the culvert barrel or tailwater depth. Overtopping discharge depends upon headwater depth and is predicted by a broad-crested weir formula. The following sections discuss the various flow conditions. Also the criteria, used in the dynamic flood routing, which link the various discharge conditions are presented.

Inlet Control

The culvert capacity under inlet control conditions is independent of the culvert barrel and tailwater depth. While the barrel slope affects the headwater depth, the effect is small and is ignored. The headwater depth provides the estimate of the energy at the culvert entrance; any velocity head in the upstream pond is ignored.

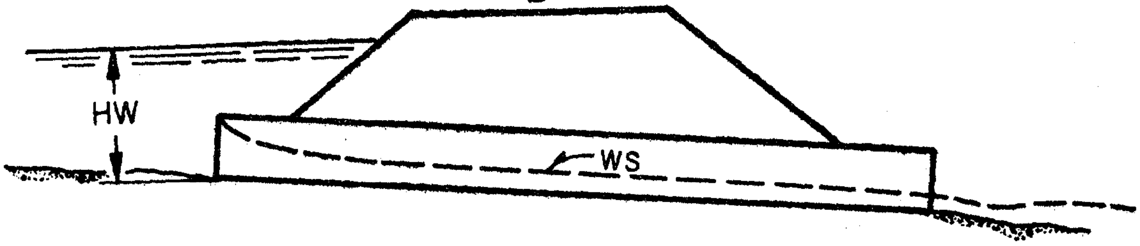
Empiric equations describing headwater-discharge relationships for box culverts under inlet control are the results of model studies conducted under the supervision of the Bureau of Public Roads. Table 7-1 summarizes the hydraulic equations for inlet control.

A



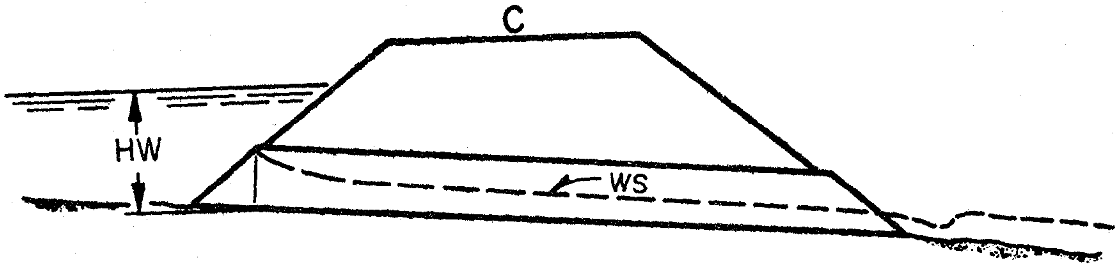
Projecting End - Unsubmerged

B



Projecting End - Submerged

C



Mitered End - Submerged

FIGURE 7-5
INLET CONTROL CONDITIONS

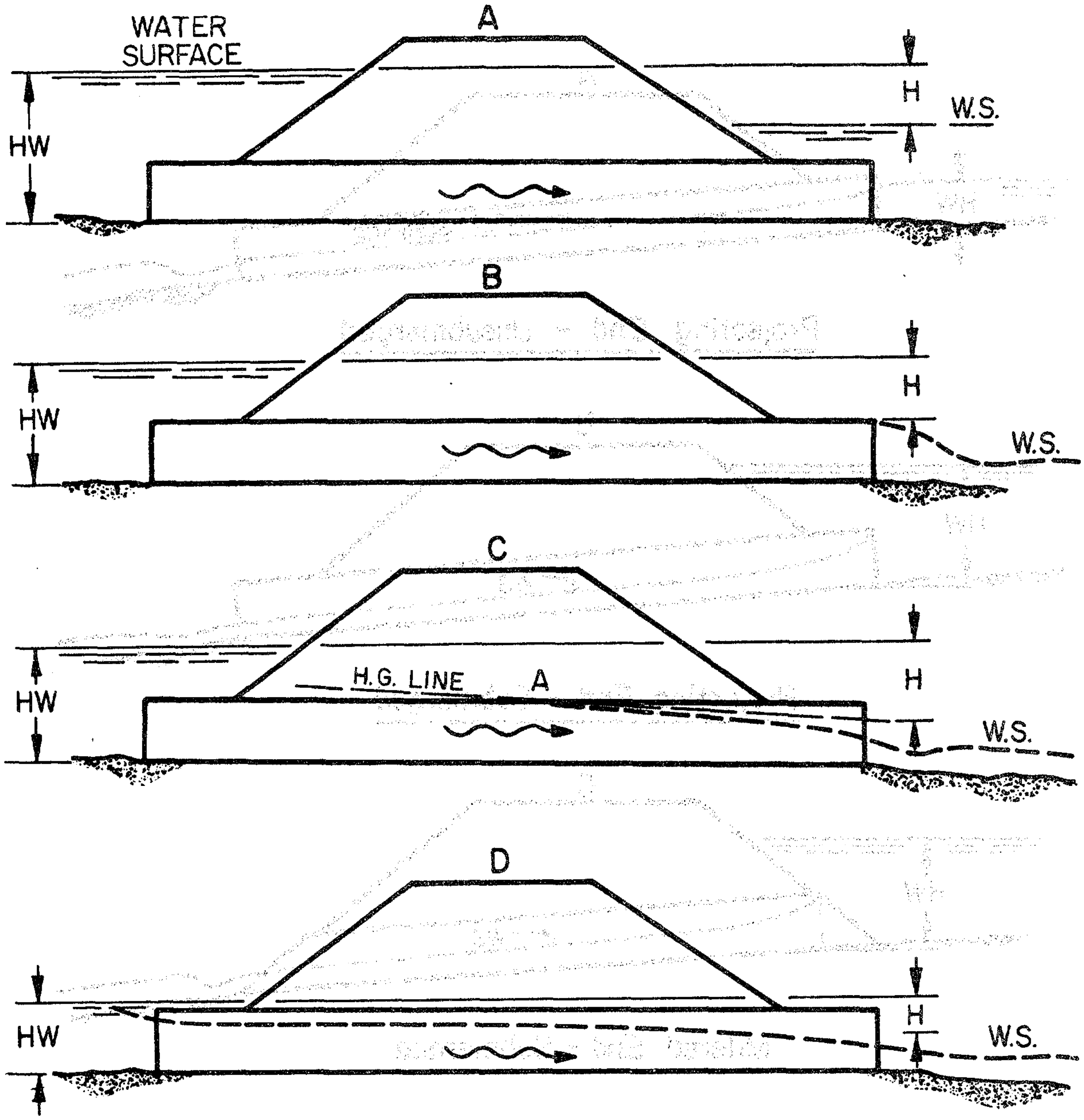


FIGURE 7-6

OUTLET CONTROL CONDITIONS

TABLE 7-1

INLET CONTROL HEADWATER-DISCHARGE RELATIONSHIPS

FOR COMMONLY USED INLET CONFIGURATIONS[1]

Description of Inlet	Type of Flow	Equation[2]	Range of Validity
1. Wingwalls, No Offset, 3/4" Chamfer on Top Edge, Flare Angle 45	Free Surface	$Q = 2.86\ BH^{3/2}$	$\frac{H}{D} \leq 1.15$
	Transition	$Q = BD^{3/2}(3.02\frac{H}{D}+0.025)$	$1.15 < \frac{H}{D} < 1.55$
	Submerged Inlet	$Q = 5.44BD^{3/2}\sqrt{\frac{H}{D}-0.793}$	$\frac{H}{D} \geq 1.55$
2. Wingwalls, No Offset, 3/4" Chamfer on Top Edge, Flare Angle 18.4	Free Surface	$Q = 2.89\ BD^{3/2}$	$\frac{H}{D} \leq 1.15$
	Transition	$Q = BD^{3/2}(2.60\frac{H}{D}+0.57)$	$1.15 < \frac{H}{D} < 1.65$
	Submerged Inlet	$Q = 5.26\ BD^{3/2}\sqrt{\frac{H}{D}-0.796}$	$\frac{H}{D} \geq 1.65$
3. Wingwalls, No Offset, 3/4" Chamfer on Top Edge, Skew Angle 15-45 Flare Angle 18.4	Free Surface	$Q = 2.87\ BD^{3/2}\ (\frac{H}{D}+0.006)^{3/2}$	$\frac{H}{D} \leq 1.15$
	Transition	$Q = BD^{3/2}\ (2.80\frac{H}{D}+0.46)$	$1.15 < \frac{H}{D} < 1.5$
	Submerged Inlet	$Q = 5.21\ BD^{3/2}\sqrt{\frac{H}{D}-0.70}$	$\frac{H}{D} \geq 1.5$
4. 90 Headwall, Beveled on All Three Edges, Bevel Angle 45	Free Surface	$Q = 2.87\ BD^{3/2}$	$\frac{H}{D} \leq 1.10$
	Transition	$Q = BD^{3/2}\ (3.48\frac{H}{D}-0.52)$	$1.10 < \frac{H}{D} < 1.5$
	Submerged Inlet	$Q = 5.64\ BD^{3/2}\sqrt{\frac{H}{D}-0.81}$	$\frac{H}{D} \geq 1.5$

[1] Barrel slope = 0.02 for all inlet types; single barrel culverts.

[2] Variable definitions: Q = culvert discharge, cfs; B = culvert width, ft; D = culvert height, ft; H = headwater, ft.

(Hydraulic & Hydrology Speciality Group, Office of R & D, BPR, November 7, 1968)

Criteria for Selecting Inlet or Outlet Control

To avoid the complex questions of hydraulic control, the analysis incorporates an accepted design procedure of selecting the discharge estimate that yields the greatest headwater depth for a given discharge. This assumption is conservative. Also, the sensitivity analysis of Chapter X shows the results of our analyses of the case studies are relatively insensitive to hydraulic control criteria. At each step of the hydrograph, routing discharge is estimated two ways: inlet control and outlet control. The control producing the greatest headwater depth is selected and used.

Overtopping Flow

During hydrograph routing the headwater depth may exceed the minimum road surface elevation which results in overtopping. Road surfaces over the stream crossings are rarely horizontal; therefore, the road surface is described using a number of horizontal segments which approximate a vertical curve. Figure 7-7 illustrates a typical stream crossing section taken along the center line of the highway. The segments capture the geometry of horizontal as well as vertical curves.

The hydraulic analysis of overtopping treats the road surface as a broad-crested weir. The roadway is assumed to be horizontal between the stations which delimit the segments. This assumption introduces error in the estimate of overtopping flow; however, the number of sections may be increased to minimize the error. Vertical sections are selected to capture the geometry of the existing grade (for embankment calculations) and to avoid large differences in the vertical elevation at the end points.

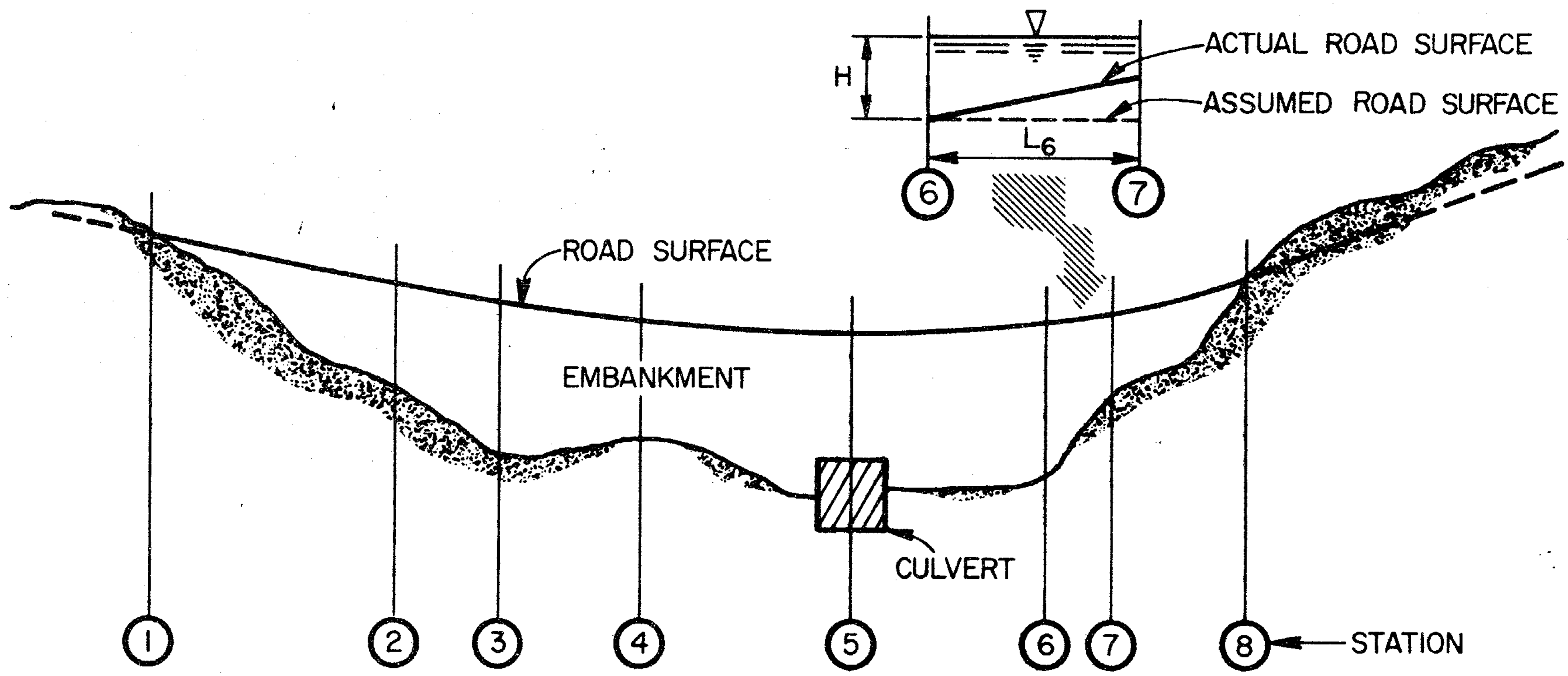


FIGURE 7-7

TYPICAL ELEVATION ALONG CENTER LINE OF HIGHWAY

Applying the equation for the discharge of a broad-crested weir, the discharge over any given roadway section becomes

$$q_i = 3.03\ l_i h^{1.5} \tag{7-9}$$

Where, l_i = length of section i in feet,

i = roadway segment, and

h = head on roadway.

The total overtopping discharge is the sum of the discharges for each roadway section,

$$Q_{\text{overtopping}} = \Sigma q_i \tag{7-10}$$

The total outflow is the sum of the culvert discharge and the overtopping discharge. Thus,

$$O = Q_{\text{culvert}} + Q_{\text{overtopping}} \tag{7-11}$$

Case Studies

The differences in natural topography at The Glade and Interstate 85 significantly affect the flood routing computations. Figure 7-8 presents the headwater-storage relationships for both case studies. At any given headwater-depth, the storage at the Interstate 85 site is considerably greater than at The Glade. For the range of headwater depths predicted by the flood routing computations, the ratio of storages (storage at Interstate 85 ÷ storage at The Glade) varies between 6 and 7. These curves were determined by planimetering topographic maps for each site.

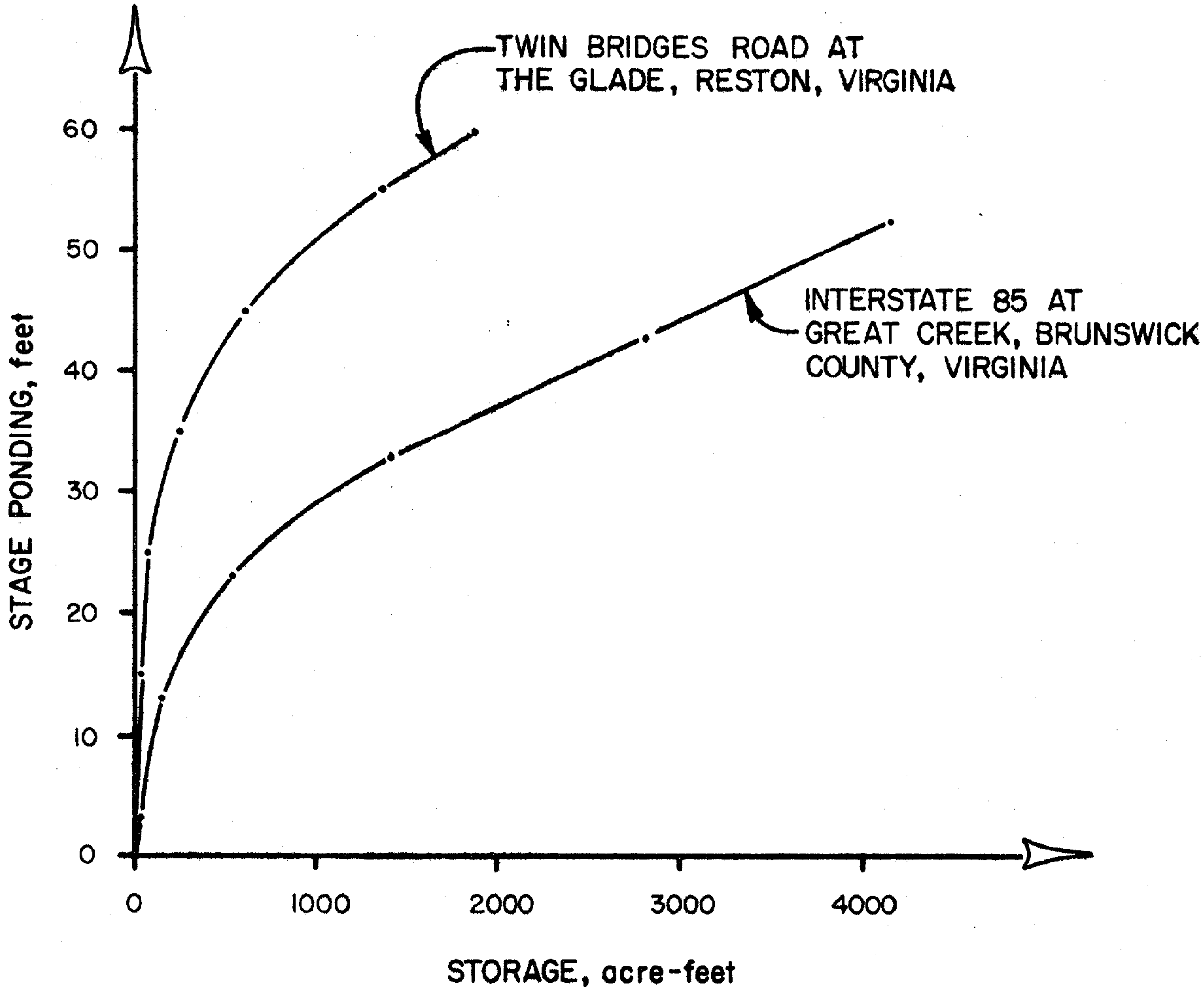


FIGURE 7-8

STAGE PONDING VERSUS STORAGE FOR CASE STUDIES

Figure 7-9 illustrates the tailwater depth versus discharge functions. The valley bottom at the Twin Bridges site is considerably narrower than the corresponding bottom at the Interstate 85 site, a fact which is reflected in tailwater discharge characteristics. These relationships are derived from Manning's Equation assuming:

1. normal depth,
2. depth equals hydraulic radius,
3. Manning's n equals 0.05 for Interstate 85, and
4. Manning's n equals 0.09 for The Glade.

The impact of these functions on the economic response is, for these sites, relatively insignificant because the culvert discharge is predominantly governed by inlet control.

Figure 7-10 illustrates typical flood routing results for the site on Interstate 85. The peak inflow for this flood in 1220 cfs and occurs 2.5 hours after the flood begins. At this site the flood duration is estimated at 6.5 hours. Upstream storage attenuates the outflow hydrograph, reducing the maximum outflow to 770 cfs at 4.0 hours for the triple 4' x 4' box culvert. The bar graph below the hydrographs indicates that outlet control governs for only .5 hours near the beginning of the flood. Inflow terminates at 6.5 hours and the upstream pond empties gradually over a two to three hour period.

