SELECTING APPROPRIATE FLOOD DESIGN FREQUENCIES FOR DRAINAGE BASINS IN WYOMING

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1. INTRODUCTION

1.1 Purpose and Tasks

The main thrust of this study is to develop a scientifically and technically sound procedure for determining the appropriate flood design frequency for drainage basins in Wyoming. The results of the study would provide a mechanism for selecting a design frequency for various roadway crossing structures such as culverts and bridges in Wyoming which can serve to complement the current Wyoming Highway Department (WHD) Design and Operating Policy.

Tasks performed to this point in time to accomplish the purpose of the study primarily involved (1) identification, definition, and collection of social, economical, and physical parameters and variables relevant to the design of highway drainage structures; (2) determination of a flood frequency associated with the least total expected cost (LTEC) design; (3) determination of an extended LTEC frequency by considering, in addition to the economic aspect, other intangible factors that affect the selection of design frequency; (4) development of a mechanism to relate LTEC design frequency and/or extended LTEC design frequency to economic, social, hydrologic/hydraulic characteristics of drainage basins which are typical to Wyoming. Types of highway drainage structures considered are box and pipe culverts and bridges.

It is important to note the fact that there have been several modifications and additions to the original proposal as the study evolved. The goal of the original proposed study was to develop some type(s) of empirical regression equations that relate LTEC to relevant economic, social, and drainage basin characteristics. Currently, it is felt that the scope can be extended to bring a new technology to bear on the development of an expert system or decision-support system to aid in the selection of a flood design frequency through LTEC. This would be the approach followed when this research project is continued.

The reasoning behind this change in scope was the fact that determination of an appropriate design frequency for highway drainage structures is an important element in the overall decision-making process and both tangible and intangible factors are important to the selection of the final design frequency in order to make a balanced decision between technical quantities (tangibles) and such items as environmental impacts, public convenience and legal liability of the state highway department (intangibles) in the final determination of the flood frequency to be used.

1.2 Scope of Report

This interim final report summarizes the work performed thus far for the cooperative research project entitled, "Selecting Appropriate Flood Design Frequencies for Drainage Basins in Wyoming" between the Wyoming Highway Department (WHD) and the Wyoming Water Research Center (WWRC). The project was terminated by the WHD on February 27, 1988. Therefore, this report contains the research work performed up to the termination date.

This report includes the following:

1. Identification and definition of relevant variables and parameters in the least total expected cost (LTEC) design of highway drainage structures.
2. Development of cost functions for pipe culverts, box culverts, and bridges.

3. Collection of relevant information on basin and channel characteristics representative of Wyoming drainages.
4. Collection of relevant information on economic variables in assessing flood damages.
5. Factors and mechanisms for determining the extended LTEC design frequency.
6. Identifying design and operational performance standards of Policy 18-6.

2. IDENTIFICATION AND DEFINITION OF RELEVANT VARIABLES AND PARAMETERS IN THE LTEC DESIGN OF HIGHWAY DRAINAGE STRUCTURES

Since the LTEC design involves the evaluation of the first cost (i.e., installation cost) and the second cost (i.e., flood related damage costs), the relevant variables and parameters associated with the first cost and the second cost were identified and are listed in Tables 2.1 and 2.2, respectively. Variables in this study were defined as those inputs which would vary from one site to another, while parameters were defined as constants which do not vary and are universally usable by all drainage structure sites. It should be noted that these lists of variables and parameters in Tables 2.1 and 2.2 are the modified version of a more extensive list extracted from published literature [1,2,3]. However, knowing that the LTEC design was to be performed during the early stage of planning, detailed information required for actual design is not always known. Therefore, the list shown in Tables 2.1 and 2.2 was chosen on the basis of availability of information to a designer at the time when the LTEC design frequency is to be determined.

Table 2.1. Variables and Parameters Relevant in Evaluating the First Cost of Roadway Crossings.

	Pipe Culverts	Box Culverts	Bridges
Parameters	Unit cost of culvert	Unit costs of concrete and steel	Unit cost of bridge
Variables	-Size of culvert -Length of culvert -Type of culvert	-Number of barrels -Length of barrel -Width per barrel -Quantity of concrete per unit length -Quantity of steel per unit length	-Length of bridge

Table 2.2 Damage Categories with Related Economic Variables and Site Characteristics

Damage Category	Economic Variables and Parameters	Site Characteristics
(1) Flood plain Property Damage - Losses to crop - Losses to buildings	- Types of crops - Economic values of crops - Types of buildings - Economic values of buildings	- Locations of crop fields - Locations of buildings - Physical layout of drainage structures - Roadway geometry - Flood characteristics - Stream valley cross-section - Slope of channel profile - Channel and flood plain roughness characteristics
(2) Damage to Pavement and Embankment - Pavement Damage - Embankment Damage	- Material cost for pavement - Material cost for embankment - Equipment costs - Man-hour costs - Repair rate for pavement and embankment	- Flood magnitude - Flood hydrograph - Overtopping duration - Depth of overtopping - Total area of pavement - Total volume of embankment - Types of drainage structure and layout - Roadway geometry - Soil characteristics of embankment
(3) Traffic-Related Losses - Increased travel cost due to the detour - Lost time of vehicle occupant - Increased risk of accidents on the detour - Increased risk of accidents on a flooded highway	- Rate of repair - Operational cost of vehicle - Distribution of income for vehicle occupant - Cost of vehicle accident - Rate of accident	- Average daily traffic volume - Composition of vehicle types - Length of normal and detour paths - Flood hydrograph - Duration and depth of over-topping - Duration of repair - Expected detour length and vehical speed during repair

3. DEVELOPMENT OF COST FUNCTIONS FOR PIPE CULVERTS, BOX CULVERTS, AND BRIDGES

3.1 Pipe Culverts

After several revisions, cost functions for pipe culverts of various materials and types were developed as shown in Table 3.1. Originally, cost functions were developed for pipe culverts for different highway systems. However, it was later found, based on the WHD's suggestion, that the differences in mean unit pipe culvert costs among different highway systems were insignificant. Therefore, pipe culverts of a given material in various highway systems were lumped into one category in developing the cost functions. It should be pointed out that pipe culverts of a given material with flared ends and without flared ends were considered separately because of the incompatibility of the cost units; the former has the unit of $/each pipe and the latter $/linear foot. Table 3.1 contains the cost functions and related statistics for pipe culverts of different materials and types based on data extracted from Wyoming Construction Weighted Average Bid price forms [4] between 1965 and 1985. Adjustment of costs in different years was made according to the Wyoming Construction Cost Index [5] shown in Table 3.2 with 1977 used as the base year.

Statistics such as standard errors associated with each cost function provide important information on the degree of uncertainty associated with the cost function. They can be used as the basis to evaluate the sensitivity of uncertainty in installation cost on the LTEC design frequency.

This set of cost functions in Table 3.1 have been built into the software for the LTEC analysis. Users have to specify the type of

Table 3.1. Revised pipe culvert cost functions for interstate - primary, secondary, urban and SC-CFM highway.

Case No.	Type of Pipe	Unit	Equation	Data Limitation	R	Standard Error	Sample Size n					
							I*	P*	U*	S*	CFM*	Total
1	Pipe	$/LF; Inches	ln(C) is the $1.51 + 0.0142D + 0.0759[\ln(D)]^2$	6" to 120"	0.932	0.2355	54	110	88	20	2	272
2	RCP (Instld)	$/LF; Inches	C is the $11.6 + 0.0212D^2$	12" to 96"	0.919	14.51	31	28	13	29	7	108
3	CMP (Instld)	$/LF; Inches	C is the $13.2 + 0.0102D^2$	6" to 96"	0.857	9.763	46	53	58	19	78	253
4	CMP Arch	$/LF; Inches	C is the $0.41 + 0.00378D^2 + 0.521D$	18"$_x$ to 142"$_x$	0.941	10.94	16	18	19	13	21	87
5	Pipe (Instld)	$LF; Inches	C is the $10.9 + 0.00898D^2$	6" to 72"	0.907	4.551	14	27	23	6	15	85
6	CMP Arch	$/LF; Inches	C is the $15.3 + 0.00732D^2$	18"$_x$ to 142"$_x$	0.877	20.36	24	40	43	2	18	124
7	RCP FE	$/EA; Inches	ln(C) is the $3.32 + 0.187[\ln(D)]^2$	12" to 90"	0.925	0.2639	31	61	36	15	6	112
8	CMP FE	$/EA; Inches	C is the $639 + 25.3D - 359[\ln(D)]^2$	12" to 84"	0.942	75.42	23	36	37	5	12	112
9	Piep FE	$EA; Inches	ln(c) is the $1.73 + 0.282[\ln(D)]$	12" to 78"	0.948	0.2827	32	80	57	19	1	188
10	Pipe FE (Instld)	$/EA; Inches	C is the $22.6 + 0.171D^2$	12" to 66"	0.955	47.31	6	19	12	4	7	48
11	CMP Arch FE (Instld)	$/EA; Inches	ln(C) is the $3.41 + 0.0409D$	18"$_x$ to 72"$_x$	0.954	0.2278	6	4	11	9	2	32
12	Relaying pipe	$/LF; Inches	C is the $7.82 + 0.00524D^2$	5" to 90"	0.833	6.214	7	12	11	6	15	51
13	CMP	$/LF; Inches	ln(C) is the $1.28 + 0.150[\ln(D)]^2$	6" to 120"	0.922	0.2699	96	118	126	16	43	394
14	RCP arch	$/LF; Inches	ln(C) is the $1.06 + 0.185[\ln(D)]^2$	22"$_x$ to 88"$_x$	0.938	0.1825	55	57	17	12	8	148

Table 3.1. (cont.)

Case No.	Type of Pipe	Unit	Equation	Data Limitation	R	Standard Error	Sample Size n I*	P*	U*	S*	CFM*	Total
15	Pipe arch FE	$/AE; Inches	C is the $302 + 0.329D^2 - 17.0D$	17"$_x$ to 81"$_x$	0.897	106.4	9	30	17	4	-	60
16	RCP	$/LF; Inches	C is the $7.55 + 0.0201D^2$	12" to 132"	0.966	11.12	118	154	104	32	29	429
17	Pipe FE (corrosion resist.)	$/EA; Inches	C is the $-32.9 + 0.237D^2$	18" to 66"	0.946	89.84	8	36	13	8	-	65
18	CSP Arch	$/LF; Inches	C is the $6.97 + 0.0138D^2$	18"$_x$ to 72"$_x$	0.940	6.846	4	6	6	-	13	29
19	Pipe (corr. resist.)	$/LF; Inches	ln(C) is the $1.94 + 0.0363D$	18" to 78"	0.898	0.2701	16	45	17	8	4	86
20	CMP Arch FE	$EA; Inches	ln(c) is the $3.44 + 0.0422D$	22"$_x$ to 72"$_x$	0.957	0.1819	7	3	12	-	2	24
21	CSP	$/LF; Inches	ln(C) is the $1.27 + 0.154[\ln(D)]^2$	6" to 90"	0.919	0.2883	24	14	10	-	21	69
22	RCP Arch FE	$/EA; Inches	ln(C) is the $3.47 + 0.164[\ln(D)]^2$	29"$_x$ to 88"$_x$	0.939	0.1620	18	8	6	-	2	34
23	RCP Arch (Instld)	$/LF; Inches	C is the $-16.7 + 1.31D$	22"$_x$ to 88"$_x$	0.925	8.639	-	7	4	8	-	19

*I - Interstate; P - Primary; U - Urban; S - Secondary; CFM - County/Farm

Table 3.2. Annual Construction Index.

YEAR	INDEX (1977=100%)
1964	34.769
1965	45.471
1966	44.671
1967	44.405
1968	46.270
1969	50.666
1970	52.265
1971	56.883
1972	55.773
1973	62.167
1974	90.187
1975	95.160
1976	99.512
1977	100.000
1978	127.500
1979	148.700
1980	158.100
1981	153.000
1982	139.800
1983	144.300
1984	146.400
1985	151.900

culvert and the material to be used. In addition, the designer needs to specify the physical layout of roadway crossing, such as roadway system (interstate, primary, etc.), number of traffic lanes, length of roadway crossing, embankment height, etc. Other physical variables such as width of each traffic lane and side slopes of embankment were "standardized" using the specifications given by the current highway design manuals. Therefore, given the physical layout of a roadway crossing and hydraulic characteristics of the channel, the culvert size which conveys a flood magnitude of a certain frequency can be calculated by some fairly simple hydraulic equations or by a more elaborate computer model such as CDS. Once the pipe diameter is determined, the cost of the pipe culvert can be computed by the appropriate cost function in Table 3.1. The unit cost of the embankment is available from reference [4]. This unit cost will be multiplied by the total volume of embankment to obtain the total cost of the embankment. The volume of embankment can be easily estimated if the physical layout of the roadway crossing is specified (Figures 3.1a and 3.1b).

The column "Data Limitation" in Table 3.1 shows the lower and upper bounds of pipe size used in this study. It is believed that the developed cost functions can be extrapolated beyond the present data range to larger culvert sizes without seriously damaging the validity of the functions. However, the data set represents the culvert sizes that have been used for highway crossings in Wyoming over the past 20 years. Of course, the use of larger pipe sizes in the future is very probable.

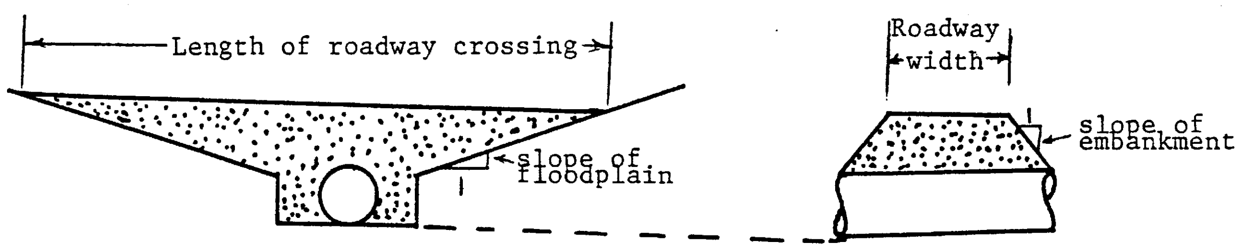


Figure 3.la. Schematic Diagram of Roadway Crossing with Culvert.

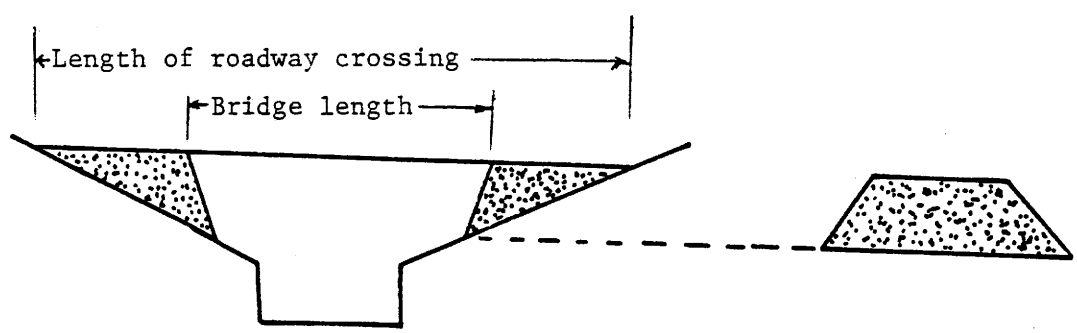


Figure 3.lb. Schematic Diagram of Roadway Crossing with Bridge.

3.2 Box Culverts

Installation cost of box culverts primarily involves cost of culvert and cost of embankment. The total cost of a box culvert can be estimated as

$$C_{box} = NL(U_c Q_c + U_s Q_s) \tag{3.1}$$

where C_{box} is the cost of a box culvert (\$), N is the number of barrels, L is the length of box culvert (ft.), U_c is the unit cost of concrete (\$/yd^3), Q_c is the quantity of concrete per unit length (yd^3/ft), U_s is the unit cost of reinforced steel (\$/lb), and Q_s is the quantity of reinforced steel per unit length (lb/ft). Variables Q_c and Q_s in Eq. (3.1) may potentially depend on the physical characteristics of the box culvert such as the width of barrel, height of culvert, and fill height. Fill height was found to have very little effect on the estimating equations for Q_c and Q_s and, consequently, was dropped from the equations. The following two equations were developed for estimating Q_c and Q_s.

$$Q_c = -0.563 + 0.0764 B + 0.189 H \tag{3.2}$$

with a correlation coefficient of 0.89, and

$$Q_s = -5.87 + 2.55 BH \tag{2.3}$$

with a correlation coefficient of 0.95, in which B is the width of barrel (ft.) and H is the height of culvert (ft.). The total cost for the roadway crossing can be estimated by adding the cost of embankment to the cost of the box culvert.

To estimate the cost associated with a box culvert having a capacity to accommodate the flood magnitude of a specified return period, variables such as N, L, B, H, roadway layout, and channel characteristics must be specified by the designer. Unit costs of concrete and steel have been built into the software as a default unless specified otherwise by the designer. However, it appears that in designing a box culvert, there are many possible combinations of N, W, and H which would yield essentially the same conveyance capacity. That is, the problem then becomes an optimization problem where the optimal N, W, and H leading to the minimum installation cost would be sought. Naturally, in the course of optimizing N, W and H, design specifications of the WHD on roadway geometry must be followed. So far, no mechanism (computation routine) has been developed to find the optimal N, W, and H.

3.3 Bridges

To develop the first cost function for bridges, the dependent variable considered was the unit cost of a bridge per square foot of bridge deck. Independent variables that were considered potentially relevant are: length and type of bridge, type of substructure, clearance height of bridge, soil condition, and miscellaneous (i.e., wire enclosed riprap, removal of old bridge, reinforced concrete approach slabs). However, due to the fact that, when the LTEC design is performed by the WHD, the details of the bridge design are normally unknown except for the length and width of the bridge. Therefore, it is unrealistic to develop a bridge cost function involving any design variables which are unknown or susceptible to changes in the latter stage of design.

A preliminary investigation was performed to examine the relationship between unit cost of a bridge per square foot of bridge deck and relevant variables. The result of the correlation study based on 85 data points provided by the WHD was that the length of a bridge has a more important role than other variables in explaining the variation of unit cost of a bridge. However, the correlation coefficient is only around 50 percent. Later, more data were requested from and provided by the WHD with the intent of trying to improve the original cost function. Currently, a total of 238 data points are available for analysis and were used. The independent variables which were considered initially were again used (length, width, and the surface area of bridge). With all 238 points included in a stepwise regression analysis, it was again found that the bridge length was still the most important variable. The resulting regression equation is

$$U_{brdg} = 159 - 34.6 \ln L + 0.323 L - 0.000316 L^2 \tag{3.4}$$

with a correlation coefficient of 58 percent and standard error of $5.885/ft^2. The unit bridge cost versus bridge length for all 238 data points are shown in Figure 3.2. As can be seen, there exists a tremendous scatterness in the data set. A series of trials were made by gradually deleting some extraordinary data points (outliers) based on the magnitude of a standardized residual in attempting to find the model structure that describes the data behavior. This is analogous to procedures that filter out noises of incoming radio signals. After sequentially deleting 96 outliers, the final model had exactly the same structure as Eq. (3.4) with a correlation coefficient of 0.914 and standard error of 1.885 \$/ft^2.

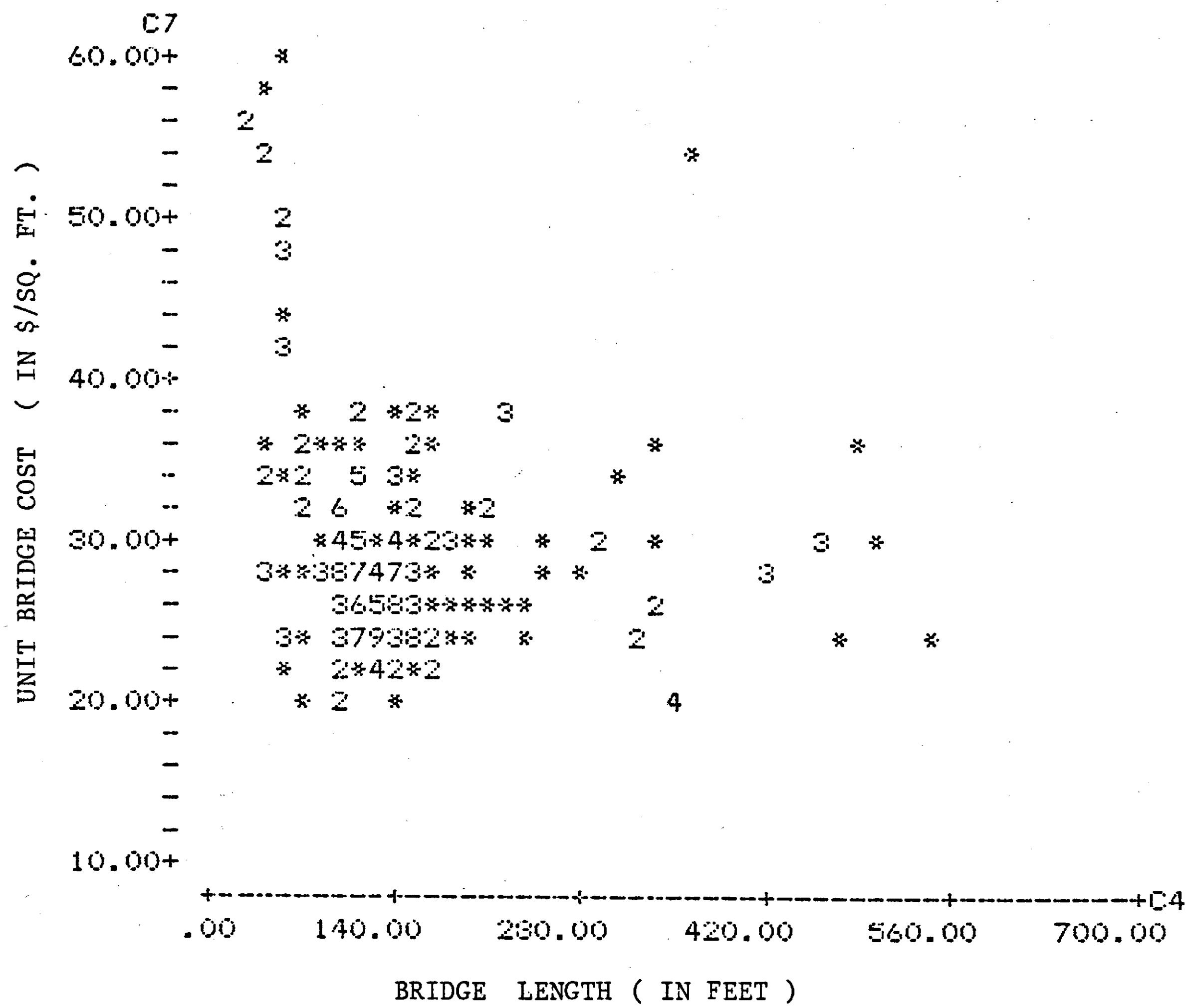


Figure 3.2. Scatter Plot of Unit Bridge Cost Versus Bridge Length.

