Design of Encroachments on Flood Plains Using Risk Analysis

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PREFACE

The purpose of this circular is to provide procedures for the design of encroachments on flood plains using risk analysis. The application of risk analysis to the design of drainage structures allows the designer to select that design which will result in the least total expected cost to the public. The use of risk analysis in the design of drainage elements for transportation facilities is an evolving technology. The methodology presented in this manual should be viewed as a first step and users are encouraged to seek improved or new methods and uses for the application of risk analysis in drainage design.

The material is presented in sections which are arranged in the step-by-step order which should be followed in analyzing a proposed crossing. The last section is a sample report outline which can be used to organize any analysis performed using the procedure. Example problems which illustrate the procedure are included in the appendices.

ACKNOWLEDGEMENTS

The concepts in this circular are based on the research and development efforts in FHWA-TS-80-226, "Hydraulic Design of Bridges with Risk Analysis," by V. R. Schneider and K. V. Wilson of the U.S. Geological Survey and FHWA-RD-75-54, "Evaluation of Flood Risk Factors in the Design of Highway Stream Crossings," by M. T. Tseng, A. J. Knepp, and R. A. Schmalz.

A special acknowledgement is also due the Colorado Division of Highways for providing appendix D. This appendix, written by Mr. Lawrence E. Dezman, represents an application of the least total expected cost design procedure to a real world problem. As such, it provides valuable guidance for the implementation of the procedures presented in this manual.

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U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

THE DESIGN OF ENCROACHMENTS ON FLOOD PLAINS USING RISK ANALYSIS

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1.0 INTRODUCTION

The design of all flood plain encroachments should include an evaluation of the inherent flood related risks to the highway facility and to the surrounding property. When this evaluation indicates that the risk warrants additional study, a detailed analysis of alternative designs is necessary in order to determine that design with the least total expected cost (LTEC) to the public.

The purpose of this manual is to provide guidance in the application of the LTEC design decisionmaking process. The LTEC design process is basically one of optimization, where economic and engineering analyses of alternative designs provide the basis for decisionmaking.

An essential ingredient in the LTEC design concept is risk analysis. Risk analysis provides the vehicle for analyzing the losses incurred for the various design strategies due to possible states of natures (flood events). All quantifiable losses are included in the risk analysis. These may involve damage to structures, embankments, surrounding property, traffic related losses and scour or stream channel damage. The product of the risk analysis is the annual economic risk associated with each design strategy.

The sum of the annual economic risk and the annual capital costs, the total construction costs multiplied by a capital recovery factor, results in the total expected cost (TEC) for each design strategy. Comparison of the TEC's for all design strategies allows the designer to select the LTEC or optimum design.

Although, the emphasis in this manual is on bridge crossings, the LTEC decisionmaking process concept is applicable to other drainage features. For example, it may be utilized in the design evaluation of culverts, longitudinal encroachments, countermeasures and foundation elevations. Bridge, culvert and spur dike design problems are presented in appendices A, B, C and D.

1.1 LTEC and Traditional Design Concepts

Risk is defined as the consequences associated with the probability of flooding attributable to an encroachment. Therefore, regardless of the design process utilized, traditional or LTEC, there is a level of risk associated with every flood plain encroachment. The manner of assessing project risk and the way in which risk influences decisionmaking illustrate the basic differences between the LTEC and traditional design processes.

In the traditional design process, the level of risk is seldom quantified but is implied through the application of predetermined design standards, e.g. design frequency, backwater limitations, limiting velocity, etc. Its role in decisionmaking is implicit and limited to the degree that risk was considered in establishing the design standards.

By way of contrast, in the LTEC design process the level of risk is a product of the analysis and is a function of individual design and site characteristics. Risk is explicitly defined and the quantified levels of risk for all the design alternatives are key factors in the decisionmaking process.

1.2 Function of Design Elements

The implications and manner of involvement of several design elements change when LTEC decisionmaking process is employed. The role of the design discharge is, for example, one of the most difficult mental blocks to overcome in making the transition from traditional to LTEC design.

1.2.1 Design and Overtopping Discharges

In the traditional design process, the design discharge is a single valued input which is generally based on arbitary standards. The design discharge is used to determine structure size and the resulting backwater elevation. Freeboard and structural depth are then added to the backwater elevation, and, in some situations, the resulting elevation may determine the highway profile. The risk associated with this design configuration is then assessed using the base flood

Under risk analysis, a range of flood events is utilized in the analysis and the end products are the LTEC design and the overtopping discharge. Thus, the discharge associated with the selected design alternative is a product of the analysis not an input parameter. The overtopping discharge is by definition that discharge described by the probability of exceedance and water surface elevation at which flow occurs over the highway, over the watershed divide, or through structure(s) provided for emergency relief. Thus, the overtopping discharge must be accommodated by the LTEC design.

The base flood is simply another flood included in the range of discharges selected for the analysis. It probably will not be the flood associated with the LTEC design, nor the largest flood considered in the analysis.

1.2.2 Freeboard

In traditional design, providing embankment freeboard may be accomplished by either adding additional embankment elevation after the design headwater is determined or by selecting a design which results in a headwater below the desired embankment freeboard. In risk analysis, this procedure has no meaning, since it would simply add another trial design for consideration.

Providing freeboard to protect bridge structures from debris- and scour-related failures is, however, required by policy, where practicable. This may be accomplished after the LTEC design is selected by specifying the desired "low steel" elevation for the bridge structure.

Freeboard can be defined in this context as the positive difference between the elevation of "low steel" and the overtopping flood elevation.

2.0 PRELIMINARY ANALYSIS OF PROJECT RISK

The LTEC design process described in the following sections requires considerable expediture of resources. Therefore, the level of analysis should be commensurate with the economic risks involved. On the bottom of the risk scale, encroachments which have little or no risk associated with them can be designed using appropriate hydraulic procedures. High risk encroachments which create large economic risks should be designed using the techniques described in this circular. The process of determining which of these responses is appropriate is discussed in this section.

The determination of whether or not to design by the LTEC process can be viewed as a screening process (figure 2.1). All encroachments are assessed by comparing preliminary data to thresholds for each of the categories shown: lacks practicable detour, hazard to people, and hazard to property. If one or more of the threshold values are exceeded, the encroachment should be designed by the LTEC process. If the threshold values are not exceeded, the encroachment can be designed using traditional design methods.

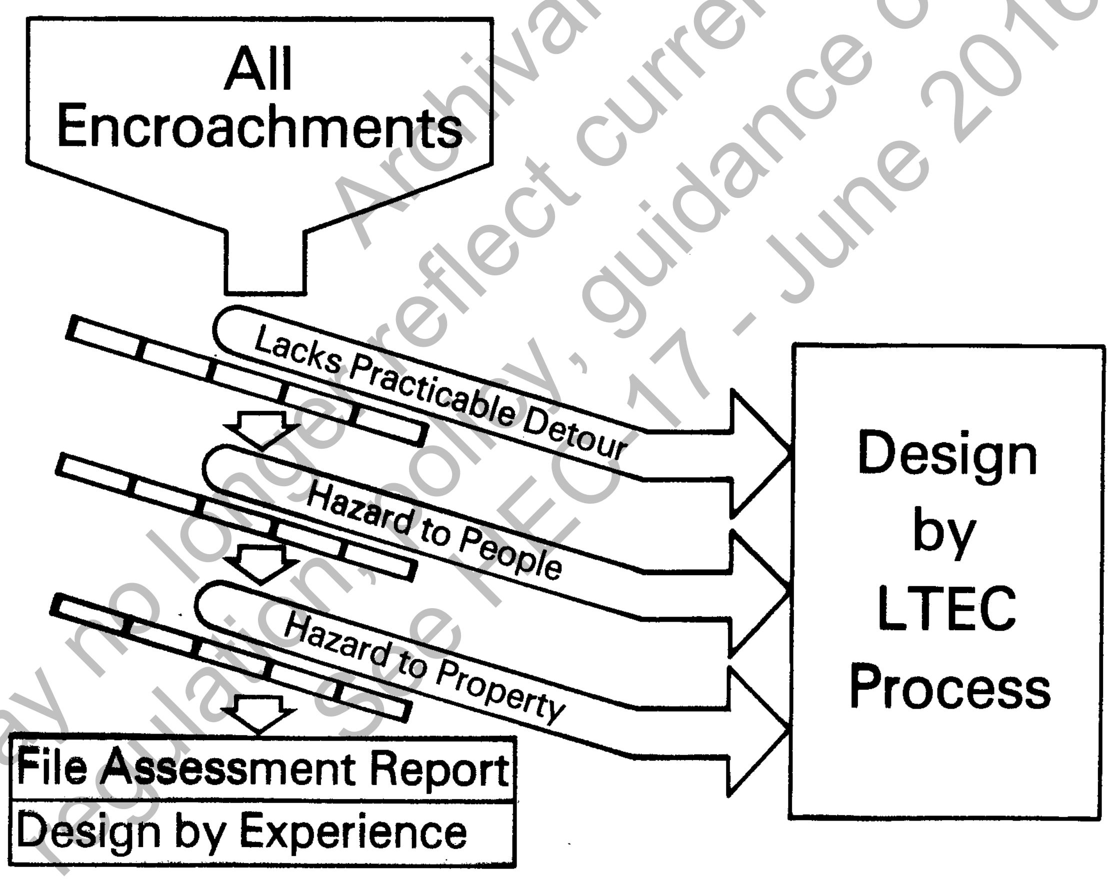

Figure 2.1 Design Risk Assessment

A form similar to figure 2.2 can be used to document the assessment or screening process. The form includes the three types of losses: traffic related, roadway, and backwater. Under each type of loss are the factors which will be used to assess that loss. For example, traffic losses are assessed indirectly by determining the number of vehicles which use the structure and evaluating how frequently, if ever, they will be delayed or detoured. This is done by estimating the overtopping flood probability for the alternative with the smallest structure and fill. If the roadway carries a large volume of traffic and will be frequently overtopped, the LTEC design process should be used. Similarly, the other losses can be estimated or calculated and compared to the thresholds. The thresholds must be established through the use of the LTEC design process; noting the values at which a factor yields little or no response.

If the assessment indicates that the LTEC process is appropriate, the encroachment can be further screened by estimating the magnitudes of the various risk costs and comparing these to the annual capital costs. This comparison is illustrated in Table 2.1. The procedure is based on an initial trial design which may be selected using traditional design concepts. The economic losses (roadway, traffic related, and backwater) are determined for the one design and their relative impact determined. If one or more of the economic losses are relatively small compared to the other losses, they may be assumed constant or ignored when analyzing the other alternatives.

Figure 2.2 RISK ASSESSMENT FOR ENCROACHMENT DESIGN

Project ____________________

Encroachment Location ____________________

Encroachment Design:
Case 1: Minimum Structure __________ Minimum Fill __________

Case 2: Minimum Structure __________ Maximum Fill __________

	Estimated Value for Encroachment	Threshold Value *
1. Hazard to People and Property:		
a. Traffic related losses		
1 ADT	______ VPD	______ VPD
2 Overtopping flood probability, Case 1	______ %	______ %
b. Roadway losses		
1 Embankment volume	______ CY	______ CY
2 Overtopping flood Probability, Case 1	______ %	______ %
c. Backwater related losses		
1 Land use		
a Number of residences	______	______
b Number of other buildings	______	______
2 Overtopping flood Probability, Case 2	______ %	______ %

2. Other Factors:
   a. Needed for emergency supply & evacuation route No ___ Yes ___
   b. Needed for emergency vehicle access No ___ Yes ___
   c. Lacks practicable detour No ___ Yes ___
   d. Encroachment on regulatory floodway No ___ Yes ___

If the answer to any of the other factors is yes or if any of the factors which indicate hazard to people or property exceed the threshold, analyze the encroachment using the LTEC design process or justify why it is not required.

* Thresholds to be determined by LTEC design experience

Table 2.1 — Preliminary analysis of flood plain encroachments

RR $	T.R $	B.W. $	A.C.C. $	Complete risk analysis
R.R. $	T.R. $	B.W. $	A.C.C. $	Ignore or assume constant backwater losses (B.W.)
R.R $	T.R. $	B.W. $	A.C.C. $	Ignore or assume constant traffic related losses (T.R.)
R.R. $	T.R. $	B.W. $	A.C.C. $	Ignore or assume constant roadway repair losses (R.R.)
R.R. $	T.R. $	B. W. $	A.C.C. $	Annual capital costs (ACC) principle factor — all losses negligible — select lowest, shortest acceptable alternative

Analysis based on first trial design

3.0 LTEC DESIGN PROCEDURE

Table 3.1 illustrates the decisionmaking process and the remaining sections of the manual contain detailed information on the various steps leading to the selection on the LTEC design.

Section 4.0 provides guidance on the selection of alternative designs and section 5.0 discusses limiting assumptions and other analysis considerations. Data collection and the hydrologic and hydraulic analysis, which is central to the entire LTEC procedure, is discussed in section 6.0. Sections 7.0 and 8.0 provide details on the mechanics of computing the economic losses and the total expected cost (TEC) for each design alternative. Section 9.0 discusses the selection of the least total expected cost (LTEC) design and section 10.0 contains a sensitivity analysis procedure to aid the designer in determining the relative significance of the variables in the decisionmaking process.

The LTEC design procedure is conceptually rather simple. Its application, however, does involve the acquisition and manipulation of a sizeable data base of information. The designer should have access to computer based techniques for the hydrologic and hydraulic computation, but "hand" calculation are recommended for the other computations until familiarity with the process is gained.

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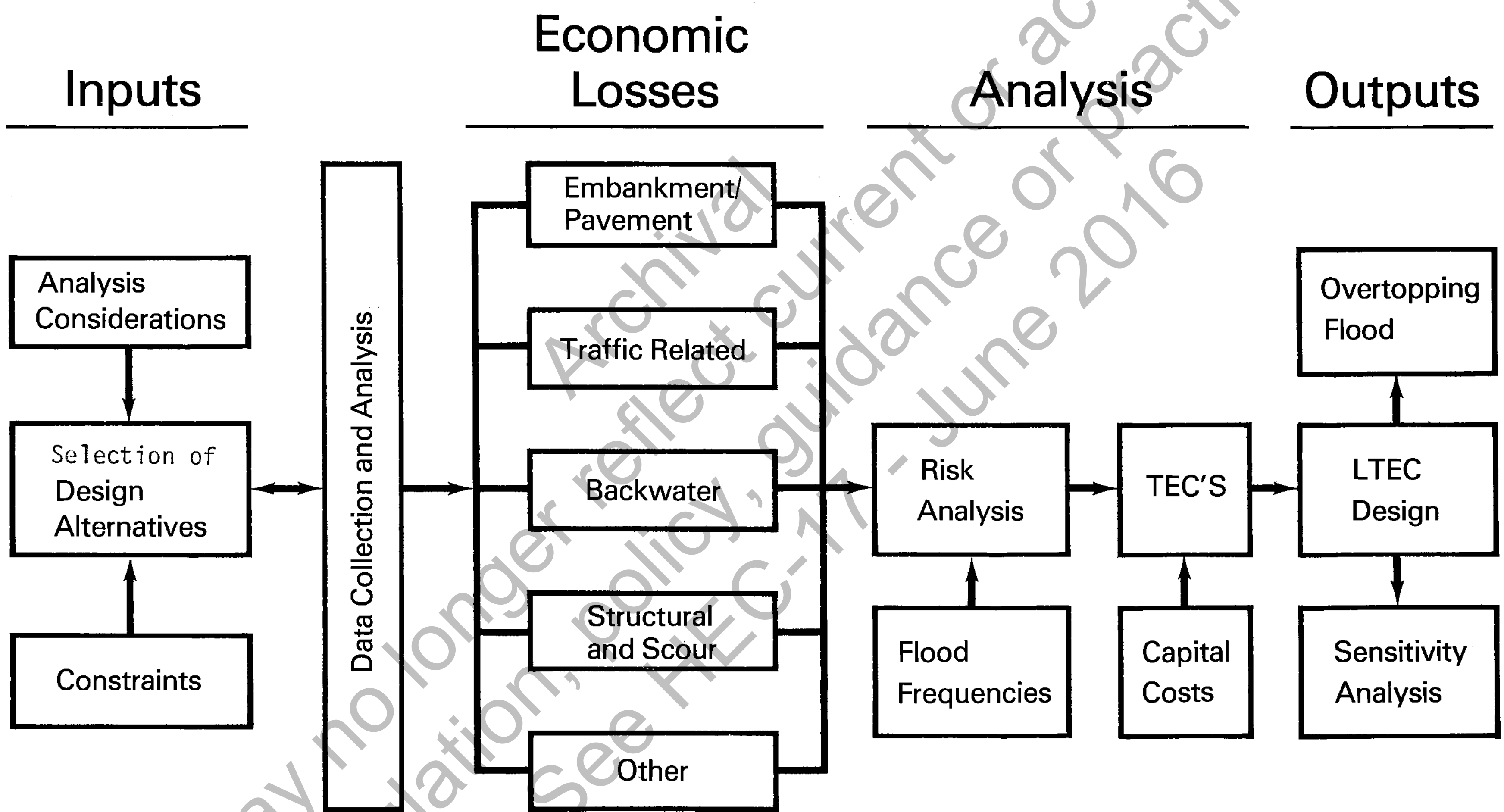
Table 3.1 — LTEC Design Decisionmaking Process



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4.0 SELECTION OF ALTERNATIVE DESIGNS

The first step in the selection process is to determine the range of practicable design alternatives. Constraints, in addition to engineering considerations, will frequently limit the available alternatives at a site. In some cases, these nonengineering constraints may severely limit the design alternatives available.

Some items which may limit the design are:

(1) Prescribed minimum design flood criteria as in the case of the Interstate.

(2) Limitations imposed by roadway geometrics such as maximum or minimum grade lines, site distance, vertical curvature, etc.

(3) Overtopping frequency of the adjoining roadway. In particular, that section of roadway involving the same watershed under consideration.

(4) Topographical features such as stream levees, elevation of the watershed divide, and clearances for highways or railroads which are bridged.

(5) Navigation clearance requirements.

(6) Flood plain ordinances or other legislative mandates limiting allowable backwater or encroachment on the flood plain.

(7) Channel stability considerations which would limit velocity or the amount of constriction.

(8) Ecological considerations such as may exist with wetlands or in other sensitive environments.

(9) Geological or geomorphic conditions or constraints including subsurface conditions.

(10) Social considerations including the importance of the facility as an emergency evacuation route in time of peril.

(11) Availability of funds to construct the facility. (This item may or may not be a consideration in a first appraisal but could ultimately govern the design selection.)

The second step in the alternative selection process is to determine the components of the various design strategies. These could include bridges, culverts, embankment, protective measures and so on, and each alternative may involve one or a combination of these components. There will also be control variables associated with each design strategy. These may include bridge length, embankment height, culvert size, degree of longitudinal encroachment and countermeasure parameters.

Consider, for example, a crossing of a wide flood plain, 2900 feet with a low water channel 440 feet wide. The alternative design strategies include a main channel structure and may include auxiliary (relief) structure(s), either culverts or smaller bridges. The control variables may be bridge length, embankment height and possibly culvert size. Also, some or all of the alternatives may require scour protection, thus requiring that additional control variables be considered.

In this illustration, the main channel opening could vary from a structure less than 440 feet in length to one 2900 feet long, the embankment height may vary from no embankment to the height controlled by the adjacent grade lines, in any desired increments. Auxiliary opening and scour protection schemes may involve one or more bridge lengths and/or culvert sizes and one or more protection systems for all or only some of the alternatives.

The Control variables are designated as follows: main channel bridge length as BL_1 --- BL_m, embankment height as EH_1 --- EH_n, auxiliary opening schemes as XO_1 --- XO_o, and scour protention schemes as SP_1 -- SP_p. The possible set of design alternatives becomes a four dimensional matrix best described by index notation, $A_{i,j,k,l}$ where i varies from 1 to n, j varies from 1 to m, k varies from 1 to o and l varies from 1 to p. If we assume n=4, m=4, o=4 and p=4 there will be 256 design alternatives in the analysis. However, if auxiliary opening schemes are varied in combination with bridge lengths, the analysis then looks like a two variable analysis as illustrated in table 4.1, and is manageable. Scour protection would be inherent in the design of the alternatives, but scour protection costs would still vary with the amount of flow constriction, the extent of over-embankment flow and the extent of auxiliary openings.

From the above example, it is apparent that the alternative selection process, especially where more than two control variables are involved, can rapidly assume unmanageable proportions. Seldom will the selection process be as open ended as in the previous example, however. In most cases, the selection of alternative design strategies will be governed by a set of control criteria based on previous experience, preliminary analysis or other constraints such as those discussed above.

The preliminary analysis described above establishes a starting point for the selection of design alternatives. This starting point may include a bridge length, an embankment height, and auxiliary openings and scour protection schemes. Other alternatives are generated by varying the control variables above and below the starting point. Auxiliary opening schemes may be varied in combinations with the main channel structures.

Most often the selection process will necessarily involve arbitrary control criteria (such as the criteria for providing scour protection). Because of this, the designer should, after selecting the LTEC design, review the criteria to determine if other design alternatives should be investigated.

TABLE 4.1 Design Alternatives for a Two Variable Analysis

BRIDGE/AUXILIARY OPENING COMBINATION	EMBANKMENT HEIGHT EH_1	EH_2	EH_3	- - -	EH_m
BL_1 & XO_k	$A_{1,1}$	$A_{1,2}$	$A_{1,3}$	- - -	$A_{1,m}$
BL_1 & XO_{k+1}	- - -	- - -	- - -	- - -	'
BL_2 & XO_k	- - -	- - -	- - -	- - -	'
BL_2 & XO_{k+1}	- - -	- - -	- - -	- - -	'
'	- - -	- - -	- - -	- - -	'
'	- - -	- - -	- - -	- - -	'
'	- - -	- - -	- - -	- - -	'
'	- - -	- - -	- - -	- - -	'
BL_n & XO_o	- - -	- - -	- - -	- - -	$A_{n,m}$

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5.0 ANALYSIS CONSIDERATIONS

Prior to developing procedures for selecting the LTEC design, it is necessary to discuss several limiting assumptions and the roles played by such key variables as the service life of the structure and the discount rate used in the analysis.

5.1 Limiting Assumptions

For bridge design, it is assumed in the analysis procedure that the bridge itself will not fail. In other words, the foundation and other critical components of the bridge are not anticipated to fail even during rare flood events. The construction costs are assumed to include allowance for designing against failure of these components. There may be damage to the bridge deck, to embankments and to scour protection measures all of which are included in the assessment of economic risks.

On the other hand, the assumption of failure should be included in the analysis for culverts and longitudinal flood plain encroachments. This will require defining failure criteria for these structures. Such criteria might be linked to degree of damage, overtopping flood magnitude, or structure size. Example B illustrates the analysis procedure and assumed failure criteria for a culvert design.

Another basic assumption is that no increase in loss of life occurs. Increased accident potential and damage are accounted for in the procedure.

The terminal or salvage value of the facility is also assumed to be zero.

5.2 Useful Life of Structure

Generally, highway properties are retired from service due to physical wear and tear or deterioration, inadequacy in load capacity or traffic volume capacity, general obsolescence and demand to make room for other improvements. Because of the uncertainties involved, the service life of highways varies over a considerable range. This variation is found not only between components of the highway such as earthwork, bridges and paving but also within components.

Generally, highway bridges and embankment have a service life in excess of 30 years. The service life of a highway pavement may be shorter. The service life for culverts can be estimated by application of procedures such as contained in reference 18.

In the LTEC design process, the construction or capital cost component is amortized over the service life of the structure. This is accomplished by multiplying the capital cost by the capital recovery factor, CRF. Since the CRF approaches the discount rate as the service life increases, examining these factors in a set of compound interest rate tables indicates that when the discount rate is 7 percent or greater there is not a significant change in the CRF when the service life is greater than 30 years.

Therefore, the service life tends to be a critical factor in the analysis when the discount rate (cost of capital) is relatively low and the service life or analysis period is relatively short.

5.3 Discount Rate

The discount rate is a factor to which the final result (LTEC design) is highly sensitive. Table 5.1 compares four alternate proposals with the service life of 20 years and discount rates of 4, 7, 10 and 12 percent. It should be noted in this evaluation, that the higher the discount rate, the higher the annual charge for capital recovery and the more the evaluation will tend toward alternatives with the least initial investment.

The Water Resources Council (WRC), an independent executive agency of the United States Government, is required to annually publish the discount rate to be utilized by Federal agencies in plan formulation and evaluation of water and related land resources projects. This rate is used for the purpose of discounting future benefits and computing costs, or otherwise converting benefits and costs to a common time base. The WRC rate is based on the average yield during the preceeding year on interest-bearing marketable securities of the United States which, at the time the computation is made, have terms of 15 years or more remaining to maturity. This average for fiscal year 1980 is 8 1/4 percent. The WRC, however, can not raise nor lower its discount rate by more than one-quarter of one percent for any year, and since the rate for fiscal 1979 was 6 7/8 percent, the published WRC discount rate for fiscal year 1980 is 7 1/8 percent.

The user may wish to utilize the WRC discount rate, a rate based on the current cost of borrowing money in a particular locality or some other value. The user should keep in mind that the cost of borrowing money is governed somewhat by perceived inflation in the economy. Since, the discount rate is simply a means of converting costs to a common time frame, the selection of the discount rate should be consistent with the method of estimating annual maintenance and user costs. If present prices are used to estimate those annual costs which will occur some time in the future, then the discount rate, which is used to convert initial investments to annual costs, must be relatively free of inflation.

TABLE 5.1

Alternative Project	Initial Investment	Annual Maintenance and Users Costs	Total Equivalent Annual Costs for n=20 yrs. (Initial Investment x CRF + Annual Costs)			
			i=4%	i=7%	i =10%	i=12%
1	$35,000	$4,550	7125	7853	8661	9236
2	50,000	2,660	6339	7380	8533	9354
3	65,000	1,490	6272	7626	9124	10,192
4	80,000	1,375	7261	8926	10,772	12,085

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6.0 DATA COLLECTION AND ANALYSIS

6.1 Data Collection

Data required for the LTEC design process can be grouped into the following general categories:

(1) land use

(2) flood plain geometry

(3) hydrologic and hydraulic

(4) geologic

(5) capital costs

(6) traffic

(7) repair costs

The first category is a new data requirement which could result in a tremendous burden on field survey crews if expedient data collection methods are not developed. Categories 2, 3 and 4 are data presently collected for drainage design. The last three categories of data are generally available in highway agencies but are not traditionally utilized in drainage design; thus, these data are being put to a new use.

The specific requirements and accuracy guidelines for each category are described below.

6.1.1 Land Use Data

Land use data needs upstream of the crossing include the location and first floor elevation of all buildings and location, area and average ground elevation of all crops. Locations need only be accurate enough to determine where the particular land use is with respect to the cross sections used in the hydraulic analysis and can usually be estimated from a quadrangle map. First floor elevations for buildings should be accurate to the nearest foot. Aerial surveys might suffice for this accuracy since roof elevations can probably be established and a drive-by ground inspection could establish the approximate distance from the roof to the first floor. Other expedient methods of establishing first floor elevations include the use of developers plans or street elevations in conjunction with a drive-by ground inspection to establish approximate distances from the street to the first floors. For example problem A in this manual, field survey crews established first floor elevations to the nearest 0.1 ft, but that accuracy is not warranted in most cases and the added burden on survey crews is likely to introduce a bottleneck in the design process. Ground elevations for crops can be established from aerial survey contour maps. Crop types should be grouped by their sensitivity to incremental flooding. Grain and hay cropland could be treated as one group, while produce cropland would be a separate group. Normally woodland can be ignored in risk analysis.

In addition to locations and elevations, property values need to be established. Right-of-way sections in highway agencies are good sources of information on property and building values. State Departments of Agriculture and the State offices of the U.S. Soil Conservation Service are excellent sources for crop value information.

Residences are assumed to have contents worth one-half the value of the building. Crops are valued at market-value less harvesting and hauling costs. In other words, the value to be used in risk analysis is the value of a mature crop in the field.

Land-use data should be summarized in a systematic format. Two techniques of tabulating land use data may be used: (1) assume property lies halfway between cross-sections or (2) assume property lies on the closest cross-section. The latter technique is probably preferable unless all the property of interest happens to fall between two cross sections. For a given cross section, all similar buildings at a predetermined elevation are grouped into one unit.

The upstream and lateral extent of land use data needed can be approximated by a preliminary hydraulic analysis considering the smallest bridge, highest embankment and largest flood.

6.1.2 Flood Plain Geometry

Flood plain geometry can be described by several cross sections upstream and downstream of the crossing and by 1-2 foot interval contour maps. Typically there should be six or seven cross sections that cover a reach of the flood plain approximately 1 valley width downstream to 2 or 3 valley widths upstream of the crossing. Contour maps can be developed from aerial and/or ground surveys.

6.1.3 Hydrologic and Hydraulic Data

6.1.3.1 Flow Magnitudes

Data from gaging stations upstream and downstream of the crossing and associated drainage areas should be assembled.

6.1.3.2 Hydrographs

A family of stage hydrographs is needed to estimate embankment overtopping times. Usually a complete family of measured hydrographs will not be available, but measured hydrographs for one or two floods can be used as patterns for other floods. If no measured hydrographs are available, synthetic hydrographs can be used.

6.1.3.3 Flow Resistance

Mannings roughness ("n") values must be determined at each cross section for the flood plain and main channel. USGS Water Supply Paper 1849 has good guidelines for estimating "n" values for main channels. The USGS is currently conducting a study for FHWA in Louisiana to develop procedures for estimating "n" values for heavily vegetated flood plains. Several hydraulic texts have estimated "n" values for various crops. Some typical "n" values are tabulated below.

Table 6.1 Typical "n" values for Flood Plains

Flood Plain Cover	Values of n*
Corn	.06 - .07**
Small Grain	.06 - .10
Pasture	.04 - .05
Brush & Waste	.08 - .12

* Values tabulated from Chow page 104 (23)
** Higher values of "n" generally relate to lower depths of flow

6.1.4 Geologic and Soils Data

6.1.4.1 Channel Morphology

Sequential aerial photographs should be examined to determine susceptibility of the channel to lateral migration and degradation.

6.1.4.2 Soils Information

Soil samples should be obtained and analyzed for grain size distribution and cohesive properties. Resistance to penetration and soil profiles should also be obtained.

6.1.4.3 Scour History

Past observations of bridge scour should be used to enhance engineering judgement in making scour predictions for various bridge alternatives.

6.1.5 Construction Costs

Construction cost estimates for each alternative are necessary to the LTEC design process. Construction costs include initial embankment, pavement and structural costs. Any maintenance costs which vary with alternatives should also be included. Estimates should include allowance for varying foundation depths to account for scour. Estimates should also allow for other scour protection measures such as spur dikes and riprap.

6.1.6 Traffic Data

Traffic data should include the following:

Design or projected ADT
Initial ADT
Traffic mix
Vehicle running cost, $/vehicle mile
Average occupancy rate, passengers/vehicle
Value of time, $/hr.
Length of normal route and detour routes
Average speed on each route
Fatality rate on each route, people per hundred million vehicle miles
Injury ratio, injuries/fatality
Property damage ratio, accidents/fatality
Unit cost of injuries, $/injury
Unit cost of property damage, $/accident

Detour routes must be identified along with the expected overtopping frequency. In other words detour routes may change as the flood frequencies vary. Also, the crossing being designed may be planned as a detour for other crossings and the ADT may be a function of flood frequency.

6.1.7 Embankment and Pavement Repair Costs

Although many highway agencies keep elaborate maintenance records, these records are not likely to be of much value in estimating repair costs. Repair costs must be estimated in much the same manner as construction costs. The repair operations must be conceptualized and related to standard construction operations when estimating costs.

To estimate repair time and cost, data should include the following:

Total volume of embankment, CY
Total area of pavement, SY
Rate of embankment repair, CY/day
Rate of pavement repair, SY/day
Unit cost of embankment repair $/CY
Unit cost of pavement repair $/SY
(assume density of asphalt pavement = 100 lb/SY/in.)
Adjustment cost for rapid repair
Mobilization cost, $

6.1.8 Summary of Data Requirements

Table 6.1 is a summary of the data requirements and includes an indication of units, possible sources and where the data are used in the analysis.

Table 6.1 Summary of Data Requirements

Data and Acceptable Accuracy	Units	Source	Where Used In Analysis
1. Buildings:			
a. 1st flr. elev, ± 1 ft	ft	Surveys	B.W. Damage
b. Value including contents, ± 10%	$	ROW Offices Realtors	" "
c. Locations, Nearest X-Sect.		Quad. Map	"
2. Crops:			
a. Area per contour interval	acre/contour elevation.	Aerial Photo. Quad. Map	" "
b. Value of mature crop in field	$/acre		"
c. Locations, Nearest X-Sect.			
3. Flood Plain Geometry:			
a. 5 or 6 X-section		Field Surveys	Hydraulic
b. Avg valley slope	ft/mi	Low Water Elev.	Analysis
c. Manning "n" values		at Gage Stations	
4. a. Gaging Records			
(1) Gaging data	sq mi	USGS, SCS	Hydro. Analysis
(2) Drainage areas		COE, WPRS*	For Gaged Sites
b. Watershed Parameters			
(1) Hydrophysiographic Zone		Ref (20)	Hydro. Analysis
(2) Isoerodent factor		Ref (20)	For Ungaged Sites
(3) Elevation Diff.	ft	Quad. Map	
(4) Drainage area	sq mi		
5. a. Hydrograph	Elev. vs time or Q vs time	USGS, SCS, COE, WPRS	Traffic Losses Embankment Damage
b. Hydrograph Factors			
(1) Length of longest watercourse, L	mi	Quad. Map	Traffic Losses
(2) Elev. difference	ft	Quad. Map	Emb. Damage
6. Soils properties			Construction Costs in Conjunction W/Scour Est.
a. Soil Types		Split Spoon Sample	
b. Grain Size Distr.		Sieve Anal.	
7. Scour History		Maintenance Records	" "
8. Channel Morphology		Sequential Aerial Photos	" "
9. Construction Costs		C & M Unit or Bridge Unit	Annual Capital Costs
10. a. Traffic Data:			Traffic Losses
(1) Initial ADT	Veh/day	Traffic and/or Planning Units	
(2) Design ADT	"	"	"
(3) Traffic Mix		"	"
(4) Vehicle Running Costs	$/mile	"	"

*WPRS is the U.S. Water and Power Resources Service, previously the U.S.Bureau of Reclamation

Table 6.1 Summary of Data Requirements (Cont'd)

Data and Acceptable Accuracy	Units	Source	Where Used In Analysis
10. (5) Occupancy Rate	Pas/Veh	Traffic and/or	Traffic Losses
(6) Value of Time	$/hr	Planning Units	"
(7) Length of:		"	"
Normal Route	miles		
Detour	miles		
(8) Avg. Speed on Each Route	mi/hr	"	"
(9) Fatality Rate on Each Route	Fatalities/ 100 mil. mi.	"	"
(10) Injury Ratio	Injury/Fatality	"	"
(11) Property Damage Ratio	Accident/ Fatality	"	"
(12) Unit Cost:		"	"
Injuries	$/Injury		
Accidents	$/Accident		
b. Traffic Loss per Hour of Detour	$/hr	"	"
11. Repair Data:			Repair Costs
a. Total Volume of Emb.Subject to Overtopping	CY	Plans	"
b. Total Area of Pavement Subject to Overtopping	SY	Plans	"
c. Mobilization Cost:	$	C & M Unit	"
(1) Dist. from Maint. Yard			
(2) Type of Equip.			
(3) In House or Contract			
d. Rate of Emb. Repair:	CY/day	C & M Unit	"
(1) Type of Equip.			
(2) Dist.to Borrow			
(3) Extent of Repair			
e. Rate of Pavement Repair	SY/day	C & M Unit	"
(1) Dist. to Supplier			
(2) Extent of Repair			
f. Unit Cost of Embankment Repair	$/CY	C & M Unit	"
g. Unit Cost of Pavement Repair	$/SY	C & M Unit	"
h. Max. Time Roadway Closed to Traffic	hr	Traffic/ C & M Unit	"
i. Adjustment Factor for Rapid Repair		C & M Unit	"
j. Length of Work Day	hr/day	C & M Unit	"

6.2 Hydrologic and Hydraulic Data Analysis

6.2.1 Flood Frequency

Gaging data is usually not available at a crossing. Gaging data at stations upstream and downstream of the crossing can be adjusted to obtain data for the crossing. This data is then input to a flood frequency distribution function to obtain probabilities for various discharges. The Log Pearson Type III distribution is a recommended distribution. Procedures for applying the Log Pearson Type III distribution are described in WRC Bulletin 17A (19).

For ungaged watersheds (usually smaller watersheds), a designer may have to rely on empirical methods such as the USGS Regional Analyses, or the Utah State method described in Report Nos. FHWA-RD-77-158 and 159 (20).

The USGS regional analyses typically include regression equations for various flood frequencies ranging from 2.33-year (the annual peak discharge) to 100-year. The analyses in several States include equations for 200 and 500-year frequencies. The Utah State method resulted from a national effort for FHWA. The method divides the nation into 24 hydrophysiographic zones as illustrated in figure 6.1. The method includes regression equations which are summarized in table 6.2, relating the 10-year flood to watershed parameters. The regression equations are in the form:

$$Q_{10} = C\ A^{e1}\ R^{e2}\ DH^{e3}$$

Where: C, e1, e2 and e3 are the regression coefficient and exponents
- A = Watershed area
- R = isoerodent factor which is related to the annual maximum 30-minute rainfall intensity
- DH = Elevation difference between the main channel at its most distant boundary and the drainage structure site

Other frequency floods are then related to the 10-year flood by the equation below:

$$Q_t = (a)\ Q_{10}^{b}$$

where:
- a = 0.46921 and b = 1.00243 for t = 2.33 yrs.
- a = 1.45962 and b = 1.02342 for t = 50 yrs.
- a = 1.64380 and b = 1.02918 for t = 100 yrs.

Application of the Utah State method should be limited to watersheds smaller than 50 square miles in the U.S. The Utah State method also includes an equation for the "probable maximum runoff peak" which is:

$$Q_p(max) = 10^{(3.92+.812(\log A)-.0325(\log A)^2)}$$

where: A = drainage area in square miles

Use of this equation should also be limited to small watersheds (less than 50-100 square miles). Q_p(max) is not related to a specific frequency flood and can be viewed as an order of magnitude indication for very large floods (say Q_{200} to Q_{500}) which should be included in a risk analysis.

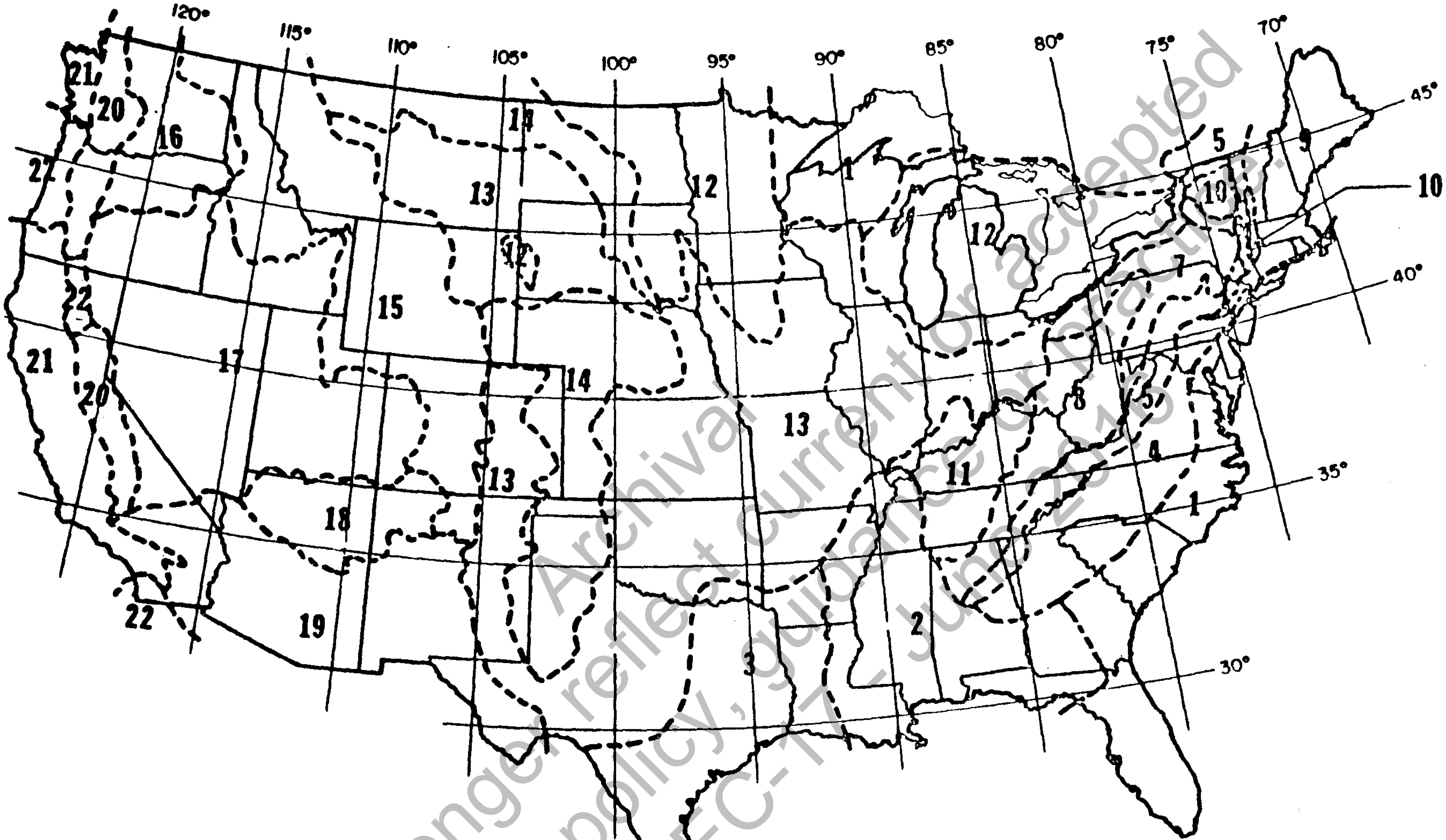


FIGURE 6.1 HYDROPHSYIOGRAPHIC ZONE MAP FOR THE CONTIGUOUS UNITED STATES
(FROM FLETCHER ET AL (20) FHWA REPORT RD-77-159, P 12)

TABLE 6.2 THE 3-PARAMETER REGRESSION EQUATION FOR EACH OF THE 24 HYDROPHYSIOGRAPHIC ZONES OF THE UNITED STATES AND PUERTO RICO. (FROM FLETCHER ET AL (20) FHWA REPORT RD-77-159, P8)

Zone	Equation
All Zone	$\hat{q}_{10} = 1.28015\ A^{0.56172}\ R^{0.94356}\ DH^{0.16887}$
1	$\hat{q}_{10} = 0.02137\ A^{0.43975}\ R^{1.16383}\ DH^{0.78453}$
2	$\hat{q}_{10} = 11.8893\ A^{0.57269}\ R^{0.44271}\ DH^{0.29510}$
3	$\hat{q}_{10} = 10410.4\ A^{0.54499}\ R^{-0.69141}\ DH^{0.32389}$
4	$\hat{q}_{10} = 76.7226\ A^{0.64795}\ R^{0.24744}\ DH^{0.03546}$
5	$\hat{q}_{10} = 1.14069\ A^{0.81060}\ R^{0.81127}\ DH^{0.16225}$
6	$\hat{q}_{10} = 10^{5.03658}\ A^{0.22735}\ R^{-2.07865}\ DH^{0.71475}$
7	$\hat{q}_{10} = 141.135\ A^{0.88572}\ R^{-0.13043}\ DH^{0.13981}$
8	$\hat{q}_{10} = 95.0775\ A^{0.58571}\ R^{0.07355}\ DH^{0.18493}$
9	$\hat{q}_{10} = 0.50051\ A^{0.69229}\ R^{0.74166}\ DH^{0.39729}$
10	$\hat{q}_{10} = 0.000613\ A^{1.30515}\ R^{3.28114}\ DH^{-0.54172}$
11	$\hat{q}_{10} = 1111.47\ A^{0.67899}\ R^{-0.76204}\ DH^{0.58914}$
12	$\hat{q}_{10} = 0.01961\ A^{0.47391}\ R^{1.68758}\ DH^{0.30700}$
13	$\hat{q}_{10} = 6.18115\ A^{0.66694}\ R^{0.87434}\ DH^{0.01023}$
or	$\hat{q}_{10} = 6.6082\ A^{0.67054}\ R^{0.87120}$
14	$\hat{q}_{10} = 0.00353\ A^{0.42562}\ R^{1.64552}\ DH^{0.82680}$
15	$\hat{q}_{10} = 412.131\ A^{1.00832}\ R^{-0.43497}\ DH^{-0.18943}$
16	$\hat{q}_{10} = 5.99340\ A^{0.69400}\ R^{0.81381}\ DH^{-0.02694}$
17	$\hat{q}_{10} = 41.2165\ A^{0.95643}\ R^{0.90116}\ DH^{-0.49291}$
18	$\hat{q}_{10} = 5399.80\ A^{0.61776}\ R^{-0.20988}\ DH^{-0.28469}$
19	$\hat{q}_{10} = 0.67503\ A^{0.44020}\ R^{1.26786}\ DH^{0.24140}$
20	$\hat{q}_{10} = 0.88267\ A^{0.94684}\ R^{1.01373}\ DH^{0.06857}$
21	$\hat{q}_{10} = 8.80096\ A^{0.90473}\ R^{0.44704}\ DH^{0.13937}$
22	$\hat{q}_{10} = 0.76272\ A^{0.69452}\ R^{0.85611}\ DH^{0.23777}$
23	$\hat{q}_{10} = 9687.77\ A^{0.99975}\ R^{0.16025}\ DH^{-0.58516}$
24	$\hat{q}_{10} = 12.8566\ A^{0.86854}\ R^{1.17343}\ DH^{-0.37794}$

6.2.2 Water Surface Profiles

One of the biggest impacts of the LTEC design process is the requirement for water surface profiles. Instead of one water surface profile, the LTEC design process requires water surface profiles for each discharge for existing conditions and for each alternative design condition.

