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DESIGN AND CONSTRUCTION OF LOW WATER STREAM CROSSINGS

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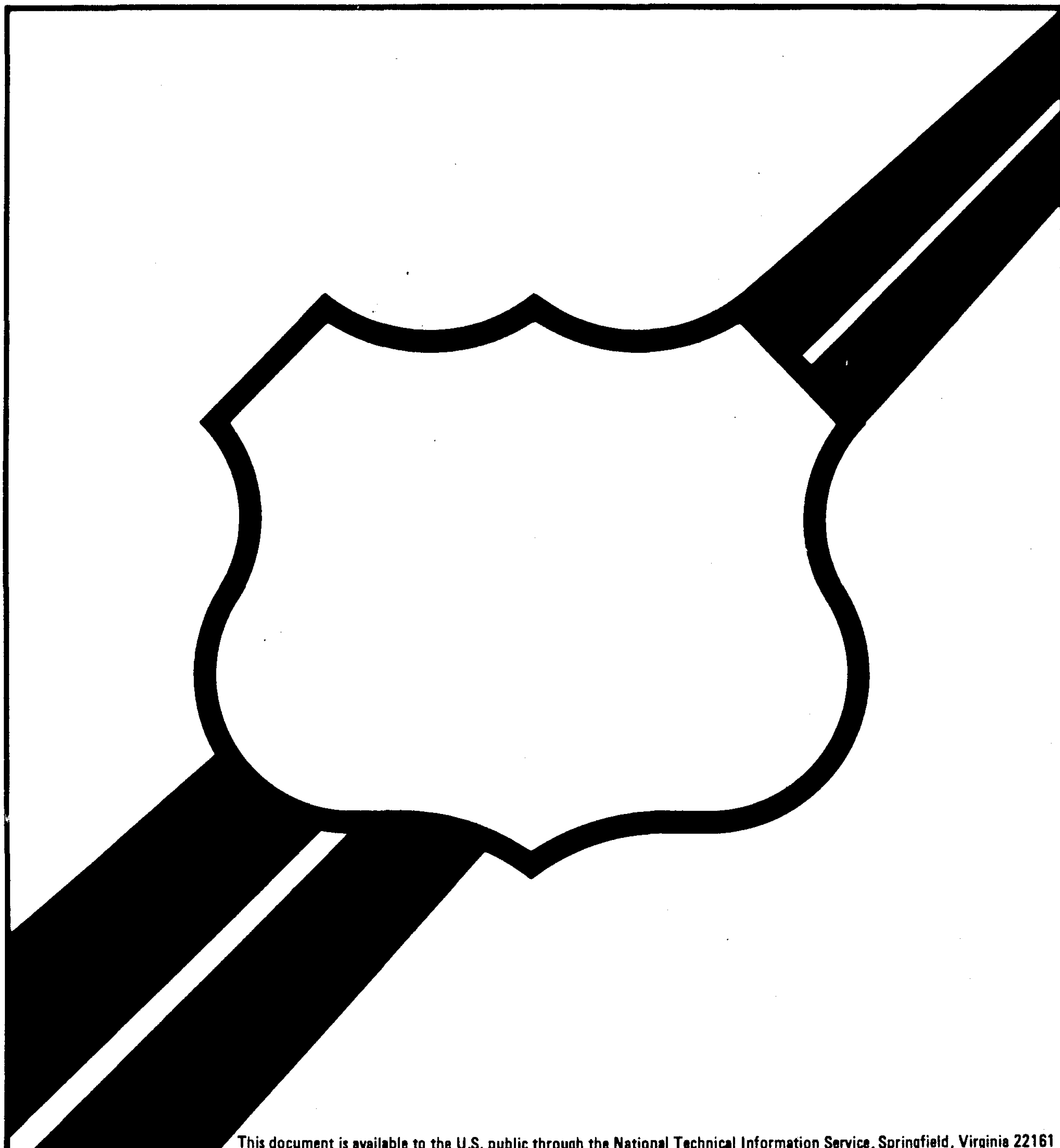

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16. Abstract

Literature on the Low Water Stream Crossings (LWSC) are reviewed and analyzed to develop a set of design criteria for such structures. Case histories of existing structures are documented and an approach to decision criteria and economic analysis on which decisions can be made are incuded. Hydraulics of LWSC and the means of erosion protection are also discussed to the extent they are applicable to the structures of this type. This report, which emphasizes theoretical views and concepts, complements a second volume: "Design Guide: Low Water Stream Crossings", which stresses application and computations.

Other reports in this series include:

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A	CROSS SECTIONAL AREA
ADT	AVERAGE DAILY TRAFFIC
A_p	TOTAL AREA OF PAVEMENT
C_a	ADJUSTMENT FACTOR FOR QUICK REPAIR
C_e	COST OF EMBANKMENT
C_f	COEFFICIENT OF DISCHARGE FOR FREE FLOW
C_p	COST OF PAVEMENT
C_s	COEFFICIENT OF DISCHARGE FOR SUBMERGED FLOW
CRF	CAPITAL RECOVERY FACTOR
D	FLOW DEPTH
D	CULVERT DIAMETER
DD	DIFFERENCE IN DISTANCE BETWEEN NORMAL ROAD AND DETOUR
DH	ELEVATION DIFFERENCE
DT	EXTRA TIME NEEDED FOR DETOUR
d	WATER DEPTH FROM THE FIRST FLOOR OF BUILDING
D_{50}	MEDIAN DIAMETER OF STONE
H	TOTAL HOODWATER HEAD MEASURED ABOVE THE CROWN OF ROADWAY
HW	HEADWATER DEPTH
i	INTEREST RATE
K	MULTIPLIERS TO MODIFY TANGIBLE COST OF LWSC TO REGULAR BRIDGE
L	CULVERT LENGTH
L	LENGTH OF BROAD CREST WEIR
L	LENGTH OF LONGEST WATERCOURSE
L(Q)	DAMAGE COST DUE TO FLOOD WITH DISCHARGE Q
L_{ep}	LOSSES TO EMBANKMENT AND PAVEMENT
L_{tr}	LOSS DUE TO INTERRUPTION OF NORMAL TRAFFIC
N	SERVICE LIFE OF STRUCTURE
NC	AVERAGE NUMBER OF OCCUPANTS PER VEHICLE
n	MANNING ROUGHNESS COEFFICIENT

LIST OF SYMBOLS
(CONT'D)

P	WETTED PERIMETER
P	PERCENT LOSS
P_e	PERCENT LOSS OF EMBANKMENT
Q	DISCHARGE
Q_{ot}	OVERTOPPING DISCHARGE
Q_t	FLOOD WITH THE RECURRENCE INTERVAL OF t YEARS
R	RISK COST DUE TO FLOOD
RC	INCREASED RUNNING COST
R_e	RATE OF EMBANKMENT REPAIR COST
R_p	RATE OF PAVEMENT REPAIR COST
S_o	SLOPE
T_b	BASE TIME OF HYDROGRAPH
TC	COST OF LOST TIME
T_r	RESTORATION TIME
T_{ot}	OVERTOPPING TIME
TW	TAIL WATER DEPTH

LOW WATER STREAM CROSSING

I. INTRODUCTION

A. BACKGROUND

Almost all governmental entities responsible for road and highway systems in the country are currently facing severe limitations of available resources. This is particularly true for the nation's low volume road system, which forms about 75% of the entire highway and road network of the country. One of the impacts of such shortage of funds is being reflected in that the construction of many new crossing structures as well as much needed replacement of many old, unsafe structures on low volume roads are faced with long postponement or cancellation. As a result, the concept of Low Water Stream Crossing (LWSC) has been accepted in many places as an alternative to the usually costly conventional high level bridges. An LWSC is defined as a stream crossing structure that is designed and constructed so that it is overtopped by floods several times a year, as opposed to the traditional practice of designing for a high flood, typically with a frequency of 20 or more years.

Since an LWSC is a low structure, it is relatively inexpensive. However, since it is overtopped by floods and therefore interrupts the traffic frequently, it causes frequent inconvenience and loss of time and use. Further, since LWSCs are exposed to the erosive forces of overtopping flood water, they need frequent repair, although total repair costs may not generally be very high.

An LWSC, therefore, can be well suited for certain areas, where lack of funds do not permit building a conventional crossing and where circumstances make some inconveniences acceptable to the users. However, several questions remain; the most important of them are: "how to decide where an LWSC is suitable", "what should be the hydrologic and hydraulic design criteria for their design", and "what structural/hydraulic means are needed for the efficient operation of an LWSC so that their total cost including repair costs are minimized". It is hoped that reasonable answers to these questions will provide some guidance to the engineers and planners in considering various aspects of an LWSC meaningfully, as well as in designing them with a certain degree of confidence.

B. OBJECTIVES

Objectives of the study as outlined by the Federal Highway Administration are as follows:

1. To document and advance the state-of-the-art and the design and construction of Low Water Stream Crossings for low volume roads.

2. To characterize and document by case history existing LWSC structures of different types.

3. To provide guidance to the planners and designers of LWSCs and other users for:

   a. Decisions on applicability of LWSC as opposed to conventional high bridges at any given site.

   b. Selection of appropriate type of LWSC structures.

   c. Efficient design of LWSC structures.

C. RESEARCH APPROACH

The approach taken in performing this research included an extensive search of literature; a number of case histories in various parts of the country were developed, including the selection and design criteria used as well as their performance history. A number of engineering practitioners engaged in the design and construction of LWSCs were interviewed along with many state, county and local officials. In addition, independent investigations were made into several aspects of design, such as erosion protection and risk analysis.

The review of literature on LWSCs covered more than 100 significant works related to the development, design and construction of such structures. These are summarized in chronological order in the "Literature Review" section and formed the initial basis which provided a starting point for the formulation of a set of desirable conditions and criteria for an LWSC.

The case histories including the selection process, design methods, construction techniques and performance records of more than forty old and recent LWSCs were studied and evaluated; and five such cases were described and discussed in the report. Although documented information on LWSC was found to be scarce and in most cases incomplete, this evaluation led to the formulation of a set of general considerations that can be applied in the design of an LWSC.

Another major input in this study came from persons in many parts of the country and abroad, who have been involved in the design, construction and maintenance of LWSCs. Present state-of-the-art of LWSC being such that these are primarily selected and designed based on the personal experience and judgments of the engineers, this input was considered essential and valuable in this attempt to consolidate the information on LWSC.

II. SUMMARY AND CONCLUSIONS

Review of available literature on low water stream crossings (LWSC), as well as communications with many county and local highway engineers reveal that numerous such crossings exist in almost all parts of the country. Such structures are particularly suitable for flat arid regions, as in the south western and mid-western U.S., over streams with wide floodplains and dry streambeds and over streams where the depth of normal flow is very small. LWSCs are also suitable for mountainous terrains and for long and narrow drainage areas where the duration of flood flow is short.

In the U.S. and overseas, LWSCs have been satisfactorily used both on low volume rural or secondary and farm to market, as well as on high volume urban roads. This leads one to conclude that their acceptability and the justification of use primarily rest on the economics of the available alternatives, which is dependent on a host of factors, including traffic volume.

The primary criteria that are presently and most commonly used in the selection of an LWSC over a conventional highl level bridge are: shortage of funds, low traffic volume and various political considerations. Due to the extremely high cost of conventional bridges, in many cases the alternative to an LWSC would be no bridge at all. In such cases, an LWSC is generally more easily accepted from a point of practical consideration and as a workable compromise.

Current practice in the design approach for an LWSC is not much different from that for a high level traditional bridge crossing. Structural features of the crossing are generally modified to make it hydraulically more efficient and to provide protection against the erosion and other hazards of the overtopping flood waters. Maximum use of available material and labor is generally made, lowering the capital cost of such structures.

Many LWSCs have been known to perform efficiently for a long period of time. A still larger number have failed due to various inadequacies in the selection, design or construction process. However, no definite systematic approach has yet been developed for the design of LWSCs. The current design practice is to design the structures primarily based on individual experience, judgment and intuition. No comprehensive study of such process has been made so far, and the information on selection, design and performance of LWSCs are widely fragmented. One difficulty in the formulation of a definite procedure for the design of LWSC is that the knowledge about hydraulics of such structure is still very limited. High level bridges and culverts have been extensively studied in the laboratory as well as in the field, and thorough knowledge about their hydraulic performance characteristics exists, which have led to the development of an organized and systematic design procedure and guidelines. Except for the limited extent to which the knowledge of hydraulics of high bridges and culverts can be applied to the LWSCs, not much is known about their hydraulic behavior, particularly during the critical period of overtopping.

The need for a specific hydraulic or structural design approach, e.g., criteria for design flood, elevation of roadway, waterway area, etc., is, however, greatly diminished if one approaches the problem from a different perspective, and begins with a decision analysis which is primarily based on an economic analysis of the LWSC and all other alternative feasible structures. According to this approach, total tangible costs including the capital cost and the risk costs associated with a particular structure-- whether a conventional bridge or an LWSC--should ultimately determine whether the structure under consideration is the desired optimum (or least total expected cost, LTEC) solution. All design characteristics, including the elevations, waterway area and capacity, extent of erosion protection needed, etc., for the most desirable structure will thus be determined from this analysis.

Such a risk-based design approach has been a fairly recent development in the practice of bridge design and is slowly being recognized by the highway engineers. And it is apparent that any meaningful attempt to the selection and design of an LWSC must include a risk analysis-based approach. The problem with such an approach, however, is that a detailed risk based economic analysis requires availability of a large amount of data of various kinds as well as an intensive and tedious computation process which takes a large amount of effort. The time and effort required by such analysis therefore may not, in many cases, be commensurate with the savings in cost it offers. It is, therefore, necessary to simplify the procedures for risk analysis, possibly by reducing data needs through the use of generalized information, by ignoring some parameters in the analysis to which the results are not too sensitive and by using approximate solutions to many complex components of the risk analysis model.

In addition to the tangible costs of an LWSC, there are many intangible factors which may play dominant roles in the selection and design process. These include: inconvenience due to stoppage of traffic, potential danger to life, loss of use of road for emergency purposes, effect on the environment, as well as many other social, environmental and legal aspects. A set of criteria that is considered most favorable for an LWSC was developed in a previous study (86) and is adopted as selection criteria for LWSCs. A similar set of criteria indicates the conditions under which a conventional high level bridge will be desirable, and serves as exclusionary criteria for LWSCs. For cases between these two extremes, detailed economic analysis may be necessary to determine an LTEC solution.

An LWSC is primarily different from a conventional bridge in that it is designed to be frequently overtopped during high flows in the stream. Therefore, the major difference in the various design features of these two types of structures is the one due to an essential need for the LWSCs to be protected against overtopping waters, as well as to allow efficient passage of water as soon after the flood as possible and with as little backwater as possible. Although not many reliable records of LWSC performance are available, based on the literature search, case study and communications with the experienced highway engineers, a number of important design considerations have been identified that will help in achieving these results.

III. LITERATURE REVIEW

In designing a stream crossing, it is natural for a designer to select a simple, economical crossing. A simple structure may however, impair traffic safety and comfort to some extent. A properly engineered design has therefore always been the one which is the most desirable compromise between the cost and comfort. But as technology advanced and better construction materials became available in the early 20th century, better and more expensive bridges were built, with less emphasis on economical design. This trend came to a temporary halt in the 1930s through 1940s due to the recession and war when many LWSCs in the United States were built on secondary roads. After the war, as the economy recovered and flourished through the 1950s and 1960s, the trend toward a better highway system handling faster traffic with more comfort surfaced. As a result, the subject of LWSCs did not receive its due share of importance until recently.

As the increase in population brought pressure for more new water crossings on the nation's network of low volume roads, and the low cost crossings hurriedly built in the early 20th century became obsolete, LWSC attracted notice and again seemed to be a simple economical solution for many low volume road crossing needs. There are three types of LWSCs commonly in use (17): 1) fords (or dips) formed by lowering the road grade to the stream bed level from bank to bank; 2) vented fords formed by partially lowering the road grades for passing floods and providing culverts for handling the day-to-day flow; and 3) low water bridges formed by partially lowering the road grade for passing floods and providing a bridge type structure to handle the day-to-day flow. LWSCs are also known as Irish bridge, Kentucky bridge, submersible bridge or causeway in various parts of the world.

An early study was performed by James (43) in 1930 on the grading of causeways. Rough riding over a causeway is generally the result of two conditions: 1) the pavement at the invert of the causeway is subjected to scouring by water passing over it. If careful and frequent maintenance is not performed, the resulting channels on the pavement cause inconvenience in much the same way as do potholes; 2) the causeway may be too short for its depth, i.e., the vertical curve may be of too small a radius. The first condition can easily be taken care of by keeping the pavement reasonably smooth and maintaining it in that condition. To remove the second condition, it is necessary to provide vertical curves of suitable radii. James deals with the techniques of determining proper grades for the approaches by selecting vertical curves of maximum radii along with a level grade at the bottom of the causeway. Equations for determining the lengths of vertical curves and their radii for different depths of causeways for speeds of 25 to 50 miles per hour (40 to 80 km. per hour) is discussed and the instruction given for actually setting out the grades in the field. Vertical curves of 200 ft (61 m) radius will ensure smooth riding for speeds up to about 25 miles per hour (40 km per hour).

The basic principles regulating the design of a submersible bridge are discussed by Mears (58). A submersible bridge must have an adequate foundation so that it may successfully resist the lateral thrust due to water pressure, and also be protected against scour. It should be heavy and massive, and without perforations or pockets in which floating debris could lodge. Spans should be moderately large, about 20 ft [6.1 m], so as to let debris, such as floating trees, etc. pass through. Steel bridges are not suitable because they are relatively light. The suitability of such bridges was illustrated by bridges constructed in central India during the late 1920s to early 1930s. Some were even submerged while still under construction due to floods caused by heavy rainfall 12 to 36 hours in duration.

The study on causeways continues, and in May 1932, the principles of causeway design (76) are elaborated. It is suggested that the height of a causeway be kept at a minimum (preferably not exceeding 3 ft [.91 m] at the deepest point) to interfere as little as possible with the natural flow conditions. Concrete is the most suitable material for paving. The slab should be 6-7 in (15 to 18 cm) thick, with all edges thickened to 9 in (23 cm), and reinforced with steel mesh. Cut-off walls should be used to prevent the undermining of the structure and to confine and stabilize the fill supporting the slab. The stream bed immediately downstream from the causeway should be paved with larger stones to protect the bed against erosion. For traffic safety, posts conspicuously marked to show the depth of water over the causeway and to define its position when under water are required. Principles, rather than standardization of details, are discussed.

In 1934, Dean (22) presents the design and construction of a submersible road bridge over the Nerbudda River near Jubbulpore, India. The factors which determine the suitability of the site for this bridge are the same as those commonly used in site selection and includes: 1) a straight stretch of river; 2) well-defined and high banks; 3) small width; and 4) good foundations.

Additionally, the method employed in designing the various bridge elements is described and calculations provided for sizing the elements and designing the piles for foundations under critical conditions, i.e., 1) when the bridge is about to be overtopped, and 2) when the bridge is under the maximum known flood.

Mirza (61) in 1944 presents a theoretical reason that as long as the river does not develop supercritical velocity, the design of the superstructure of a submersible bridge or causeway does not call for any special treatments other than those observed for an insubmersible or high-level bridge. Causeways with monolithic bases, such as arches built with concrete or concrete pipes, are suggested for unstable soils. Palit (79) advocates that a series of R.C. box culverts will be cheaper and more desirable. His opinion is later supported by Raghavachary (78) who compares advantages and disadvantages between concrete-pipe and R.C. box type structure on unstable foundation soils. Across wider streams, low cost low water bridges are suitable. One such bridge is a low water bridge over Dry Fork Creek in Missouri (6). The Dry Fork Creek is crossed by a rural delivery route two miles east of Bern in Gascomade County. There was no bridge; the 100 ft (30.5 m) channel was sometimes impassable for days and hazardous to the mail carrier. Consequently, the county decided to build a low-water crossing 150 ft (46 m) long, 18 ft (5.5 m) wide, including two 25-ft (7.6 m) approaches. The bridge deck consists of 10 in (25 cm) R.C. slab resting on nine vents and two walls, and the finished floor is 36 in above low water level, 2 in (5 cm) higher on upstream side than on downstream side. The entire bridge is supported on white oak piles driven to solid bed rock and spaced at 3 ft (0.9 m) center-to-center with six of them under each vent and each wall.

In the 1950s, LWSCs were favored by bridge engineers in India. Lall (48), basing his study on the investigation of failures of various causeways, described the principles of causeway design for sandy beds and uniform soils. Also noted are some of the primary factors which can initiate bridge structure failure, ultimately resulting in great damage or elimination of the whole causeway. Example of damages to the Subarnarekha Causeway in India, constructed in 1906, is cited. He provides several design details to make the new causeways less susceptible to damages inflicted by flood water. The following conclusions are drawn: 1) railing must not be provided; 2) the foundation of a causeway needs adequate protection against scour and undermining. This can be done by providing curtain walls between the two cut-offs extending down to their lowest levels so as to completely box up the foundation of the causeway.

Several observations made by Mitra and others (62) points out that more than a dozen causeways constructed on sandy soils in Hyderabad State in India during the 1930s and 1940s functioned quite satisfactorily. Some of the suggestions for designing such causeways are:

(a) Providing vents for the full width of the stream.

(b) Providing a foundation raft and apron thick enough to distribute the load uniformly and withstand impact and scour.

(c) Preventing sand under the raft from being scoured by providing cut-off walls upstream and downstream up to scour depth, and aprons of sufficient width and weight upstream and downstream with staggered blocks.

(d) Wedging or tieing the wing and return walls into the natural banks, for about 35 to 50 ft (10.7 to 15.2 m).

(e) Providing guide stones 12 in (30 cm) high, 4 to 6 ft (1.2 to 1.8 m) apart, instead of rigid or collapsible railings.

Gupta (38) describes the development of vented causeways constructed on sand foundations across Indian rivers. The design aspects of a causeway are discussed with reference to their stability against scour under the founda-ation, safe bearing-pressure distribution, scour on the downstream side, and horizontal pressure due to moving water. Some empirical formulas for calculating design stresses and critical floor length-cut off depth ratio to prevent undermining are presented. He contends that submersible bridges are especially suitable where rivers have a very small flow during the greater part of the year and are subject to rapidly rising and falling floods so that the duration of traffic stoppage is short. A submersible bridge may be provided if peak floods do not interrupt traffic on more than six occasions during a year.