Figure 7-11 presents the results of flood routing computations for the fifth of nine floods used in the analysis of The Glade site. The peak inflow of 1693 cfs occurs at two hours. Flood durations at this site are

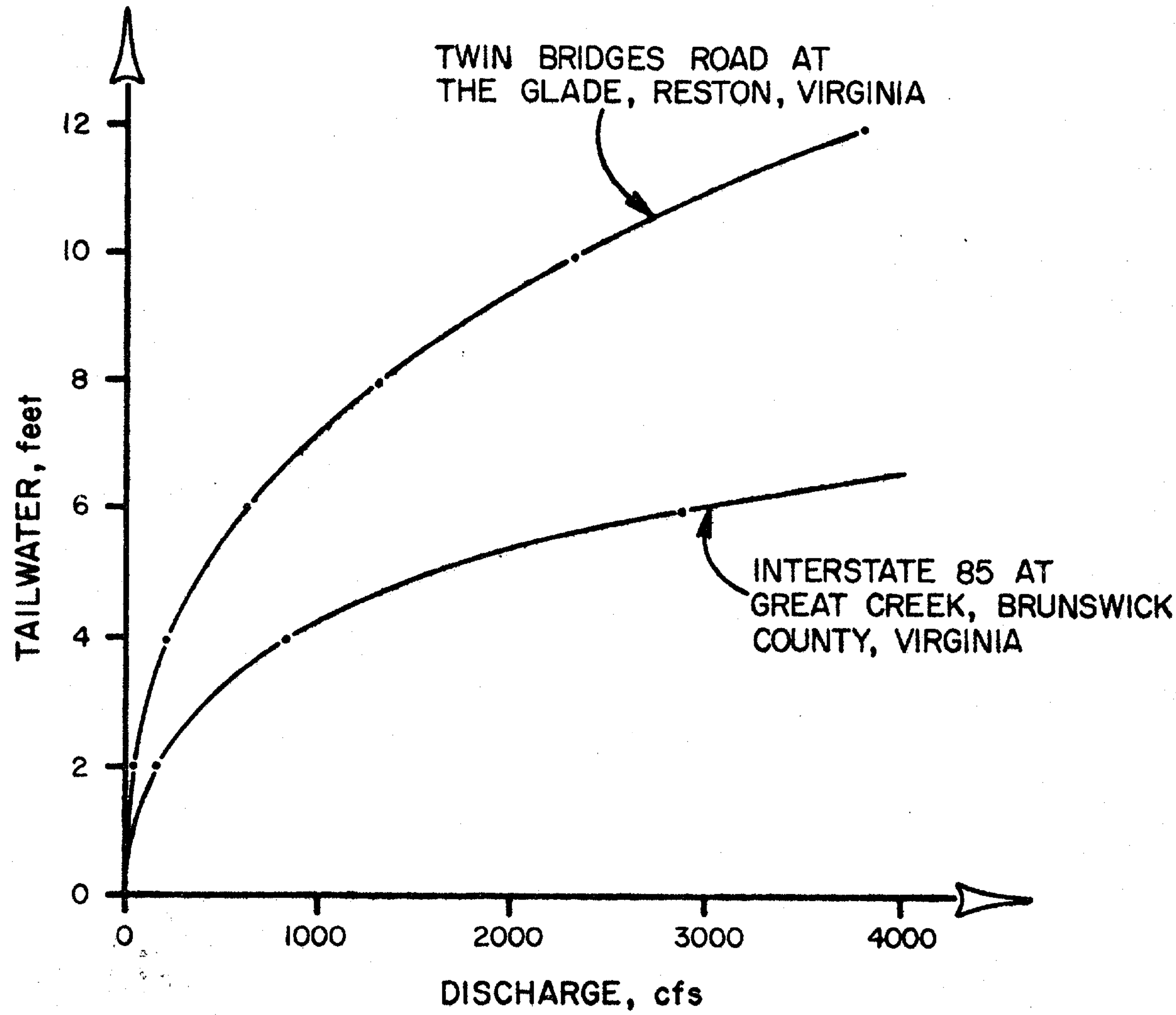


FIGURE 7-9

TAILWATER DEPTH VERSUS DISCHARGE FOR CASE STUDIES

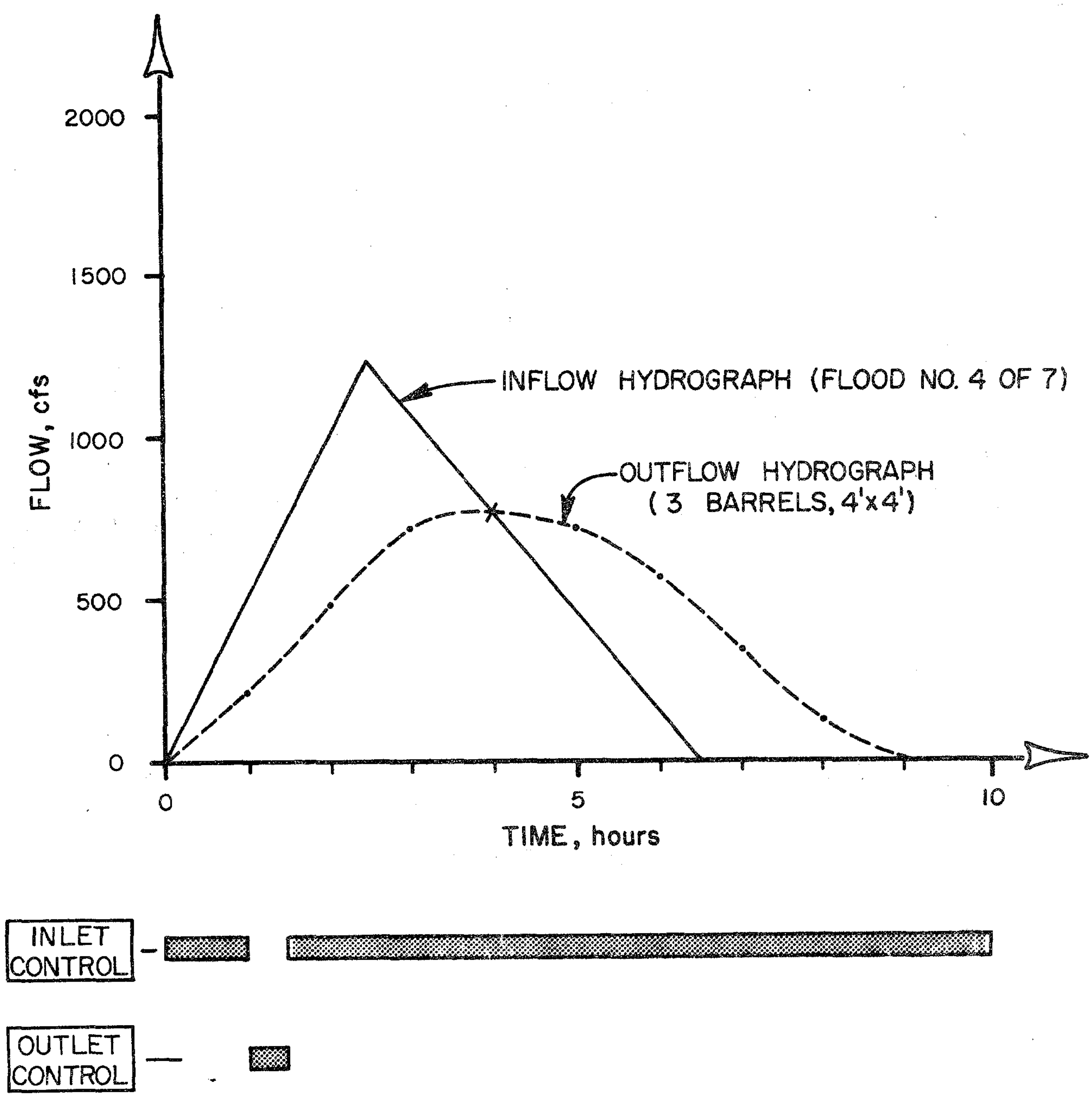


FIGURE 7-10

TYPICAL FLOOD ROUTING RESULTS - INTERSTATE 85, GREAT CREEK, BRUNSWICK COUNTY, VIRGINIA

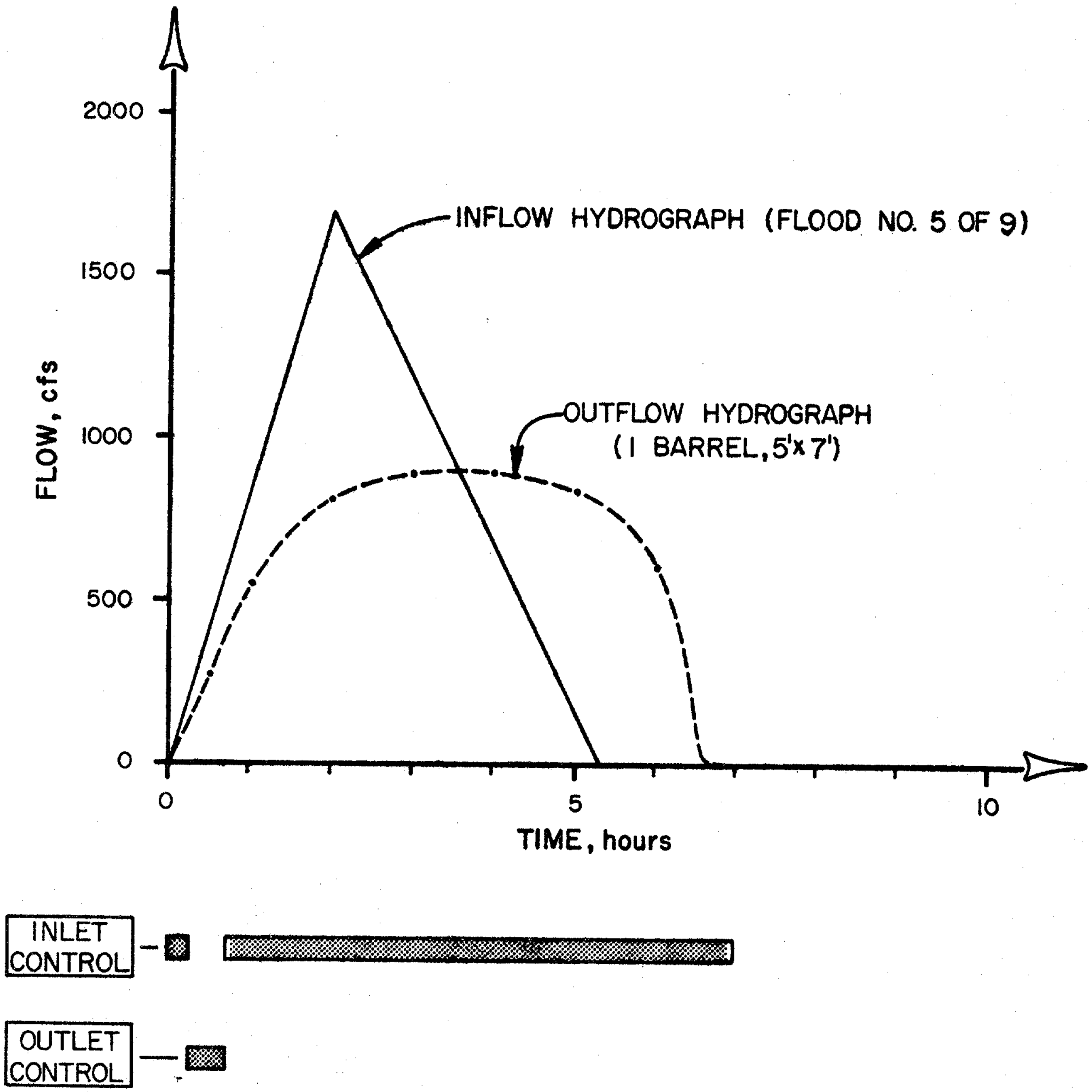


FIGURE 7-11

TYPICAL FLOOD ROUTING RESULTS - TWIN BRIDGES ROAD, AT THE GLADE, RESTON, VIRGINIA estimated at 5.3 hours. Maximum outflow for the single 5' by 7' box culvert is 900 cfs at 3.5 hours. Outlet control governs for approximately 0.5 hours near the beginning of this flood. Outflow essentially ceases after 6.5 hours. As a consequence of the headwater-storage characteristics at this site, the upstream pond empties completely in 1.3 hours after the inflow terminates, a marked difference from the Interstate 85 site.

CHAPTER VIII LOSSES

The development of the loss function is central to the analysis. The purpose of this chapter is to describe the methods that are used to obtain an economic measurement for damages from different magnitudes of runoff hydrographs. The losses associated with the failure of a culvert fall into three main categories:

1. Damage to the site

The roadway may be damaged due to overtopping and the fill may erode due to high velocity flows. In extreme cases, the complete site may "washout" causing the replacement of the complete roadway, fill volume and the box culvert.

2. Loss incurred by the traffic that uses the crossing

When overtopping occurs different degrees of traffic situations can occur. The overtopping may be of small enough magnitude so that it only has the effect of impeding the flow of traffic. In some cases, however, the degree of the overtopping may necessitate the routing of traffic onto an alternate road. This results in lost time to the occupants of the vehicles and very likely an increase in operating cost. The detour might last many days if the damage to the site is great enough to render it unsafe for passage. The addition of an unexpected obstacle in the roadway, such as a roadblock to stop traffic from using the flooded crossing, presents a hazard and increases the chance of a traffic accident occurring.

3. <u>Damage associated with the development of a large flood stage upstream from the roadway crossing.</u>

The amount of damage depends to a large degree on the type of land use upstream of the culvert site. Depending on the location of the culvert, there might be woodlands, pasture lands, farm crops, private homes, industries or any combination of these located in the flood area. Obviously, there are different levels of damage done to these land uses by flooding. For example, pasture lands may suffer little damage from water, while a private home or industry may be destroyed by a large degree of flooding.

Three hydraulic quantities are used to evaluate the loss at a given site:

1. the duration of the overtopping,
2. the amount of erosion, and
3. the maximum upstream stage.

The three quantities are associated with a particular inflow hydrograph.

The duration of the overtopping is obtained using the flood routing technique described in Chapter VII. The velocity, as a function of time, on the downstream embankment is calculated by using Manning's Equation in conjunction with the overtopping discharge; these velocities are used to calculate the volume of the fill at the site that eroded. Two approaches to compute this volume are:

1. A shear stress approach for estimating embankment failure; this method estimates an erosion rate

from shear stress, converts the erosion rate to a volume and accumulates the volume over time. The methodology pertains to cohesive soils.

2. An empirical equation found in the literature of the form:

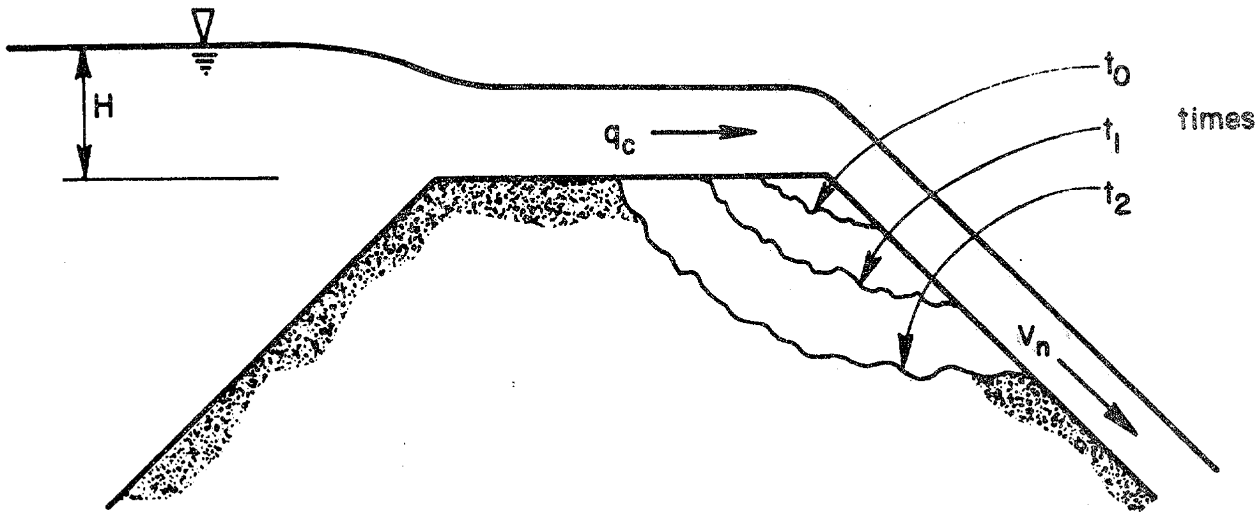
$$E = \alpha V^{\beta}$$

Where, E = Erosion (tons/ft./day),
V = Mean velocity, and
α, β = Empirical constants.

This empirical approach deals with cohesionless soils and estimates an erosion rate from the velocity of the water on the downstream embankment slope. Using the erosion rate and the appropriate time interval, a total erosion volume is computed.

Comparison of the two methods indicates that values of a $\alpha = 0.25$ and $\beta = 3.8$ provide reasonable estimates. These values represent a compromise of the two extremes of a cohesive and a cohesionless soil. Figure 8-1 shows the erosion failure mechanism assumed to occur.

As indicated on Figure 8-1, an erosion threshold velocity, V_e, is assumed. The concept is that initially at the start of overtopping, when the velocity is increasing, erosion does not occur. As the normal velocity on the downstream embankment slope increases, it reaches and passes through an erosion threshold value, V_e. Erosion commences with the attainment of V_e and continues to occur throughout the remainder of the overtopping period. Figure 8-2 gives information useful in estimating V_e.



DEFINITIONS

q_c	- (cfs/f)	- CRITICAL DISCHARGE
v_n	- (f/s)	- MANNING'S FORMULA VELOCITY
v_e	- (f/s)	- THRESHOLD EROSION VELOCITY
c	- (tons/f/day)	- SEDIMENT CONCENTRATION IN OVERFLOW

FAILURE PROGRESSION

- DISCHARGES, q_c INCREASES UNTIL THRESHOLD EROSION COMENCES, DISCHARGE CONTINUES TO INCREASE AND THEN DISCREASES WHILE EROSION PROGRESSES.
- NO EROSION, q_c RISING, $v_n < v_e$
- EROSION THRESHOLD, $v_n = v_e$
- OVERFLOW CONCENTRATION, $C = av^b$
- $t_0 < t_1 < t_2$, etc.

FIGURE 8-1

EMBANKMENT FAILURE MECHANISM

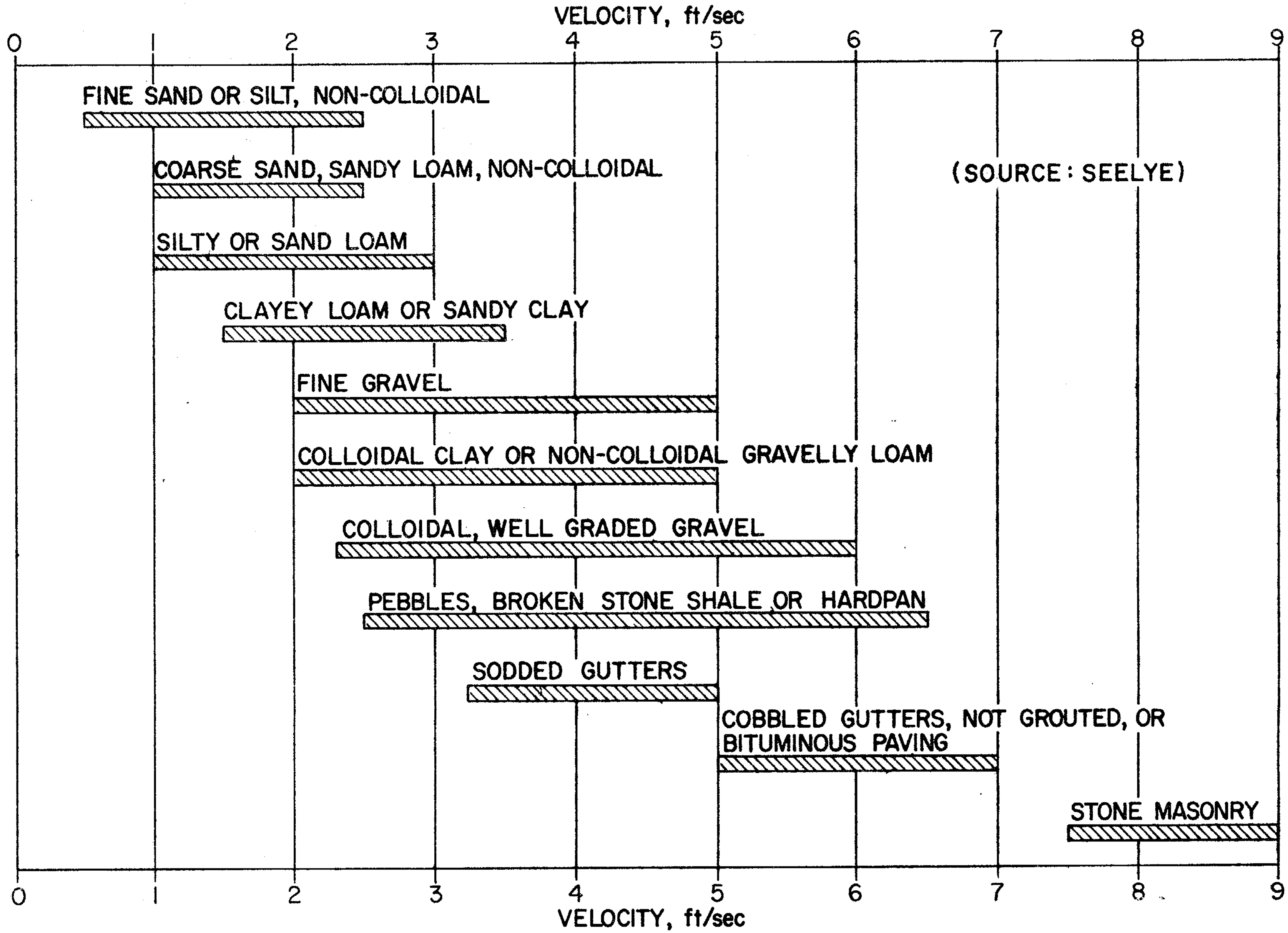


FIGURE 8-2

RANGE OF VELOCITIES FOR DIFFERENT SOILS ABOVE WHICH CHANNEL EROSION WILL OCCUR

The time duration of the overtopping and the eroded fill volume determine the three loss types: site loss, traffic loss, and stage-damage loss.

The damage to the site is divided into three sub-categories:

1. the loss of fill volume,
2. the damage to the roadway, and
3. the damage to the box culvert.

