# DESIGN GUIDE: LOW WATER STREAM CROSSING

Research, Development, and Technology

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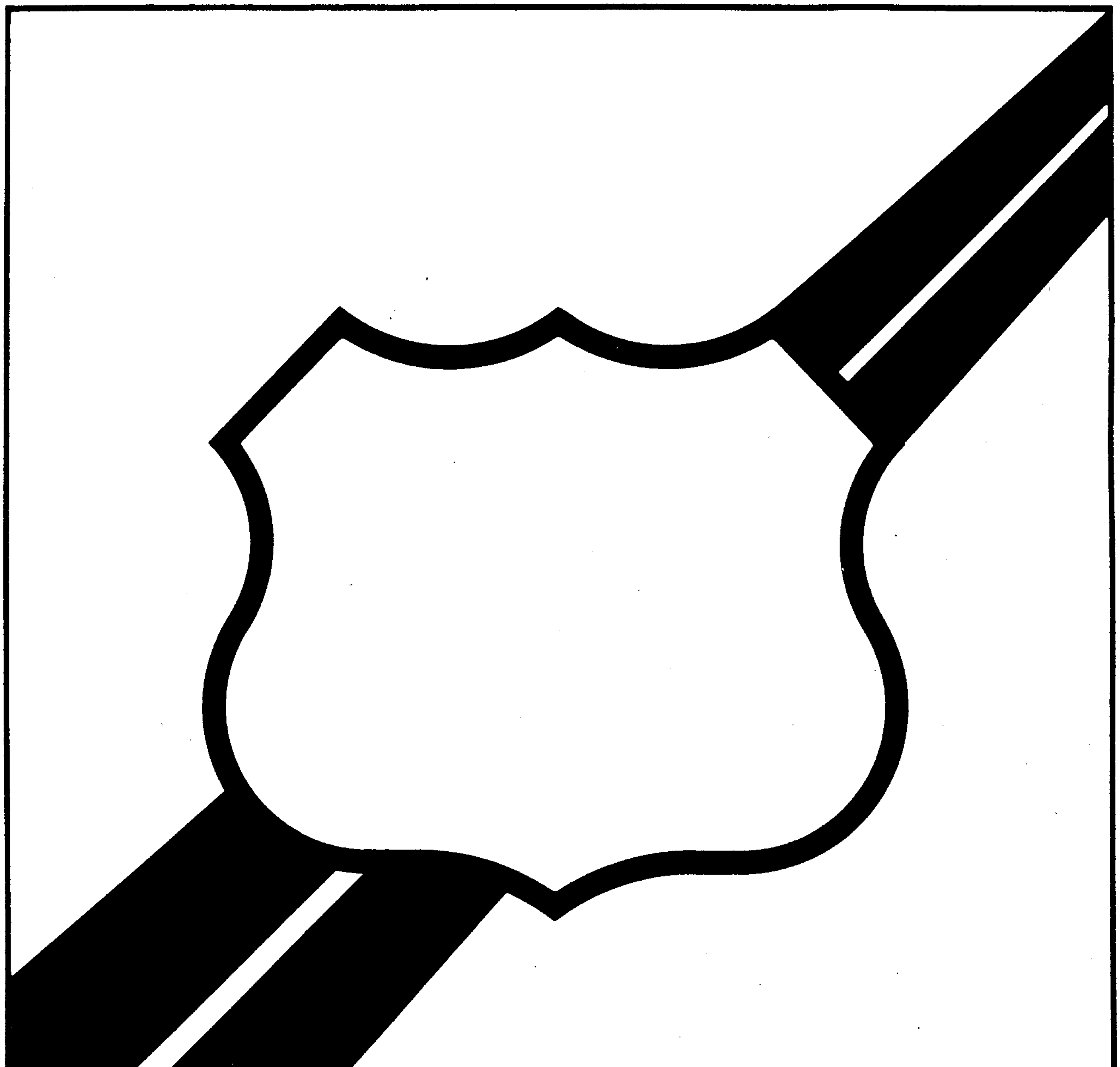

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16. Abstract


Commonly used Low Water Stream Crossings (LWSC) of different types are described and the characteristics of their efficient design are discussed. Selection factors for LWSCs are listed, along with a set of general and specific design considerations. A method, based on economic risk analysis, is recommended for the final selection and design of fords, vented fords and low water bridges. Design examples are provided for each of the three structures.

This Design Guide emphasizes the application aspects and complements the volume: "Design and Construction of Low Water Stream Crossings", which stresses theoretical concepts and views.


Other reports in this series include:

FHWA/RD-83/015 - Design and Construction of Low Water Stream Crossing. Executive Summary.
FHWA/RD-82/163 - do Final Report (Unpublished - Avail. NTIS)

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# FOREWORD

During 1979 to 1982, a study was conducted on the design and construction of Low Water Stream Crossings (LWSC) for the Federal Highway Administration (FHWA). Two reports were produced from this study: (1) a Final Report containing five case studies, annotated bibliography and summary of present knowledge regarding LWSC, and (2) a Guide for design and construction of LWSC. The Guide is developed based on the Final Report and written in a way that the engineers engaged in actual design may use it without having to go through detailed analysis or study. Because of this briefness, the Guide may lack explanation of concepts and theory on which the equations and discussions are based. The Final Report should, therefore, be referred to when additional information is needed.

In addition to the Final Report, the following Hydraulic Engineering Circulars prepared by the Federal Highway Administration are recommended as supplemental materials for designing an LWSC:

| | |
|---|---|
| HEC No. 5 | Hydraulic Charts for the Selection of Highway Culverts |
| HEC No. 10 | Capacity Charts for the Hydraulic Design of Highway Culverts |
| HEC No. 17 | The Design of Encroachments on Flood Plains Using Risk Analysis |

These documents may be obtained from the U.S. Department of Transportation, FHWA, Bridge Division, Washington, D.C. 20590 or from the National Technical Information Service, Springfield, Virginia 22161.

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# LIST OF SYMBOLS

| | |
|---|---|
| $A$ | CROSS SECTIONAL AREA |
| $A_p$ | TOTAL AREA OF PAVEMENT SUBJECT TO OVERFLOW |
| $C_a$ | ADJUSTMENT FACTOR FOR QUICK REPAIR |
| $C_e$ | COST OF EMBANKMENT |
| $C_f$ | COEFFICIENT OF DISCHARGE FOR FREE FLOW |
| $C_p$ | COST OF PAVEMENT |
| $C_r$ | TOTAL COST OF STRUCTURE |
| $C_s$ | COEFFICIENT OF DISCHARGE FOR SUBMERGED FLOW |
| $D$ | DIAMETER OF PIPE |
| DD | INCREASED DISTANCE FOR DETOUR |
| DT | INCREASED DRIVING TIME FOR DETOUR |
| $D_{50}$ | MEDIAN DIAMETER OF STONE |
| $d$ | WATER DEPTH ABOVE THE FIRST FLOOR |
| $H$ | TOTAL HEAD UPSTREAM MEASURED ABOVE THE CROWN OF ROADWAY |
| $h^*$ | BACKWATER |
| $I$ | IMPACT FRACTION |
| $I_f$ | IMPACT FACTOR |
| $i$ | INTEREST RATE |
| $L$ | LENGTH OF OVERFLOW |
| $L$ | LENGTH OF PORTION OF SPAN WHICH IS LOADED TO PRODUCE THE MAXIMUM STRESS |
| $L_f$ | LOAD FACTOR |
| $L_{ep}$ | LOSSES TO EMBANKMENT AND PAVEMENT |
| $L_{rc}$ | INCREASE RUNNING COST |
| $L_{tc}$ | COST FOR LOST TIME OF VEHICLE OCCUPANTS |
| $L_{tr}$ | COST FOR INTERRUPTION OF NORMAL TRAFFIC |
| $L(Q_i)$ | LOSS CAUSED BY FLOOD $Q_i$ |
| $N$ | SERVICE LIFE OF STRUCTURE |
| $N_c$ | AVERAGE NUMBER OF OCCUPANTS PER VEHICLE |