In the United States, articles regarding LWSC begin to appear more often after 1950. Trice (97) discusses a basic design and construction procedure for a low cost concrete bridge, similar in principle, to those of previous authors (48, 62).

In developing a network of low cost roads throughout Texas, Bingham (7) develops guidelines for design and construction of low water dips. For those stream crossings where the amount of traffic justifies a bridge, but sufficient funds are not available, construction of a well designed dip is stressed, which will be far better than an inadequate bridge. Prior to designing a dip, the collection of pertinent data is recommended, which should include stream section and roadway section; water elevation at normal river stage; stream meander, profile and topography, highway elevation; and drainage area. In addition, provisions for smooth and safe grades are suggested and the methods for determining the proper road grades corresponding to the design speed of the road are described. Some examples are also included.

Several low cost concrete bridges in Burleson County, Texas are described in "Roads and Streets" (51). In the Engineering News Records (19), construction of a 480 ft (146.3 m) span and 12 ft (3.7 m) wide submersible bridge over Haina River in the Dominican Republic is described. The bridge has been constructed with precast concrete pipes tied and reinforced together with a rigid frame of steel rails and reinforcing steel bars. During extreme flood periods water may rise as much as 4 ft over the whole bridge.

As the economy in the U.S. prospered in the late 1950s through 1960s, interest in LWSC dwindled and the construction of higher level and more expensive bridges became the trend. This can be seen from Wilson's paper (113), which stresses the replacement of drainage dips by pipe culverts as a result of increased highway traffic volume. He also points out that installation of pipe is more economical than construction of a bridge.

In discussing the advancement of construction techniques and materials, some innovations related to the design of small bridges are described by Kiedaisch (47). These changes include elimination of the cap detrusion for slab spans for 26 ft (7.9 m) or more, and introduction of prestressed designs using slabs, T-beams, box girders and I-girders.

As the field data of LWSC accumulates, steady progress is made in evaluating the functions of the components of existing structures. Raju (79) points out that these structures present unique hydraulic problems in addition to those encountered by other hydraulic structures, such as dams, spillways, weirs, etc. The field investigations carried on in the past indicate three chief factors contributing toward the causes of problems in a causeway: 1) design of the causeway; 2) configuration of the river, including beds and banks; and 3) conditions of flood flow. He cautioned that in constructing a causeway below a bend in the river, the vents should be distributed along the causeway to avoid a large vortex flow downstream of the causeway.

Chatterjee (14) constructed vented causeways by utilizing empty bitumen drums. During the summer months when the river was dry, its bed was excavated to a depth of 4 to 5 ft (1.2 to 1.5 m), and a width of at least 20 ft (6.1 m). A concrete mix in the ratio of 1:4:8 was laid into the excavation to a depth of 4 ft (1.2 m). Over this hard foundation, a layer of rich mortar made of sand and cement was laid. The empty bitumen drums, with indentations at both ends removed, were placed in rows along the course of the river. A 6 to 9 in (15 to 23 cms) gap was left between each row of drums and filled in with 1:2:4 concrete mix. Above the drums, a 1 ft (30 cm) layer of concrete was spread to load the drums. The surface was finished with 3 in of a 1:1-1/2:3 mix wearing course. Expansion joints were kept at intervals of 20 ft (6.1 cm). Low water crossings of this type up to 250 ft (76.2 m) in length, 16 ft (4.9 m) in width, with 60 ft (18.3 m) long approaches have been constructed.

Iyer and others (42) describe in a paper in 1955, the details of design and construction of submersible bridges over raft foundations. The elements described include siting, design of foundations, and design of a superstructure composed of a deck slab, wearing coat, expansion joints and guard stones. Additionally, pertinent information with respect to constructional organization, costs of specific items of work, and calculations for designing a bridge are provided. Lacey's formula for scour depth is used to determine scour depth at abutments.

Bennett and others (5) suggest that the critical condition in a submersible bridge is when the floodwater is about to overtop the structure, since under high flood the velocity through the vents is not higher than that in the upstream side due to submergence of the structure. Therefore, scour depth may not be more than that under ordinary flood conditions. In reducing flow resistance, the upstream and downstream face of the bridge slab should be rounded.

Based on several investigations, Chanda and others (13) conclude that:

(1) The floor level of a submersible bridge should be kept at some depth below the low bed level to prevent the floor acting as a weir when degradation of bed level takes place;

(2) The length of the downstream apron should be calculated for 1-1/2 D where D is the maximum scour depth under critical conditions, i.e., when the flood water is just about to overtop the structure;

(3) Rounding the end faces of the slab and a one-way camber of the roadway should increase the coefficient of discharge of the deck.

These works are followed by studies by Victor (106), Singh (87) and Kand (44) adding details on design and construction of submersible bridges. These details include design of bearings and hand rails, and essential hydraulic design criteria. Also included are methods for related hydraulic calculations using empirical and rational formulas to determine the various proposed levels including level of deck slab bottom, finished road level, etc. It is pointed out that when the flooded river is about to overtop the bridge, it will be under the combined action of forces produced by water currents, buoyancy, and uplift pressure of the air entrapped between the girders and the deck slab bottom. Therefore, proper air escapes, such as vents in the sides or in the deck itself, should be provided.

Most of these works abroad are focused on submersible bridges with less effort on investigating smaller low water crossings, such as fords and vented fords, perhaps because of relatively small costs of constructing such structures. In the U. S., many secondary roads have been built since the 1940's as the main highway system rapidly expanded. This required many new LWSCs and replacement of outdated LWSCs.

Smith (88) counts about 250 bridges of more than 20 ft (6 m) in 1,200 miles (750 km) of county road in Walla Walla County, Washington. To operate

operate and maintain this road system with limited funds, he recognizes the necessity for such low-level low-cost bridges.

In designing a small stream crossing in Minnehaha County, South Dakota, Scurr (83) compares the costs of 11 alternative structures which could carry the design run-off. Although factors such as traffic safety and property damage from flooding should be considered in the final decision, it is stressed that the most economical structure satisfying all engineering and local requirements should be selected. Comparison of construction costs reveals that the smaller culvert is not always more economical than one with a larger opening and that there is not always a direct reltionship between length of the structure and its cost.

Three ways of lowering bridge costs are suggested by Dean (23): 1) consideration of H-10 design loading on Class III roads; 2) reduction of two-lane roadway width from 24 ft to 20 ft (7.3 to 6 m); and 3) lowering height of the structure.

Provisions for low-cost bridges for lightly traveled roads are also stressed by Dudley (26). Use of surplus material and utilization of one-lane bridges are alternatives for reducing bridge construction cost.

Smith (88) suggests installation of multiple arches placed upon a concrete foundation with concrete wing walls, use of timber treated with creosote or commonly used preservative, and use of precast and pre-stressed concrete.

Instead of replacing old, small bridges, Laumann (49) contends that larger culverts safely pass design flood can be used at a cost substantially less than that for a bridge structure replacement. Instead of replacing an old bridge over Mound Creek in Brown County, Minnesota, he estimates a cost saving of 60% could be obtained, if culverts were used.

Low water crossings over small mountain streams have become increasingly popular in recent years. Mountain roads can operate with occasional road closures. Coghlan and Davis (17) concludes that it is reasonable to drive a vehicle through 3 or 4 in (7.6 to 10.2 cm) of water and that a road closure of one to three days at a time, not totaling more than 15 days a year, may be allowed in forest roads.

Use of gabions for low water crossings on primitive or secondary forest roads are suggested by Leydecker (50). Gabions are placed at the final grade line, with the upstream edge of the gabion alongside the downstream edge of the road. The gabions are then backfilled and stream gravel is pushed up behind the gabions to form the roadway. Essentially, the gabions form a 6 to 12 in (15 to 30 cm) high, porous dam which retains the stream gravel.

Winfrey (114) points out that in the economic analysis of bridges, it is necessary to consider damage costs from floods which exceed the conveyance capacity of bridges. Annual cost, combining annual equivalent cost of all investment should be used as a guide in decision making.

Morgali (64) presents procedures for determining the most economical design for bridges and roadways crossing the flood plain, where damage costs due to floods are considered.

Tseng and others (98) discuss an engineering systems approach, based on risk analysis, to determine the most cost-effective design configuration for a highway stream crossing. To evaluate various design alternatives, the methodology integrates economic factors with the most commonly used hydrologic and hydraulic parameters by linking the initial construction costs and flood related highway losses, including property damage from backwater flooding, with the size of the bridge and design flood frequency. To illustrate the application of this risk analysis, a problem applying to the Tallahalla Creek Bridge site near Waldrup, Mississippi is included.

These studies are followed immediately by a study by Schneider and Wilson (82) on hydraulic design of bridges with risk analysis. Based on these studies, a hydraulic engineering circular provides procedures for design of encroachments on flood plains using risk analysis. Since then, this method of economic analysis of stream crossings has increasingly gained general support and popularity.

IV. TYPES OF LOW WATER STREAM CROSSINGS

LWSCs commonly designed and constructed in the United States can be broadly characterized as:

A. Fords (commonly known as dips)
B. Vented fords (dips with vents/drain-pipes and culverts)
C. Low water bridges

These three types of LWSCs are briefly described in this section to provide an overview of the various features and design considerations of such structures. Detailed discussions of design criteria and considerations are provided in the Design Guide, and are not repeated here.

A. FORDS

Fords are commonly constructed across drainages that are dry during the most part of the year or where the day-to-day flow is in the order of a few inches of depth. They are generally laid on the stream beds, the water flowing over them and are generally formed by lowering the grades of approaching roads to the streambed level from bank to bank.

The simplest type of ford normally consists of an unsurfaced crossing formed by levelling the streambed for the width of the roadway. In some fords a row of boulders along the downstream edge of the roadway is provided, which is backfilled with gravel, thus forming a road surface. A slightly improved ford may, however, utilize two small endwalls, one on each edge of the roadway, or a wall on the upstream side and gabion along the downstream edge of the roadway.

Generally, simple fords initially constructed for low-volume roads are gradually upgraded as their utilization increases with increase in traffic count. Some fords may need upgrading after initial construction due to the fact that once the stream gradient has been altered by boulders or gabions, erosion of the streambed just below the downstream side of the crossing develops, resulting in formation of a plunge pool unless the streambed gradient is flat for a considerable distance downstream or the streambed is largely composed of bedrock and large boulders. As the erosion process continues unchecked, the plunge pool can grow larger in size, thereby undermining the roadway support and creating severe maintenance problems which could lead to ultimate failure of the crossing. In order to have a successful and maintenance-free ford, the experience gained from past performance of fords dictates that a ford should include the following components:

1. An unerodible paved roadway over which vehicles can smoothly run.

2. Two end cutoff walls, one on each edge of the roadway, of sufficient depth to provide support to the pavement and counter any kind of subsoil flow.

3. A rockfilled gabion or other endwall on the downstream side to check scouring of the streambed.

4. Markers enabling drivers to spot the limits of the roadway when it is flooded.

Fig. 1 shows the details of several typical fords.

Pavement. The pavement of the roadway can be rigid, such as, concrete or asphalt-treated surface, which cannot withstand movement of the bed soil underneath without suffering some structural damage. Or it can be a flexible type, which can deform itself when there is soil movement. The pavement in either case provides vehicles with a stable tractive surface and protects the crossing from erosion.

Generally, roadway pavements of reinforced concrete with expansion joints are preferred because of the inherent advantage of adequate strength to carry traffic over weak soils. However, due to high initial construction cost and erosion of the streambed through cracks in the concrete which causes movement of the soil underneath and deterioration of the pavement, their use over erodible soils is discouraged unless the foundation is treated. Asphalt-treated surfaces provide a smooth driving surface, but because of its lighter weight, an adequate layer of asphalt and aggregate, in the range of 4 to 6 in (10 to 15 cm) needs to be used. Fig. 1 shows the different types of improved fords discussed.

Gabions are boxes made of steel wire fabric and filled with stones, constituting a solid mass so heavy that water cannot displace them. At the same time, they are flexible enough to adjust and strengthen themselves if the soil beneath is eroded. Thus, gabion-walled fords (Fig. 2) provide both flexibility and resistance to erosion. On the other hand, vehicles passing over gabions often break down the wires on the driving surface requiring repair. The side baskets continue to hold the entire length of gabions.

<u>Endwalls</u>. The primary role of endwalls is to protect the edges of fords from being eroded away and to provide support to the pavement during flooding when scouring activity is enhanced. The endwalls also play an important role in reducing the piping action of seeping water under the pavement, thus reducing the uplift force of the water against the pavement. Walls should normally be carried below the scour depth. If the streambed is rocky and the required depth of the wall is small, it can be made of stone masonry. If either the depth is relatively high or the bedsoil erodible, stone masonry walls are replaced by concrete walls. In such instances, provisions for gabions and aprons on both upstream and downstream sides are recommended.

<u>Markers.</u> Markers generally consist of small posts placed equi-distant along each edge of the roadway. Post height is normally related to the depth of flood water at the dip.

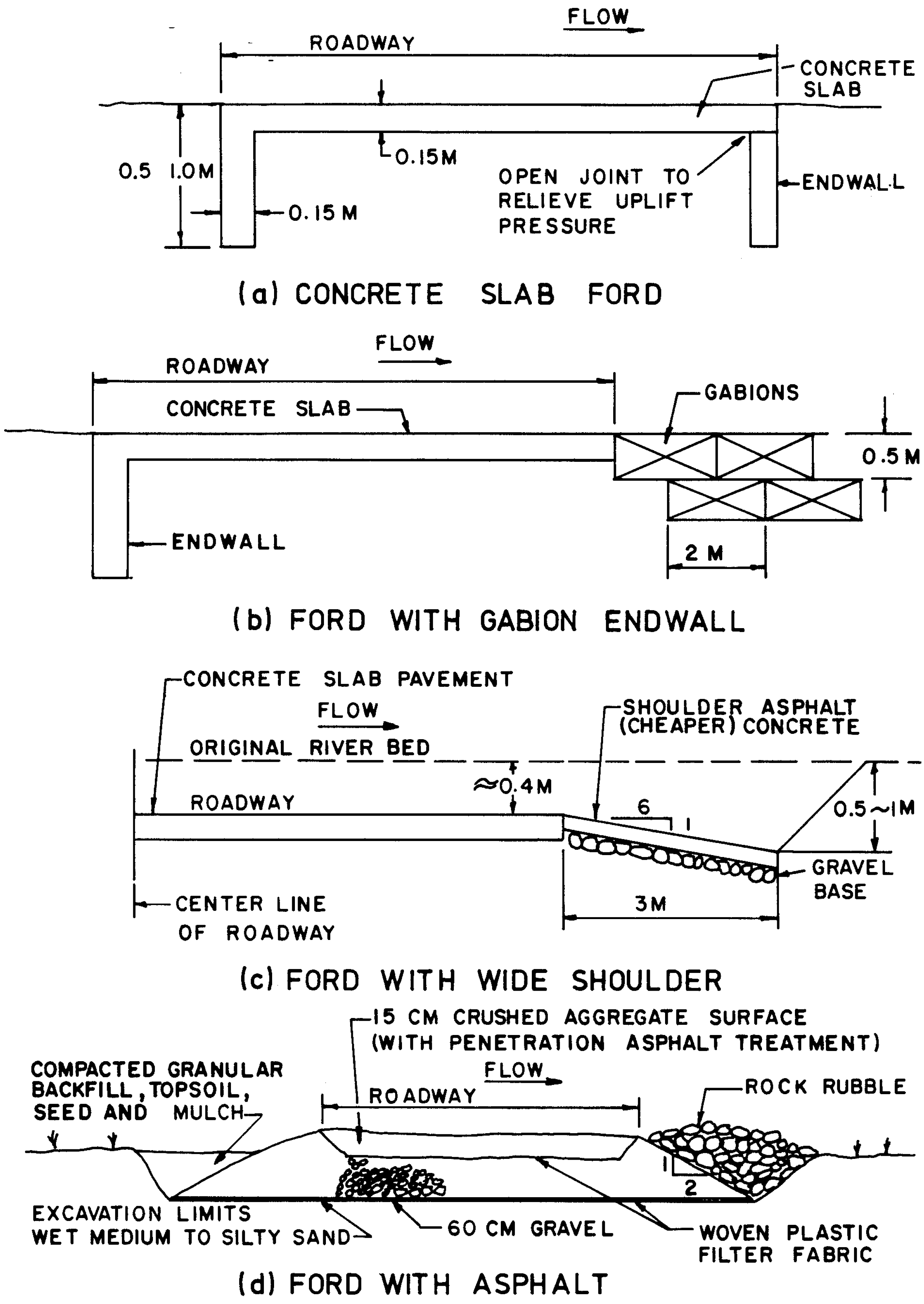


FIG. I TYPICAL FORDS

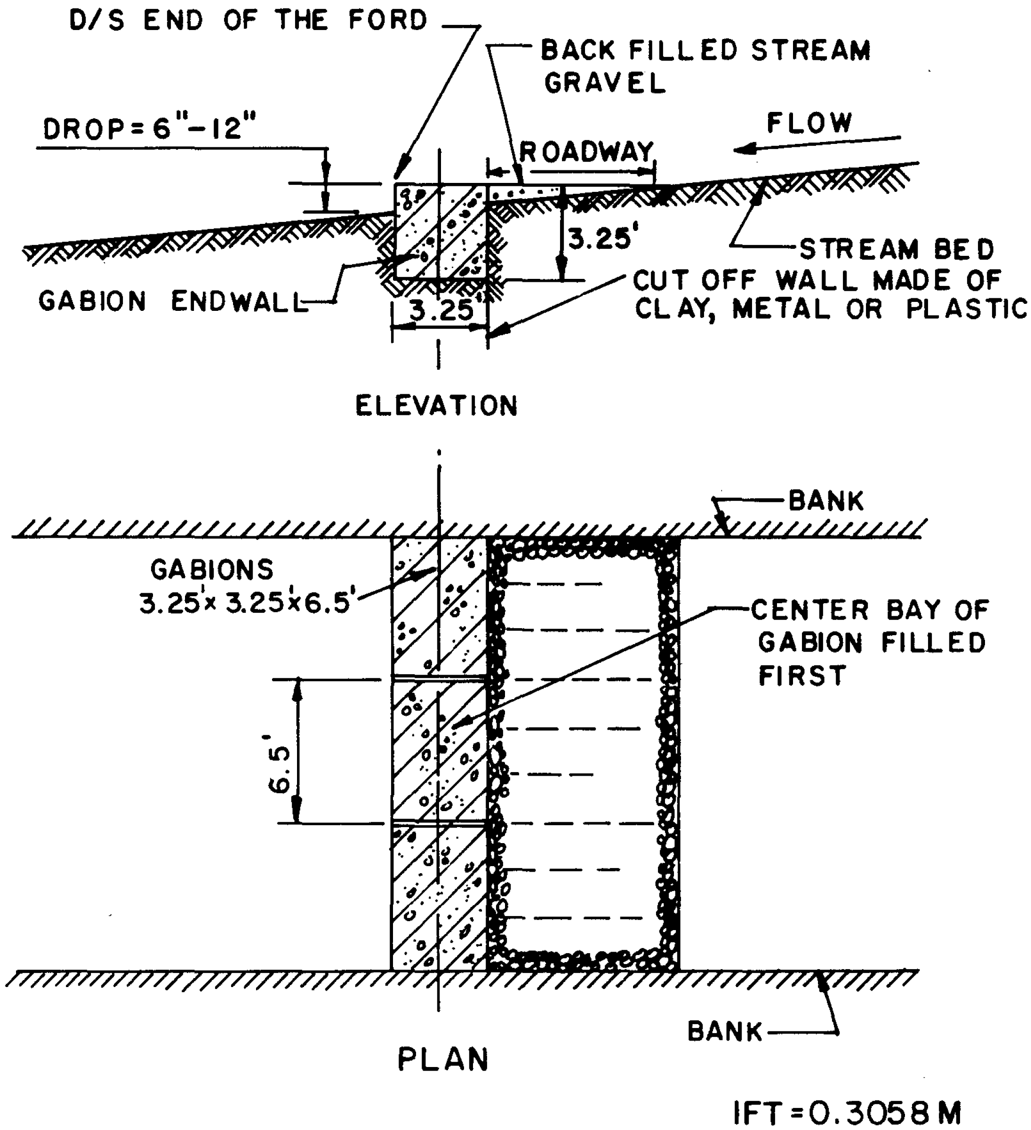


FIG. 2 GABION WALLED FORD

B. VENTED FORDS

Vented fords are dips with vents (pipes) to provide passage for day-to-day flow. They are formed by partially lowering the grades of approaching roads and providing drain pipes underneath. Their use over simple fords is preferred where day-to-day flow exceeds fordable depth, normally 4-6 in or 10-15 cm.

Construction procedures employed for vented fords are normally simple. They generally involve pouring concrete over corrugated metal pipes or precast concrete pipes laid either directly over the riverbed or over the concrete foundation laid in trenches excavated in the riverbed. The depth and length of the trenches are dependent on size of the pipes and their length, which in turn depend on design flow and road width. Instead of pouring concrete over the vents, they can also be encased with gravel fill or rubble masonry. Generally, both ends, upstream and downstream, are beveled for smooth passage of water, and a concrete slab is laid over the vents to provide a smooth finished roadway. Hot or cold mixed asphalt and concrete grouted rubble stones can also be used to pave the concrete roadway. Corrugated metal pipes are most commonly employed for construction of vented fords. Other types of pipes which can be used, depending upon availability of construction materials and relative costs, include iron pipes, PVC pipe, concrete pipes, and iron pipes, etc. Other factors which can affect selection of a specific type of vent include design life of the crossing, extent of possible damage during floodings, and annual maintenance cost.