Deletion of 96 data points represents a 40 percent reduction in the total number of data points. Although the regression equation associated with the reduced data set has a much higher correlation coefficient and much less standard error, it can only be regarded as artificial and does not truly represent the behavior of the total data set. Furthermore, there are many important characteristics of bridges such as bridge type, geologic condition and substructure type that cannot be specified when the LTEC design is performed. It is felt that the model derived on the basis of the total data set is more representative of the realistic design situation. The high value of standard error is primarily caused by the fact that many of the important characteristics associated with bridge costs are unknown at the time the LTEC design is to be performed. The total data set lumps bridges of many different types and their individual characteristics. It perhaps would be desirable to develop cost functions for bridges of different characteristics as was done for pipe culverts. However, bridges involve many variables that are not known beforehand.

Examining Figure 3.2, it appears that unit bridge cost per square foot has two distinct behaviors. That is, for bridges, with length shorter than some threshold value, the unit bridge cost decreases sharply as bridge length increases; when bridge length is longer than this threshold value, the unit bridge cost increases only slightly with bridge length. A piece-wise linear cost function was developed using a pattern-search technique called the Hooke-Jeeve method [6] to minimize the standard error of estimates. The resulting equation is

$$U_{bridge} = \begin{cases} 65 - 0.4813\ L, & L \leq 83' \\ 23.75 + 0.01875\ L, & L > 83' \end{cases} \tag{3.5}$$

with a standard error of 5.855 \$/ft which is a little bit smaller than that of Eq. (3.4).

Since highly accurate cost functions for bridges cannot be derived, the function developed can still be used pending acceptable findings using a sensitivity analysis during one of the tasks not completed. That is, sensitivity of bridge cost on the LTEC design frequency must be examined. This can easily be accommodated in the software development when a bridge structure is considered. After all, design frequency is what is being sought and not the annual cost. It is intuitively conceivable, but yet to be proved, that the LTEC design frequency should be less variable than that of the cost. The total cost of a roadway crossing using a bridge can be calculated by adding the cost of the embankment and the cost of the bridge structure (Figure 3.1b).

If the sensitivity analysis finds that the resulting variability is too great for acceptable design practice, a closer look at other variables associated with the original data set (outliers) may be necessary. This should be considered as a part of one of the final tasks of the project if funding is later continued.

4. COLLECTION OF RELEVANT INFORMATION ON BASIN AND CHANNEL CHARACTERISTICS

LTEC design of highway drainage structures requires integrated analysis of hydraulics and hydrology. Hydrologic analysis provides estimations of flood magnitude of various frequencies which serve as part of the input in the LTEC design. In this study, flood magnitude of different return periods in the LTEC design will primarily be estimated by regional regression models to be developed by the USGS. According to three previous analyses [7,8,9] performed by the USGS for regional flood frequency relations in Wyoming, the independent variables used are drainage area and maximum basin relief or channel characteristics. The drainage area is the main independent variable used in the four hydrologic regions, as classified by the USGS, for the State of Wyoming.

Evaluation of annual expected flood damage for a proposed roadway crossing requires knowing the backwater profile upstream of the crossing site when subject to a flood of a certain return period. This, in turn, depends on a number of hydraulic characteristics of the stream and roadway geometry. Geometry of channel cross-sections at about 250 actual pipe/box culverts and bridge sites were extracted from "Plan and Profile of Proposed State Highways" [10] provided by the WHD.

Elementary hydraulic parameters considered include (1) top width of main channel, (2) top width/average depth ratio, (3) average slope of channel bottom, and (4) slopes of floodplain (left and right) transverse to the flow direction. The original database extracted from [10] may not be representative enough to cover the wide range of channel characteristics which may be encountered by future highway projects. As a result, the data were expanded to consider many additional sites from the various sources of published literature which contained hydraulic information for streams in Wyoming [7,8,9]. Any missing data which are not directly provided for in the literature, particularly the slope(s) of the floodplain on both sides of the main channel perpendicular to the flow direction, were filled by measuring them on USGS 7½-minute topographic maps.

Summary of basin/channel characteristics considered as typical to Wyoming are shown in Table 4.1, including all the sources from which the data were obtained. Based on the data set, it was found that the large majority of streams can be classified as wide open channels which indicates the top width/depth ratio exceeds 10. Knowing that wide open channels hydraulically behave like rectangular channels [11,12], it was determined for the purpose of simplifying the task of describing actual channel geometry that an idealized channel cross-section as shown in Figure 4.1 be used to perform hydraulic calculations.

It should be pointed out that the actual sites for bridges, box culverts, and pipe culverts available from "Plan and Profile of Proposed State Highway" [10] was less than the sample size shown in Table 4.1. This is because sometimes there might be two or three cross-sectional profiles in the neighborhood of a given actual site. Therefore, each cross-sectional profile in the neighborhood of an actual site was measured regarding its channel characteristics and treated as an individual data point. The assumption made here was that a highway drainage structure (bridge, box culvert, or pipe culvert) could possibly have been located at one of the neighborhood sites instead of the actual site and the sites would have similar physiographic basin characteristics. Otherwise, a need for more information on basin/channel characteristics from the Wyoming Highway Department would have had to have been obtained.

To have a reliable assessment of floodplain slopes using the plan and profile sheets of the Wyoming Highway Department, the floodplain

Table 4.1 Revised simple statistics for hydraulic characteristics in Wyoming.

Variable	Mean	Standard Deviation	Maximum Value	Minimum Value	at 75% Value	at 25% Value	Sample Size	Sources
Width of main channel	92.4	75.0	338.6	9.2	113.6	42.4	51	Green River Basin (3)
	36.7	30.0	180.0	2.0	48.7	15.0	152	Wyoming (5)
	124.0	100.4	566.0	20.0	146.2	59.5	102	Bridgy (7)
	51.2	35.1	150.0	8.0	60.0	28.0	60	Box culvert (7)
	53.1	53.4	290.0	6.0	66.0	19.0	92	Pipe culvert (7)
Ratio of top width to depth (ft/ft)	30.1	15.6	88.2	5.4	35.3	21.0	51	Green River Basin (3)
	10.05	7.30	53.75	1.50	13.72	4.92	152	Wyoming (5)
	15.8	10.8	66.7	3.20	22.2	7.7	102	Bridge (7)
	22.1	28.6	132.0	2.0	20.8	7.4	60	Box Culvert (7)
	30.3	30.7	175.0	5.2	38.2	12.7	92	Pipe culvert (7)
Main channel slope (ft/mi)	47.7	55.0	213.0	4.3	59.3	9.1	51	Green River Basin (3)
	139.5	103.0	543.0	8.0	197.7	66.8	152	Wyoming (5)
	31.8	34.6	142.0	2.4	33.2	10.6	100	Bridge (7)
	23.9	11.2	52.8	16.4	21.6	16.9	23	Box culvert (7)
	250	312	1584	4	299	79.0	94	Pipe culvert (7)
	605	191	929	240	733	436	22	USGS (4)
Maximum relief Rmax (ft)	403	175	752	5	545	317	22	USGS (4)
Drainage area (sq mi)	834	1887	9740	6	500	53	51	Green River Basin (3)
	222	541	5270	1	173	10	152	Wyoming (5)
	445	1008	7492	1	381	34	97	Bridge (7)
	33	105	464	0.270	5	1	50	Box culvert (7)
	0.181	0.334	2.0	0.00156	0.234	0.0172	94	Pipe culvert (7)
	3.36	2.86	10.80	0.69	5.20	1.37	22	USGS (4)
Floodplain slope Right side (ft/ft)	0.075	0.141	1.00	0.001	0.071	0.009	95	Bridge (7) (6)
	0.111	0.120	0.50	0.002	0.167	0.033	86	Box culvert (6)
	0.0685	0.0502	0.2500	0.0008	0.0909	0.0333	107	Pipe culvert (7)
Floodplain slope Left side (ft/ft)	0.0645	0.0972	0.5	0.001	0.0679	0.0096	94	Bridge (7) (6)
	0.097	0.118	0.5	0.005	0.111	0.027	86	Box culvert (6)
	0.0653	0.0441	0.250	0.0011	0.0833	0.0315	108	Pipe culvert (7)(6)

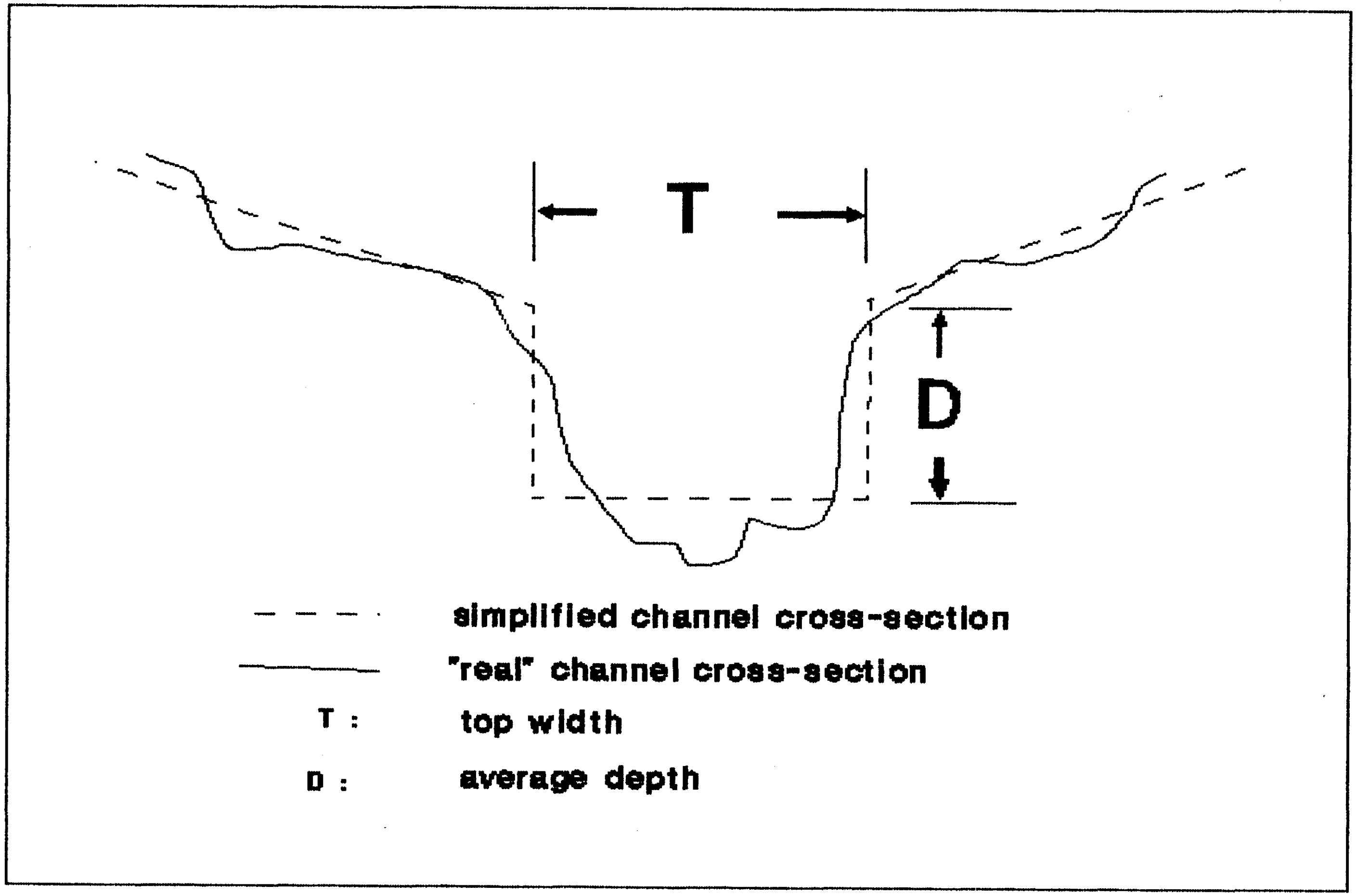


Figure 4.1. Idealization of Channel Cross-Section.

slope was measured if the extent of the floodplain on both sides of the channel exceeded at least twice the top width of the channel. With this criterion, floodplain slopes transverse to flow direction were only measurable at 37 bridge sites [10] and none of the box culvert sites. As an alternative, floodplain slopes presented in the last two rows of Table 4.1 were obtained by measuring floodplain slopes from USGS 7½-minute topographic maps. The intention was to make a total sample size of floodplain slopes of approximately 90 for each structure type. Table 4.2 indicates the number of floodplain slopes that are directly measured from [10] and indirectly synthesized from USGS topographic maps for different structures. The purpose of using synthesized data for floodplain slopes was to avoid requesting the Wyoming Highway Department staff to retrieve data from their microfilm files.

To expand the representativeness of floodplain slopes associated with different structure types in Wyoming, drainage basins of various sizes all over Wyoming were selected and their floodplain slopes measured. All the sites selected in the synthesis process were considered as potential sites for future roadway crossings. However, the question in the synthesis procedure is: "For a selected basin, what type of structure is to be used for the roadway crossing."

To answer the question, the step taken was to examine the structure types and the corresponding basin/site characteristics based on available data. The most easily available basic characteristic associated with structure sites was the drainage basin area. Based on the information available from [10], histograms (Figure 4.2) were constructed showing the distribution of the number of structures of a given type versus drainage area. It appears, from Figure 4.2, that the type of

Table 4.2. Number of data for floodplain slope.

# of Points of Data						Source of Data
Bridge		Box Culvert		Pipe Culvert		
R	L	R	L	R	L	
38	37	0	0	87	88	WHD
57	57	86	86	20	20	Topographic Map Wyoming
95	94	86	86	107	108	Total

Note: R - # of points of data on right side of river
L - # of points of data on left side of river

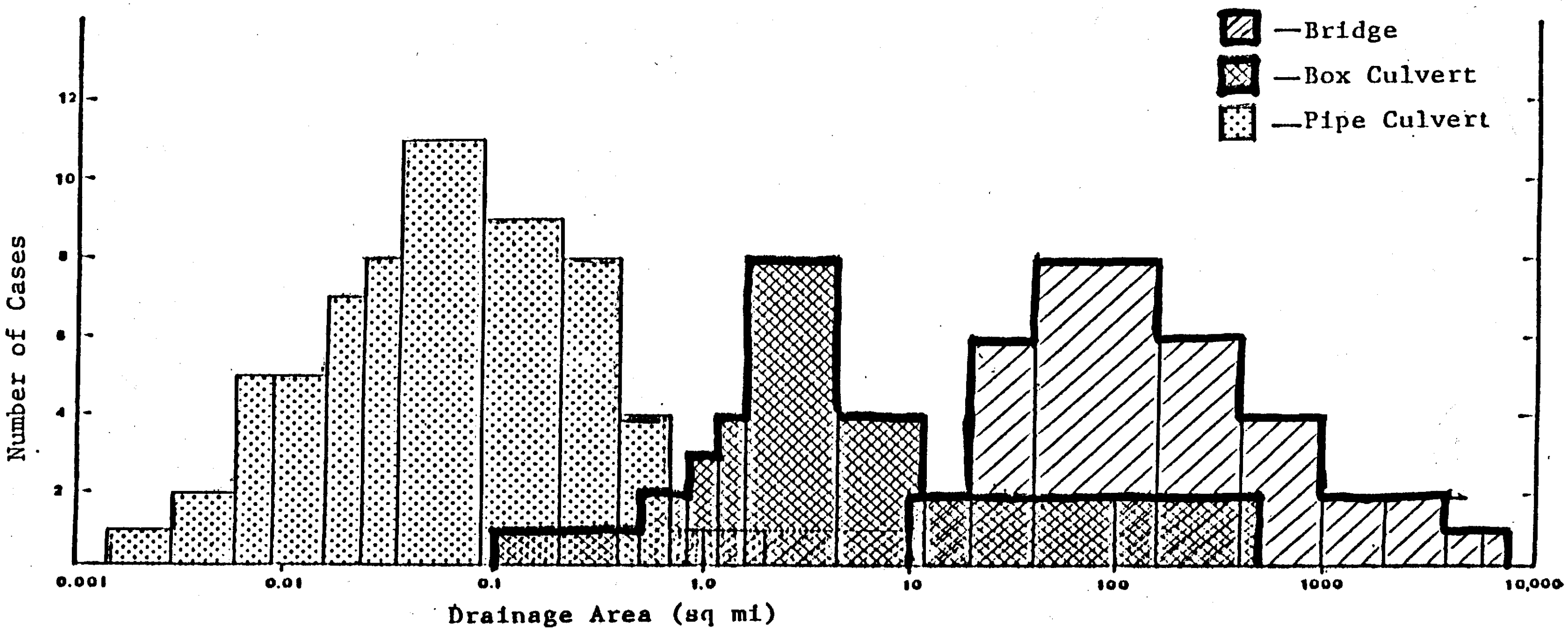


Figure 4.2. Histogram of Bridges, Box Culverts, and Pipe Culverts.

highway drainage structure was closely related to the corresponding drainage area. Although there is some overlapping with drainage area between pipe culverts and box culverts as well as box culverts and bridges, clear distinction is not difficult. Based on Figure 4.2, the criterion given in Table 4.3 was used as the basis for selecting sites for expanding the database on floodplain slope for a given highway drainage structure type. For the sites with drainage basin areas falling in the overlapping region of two different types of structure, it was assumed that hydraulically the two structure types are often equally suitable with other site specific factors dictating the structure type selected.

As pointed out by Wacker [13], determination of structure type depends on a number of things such as drift, upstream property, grade-line, low channel width, land use, and hydrologic region. Furthermore, a suggestion was given to use an index discharge such as mean annual discharge instead of the drainage area as described above. Although the original intention of using area-type criterion was to help in selecting representative sites for the various structure types (so that information on floodplain slopes in the database can be expanded through the synthesis procedure) an attempt to decide which structure type should be used based on an index discharge is possible. The justification for doing so is: having observed from the records that both culverts and bridges have been constructed in the overlap area, it can actually be assumed that, given a site with drainage area in the overlap area, it is possible to use either a bridge or a culvert at the given site depending on the unique site characteristics. The computer program which would be developed (in a task yet to be completed) for generating the

Table 4.3. Criteria for Structural Classification.

	Bridge	Box Culvert	Pipe Culvert
Area of Drainage Basins (sq mi)	> 15	0.5 to 50	< 3

hypothetical database should consider the two types of drainage structures that occur within the overlapping area.

Use of index discharge as suggested would be a better criterion because it contains physiographic/hydrologic information rather than just physiographic information alone (as is the case with using just drainage area). However, use of hydrologic variables as criterion would require that these variables be updated periodically as the data set is increased with time (mean annual discharge will change with 10 more years of record). This may lead (but the probability is low) to a situation where structures built several years previously would not be consistent with present criterion. If this were to happen, consideration should be given to using some quantity that would be less variable over time.

5. COLLECTION OF RELEVANT INFORMATION ON TANGIBLE ECONOMIC VARIABLES IN ASSESSING FLOOD DAMAGES

Flood damages and associated economic variables of roadway crossings include those from (1) floodplain property damage, (2) damage to pavement and embankment, and (3) traffic related losses.

5.1 Damage to Floodplain Properties

Damage to upstream floodplain properties is primarily caused by the backwater due to the presence of roadway crossings. Two major components are included in this damage category: (a) damage to crops and (b) damage to buildings and their contents.

Crop damage can be estimated by the following equation

$$\text{Crop damage} = \sum_{i}^{N} U_i \, A \, P_i \, Y_i \, Q_i \tag{5.1}$$

in which U_i is the unit price of crop i, in dollars/ton; A is the total floodplain area inundated, in acres; P_i is the percentage of flooded area planted with crop i; Y_i is the yield per acre of crop i, in tons/acre; and Q_i is the percentage of damage to crop i. The total area of floodplain inundated is a function of flood magnitude, hydraulic characteristics of the channel, and geometry of the highway crossing. Its determination requires hydraulic analysis of the backwater profile for a particular design of highway drainage structure of a specified return period.

Crops typically found in Wyoming and their corresponding unit price and yield per acre for irrigated and nonirrigated lands can be obtained elsewhere [14] and are summarized in Table 5.1. Crop yield information for irrigated land is tabulated in Table 5.2. Comparison of Tables 5.1 and 5.2 indicate that average crop yield of irrigated land is higher than for irrigated and nonirrigated lands together. Conceivably, it is more convenient to irrigate farm land located in the floodplain, unless farmers are prohibited from doing so due to legal restrictions. Therefore, it might be more reasonable to use information in Table 5.2 in assessing crop damages since the floodplain area is most likely irrigated. Crop losses due to floodwaters, basically, depend on the duration and depth of inundation. Representative percentages of damage to crops due to flooding are shown in Table 5.3.

Due to the site specific nature of the variable P_i in Eq. (5.1), it must be a part of the input and must be specified for each particular site under study. Presently, U_i, Y_i and Q_i for various crops are treated as the parameters and their typical values should be built

Table 5.1. Unit price of crops in Wyoming.

Crops	*Yield (Yi)	**Price (Ui)	(Yi)*(Ui) dollars/acre
Alfalfa hay	2.35 ton/ac	66.5 $/ton	156.28
Other hay	1.20 ton/ac	66.5 $/ton	79.8
Corn-grain	93.6 bush/ac	2.95 $/bush	276.12
Corn-silage	16.05 ton/ac	-	-
Sugar beets	2.34 ton/ac	34.28 $***/ton	695.63
Barley	62.7	3.20	200.64
Oats	46.5	1.70	79.05
Wheat	26.5	3.30	87.45
Dry beans	1,836 lbs/ac	14 $/100 lbs	257.04 $/acre

NOTE: *Values in yield (Yi) are the average of 10-year record (from 1975 to 1984 yield per harvested acre).

**Unit price of crop i in 1984 dollars.

***Data in 1983.

Table 5.2. Yield of crop for irrigated land in Wyoming (1983-1984).

Crop	Yield 1985	1984	Avg.	(Yi)*(Ui) $/acre	Unit of Yield
Wheat	54.9	54.0	54.5	179.85	Bushels/acre
Barley	72.6	71.5	72.05	230.56	Bushels/acre
Oats	62.1	62.6	62.4	106.08	Bushels/acre
Dry beans	1,800	2,050	1,925	269.50	lb/acre
Sugar beets	19.2	20.0	19.6	670.32	Ton/acre
Corn	104	100	102.0	300.9	Bushels/acre
Alfalfa hay	2.95	2.9	2.93	194.85	Ton/acre
Other hay	1.50	1.48	1.49	99.09	Ton/acre

Note: Unit price using data in 1984.