## LIST OF SYMBOLS
(Cont'd)

| | |
|---|---|
| $n$ | MANNING'S ROUGHNESS COEFFICIENT |
| $P$ | WETTED PERIMETER |
| $P$ | PERCENT DAMAGE TO BUILDING |
| $P_e$ | PERCENT OF EMBANKMENT LOSS |
| $P_i$ | EXCEEDENCE PROBABILITY OF FLOOD $Q_i$ |
| $P_p$ | PERCENT OF PAVEMENT LOSS |
| $P_r$ | PERCENT OF STRUCTURE LOSS |
| $Q$ | DISCHARGE |
| $Q_i$ | DISCHARGE OF i-YEAR FLOOD |
| $R$ | HYDRAULIC RADIUS |
| $S$ | SLOPE |
| $T_r$ | RESTORATION TIME |
| $t_b$ | BASE TIME FOR HYDROGRAPH |
| $t_{ot}$ | OVERTOPPING TIME |
| $V$ | FLOW VELOCITY |
| $V_e$ | TOTAL VOLUME OF EMBANKMENT SUBJECT TO OVERFLOW |
| $W_D$ | DEAD LOAD |
| $W_L$ | LIVE LOAD |
| $Y_2$ | BRINK FLOW DEPTH |

# 1.0 INTRODUCTION

## 1.1 BACKGROUND

Low Water Stream Crossings (LWSCs) are defined as stream crossing structures deliberately designed and constructed to be frequently overtopped by high floods. Such structures include fords (or dips), vented fords and simple submersible bridges.

For low volume roads, the use of LWSCs are often ideal, particularly when funds are limited. Unfortunately, systematic scientific study on LWSCs on which selection criteria, hydraulic, and structural design could be based, is lacking at this time. It is hoped that this Design Guide will fill this void and provide guidance to engineers in designing LWSCs.

## 1.2 SCOPE

Information has been collected from existing literature and field observations and analyzed for material useful in LWSC design and construction. The manual contains: (1) decision-making factors for the selection of LWSC, (2) methodology for selecting the optimum design, (3) hydraulics of LWSC and erosion protection measures, (4) methods and criteria for design and construction, and (5) design examples.

## 1.3 LIMITATIONS

This Guide primarily covers small structures not requiring extensive structural analysis, or detailed analysis of hydraulics and erosion control measures. The Guide deals generally with common cases and conditions. For unusual cases, such as very unstable stream, it should be used with caution.

The design criteria are based on a body of experience which should be used selectively to suit local conditions.

# 2.0 SELECTION FACTORS AND TYPES OF LOW WATER STREAM CROSSINGS

For economical reasons, LWSCs are generally found suitable for rural areas and mountainous terrains where traffic is infrequent and frequency of flooding is low. The economic gain in lowering and simplifying the structure is, however, coupled with an increase in risk of structural damage and other losses due to overtopping floods. Traffic is interrupted and hazards increase during and after a flood. Therefore, in selecting a LWSC, economic factors as well as factors related to traffic must be considered. Important factors in selecting LWSC in preference to a regular high level crossing are detailed in this section.

## 2.1 SELECTION FACTORS

An opinion survey (8) of experienced engineers was conducted by Shen at the Colorado State University on the selection factors and criteria for LWSC. Based on 60 responses from 40 states, Shen summarized a number of criteria as listed in Table 1 as a guide for selecting LWSC. Column 2 of this table shows the conditions under which an LWSC would be most desirable and Column 3 shows the conditions when any consideration of LWSC would be excluded.

For cases fitting the most favorable conditions, therefore, a decision to select an LWSC can be readily made after due consideration is given to other social and environmental factors discussed later, without having to go through detailed economic or other analyses. For cases with conditions falling between the most and the least favorable conditions, however, further analyses, mainly an economic analysis, may be necessary to evaluate the savings in total cost for an LWSC, which will determine its suitability.

TABLE 1

SELECTION FACTORS

| Criteria | Most Favorable for LWSC | Least Favorable for LWSC |
|---|---|---|
| Average Daily Traffic (ADT) | less than 5 vehicles | 200 vehicles |
| Average annual flooding | less than 2 times | 10 times |
| Average duration of traffic interruption per occurrence | less than 24 hours | 3 days |
| Extra travel time for alternate route | less than 1 hour | 2 hours |
| Possibility of danger to human life | less than 1 in 1 billion (with excellent warning systems) | 1 in 100,000 |
| Property damage | none | One million dollars |
| Frequency of using it as an emergency route | none | Once/month |

Besides the above mentioned factors, there are other unquantifiable factors to be considered for decision making. Many states restrict the construction of low crossings along school bus routes or defense roads. For remote areas where highway patrols cannot be reached easily in an emergency, a good warning system for during the period of overflow should be considered. Ice cover on the roadway at a sag point of a low crossing is hazardous even to slow moving traffic. Ice problems should be reviewed carefully for a low-water crossing in cold regions.

Usually of short length, an LWSC is vulnerable to stream migration. Aggradation of streambed may plug the openings and reduce the flow capacity. Degradation of streambed downstream, on the other hand, may induce serious erosion of foundation soil and ultimately cause structural failure. An LWSC, therefore, should be located only in a reach where the stream is stable.

Because of its low clearance, an LWSC is also vulnerable to debris and ice. In numerous cases, floating debris has plugged openings of low bridges and culverts. A higher-level regular bridge is recommended where debris and ice pose serious problems.

An LWSC should not be encouraged as a permanent structure along roads where large increase in traffic is expected caused by economical, social and other changes. Environmental impact on fish and wildlife must also be considered and appropriate agencies must be contacted before a decision is made.

## 2.2 TYPES OF LOW-WATER STREAM CROSSINGS

There are three common types of low-water stream crossings: ford (dip), vented ford and low bridge.

### 2.2.1. Ford

Fords are commonly constructed across drainages which are dry during the most part of the year or where day-to-day stream flow is only a few inches in depth. Fords are founded on river and stream beds, and are generally formed by lowering the grades of approaching roads to streambed level from bank to bank. A typical concrete-slab ford is shown in Fig. 1.

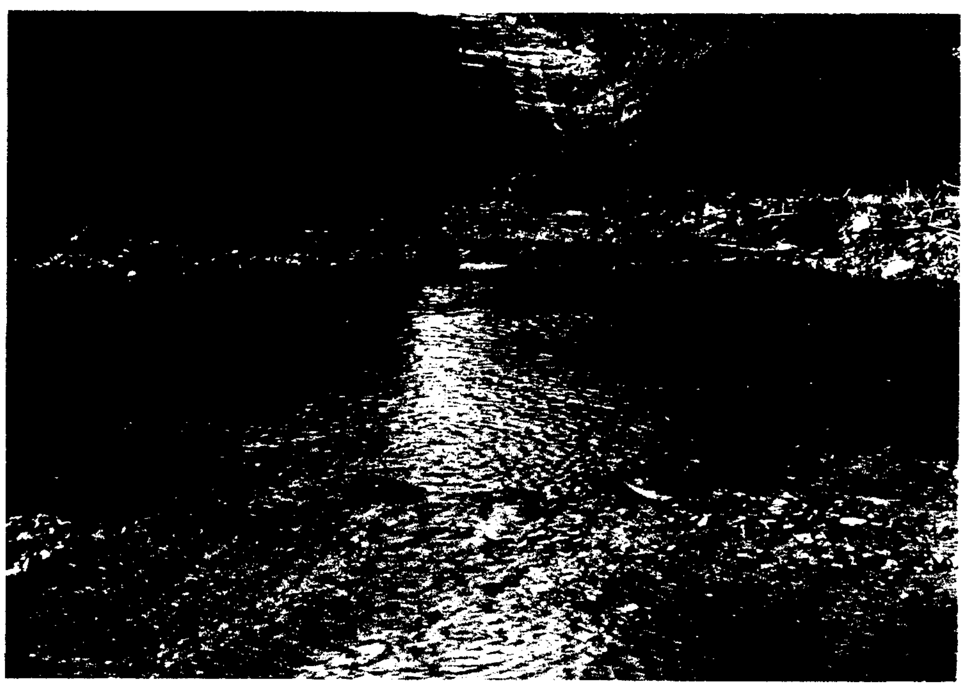


FIG. 1 UNIMPROVED FORD OVER NATURAL STREAM BED

### 2.2.2 Vented Ford

Vented fords are simply dips with vents or openings below the road surface to provide passage of day-to-day flow. Their use over simple fords is preferred where day-to-day flow exceeds normal fordable depth of 4 to 6 in. (10 to 15 cm). They are formed by partially lowering the grades of approaching roads and providing drain pipes underneath. The pipes may be embedded with soil (flexible type) or encased with concrete (rigid type). The typical vented fords are shown in Figs. 2 and 3.