Unfortunately, most of the existing computer programs are not geared to the LTEC requirements. Existing programs include the COE HEC-2 and the USGS E431. The FHWA program (HY-4) does not provide sufficient water surface elevation definition to be used for risk analysis. HEC-2 has a special bridge routine as well as a general bridge routine, but the general bridge routine is often more applicable to design problems than the special routine. The E431 program has incorporated the FHWA backwater procedure and should provide reliable water surface profiles upstream of a bridge. One problem with either program is that a tremendous amount of printout must be scanned to pick up a few key elevations that are necessary for risk analysis. Also, items like overtopping depths and velocities are not outputs of these programs. FHWA is currently sponsoring a study with USGS to develop an updated bridge backwater program that will combine the best algorithms of E431 and HEC-2 and will provide convenient printout for a risk analysis.

6.2.3 Stage-Discharge Relationship

If a computer is used to determine water surface profiles, rating curve information for each alternative can readily be obtained. The E431 program does not print the upstream water surface elevations right at the bridge section, but it does print the key numbers needed to calculate the elevations as follows:

Where: WSBR = WSU - HF

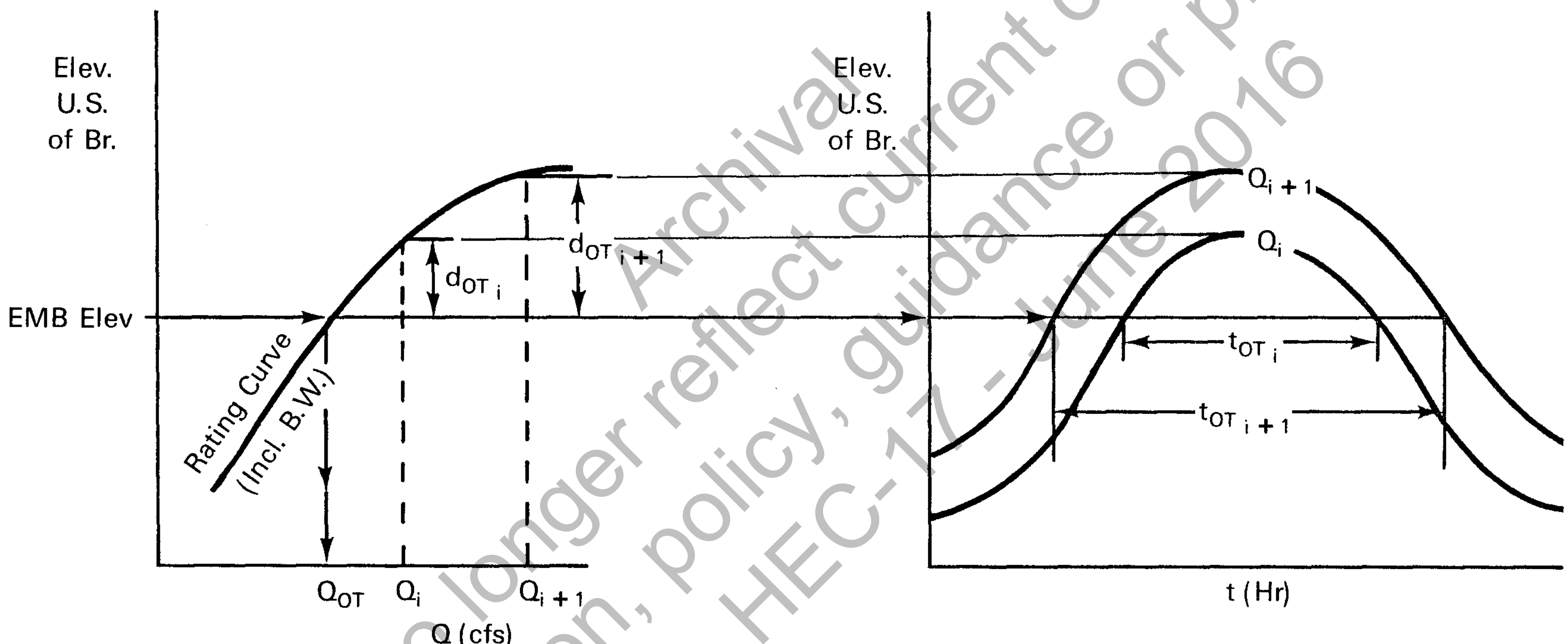
WSBR = water surface elevation just upstream of bridge
WSU = water surface elevation at the approach section
HF = friction head loss between the approach section and the bridge

The program prints HF for the natural condition, but it leaves a blank for HF in the bridge condition, therefore:

HF = DISTU(Q/KU)^2
DISTU = distance from bridge to the approach section where WSU is computed (usually DISTU = Bridge Length)
KU = the upstream conveyance for the bridge condition

If just a few alternatives (say four or five) are being considered, the rating curves should be plotted for each as indicated in figure 6.2a. Then overtopping discharges and overtopping depths can be determined graphically as illustrated.

Note: These curves would be unique for ea. BR. length ~ EMB. elev.



(a.) Rating Curve

(b.) Stage Hydrography

FIGURE 6.2 OVERTOPPING DISCHARGE, DEPTHS AND TIMES FROM UNIQUE RATING CURVE AND STAGE HYDROGRAPH

If a large number of alternatives are being considered, it is expedient to determine the overtopping discharges by interpolating betweeen water surface elevations to determine where overtopping would occur.

E431 will sometimes indicate flow over the roadway before overtopping occurs because it computes flow over the roadway from a weir equation which uses the total head as a parameter. E431 assumes flow over the roadway if the total head is greater than the embankment elevation. E431 apparently does not check the water surface elevation just upstream of the bridge to determine where overtopping occurs. Nevertheless, the proper overtopping discharge can be estimated by interpolating between water surface elevations as discussed above.

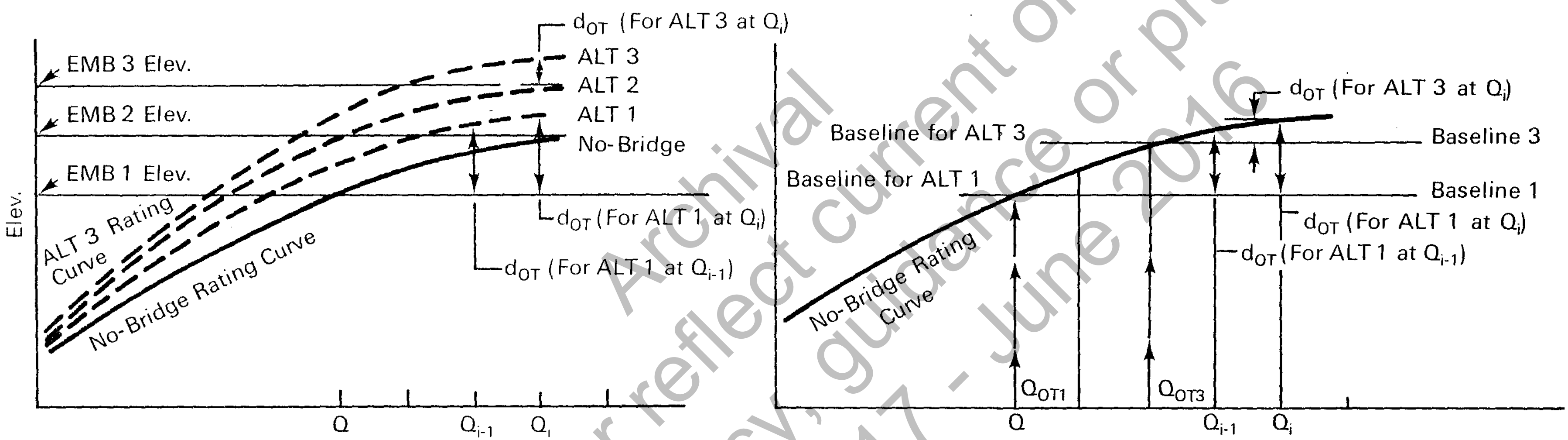
6.2.4 Overtopping Depths and Times

Figure 6.2 illustrates the direct method of determining overtopping depths and times as well as the overtopping discharge. Figure 6.2a is the rating curve just upstream of the bridge while figure 6.2b is the family of stage hydrographs for all the floods considered in the analysis. Once these curves are drawn, a base line can be drawn at the embankment elevation and used to determine the overtopping discharge and depths from the rating curves and the overtopping times from the hydrographs. The problem with the direct method is that rating curves and the entire family of stage hydrographs must be developed for each alternative. To facilitate the design process, short-cut methods are described below for determining overtopping depths and times assuming overtopping discharges are known.

6.2.4.1 Short-cut Method for Determining Overtopping Depths

Figure 6.3a illustrates the direct method of determining overtopping depths for a number of alternatives. This figure represents several rating curves plotted on one graph. The important characteristic to observe in figure 6.3a is that the rating curves tend to parallel the natural rating curve after overtopping occurs; an overtopped roadway essentially acts like a weir.

If this parallel characteristic is assumed, overtopping depths can be approximated directly from the natural rating curve as illustrated on figure 6.3b. The absolute elevations are ignored and baselines are established by drawing lines through the rating curves at the appropriate overtopping discharges (assumed to be known from the stage-discharge interpolations). Then, all of the overtopping depths can be estimated from a single rating curve.



(a) Sketch of Basic Graphical Methods

(b) Sketch of Short-Cut Graphical Methods

FIGURE 6.3 OVERTOPPING DEPTHS FROM RATING CURVES

6.2.4.2 Short-cut Method for Determining Overtopping Times

A short-cut method for estimating overtopping times is based on the assumption that hydrographs maintain approximately the same shape for all bridge conditions. Then for a known overtopping discharge and depth, the overtopping time can be estimated from a single family of either discharge or stage hydrographs, whichever is more readily availble. Furthermore, since stage hydrographs may be assumed parallel curves for various flood levels, overtopping times can be estimated from a single hydrograph curve for any major flood.

Stage hydrographs are readily available at continuous gaging stations, while discharge hydrographs are computed more readily for ungaged sites. Figure 6.4 illustrates the use of a stage hydograph in conjunction with overtopping depths to estimate overtopping times. The procedure is to scale a distance equal to the overtopping depth from the peak of the stage hydrograph and to measure time as the distance between the legs of the hydrograph. There is no need to draw a whole family of parallel stage hydrographs since any one would give the same results using this procedure. Figure 6.5 illustrates the case of a family of discharge hydrographs used in conjunction with overtopping discharges to estimate overtopping times. The procedure is to draw a line through the family of discharge hydrographs at the overtopping discharge and measure overtopping times between the legs of the respective hydrographs along this line.

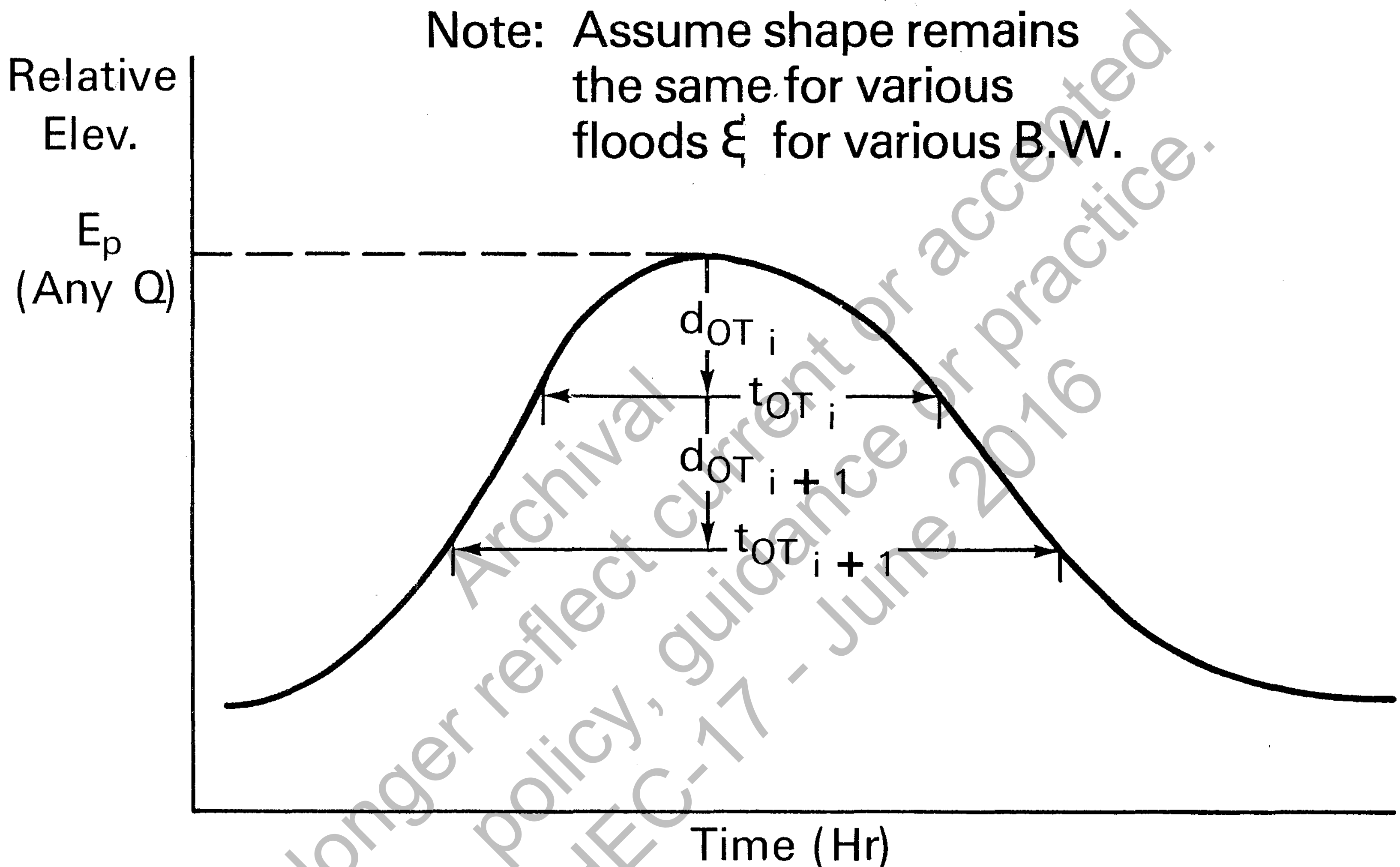


FIGURE 6.4 OVERTOPPING TIMES FROM A STAGE HYDROGRAPH

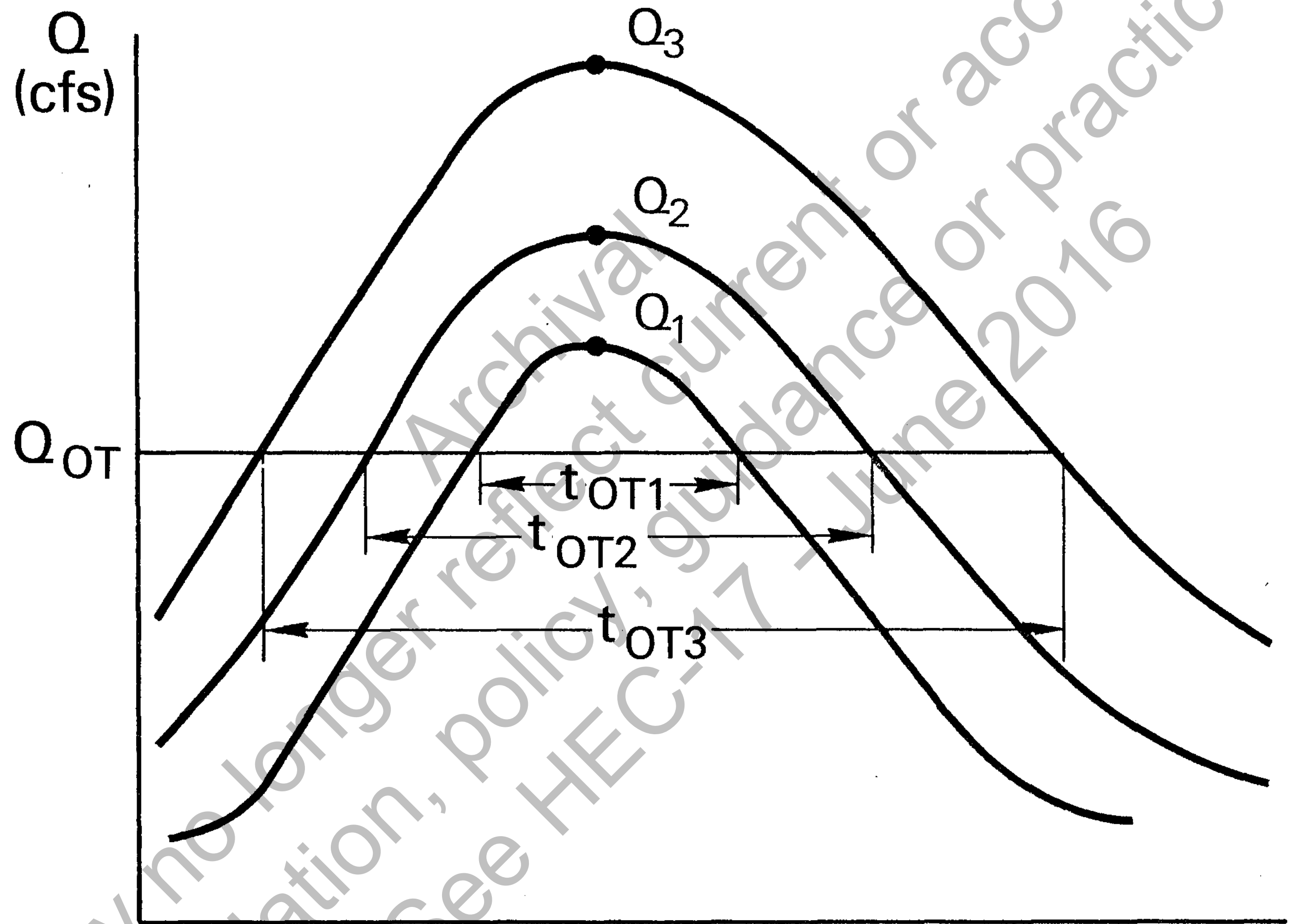


FIGURE 6.5 OVERTOPPING TIMES FROM A FAMILY OF DISCHARGE HYDROGRAPHS

6.2.5 Hydrographs

There are several techniques for developing hydrographs from limited data. While discharge hydrograph may be adequately approximated by a triangular shape, a stage hydrograph tends to be more of a trapezoidal shape with a longer duration near the peak.

Hydrographs may be developed by using a measured outflow hydrograph as a pattern, or where no hydrograph is available, by assumming the hydrograph shape.

6.2.5.1. Using a Measured Hydrograph as a Pattern

If a measured hydrograph is available, curves similiar to the measured hydrograph can be drawn to complete the family of hydrographs. First, mark each flood peak as illustrated in Figure 6.6a, and then draw similiar curves through the peaks as illustrated in Figure 6.6b.

The measured hydrograph should be based on measurement from an existing bridge or transfer of data from another bridge on the same stream. To be precise, the hydrographs should represent upstream stage-time relationships that reflect ponding responses from each alternate design. In a practical sense, however, several compromises as presented in the "overtopping times" section, are acceptable. First, the range of bridge alternates is not likely to significantly change the shape of the hydrographs so that one set is adequate. Second, relative rather than absolute stages are important so that even a downstream hydrograph will suffice. Finally, transferring a hydrograph from another bridge on the same stream is preferable to relying strictly on a computed hydrograph.

6.2.5.2. Computed Hydrographs

Measured hydrographs are not available on some streams and an analytical hydrograph will have to be developed. A hydrograph can be developed by a combination of methods attributed to the U.S. Water and Power Resource Service (21) and the Soil Conservation Service (22). Referring to Figure 6.7a, the time to peak, Tp, is estimated by

$$T_p = T_c = \left(\frac{11.9\, L^3}{H}\right)^{0.385}$$

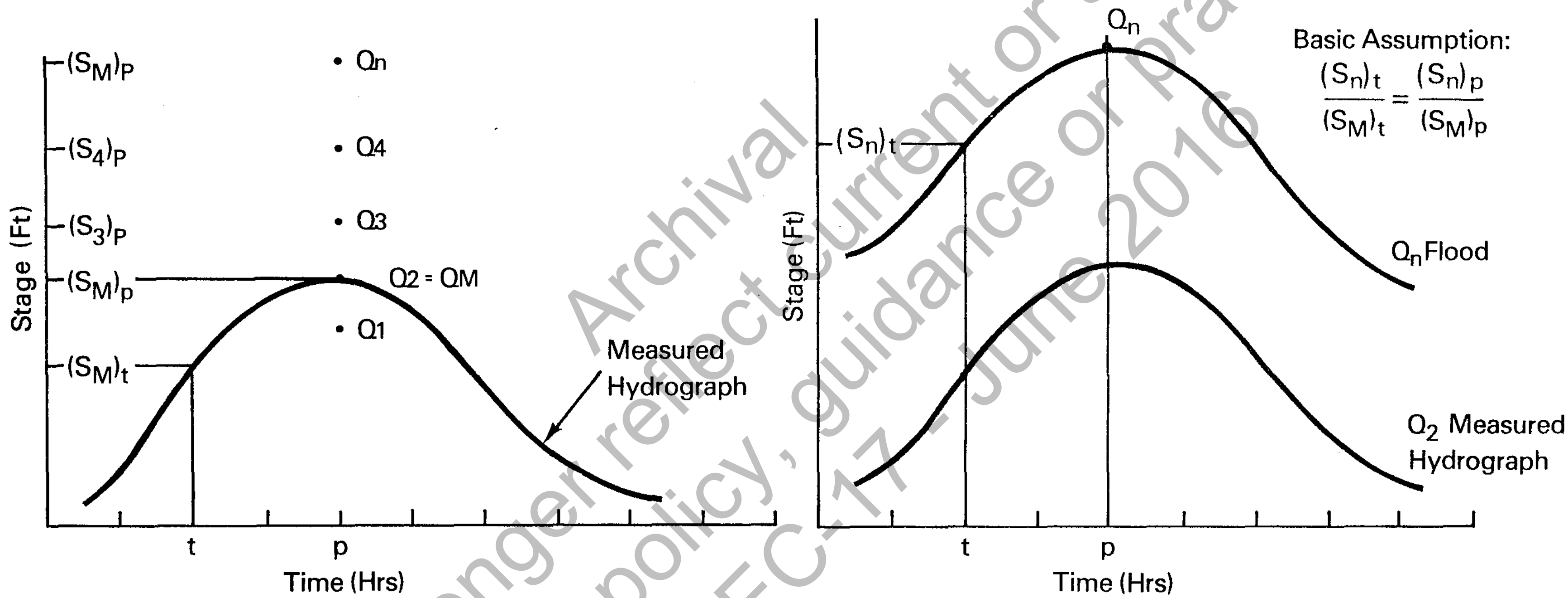
where: T_p = time to peak, hr.
L = length of longest watercourse, mi.
H = elevation difference, ft.
T_c = time of concentration, hr.

An average constant can be used to compute the time of recession, Tr, so that:

$$T_r = 1.67\, T_p$$

For a given peak discharge, Qp, a triangular discharge hydrograph, as illustrated in Figure 6.6a, is defined by Tp and Tr.

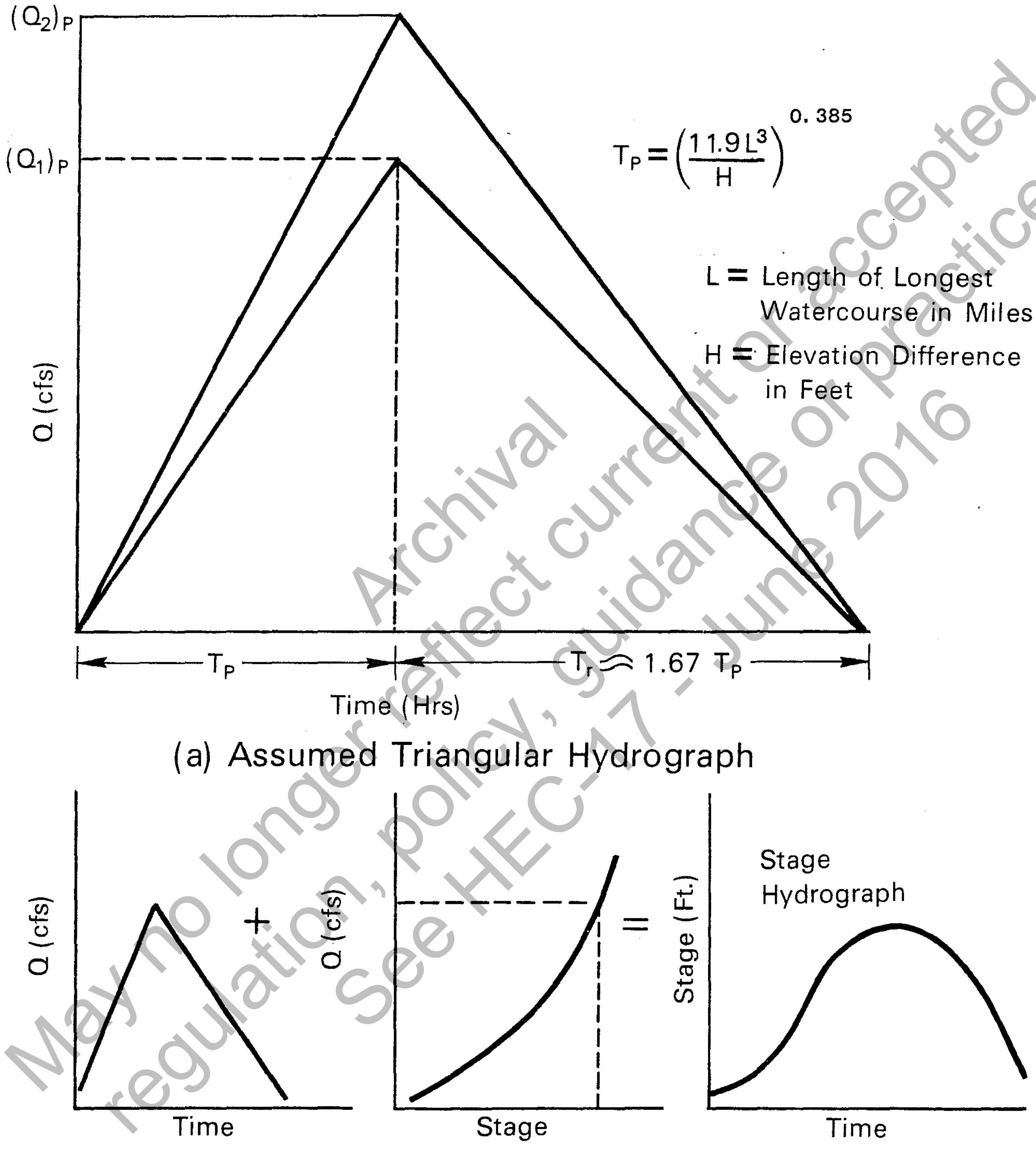
A stage hydrograph can be developed from a triangular discharge hydrograph by combining it with a rating curve, as illustrated in Figure 6.7b.



(a) Locating stage-discharge points

(b) Drawing parallel curves through stage — discharge points

FIGURE 6.6 CONSTRUCTING A FAMILY OF HYDROGRAPHS FROM A MEASURED HYDROGRAPH



(a) Assumed Triangular Hydrograph

(b) Development of a Stage Hydrograph

FIGURE 6.7 COMPUTED HYDROGRAPHS

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7.0 COMPUTATION OF ECONOMIC LOSSES

The methods presented for assessing economic losses are based on the best available information considering the degree of detail warranted, ease in application, general applicability and availability of data.

Potential economic losses may result from:

(1) Loss of pavement and embankment.

(2) Interruption of normal traffic flow.

(3) Damage to surrounding property due to backwater.

(4) Structural damage including scouring of foundations.

Subsequent parts of this manual contain suggested methods for assessing the various economic losses. To illustrate these methods, the following example problem will be used throughout the discussion.

7.1 Example Problem

A crossing of Row Creek is proposed. The hydrologic and hydraulic calculations have been accomplished and the alternative bridge length and embankment heights selected. Only one of the alternative designs, a bridge length of 440 feet and embankment height elevation 153, is used to illustrate the economic loss evaluation procedures. The reader should recognize that the economic loss evaluation procedures would have to be repeated for each alternative design to complete the LTEC design decisionmaking process. Additional data are presented as needed for the various economic loss assessment procedures.

7.2 Embankment Damage

Flood flows which overtop bridge approach embankments for sustained periods of time may result in erosion of shoulders and fill material and loss of the pavement surface. When such damage occurs, economic losses due to traffic interruption and the need to replace the fill and pavement result. In order to assess these losses, the mechanics of erosion must be known. Unfortunately embankment erosion has not been studied on a broad scale and the existing literature can only supply limited information on the subject.

Some experience is available from highway agencies concerning the duration and depth of overtopping which will cause erosion. Figure 7.1 is based on this experience. Users are encouraged to develop information on roadway damage due to overtopping for application in evaluation of the economic losses to embankment and pavement.

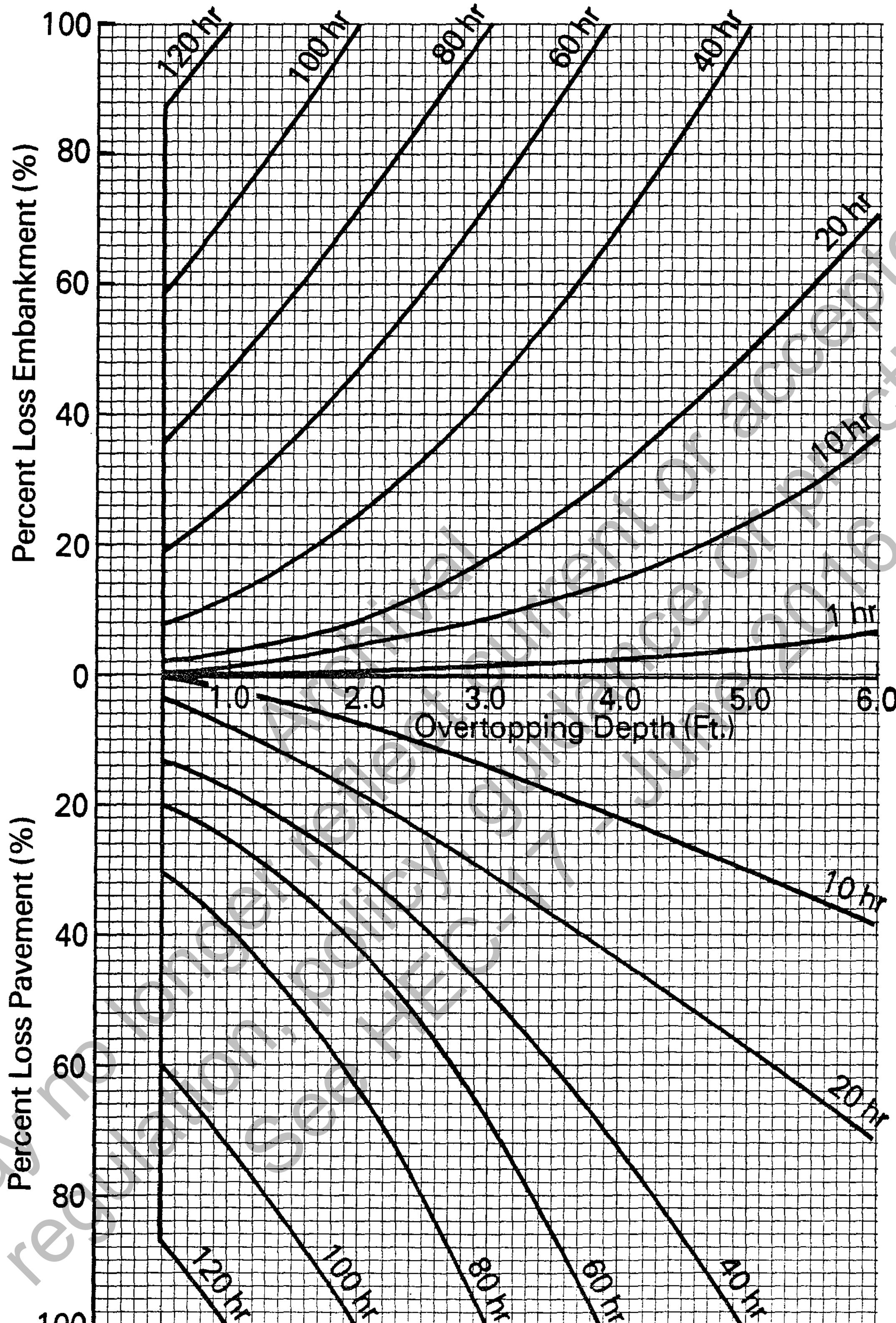


FIGURE 7.1 EMBANKMENT-PAVEMENT LOSSES

The embankment-pavement loss data used to construct table 7.1 were obtained from highway sections 48 feet wide, 40 feet of asphalt pavement, well vegetated 3 to 1 side slopes, and sandy-clay fill material. The data set included estimates for embankment erosion and pavement loss for 0.5, 1.0, 2.0, 3.0, 4.0 and 5.0 feet of overtopping. The data for the 0.5 foot condition included estimates of losses for overtopping time from 12 to 72 hours. The data for 1.0 and 2.0 feet of overtopping included estimates for 1 and 72 hours, and the data for 3.0, 4.0 and 5.0 feet of overtopping included estimates for 1 and 60, and 1 and 48 and 1 and 36 hours of overflow, respectively. Since the data set for the 0.5 foot of overflow was fairly complete, it was utilized to establish trends for the other depths of overtopping.

To determine embankment and pavement losses, enter table 7.1 with the time and depth of overtopping to determine the percent of embankment and pavement loss. The time and depth of overtopping for the various flood events are outputs of the hydraulic analysis. The economic losses at the roadway (LAR) are then computed from the equation:

$$LAR = [P_e C_e V_e + P_p C_p A_p] C_a + M_c$$

E_w = Embankment width, ft.

P_e = Percent of embankment loss from figure 7.1 multiplied by $\frac{48}{E_w}$.

C_e = Cost of embankment, \$/CY.

V_e = Total volume of embankment subject to overflow, CY.

P_w = Pavement width, ft.

P_p = Percent of pavement loss from figure 7.1 multiplied by $\frac{40}{P_w}$.

C_p = Cost of pavement, \$/SY.

A_p = Total area of pavement, subject to overflow, SY.

C_a = Ajustment factor for rapid repair.

M_c = Mobilization cost, \$.

Example: For the Row Creek crossing, flood frequencies of 10, 25, 50, 100 and 200 are to be used in the analysis. Table 7.1a summarizes the time and depth of overtopping analysis, and the percent losses for the embankment and pavement. Table 7.1b summarizes the roadway loss calculations. The costs used were \$1.47/CY of embankment and \$4.86/SY for pavement. The adjustment factor for rapid repair is 1.3 making the costs \$1.91/CY of embankment and \$6.32/SY of pavement. The mobilization costs are negligible for the 100-year flood event and \$5,000 for the 200 year event.

TABLE 7.1a - Overtopping Data

Discharge cfs	Frequency yr	Overtopping Depth (ft)	Overtopping Time (hr)	Percent Loss Embankment Fig 7.1	P_e	Percent Loss Pavement Fig 7.1	P_p
20,000	10	0	0	0	0	0	0
25,000	25	0	0	0	0	0	0
30,000	50	0.2	7	0	0	0	0
35,000	100	0.7	20	3	3.28	7	11.667
40,000	200	1.3	40	17	18.58	23	38.334

The roadway is 2-lane with a 10-foot shoulders and 3:1 side slopes. At elevation 153, the embankment volume for 2900 feet of roadway is 10,740 CY and the pavement area is 7,733 SY. For the 50-year event, roadway clean up is necessary. This is assumed to cost $2,000.

TABLE 7.1b - Roadway Losses

Discharge cfs	Frequency yr	Losses Embankment (CY)	$	Pavement(SY)	$
20,000	10	-	-	-	-
25,000	25	-	-	-	-
30,000	50	-	-	-	-
35,000	100	351.2	673	902.4	5,700
40,00	200	1,995.5	3,813	2,964.1	18,729

Total Roadway Repairs are:

$(LAR)_{50}$ = $2,000

$(LAR)_{100}$ = $673 + 5,700 = $6,373

$(LAR)_{200}$ = $3,813 + 18,729 + 5,000 = $27,542

7.3 Traffic-Related Losses

A major part of the economic losses due to inundation of a stream crossing involve traffic termination and delays caused by the need to detour traffic to an alternate route or routes.

The time that traffic is not allowed to utilize the crossing is equal to the sum of the overtopping time and the time to repair significant damage at the site. The overtopping time for the various flood flows is an output of the hydrologic analysis. The time to repair must be developed from past experience in repairing such damage.

The distribution and magnitude of the average daily traffic which will use the bridge must also be estimated. For the purpose of this analysis, it is reasonable to assume that the traffic volume will be a gradually varying series. A gradually varying series can be converted to an equivalent uniform annual series by procedures described by Grant and Ireson (14). The equivalent uniform annual series for the average daily traffic, ADTE, that represents a growing traffic series is:

$$ADTE = ADTI + G(gf)$$

Where: ADTI = initial ADT at end of first year.

G = growth rate of traffic volume, (ADTN-ADTI)/n.

ADTN = projected ADT at end of "n" years.

gf = factor to convert a gradually varying series to an equivalent annual series.

$$gf = \frac{1}{i} - \left[\frac{n}{(1+i)^n - 1}\right]$$

i = discount rate, percent.

n = service life of structure, yr.

Example: For the Row Creek crossing, the service life is 30 years for all components, the initial ADT at the end of the first year is 5000 vehicles per day, the projected ADT at the end of 30 years is 9,500 vehicles per day and the discount rate is 7 1/8 percent. The ADT equivalent is therefore:

$$ADTE = ADTI + G(gf)$$

$$\text{Where } G = (ADTN - ADTI)/n$$

$$= (9{,}500 - 5{,}000)/30 = 150$$

$$gf = \frac{1}{i} - \left[\frac{n}{(1+i)^n - 1}\right]$$

$$= \frac{1}{0.07125}\left[\frac{30}{(1+0.07125)^{30} - 1}\right] = 14 - \frac{30}{6.88}$$

$$= 10.2$$

$$ADTE = 5000 + 150(10.2) = 6530$$

7.3.1 Traffic Restoration Time

Traffic restoration time for each flood event is added to the overtopping time for each event to obtain the total detour time. The traffic restoration time can be estimated from:

$$T_r = V_e(P_e - P_{er})\frac{24}{R_e} + A_p(P_p - P_{pr})\frac{24}{R_p} + \text{Mobilization Time}$$

Where: T_r = Traffic restoration time, hr.

V_e = Total embankment subject to overflow, CY.

P_e = Percent of embankment loss.

P_{er} = Percent of embankment repair where traffic interruption does not occur.

R_e = Rate of embankment repair, CY/day.