By geometrics, it is possible to develop a graph, Figure 8-3, that relates the percent erosion at the site and the height of fill at the culvert to the percent roadway failure. Given this graph for a specific fill height, it is straightforward to develop a curve, Figure 8-4, that relates percent erosion to percent of roadway failure. The percent damage to the box culvert is also represented in Figure 8-4. It is assumed that until 90 percent of the volume of fill is eroded, no damage is done to the culvert. When 100 percent erosion occurs, it is assumed that the culvert barrel is destroyed. The economic loss incurred by damage to the site (L_s) is computed as:

$$L_s = (P_2 C_r + P_3 C_c + P_1 C_f)\ C_a \qquad (8\text{-}2)$$

Where, P_1 = percent of original volume eroded

P_2 = percent of roadway lost

P_3 = percent of culvert damage

C_r = original cost of roadway

C_c = original cost of culvert

C_f = original cost of fill volume

C_a = cost adjustment factor

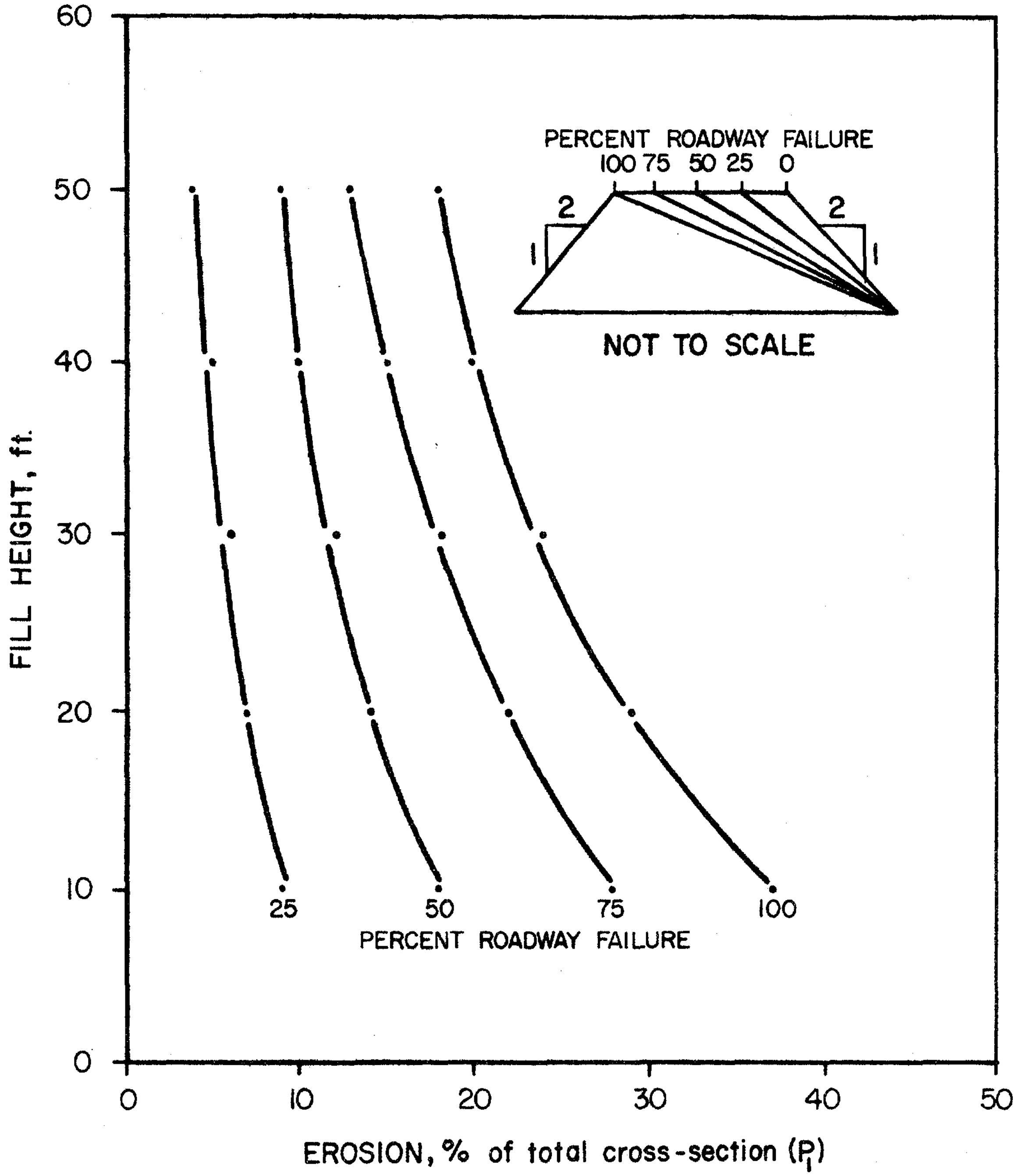


FIGURE 8-3

54 FOOT ROADWAY - EROSION FUNCTION

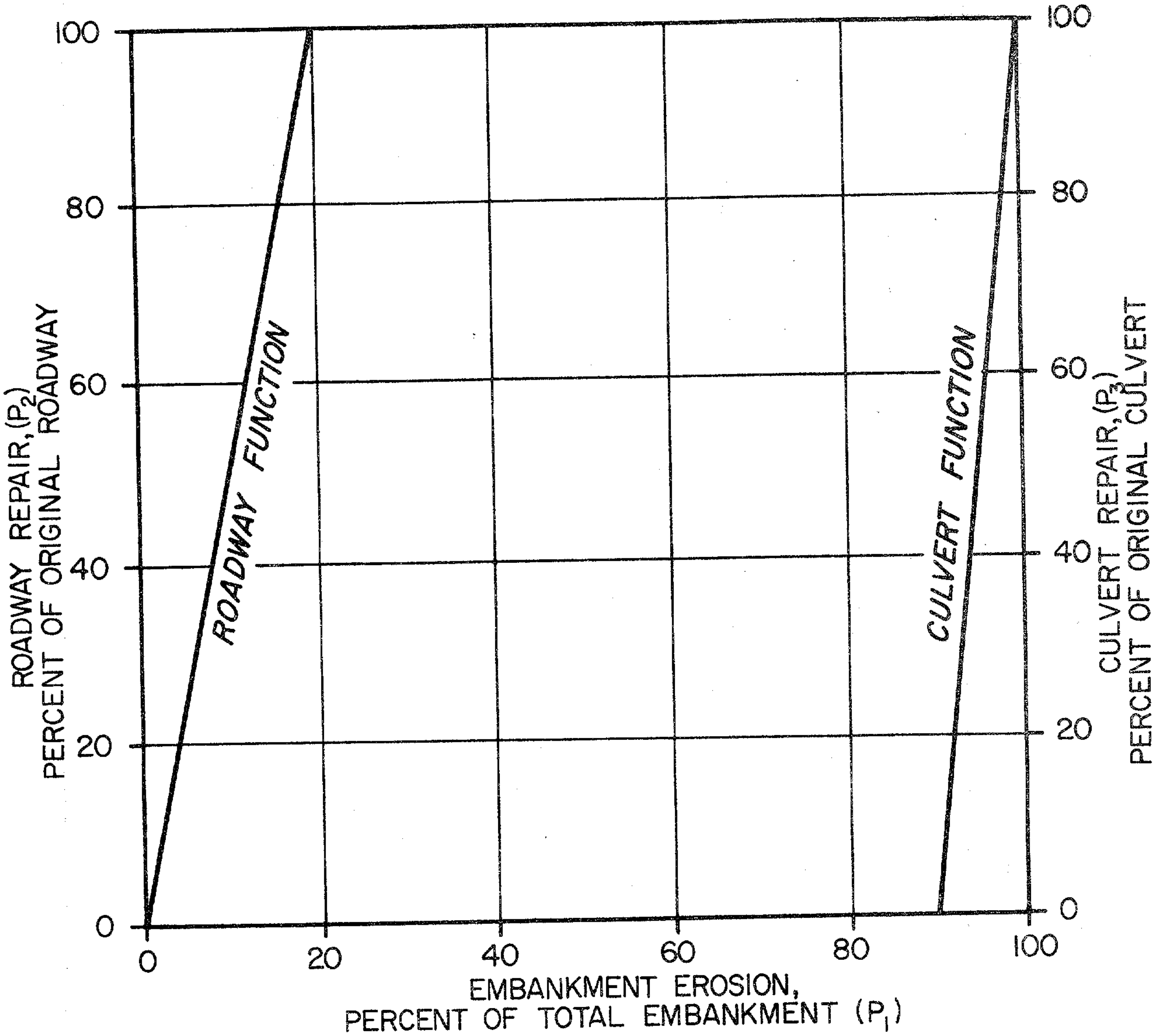


FIGURE 8-4

ROADWAY-REPAIR AND CULVERT-REPAIR FUNCTIONS

The cost adjustment factor is used to increase the cost of the original construction. This is done to allow for an increase in contracting cost to have a site quickly repaired.

The second component of the total economic loss due to the failure of a culvert is the traffic-related losses. An estimate of the total time that the traffic is not allowed to travel at its normal rate over the crossing is required. This time is assumed to be equal to the sum of the duration of the flood overtopping the road and additional time required to repair significant damage to the site. The duration of overtopping is computed in the flood routing procedure; the time of repair is estimated from Figure 8-5, which is developed by the analyst. The distribution and magnitude of the average daily traffic is also required. There are four sub-categories of traffic-related losses. They are:

1. additional running cost,
2. lost time of vehicle occupants,
3. expected accidents on additional detour miles, and
4. the expected accidents due to the unexpected obstacle.

The parameters necessary to evaluate these losses are:

X_1 = Duration of Detour = Duration of Overtopping + Repair Time (hrs)

X_2 = Average Daily Traffic, ADT, (Vehicles/Day)

X_3 = Passenger Cars (Fraction of ADT)

X_4 = Commercial Delivery Vehicles (Fraction of ADT)

X_5 = Single Unit Trucks (Fraction of ADT)

X_6 = Gasoline Service Trailers (Fraction of ADT)

X_7 = Diesel Semi-Trailers (Fraction of ADT)

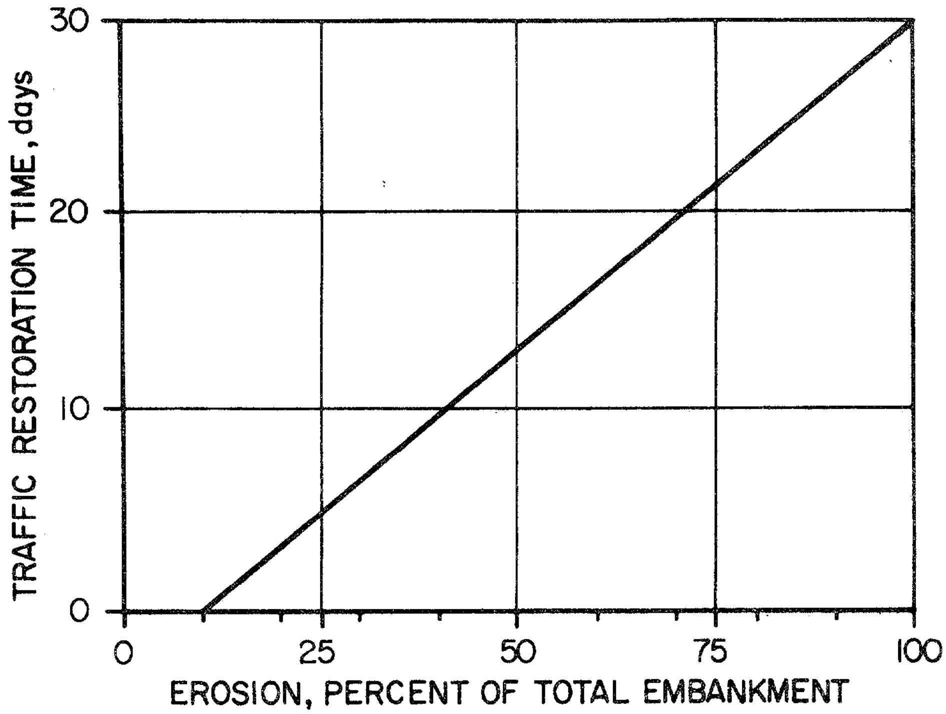


FIGURE 8-5

ASSUMED TRAFFIC RESTORATION FUNCTION

X_8 = Length of Detour (Miles)

X_9 = Speed on Detour (Miles/Hr)

X_{10} = Occupancy Rate (People/Vehicle)

X_{11} = Accident Distribution Ratio - Normal Conditions (Personal Injuries/Death)

X_{12} = Accident Distribution Ratio - Normal Conditions (Property Damage/Death)

X_{13} = Accident Distribution Ratio - Unexpected Obstacle (Personal Injuries/Death)

X_{14} = Accident Distribution Ratio - Unexpected Obstacle (Property Damage/Death)

X_{15} = Death Rate (People/100 Million Miles)

X_{16} = Death Rate Factor For Unexpected Obstacle (Multiplier to X_{15})

C_1 = Cost of a Death ($)

C_2 = Cost of a Personal Injury ($)

C_3 = Cost of Property Damage ($)

C_4 = Value of Time ($/Hr)

Parameters X_1 through X_{10} are different for each site considered, but X_{11} through C_4, except X_{16}, represent national statistics. X_{16} is a multiplier that is applied to the death rate to increase it due to the increased hazard of an unexpected obstacle. The value of the parameter varies depending on the site conditions and has to be evaluated using engineering judgment. The running cost of a passenger car (C_5) in dollars per 1000 vehicle miles, is a function of speed and is shown in Figure 8-6. The equation of the curve is:

$$C_5 = 42.5 - .455X_9 + .0068X_9^2 \tag{8-3}$$

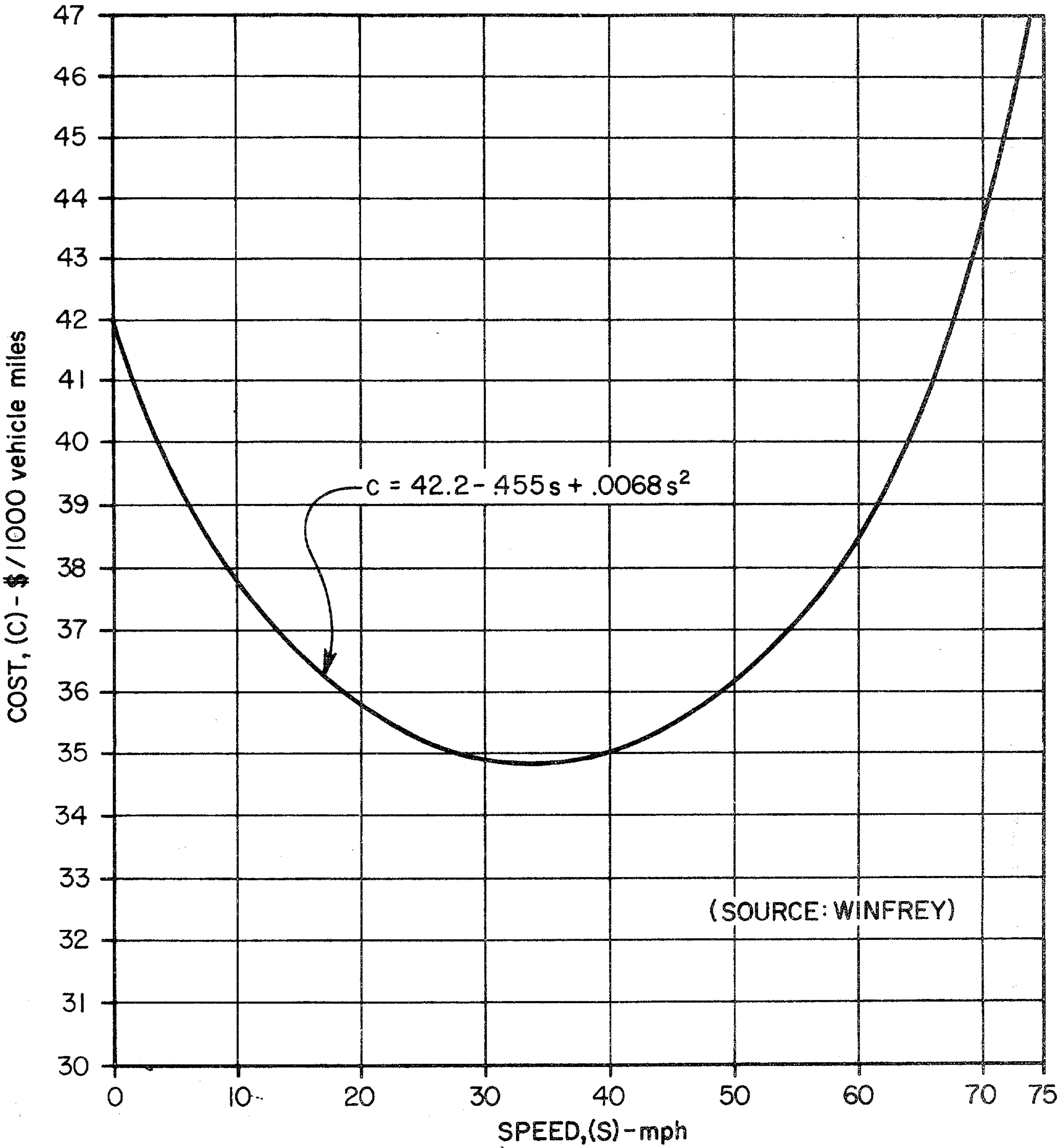


FIGURE 8-6

PASSENGER CAR RUNNING COSTS

To adjust this passenger car running cost for varying types of vehicle distributions, Equation 8-3 becomes:

$$C_5=(42.5-.455X_2^2+.0068X_9^2)\ (X_3+1.2X_4+2.0X_5+3.2X_6+3.1X_7). \qquad (8\text{-}4)$$

To estimate the losses associated with running costs, it is necessary to compute the running cost over the normal route and the detour. The difference in these two cost values represents the additional cost to the user of having to detour due to the failure of the culvert. The equation for computing the running cost (\$) is:

$$XL_1 = (X_1 \cdot X_2 \cdot X_8 \cdot C_5/24{,}000). \qquad (8\text{-}5)$$

The time lost by the vehicle occupants is the delay or the additional time it takes to detour the site. The value of lost time (\$) is computed by calculating the difference of the time value of the detour and the original route. The equation used for this calculation is:

$$XL_2 = (X_1 \cdot X_2 \cdot X_8 \cdot C_5)/24{,}000. \qquad (8\text{-}6)$$

The expected accident cost (\$) due to the difference in the dollar value of accidents on the detour and on the original route is calculated by:

$$XL_3=(X_1 \cdot X_2 \cdot X_8 \cdot X_{15}/2{,}400{,}000{,}000) \cdot (C_1+X_{11} \cdot C_2+X_{12} \cdot C_3). \qquad (8\text{-}7)$$

The expected accident cost due to an unexpected obstacle is computed by assuming one mile of road has a one-hour exposure to a higher death rate defined as the death rate, X_{15}, times a death rate multiplier for unexpected obstacles, X_{16}. Thus, the higher rate is $X_{15}X_{16}$. The accident distribution ratios are X_{13} and X_{14} which may vary from those for the normal death rate, X_{11} and X_{12}. The equation for calculating this loss ($) is:

$$XL_4 = (X_2 \cdot X_{15} \cdot X_{16}/2{,}400{,}000{,}000) \cdot (C_1 + X_{13} \cdot C_2 + X_{14} \cdot C_3). \qquad (8\text{-}8)$$

By applying Equations 8-3 through 8-8 to a set of data describing a given site, it is possible to compute the total dollar value of the traffic loss due to flooding at the culvert site. Table 8-1 shows the traffic-related data for the two case studies.

To develop a method to evaluate the adjacent property loss due to flooding, it is necessary to obtain information that relates damage to the type of land development in the upstream flood plain. Information is shown in Tables 8-2 through 8-7 relating losses in terms of 1970 dollars to the following land uses:

1. Agricultural,
2. Manufacturing,
3. Single Family Residences,
4. Retail Businesses,
5. Selected Services, and
6. Wholesale Businesses.

The method for the development of these tables is shown in Appendix B.

There are several notable characteristics of these data. First, flood losses do not always increase for depths exceeding four feet. For agriculture, the unit flood loss is not expected to significantly exceed the unit loss at a depth of three feet on the basis that most crops do not grow taller than this height. A similar trend of limited unit losses is expected for

TABLE 8-1

TRAFFIC LOSS DATA

Variable	Interstate 85	The Glade	Units
X_2	16000	466	Vehicles/Day
X_3	-85	.955	Fraction of ADT
X_4	.01	.043	Fraction of ADT
X_5	.02	0	Fraction of ADT
X_6	.03	.002	Fraction of ADT
X_7	.09	0	Fraction of ADT
X_8	1.20	1.41	Miles
X_9	55	25	Miles/Hr
X_{10}	1.7	2	People/Vehicle
X_{11}*	30	30	Personal Injuries/Death
X_{12}*	300	300	Property Damage/Death
X_{13}*	15	15	Personal Injuries/Death
X_{14}*	150	150	Property Damage/Death
X_{15}*	5.5	5.5	People/100 Million Miles
X_{16}	1000	500	---
C_1*	50000	50000	$
C_2*	2000	2000	$
C_3*	400	400	$
C_4*	2	2	$/Hr

* Accident Facts, National Safety Council, Chicago, 1968.

TABLE 8-4

ESTIMATED FLOOD LOSS DATA FOR
SINGLE FAMILY RESIDENCES,[1] 1970[2]

Estimated Market Value of Property[3]	Direct and Indirect Damages To Structure & Contents For Inside Water Depths of:			
$	1'	2'	3'	4'+
< 8,000	1,690	2,170	2,810	4,370
8,000 - 12,000	2,620	3,310	4,360	6,780
12,000 - 16,000	3,610	4,650	6,010	9,350
16,000 - 20,000	4,610	5,930	7,670	11,930
20,000 - 24,000	5,550	7,240	9,240	14,050
24,000 - 29,000	6,460	8,310	10,750	16,730
29,000 - 34,000	7,390	9,510	12,300	19,140
34,000 - 43,000	8,730	11,230	14,530	22,610
43,000 - 57,000	11,270	14,510	18,770	29,200
> 57,000	14,280	18,380	23,780	37,000

[1] U. S. Department of Commerce, Bureau of The Census, United States Census of Housing, 1960 Virginia, State & Small Areas.

Stanford Research Institute, A Study of Procedure in Estimating Flood Damage To Residential, Commercial and Indsutrial Properties In California, January 1960.

[2] Monetary data is converted to a 1970 base by applying average annual compound interest rates earned on long-term U. S. Government securities to the basic data.

[3] Value classifications for 1960 expanded to 1970 classifications on basis of average annual compound interest rates earned on long-term U. S. Government securities. These classifications do not include land values.

TABLE 8-5

ESTIMATED FLOOD LOSS DATA
FOR RETAIL BUSINESSES IN VIRGINIA,[1] 1970

Retail Trade	Annual Sales[2] Per Employee	Direct and Indirect Flood Damage[2] In Dollars Per $1000 of Annual Sales Per Establishment at Water Depths of:			
	$	1'	2'	3'	4' +
1. Building Materials, Hardware, and Farm Equipment Dealers	41,700.	22.	34.	46.	93.
2. General Merchandise, Group Stores	31,100.	14.	22.	30.	61.
3. Food Stores	48,400.	10.	15.	20.	41.
4. Automotive Dealers	59,300.	46.	70.	95.	194.
5. Gasoline Service Stations	36,500.	14.	22.	29.	60.
6. Apparel & Accessory Stores	25,500.	19.	29.	39.	80.
7. Furniture, Home Furnishings, and Equipment Stores	32,800.	18.	28.	38.	77.
8. Restaurants	11,900.	15.	24.	32.	65.
9. Drug Stores & Proprietary Stores	25,800.	17.	26.	36.	73.
10. Miscellaneous Retail Stores	35,000.	14.	21.	29.	58.
11. Nonstore Retailers	32,800.	14.	21.	29.	58.

[1] U. S. Department of Commerce, Bureau of The Census, 1967 Census of Business, Retail Trade, Virginia.

U. S. Department of Commerce, Bureau of The Census, Annual Sales, Year End Inventories, and Accounts Receivable of Retail Stores, By Kind of Business for 1967.

Stanford Research Institute, A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, Supplementary Report, 1960.

Stanford Research Institute, A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, Basic Report, 1960.

[2] Monetary data is converted to a 1970 base by applying an average annual compound rate of interest earned on long-term U. S. Government securities to the basic data.