FIG. 2 VENTED FORD - FLEXIBLE TYPE

FIG. 3 VENTED FORD - RIGID TYPE

### 2.2.3. Low Bridge

Low-water bridges, like vented fords, are formed by partially lowing the grades of approaching roads and providing adequate passage of day-to-day flow through the openings. However, instead of vents or culverts, a bridge deck is provided to pass a larger flow. The choice for this type over a vented ford is made when the daily flow cannot be handled by the latter efficiently and economically because of hydrologic or terrain con-ditons. Particularly in a stream with potential debris problems, low bridges are preferred over vented fords. A deep and narrow stream channel with steep overbanks is also suitable for a low water bridge. A simple low bridge is shown in Fig. 4.

FIG. 4 SIMPLE LOW WATER BRIDGE

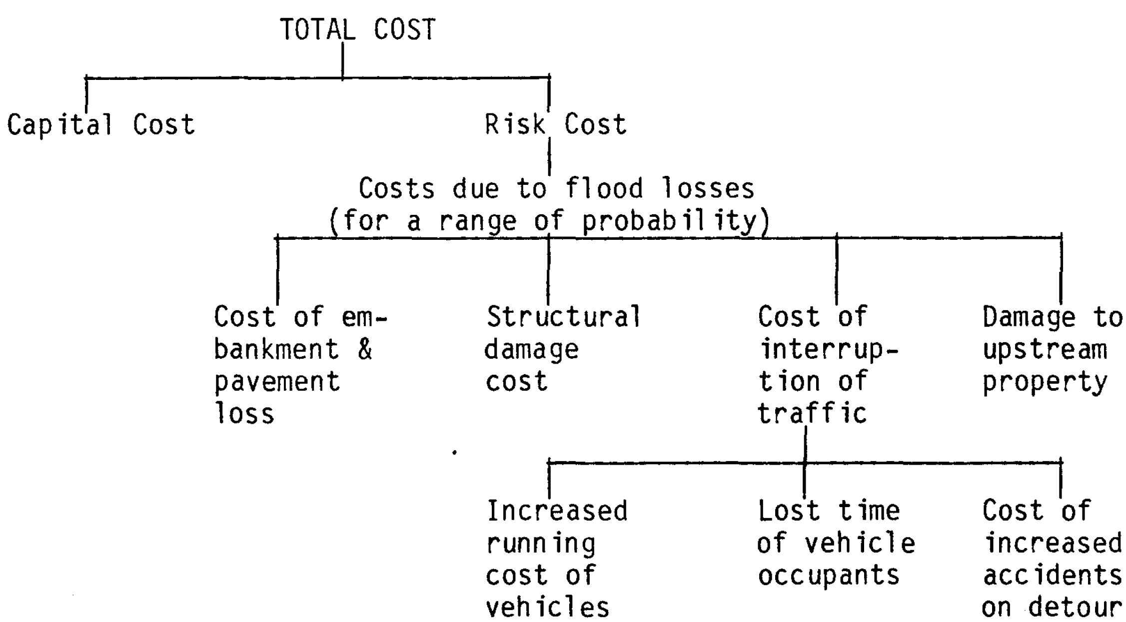
## 2.3 FINAL SELECTION USING RISK ANALYSIS

The final selection of the type and the design of an LWSC should be based on an economic analysis. A number of alternative designs, including conventional high crossings and LWSCs are considered first. For each design, the cost of flood damage is estimated by risk analysis. The costs of damage to structures, embankments, surrounding properties, traffic-related losses, and scour, etc., are included in this risk cost. The sum of the annual risk cost and the annual capital cost results in the total expected cost (TEC). Comparison of the TEC's for all design alternatives allows the designer to select the final design that is most economical. The details of risk analysis and example problems are presented in Chapters 3 and 5 respectively.

For a small structure with the total estimated cost of less than about $20,000, a detailed risk analysis may not be practicable in view of the rigorous efforts required to pursue a meaningful analysis. Instead, good engineering judgment based upon experience and intuition may serve the same purpose.

# 3.0 RISK ANALYSIS

Balanced design, including risk cost as a part of the total cost, has gained popularity among engineers in various fields. Traditionally, for a bridge design, the design flood is first selected 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 design alternative, the annual capital cost and risk cost are computed. All alternatives are then compared for total cost in selecting the most economical design, not counting the routine maintenance costs which is required for all structures. Total annual cost of any of the alternatives comprises of the annual capital cost and the annual risk cost. Annual risk cost is essentially the cost due to losses by floods of various frequencies to which the structure is exposed. Elements of the total cost of an LWSC are as shown in the following diagram:



Descriptions of these cost elements and how they can be determined are included in this section. Materials presented herein are primarily abstracted or developed from the Hydraulic Engineering Circular No. 17, "The Design of Encroachments on Floodplains Using Risk Analysis" (4).

## 3.1 ANNUAL CAPITAL COST

Capital costs mainly include structural, pavement, and embankment costs. The cost of embankment protection, such as riprap, and special warning signs required for low-water stream crossings, should also be included. To determine annual capital cost, the capital cost is multiplied by an appropriate capital recovery factor. The capital recovery factor (CRF) is computed by the equation:

$$CRF = i/[1-(1+i)^{-N}] \tag{1}$$

where $i$ = Interest rate

$N$ = Service life of the structure (year)

$$\text{And, annual capital cost} = (CRF) \times (\text{Capital Costs}) \tag{2}$$

## 3.2 ANNUAL RISK COST

To obtain annual risk costs, the following approximate equation may be used:

$$\text{Risk costs} = \sum_{i=1}^{n} (P_i - P_{i+1})\left[\frac{L(Q_i) + L(Q_{i+1})}{2}\right] \tag{3}$$

where $P_i$ = Exceedance probability of flood $Q_i$

$L(Q_i)$ = Loss caused by flood $Q_i$ (dollars)

$Q_i$ = Discharge of 1-year flood (cfs)

The first $Q_i$ is the discharge of 1-year flood and the last $Q_{n+1}$ is the discharge having a recurrence interval of 200 years or more. The floods of recurrence intervals of 1-, 2-, 6-, 10-, 50- and 200-years may be used to estimate losses and then to calculate the risk costs by using the above equation.

### 3.2.1 Losses Caused by Flood

Potential flood losses for an LWSC include:

Loss of embankment and pavement
Structural damage
Interruption of normal traffic flow
Damage to upstream property due to backwater

#### 3.2.1.1 Loss of Embankment and Pavement

Floods overtopping roadway for sustained periods of time cause damage to the pavement and embankment. The extent of damage can generally be expressed in terms of flow depth and sustained time. Such data should be available before pursuing the analysis. The loss, $L_{ep}$, to the embankment and pavement may be estimated from the equation:

$$L_{ep} = (P_e \cdot C_e \cdot V_e \cdot + P_p \cdot C_p \cdot A_p) \ \frac{C_a}{100} \tag{4}$$

where $P_e$ = Percent of embankment loss (%)
$C_e$ = Cost of embankment (\$/CY)
$V_e$ = Total volume of embankment subject to overflow (CY)
$P_p$ = Percent of pavement loss (%)
$C_p$ = Cost of pavement (\$/SY)
$A_p$ = Total area of pavement subject to overflow (SY)
$C_a$ = Adjustment factor for quick repair

3.2.1.2 Structural Damage:

The cost of structural damage may be computed by

Structural damage cost = $C_r P_r$ (5)

where $C_r$ = Total costs of structure, dollars
$P_r$ = Percent of structure loss

The percent of damage to structure will vary depending on the type of structure, and can generally be expressed in relation to flow depth and time of submergence. For a vented ford, where culverts are embedded in the soil embankment, structural damage may be assumed to be of the same percentage as the damage to the embankment.