The construction of vented fords across a river usually result in narrowing of the natural channelway at the crossing site, and creating flow disturbances with severe erosion potential. This has led to development of a typical configuration with sloped culvert entrance, sloped embankment, and splash apron along the downstream edge. The sloped entrance and embankment catch less debris and have a natural self-cleaning tendency during high water ensuring efficient operation. The splash apron on the downstream side moves the plunge pool further downstream, preventing the undermining of the roadway and culverts. However, where the soil is erodible, cutoff walls that prevent subsoil flow, are preferred over splash aprons. Fig. 3 shows typical details of a vented ford.

Large-span vented fords, with a span longer than 50 ft (15.2 m), are generally cast-in-place concrete structures which encase pipes or culverts. These crossings are designed with curbs or wheel guards. Experience has shown that if provision for curbs or wheelguards is made, sloping the curbs toward the downstream side would help reduce the collection of ice and debris during periods of overtopping. The approaches and roadway slab must be well anchored to prevent uplift and carrying away of the structure during flooding.

C. LOW BRIDGES

Low water bridges, like vented fords, are formed by partially lowering the grades of the approaching roads and providing adequate passage for day-to-day flow through the openings. However, instead of vents or culverts, a bridge deck structure is provided. The choice for this type over a vented ford is made when day-to-day flow cannot be handled by the latter efficiently and economically because of environmental or terrain conditions. For drainage basins with high debris potentials or in an environmentally sensitive area, e.g., where alteration of streambed is not acceptable, low water bridges are especially suitable.

Like a high level bridge, a low water bridge consists of the following components: a foundation to transmit the load from and above the structure to the natural soil below, a substructure to support the roadway slab and providing an adequate opening for passage of normal flow, a superstructure consisting of the roadway slab, curbs/railings, etc., and the approaches.

Rock or a hard soil strata which is not erodible at the flood flow velocity, is obviously most suitable for an LWSC when found at a shallow depth. For erodible soils, piles may be used, depending on the overall cost of the structure.

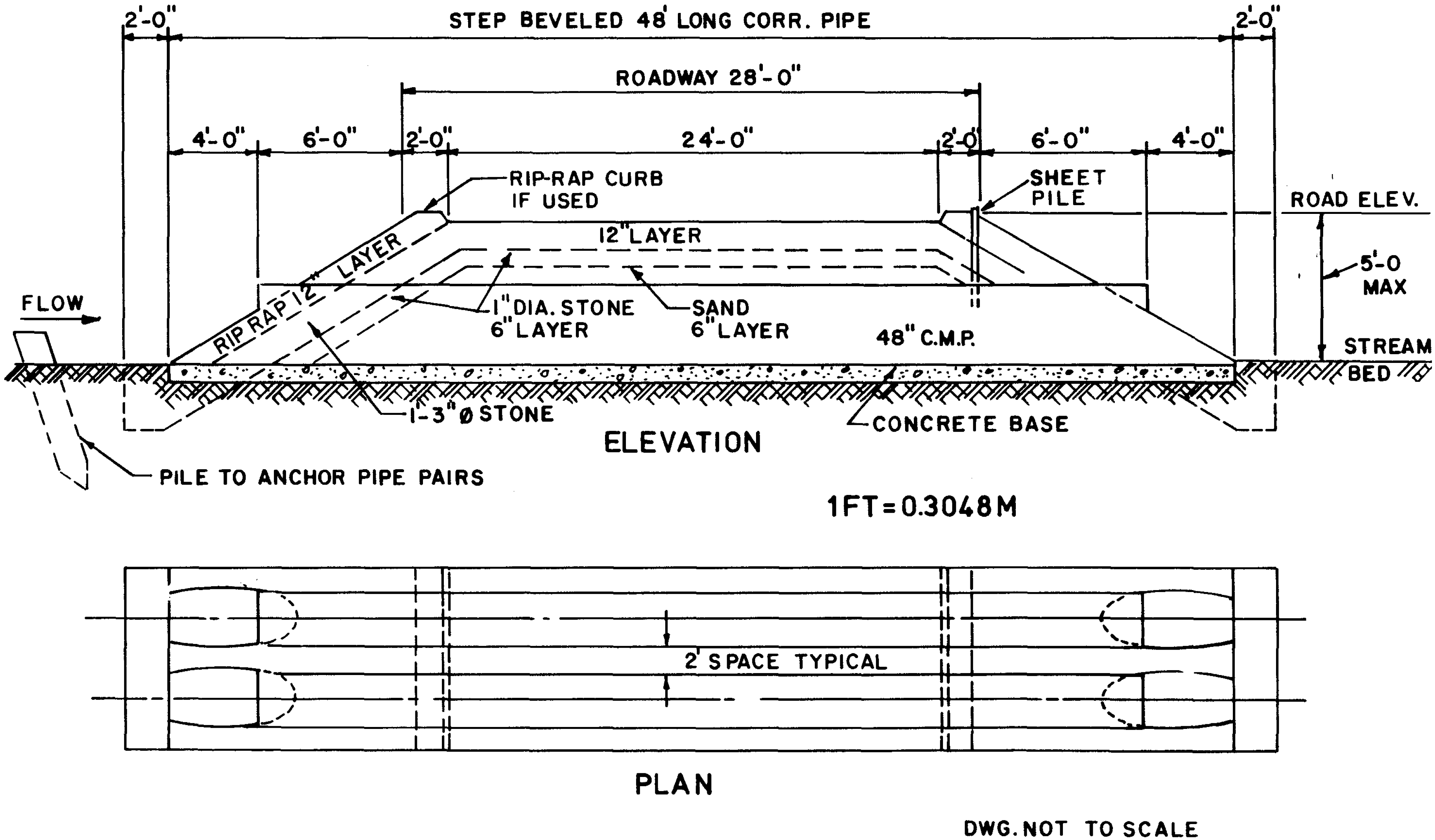


FIG. 3 VENTED FORD, GRUNDY COUNTY, IOWA

FIG. 3 VENTED FORD : JEFFERSON CO., IOWA

For superstructure, reinforced concrete slabs with mass concrete or masonry pipes and abutments are most commonly used in the United States. The deck slabs are heavy to withstand the uplift pressure of the water and the entrapped air below the slab as well as the lateral pressure exerted by the upstream water. Deck slabs are firmly anchored with the abutments and tied with the approaches.

Markers or guide posts on low water bridges are generally discouraged when constructed across streams where frequent occurrence of floating trees and debris can result in greater damages.

It is desirable to have approaches in cut sections since embankments or approaches in fill are liable to be washed away during each flooding. Silting may occur in such sections but can be reduced appreciably if approaches are aligned at a slight angle to the center line of the bridge so that the gradient falls in the direction of flow in the stream. However, where silting is not a problem, straight approaches would be preferable because of ease in traffic flow and construction of wing/return walls. Figs. 11 and 12 show pictures of two typical low water bridges.

V. CASE HISTORIES AND EVALUATION

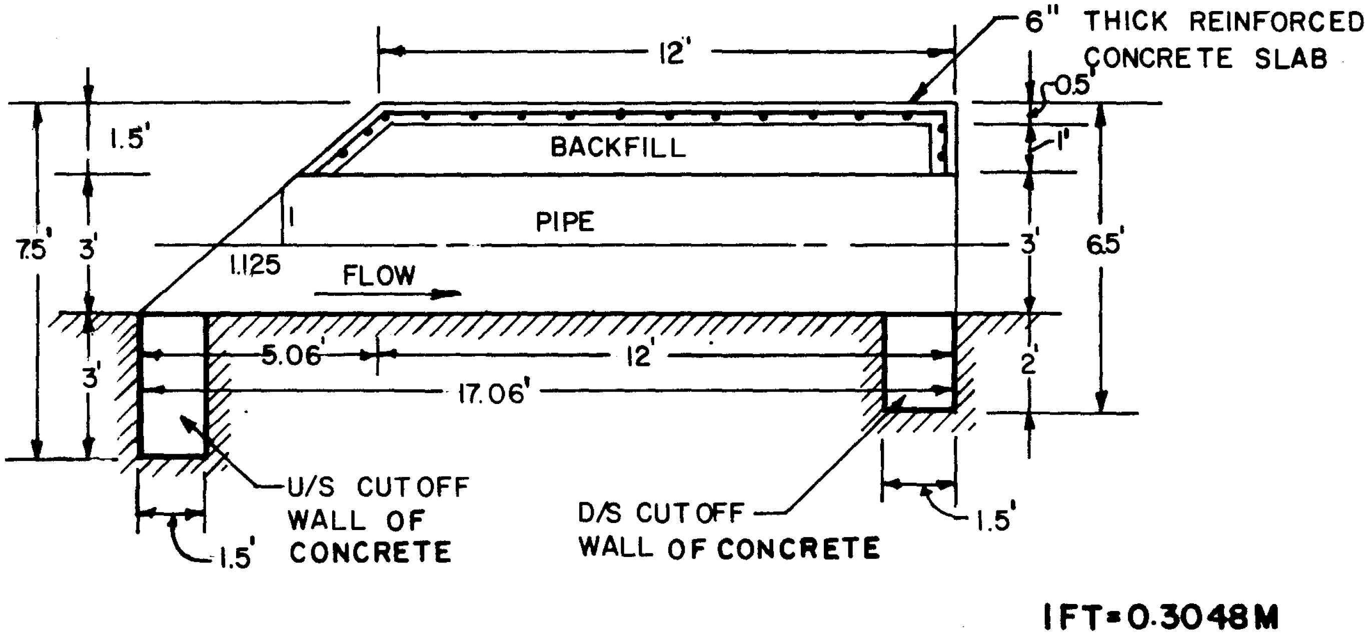
Although many LWSCs have been known to be in use for long periods of time in many parts of the country, well documented information on their selection, design, cost, construction and performance is often so scarce and fragmented that it cannot be readily used in design. Perhaps the low construction cost and the low level of technology that is thought to be necessary for designing an LWSC are the reasons for such lack of interest. It is found that no definite systematic approach has yet been developed for the selection and design of such structures. Rather, most of the current designs are performed by the engineers based on their personal experience and intuition. In order to consolidate the information on selection procedure, design methodology and performance, several state transportation departments and county highway agencies were visited to collect and assemble data related to design and performance of LWSCs. Case histories for more than 40 old and recent LWSCs were studied and five of these cases that are considered signifcant and and may be useful to highway engineers for designing LWSCs are included in this section.

CASE 1: ROUTE 752 OVER THE TYE RIVER, VIRGINIA

This 66 ft (20 m) crossing is located about two miles (3.2 km) north of Massies Mill, Virginia, on Route 752 over the Tye River, servicing farm vehicles. In July-August, 1976, the traffic count was six vehicles per day. The structure consists of 14 lines of 36 in (0.91 m) diameter corrugated metal pipes enclosed within reinforced concrete slabs. The details are shown in Fig. 4.

Neither hydrologic nor hydraulic analysis was made in designing the structure. The slab thickness and depth of the cutoff walls were arbitrarily determined. For reinforcement, No. 4 bars were placed with a spacing of one foot center-to-center, again without detailed computation.

(a) VIEW FROM DOWNSTREAM



(b) CROSS SECTION

FIG. 4 VENTED FORD ON ROUTE 752 AT THE TYE RIVER, VA

It was built in 1970 at a cost of $21,000. For economic reasons and low traffic, the structure is low, and reportedly under water approximately eight to ten times a year.

In 1972, highwater from hurricane Agnes overtopped 5 ft (1.5 m) above the grade, washed out approaches, and plugged pipes with debris consisting mostly of small trees and bushes. In 1975, it was noticed that galvanization on the pipes was worn away by rocks, and that the pipes were beginning to corrode. Because of aggradation upstream of the structure, approximately 50% of the pipes on the left bank were blocked by loose stones.

In the spring of 1978, due to high water reaching 3 ft (0.91 m) above the finished grade, both approaches were washed out approximately 12 ft (3.7 m) adjacent to the structure. Again, much debris was deposited at the inlets, although the main structure suffered no damage.

Since the structure was originally built without an abutment wall, the backfill under the concrete slab roadway and between the pipes was undermined after the approaches were washed out. The undermining was also enhanced by 6 in (15 cm) degradation downstream that apparently reduced the effectiveness of the 3 ft (0.91 m) cutoff wall downstream of the structure. To fill the voids under the slab, holes were drilled in the concrete slab roadway and concrete was poured in.

Discussion

The cost of this structure seems to be somewhat high for an ADT of less than 10. However, it withstood an overtopping flow of 5 ft (1.5 m) above the roadway although its designed frequency of overtopping was much less than one year. Although ~~The~~ erosion of approach fill is an inherent problem ~~for structures~~ designed to be overtopped frequently, both approaches of the structure suffered little damage, except for larger floods, because they were gravel filled.

The cutoff walls, particularly the upstream one should have been deep enough to reduce undermining by seepage of water. Also, a firmly built cutoff wall might have served as the foundation and increased the stability of the structure. It is also seen that concrete pipes are more suitable for gravel bed streams. In addition, the constraints of available cover may also dictate the use of a concrete pipe.

CASE 2: CROSSING OVER MOUND CREEK, MINNESOTA

A crossing over Mound Creek in Brown County, Minnesota, was built in 1971 at a cost of $12,000. It consists of two large 11' 5" X 7' 3" (3.5 m X 2.2 m) reinforced concrete pipe arch culverts (Fig. 5). The average daily traffic in 1980 was about 35. The embankment was raised so that about 60% of the 50-year flood would overtop the structure. This design, compared with traditional bridge design, claimed to have saved approximately $18,000.

Some structural damage is expected to occur during overtopping, although properly balanced hydraulic design can prevent serious damage. The required length and elevation of the roadway must be determined so that overflow velocities can be kept below the available scour velocity. This design considered the allowable scour velocity of overtopping flow to be 6 ft/sec (1.8 m/sec) over the compacted gravel-surfaced road. The elevation of the roadway at the overflow section was carefully determined so that it would be close to the same elevation as the tailwater to prevent overflow from eroding the downstream slope of the embankment. Where the tailwater was below the shoulder of the road by more than one foot (30 cm), riprap was used to prevent serious erosion. The designer also considered backwater of up to 2 ft (61 cm) as acceptable for the design.

FIG. 5 STATELY TOWNSHIP ROAD CROSSING OF MOUND CREEK IN BROWN COUNTY, MINNESOTA

Since neither hydrologic nor hydraulic data of the creek was available, the flood frequency and stage-discharge relationship were developed analytically. For magnitude and frequency of floods, Minnesota Highway Drainage Manual was used, and for stage-discharge analysis, Manning's equation was applied. Based on these computed data, hydraulic analysis was carefully made to assure that the intended structure conformed to preset requirements.

For a given discharge, the backwater (headwater elevation) was assumed. Based on this assumed backwater, discharges over the roadway and through the culverts were computed. If the summation of these discharges differed from the given discharge, another backwater was assumed and the computation repeated until the two values were reasonably close. The analyses reveal that overflow would occur on an average of once every two to five years. For a 100-year flood, the overflow depth would be 1.3 ft (0.4 m) with a mean velocity of about 3 ft/sec (0.91 m/sec).

Since its construction in 1971, the structure was overtopped twice. In both cases no repair was needed. (The picture of the culvert was taken in 1980).

Discussion

This structure differs from Case 1 in the embankment materials. The culvert in Case 1 was encased in concrete with the entire structure acting as one unit. In this case, the embankment is composed of sand and gravels. Since their unit cost is lower, the embankment may be constructed higher to accommodate larger culverts and reduce frequency of overtopping the roadway. Since the embankment is erodible, a certain degree of damage and thus repair is expected in case of overtopping. However, with proper design, construction cost savings may offset the expected damage and maintenance costs.

The construction of this structure is relatively simple and fast, reducing time of inconvenient detours or bypasses. Any future road widening or alteration can easily be accomplished by simply adding sections of the culverts and embankments. If culvert flow capacity is inadequate, additional barrels could be added relatively easily.

This type of structure, however, relies on accurate data, and somewhat lengthy computations are involved to perfect the design.

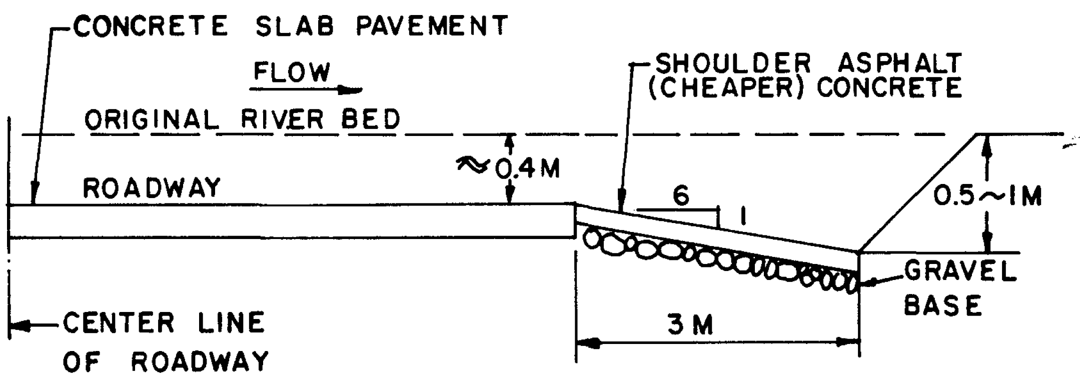
CASE 3: FORDS ACROSS DRY CREEKS IN ARIZONA

A secondary road crosses a dry creek bed by making a ford (also called a dip). Occasional flows caused by rainfall pass over the ford, when the road may be closed to traffic, depending on the depth of flow. It is the simplest type of crossing, most suitable for dry or ephemeral streams, and when the streambed is so wide that a high level bridge will normally be prohibitively costly. Many such crossings can be seen throughout the country, and more than 15 fords across dry creeks, particularly in Pima and Maricopa Counties in Arizona and Rock County in Minnesota, were investigated for their design criteria and performance. The estimated average daily traffic for these crossings varied from 50 to over 100. The findings are summerized here as Case 3.

Two factors must be considered in the design of a ford: 1) uplift force of water under the pavement, and 2) erosive action of flow over the roadway, especially on the downstream side. Because of their simplicity and low construction costs, no serious attempts have been made to estimate magnitude of uplift force nor to study in detail the erosive action of flow passing over the pavement. Concrete slabs of more than 3 in (7.5 cm) thickness have generally been used to prevent the pavement from floating up and sliding downstream. Old, cracked asphalt (as well as concrete) pavements were particularly susceptible to water flowing over them. An old asphalt ford on Litchfield Road over a tributary of the Buckeye Canal, Maricopa County, Arizona was washed out and rebuilt with concrete slab pavement (Fig. 6).

Erosion downstream of the roadway remains an unsolved problem particularly for low cost fords with limited construction funds. For some small fords, this is considered a natural situation and an average annual maintenance cost of about \$20 to \$30 per 100 sq. yds. of the fords had been spent in 1980 in Maricopa County for post-flood repairs. For larger fords, the downstream road shoulders are made with concrete slabs having a milder slope of six horizontal to one vertical up to 10 ft (3 m) beyond the edge of the roadway. In some

(a) LITCHFIELD ROAD DIP



(b) CROSS SECTION

FIG. 6 LITCHFIELD ROAD DIP AT BUCKEYE CANAL TRIBUTARY,
MARICOPA COUNTY, ARIZONA

cases, efforts have been made to reduce the erosive energy of the flowing water by embedding rocks in the concrete shoulders. For the reconstructed Litchfield Road Dip, the new roadway was placed 1-1/2 ft (45 cm) below the original bed as shown in Fig. 3(b), placing the edge of the shoulder about 3 ft (0.91 m) below the original river bed. During flood, water depth will be at least 1-1/2 ft (45 cm) and 3 ft (0.91 m) above the roadway and the edge of the shoulder respectively, thus minimizing erosion of the shoulder. One problem in this feature is that the deposited sediment on the roadway must be cleared out after each flood before traffic can be resumed. County engineers claim that cleaning the sediment by bulldozer is much easier and faster than filling the eroded materials. Such arrangement however is suitable only for dry creeks, but may create severe problems for other stream crossings.

Perhaps the most devastating problem encountered with fords is the instability of streams. Fords are small in size and vulnerable to slight change in the streambed. Less than 1 ft (30 cm) of streambed degradation may totally expose the pavement and undermine the foundation. This type of failure is quite common among fords across dry streams in flood plains.

A 100 ft (30.5 m) wide ford on Magee Road over Canada Del in the outskirt of Tucson, Arizona has a 3 in (7.5 cm) thick concrete slab. Canada Del is a dry stream which was relatively stable downstream of the crossing before urban development. After improvement of drainage network in the urban areas, the streambed downstream of the crossing drops a few inches every flood due to degradation. The streambed degraded more than 2 ft (0.61 m) in 1980, completely exposing the road foundation. In order to prevent the foundation from slumping down, two 1-1/2 ft (45 cm) thick rock walls were placed but they did not work as anticipated and failed (Fig. 7).

A 600 ft (183 m) ford at Clay Croft Crossing is situated over the confluence of Tangue Verde Wash and Pantano Wash, in the suburbs of Tucson, Arizona. The drainage area is about 800 square miles and the water flows in the stream only from November through January. Three feet (0.91 m) diameter culverts are used to drain the water during these months. The ford (vented)

FIG. 7 DIP ON CANADA DEL MAGEE ROAD, NEAR TUCSON, ARIZONA

partially) is made of about 6 in (15 cm) thick cement clay mixture. Since this crossing was built (definite year of construction is unknown), the streambed upstream of the crossing has been silted up and the downstream side scoured due to aggradation and degradation process. In the summer of 1980, the streambed upstream had the same elevation as the pavement, while the downstream side was scoured to about 10 ft (3 m) below the pavement. The pavement was badly damaged by the last flood, as shown in the photo in Fig. 8. The pavement and shoulder were also cracked in several segments and the foundation was undermined.

A 20 ft (6.1 m) ford over Pantano Wash at Vail, Arizona was built 25 years ago with a concrete slab of about 6 in (15 cm) thickness. It experienced at least 50 large floods without failure. This is attributed to the following factors: 1) the ford is built where streambed is relatively stable; 2) the reach is straight and bonded with well defined firm banks; and 3) the structure is a well-built thick concrete slab, perhaps reinforced with steel since no cracks were seen on the slab.