TABLE 8-6A

ANNUAL REVENUE DATA

Service Specialty	Annual Revenue Per Employee $
1. Miscellaneous Services	14,300
2. Personal Services	7,900
3. Hotels, Motels, Tourist Courts & Camps	11,200
4. Automobile Repair Services & Garages	16,850
5. Amusement & Recreation Services (Excluding Motion Pictures)	12,800
6. Miscellaneous Repair Services	10,900
7. Motion Pictures	10,700

TABLE 8-6B

ESTIMATED FLOOD LOSS DATA FOR SELECTED SERVICES IN VIRGINIA,[1] 1970[2]

Water Depth Feet	Estimated Direct and Indirect Flood Damage Per $1000 of Annual Revenue
1	13.20
2	20.40
3	39.00
4	56.60

[1] U. S. Department of Commerce, Bureau of The Census, _1963 Census of Business, Selected Services, Virginia_.

U. S. Department of Commerce, Bureau of The Census, _1967 Census of Government Taxable Property Values_, Vol. 2.

Stanford Research Institute, _A Study of Procedures in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California_, Basic Report, January 1960.

[2] Monetary data is converted to a 1970 base by applying average annual compound interest rates earned on long-term U.S. Government securities to the basic data.

TABLE 8-7

ESTIMATED FLOOD LOSS DATA
FOR WHOLESALE BUSINESSES IN VIRGINIA,[1] 1970

Wholesale Business	Annual Revenue Per Paid Employee	Direct and Indirect Flood Damages[2] Per $1000 of Annual Revenue Per Establishment At Outside Water Depths of:			
	$	1'	2'	3'	4'
1. Groceries and related products	116,400	17.35	27.35	50.00	62.50
2. Petroleum and petroleum prodcuts	227,000	17.90	28.20	51.55	64.40
3. Farm products - raw materials	324,000	25.70	40.55	74.10	92.55
4. Motor vehicles & automotive equipment	117,100	18.90	29.80	54.40	74.70
5. Electrical goods	204,500	17.70	27.90	51.00	63.70
6. Machinery, equipment, & supplies	76,400	20.90	32.95	60.20	75.25
7. Lumber & construction materials	104,500	19.35	30.50	55.75	69.65
8. Metals & Minerals not elsewhere classified	276,000	17.60	27.75	50.70	62.95
9. Hardware, plumbing, heating Equipment & Supplies	85,500	16.10	25.35	46.35	57.90
10. Paper and paper products	111,100	18.00	28.40	51.90	64.85
11. Drugs, chemicals & allied products	93,800	7.60	11.95	21.85	27.30
12. All other	84,900	19.45	30.70	56.10	70.05

[1] U. S. Department of Commerce, Bureau of The Census, 1967 Census of Business, Wholesale Trade, Virginia
U. S. Department of Commerce, Bureau of The Census, 1967 Census of Government, Taxable Property Values, Vol. 2
Stanford Research Institute, A Study of Procedure in Estimating Flood Damage To Residential, Commercial, and Industrial Property in California, January 1960

[2] Monetary data is converted to a 1970 base by applying an average annual compound rate of interest earned on long-term U. S. Government securities to the basic data.

single family residences and selected services. Unit losses for manufacturing, on the other hand, can be expected to increase for depths greater than four feet because stock piles of raw materials and plant assets are commonly in excess of heights of four feet. Similarly, increasing unit losses are expected for retail and wholesale businesses. For these latter land uses, extrapolating available data to depths of six or seven feet appears justifiable.

Another characteristic of the flood data that should be noted is related to geographical application. The flood loss data for agriculture (Table 8-2) is limited in application to the seven specific counties in the State of Virginia, whereas flood loss data on single family residences (Table 8-4), are applicable to the entire United States. The flood loss data for the remaining categories, namely manufacturing (Table 8-3), retail business (Table 8-5), selected services (Table 8-6), and wholesale business (Table 8-7), are applicable to the State of Virginia.

Tables 8-2 to 8-7 are used in conjunction with an inventory of the land use in the upstream flood plain to develop a stage flood loss function. The approach is to compute losses for land area covered by varying degrees of depth of flood waters. The results of this computation for the two case studies are shown in Figures 8-7 and 8-8. A greater area loss is realized from a flood in The Glade area as compared to the Interstate 85 crossing; this is explained by the fact that the Interstate 85 site is located in farmland, mostly pasture, while a large number of single family dwellings are located upstream from The Glade site. Floods cause less damage to pasture

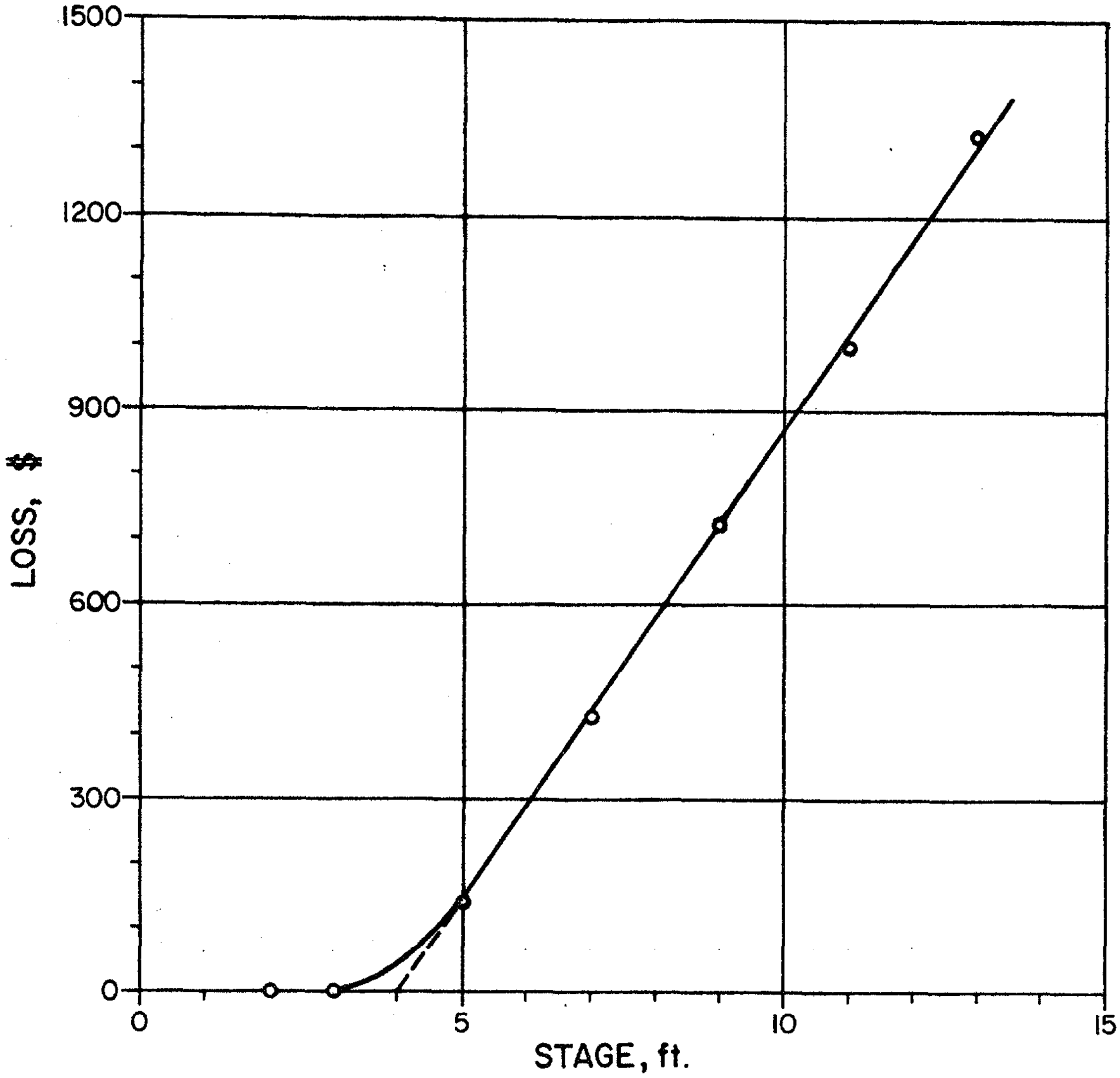


FIGURE 8-7

STAGE - DAMAGE FUNCTION, INTERSTATE 85 OVER A BRANCH OF GREAT CREEK

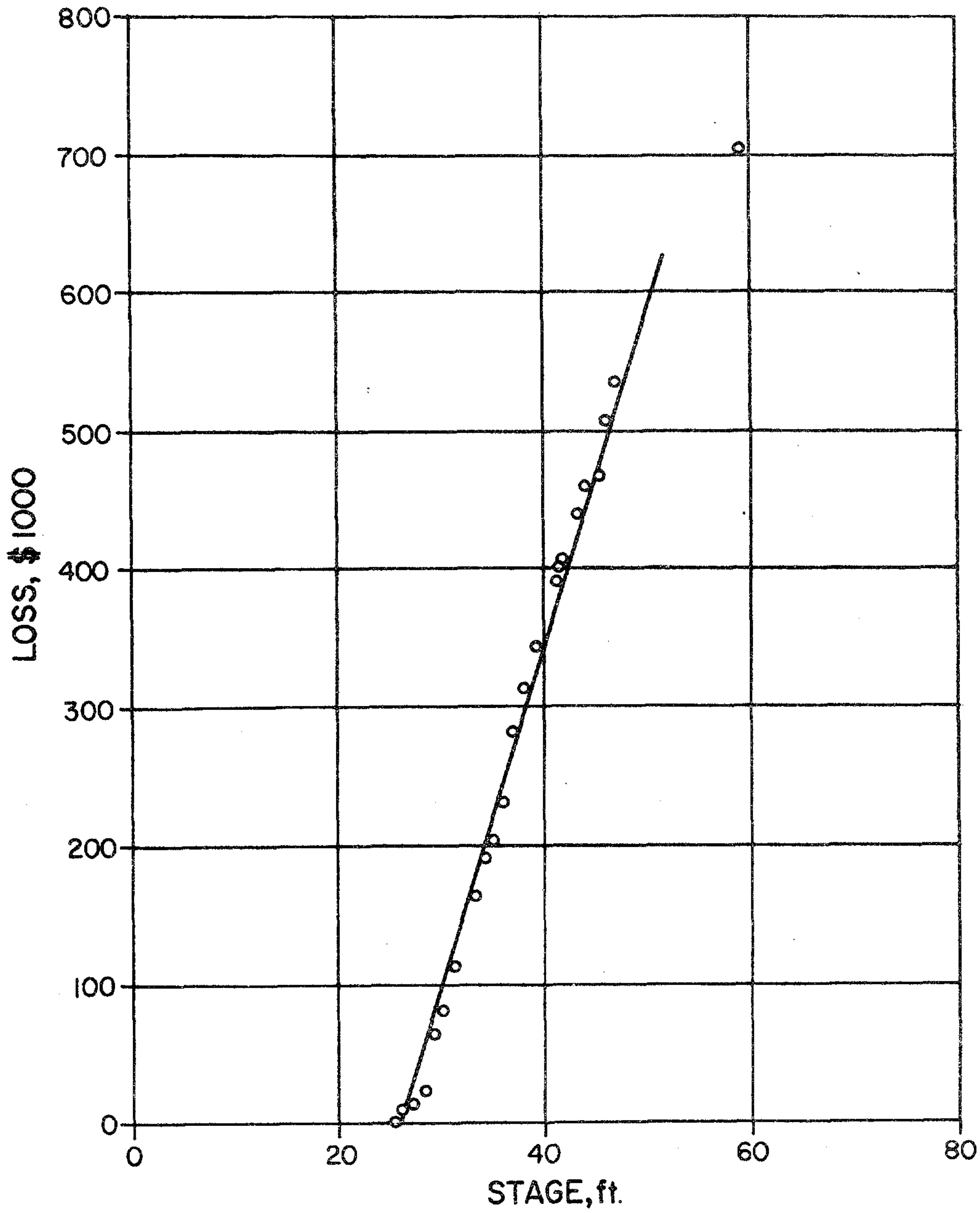


FIGURE 8-8

STAGE - DAMAGE FUNCTION, THE GLADE

land than to family residences. A significant result is that both the Figure 8-7 and Figure 8-8 loss functions are linear.

Different categories of the total loss function predominate at the case study site. At the Interstate 85 site the average daily traffic is so large, 16,000 vehicles, that the traffic loss component outweighs the others by at least a factor of five to one. In contrast, The Glade area loss function predominates because of the large number of single-family dwellings located upstream from the site.

The importance of the various parameters that define the loss functions (site loss, traffic loss, and stage-damage loss) is further discussed in a following chapter on the Sensitivity Analysis.

CHAPTER IX OPTIMIZATION

General

The purpose of this chapter is to show how the design of culverts for risk and cost minimization can be automated. The topics center on the feasibility of automation and how automation fits in the overall analysis. A wide range of designs are investigated to gain insight into the nature of the economic responses.

The overall strategy is to simplify the description of the problem and to automatically manipulate the simplification. The simplification, which is a revised version of the problem description, is done to gain:

1. a rapid means of investigating designs at low analysis costs,
2. the ability to study a large number of designs to infer the nature of the economic minimum, and
3. a pilot description of the problem upon which to test and build the complete problem description.

Every attempt is made to insure that the revised problem description captures the essentials of the design situation.

In actual design, the optimization step may be unnecessary; the fully described problem can be manually manipulated to arrive at a good design. Thus, this chapter is primarily intended to demonstrate the nature of optimal designs to provide the designer with background information to aid in searching for a good design. However, in complicated design situations, when costs are critical, optimization of a simplified problem may be desirable. Such a step can be the first design task which determines a preliminary design; the next step is to subject the preliminary design to careful analysis using the full problem description.

In brief, this chapter explores economic responses for a simplified problem, determines the feasibility of automation of optimization, and finds preliminary designs. The procedure is to modify the description of the problem, determine the nature of various solutions, and automate the search.

Simplifications

Four problem description simplifications are employed which involve geometrics, hydraulics, unit quantities, and numerical solutions.

1. Part A of Figure 9-1 shows the simplified rectangular elevation along the centerline of the road. The cross section along the centerline of the culvert is unchanged. Equivalent roadway lengths (l) are 333 feet for Interstate 85, and 400 feet for The Glade.

2. Hydraulic discharge assumptions are shown in Part B of Figure 9-1; condition 1 corresponds to orifice flow and condition 2 corresponds to weir flow. Entrance losses are neglected. The broad-crested weir overflow extends over the entire roadway length (l).

3. Unit quantities for steel and concrete cost estimating are derived from a statistical fit to data taken from California standard plans. Figures 9-2 and 9-3 show the steel and concrete curves for 104 standard plans for single box culverts; unit amounts for multiple box culverts are obtained by applying the appropriate multiplier (1, 2, 3, or 4) depending on the number of barrels. The functional form used in the statistical least squares fit, derived from analysis of bending moments in the culvert associated with soil pressure and traffic loads, displays a high degree of correlation (0.96 for steel and 0.98 for concrete). The individual points plot as a straight line.

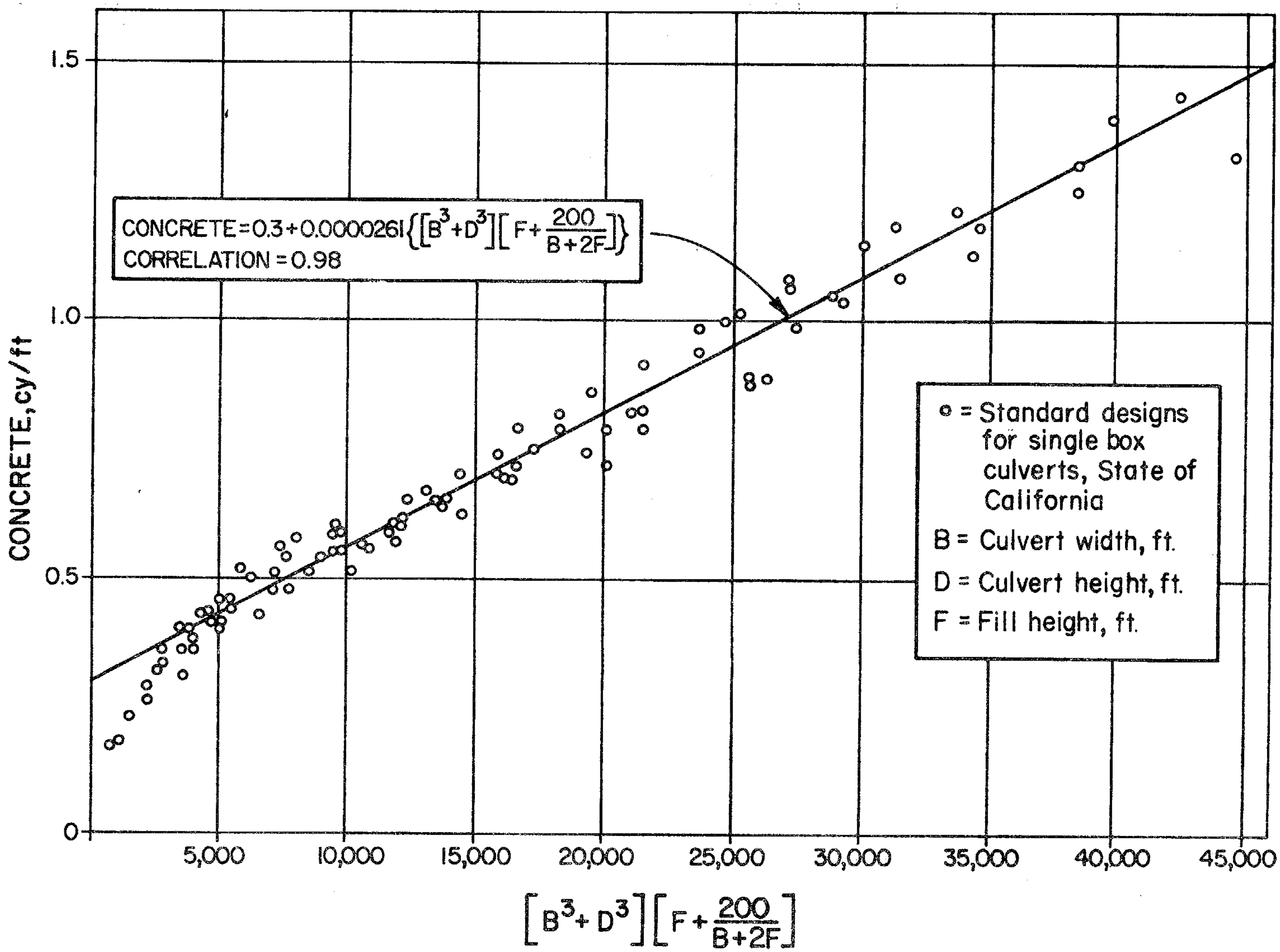


FIGURE 9-3

CONCRETE CURVE

4. The iterative solution (see Figure 7-1) is simplified to a one-step process (an Euler solution). The time step used is reduced to a small value and the ending storage is directly calculated without iteration; that is, given inflow and starting storage, outflow is calculated as a function of starting storage (without iteration), and the ending storage is derived by a mass balance for the time increment.

In addition to the four changes in logic, the stage-storage and stage-damage curves are represented as quadratic functions.

Preliminary Analyses

Experimentation, using the simplified problem description, is conducted using the Interstate 85 case study data. A large number of evaluations to determine construction costs, risks and total costs (construction costs plus risks) are performed. The purpose of finding these solutions for selected designs is to gain enough information to conceive and implement an automated procedure.

Consider Figure 9-4 which shows the culvert size (B and D) combinations contained in the Virginia Standards. The design region falls within the area defined by the lines D = 3 + B, B = 14, D = 3, and B = 3. Designs tend to have B greater than D and are exclusively for even foot values of B and D. For each design in the design region of Figure 9-4 and for one, two, three, and four-barrel configurations, analyses are made. Recall that Interstate 85 data are involved; a fill height (F) of 13 feet is used.