3.2.1.3 Interruption of Normal Traffic Flow

There are three significant sub-categories of traffic-related losses:

Increased running cost due to detour
Lost time of vehicle occupants
Increased accidents on detour

3.2.1.3.1 Increased Running Cost ($L_{rc}$)

Increased running cost is the difference between running cost on the detour and the normal route over the period of restoration. The restoration time Tr, is the sum of flood overtopping time, repair time of structure, and waiting time.

The increased running cost can be obtained from the following equation:

$$L_{rc} = (T_r)(ADT)(DD)(\text{unit cost of travel}/1{,}000) \quad (6)$$

where DD = Increased distance (miles)

$T_r$ = Restoration time (days)

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

3.2.1.3.2 Cost for Lost Time of Vehicle Occupants ($L_{tc}$)

The cost for lost time of vehicle occupants can be estimated by the following equation:

$$L_{tc} = (T_r)(N_c)(ADT)(DT)(\text{unit cost of occupants' time}) \quad (7)$$

where $N_c$ = Average number of occupants per vehicle

DT = Increased driving time for detour (day)

The unit cost of occupants' time is in dollars per day.

3.2.1.3.3 Increased Accidents on Detour

Increased chance of accidents on the detour is not considered significant on secondary roads and the increased accident cost may be neglected.

3.2.1.3.4 Cost for Interruption of Normal Traffic ($L_{tr}$)

The total risk cost for interruption of normal traffic is the sum of the increased running cost and the cost for lost time of vehicle occupants, that is,

$$L_{tr} = L_{rc} + L_{tc} \quad (8)$$

3.2.1.4 Damage to Upstream Property Due to Backwater

Where a bridge crossing causes backwater resulting in damage to property in the vicinity of the crossing, the highway agency is responsible for the portion of the damage attributable only to the crossing.

Backwater computation could require a large effort depending on the accuracy needed. For a small bridge crossing where the drainage area is small and stream data are unavailable, simplified methods for backwater computation can be used. The backwater may be assumed as a pond with its watersurface elevation the same as the headwater elevation at the crossing.

Backwater damage losses include losses to crops and buildings in the floodplain. However, the value of crops is generally much less than that of buildings and losses to buildings often outweigh crop losses. Therefore, if no building is involved within the floodplain created by backwater, the backwater losses may be neglected.

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

$$\text{Loss to building} = (\text{cost of building}) \times (\text{percent damage}) \qquad (9)$$

The cost of the building should include the market value of the building and the property contained in the building. Generally, 50% of the building cost is considered appropriate for the property value. The percent of damage, P, to the building for water depth less than 3 ft (0.91 m) from the first floor can be approximated by the following equations:

For residence with basement:

$$P = 12.5 + 5d \qquad (10)$$

For residence with no basement

$$P = 8 + 5d \qquad (11)$$

where d = water from the first floor (ft)

For water depth exceeding 3 ft (0.91 m), the approximate percentage of damage may be estimated by subtracting 1% for each foot (0.305 m) of depth over 3 ft (0.91 m) from the value computed by the above equations.

The highway agency is not generally held responsible for flood damage incurred under normal flow conditions before the bridge is constructed. Therefore, countable losses should be the losses computed as above minus the losses which would result under existing conditions.

## 3.3 TOTAL EXPECTED COST (TEC)

The total expected cost (TEC) per year should include annual capital cost, risk cost and routine maintenance cost. This cost should be compared for all alternative designs to assist engineers in the final selection. Logically, the design alternative with the least total expected cost (LTEC) should be selected as the final design.

## 3.4 DATA NEEDED

For designing small crossing structures, with a total cost of less than $20,000, an exhaustive economic analysis may not be required since the cost saving resulting from the analysis may not justify the considerable effort needed for the analysis. For a larger structure, however, a substantial saving can be obtained by a careful risk-based economic analysis of various alternate designs and selecting the least total cost solution. Data required for the risk analysis include construction cost, site geometry and land use, hydrologic and hydraulic data, traffic data, and flood loss data.

TABLE 2. SUMMARY OF DATA REQUIREMENTS

| Data | Source | Where Used in Analysis |
| --- | --- | --- |
| 1. Construction costs (unit prices of materials) for all structural components | C & M Unit | Capital cost |
| 2. Site geometry and land use | | |
| Contour map | USGS, county, Twp. | Hydraulic analysis |
| Stream cross sections | Field survey | Backwater computation |
| Crops (kind, area, location, elevation) | USDA, Field Survey | Backwater damage estimation |
| Buildings (value, location, elevation) | Field survey | Backwater damage Estimation |
| 3. Hydrologic and hydraulic data: | | |
| Gaging data (stage and discharge) | USGS, SCS, Drainage Manuals | Stage-discharge relationships |
| Watershed parameters | USGS, SCS maps | Hydrograph |
| Flood frequency and magnitude | USGS, state highway | Annual risk costs |
| 4. Traffic data | | |
| Design ADT | Traffic and/or planning units | Traffic detour cost estimation |
| Traffic mix | " | " |
| Vehicle running cost | " | " |
| Average occupancy | " | " |
| Value of time | " | " |
| Length of normal route | " | " |
| Length of shortest detour route | " | " |
| Average speed of traffic | " | " |
| 5. Flood loss data | | |
| Agricultural products | Local agencies, USDA, FEMA, US Army Corps, FHWA | Risk analysis |
| Buildings | | " |
| Bridges and components | | " |

# 4.0 DESIGN METHODOLOGY

## 4.1 GENERAL CONSIDERATIONS FOR LWSC

A number of considerations is listed in Table 3 for the design of LWSCs. These considerations are based on the behavior of LWSCs during a flood, their performance, and the opinions and the common practice of the experienced engineers. Design considerations listed in Table 3 are general in nature, and the criteria listed may vary depending on available funds and traffic count.

TABLE 3. GENERAL CONSIDERATIONS

| CONSIDERATIONS | CRITERIA |
|---|---|
| A. HYDROLOGIC & HYDRAULIC | |
| 1. Frequency of overtopping | Less than 10 times per year |
| 2. Duration of combined overflow and repair time | Less than 3 days |
| 3. Overtopping depth | Less than 1 ft (for ADT <100) for 2-year flow |
| B. GEOMORPHIC AND LAND USE | |
| 1. Drainage area shape | Long and narrow (>3 to 4 times width in length) |
| 2. Stream and basin slope | Steep |
| 3. Channel and overbank | Low valley storage upstream<br>Located in a stable stream reach |
| C. STRUCTURAL | |
| General | |
| 1. Vertical curve at dip | Mild and gradual, as in Table 4 |
| 2. Orientation of structure | Straight; skew should be avoided as much as possible |

TABLE 3. GENERAL CONSIDERATIONS (Cont'd)

| CONSIDERATIONS | CRITERIA |
|---|---|
| 3. Approach length | Long, to provide sufficient site distance for warning signs |
| 4. Height of pavement above the stream bed | Less than 4 ft |
| <u>Fords</u> | |
| 1. Normal daily flow depth | Less than 4 to 6 in |
| 2. Pavement material | May vary from riverbed gravel to concrete |
| 3. Erosion protection | End walls and gabion protection may be desirable |
| | Wide, sloped shoulders in downstream may be helpful |
| <u>Vented Fords</u> | |
| 1. Pavement and fill materials | Should be dense packed; heavy, to withstand erosion and washout. May be encased in concrete for narrow crossings |
| 2. Vents | Pipes of various materials can be used. Should be anchored in the ground; both ends bevelled to allow easy passage of debris |
| | More than one vent should be used but fewer lines of larger pipes desirable |
| 3. Erosion protection | Cut-off walls and splash aprons may be needed. Rip rap protection of slope may be considered |
| <u>Low Bridges</u> | |
| 1. Pavement | Light and loose pavements such as bituminous or gravel pavements are not desirable |