Table 5.3. Percent damage to crops.

Crops	% damage			
	≤ 24 hr inundation		≥ 24 hr inundation	
	0-2ft	2 ft	0-2ft	2 ft
Corn	54	88	75	100
Soybeans	92	150	150	100
Oats	67	97	81	100
Hay	60	82	70	97
Pasture	50	75	60	90
Winter Wheat	57	87	72	100

into the computer program as default-valued variables based on Tables 5.1 through 5.3 unless otherwise specified by the user. At the present time, when the computer program subroutine for generating crop type is utilized, only one type of crop is generated for a given site. This simplified modification was done at the request of Mr. Wacker with WHD [13].

Evaluation of damage to buildings in floodplains due to backwater effects requires information on the number of buildings in the floodplain, their locations, types and values. Broadly speaking, building types typically found in floodplains are residential buildings with and without basements, commercial buildings, agricultural structures, commercial outdoor storage areas, and mobile homes. Flood damage to buildings in floodplains can be estimated by Eq. (5.2) as

$$\text{Building damage} = \sum_{j}^{N} V_j P_j \tag{5.2}$$

where N is the total number of buildings at risk in the floodplain, V_j is the estimated value of building j, and P_j is the percentage of damage to building j due to inundation. The variable P_j in Eq. (5.2) in general is a function of building type and inundation depth. Some general relations for P_j and inundation depth were available from the literature [1,15,16] and are shown in Figures 5.1 through 5.3. No data have been collected specifically for Wyoming to develop similar curves. However, Figures 5.1 and 5.3 show the percent damage, rather than damage in monetary value, which would make them more suitable for use in this study. Recently, water depth-damage relations as used in 1987 for assessing flood insurance were obtained from FEMA. Tables 5.4 and 5.5

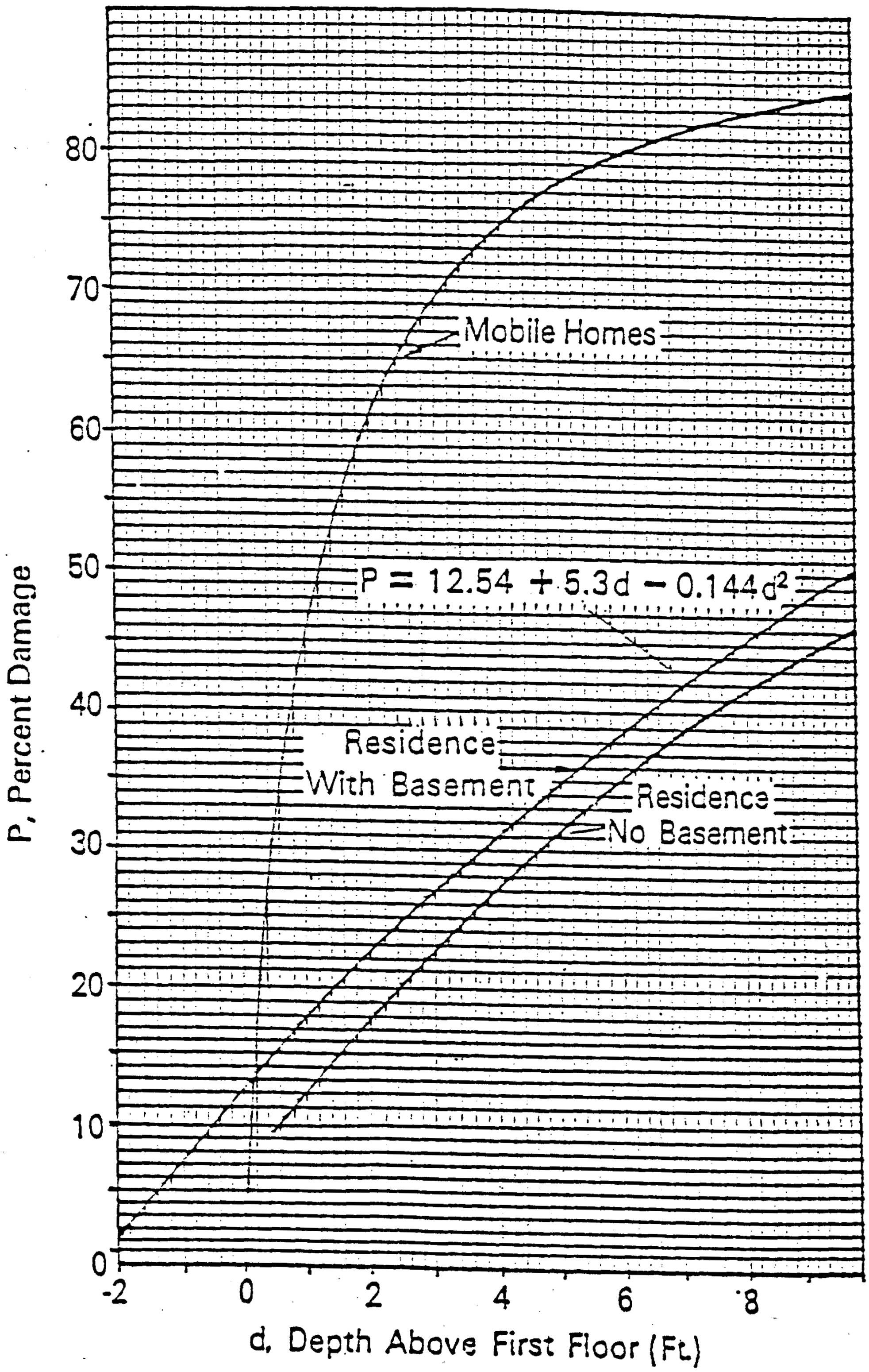


Figure 5.1. Percent Damage, Mixed Residences (1).

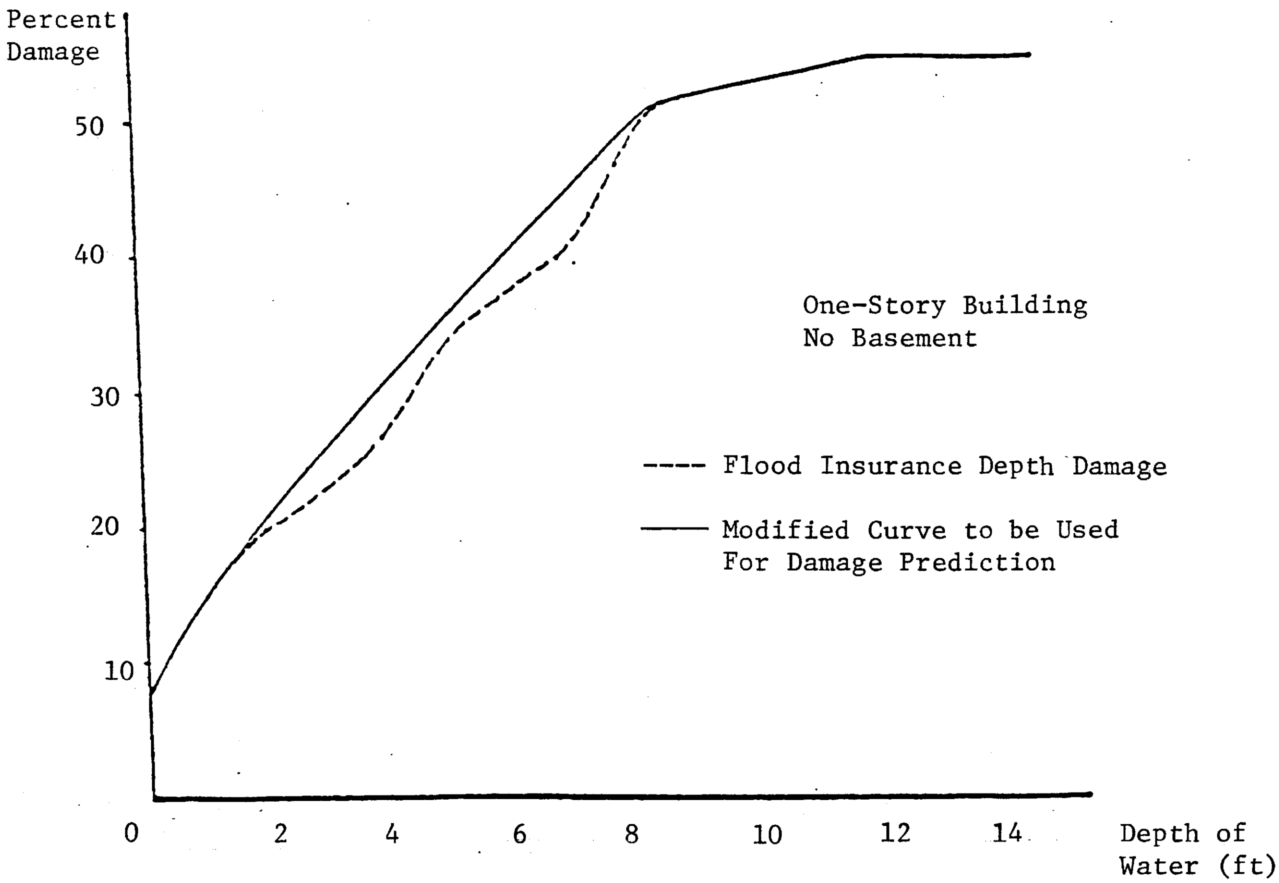


Figure 5.2. Typical Flood Damage Versus Depth of Inundation Curve (15).

Water depth above floor	Damage ratio, % of total value
-1	0
0	3
1	9
2	13
3	25
4	27
5	28
6	33
7	34
8	41
9	43
10	45
11	46
12	47
13	48
14	49
15	50

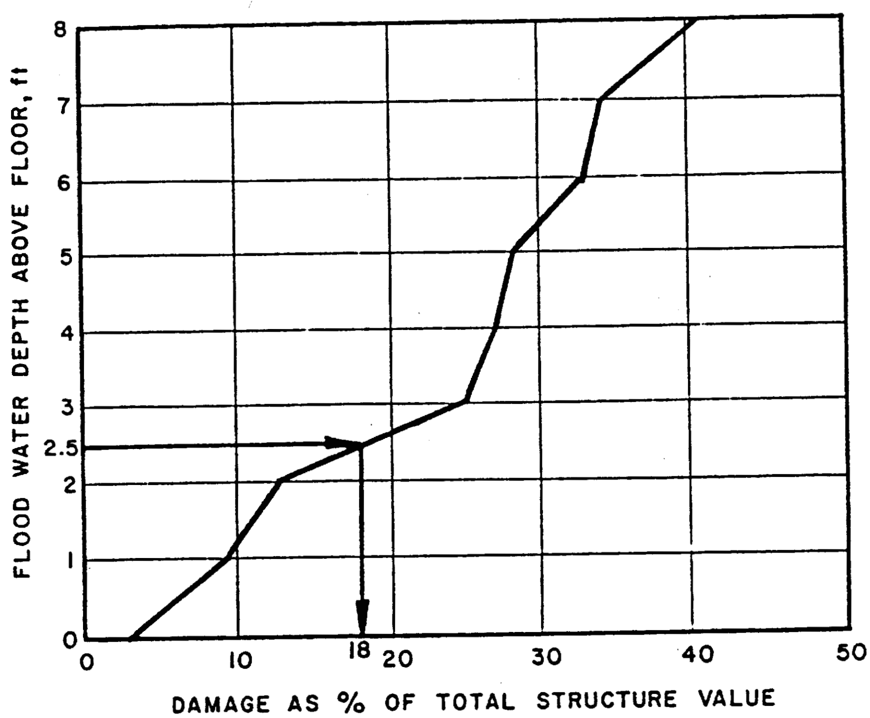


Figure 5.3. Flood Elevation Versus Damage to Structure (16).

Table 5.4. Depth-Percent Damage Values for Building Coverage.

Water Depth (ft.)	Types of Residential Housing					
	One Floor W/O Bsmnt.	Two Floor W/O Bsmnt.	Two Floor W/ Bsmnt.	Split Level W/O Bsmnt.	Split Level W/ Bsmnt.	Mobile Home
-4						
-3						
-2			4.05		3.00	
-1			8.08		5.01	
0	7.46	5.05	10.85	3.01	6.02	8.24
1	13.55	9.01	15.15	8.99	15.68	44.34
2	20.61	13.06	20.76	13.00	18.84	63.26
3	26.85	18.00	23.44	24.98	21.92	73.31
4	28.76	19.98	27.83	26.97	26.94	78.48
5	29.91	21.98	32.74	27.99	31.93	79.75
6	40.70		37.73	32.99	34.95	80.86
7	42.82	25.98	43.64	34.00	35.95	81.98
8	43.98		48.61	40.99	43.96	
9	44.99		50.84	43.00	47.96	
10	46.32	37.98	52.88	44.99	49.96	
11	47.06		54.95		51.99	
12	48.26		56.90	47.00	53.98	
13	49.01		58.95		56.00	
14	49.98		59.97		57.99	
15	50.10				59.00	
16	50.10				60.00	
17	50.10					
18	50.10					

Table 5.5 Depth-Percent Damage Values for Contents Coverage.

	Residential Contents			Commercial Contents		
Water Depth (ft.)	First Floor Only	First Floor & Above	Mobile Home	First Floor Only	First Floor & Above	Mobile Unit
-4						
-3						
-2						
-1						
0	11.20	7.32	3.25	10.20	7.17	3.03
1	22.84	10.38	26.58	17.42	9.75	26.96
2	31.39	17.96	49.12	23.53	17.72	49.92
3	34.09	22.54	64.08	29.37	22.62	64.89
4	36.70	28.14	70.35	35.21	28.34	70.89
5	40.54	33.06	75.60	40.05	33.15	75.89
6	44.88	38.92	77.68	45.01	39.26	77.97
7	49.86	43.91	78.80	49.99	44.03	78.98
8	54.77	49.79	80.73	55.03	50.04	80.99
9	59.88		82.88	59.98		82.99
10	59.83	57.93		60.01	57.98	
11						
12						
13						
14						
15						
16						
17						
18						

show the depth-percent damage values of building coverage and contents coverage, respectively, for various residential and commercial buildings. Tables 5.4 and 5.5 represent the most current information on flood water depth-damage relations available. Since both tables are used nationwide for assessing flood damage, it was decided to adopt them in this study for the State of Wyoming. Using Tables 5.4 or 5.5, one is able to estimate damage to buildings by backwater effects once the information on the value of the building is determined. A computer program incorporating information contained in Tables 5.4 and 5.5 was developed.

It is generally difficult to know precisely the value of a building because it is dependent on its contents and the condition of the building itself. Therefore, it would be practical to leave the building value along with its type as an input variable to be specified by the user or designer.

To use Tables 5.4 and 5.5 and Eq. (5.2) for estimating flood damage to buildings requires the specification of elevation of the first floor flood entry point for each building at risk in the floodplain. This would enable the computation of inundation depth above the first floor under various flood magnitudes.

A much more extensive and comprehensive list of flood damage or losses due to floods is given by the recent Bureau of Reclamation's Technical Memorandum No. 7 [17]. The BuRec Technical Memorandum No. 7 with this detailed listing deals with flooding due to the failure of dams with consequences several orders of magnitude more severe than that of backwater caused by roadway crossings. The inclusion of such items as employment and income losses, utilities, farm equipment, lost productive capacity of land, and many other items may not be entirely realistic for this study. This study might, however, provide some idea for a correction factor in cases where some losses are not explicitly accounted for by this study.

To summarize the discussion in this section, input variables for a site specific situation and information representative of state-wide conditions in estimating floodplain property damage are listed in Table 5.6.

5.2 Flood Damage to Pavement and Embankment

When floodwater overtops roadway crossings, damage to pavement and embankment could occur due to erosion. In principle, losses of pavement and embankment due to a flood can be estimated as

$$L_{P\&E} = \frac{E_w P_e C_e V_e}{48} + \frac{P_w P_p C_p A_p}{20} + C_a + M_c \qquad (5.3)$$

in which $L_{P\&E}$ is the economic loss of pavement and embankment, E_w is the embankment width (ft.), P_e is the percentage of embankment loss, C_w is the cost of the embankment (dollars/yd^3), V_e is the total volume of embankment subject to overflow (yd^3), P_w is the pavement width (ft), P_p is the percent of the pavement loss, C_p is the cost of pavement (dollars/yd^2), A_p is the total area of pavement subject to overflow (yd^2), C_a is the adjustment for rapid repair, and M_c is the mobilization cost (dollars).

Variables in Eq. (5.3) relative to the physical layout of the roadway such as E_w, P_w, V_e, and A_p are determined internally by the computer software using default values based on a design standard for various highway systems unless they are otherwise specified. Percentage

Table 5.6. Economic Variables in Assessing Flood Damages to Upstream Properties.

	Crop Damage	Building Damage
Site-specific input variables	-Percentage of acreage distribution of various crops	-Numbers of buildings in floodplain at risk crops in floodplain -Values of buildings -Types of buildings -Elevation of doorway for each building
Statewide Representative Information	-Yield for crops (Table 5.2) -Unit price for crops (Table 5.2) -Percentage of damage for crops (Table 5.3)	-Percentage damage and inundation depth (Tables 5.4 and 5.5)

of embankment loss and pavement loss information for different embankment surfaces under various overtopping conditions are available from [28] which have been incorporated in the computer program. Costs for conservatively estimating embankment and pavement repair are available from the "Average Bid Price" [4] documents of the Wyoming Highway Department which can be built into the program as default-valued parameters. A list of parameters and variables for estimating damage to pavement and embankment is given in Table 5.7.

5.3 Traffic Related Losses Due to Flooding

Primarily, three components make up the total traffic related losses due to flooding: (i) increased vehicle running cost, (ii) vehicle occupant time losses, and (iii) accident costs related to the flooding.

(5.3.1) Increased Vehicle Running Cost. This cost item results from taking a detour route rather than the normal route due to traffic interruption caused by overtopping floodwater and the following roadway restoration activity. The increased vehicle running cost can be estimated by

$$TL_{RC} = \frac{TD(ADTE)\ (\Delta L)(U_{RC})}{24} \tag{5.4}$$

where TL_{RC} is the traffic loss due to increased vehicle running cost (dollars), TD is the total delay time (hrs) which is the sum of overtopping duration, T_{ot}, and roadway restoration time, T_r; ADTE is the equivalent average daily traffic (vehicle/day); ΔL is the increased distance of travel between the detour and normal routes (miles); and U_{RC} is the unit vehicle running cost (dollars/vehicle/mile).

Table 5.7. List of Parameters and Variables for Assessing Pavement and Embankment Damage.

Default Parameters (To be set in software)	Parameters (To be computed)	Variables (To be specified by users)
-Unit cost of embankment	-Percentage of embankment loss	-Embankment width*
	-Length of embankment	
-Mobilization cost	-Volume of	-Pavement width*
	-Percentage of loss	-Embankment height or grade line*
-Adjustment for rapid repair	-Total area of pavement subject to overflow	-Embankment soil type

*Can be determined by adopting standard design roadway geometry unless otherwise specified.

When a vehicle is running on the road, the vehicle owner must pay taxes, insurance, maintenance, and fuel to be able to operate the vehicle. Therefore, the unit vehicle running cost should include all these various cost considerations. A study has been made by the U.S. DOT and FHWA which shows the cost [18] of owning and operating an automobile (Table 5.8). Data shown in Table 5.8 can be used as the basis for determining U_{RC} in Eq. (5.4).

Variable ΔL is the increased driving mileage due to the detour and must be specified by the designer. Information on average daily traffic (ATDE) in Wyoming can be obtained from [19]. To compute the delay time, it is required to estimate overtopping duration and roadway restoration time. The overtopping duration depends on the highway drainage opening and the shape and magnitude of the flood hydrograph while the restoration time depends on the extent of flood damage to the pavement and embankment and the rate of repair. The roadway restoration time can be estimated as

$$Tr = \frac{24V_eP_e}{R_e} + \frac{24A_pP_p}{R_p} + M_t \qquad (5.5)$$

in which R_e and R_p are rates of embankment repair (yd^3/day) and pavement repair (yd^2/day), respectively, and M_t is the mobilization time (hrs). The other variables have been defined previously.

(5.3.2) Time Loss of Vehicle Occupant. Information required for estimating the cost of time loss of a vehicle occupant, in general, is more difficult to obtain. To the researchers' knowledge, there are no immediate and direct data available for cost associated with the vehicle occupant(s) time (the average cost of a vehicle occupant in

Table 5.8. Unit cost of owning and operating automobiles.

Vehicle Size	U_{RC}^{*} (\$/vehicle/mile)
Large	0.3062
Intermediate	0.2784
Compact	0.2331
Subcompact	0.2271
Passenger van	0.3925

*The cost is obtained by averaging over a 12-year period for medium- priced vehicle operated in Baltimore area which was considered to be in the middle range.

terms of cost due to the delay) and a typical carrier occupancy composition. If these data were available, then the cost associated with time loss of vehicle occupancy could be estimated as

$$TL_{VO} = \frac{(TD)(\Delta L)(ATDE)(OR)\ U_{OC}}{24S} \tag{5.6}$$

in which TL_{VO} is the total cost of vehicle occupant time loss traveling on the detour (dollars), OR is the occupancy rate (persons/vehicle), U_{OC} is the unit cost per occupant (dollars/person/hr), and S is an average vehicle speed (mile/hr) over the detour length. As a first approximation, the trucking business might be used with some modifications to obtain ballpark estimates for some of the above values. This item could be included as part of the research to be done if the project is continued.

(5.3.3) Increased Accident Cost. Based on the published statistics on traffic accidents in Wyoming [19], it is possible to deduce accident rate and cost associated with injury, death, and property damage due to detours. The increased accident cost can be estimated by

$$TL_{ACC} = \frac{(TD)(ADTE)(\Delta L)(DR)(AIF)}{2.4 \times 100} \tag{5.7}$$

in which TL_{ACC} is the increased cost of an accident due to the detour (dollars), DR is the death rate (person/10^6 vehicle miles), and AIF is the accident-injury factor defined as

$$AIF = (IR)(U_{Inj}) + (DMR)\ (U_{damg}) \tag{5.8}$$

where IR is the injury rate (injuries/death), U_{Inj} is the unit cost per injury (dollars/injury), DMR is the damage ratio (injuries/death), U_{damg} is the unit cost of damage (dollars/damage claimed). A list of parameters and variables for assessing traffic and accident-related costs is given in Table 5.9.