FIG. 8 CLAY CROFT DIP AT THE CONFLUENCE OF TANGUE VERDE WASH AND PANTANO WASH NEAR TUCSON, ARIZONA

In LWSC, saving construction cost is generally coupled with an increase in risk costs, which include cost of repair and the cost required for traffic safety. Warning signs are posted at fords and emergency blockades generally placed by law enforcement officers when the fords have water running. However travelers are sometimes unaware of the dangers or they deliberately ignore the signs. Some incidents occurred in Arizona where travelers ignored the blockades placed by state patrols at the LWSCs resulting in five deaths in two accidents.

Discussion

A ford should cross a stream where the streambed is stable. The reach should preferably be straight making a right angle with the roadway. If a ford is forced to cross an unstable stream, the pavement may be lowered slightly below the original streambed. The road should then be cleared of deposits after every flood.

Within the limit of available funds, a ford consisting of mass reinforced concrete slabs with very mildly sloped, wide road shoulder on the downstream side is preferred. One or 2 ft (30 or 60 cm) deep upstream as well as downstream cutoff walls are provided to reduce undermining.

A ford should not have a very sharp vertical curve. Traffic hazards may occur during both the dry and wet season if cars are traveling at high speed. Travelers often ignore dip signs and physical blocking of the road may be necessary during a flood.

CASE 4: SASKATCHEWĀN RURAL AFFAIRS, SASKATCHEWAN, CANADA

Most low water stream crossings in Saskatchewan, Canada are 60 to 120 ft (18 to 36 m) in length. The culverts are designed for average summer flow so that crossings are not inundated during the summer months.

The center core of the LWSC used in Saskatchewan is usually constructed of rock materials when locally available; otherwise earth fill is used. The disadvantage of using earth comes from danger of piping failure. When failure due to piping occurs, the concrete slab on the roadway and side slopes normally gets cracked and break up because of loss of support below. To avoid piping failure, gravels are placed over the earth over which concrete slab is placed. Care is taken to ensure that no uneven settlement can take place causing cracks in the slab. An effective measure to control piping is construction of an upstream cut-off wall at the base of the upstream sideslope to check seepage water. This is normally extended over the width of the main channel bottom and is arbitrarily made about 2 to 3 ft (0.6 to 0.91 m) deep.

Another important factor in designing an LWSC is the height of the crossing. Preferable crossing height at the center of the stream channel should not be more than 3 or 4 ft (0.91 or 1.2 m). Crossings higher than 3-4 ft (0.91-1.2 m) act more as a dam, and erosion of the road grades at the end of the crossing occurrs. In Saskatchewan, the crossings have been constructed with a dip in the center section so that the center is 1 to 1.5 ft (30 to 45 cm) lower than the ends of the crossing.

LWSC's seem to perform best when the ends can be terminated at a well defined creek bank as opposed to installing them on wide flood plains with no well defined banks.

Another variation of crossing used with some success in Saskatchewan is to omit placement of the concrete slab on the roadway surface and sideslopes, and instead having gravel on top of the rock for a driving surface for traffic. Normally this gravel driving surface must be replaced each year after being swept away by spring floods.

An innovative idea which has been used is to drive galvanized corrugated steel sheeting on the upstream and downstream vertical walls of a crossing. The space between the walls was filled with rock and a concrete slab was cast for a driving surface on top of the rock.

The following are some structural provisions which have proved to be very useful:

1. In case of erodible granular material, an upstream cut-off wall is very useful. In cases where there is cobble or rock, the upstream cut-off wall can be omitted. Failures due to piping have resulted in many crossings which did not have cut-off walls. In erodible material, this wall should be extended to more than 2.5 ft (0.76 m) in depth. Similarly, in highly erodible material, the cut-off wall should be extended beyond the width of the main channel and tapered up towards the ends of the crossing.

2. A further protective measure is to construct an end block at both ends of the crossing to serve as an erosion barrier in cases where erosion does not extend very deep. This end block would consist of increased depth of the roadway slab to 2 ft (0.61 m) or greater for a 1 ft (30 cm) width across the end of the roadway slab. The end block could be extended to the outside edges and constructed monolithically with the upstream seepage barrier. Typical cross sections of the two types of LWSC used by Saskatchewan Rural Affairs Department are shown in Figs. 9 and 10.

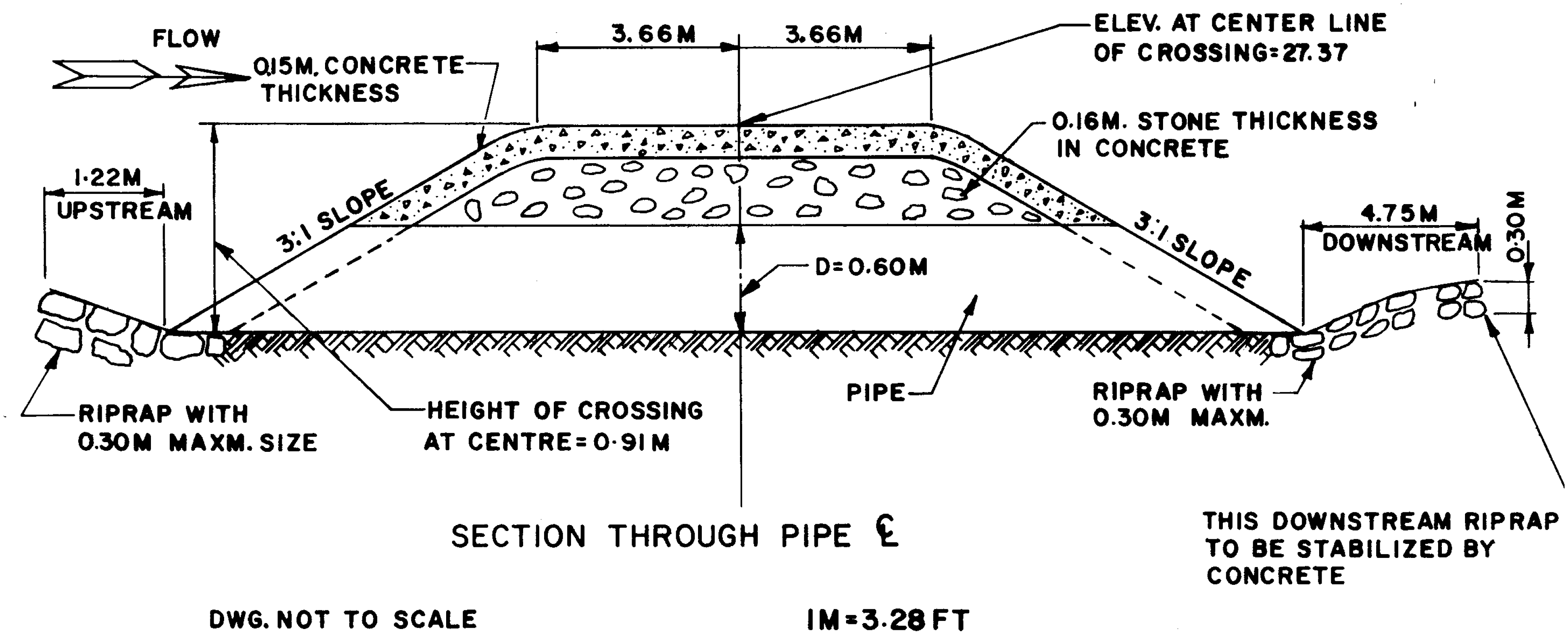


FIG. 9 VENTED FORD, SASKATCHEWAN: TYPE I

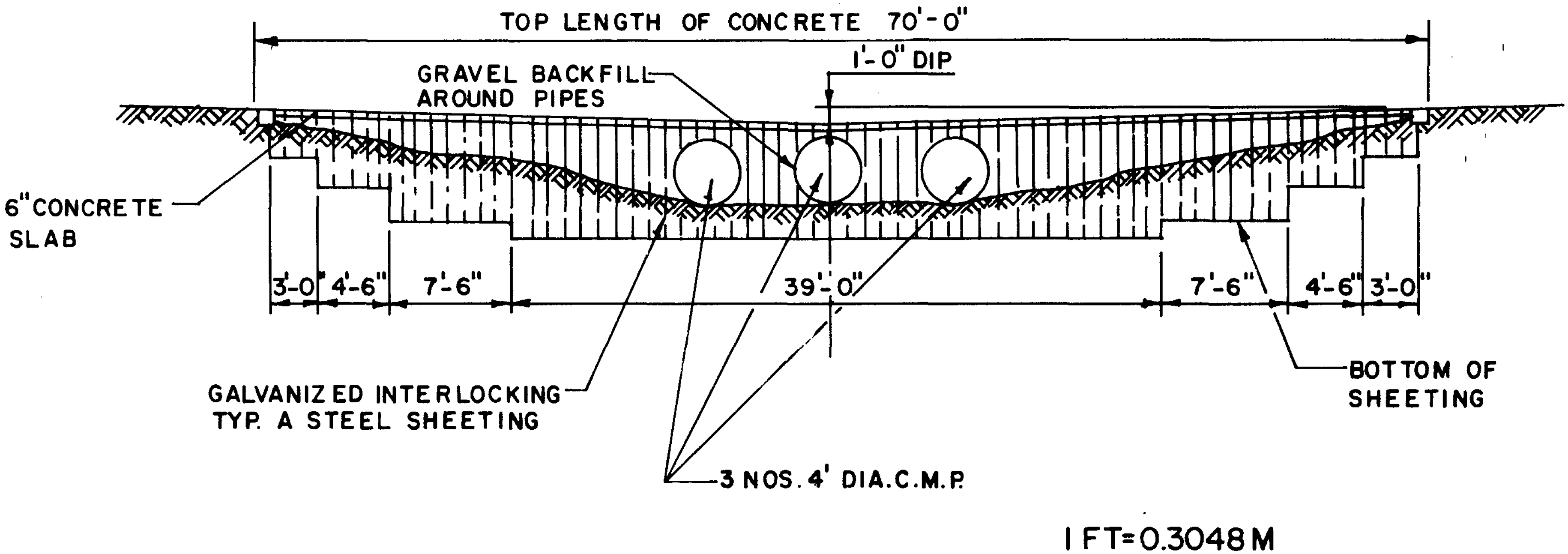


FIG. 10 VENTED FORD, SASKATCHEWAN: TYPE 2

Discussion

Two important considerations for a low level crossing for a particular site are the depth of water at the proposed location and the availability of rock in the area. The construction of an LWSC is not usually desirable if the depth of water involved is more than 3 to 4 ft (0.9 to 1.2 m). Higher depth would complicate construction and require a high embankment. Whenever there is a potential supply of rock in the area, the center core of the crossing should be constructed of rock rather than earth fill.

CASE 5: LOW BRIDGES

Two low bridges on County Route 644 at the confluence of the Hazel and Hughes Rivers at the southeast corner of Rappahanock County, Virginia were built in the early 1920s (Figs. 11 and 12). The bridge over the Hazel River is a one-span concrete slab bridge, measuring 27.5 ft (8.4 m) in length and 11.75 ft (3.7 m) in width. The bridge over the Hughes river is a two-span concrete slab bridge, 46.7 ft (14.2 m) in total length and 11.75 ft (3.7 m) in width. The thickness of the slab for both bridges is 1.5 ft (45 cm). The edge of the slab deck is raised 0.5 ft (15 cm) with 1 ft (30 cm) width to prevent passing vehicles from falling off the bridges. A pair of thick, well-built wing wall abutments extend 5 ft (1.5 m) beyond the edge of the bridge to support the deck and to protect the approach fills against erosion. As both bridges are slightly skewed, the wing walls were built with different angles, 30° on the concave side of the bank and 45° on the convex side, to guide flow smoothly through the bridges. The clearance under the deck to the riverbed is about 4 to 5 ft (1.2 to 1.5 m).

Both rivers appear to be moderately stable with no apparent horizontal or vertical shift of the river channels due to meandering. The rivers run through wooded areas and dead trees are seen strewn along the riverbanks waiting to be picked up by a high-stage flow. It is certain that debris problems prevail along these rivers. Under the decks of both bridges, large branches of trees were found still trapped, perhaps having been there for a time.

This county road, used mainly by farmers and school buses, had an average daily traffic count of 41 in 1978. The normal route distance is about 1.6 miles a(2.6 km) and a 9 mile (14.5 km) detour route is available when bridges are impassable. According to more than 50 years' observation by a farmer living nearby, the bridges have been inundated, on the average, three times a year, with overflow depth of about 6 ft (1.8 m). Each inundation takes about 12 hours with a total length of inundation time throughout the year of approximately one week. The average annual costs for repairing damage and cleaning debris were about $600 for both bridges.

Discussion

Despite frequent overflow for more than a half century, the bridges stood up well without requiring major repair. During the 1972 catastrophic flood caused by Hurricane Agnes, these structures were under water approximately 10 ft (3 m) at flood peak. The structures remained undamaged although minor repair work on the approach fills was needed to restore traffic.

Both bridges are constructed of massive concrete having thick slab decks supported by a pair of well-built abutments extending 5 ft (1.5 m) from the edge of the deck, making suitable angles with the deck to deflect stream flow smoothly around the abutments. The structures are located in the stable reaches of the rivers. The foundations of the structures are built on firm riverbed on which very little scour hole was observed at the abutments. Even where a tree branch is trapped under the deck, no serious scour is found. These are believed to be some of the contributing factors for the long survival of the structures.

FIG. 11 LOW BRIDGE OVER THE HAZEL RIVER, VIRGINIA

FIG. 12 LOW BRIDGE OVER THE HUGHES RIVER, VIRGINIA

A 5 ft (1.5 m) clearance under the deck seems too narrow for large debris to pass and is susceptible to debris problems. However, since these bridges were designed to be overtopped even by small floods, larger floating debris ordinarily carried by larger floods would pass over the road without being trapped under the deck.

For a low bridge, particularly where debris problems exist, railings on the deck should be avoided. Raising the edge of the deck slightly as in these bridges may be advisable to prevent the passing vehicles from falling off the bridge when the road surface is slippery. Slots could be provided in the slab for proper drainage.

VI. SELECTION OF LOW WATER STREAM CROSSINGS

Many LWSCs have been built at locations having low traffic volume and infrequent flooding. Substantial financial savings have been achieved by selecting such structures over the traditional high level crossings. Since an LWSC is a low level structure, it is relatively inexpensive. However, since it can not ensure uninterrupted traffic across the stream, it causes frequent inconvenience and loss of time and use. Further, since an LWSC is frequently overtopped and thus highly susceptible to damage due to erosion, it requires frequent repair and maintenance; although since the structure itself is low-cost, repair costs are not very high.

It is, therefore, obvious that a trade-off between relatively high capital cost, but less frequent repair need, and relatively uninterrupted traffic offered by a conventional bridge on the one hand, and a lower construction cost, but frequent repair needs and traffic interruptions causing loss of time and convenience on the other, determines whether a crossing under consideration should be an LWSC or not. The tangible factors that go into both sides of this equation, such as, capital cost, maintenance cost, flood damage cost, erosion protection cost, cost of debris and sediment removal, increased traffic time, etc., can be estimated. However, there are many intangible factors which play dominant roles in this trade-off and are difficult to assess. They include: inconvenience due to blocking of the road, potential danger to life as well as other political, social and legal considerations. The decision maker must evaluate all tangible and intangible factors in each case to determine the type of crossing best suitable for a site.

A. DECISION ANALYSIS

In secondary road stream crossings, highway engineers are often faced with the question as to whether a low water stream crossing should be considered. As discussed earlier, this question can only be answered when an economic decision analysis is performed by taking into account all the tangible and intangible factors that govern the selection of such structures.

Undoubtedly, when the total tangible costs of the alternatives, say an LWSC and a high level bridge, are approximately equal, a traditional high level design would be selected. But the question remains that how heavily the potential financial savings of using an LWSC should be weighed against all intangible factors.

In an effort to examine the opinions and experience of the engineers involved in the selection of such structure, a study (86) was conducted by Shen at the Colorado State University. Based on 60 responses from experienced highway engineers in 40 states, Shen found a set of criteria and conditions under which an LWSC was overwhelmingly favored by the respondents, as well as another set of conditions when a conventional high level bridge would be absolutely desirable. These criteria, termed as most favorable and least favorable criteria for the selection of an LWSC, are listed in Table 1.

For cases meeting the most favorable criteria, therefore, a decision to select an LWSC can be readily made after due consideration is given to other social, environmental and legal factors that may affect the selection. No further detailed economic analysis are necessary for such a condition. For cases, meeting the least favorable criteria, likewise, a decision for a conventional high level bridge will be taken. For cases falling between these two extremes, however, further analysis may be needed to evaluate the total expected costs of the two different types of structures.

TABLE 1. LWSC SELECTION CRITERIA

CRITERIA	Most Favorable for LWSC	Least Favorable for LWSC
Average Daily Traffic (ADT)	less than 5 vehicles/ day	more than 200 vehicles/ day
Average Annual Flooding	less than 2 times/ year	more than 10 times/year
Average duration of traffic interruption per occurrence	less than 24 hours	more than 3 days
Extra travel time for detour	less than 1 hour	more than 2 hours
Possibility of danger to human life	less than 1 in 1 billion	more than 1 in 100,000
Property damage	none	1 million dollars
Frequency of using it as an emergency route	none	once/month

More than 85 percent of the respondents indicated that they would be willing to consider changing their constraints for the selection of LWSC if more than 75 percent expressed different opinions from their own.

Analysis of these responses was extended by defining "Preference Index", q, as the quantitative extent of preference in choosing an LWSC over a high bridge. The preference index which is essentially a cost saving ratio needed to deviate from a traditional design and go to a higher risk LWSC design, varies from zero, when no LWSC will be considered, to one where LWSC is a definite cosideration. For the most favored conditions in Table 1, the preference index becomes 0.72. For other conditions, the Preference Index can be estimated by using the following equations:

$$q = K_1 \cdot K_2 \cdot K_3 \cdot K_4 \cdot K_5 \cdot K_6^{(0.72)} \qquad (1)$$

where K's are the multipliers to modify the value of Preference Index, q, for variations of conditions different from the most favorable conditions.

The variations of K-values are listed in Tables 2-7.

The Preference Index is compared with the ratio of the total annual tangible costs of LWSC to high bridges. If the preference index exceeds the ratio, then the decision maker is advised to choose LWSC. Details of the analysis with a sample problem are presented in Appendix B.

In selecting an LWSC for conditions falling between the most and least favorable limits of Table 1, Tables 2-7 can be used to determine the Preference Index, q. A comparison of 'q' with the ratio of total annual tangible cost of the LWSC to that of the high bridge, will determine whether an LWSC will be desirable under the circumstances. Further economic analysis may, at that time, be conducted to select the appropriate structure.

TABLE 2. VARIATION OF K_1 WITH TRAFFIC COUNT

K_1 = 0.9 for 50 ADT
K_1 = 0.81 for 90 ADT
K_1 = 0.73 for 130 ADT
K_1 = 0.66 for 170 ADT
K_1 = 0.59 for 210 ADT

TABLE 3. VARIATION OF K_2 WITH AVERAGE ANNUAL FREQUENCY OF FLOODING

K_2 = 0.90 for average number of annual floodings of 4.
K_2 = 0.81 for average number of annual floodings of 6.

. . . the value of K_2 is decreased by 10 percent of every increase of 2 annual floodings.

TABLE 4. VARIATION OF K_3 WITH QUALITY OF HYDROLOGIC ANALYSIS

K_3 = 0.8 for average hydrologic analysis.
K_3 = 0.4 for poor hydrologic analysis.

TABLE 5. VARIATION OF K_4 WITH AVERAGE DURATION OF TRAFFIC INTERRUPTIONS

K_4 = 0.8 for average traffic interruption greater than 48 hrs.
K_4 = 0.6 for average traffic interruption greater than 72 hrs.

TABLE 6. VARIATION OF K_5 WITH EXTRA TIME FOR DETOUR TRAVEL

K_5 = 0.7 for alternate route with travel time longer than 2 hrs.
K_5 = 0.6 for alternate route with travel time longer than 3 hrs.

TABLE 7. VARIATION OF K_6 WITH POSSIBILITY OF DAMAGE TO HUMAN LIFE

K_6 = 0.6 for possibility greater than 1 in 1 million. K_6 = 0.2 for possibility greater than 1 in 10,000 (One may wish to include the warning system in this assessment of K_6).

B. RISK ANALYSIS

As stated earlier, a detailed economic analysis is necessary not only to confirm the savings in the selection of an LWSC over a traditional high level bridge, but to determine a balanced design for the crossing or a Least Total Cost (LTEC) Solution. Such analysis takes into account all the tangible costs of different possible alternates. The total tangible costs consist of capital cost and risk costs. Risk cost is essentially the sum of all costs due to floods of a range of frequency to which the structure is exposed.

Such design approach, including risks as part of the total cost, has become increasingly popular among design engineers in various fields. For a bridge design, traditionally the design flood is selected first, and alternate designs are made on the basis of the design flood. For balanced design, however, flood magnitude is considered as a variable, and for each alternate design, the annual capital cost and risk cost are computed. Total expected costs for all alternatives are then compared to select the most economical design. A simplified risk analysis procedure developed for low volume roads consists of determining the Total Expected Cost (TEC)'s of a number of alternatives by summing up the annualized capital costs and the annual risk costs for each of the alternatives. Capital costs, including structural, pavement and embankment costs, can be annualized by multiplying it with the Capital Recovery Factor. Annual Risk cost can be computed from the following:

$$\text{Annual Risk Cost} = \Sigma \ (P_i - P_{i+1}) \frac{[L(Q_i) + (Q_{i=1})]}{2} \tag{2}$$

where, P_i = Exceedence Probability of Flood Q_i

Q_i = Discharge of i-year Flood, in cfs.

$L(Q_i)$ = Loss caused by Q_i, in \$

Potential flood losses to be considered in the computation of L(Q) includes loss of embankment and pavement, structural damage, interruption of normal traffic, damage to upstream property due to flooding, etc.

The material presented hereafter in this chapter is mostly extracted and adapted from the Hydraulic Engineering Circular No. 17 published by the Federal Highway Administration (33) to fit LWSCs. Some equations are simplified for easier understanding and application.

Although the procedure for balanced design is complex and exhaustive, it is believed that a substantial saving, particularly for larger structures, can be accomplished through such process and should form the basis for decision analysis for any structure.