The economic responses of the resultant designs are plotted versus the culvert area (B x D); this is a convenient method to obtain two-dimensional plots. These plots are shown in Figures 9-5, 9-6, 9-7, and 9-8 for one, two,

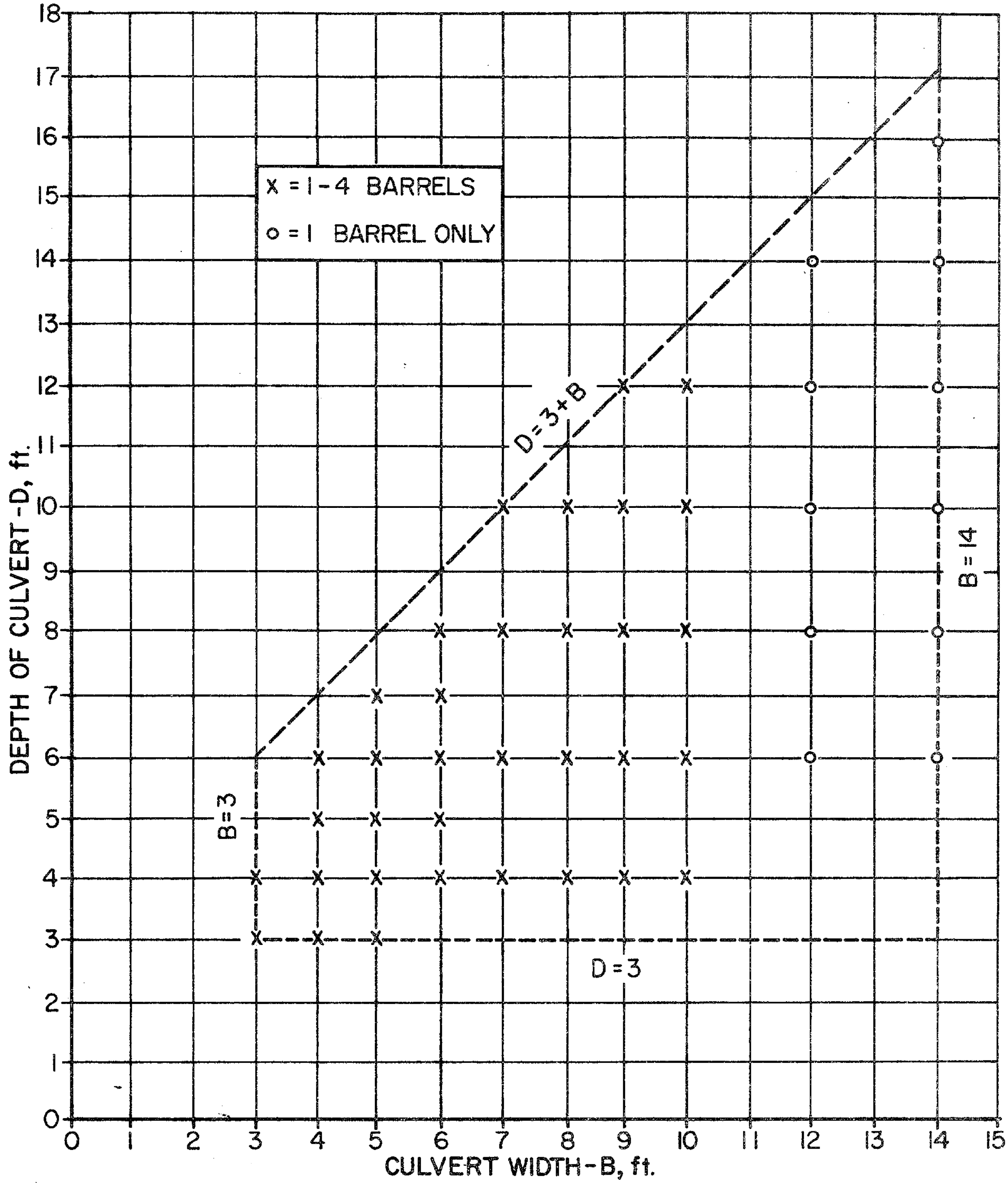


FIGURE 9-4
VIRGINIA CULVERT COMBINATIONS

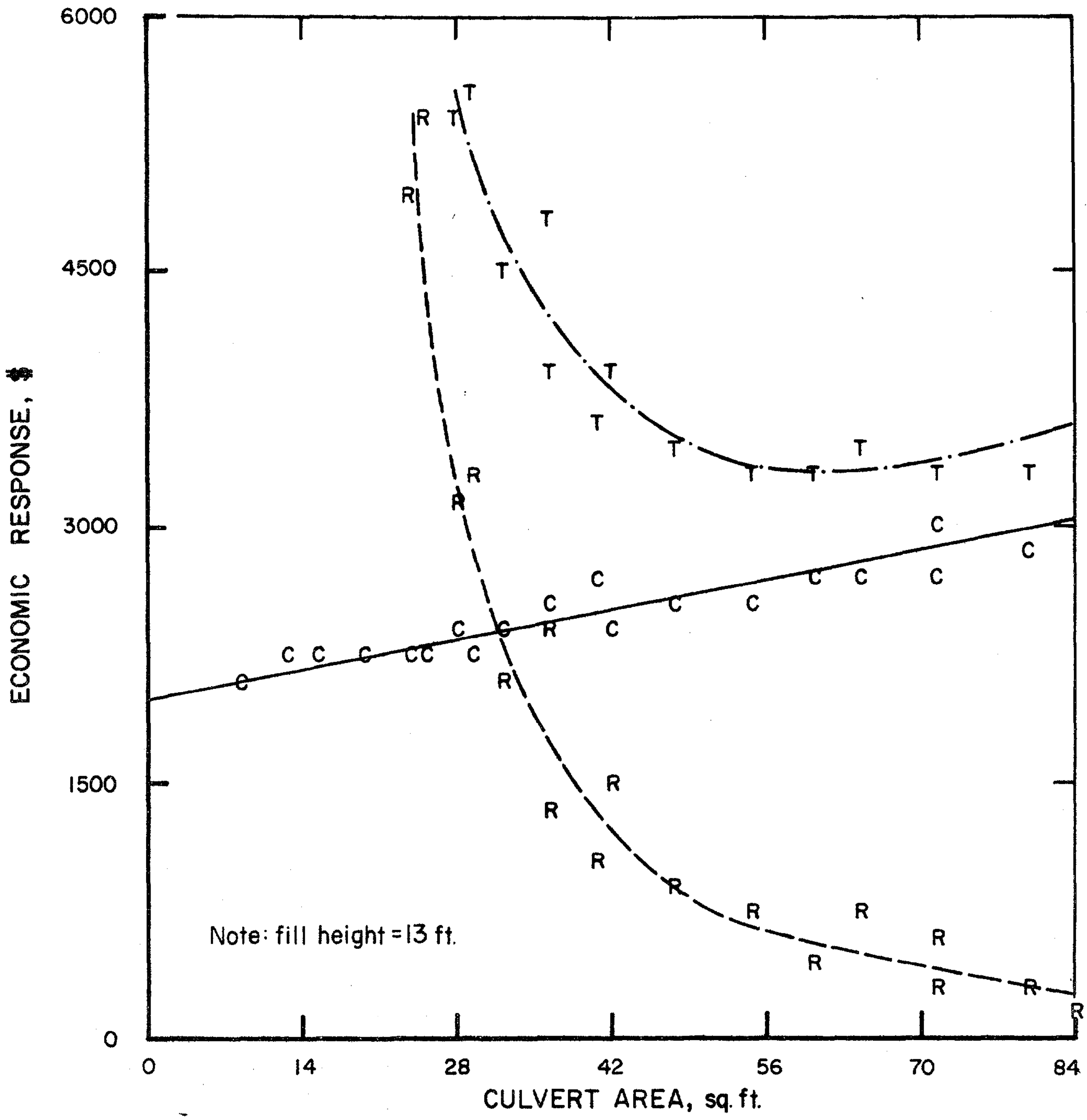


FIGURE 9-5

TWO-DIMENSIONAL ECONOMIC RESPONSE AT I-85 SITE FOR ONE BARREL

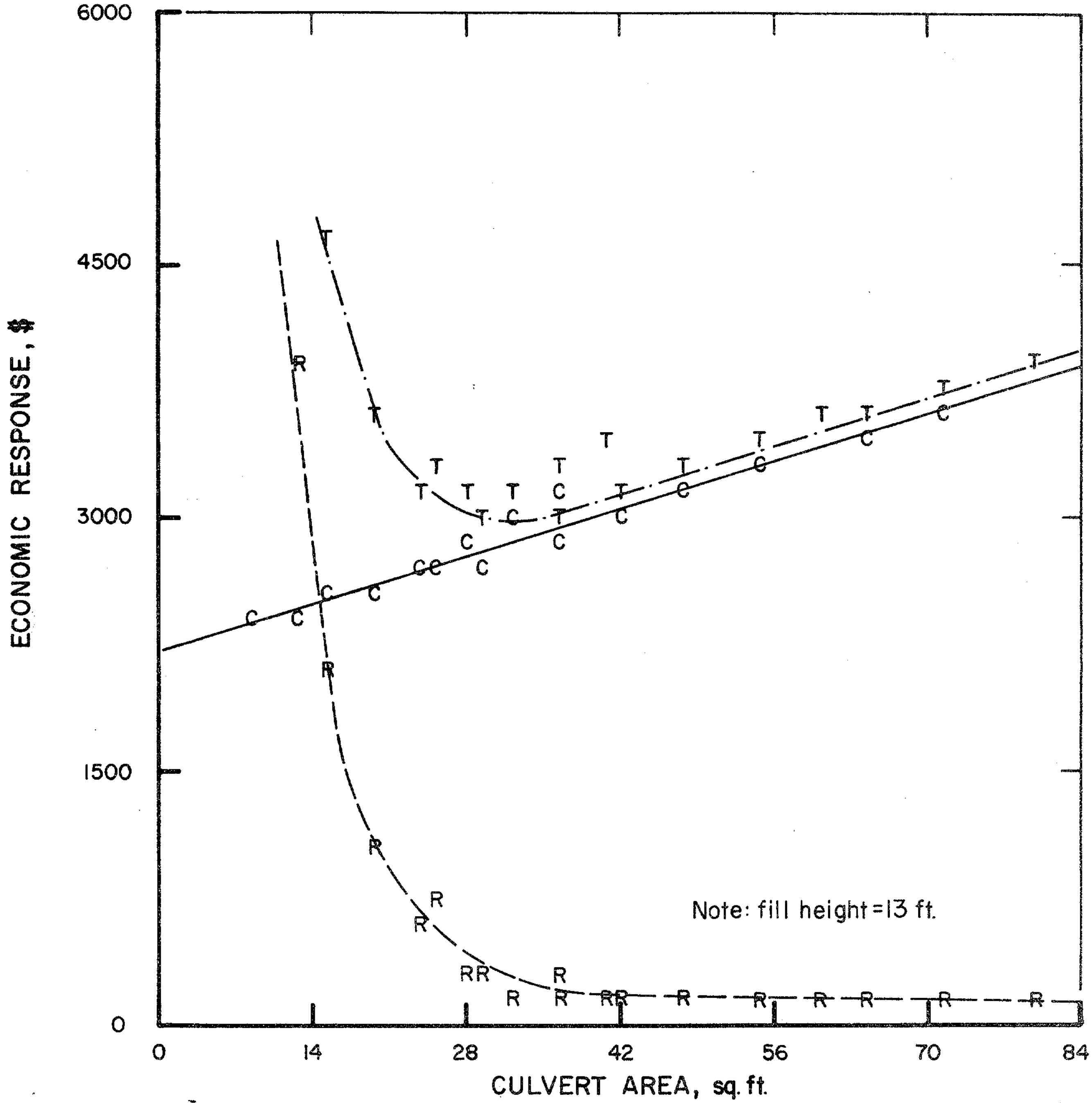


FIGURE 9-6

TWO-DIMENSIONAL ECONOMIC RESPONSE AT I-85 SITE FOR TWO BARRELS

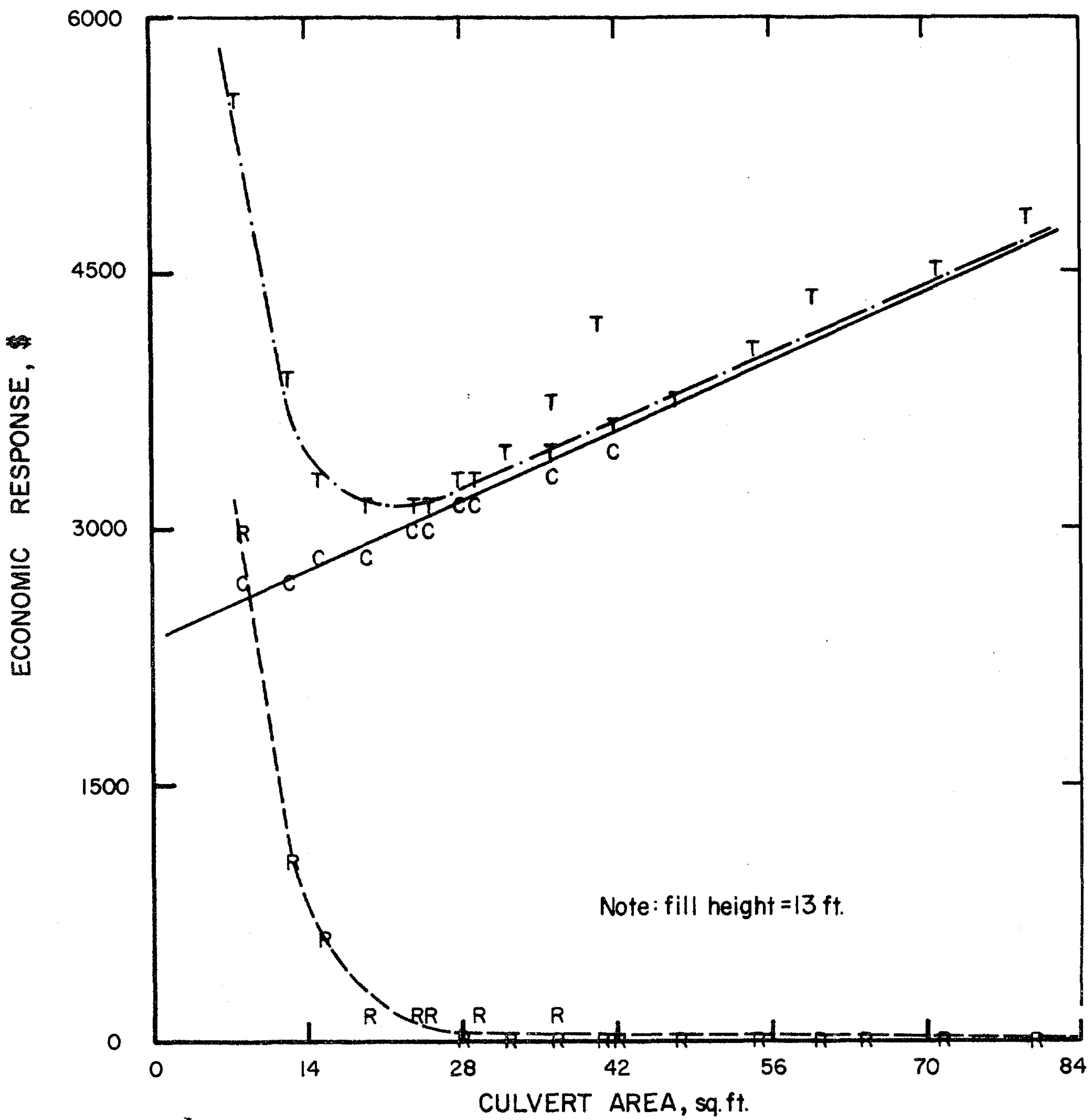


FIGURE 9-7

TWO-DIMENSIONAL ECONOMIC RESPONSE AT I-85 SITE FOR THREE BARRELS

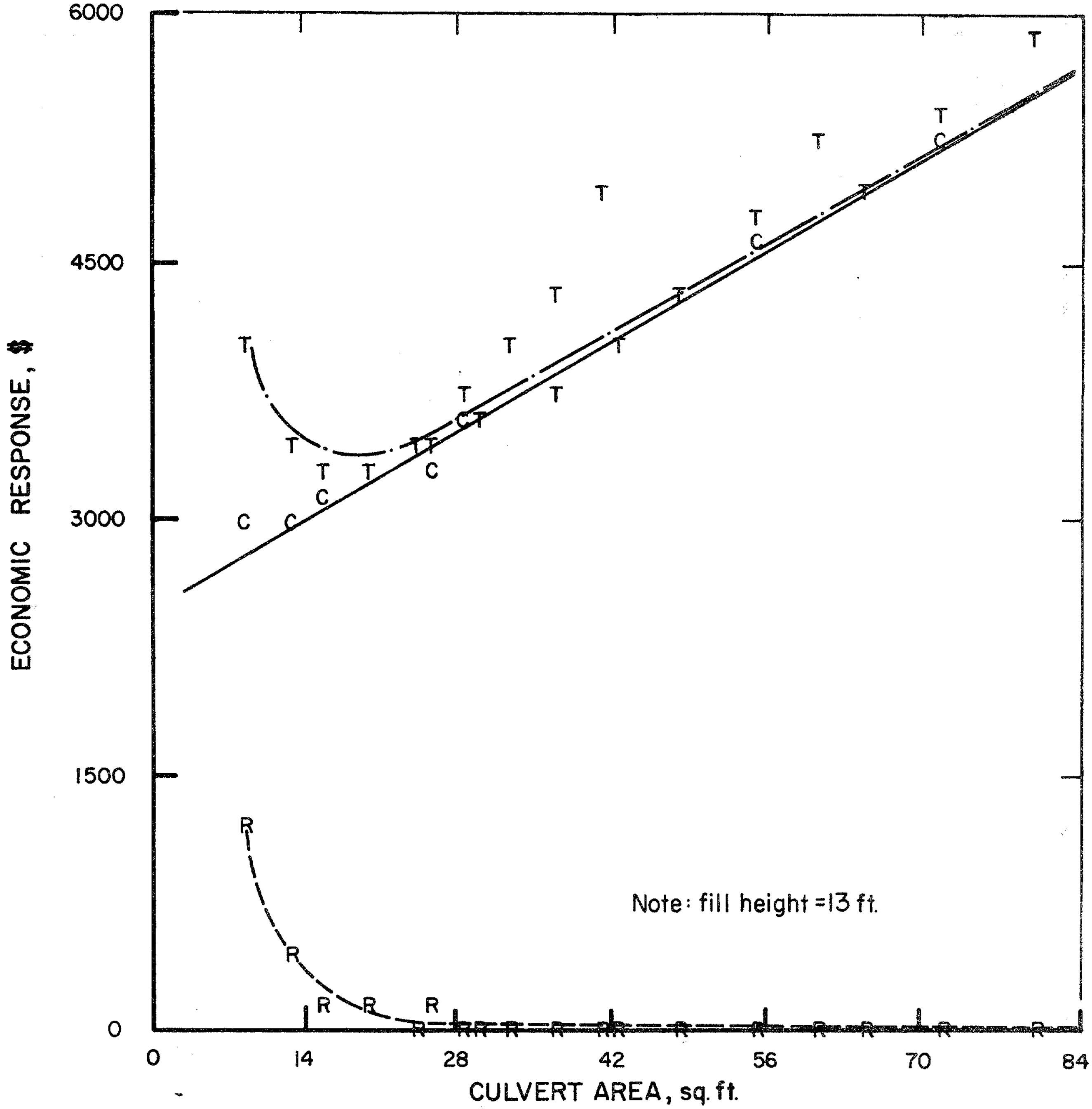


FIGURE 9-8

TWO-DIMENSIONAL ECONOMIC RESPONSE AT I-85 SITE FOR FOUR BARRELS three, and four barrels, respectively. Note that the plotting symbols, R (risk), C (construction cost), and T (total = R + C), do not fall on the smooth curves (drawn in by eye), rather scatter is shown. The scatter derives from the fact that area is not a completely adequate variable to predict the total variation. In other words, the economic response varies with both B and D as well as F which is fixed in this case.

However, the four figures demonstrate the essence of the risk analysis. Construction costs increase approximately linearly as area increases. Similarly, risk rapidly decreases from high values at small culvert openings to very low risks at large culvert openings. The risk versus waterway area curve is nonlinear and is approximately hyperbolic. The shape of these curves accords with intuition and bears out a basic assumption of this study; risks can be considered in culvert design. In this study, risks imply the expected value of economic losses.

Sequentially considering Figures 9-5, 9-6, 9-7, and 9-8, it is noted that the low point of the total cost curve ("T") moves from right to left for increasing number of culvert barrels. This phenomenon is present because the abscissa of each curve is the area of one box regardless of the number of barrels being considered. However, the analysis indicates that the total area of the opening (kBD where k = the number of barrels) is roughly the same for each total cost curve given in Figures 9-5, 9-6, 9-7, and 9-8. For example, the $B \cdot D$, which corresponds to the low point for one barrel, is about 56 square feet and the corresponding number for two barrels is near 28 square feet; 2 x 28 = 56.

It is significant that each set of curves has the same general shape. Also, highly important is the fact that a minimum exists regardless of the number of barrels considered.

Figure 9-9 shows the results of relaxing the 2-D representation. The contours of this figure are lines of equal total cost. Three-dimensional variation is captured. Only one minimum is observed, and the contours describe a relatively smooth total cost surface. Figure 9-9 presents results for one barrel; similar curves for two, three, and four barrels are obtained. Figure 9-9 and the 2-D curves show that total costs rise gradually on the high B·D side of the minimum and rise very rapidly on the low side. This leads to the qualitative inference that the cost to society of overdesign (more waterway openings than necessary) is much less than a comparable underdesign.

In summary, this preliminary analysis, although not automated, yielded considerable information. Automation appears feasible because:

1. low points exist,
2. multiple low points are not observed, and
3. the 3-D total cost surface is regular and smooth; no anomalies are observed.

Automation should consider:

1. the discrete nature of the designs; that is, designs are specified at integral values of B and D in feet,
2. the boundaries within which standard designs exist (B = 3 + B, B = 14, D = 3, B = 3), and

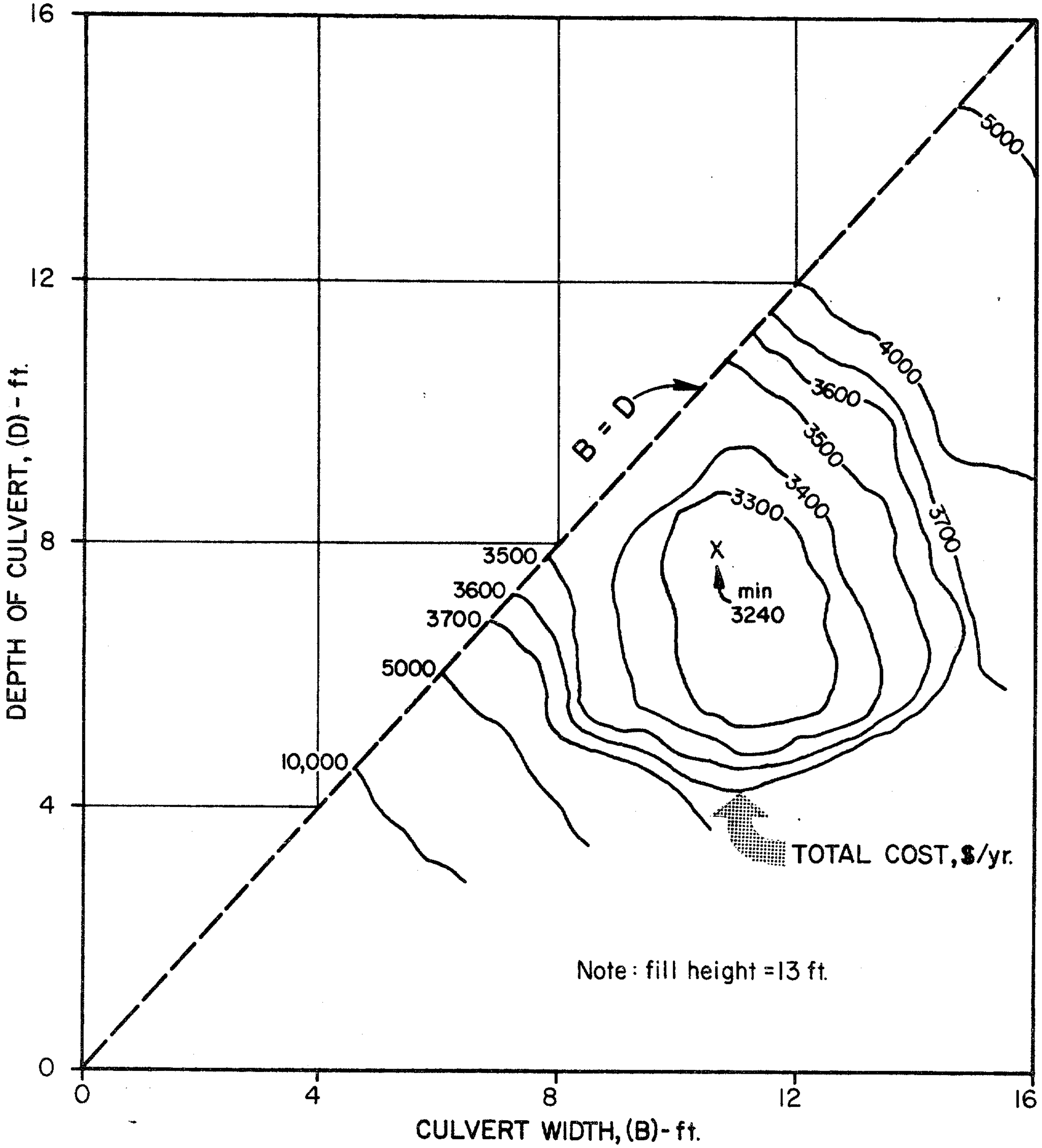


FIGURE 9-9

THREE-DIMENSIONAL ECONOMIC RESPONSE AT I-85 SITE FOR ONE BARREL

3. the area of flow is a primary design factor and is relatively constant for optima defined for designs having variable numbers of barrels.