In addition to [20], reference [21] contains the results of a sample survey of policy reported injuries and fatalities due to automobile accidents in the 48 contiguous states and the District of Columbia. The study represents 500,000 fatalities and seriously injured persons and reports the following: "Average economic losses for seriously injured persons were $4,200. Average economic losses to fatality cases were $2,300, exclusive of lost earnings. Economic losses to families who had one or more seriously injured members or fatalities averaged $4,200 to date of interview plus $6,100 in future lost earnings." It should be noted that the dollar figures mentioned here are 1970 dollars. Adjustment should be made to present day values with the realization that these costs are increasing substantially above the inflation rate of the dollar due to court decision awards in these types of cases.

6. DETERMINATION OF EXTENDED LTEC FREQUENCY

The purpose of highway construction is to serve the public. However, the action may have negative effects on other aspects important to the public if care is not exercised in design and construction. As contained in the Federal-Aid Highway Program Manual (FHPM) [22], federal-aid highway policy states: "It is the policy of the Federal Highway Administration that in the development of a project, a

Table 5.9. List of Parameters and Variables for Assessing Traffic Related Costs.

Default Parameters	Parameters to be Computed	Input Variables
-Unit vehicle running cost	-Total delay time	-Increased travel distance
-Mobilization time	-Overtopping time	-Average daily traffic
	-Roadway restoration time	-Rate of embankment repair
	-Accident-injury factor	
-Occupancy rate		
-Unit cost of occupant		
-Vehicle speed		
-Death rate		
-Injury rate		
-Unit cost per injury		
-Damage ratio		
-Unit cost of damage		

systematic interdisciplinary approach be used to assess engineering considerations and beneficial and adverse social, economic, environmental, and other effects; that efforts be made in developing projects to improve the relationship between man and his environment, and to preserve the natural beauty of the countryside and natural and cultural resources; that project development involve consultation with local, state and federal agencies, and the public; that decisions be made in the best overall public interest based on a balanced consideration of the need for fast, safe and efficient transportation, public services, and social, economic, and environmental effects, and national environmental goals."

Determination of an appropriate design frequency for highway drainage structures is an important element in the overall decision-making process. In addition to economic costs of the project as described in Section 5, there are other aspects to consider such as the effect of drift and ice, environmental impact, public convenience, and legal liability of the state highway agency which are intangible and might be equally if not more important in the decision-making process. Therefore, determination of an appropriate design frequency requires inclusion of many important tangible as well as intangible factors so that a balanced decision can be achieved.

6.1 Intangible Factors Affecting the Design Frequency

Cost is the main factor considered in common LTEC design practice. More specifically, most LTEC design practices determine the capacity and/or design return interval of drainage structures that minimize the total expected annual cost. However, the use of an expected value does not reflect the variability of the annual cost and other statistical characteristics associated with it. There are other intangible factors, in addition to the expected cost, that should be included in order to have a more complete picture of the annual cost of a drainage structure for a given capacity or return interval.

(6.1.1) Environmental and Hydraulic Effects. Construction of drainage structures for highway crossings frequently involve encroaching on the natural floodplain. In general, the presence of roadway crossings with encroachment would result in, for better or worse, a change in hydraulic characteristics such as flow distribution, flow velocity, and sediment transport capacity. Stream response to changes in these characteristics may be confined to the local area or may extend for many miles upstream and downstream of the site.

It is difficult quantitatively and definitively relating the design return interval to the potential hydraulic effects on a stream system. Heuristically, increases in the design return interval of a highway drainage structure, in general, represents less encroachment on the floodplain and, therefore, less disturbance to the natural hydraulic characteristics of flow.

Encroachment on the floodplain commonly does not affect the hydraulics of a stream during a normal low flow period. Effect on hydraulics becomes more pronounced under high flow conditions because hydraulic encroachment acts as a constriction in the flow path. The presence of encroachment tends to increase flow velocity in the vicinity of a structure site which increases the ability of the flow to erode the stream bank and bed. Therefore, after a major flood event, the stream somewhere downstream of a highway drainage structure site designed with a low return interval may become braided and unstable. Good discussions of general response of a stream system to the presence of a roadway crossing can be found elsewhere [22,23]. Table 6.1 summarizes the effect of bridges on meandering dynamically stable channels [24]. Where the channel is in the transition range between a stable or braided regime, the structure may cause a threshold to be exceeded and force a channel to become unstable and braided. Where the channel is already unstable and braided, the hazards to the structure may be significantly increased, but the environmental hazards will not materially change.

Impact of roadway crossings on the environment primarily arises from the potential increase in sediment concentration as the result of change in hydraulic characteristics. Highway drainage structures designed with a lower return period are more susceptible to being overtopped by major floods. When roadways are overtopped by floods, large quantities of embankment material may be eroded and carried into the stream. Too much sediment in a stream might have, at least temporarily, destructive effects on fish and wildlife habitat. There could be other changes in stream systems induced by the roadway crossing that might have some impact, for better or worse, on the aquatic ecosystem.

As stated above, assessment of hydraulic effects and environmental impacts associated with different design return intervals can best be made through subjective and personal judgement. If a stream over which a roadway crossing is to be constructed does not contain any environmental sensitive reaches, then the decision can be made mainly on tangible factors and perhaps other intangible factors. On the other hand, care must be exercised in judging the effects of the drainage structure on the ecological system and overall stream system.

Table 6.1. Effect of Bridges on Meandering Dynamically Stable Channels [23].

Structure	Effect	Result
Embankment	-Obstruct drainage in flood plain & increase flow intensity through opening	-Local scour at piers & abutments -Increase hydrodynamic force on piers -Increase upstream water level, & magnitude & frequency of floods upstream
Abutment	-Obstruct migrating meander & change pattern	-Extensive bank erosion downstream
	-Deflect flow pattern & increase local flow intensity	-Local scour at abutments
	-Reduce width of waterways & increase flow intensity through opening	-Local Bank erosion downstream -Increase scour at abutments, piers & in waterway -Bank eriosion downstream -Increase upstream water level & magnitude & frequency of floods upstream
Pier	-Deflect flow pattern & increase local flow intensity	-Local Scour at piers -Increased hydrodynamic forces on piers
	-Reduce width of waterway & increase flow intensity	-Increase scour at pier, abutment & in waterway -Increase upstream water level & frequency of flood upstream

(6.1.2) Public Serviceability. This is a term devised for the purpose of this research which covers broad and general serviceability of roadways to the public. It includes primarily the notion of traffic interruption due to extraordinary circumstances such as flooding. Traffic interruptions could be a severe occurrence. The seriousness of the situation may largely depend on the traffic volume, traffic delay incurred, availability of alternative routes, and overall importance of the route, including the provision of emergency and rescue [Section 5 of Chapter 2, Ref. 22].

Although the tangible aspect of traffic interruption can be estimated as described in Section 5.3, there are intangible aspects of the serviceability of a highway which cannot be measured in terms of monetary value. These aspects may include physiologic feeling of highway users, what level of importance the highway users have become accustomed, and the importance of the route to national defense and to the economic well-being of a community if traffic interruption occurs.

Simplistically the public serviceability of a highway at a roadway crossing can be measured by its ability to provide continuous service to the public without being interrupted by flooding. This measure is then closely related to the design return interval used for drainage structures. The larger the design return interval for a drainage structure, the less frequent will the traffic be interrupted by flooding which naturally would have a higher serviceability and cause less inconvenience, both tangible and intangible, to the traveling public.

(6.1.3) Legal Litigation. State highway departments must design roadway crossings with extreme care to best serve the general public. It is, however, a hard fact that sometimes the engineers and/or Highway Department may be involved in legal litigation. In general, legal litigation could arise from many possible causes. Chapter V of AASHTO's Highway Drainage Guideline [22] provides a brief yet comprehensive discussion of various laws and regulations affecting highway drainage design.

Even if a highway engineer carefully practices drainage design for roadway crossings with all legal regulations in mind, it is possible that someone would sue the design engineer and/or the Highway Department for improper design of roadway drainage structures. This would generally occur after a major storm event which causes some flood damage to properties or creates a hazardous condition at roadway crossings that endanger the life of motorists.

Among many things, the design return period used for highway drainage structures more or less measures the likelihood of the State Highway Department getting involved in a legal battle regarding the appropriateness of its drainage design policy. Intuitively, use of a larger return period would result in less of a chance of being involved in legal litigation for the Highway Department which as a practical matter could be desirable from the Highway Department's point of view. Conversely, attempting to avoid litigation by designing for very large return periods at all drainage sites would generally be uneconomical.

(6.1.4) Other Factors. In addition to the four intangible factors mentioned above that may have significant impact on the determination of design frequency for highway drainage structures, the following intangible factors may also be added to the list: potential loss of life, national defense highway, and impact on local economy. These additional factors were extracted from Table 1 of reference [22].

In summary, the list of intangible factors mentioned in this section are only meant to be tentative for the purpose of discussion and consideration in devising a reasonable and prudent methodology for reducing or extending a design frequency determined by using tangible factors with the LTEC design procedure. All these intangible factors (attributes) are noncommensurable and, most of them, are in conflict with the economical consideration of drainage structure design. Consideration of all or part of these factors would provide a much more complete picture of the problem than the conventional LTEC design procedure which considers only the economic aspect of the problem. It can be realized that use of a multiple-attribute approach enhances more realistic decision-making and the design frequency so determined will be more acceptable in practice.

6.2 Mechanisms for Multiple-Attribute Decision Making

There are methods with various degrees of sophistication developed for multiple-attribute decision making. Had this research project continued, a simple yet quite effective and popular method called the "simple additive weighing technique" would have been initially employed to help determine and evaluate intangibles for inclusion in an extended LTEC design frequency. The technique involves an analysis of an information matrix consisting of a decision-maker's subjective evaluation of his/her preference by assigning ratings to each of the attributes involved for a number of alternatives under consideration.

A typical information matrix for a multi-attribute decision-making problem is shown in Figure 6.1. The relative merit of each alternative is judged on the basis of its final rating computed as

$$F_i = \sum_{j=1}^{M} R_{ij} W_j / \sum W_j \qquad \text{for all } i=1,2,\ldots N \tag{6.1}$$

in which F_i is the final rating for alternative i, R_{ij} is the rating for alternative i with respect to attribute j, and W_j is the weight for attribute j representing the relative importance of attribute j, N and M are, respectively, the total number of alternatives and attributes.

In relating to the problem of determining an extended LTEC design frequency, the attributes are the economic (tangible) factors and those intangible factors discussed in the previous subsection and the alternatives are the various design frequencies considered by design engineers and/or policy makers of the State Highway Department. In determining the list of alternative design frequencies to investigate, one should use the economic LTEC design frequency as the lower bound. Consideration of additional attributes would generally lead to the use of a larger design return interval or frequency. Determination of the list of design return periods in excess of the economic LTEC design frequency to be investigated is rather arbitrary. At present, the 500-yr event is commonly used as an upper limit in highway bridge design considerations and some FEMA studies.

6.3 Some Issues to be Resolved

Determination of an extended LTEC design frequency using the multi-attribute decision-making approach as proposed is a plausible and

		ATTRIBUTES			
		A_1	A_2	---------	A_M
ALTERNATIVES	a_1	R_{11}	R_{12}	- - - - -	R_{1M}
	a_2	R_{21}	R_{22}	- - - - -	R_{2M}
	-	-	-		-
	-	-	-		-
	a_N	R_{N1}	R_{N2}	- - - - -	R_{NM}
Weight		W_1	W_2	- - - - -	W_M

Figure 6.1 Information Matrix.

viable way of problem solving. There is no known study to the investigators' knowledge that has examined the multi-dimensional aspects of the LTEC design, let alone the solution approach. This could be a new area of challenge in the LTEC design of highway drainage structures or risk-based design philosophy as a whole. Several important issues remain to be resolved for implementing the proposed technique:

(a) Who is (or are) the decision-maker(s)?--Administrators in the Highway Department may very likely have different views than that of engineers with regard to the relative importance of attributes. Therefore, depending on who is "playing the game", the conclusions are bound to vary from one "player" to another. Since the study is aimed at determining a unified and consistent drainage design policy for the Highway Department, it is logical to consider the entire Highway Department, as a whole, the sole decision-maker. The simple additive weight technique described above is suitable to a single decision-maker. The theories and techniques in game theory are developed for cases of multiple decision-makers who have conflicting views and interests in attributes among them. The nature of the problem in game theory is negotiation, which is not the case with our problem.

(b) What are the attributes to be considered?--Based on the list of factors mentioned above which might have impact on selection of design frequency, the investigators and the WHD staff need to decide a viable list of attributes for implementing the proposed technique.

(c) How to design a procedure for an operation survey?--This issue mainly concerns the designing of a set of questionnaires under various conditions for which decision-maker's judgments on weights and ratings are to be asked. The difficult part is to devise a questionnaire set that is easily and intuitively understandable for all participants to be involved in the survey. Furthermore, a need to determine who will be the participants is required. At this point in time, a draft questionnaire has been developed on this subject area.

(d) How to synthesize and analyze survey results?--Results of a survey showing various participants' judgments on weights and ratings will be varied. Even for a single decision-maker (or participant), it is not difficult to imagine that he/she might not be able to assign an exact value to each of the weights or ratings. Consequently, it would be more realistic and reasonable to allow the decision-maker to assign lower, upper and most likely values for the weights and ratings to reflect the degree of uncertainty in his/her preference and judgement. For this case, a Master of Science thesis by Spett [30] under Dr. Tung's direction on the methodologies using probabilistic theory and fuzzy set theory has been developed to perform such decision-making under uncertainty.

7. IDENTIFYING DESIGN AND OPERATIONAL PERFORMANCE STANDARDS

In regard to the current WHD Operating Policy 18-6 which would be impacted by this research, it is expected that design flood frequencies currently adopted in the policy will be affected. At present, the design flood frequencies in Operating Policy 18-6 are arbitrary and based on practices that have "evolved" over the years. Conventionally, the design flood frequency as determined from the LTEC analysis is purely based on economic efficiency involving evaluation of tangible items. As discussed earlier, there are many intangible criteria in addition to tangible ones that might affect the final adoption of a design flood frequency in the operating policy. It is believed that in our present legal environment either one of the above procedures presently being used for determining a design flood frequency would not be acceptable in a court of law.

Determination of economic LTEC design frequency as related to economical and physical characteristics of a site under investigation will be developed in the later phases of this research should this project be continued. To determine the extended LTEC design frequency, which includes relevant intangible factors, a mechanism is discussed in the previous section for consideration. The difference between the extended LTEC recurrence interval (RI) and economic LTEC RI represents the "intangible RI", a terminology used on page 6 of the draft WHD Operating Policy 18-6, which is being considered for use as a guideline in Policy 18-6 for various functional highway classifications. Note that this intangible RI depends on the economic LTEC design RI and other intangible criteria. A question can be raised about the feasibility or practicality of including such an intangible recurrence interval as a hard guideline in the policy because judgements given to each of the intangible factors would vary from one site to another. That is, each design problem is quite site specific. Furthermore, the problem associated with Table 1 of the draft WHD Operating Policy 18-6, which relates the intangible RI to highway classification and structure type, is that not enough flexibility is provided to account for the physical, economical, and other intangible characteristics of a site under investigation.

As an alternative (for the purpose of discussion), it seems practical to perform the task of determining the LTEC design frequency in two parts:

Part (I) Determine the economic LTEC design frequency based on tangible physical and economic characteristics of the site. This can be achieved by referring to the design policy which includes the results that would be derived from this research should this project be continued.

Part (II) Determine the extended LTEC design frequency as outlined in the following steps:

1) Based on the result of part (I), determine a list of design frequencies to be considered.
2) Identify the list of important tangible and intangible factors (or attributes).
3) Assign ratings and weights. Otherwise, default values based on survey results could be used.

In the part (II) exercise, interactive computer software would have to be developed to assist engineers. This was a task of this project had it been continued. Default values of ratings and weights would be used to obtain the extended LTEC design frequency which would serve as the basis to compare the consistency of each individual engineer's judgement.

8. FAMILIARIZATION OF COMPUTER CODES FOR CULVERT AND BRIDGE DESIGNS

Computer programs and design manuals were obtained from the WHD. The computer programs for culvert design (CDS) and backwater (HY-7) for bridge design were obtained from the WHD and placed on an IBM PC type machine at the Wyoming Water Research Center.

Familiarization with the computer programs and design manual procedures has occurred. Interaction with WHD personnel familiar with the computer programs and design procedures also occurred.

Sample data files for CDS and HY-7 were tested. The basic function of the two computer programs are:

1. CDS - Culvert Design System
    a. Design options - selects a culvert size and number of barrels compatible with the engineering data, environmental constraints and site geometry.
    b. There is an upper limit of six barrels for commercial culverts (round concrete, round metal, arch concrete, arch metal) or five for concrete box culverts.
    c. Review option - provides hydraulic performance data for a specific culvert identified by the user.
    d. Program allows for upstream pond storage.
2. HY-7 - the seventh member in a series of FHWA computer programs for hydraulic analysis (FHWA Program HY-7).

    HY-7 provides a water surface profile computational tool specifically applicable to bridge waterway design using risk analysis concepts.

In attempting to obtain reliable decision rules for the selection of an appropriate design frequency for highway drainage structures in

Wyoming, generation of a large number of hypothetical sites, perhaps 200, for various drainage structure types as well as for various hydrologic regions in Wyoming is suggested when this project is continued at some future date. The approach to be taken would be to apply the two computer design codes to each hypothetical site generated. The contention is, however, that it would be a very slow and time-consuming task if the computations are to be made on a PC (the new 386 machines may eliminate this problem). Efforts therefore were made to upload the two programs to the University of Wyoming Cyber 760 and 840 mainframe computer. Currently, the program CDS is operational on the Cyber 840, while the HY7 is yet to be worked out. The mainframe version of the CDS has been modified in many places to handle the extra computations required for later analysis. Since the University of Wyoming Computer Center will remove all Cyber computers by December, 1988, all computer programs developed will be transferred to the new VAX mainframe computer systems.

9. GENERATING HYPOTHETICAL DRAINAGE SITES

Based on the basin/channel characteristics collected under Task 1, the selection of appropriate hypothetical drainage sites representative of Wyoming drainage basins is possible. However, the characterization of hypothetical drainage sites to which the economic analysis is to be performed should not be postulated arbitrarily. The reason is that, in reality, basin/channel characteristics are not entirely independent. As a result, it is proposed to employ a Monte Carlo simulation in a multivariate setting to generate such hypothetical drainage sites. In doing so, correlation structures between relevant basin/channel characteristics such as basin area, channel slope, channel top width, top width-depth ratio, etc., can be preserved. Consequently, unrealistic hypothetical drainage sites will be eliminated, and not generated or analyzed.

The first step in multivariate simulation to generate hypothetical drainage sites is to examine the statistical properties of the relevant basin/channel characteristics including their correlation structures. Based on the available data collected in Task 1, the statistical properties of the relevant basin/channel characteristics and their correlations are shown in Tables 9.1 and 9.2. It should be pointed out that the summary of the statistics and correlation matrix shown in Tables 9.1 and 9.2, respectively, are for the log-transformed variables. The reason for use of the log-transformed variables was that the original data when transformed are much closer to approximating a normal distribution than were the original data values (Figures 9.1a-f). Log-transformation of the basin/channel variables leads to a much more symmetric distribution. This observation further facilitates the use of readily available multivariate normal random number generation.

A simple hypothesis test was performed to assess whether the true correlation coefficient ρ between each pair of basin/channel characteristics (on the log scale) is zero. That is, the hypothesis test problem considered was

$$\text{Ho: } \rho = 0 \quad \text{versus} \quad \text{Ha: } \rho \neq 0$$

Under the normality assumption (which was verified for the log-transformed variables), the test statistic is

Table 9.1 Summary Statistics of Basin/Channel Characteristics at Log-Transformed Scale.

	N	Mean	STDEV
ln (W)	413	3.672	0.982
ln (W/D)	410	2.327	0.766
ln (Sr)	311	-3.218	1.357
ln(Sl)	308	-3.211	1.268
ln (A)	479	3.421	2.141
ln (Sc)	356	4.182	1.330

Note: W = channel top width (ft.)

W/D = channel top width-depth ratio (ft./ft.)

Sr = floodplain slope on the right (ft./ft.)