The design procedure, however, requires considerable effort in obtaining and processing accurate data. Therefore, it is essential that the level of effort be commensurate with the probable cost saving that may result from the analysis. For smaller structures, it is particularly important to identify essential key variables for risk analysis so as to reduce the level of effort while retaining an acceptable degree of accuracy.

C. DATA REQUIRED

Recommended list of data needed for larger bridges is presented in Reference 33. For small stream crossing, this list must be reduced in accordance with the sensitivity of the variables to the particular site under study. Data required for balanced design of small stream crossings are as follows:

1. Site geometry and land use
2. Traffic
3. Hydrologic and hydraulic
4. Capital costs

1. Site Geometry and Land Use:

A map with 1 or 2 ft (30-60 cm) contour interval and a streambed cross section at the crossing will be needed. Land use upstream of the crossing, including crops and nearby buildings must be located on the contour map to determine the risks connected with flooding at various water levels. Pasture and woodland can be ignored in risk analysis.

2. Traffic:

The ADT, traffic mix, length of normal and detour route, and vehicle running cost are required. These data can be obtained from highway agencies or from the county office. On some occasions, the design may be constrained by type of vehicle, such as school bus and emergency vehicles.

3. Hydrologic and Hydraulic Data:

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

These data are, however, seldom available for smaller streams where LWSCs are considered. In such case, the stage-discharge relationship must be obtained from the slope-area method. Manning's roughness coefficient 'n' can be estimated for the channel and overbanks from field visits.

From the channel cross section, the cross sectional area and wetted perimeter of flow at any given depth can be determined. The slope of stream can be estimated from the contour map. Applying Manning's formula, then, the discharge can be computed for various stages from:

$$Q = \frac{1.49}{n} \left(\frac{A}{P}\right)^{2/3} S_0^{1/2} A \qquad (3)$$

where: Q = Discharge, cfs

A = Cross sectional area, ft^2

P = Wetted perimeter, ft

S_0 = Slope of stream ft per ft

For a stream with wide flood plain Manning's roughness coefficient must be determined for each sub-section and the discharge computed separately for the flood plain and main channel. The total discharge is the sum of the discharges in the overbanks and in the main channel.

Flood Frequency. In estimating flood frequency, the regression equations presented in HEC 17 (33) or the USGS method can be used. These methods give a 10-year flood, Q_{10}, in various regions and states throughout the United States. It would be advisable, however, that Q_{10} be computed by using actual data collected at the stream crossing or nearby river.

Floods of other frequencies are then computed by the following equation:

$$Q_t = a \; (Q_{10})^b \tag{3}$$

where: $a = 0.4921$ and $b = 1.00243$ for $t = 2.33$ yrs.
$a = 1.4596$ and $b = 1.02342$ for $t = 50$ yrs.
$a = 1.6438$ and $b = 1.02918$ for $t = 100$ yrs.

Floods of intermediate frequencies can be interpolated from these four points.

Hydrograph. A hydrograph is required to estimate the overtopping time. If a measured hdyrograph is available, overtopping time can be obtained directly from this hydrograph. If a measured hydrograph is not available, a triangular hydrograph developed by the Soil Conservation Service may be used for an approximation. For a triangular hydrograph, the base, T_b, of the hydrograph becomes approximately

$$T_b\text{, hours} = 6.92 \; \frac{L^{1.16}}{(DH)^{0.385}} \tag{5}$$

where: L = length of longest watercourse, in miles.
DH = elevation difference, in ft. between the main channel at its most distant boundary and drainage structure site.

3. Capital Costs

Capital costs include structural, pavement, and embankment costs. The cost of embankment protection, such as riprap, and special warning signs required for low water stream crossings are also included in capital costs. Data on such costs can be available from the construction unit of any highway agency and may vary from place to place. Therefore, site specific data have to be collected to make a meaningful cost analysis.

D. COMPUTATION OF LOSSES

Potential losses for an LWSC are:

1. Loss of embankment and pavement
2. Structural damage
3. Traffic related losses
4. Damage to upstream property due to flooding

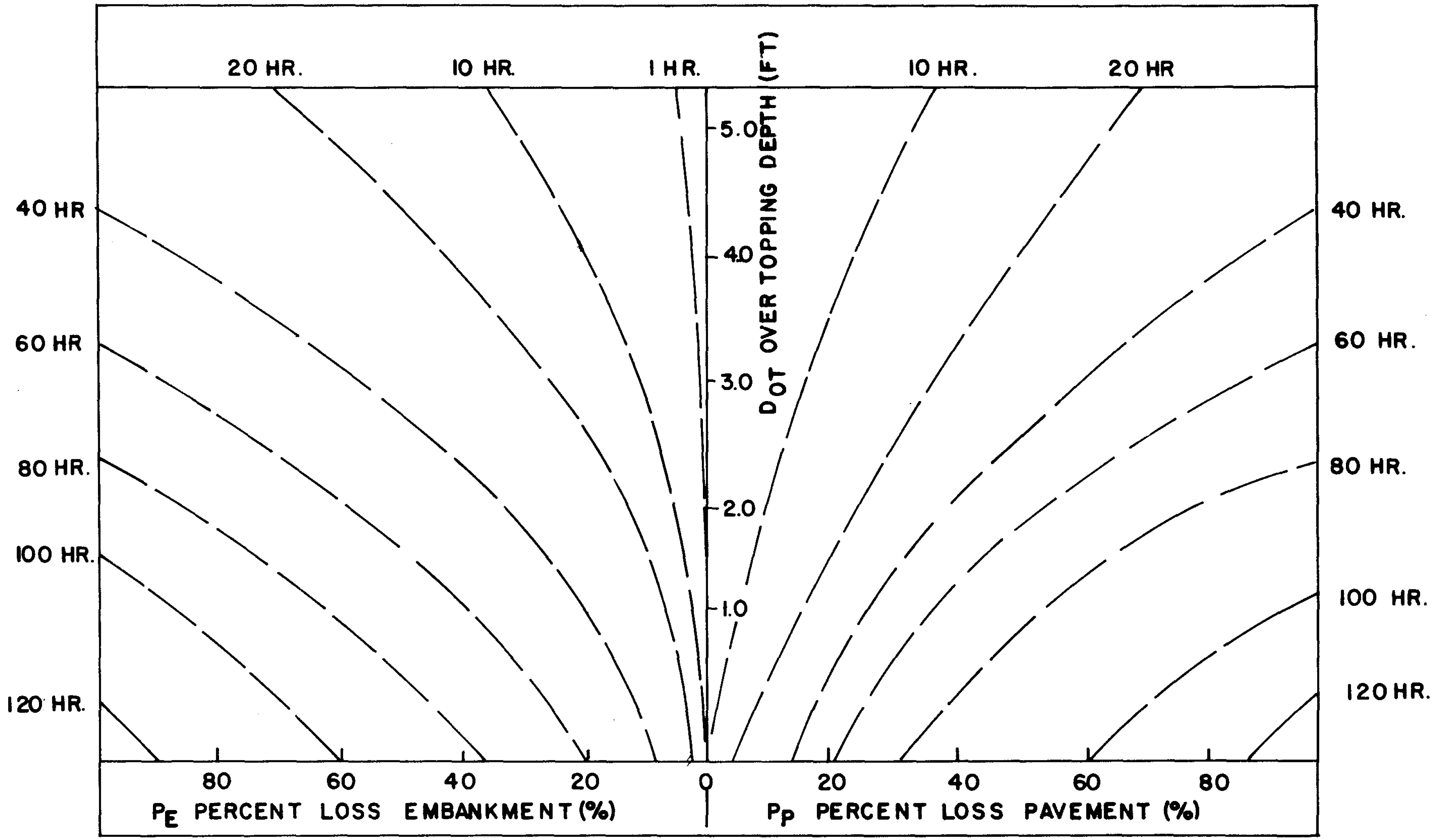
1. Loss of Embankment and Pavement

Flood flow over roadway for sustained periods often causes substantial damage to the pavement and embankment. The extent of damage in terms of flow and sustained time has not been studied extensively. However, tentative data relating the percentage of damage, overflow depth and sustained time are shown in Fig. 13, which has been developed by the Federal Highway Administration for the evaluation of economic losses to embankment and pavement (33).

The losses, L_{ep}, to the embankment and pavement may be estimated from the equation:

$$L_{ep} = (P_e C_e V_e + P_p C_p A_p) \frac{Ca}{100} \qquad (6)$$

where: P_e = percent of embankment loss
C_e = cost of embankment, \$/cu. yd.
V_e = total volume of embankment Subject to overflow, cu yd



1 FT = 0.3048 M

FIG. 13 PAVEMENT EMBANKMENT LOSS

P_p = percent of pavement loss
C_p = cost of pavement, \$/sq. yd.
A_p = total area of pavement subject to overflow, sq. yd.
C_a = adjustment factor for quick repair

2. Structural Damage

For a vented ford where culverts are used, the structural damage incurred from inundation may be considered the same percentage as damage on the embankment. Culverts are likely to be washed out when embankment materials over them are eroded. The cost of structural damage may then be computed by the equation:

$$\text{Structural damage cost in \$} = C_s P_e \tag{7}$$

where: C_s = total costs of structure, in \$
P_e = percent of embankment loss

3. Traffic Related Losses

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

(1) Increased running cost due to detour
(2) Lost time of vehicle occupants
(3) Increased accidents on detour

For secondary roads in rural areas, the last two categories are insignificant and may be ignored in evaluating economic risk costs.

Increased Running Cost. The increased running cost is the difference between running cost on the detour and the normal route for the period of detour travel after the flood. It is related to ADT, travel distance, duration of detour, and speed of vehicle. Much of these data are available except that duration of detour needs to be calculated. Duration of detour will normally include the overtopping time and restoration time.

The restoration time should be the sum of actual restoration time of structure and waiting time for restoration. The total restoration time, T_r in days may be estimated by:

$$T_r = V_e P_e / R_e + A_p P_p / R_p + T_w \tag{8}$$

where: R_e = rate of embankment repair, CY/day (cubic yard per day)

R_p = rate of pavement repair, SY/day (square yard per day)

T_w = waiting time in days

For secondary roads, the waiting time may be substantially longer than actual restoration time for repair because of their low order of priority. In such cases, the restoration time can be simply assumed as waiting time.

The overtopping time, T_{ot}, can be estimated by:

$$T_{ot} = \frac{Q - Q_{ot}}{Q} T_b \tag{9}$$

where Q_{ot} = overtopping discharge, cfs (cubic feet per second)

The overtopping discharge is discharge when the water stage is just at roadway elevation and the water is about to overtop the roadway.

For a small-drainage area, the base time, t_b, is short and thus the overtopping time becomes short. In such case, the overtopping time as compared to restoration time may be very small and can be omitted in computing the total risk cost due to interruption of normal traffic.

Average daily losses due to increased running costs, RC, can be computed by the following equation:

$$RC = (ADT)(DD)(\text{unit cost of travel})/1{,}000 \tag{10}$$

where DD = increased distance in miles

The unit cost of travel is in dollars per 1,000 vehicle miles.

Lost Time of Vehicle Occupants. The average daily lost time of vehicle occupants can be estimated by multiplying the extra time needed for detour, DT, the average number of occupants per vehicle, NC, and ADT. Then, the cost of lost time, TC, per day can be determined by multiplying the average lost time by the unit cost for time of the occupants.

$$TC = (NC)(ADT)(DT)(\text{unit cost of occupants' time}) \tag{11}$$

Increased Accident Cost. For secondary roads, where increase in accidents on detour is minimal, the increased accident cost may be omitted.

The total risk cost due to interruption of normal traffic flow, may be then estimated from the following equation

$$L_{tr} = (RC + TC) \times (T_r + T_{ot}) \tag{12}$$

where: L_{tr} = loss due to interruption of normal traffic, in dollars

T_{ot} = overtopping time of flood, in days

4. Damage to Upstream Property Due to Backwater

A bridge crossing generally causes backwater and results in damage to property adjacent to the crossing. The highway agency is responsible for the portion of the damage attributable only to the crossing. For the assessment of such damage, computation of backwater resulting from the proposed structure is needed. Backwater computation can be quite exhaustive depending on the accuracy required. Therefore, for a small bridge crossing where the drainage area is small and stream data unavailable, the backwater computation may be simplified. The backwater surface may be estimated analytically based on discharge, configuration of the structure, and tailwater surface elevation. Details are presented in Section VII.

Backwater damage losses generally include losses to crops and buildings. However, the crop value is much less than building value, and the losses to ordinary buildings outweigh crop losses. Therefore, if no building is involved upstream within the backwater affected region, backwater losses may be omitted for small structures in rural areas.

When buildings are involved, losses can be computed by the following equation:

$$\text{Loss to building} = (\text{cost of building}) \times (\text{percent damage}) \qquad (13)$$

Building cost should include market value of the building and the property it contains. Generally 50% of building cost is considered appropriate for the property value. The percent damage, P, to the building for water depth less than 3 ft (0.9 m) in the first floor can be approximated by the following equations:

For residence with basement (14)

$$P = 12.5 + 5d$$

For residence with no basement (15)

$$P = 8 + 5d$$

where d = water depth in the first floor, ft.

The highway agency is not generally responsible for flood damage incurred under normal flow conditions before the bridge is constructed. Therefore, losses attributable to the crossing should be the losses computed above minus losses which would result under natural conditions.

For water deeper than 3 ft (0.9 m) the approximate percentage of damage may be estimated by subtracting 1% for each foot of depth over 3 ft (0.9 m) from the value computed by the above equations.

E. COMPUTATION OF TOTAL EXPECTED COST

The total expected costs include annual capital costs and risk costs. Maintenance costs should also be included in the above, but since a certain amount of routine maintenance is necessary for all structures, it is not considered in this analysis.

1. Capital costs

For a LWSC, capital costs are composed of:

(1) Bridge (and/or culverts) and foundations
(2) Embankments

(3) Roadway pavement

(4) Protective measures (riprap, aprons, debris racks, etc.)

To determine the annual capital costs, total capital costs are multiplied by an appropriate capital recovery factor. The capital recovery factor (CRF) is computed by:

$$CRF = \frac{i}{1 - (1+i)^{-n}} \tag{16}$$

where i = interest rate

n = service life of the structure, yr

The annual capital cost is then:

Annual capital costs = (CRF)(capital costs)

2. <u>Risk costs</u>

To obtain risk costs, the product of the loss function and the probability function must be integrated over the entire spectrum of probability to which structure is exposed. These functions, however, are random in nature and cannot be expressed in simple forms. Therefore, the following approximation is recommended for evaluation of risk costs (R):

$$R = (P_i - P_1) \quad \frac{[L(Q_i) + L\ (Q_{i+1})]}{2} \tag{17}$$

where $L\ (Q_{n+1})$ is assumed to equal $L(Q_n)$

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

P_i = exceedance probability of flood Q_i

$L(Q_i)$ = loss caused by flood Q_i

The first Q_1 is the discharge of a one-year flood and the last Q_{n+1} is the discharge having a recurrence interval of 200 year or more.

F. LEAST TOTAL EXPECTED COST

The sum of the annual risk cost and annual capital costs results in the total expected cost (TEC). Comparison of the TECs for all design alternatives allows the designer to select the design with the least total expected cost (LTEC). Logically the design with LTEC should be the final design unless dictated by other social and political factors.

VII. HYDRAULICS AND EROSION PROTECTION

A. HYDRAULICS OF LWSC

For vented fords and low bridges, there are three distinctly different hydraulic phases of operation. During low dry weather flow, there is a period when these structures will pass the flow through the openings as an open channel under atmospheric pressure. It is commonly known as low flow condition. For some flows, and at some point on the rising limb of the stage hydrograph, this low flow condition changes into a pressure flow, when the upstream headwater affects the flow through the openings, increasing velocity and discharge. When the rising flood elevation exceeds the elevation of road grade, overflow or weir flow along with the pressure flow is initiated and a combined pressure-weir flow condition occurs until flood recedes. For a ford, however, with no opening below the road, only weir flow condition takes place, with the flow depth varying for various flow magnitudes.

During low flow and pressure flow conditions, the flow behavior is relatively simple and its characteristics and behavior have been well documented. When the flow enters into the combined pressure and weir flow condition and overtopping of road surface occurs, flow pattern becomes complicated. It has been seen that when the head is low, nappe of the overflowing water clings to the downstream face of the structure, and burrows into the streambed immediately downstream. With the increase in head, however, this nappe springs clear of the face of the structure at some flow, and falls at an angle to the stream bed. The nappe is then broken into vortices by the impact of the high velocity flow and acts as erosive force in that region.

In the following paragraphs, general discussions of flow over broad crested weirs and pressure flow through culverts are presented to cover the types of flow experienced by the three types of LWSC.

Weir Flow

Flow over a ford can be estimated by using the discharge formula for a broad crested weir, which is

$$Q = C_f \cdot L \cdot H^{3/2} \tag{18}$$

where

Q = discharge, cfs

C_s = coefficient of discharge for submerged flow.

C_f = coefficient of discharge for free flow where the ratio C_s/C_f is called the submergence factor.

L = length of inundated road way, ft

H = total upstream head measured above the crown of the roadway, ft

The coefficients can be obtained from Figure 14.

Where the depth of flow varies along the roadway, it is advisable to divide the inundated portion into several sub-sections and compute the discharge of each section separately. The sum of the discharges over the sub-sections is the total discharge through the crossing.

For the design of protection measures against erosion, the velocity at the downstream edge of the roadway embankment should be used. The mean velocity, V_2, at the brink (downstream edge of the roadway) can be computed by:

$$V_2 = \frac{Q}{Ly_2} \tag{19}$$

The brinkflow depth, y_2, can be determined in accordance with the elevation of the tailwater as:

a. If the tailwater is higher than the roadway elevation, brink depth y_2, equals the tailwater elevation minus roadway elevation.

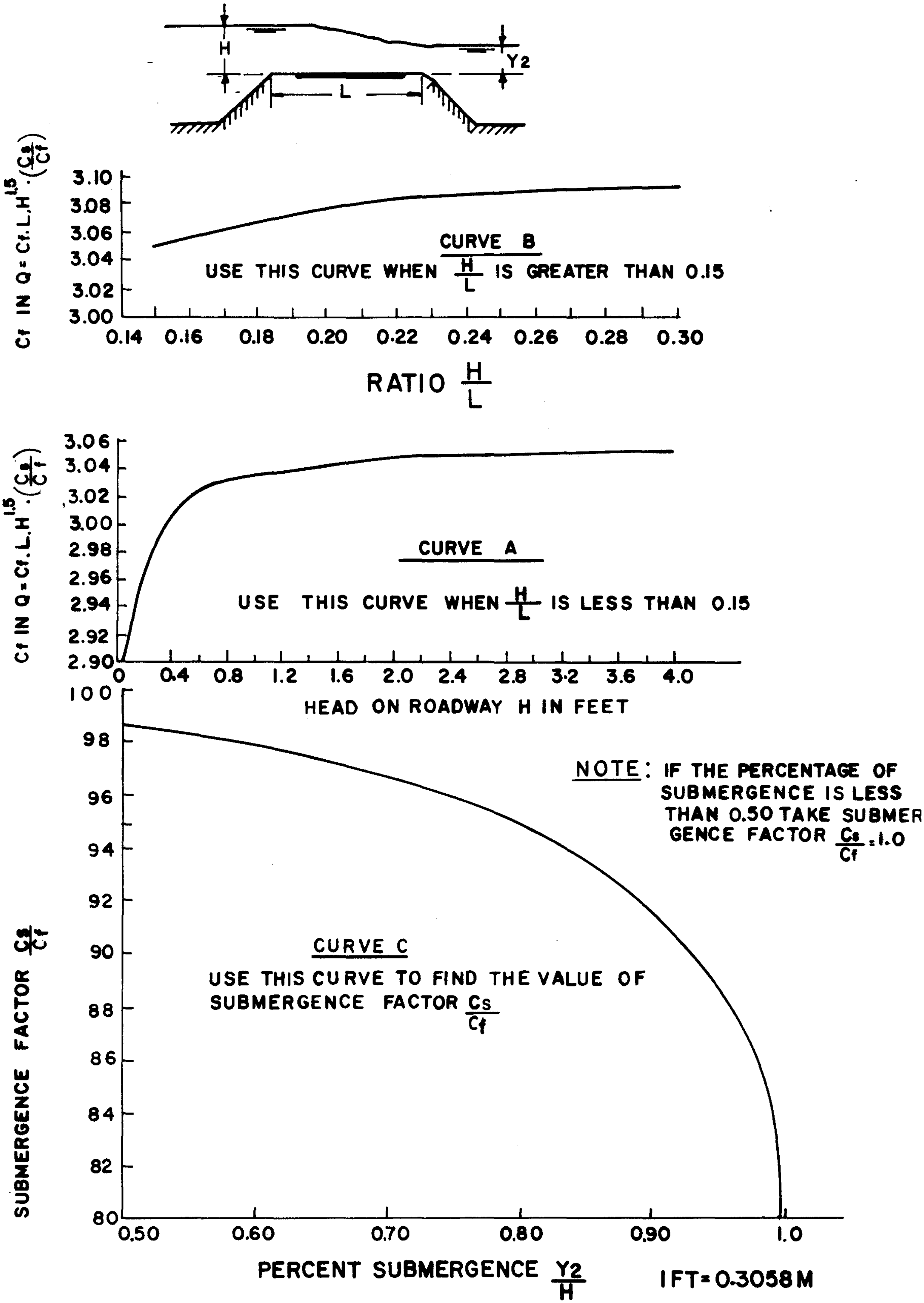


FIG. 14 CURVES FOR WEIR FLOW COEFFICIENTS

b. If tailwater is lower than the roadway elevation, brink flow depth, $y_2 = 0.48\ H$, where H = upstream depth of water above the roadway.

Where flow depth varies along the roadway, the critical section should be used to estimate the maximum brink velocity. Design of erosion protection measures should be based on this maximum velocity.

Pressure Flow Through Culverts

For a vented ford, normally the flow is through the culvert, undergoing a low or pressure flow. But during flood, the water may overtop the road and then the flow will be comparable to broad crested weir at the top and pressure flow through the culvert.