TABLE 3. GENERAL CONSIDERATIONS (Cont'd)

| CONSIDERATIONS | CRITERIA |
|---|---|
| 2. Bridge Deck | Must be heavy to withstand drag, uplift and lateral forces due to overflow and upstream water<br><br>Must be secured to the substructure<br><br>Upstream and downstream edges should be rounded<br><br>Rounded edges with one way camber |
| 3. Erosion protection | Cut-off walls and impervious aprons may be desirable |
| D. <u>SIGNS AND MARKERS</u> | |
| 1. Signs | Must have adequate warning signs Fig. 19 shows sample signs |
| 2. Road Markers | Guard rails are not recommended, to avoid collecting debris<br>Sloped curb rails may be used<br>Road markers may be desirable |

NOTE: 1 ft = 0.305 m

TABLE 4. MAXIMUM SPEED, DEPTH OF DEPRESSION AND RADII OF VERTICAL CURVES FOR A FORD

| MAXIMUM SPEED MPH (KPH) | MAXIMUM RADIUS VERTICAL CURVE FT (m) | MAXIMUM DEPTH DEPRESSION FT (m) |
|---|---|---|
| 25 (40) | 200 ( 61) | 2.0 (0.61) |
| 30 (48) | 300 ( 91) | 3.0 (0.91) |
| 35 (56) | 400 (122) | 4.0 (1.22) |
| 40 (64) | 500 (152) | 5.0 (1.52) |

## 4.2 DESIGN OF FORDS

### 4.2.1 Design Considerations

The simplest type of ford normally consists of an unsurfaced crossing formed by levelling the streambed over the width of the roadway. In some fords, a row of boulders along the downstream edge of the roadway is also provided, which are backfilled with loose gravel or gabion as shown in Fig. 5.

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 (6). Instead of endwalls, road shoulders may also be covered with asphalt or concrete slab to prevent erosion of the road foundation. Some examples of these types of improved fords are shown in Fig. 6.

Depending on available funds and traffic volume, fords may be further improved like structures shown in Fig. 7. The road surfaces are of a thick concrete slab and are sloped toward downstream for drainage purpose. A successful and maintenance-free ford, should include the following components:

-- An unerodible paved roadway over which vehicles can smoothly run.
-- Two end cutoff walls, one on each edge of the roadway, of sufficient depth to provide support to the pavement and counter subsurface flow.
-- A rockfilled gabion or concrete shoulder on the downstream side to check scouring of the streambed.
-- Markers to help drivers spot the limits of the roadway when flooded.