A = drainage area (sq. miles)

Sc = channel slope (ft./mile)

Table 9.2 Correlation Matrix of Log-Transformed Basin/Channel Characteristics.

	ln(W)	ln(W/D)	ln(Sr)	ln(Sl)	ln(A)	ln(Sc)
ln (W)	1.000	0.693*,**	-0.359*,**	-0.214*,**	0.499*,**	-0.498*,**
ln (W/D)		1.000	-0.177*,**	-0.052	0.235*,**	-0.275*,**
ln (Sr)			1.000	0.721*,**	-0.239*,**	0.521*,**
ln (Sl)				1.000	-0.202*,**	0.490*,**
ln (A)					1.000	-0.533*,**
ln (Sc)						1.000

Note: * = significant at 5%

** = significant at 1%

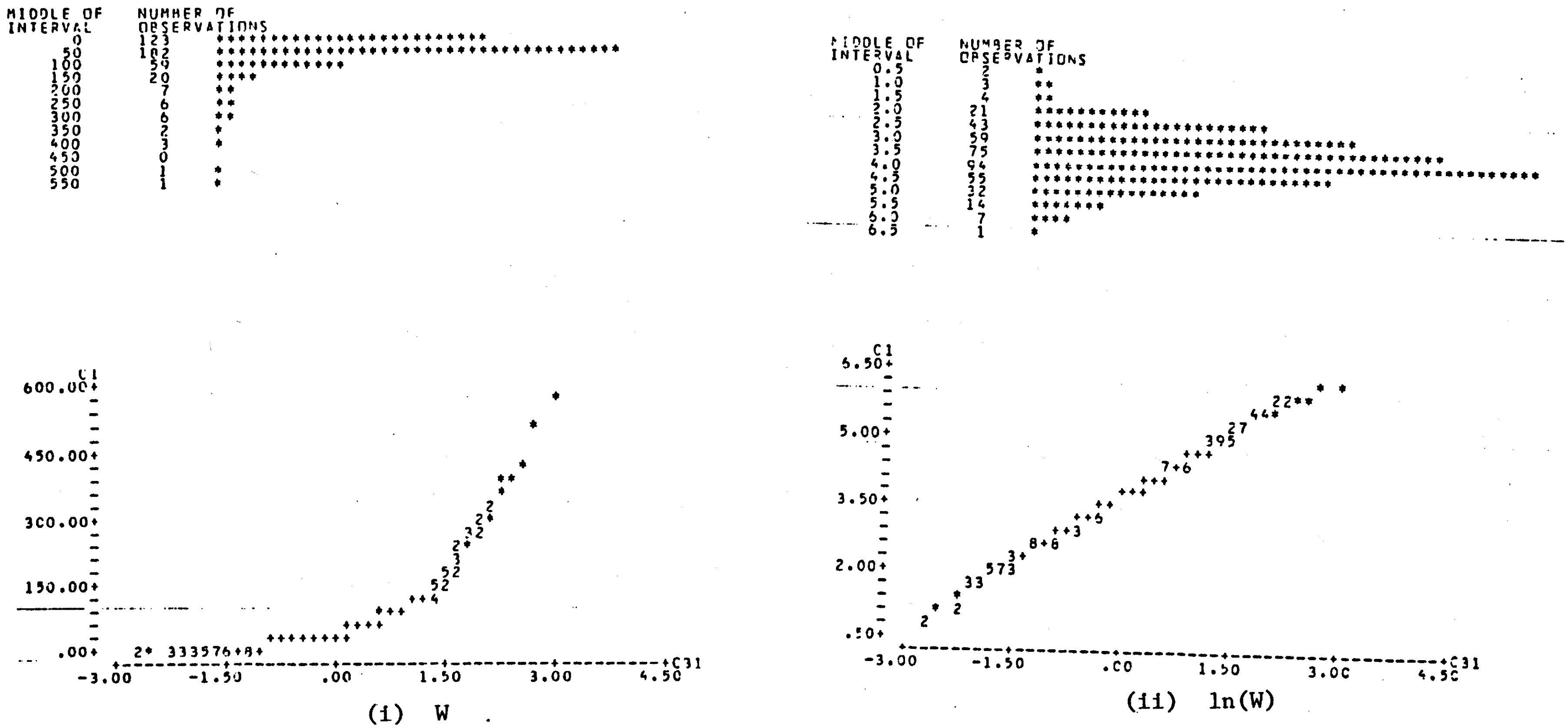
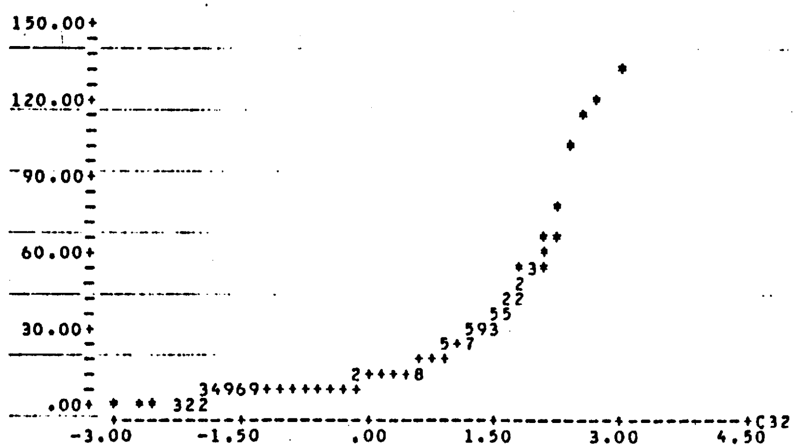


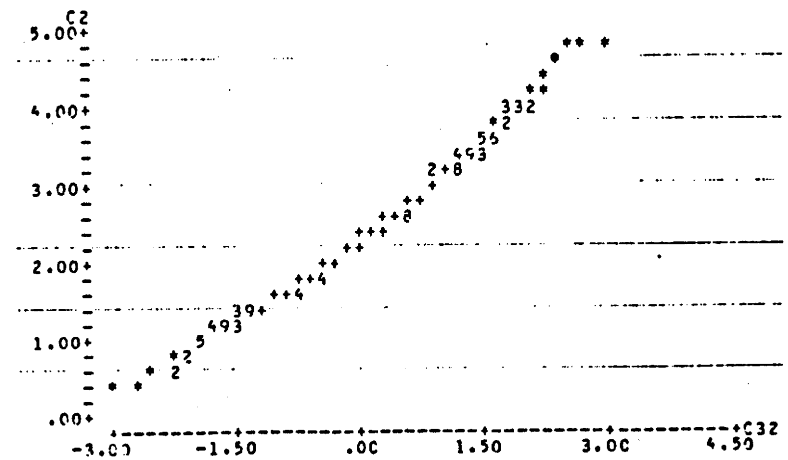
Figure 9.1a. Histograms and Normal Score Plots for (i) W and (ii) ln(W).

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
0	71
10	225
20	59
30	29
40	11
50	3
60	5
70	2
80	1
90	0
100	1
110	1
120	1
130	1

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
0.5	5
1.0	23
1.5	72
2.0	98
2.5	105
3.0	54
3.5	36
4.0	12
4.5	3
5.0	2



(i) W/D

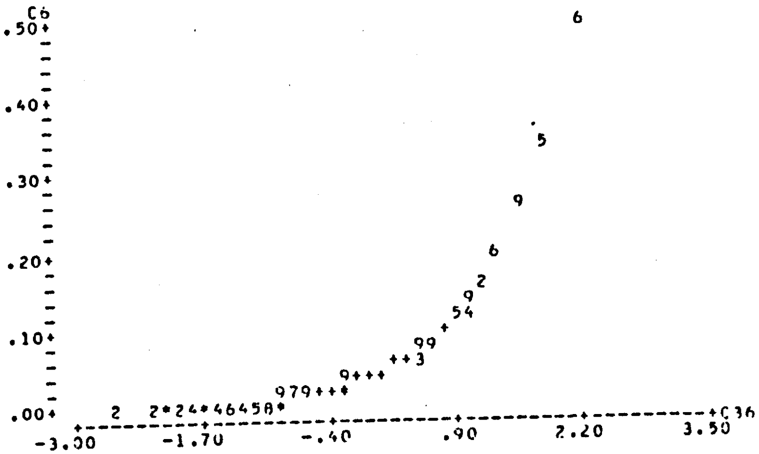


(ii) ln(W/D)

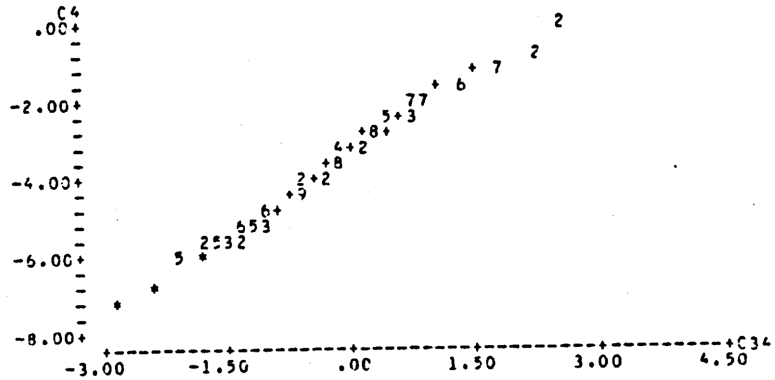
Figure 9.1b. Historgrams and Normal Score Plots for (i) W/D and (ii) ln(W/d).

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
0.00	81
0.05	90
0.10	26
0.15	15
0.20	6
0.25	9
0.30	0
0.35	5
0.40	0
0.45	0
0.50	6

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
-7.0	1
-6.5	1
-6.0	7
-5.5	17
-5.0	15
-4.5	21
-4.0	26
-3.5	24
-3.0	50
-2.5	22
-2.0	30
-1.5	16
-1.0	7
-0.5	2
0.0	2



(i) Sr

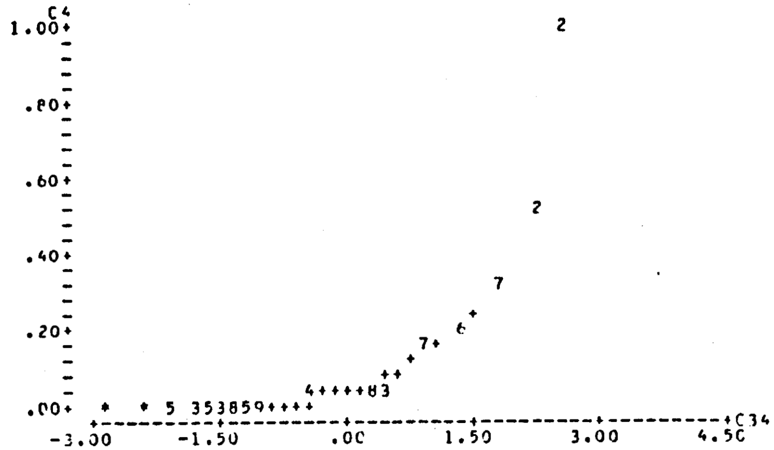


(ii) ln(Sr)

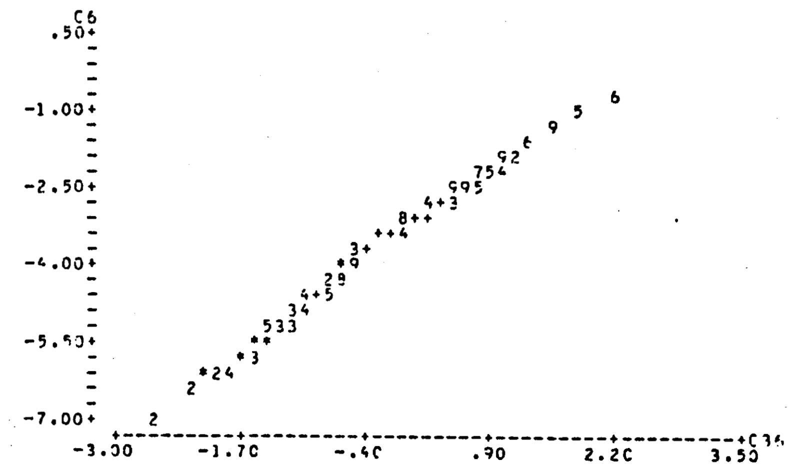
Figure 9.1c. Histograms and Normal Score Plots for (i) Sr and (ii) ln(Sr).

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
0.0	131
0.1	67
0.2	22
0.3	17
0.4	0
0.5	2
0.6	0
0.7	0
0.8	0
0.9	0
1.0	2

MIDDLE OF INTERVAL	NUMBER OF OBSERVATIONS
-7.0	2
-6.5	2
-6.0	9
-5.5	10
-5.0	12
-4.5	24
-4.0	21
-3.5	34
-3.0	45
-2.5	33
-2.0	20
-1.5	15
-1.0	5
-0.5	6

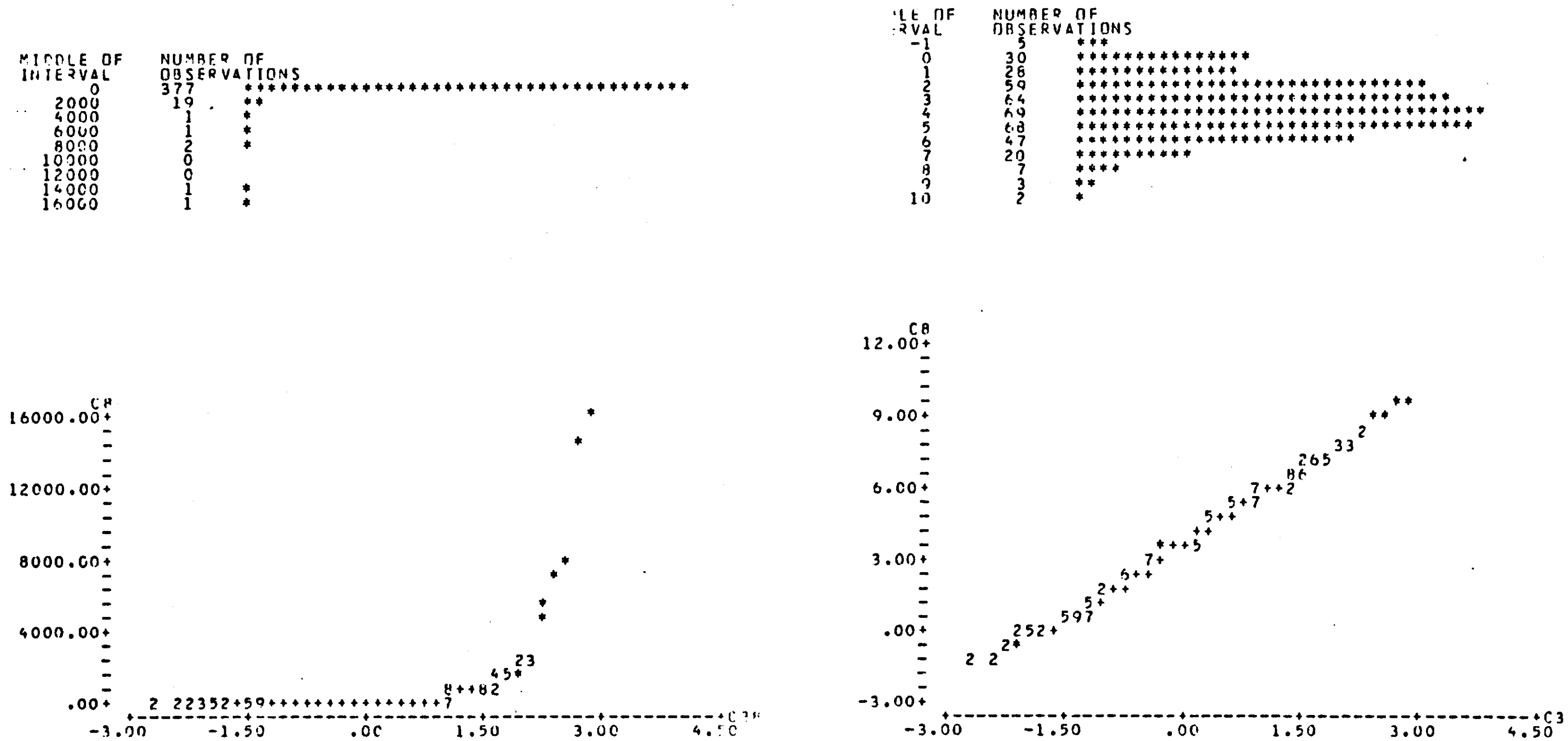


(i) S_1



(ii) $\ln(S_1)$

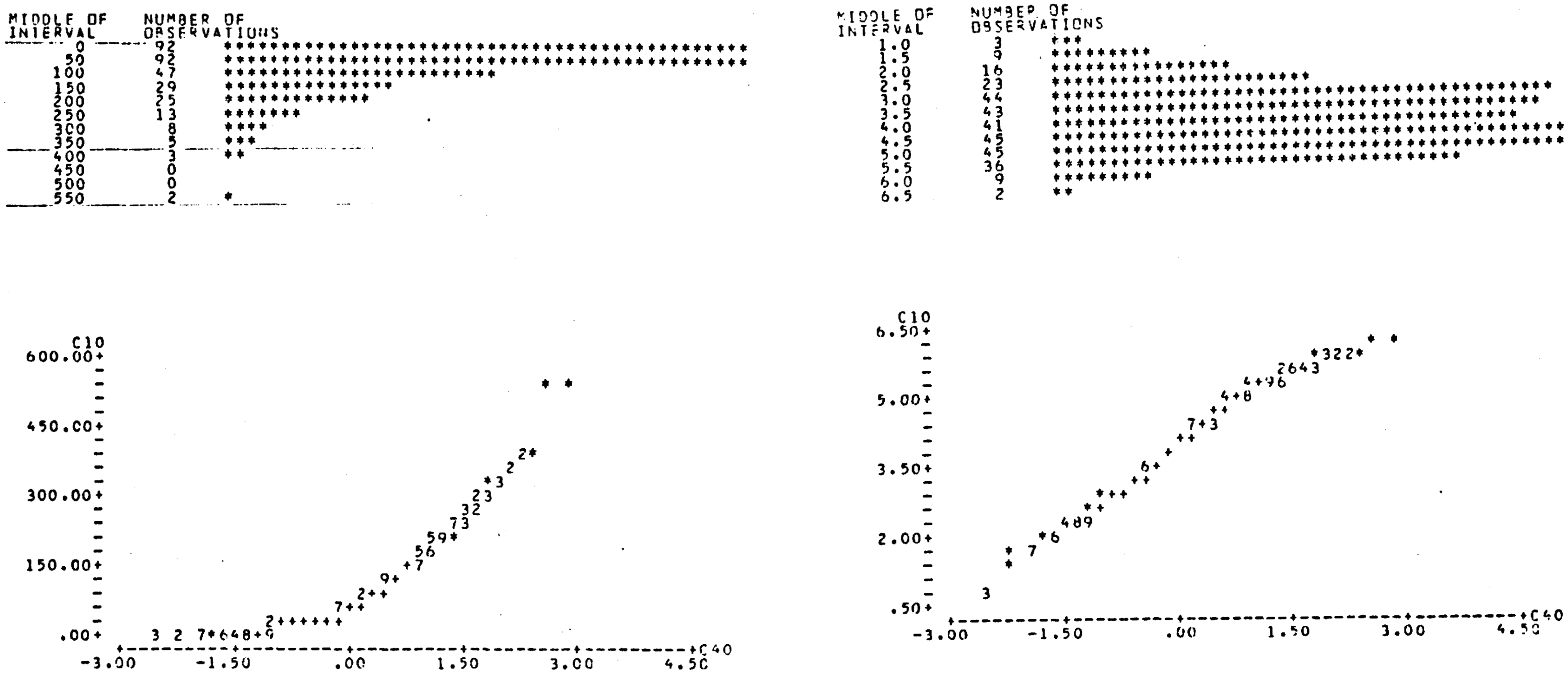
Figure 9.1d. Histograms and Normal Score Plots for (i) S_i and (ii) $\ln(S_i)$.



(i) A

(ii) ln(A)

Figure 9.1e. Histograms and normal score Plots for (i) A and (ii) ln(A).



(i) Sc

(ii) ln(Sc)

Figure 9.1f. Histograms and Normal Score Plots for (i) Sc and (ii) ln(Sc).

$$T = r \sqrt{n-2} / \sqrt{1-r^2} \tag{9.1}$$

in which the test statistic T has a t-distribution with n-2 degrees of freedom where n is the sample size and r is the sample correlation coefficient between two random variables. From Table 9.2, it is shown that all pairs except (ln(W/D), ln(Sl)) have non-zero correlation coefficients which are statistically significant at both the 1% and 5% levels. Based on the correlation matrix shown in Table 9.2, it is possible to generate basin/channel characteristics for the hypothetical drainage sites.

10. GENERATING ECONOMIC VARIABLES FOR HYPOTHETICAL DRAINAGE SITES

To evaluate the second cost associated with a drainage structure design, economic variables in a given hypothetical drainage site must be specified. The economic variables that are relevant in the LTEC design of highway drainage structures are crop distribution and average daily traffic (ADT), as well as the number, value, type an location of buildings.

Data for ADT on Wyoming highways of various system types are available from "Vehicle Miles" [19], published annually by the WHD. The range of ADT of various highway systems in Wyoming remains to be compiled. A uniform distribution would be assumed to generate the ADT. The bounds that define the range of ADT in Wyoming for a given highway system should consider conceivable future growth.

To generate the percentage of area of crop i in an inundated area, a simplified procedure described below was used. There are assumed to be a total of K types of crops planted in Wyoming. In fact, there are eight important economic crops planted in Wyoming which include wheat, barley, oats, dry beans, sugar beets, corn, alfalfa hay, and other hay. The percentages of planted area of those crops in Wyoming averaged over ten years (1975-1984) are shown in Table 5.2. Based on these percentages, random integer numbers, say 1-8, can be generated using a multinomial distribution. The percentage of area of a certain crop planted in the study sites can then be determined based on the proportion of the number of the crop type in the randomly generated sample. For example, using a multinomial distribution it is possible to obtain, out of a total of eight, two cases of wheat, one of oats, one of barley, three of alfalfa hay, and one other hay, for a particular hypothetical drainage site. It would then be assumed that the percentage of the inundated hypothetical floodplain is planted with 2/8 of wheat, 1/8 of oats, 1/8 of barley, 3/8 of alfalfa hay, and 1/8 of other hay.

Finally, to generate possible buildings located in the hypothetical floodplain presents the most challenging task at the moment because the problem is four-dimensional and involves the types of buildings, their numbers, values, and locations. The number of hypothetical buildings (regardless of their types) in an inundated hypothetical floodplain can be generated by the Poisson distribution given that the average number of buildings per unit area (building density) and susceptible flooding area are known. The susceptible flooding area, for simplicity, can be delineated as illustrated by Figure 10.1. The section 70-100 top width upstream of the drainage structure site was considered to be where the backwater effect for the largest discharge under investigation vanishes. At the time this project was being placed on hold, the main difficulty was the lack of immediately available data for estimating the building

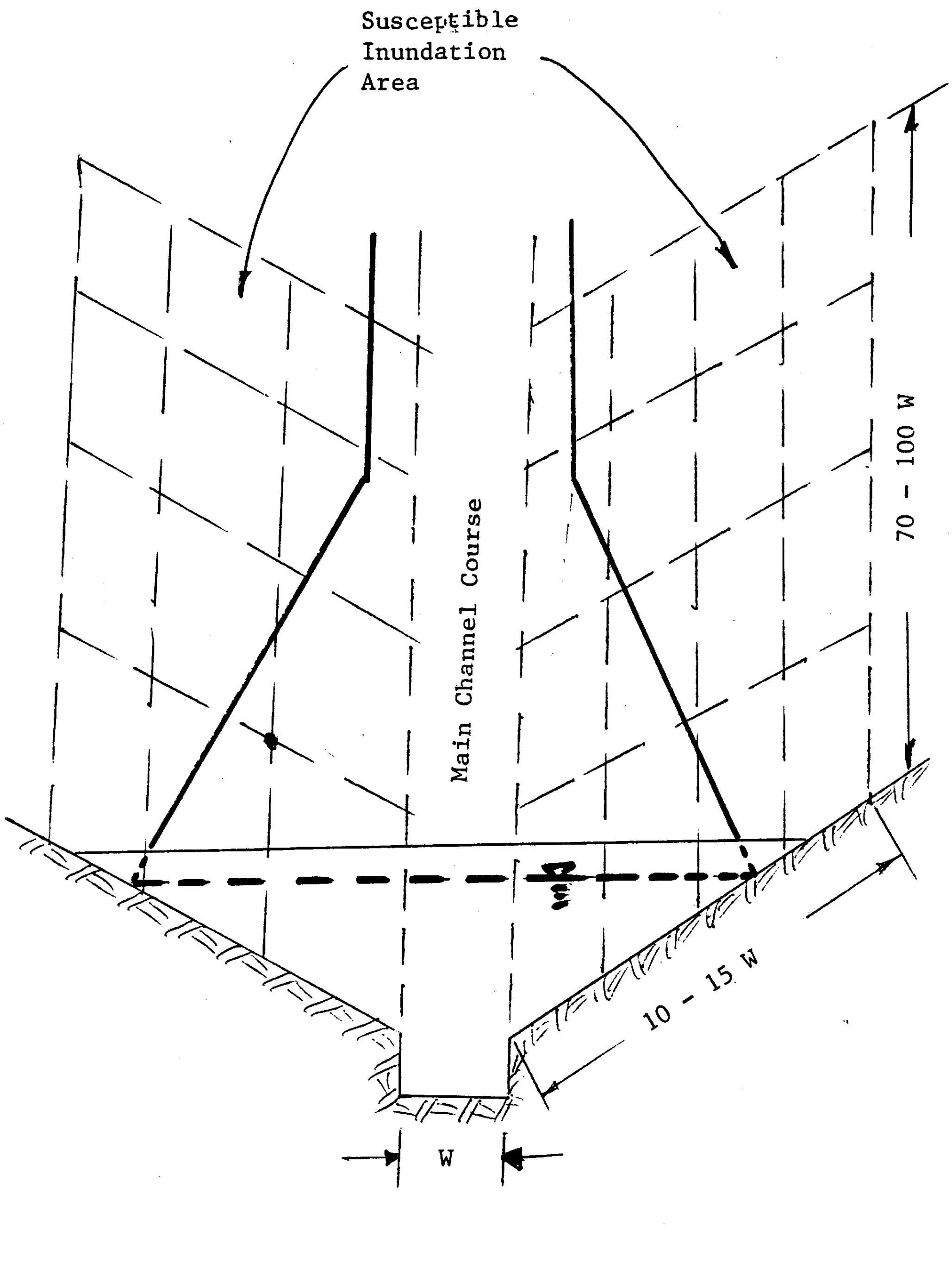


Figure 10.1 Schematic Sketch of Delineation of Susceptible Inundation Area.

density that is representative to Wyoming. In fact, the building density may not even be a constant for a given drainage site; it might increase with the distance away from the stream channel.