There are two types of culvert flows: a) flow with inlet control when the tailwater is low, and b) flow with outlet control when the tailwater is high. For vented fords, normally outlet control prevails because the structure is low and the tailwater is high so that the culvert outlet is submerged. The relationships between differential head of water and discharge for culverts of various types and sizes has been established based on culvert research data. Federal Highway Administration's publication (31): Hydraulic Engineering Circular No. 5, "Hydraulic Charts for the selection of Highway Culverts", presents these relationships in the form of design charts. These charts can be used to determine the flow capacity of culverts of various types and sizes under inlet and outlet conditions.

The exit velocity of culvert flow can be estimated by dividing the discharge by the cross-sectional area of the culvert. The erosion protection measures are dependent upon this exit velocity.

When a vented ford is overtopped, the discharges through the culverts and over the ford can be determined by using a trial-and-error method. First, a headwater depth is assumed and the discharges through the culverts and over the ford for the assumed headwater are determined separately. The sum of these discharges should equal the total discharge in the stream. If not, this procedure should be repeated until the sum of the discharges matches the stream discharge.

For erosion protection measures, the brink velocity of overflow and the exit velocity at the culvert outlet are computed. The design of protection structure for the embankment are made based on the brink velocity of overflow. Exit velocity is used for designing a protection structure at the culvert outlet.

B. EROSION

One of the most important causes of the failure of stream crossing structures and a factor of great concern in the design of LWSC's, especially in streams with erodible beds, is the erosive effect of the flow through and over the structures. Due to narrowing of the natural channel opening at the stream crossing, the velocity of the flow at the crossing increases significantly. The velocity creates a scouring effect on the bed and embankments, and depending on the bed and embankment materials, may undermine the structure or foundation by erosion. The flow patterns through an LWSC at different phases of flow also cause vortices in the immediate vicinity of the structure, resulting in the creation of scour holes and causing failure of the structure foundation or embankment. Such vortices are frequently associated with jets of water flow through the constriction and over the roadway.

Apart from erosion due to the surface flow, subsoil flow can also cause or contribute towards the cause of the failure of the foundation through uplifting and by undermining of the soil by piping.

Erosive action on the road approaches caused by bends on the upstream side of the structures is also important. During heavy floods, high velocity jets cause continuous erosion of the bank at the approach segment, by clinging to the concave bank due to centrifugal action. The presence of the LWSC structure itself accelerates this process, and increases the silting on the other side. One such instance is known where the river was found to cut into the bank to a considerable distance upstream of the structure. Deflection of the flow with river training works at the bend is one way to prevent such failures. Such measure, however, if needed, may not be very cost effective for our LWSC.

Other than minimizing the erosive forces by careful design so that the velocities of flow through and above such structures remain within a tolerable range, the prevention of foundation and embankment failures can be achieved with proper foundation and structural design. Several methods that can be used are:

- Building the foundation deep enough and beyond the possible reach of scour holes.
- Providing a foundation on piles (timber, steel beams, etc.)
- Protection of embankment slopes by riprap.
- Provision of upstream and/or downstream aprons.

Possible scour depth at any crossing is a function of the velocity and the foundation material. For the protection of foundations of LWSCs in erodible soils, a depth of 4 to 5 ft is typically considered adequate.

Design of the pile foundation will also depend to a great extent on the soil and the depth to the unerodible material strata, and may be very expensive for an LWSC.

Riprap is the most common measure used for the protection against erosion of embankment as well as streambed, and therefore, the foundation. Discussions on the type and design of riprap protection is discussed in the following pages.

Riprap

Riprap has been used to prevent erosion, scour or shedding of the top layer of a structure or embankment. Covering the surface with hard materials, such as asphalt, concrete or soil cement is often costly and impractical for LWSCs where economy is the main consideration. In such cases, riprap has been used with a large degree of success by highway engineers in many places.

The following types of riprap are suitable for slope protection:

1. Dumped riprap
2. Hand-placed riprap
3. Wire-enclosed riprap or gabions
4. Grouted riprap
5. Concrete riprap in bags (sacked concrete)
6. Concrete slab riprap

These riprap types have been listed in ascending order of construction costs. For LWSCs where economical design is the chief requirement, dumped riprap and hand-placed riprap are preferred over other ripraps and will be discussed in this section.

1. Dumped riprap is graded stone on a slope, prepared in such a manner that segregation of stones does not take place. Dumped stone riprap is the most flexible type and will adjust itself to uneven bank settlement. Damage by floods can be repaired by simply adding new stones to original elevation.

2. Hand-placed riprap is laid carefully by hand or by derrick following a more or less definite pattern with the voids between larger stone filled with smaller stone, and the surface kept relatively even. Hand placed riprap is firmer than dumped riprap, although it is susceptible to movement of the surface it is supposed to protect. A performance survey by the Corps of Engineers of the majority of large dams in the United States showed that hand-placed riprap was not as satisfactory as an equivalent thickness of dumped riprap. The equivalent thickness was specified as one-half that required for dumped riprap.

An underlying layer of a filter of graded pervious materials like gravels, sand, etc. is essential for ripraps, unless the bank material meets the filter requirements. Filter fabrics can be used instead of graded soil filter because they are economical, handier and durable. Similar to graded filters, plastic filters are highly permeable for water while still keeping the soil away. Plastic filter fabrics have been used to protect structures from erosion in connection with rivers, lakes, canals, dams and drainage systems of all types. They are chiefly used in bank protection, subdrainage protection and foundation soil. It has been seen that these filters retain their strength after long exposure to saline or fresh water. There are reports of successful applications of plastic filters to protect beaches in Florida, control erosion problems at bridges by the U.S. Corps of Engineers, and to support roads in spongy Alaskan soils.

Fabrics are manufactured from synthetic fibers. They can be either a woven type where the filtering area is distinct, or nonwoven type where the filtering area is winding. They are made of synthetic fibers like PVC, nylon, polyester, polypropylene yarns which are generally termed as plastic. The wovens are made of interfaced filaments or multifilament yarns. In the case of nonwoven fibres, filaments are randomly arranged into sheets. Filaments may be bonded in one of the following ways:

1. Resins are added to join the filaments.
2. Filaments are heated and compressed so that they are fused together.
3. Filaments are punched by means of barbed needle.

The handling of fabrics depends on project size. On a small job, crews place the fabric manually. On a road project, after the fabric is placed, backfilling, spreading and compaction with standard equipment completes the job. For a paving work by asphalt, a spreader places the mix over the fabric.

Size of Stone

The size of riprap needed to protect an erodible surface can be determined by using Fig. 15. This figure shows the relationship between stone size, d_{50}, and the velocity of flow on surface slope (in the direction of flow) as the third variable.

Fig. 15 is developed for stone having unit weight of 165 pounds per cu ft and with the angle of repose of 41°. If the angle of repose of stone is other than 41°, the following correction should be made:

$$d_{50} = d_{50} \cdot (\mathrm{Sin}\,\phi \;/\; \mathrm{Sin}\,41°). \;\; [\mathrm{Sin}\,(41° - \phi)/\mathrm{Sin}\,(\phi - \alpha)] \tag{20}$$

where d_{50} = corrected stone size, ft
= angle of repose of stones
= embankment slope

As discussed in the previous section, flow velocity should be determined properly at a section where the erosion condition is most critical.

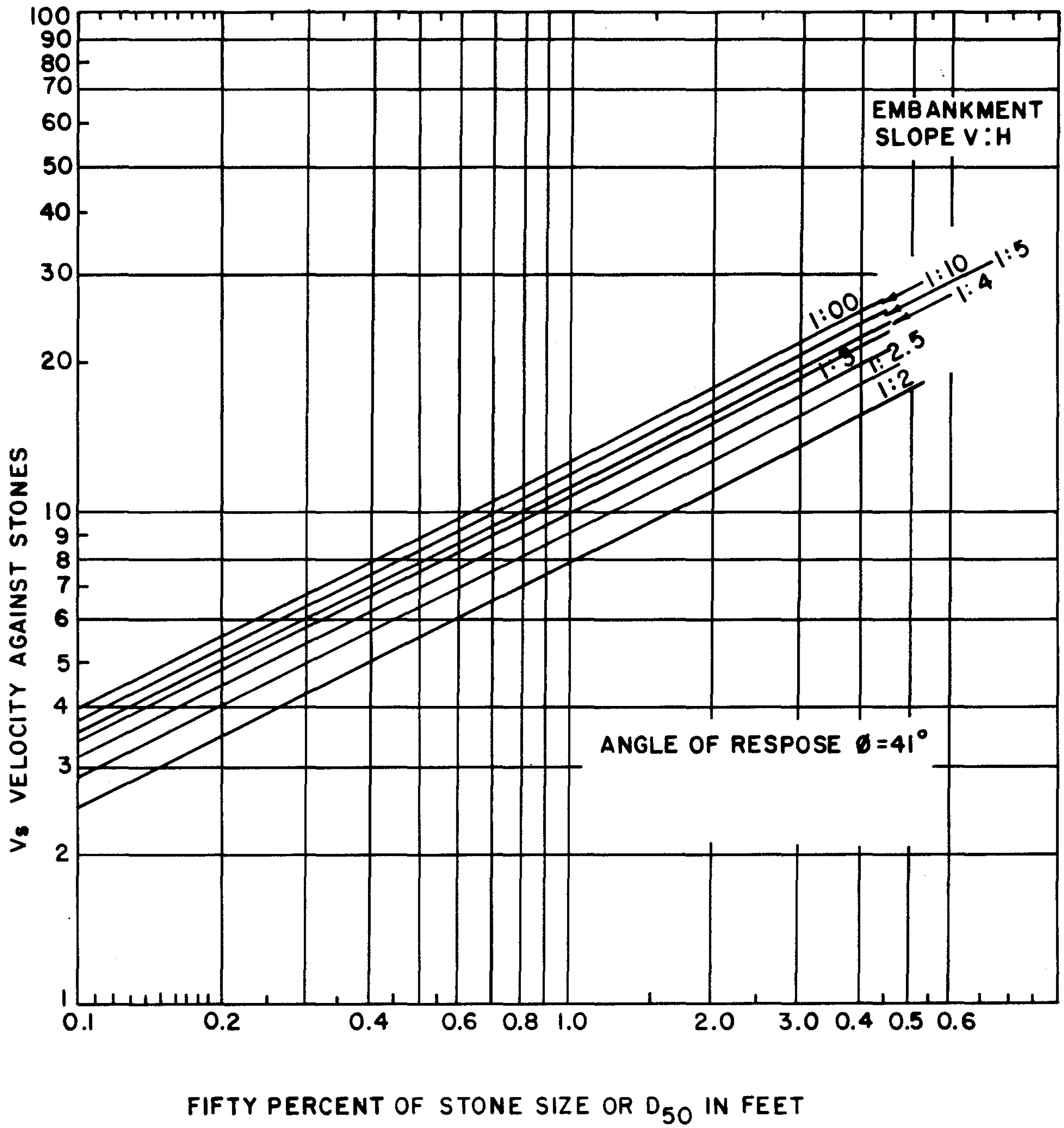


FIG. 15 RIPRAP SELECTION CURVE

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APPENDIX A

ANNOTATED BIBLIOGRAPHY

GENERAL

Coghlan, G., and Davis, Neil, "Low Water Crossings" In Proc., 2nd Intl. Conference on Low Volume Roads, August 20-23, 1978, Ames, Iowa, pp. 98-103.

This paper provides a rationale for planning and constructing low water crossings. The types of low water crossings included for discussions and illustrated by figures are: (1) fords (or dips) formed by lowering the road grade to the stream bed level from bank to bank; (2) vented fords formed by partially lowering the road grades for floods and providing culverts for handling the day-to-day flow; and (3) low water bridges formed by partially lowering the road grade for flooding and providing a bridge type structure to handle the day-to-day flow that cannot be handled by culverts. Examples are given of good and poor designs using different types of materials and consideration of a variety of environmental factors.

Dean, W. E., "Low Cost Bridge Construction," Florida Highways, 17(12): 4, 40-42, Nov. 1949.

Three ways of lowering bridge cost are suggested: (1) consideration of H-10 design loading on Class III roads; (2) reduction of 2-lane roadway width from 24 ft. to 20 ft.; (3) lowering the height of the structure.

Dudley, C., "Low-Cost Bridges for Lightly Travelled Roads" In, Proc., Calif. Inst. on Street and Highway Problems, Univ. Calif., 1953, pp. 51-52.

Provision for low-cost bridges for lightly travelled roads is stressed. Use of war-surplus material and utilization of one-lane bridges could be alternatives for reducing bridge construction cost.

Goverdhan Lal, "The Design of Small Bridges and Culverts," Journal IRC, Paper No. 167, Vol. XVIII-2, December 1953, pp. 123-214.

Procedures for computing flow from small basins are described in this paper. In addition, data requirements and methods for designing small bridges and culverts are discussed.

"The Grading of Causeways," Main Roads, 6(2): 42-43, Feb. 1935.

The grading of approaches to causeways is described. A mathematical method for determining the lengths of vertical curves and their radii for different depths of causeways for speeds 25 to 50 m.p.h. is discussed. The results are shown graphically and can be used for setting out the grades actually in the field.

James, J., "The Grading of Causeways," Main Roads, 2(2): 28, Oct. 1930.

Rough riding over a causeway is generally the result of two causes: (1) the pavement at the invert of the causeway is subjected to scouring by water passing over it, and, if not carefully and frequently maintained, the resultant channels cause shocks in much the same way as do potholes; (2) the causeway may be two short for the depth, i.e. the vertical curve of which it is formed may be of too small a radius. The first cause can be easily taken care of by keeping the pavement reasonably smooth and maintaining in that condition. To remove the second cause it is necessary to provide vertical curves of suitable radii. Therefore, this paper deals with the techniques of determing the proper grades for the approaches by selecting vertical curves of maximum radii along with a level grade at the bottom of the causeway.

Khosla, A. N., Bose, N. K., and Taylor, E. M., Design of Weirs on Permeable Foundations, Central Board of Irrigation Publication No. 12, New Delhi, India, June 1954.

The flow of water through the subsoil resulting in the hydraulic gradient and uplift pressures has been the determining factor in the design of weirs and other hydraulic structures. This document, therefore, concisely presents fundamental theories (such as theory of seepage flow) and procedures which are basically utilized for designing weirs on permeable foundations. The mathematical equations which form the basis for general practices and principles of design are based on experimental verifications of various models and prototypes.

The document is divided into eleven self-contained chapters and incorporates the essence of all the contemporary literature available at that time.

McCullogh, C. B., Economics of Highway Bridge Types, Gillette Pub. Co., 1929.

This document contains fundamentals of economic analysis and procedures for the selection of ordinary highway bridge structures.

Morgali, James, and Oglesby, C. H.,"Procedures for Determining the Most Economical Design for Bridges and Roadways Crossing Flood Plain". In, Highway Research Board Bulletin No. 320, 1962, pp. 40-65.

This paper presents a procedure for determing the most economical configuration of bridges and their components.

Scurr, K. R., "Economics of Small Highway Structures," County and Township Roads, Sept.-Oct., 1953, pp. 24-28.

In this paper, needs for engineering and economic considerations for designing each individual stream crossing are stressed. Several specific and typical examples taken from South Dakota Bridge Department project files are discussed. These examples explain various approaches which should be taken to select the best and most economical type and size of structure for each specific site.

Smith, L. B., "Bridge Problems On County Roads" In, Proc. 4th Annu. Road Builders Clinic, State College of Washington, Pullman, Wash., March 26-27, 1953, pp. 67-68.

Some ways of reducing the cost of constructing bridges over county roads are suggested, including installation of multi-plate arches placed upon a concrete foundation with concrete wing walls, use of timber treated with cresote or commonly used preservative, and use of precast and pre-stressed concrete.

State of California, Manual of Bridge Design Practice" Dept. Public Works, Div. of Highways, Bridge Dept. 3rd ed., 1971.

Includes bridge design criteria.

Tokerud, Roy, "Adequacy of Bridge Design" In, Proc. Northwest Roads and Street Conf. Oregon St. Univ. 1966, pp. 24-36.

The adverse effects of scour and drift forces acting on the bridge structures are described. Development of some specific design criteria by taking into account the additional stresses due to actions of drift, ice, scour and buoyancy is suggested. The criteria for single-shaft piers footings developed by the Bridge Department of the Oregon State Highway Department are illustrated.

Tseng, M. T., Knepp, A. J., and Schmalz, R. A., Evaluation of Flood Risk Factors in the Design of Highway Stream Crossing: Vol. IV, Economic Risk Analysis for Design of Bridge Water Ways. FHWA Final Report No. FHWA RD-75-54, Federal Highway Admin., Of. of Research and Development, Washington, D.C. June 1975.

This document presents an engineering systems approach, based on risk analysis, to determine the most cost-effective design configuration for a highway stream crossing. To evaluate various design alternatives the methodology employed integrates economic factors with the most commonly used hydrologic and hydraulic parameters by linking the initial construction costs and flood related highway losses including property damages due to backwater flooding with the size of the bridge and the design flood frequency.

For the purpose of risk analysis the annual bridge construction cost consists of the sum of the costs of the bridge structure and its components amortized over the life of the bridge at a given interest rate. Such a cost generally would depend upon the type of bridge, its length and width, clearance, span, geology of the underlying river bed, height of the approach embankments, type of highway, and the width of the valley at the crossing site. However, in this model only two most important decision variables, namely, size of the bridge opening and height of the approach embankment have been used. Other decision variables related to scour around piers and abutments and protective measures on embankments are not included in the analysis due to non-availability of adequate information and are, therefore, left for future studies.

The risk defined in this model equals the sum of the products of the weighted probability of flood occurrences and flood-related economic losses, including structure damages, traffic-related losses (such as traffic stoppages and delays caused by an inconvenient detour), and flood losses in terms of accidental deaths, personal injuries and property damages.

To illustrate the application of this risk model an example problem applied to the Tallahalla Creek Bridge site near Waldrup, Mississippi is included.

"Unusual Type Road Closure in Modoc County, California," Highways and Public Works L32 (3o4): 51, 1953.

Maintenance problems encountered due to cold weather and heavy winds resulting in 8 to 10 ft. high snow piles at Cedarville Causeway in California are discussed.

DIP OR FORD

Bingham, M., Design and Construction of Low Water Dips. Texas Highway Department Construction and Maintenance Bulletin, No. 6, May 1951.

For those stream crossings where the amount of traffic justifies provision for a bridge but sufficient funds are not available, construction of a well designed dip is stressed; for a well designed and constructed dip would be far better than an inadequate bridge. Prior to designing a dip the collection of pertinent date is recommended. The data items include: (1) stream section and roadway section; (2) water elevation at normal river stage; (3) stream meander, profile and topography; (4) highway elevation; and (5) drainage area. In addition, provisions for smooth and safe grades are suggested and methods for determining the proper road grades corresponding to the design speed of the road are described. Some examples are also included.

Annon., "Fording in the Modern Manner," Eng. News Record, 126 (1): 17, 1941. p. 17.

To cut down the cost of a stream crossing on a secondary road a ford was built in place of a bridge.

Leydecker, A. D., Use of Gabions for Low Water Crossings on Primitive Secondary Roads U.S. Forest Severice Field Notes, Vol. 5, Nos. 5 and 6, May-June 1977.

This paper stresses utilization of gabions for low water crossings on primitive or secondary forest roads because they are simple in design and less expensive. The construction of a stream crossing using gabions is also described.

Bennett, C. M., et al., "Discussion on Construction of Submersible Bridge Across River Mahanandi near Bhanjnagar in Orissa," Journal IRC, 19(4): 680-688, 1955.

All the hydraulic, structural and economical aspects of submersible bridge design are discussed. For example, resistance to flow can be reduced if the u/s face of bridge slab were made triangular in section and bottom edge rounded. The critical condition in a submersible bridge is when the floodwater is about to overtop the structure, since, under high flood due to submergence of the structure, the velocity through the vents is not higher than in the u/s side. Therefore, scour depth might not be more than under ordinary flood conditions.

Cottman, N. H., "Fivefold Increase Obtained in the Capacity of a Small Bridge by Using a Shaped Minimum Energy Subway," Australian Road Research 6(4): 42-45, 1976.

Design of a 15.2 m long, two-span, U-slab floodway based on specific energy design criteria developed by Prof. G. R. McKay of Queensland is described. The author's contention is that the design of the bridge resulted in as much as fivefold increase in flow capacity of the bridge at a cost of only 40 percent of the cost of extending the bridge to an adequate length in the normal way.

Anon., "Concrete Pipe and Steel Rail Make Effective Flood Bridge," Eng. News Record 142(17): 47-48, 1949.

Construction of a 480 ft span and 12 ft wide submersible bridge over Haina River in the Dominican Republic is described. The bridge has been constructed by using precast concrete pipes tied together and reinforced with a rigid frame with the help of steel rails and reinforcing steel bars. During extreme flood periods, water may rise as much as 4 ft over the whole bridge.

Gupta, B. B., "Some Causeways on Sand Foundations Across Rivers in India," Journal Institute of Civil Engineers (London), Paper No. 5739 33(2): 141-152, 1949.

This paper describes the development of vented causeways constructed on sand foundations across Indian rivers. The design aspects of a causeway are discussed with reference to their stability against scour under foundation, safe bearing-pressure distribution, scour on the downstream side, and horizontal pressure due to moving water. Some empirical formulae for calculating the design stresses and other elements of design are given.

Maurice, Frank J., "Submersible Bridges and Causeways," Highways and Bridges 4(177): 8, 1937.

General concept of application of submersible bridges and causeways is provided and application of corrugated pipes in causeway design is described.

Naik, B.G., "Failure of Old Causeway," Journal IRC, Paper No. 101, September 1955, p. 8.

Three main causes for the failure of a 17 vent-causeway (each vent spanning 10 ft.) are indicated: (1) inadequate foundations, (2) scour cumulation, and (3) incorrect layout.

Raghavachary, K. S., et al., "Discussion of Stable Causeways on Unstable Foundation Soils," Journal IRC 9(4): 37-48, 1945.

Comparison of advantages and disavantages between Surkhi concrete-pipe causeway and R. C. Box type structure causeway on unstable foundation soils is given. The superstructure design is also briefly discussed.

Raju, S. P., "Causeways or Submersible Bridges," Journal IRC 17(3): 354-370, 1952.