Perhaps the most significant finding of this preliminary analysis is that overdesign is preferable to underdesign with respect to the area of culvert opening.

Optimization Scheme

Based on the preliminary analysis, an optimization procedure is designed and tested. The technique fixes the number of barrels (k) and the fill height (F) and conducts a discrete gradient search for minimum total cost in the B,D plane. The zone of search is bounded as shown in Figure 9-4; only B,D combinations having integer values are considered. One, two, three, and four barrel cases are automatically considered. The procedure must be implemented for each value of fill height considered; thus, the optimization of fill height requires a series of parametric solutions (where F is the parameter being varied). The discrete gradient search is illustrated in Figure 9-10 and has four steps:

1. Check neighborhood. Given a starting point, the adjacent points are studied. Each point is a design. Each design is evaluated for total cost (the sum of construction costs plus risk). The simplified problem description is used to derive the total cost.

2. Find minimum. The least total cost design of the eight neighboring designs is identified. For example, in Figure 9-10, point 6 is the minimum. The direction from the starting point to the minimum (point 6) establishes a direction of motion.

3. Move from neighborhood to neighborhood. As long as succeeding points have lesser costs, continue in the direction of motion found in Step 2. Upon reaching

STEP 1: CHECK NEIGHBORHOOD

STEP 2: FIND MINIMUM (POINT 6, 3014)

STEP 3: MOVE FROM NEIGHBORHOOD TO NEIGHBORHOOD

STEP 4: TERMINATE UPON NO IMPROVEMENT

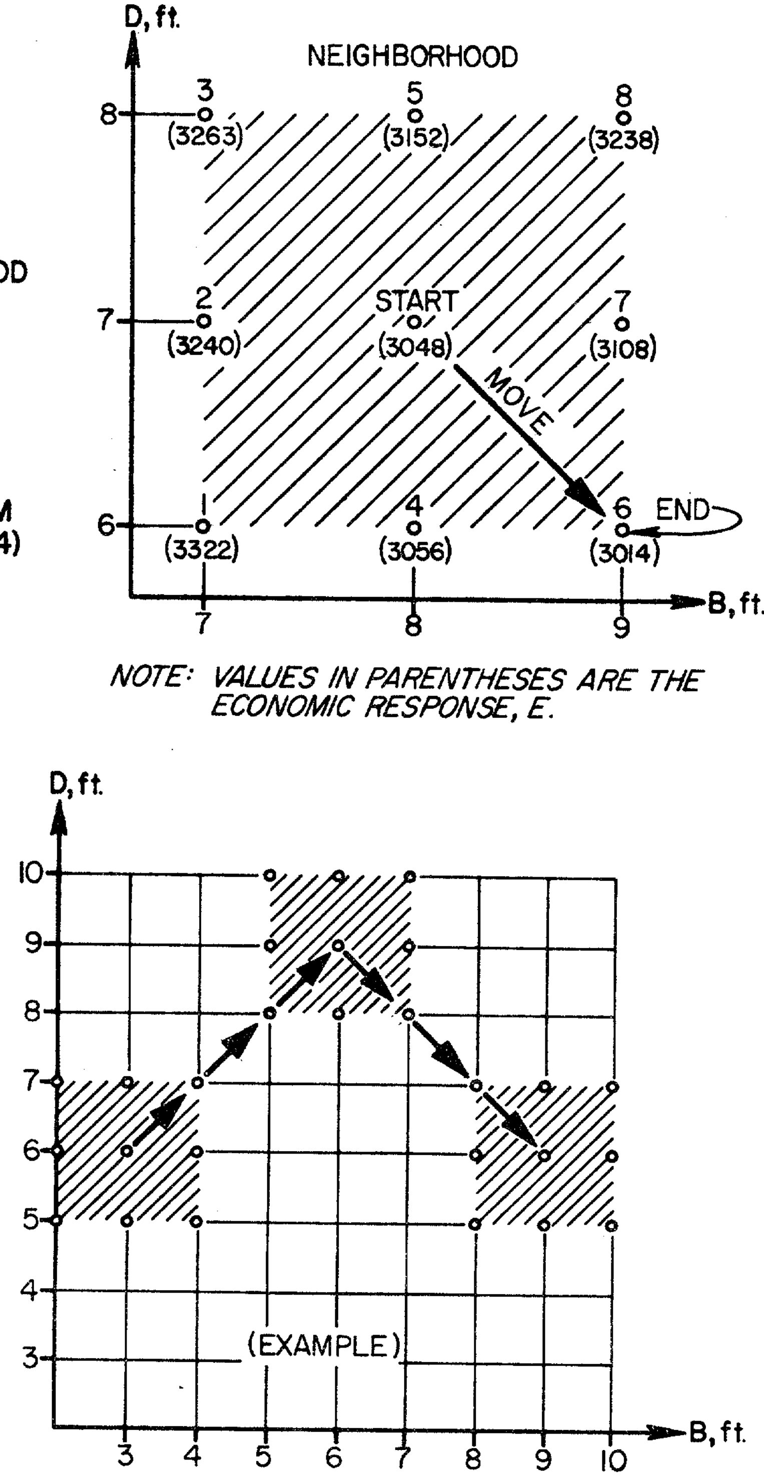


FIGURE 9-10

DISCRETE GRADIENT SEARCH

How can a starting point for the two-barrel case be obtained using the optimum solution of the one-barrel case? For this case, assume that the starting point is a pair of square culverts. Recall that the optimal area of flow (80 ft^2; see preceding paragraph) is relatively constant for designs having variable numbers of barrels. Therefore, given a nearly optimal area (in this case, 80 ft^2), one can solve, kBD = area; for Interstate 85 with two barrels, the starting point is B=D=6 to the nearest integer. Thus, the discrete gradient search initiates from B=D=6.

The logic applies to three and four-barrel solutions as well. Given a two-barrel optimum, a three-barrel starting point can be established. Given a three-barrel optimum, a four-barrel starting point can be established. Note that the order of consideration is one, two, three, and four-barrel situations. Study of the reverse order of consideration indicates that the same solutions are found with negligible differences in the amount of computational effort.

In brief, a tested and feasible, automated optimization scheme for minimizing the sum of construction costs plus risks for box culverts consists of the following steps:

1. Solve the one-barrel case.

   A. The starting point is obtained by assuming a square culvert, and solving the equation, $BD=Q(\max)/\sqrt{2gF}$, where Q(max) is the maximum flow in the most severe hydrograph and F is the fill height.

   B. A discrete gradient search moves from the starting point to the one-barrel optimum combination of B,D.

2. Solve the two, three, and four-barrel cases in the order stated.

   A. The starting point is obtained by assuming multiple square culverts, and solving the equation, kBD = area, where k is the number of barrels and area is the total waterway area found in the preceding case.

   B. A discrete gradient search moves from the starting point to the appropriate optimum combination of B,D; each of the barrels in a multiple box solution is assumed to have identical values for B and D.

The procedure works for the simplified problem description for the two case studies, Interstate 85 and The Glade.

Preliminary Results

The main finding is that an automated optimization or *computerized design is feasible*. The underlying simplified problem description serves two roles: first, it permits the feasibility study to proceed in parallel with the development of a finely detailed problem representation; and secondly, it provides cost effective information upon which to base decisions concerning the detailed representation. An insight into the cost and risk responses and an appreciation for the interaction of the main variables is also a result of this chapter.

Table 9-1 shows the optimum designs for both cases and is based upon the automated design. The results are used to pinpoint neighborhoods to study using the complete problem definition; the sensitivity analysis is conducted in the next chapter. It is found that, while the total costs differ significantly between the simplified and complete problem, the optimal design selection of waterway openings are very close to being the same.

TABLE 9-1

AUTOMATED OPTIMA

SIMPLIFIED PROBLEM DESCRIPTION

INTERSTATE 85

Width B	Depth D	Fill Ht. F	Barrels k	Yearly Cost	Responses Risk	$ Total
10	8	13	1	2887	353	3240
6	6	13	2	2819	229	3048
5	4	13	3	2897	191	3088
4	4	13	4	3102	136	3238

THE GLADE

Width B	Depth D	Fill Ht. F	Barrels k	Yearly Cost	Responses Risk	$ Total
6	5	53	1	13509	1463	14971
5	4	53	2	14695	635	15249
4	3	53	3	15083	820	15903
3	3	53	4	15739	820	16559

It is informative to compare the Table 9-1 results with conventional designs. A conventional design in this case is defined to be one which uses a static design flow having a one-in-fifty year return period in conjunction with a headwater depth criteria (10 feet above invert for Interstate 85 and 5 feet above crown for The Glade). These data, design tables, and engineering judgments yield conventional designs of $k = 2$, $B = 8$, $D = 7$ for Interstate 85, and $k = 2$, $B = D = 8$ for The Glade. Different engineers may select different headwater depth criteria or apply other constraints; however, resultant designs should be approximately the same.

The conventional designs are larger than the automated designs. This suggests that the one-in-fifty year return period is conservative and yields over designs. However, only two cases are considered and The Glade site is rather unusual and may bias the inference.

Consider the effect of fill height (F). A systematic study is conducted which varies F (for Interstate 85) and optimizes for each value of F. The results are plotted in Figure 9-12. A fill of 13 feet is called for in the preliminary design. If the road center line elevation could be dropped two feet, an optimum could be obtained. This may be impossible for long range alignment or other reasons. However, a pronounced minimum is observed for fill height. This situation may be of interest to designers should fill height be adjustable. Note that at low fills the cost plus risk goes upward; this is because the risks outweigh the cost savings for low fill heights.

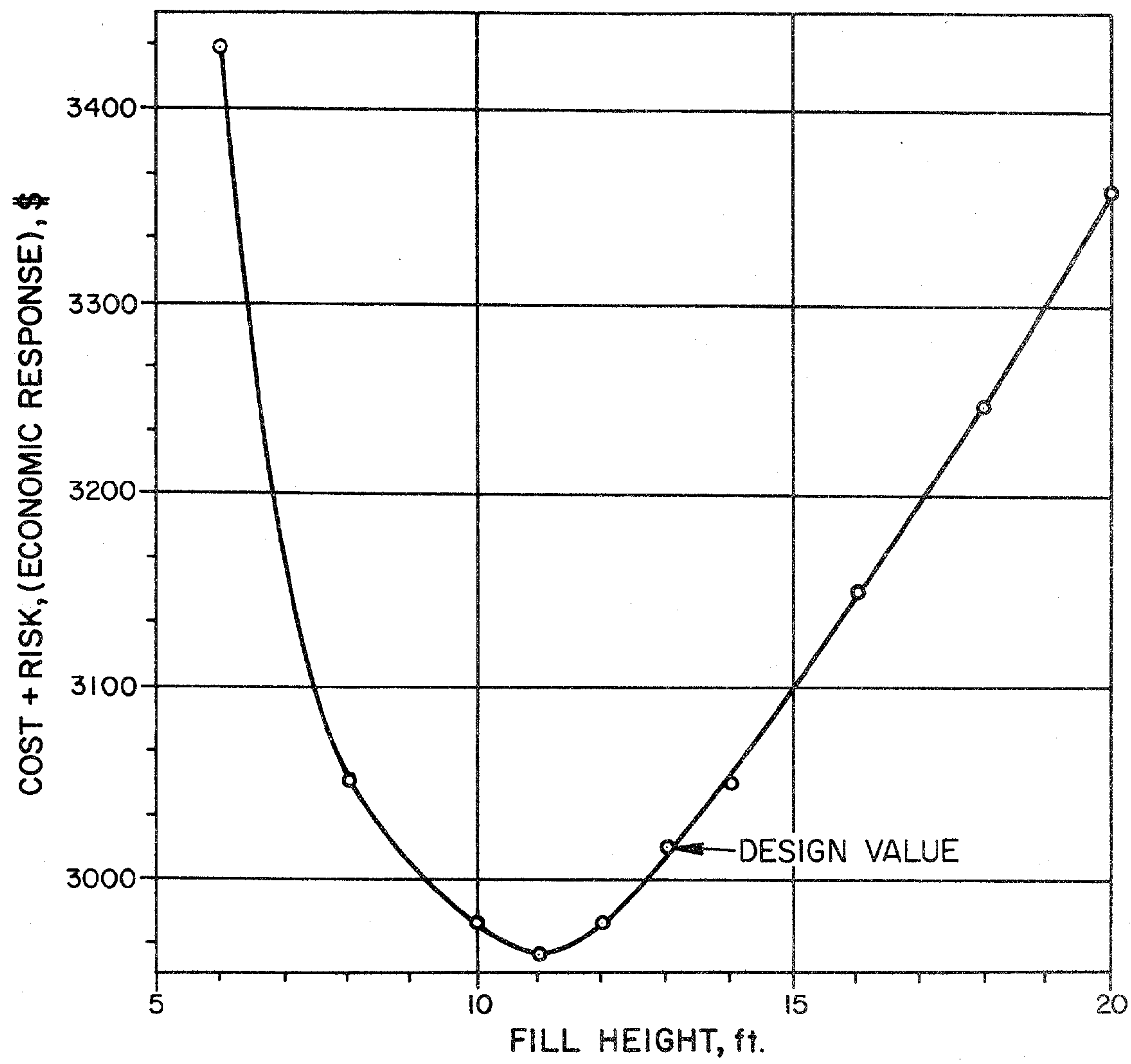


FIGURE 9-12

EFFECT OF FILL HEIGHT ON OPTIMUM ECONOMIC RESPONSE FOR THE I-85 CASE STUDY

Refined Results

Investigation of the simplified problem solutions, shown in Table 9-1, indicates that cost components associated with fill volumes and road surface (guardrail to guardrail) are biased. This fact was derived from comparing the simplified problem results with like results for the full problem description; the basic assumption is that the full problem description results in more accurate answers.

The bias is traced to the equivalent roadway length, l, shown in Figure 9-1. The bias is reduced by employing the following strategy:

1. Compute the volume of the fill using the exact dimensions of the site.
2. Select the value of l such that the simplified cross section has a fill volume equivalent to that computed in Step 1.

This refinement gives an l of 442 feet for Interstate 85 and 300 feet for The Glade; corresponding fill volumes are 33,000 and 119,000 cubic yards, respectively. The actual length of roadway, L (600 feet for Interstate 85 and 634 feet for The Glade), is used for estimating the pavement, shoulder, and guardrail costs. The equivalent length, l, is used to estimate fill costs and is employed as the length of the overtopping, broad-crested weir flow.

With the refined equivalent roadway length (and, in the case of Interstate 85, an adjustment to the final design fill height of 14.6 feet), the simplified problem yields the revised automated optima shown in Table 9-2. These results are not greatly different from the Table 9-1 values; the differences reflect the elimination of the bias.

TABLE 9-2

REVISED AUTOMATED OPTIMA

SIMPLIFIED PROBLEM DESCRIPTION

INTERSTATE 85

Width B	Depth D	Fill Ht. F	Barrels k	Yearly Responses ($) Cost	Risk	Total
9	7	14.6	1	4085	329	4414
6	5	14.6	2	4122	202	4328
5	4	14.6	3	4299	153	4452
4	3	14.6	4	4421	200	4621

THE GLADE

Width B	Depth D	Fill Ht. F	Barrels k	Yearly Response ($) Cost	Risk	Total
6	5	53	1	11,218	1,463	12,681
4	4	53	2	11,729	1,230	12,959
4	3	53	3	12,793	820	13,613
3	3	53	4	13,449	820	14,269

The solutions to the simplified problem are used as points of departure for a study of full problem description results. Solutions to the full problem are obtained in the neighborhood of the departure point designs. Thus, better designs are sought by making incremental improvements in the vicinity of the simplified problem solutions; the betterment is obtained through the superior cost and risk measurement attributes of the full problem description. The results of this neighborhood search are shown in Table 9-3.

The solutions to the full problem description are given in the columns marked "complex" in Table 9-3. These solutions are determined through the use of the full problem description. To compare these solutions and measure economic estimating differences, simplified solutions for the same design are listed under the columns marked "simple." This table leads to an understanding of the value of using highly detailed and complex problem descriptions rather than simplified versions of the same problem. Note that, in terms of the total costs, the simplified version gives a 5.3 per cent underestimate for Interstate 85 and a 2.5 per cent underestimate for The Glade.

A strategy for optimization and the selection of designs that follows from the logic of this chapter is:

1. Employ a simplified version of the problem in conjunction with an automated search to derive a *good design;* the good design corresponds to the optimal solution to the simplified problem.

2. Use a full problem description (having a degree of complexity commensurate with the budget to solve the problem) to solve for designs in the vicinity of the *good design* to find a *better design*; this search employs manual selection of B and D.

TABLE 9-3

RESPONSE COMPARISON

	INTERSTATE 85		THE GLADE	
	Simple	Complex	Simple	Complex
Total ($/year)	4,509	4,762	13,115	13,448
Cost	4,195	4,243	11,913	11,754
Risk	314	519	1,201	1,694
Cost Components ($)				
Surface	34,632	34,633	10,200	10,787
Fill	15,412	15,398	118,015	118,644
Culvert	14,503	15,132	55,067	51,062
Risk Components ($/year)				
Repair	78	192	0	0
Stage Damage	215	297	1,202	1,694
Traffic Loss	21	30	0	0

Design Variables	INTERSTATE 85	THE GLADE
B	4	5
D	4	7
F	14.6	53
Barrels	3	1

Notes:
1. Simple denotes simplified problem description.
2. Complex denotes complete or full problem description.

A note of clarification concerns the automation of the evaluation of a design. Both the simple and complex problem descriptions are automated; given a design, a computerized computation gives the resultant costs and risks. However, only in Step 1 above is the search to select a design automated.

CHAPTER X SENSITIVITY ANALYSIS

General

Having established culvert design sizes for each case study, the sensitivity of these central designs to changes in individual design assumptions and parameters is tested. The relative importance of each in terms of total cost, or the sum of construction cost plus risk, may be assessed to form a basis for decisions. Regardless of the engineering feature being designed, culverts are just one example, there are going to be uncertainties in the designer's mind concerning the values to select for the design parameters such as interest rate, highway speed, flood peak magnitudes, etc. Sensitivity results permit the designer, or the authority who is developing criteria for the designer, to cope with and hedge against the uncertainty known to be present, but seldom explicitly considered.

Uncertainty can also be associated with, or thought of as, the error or difference between forecasted and subsequently observed future values. For example, flood peak magnitudes, durations, and probabilities of occurrence (or the reciprocal, the return period) are estimates; good or bad depending upon the amount of available information, they are still estimates. Another example is the unit cost assigned to concrete and steel; these are known to vary with time as well as location. The list of factors subject to estimating errors is long and virtually every parameter and measurement associated with culvert design is a candidate for some degree of error. Not only does sensitivity analysis permit hedging, it identifies critical variables and factors for the purpose of singling them out for careful consideration in the design process to reduce uncertainty. Furthermore, research and development priorities can be established in design agencies to study and resolve estimating problems associated with sensitive design factors.

The central designs are specified to be the optimal designs identified in the previous chapter. For Interstate 85, B=4, D=4, F=14.6 (fill height) and k=3 (barrels); for The Glade, B=5, D=7, F=53.0 and k=1. Changes in parameters and design assumptions are investigated. One parameter is studied individually, while all others are held at the values upon which the central design is based. Parameters such as the flood peak magnitudes, which are made up of several values, are varied proportionately. For example, increasing the peaks by 25 per cent is defined as operating on all values by multiplying them by a factor 1.25.

Additionally, the sensitivity of overall designs, in their cost and risk categories, is studied. The optimal designs are contrasted with the conventional designs specified in the last chapter. Recall that the conventional designs are: for Interstate 85, B=8, D=7, F=14.6 and k=2; for The Glade, B=8, D=8, F=53 and k=2. The optimal and conventional designs are also compared with high risk designs.

Sensitivity of Assumptions and Parameters

Using the full problem description, Tables 10-1 and 10-2 are developed. Table 10-1 applies to Interstate 85 and ranks 44 design factors according to the average absolute change in total cost (economic response) associated with plus and minus 25 per cent changes in the appropriate factors. In other words, the absolute value of the total cost associated with a plus 25 per cent change in a factor is averaged with the absolute value associated with a minus 25 per cent change to obtain the average absolute value. The average absolute value determines the ranking. Table 10-2 applies to The Glade and is much shorter, having only 18 entries. The brevity of Table 10-2 derives from the fact that the fill height is sufficiently high (53 feet) to preclude overtopping from even the largest flood. This phenomenon blocks numerous factors from influencing the total cost.

Consider the factors from Tables 10-1 and Table-2 having ten per cent or more average absolute change. These are: interest rate, unit cost of road surface, unit cost of fill material, flood loss (stage-damage), road elevation (fill height), and flood peak magnitude. There are six factors listed which represent both case studies. An inference of this study is that these factors merit careful consideration in the design of culverts to minimize the construction costs plus risks.

Consider the effect of culvert inlet design on the total cost; Table 10-3 summarizes inlet design sensitivity results using the full problem description.

TABLE 10-1

RANKING OF SENSITIVITY

INTERSTATE 85

Rank	Variable	Percent Change In Economic Response[1]		
		AAV	+25%	-25%
1	Interest rate	22.7	+22.8	-22.6
2	Flood peak magnitude	15.3	+22.8	- 7.8
3	Unit cost of road surface	13.2	+13.2	-13.2
4	Fill height	8.2	+ 6.3	+10.1
5	Unit cost of fill	5.8	+ 5.8	- 5.8
6	Stage-damage function	4.0	+ 5.2	- 2.8
7	Highway design speed	3.9	0	+ 7.8
8	Unit cost of concrete	3.9	+ 3.9	- 3.9
9	Detours design speed	3.7	0	7.4
10	Empirical exponent (β) in erosion function	2.2	+ 2.2	+ 2.2
11	Downstream embankment slope	2.0	+ 2.6	- 1.5
12	Upstream embankment slope	1.8	+ 1.8	- 1.8
13	Highway travel distance	1.6	+ 0.2	3.0
14	Unit cost of reinforcing steel	1.5	+ 1.5	- 1.5
15	Manning's 'n' for downstream embankment slope	1.4	- 0.2	+ 2.5
16	Unit weight of fill	1.2	- 0.1	+ 2.4
17	Detour travel distance	1.1	+ 1.9	- 0.3
18	Roadway repair time as a function of erosion	1.1	+ 1.1	- 1.1
19	Time to flood peak	1.1	+ 2.1	- 0.1
20	Repair cost factor	1.0	+ 1.0	- 1.0
21	Road surface repair as a function of erosion	0.8	+ 0.8	- 0.8

[1] All factors vary ±25%. AAV stands for average absolute value. Tabular entries represent percent change in the total cost (or the economic response) associated with the factor change given as the column lable. AAV is the average of the absolute values of the two adjacent entries.