Assume that a hypothetical building density unique to Wyoming can be determined when this project resumes and from this the total number of buildings, regardless of their types, are generated. Their locations can then be determined as follows. First, the susceptible inundation area is divided into a number of strips parallel to the channel. The width of each strip can be 100-140 feet including the length of a house with yard and road. Again, a Poisson distribution based on building density on each strip is used to generate the number of hypothetical buildings in each strip. The actual number of buildings in each strip is determined by

$$N_i^* = I\left\{ N\, N_i / \sum_{i=1}^{M} N_i \right\} \tag{10.1}$$

where $I\{\ \}$ is the round-off integer for the value in $\{\ \}$, N_i^* is the actual building number in the i-th strip, N is the total number of buildings in the susceptible inundated area, N_i is the number of buildings generated for the i-th strip by simulation, and M is the total number of strips.

Second, once the number of buildings in each strip is determined, lines perpendicular to the channel are drawn which divide the entire susceptible inundation area into a number of blocks (Figure 10.1) with each block being considered as a potential land parcel in which one unit of a hypothetical building is to be accommodated. To determine exactly the location of buildings in each strip, it must be assumed that each building is equally likely to be located in each parcel in the designated strip. A random number is generated from a uniform distribution, U(0,1), for each parcel along a strip. The parcels associated with the highest N_i^* values are assigned, in each strip, with a building.

Upon determination of hypothetical building locations, their types have to be specified. To do so, a multinomial distribution can be applied. However, it would require the specification of the probability that each building type as listed in Table 5.4 could be selected. The value of the buildings and the contents of each may be obtained by consulting with real estate and insurance agencies.

11. GENERATING A DATABASE FOR DRAINAGE STRUCTURE DESIGN

Upon resuming this project, the hypothetical drainage sites representative to Wyoming and their economic variables will be generated and then classified into three groups within the three drainage structure types, i.e., pipe culvert, box culvert and bridge. The classification criteria will be based on the size of drainage area as described in section 4 (Table 4.3) for simplicity. Manning's roughness coefficient for the main channel as well as for the floodplain will be generated based on the information given in Chow's open channel hydraulics book or a similar tabulated set of values. Design specifications for embankment geometry and roadway width for drainage structures would be obtained from the WHD.

All these data would be used to define the input record for the culvert design system (CDS) and bridge program (HY-7). A flowchart illustrating the concept of the proposed approach to generate the data base for culvert design using CDS is shown on Figure 11.1. Implementation of the algorithm as shown on Figure 11.1 would be made on the

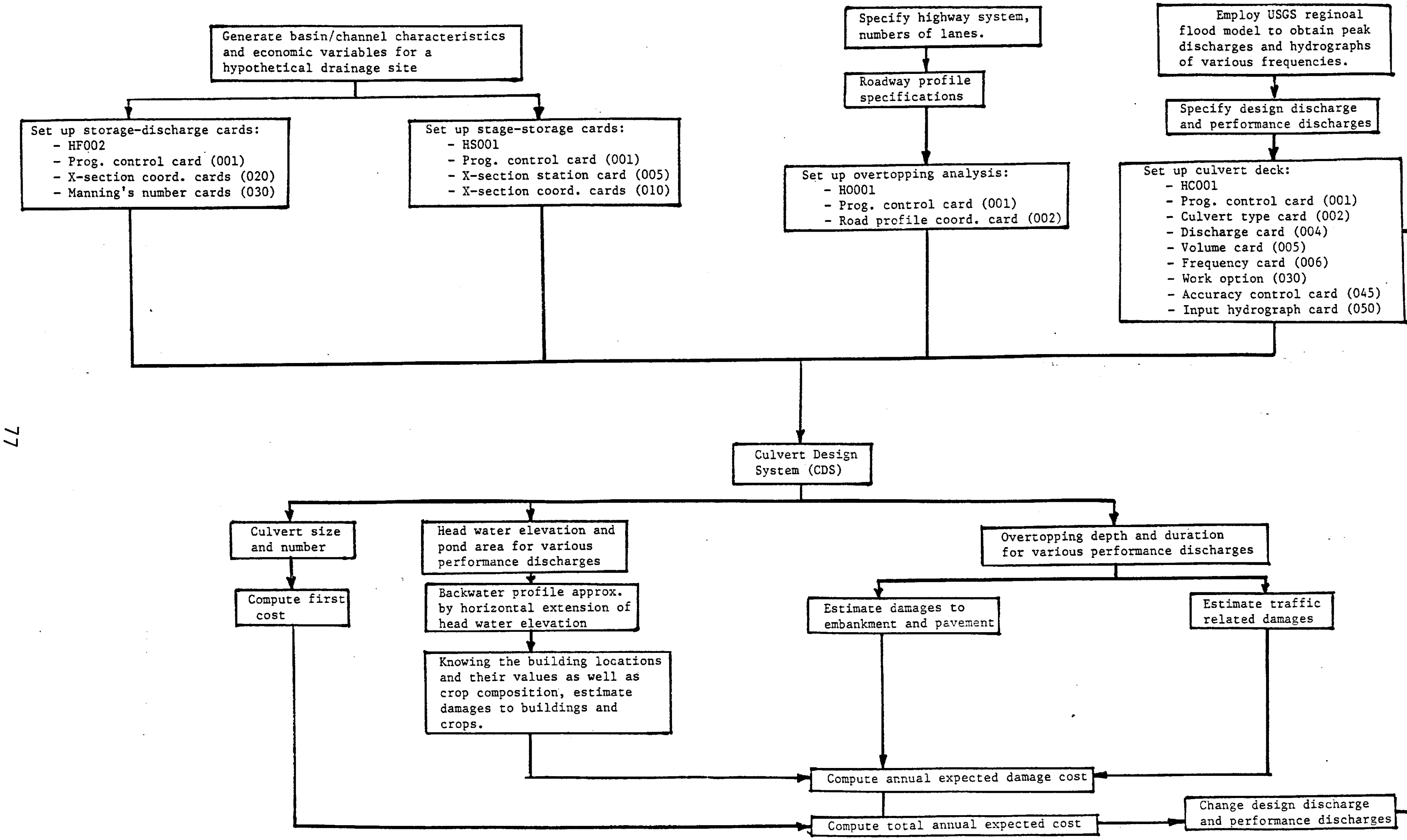


Figure 11.1. Flowchart of Generating Data Base for Drainage Structure Design

University of Wyoming mainframe computer for fast computation. Currently, the source code of CDS has been uploaded to the Cyber and has been made executable using the example data set given in Table 11.1. Both CDS and HY-7 would have to be moved to the new University VAX system or a larger and faster microcomputer (386 machine) if this project is continued. Channel geometry and roadway profile information corresponding to the data in Table 11.1 are shown on Figure 11.2.

12. DEVELOPING PROCEDURES FOR ASSESSING ANNUAL EXPECTED FLOOD DAMAGE

Strictly speaking, the annual expected flood damage cost in the LTEC design should be the incremental damage as the result of the presence of the roadway crossing. Annual expected flood damage for the preconstruction condition should be calculated. Area of inundation in the floodplain for the preconstruction can be estimated to correspond to the depth computed from the uniform flow equation. A schematic diagram illustrating the inundated areas before and after the construction for a given flood discharge is shown on Figure 12.1. The following theoretical description is condensed from Bao et al. [25].

$$E(D) = \int_{qo}^{\infty} D(q)\ f(q)\ dq \qquad (12.1)$$

where E(D) is the annual expected damage cost, q_o is the threshold discharge beyond which flood damage occurs, f(q) is the probability density function of the annual flood, and D(q) is the damage function consisting of the following four components,

$$D(q) = D_B(q) + D_C(q) + D_E(q) + D_T(q) \qquad (12.2)$$

Table 11.1 Example Data Set for CDS.

100 TEST DATA SET

HF002						
001	1.0	0.0	0.0	0.002		
020	0.0	1010.0	1000.0	1005.0	1000.0	1000.01
020	1050.0	1000.0	1050.0	1005.0	2050.0	1010.0
030	1000.0	0.050	1050.0	0.035	2050.0	0.050
HS001						
001	1.0	1.0		0.002		
005	5000.0					
010	0.0	1020.0	1000.0	1015.0	1000.0	1010.01
010	1050.0	1010.0	1050.0	1015.0	2050.0	1020.0
HO001						
001	2	1.0	10.0			
002	0.0	1010.0	2050.0	1010.0		
HC001						
001	30.0	1.0	0.0	0.0	100.0	0.002
002	51.0	0.0	0.0	0.0	0.0	
004	400.0	800.0	1200.0			
005	0.0	0.0	0.0			
006	25.0	50.0	100.0			
030	10.0	8.0				
045	30.0	0.0	0.0			
050	0.0	0.0	90.0	400.0	270.0	0.0
045	30.0	0.0	0.0			
050	0.0	0.0	120.0	800.0	360.0	0.0
045	30.0	0.0	0.0			
050	0.0	0.0	150.0	1200.0	480.0	0.0
999						

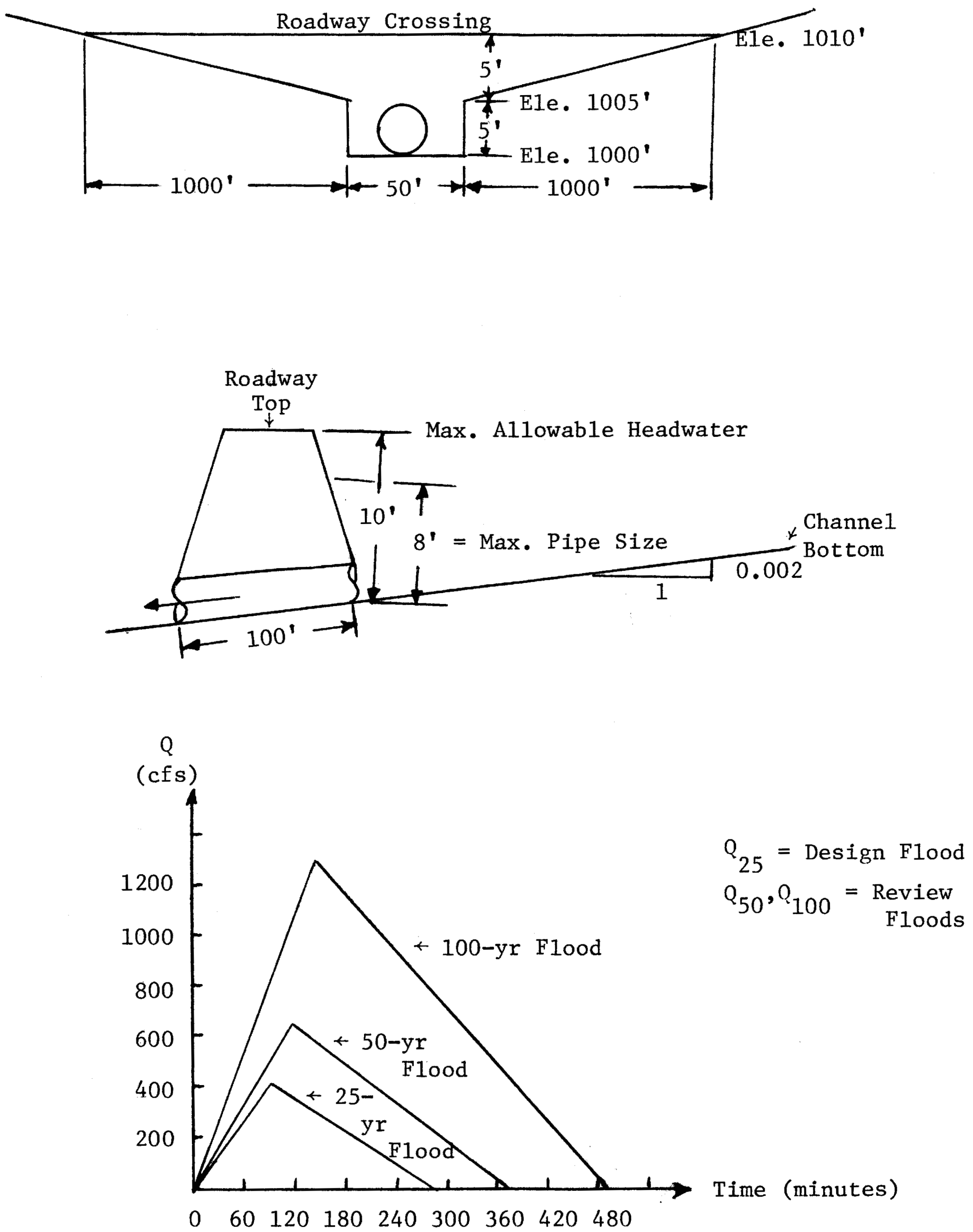


Figure 11.2. Geographical Sketches of Example Channel Cross-Section, Roadway Profile, and Hydrographs.

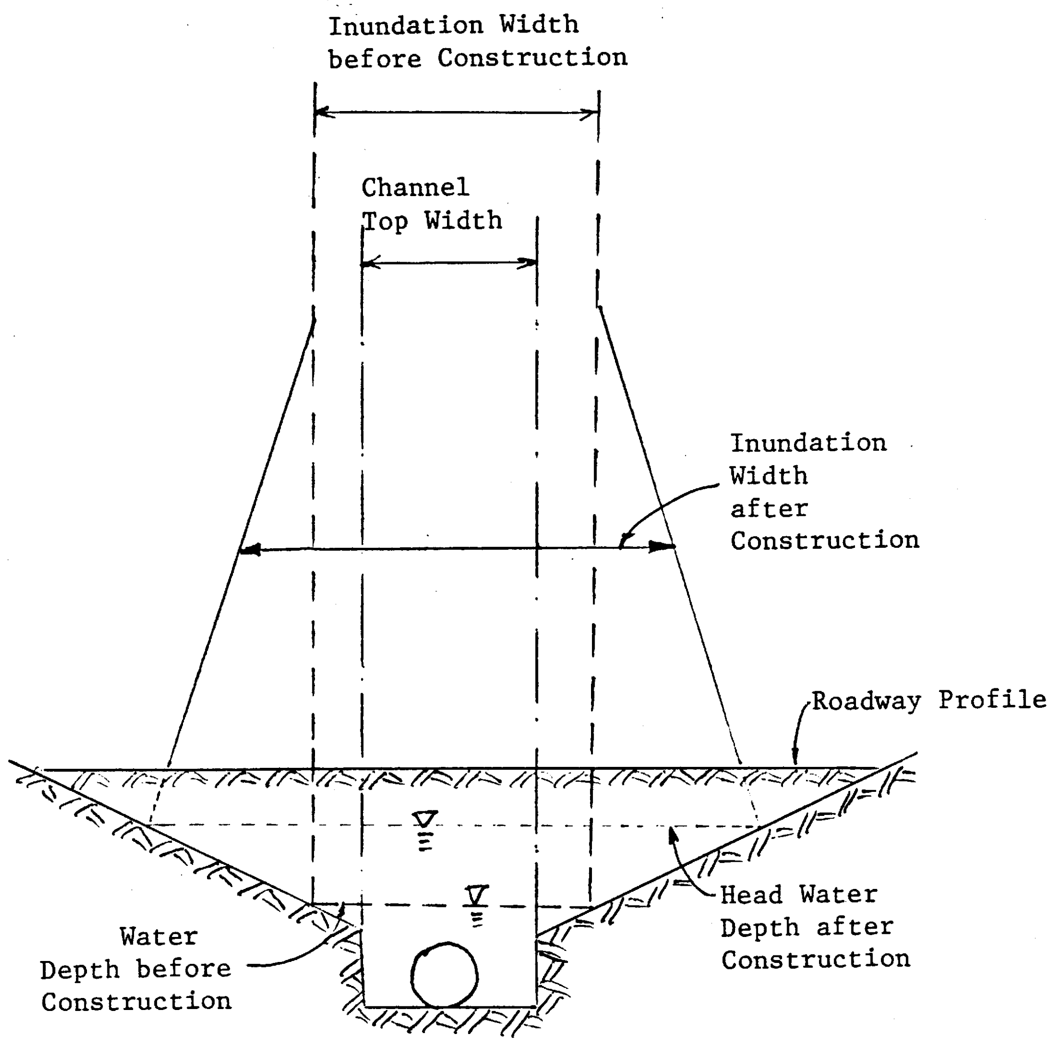


Figure 12.1. Schematic Diagram Showing the Inundated Area Before and After Construction.

in which $D_B(q)$ is the damage to buildings, $D_C(q)$ is the damage to crops, $D_E(q)$ is the damage to the embankment and pavement, and $D_T(q)$ is the traffic-related damage.

Using the regional flood frequency equations developed by the USGS, as they are developed for various discrete return periods, Eq. (12.1) can be replaced by

$$E(D) \approx \sum_i \overline{D_i(q)} \, \Delta F_i \tag{12.3}$$

where ΔF_i is the incremental probability for the i-th interval of the frequency scale and $\overline{D_i(q)}$ is the average damage for the i-th interval.

However, it must be recognized that flood magnitude obtained from the regional regression equations (USGS) are only estimates of the true but unknown discharges. They are associated with uncertainties indicated by the standard errors. As was developed by the USGS for Wyoming previously, the flood magnitude of return period T is related to basin physiographical characteristics in a multiplicative fashion,

$$Q_T = a_o \prod_{i=1}^{k} X_i^{a_i} \tag{12.4}$$

where the X_i's are basin characteristics and the a_i's are regression coefficients. In such cases, the standard errors associated with Eq. (12.4) in general are expressed in terms of the percentage errors. Under the normality assumption, flood discharge of a given return period can be considered to have a log-normal distribution. The uncertainty of discharges associated with the regional regression equations is shown in

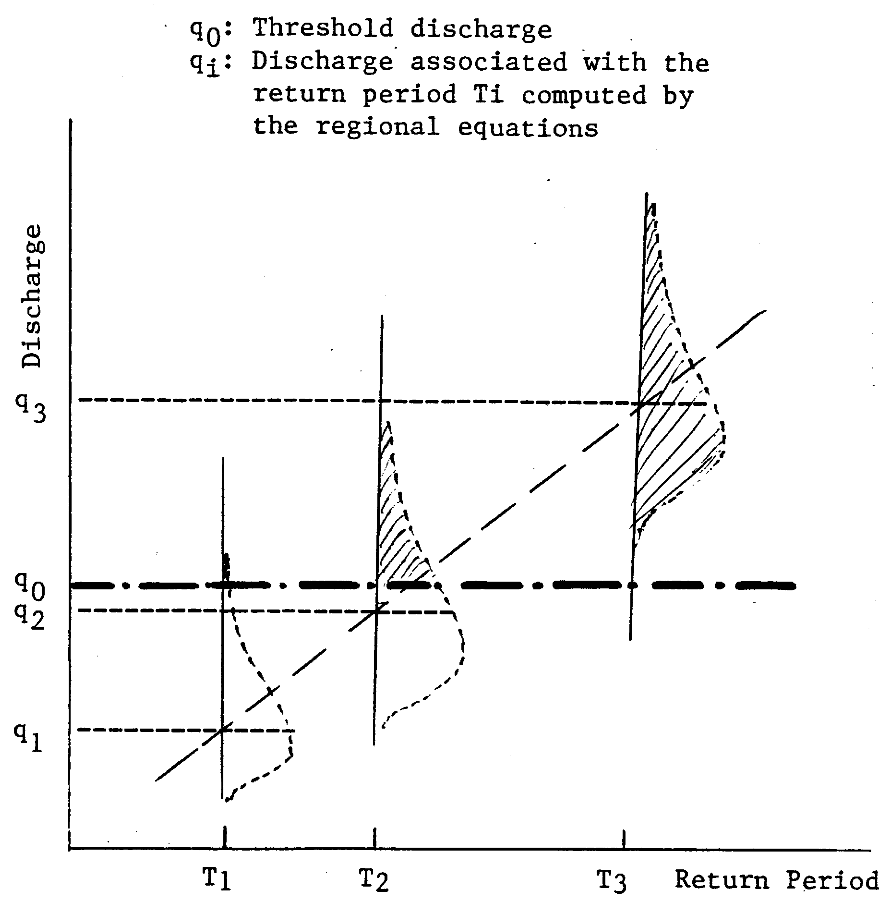


Figure 12.2 Illustration of Uncertainty in Regional Frequency Equations.

Figure 12.2. The flood discharge Q_T computed by Eq. (12.4) represents the median in the distribution of Q_T.

Therefore, for each return period T, there exists an expected damage associated with the encroachment of the floodplain,

$$E(D_T) \approx \sum_j \overline{D_j(q)}\ \Delta G_j \tag{12.5}$$

in which ΔG_j is the j-th incremental probability associated with the distribution of Q_T and D_T is the flood damage associated with the T-yr event. The annual expected flood damage cost, combining Eqs. (12.3) and (12.5), can be calculated by

$$E(D) \approx \sum_i \overline{E(D_{T_i})}\ \Delta F_i = \sum_i \overline{[\sum_j \overline{D_j(q)}\ \Delta G_j]}\ \Delta F_i \tag{12.6}$$

where $E(D_{T_i})$ is the average of the expected damage corresponding to the incremental frequency ΔF_i.