A_p = Total pavement area subject to overflow, SY.

P_p = Percent of pavement loss.

P_{pr} = Percent of pavement repair where traffic interruption does not occur.

R_p = Rate of pavement repair, SY/day.

Total detour time for each flood event is: T_r + overtopping time.

Example: For the Row Creek Crossing, the rates of repair are 3270 CY/day for embankment and 3334 SY/day for pavement. The percent of time where traffic delays do not occur are 1.09 percent for embankment and 8.34 percent for pavement repairs. The mobilization time is negligible for the 100-year event and 8 hours for the 200-year event. For the 50-year event, even though no damage occurs, clean up of the roadway is required. This is assumed to be possible without traffic interruption. Therefore, the detour time is the overtopping time for the 50-year event.

$$\text{Time of detour } (T_d) = t_r + \text{overtopping time}(t_{ot})$$

$$T_{d50} = 0 + 7 = 7 \text{ hours}$$

$$T_{d100} = 1.7 + 1.9 + 0 + 20$$

$$= 23.6 \text{ hours}$$

$$T_{d200} = 13.8 + 16.7 + 8 + 40$$

$$= 78.5 \text{ hours}$$

There are three subcategories of traffic-related losses which might occur:

(1) Increased running cost due to the detour.

(2) Lost time of vehicle occupants.

(3) Increased accidents on the detour.

Much of the data necessary to assess these losses is available in the highway agencies planning and/or traffic engineering units.

7.3.2 Increased Running Cost

This cost represents the difference between the running cost on the detour and the normal route. It is a function of ADTE, travel distance, duration of detour, design speed and vehicle distribution. The vehicle distribution, number of cars, trucks, semitrailers etc., used in the analysis may be a standard distribution or one based on an actual traffic distribution in the study area. Running costs are computed for the passenger cars in the distribution and these costs adjusted to reflect the costs for the other classes of vehicles in the traffic distribution.

To obtain the losses due to increased running costs, it is necessary to compute the running costs over the normal route and the detour. The difference in these values is the additional cost or economic loss to the users of the facility.

Running cost (RC) is computed by:

$$RC = \frac{(\text{Time})(\text{ADTE})(\text{Length})(\text{Unit Cost})}{24{,}000}$$

Losses from running costs (RCL) are then:

RCL = (RC for Detour) - (RC for Normal Route)

Where:

Time (Duration of detour) is in hours
ADTE is in vehicles per day
Length is in miles
Unit costs are in dollars per 1000 vehicle miles

Example: For the Row Creek example, the travel distances are 5 miles for the normal route and 10.3 miles for the detour route. Travel speeds are 55 mph for the normal route and 35 mph for the detour. The types of vehicles in the traffic distribution are:

(1) Passenger cars (70% of ADTE)
(2) Commercial delivery trucks (20% of ADTE)
(3) Semitrailer trucks (10% of ADTE)

The running cost for passenger cars is \$37/1000 vehicle mile at 55 mph and \$34.50/1000 vehicle mile at 35 mph. The adjustment factors for the cost of the vehicles in the traffic distribution are 1.5 for commerical delivery vehicles and 3.2 for semitrailers.

The running costs are:

$$RC = \frac{\text{(Time of Detour)(ADTE)(Length)(Unit Cost)}}{24{,}000}$$

The Unit Cost for the normal route is:

$$UC = 37\ [.7+1.5(.2)+3.2(.1)]$$

$$UC = \$48.84/1000 \text{ vehicle miles,}$$

and for the detour

$$UC = 34.50[.7+1.5(.2)+3.2(.1)]$$

$$UC = \$45.54/1000 \text{ vehicle miles.}$$

The running costs for the normal route are:

$$RC_{(Q)} = \frac{T_{dQ}(6530)(5)(48.84)}{24{,}000} = (T_{dQ})(66.4)\ \$/hr$$

The running costs for the detour are:

$$RC_{(Q)} = \frac{T_{dQ}(6530)(10.3)(45.54)}{24{,}000} = (T_{dQ})(127.6)\ \$/hr$$

The increased running costs due to the detour use are:

$$RCL_{(Q)} = T_{dQ}(127.6-66.4) = (T_{dQ})(61.2)\ \$/hr$$

7.3.3 Time Losses

These losses are a function of ADTE, detour duration, traveled distance, vehicle occupancy rate, design speed and the value of individuals time. The occupancy rate and the value of time may be averages which apply to all individuals in the classes of vehicles in the traffic distribution or different values for each.

The occupancy rate data will most often be site specific and the value of time data may be based on statistical results such as those in the table below.

Income Level/Year	Value of Time $/hr
Under $4000	$2.26
4000 - 5999	2.73
6000 - 7999	3.19
8000 - 9999	3.64
10000 - 11999	4.11
12000 - 14999	4.75
15000 - 20000	5.03
Over 20000	5.49

The value of time is computed from:

$$TC = \frac{(\text{Time})(\text{Length})(\text{ADTE})(\text{Occupancy Rate})(\text{Unit Cost})}{(\text{Speed})(24)}$$

The vehicle occupant's dollar loss due to use of the detour is:

TCL = (TC of Detour) - (TC for normal route)

Where:

Time is in hours (Detour Time)
Length is in miles
Speed is in miles per hour
ADTE is in vehicles per day
Occupancy rate is in people per vehicle
Unit cost is in dollars per person per hour

Example: For the Row Creek crossing, the occupancy rate is two people/vehicle, and the value of time is $4.75/hr, which is based on an average income level of $12,000 to $15,000 for this section of the State. The value of time for the normal route is:

$$TC_{(Q)} = \frac{T_{dQ}(5)(6530)(2)(4.75)}{(55)(24)} = (T_{dQ})(235) \text{ \$/hr}$$

and for the detour is:

$$TC_{(Q)} = \frac{T_{dQ}(10.3)(6530)(2)(4.75)}{(35)(24)} = (T_{dQ})(760.7) \text{ \$/hr}$$

The loss of time in dollars due to use of the detour is:

$$TCL_{(Q)} = T_{dQ}(760.7 - 235) = (T_{dQ})(525.7) \text{ \$/hr}$$

7.3.4 Accident Costs

Increased accident costs are based on death rate statistics. For each death, there are assumed a certain number of personal injuries and property damage accidents. The personal injuries and property damage losses are obtained by applying property damage and personal injury rates to the costs of damage and injury. The rate and cost data may be site specific or based on national statistics. These losses are computed on a vehicle mile basis and are a function of ADTE, length of detour, duration of detour, ratios of personal injuries and property damage accidents to deaths, costs of personal injuries and property damage accidents.

The cost of accidents is computed by:

$$AC = \frac{1}{2.4\text{x}10^9}[(\text{Time})(\text{ADTE})(\text{Length})(\text{Death Rate})][(\text{Accident/Injury Factor})]$$

$$\text{Accident/Injury Factor} = [(\text{Injury Ratio})(\text{Unit Cost of Injury})+(\text{Damage Ratio})(\text{Unit Cost of Damage})],$$

and the dollar loss due to increased accident exposure on the detour is:

ACL = (AC for detour) - (AC for normal route)

Where:

Time is in hours (Time of Detour)
ADTE is in vehicles per day
Length is in miles
Death ratio is in people per 100 million vehicle miles
Injury rate is in injuries per death
Damage ratio is in damage per death
Unit cost of injury is in dollar per injury
Unit cost of damage is in dollars per damage claim

Even though the cost for loss of life is not included in this analysis, the user may encounter situations where it would be appropriate to include this loss. One problem with considering loss of life is arriving at a reasonable value to use in the analysis. Suggested values vary considerably. To include loss of life requires adding a cost per death term to the Accident/Injury Factor part of the accident cost equation:

Accident/Injury Factor = [(Cost Per Death)+(Injury Ratio)(Unit Cost of Injury)+(Damage Ratio)(Unit Cost of Damage)]

Example: The additional data needed for computing the cost of accidents (no increased loss of life is assumed) at Row Creek are:

	Normal Route	Detour Route
Death Rate	5	5
Injury Rato	17	22
Damage Ratio	175	250
Cost of Injury	3500	3500
Cost of property Damage	600	600

The cost of accidents on the normal route is:

$$AC_Q = \frac{(T_{dQ})(6530)(5)(5)}{2.4 \times 10^9}[(17)(3500)+(175)(600)] = (T_{dQ})(11.2)\ \$/hr$$

and the cost of accidents on the detour is:

$$AC_Q = \frac{(T_{dQ})(6530)(10.3)(5)}{2.4 \times 10^9}[(22)(3500)+(250)(600)] = (T_{dQ})(31.8)\ \$/hr$$

The increase cost of accidents due to using the detour are:

$$ACL_Q = (T_{dQ})(31.8 - 11.2) = (T_{dQ})(20.6)\ \$/hr$$

Summation of Traffic-Related Losses

The total traffic-related losses which are input to the economic risk analysis are computed by the following equation:

$$TRL_Q = T_{dQ}(\text{Running Cost + Lost Time Cost + Accident Cost})$$

$$TRL_{50} = 7(61.2 + 525.7 + 20.6) = 7(607.5) = \$4,252$$

$$TRL_{100} = 23.6(607.5) = \$14,337$$

$$TRL_{200} = 78.5(607.5) = \$47,688$$

In application, the user would be required to compute the time of detour for the various flood events used in the analysis. The traffic losses represented by the cost per hour value ($607.50) would in most cases, be obtained from traffic engineering and/or planning units.

It is assumed in this analysis that the detour or detours selected are available for traffic use when flooding occurs at the study site. In application, the designer should analyze candidate detours to determine the likelihood of possible detours being available for traffic service when needed. This involves determining the overtopping frequency for crossings on the detour(s) and making a judgment as to the probability of coincidental flooding at the detour(s) and study site.

7.4. Backwater Damage Losses

The construction of bridges and culverts on flood plains most often involves constriction of the natural floodway. This constriction results in an increased water surface elevation or backwater upstream, which may contribute to incremental damage to property adjacent to the crossing.

The highway agency should not be held responsible for flood damage incurred under normal flow conditions before the bridge, embankment or culvert is in place, regardless of flood magnitude. The highway agency is responsible, however, for the additional or incremental flood damage which results from backwater associated with the construction of a stream crossing.

The magnitude of the incremental damage due to backwater depends on the degree of constriction of the natural floodway and the specific land uses on the flood plain. Traditionally, flood plains have been desirable areas for development. Flood plains are often used for farming, as pasture lands, and all too often, private homes and industries are located on flood plains. Different levels of damage are associated with these flood plain uses. For example, pasture and woodlands may incur little damage from flooding, while private homes or industries may be destroyed by severe flooding.

Assessing backwater damage requires collecting considerable field data for input to the hydraulic calculations and to define specific flood plain uses. The data collection and analysis effort expended should depend on the complexity and importance of the individual encroachment. For example, a rural site with little flood plain development may justify only a minimal effort while a complex urban flood plain encroachment may justify considerable data collection and analysis.

The assessment procedure requires that a site map be prepared from information obtained from aerial and/or ground surveys. This flood plain map should include contours at an appropriate interval and indicate the locations of cross sections, crop lands, pastures, buildings, etc. In order to assure sufficient map coverage it will be necessary to estimate the maximum extent of backwater for the worst condition, i.e. highest flood and maximum contraction.

From the site map, the area between successive cross sections for each contour interval for each crop, and the number and first floor elevation of all buildings between cross sections is determined.

Property values and crop data are also necessary inputs to the analysis. Right-of-way units, tax assessment records and the State offices of the U.S. Soil Conservation Service are good sources for obtaining property value and crop information. To compute the value of crops requires determining the yield per acre and the price per unit of yield. Table 7.2 represents a composite of data obtained from the SCS for computing crop losses due to flooding. The data used in preparing table 7.2 are from the northeast region of the United States. The table was prepared to illustrate the crop damage assessment procedure and is not recommended for general use. Similar information for specific areas is available from the State offices of the SCS.

The value of the contents of residences is considered to be 50 percent of the value of the residence. The value of the contents of other buildings must be determined by site investigation.

The percent damage with depth of inundation values for residences are indicated of figure 7.2. The percent damage values for other buildings must be determined on an individual basis.

The step-by-step procedure for determining backwater damage losses is:

(1) Prepare site map from aerial and/or ground survey data.

(2) Identify the various crops and buildings on the flood plain.

(3) Compute the acreage for the various crops between cross sections for each increment of elevation and determine first floor elevations for all buildings.

(4) Compute the value of the various crops - dollars per acre times number of acres.

(5) Determine the value of each building and contents and sum values for each first floor interval.

(6) Compute the average water surface elevations between cross sections for each flood event for the natural and backwater conditions.

Table 7.2 — Percent Damage To Crops

Crop	% Damage			
	Less than 24 Hours Inundation		More than 24 Hours Inundation	
	0 to 2 Feet	Over 2 Feet	0 to 2 Feet	Over 2 Feet
Corn	54	88	75	100
Soybeans	92	100	100	100
Oats	67	97	81	100
Hay	60	82	70	97
Pasture	50	75	60	90
Winter Wheat	57	87	72	100

(7) Compute the losses to crops.

(a) Tabulate the contour intervals and incremental dollar values from steps 3 and 4.

(b) Tabulate the average water surface elevations for each flood from step 6.

(c) Compute the depth of inundation for each contour interval for each flood, i.e. the average water surface elevation minus the lower contour elevation.

(d) From table 7.2 determine and tabulate the percent loss for each increment of inundation.

(e) Determine the damage for natural conditions for each flood event. This is the sum of the products of the percent damage and incremental value of crops for each contour interval.

(f) Repeat steps b through e for backwater conditions.

(g) Compute backwater damage - the damage due to backwater minus damage which occurred under natural conditions.

(h) Repeat steps b through g for each design alternative.

(8) Compute losses to buildings.

(a) Tabulate the first floor elevation intervals selected and the dollar value of all buildings within each interval, steps 3 and 5.

(b) Tabulate average water surface elevations between cross sections for each flood.

(c) Compute the depth of inundation for each first floor interval, i.e. the average water surface elevation minus the representative first floor elevation.

(d) From figure 7.2 determine the percent loss for each depth of inundation.

(e) Determine the damage for natural conditions. For each flood event, this is the sum of the products of the percent damage and incremental value of buildings for each representative first floor elevation.

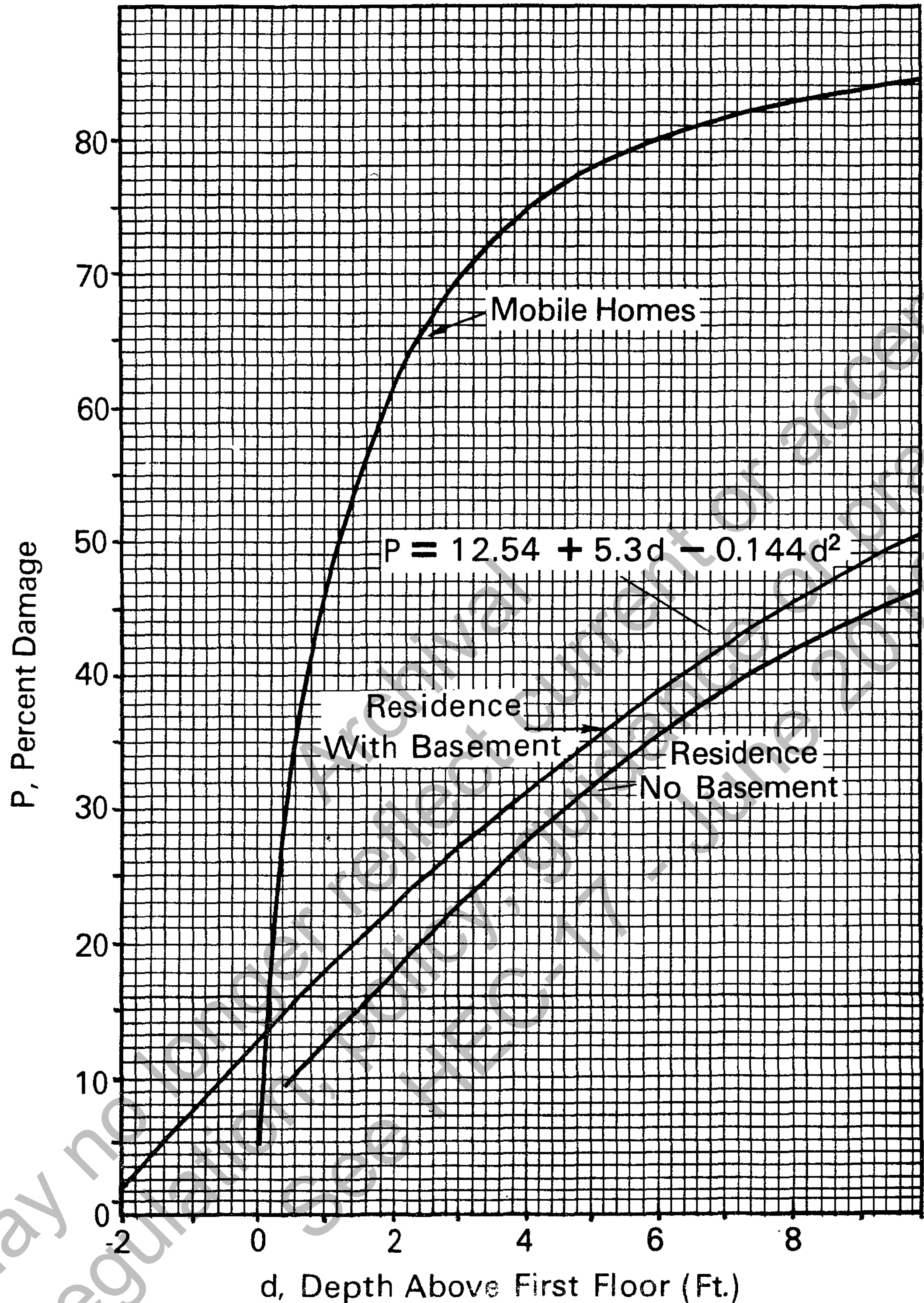


FIGURE 7.2 PERCENT DAMAGE, MIXED RESIDENCES

(f) Repeat steps b through e for backwater conditions.

(g) Compute backwater damage as in step 7(g).

(h) Repeat steps b through g for each design alternative.

(9) For each design alternative, for each flood event sum the backwater damage values for crops and buildings.

Example: Following the above step-by-step procedure, the backwater damage losses are computed for the Row Creek crossing.

Step 1. The site map has been prepared, figure 7.3.

Step 2. The crops and buildings are identified on the site map, and are:

Symbol	Land Use
1.	Residence/with basement
2.	" "
3.	" "
4.	" "
5.	" "
6.	" "
7.	" "
8.	" "
9.	" "
I	Corn
II	Soybean
III	Pasture
IV	Hay

Step 3. The acreage and first floor elevation are:

Contour	x-Section	Crop	Acres
148-149	A-B	Corn	0.1
149-150	"	"	0.2
150-151	"	"	0.25
151-152	"	"	0.2
152-153	"	"	0.35

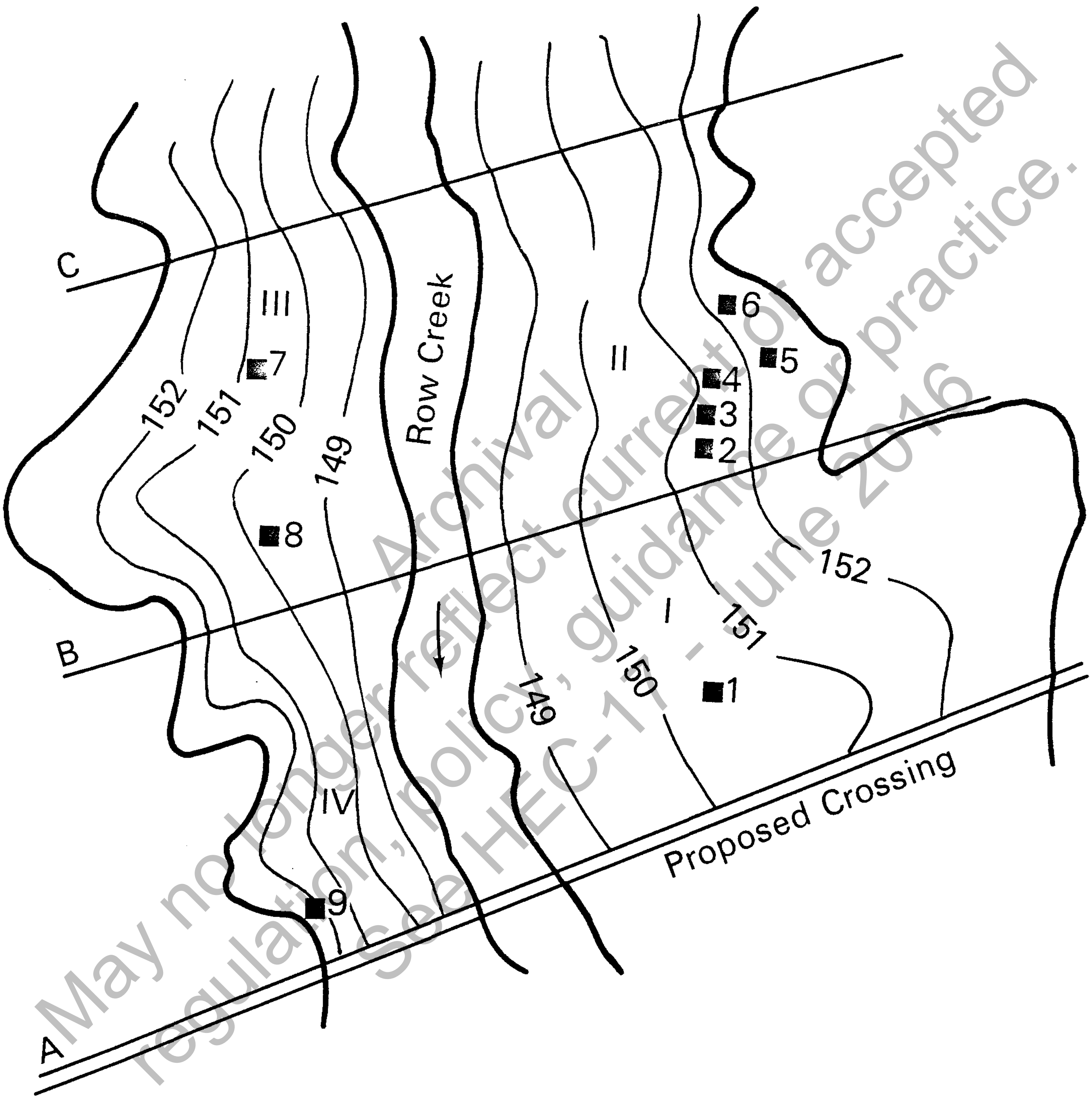


FIGURE 7.3 ROW CREEK CROSSING

Contour	x-Section	Crop	Acres
148-149	A-B	Hay	0.1
149-150	"	"	0.07
150-151	"	"	0.1
151-152	"	"	0.06
152-153	"	"	0.05

Contour	x-Section	Crop	Acres
148-149	B-C	Soybean	0.12
149-150	"	"	0.15
150-151	"	"	0.17
151-152	"	"	0.10
152-153	"	"	0.12

Contour	x-Section	Crop	Acres
148-149	B-C	Pasture	0.1
149-150	"	"	0.18
150-151	"	"	0.17
151-152	"	"	0.1
152-153	"	"	0.2

x-Section	Symbol	First Floor Elevation
A-B	1	150.6
A-B	9	152.0
B-C	2	151.5
"	3	151.4
"	4	151.3
"	5	151.9
"	6	151.8
"	7	150.7
"	8	149.6

Step 4. Data for mature crops in the area indicate that the corn yield is 40 bushels/acre and $4.00/bushel making its value $160.00/acre. The value of soybean at 25 bushels/acre and $5.25/bushel is $131.25/acre. The value of pasture is $200.00/acre and hay yield is 4 ton/acre at $75.00/ton or $300/acre.

Contour	x-Section	Crop	Acres	$/Acre	Incremental Value
148-149	A-B	Corn	0.1	160	16
"	"	Hay	0.1	300	30
149-150	"	Corn	0.2	160	32
"	"	Hay	0.07	300	21
150-151	"	Corn	0.25	160	40
"	"	Hay	0.1	300	30
151-152	"	Corn	0.2	160	32
"	"	Hay	0.06	300	18
152-153	"	Corn	0.35	160	56
"	"	Hay	0.05	300	15

Contour	x-Section	Crop	Acres	$/Acre	Value
148-149	B-C	Soybean	0.12	131.25	16*
"	"	Pasture	0.1	200	20
149-150	"	Soybean	0.15	131.25	20*
"	"	Pasture	0.18	200	32
150-151	"	Soybean	0.17	131.25	22*
"	"	Pasture	0.17	200	34
151-152	"	Soybean	0.10	131.25	13*
"	"	Pasture	0.10	200	20
152-153	"	Soybean	0.12	131.25	16*
"	"	Pasture	0.2	200	40

*Rounded to nearest dollar

Step 5. The value of buildings and contents is:

x-Section	Symbol	Representative First Floor Elevation*	Value Building	Value Contents	Total
A-B	1	151	60,000	30,000	90,000
"	9	152	15,000	3,750	18,750
B-C	8	150	20,000	150,000	170,000
"	3	151	90,000	45,000	
"	4	151	8,000	2,000	
"	7	151	80,000	40,000	265,000
"	6	152	2,000	2,000	
"	2	152	10,000	2,500	
"	5	152	80,000	40,000	136,500

*Rounded to nearest foot

Step 6. The average water surface elevations between cross sections are:

Discharge cfs	Frequency yr	x-Section	Average Elev. Natural	Average Elev. Backwater
20,000	10	A-B	150.9	151.1
25,000	25	"	151.7	152.4
30,000	50	"	152.0	153.5
35,000	100	"	152.6	154.0
40,000	200	"	152.9	154.4
20,000	10	B-C	152.4	152.7
25,000	25	"	153.0	153.5
30,000	50	"	153.4	153.9
35,000	100	"	153.7	154.4
40,000	200	"	154.2	154.9

Step 7. Compute losses to crops.

a. On table 7.3, 7.4, 7.5, and 7.6 enter the contour intervals and total dollar values for crops within interval.

b. Also on these same tables enter average water surface elevation between cross sections.

TABLE 7.3 BACKWATER LOSSES (STEP 7)

CROP - CORN

Q (cfs)	Cross Section	Avg. W.S. Elev.	Contour Interval 148-149	149-150	150-151	151-152	152-153	Total Damage ($)	Delta Damage ($)
			Incremental Value 16	32	40	32	56		
			% Damage Natural Condition						
20,000	A-B	150.9	88	54	54	0	0	53.	
25,000		151.7	88	88	54	54	0	81.	
30,000		152.0	88	88	88	54	0	95.	
35,000		152.6	100	100	100	75	75	154.	
40,000		152.9	100	100	100	75	75	154.	
			% Damage Backwater Condition						
20,000	A-B	151.1	88	88	54	54	0	81.	28.
25,000		152.4	88	88	88	54	54	125.	44.
30,000		153.5	88	88	88	88	54	136.	41.
35,000		154.0	100	100	100	100	100	176.	22.
40,000		154.4	100	100	100	100	100	176.	22.

TABLE 7.4 BACKWATER LOSSES (STEP 7)

CROP - SOYBEAN

Q (cfs)	Cross Section	Avg. W.S. Elev.	148-149	149-150	150-151	151-152	152-153	Total Damage ($)	Delta Damage ($)
			Incremental Value						
			16	20	22	13	16		
			% Damage Natural Condition						
20,000	B-C	152.4	100	100	100	92	92	85.	
25,000		153.0	100	100	100	100	92	86.	
30,000		153.4	100	100	100	100	92	86.	
35,000		153.7	100	100	100	100	100	87.	
40,000		154.2	100	100	100	100	100	87.	
			% Damage Backwater Condition						
20,000	B-C	152.7	100	100	100	100	92	86.	1.00
25,000		153.5	100	100	100	100	92	86.	-
30,000		153.9	100	100	100	100	92	86.	-
35,000		154.4	100	100	100	100	100	87.	-
40,000		154.9	100	100	100	100	100	87.	-

TABLE 7.5 BACKWATER LOSSES (STEP 7)

CROP - PASTURE

Q (cfs)	Cross Section	Avg. W.S. Elev.	Contour Interval 148-149	149-150	150-151	151-152	152-153	Total Damage ($)	Delta Damage ($)
			Incremental Value 20	32	34	20	40		
			% Damage Natural Condition						
20,000	B-C	152.4	75	75	75	50	50	95.	
25,000		153.0	75	75	75	75	50	100.	
30,000		153.4	75	75	75	75	50	100.	
35,000		153.7	90	90	90	90	60	119.	
40,000		154.2	90	90	90	90	90	131.	
			% Damage Backwater Condition						
20,000	B-C	152.7	75	75	75	50	50	95.	-
25,000		153.5	75	75	75	75	50	100.	-
30,000		153.9	75	75	75	75	50	100.	-
35,000		154.4	90	90	90	90	90	131.	12.00
40,000		154.9	90	90	90	90	90	131.	-

TABLE 7.6 BACKWATER LOSSES (STEP 7)

CROP - HAY

Q (cfs)	Cross Section	Avg. W.S. Elev.	Contour Interval 148-149	149-150	150-151	151-152	152-153	Total Damage ($)	Delta Damage ($)
			Incremental Value 30	21	30	18	15		
			% Damage Natural Condition						
20,000	A-B	150.9	82	60	60	0	0	55.	
25,000		151.7	82	82	60	60	0	70.	
30,000		152.0	82	82	82	60	0	77.	
35,000		152.6	97	97	97	70	70	102.	
40,000		152.9	97	97	97	70	70	102.	
			% Damage Backwater Condition						
20,000	A-B	151.1	82	82	60	60	0	70.	15.
25,000		152.4	82	82	82	60	60	86.	16.
30,000		153.5	82	82	82	82	60	90.	13.
35,000		154.0	97	97	97	97	97	111.	9.
40,000		154.4	97	97	97	97	97	111.	9.

TABLE 7.7 BACKWATER LOSSES (STEP 8)

BUILDINGS

Q (cfs)	Cross Section	Avg. W.S. Elev.	First Floor Elev. 150 — Incremental Value 0	First Floor Elev. 151 — Incremental Value 90,000	First Floor Elev. 152 — Incremental Value 18,750	Total Damage ($)	Delta Damage ($)
			% Damage Natural Conditions				
20,000	A-B	150.9	-	12.0	7.0	12,113	
25,000		151.7	-	17.0	11.0	17,363	
30,000		152.0	-	18.0	13.0	18,638	
35,000		152.6	-	21.0	16.0	21,900	
40,000		152.9	-	22.0	18.0	23,175	
			% Damage Backwater Conditions				
20,000	A-B	151.1	-	13.0	8.0	13,200	1,087
25,000		152.4	-	19.5	14.5	20,269	2,906
30,000		153.5	-	25.0	20.0	26,250	7,612
35,000		154.0	-	27.0	22.5	28,519	6,619
40,000		154.4	-	29.0	24.5	30,694	7,519

Q (cfs)	Cross Section	Avg. W.S. Elev.	Incremental Value 170,000	265,000	136,000	Total Damage ($)	Delta Damage ($)
			% Damage Natural Conditions				
20,000	B-C	152.4	24.5	19.5	14.5	113,117	
25,000		153.0	27.0	22.5	18.0	130,095	
30,000		153.4	29.0	24.5	19.5	140,842	
35,000		153.7	30.0	25.5	21.0	147,240	
40,000		154.2	33.0	28.0	23.5	162,377	
			% Damage Backwater Conditions				
20,000	B-C	152.7	25.5	21.0	16.0	120,840	7,722
25,000		153.5	29.5	25.0	20.0	143,700	13,605
30,000		153.9	31.0	26.5	22.0	152,955	12,112
35,000		154.4	33.0	29.0	24.5	166,392	19,152
40,000		154.9	35.0	31.0	26.5	177,822	15,445

c. Compute the depths of inundation for each contour interval. i.e., the average water surface elevation minus the lower contour interval elevation.

d. Using the values computed in c, refer to table 7.2 to determine percent of damage and enter in tables 7.3, 7.4, 7.5, and 7.6 The inundation time is less than 24 hours for all floods less than 35,000 cfs.

e. Multiply the percent damage by the total crop value for each contour interval and sum these products for each flood event to obtain the damage loss for each flood.

f. Repeat the calculation for the backwater condition.

g. Determine the delta damage, i.e., backwater damage minus natural condition damage.

Step 8. Compute losses to buildings.

a. The representative first floor elevation, rounded to the nearest foot, and the sum of the dollar values for all building between cross sections for each representative elevation are entered on table 7.7.

b. Enter average water surface elevation for each contour interval on table 7.7.

c. Compute the depth of inundation, i.e., average flood elevation minus representative first floor elevation.

d. Determine percent damage for each depth of inundation from figure 7.2 and record on table 7.7.

e. Compute total damage - i.e., multiply the percent damage by the incremental value and sum these products for each flood.

f. Compute the damage due to backwater.

g. Compute the delta damage - the damage due to backwater condition minus the damage for natural condition.

Step 9. The table below summarizes the results of the backwater damage loss calculations. The values in the table are input to the risk analysis computations.

Summary of Backwater Damage Losses

	Backwater Losses					
Q	Corn	Soybeans	Pasture	Hay	Buildings	Total
20,000	$28.	$1.	-	$15.	$8,809	$8,853
25,000	44.	-	-	16.	16,511	16,571
30,000	41.	-	-	13.	19,724	19,788
35,000	22.	-	$12.	9.	25,711	25,754
40,000	22.	-	-	9.	22,964	22,995

7.5 Structural Damage

A highway stream crossing may sustain flood-related structural damage as a result of:

(1) Damage to the bridge superstructure due to debris and inundation of the bridge deck.
(2) Scour around bridge piers and abutments.

7.5.1 Damage to Bridge Superstructure

The inundation of a bridge deck by flowing water results in additional stress on the superstructure. The potential damage to a bridge superstructure is aggravated by accumulation of trash, and the impact of large floating debris. Although damage due to inundation of superstructures has been reported during most major floods, the data are insufficient to predict the precise effects of flood forces and the extent of damage, particularly since data on the effect of debris are not easily obtained.

While damage from inundation of bridge superstructures is difficult to assess, it may be too important to ignore in assessing flood losses. For lack of a more refined procedure, the following equation is suggested to estimate the damage due to superstructure inundation:

$$L_b = ay$$

Where

a = a coefficient, dollars/foot of inundation.
y = depth of submergence of the bridge deck, ft.

The coefficient must be estimated based on past experience in cleaning up, repairing damage and placing back in service bridges inundated by floods.

Example. It has been determined that the unit cost of structural damage due to inundation is $4,400 per foot of inundation at the Row Creek crossing. The losses due to inundation are:

Q	Inundation (ft)	Losses
20,000	-	-
25,000	-	-
30,000	0.3	1320
35,000	0.7	3080
40,000	1.3	5720

7.5.2 Damage due to scour at bridge foundations and spur dikes.

In major floods, scour damage often occurs around piers, abutments and spur dikes. The extent of scour is affected by both hydraulic and geologic conditions and is aggravated by debris accumulation.

Losses due to scour may be incorporated into the analysis in much the same way as was used to assess losses due to superstructures inundation. Scour damage is assumed to result in an economic loss determined by multiplying the scour extent by the unit cost to repair the damage:

$$L_s = sc$$

Where:

s = scour extent, CY.

c = unit cost to repair the damage, \$/CY.

There are numerous methods available to estimate the extent of scour for various flow conditions and, again, past experience must be utilized to estimate the unit cost to repair the damage.

Example: Based on past experience at similar crossings, the cost to repair scour damage is estimated to be \$100/CY of scour. Scour loss estimates for the Row Creek crossing are:

Q	Scour Depth	Scour Volume	Losses
(cfs)	(ft)	CY	(\$)
20,000	8	25	2,500
25,000	16	175	17,500
30,000	18	254	25,400
35,000	20	349	34,900
40,000	22	464	46,400

8.0 COMPUTATION OF TOTAL EXPECTED COST (TEC)

8.1 Capital Costs.

All of the initial costs incurred in completing a structure are included in the capital cost. For a bridge these may include:

(1) Bridge and foundations.
(2) Approach embankments.
(3) Roadway pavement.
(4) Protective measures including countermeasures (spur dikes, riprap, etc.).

Annual maintenance and operating costs are not included in the analysis unless they vary with the alternatives being considered.

The construction cost components are summed to obtain the total initial cost which must be amortized over the life of the structure. All computations are made in terms of constant dollars by using a discount rate instead of the prevailing interest rate in the computations. Interest rates normally include a perceived inflation factor, which if used in the computation would require applying inflation factors to replacement and maintenance costs that may not be incurred until some future time. By using a discount rate, all costs can be estimated at today's prices. The total construction or capital costs are multiplied by a capital recovery factor to obtain the annual amortization series. The capital recovery factor (CRF) is defined as "an annuity whose present value is one." The CRF is computed from:

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

Where: i = discount rate, percent.
n = service life of the structure, yr.

Tables of CRF's values for various discount rates can be found in most texts on economics.

The total cost and annual capital cost are computed for each design strategy.

Example. For the 440 foot alternative design of the Row Creek crossing, the capital cost is:

Bridge and Foundation	$1,355,200
Approach Embankments	18,795
Road Pavement	104,395
	$1,478,390

With a discount rate of 7 1/8 percent, the CRF is:

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

$$= \frac{0.07125}{1-(1+0.07125)^{-30}}$$

$$= 0.0816$$

The annual capital cost is therefore:

0.0816 x 1,478,390 = $120,636

8.2 Risk Costs.

To determine the total expected cost for each design requires computing the risk costs for each design strategy.

An assumption that influences this computation is that damage will be repaired so it has the same opportunity to recur year after year. This assumption means that the probability of a damage loss is the probability that a flood will be expected in any given year.

Determining the risk costs for a given design strategy requires two functions: (1) the loss function, and (2) the probability density function of the flood events.

The loss function represents the sum of the economic losses associated with the various flood events. The probability density function is a mathematical expression of how the probabilities are distributed over the range of flood events.

Since the floods are considered a continuous random variable, the probability that a given event is within a closed interval must be considered:

$$P[a<Q<b] = \int_a^b f(Q)dQ$$

Where: $f(Q)$ = the probability density function (flood frequency relationship)

For continuous distributions of the flood and loss functions, the economic risk, R, is defined as:

$$R = \int_0^\infty L(Q)f(Q)dQ$$

Where $L(Q)$ = loss function

and $\int_0^\infty f(Q)dQ = 1,\ f(Q) \geq 0.$

The following approximation is utilized to evaluate the economic risk integral:

$$R = \sum_{i=1}^{n} (P_i - P_{i+1})\frac{[L(Q_i) + L(Q_{i+1})]}{2}$$

Where: $L(Q_{n+1})$ is assumed to equal $L(Q_n)$

and $P_{n+1} = 0$ (Probability of infinite flood)

This makes the last term in the above summation equal to $P_nL(Q_n)$.

P_i = exceedance probability of the flood Q_i,

$L(Q_i)$ = dollar damage caused by flood Q_i,

P_n = exceedance probability of the largest flood considered in the analysis, which results in damage $L(Q_n)$.

This approximation has an inherent assumption that the upper end of the integral can be characterized by the largest flood in the analysis. This assumption is acceptable when the flood has very low probability.

The Row Creek example will serve to illustrate the procedure.

The table below summarizes the potential losses for the 440 foot bridge alternative.

Summary of Economic Losses

Q (cfs)	Freq (yr)	Losses: Embankment & Pavement ($)	Traffic Related ($)	Backwater Related ($)	Super-structure ($)	Scour Related ($)	Total Losses ($)
15,000	5	-	-	0	-	-	0
20,000	10	-	-	8,853	-	2,500	11,353
25,000	25	-	-	16,571	-	17,500	34,071
30,000	50	2,000	4,252	19,788	1,320	25,400	52,760
35,000	100	6,373	14,337	25,754	3,080	34,900	84,444
40,000	200	27,542	47,688	22,995	5,720	46,400	150,345

No losses occurred for any flood equal to or less than the 5-year event.

The economic risk associated with each flood event is calculated as follows:

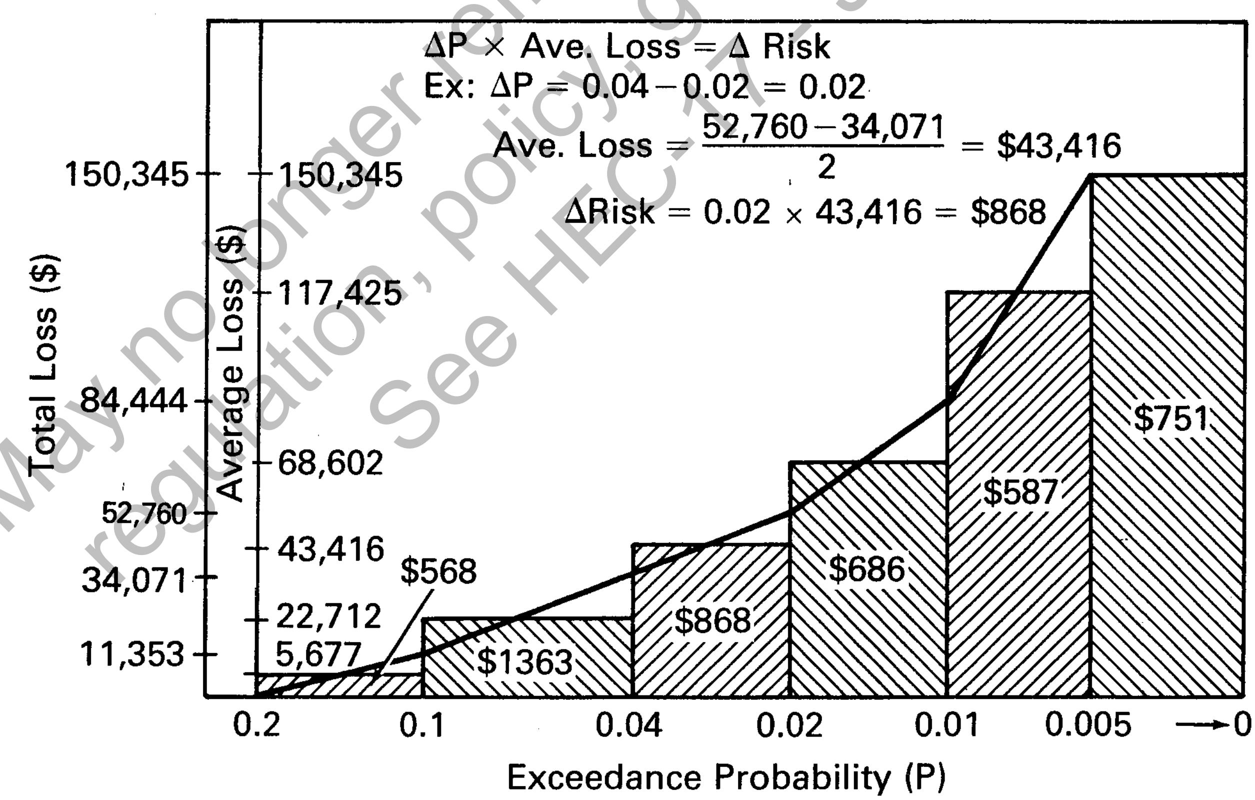
Ecomomic Risk

Q (cfs)	Freq (yr)	Exceedance Probability	Total Losses ($)	Average Loss ($)	Delta Probability	Annual Risk ($)
15,000	5	0.2	0			
				5,677	.10	568
20,000	10	0.1	11,353			
				22,712	.06	1,363
25,000	25	0.04	34,071			
				43,416	.02	868
30,000	50	0.02	52,760			
				68,602	.01	686
35,000	100	0.01	84,444			
				117,425	.005	587
40,000	200	0.005	150,345			
				150,345	.005	751
		0	150,345			

The annual economic risk for the Row Creek crossing is therefore:

$$R = 568 + 1{,}363 + 868 + 686 + 587 + 751 = \$4{,}823$$

A graphical illustration of the procedure for computing the annual ecomomic risk is shown below.