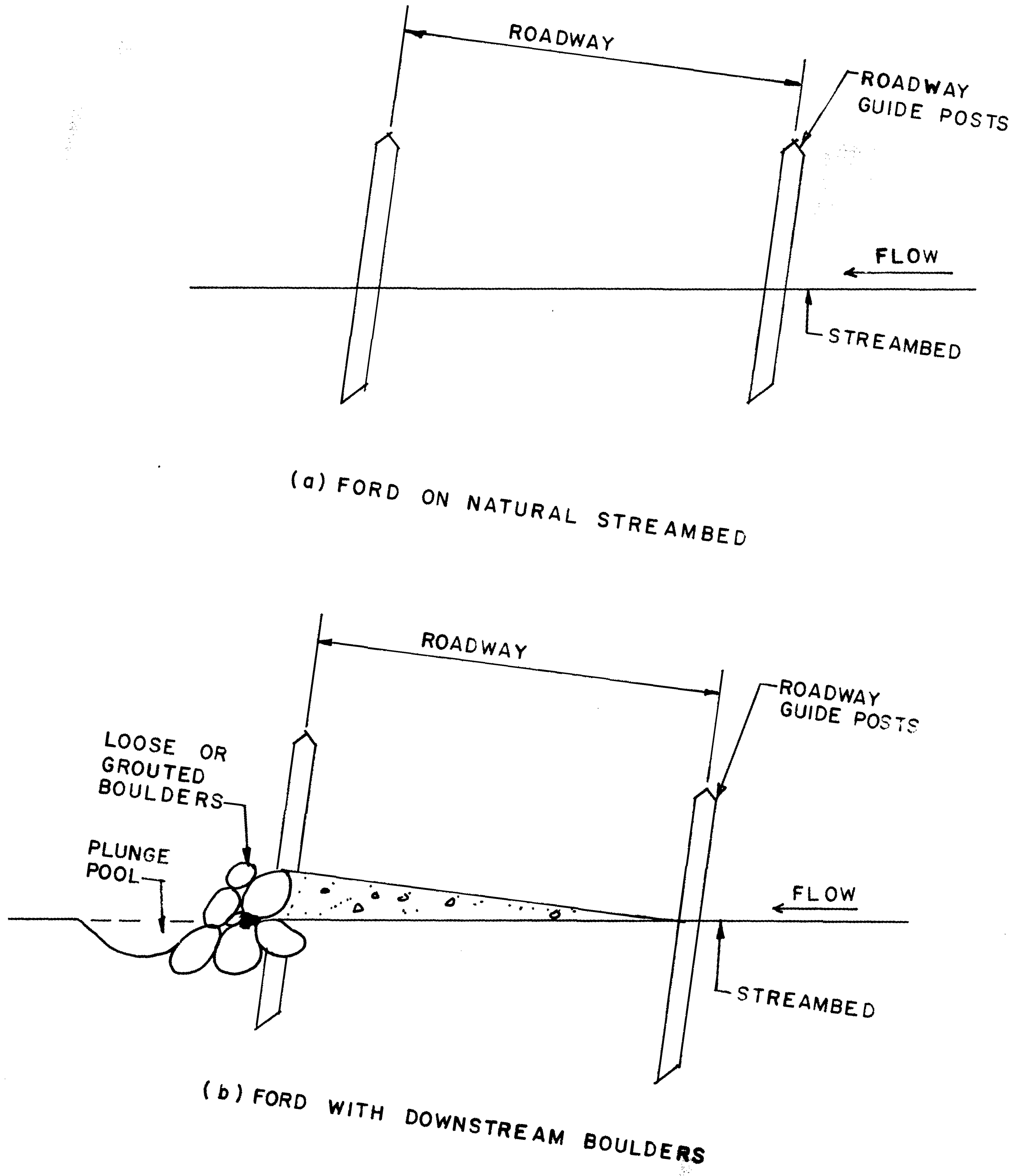



FIG. 5 UNIMPROVED FORDS

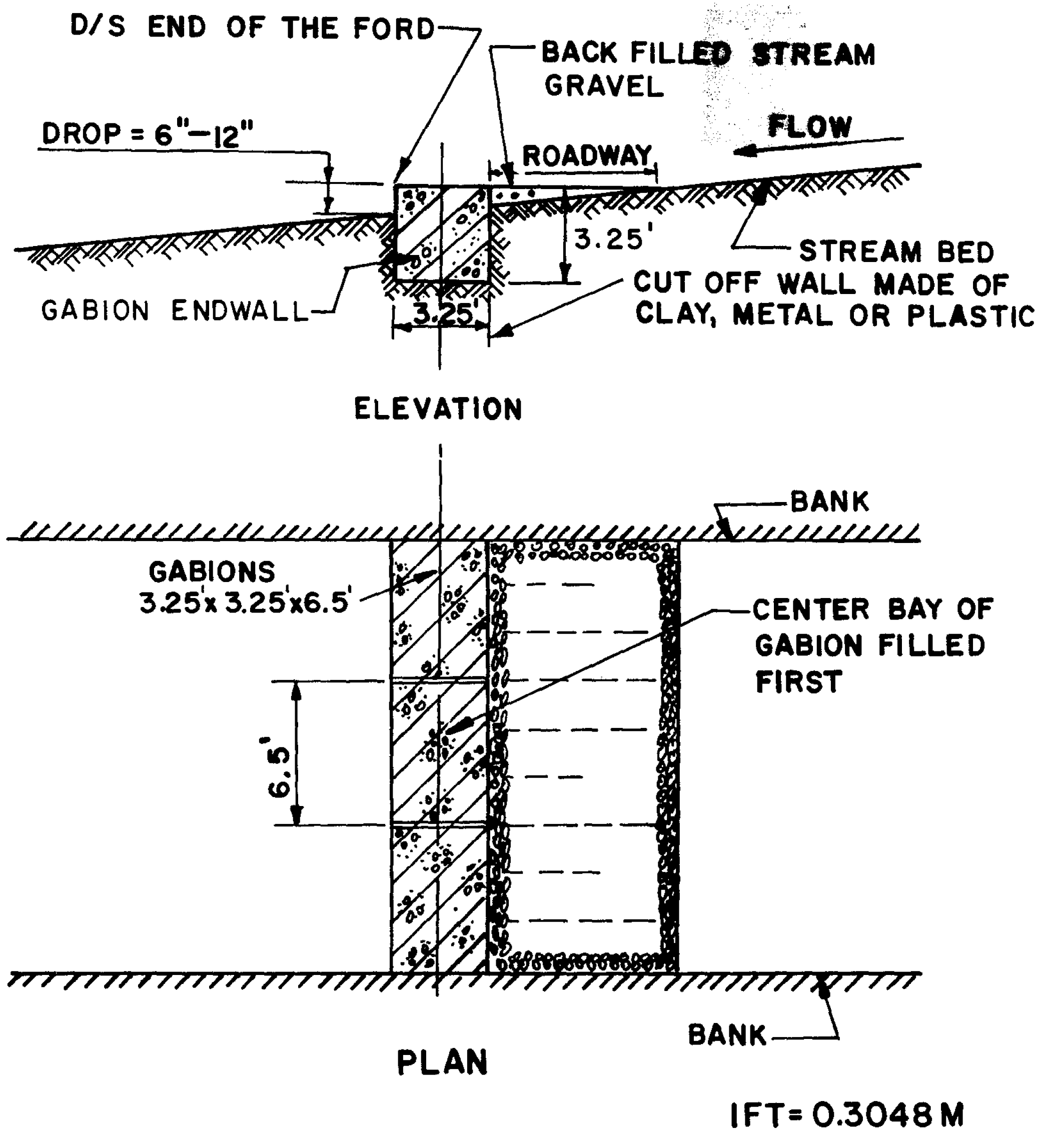
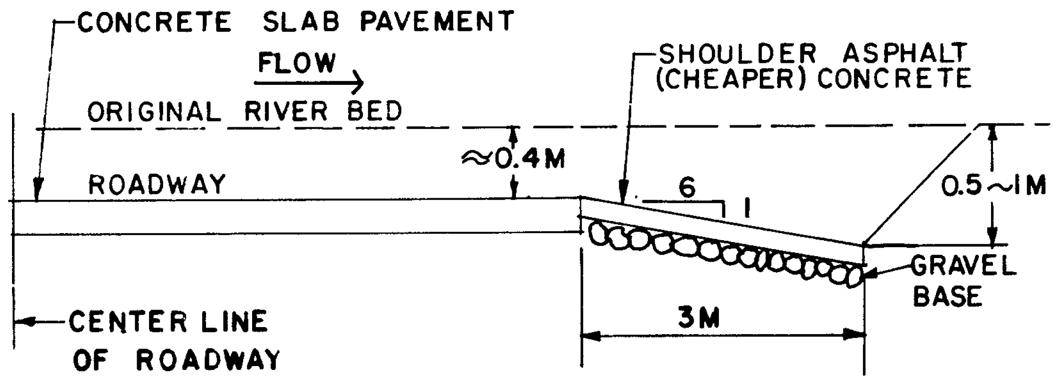
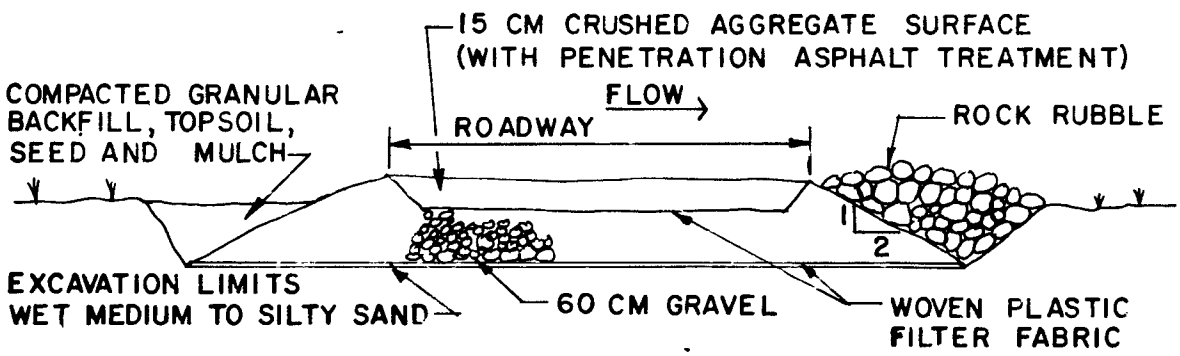