Equation (12.6) takes into account the uncertainty associated with Q_T from the regression model while Eq. (12.3) does not. In theory, Eq. (12.6) should yield a more accurate assessment of the actual annual expected flood damage. Furthermore, the annual expected flood damage calculated by Eq. (12.6) is always higher than that calculated by Eq. (12.3). Because Eq. (12.3) does not account for the uncertainty associated with Q_T, the annual expected damage calculated by it would always underestimate the actual value.

13. DESCRIPTIONS OF DEVELOPED COMPUTER SUBROUTINES FOR GENERATING HYPOTHETICAL DRAINAGE SITES AND FOR CALCULATING FLOOD RELATED DAMAGES

This section summarizes the developed simulation models for generating hypothetical site characteristics and the corresponding economic variables. Relevant information at a given drainage site for the LTEC design of a highway drainage structure can be generated by the following subroutines that have been completed to this point of the project:

1. "SIMSITE" - A simulation subroutine designed to generate physical basin/channel characteristics of a hypothetical drainage site representative of those in Wyoming. The site characteristics generated include basin area, channel slope, top width of main channel, width - depth ratio, floodplain slopes, Manning roughness coefficients of main channel and floodplain, and geographical factors.
2. "SIMTRAF" - A simulation routine for generating traffic conditions at the hypothetical drainage site. It produces information such as ADT, distance to nearest detour, accident ratio, road type, mobilization time, average vehicle speed, unit cost of accident, unit cost of occupancy, occupancy rate per vehicle, and vehicle composition.
3. "SIMCROP" - Developed to generate crop type in the floodplain susceptible to flood damage. Productivities and economical values of various crops are built into the program as the interval parameters.
4. "SIMEP" - Developed to generate information about the roadway crossing geometry such as embankment height, side-slopes of embankment, road surface condition (paved or not paved), type of vegetal cover, type of embankment base soil, flood condition, top width, width of pavement, thickness of pavement, unit cost of pavement, unit cost of embankment, and mobilization cost.

5. "SIMBLDG" - Simulate the number of buildings in the floodplain, their types, values, and locations.

The above five simulation modules have been developed for generating basin/channel characteristics as well as economic variables in a hypothetical drainage site and are operational. Other subroutines have also been developed to compute the various costs for the different drainage structure (bridge, box culvert or pipe culvert) designs. Table 13.1 lists the subroutines developed so far that are also operational and it briefly describes the function of each individual subroutine. To provide an overall picture for conceptualizing the methodological framework, Figure 13.1 is drawn to show the linkage among various subroutine modules in the task of the total research effort. Those that are cross-hatched are currently in place as of this time.

In the following subsections, a more detailed description about each subroutine module is given. Furthermore, procedures to quantify the parameters in each subroutine module are described.

13.1 Subroutine Module "SIMSITE":

"SIMSITE" was developed to generate basin/channel characteristics of hypothetical drainage sites representative of Wyoming. It was developed in accordance with the new USGS regional flood study [26] in that the State of Wyoming is divided into three hydrological regions:

(1) Mountainous Region

Features:
- small peakflow
- large annual runoff
- contributed by snowmelt

Table 13.1. List of Subroutines Developed.

No.	Name	Functions
1	AREA	Calculating the inundation area of upstream floodplains and average elevation of inundation area
2	FSTC	Computing the installation cost in LTEC design of highway drainage structures (bridge, box culvert, or pipe culvert)
3	DAMTRAF	To compute traffic-related losses due to flooding
4	DAMEP	To compute flood damage to embankment and pavement
5	DAMBLDG	To compute flood damage to buildings
6	DAMCROP	To compute flood damage to crops
7	SIMSITE	Simulation to generate database for hypothetical drainage basin in the State of Wyoming
8	SIMTRAF	To generate database for calculating the traffic-related damage due to the flooding by simulation
9	SIMCROP	To generate database for calculating the flood damage to crops by simulation
10	SIMEP	To generate the database for calculating the flood damage to embankment and pavement
11	SIMBLDG	To generate the database for calculating the flood damage to the buildings
12	NMLDEP	To compute the normal depth of water in the main channel under natural condition

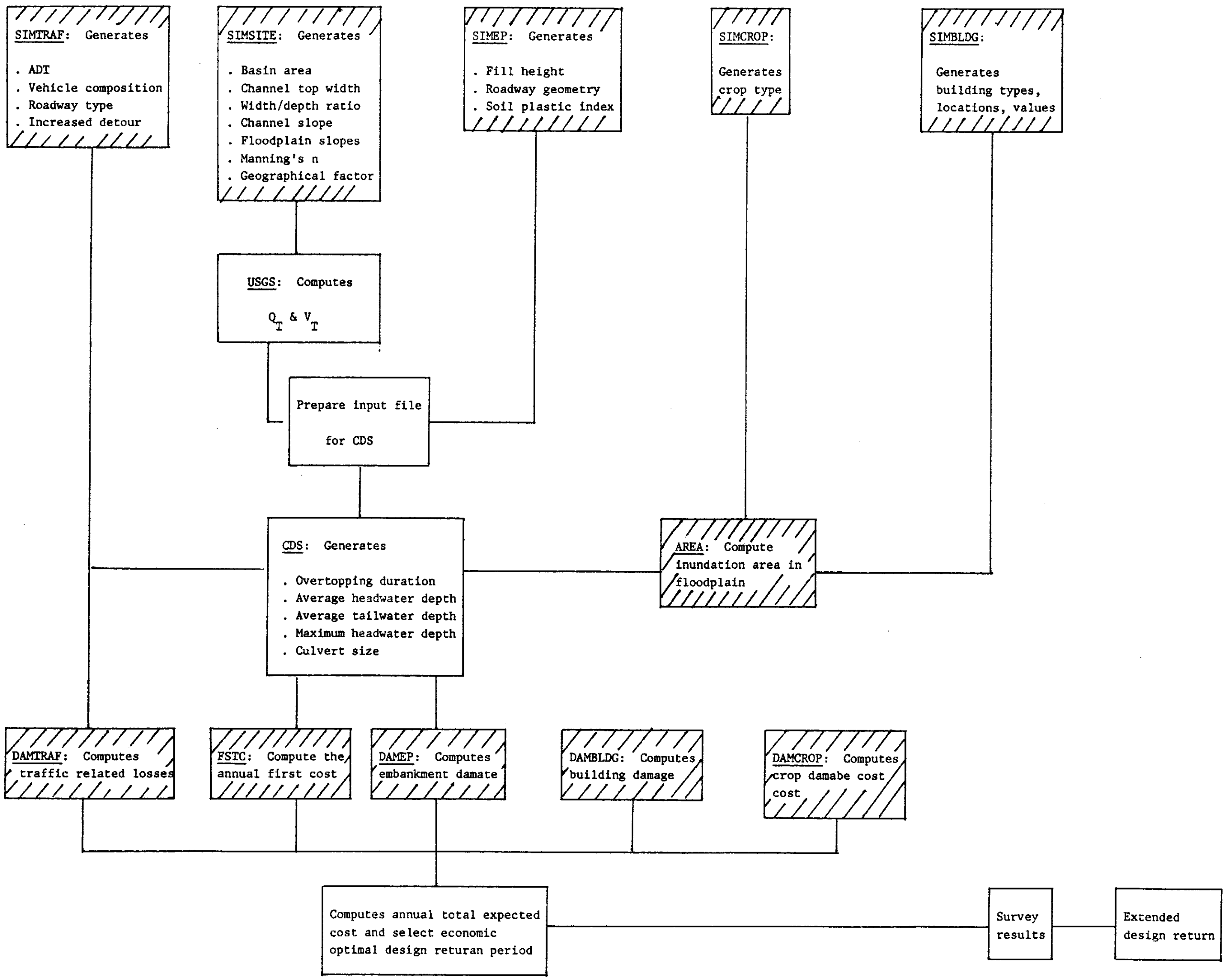


Figure 13.1. Simulation Model Flowchart for Total Project.

(2) Plains Region (Northern and Eastern Plains and Deserts)

Features: - high peak flow

- varies from year-to-year

- contributed by rain storm

(3) High Desert Region (South-central and Southwestern Plains and Desert

Features: - peak flow is smaller than region (2)

- contributed by wide spread rain storms and snow

In the recent USGS regional study [26], two methods are used in developing regional flood frequency relations: (a) Basin-Characteristic method and (b) Channel-Geometry method. Forms of regression equations that relate flood magnitude to basin or channel characteristics are listed in Table 13.2. In this research, the Channel-Geometry method was used in subroutine "SIMSITE". The regional hydrological variables considered were: (1) A: drainage area, (2) W: main channel top width, (3) WD: width-depth ratio, (4) SR: floodplain slope on the right-hand side, (5) SL: floodplain slope on the left-hand side, (6) SB: basin slope, and (7) GF: geographic factor.

For Mountainous region variables, A and W were generated by a multivariate normal random number generator on a log-transformed scale while WD, SR, SL, and SB were generated by multiple regression on A and W. Similarly, for the Plains Region, A, W, GF and SB were generated by a multivariate normal random number generator while SR, SL, and WD were generated by multiple regression equations. For the High Desert Region, A, W, and GF were generated by a multivariate normal random number generator while the remaining characteristics were calculated by the regression equations. The database for assessing statistical properties

Table 13.2 Regression Equations for Estimating Flood Flows in the State of Wyoming.

Regions		Basin-Characteristic Method	Channel-Geometry Method
Mountainous		Pt = f(A, ELEV)*	Pt = f(W)
	or	Pt = f(A, PR)	
Plains		Pt = f(A, SB, GF)	Pt = f(W, GF)
High Desert		Pt = f(A, PR, GF)	Pt = f(W, GF)

* Pt = annual peak flow with t-yr return period (cfs)

A = contributing drainage basin area (sq. miles)

ELEV = mean basin elevation (ft)

PR = average annual precipitation (inches)

SB = drainage basin slope (ft/mile)

GF = geographic factor

W = main-channel width (ft)

Table 13.3. Statistics of Site Characteristics in Mountainous Region.

(a) Summary of Statistics:

	ln(A)	ln(W)
Mean	4.20	3.491
Stdev	1.40	0.532
Max	7.05	5.193
Min	-0.65	0.693

(b) Correlation:

(ln(A), ln(W)) = 0.836

(c) Regression Equations:

Site* Variables	Regression Equations	r	Se
WD	= exp(0.352 + 0.567 ln(W) - 0.373 ln (A))	0.762	0.4812
SR	= exp(-1.6 - 0.338 ln(W) - 0.166 ln(A))	0.455	1.188
SL	= exp(-2.17 - 0.238 ln(W) - 0.113 ln(A))	0.30	1.333
SB	= exp(7.61 - 0.46 ln(W) - 0.142 ln(A))	0.615	0.8674

*See Table 13.6 for descriptions of the site variables.

additional information from the recent USGS regional study [26]. Statistics of site characteristics and the regression equation developed for the three hydrologic regions were given in Tables 13.3 through 13.5. Sample printouts from "SIMSITE" are shown in Tables 13.6 through 3.8.

13.2 Subroutines "SIMEP" and "DAMEP":

Subroutine "SIMEP" was developed to generate characteristics of roadway crossing geometry and the surface properties of the highway embankment. Variables describing the roadway crossing geometry are fill height, side slopes of embankment, and embankment top width. Investigation of the relation between fill height and other channel characteristics such as channel depth or width based on various actual sites [10] failed to identify the existence of such a relationship. Embankment side slopes were determined based on the design standard stated in reference [27]. The top width of an embankment was dependent on the number of traffic lanes on the roadway crossing. In the program to generate the hypothetical roadway geometry, it was assumed that four lanes are used for interstate highways, and two lanes for other highway systems. It may be more reasonable to relate the number of traffic lanes to the ADT if this project is continued rather than the assumption presently being made. The width of each traffic lane was 12 feet and the right shoulder was 10 feet for interstate, 3 to 8 for other highways.

In addition to roadway crossing geometry, soil property represented by the plastic index (PI) was generated in this subroutine for the purpose of calculating average erosion rate for the base soil of the embankment. A series of curves to estimate total volume of

Table 13.4 Statistics of Site Characteristics in Plains Region

(a) Summary of Statistics:

	ln(A)	ln(W)	ln(SB)	ln(GF)
Mean	2.07	2.750	6.513	0.165
Stdev	1.87	0.685	0.465	0.239
Max	6.25	4.190	7.307	0.470
Min	-0.43	1.792	5.476	-0.223

(b) Correlation Matrix:

	ln(A)	ln(W)	ln(SB)	ln(GF)
ln (A)	1.0			
ln (W)	0.752	1.0		
ln (SB)	-0.053	-0.072	1.0	
ln (GF)	-0.317	-0.065	0.212	1.0

(c) Regression Equations:

Regression Equations	r	Se
$WD = \exp(.111+0.584 \ln(W)-.0318 \ln(A)+.0387 \ln(SB))$	0.763	0.4819
$SR = \exp(-5.86-0.022 \ln(W)-0.0691 \ln(A)+.687 \ln(SB))$	0.637	1.032
$SL = \exp(-7.50-0.157 \ln(W)-.0085 \ln(A)+.859 \ln(SB))$	0.612	1.109

Table 13.5. Statistics of Site Characteristics in High Desert Region.

(a) Summary of Statistics:

	ln(A)	ln(W)	ln(GF)
Mean	3.30	3.006	0.152
Stdev	2.77	0.834	0.234
Max	8.57	4.787	0.470
Min	-0.43	1.792	-0.223

(b) Correlation Matrix:

	ln(A)	ln(W)	ln(GF)
ln(A)	1.0		
ln(W)	0.781	1.0	
ln(GF)	-0.217	-0.003	1.0

(c) Regression Equations:

Regression Equations	r	Se
WD = exp(0.352 + 0.567 ln(W) - 0.0373 ln(A))	0.762	0.4812
SR = exp(-1.6 - 0.338 ln(W) - 0.166 ln(A))	0.455	1.188
SL = exp(-2.17 - 0.238 ln(W) - 0.113 ln(A))	0.30	1.133
SB = exp(7.61 - 0.46 ln(W) - 0.142 ln(A))	0.615	0.8674

Table 13.6. 20 samples of hypothetical drainage basin generated by subroutine "SIMSITE" for mountainous region in the State of Wyoming

A	W	WD	SR	SL	SB
46.68	27.75	9.92	.0479	.0587	393.03
4.35	27.70	15.28	.0037	.4807	757.73
2082.44	187.38	37.25	.0175	.0031	305.05
2047.39	237.55	22.67	.0071	.0227	385.73
19.34	41.52	7.70	.0312	.0059	409.74
145.85	23.00	10.26	.0968	.0421	136.26
14.24	33.64	10.64	.0722	.0027	117.05
49.90	23.55	5.52	.1462	.0401	231.97
486.56	56.46	7.26	.0392	.0568	265.27
107.86	37.25	4.13	.1269	.0358	450.07
17.11	19.97	9.70	.1224	.0029	117.39
9.59	35.24	22.62	.0282	.1792	353.79
34.32	22.11	8.37	.0135	.0641	250.84
313.55	56.24	16.56	.0093	.0973	116.66
1.85	11.85	5.00	.1717	.3140	390.25
26.65	35.68	10.23	.0912	.0025	416.74
390.95	59.88	16.73	.0021	.0031	364.43
5.97	28.48	11.99	.0785	.0553	641.18
7.82	7.44	2.69	.1107	.0534	720.99
210.17	66.26	15.36	.2057	.1025	207.00

Regional hydrological parameters:

A = drainage basin area (sq. miles)

W = channel top width (ft)

WD = ratio of channel width to depth (ft/ft)

SR = floodplain slope right hand side (ft/ft)

SL = floodplain slope left hand side (ft/ft)

SB = drainage basin slope (ft/mile)

Table 13.7 15 samples of hypothetical drainage basin generated by subroutine "SIMSITE" for the plains region in the State of Wyoming

A	W	GF	WD	SR	SL	SB
3.39	20.72	1.15	9.93	.2825	.3736	666.01
116.52	29.57	1.21	21.40	.4772	.0971	836.57
.42	14.75	1.22	5.23	.2466	.0595	762.69
39.33	9.72	1.16	5.41	.0995	.2940	705.28
.20	8.78	1.23	8.06	.0792	.7332	745.03
.02	10.75	1.23	6.88	.2728	.2133	732.42
24.37	18.26	1.18	10.08	.2121	.0382	738.94
.31	19.73	1.19	11.82	.5902	.0266	663.20
60.05	22.27	1.16	13.51	.2136	.0982	611.46
9676.25	37.99	1.15	7.98	.0758	.5691	644.68
20.20	9.03	1.19	6.22	.2638	.1659	728.50
536.35	29.12	1.16	7.18	.3000	.5642	681.83
.03	8.62	1.22	6.29	.6893	.0850	711.36
122.62	28.67	1.18	18.66	.1070	.1195	812.79
.01	6.47	1.20	6.07	.7697	.0391	705.56

Regional hydrological parameters:

A = drainage basin area (sq. miles)

W = channel top width (ft)

GF = geographic factor

WD = ratio of channel width to depth (ft/ft)

SR = floodplain slope right hand side (ft/ft)

SL = floodplain slope left hand side (ft/ft)

SB = drainage basin slope (ft/mile)

Table 13.8. 20 samples of hypothetical drainage basin generated by subroutine "SIMSITE" for the high desert region in the State of Wyoming.

A	W	GF	WD	SR	SL	SB
6.91	12.34	.96	6.73	.0865	.0883	748.45
80.45	13.58	.93	1.83	.3408	.1196	137.52
.02	9.33	.94	3.37	.2398	.2031	227.55
.29	13.85	.96	10.91	.1062	.0332	1542.66
54.22	16.16	.95	5.44	.0358	.0190	145.14
2.79	21.41	.96	11.58	.0058	.0096	277.29
134.38	13.06	.93	8.91	.0506	.0139	863.18
.22	13.07	.96	5.91	.0816	.0411	1533.48
3.06	14.90	.97	8.11	.2906	.1122	616.06
3857.18	12.96	.91	3.54	.0046	.0210	217.73
.03	9.02	.97	4.81	.3786	.2792	247.43
.04	10.71	.95	8.27	.2469	.1216	2020.09
280.84	37.98	.97	10.73	.0221	.0338	654.87
.21	5.35	.93	4.90	.2439	.2624	306.22
83.47	21.78	.92	10.74	.5276	.0193	203.31
4682.50	25.35	.95	2.96	.0826	.1174	249.43
750.07	10.77	.90	4.86	.0032	.3990	215.65
1.22	12.81	.95	7.92	.0391	.1095	290.57
.09	3.44	.97	4.20	.1188	.9381	523.32
.01	3.89	.95	1.97	.4856	.1932	1699.67

Regional hydrological parameters:

A = drainage basin area (sq. miles)

W = channel top width (ft)

GF = geographic factor

WD = ratio of channel width to depth (ft/ft)

SR = floodplain slope right hand side (ft/ft)

SL = floodplain slope left hand side (ft/ft)

SB = drainage basin slope (ft/mile)

embankment erosion are given by Chen and Anderson [28]. These curves were digitized and stored in the subroutine "DAMEP" to estimate damage to the embankment and pavement. Other variables required to estimate erosion loss of the embankment such as overtop depth, tailwater/ headwater ratio, and overtopping duration can be obtained from CDS. Computation procedures for calculating embankment damage are shown in Figure 13.2.

13.3 Subroutines "SIMBLDG" and "DAMBLDG":

Subroutine "SIMBLDG" generates the number of hypothetical buildings, their locations, values, and types in floodplains susceptible to inundation. The procedures for generating building numbers and their locations using the Poisson distribution were described in Section 10. It requires, however, the knowledge of building density at the site. Furthermore, for an urban drainage basin, the composition of residential and commercial buildings needs to be known. The following two subsections briefly describe the process used to develop a rough idea of these parameters.

(13.3.1) Determination of Building Density (Rural and Urban) in the State of Wyoming. One hundred forty five 7.5-minute topographical maps in Wyoming were randomly selected to estimate the building density. A total of 1,323 rural drainage sites and 22 urban drainage sites were encountered. Building density is summarized in Table 13.9. It can be seen from Table 13.9 that the average building density is about the same as the variance, therefore, the Poisson distribution is justified to generate the number of buildings.

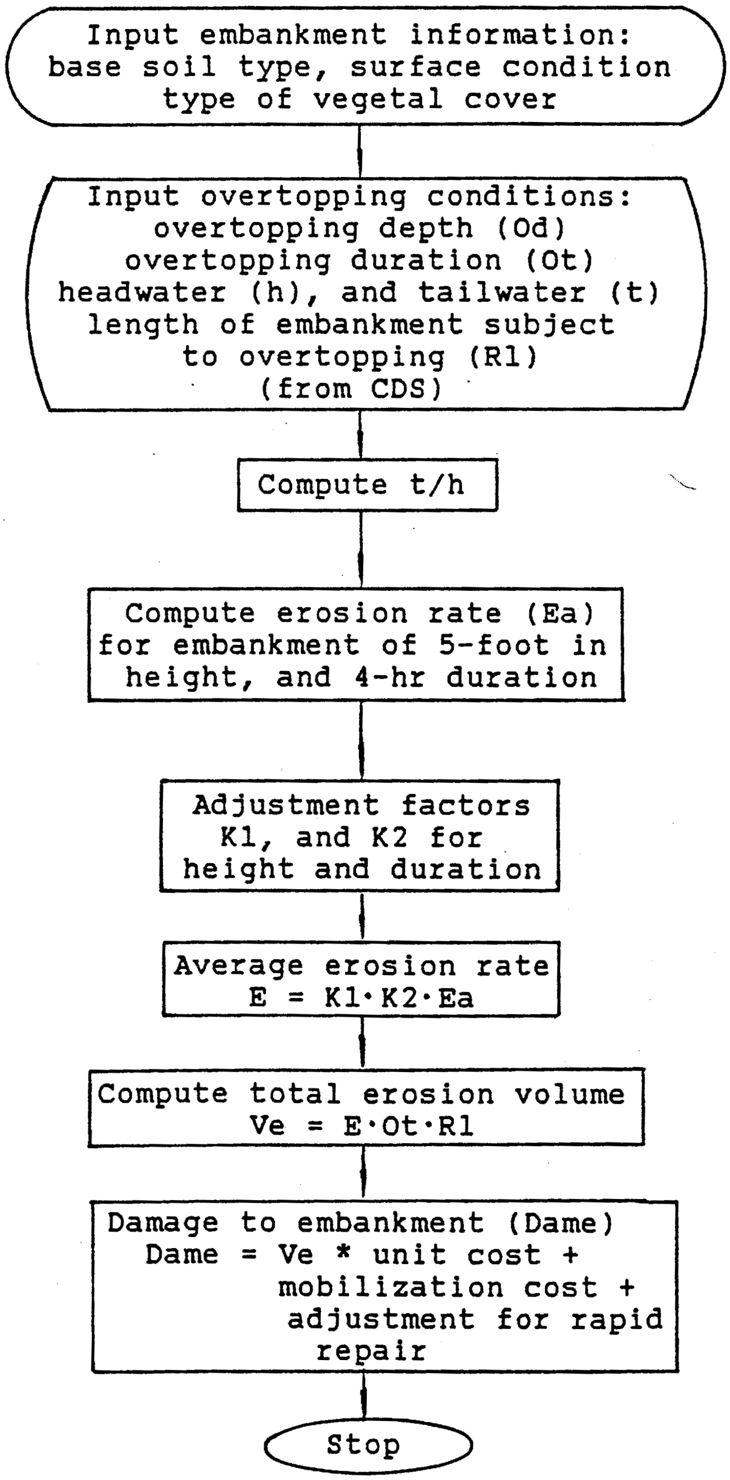


Figure 13.2. Flowchart for Calculating Flood Damage to Embankment and Pavement.