8.3 Total Expected Cost

The sum of the annual capital cost and annual risk cost equals the total expected cost for each design strategy. For the example in the previous section, the TEC is:

Annual Construction Cost	\$120,636
Risk Cost	4,823
TEC	\$125,459

The above procedures illustrate the analysis for one alternative crossing of Row Creek and would need to be repeated for each design alternative to obtain the set of TEC's for selecting the LTEC design. The design goal is to determine that strategy (TEC) which minimizes the total expected cost, i.e., the LTEC design.

9.0 LEAST TOTAL EXPECTED COST DESIGN

Generally in the design of a bridge crossing, the two most important variables are the bridge length (L) and the approach embankment height (H). Assuming an infinite number of combinations of bridge sizes and embankment heights, the total expected cost for each combination is:

$$TEC = R(L,H) + C(L,H)$$

$$= \text{Annual Risk Cost (R)} + \text{Annualized Capital Cost (C)}$$

Plotting all the TEC's would result in the response surface shown in figure 9.1. The optimum or LTEC design would then correspond to the point on the response surface where L and H are a minimum, L_{opt}, H_{opt}.

In practice, the three dimensional surface is replaced by the two families of curves shown in figure 9.2.

In figure 9.2., the TEC's for the various embankment heights are plotted for the various bridge lengths. The LTEC design corresponds to the lowest point on the lowest curve. In many cases, it will be found that the minimum bridge length curve is very flat over a range of embankment heights around the optimum point. In such cases, the designer may wish to recommend a design range rather then a single design. This decision is subjective considering the uncertainties involved in the evaluation process. The overtopping discharge and return interval for the selected design are obtained from the second set of curves on figure 9.2.

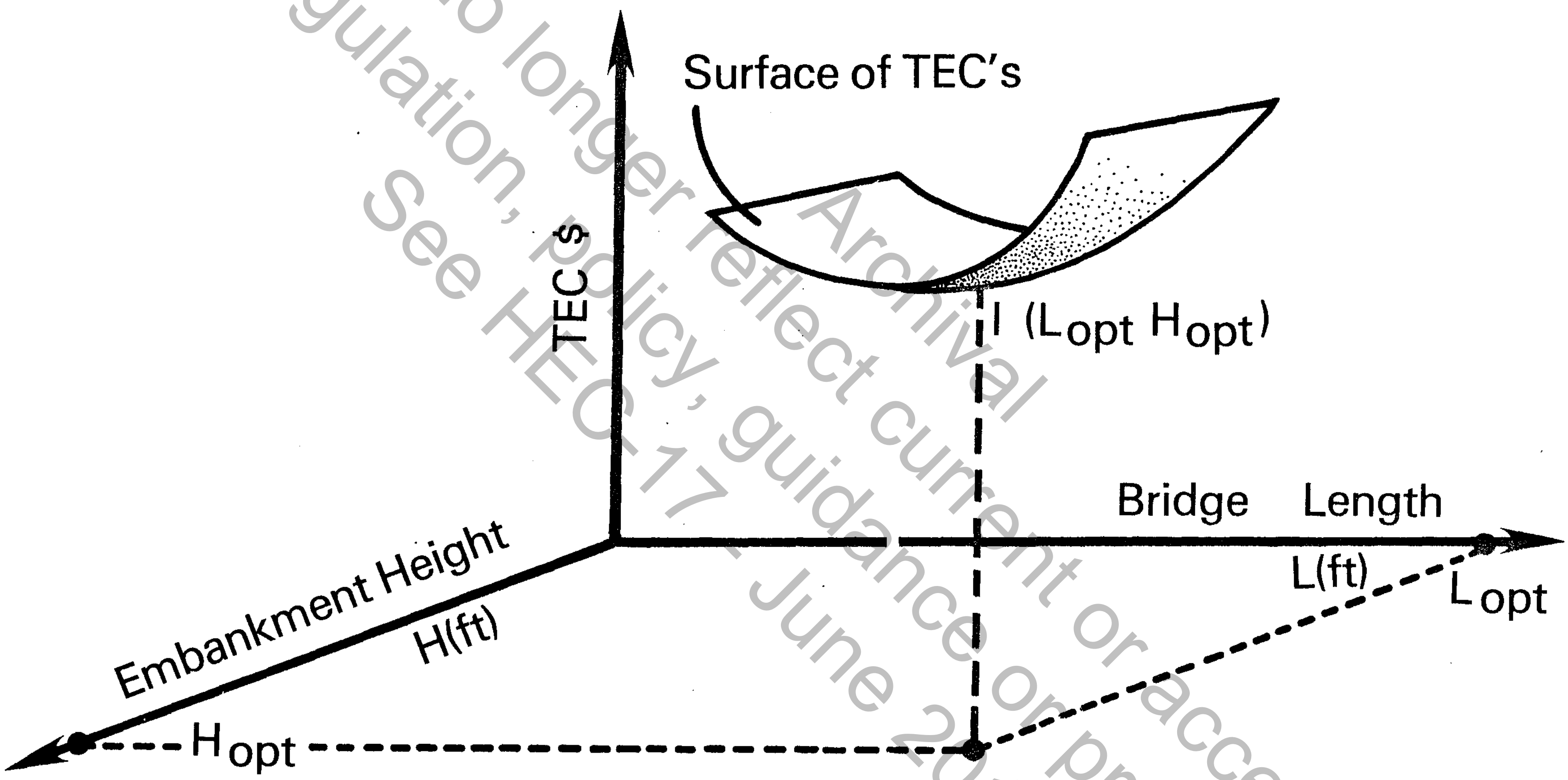


FIGURE 9.1 RELATIONSHIP OF TEC'S TO LTEC DESIGN DECISION

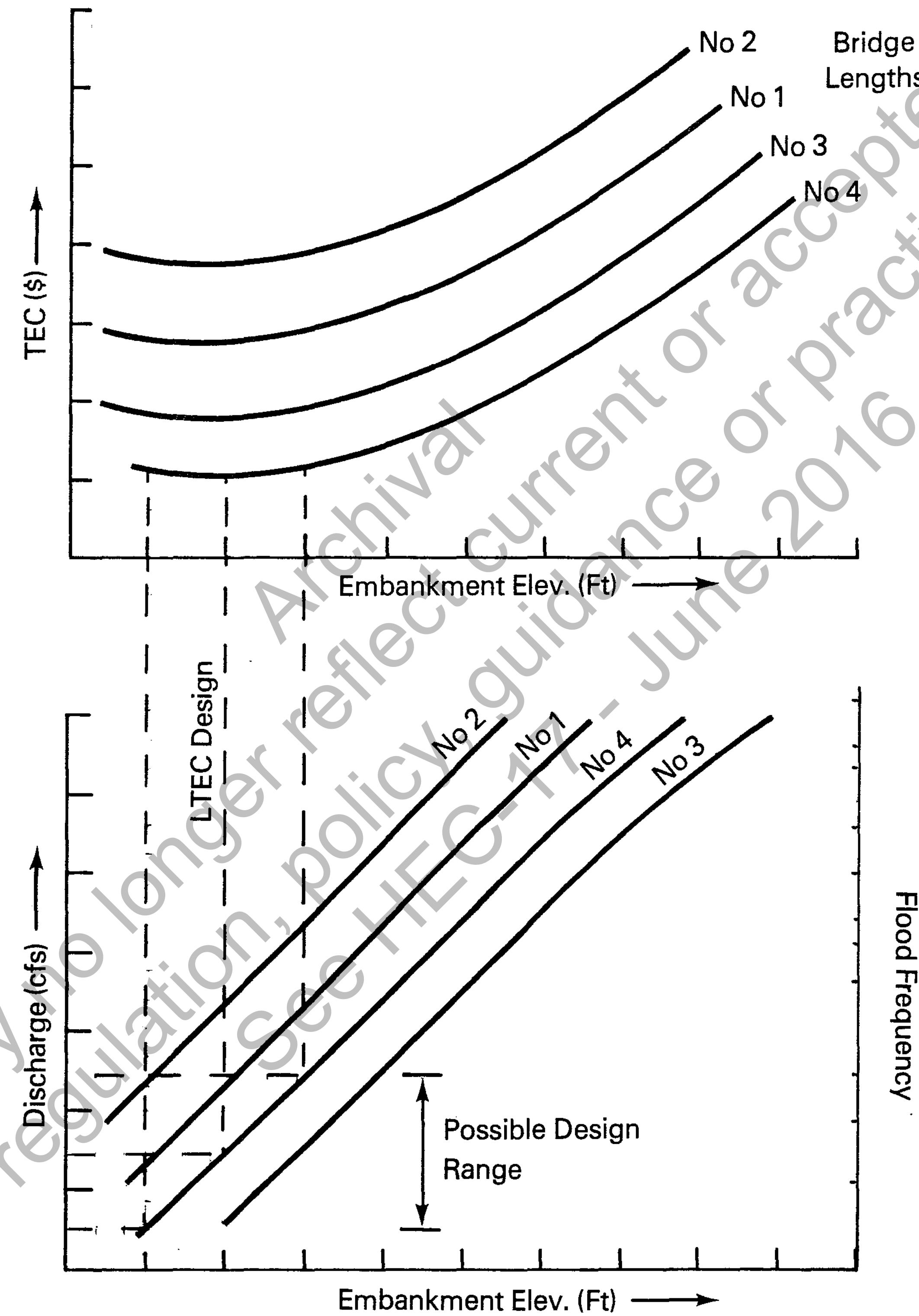


FIGURE 9.2 LTEC AND OVERTOPPING DISCHARGE RELATIONSHIPS

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10.0 SENSITIVITY ANALYSIS

In the development of the methodology for the LTEC decisionmaking process, assumptions are made for several physical processes. For example, estimates are made from available information for such design parameters as embankment losses, traffic level and characteristics, and hydrologic data. These assumptions and estimates are subject to errors. Hence, the use of this information in calculating the total expected cost at a bridge site may induce solution errors, which in turn may affect the decisionmaking process.

The number of parameters involved in the analysis is large and the governing hydrologic and hydraulic processes are complex. Moreover, the economic data used for the analysis varies from site to site and also with time. The uncertainties associated with the analysis need not discourage the use of the method as long as the limitations inherent in the method of analysis are identified. Sensitivity analysis is the means by which these limitations may be assessed.

The sensitivity analysis presented here is a simple process involving changing the variables, one at a time, to determine the relative effect of the variable change on the total expected cost. The optimum design or LTEC design is the baseline for the analysis. Initially, the designer may include all variables in the analysis; however as experience is gained, the designer should be able to emphasize those variables which are most significant.

The designer must exercise judgment in selecting the variable ranges in the analysis. For example, the confidence limits associated with the flood frequency analysis may be defined and input as the range of expected floods. Once the variables are selected and their range determined, the relative effect on the total expected cost is computed.

To illustrate the process, assume that the optimum or LTEC design has an annual expected total cost of \$53,655 and the designer wishes to analyze the sensitivity of the variables involved in the analysis.

The variables included in this sensitivity analysis are:

1. Capital costs - The bridge cost were obtained from a regression analysis of the costs of bridges throughout the United States over a 21-year period. The regression equation is:

   $$\text{Bridge Cost} = a_0 + a_1 x$$

   Where: a_0 = constant

   a_1 = coefficient for lengthh

   x = bridge length

   The roadway cost, RWC, and the embankment cost, CY, are the other factors in the capital cost.

   The coefficients a_0 and a_1 and the RWC and CY values were varied plus and minus 25 percent in the sensitivity analysis.

2. Discount Rate - The discount rate was changed from the baseline value of 7 percent to 5 and 9 percent.

3. Cost of Time - The cost of time was varied from \$2.73 per hour for the baseline to \$3.64 and \$4.57 per hour.

4. Traffic Level - The traffic level was varied from the baseline value of 16000 vehicles per day to 12000 and 20000 vehicle per day.

5. Backwater Damage - The backwater damage was increased by factors of 10 and 100 over the baseline value.

6. Flow Frequency - The baseline flow frequency relationship, figure 10.1, was increased and decreased 25 percent. New probabilities for the flood flows used in the baseline analysis were obtained for these conditions and used in the analysis.

The changes in the TEC resulting from changing the variables in the analysis are shown in table 10.1. Although not included in this illustration the optimum design remained the 300-foot bridge with 315-foot embankment elevation for all the sensitivity analysis runs.

The effects of the various parameter changes on the TEC for the 300-foot bridge with 315-foot embankment elevation were determined by applying the equation:

$$\left(\frac{\text{Delta TEC}}{\text{LTEC}}\right)\left(\frac{\text{V}}{\text{Delta V}}\right)$$

Where: LTEC is the baseline cost,

Delta TEC is the change in TEC with the new value of the variable,

V is the original value of the variable, and

Delta V is the change in the variable.

The results of this analysis are shown in table 10.2. The analysis indicates that the TEC is most sensitive to the capital cost, interest rate and flow frequency and rather insensitive to the traffic, flood loss and cost of time.

It should be noted that even though embankment losses were not considered directly in this analysis, the cost of the roadway and embankment fill are used in determining erosion losses and thus had an effect on the economic risk component of the TEC.

The results obtained from a sensitivity analysis will vary with individual crossings. The purpose of the analysis is to indicate where the greatest benefits can be obtained through additional effort in defining input parameters. Also, with experience gained through applying sensitivity analysis, it should be possible to determine realistic thresholds for use in initial project evaluation to determine the level of study effort which is commensurate with the risks for various site conditions.

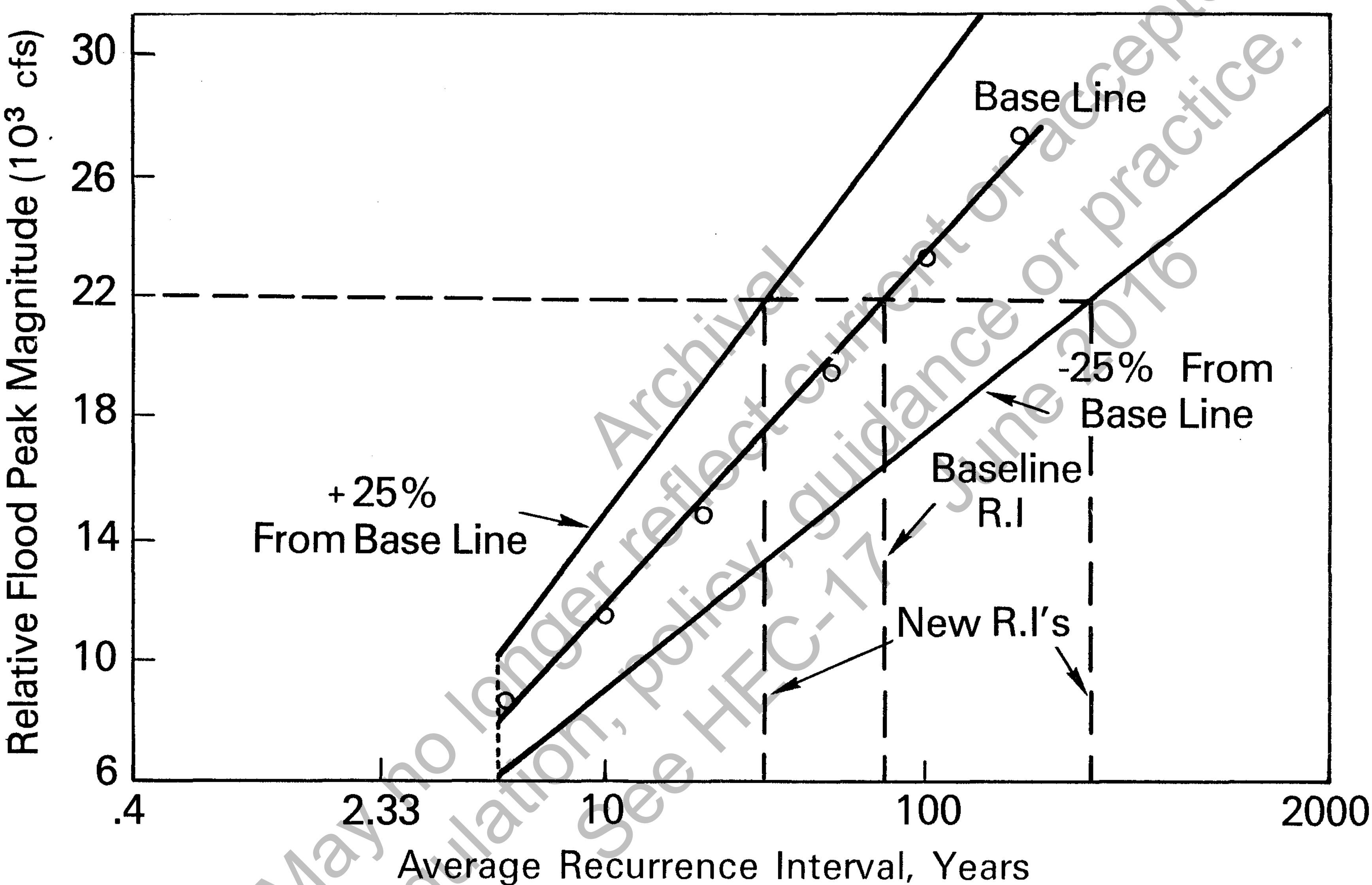

FIGURE 10.1 FLOW FREQUENCY RELATIONSHIP

TABLE 10.1 - TEC Response to Variable Changes

*CAPITAL COSTS				INTEREST RATE	COST OF TIME	TRAFFIC LEVEL	BACKWATER DAMAGE	DELTA PROBABILITY	TEC
$a_0 x10^3$	$a_1 x10^3$	RCW	CY						
4.83	1.547	58.0	2.0	7	2.73	16000	BASELINE	0.01	
								0.02	
								0.01	
								0.01	**53655
6.038	1.934	73.0	3.0	7	2.73	16000	BASELINE	"	69467
3.622	1.16	43.0	1.0	"	"	"	"	"	37844
4.83	1.547	58.0	2.0	5	"	"	"	"	38991
"	"	"	"	9	"	"	"	"	68497
"	"	"	"	7	3.64	"	"	"	53687
"	"	"	"	"	4.57	"	"	"	53720
"	"	"	"	"	2.73	12000	"	"	53490
"	"	"	"	"	"	20000	"	"	53820
"	"	"	"	"	"	16000	x10	"	59875
"	"	"	"	"	"	"	x100	"	122064
4.83	1.547	58.0	2.0	7	2.73	16000	BASELINE	0.02	
								0.03	
								0.022	
								0.028	55217
4.83	1.547	58.0	2.0	7	2.73	16000	BASELINE	0.006	
								0.0087	
								0.0033	
								0.002	52692

* 100 Year Project Life Used in Analysis

**Optimum Design or LTEC design
300-foot Bridge
315-foot Embankment Elevation

TABLE 10.2 - Sensitivity Analysis Results

PARAMETER	LTEC	NEW TEC	DELTA TEC	DELTA TEC / LTEC	DELTA V / V	*RELATIVE RESPONSE
Capital Cost	53655					
+25 percent		69467	15812	0.295	0.25	1.179
-25 percent		37844	15811	0.295	0.25	1.179
Discount Rate (7%)	53655					
5 percent		38991	14664	0.273	0.286	0.956
9 percent		68497	14842	0.277	0.286	0.97
Cost of Time ($2.73)	53655					
$3.64/hr		53687	32	0.0006	0.33	0.0018
4.57/hr		53720	65	0.0012	0.67	0.0018
Traffic Level (16,000)	53655					
12,000 Veh/day		53490	165	0.0031	0.25	0.0124
20,000 Veh/day		53820	165	0.0031	0.25	0.0124
Backwater Damage	53655					
X 10		59875	6220	0.116	9.00	0.0129
X 100		122064	68409	1.275	99.00	0.0129
Probability	53655					
+25 percent		55217	1562	0.0291	0.25	0.1164
-25 percent		52692	963	0.0179	0.25	0.072

$$* \text{ Relative Response} = \left(\frac{\text{DELTA TEC}}{\text{LTEC}}\right)\left(\frac{\text{V}}{\text{DELTA V}}\right)$$

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11.0 SAMPLE REPORT OUTLINE

A suggested outline for a report to document the results of a LTEC design process is shown in the following table. Tables, graphs, computer printouts, check sheets or drawings should be used to document parts of the analysis in order to minimize the time involved in preparing a report.

TYPICAL REPORT OUTLINE

1.0 Introduction - Project Description

2.0 Site Data
- 2.1 General Description of Watershed and Stream
- 2.2 Flood Plain Geometry
  - 2.2.1 Cross Sections
  - 2.2.2 Contours
- 2.3 Land Use
- 2.4 Hydrologic and Hydraulic
  - 2.4.1 Flow Magnitude
  - 2.4.2 Flow Hydrographs
  - 2.4.3 Flow Resistance
- 2.5 Geologic
  - 2.5.1 Surface Geology
  - 2.5.2 Channel Morphology
  - 2.5.3 Soils Information
  - 2.5.4 Scour History

3.0 Selection of Alternative Designs

4.0 Analysis Considerations
- 4.1 Useful Life of Facility
- 4.2 Discount Rate
- 4.3 Loss of Life
- 4.4 Facility Failure
- 4.5 Data Limitations
- 4.6 Other Considerations
- 4.7 Cost Data

5.0 Cost Data
- 5.1 Structure Cost
- 5.2 Cost of Protective Measures
- 5.3 Embankment and Pavement Repair Costs
- 5.4 Traffic Related Costs
- 5.5 Property Values

6.0 Hydrologic and Hydraulic Data Analysis
- 6.1 Flow Frequency/Probability
- 6.2 Water Surface Profiles
  - 6.2.1 Existing Conditions
  - 6.2.2 With Alternative Designs
- 6.3 Stage-Discharge Relationships
  - 6.3.1 Existing Conditions
  - 6.3.2 With Alternative Designs
- 6.4 Hydrographs
  - 6.4.1 Time of Overtopping
  - 6.4.2 Depth of Overtopping
- 6.5 Flow Distribution

7.0 Economic Losses Analysis
- 7.1 Structural Losses
  - 7.1.1 Superstructure
  - 7.1.2 Substructure
- 7.2 Embankment and Pavement Losses
- 7.3 Traffic Related Losses
  - 7.3.1 Traffic Restoration Time
  - 7.3.2 Running Cost
  - 7.3.3 Time Cost
  - 7.3.4 Accident Losses
- 7.4 Backwater Damage Losses
- 7.5 Other Losses

8.0 Total Expected Cost
- 8.1 Capital Cost
- 8.2 Risk Costs
- 8.3 Total Expected Cost

9.0 Selection of Least Total Expected Cost Design

10.0 Sensitivity Analysis

11.0 Summary and Conclusions

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APPENDICES

EXAMPLE PROBLEMS

Example Problem A - RISK ANALYSIS SAMPLE PROBLEM - U.S. 11 CROSSING LEAF RIVER AT HATTIESBURG, MISSISSIPPI

1.0 Project Descripton

U.S. Highway 11, previously a national artery between New Orleans and Washington, D.C., became a local highway after construction of Interstate Highway 59. That segment of U.S. Highway 11 crossing Leaf River in Hattiesburg is now a city artery and has a traffic volume exceeding 12,000 vehicles per day.

The existing bridge is 989 ft long with roadway (embankment) elevation at the flood plain level, 147 ft elevation.

The bridge is being replaced. The purpose of this analysis is to determine the optimum bridge length/embankment elevation combination. (The highway department selected a 1000 ft bridge and a 149 embankment elevation.) The bridge site is shown on the State road map in figure A.1.

2.0 Site Data

2.1 General Description of Stream

The bridge site is over the Leaf River approximately 400 feet downstream of the confluence of the Leaf River and the Bowie River. The total drainage area at the bridge site is 1760 sq miles of which 660 sq miles is contributed by the Bowie River Watershed. Leaf River is a moderately meandering stream which has a well-defined 400-ft-wide main channel and a 7000 to 9000-foot-wide valley. The main channel conveys most of the flood flows and conveys the 17-year frequency flood (66,000 cfs) at bankfull stage. The Leaf River valley slope is approximately 2.5 ft/mi, and the Bowie river valley slope is around 3 ft/mi. Gravel and sand mining operations exist at several sites in the Leaf and Bowie River flood plains upstream and downstream of the bridge site.

2.2 Flood Plain Geometry

Seven cross sections (figures A.7A - A.7G) were taken at locations marked on the aerial photograph (figure A.2). A Southern Railway bridge is located between sections 2 and 3; the highway bridge is located between sections 4 and 5; and the residences subject to backwater damage are located between sections 5 and 6.

A 10-ft contour interval quadrangle map and flood insurance records of building elevations were used in lieu of a detailed contour map.

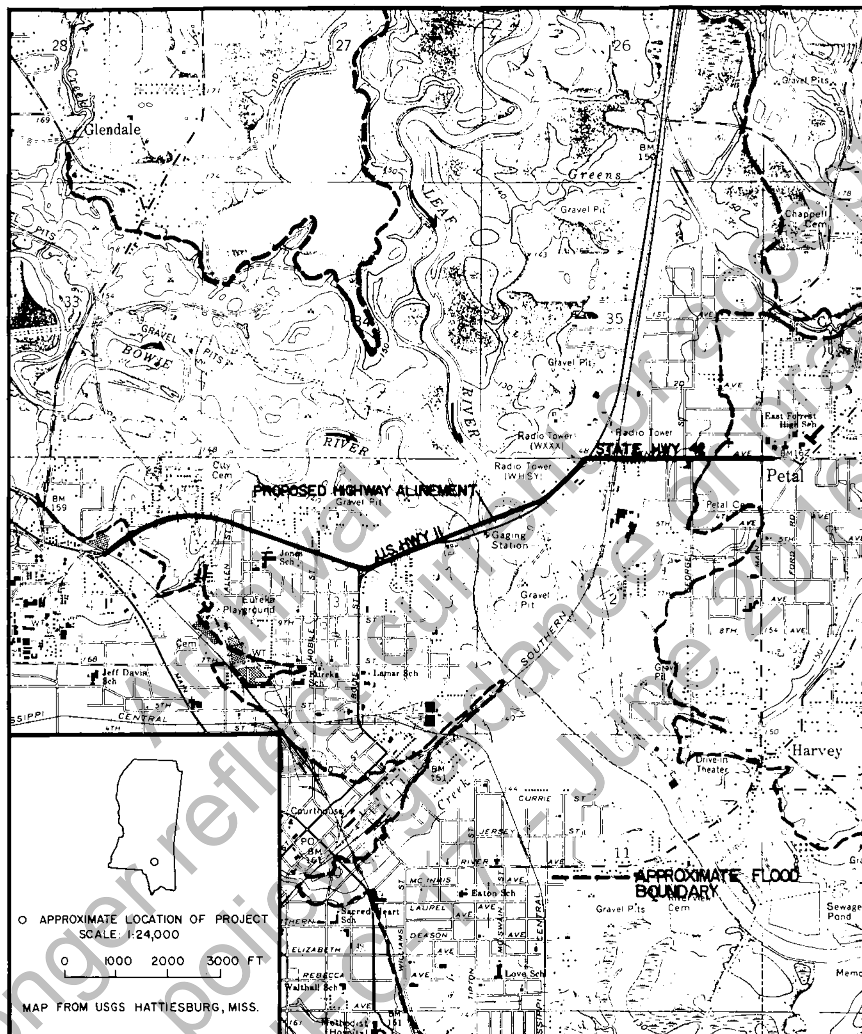

FIGURE A.1 GENERAL LOCATION MAP, LEAF RIVER AT U.S.HIGHWAY 11 HATTIESBURG, MISS.

FIGURE A.2 LOCATION OF THE CROSS SECTIONS USED IN THE HYDRAULIC ANALYSIS

2.3 Land Use

The valley is approximately half open and half wooded, with cleared fields and pasture land occasionally reaching the banks of the river. In the immediate vicinity of U.S. Highway 11, the flood plain is mostly cleared upstream and downstream. There is a strip of woods 500 to 1000 ft wide along the east bank both upstream and downstream and along the west bank downstream. The crossing is located in the developed areas of Hattiesburg and Petal and there are numerous houses which may be affected by backwater created by the proposed crossing as shown by the aerial photograph (figure A.2).

2.4 Hydrologic and Hydraulic Data

2.4.1 Flow Magnitude

The U.S. Geological Survey has operated a countinuous-recording gaging station at this site since 1939 and the National Weather Service has operated a gage at this site since 1904. Hydrologic data are summarized in table A.1 below.

Table A.1 Summary of Hydrologic Data

Flood	Prob. of Exceedence in a given year	Q (cfs)	Crest Elev. (ft)	Comment
April 1974	0.6%	121,000	152.3	flood of record
April 1980	-	-	151.9	2nd highest flood of record
April 1919	-	87,900	-	-
April 1921	-	82,800	-	-
April 1943	-	71,300	-	-
Feb. 1961	4.2%	72,200	149.8	-
Q_{50}	2%	90,500		at bridge site
Q_{100}	1%	110,000		at bridge site
Q_{50}	2%	59,000		Leaf River above confluence
Q_{100}	1%	70,000		Leaf River above confluence

2.4.2 Hydrographs

The shape of the 1974 flood hydrograph was determined to be typical by comparison with other extreme floods and was used as a pattern for estimating hydrographs (figure A.5) for other floods. These hydrographs were used in analyzing the periods of overtopping and resulting damages for the various alternatives.

2.4.3 Flow Distribution

The flow distribution was calculated based on the relative conveyance of the highway centerline cross section. The distribution is made by the step-backwater method computer program using Manning's equation.

During the 2-percent chance flood, 54,700 ft^3/sec is in the main channel; 22,500 ft^3/sec is distributed on the west flood plain, and 13,300 ft^3/sec is on the east flood plain. During the 1-percent chance flood, 58,200 ft^3/sec is in the main channel; 32,000 ft^3/sec is on the west flood plain, and 14,800 ft^3/sec on the east flood plain. According to these figures, approximately 53-percent of the 1-percent chance flood is confined to the main channel.

2.5 Geologic Data

2.5.1 Surface Geology

Surface geology in the vicinity is the Hattiesburg formation which consists of perhaps 200 ft of clay, sandy-clay, sand and gravel, with some thin ferruginous (containing iron) layers. The beds of sand and gravel are capable of conveying large quantities of ground water.

2.5.2 Channel Morphology (see "General Description of Stream")

2.5.3 Soils Information

The steambed is composed of dense sand and gravel, and the banks are formed of medium dense sand with silty-clay overburden 5 to 10 ft thick. A layer of hard blue silty clay outcrops in the streambed a few hundred feet downstream.

2.5.4 Scour History

The channel position has been generally stable since gaging records began in 1939. Scour and fill of as much as 10 to 12 ft has occurred in the channel and especially along the west edge of the channel.

A large scour hole usually exists at the confluence of Leaf and Bowie Rivers 400 ft upstream from the site. Turbulence created by the mixing of the water from the two streams creates a potential scour problem just upstream from U.S. Highway 11. The surveyed centerline of the proposed crossing about 150 ft upstream from the existing crossing showed a bottom elevation of 104 ft compared to 116 ft at the bridge site. These data indicate that the proposed crossing may be on the edge of a scour hole created by the confluence of the two streams.

3.0 Selection of Design Alternatives

Thirty design alternatives were selected for the analysis. The design alternatives included the combinations of six bridge lengths (280', 440', 600', 800', 1200' and 1600') and five embankment elvations (147', 149', 151', 153', and 155'). The lowest embankment elevation is at flood plain level.

4.0 Analysis Considerations

The following assumptions were used for the analysis:

4.1 The useful life of the structure - 25 years
4.2 The discount rate - 7 1/8%
4.3 The loss of life is the same for the detour and for the normal route and can be neglected in the analysis.
4.4 The foundations are set to withstand the maximum expected flood so that the bridge itself will not fall.

5.0 Construction Costs

5.1 Structure Costs

Construction costs were estimated by the Mississippi Department of Highways. Costs included spur dikes and abutment protection as required according to the velocity results of the hydraulic computations. Costs also include extra foundation expenses required to allow for contraction and pier scour for the smaller openings. Cost data are in table A.8.

5.2 Cost of Protective Measures

Bank protection for this site was estimated to cost $500,000.

5.3 Embankment and Pavement Repair Costs

Repair costs were estimated from the following equation:

Repair costs = Mobilization Cost + [(EMB Vol)(Pemb)(UCemb) + (Pavement Area)(Ppave)(UCpave)]Ca

The following assumptions were used:

Mobilization Cost = $375
EMB Vol = AX(7000 - BR. Length) in CY
AX = 0.916 CY/ft for Emb Elev = 147'
AX = 4.0 CY/ft for Emb Elev = 149'
AX = 8.88 CY/ft for Emb Elev = 151'
AX = 14.67 CY/ft for Emb Elev = 153'
AX = 21.33 CY/ft for Emb Elev = 155'
Pavement Area = 2.667 x (7000 - BR length) in SY
UCemb = $2.00/CY
UCpave = $14.40/SY
Ca = 1.0 (adjustment coef. for accelerated repairs)

5.4 Traffic Delay Costs

A suggested route for travel from Hattisburg to Petal when U.S. 11 is out of service for any reason is as follows:

(1) Turn left on U.S. 49 at the intersection with U.S. 11,
(2) take I-59 North at its intersection with U.S. 49,
(3) exit I-59 at the Moselle exit, proceeding to U.S. 11,
(4) turn right at U.S. 11 and proceed to Petal.

The total length of the detour is 25 miles or 23 miles longer than the normal route.

The following traffic data were used in the analysis:

Present ADT = 12,680 vehicles
Projected ADT = 12,680 vehicles
Speed on detour = speed on normal route = 55 mph
Length of detour = 25 miles
Length of normal route = 2 miles
Mileage cost = $.20/mile
Value of lost time = $3.60/hour per occupant
Occupancy rate = 1.25 occupants per vehicle
Increased accident costs are negligible

Using the above figures, the total additional cost of moving 12,680 vehicles on the detour is $82,189 per day, or $3424 per hour.

The detour time is the sum of the overtopping time, t_{ot}, and the traffic restoration time, t_{tr}.

$$\text{detour time} = t_{ot} + t_{tr}$$

For this example, the traffic restoration time was estimated from:

$$t_{tr} = (\text{No. full days to repair emb})24 + (\text{partial days to repair emb})10 + (\text{No. full days to repair pave})24 + (\text{partial days to repair pave})10$$

Where: days to repair emb = emb vol(P_{emb}/Rate emb)
days to repair pave = pave area(P_{pave}/Rate pave)

Assume: 10 hour/work days
Rate emb = 2000 CY/day
Rate pave = 3000 SY/day

Traffic losses are computed by multiplying $3424 per hour by detour time in hours.

5.5 Property Values and Damage Costs

There are 301 residences which could be damaged due to backwater from the bridges. Velocities on the flood plain in the vicinity of the residences are low (less than 1 fps) therefore, damage from velocity was not considered. Data were collected on types of residences, first floor elevation, location in the flood plain, and value of the residence. These data along with the relationships between flood depth and damages shown in figure 7.2 were used to compute damages to the structures and contents caused by backwater flooding. For this example, all residences were assumed to have basements. Backwater flood damage is the difference between the damage with the highway crossing in place and the natural condition (no highway).

A computer program developed by the Corps of Engineers can be used to estimate backwater damage. The program requires first floor elevation, location, and the value of each residence. The results are in terms of benefits from a flood control project rather than in terms of damages since the Corps' flood control projects are assumed to be beneficial. To use the program from computing bridge backwater damages, the worst bridge is used as the program "EXISTING CONDITION." Damages are then related to the worst case. Even without the computer program, backwater damage can be estimated with a reasonable amount of effort. First flood elevations are summarized in Table A.2.

Table A.2. Summary of 1st Floor Elevations

Bracketing Cross Sections	1st Floor Elev. Truncated to Nearest Ft.	Number of Houses	Total Value (Approx. $37,500 each)
5-6	148	7	$ 262,500
(see figure A.2)	149	42	1,575,000
	150	55	2,062,500
	151	64	2,400,000
	152	73	2,737,500
	153	43	1,612,500
	154	16	600,000
	155	1	37,500

Total Number of Houses = 301

Houses were grouped at incremental elevations as single units with values equal to the total of all the houses within the incremental elevation. Since all the houses are between sections 5 and 6 in this example, average water surface elevations between those two sections were used to compute the backwater damage.

6.0 Hydrologic and Hydraulic Data Analysis

6.1 Flow Frequency and Probability

The Log-Pearson Type III flood frequency analysis was used to analyze the hydrologic data. Results of the Log-Pearson type III analysis are plotted in figure A.3. Based on this analysis the following floods were selected for the study.

Q(cfs)	Frequency (yr)	Probability
54,200	10	.10
62,000	15	.067
68,000	20	.05
73,600	25	.04
90,500	50	.02
110,000	100	.01
121,000	160	.00625
131,000	200	.005
164,000	500	.002

6.2 Water Surface Profiles

The USGS step backwater program (E431 by Shearman 1976) was used for hydraulic computations for discrete discharges on the discharge-frequency curve (discharges ranged from 54,200 cfs to 164,000 cfs). The computations included the natural condition and each bridge length (ranging from 280 ft to 1600 ft) at each embankment elevation (ranging from 147 ft to 155 ft). Average water surface elevations, between sections 5 and 6, where residences are located, are in Table A.3.

6.3 Stage-Discharge Relationship

The USGS has operated a continuous-record gaging station just downstream of the bridge site since 1939. Figure A.4 is the downstream partial rating curve for the bridge site. The downstream rating curve is a close approximation to the natural rating curve and can be used to estimate overtopping depths according to the short cut method described in section 6 of this manual.

The upper portion of this rating curve is approximated closely by the following regression equation:

$$Elev = 136.4618 + 0.1906(Q/1000) - .000489(Q/1000)^2$$

Using this equation in lieu of the rating curve to estimate overtopping depths yields:

$$d_{ot} = \text{Elev at } Q - \text{Elev at } Q_{ot}$$

$$= 0.1906(Q/1000 - Q_{ot}/1000) - .000489[(Q/1000)^2 - (Q_{ot}/1000)^2]$$

6.4 Hydrographs

The family of stage hydrographs for this site (shown in figure A.5) was obtained by drawing hydrographs similar to the one measured hydrograph for the 50-year flood. Since they are similar by construction, the shapes near the peak are similar; therefore, a normalized hydrograph shape relating time of overtopping to depth of overtopping could be derived as illustrated in figure A.6. Using regression equations to approximate the curves yields the following equation for the time of overtopping:

$$t_{ot} = 28.5\ d_{ot}^{0.52}$$

The equation for d_{ot} and t_{ot} are site specific relationships for this example only. They were used for computational expedience since there were so many design alternatives in this example.

7.0 Economic Losses Analysis

7.1 Structural Losses

Structural losses for this site were negligible.

7.2 Embankment and Pavement Losses

Embankment and pavement losses (roadway repair costs) are computed in table A.4.

7.3 Traffic Delay Losses

Table A.4 is a work table for computing traffic losses and repair costs for expected damages to the embankment and pavement. Traffic losses and repair costs are both influenced by the depth and time of overtopping so they naturally go together.

7.4 Backwater Damage Losses

Table A.5 is a typical detailed work table for backwater damages. The last column (Delta Damage) of this table is determined by substracting the total damage for the natural condition from the total damage for a given bridge configuration. The detailed calculations of table A.5 were programmed on a pocket calculator so that the Delta Damage column could be calculated directly by computing initial storage values and average water surface elevations for each bridge configuration. Table A.6 is the summary table for backwater damages (the Delta Damages) calculated with a pocket calculator.

7.5 Other Losses

Table A.7 is a summary of "Other" losses which in this case were limited to loss of abutment protection when velocities through the bridge opening exceeded 9 ft/sec. Half the abutment protection was assumed to be lost when velocities were between 9 and 10 ft/sec; all of it was assumed to be lost when velocities exceeded 10 ft/sec.

8.0 Total Expected Cost

The total expected cost (TEC), for each bridge configuration consists of four risk costs and the annual capital cost. Tables A.4 - A.8 were used to generate the total expected cost data.

8.1 Capital Costs

Table A.8 is a summary of the capital cost for each bridge configuration. The total construction costs were multiplied by the capital recovery factor (CRF = 0.08678 for a 25-year service life and a 7 1/8% discount rate) to determine the annual capital cost.

8.2 Economic Risk Costs

The risk cost tables (A.4 - A.7) all have a similarity in format in that all have a risk cost associated with each discharge selected for the analysis. These costs are discrete points of the probability loss function that must be integrated to get the TEC components. Table A.9 shows the process used to approximate the integral. Again, the computations of this table were programmed on a pocket calculator so that the TEC components (the totals at the bottom of Table A.9) could be computed directly by inputting initial values of probabilities and individual cost values from the cost tables.

8.3 Total Expected Cost

Table A.10 is the TEC table used to analyze alternatives. The total expected cost is the sum of the annual capital cost and the four risk costs. Table A.10 also includes the overtopping flows for each configuration. The overtopping flows were determined by interpolating between the water surface elevations just upstream of the bridge determined from the hydraulic analysis.