TABLE 10-1

continued

Rank	Variable	Percent Change In Economic Response[1]		
		AAV	+25%	-25%
22	Empirical coefficient (α) in erosion function	0.5	+ 0.1	+ 0.85
23	Flood duration	0.5	+ 0.8	- 0.2
24	Amortization period	0.4	+ 0.7	0.1
25	Culvert skew angle	0.4	+ 0.5	- 0.3
26	Death rate	0.2	+ 0.2	- 0.2
27	Manning's 'n' for culvert barrel	0.1	0	- 0.2
28	Culvert slope	0.1	0	- 0.2
29	Death rate factor	0.1	+ 0.1	- 0.1
30	Culvert entrance loss coefficient	0.1	+ 0.2	0.0
31	Cost of property damage in auto accident	0.1	+ 0.1	- 0.1
32	Unit cost of structural excavation	0.1	+ 0.1	- 0.1
33	Cost of death in auto accident	0.1	+ 0.1	- 0.1
34	Average daily traffic	<0.05	--	--
35	Cost of personal injury in auto accident	<0.05	--	--
36	Ratio of the number of property damage accidents to the number of accidents in which a death occurs under normal driving conditions	<0.05	--	--
37	Ratio of the number of property damage accidents to the number of accidents in which a death occurs in accidents caused by an unexpected obstacle	<0.05	--	--
38	Threshold erosion velocity	<0.05	--	--

TABLE 10-1

continued

Rank	Variable	Percent Change In Economic Response[1]		
		AAV	+25%	-25%
39	Ratio of the number of personal injury accidents to the number of accidents in which a death occurs under normal driving conditions	<0.05	--	--
40	Ratio of the number of personal injury accidents to the number of accidents in which a death occurs caused by an unexpected obstacle	<0.05	--	--
41	Value of time to average traveler	~ 0	--	--
42	Culvert repair as a function of erosion	~ 0	--	--
43	Tailwater as a function of discharge	~ 0	--	--
44	Vehicle occupancy	~ 0	--	--

TABLE 10-2

RANKING OF SENSITIVITY

THE GLADE

Rank	Variable	Percent Change In Economic Response[1]		
		AAV	+25%	-25%
1	Stage-damage function	50.5	+90.0	-11.0
2	Fill height	29.4	+32.8	-26.0
3	Interest rate	21.6	+21.8	-21.4
4	Unit cost of fill	14.4	+14.4	-14.4
5	Flood peak magnitude	13.7	+18.5	- 8.9
6	Upstream embankment slope	7.9	+ 7.9	- 7.9
7	Downstream embankment slope	6.6	+ 6.6	- 6.6
8	Unit cost of concrete	4.9	+ 4.9	- 4.9
9	Flood duration	4.8	+ 5.3	- 4.2
10	Unit cost of road surface	1.3	+ 1.3	- 1.3
11	Unit cost of reinforcing steel	1.1	+ 1.1	- 1.1
12	Manning's 'n' for culvert	.7	+ 1.4	0
13	Amortization period	.4	- 0.1	+ 0.6
14	Time to peak inflow	.3	+ 0.3	- 0.3
15	Unit cost of excavation	~ 0	--	--
16	Culvert slope	~ 0	--	--
17	Culvert entrance loss coefficient	~ 0	--	--
18	Tailwater depth	~ 0	--	--

[1] All factors vary ± 25%. No other variables have any effect on the answer because no overtopping occurs at The Glade. AAV stands for average absolute value. Tabular entries represent percent change in the total cost (or the economic response) associated with the factor change given as the column lable. AAV is the average of the absolute values of the two adjacent entries.

TABLE 10-3

SENSITIVITY OF ECONOMIC RESPONSE
TO INLET STRUCTURE DESIGN

INLET TYPE	DESCRIPTION	ECONOMIC RESPONSE ($)		% CHANGE FROM TYPE 1	
		I 85	THE GLADE	I 85	THE GLADE
1	Wingwalls, 3/4 inch chamfer on top edge, Flare angle = 45 degrees, No offset.	4762	13,448	---	---
2	Wingwalls, 3/4 inch chamfer on top edge, Flare angle = 18.4 degrees, No offset.	4839	13,560	+1.6	+0.8
3	Wingwalls, 3/4 inch chamfer on top edge, Flare angle = 18.4 degrees, Skew angle 15-45 degrees.	4790	13,558	+0.6	+0.8
4	90 degree headwall, Bevel on all three edges, Bevel angle = 45 degrees, Vertical bevel = 1/2 inch per foot of rise, Horizontal bevel = 1/2 inch per foot of span.	4713	13,296	-1.0	-1.1

Changes in total cost vary from -1.1 to 1.6 per cent. The ninety degree headwall design yields the lowest total cost; however, regardless of selection, inlet design has only modest impact on the total cost.

Broaden consideration to factors from Tables 10-1 and 10-2 having a two per cent change or more. This increases the combined list to 12 factors (note that the upstream and downstream embankment slopes are counted as one factor). A grouping of the 12 factors into three categories (economic, engineering, and hydrologic-hydraulic) is presented in Table 10-4. Six of the factors appear in both cases as marked in Table 10-4. This listing highlights important areas for research priorities which can be summarized as:

1. determination of the influence of interest rate and criteria for its selection,
2. estimation of unit costs,
3. estimation of stage-damage curves (techniques for this are presented in this report, but adequate testing remains to be done),
4. investigation of how to forecast future highway speeds,
5. consideration of methods which predict inflow hydrographs, and
6. investigation and definition of erosion phenomenon.

This list does not reflect a priority ranking; furthermore, certain elements of Table 10-4 are not included (fill height, for example) because they are considered to be the result of design, rather than information (or parameters) upon which the design is based.

TABLE 10-4

CLASSIFICATION OF TWELVE MOST SENSITIVE VARIABLES

ECONOMIC	
Interest Rate[3]	
Unit Cost Road Surface[1]	
Unit Cost Fill[3]	
Stage-Damage Curve[3]	
Unit Cost of Concrete[3]	
ENGINEERING	
Highway Speed[1]	
Detour Speed[1]	
Fill Height[3]	
Embankment Slopes[2]	
HYDROLOGIC-HYDRAULIC	
Flood Peak Magnitudes[3]	
Erosion Function[1]	
Flood Duration[2]	

[1] Interstate 85 Site.

[2] The Glade Site.

[3] Interstate 85 and The Glade Sites.

Sensitivity of Designs

Using the simplified problem description, Table 10-5 is developed. An optimal design, conventional design, and high risk design are contrasted for each case study. The optimal design is based on construction cost plus risk minimization, using a number of inflow hydrographs, dynamic flood routing, and loss estimation; whereas the conventional design is based on sizing the waterway to accommodate the one-in-fifty year return period peak flood flow. Given the conventional design, the associated construction costs and risk components are estimated as shown in Table 10-5. Also shown are the cost and risk estimates for designs purposely chosen to have high risks.

The striking result is that the conventional designs have larger waterway openings than do the optimal designs. Also the risk costs are much lower for the conventional designs. In fact, conventional designs give very low risk costs for both case studies.

The conventional design discharge which corresponds to the optimal designs in Table 10-5 is estimated. The question is, 'what static flow, corresponding to a particular return period as shown in Figures 6-2 and 6-3, could result in the derived optimal designs?' The procedure used to answer this question is to work the conventional design procedure backwards, assuming that inlet control governs. To start, the sizes of the box culverts are given (the optimal designs). The associated headwater criteria are imposed (ten feet above the invert for Interstate 85 and five feet above the crown

TABLE 10-5

DESIGN SENSITIVITY

	I - 85 Design			The Glade Design		
	O* ptimal	C* onventional	High Risk	O* ptimal	C* onventional	High Risk
Design Variables						
Culvert Width (B)	4	8	4	5	8	4
Culvert Height (D)	4	7	4	7	8	4
Fill Height (F)	14.6	14.6	8	53.0	53.0	17.0
Number of Barrels (K)	3	2	2	1	2	1
TC = Total Cost ($/yr)	4,500	4,800	22,000	13,100	21,600	22,800
C = Cost (% of TC)	91	97	15	91	100	11
Roadway (% of C)	54	48	68	5	3	27
Fill (% of C)	23	31	16	65	35	52
Culvert (% of C)	23	21	16	30	62	21
R = Risk (% of TC)	9	3	85	9	0	89
Repair (% of R)	25	0	58	0	--	93
Stage-Damage (% of R)	69	100	40	100	--	0
Traffic (% of R)	6	0	2	0	--	7

*Restrained to the given fill heights (14.6 and 53.0) for The Glade). Given the size and the headwater criteria, a conventional design discharge is computed (for Interstate 85, 660 cfs and for The Glade, 450 cfs). Using the conventional design discharges and the flood peak frequency curves (Figures 6-2 and 6-3), the corresponding return periods are determined as one-in-five years for Interstate 85 and one-in-one year for The Glade. Therefore, if a designer uses discharge estimates corresponding to a return period of one-in-five years for Interstate 85 and one-in-one year for The Glade, his designs would approximate the optimal designs; recall that optimal designs minimize construction costs plus risks.

The return period differences are probably attributable to the utilization and acceptance of ponding in the risk-oriented design. In other words, headwater ponding, if it is permissible, reduces required waterway openings to much smaller values than the conventional design procedures. The frequent return periods associated with optimal designs are a measure of the smaller drain opening requirements. Also, the risk costs are incompletely assessed in this analysis. There are possible losses (and, hence, risks) associated with maintenance costs, exit velocity energy dissipation, and seepage failures, not to mention intangible losses. These factors are neglected, but their inclusion should move the optimal return period to values toward the one-in-fifty year frequency.

An inference is that conventional design is risk adverse. One interpretation is that the difference in conventional and optimal total costs is what the designer has society pay to sustain his risk adversion. It is probably more realistic to deduce that the difference represents a safety factor to account for design uncertainties, intangible costs, and legal or institutional restraints imposed to cover broad design categories. A benefit of this study is that specific cases can be analyzed to establish reasonable values of this factor of safety.

The cost to society is the total cost; that is, construction cost plus risk. The analysis indicates the possibility that from an economic standpoint, in the absence of side restraints, society should assume some risks (in both cases, the risk levels are 9 per cent of the cost) to obtain minimal total costs. The optimal risks suggested are greater than those currently implied by conventional design. Of course, restraints exist and two case studies do not provide a strong enough information base to form strong inferences. Further case studies are strongly suggested to provide sufficent information to deduce appropriate risk levels and safety factors.

The high risk designs in Table 10-5 are included for comparison. These designs are arbitrarily selected to show the economic consequences of a poor design.

Four observations concerning Table 10-5 could have broad design implications:

1. the optimal design risks exceed the conventional design risks,

2. the distribution of total costs to construction costs and risks is the same in both case studies (91 per cent, 9 per cent),

3. the repair (culvert fill and roadway) and stage-damage components of the risk both have numerous high entries in the table which could indicate the relative significance of these loss categories, and

4. the traffic components (accidents, increased running costs, lost time) of the risk (the last row) are small for all designs which could indicate the relative insignificance of this loss category.

Sensitivity of Risks and Traffic Turbulence

Table 10-6 shows the results of a sensitivity study of the simplified version of the culvert analysis program for Interstate 85 and The Glade. This analysis is designed to show the effect of underestimating the expected losses associated with large floods. This is done by multiplying the total expected losses by an increasing factor until a change in the optimum design occurs.

A one thousand per cent increase is used as a starting point for both sites. This increase results in a large change in the optimum design; therefore, the increase is reduced until the point of change from the initial optimum design occurs. The analysis indicates that a twenty per cent underestimation of the expected losses at The Glade does not change the optimum design; while a ninety per cent underestimation of the Interstate 85 losses does not change the design.

TABLE 10-6

THE EFFECT OF RISK ON THE MINIMUM COST DESIGN

Increase In Risk (%)	Number of Barrels	Culvert Width (ft)	Culvert Height (ft)	Total Cost ($/yr)
		INTERSTATE 85		
0	2	6	5	4,330
90	2	6	5	4,510
200	2	7	4	4,530
1000	2	9	4	5,670
		THE GLADE		
0	1	6	5	12,680
20	1	6	5	12,970
30	1	6	6	13,090
1000	2	6	5	15,020

The simplified culvert analysis for the Interstate 85 site has a constant detour speed of 55 M.P.H. Experience and study of the Highway Capacity Manual indicates that this value should not be constant, but should reflect some traffic turbulence and overloading when a culvert is flooded out and the main route is closed.

The step function in Table 10-7 is used to estimate the turbulence effect. This function is incorporated into the simplified culvert analysis program for Interstate 85 to determine if there is any change in the optimum design. Since the optimum design corresponds to the situation where there is practically no overtopping, the optimum design does not change. In reviewing the possible designs, the step function causes a maximum increase of 1.2 per cent in the total costs. Note that these results are obtained from a very limited sample (one case study); the traffic turbulence does not influence The Glade design since no traffic losses occur because there is no overtopping.

TABLE 10-7

ASSUMED DETOUR SPEED TRANSITION

Detour Duration Hrs	Effective Detour Speed Mi/Hr
0.5	5
0.5 - 1.0	15
1.0 - 2.0	25
2.0	40

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APPENDIX A

DATA SUMMARY FOR CASE STUDIES

TABLE A - 1. INTERSTATE - 85 DATA

TABLE A - 2. THE GLADE DATA

TABLE A-1

INTERSTATE - 85 DATA

Location: Interstate - 85 Crossing over A Branch of the Great Creek, Brunswick County, Virginia.

Geometric Data:

Road Width	108	(ft)
Road Length	600	(ft)
Fill Height Above Culvert	14.63	(ft)

Embankment Slopes

Upstream	2:1	(ft/ft)
Downstream	2:1	(ft/ft)

Center Line Elevations

Culvert is located at Station 3+05

Station	Roadway Elevation	Grade Elevation
0+25	260.10	260.10
0+50	260.05	258.30
1+00	260.01	251.60
1+50	260.03	247.20
2+00	260.11	246.80
2+50	260.24	246.50
3+00	260.42	246.50
3+05	260.43	245.80
3+50	260.67	246.80
4+00	260.96	247.00
4+50	261.32	247.25
5+00	261.73	247.10
5+50	262.19	253.00
6+00	262.71	259.10
6+50	263.00	263.00

Hydrologic Data

Triangular Hydrographs

Hydrograph Number	Peak Flow (cfs)	Probability of Yearly Occurrence
1	439	.4800
2	730	.3300
3	1003	.0600
4	1220	.0200
5	1395	.0100
6	1565	.0050
7	1995	.0035

Time Factors Applicable to All Hydrographs

Time to Peak	2.5	(hrs)
Time From Peak to End of Flood	4.0	(hrs)
Flood Duration	6.5	(hrs)

Hydraulic Data

Manning's Roughness Coefficients:

Culvert	.012	
Natural Watercourse	.050	
Fill Slope	.030	
Slope of Culvert	.010	(ft/ft)

Hydraulic Date (continued)

Stage Versus Discharge

Stage (ft)	Natural Watercourse Discharge (cfs)
0	0
3	229
4	819
5	1660
6	2880
7	4500
8	6450

Stage Versus Storage

Stage (ft)	Storage (acre-ft)
0	0
3	7
13	112
23	423
33	1107
43	2303
53	4167

Cost Data

Fill Volume	.47	($/cy)
Excavation Volume	2.66	($/cy)
Roadway (guardrail to guardrail)	57.72	($/ft)
Steel	.15	($/lb)
Concrete	57.70	($/cy)
Unit Quantities: Steel and Concrete are taken from Highway Culvert Design Standards.		
Amortization Period	100	(yrs)
Interest Rate	6.5	(%/yr)

Loss Data

Repair Factor		
Ratio of Repair Cost to Initial Cost	2.0	
Traffic		
Average Daily Traffic	16,000	(veh/day)
Vehicle Distribution		
Passenger car	.85	(FADT)[1]
Commercial delivery vehicle	.01	(FADT)
Single unit truck	.02	(FADT)
Gasoline semi trailer	.03	(FADT)
Deisel semi trailer	.09	(FADT)

[1] Fraction of Average Daily Traffic.

Loss Data (continued)

Occupancy Rate	1.7	(people/veh)
Travel Distance		
Normal Route	4.0	(miles)
Detour	5.2	(miles)
Speed		
Normal Route	70	(mph)
Detour	55	(mph)
Death Rate Under Normal Driving Conditions	5.5	(deaths/100 Million miles)
Accident Distribution Ratios For Normal Driving Conditions		
Personal Injuries Per Death	30	
Property Damage Per Death	300	
Accident Costs		
Death	50,000	($)
Personal Injuries	2,000	($)
Property Damage	400	($)
Value of Lost Time	2	($/hr)
Running Costs		

$$C=42.2 - 0.455S + 0.0068S^2$$

where

C = running cost of passenger car $/1000 miles
S = speed of vehicle - mph

Loss Data (continued)

Ratio of Unexpected Obstacle Death Rate to Normal	
Death Rate	1,000
Accident Distribution Ratio for Unexpected Obstacles	
Personal Injuries Per Death	15
Property Damage Per Death	150

Erosion Relationships

Predictive Equations

$$E = \alpha v^{\beta}$$

where

E = Erosion tons of fill per day per ft of roadway
v = flow velocity, fps
α = empirical constant = 0.25
β = empirical constant = 3.80

Loss Data (continued)

Road Repair, Culvert Repair, and Traffic Restoration Time vs Embankment Erosion

Embankment Erosion Per Cent of Total Embankment	Road Repair Per Cent of Original Cost	Culvert Repair Per Cent of Original Cost	Traffic Restoration Time (days)
0	0	0	0
10	50	0	0
20	100	0	3.33
30	100	0	6.66
40	100	0	9.99
50	100	0	13.32
60	100	0	16.65
70	100	0	19.98
80	100	0	23.31
90	100	0	26.64
100	100	100	30.00

Stage - Damage Curve

Stage Above Culvert Invert (ft)	Flood Loss $
0	0
1	0
3	0
5	148
7	435
9	738
11	1041
13	1344
20	1344

TABLE A-2

THE GLADE DATA

Location: Twin Bridges Road Over The Glade, Fairfax County, Virginia

Geometric Data:

Road Width	54	(ft)
Road Length	634	(ft)
Fill Height Above Culvert	53	(ft)
Embankment Slopes		
Upstream	3:1	(ft/ft)
Downstream	2 1/2:1	(ft)

Center Line Elevations

Culvert located at station 15+86.4

Station	Design Elevation	Existing Elevation
11+50	344.25	344.25
12+00	341.25	337.50
12+50	338.25	328.50
13+00	335.25	317.90
13+50	332.25	306.00
14+00	329.35	294.00
14+50	327.15	282.50
15+00	325.75	272.00
15+50	325.15	272.40
15+86.4	325.22	273.00
16+00	325.35	274.50
16+50	326.25	281.00
17+00	327.18	296.25
17+50	328.18	315.00
17+84	328.86	328.86

Hydrologic Data

Triangular Hydrographs

Hydrograph Number	Peak Flow (cfs)	Probability of Yearly Occurrence
1	697	0.405
2	995	0.330
3	1226	0.060
4	1403	0.020
5	1693	0.010
6	2118	0.005
7	2790	0.0035
8	3266	0.00055
9	3746	0.00050

Time Factors Applicable to All Hydrographs

Time to Peak	2.5	(hrs)
Time Peak to Return	4.0	(hrs)
Base Time	6.5	(hrs)

Hydraulic Data

Manning's Roughness Coefficients:

Culvert	0.012
Natural Watercourse	0.090
Fill Slope	0.030

Slope of Culvert	0.010

Hydraulic Data (continued)

Stage (above Culvert Invert) Relationships

Stage (ft)	Natural Watercourse Discharge (cfs)
0	0
2	32
4	204
6	604
8	1302
10	2362
12	3843
14	5800

Stage Versus Storage

Stage (ft)	Upstream Storage (af)
0	0
5	1
15	11
25	66
35	230
45	600
55	1368
60	1867

Cost Data

Fill Volume	1.00	($/y3)
Excavation Volume	8.00	($/y3)
Roadway (guardrail to guardrail)	17.00	($/ft)
Steel	0.18	($/Lb)
Concrete	125.	($/y3)
Unit Quantities:		
Steel	Taken From Highway Culvert Design Standards	(Lb/ft)
Concrete		(cy/ft)
Amortization Period	100	(yrs)
Interest Rate	6.5	(%/yr)

Loss Data

Parameters		
Construction		
Ratio of Repair Cost to Initial Cost	1.50	
Traffic		
Average Daily Traffic	466	(veh/day)
Vehicles (Fraction of ADT)		
Passenger car	.955	(FADT)[1]
Commercial delivery vehicle	.043	(FADT)
Single unit truck	.000	(FADT)
Gasoline semi trailer	.002	(FADT)
Diesel semi trailer	.000	(FADT)

[1]Fraction of Average Daily Traffic.