Since causeways are designed for the dual functions of passing normal discharges through the vents below the roadway and flood discharges both through the vents and over the roadway itself, these structures present hydraulic problems which are unique to them and are in addition to those encountered by other hydraulic structures, such as dams, spillways, barrages, etc. Field investigations carried on in the past indicate three factors that create problems in a causeway, and include: (1) design of the causeway, (2) configuration of the river including beds and banks, and (3) conditions of flood flow.

In order to know more about these problems and find their solutions, both field and laboratory studies are conducted. The results of these analyses are also described.

Texas Highway Department, Manual Bridge," Bridge Division, 1964.

Contains a section on hydraulic requirements of small structures, multiple box culverts, and selection of culvert sizes. Recommended are flood frequencies for minor and major streams with respect to A.D.T.

W.H.E., "Small Bridge for Agricultural Traffic," The Surveyor 103(2724): 159, 1944.

To serve the most remote farm areas, bridges of comparatively shorter span and width and capable of carrying heavy tractors and similar machinery are required. However, scarcity of labor and materials and subsoil of poor bearing quality pose some problems in constructing such bridges. Use of box culverts type structures without wing walls are recommended.

Wilson, L. D., "Replacing Drainage Dips," Pacific Road Builder and Engineering Review 89(1): 33-34, 1957.

This paper stresses the replacement of drainage dips by pipe culverts as a result of increase in highway tracffic volume. Also pointed out is that installation of pipe is more economical than construction of a bridge.

Benz, E. G., "Constructing a Low Water Bridge in Missouri," Public Works Jan. 1945, p. 24.

Construction details of a low water bridge over Dry Fork Creek in Missouri are described. Dry Fork Creek is crossed by a rural delivery route two miles east of Bern in Gascomade County. There was no bridge originally over the 100 ft. channel and the route was sometimes impassible for days making it hazardous to the mail carrier. Consequently, the county decided to build a low-water crossing 150 ft. long, 18 ft. wide, including two 25-ft. approaches. The bridge deck consists of 10" R. C. slab resting on nine vents and two walls; the finished floor is 36" above low water level, 2" higher on upstream side than on the downstream side.

The entire bridge is supported on white oak piles driven into solid bed rock and spaced at 3 ft. center to center with 6 of them under each vent and each wall.

Brink, S. A., Use of Cattleguard Superstructures for Low Water Crossings on Secondary Forest Roads, Eldorado National Forest, California.

Description of a low water crossing constructed by employing cattleguard superstructures is given. The 180 ft span structure consisted of the approach slabs, ten concrete piers, twenty 8 by 16 ft. cattleguards, and two 8 by 8 ft. cattleguards.

The low water crossing was designed to carry approximately 80 percent of the stream flow under the cattleguards. The concrete pier footings were designed with a soil bearing pressure of 1 ton/sq ft and were placed 5 ft. below streambed to prevent scouring. The concrete approach slabs, placed one on each end of the structure, were designed also with cut off walls 5 ft below the streambed and were heavily rip-rapped, both upstream and downstream, to protect the roadway during high water.

Overall the structure resulted in a pleasant aesthetic appearance, low initial and maintenance costs, and minimal environmental impact.

Chanda, R. P., et al., "Discussions on Paper by E. V. S. Iyer and D. C. Panda--Construction of a Submersible Bridge Across River Mahanadi near Banjanagar (Ganjam) in Arissa," Journal IRC, 19(1): 681-688, 1955.
681-688.

Several observations made by Mr. Chanda and others are discussed. The major ones are:

1. The floor level of a submersible bridge should be kept at some depth below the low bed level to prevent the floor from acting as a weir when retrogression of levels takes place.

2. The length of the downstream apron should be calculated for 1-1/2 D where D is the maximum scour depth under critical conditions, i.e. when the flood water is just about to overtop the structure.

3. Rounding the end faces of the slab and a one-way camber of the roadway to increase the coeficient of discharge of the deck.

Chatterjee, S., "Bitumen Drums used in Vented Causeways," Contractors and Engineers 50(11): 57-58, 1953.

Construction of vented causeways by utilizing empty bitumen drums is described. During the summer months when the river is dry, its bed is excavated to a depth of 4 to 5 ft and to a width of at least 20 ft. A concrete mix in the ratio of 1:4:8 is laid into the excavation to a depth of 4 ft. Over this hard foundation, a layer of rich mortar made of sand and cement is laid. Then the empty bitumen drums, with indentations at both ends removed, are placed in rows along the course of the river. A 6 to 9 in gap is left between each row of drums. These gaps are filled in with 1:2:4 concrete mix. Above the drums, a 1 ft. layer of concrete is spread to load the drums. Surface is finished with 3 in of a 1:1-1/2:3 mix wearing course. Expansion joints are kept at intervals of 20 ft.

Low water crossings of this type up to 250 ft length, 16 ft width by 60 ft long approaches have been constructed.

Dean, A. W. H., "Construction of a Submersible Road Bridge Over the Nerbudda River near Jubbulpore, Central Provinces, India," In Proc. Inst. of Civil Engineers, London, Vol. 239, 1934-35, Paper No. 4915, pp. 178-202.

This paper contains the complete description of the road bridge over Nerbudda River near Jubbulpore, India. The factors which determined the suitability of the site for this bridge were the same commonly used for site selection purposes and included: (1) fairly straight stretch of river; (2) well-defined and fairly high banks; (3) small width; and (4) good foundations.

Additionally, the methodology employed for designing the various elements of the bridge is described and calculations used for sizing the elements and designing the piles for foundations under critical conditions (i.e. (1) when the bridge is about to be overtopped, and (2) when the bridge is under the maximum known flood) are provided.

Anon., "Inspection Tour Notes: Submersible Bridge Across River Narmada at Mortakka," *Journal IRC* 20(4): 509-511, 1957.

This paper describes the design features and construction of a "high type" submersible bridge consisting of 24 R.C.C. arches with clear spans of 88.75 ft each.

Iyer, E. V. S., and Panda, D.C., "Construction of a Submersible Bridge Across River Mahanadi near Bhanjanagar (Ganjam) in Orissa," *Journal IRC*, Paper No. 173 19(1): 91-109, 1955.

This paper deals with the details of design and construction of a submersible bridge over raft foundation. The elements include siting, design of foundations, and design of a superstructure composed of a deck slab, wearing coat, expansion joints and guard stones. Additionally, pertinent information on constructional organization, costs of specific items of work, and calculations for designing a bridge are provided.

Kand, C. V., "Submersible Bridges in Madhya Pradesh," *Journal IRC*, March 1969, pp. 135-173.

Several design and economic aspects of submersible bridges in Madhya Pradesh in India are presented in this paper. The design aspects include moments on substructure under critical conditions, span, afflux, scour depth, and provisions for well foundations, abutments, return walls, approaches, and bearings. The cost comparisons with high level bridges are also provided.

Kelly, John, "Proposed Submersible Bridges," *New Zealand Engineering and Mining Journal* 13(15), 1936.

A brief editorial on bridges and floods is provided.

Kiedaisch, W. C., "Recent Developments in Small Bridge Design." *In Proc. California Street and Highway Conference*, Univ. of Calif. 1960, pp. 85-89.

A description of some of the innovations developed in early 1960's and related to the design of small bridges are described. These changes include elimination of the cap detrusion for slab spans for 26 ft or more, and introduction of prestressed designs using slabs, T-beams, box girders, and I-girders.

Lall, Y. K., "Some Causeways On Sandy Bed," Journal IRC, Paper No. 140 13(3): 347-376, 1949.

This paper briefly narrates the basic principles of causeway design on sandy beds and unfirm soils. Additionally, some of the primary factors which generally could initiate bridge structure failure ultimately resulting in great damage or elimination of the whole causeway are described. Damages to the Subarnarekha Causeway,constructed in 1906 in India are depicted as an example. Several design aspects which could make the new causeways less prone to damages inflicted by flooded river are provided.

Laumann, F. J., "Economical Bridge Replacement for Secondary Roads," Concrete Pipe News 26(6): 116-118, 1974.

In this paper, replacement of old and deteriorated bridges built on low-traffic county roads and secondary roads, which have become less useful due to insufficient road width, by submersible bridges is suggested. A properly balanced hydraulic design of this bridge type is not only economical but can prevent serious damage to the roadway in the overflow portion of the bridge. Design calculations for Stately Township Road crossing of Mound Creek in Brown County, Minnesota, constructed in 1971, are provided as an example. In addition, survey information required for proper analysis and the advantages of reinforced concrete arch type culvert structure versus a bridge are provided.

Anon., "Low Cost Bridges (Texas)," Roads and Streets 93(6): 39-40, 1950.

Several low cost concrete bridges in Burleson County, Texas, are described.

Manual d'Execution de Petits Ouvrages Routiers en Afrique, Republique Francaise, Ministere de la Cooperation, 1975.

This manual (in French) prepared by the cooperation of the Ministry of the Republic of France on the construction of small roads in Africa contains a chapter (Chapter 10) on "Causeways and Submersible Bridges".

In this chapter description of design practices and construction procedures developed for various components of low water crossways are provided. The major elements discussed include pavements--concrete and asphalt with expansion joints, side walls, gabions, and aprons. Several measures to protect causeways against any kind of damage from scouring of bed material are suggested and the methodology to determine the elevation of the pavement is included.

When the stream bed conditions indicate that scouring will be minimum and bed soil can easily support the foundation, the provision of a submersible bridge far outweighs the high level bridge. Calculations show that the overturning moment of a high level bridge from scouring of piers will be almost 800 times more than the overturning movement of a submersible bridge built on firm bed. Use of metal sheets, piles, and hooked plates to control scouring of the bed material is suggested.

Mears, R. P., "Submersible Bridges," Journal of the Institution of Structural Engineers 9(12): 386-396, 1931.

The basic principles regulating the design of a submersible bridge are discussed in this paper. The suitability of such bridges under specific physical and climatic conditions is illustrated with the help of examples of bridges constructed in Central India during the late 1920's to early 1930's. Some of these bridges were even submerged while still under construction, due to floods caused by bursts of heavy rainfall of 12 to 36 hours' duration.

Mirza, M. A., "Stable Causeways on Unstable Foundation Soils," Journal IRC, Paper No. IX-100, 9(2)" 55A-69A, 1944.

This paper presents a theory that as long as the bed of a river does not develop the hypercritical velocity of about 23 ft per second for most of the southern rivers of India, the design of the superstructures of a submersible bridge or causeway would not call for any special treatments other than those observed for an insubmersible bridge or causeway would not call for any special treatments other than those observed for an insubmersible or high-level bridge. And by the same reasoning, the submersible structures founded on unstable soils, such as sand soils, can survive provided adequate protection is furnished to the foundation against disturbing action of flowing water, such as percolation or piping, scour or erosion, and excessive loading, i.e. loading beyond the bearing capacity of the bed soil. Causeways with monolithic bases such as arches built with surkhi concrete or surkhi concrete pipes are suggested for unstable soils.

Mitra, C. D., et al., "Discussion on a Paper by Y. K. Lall--Some Causeways on Sandy Bed," Journal IRC 13(4): 183-192, 1949.

Several observations made by Mr. C. D. Mitra and others are discussed. It was pointed out that more than a dozen causeways constructed on sandy soils in Hyderabad State during the 1930's and 1940's had been functioning quite satisfactorily. Some suggestions for designing such causeways include:

1. Providing ventway for the full width of the stream.
2. Providing a foundation raft and apron thick enough to distribute the load uniformly and withstand impact and scour.
3. Preventing the sand under the raft from being scoured by providing curtain walls upstream and downstream up to scour depth and aprons of sufficient width and weight upstream and downstream with staggered blocks.
4. Wedging the wing and return walls into the natural banks, say from 35 to 50 ft.
5. Providing guide stones 12 in high 4 to 6 ft apart instead of rigid or collapsible railings.

Anon., "Precasting Fosters High Stability in Bridge Built to be Submerged Orange River Bridge," Eng. News Record 150(26): 53-54, 1953. 53-54.

Construction of a 3000-ft. span bridge over the Orange River in South West Africa using precast concrete shells and piles is described. The bridge has been built to serve an isolated mining area to which an interruption of road traffic for a week or two, during high floods would not be serious. To reduce the amount of debris held by the bridge, the structure was made as low as possible and curbs of reinforced concrete 1 ft. high were used in place of bridge rails.

Anon., "Principles of Causeway Design," Main Roads 23(9): 136-142, 1932.

Some basic principles of designing and constructing causeways over different types of streams and their streambeds are described. Safety of a causeway by constructing curtain walls and paving the streambed on upstream and downstream sides with large stones which cannot be carried along in the highest flood is stressed. Different types of causeway construction are depicted by figures.

Rader, E. M., "Rickenbacker Causeway Completed," Florida Highways 15(12): 43, 1947.

A brief general description of Rickenbacker Causeway located in Dade County, Florida, is provided.

Singh, S. P., "Design of Submersible Bridges," Journal IRC 29(3): 489-505, 1965.

This paper presents basic information related to the design of submersible bridges consiting of deck slab and supported by piers and abutments. Four types of bed materials which are normally encountered in designing foundations for piers and abutments are briefly discussed and the most effective type of foundation for each one of these materials is suggested. The natural beds include hard rock, hard soil other than rock, sandy bed, and weak soil such as silt or clay. Some additional design aspects which need to be thoroughly considered designing a submersible bridge are also discussed, including design of bearings and hand rails, impact of floating debris, and the hydrodynamic effect of water acting on the whole span of the bridge under critical condition.

The bridge approaches the critical condition when the flooded river is just about to overtop the bridge. At this stage, the bridge will be under the combined action of forces produced by water currents, buoyancy, and uplift pressure of air entrapped between the girders and deck slab bottom if the proper air escapes such as vents in the sides or in the deck itself, are not provided. The design aspects are discussed individually and a procedure generally used for designing a submersible bridge is presented.

Trice, B. A., "A Low Cost Concrete Bridge," Roads and Streets, October 1948, p. 83.

A basic design and construction procedure for a low cost concrete bridge is described.

Vaswani, H. P., "Submersible Road Bridges," Civil Engineers and Public Works Review 48(570): 1139-1140, 1953.

Brief notes from papers prepared by experienced engineers responsible for the design and construction of submersible bridges are contained. Moleswarth's equation to compute afflux at the u/s end is also provided.

Victor, D. J., "The Investigation, Design and Construction of Submersible Bridges," Journal IRC 24(1): 181-214, 1959.

This paper contains basic information normally needed for designing a submersible bridge. Discussed are various points which need careful consideration prior to designing a bridge at a particular location, including: (1) proper site selection; (2) essential hydraulic design criteria in terms of flood discharge and related hydraulic data calculations using various empirical as well as rational formulae for determining various proposed levels, including level of the deck slab bottom, finished road, level, etc; and (3) various forces acting on a submersible bridge under critical conditions.

In addition, various other design aspects, including most common types of foundations, piers and abutments, deck slab, kerbs and guideposts, and approaches are briefly described; areas which need further research are indentified; and a listing of the basic data to be gathered prior to undertaking the project for planning, designing, and constructing a submersible bridge is included as an appendix.

Walsh, R. E., "Economics In Small Bridge Construction," Surveyor 109(3063): 579-582, 1950.

Economical ways of constructing small bridges are described. Examples for material savings are provided for reinforced concrete box culverts, portal frame complexes and temporary bridges. Several money-saving ways are also depicted.

Walton, G. F., and Gupta, S. B., "New Concrete Submersible Bridge, Bridging the River Nerbudda", Highways and Bridges 1(21): 10, Oct. 30, 1934.

This article discusses in detail a submersible bridge over the River Nerbudda, providing a link between Jubbalpore and Nagpur.

APPENDIX B

DECISION ANALYSIS AND CRITERIA FOR THE SELECTION PROCESSES

LWSCs have been used extensively at locations with low traffic volumes and infrequent floodings. Substantial financial savings can be achieved by building these structures rather than high bridges. The tangible factors (such as capital costs, maintenance costs, flooding damage, protection from erosion, removal of debris and sediment, increase of traffic time, etc.) are estimable. However, the intangible factors (such as inconvenience, danger, the social, political, and legal aspects, etc.) are difficult to assess. Undoubtedly, regular high bridges would be selected under conditions where the total tangible cost of LWSC is approximately equal to the total tangible cost of regular high bridges. The question is how heavily should the potential financial savings of using LWSC be weighed against all the intangible factors. Ultimately, the decision maker must evaluate all tangible and intangible factors in each case to determine which type of structure to use. He must choose the method, conduct his own analysis, and make his decision. He may also be required to defend his decision.

A. General Decision Model

After extensive searching of current literature and interviewing of many state highway department staffs, it became apparent that no design guide is available to estimate intangible costs. An analysis of a public opinion survey conducted by Shen can serve as a rather useful guide. His summaries are based on 60 responses from 40 states and are described as follows:

Highway engineers often have two choices of structures carrying traffic over a stream: 1) regular bridge, or 2) low water crossing structure. This can be illustrated in Fig. 16.

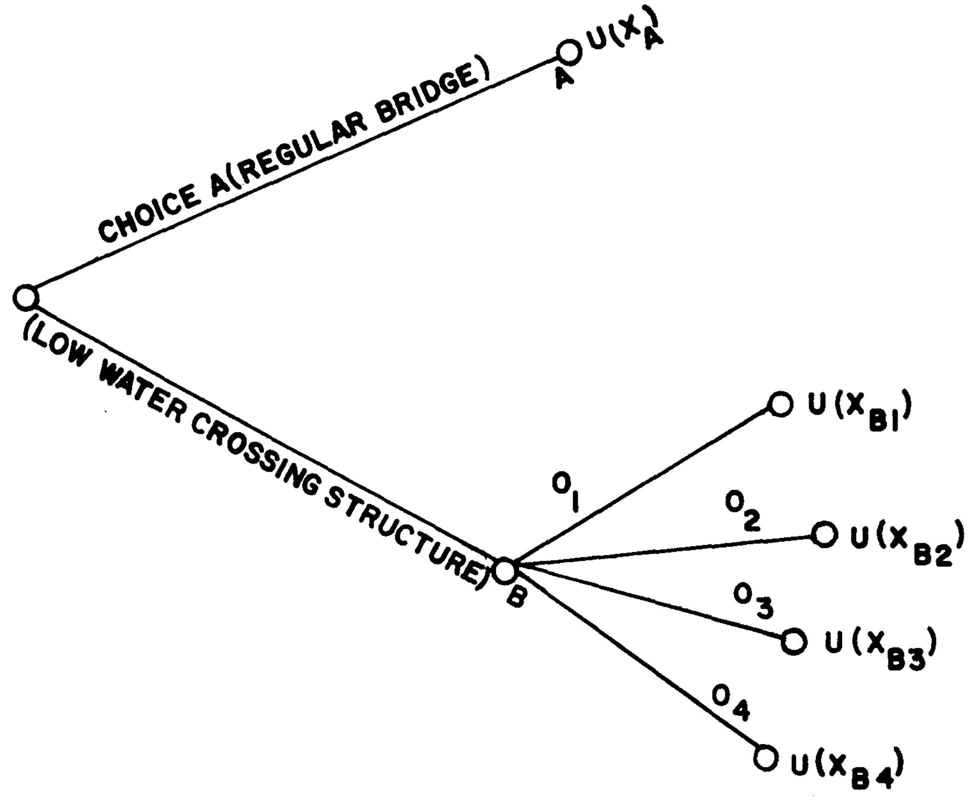


FIG. 16 DECISION MODEL

The engineers may choose A for a regular bridge to reach point A. Let us assume that this regular bridge will not fail, and the total tangible cost (including construction, maintenance, etc.) of the bridge is $U(X_A)$. In this case, there is no uncertainty.

The engineers may also choose B for a low water crossing structure. Let O_i be the probability for occurrence of certain floods with magnitudes falling within a certain range, and let $U(X_{Bi})$ be the total tangible costs (including construction, maintenance, traffic delay, damage to surroundings, repairs etc.) for this particular selected low water crossing structure where floods within a certain range occur (corresponding to the probability of O_i). All of these tangible costs can be calculated as total tangible costs within a selected time period, say 50 years, and amortized to the first year for easy comparison.

Now, the expected total tangible cost for the selection of A (a regular bridge) is $U(X_A)$, and the expected total tangible cost for the selection of B (a low water crossing structure) is:

$$\sum_i O_i \quad U(X_{Bi}) \tag{B-1}$$

Let us define a variable q to be the ratio between the two tangible expected costs as follows:

$$q = \frac{\sum_i O_i \quad U(X_{Bi})}{U(X_A)} \tag{B-2}$$

Technically, if all costs are properly calculated in the function of U, one should choose B--a low water crossing structure--if q is less than 1. There are procedures available to estimate the tangible costs; however, all intangible costs are difficult to determine. Let us assume that the cost function U only includes tangible costs and we shall use an opinion survey from decision makers to deal with the intangible costs.

Example on the Selection of q, the Preference Index:

If decision maker A typically does not like to use LWSC, he will select an LWSC if, and only if, the tangible costs of an LWSC are less than 10 percent of the tangible costs of a regular bridge. In this first case

$$q_1 = \frac{\sum_i O_i \;\; U(X_{Bi})}{U(X_A)} = 0.1 \qquad \text{(B-3)}$$

If decision maker B in general does like to use LWSC more than decision maker A, he will select LWSC if the tangible costs of an LWSC are less than 50 percent of the tangible costs of a regular bridge. In this second case:

$$q_2 = \frac{\sum_i O_i \;\; U(X_{Bi})}{U(X_A)} = 0.5 \qquad \text{(B-4)}$$

From the above two cases, it is clear that the greater the value of q, the higher the preference for LWSC. Furthermore, if a decision maker assigns 0 as the value of q, it would indicate that he would not choose LWSC regardless of amount of cost savings. On the other hand, a q value of 1 would indicate that all the intangible costs of LWSC have been ignored. Thus, the value of q should vary between 0 and 1. The value of q can be treated as a Preference Index.

Shen found from all responses to his public opinion survey that the weighted average (weighted according to their importance) of a Preference Index, q, is 0.72 for the conditions shown in the following table (in other words, for a 28 percent savings of total tangible costs, decision makers should consider LWSC).