9.0 The LTEC Design

Figure A.8 is a graphical representation of the TEC table. The least TEC (LTEC, design) is a 440' bridge with a 149' embankment elevation. The best 50-year design would be 440' bridge with a 150.5' embankment elevation, but, the delta cost for the 50-year design would be $19,000/yr over the LTEC design. The present worth of this delta cost is $218,900 ($19,000/CRF). The LTEC design will be overtopped by a flood with a 32-year return interval.

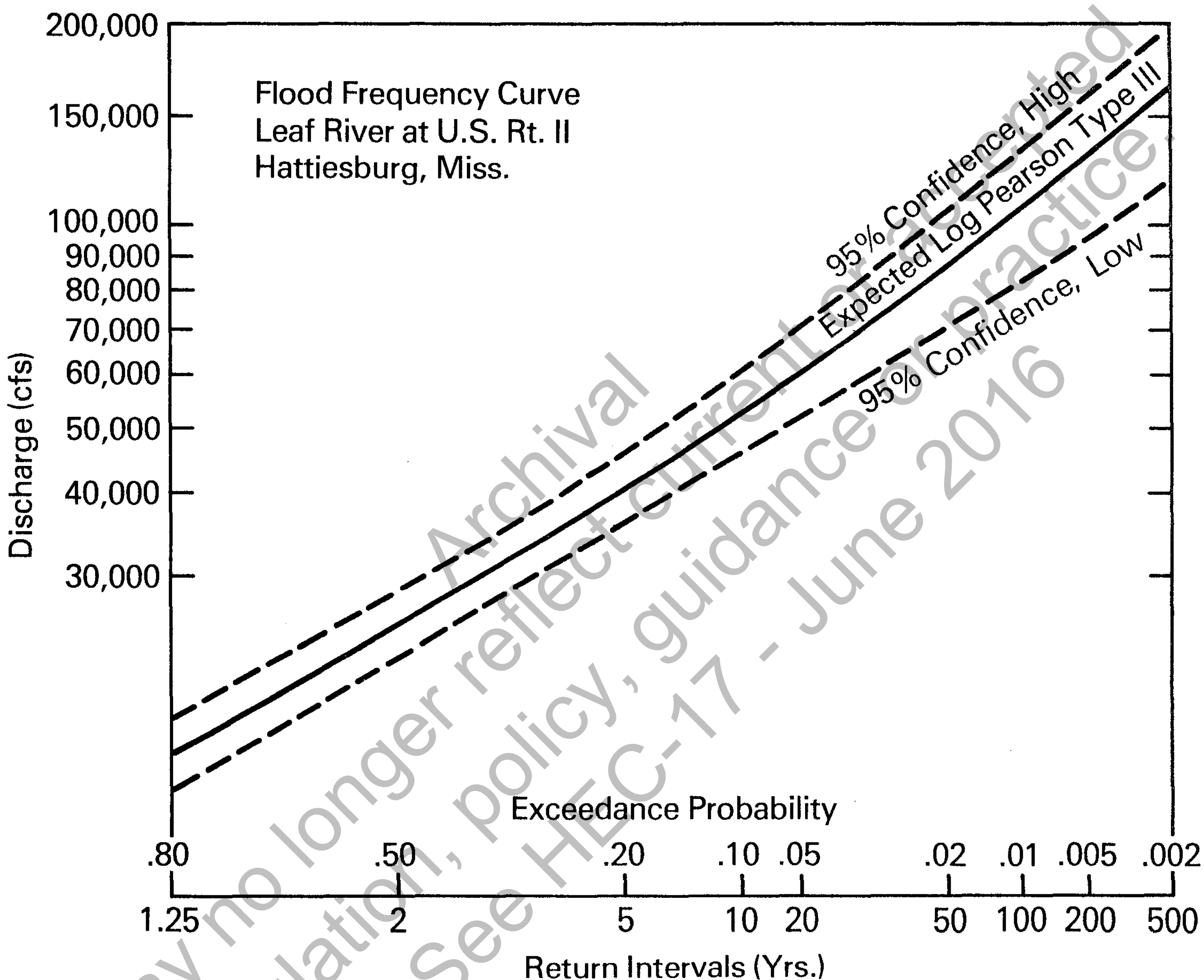

FIGURE A.3 DISCHARGE FREQUENCY CURVE

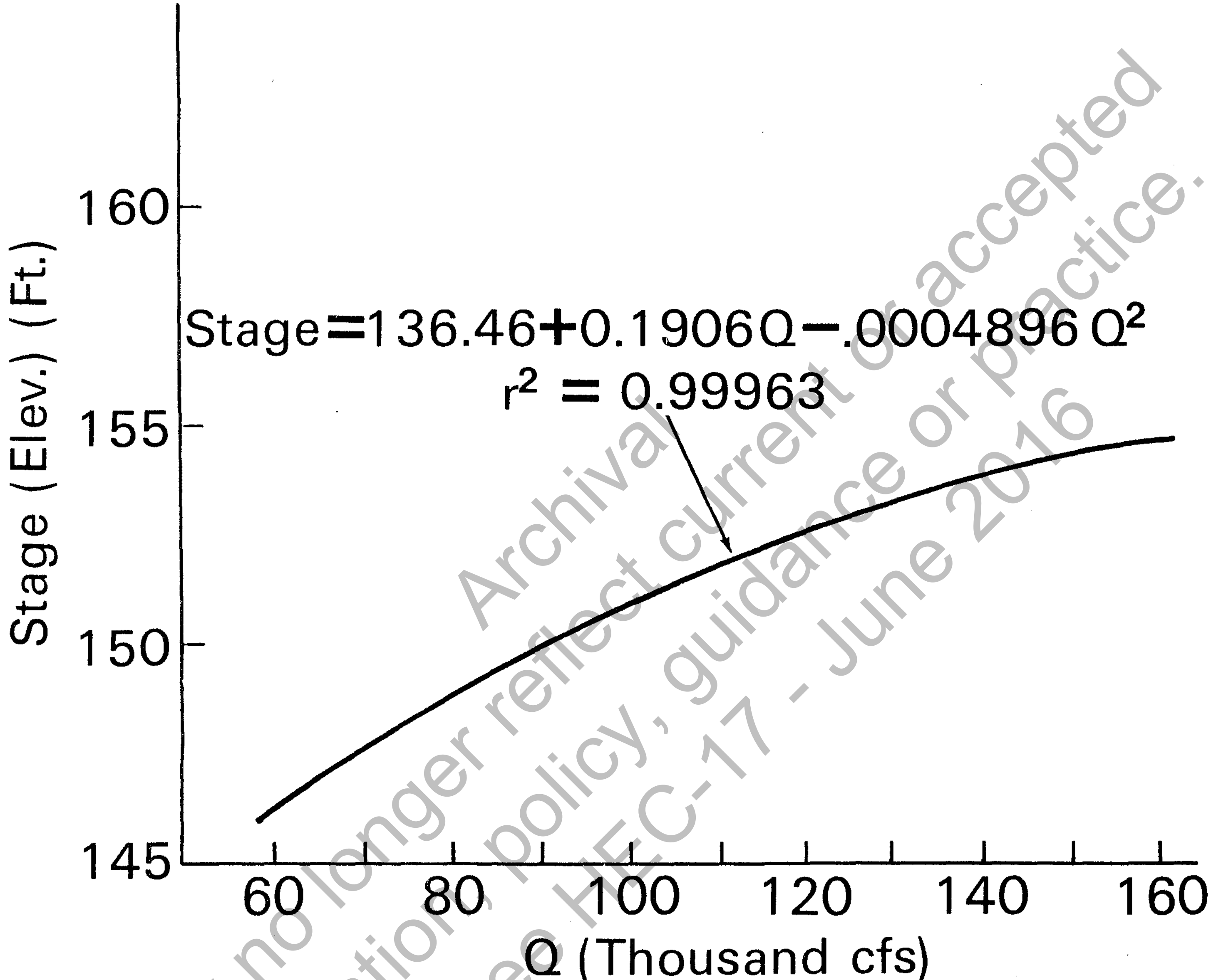

FIGURE A.4 RATING CURVE (UPPER END)

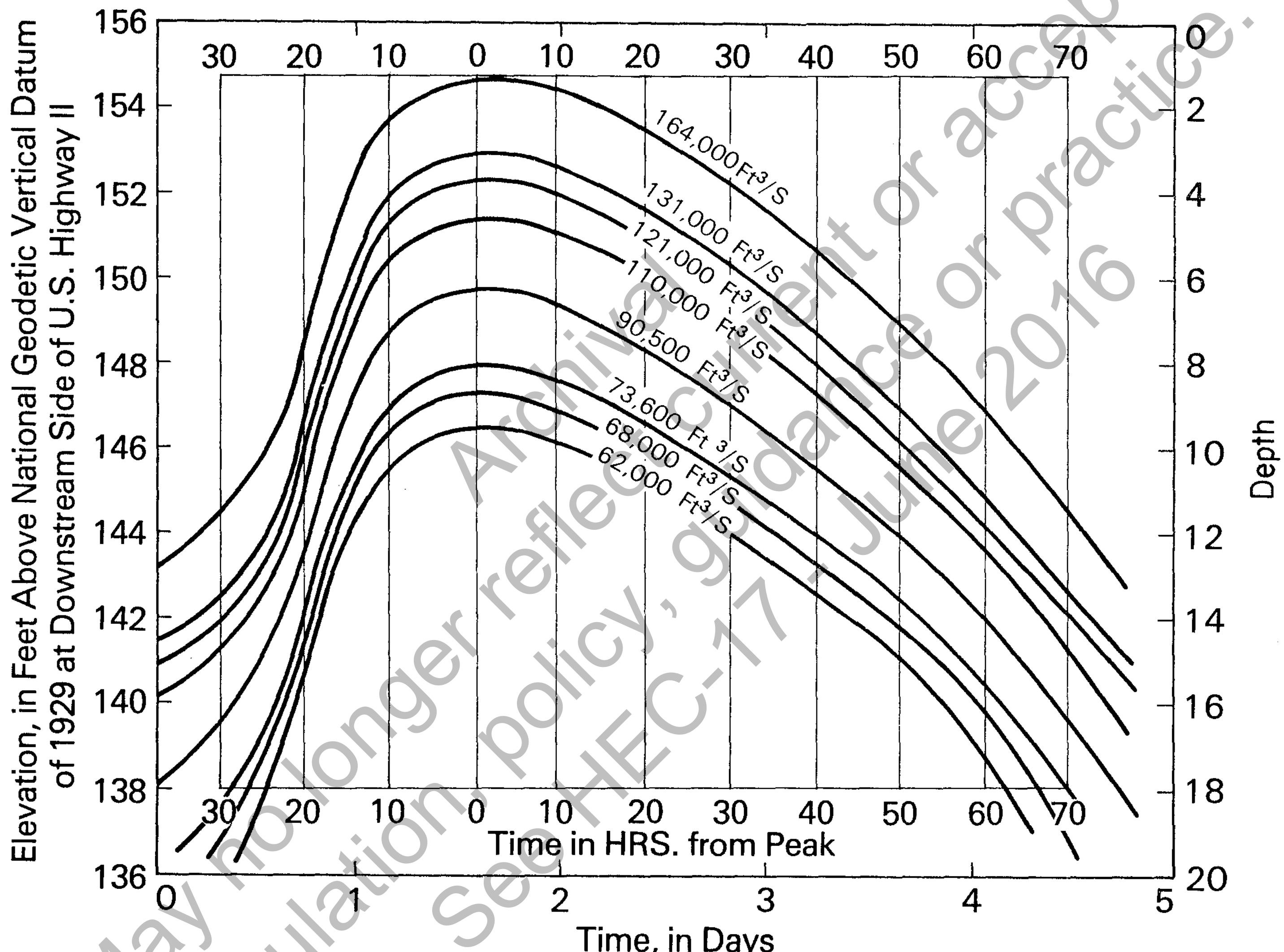

FIGURE A.5 STAGE HYDROGRAPH, LEAF RIVER AT U.S. HIGHWAY 11
AT HATTIESBURG, MISSISSIPPI

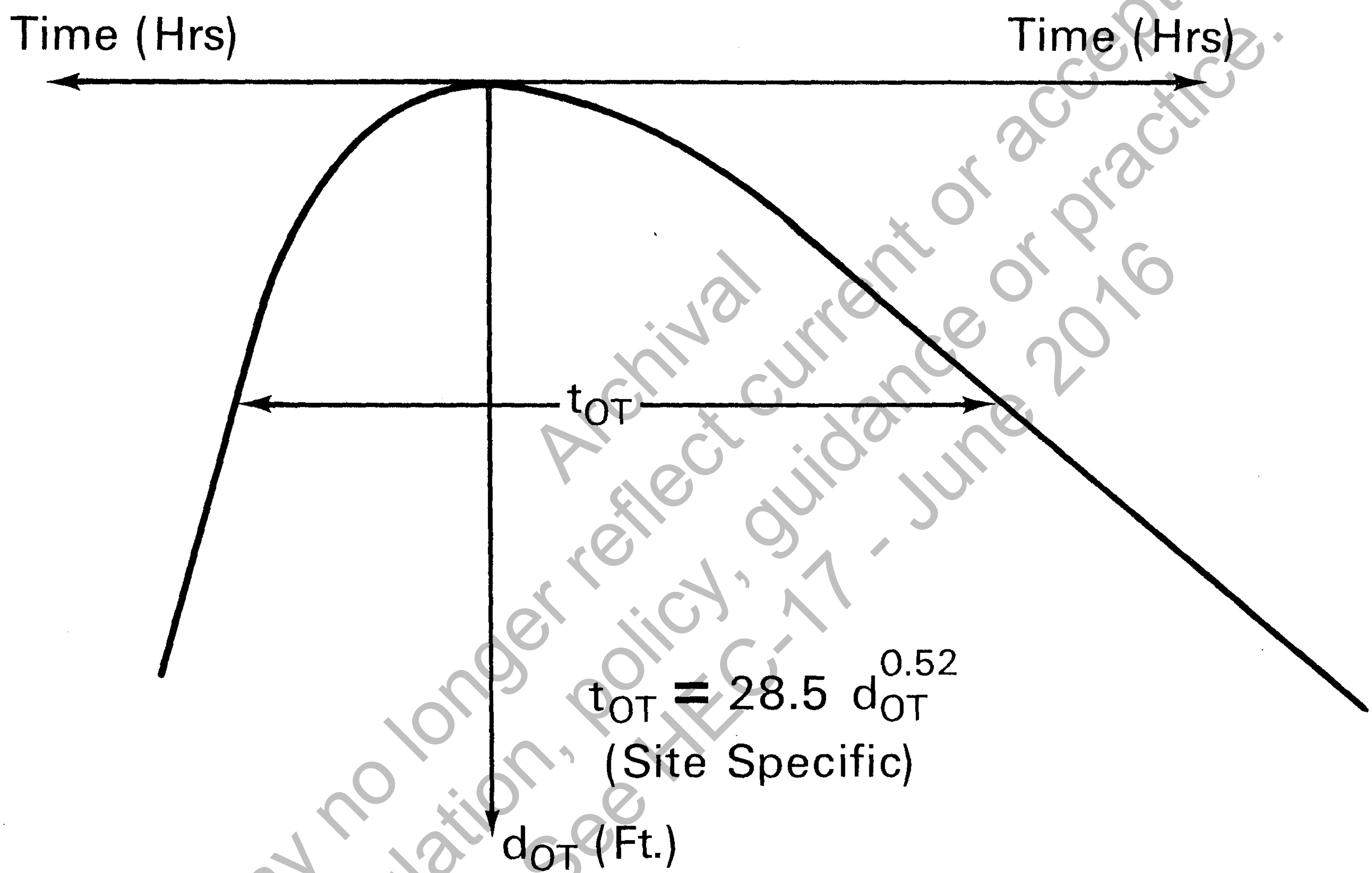

FIGURE A.6 SINGLE HYDROGRAPH USED TO ESTIMATE OVERTOPPING TIMES

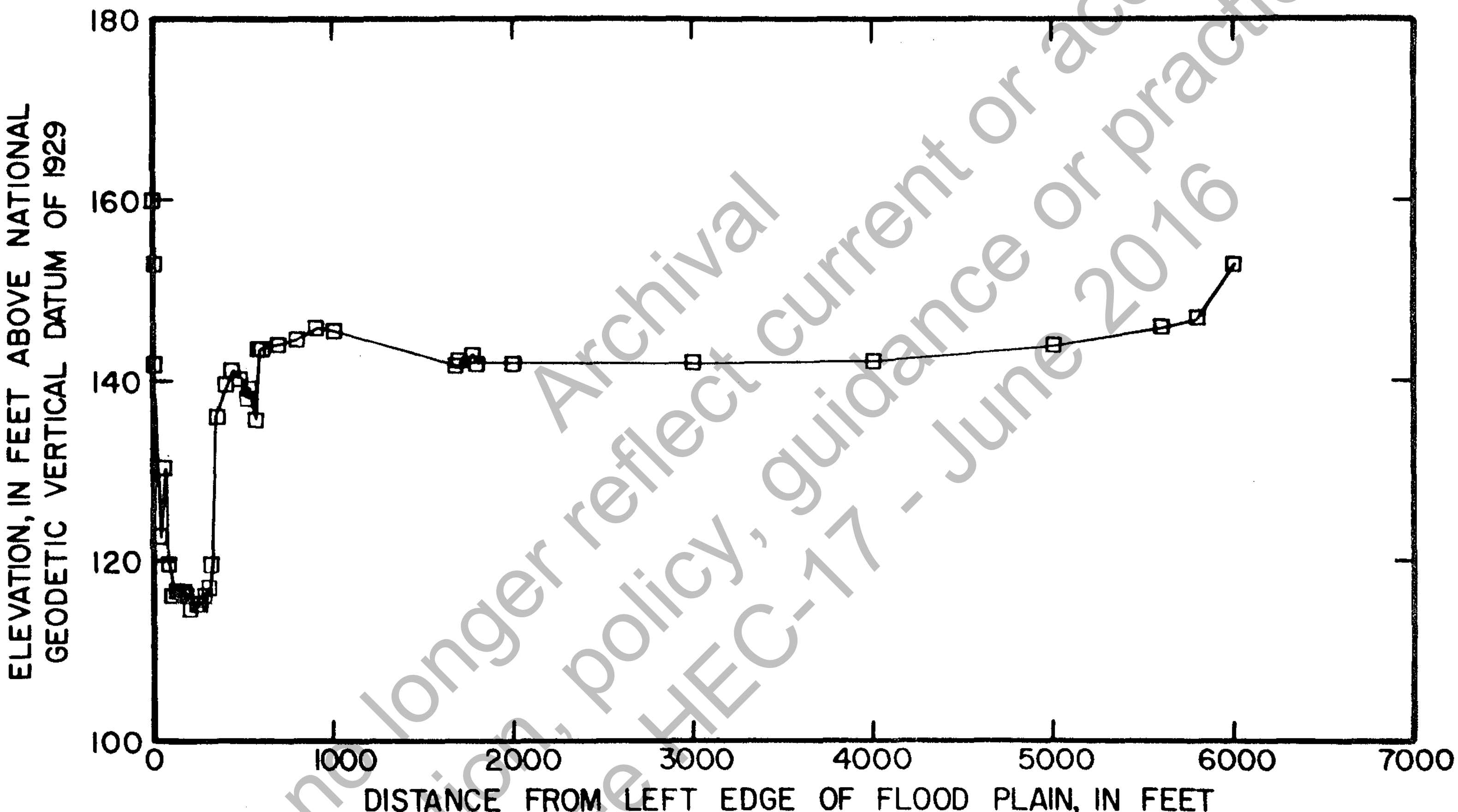

FIGURE A.7A CROSS SECTION 1 LOCATED UPSTREAM FROM RIVER AVENUE (STATION 0)

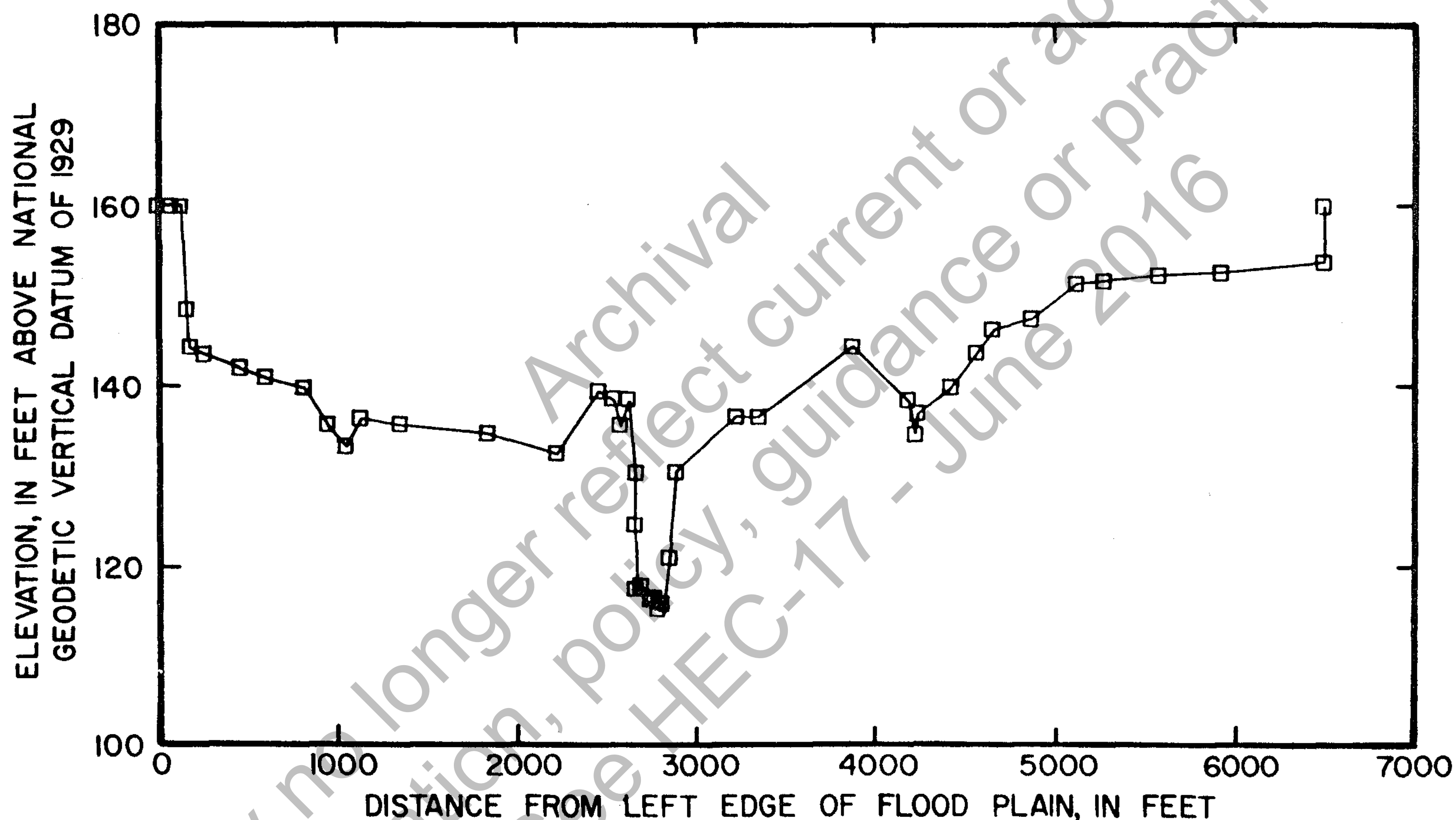

FIGURE A.7B CROSS SECTION 2 LOCATED DOWNSTREAM FROM THE SOUTHERN RAILROAD BRIDGE (STATION 3,300 FT)

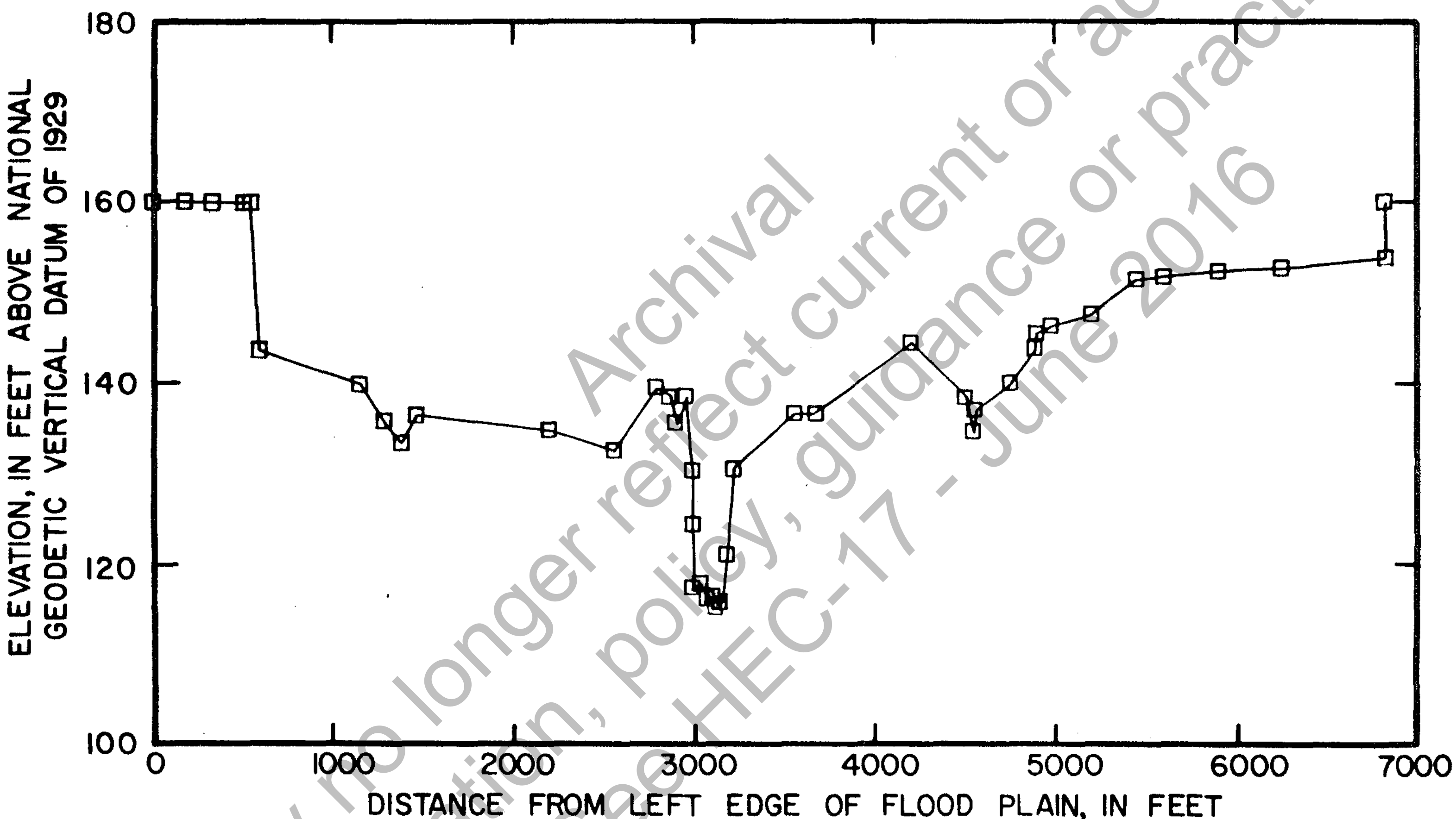

FIGURE A.7C CROSS SECTION 3 LOCATED UPSTREAM FROM THE SOUTHERN RAILROAD BRIDGE (STATION 4,500 FT)

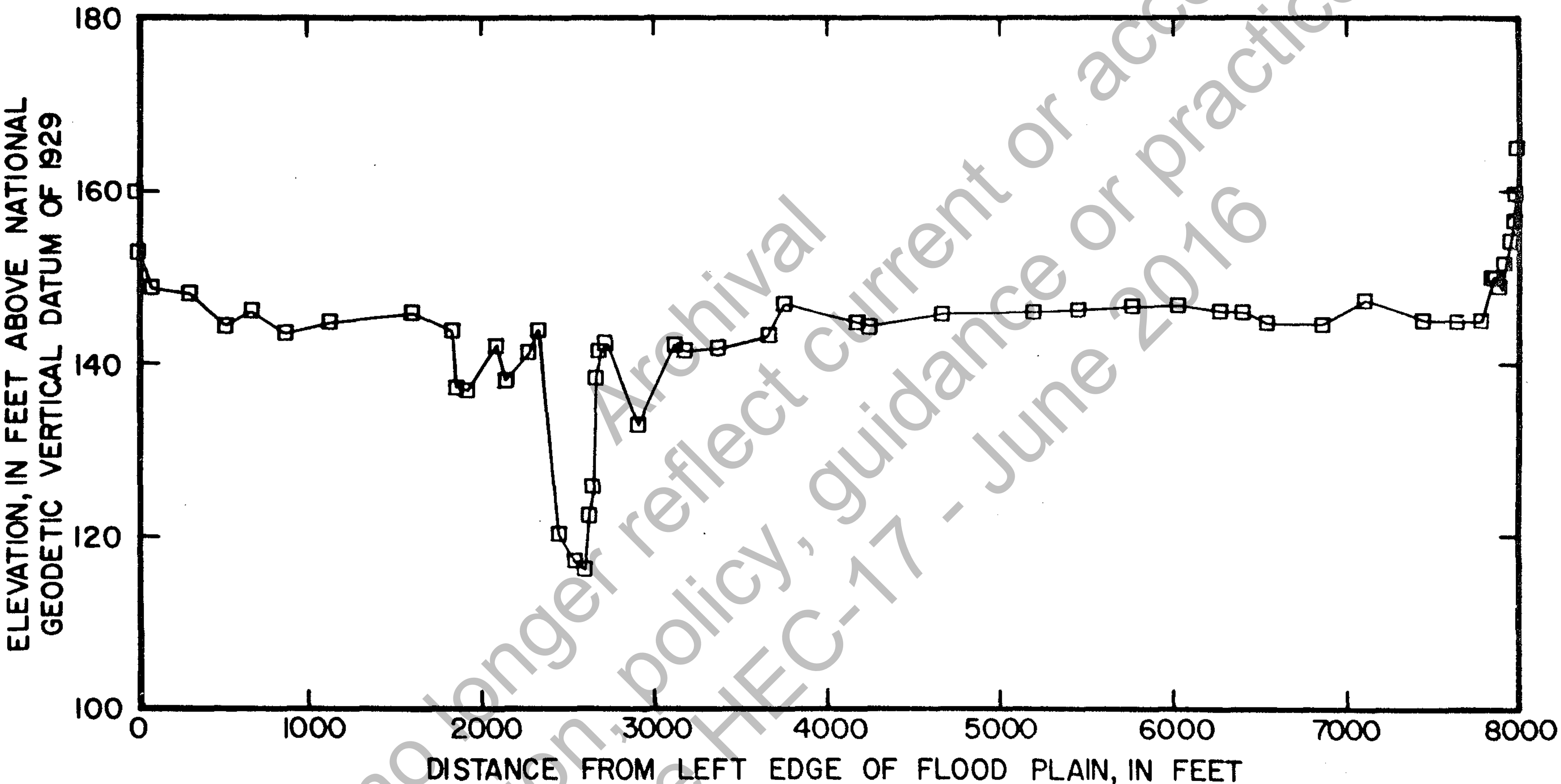

FIGURE A.7D CROSS SECTION 4 LOCATED DOWNSTREAM FROM THE PROPOSED U.S. HIGHWAY 11 (STATION 5,900 FT)

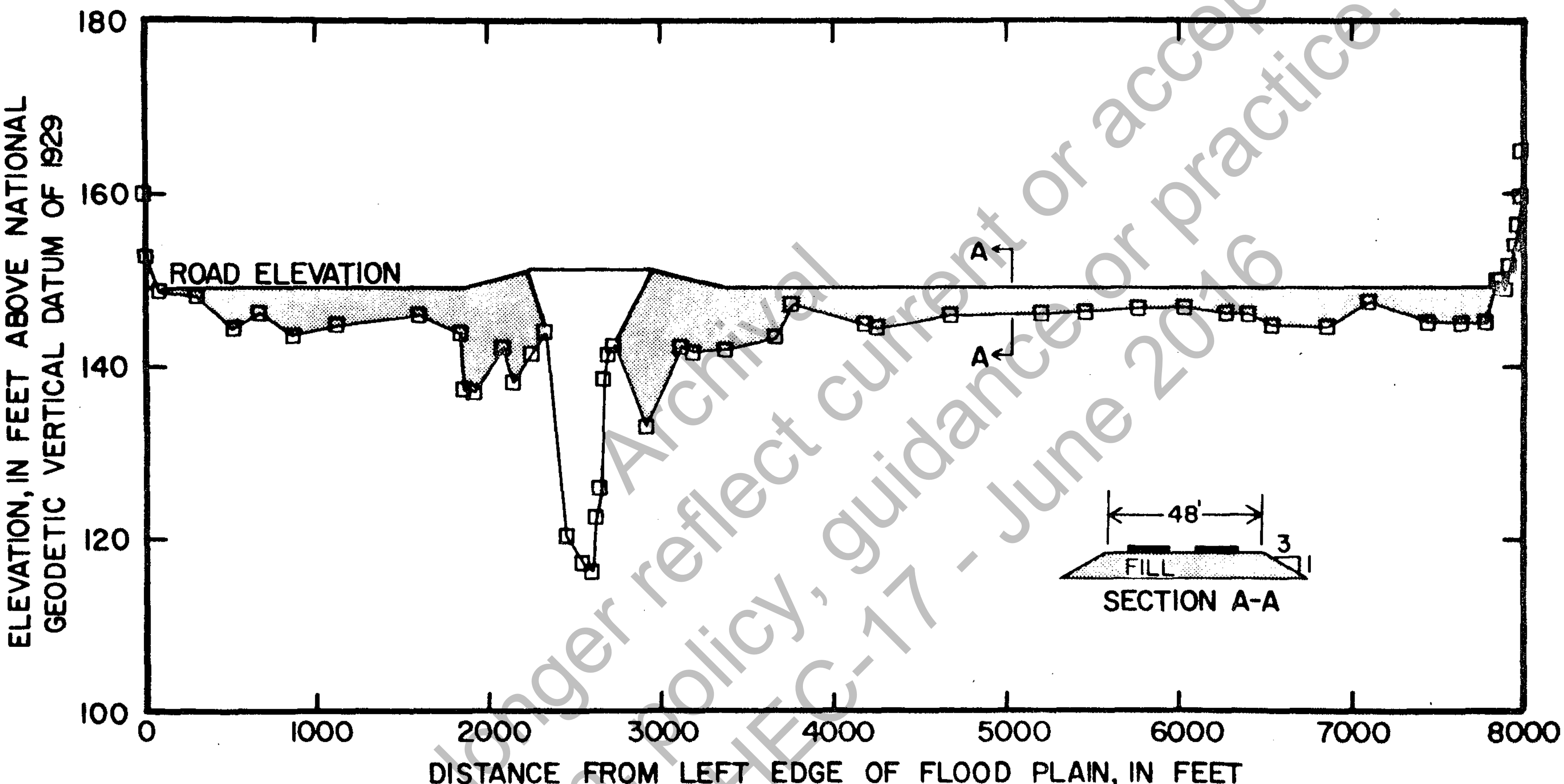

FIGURE A.7E CROSS SECTION 5 LOCATED UPSTREAM FROM THE PROPOSED U.S.HIGHWAY 11 (STATION 6480 FT) WITH A TYPICAL HIGHWAY SECTION SUPERIMPOSED

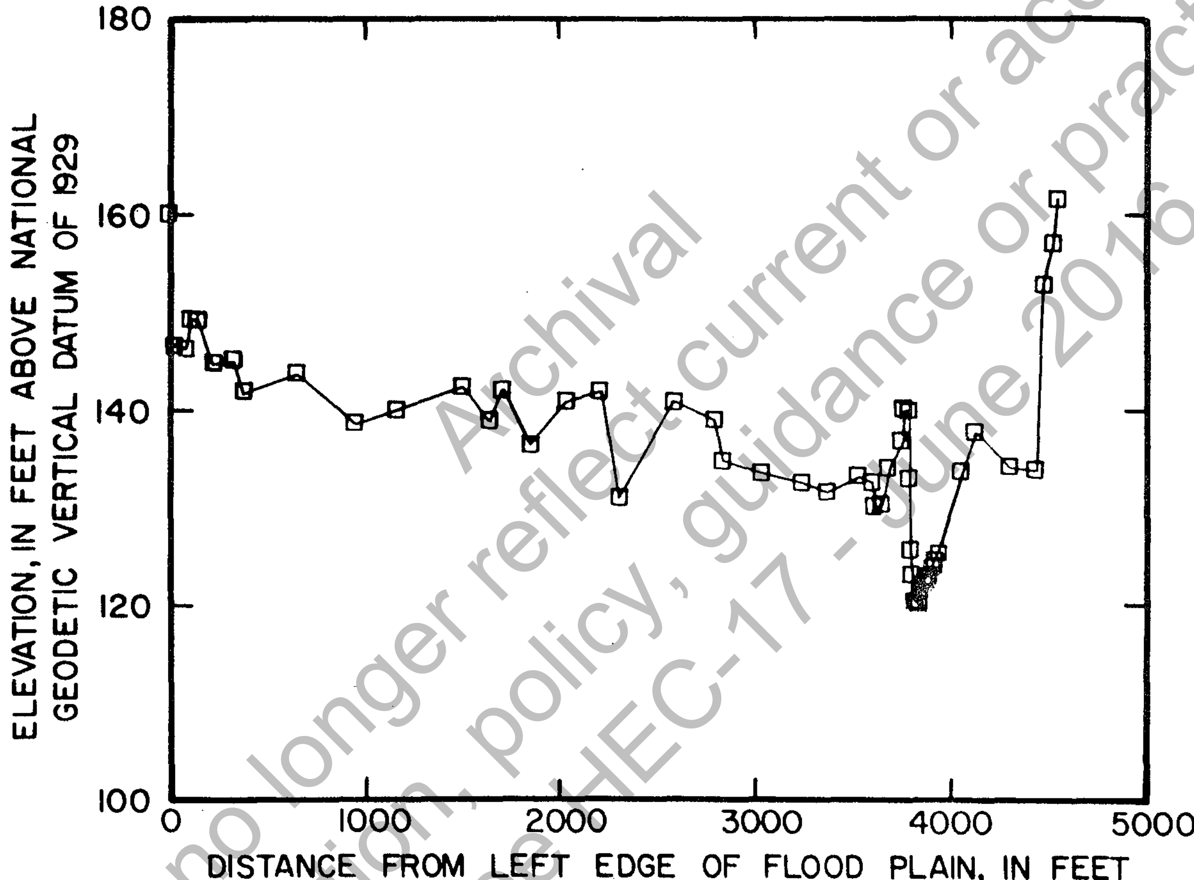

FIGURE A.7F CROSS SECTION 6 LOCATED ON THE LEAF RIVER UPSTREAM FROM THE CONFLUENCE WITH BOWIE RIVER (STATION 11,200 FT)

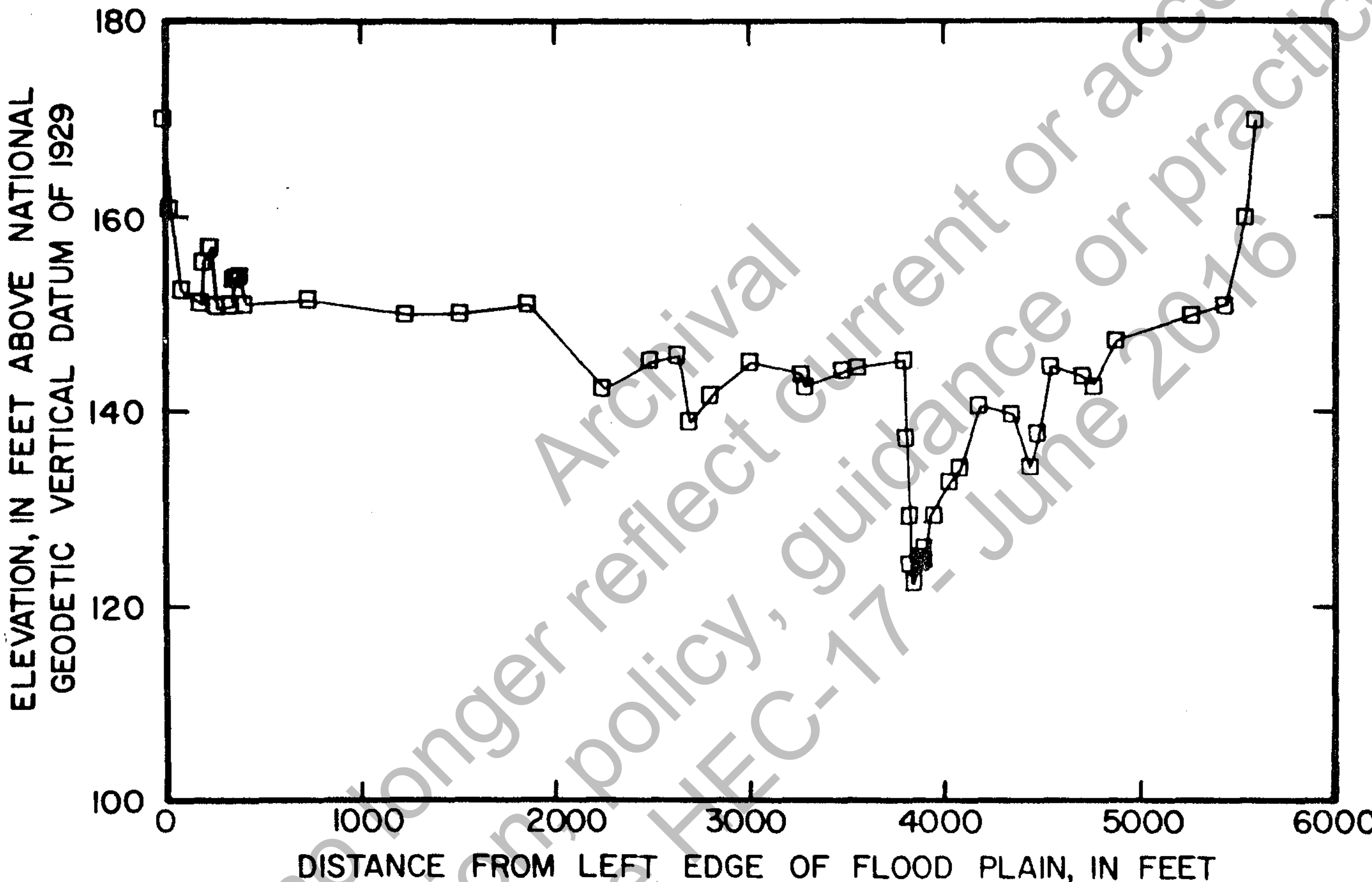

FIGURE A.7G CROSS SECTION 7 LOCATED ON THE LEAF RIVER, 5,780 FT. UPSTREAM FOR CROSS SECTION 6 (STATION 16,980 FT)