FIG.5 (c) GABION FORD

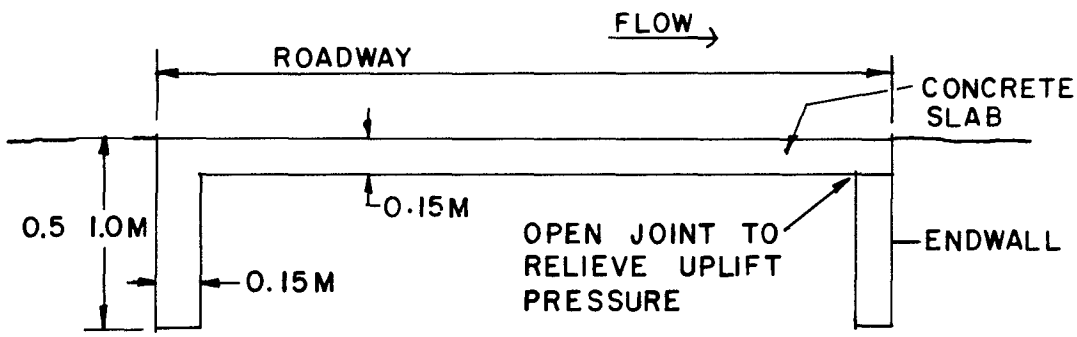



(a) FORD WITH WIDE SHOULDER

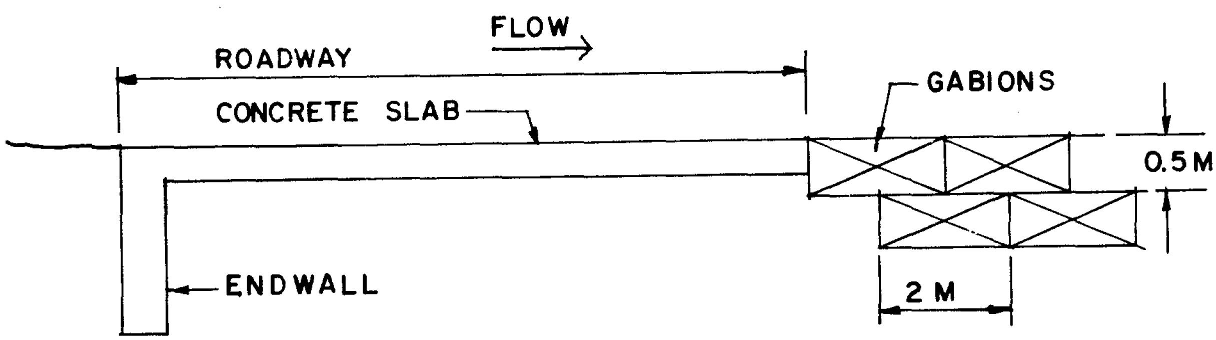



(b) FORD WITH ASPHALT

Fig. 6 SLIGHTLY IMPROVED FORDS

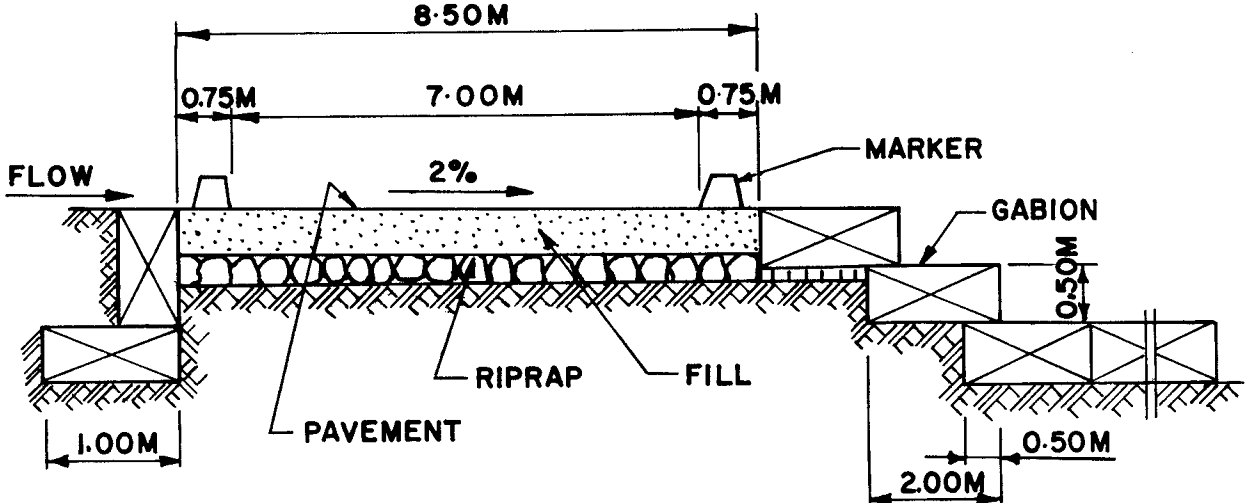



(a) CONCRETE SLAB FORD

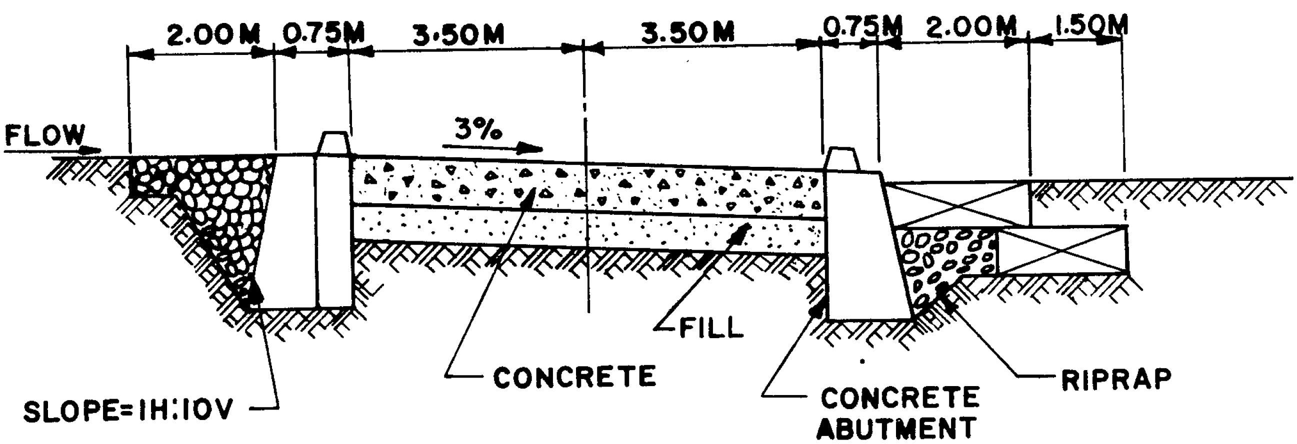



(b) FORD WITH GABION ENDWALL

FIG. 7 IMPROVED FORDS




(c) FORD ON SCOURABLE BED




(d) FORD ON UNSCOURABLE BED

FIG. 7 (CONT'D) IMPROVED FORDS

Roadway pavement can either be a concrete or asphalt-treated surface or of gravels, rocks and gabions. 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 their inherent advantage of providing adequate strength to carry traffic, even 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, such pavement may not be desirable for many cases. When used over erodible soils, it should include a proper filter to prevent piping and consequent undermining of the pavement. Asphalt treated surfaces can also be used as pavement. However, because of their lighter weight, an adequate layer of asphalt and aggregate, in the range of 4 to 6 in (10 to 15 cm) must be used.

Gabions are boxes of steel wire fabric filled with stones, constituting a solid mass so heavy that it becomes difficult for water to displace them. At the same time, they are flexible enough to adjust and strengthen themselves when the soil underneath is eroded. Therefore, gabion-walled fords provide both flexibility and resistance to the erosion process. However, vehicles passing over them may break down the wires on the driving surface, requiring occasional refixing. An example of a gabion ford performing satisfactorily in the United States is shown in Fig. 5(c).

The primary roles of endwalls are to protect the edges of fords from erosion and to provide support to the pavement during flood when scouring activity is enhanced. Walls should normally rest on solid bedrock or non-erodible soil or carried to a depth below the expected scour depth. If the streambed is rocky and the depth is small, the walls could simply be stone masonry. If either the depth is relatively high or the bedsoil erodible, stone masonry walls are replaced by concrete walls. When the bedsoil is erodible and depth relatively high, gabions and aprons on both sides, upstream and downstream, are recommended. For downstream sidewall, an open joint should be provided to relieve the uplift pressure of seeping water.

Markers generally are small posts equidistant along each edge of the roadway. Their height usually depends upon the depth of flood water over the ford which may be considered safe for passage of vehicles, and may be up to 4 to 6 in (10 to 15 m). If the markers are not visible (or totally under water), drivers would then be warned that the water depth is too high and dangerous for crossing.

### 4.2.2 Hydraulics of Fords

Normally the roadway of a ford should follow the natural contour of the streambed to assure the minimum disturbance to the streamflow. Sometimes, for fords across a dry creek where flow is very infrequent and for streams where floodwater carries large amount of sediments, the roadways are slightly depressed below the streambed to protect the road foundation from being eroded by floods. For this type of fords, the flow over the depressed road surface differs from the stream flow at the beginning of a flood. However, the depression is quickly silted and the level rises to the streambed when natural flow condition is established. The flow condition over a flat or depressed ford, therefore, should be considered the same as the flow under a natural condition (without the structure).

Where the highway is to cross an abrupt dip, the roadway should be raised to make a smooth vertical curve so as to reduce the impact of passing vehicles. The flow over such a raised ford is comparable to the flow over a broad crested weir. In the following sub-sections, discharge-depth relationship and exit velocity of the flow over a raised ford are discussed.

#### 4.2.2.1 Discharge-Depth Relationship

The flow over a raised ford, which behaves like a broad-crested weir, is given by the equation:

$$Q = C_f.L.H^{1.5}.(C_s/C_f) \qquad (12)$$

where

$Q$ = Discharge

$C_s$ = 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 overflow section (ft)

$H$ = Total head upstream measured above the crown of the roadway (ft)

The values of $C_s$ and $C_f$ can be obtained from Fig. 8

#### 4.2.2.2 Exit Velocity

The exit velocity at the downstream side of the roadway embankment should be computed based on the downstream flow depth. The downstream flow depth, $y_2$, can be determined in accordance with the elevation of the tailwater:

1. If the tailwater is higher than the roadway, the downstream flow depth, $y_2$ = tailwater elevation minus roadway elevation.

2. If the tailwater depth is lower than the roadway elevation, the downstream flow depth, $y_2 = 0.48H$ where $H$ = upstream depth of water from the roadway crown.