TABLE B-1: FAVORABLE CONDITIONS FOR LWSC (q = 0.72)

Average Daily Traffic (ADT)	less than 5
Average annual flooding	less than 2
Average duration of traffic interruption	less than 24 hr.
Extra travel time for alternate route	less than 1 hr.
Possibility of danger to human life	less than 1 billion (with excellent warning systems)

The United States was divided into four zones: Eastern (east of Chicago), Midwestern (east of Colorado), Mountain and Western (California, Oregon and Washington). Responses from the Eastern zone generally assigned a 10 percent higher value of q to nearly all the factors. Responses from the

Midwestern zone generally assigned a slightly lower value (about 5 percent) to the q values of many factors.

Figs. 17 and 18 show variations of the Preference Index, q, with the amount of daily traffic, average frequency of annual flooding, quality of hydrologic analysis, duration of interruptions of traffic, length of alternate route, possible damage to human life, property damage, fundings, warning systems, use as an emergency route, and dimensions of bridges or structures. The slopes of all these curves seem to be rather smooth, indicating no critical limits beyond which the Preference Index, q, would drop sharply.

From these figures, one may define a set of multipliers K to modify the value of q for each variation of conditions different from the favorable conditions given in Table B-1.

Decision makers are usually reluctant to establish a set of absolute constraints below which no LWSC will be considered. From examining the responses, however, the following set of absolute constraints may be formed:

TABLE B-2: ABSOLUTE CONSTRAINTS FOR LWSC

Traffic per day	200
Average annual flooding frequency	10
Average traffic interruption duration	3 da.
Possibility of damage to human life	1 in 10,000
Extra time for alternate route travel	2 hr.
Frequency of use as an emergency route	once per mo.
Property damage	one million dollars
Any absolute constraint due to legal, social and political considerations.	

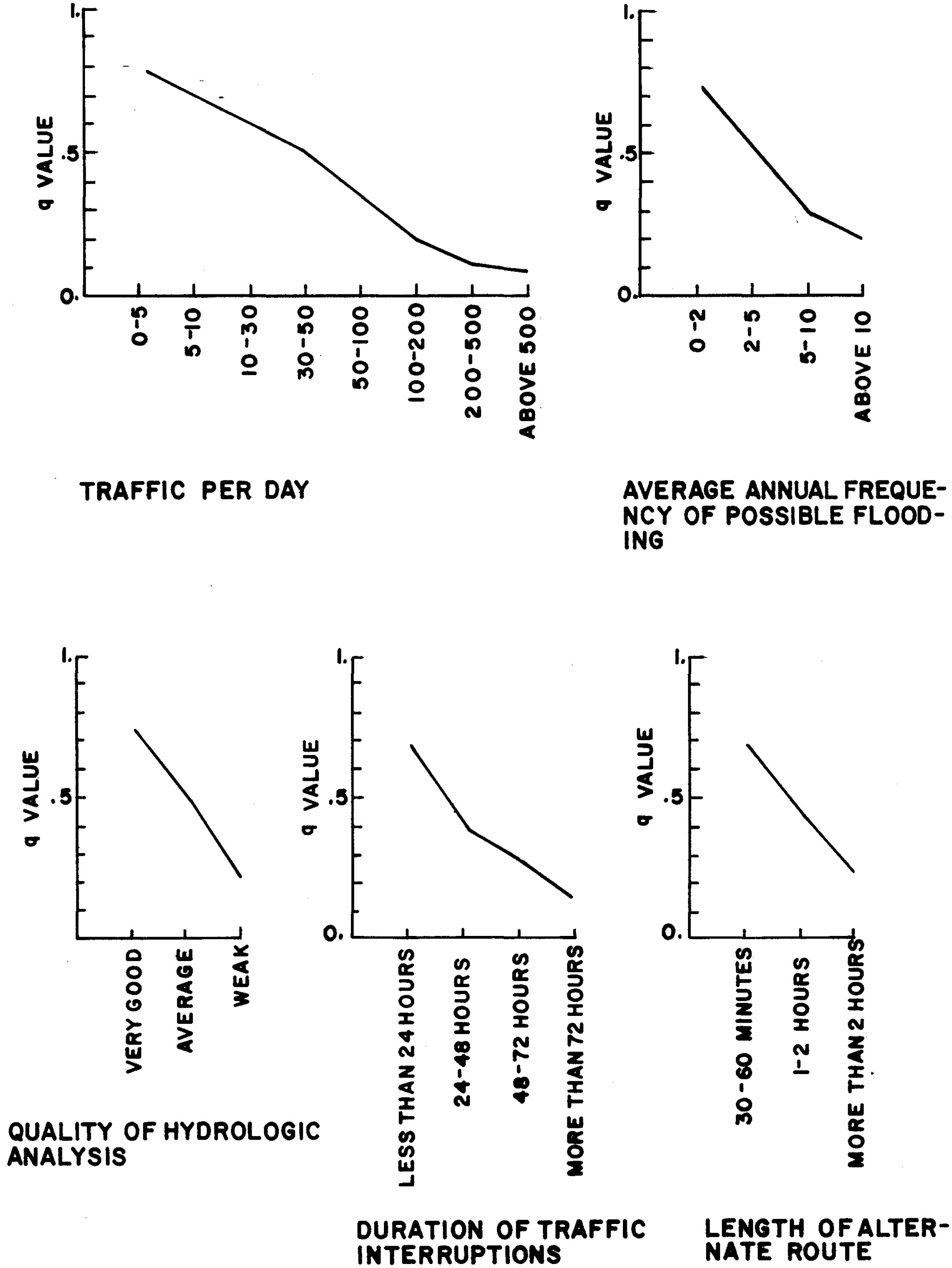


FIG. 17 RELATIONSHIP OF 'q' WITH VARIOUS SELECTION FACTORS

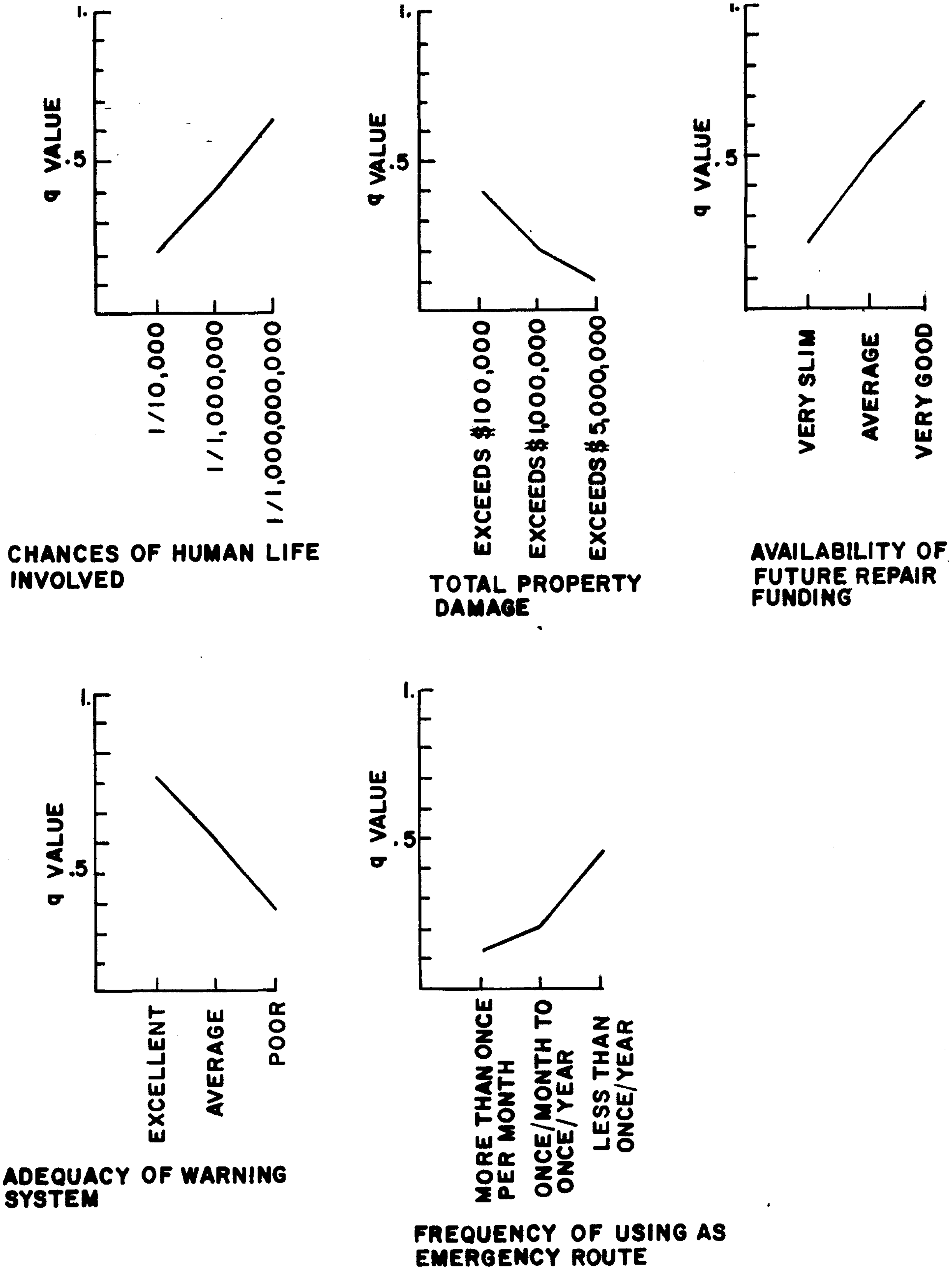


FIG. 18 RELATIONSHIP OF 'q' WITH VARIOUS SELECTION FACTORS

More than 85 percent of the respondents indicated that they would be willing to consider the change of their own constraints for the selection of LWSC if more than 75 percent of others expressed different opinions from their own.

More than 75 percent of the respondents indicated that more LWSC would be used if proper criteria for the selection of LWSC can be defined.

A majority of the respondents indicated that LWSC should not be avoided whenever possible.

B. Evaluation of Tangible Costs

Tseng, Knepp and Schmalz (98), Schneider and Wilson (82), and Corry, Jones and Thompson (33) presented procedures for the estimation of all the tangible costs, including the sum of the expected losses due to different flood frequencies.

The total tangible costs may be divided into 1) total site investigation, design, site preparation, and construction costs and 2) inspection, maintenance, repair, and total expected flood damage costs.

The site investigation costs include site visits; data collection; data analysis for geological, soil, hydrologic and hydraulic investigations. The design costs include costs of design by engineers, geologists and soil scientists. Construction costs consist of the cost of superstructure and substructure of LWSC or the regular bridge; site preparation and excavation of the approach and roadway; and protective measures such as ripraps, dikes, etc. to reduce scour. This group of costs may be classified as initial capital costs.

The second group of costs consists of field inspections, regular maintenance (including possible removal of floating debris and sediment deposition), regular repairs for normal flows, and expected flood damage costs.

The regular repairs for normal flows and expected flood damages for high flows may include structural damages (both superstructure and substructure); embankment erosion (upstream, flow overtopping, and downstream); pier scour for bridges or culvert downstream scour for vented fords; traffic-related losses (time loss due to extra travel time for alternate routes, accident costs, etc.); and flood damages (flooding of houses, farms, and other properties).

It is difficult to define cost functions for all the above factors in different situations. Nevertheless, detailed studies should be made to define these costs better for highway engineers than as provided by current knowledge. For instance, damages due to flooding of areas of different types of land use and erosion due to overtopping are two factors which require particular immediate attention.

C. Procedure for the Selection of Either LWSC or Regular High Bridges

Step 1: Data collection

(a) Location and topographic maps
(b) Hydrological and climatological data (rainfall, runoff, snow, ice, etc.)
(c) Geological data (foundation, seepage, etc.)
(d) Soil data (bank and bed material characteristics)
(e) Traffic data (average daily traffic, speed, accidents, emergency route, etc.)
(f) Economic data (material and labor costs)
(g) Detail topographical data by surveying
(h) List factors that may affect the total tangible and

intangible costs (legal, social, political, sediment movement, floating debris, land use, etc.)

Step 2: Field inspection (for both tangible and intangible factors)

Step 3: Hydrologic, hydraulic, soil and geological analysis

Step 4: Calculate total tangible costs for Regular high bridges and a type of LWSC.

This would include all tangible costs applicable to a particular situation. As stated in the previous section, these tangible costs include: 1) total site investigation, design, site preparation, and construction costs, and 2) inspection, maintenance, repair and total expected flood damage costs. Some of these costs are described by Tseng, Knepp, and Schmalz (99).

Step 5: Calculate the value of q* which is:

$$q^* = \frac{\text{total tangible costs of LWSC}}{\text{total tangible costs of regular bridge}}$$

Step 6: Determine average traffic per day, average annual flooding frequency, average traffic interruption duration, possibility of human life involvement, extra travel time for alternate route, frequency of use as an emergency route, and the total property damage due to LWSC.

Step 7: Compare data on intangible factors for a particular site as defined in Step 6, with absolute constraints for LWCS as given in Table 8. Determine if any absolute constraints are violated. The decision maker must now make his own decision whether or not to eliminate the use of LWSC as a valid possibility.

Step 8: Obtain the various K factors to modify the preferance index q based on data collected for the intangible factors, as described in Step 6 from Tables 2, 3, 4, 5, 6, and 7.

Step 9: Calculate

$$q = \text{product } (K_1 \cdot K_2 \cdot \cdot \cdot) \cdot 0.72 \qquad \text{(B-5)}$$

based on the K_i values given in Step 8, and with the value of q* as calculated in Step 5.

Step 10: Based on the values of q* and q, the decision maker must use his own judgement on the selection of LWSC. If he should agree with all other respondents to the questionnaire, he may choose the LWSC if q = q*. However, as stated previously, the decision maker must evaluate all tangible factors in a given case to decide which type of structure to use. He must choose the method, conduct his own analysis and make his own decision. He may also be required to defend his decision.

EXAMPLE PROBLEM:

Following the procedure as given by Tseng, Knepp, and Schmalz (98), Schneider and Wilson (80), and Corry, Jones, and Thompson (33), it is determined that the total tangible costs of a regular high bridge are:

i)	Annual capital costs	\$ 50,000
ii)	Annual maintenance costs	\$ 2,500
iii)	Annual risk costs	\$ 1,000
	Total	\$ 53,500

The total annual tangible costs of a LWSC are:

i)	Annual capital costs	\$ 10,000
ii)	Annual maintenance, monitoring repair costs (including risks)	\$ 5,000
	Total	\$ 15,000

Then, a survey is made, and the following information obtained for local conditions through investigation:

i) ADT = 100
ii) Average annual flooding = 4
iii) Quality of hydrologic analysis is average.
iv) Average traffic interruption is about 48 hr.
v) Extra time for alternate route is 1 hr.
vi) Chances of possible damage to human life are 1 in 1 million.
vii) No absolute constraints listed in following Table 8 are violated.

Next, is the calculation of $q^* = 0.72k = K_1 . K_2 . K_3 . K_4 . K_5 . K_6 . 0.72$ from Tables 2, 3, 4, 5, 6, and 7:

It was found that

$K_1 = 0.79$, $K_2 = 0.90$, $K_3 = 0.80$, $K_4 = 0.80$, $K_5 = 1.0$, $K_6 = 0.60$
and
$q^* = 0.79 \,.\, 0.90 \,.\, 0.80 \,.\, 0.80 \,.\, 1.0 \,.\, 0.60 \,.\, 0.72 = 0.20$

The value of 0.72 is obtained from Table 1 for the most favorable conditions.

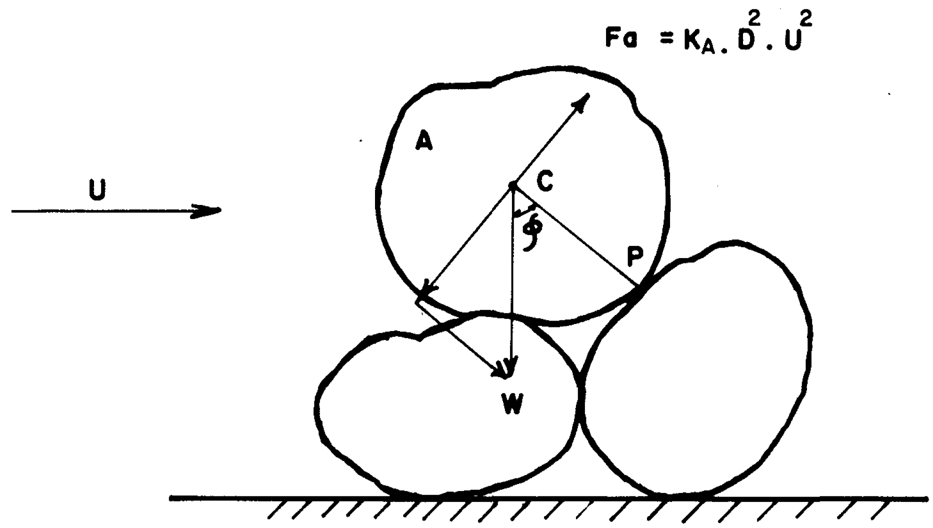
The next step is as follows:

$$\frac{\text{Total annual tangible costs of LWSC}}{\text{total annual tangible costs of a regular high bridge}} =$$

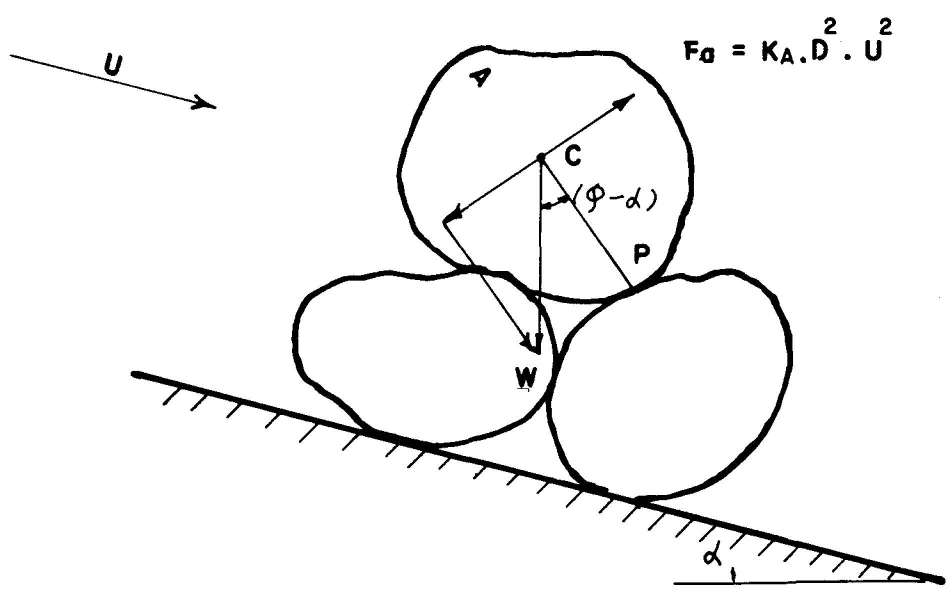
$$\frac{15{,}000}{53{,}500} = 0.28$$

The decision maker must now make his own decision based on the average of all opinions, as outlined by Shen (85). Thus, following the average of all opinions, one would use the LWSC if the ratio between total annual tangible costs of LWSC and total annual tangible costs of regular high bridges is less than q^* (0.20). In this particular case the ratio is $q = 0.28$.

The decision maker is advised at this point to study Shen's report and compare his case with other cases. If he agrees with the average of all other opinions collected in the survey, he would use a regular bridge in this instance, because $0.28 > 0.20$ [$q^*(0.20) < q(0.28)$].



(b) ON HORIZONTAL SURFACE



(b) ON SLOPING SURFACE

FIG. 19 RIPRAP STONE ON EMBANKMENTS

APPENDIX C

ANALYSIS OF FORCES ON A STONE

Stone A in flowing water is subjected to lift and drag forces (Fig. 21). The components of these forces perpendicular to the line CP are the acting forces to dislodge the stone. The resultant of these components can be expressed as:

$$F_a = K_a\ d^2 U_s{}^2 \tag{C-1}$$

where:

f_a = resultant of the acting forces

d = diameter of stone

V_s = incipient flow velocity

The resisting force is the component of submerged weight of the stone in the direction perpendicular to CP, which may be expressed by:

$$F_v = K_v d^3 \mathrm{Sin}\phi \tag{C-2}$$

where:

F_v = resisting force

ϕ = angle of repose

Referring to Fig. 21, for riprap laid on a flat surface, an equilibrium condition exists when the resisting force equals the acting force, or:

$$K_r d_o{}^3\ \mathrm{Sin}\ \phi\ = K_a d_o{}^2 U_s{}^2 \tag{C-3}$$

or

$$d_o = \frac{K_a}{K_r}\ \frac{U_s{}^2}{\mathrm{Sin}\phi} \tag{C-4}$$

For riprap on a slope of α (Figure C-2), the resisting force becomes:

$$K_r = K_r d^3 \operatorname{Sin}(\phi - \alpha) \tag{C-5}$$

and the equilibrium equation results in:

$$K_r d^3 \operatorname{Sin}(\phi - \alpha) = K_a d^2 V_s^2 \tag{C-6}$$

or $$d = \frac{K_a}{K_r} \frac{U_s^2}{\operatorname{Sin}(\phi - \alpha)} \tag{C-7}$$

Combining equations (C-4) and (C-7), the relationship between the diameters of stone for flat bed and sloping embankment can be developed as:

$$\frac{d}{d_o} = \frac{\operatorname{Sin}\phi}{\operatorname{Sin}(\phi - \alpha)} \tag{C-8}$$

The angle of repose of ordinary stone slightly increases with its size. It is about 41° for 1 ft diameter, increases to about 42° for 2 ft diameter, and approaches a constant value of about 43° for a further increase in diameter. The correction factors for 41° and 43° vary slightly: the difference is about 1% for an embankment slope of 1:10 to 8% for a slope of 1:2 V:H. The correction factor reduces with the angle of repose. Fig. 19 shows the relationship between incipient flow velocity and stone size for the angle of repose of 41°. For larger stone, the size selected may be reduced slightly.