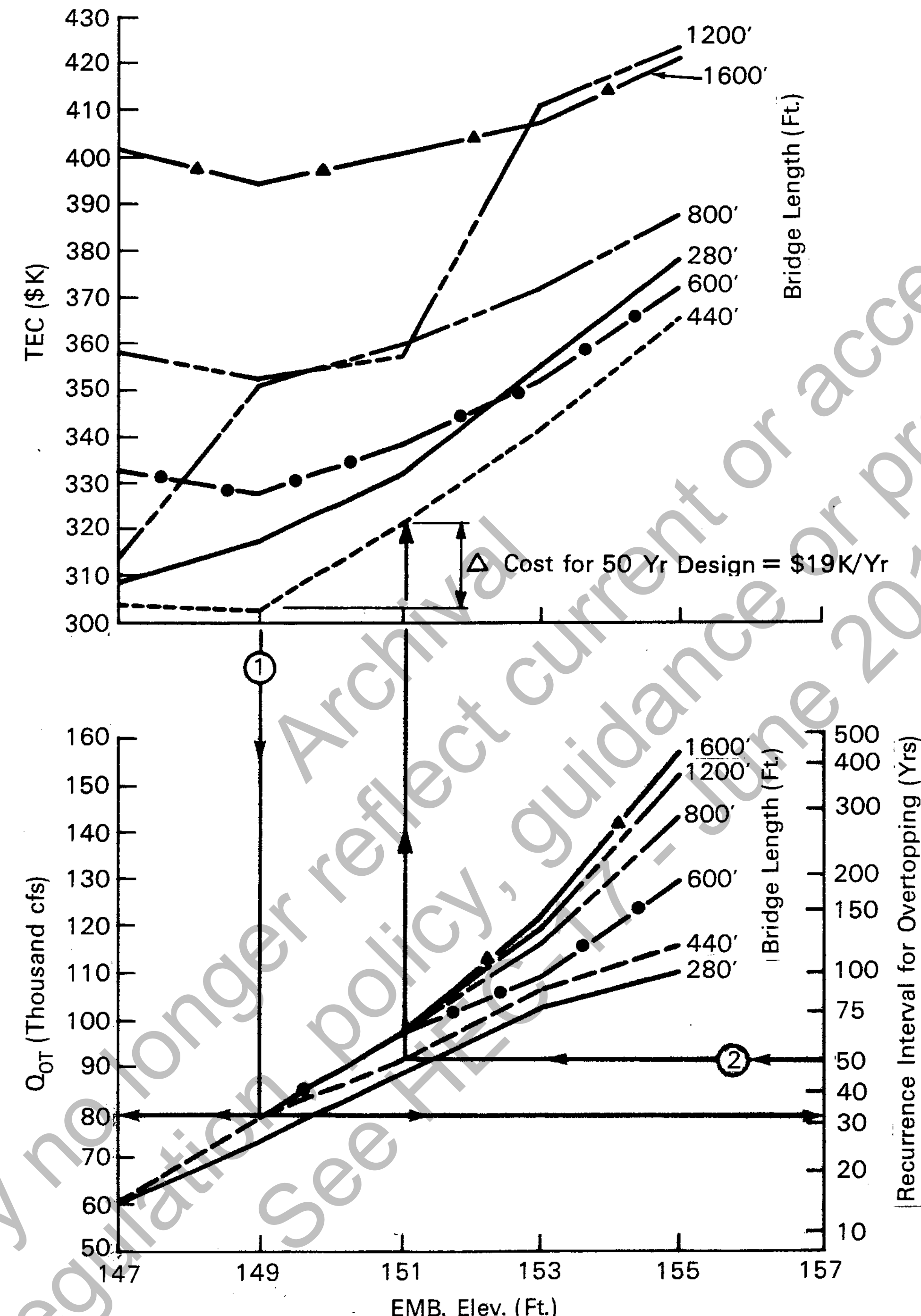

FIGURE A.8 SELECTION OF THE DESIGN DISCHARGE, BRIDGE LENGTH, AND EMBANKMENT ELEVATION GIVEN THE TOTAL EXPECTED COSTS

Table A.3 Average W.S. Elevations After Adjustments

Q_{ot} cfs	Bridge Length (ft)	Embank Elev. (ft)	Avg. W.S. Elev. between Section 5 & 6 Q (cfs)								
			54200	62000	68000	73600	90500	110,000	121,000	131,000	164000
Natural	Profiles		146.97	148.09	148.72	149.28	150.87	152.47	153.27	153.99	155.13
60,000	280	147	146.97*	148.23*	148.79*	149.35*	150.94*	152.51*	153.32*	154.04*	155.46
73,000		149	"	148.25*	148.94*	149.70*	150.97*	"	"	"	155.46
88,500		151	"	"	"	149.84	151.90	152.83	153.38	"	155.46
103,600		153	"	"	"	"	152.41	154.09	154.50	154.87	156.10
110,000		155	"	"	"	"	"	155.37	155.86	156.25	157.34
60,000	440	147	146.97*	148.23*	148.79*	149.35*	150.94*	152.51*	153.32	154.04*	155.46
78,000		149	"	148.25*	148.94*	149.70	150.97	"	"	"	"
90,100		151	"	"	"	"	151.78	152.78	153.38	"	"
106,900		153	"	"	"	"	152.03	153.91	154.39	154.79	156.02
116,900		155	"	"	"	"	"	154.56	155.60	156.07	157.18
60,000	600	147	146.97*	148.23*	148.79*	149.35*	150.94*	152.51	153.32*	154.04*	155.46
78,000		149	"	148.25*	148.92	149.52*	150.97*	"	"	"	"
97,000		151	"	"	"	"	151.53	152.73*	153.38*	"	"
117,000		153	"	"	"	"	"	153.58	154.18	154.65	155.77
143,500		155	"	"	"	"	"	153.65	154.78	155.68	156.60
60,000	800	147	146.97*	148.23*	148.79*	149.35	150.94*	152.51*	153.32*	154.04*	155.46
78,000		149	"	148.25*	148.92	149.52*	150.97*	"	"	"	"
97,000		151	"	"	"	"	151.31	152.73*	153.38*	"	"
117,000		153	"	"	"	"	"	153.22	154.01	154.54	155.77
143,500		155	"	"	"	"	"	"	154.22	155.13	156.60
60,00	1200	147	146.97	148.23	148.79	149.35	150.94	152.51	153.32	154.04	155.46
78,000		149	"	148.25	148.92	149.52	150.97	152.51	"	"	"
97,000		151	"	"	"	"	151.24	152.73	153.38	"	"
119,000		153	"	"	"	"	"	152.98	153.85	154.45	155.63
153,200		155	"	"	"	"	"	"	153.89	154.70	156.24
60,000	1600	147	146.97*	148.23*	148.79*	149.35*	150.94*	152.51*	153.32	154.04	155.46
78,000		149	"	148.25*	148.92*	149.52*	150.97*	152.51*	"	"	"
97,000		151	"	"	"	"	151.24*	152.73*	153.38*	"	"
121,000		153	"	"	"	"	"	152.98*	153.84	154.45*	"
157,100		155	"	"	"	"	"	"	153.85	154.63	156.01

*Some values of avg. w.s. elev. were adjusted because it is unreasonable for the U.S. elevations to increase with larger opening when everything else is constant.
NOTE: Elevations for the 1200 foot bridge were used as pivots.

Table A.4 Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
280 ft	54,200			0/0	0	0	0
147 ft	62,000	0.3	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.9	9.4	130.9K	31.4K
	73,600	1.7	37.9	20.0/25.3	35.3	249.4K	68.1K
	90,500	3.6	55.	77.9/77.8	120	600.0K	210.7K
	110,000	5.4	68.3	100/100	120	644.8K	270.8K
	121,000	6.2	73.8	100/100	120	663.5K	270.8K
	131,000	6.9	77.8	100/100	120	677.3K	270.8K
	164,000	8.4	86.4	100/100	120	706.6K	270.8K
280 ft	54,200					0	0
149 ft	62,000					0	0
73,000 cfs	68,000					0	0
	73,600	0.1	7	0/0	0	24.7K	0
	90,500	1.9	40.2	25.3/30.7	108	506.6K	93.2K
	110,000	3.7	56.	85.0/83.6	120	604.7K	261.8K
	121,000	4.6	63.	100/100	120	626.5K	312.2K
	131,000	5.3	67.	100/100	120	642.4K	312.2K
	164,000	6.8	77.	100/100	120	675.2K	312.2K
280 ft	54,200					0	0
151 ft	62,000					0	0
88,500 cfs	68,000					0	0
	73,600					0	0
	90,500	0.2	12.4	0/0	0	42.9K	0
	110,000	2.0	41.	27.1/32.5	120	551.2K	116.7K
	121,000	2.9	49.	52.0/55.7	120	579.6K	206.2K
	131,000	3.5	55.	76.7/76.8	120	599.1K	290.2K
	164,000	5.1	66.3	100/100	120	637.8K	377.8K
280 ft	54,200					0	0
153 ft	62,000					0	0
103,600 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000	0.55	20.9	2.5/4.5	29.1	171.2K	17.1K
	121,000	1.4	34.0	14.0/18.9	120	527.3K	76.7K
	131,000	2.1	41.7	28.8/34.2	120	553.6K	145.6K
	164,000	3.6	55.5	79.5/79.1	120	601.1K	361.4K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
280 ft.	54,200					0	0
155 ft	62,000					0	0
110,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000	.8	26.3	5.6/8.8	101.5	437.4K	39.3K
	131,000	1.5	35.5	16.4/21.4	120	532.5K	102.6K
	164,000	3.1	51.0	58.6/60	120	585.4K	327.1K
440 ft.	54,200					0	0
147 ft	62,000	.26	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.9	9.2	130.1K	30.7K
	73,600	1.7	37.6	20.0/25.3	34.8	247.8K	66.5K
	90,500	3.6	55.2	77.9/77.8	120	600.0K	205.7K
	110,000	5.4	68.3	100/100	120	644.8K	264.3K
	121,000	6.2	73.8	100/100	120	663.5K	264.3K
	131,000	6.9	77.8	100/100	120	677.3K	264.3K
	164,000	8.4	86.4	100/100	120	706.6K	264.3K
440 ft.	54,200					0	0
149 ft	62,000					0	0
78,000 cfs	68,000					0	0
	73,600					0	0
	90,500	1.4	33.4	13.1/17.8	55.5	304.3K	52.1K
	110,000	3.2	51.8	62.2/64.6	120	588.3K	195.7K
	121,000	4.0	58.7	96.6/92.9	120	611.8K	285.2K
	131,000	4.7	63.6	100/100	120	628.7K	304.8K
	164,000	6.2	73.7	100/100	120	663.2K	304.8K
440 ft.	54,200						
151 ft	62,000						
90,100 cfs	68,000						
	73,600					0	0
	90,500	.04	5.4	0/0	0	18.5K	0
	110,000	1.8	39.2	23.2/28.6	120	545.1K	99.3K
	121,000	2.7	47.8	46.6/50.9	120	574.4K	183.0K
	131,000	3.4	53.6	70.3/71.4	120	594.5K	262.2K
	164,000	4.9	65.1	100/100	120	633.9K	368.8K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
440 ft.	54,200						
153 ft	62,000						
106,900 cfs	68,000						
	73,600					0	0
	90,500					0	0
	110,000	0.26	14.2	0/0	0	48.6K	0
	121,000	1.1	30.2	9.2/13.3	108	472.8K	51.5K
	131,000	1.8	38.6	21.9/27.2	120	543.0K	111.2K
	164,000	3.3	53.2	68.2/69.7	120	593.0K	307.2K
440 ft	54,200					0	0
154 ft	62,000					0	0
116,900 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000	0.3	15.4	0.8/1.8	7.0	76.4K	7.4K
	131,000	1.0	28.2	7.2/10.9	120	507.4K	47.9K
	164,000	2.5	46.0	40.7/45.5	120	568.3K	228.9K
600 ft	54,200					0	0
147 ft	62,000	0.26	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.7	8.9	129.4K	30.0K
	73,600	1.7	37.6	20.0/25.3	34.2	246.0K	64.9K
	90,500	3.6	55.2	77.9/77.8	120	600.0K	200.7K
	110,000	5.4	68.3	100/100	120	644.8K	257.9K
	121,000	6.2	73.8	100/100	120	663.5K	257.9K
	131,000	6.9	77.8	100/100	120	677.3K	257.9K
	164,000	8.4	86.4	100/100	120	706.6K	257.9K
600 ft	54,200					0	0
149 ft	62,000					0	0
78,000 cfs	68,000					0	0
	73,600					0	0
	90,500	1.4	33.4	13.1/17.8	54.9	302.0K	50.8K
	110,000	3.2	51.8	62.2/64.6	120	588.3K	191.0K
	121,000	4.0	58.7	96.6/92.9	120	611.8K	278.3K
	131,000	4.7	63.6	100/100	120	628.7K	297.4K
	164,000	6.2	73.7	100/100	120	663.2K	297.4K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
600 ft.	54,200						
151 ft	62,000						
97,000 cfs	68,000						
	73,600					0	0
	90,500					0	0
	110,000	1.2	30.8	9.9/14.1	64.1	325.1K	46.3K
	121,000	2.0	41.0	27.2/32.7	120	551.4K	111.6K
	131,000	2.7	47.7	46.3/50.6	120	574.1K	177.4K
	164,000	4.2	60.2	100/100	120	617.2K	359.8K
600 ft.						0	0
153 ft	62,000					0	0
109,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000	.08	7.8	0/0	0	26.8K	0
	121,000	0.9	27.6	6.7/10.2	79.1	356.1K	38.0K
	131,000	1.6	36.5	18.0/23.2	120	535.9K	91.3K
	164,000	3.1	51.7	61.6/64.1	120	587.8K	273.5K
600 ft.	54,200					0	0
155 ft	62,000					0	0
130,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000					0	0
	131,000	.06	6.8	0/0	0	23.2K	0
	164,000	1.6	36.3	17.6/22.8	120	535.2K	104.6K
800 ft.	54,200						
147 ft	62,000	0.26	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.7	8.6	128.4K	29.0K
	73,600	1.7	37.6	20.0/25.3	33.6	243.8K	62.8K
	90,500	3.6	55.2	77.9/77.8	120	600.0K	194.5K
	110,000	5.4	68.3	100/100	120	644.8K	249.8K
	121,000	6.2	73.8	100/100	120	663.5K	249.8K
	131,000	6.9	77.8	100/100	120	677.3K	249.8K
	164,000	8.4	86.4	100/100	120	706.6K	249.8K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
800 ft.	54,200					0	0
149 ft	62,000					0	0
78,000 cfs	68,000					0	0
	73,600					0	0
	90,500	1.35	33.5	13.1/17.8	40	251.2K	49.2K
	110,000	3.2	51.8	62.2/64.6	120	588.3K	185.0K
	121,000	4.0	58.7	96.6/92.9	120	611.8K	269.6K
	131,000	4.7	63.6	100/100	120	628.7K	288.1K
	164,000	6.2	73.7	100/100	120	663.2K	288.1K
800 ft.	54,200						
151 ft	62,000						
97,000 cfs	68,000						
	73,600					0	0
	90,500					0	0
	110,000	1.16	30.8	9.9/14.1	63	321.2K	44.9K
	121,000	2.0	41.0	21.2/32.7	120	551.4K	108.1K
	131,000	3.7	47.7	46.3/50.6	120	574.1K	172.0K
	164,000	4.2	60.2	100/100	120	617.2K	348.6K
800 ft.	54,200						
153 ft	62,000						
117,000 cfs	68,000						
	73,600						
	90,500						
	110,000						
	121,000	0.3	15.2	0/0	0	51.9K	0
	131,000	1.0	28.1	7.1/10.8	80.2	370.7K	38.9K
	164,000	2.5	45.9	40.5/45.3	120	568.0K	181.8K
800 ft.	54,200						
155 ft	62,000						
143,500 cfs	68,000						
	73,600						
	90,500						
	110,000						
	121,000						
	131,000					0	0
	164,000	0.82	25.8	5.3/8.4	81.4	367.0K	34.3K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
1200 ft	54,200					0	0
147 ft	62,000	0.26	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.9	8.1	126.5K	27.2K
	73,600	1.7	37.6	20.0/25.3	32.3	239.5K	58.8K
	90,500	3.6	55.2	77.9/77.8	120	600.0K	181.9K
	110,000	5.4	68.3	100/100	120	644.8K	233.7K
	121,000	6.2	73.8	100/100	120	663.5K	233.7K
	131,000	6.9	77.8	100/100	120	677.3K	233.7K
	164,000	8.4	86.4	100/100	120	706.6K	233.7K
1200 ft	54,200					0	0
149 ft	62,000					0	0
78,000 cfs	68,000					0	0
	73,600					0	0
	90,500	1.4	33.3	13.1/17.8	38.3	245.5K	46.1K
	110,000	3.2	51.8	62.2/64.6	120	588.3K	173.1K
	121,000	4.0	58.7	96.6/92.9	120	611.8K	252.2K
	131,000	4.7	63.6	100/100	120	628.7K	269.5K
	164,000	6.2	73.6	100/100	120	663.2K	269.5K
1200 ft	54,200					0	0
151 ft	62,000					0	0
97,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000	1.2	30.8	9.9/14.1	60.7	313.5K	42.0K
	121,000	2.0	41.0	27.2/32.7	120	551.4K	101.2K
	131,000	2.7	47.7	46.3/50.6	120	574.1K	160.0K
	164,000	4.2	60.2	100/100	120	617.2K	326.1K
1200 ft	54,200					0	0
153 ft	62,000					0	0
119,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000	0.15	10.5	0/0	0	35.9K	0
	131,000	0.82	25.7	5.2/8.3	54.4	274.4K	27.8K
	164,000	2.4	44.4	36.1/41.2	120	563.0K	153.6K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
1200 ft	54,200						
155 ft	62,000						
153,200 cfs	68,000						
	73,600						
	90,500						
	110,000						
	121,000						
	131,000						
	164,000	0.38	17.3	1.3/2.6	9.3	91.1K	9.4K
1600 ft	54,200						
147 ft	62,000	0.26	14.2	0/0	0	48.6K	0
60,000 cfs	68,000	1.0	28.8	7.8/11.7	7.5	124.6K	25.3K
	73,600	1.7	37.6	20.0/25.3	31.1	235.2K	54.8K
	90,500	3.6	55.2	77.9/77.8	112	574.7K	169.4K
	110,000	5.4	68.3	100/100	120	644.8K	217.6K
	121,000	6.2	73.8	100/100	120	663.5K	217.6K
	131,000	6.9	77.8	100/100	120	677.3K	217.6K
	164,000	8.4	86.4	100/100	120	706.6K	217.6K
1600 ft	54,200						
149 ft	62,000						
78,000 cfs	68,000					0	0
	73,600					0	0
	90,500	1.35	33.3	13.1/17.8	36.7	239.7K	42.9K
	110,000	3.2	51.8	62.2/64.6	120	588.3K	161.2K
	121,000	4.0	58.7	96.6/92.9	120	611.8K	234.8K
	131,000	4.7	63.6	100/100	120	628.7K	251.0K
	164,000	6.2	73.7	100/100	120	663.2K	251.0K
1600 ft.	54,200					0	0
151 ft	62,000					0	0
97,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000	1.16	30.8	9.9/14.1	58.5	305.8K	39.2K
	121,000	2.0	41.0	27.2/32.7	120	551.4K	94.2K
	131,000	2.7	47.7	46.3/50.6	120	574.1K	149.8K
	164,000	4.2	60.2	100/100	120	617.2K	303.7K

Table A.4 (cont'd) Work Table for Traffic Losses and Repair Cost

Design Features: BR. Length/ EMB. Elev/Q_{ot}	Q (cfs)	d_{ot} (ft)	t_{ot} (hr)	P_{emb}/P_{pave} (%)	t_{tr} (hr)	Traffic Loss ($)	Repair Cost ($)
800 ft.	54,200					0	0
153 ft	62,000					0	0
121,000 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000					0	0
	131,000	0.67	23.2	3.6/6.2	31.3	186.7K	18.9K
	164,000	2.2	43.0	32.1/37.4	120	558.0K	128.7K
800 ft	54,200					0	0
155 ft	62,000					0	0
157,100 cfs	68,000					0	0
	73,600					0	0
	90,500					0	0
	110,000					0	0
	121,000					0	0
	131,000					0	0
	164,000	0.23	13.3	0/0	0	45.6K	0

Table A.5 Typical Detailed Work Table for Property Damages Rt. 11 Crossing Leaf River at Hattiesburg Miss. Residential Property with basements (assumed)

d = Avg. W.S. Elev - 1st Flr. Elev

$$p = 12.54 + 5.30d - 0.144d^2$$

Q (cfs)	Avg. W.S. Elev.	p, Percent Damage — 1st Floor Elev./Incremental Value ($K) 148 262.5	149 1,575	150 2,062.5	151 2,400	152 2,737.5	153 1,612.5	154 600	155 37.5	Total Damage ($)	Delta Damage ($)
No Bridge Sections 5&6											
54,200	146.97	6.93	1.19	0						36,891	
62,000	148.09	13.02	7.60	1.89	0					192,846	
68,000	148.72	16.28	11.04	5.52						330,544	
73,600	149.28	19.09	14.01	8.65	3.0	0				521,151	
90,500	150.87	26.56	21.95	17.01	11.85	6.37	0.60	0		1,235,199	
110,00	152.47	33.35	29.20	24.75	20.02	15.0	9.69	1.09	0	2,129,830	
121,000	153.27	36.47	32.55	28.53	23.83	19.04	13.96	8.59	2.94	2,563,523	
131,000	153.99	39.12	35.40	31.39	27.10	22.52	17.65	12.49	7.04	2,936,622	
164,000	155.13	41.80	39.62	35.94	31.97	27.72	23.18	18.35	13.23	3,493,022	
BR. Length = 280 ft, Embank Elev = 147 ft, Sections 5&6											
54,200	146.97	6.93	1.19	0						36,891	0
62,000	148.23	13.75	8.37	2.71	0					223,831	30,984
68000	148.23	16.64	11.42	5.92	0.12	0				348,537	17,993
73,600	149.35	19.43	14.38	9.03	3.40	0				545,454	24,303
90,500	150.94	26.88	22.28	17.39	12.22	6.76	0			1,274,907	39,797
110,000	152.51	33.51	29.37	24.94	20.21	15.21	9.91	4.32	0	2,151,952	22,122
121,000	153.32	36.66	32.75	28.55	24.06	19.29	8.87	3.23	0	2,589,980	26,456
131,000	154.04	39.30	35.39	31.60	27.32	22.75	17.90	12.75	7.32	2,961,949	25,287
164,000	155.46	44.06	40.77	37.19	33.31	29.15	24.71	19.97	14.95	3,646,162	153,160
BR. Length = 280 ft, Embank Elev = 149 ft, Sections 5&6											
54,200	146.97	6.93	1.19	0						36,891	0
62,000	148.25	13.86	8.48	2.82						228,240	35,393
68,000	148.94	17.39	12.22	6.76	1.19	0				401,840	71,296
73,600	149.70	21.13	16.18	10.94	5.41	0				665,638	144,487
90,500	150.97	27.01	22.42	17.55	12,38	6.93	1.19	0		1,291,879	56,679
110,000	152.51	33.51	29.37	24.94	20.21	15.21	9.91	4.32	0	2,151,952	22,122
121,000	153.32	36.66	32.75	28.55	24.06	19.29	14.22	8.87	3.23	2,589,980	26,456
131,000	154.04	39.30	35.59	31.60	27.32	22.75	17.90	12.75	7.32	2,961,949	25,286
164,000	155.46	44.06	40.70	37.31	33.31	29.15	24.71	19.97	14.85	3,646,162	153,160

Table A.6 Summary of B.W. Damage to Upstream Property, Route 11 Crossing Leaf River at Hattiesburg, Mississippi

Q (cfs)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)
Bridge L.:	280		280		280		280	
Emb. Elev.	147		149		151		153	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.23	31.0	148.25	35.4	148.25	35.4	148.25	35.4
68,000	148.79	18.0	148.94	71.3	148.94	71.3	148.94	71.3
73,600	149.35	24.3	149.70	144,5	149.84	203.4	149.84	203.4
90,500	150.94	39.7	150.97	56.7	151.90	573.8	152.41	861.3
110,000	152.51	22.1	152.51	22.1	152.83	197.4	154.09	857.3
121,000	153.32	26.4	153.32	26.4	153.38	58.1	154.50	627.2
131,000	153.04	25.3	154.04	25.3	154.04	25.3	154.87	433.2
164,000	155.46	153.2	155.46	153.2	156.46	153.2	156.10	440.1
Bridge L.:	280		440		440		440	
Emb. Elev.	155		147		149		151	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.25	35.4	148.23	31.0	148.25	35.4	148.25	35.4
68,000	148.94	71.3	148.79	18.0	148.94	71.3	148.94	71.3
73,600	149.84	203.4	149.35	24.3	149.70	144.5	149.70	144.5
90,500	152.41	861.4	150.94	39.7	151.97	56.7	151.78	504.9
110,000	155.37	1,474.9	152.51	22.1	152.51	22.1	152.51	170.1
121,000	155.86	1,263.5	153.32	26.4	153.32	26.4	153.38	58.1
131,000	156.25	1,061.8	154.04	25.3	154.04	25.3	155.04	25.3
164,000	157.34	958.2	155.46	153.2	155.46	153.2	155.46	153.2
Bridge L.:	440		440		600		600	
Emb. Elev.	153		155		147		149	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.25	35.4	148.25	35.4	148.23	31.0	148.25	35.4
68,000	148.94	71.3	148.94	71.3	148.79	18.0	148.92	64.2
73,600	149.70	144.5	149.70	144.5	149.35	24.3	149.52	83.0
90,500	152.03	647.9	152.03	647.9	150.94	39.7	150.97	56.7
110,000	153.91	766.2	154.56	1,090.3	152.51	22.1	152.51	22.1
121,000	154.39	573.2	155.60	1,146.5	153.32	26.4	153.32	26.4
131,000	154.79	394.8	156.07	983.3	154.04	25.3	154.04	25.3
164,000	156.02	404.5	157.18	894.1	155.46	153.2	155.46	153.2

Table A.6 (Cont'd) Summary of B.W. Damage to Upstream Property, Route 11 Crossing Leaf River at Hattiesburg, Mississippi

Q (cfs)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)
Bridge L.:	600		600		600		800	
Emb. Elev.	151		153		155		147	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.25	35.4	148.25	35.4	148.25	35.4	148.23	31.0
68,000	148.92	64.2	148.92	64.2	148.92	64.2	148.79	18.0
73,600	149.52	83.0	149.52	83,0	149.52	83.0	149.35	24.3
90,500	151.53	368.4	151.53	368.4	151.53	368.4	150.94	39.7
110,000	152.73	142.9	153.58	596.4	153.65	632.7	152.51	22.1
121,000	153.38	58.1	154.18	468.8	154.78	736.2	153.32	26.4
131,000	154.04	25.3	154.65	327.2	155.68	809.6	154.04	25.3
164,000	155.46	153.2	155.77	293.8	156.60	655.0	155.46	153.2
Bridge L.:	800		800		800		800	
Emb. Elev.	149		151		153		155	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.25	35.4	148.25	35.4	148.25	35.4	148.25	35.4
68,000	148.92	64.2	148.92	64.2	148.92	64.2	148.92	64.2
73,600	149.52	83.0	149.52	83.0	149.52	83.0	149.52	83.0
90,500	150.97	56.7	151.31	247.1	151.31	247.1	151.31	247.1
110,000	152.51	22.1	152.73	142.9	153.22	407.2	153.22	407.2
121,000	153.32	26.4	153.38	58.1	154.01	383.3	154.22	488.8
131,000	154.04	25.3	154.04	25.3	154.54	273.7	155.13	556.3
164,000	155.46	153.2	155.46	153.2	155.77	293.8	156.60	655.0
Bridge L.:	1200		1200		1200		1200	
Emb. Elev.	147		149		151		153	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.23	31.0	148.25	35.4	148.25	35.4	148.25	35.4
68,000	148.79	18.0	148.92	64.2	148.92	64.2	148.92	64.2
73,600	149.35	24.3	149.52	83.0	149.52	83.0	149.52	83.0
90,500	150.94	39.7	150.97	56.7	151.24	208.2	151.24	208.2
110,000	152.51	22.1	152.51	22.1	153.73	142.9	152.98	278.6
121,000	153.32	26.4	153.32	26.4	153.38	58.1	153.85	301.9
131,000	154.04	25.3	154.04	25.3	154.04	25.3	154.45	229.6
164,000	155.46	153.2	155.46	153.2	155.46	153.2	155.63	230.7

Table A.6 (Cont'd) Summary of B.W. Damage to Upstream property, Route 11 Crossing Leaf River at Hattiesburg, Mississippi

Q (cfs)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)	Avg. W.S. Elev. (ft)	Delta Damage ($1000)
Bridge L.:	1200		1600		1600		1600	
Emb. Elev.	155		147		149		151	
54,200	146.97	0	146.97	0	146.97	0	146.97	0
62,000	148.25	35.4	148.23	31.0	148.25	35.4	148.25	35.4
68,000	148.92	64.2	148.79	18.0	148.92	64.2	148.92	64.2
73,600	149.52	83.0	149.35	24.3	149.52	83.0	149.52	83.0
90,500	151.24	208.2	150.94	39.7	150.97	56.7	151.24	208.2
110,000	152.98	278.6	152.51	22.1	152.51	22.1	152.73	142.9
121,000	153.89	322.3	153.32	26.4	153.32	26.4	153.38	58.1
131,000	154.70	251.4	154.04	25.3	154.04	25.3	154.04	25.3
164,000	155.24	501.1	155.46	153.2	155.46	153.2	155.46	153.2
Bridge L.:	1600		1600					
Emb. Elev.	153		155					
54,200	146.97	0	146.97	0				
62,000	148.25	35.4	148.25	35.4				
68,000	148.92	64.2	148.92	64.2				
73,600	149.52	83.0	149.52	83.0				
90,500	151.24	208.2	151.24	208.2				
110,000	152.98	278.6	152.98	278.6				
121,000	153.84	296.8	153.85	301.9				
131,000	154.45	229.6	154.63	317.5				
164,000	155.46	153.2	156.01	400.6				

Table A.7 Losses to Abutment Protection

Note: Assume Loss of Half of Bank Protection for $9 \leq V_{bo} \leq 10$ ft/sec

Assume Loss of All Bank Protection for $V_{bo} > 10$

Q (cfs)	V_{bo} ft/sec	Loss ($)	V_{bo} ft/sec	Loss ($)	V_{bo} ft/sec	Loss ($)	V_{bo} ft/sec	Loss ($)
Bridge L.:	280		280		280		280	
Emb. Elev.	147		149		151		153	
54,200		0	7.16	0	7.16	0	7.16	0
62,000		0	7.85	0	7.85	0	7.85	0
68,000		0	8.41	0	8.41	0	8.41	0
73,600		0	8.81	0	8.91	0	8.91	0
90,500	7.75	0	9.16	250K	9.27	250K	10.33	500K
110,000	8.03	0	9.50	250K	9.60	250K	10.70	500K
121,000	8.13	0	9.66	250K	9.77	250K	10.89	500K
131,000	8.29	0	9.80	250K	9.91	250K	11.05	500K
164,000	8.45	0	9.99	250K	10.10	500K	11.26	500K
Bridge L.:	280		440		440		440	
Emb. Elev.	155		147		149		151	
54,200	7.16	0	6.66	0	6.66	0	6.66	0
62,000	7.85	0	6.75	0	7.26	0	7.26	0
68,000	8.41	0	6.99	0	7.76	0	7.76	0
73,600	8.91	0	7.11	0	8.19	0	8.19	0
90,500	10.33	500K	7.48	0	8.61	0	8.61	0
110,000	>10	500K	7.25	0	9.04	250K	9.04	250K
121,000	>10	500K	7.38	0	9.26	250K	9.26	250K
131,000	>10	500K	7.49	0	9.44	250K	9.44	250K
164,000	>10	500K	7.63	0	9.49	250K	9.49	250K
Bridge L.:	440		440		600		600	
Emb. Elev.	153		155		147		149	
54,200	6.66	0	6.66	0	5.72	0	2.72	0
62,000	7.26	0	7.26	0	5.95	0	6.17	0
68,000	7.76	0	7.76	0	6.08	0	6.54	0
73,600	8.19	0	8.19	0	6.21	0	6.86.	0
90,500	9.34	250K	9.34	250K	6.59	0	7.27	0
110,000	9.84	250K	10.55	500K	6.96	0	7.69	0
121,000	10.05	500K	10.81	500K	7.15	0	7.90	0
131,000	10.25	500K	11.02	500K	7.31	0	8.07	0
164,000	10.30	500K	11.07	500K	7.35	0	8.12	0

Table A.7 (cont'd) Losses to Abutment Protection

Note: Assume Loss of Half of Bank Protection for $9 \leq V_{b.o.} \leq 10$ ft/sec
Assume Loss of All Bank Protection for $V_{b.o.} > 10$

Q (cfs)	$V_{b.o.}$ ft/sec	Loss ($)	$V_{b.o.}$ ft/sec	Loss ($)	$V_{b.o.}$ ft/sec	Loss ($)	$V_{b.o.}$ ft/sec	Loss ($)
Bridge L.:	600		600		600		800	
Emb. Elev.	151		153		155		147-153	
54,200	5.72	0	5.72	0	5.72	0		0
62,000	6.17	0	6.17	0	6.17	0		0
68,000	6.54	0	6.54	0	6.54	0		0
73,600	6.86	0	6.86	0	6.86	0		0
90,500	7.73	0	7.73	0	7.73	0		0
110,000	8.17	0	8.17	0	8.65	0		0
121,000	8.39	0	8.39	0	9.13	250K		0
131,000	8.58	0	8.58	0	9.33	250K		0
164,000	8.63	0	8.63	0	9.38	250K		0
Bridge L.:	800		1200		1600			
Emb. Elev.	155		147-155		147-155			
54,200	5.17	0		0		0		
62,000	5.48	0		0		0		
68,000	5.77	0		0		0		
73,600	6.02	0		0		0		
90,500	6.66	0		0		0		
110,000	7.36	0		0		0		
121,000	7.74	0		0		0		
131,000	8.07	0		0		0		
164,000	8.47	0		0		0		

$V_{b.o}$ = Velocity through the bridge opening - taken directly from the E431 program printout for $Q \leq Q_{ot}$, but calculation based on area of flow in the bridge opening for $Q > Q_{ot}$ i.e. $(V_{b.o.})_{w/ot} = \left[\frac{(\text{Area of Flow in b.o.})_{w/ot}}{(\text{Area of Flow in b.o.})_{w/o\ ot}}\right]^{2/3} (V_{b.o.})_{w/o\ ot}$

Table A.8 Annual Capital Cost Computations

CRF (25 years, 7 1/8%) = 0.08678

Bridge Length (ft)	Embankment Elevation (ft)	Construction Cost Bridge ($)	Embankment & Pavement ($)	Abutment Protection ($)	Total ($)	Annual Capital Cost for 25 yr, 7 1/8% ($/yr)
280	147	2512K	126K	500K	3138K	272.3K
	149	2526K	217K	500K	3243K	281.4K
	151	2526K	365K	500K	3391K	294.3K
	153	2526K	531K	500K	3563K	309.2K
	155	2526K	734K	500K	3760K	326.3K
440	147	2438K	126K	500K	3064K	265.9K
	149	2452K	216	500K	3168K	274.9K
	151	2452K	363	500K	3315K	287.7K
	153	2451K	534	500K	3485K	302.4K
	155	2451K	729	500K	3680K	319.3K
600	147	2779K	126K	500K	3505K	295.5K
	149	2793K	214K	500K	3507K	304.3K
	151	2793K	357K	500K	3650K	318.0K
	153	2793K	524K	500K	3817K	331.2K
	155	2793K	715K	500K	4008K	347.8K
800	147	3070K	126K	-	3196K	277.3K
	149	3083K	211K	500K	3794K	329.2K
	151	3083K	350K	500K	3933K	341.3K
	153	3083K	513K	500K	4096K	355.4K
	155	3083K	699K	500K	4282K	371.6K
1200	147	3582K	126K	-	3708K	321.8K
	149	3594K	205K	-	3799K	329.7K
	151	3595K	336K	-	3931K	341.1K
	153	3596K	489K	500K	4585K	397.9K
	155	3596K	655K	500K	4761K	413.1K
1600	147	4094K	126K	-	4220K	366.2K
	149	4104K	199K	-	4303K	373.4K
	151	4107K	322K	-	4429K	384.3K
	153	4107K	466K	-	4573K	396.8K
	155	4107K	632K	-	4739K	411.2K

Table A.9 Work Table for Risk Analysis (For All Hand Calculations)

Trial Design Discharge: ______
Embankment Elevation: 147
Bridge Length: 280

Est. Capital Cost: $3,138,000
Service Life: 25 Years
Annual Capital Cost: $272,312

Discount Rate: 7 1/8 %

		Cost ($)					Average Costs for P Interval ($)				Annual Risks = Avg. Cost X P ($/yr)			
Q	P	Traffic Inter.	Emb & Pave. Repairs	B.W. Damage	Other	Delta P	Traffic	Emb.	B.W.	Other	Traffic	Emb.	B.W.	Other
54,200	.1000	0	0	0	0									
						.0334	24.3K	0	15.4K	0	809	0	516	0
62.000	.0666	48.6K	0	30.9K	0									
						.0167	89.8K	15.7K	24.4K	0	1,499	262	408	0
68,000	.0500	130.9K	31.4K	18.0K	0									
						.0100	190.2K	49.8K	21.2K	0	1,902	498	212	0
73,000	.0400	249.4K	68.1K	24.3K	0									
						.0200	424.7K	139.4K	32.0K	0	8,494	2,788	641	0
90,500	.0200	600.0K	210.7K	39.8K	0									
						.0100	622.4K	240.8K	31.0K	0	6,224	2,408	310	0
110,000	.0100	644.8K	270.8K	22.1K	0									
						.0038	654.2K	270.8K	24.2K	0	2,486	1,029	92	0
121,000	.0062	663.5K	270.8K	26.4K	0									
						.0012	670.4K	270.8K	25.8K	0	804	325	31	0
131,000	.0050	677.3K	270.8K	25.2K	0									
						.0030	692.0K	270.8K	89.2K	0	2,076	812	268	0
164,000	.0020	706.6K	270.8K	153.2K	0									
						.0020	706.6K	270.8K	153.2K	0	1,413	542	306	0
										Totals:	$25.71K Traffic	$8.66K Embank	$2.78K (B.W.)	0 Other

Table A.10 TOTAL EXPECTED COSTS (TEC'S)

Bridge Length (ft)	Embank. Elev. (ft)	Annual Risks = $\sum(\Delta p \times \text{Avg Loss})$ Traffic $/yr	Repairs $/yr	B.W. $/yr	Other $/yr	Annual Capital Cost $/yr	TEC $/yr	Overtopping Flows Q_{ot} (cfs)	Freq. (yrs)
280	147	25.71K	8.66K	2.78K	0	272.3K	309.5K	60,000	13
	149	17.42K	5.73K	5.66K	7.5K	281.4K	317.7K	73,600	25
	151	9.39K	3.25K	15.59K	8.38K	294.3K	331.0K	88,500	43
	153	5.77K	1.88K	27.74K	15.0K	309.2K	354.6K	103,600	83
	155	4.26K	1.46K	36.73K	15.0K	326.3K	378.8K	110,000	100
440	147	25.67K	8.46K	2.78K	0	265.9K	302.8K	60,000	13
	149	13.80K	4.55K	5.66K	3.75K	274.9K	302.6K	78,000	29
	151	8.94K	2.98K	13.49K	7.50K	287.7K	320.6K	90,100	50
	153	4.73K	1.44K	22.59K	9.52K	302.4K	340.8K	106,900	90
	155	3.25K	0.92K	29.31K	11.25K	319.3K	364.1K	116,900	125
600	147	25.64K	8.25K	2.78K	0	295.5K	332.2K	60,000	13
	149	13.76K	4.44K	4.65K	0	304.3K	327.2K	78,000	29
	151	6.99K	2.23K	10.24K	0	318.0K	337.5K	97,000	62
	153	4.28K	1.24K	15.52K	0	331.2K	352.3K	109,000	95
	155	1.92K	0.37K	18.78K	2.02K	247.8K	370.9K	130,000	192
800	147	25.59K	7.99K	2.78K	0	277.3K	313.7K	60,000	13
	149	13.00K	4.30K	4.65K	0	329.2K	351.2K	78,000	29
	151	6.96K	2.16K	8.42K	0	341.3K	358.9K	97,000	62
	153	2.90K	0.72K	12.07K	0	255.4K	371.1K	117,000	125
	155	1.28K	0.12K	14.19K	0	371.6K	387.2K	143,500	285
1200	147	25.50K	7.48K	2.78K	0	321.8K	357.6K	60,000	13
	149	12.91K	4.03K	4.65K	0	329.7K	351.3K	78,000	29
	151	6.91K	2.02K	7.83K	0	341.1K	357.9K	97,000	62
	153	2.64K	0.60K	10.08K	0	397.9K	411.2K	119,000	142
	155	0.32K	0.03K	11.33K	0	413.1K	424.8K	153,200	370
1600	147	25.03K	6.96K	2.78K	0	366.2K	401.0K	60,000	13
	149	12.83K	3.75K	4.65K	0	373.4K	394.7K	78,000	29
	151	6.85K	1.88K	7.83K	0	384.3K	400.9K	97,000	62
	153	2.34K	0.49K	9.79K	0	396.8K	409.4K	121,000	160
	155	0.16K	0	10.86K	0	411.2K	422.2K	157,100	416

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Example Problem B - Culvert Problem

This example illustrates the application of risk analysis to the design of a culvert.

Problem Conditions: It is desired to design a circular culvert under a two-lane highway. Culvert length is 100 feet. The equivalent average daily traffic is 3000 vehicles per day. The discount rate used in 7 1/8 percent and the useful life of the structure is 35 years.

The flood range used in the analysis is:

Return Interval	Exceedance Probability	Discharge
5	0.02	100
10	0.10	150
20	0.05	170
40	0.025	190
80	0.0125	200
160	0.00625	230

The alternative designs included are:

Culvert Diameter (in)	Elev. Top of Fill (ft)
48	316
54	316
60	316
66	316

The economic losses due to traffic interruption, backwater and damage to the embankment have been assessed and the results are given below.