Table 13.9. Building density in the State of Wyoming.*

	Rural	Urban
Percentage of bldg occupants on site	6.80%	100%
Average bldg density	2.233 bldgs/site	2.01 bldgs/acre
Standard deviation	1.423	1.21

*The topographic maps investigated are compiled at different times. The average year of the maps publication is 1961 with a standard deviation of about six years.

(13.3.2) Determination of Ratio of Residential to Commercial Buildings in the State of Wyoming. Buildings are divided into residential and commercial types, in general, with the main subcategories listed in Table 13.10. For generating a hypothetical database for an urban drainage basin, the percentage of residential and commercial buildings in the State of Wyoming needs to be determined. Twenty-one cities in Wyoming as shown in Table 13.11 were selected from Mountain Bell phone books. For each city in the phone book, about 20 to 30 percent of its pages were randomly selected as the sample set to estimate the average number of residential buildings and commercial buildings per page. The total number of residential and commercial buildings of the city were then estimated by multiplying the average number per page by the number of total pages for that city. The percentages of residential and/or commercial buildings of the city were then calculated. The weighted average percentages, accordingly, for Wyoming were calculated by using the sum of residential and/or commercial buildings from the 21 cities (Table 13.11).

Once the number of buildings (regardless of their types) for a given hypothetical site is generated, the number of residential and commercial buildings is determined according to the ratio given in Table 13.11. As far as the hypothetical building type is concerned, a multinomial distribution was used to generate the type of residential building and commercial business. A sample output from "SIMBLDG" is given in Table 13.12. A flowchart showing the logic in "SIMBLDG" is given in Figure 13.3. The classification of the commercial buildings and their contents were cited from [29]. The classification of residential buildings is the same as in section 5. The data on the percent of

Table 13.10 Building Code Index.

Bldg Type	Description	Bldg Type	Description
Residential		Commercial	
1	One story w/o basement	34	Gas Company*
2	Two story w/o basement	35	Garage
3	Two story w/ basement	36	Greenhouse
4	Split level w/o basement	37	Grocery Store
5	Split level w/ basement	38	Grocery Store (Kwik)
6	Mobile home	39	Gift Shop
Commercial		40	Gun shop
1	Antique shop	41	Hall
2	Appliance shop	42	Hardware
3	Auto dealer	43	Hobby shop
4	Auto junkyard	44	Hotel
5	Auto parts	45	Jewelry
6	Auto repair	46	Laundry
7	Auto transmission service	47	Library
8	Auto muffler service	48	Liquor store
9	Bakery	49	Lumber yard
10	Bank	50	Meat market
11	Barber shop	51	Motel
12	Beauty shop	52	Music store
13	Boat store	53	Newspaper printing
14	Bowling alley	54	Nursing home
15	Book store	55	Nursery (plant)
16	Business (general)	56	Office building
17	Church	57	Plumbing supply
18	City hall	58	Police station
19	Cleaners	59	Post office
20	Clinic (medical)	60	Private club
21	Construction company	61	Real estate office
22	Country club	62	Radio station
23	Clothing	63	Restaurant
24	Dentist's office	64	Restaurant drive-in
25	Department store	65	School
26	Doctor's office	66	Tavern
27	Drug store	67	Theater
28	Fire Station	68	Transport company
29	Flooring and carpeting	69	Trailer Sales
30	Florist	70	Television repair
31	Food processor	71	Variety store
32	Funeral home	72	Warehouse
33	Furniture	73	Welding supply

*No data available for the percent damage of the contents.

Table 13.11. Estimation of Ratio of Residential Buildings to Commercial Buildings in the State of Wyoming.

No.	City	Estimate of Resid Bldg	Estimate of Commer Bldg	% of Resid Bldg	% of Commer Bldg
1	Buffalo	2,006	566	78.0	22.0
2	Casper	23,125	6,726	77.5	22.5
3	Cheyenne	23,631	6,031	79.7	20.3
4	Cody	5,110	1,156	81.6	18.4
5	Douglas	2,598	947	73.3	26.7
6	Evanston	4,557	984	82.2	17.8
7	Gillette	9,434	1,839	83.7	16.3
8	Green River	4,221	537	88.7	11.3
9	Kemmerer	1,594	473	77.1	22.9
10	Lander	3,823	986	79.5	20.5
11	Laramie	11,645	1,717	87.2	12.8
12	Newcastle	1,953	539	78.4	21.6
13	Powell	3,168	686	82.2	17.8
14	Rawlins	3,392	702	82.9	17.1
15	Riverton	5,096	1,323	79.4	20.6
16	Rock River	180	35	83.7	16.3
17	Rock Springs	8,222	1,798	82.1	17.9
18	Saratoga	770	187	80.5	19.5
19	Sheridan	8,700	1,842	82.5	17.5
20	Thermopolis	2,184	411	84.2	15.8
21	Worland	2,802	694	80.1	19.9
Weighted Average for Wyoming		128,211	30,179	80.9	19.1

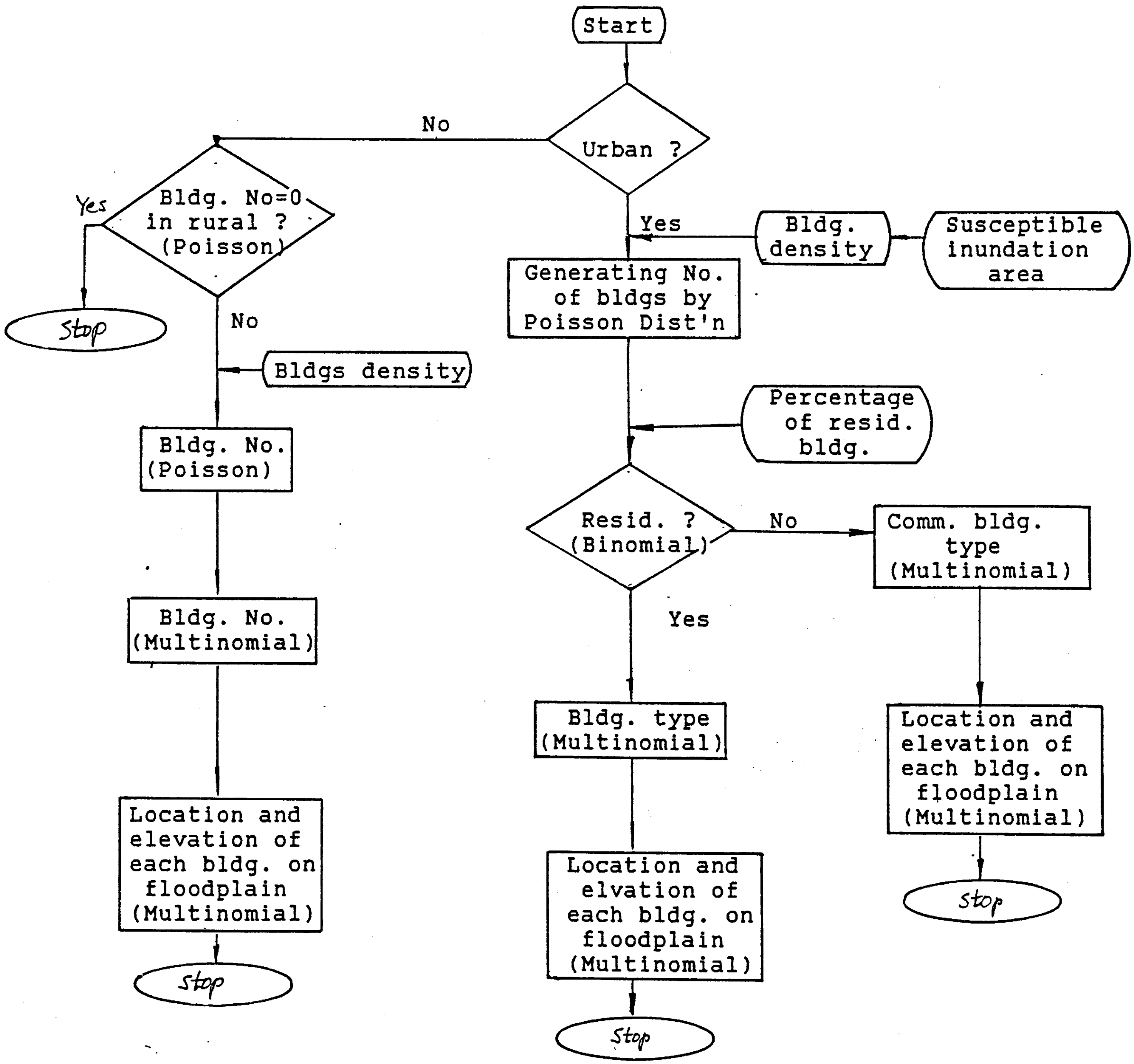


Figure 13.3. Flowchart of SIMBLDG.

Table 13.12. Sample of Subroutine "SIMBLDG" - Generating No. of Buildings on Floodplain.

Area (Sq Mi)	Width (ft)	Width to Depth Ratio	Floodplain Slope Left (ft/ft)	Floodplain Slope Right (ft/ft)	Basin Slope (ft/mi)
13.52	19.61	10.19	.2307	.1016	459.03

Bankfull Elevation (ft)	Number of Buildings	
	Residential	Commercial
1000.00	16	3

Residential Bldg Type	Elevation	Commercial Bldg Type	Elevation
3	1023.07	38	1080.92
2	1000.00	28	1083.90
1	1020.32	7	1072.22
4	1054.83		
2	1040.46		
1	1040.46		
5	1036.24		
6	1034.78		
6	1055.10		
1	1043.47		
1	1063.79		
1	1063.79		
1	1060.86		
4	1083.93		
5	1115.69		
6	1079.71		

damage by flood to 73 types of commercial buildings shown in Table 13.13 were used as the internal database in the program. The ratio of content value to the building was also cited from [29] and is shown in Table 13.14. The equation in subroutine "DAMBLDG" to calculate the flood damage to a building in the floodplain is

$$D_b(q) = \sum_i VR_i[PR_i(q) + RR_i*CR_i(q)] + \sum_i VC_i[PC_i(q) + RC_i(q)] \qquad (13.1)$$

where VR_i is the value of residential building No. i

PR_i is the percent of damage to residential building No. i

PR_i is the ratio of content value to the building value for residential building i

CR_i (q) is the percent of damage to residential contents of building i

VC_j is the value of commercial building j

PC_j (q) is the percent of damage to commercial building j

RC_j is the ratio of contents value to building value for commercial building j

CC_j (q) is the percent of damage to contents of commercial building j

13.4 Subroutines "SIMCROP" and "DAMCROP":

When generating crop type by "SIMCROP" only one type of crop was generated for each site. In estimating crop damage due to flooding, eight different crops including (1) wheat, (2) barley, (3) oats, (4) dry beans, (5) sugar beets, (6) alfalfa hay, (7) other hay, and (8) corn-grain/corn-silage were considered.

Flood damage to a crop is dependent on inundation depth as well as duration. Later when this research is continued, it is proposed to use averaged inundation depth from the CDS in the damage calculation.

Table 13.13. Building Code Index.

Bldg Type	Description	Bldg Type	Description
Residential		Commercial	
1	One story w/o basement	34	Gas Company*
2	Two story w/o basement	35	Garage
3	Two story w/ basement	36	Greenhouse
4	Split level w/o basement	37	Grocery Store
5	Split level w/ basement	38	Grocery Store (Kwik)
6	Mobile home	39	Gift Shop
Commercial		40	Gun shop
1	Antique shop	41	Hall
2	Appliance shop	42	Hardware
3	Auto dealer	43	Hobby shop
4	Auto junkyard	44	Hotel
5	Auto parts	45	Jewelry
6	Auto repair	46	Laundry
7	Auto transmission service	47	Library
8	Auto muffler service	48	Liquor store
9	Bakery	49	Lumber yard
10	Bank	50	Meat market
11	Barber shop	51	Motel
12	Beauty shop	52	Music store
13	Boat store	53	Newspaper printing
14	Bowling alley	54	Nursing home
15	Book store	55	Nursery (plant)
16	Business (general)	56	Office building
17	Church	57	Plumbing supply
18	City hall	58	Police station
19	Cleaners	59	Post office
20	Clinic (medical)	60	Private club
21	Construction company	61	Real estate office
22	Country club	62	Radio station
23	Clothing	63	Restaurant
24	Dentist's office	64	Restaurant drive-in
25	Department store	65	School
26	Doctor's office	66	Tavern
27	Drug store	67	Theater
28	Fire Station	68	Transport company
29	Flooring and carpeting	69	Trailer Sales
30	Florist	70	Television repair
31	Food processor	71	Variety store
32	Funeral home	72	Warehouse
33	Furniture	73	Welding supply

*No data available for the percent damage of the contents.

Table 13.14. Ratio of Content Value to its Building Value.

Building category	Ratio of content value to the building value
Residential	0.50
Retail	1.00
Schools and churches	0.70
Offices	0.65
Auto services	0.60
Manufacturing	0.40
Warehousing (light)	1.50
Warehousing (heavy)	0.65

13.5 Subroutines "SIMTRAF" and "DAMTRAF":

In "SIMTRAF", type of road, increased travel distance due to detour, average daily traffic equivalent (ADTE) and vehicle composition were generated. Types of roadway systems include interstate, primary, secondary, state highway only, service road, and urban street system. Presently in the subroutine, the increased travel distance of a detour was assumed to be 1-10 miles in urban areas and 0-50 miles in rural areas which is probably not very representative and will need revision. Vehicle types were classified according to their sizes [18], i.e. large (for trailer trucks and buses), intermediate (eg. pickup), compact/ subcompact (passenger cars), and van.

From the available information [19], the ADTE in Wyoming is summarized in Table 13.15. Appropriate bounds will be placed (open to suggestion) so that random ADTE can be generated for a hypothetical site.

So far, the investigators have not found a useful publication that provides vehicle composition on Wyoming highways. An estimation can possibly be made based on traffic accident reports [19]. Estimation of vehicle composition based on the comprehensive accident report is valid if the assumption that the percentage of accidents of a specific vehicle type is directly proportional to the number of vehicles of that type on the road and each vehicle of a given type is equally likely to have an accident. Based on this assumption, the vehicle composition on Wyoming highways is estimated and is shown in Table 13.16.

Using the same assumption as above for estimating the vehicle composition, the vehicle occupant composition can be estimated in a similar manner. According to occupation classification in [20], the

Table 13.15. Equivalent Average Daily Traffic in the State of Wyoming.

	ADTE	
Road Type	Rural Area	Urban Area
Interstate	4,360	7,796
Primary	1,294	8,944
Secondary	486	4,210
State highway only	356	2,345
Service road	242	1,591
Urban system	---	5,428

Table 13.16. Percentage of Vehicle Type on Wyoming Highways.

	Vehicle Composition (%)										
	Road Type										
	Rural					Urban					
Vehicle Type	*A	B	C	D	E	A	B	C	D	E	F
+1	31.4	17.9	14.2	16.6	30.0	19.4	5.5	4.8	3.4	8.2	4.5
2	20.0	24.0	25.1	24.4	20.4	23.5	27.6	27.8	28.3	26.8	27.9
3	46.4	55.5	58.0	56.4	47.3	54.5	63.9	64.4	65.3	62.1	64.6
4	2.2	2.6	2.7	2.6	2.3	2.6	3.0	3.0	3.0	2.9	3.0

* A = interstate

B = primary

C = secondary

D = state highway only

E = service road

F = urban system

\+ 1 = large

2 = intermediate

3 = compact/subcompact

4 = van

Table 13.17. Estimation of Weighted Average Unit Cost of Vehicle Occupants on Wyoming Highways.

Occupations	Hourly Wage ($/hr) (1)	Percentage in Accident Fatal (2)	Injury (3)	Property (4)	Ave. (5)
Unknown+	12.0	40.0	26.5	24.4	30.3
Military	12.0	1.0	1.2	1.2	1.1
Unemployed	2.0	4.0	5.3	4.7	4.7
Miscellanies	20.0	0.5	0.8	0.9	0.7
Retired	3.0	6.5	3.4	3.3	4.7
Student	2.0	8.0	11.0	10.9	10.1
Laborer	10.0	6.0	6.8	5.4	6.1
Craftsman	18.0	3.5	5.8	6.5	5.3
Domestic	2.0	2.0	4.6	4.9	3.8
Transportation	15.0	12.5	10.5	7.6	10.2
Agri, ranch, forest	15.0	2.0	2.0	1.6	1.9
Service work	12.0	3.0	3.1	3.8	3.3
Clerical-sales	15.0	1.0	5.0	7.1	4.4
Professional management	25.0	8.5	10.9	14.3	11.2
Energy	15.0	1.5	2.9	2.3	2.2
		100.0	100.0	100.0	100.0

Weighted unit occupancy cost = $\Sigma(1)*(5)/100$ = 13.95 ($/hr)

+Hourly wage of unknown is assumed to be the average hourly wage of all other occupations.

weighted average unit cost of occupancy (in $/hr/person) can be estimated as in Table 13.17. The column containing occupant's hourly wage was our best guess which, of course, is subject to revision and may be better based on trucking values as indicated earlier.

13.6 Other Subroutines:

Other subroutines such as "AREA", "FSTC" and "NMLDEP" were also developed and are operational. Subroutine "AREA" is used to determine the extent of inundation area in the floodplain. Subroutine "FSTC" is used to calculate the first cost of drainage structures according to structure type and size as well as embankment and pavement considerations for the roadway crossing. Subroutine "NMLDEP" calculates the normal depth of water in the channel and/or floodplain for any given discharge before the presence of a roadway crossing.

14. SUMMARY AND RECOMMENDATIONS

This interim final report represents the progress made to the date of termination by WHD on developing a methodology and appropriate data to produce a design procedure for selecting appropriate flood design frequencies for drainage basins in Wyoming. The progress that was made to the date of termination was substantial and was producing the information that was needed to develop a sound, technically supported state-of-the-art methodology and associated computer program for flood design frequencies. The methods being used are considered to be highly defensible both from a scientific and legal standpoint. It is hoped that the WHD will find the information contained in this interim report of significant enough value that the project can be re-initiated at some time in the near future as funds become available.

Following is a list of the main tasks that were to be accomplished by this project as originally envisioned in the cooperative agreement with a statement indicating where the project was as of termination.

Task	Status
Task 1 - Definition and Acquisition of Variables	
Subtask 1A - Identification and Definition of Relevant Variables and Parameters	Completed
Subtask 1B - Selection of Hypothetical Drainage Sites and Data Acquisition	Completed
Subtask 1C - Identifying Design and Operational Performance Standards	Completed
Subtask 1D - Preparation of Interim Report	Completed
Task 2 - Developing and Applying a Technical Procedure to Identify LTEC Design Flood Frequencies	
Subtask 2A - Familiarization of WHD's Hydraulic Design Procedures for Culvert and Bridge Analyses	Completed
Subtask 2B - Development of a Database for Structural Design, Hydraulic Performance Characteristics and Various Flood Related Damages	Completed
Subtask 2C - Development of Procedures for Assessing Annual Expected Flood Damage Costs	Completed
Subtask 2D - Test the Developed Procedures for Flood Damage Evaluation	Partially Completed
Subtask 2E - Identifying the Economic LTEC Design Flood Frequency	Procedures Defined
Subtask 2F - Evaluate the Effect of Different Types of Uncertainty Considered in Analysis on the Economic LTEC Design	Completed
Subtask 2G - Determination of the Extended LTEC Design Frequency by Including Intangible Costs	Substantial Progress

Subtask 2H - Generating LTEC Flood Frequencies vs. Site Characteristics Database *

Subtask 2I - Preparation of Interim Report

Task 3 - Analyzing the Synthesized Data

Subtask 3A - Establish a Function Relationship Between LTEC Design Flood Frequencies and Site Conditions *

Subtask 3B - Establish Functional Relationship Between Hydraulic Performance Characteristics of Structures and Site Conditions *

Subtask 3C - Preparation of Interim Report *

Task 4 - Policy and Practice Recommendations

Subtask 4A - Comparisons of Current and Proposed Operating Policies for Determining Design Flood Frequency *

Subtask 4B - Identification of Sensitive Sites *

Subtask 4C - Preparation of Final Report *

From the progress made to the date of termination of this project, a number of recommendations to be considered are indicated below when this project is refunded.

1. Development of a computational routine to find the optimal N, W and H for box culverts that will minimize installation cost and meet WHD design specifications.

2. For the unit cost of bridge regression function, a closer look at several other variables besides the length of the bridge and the other quantities initially regressed should be considered with the outlier data set to see if a commonality variable(s) could be found to strengthen the regression function.

3. In some situations, either a bridge or a culvert system could be used to pass flow at a given site and consideration should be given to both types of drainage structure when generating the hypothetical drainage site database in the area that would overlap in consideration of these two structure types.
4. Development of a correct factor for some types of losses due to flooding that cannot be accounted for explicitly (not intangibles) in this study (i.e., employment and income losses, farm equipment, etc.).
5. Consideration of the trucking business as a first approximation with some modifications to obtain an estimate of the cost of time loss of a vehicle occupant as a result of a detour.
6. Re-evaluation of the selected attributes being utilized as a portion of the decision-making process for intangibles of the extended LTEC design frequency methodology to ensure that all quantities essential to the decision-making process are included.
7. To analyze the types of buildings that may be located in the floodplain of a given site, information on commercial buildings needs to be developed as well as investigation of the arrangement of building types at distances away from the main-stream channel within the floodplain.
8. Development of a regression function relating average daily traffic (ADT) to roadway top width rather than the present scheme being used.

9. Consider some different variables for development of a regression relationship for vehicle composition (types) and vehicle occupancy.

It is difficult at this time to estimate what it would cost to finish this project if it were refunded with the tasks remaining and the above recommendations that should be considered. At this particular time, the provisions for cost sharing will have to be decreased drastically from the previous agreements because part of the costs associated with the principal investigators will be required to be included in the costs to WHD. A ballpark estimate would be $65,000. The estimated time for completion of the project would be approximately two years.

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