Loss Data (continued)

Occupancy Rate	2.00	(people/veh)
Travel Distance		
Normal Route	1.56	(miles)
Detour	2.97	(miles)
Speed		
Normal Route	25	(mph)
Detour	25	(mph)
Death Rate Under Normal Driving Conditions	5.5	(deaths/100 Million Miles)
Accident Distribution Ratios For Normal Driving Conditions		
Personal Injuries Per Death	15	
Property Damage Per Death	150	
Accident Costs		
Death	50,000	($)
Personal Injuries	2,000	($)
Property Damage	400	($)
Value of Lost Time	2	($/hr)

Running Costs

$$C=42.2 - 0.455S + 0.0068S^2$$

where

C = running cost of passenger car $/1000 miles
S = speed of vehicle - mph

Ratio of Unexpected Obstacle Death Rate to Normal Death Rate	1,000

Loss Data (continued)

Accident Distribution Ratio For Unexpected Obstacles

Personal Injuries Per Death	15
Property Damage Per Death	150

Erosion Relationships

Predictive Equations

$$E = \alpha v^{\beta}$$

where

E = Erosion tons of fill per day per ft of roadway
v = Flow velocity, fps
α = Empirical constant = 0.25
β = Empirical constant = 3.80

TABLE A-2, Continued

Loss Data (continued)

Road Repair, Culvert Repair, and Traffic Restoration Time vs Embankment Erosion

Embankment Erosion Per Cent of Total Embankment	Road Repair Per Cent of Original Cost	Culvert Repair Per Cent of Original Cost	Traffic Restoration Time (days)
0	0	0	0
10	50	0	0
20	100	0	3.33
30	100	0	6.66
40	100	0	9.99
50	100	0	13.32
60	100	0	16.65
70	100	0	19.98
80	100	0	23.31
90	100	0	26.64
100	100	100	30.00

Stage - Damage Curve

Stage Above Culvert Invert (ft)	Flood Loss $
0	0
26	0
27	8,730
32	78,760
37	232,550
42	395,450
47	505,450

APPENDIX B

DEVELOPMENT OF
UNIT FLOOD LOSSES

TABLE B-1

FLOOD LOSS DAMAGE FOR AGRICULTURE

Variable	Description	Equation	Reference
X_1	Selected Virginia counties	DATA	
X_2	Cropland harvested (acre)	DATA	1
X_3	Irrigated land (acre)	DATA	2
X_4	Cropland in fruits & nuts (acre)	DATA	3
X_5	Value of field crops sold other than vegetables, fruits & nuts - 1964	DATA	4
X_6	Value of vegetables sold	DATA	5
X_7	Direct flood damage to field crops at various water depths - % of total value of crops	DATA	6
X_8	Indirect flood damage to field crops % of total value of crops	DATA	7
X_9	Interest at an average rate of 5% accumulated for 6 years (1964-1970)	DATA	8,a
X_{10}	Total land under cultivation (acres)	X_2+X_3	--
X_{11}	Total land under cultivation less cropland in fruit & nuts	$X_{10}-X_4$	--
X_{12}	Total value of crops sold including vegetables	X_5+X_6	--
X_{13}	Total value of field crops sold per acre in 1964	X_5/X_{11}	
X_{14}	Average $ value of direct damage per acre to field crops at various water depths	$X_7 \cdot X_{13}$	--
X_{15}	Total average $ value of direct and indirect damage per acre to field crops at various water depths	$(100+X_8) \cdot X_{14}$	--
X_{16}	Increased values of damage per acre to field crops from 1964-1970 at various water depths	$X_9 \cdot X_{13}$	
X_{17}	Total average $ damage per acre to field crops in 1970 at various water depths	$X_{15}+X_{16}$	--

a. This accumulated rate, based on the approximate average annual interest rate paid on long-term U.S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in the value of field crops, and therefore in the losses to such crops damaged by floods, during the 6-year period, 1964 - 1970.

Sources:

1. <u>1964 United States Census of Agriculture - Virginia</u>, Vol. 1, Part 24, Statistics for Counties, Table 1, Line 17.

2. <u>Ibid</u>., Table 1, Line 74.

3. <u>Ibid</u>., Table 13, p. 424 ff, Line 69.

4. <u>Ibid</u>., Table 5, Line 65.

5. <u>Ibid</u>., Table 5, Line 66.

6. These figures are Resources Development Associates estimates based largely on information contained in the <u>Economic Guide for Watershed Protection</u> and Flood Prevention (Chapter 3), published by the Soil Conservation Service of the U. S. Department of Agriculture, March 1964.

7. <u>A Time-Dependent Planning Process for Combining Structural Measures, Land Use, and Flood Proofing to Minimize the Economic Cost of Floods</u>, James, L.D., a Project on Engineer - Economic Planning, Stanford University, August 1964, p. 22.

8. Federal Reserve Bulletin.

TABLE B-2

FLOOD LOSS DATA FOR MANUFACTURING

Variable	Description	Equation	Reference
X_1	Industry groups	DATA	a
X_2	Number of establishments	DATA	1
X_3	Production workers	DATA	1
X_4	Value added by manufacturing in 1963	DATA	1
X_5	Value added by manufacture in the U. S. - 1966	DATA	2
X_6	Manufacturers' inventories in the U. S. - 1966	DATA	3
X_7	Interest at an average rate of 5% accumulated for 7 years	DATA	4,c
X_8	Assessed valuation of industrial property in Va. - 1966	DATA	5
X_9	Ratio of assessed valuation of industrial property to the value of measurable sales in Va. - 1966	DATA	6,d
X_{10}	Distribution of industrial property in Va. on the basis of value added by manufacture in Va.	DATA	e
X_{11}	Interest at an average annual rate of 5-1/2% accumulated for four years	DATA	4,c
X_{12}	Estimated average direct damage in dollars per $1,000 value of manufac-manufacturing inventories and properties at varying water depths	DATA	7,f
X_{13}	Estimated average indirect damage to manufacturing inventories and properties expressed in terms of direct damage to inventories and properties	DATA	7,g
X_{14}	Average value added by manufacture per establishment in Va. - 1963	X_4/X_2	--
X_{15}	Manufacturers' inventories in the U.S. as a fraction of value added by manufacture - 1966	X_6/X_5	--
X_{16}	Estimated manufacturers' average inventories per establishment in Va. - 1963	$X_{14} \cdot X_{15}$	--

Variable	Description	Equation	Reference
X_{17}	Increased value of manufacturers' average inventories per establishment from 1963 - 1970	$X_7 \cdot X_{16}$	--
X_{18}	Total value of manufacturers; average inventories per establishment in 1970	$X_{16}+X_{17}$	--
X_{19}	Estimated total value of all industrial property in Va. - 1966	X_8/X_9	--
X_{20}	Estimated value of manufacturing properties in Va. by industrial groups - 1966	$X_{10} \cdot X_{19}$	--
X_{21}	Estimated average value of manufacturing properties per establishment in Va. by industrial groups - 1966	X_{20}/X_2	--
X_{22}	Increased average value of manufacturing properties per establishment in Va. by industrial groups from 1966 - 1970	$X_{11} \cdot X_{21}$	--
X_{23}	Total average value of manufacturing properties per establishment in Va. by industrial groups - 1970	$X_{21}+X_{22}$	--
X_{24}	Average estimated value of manufacturing inventories and properties per establishment in Va. - 1970	$X_{18}+X_{23}$	--
X_{25}	Estimated average direct damage of manufacturing inventories and properties per establishment for varying water depths - 1970	$X_{12} \cdot X_{24}$	--
X_{26}	Estimated average indirect damage of manufacturing inventories and properties per establishment for varying water depths - 1970	$X_{13} \cdot X_{24}$	--
X_{27}	Total estimated average direct and indirect damage of manufacturing inventoies and properties per establishment for varying water depths -1970	$X_{25}+X_{26}$	--
X_{28}	Total flood damage per dollar of value added by manufacture per establishment at varying water depths	X_{27}/X_{14}	--

Footnotes: (Generation of Manufacturing Flood Loss Table)

a. Arranged according to size of value added by manufacture in Virginia (see X_4).

b. A large figure, but it includes several large industries in the United States (steel, automobiles, etc.) that are not large in Virginia.

c. This accumulated rate, based on the approximate annual average interest rate period on long-term U. S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in values of manufacturers' inventories and properties for the periods 1963 to 1970 (for inventories), and 1966 to 1970 (for properties).

d. This figure is for industrial and commercial properties combined. Separate figure not given for industrial properties.

e. Resources Development Associates decided that the most reasonable procedure in distributing the $1,361,000,000 of industrial property valuations in Virginia (this is a total figure with no breakdowns for industrial groups) is on the basis of value added by manufacture for each of such groups in Virginia as set forth in X_4 .

f. In the source referred to, this figure was $42. Referring, however, to the graphic plot of points (Fig. 23, p. 62) from which this figure was derived, a more reasonable figure for 4', based on the curve as drawn, is $120.

g. These figures are derived from a curve drawn through the points representing the percent of <u>indirect</u> to <u>direct</u> damages obtained from Table 2, p. 53 of the source as mentioned.

Sources: (Generation of Manufacturing Flood Loss Table)

1. Census of Manufacturers 1963, Bureau of the Census, Table 5, pp. 47 - 9 - 11.

2. Annual Survey of Manufacturers, 1966, Bureau of the Census, pp. 29-52.

3. Ibid., pp. 89-109.

4. Federal Reserve Bulletin.

5. Taxable Property Values, 1967 Census of Governments, Bureau of the Census, Table 4, p. 34.

6. Ibid., Table 9, p. 47.

7. A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, Stanford Research Institute, 1960, Table 2, p. 53, and Fig. 23, p. 62.

TABLE B-3

FLOOD LOSS DAMAGE FOR SINGLE FAMILY RESIDENCES

Variable	Description	Equation	Reference
X_1	Selected counties in Virginia	DATA	--
X_2	Owner occupied homes by value classification - 1960	DATA	1,a
X_3	Medium points of estimated property values per home	DATA	2
X_4	Number of homes - 1960	DATA	1
X_5	Medium average estimated property value per home - 1960	DATA	1
X_6	Estimated average market value of contents as % value of home	DATA	3
X_7	Estimated average percent of direct damage to structure and contents at varying water depths	DATA	4
X_8	Estimated average percent of indirect damage to structure and contents at varying water depths	DATA	5
X_9	Interest at an annual average rate of 5% accumulated for 10 years	DATA	6,b
X_{10}	Average structure value of home; property value less estimated lot value	$.90 \cdot X_3$	c
X_{11}	Average market value of contents of home	$X_6 \cdot X_{10}$	--
X_{12}	Estimated average market value of structure and contents of home	$X_{10}+X_{11}$	--
X_{13}	Estimated average market value of contents of home in basement	$.10 \cdot X_{11}$	d
X_{14}	Average market value of structure and contents of home excluding basement contents	$X_{12}-X_{13}$	--
X_{15}	Estimated average direct damage to structure and contents for varying water depths	$X_7 \cdot X_{14}$	--
X_{16}	Average direct damage to structure and contents at varying water depths plus complete loss of basement contents	$X_{13}+X_{15}$	--

Variable	Description	Equation	Reference
X_{17}	Average indirect damage to structure and contents per home at varying water depths	$X_8 \cdot X_{16}$	--
X_{18}	Total average damage to structure and contents per home in 1960	$X_{16}+X_{17}$	--
X_{19}	Increased value of direct and indirect damage to structures and contents per home from 1960 to 1970	$X_9 \cdot X_{18}$	--
X_{20}	Total average of direct and indirect damage to structures and contents per home in 1970 at varying water depths	$X_{18}+X_{19}$	--
X_{21}	1960 average home value increase	$X_9 \cdot X_{10}$	--
X_{22}	1970 average home value	$X_{10}+X_{21}$	--

a. Based on each home owner's estimate of the value of his property.

b. This accumulated rate, based on the approximate annual average interest rate paid on long-term U. S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in values of residential structures and contents for the 10-year period, 1960 - 1970. (Figures on residential property values for 1970 should become available in early 1972, in the U. S. Census of Housing for 1970.)

c. This is Resource Development Associates' estimated structure value of the home, equal to the property values as given in Column C, less 10% for the value of the lot which is generally not severely damaged by flooding. In short, these figures are 90% of those in X_3.

d. Resource Development Associates' estimate. Contents of home in basement are considered a total loss if property is flooded to slightly above ground level or more.

Sources: (Generation of Residential Flood Loss Table)

1. United States Census of Housing, 1960, Virginia, State and Small Areas, Bureau of the Census, U. S. Department of Commerce, Tables 17 and 30. (Fairfax County, p. 48-56, Fairfax City, p. 48-66; Roanoke County, p. 48-55; Brunswick County, p. 48-97; Caroline County, p. 48-98; Nelson County, p. 48-102; Tazewell County, p. 48-104; Virginia Beach City, p. 48-53).

2. Simple arithmatic by Resource Development Associates, plus RDA's estimate of median points for properties valued at "less than $5,000" and "35,000 or more."

3. A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, by Homan, A., Gerloff, and Waybur, Bruce (Stanford Research Institute, January 1960), Fig. 4, p. 35.

4. Ibid., Fig. 2, p. 33.

5. Ibid, Fig. 9, p. 40.

6. Federal Reserve Bulletin.

TABLE B-4

FLOOD LOSS DAMAGE FOR RETAIL BUSINESSES

Variable	Description	Equation	Reference
X_1	Selected Virginia Counties	DATA	--
X_2	Kind of business group	DATA	--
X_3	Number of establishments	DATA	1,a
X_4	Annual Sales	DATA	1,a
X_5	Annual Sales to inventories (at cost) ratios	DATA	2,b
X_6	Average value of structure as percent of inventory valuation at cost per establishment	DATA	3,d
X_7	Estimated average direct damage in dollars per $1000 value of contents & structures at varying water depths	DATA	4
X_8	Estimated average indirect damage to contents & structures expressed in percent of direct damage to such contents & structure	DATA	5e
X_9	Interest at an average annual rate of 6% accumulated for 3 years	DATA	f
X_{10}	Average Annual sales per establishment	X_4/X_3	--
X_{11}	Average inventories per establishment at cost	X_{10}/X_5	--
X_{12}	Average value of structure per establishment	$X_{11} \cdot X_6$	--
X_{13}	Average value of content & structure per establishment	$X_{11}+X_{12}$	--
X_{14}	Estimated average direct damage of contents & structure per establishment for varying water depths	$X_7 \cdot X_{13}$	--
X_{15}	Estimated average indirect damage of contents & structure per establishment for varying water depths	$X_8 \cdot X_{14}$	--
X_{16}	Total direct and indirect damage to contents & structure per establishment in 1967 at varying water depths	$X_{14}+X_{15}$	--

TABLE B-4, Continued

Variable	Description	Equation	Reference
X_{17}	Increased value of direct & indirect damage per establishment from 1967 to 1970 at varying water depths	$X_9 \cdot X_{16}$	--
X_{18}	Total of direct & indirect damage to structures & contents per establishment in 1970	$X_{16}+X_{17}$	--
X_{19}	Total estimated flood damage per $1000 of annual sales per establishment	X_{18}/X_{10}	--

a. For all establishments, whether or not they had a payroll. (Those that did not usually were operated by one person, or a family.)

b. These are year-end inventories, at cost, and are national figures.

c. For 1966 (Statistical Abstract of the U. S, 1968, p.769)

d. These figures are based on a 1958 survey of 24 California cities under 30,000 population.

e. In the Indirect Flood Damage figures as cited in the footnote, the dollar figures have been converted by WRE to percents of Direct Flood Damage.

f. This accumulated rate, based on the approximate average annual interest rate paid on long-term U. S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in values of retail trade structures and contents for the 3 year period, 1967 to 1970. (Figures on retail trade property values for 1970 should become available in the latter part of 1972 in the U. S. Census of Business for 1970.)

TABLE B-4, Continued

Sources:

1. Bureau of the Census, U. S. Department of Commerce, 1967 Census of Business, Retail Trade, Virginia, Table 3, pp.48-8 to 48-15.

2. Bureau of the Census, U. S. Department of Commerce, Annual Sales, Year-End Inventories, and Accounts Receivable of Retail Stores, by Kind of Business for 1967, Series: BR 13-67, Table 9, p.14.

3. Stanford Research Institute, A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, Supplementary Report, January 1960, Table 39, p.56.

4. Stanford Research Institute, A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California, Basic Report, January 1960, Table 2, p.53; and Fig. 21, p.60.

5. Ibid, Table 2, p.53.

6. Federal Reserve Bulletin

TABLE B-5

FLOOD LOSS DAMAGE FOR SELECTED SERVICES

Variable	Description	Equation	Reference
X_1	Types of Services	DATA	--
X_2	Number of Establishments	DATA	1
X_3	Annual Receipts (1963)	DATA	1
X_4	Assessed Valuation of Commercial property in Virginia in 1966	DATA	2,a
X_5	Ratio of the assessed valuation of commercial property in Virginia in 1966 to measurable sales	DATA	3,b
X_6	Distribution of selected service property valuations in Virginia in 1966 on the basis of annual receipts	DATA	--
X_7	Estimated average direct flood damage in dollars per $1000 value of properties at various water depths	DATA	4,c
X_8	Estimated average indirect damage to properties expressed in percent of direct damage at various water depths	DATA	5,d
X_9	Interest at an annual average rate of 5% accumulated for seven years (1963-70)	DATA	6,e
X_{10}	Average annual receipts per establishment	X_3/X_2	--
X_{11}	Estimated total value of all selected service properties in Virginia in 1966	X_4/X_5	--
X_{12}	Estimated value of selected service properties in Virginia by kinds of service	$X_6 \cdot X_{11}$	--
X_{13}	Estimated value of selected service peoperties per establishment in Va. in 1963	X_{12}/X_2	--
X^{14}	Average direct flood damage of selected service properties per establishment of various water depths	$X_7 \cdot X_{13}$	--

TABLE B-5, Continued

Variable	Description	Equation	Reference
X_{15}	Average indirect flood damage of selected service properties per establishment at various water depths	$X_8 \cdot X_{14}$	--
X_{16}	Total average direct & indirect flood damage of selected service properties per establishment at various water depths	$X_{14}+X_{15}$	--
X_{17}	Total average direct & indirect flood damage of selected service properties per $1000 of annual receipts per establishment (1963)	X_{16}/X_{10}	--
X_{18}	Average estimated direct & indirect flood damage to selected service properties per $1000 of annual receipts per establishment for various water depths in 1970 value	$(100+X_9)\cdot X_{17}$	--

a. Total valuation of commercial properties was $1,262,000,000. 4.5% of this figure, or $56,790,000 is estimated by RDA to represent selected service properties, on the basis of the percent of selected service annual receipts in Virginia to the total of retail, wholesale, and selected services sales-- all of which are considered as commercial properties.

b. This figure is for commercial and industrial properties combined. Separate figures are not given.

c. Damage figures for 3' and 4' taken from Fig. 22, p.61 of study referred to under sources

d. In the indirect flood damage figures, as cited in the footnote, the dollar figures have been converted by WRE to percents of Direct Flood Damage.

e. This accumulated rate, based on the approximate average annual interest rate paid on long-term U. S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in average value of selected service properties for the 7 year period, 1963 to 1970. (The 1967 Census of Business, Selected Services, for Virginia should be available in the Spring of 1970.)

Sources:

1. 1963 Census of Business, Selected Services, Virginia, Table 1, p.48-5.

2. 1967 Census of Governments, Taxable Property Values, Vol. 2, Table 4, p.34.

3. Ibid, p.47.

4. A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California (Basic Report), Stanford Research Institute, January 1960, Table 2, p.53. Fig 22, p.61.

5. Ibid, Table 2, p.53

6. Current issues of the Federal Reserve Bulletin and/or the Survey of Current Business

TABLE B-6

FLOOD LOSS DAMAGE FOR WHOLESALE BUSINESSES

Variable	Description	Equation	Reference
X_1	Major kind of business	DATA	1
X_2	Number of establishments	DATA	1
X_3	Annual sales	DATA	1
X_4	Inventories	DATA	1,a
X_5	Assessed value of commercial property in Virginia - 1966	DATA	2,b
X_6	Ratio of the assessed valuation of commercial property in Virginia in 1966 to measurable sales	DATA	3,c
X_7	Estimated average direct flood damages in dollars/$1000 value of inventories and properties at varying water depths	DATA	4
X_8	Estimated indirect damage to inventories and properties	DATA	5,d
X_9	Interest at an average rate of 6% accumulated for 3 years (1967-1970)	DATA	6,e
X_{10}	Distributions of wholesale property valuations in Virginia on the basis of annual sales	DATA	---
X_{11}	Average annual sales per establishment	X_3/X_2	---
X_{12}	Average inventories per establishment	X_4/X_2	---
X_{13}	Estimated total value of all wholesale properties in Virginia - 1966	X_5/X_6	---
X_{14}	Estimated values of wholesale properties in Virginia by major kinds of businesses in 1966	$X_{10} \cdot X_{13}$	---
X_{15}	Estimated value of wholesale property per establishment in Virginia - 1966	X_{14}/X_2	---
X_{16}	Total value of inventory & property per wholesale establishment - Virginia 1966	$X_{12}+X_{15}$	---

Variable	Description	Equation	Reference
X_{17}	Direct flood damage of inventory & property per establishment at varying water depths	$X_7 \cdot X_{16}$	---
X_{18}	Estimated indirect damage of inventory & property per establishment at varying water depths	$X_8 \cdot X_{17}$	---
X_{19}	Total flood damage of inventory & property per establishment in Virginia - 1967	$X_{17}+X_{18}$	---
X_{20}	Increased value of flood damage per establishment from 1967 to 1970	$X_9 \cdot X_{19}$	---
X_{21}	Total flood damage of inventory & property per establishment in 1970	$X_{19}+X_{20}$	---
X_{22}	Total flood damage per $1000 of annual sales per establishment	X_{21}/X_{11}	---

a. Year end inventories.

b. Total valuation of commercial properties was $1,262,000,000. 47.7% of this figure, or $602,000,000, is estimated by Resources Development Associates (RDA) to represent wholesale properties, on the basis of the percent of wholesale annual sales in Virginia to the total of retail, wholesale, and selected services sales - all of which are considered as commercial properties.

c. This figure is for commercial and industrial properties combined. Separate figures are not given.

d. In the Indirect Flood Damage figures as cited in the footnote, the dollar figures have been converted by Water Resources Engineers (WRE) to percents of Direct Flood Damage. These figures are derived from a curve drawn through the points representing the percent of indirect to direct damages obtained from Table 2, p. 53 of the source as mentioned.

e. This accumulated rate, based on the approximate average annual interest rate paid on long-term U. S. Government securities, as reported in the Federal Reserve Bulletin, represents the probable increase in average values of wholesale trade inventories and properties for the 3 year period 1967 to 1970.

Sources: (Generation of Wholesale Trade Flood Loss Table)

1. 1969 Census of Business, Wholesale Trade, Virginia, Table 1, p. 48-5.
2. 1967 Census of Governments, Taxable Property Values, Vol. 2, Table 4, p. 34.
3. Ibid., p. 47.
4. A Study of Procedure in Estimating Flood Damage to Residential, Commercial, and Industrial Properties in California (Basic Report), Stanford Research Institute, January 1960, Fig. 23, p. 62.
5. Ibid., Table 2, p. 53.
6. Federal Reserve Bulletin.

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