Economic Losses

Culvert Diameter	Fill Elev.	Exceedance Probability 0.20	0.10	0.05	0.025	0.0125	0.00625
48	316	0	150	375	490	650	928
54	316		0	105	275	460	710
60	316				0	159	510
66	316					0	248

The annual capital and maintenance costs are:

Culvert Diameter	Capital Cost	Annual Capital Cost	Annual Maintenance Cost	Annual Culvert Cost
48	4090	355	25	380
54	5340	463	20	483
60	6600	573	15	588
66	8320	722	10	732

The annual risk costs for the 48-inch culvert are:

Q	Probability	Losses	Average Losses	Delta Probability	Annual Risk
100	0.20	0			
			75.00	0.10	7.50
150	0.10	150			
			262.50	0.05	13.13
170	0.05	375			
			432.50	0.025	10.81
190	0.025	490			
			570.00	0.0125	7.13
200	0.0125	650			
			789.00	0.00625	4.93
230	0.00625	928			
			928.00	0.00625	5.80
	0	928			

Risk = 7.50 + 13.13 + 10.81 + 7.13 + 4.93 + 5.80

= $49.30

The annual risk costs for all the alternative designs are included in the TEC table below.

Culvert Diameter	Annual Capital Cost $	Annual Risk Cost $	Total Expected Cost $
48	380	49.30	429.30
54	483	20.07	503.07
60	588	6.28	594.28
66	732	2.32	734.32

The LTEC design is therefore the 48-inch culvert.

In the above analysis, it was assumed that the culvert did not fail under any flood condition. If the culvert is assumed to fail when the embankment losses are greater than 50 percent the following results are obtained:

Culvert Diameter	Annual Capital Cost $	Annual Risk Cost $	Total Expected Cost $
48	380	202.66	582.66
54	483	20.07	503.07
60	588	6.28	594.28
66	732	2.32	734.32

In this case, the LTEC design changes to the 54-inch culvert. The culvert failure was treated as an additional loss by adding the cost to replace - using initial cost data - in the computation of the annual risk costs. The failure criteria was only triggered for the 48-inch culvert design for floods of 190 cfs or greater. The computations for the 48-inch culvert are shown below.

Q	Probability	Losses	Average Loss	Delta Probability	Annual Risk
100	0.20	0			
			75.00	0.10	7.50
150	0.10	150			
			262.50	0.05	13.13
170	0.05	375			
			2477.50	0.025	61.93
190	0.025	4580			
			4660.00	0.0125	58.25
200	0.0125	4740			
			4879.00	0.00625	30.49
230	0.00625	5018			
			5018.00	0.00625	31.36
	0.0	5018			

Risk = 7.50 + 13.13 + 61.93 + 58.25 + 30.49 + 31.36 = $202.66.

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Example Problem C - Design Component Problem

The LTEC procedures may be applied in making decisions on design components. For example, a designer may wish to assess the problem of whether or not to add spur dikes to an exising bridge that has experienced significant damage due to abutment scour for a 40-year return interval flood event.

The following data are available:

1. Remaining life of bridge = 25 years.

2. Repair cost for abutments vary from zero for the 20-year event to $75,000 for floods greater than 40-year event. Exceedance probability = 0.025.

3. Cost to construct spurs dikes = $10,000.

The design alternatives are to leave the site as is or to add spur dikes.

Assumptions:

1. Spurs will be half destroyed by any flood greater than a 20-year event. Exceedence probability = 0.05.

2. Repair cost to the abutments are constant for all floods greater than the 40-year event.

3. Repair costs to the spur dikes vary from zero for the 10-year event to 1/2 the initial cost for the 20-year and larger flood events. For the 100-year and larger floods, abutment damage of $75,000 will occur and the spur dikes will be completely lost.

4. Backwater damage is negligible.

5. Traffic delay costs are negligible.

Analysis:

Annual capital cost of spur dikes:

$10,000 [(CRF) (7 1/8 %; 25 yrs)]

Capital Recovery Factor for 7 1/8 % discount rate for 25 years = 0.08679

10,000(0.08679) = $867.90/year.

The annual risk costs for the alternatives are:

Frequency	Probability	Losses	Average Losses	Delta Probability	Annual Risk
		Condition With Spur Dikes			
10	0.10	0			
			2500	0.05	125.00
20	0.05	5000			
			5000	0.025	125.00
40	0.025	5000			
			45000	0.015	675.00
100	0.01	85000			
			85000	0.01	850.00
	0	85000			
			Total-------------------		$1775.00
		Condition Without Spur Dikes			
10	0.10	0			
			0	0.05	0
20	0.05	0			
			37500	0.025	937.50
40	0.025	75000			
			75000	0.025	1875.00
	0	75000			
			Total-------------------		$2812.50

The TEC's are then:

Alternative	Annual Capital Cost ($)	Annual Risk Cost ($)	TEC ($)
With Spurs	867.90	1775.00	2642.90
Without Spurs	0	2812.50	2812.50

Based on this analysis, the designer would probably elect to recommend spur dikes. However, he may also wish to investigate other design alternatives such as riprapping the spur dikes, which would increase the capital cost but reduce the risk costs.

Example Problem D. Economic Risk Analysis of the State Highway 63 Crossing of the South Platte River, South of Atwood, Colorado

This appendix documents the application of the LTEC design procedure to the South Platte River crossing. Mr. Lawrence E. Dezman, P.E., Hydraulics Squad, Colorado Division of Highways was the principal investigator and author of the design report. The report is presented here as provided by the Colorado Division of Highways.

I. INTRODUCTION

Three timber bridges on State Highway (S.H.) 63 over the South Platte River near Atwood, Colorado were scheduled for replacement by the Colorado Division of Highways (CDOH) under the highway bridge replacement and rehabilitation program. Federal-Aid Highway Program Manual volume 6, chapter 7, section 3, subsection 2 requires that a bridge be sized by economic risk analysis if the level of study is commensurate with the expense. Selection of bridge length over the South Platte has not been clear cut. In the past, multiple bridges were built to span main channels or one bridge was built to span several channels. Replacement of these three bridges in kind would require most of the budgeted $2.5 million dollars. The existing bridges had little capacity considering the length due to a lack of clearance beneath. A logical design methodology was needed by which several different alternate bridge types and lengths could be compared. Hydraulic Engineering Circular No. 17 (HEC-17, 1980) provided that methodology.

The thrust of this report is to document sources of data and stress, by example, that imaginative investigation is a major component of a risk analysis study. The concept and mechanics of risk analysis are discussed briefly herewith, but the details are left to other publications, such as HEC-17.

II. CONCEPT

Economic risk analysis is a scheme by which a design alternate with the least total annual cost is identified. The total cost has two components: the first is the sum of all annual risk costs occasioned by a particular alternate, the second is the annual construction cost.

Flood damage is analyzed to determine risk costs. Flood damages considered in this analysis include building and cropland damage due to backwater, scour damage at spur dikes and abutments, embankment and pavement damage due to flow overtopping and costs to the traveling public due to traffic interruption. Floods considered in this analysis ranged from 20% chance of occurrence in any year to 0.2%. The damages caused by each flood were multiplied by the flood probability in one year to find that flood's risk cost. Flood risk costs were then summed over the range of floods in a discrete integration to determine an alternate's annual risk.

The annual construction cost is the amortization of all costs associated with constructing an alternate. Costs for spur dikes, bridge structure, roadway embankment, pavement, embankment removal and riprip were included. Costs for items common to all alternates and not subject to flood damage were omitted. Discount rates and amortization periods used in computing annual construction costs are discussed subsequently.

If only one design item, such as bridge length, is varied, the results can be graphically shown on a two-dimensional plot (see Figure 1). The low point on the total annual cost curve marks the most economical bridge length if all the important costs have been considered. Plotting the recurrence interval for the overtopping flood permits one to obtain a "feel" for the level of design. Other considerations, thought to be unquantifiable, can be measured against cost by using Figure 1 (e.g., the increased cost of a longer bridge to avoid public criticism of a shorter-than-existing replacement).

A sensitivity analysis is done upon completion of the economic risk analysis to assess the impact analysis assumptions have on results. It can be used to determine if more effort is warranted in gathering data or defining a physical process. For example, if embankment losses (of which little is known) are a major portion of the annual risk, one would lack confidence in study results. A sensitivity analysis enables the designer to compare the impact of these losses against the impact of other, better defined, losses. If the relative impact of embankment losses is small, confidence in study results is restored.

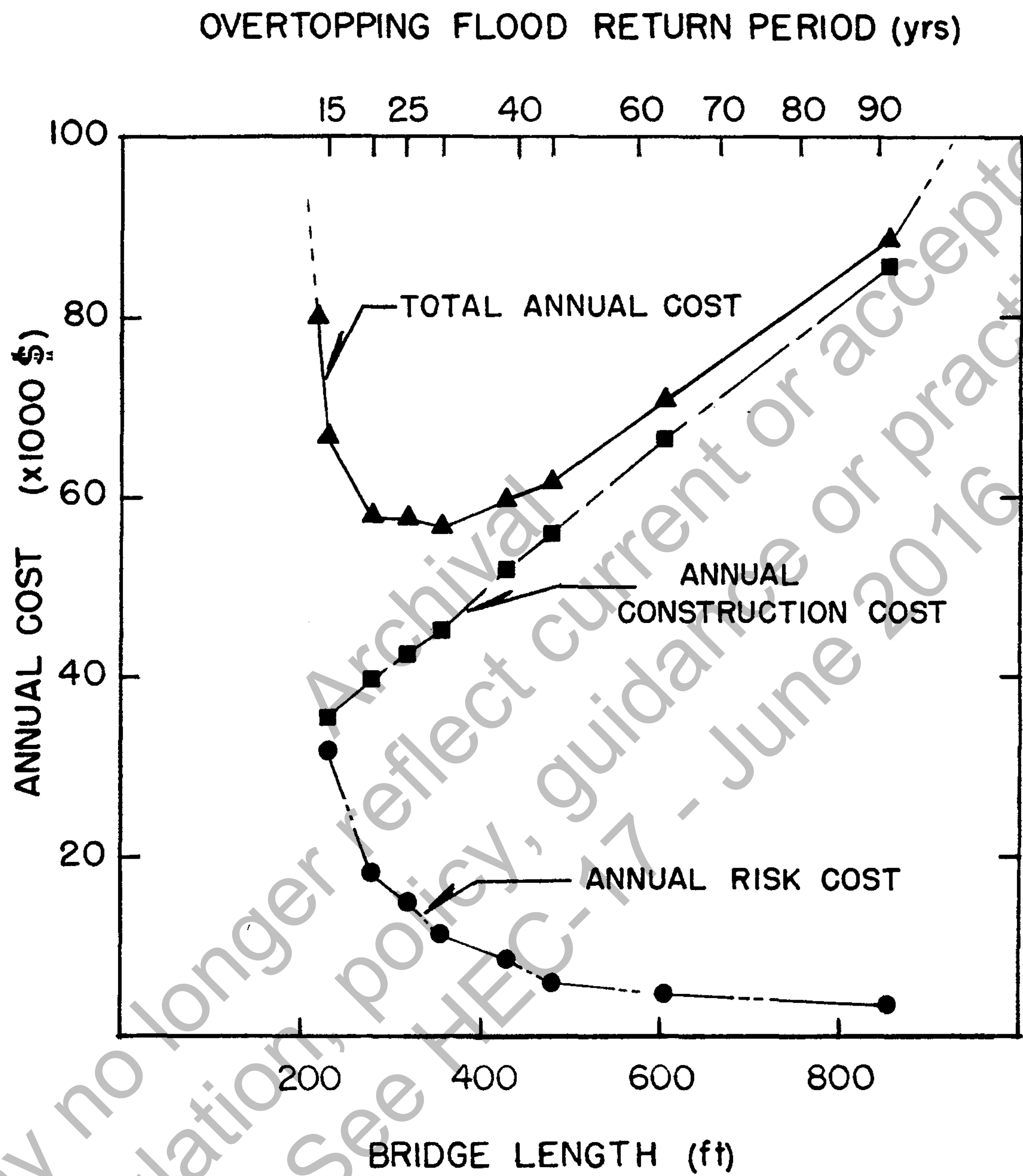

Figure 1. Annual Cost vs. Bridge Length and Overtopping Flood Return Period

III. LOCATION AND PHYSICAL DESCRIPTION

Location.

The site is located near Atwood in Logan County, Colorado, 120 miles northeast of Denver. Highways U.S. 6 and I-76 parallel the South Platte River from Fort Morgan to the Colorado-Nebraska border (see Figure 2). State Highway S.H. 63 provides one of several links between the two highways as well as access to the town of Akron to the south. The bridge will be situated in T.7 N., R.53 W., Section 26, at an elevation of 3990 feet above mean sea level.

Basin Description.

The South Platte drains 17800 square miles from alpine tundra at 14000 feet in the Mosquito Range to the site near Atwood. The upper river basin flows east to southeasterly, exiting the mountains southwest of Denver. It then flows northeasterly from the foothills, through Denver and across the plains to the rolling hills flanking the flood plain at the site. Along its course below metropolitan Denver, the river flows through the towns of Greeley, Fort Morgan and Brush.

Many mountain streams and ephemeral washes are tributary to the South Platte above the site. The major mountain streams include the Big and Little Thompson Rivers, St. Vrain Creek, Cache la Poudre River, Boulder Creek and Clear Creek. All of these enter the South Platte on the plains as the river makes its way northeasterly past the Front Range of the Rocky Mountains. Major plains streams include Bijou, Kiowa, Lone Tree, Cherry and Plum Creeks.

Many reservoirs regulate both the main stem and tributary streams. Notable among these are Chatfield Lake, Cherry Creek Lake and Elevenmile Canyon Reservoir. Elevenmile Canyon is one of several facilities either being operated or constructed on the South Platte for Denver's water supply. The U.S.G.S. (1980), in the text preceeding gage data for Balzac (#06760000) states, "Natural flow of stream affected by transmountain and transbasin diversions, storage reservoirs, power developments, ground-water withdrawals and diversions above station for irrigation of about 1065000 acres, and return flow from irrigated areas."

Channel and Floodplain Description.

The South Platte's channel in the vicinity of the site has a coarse sand to fine gravel bed (D_{50}=1mm.). Alluvial deposits extend more than 80 feet below the existing thalweg. Valley slope through the reach is 7 feet per mile.

The channel morphology has changed over the past 80 years. Shen (1971) states,

> "The South Platte River has always been cited as a classic example of a braided stream. About 55 miles above its

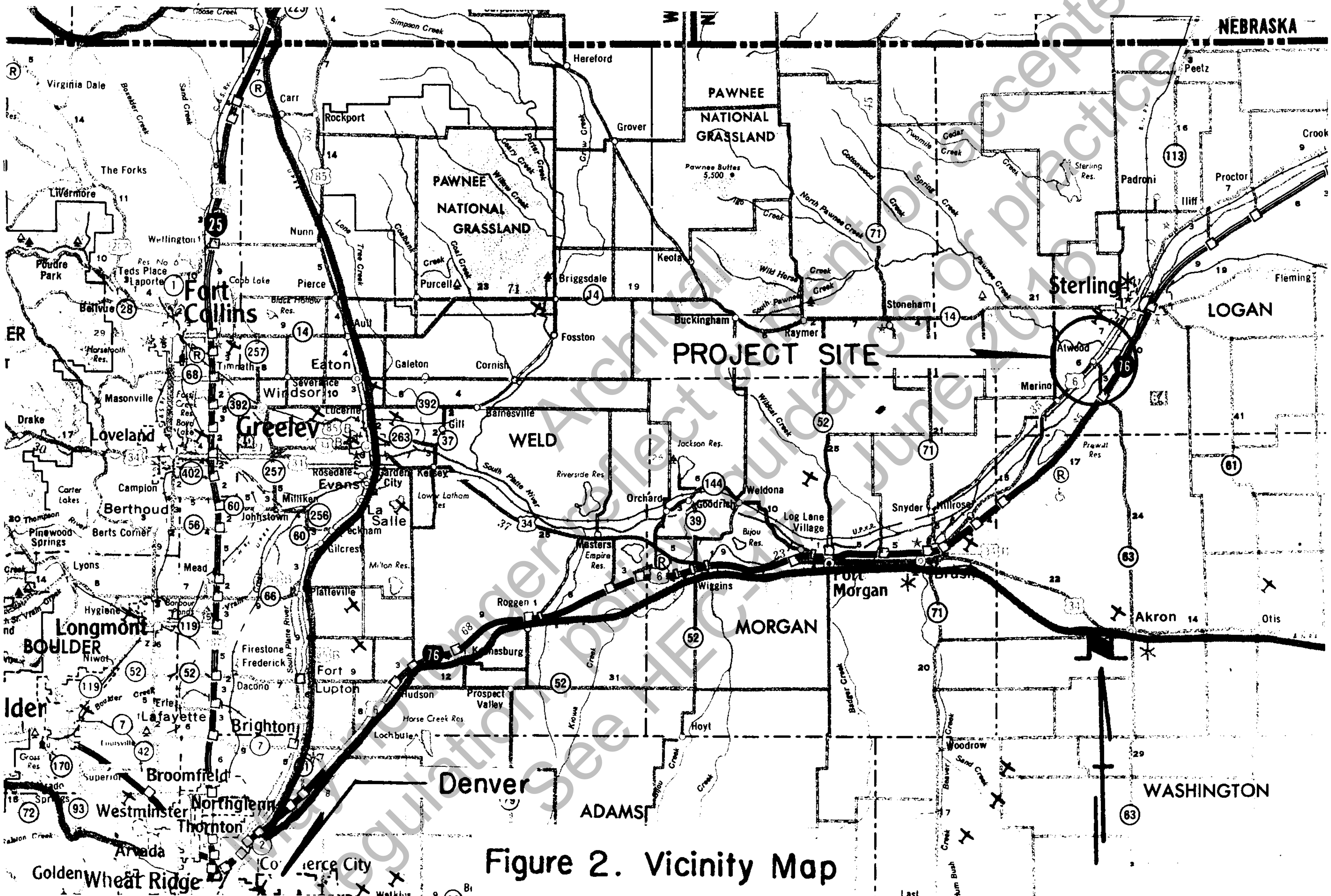

Figure 2. Vicinity Map

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S of Atwood at S Platte R

junction with the North Platte River, the South Platte River was about a half mile wide in 1897, but it has narrowed to about 200 feet wide by 1959.

The tendency of both rivers is to form one narrow well-defined channel in place of the previously wide braided-channels. In addition, the new channel is generally somewhat more sinuous than the old."

Shen (1971) attributes the majority of the present narrow channel to a decrease in the annual momentary maximum discharges. Close scrutiny of stream gage data shows no statistically significant trends in annual momentary maximums. These data represent a lengthy record dating back to 1902 for the Julesburg gage (#06764000), 58 miles downstream and to 1918 for Balzac, 15 miles upstream.

Historical accounts indicate that prior to 60 years ago, the South Platte flood plain was, unlike today, devoid of trees. The trees, mainly willows and cottonwoods, presumably have spread from intermittent plantings along the river to cover the river bottom (see Figure 3). Expanding use of water throughout the basin during this period has stabilized low flows, thereby providing a constant source of water for phreatophytic vegetation. It is conjectured that increased vegetation has decreased sediment transport capacity by capturing sediments and stabilizing them with plant roots. Consequently, this reach of the river is aggrading. Local residents support this idea by citing farm-related hydraulic appurtenances along the South Platte that have been sanded in at an increasingly rapid rate since they were built.

Since aggradation of a river channel can have large effects on bridge capacity, its occurrence was investigated further. Stream gage meter notes kept by the Colorado Division of Water Resources (DWR) were examined to find evidence of aggradation. Each stream gage administered by DWR on the South Platte is rated biweekly by measuring depth and velocity at intervals across the river cross section. The thalweg is identified at the maximum depth of flow. Data over the period from 1949 to 1978 at the Balzac gage shows a 3.4 foot increase in thalwag elevation. Meter notes for Balzac are available back to 1917, unfortunately a reliable gage datum is not available prior to 1935.

Concurrent with the vegetation and aggradation, the river's path has become more sinuous as shown on aerial photos and U.S.G.S. contour map (see Figure 4). Even floodplain flows, which formerly proceeded straight downslope, follow a sinuous path. An oxbow lake exists less than 2000 feet downstream of the bridge site offering further evidence of sinuosity.

The three mile wide flood plain near the site is predominantly loamy alluvium underlain by sand and gravel. Uplands flanking the flood plain are 50 to 100 feet higher. The northern uplands are of wind-blown sandy loam and alluvial sand. The southern uplands are

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LOOKING UPSTREAM FROM EXISTING BRIDGE

LOOKING DOWNSTREAM FROM EXISTING BRIDGE

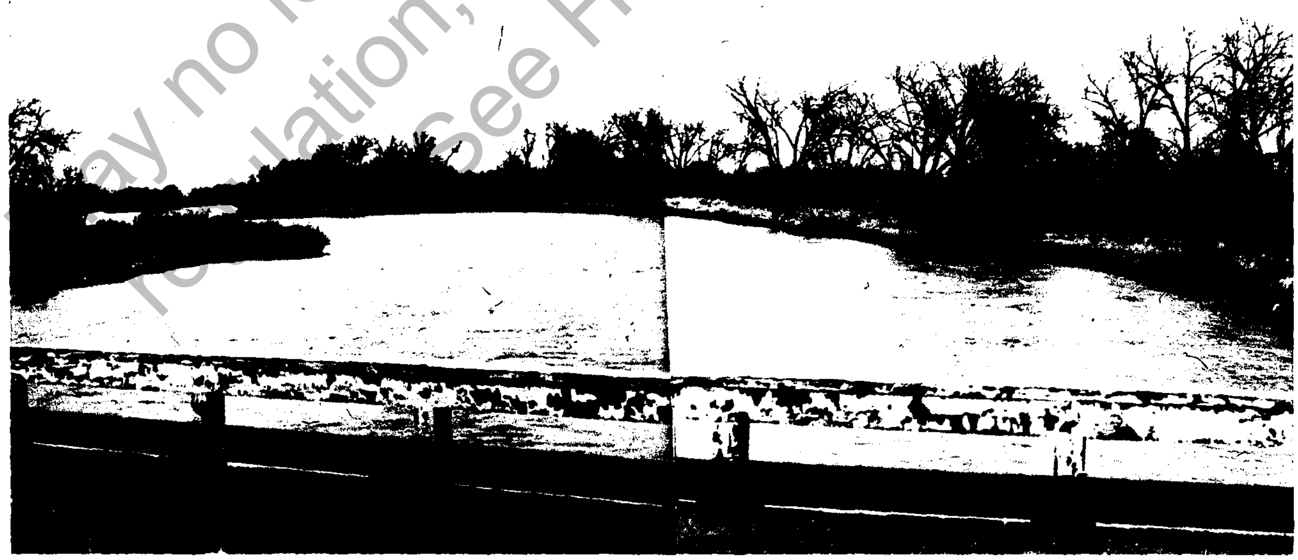

Figure 3. Photos of Flood Plain

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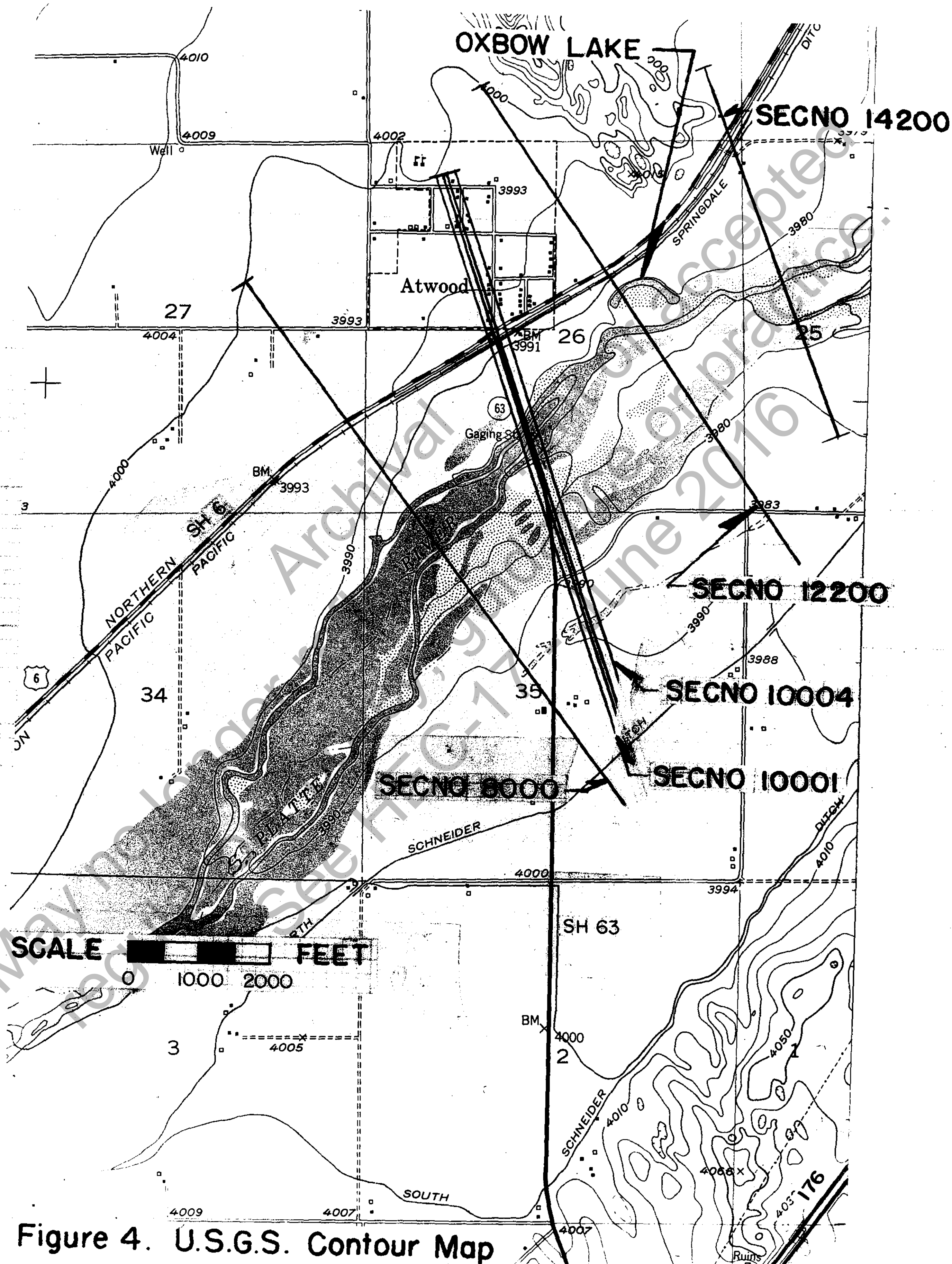

Figure 4. U.S.G.S. Contour Map

covered by blow sand (Amen, et al, 1977). Land immediately adjacent to the river is primarily rangeland while the remainder is irrigated cropland, pastureland and hayland (Soil Conservation Service, SCS, 1972).

Site Description.

The existing treated timber bridges span the three major channels of the river present when they were built in 1937 (see Figure 5). The S.H. 63 embankment crosses much of the flood plain at a constant elevation of 3990 above mean sea level. The Union Pacific Railroad and U.S. 6 on the north edge of the flood plain are within a couple feet of 3990; there is little topographic relief in the area. Neither the railroad and highway on the north nor the farm road on the south significantly affect conveyance.

The longest existing bridge over the south channel carries the least flow because the stream bed has aggraded. The middle bridge (which is also the middle length) is over the present main channel. The majority of non-overtopping flows are carried by the middle and northern bridges. In May 1980, converging south bank floodplain flow was observed to bypass the southern bridge flowing northerly to the middle bridge. Apparently a downstream control in the south channel has caused aggradation and severe reduction in bridge capacity.

The bridges are supported on timber pile bents, spaced 23 feet center to center. Vegetation growing on the sandy flood plain is easily eroded and provides an abundant source of debris. These bridges comb debris from the flow, necessitating a 24 hour maintenance vigil during high water.

Figure 5. Aerial Photo of Site

IV. ANALYSIS CONSIDERATIONS

Useful Life.

Useful life is necessary in the capital recovery factor to amortize construction cost. Annual construction cost, being more than 50% of all alternates' total annual costs as will be shown later, is very sensitive to the useful life. Careful selection is imperative.

Winfrey (1969) states, "With the exception of retirement by accident and disaster, the highway property is retired only upon decision of management." He further states, "There are no specific or universal guide posts upon which the decisions to retire property from service are based." He cites service lives of 50 to 75 years for bridges and other major structures. The American Association of State Highway and Transportation Officials (AASHTO, 1977) offers, ". . . the design year for highway improvements is typically 25 years or less, and traffic projections are not ordinarily made beyond that point, so an analysis period of 15 to 25 years is generally used."

The existing timber bridges at the site were constructed in 1937, giving them a service life of 44 years (construction is to occur in 1981). Other bridges along the South Platte are of similar age. This may be an indicator of CDOH policy on bridge longevity.

A useful life of 30 years was used in the analysis. Selection of shorter useful life makes a shorter bridge more attractive. The effect of 20 and 40 year lives was considered in the sensitivity analysis.

Discount Rate.

The discount rate is used with the useful life in amortizing construction costs. Various authorities advocate widely differing discount rates for use in public project economic studies. The Water Resources Council (WRC) is charged with annually publishing the discount rate that Federal agencies must use for water projects. This rate is tied to United States interest bearing securities but cannot be changed by more than one quarter percent per year. For 1980 the WRC interest rate is 7-1/8 percent.

Winfrey (1971) advocates rates based on, ". . . consideration of several factors such as current rates being paid by road users for personal financing, car purchase financing, business financing, governmental financing and earning rates that road users are able to achieve through investment in stocks, bonds, business ventures, and financial transactions." At current rates, 12% would be representative of these rates of returns.

Interest rates can be formulated as follows:

Interest rate (%) = inflation (%) + profit (%) + availability premium (%) + risk premium (%)

The last two factors are the real cost of borrowing (renting) money. The rate paid to obtain a portion of a limited money resource is the availability premium while the risk premium covers the chance that a borrower will default.

According to AASHTO (1977), ". . . if future benefit and costs are calculated in constant dollars, only the real cost of capital should be represented in the discount rate used. The real cost of capital has been estimated at about 4 percent in recent years for low-risk investments."

A discount rate of four percent will be used for this study. The WRC rate and Winfrey's suggestion were not used because of their inclusion of inflation factors. The rate, however, will be varied to the extremes noted for the sensitivity analysis. The use of a low discount rate will have the effect of lowering the annual construction cost, thereby making a more expensive (longer) structure more attractive.

Loss of Life Potential.

The potential for loss of human life in the backwater area is low because no buildings are in the direct path of flood waters. Loss of life potential for the vehicles moving over the crossing while a flood overtops the embankment is also small. Flood waters can rise rapidly on the South Platte, but the long-level embankment provides much flow relief with small increases in stage. Travelers over the detour route will be exposed to a greater chance of death since records show no fatal accidents have occurred on S.H. 63 between I-76 and U.S. 6. Deaths have occurred on the detour. This will be discussed under traffic losses.

Possibility of Loss of Structure.

Attention to two hydraulic conditions can confirm the assumption of a non-failing structure. These are lateral loads exerted on the superstructure by flowing water and debris and scour holes at bridge piers. A vertical curve will keep much of the superstructure above all but the most extreme floods. At overtopping elevation there will be more than three and one-half feet of freeboard to pass debris floating on the South Platte during high water. Pile caps will be constructed at the scour elevations predicted for a 100 year event. Aggradation of the river bed will effectively provide more scour protection as time passes.

V. COST AND DAMAGE DATA

Cost data were obtained from several sources, many of them within the Division of Highways. An effort was made to obtain detailed cost data from first-hand sources to maximize the applicability of the sensitivity analysis. Subsequent studies may build on these data so than an accurate cost data file can be maintained within the Hydraulic Squad office.

All unit costs for construction wereobtained from the Staff Cost Estimates Squad. Costs related to maintenance were obtained from Staff Maintenance's Maintenance Management System.

Structure costs were estimated by Staff Bridge Branch for preliminary designs submitted for that purpose. They computed sub- and superstructure costs for various typical bridges and reported them on a square foot basis. These costs did not include revetment, spur dikes and roadway surfacing, all of which are a function of bridge length.

Land use maps (SCS, 1973) show rangeland immediately adjacent to the river and cropland on either side. The Sterling SCS office recommended the Colorado State University Extension Service as the source of crop value information. An interview with Jim Reed, the County Extension Director, yielded the information on Table 1. He indicated that a normal crop management plan for the area would have a plot of ground two years in beets, two years in beans, six years in corn, four years in alfalfa, then back to beets. This plan, combined with the values in Table 1, gives an acre of cropland a value of $386 per year.

According to Reed, depth of inundation has minimal effect on crop damage. Duration of inundation does. If inundation and root zone saturation last for greater than two days, 80 to 100% of the crop is lost. The crop may continue to grow, but disease ruins the value of the crop. Inundation for less than one day will cause no damage. This information is plotted on Figure 6. Rangeland would not be damaged by flooding.

Staff Right of Way assisted in valuating flood-prone buildings. Twelve separate properties were identified from site reviews and aerial photographs. Houses were valued at $25000 (average for owner and tennant occupied houses), sheds at $5000 and barns at $30000. The most valuable property upstream of the site is an alfalfa hay processing mill. Book values for similar buildings (Marshall-Swift, 1980) range from $3700 to $12500 per ton of capacity per day as a function of accessibility and conformance with EPA and OSHA standards. A value of $350000 was chosen based on the plant's 17000 ton per year capacity and lack of railroad siding. The previous owner of the mill subsequently confirmed this value. The value of processed hay stored in six silos adjacent to the mill was computed at $9750 per foot of inundation based on market value.

TABLE 1. CROP MANAGEMENT AND VALUES

Crops Grown along the South Platte River in
the Area six to eight miles east of Sterling

CROP	YIELD/ACRE	PRICE/UNIT OF YIELD
beets	16- 20 tons	$20/ton
beans	40- 50 bushels	$15/bushel
corn	100-160 bushels	$3-3.5/bushel
alfalfa	4- 6 tons	$40/ton

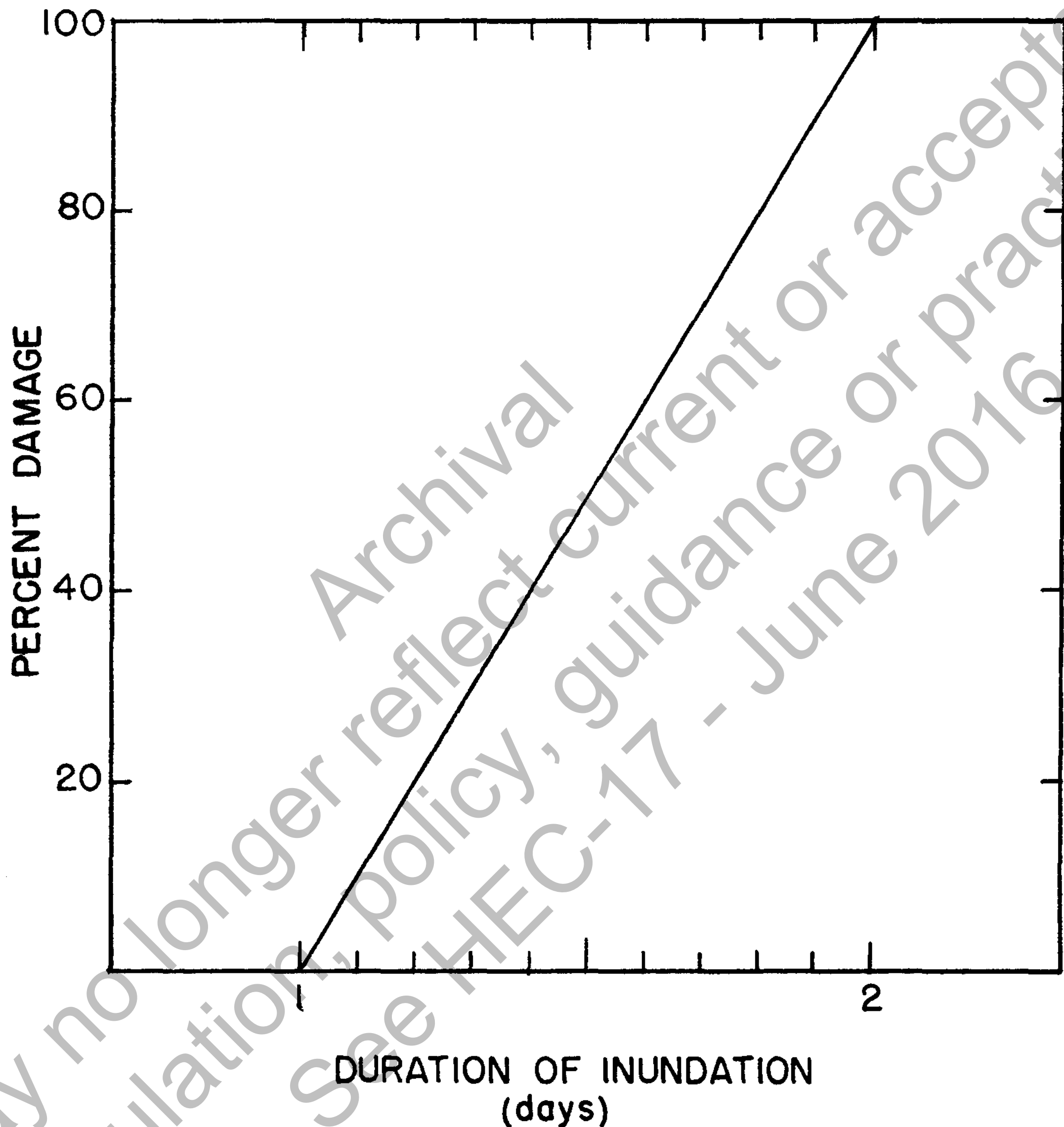

Figure 6. Crop Damage Function

Staff Relocation Section aided with valuating contents of buildings. Their experience indicated, for the range of building types subject to flooding, that contents were worth 50% of building values. Figure 7 is the building damage function given in HEC-17 with the outbuilding damage function developed by the author and Staff Relocation.

Scour costs were determined by working out the following repair scenario with Staff Maintenance Branch. As bed material is lost, maintenance end dumps and blades into place broken concrete riprap. The nearest stockpile of concrete is in the Sterling maintenance yard, seven miles away. Ten minutes of front end loader time is required to fill the available 2-1/2 ton or mid-range trucks. Trucks travel to the site at 50 mph where it will take ten minutes to unload. Ten minutes of motor grader time are necessary to move the concrete to the correct position. Use of this scenario with the costs noted in Table 2 yields a repair cost of $4.13 per cubic yard of scour damage. Scour volume is computed as a function of depth. Scour depth will be discussed later.

Average daily traffic (ADT), turning movements and vehicular classifications were obtained from the Traffic Survey Section of the Division of Transportation Planning. CDOH (1978) was the source of accident data for S.H. 63 and detour routes. Cost of accidents was obtained from the National Safety Council (1980). Traffic data is summarized in Table 3.

Time costs were computed by converting average income data from the Demographic Profile provided by the Colorado Department of Health (1979) to hourly rates per person per car. Since it was not expected that traffic costs would have a large effect on the results, the cost per hour of a vehicle occupant's time was conservatively estimated to be $4.00 per hour. This would be conservative in Logan County because the average family of four earned slightly more than $12000 in 1978. The number of occupants per car was estimated at 1.8.

Percentage of embankment and pavement loss was estimated from Figure 8. Figure 8 is reproduced from HEC-17 and is applicable to this site because it is based on observations of the same type of roadway template; i.e., two lane road with paved shoulders and 3 to 1 vegetated side slopes of sandy soil.

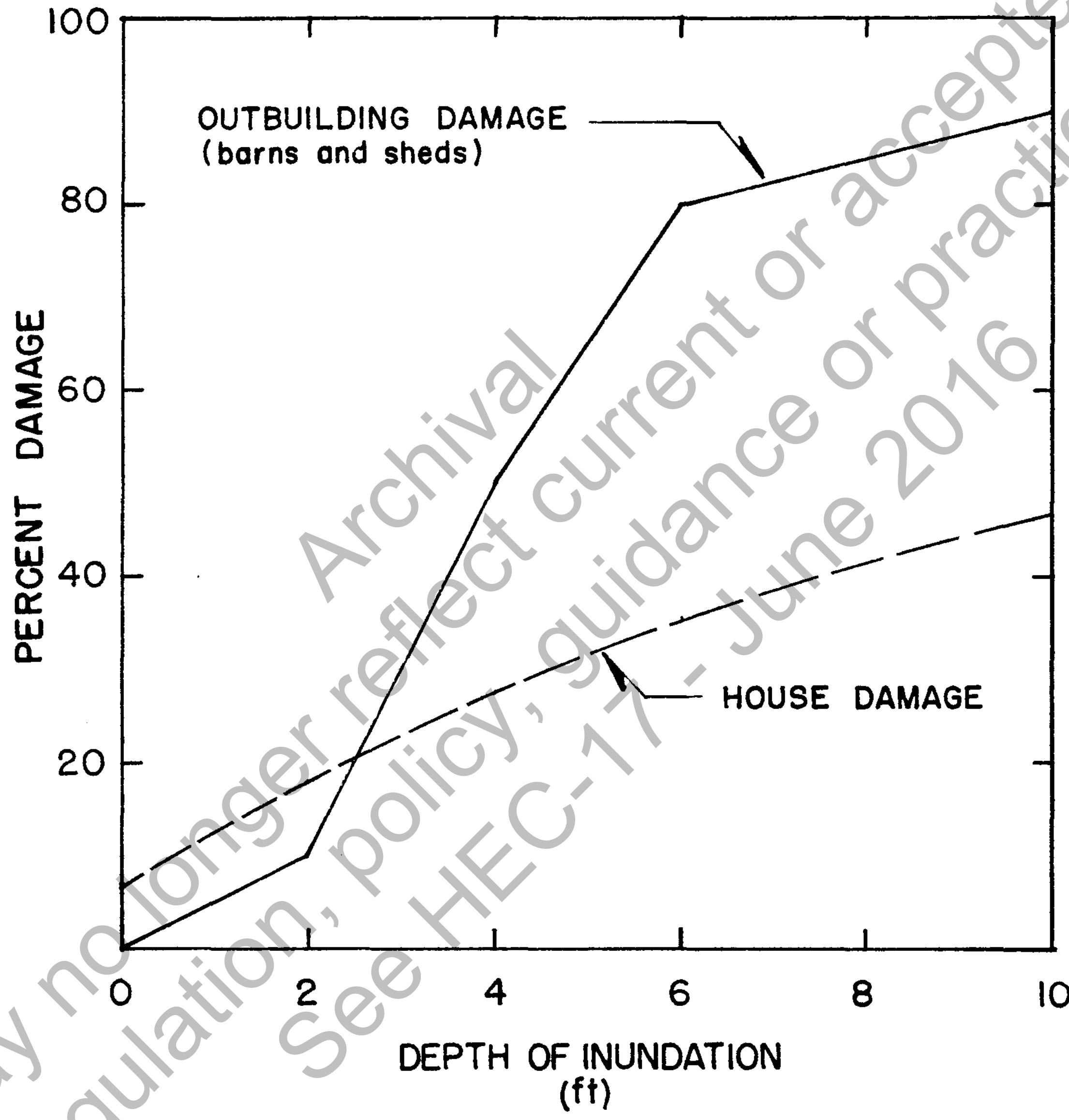

Figure 7. Building Damage Function

TABLE 2. Maintenance Equipment and Labor Data Used in Computing Scour-Repair Costs. (From Maintenance Management Systems data for 1980)

ITEM	COST ($/HR.)
Operator Costs	12.69
1-1/2 Cubic Yard Front End Loader	10.88
2-1/2 Ton Dump Truck	5.95
Mid-Range Truck	6.19
Motor Grader	13.92

TABLE 3. TRAFFIC DATA SUMMARY

Average Daily Traffic (ADT)

YEAR	VEHICLES/DAY
1980	1550
1990	1990
2000	2250

TRAFFIC MIX VEHICLE	FRACTION	RUNNING COST FACTOR
Car	0.51	1.0
Pick-up Truck	0.32	1.2
Single Unit Truck	0.10	1.5
Combination Truck	0.07	3.2

Unit Cost of Operation (including tires, oil, gas, parts, repair) - \$0.12/vehicle-mile.

ACCIDENT COSTS		(\$/ACCIDENT)
Property Damage only	(pdo)	850
Injury Accidents	(inj)	5800
Fatalities	(fat)	150000

ACCIDENT RATES (Accidents/10^6 vehicle miles)

ROAD	TERMINI	pdo	inj	fat
SH 63	US 6/I-76	1.884	0.122	0
I-76	SH 63/Sterling	0.506	0.226	0.028
US 6	Sterling/Atwood	3.981	0.826	0.020

Vehicle Occupany Rate - 1.8 persons/vehicle

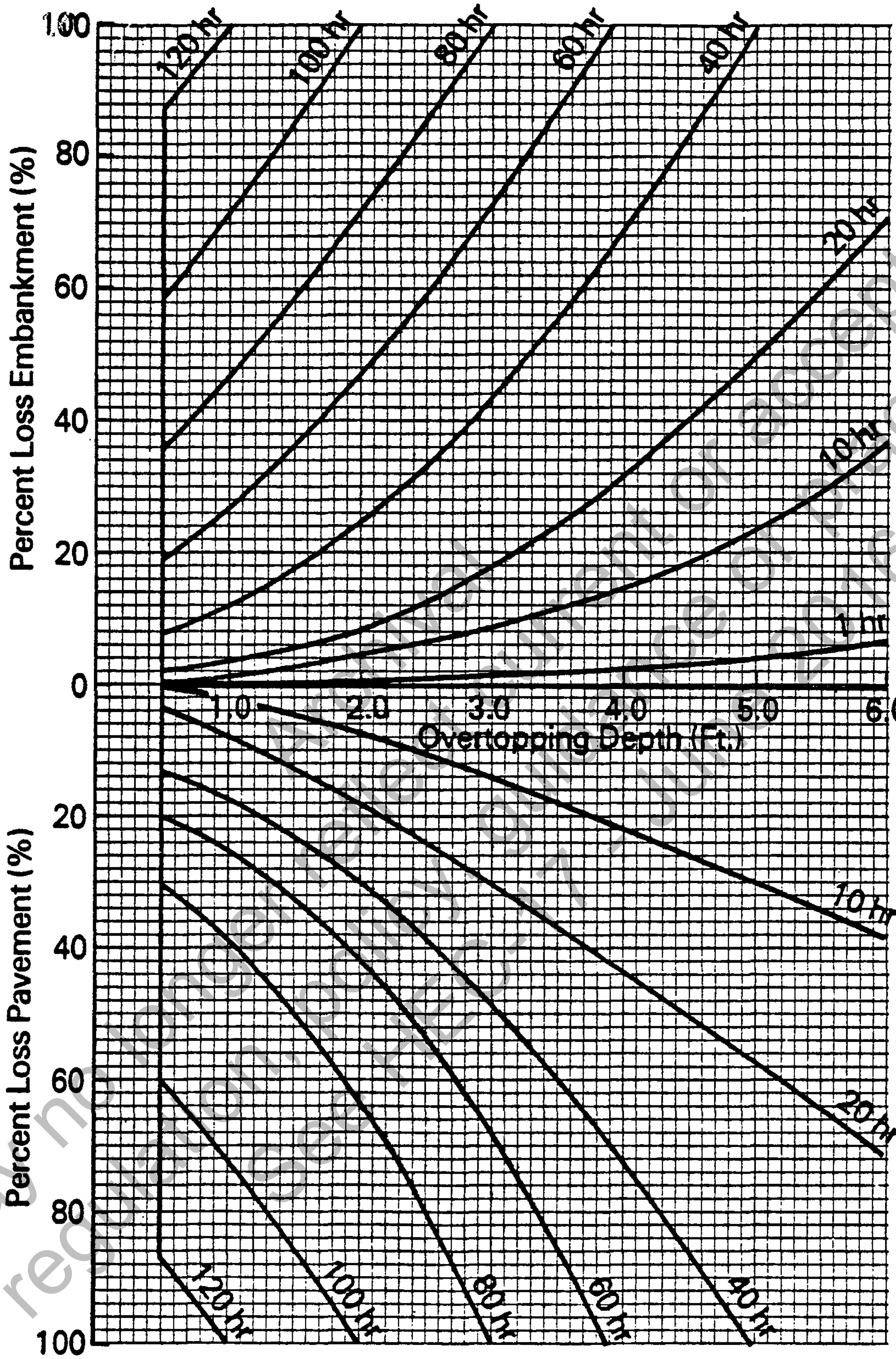

Figure 8. Embankment - Pavement Damage Function

VI. HYDROLOGY AND HYDRAULIC ANALYSIS

Flood Frequency Analysis.

Flood frequency analysis was based on stream gage data for the gages at Balzac and Julesburg. Balzac, upstream of the site, has 60 years of record; Julesburg, downstream has 71 years. Flood frequency predictions were accomplished using log-Pearson III analysis as recommended by the WRC (1977). It should be noted that the flood of record at Balzac is 123000 cfs, recorded in June 1965. Results are tabulated on Table 4 and displayed graphically on Figure 9.

Hydrographs.

Hydrographs were based on gage data from Balzac. Six flood events with peak flows (Q_p) ranging from 7400 to 123000 cfs were superposed on one graph (see Figure 10). The average daily flows (Q_{avg}) were plotted with the peak day placed at zero time. The purpose of this plot is to compare and ascertain average characteristics so that hydrographs for predicted peaks can be constructed. An eight and one-half day duration with peak flow on third day after significant rise is such a characteristic. Also observed from Figure 10 was a relationship between R($Q_p \div Q_{avg}$ for peak day) and Q_p given by the regressed equation

$$R = 2.97 \times 10^{-5} \, Q_p(\text{cfs}) + 0.7661$$

with a correlation coefficient of 0.9972. Also observed was that Q_{avg} at a three day hydrograph width (Figure 10) was 70% of Q_{avg} for peak day.

All hydrograph characteristics noted are a function of Q_p making it possible to construct hydrographs for all floods in Table 4. Constructed hydrographs are shown on Figure 11.

Hydraulic Analysis.

All bridge alternates, the natural and existing conditions were analyzed using the U.S. Army Corps of Engineers' HEC-2 Water Surface Profile computer program. Special Bridge Routine was used for all bridges. Existing bridges were calibrated against field survey obtained in May 1980 during a discharge of 10000 cfs.

Terrain data was obtained from field survey at the site and a U.S.G.S. contour map (HEC-2 cross sections, SECNO's, noted on Figure 4). Field survey delineated the channel in the vicinity of the bridge. The contour map was sufficiently accurate in this flat country to describe the overbanks.

Hydraulic roughness, described by Manning's n in HEC-2, was chosen at 0.035 for the main channel and 0.070 for the vegetated overbanks. The main channel n is probably conservative considering that sand channel bed forms will plane out at high flows.

TABLE 4. PREDICTED FLOODS

EXCEEDANCE PROBABILITY	RETURN PERIOD (yrs)	FLOOD MAGNITUDE (cfs)
20	5	9200
10	10	15500
4	25	29000
2	50	44000
1	100	68000
0.5	200	104000
0.2	500	168000

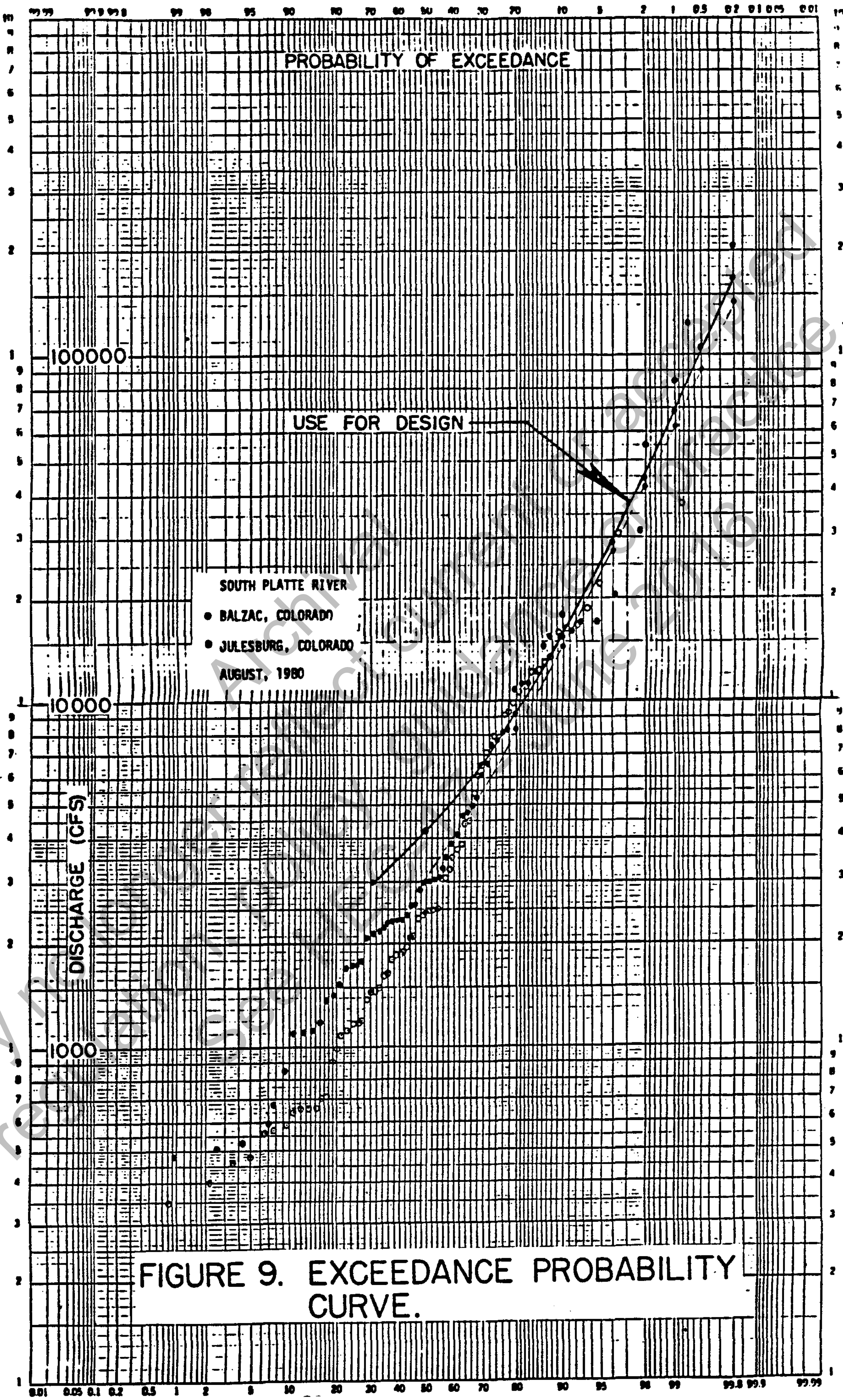

FIGURE 9. EXCEEDANCE PROBABILITY CURVE.

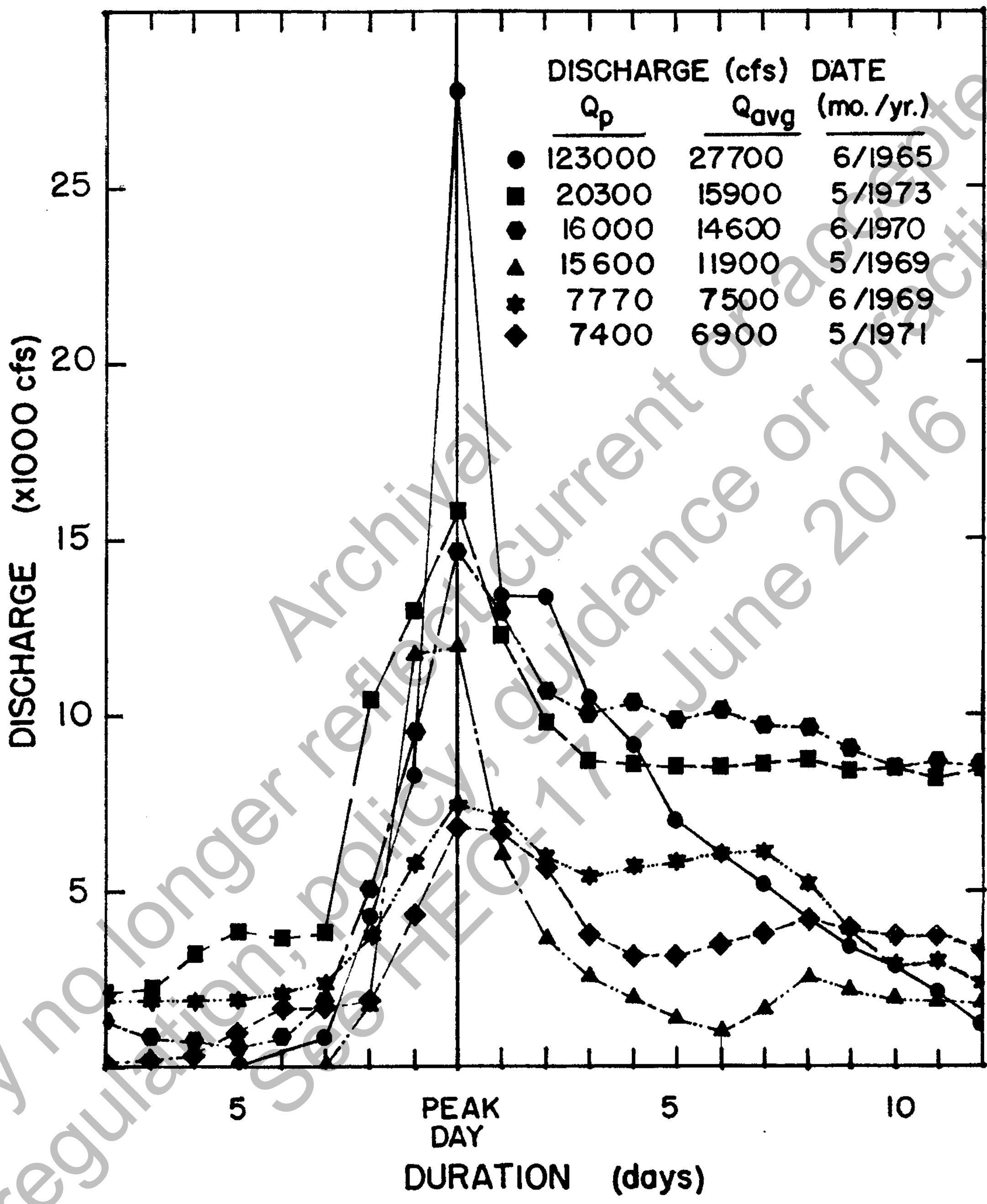

Figure. 10. Balzac Hydrographs

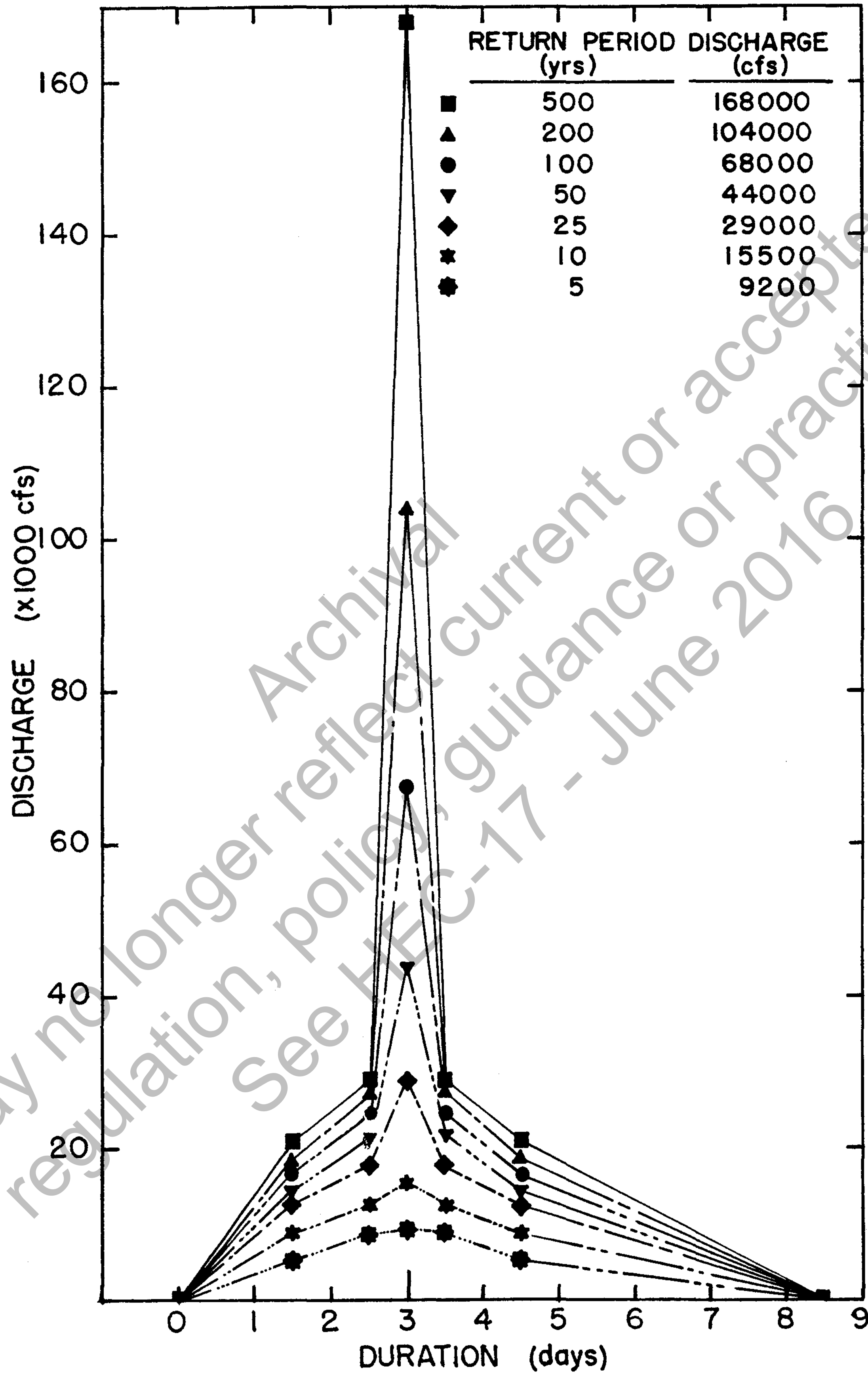

Figure 11. Constructed Hydrographs

Subcritical flows occur through this reach on a channel slope of 0.14%. Consequently, downstream channel and overbank geometry control the river stage. The downstream portion of the model was calibrated, as noted above, for a discharge of 10000 cfs. No calibration data, barring flood stories by Maintenance and local residents, was available for high flows. This shortcoming is not serious, however, since several of the alternates would cause a hydraulic jump downstream. The bridge acting as hydraulic control eliminates the impact of downstream stage.

All bridge alternates modeled were spill-through abutment type. The number of piers was computed by dividing approximate bridge length by 105 feet (typical span length for Colorado G-54 girders) and rounding up to the nearest integer. Pier widths were assumed to be 2.5 feet. A stage vs. area relationship was determined for each alternate to accurately describe the trapezoidal area approximation. Overflow length was determined by subtracting an alternate's bridge and approach length from the existing level roadway length.

Results from computer runs were used as follows:

1. Stage vs. discharge curves were plotted for each alternate for SECNO's 8000 and 10001 (Figure 12 shows stage vs. discharge for SECNO 8000).

   a. SECNO 8000 was used in determining backwater damage,

   b. SECNO 10001 was used to determine overtopping discharges for traffic, embankment and pavement losses.

2. Detailed output of flow distribution, depth and velocity was used in determination of:

   a. spur dike length, and

   b. scour depth

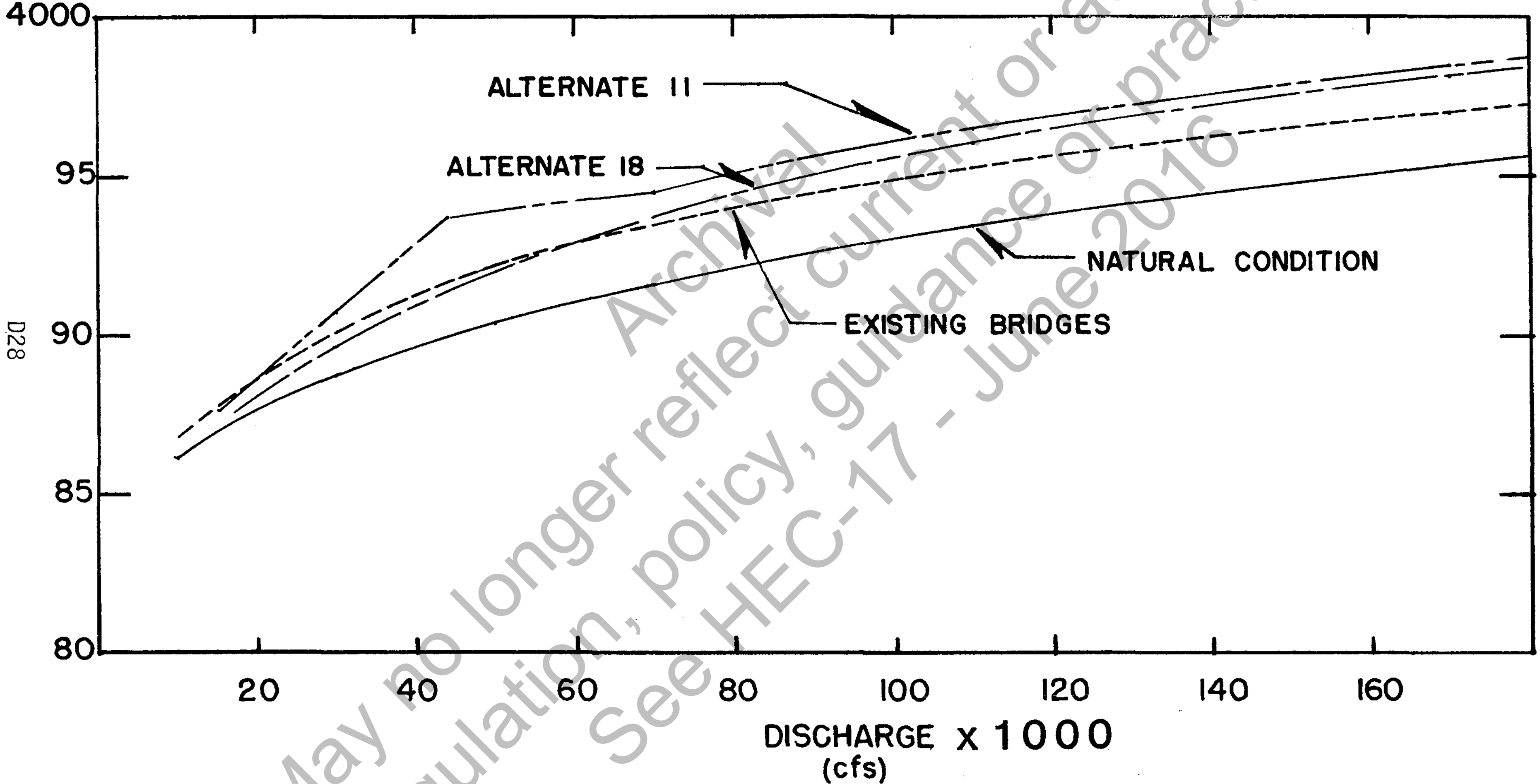

Figure 12. Stage vs. Discharge Curve, SECNO 8000

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VII. ALTERNATES CONSIDERED

Alternates ranging from no-build option to 1000 feet of bridge were considered. The no-build option (zero bridge length) was considered because of the availability of alternate routes. Bridges totaling 1000 feet were considered to replace the existing bridges with similar lengths.

This range would encompass all reasonable alternates and ensure that a low point on the total annual cost curve would be between them (see Figure 1).

Since this is a rather large range, engineering judgment and calculations were used to locate the lowest total annual cost alternate as rapidly as possible. With the exception of the no-build, the shortest bridge analyzed had a channel width at overbank elevation of 200 feet (bridge length 229 feet). This bridge would slightly constrict the main channel near the existing middle bridge (see Figure 5). Backwater damages were expected to be high, overtopping frequent and maintenance costly. The longest alternate considered was to replace the existing bridges in kind. In this case annual construction cost was expected to be high and, due to low clearance, the risk costs would also be high. The first cost estimate for bridge structure was done for this alternate and yielded an annual construction cost thought to be higher than that expected for the lowest total annual cost alternate. The high cost was due primarily to the necessity of using a steel structure to stay in keeping with the existing geometry rather than the more frequently used pre-stressed-concrete structure. The level grade across the flood plain would preclude raising the roadway grade and the lack of clearance below would eliminate using a deeper-than-existing structure (2.3 feet). When compared to concrete, twice the number of girders with one-half the span length would be needed. Both super and substructure costs would be high. As a result, the steel structure would cost 60% more per square foot.

Several design concepts were considered at this point. Three concrete structures could be constructed that would require a raised roadway profile grade. The grade could be raised in three ways: the first would require raising the grade across the entire flood plain, the second would require a vertical curve over each bridge, and the third would require one vertical curve over the north and middle bridges and another over the south bridge. All of these were undesirable. Raising the grade across the entire flood plain would raise the elevation of the overtopping flood; this would drastically increase damage for extreme events. Three vertical curves would not be significantly different than raising the entire grade when approach length is considered. All three concepts were deemed undesirable.

The third concept mentioned above, with modification, had some promise. It involved elimination of the south bridge over a channel with little conveyance and making the north and middle bridges into a single bridge opening with one vertical curve. This concept would provide sufficient conveyance beneath the bridge, would maintain clearance for debris passage, would ride comfortably, but not significantly increase overtopping damage.

Eight single bridge alternates were analyzed in detail with lengths from 229 feet to 854 feet. Bridge lengths between these were chosen to halve the difference between alternates that were expected to bring the least total annual cost alternate. This halving process continued until the sag curve was defined and the low point identified (see Figure 1).

Spur dikes were assumed for all alternates. Abutment scour was anticipated to be a severe problem for a single bridge constricting floodplain flows from over 3000 feet of width to less than 854 feet. The anticipated scour was based on experience gained in spring 1980 when substantial maintenance was required to keep existing abutments from failing.

VIII. ECONOMIC LOSS ANALYSIS

Two types of analyses were undertaken. The first was the conventional risk analysis as outlined in HEC-17 which compares backwater losses caused by bridge altern-tes to those caused by the natural condition (see Figure 1 for results). The second analysis compares backwater losses against the existing timber bridge situation. the purpose of this approach is to quantity a highway agency's risk if local residents bring suit for damages caused by a new drainage structure. Economic losses for all conditions will be discussed first.

Crop Losses.

The crop damage component of annual risk is based on percent crop damage due to duration of inundation (Figure 6) and the extent of land inundated. Duration of inundation is obtained from constructed hydrographs (Figure 11) and stage vs. discharge curve for SECNO 8000 (Figure 12). This river section was selected for backwater related damages since it is near the middle of damageable land and the slope of water surface will not cause measurable errors. The extent of land inundated was obtained from contours superposed on an aerial photo. Crop value of each contour interval was computed using $386 per acre (Section V). All bridge alternates, natural and existing conditions, were analyzed.

Building Losses.

A similar procedure was followed for building damage component of annual risk. Building damage is based on the percent damage to buildings and contents as a function of depth of inundation (Figure 7). The aerial photo mentioned above was used to identify buildings so that a value for each contour interval could be computed. Figure 12 yielded depth of inundation of each contour interval. In addition, alfalfa haymill products were damaged at a rate of $9750 per foot of inundation. Again, all alternates, natural and existing conditions were analyzed.

Embankment and Pavement Losses.

Embankment and pavement losses are a function of depth and duration of overtopping and roadway length exposed to damage. Embankment and pavement lengths were determined by subtracting bridge length from roadway length. Depth of overtopping was taken from the stage vs. discharge curve for SECNO 10001. Hydrograph width (Figure 11) was scaled at overtopping discharge to obtain duration of overtopping. This procedure yields conservative losses since maximum depth of overtopping would be instantaneous, not for the entire overtopping duration as computed.

Traffic Related Losses.

Traffic related losses were computed for all alternates and the no-build option. The no-build option was considered because the practicality of rebuilding a crossing with a terminus at the small town of Atwood seemed questionable. The traffic projection of 1900 vehicles per day in 1990 seemed incongruous with the physical situation. Study of traffic movements and conversations with local residents, maintenance and construction personnel gave clues to trip motivations for those persons using the crossing. Most of the traffic across the present bridge use it for access to the west side of Sterling from I-76 eastbound. The same destination can be achieved by using the Sterling/I-76 or Hillrose/I-76 interchanges (see Figure 4). At the present time, only 100 vehicles per day "must" use this crossing. An ADT of 150 vehicles per day was used in the analysis to account for traffic volume growth.

Traffic loss has three components: running costs, time costs and accident costs. Running and time costs were computed with an ADT of 150 vehicles per day. Accident costs were based on an ADT of 1900 vehicles per day. The rationale being the lack of bridge (either through washout or no-build option) would not change travel motivation, only the route. Detour duration is a factor in all three components and is a function of overtopping time. Maintenance is capable of restoring a crossing to service hours after overtopping waters ebb. Detour duration begins as overtopping occurs and ends one day after flood flows drop below 10000 cfs. The one day is administrative time accounting for the possibility that Maintenance would be busy repairing other flood-damaged bridges. Overtopping duration is measured from flood hydrographs (Figure 11).

The most likely detour route would take (northbound) travelers from the S.H. 63/I-76 interchange northeasterly on I-76 to the Sterling/U.S. 6 exit, through Sterling and back to the southwest on U.S. 6 to Atwood (see Figure 2). Southbound travelers would reverse this route. The net increases in travel distance and time are 15.2 miles and 23 minutes, respectively.

Running costs are a function of detour duration, ADT, detour distance and unit cost of vehicle operating time. Using the values in Table 3, running costs were computed to be $347 per day of detour duration.

Time costs are a function of detour duration, ADT, vehicle occupancy rate, unit cost of vehicle occupant time and the net time increase in driving the detour. With values from Table 3, time costs were computed to be $412 per day of detour duration.

Accident costs are a function of accident rates, ADT, costs per accident and detour duration. From values in Table 3, an accident cost of $107 per day of detour duration was computed.

The three components of traffic related losses were summed to yield $866 per day of detour duration. Detour duration is dependent on the shape of flood hydrograph and length of bridge alternate.

Scour Losses.

Scour losses are a function of scour volume as noted in Section V. Scour volume is dependent on depth and an assumed scour hole shape.

Scour depth was computed using Laursen's semi-empirical, analytically based long-contraction scour formula (Hopkins, et.al., 1980). This approach was taken to avoid problems encountered when curve-fitting type scour formulas were used with the widely varying flow conditions expected at the site.

Conventional Analysis.

Backwater losses (crop and building losses) for the natural conditions are subtracted from those for bridge alternates. The difference is the damage caused by the bridge and is the responsibility of the highway agency. Table 5 contains all components of economic loss and the resulting annual risk for Alternate 11.

Highway Agency Analysis.

This analysis utilizes the difference between a bridge alternate's backwater losses and those that would be caused by the existing bridges. The existing bridges were used as a reference since, due to their longevity, it has become the historic drainage condition. A conservative approach was taken that assumes each property damaged by a bridge alternate's backwater would sue, whether or not it would be damaged by the existing bridges. Therefore, a cost of litigation would be added to damage for each property flooded. According to the Office of the Attorney General, the cost of litigation would average $12500 per property. This amount covers expert witness and legal fees. Damages are accounted for as mentioned earlier.

The purpose of this analysis is to quantify a highway agency's risk, therefore, only the cost to the agency should be included in the annual risk. The applicable annual risk components are backwater damages (crop and building), embankment and pavement losses, and scour damage. Table 6 summarizes economic losses and the resulting annual risk for Alternate 11.

TABLE 5. Economic Losses and Annual Risk for Alternate 11 - Conventional Analysis*

Overtopping discharge = 34000 cfs
Overtopping flood return period = 30 years

Return Period (yrs)	Dis-charge (cfs)	Embankment & Pavement Loss ($)	Traffic Loss ($)	Backwater Loss ($)	Scour Loss ($)	Total Loss ($)	Average Loss ($)	Exceedance Probability	Delta Probability	Annual Risk ($)
5	9200	0	0	0	22	22		0.2		
							108		0.1	11
10	15500	0	0	150	44	194		0.1		
							9712		0.06	583
25	29000	0	0	18999	231	19230		0.04		
							124548		0.02	2491
50	44000	23906	2598	202230	1132	229866		0.02		
							326057		0.01	3261
100	68000	100225	3204	316264	2554	422247		0.01		
							463107		0.005	2316
200	104000	218214	3377	276963	5413	503967		0.005		
							556425		0.003	1669
500	168000	356319	3637	236417	12509	608882		0.002		
							556425		0.002	1113
									TOTAL	$11444

* based on analysis outlined in HEC-17

TABLE 6. Economic Losses and Annual Risk for Alternate 11 - Highway Agency Analysis (1)

Overtopping discharge = 34000 cfs
Overtopping flood return period = 30 years

Return Period (yrs)	Dis-charge (cfs)	Embankment & Pavement Loss ($)	Backwater (2) Loss ($)	Scour Loss ($)	Total Loss ($)	Average Loss ($)	Exceedance Probability	Delta Probability	Annual Risk ($)
5	9200	0	0	22	22		0.2		
						33		0.1	3
10	15500	0	0	44	44		0.1		
						138		0.06	8
25	29000	0	0	231	231		0.04		
						80685		0.02	1614
50	44000	23906	136100	1132	161138		0.02		
						200084		0.01	2001
100	68000	100225	136251	2554	239030		0.01		
						331750		0.005	1659
200	104000	218214	200843	5413	424470		0.005		
						497224		0.003	1492
500	168000	356319	201149	12509	569977		0.002		
						497224		0.002	993
								TOTAL	$7770

(1) as outlined in Section VIII
(2) includes damages and cost of litigation

IX. ANNUAL CONSTRUCTION COSTS

Construction cost estimates included cost of the structure, spur dikes, revetment, roadway embankment, embankment removal and pavement. All of these items are dependent on bridge length and have different values for each bridge alternate. Note: These costs remain the same for either the conventional or highway agency analyses mentioned in Section VIII.

Spur dike length was selected as recommended in "Hydraulics of Bridge Waterways," (1978). Revetment for spur dikes and bridge abutments was sized by tractive force theory. Construction of a single bridge would require filling in the existing north and south bridge sites (see Figure 5). Some alternates were long enough to require removal of existing embankment.

The discount rate of 4 percent and useful life of 30 years noted in Section IV were used to compute annual construction cost. Total and annual construction costs for each bridge alternate analyzed is given in Table 7.

TABLE 7. Construction Cost Summary

Bridge Length (ft)	Alternate	Total Construction Cost* ($)	Annual Construction Cost ($/yr)
229	13	607775	35148
279	12	683870	39548
319	16	732826	42379
354	11	778003	44992
429	15	884639	51159
479	18	963941	55745
604	14	1144389	66180
854	10	1473714	85225

* including 10% for engineering and contingencies

X. SENSITIVITY ANALYSIS

The following components were tested in the Sensitivity Analysis; capital cost, discount rate, useful life, building damage, embankment and pavement losses, scour loss and traffic related losses. Table 8 gives a ranking of components from most to least sensitive and response index. The most sensitive component was assigned a response index of 100. Other relative responses were adjusted accordingly.

A few comments on the range of variation considered for each component are in order. Capital costs were varied to the limit of accuracy ($\pm$15%) recommended by Staff Bridge Branch. Discount rate and useful life were varied as noted in Section IV. Building damage was varied to reflect the maximum error expected in appraisal and/or backwater damage. Embankment and pavement losses were assumed to be at most 50% in error.

Scour damage was varied by a factor of ten on the possibility that contraction and bed form movement had not been fully analyzed. Traffic losses were increased by a factor of 12.66, the ratio of 1990 ADT to ADT of persons who "must" use the crossing as noted in Section VIII ($1900 \div 150 = 12.66$).

TABLE 8. Sensitivity Analysis Results

Component	Original Value	New Value or Percent Change	Response Index
Useful Life	30 yrs	20 yrs	100
Capital Cost	$\$30/ft^2$	± 15%	78
Discount Rate	4%	12%	70
Discount Rate	4%	7-1/8%	64
Embankment and Pavement Loss	*	± 50%	62
Discount Rate	4%	2%	56
Useful Life	30 yrs	40 yrs	46
Building Damage	*	± 50%	22
Scour Damage	*	+1000%	0.3
Traffic Loss	*	+1266%	0.2

* See Table 4

XI. RESULTS

Results are summarized on Table 9. The traveling public would be exposed to the same inconvenience and perils whether in the detour or no-build situation. Using the factor cited in Section VIII, $866 per day of detour duration, and a duration of one year, the no-build option's yearly cost would be $316090. No other expenses, with the possible exception of bridge removal, are applicable. Comparing $316090 at bridge length zero on Figure 1 to the Total Annual Cost curve, the no-build option would be 5.6 times more expensive than the least expensive alternate at 354 feet (Alternate 11). Obviously, a bridge is warranted. The total annual cost of Alternate 11 is $56436. Replacement of the existing bridges with a similar length (1000 feet) would cost $40000 per year more.

Examination of Table 9 raises an interesting question. The total annual cost to the highway agency (CDOH) for the no-build option is zero; all costs are borne by the traveling public. This is the weakness in considering the costs for the highway agency - there would be no motivation to build a bridge if these results were taken literally. Given that a bridge is to be built, these results may be used to assess the agency's legal exposure. For this crossing, the highway agency approach did not change the lowest total annual cost alternate. In fact, the magnitude of annual costs changed very little. This is due to the cost of litigation being offset by decreased damages as a result of change in reference conditions and elimination of traffic related losses.

CDOH chose a bridge length of 479 feet (Alternate 18). This choice was based on preserving the existing backwater condition while taking advantage of greater hydraulic efficiency of a single long span structure with spur dikes.

Economic risk analysis enables us to quantify the redistribution of expenses resulting from this choice. In Table 9 it can be seen that the total annual cost increase between bridge lengths 354 feet and 479 feet is $5275. This is accounted for by an increase in annual construction cost of $10753 (Table 7) and a decrease in annual risk cost of $5478 (Table 9). CDOH will bear the increased construction cost. The decreases will benefit both CDOH and the public. Annual risk costs for embankment and pavement losses are reduced by $1585, benefitting CDOH. Annual risk costs for traffic and backwater losses will be reduced by $3893 and will benefit the public. Consequently, CDOH expenses will be increased by $9168 in order to save the public $3893.

TABLE 9. Economic Risk Analyses Results

Bridge Length (ft)	Alternate	Annual Risk Conventional(1)	$ Highway Agency(2)	Annual Total Conventional(3)	Cost ($) Highway (CDOH) Agency(4)
0	No Build	316090	0	316090	0
229	13	31721	28625	66869	63773
279	12	18185	15750	57733	55298
319	16	14907	12545	57286	54924
354	11	11444	7770	56436	52762
429	15	8559	5491	58718	56650
479	18	5966	2894	61711	58639
604	14	4658	1957	70838	68137
854	10	3256	520	88481	85745
1000	In Kind	--	--	100000(5)	--

(1) based on analysis outlined in HEC-17
(2) highway agency risk analysis as outlined in Section VIII
(3) (1) plus appropriate value from Table 7
(4) (2) plus appropriate value from Table 7
(5) extrapolated from Figure 1

XII. CONCLUSIONS

The bridge selected for final design will save approximately $40000 per year over an in-kind replacement. Although the lowest total annual cost alternate was not chosen, the logical design approach and thorough investigation convinced CDOH that a short, economical bridge is a reasonable choice for the South Platte River.

The highway agency risk approach would be more informative at a site that has greater traffic losses. This approach is designed to answer the question, at what bridge length is the highway agency's legal exposure minimized? The highway agency would be able to decide on the financial risk it is willing to take based on documentation rather than conjecture. This approach should be pursued in subsequent analyses.

Sensitivity analysis results in Table 8 are indigenous to this crossing. The most sensitive parameters were thoroughly researched to retain confidence in results. A few general comments on sensitivity analysis are in order. Useful life of various structures may be obtained from highway agency records; it should not be arbitrarily chosen. A highway agency's economist should be charged with selection of discount rate based on the agency's experience.

As mentioned in the introduction, imagination is a major part of risk analysis. The great amount of data collected and results computed make the designer intimately familiar with the project. He would be remiss if he did not try different combinations of the data to answer the inevitable questions asked by the principal engineers of his agency.

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