Then, the exit velocity, $V_2 = \dfrac{Q}{y_2.L}$ (13)

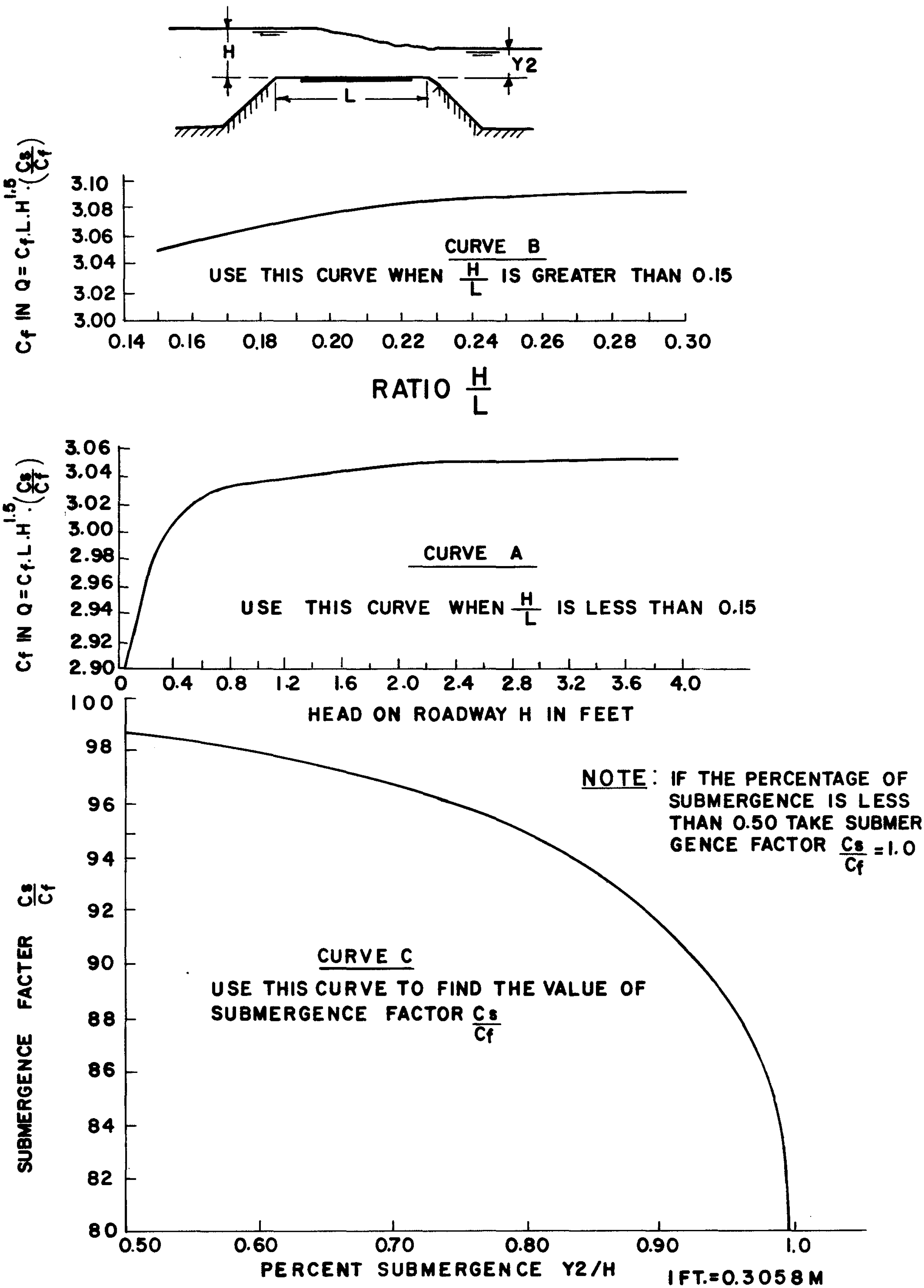



FIG. 8 COEFFICIENTS FOR WIER FLOW

Erosion protection measures, if needed, should be selected and designed based on the exit velocity.

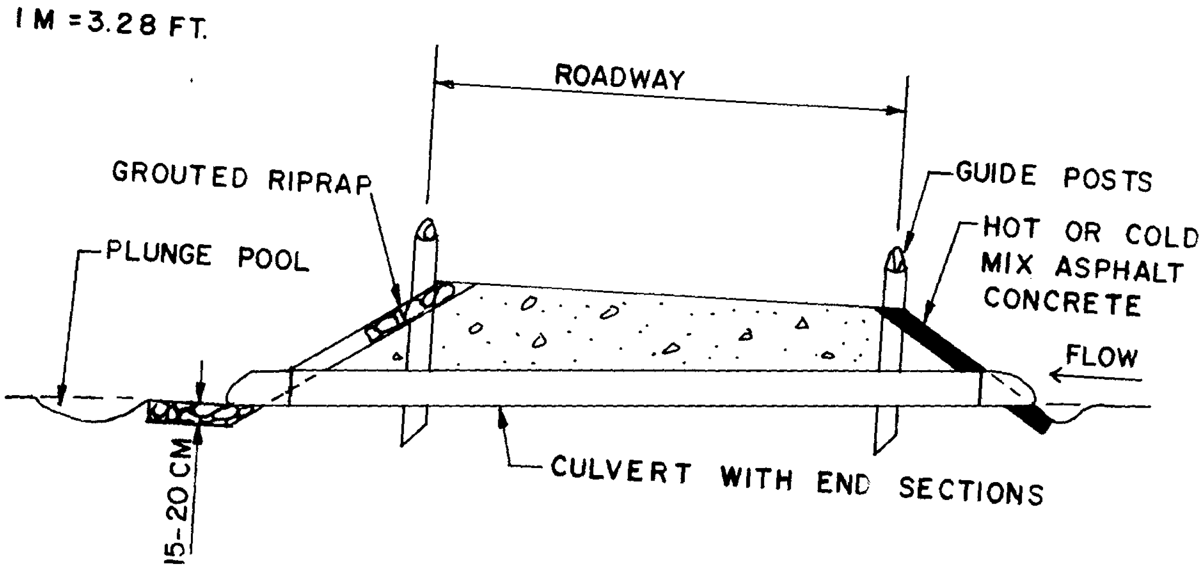
Where the depth of flow varies along the roadway, it is advisable to divide the inundated portion into reaches, and compute the discharge and velocity of each reach separately. For the design of protection measures against erosion, the maximum of the various computed mean velocities should be used.

## 4.3 DESIGN OF VENTED FORDS

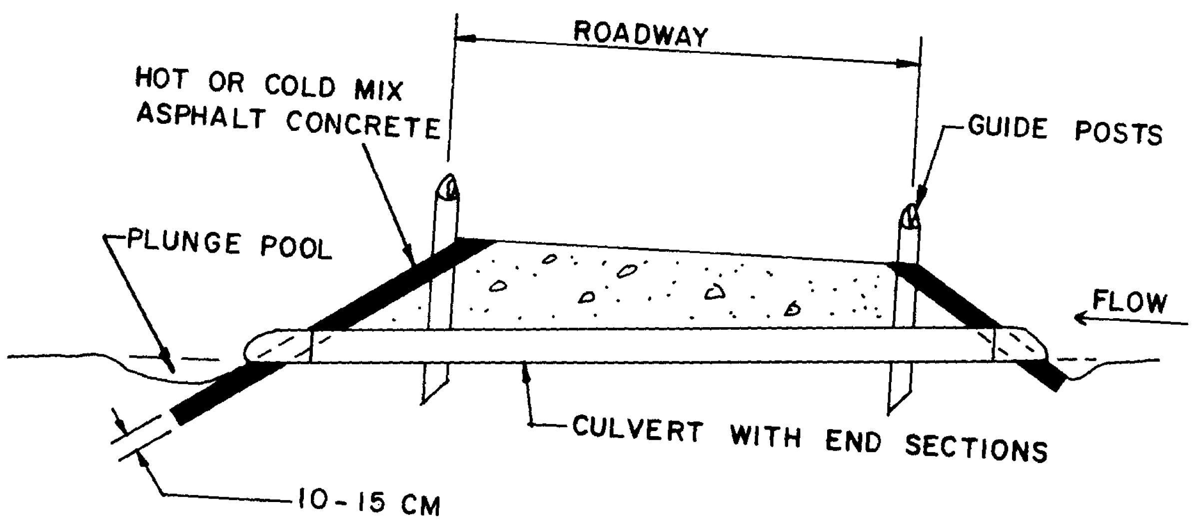
### 4.3.1 Design Considerations

When the roadway of a ford is raised from the original streambed, a vent may be added to the ford to drain water that will be ponded upstream of the structure. The vent may be composed of pipes or culverts of various shapes generally laid directly over the riverbed. The size of the pipes and their length depend on design flow and road width. The vents or the pipes may be encased in concrete, they can also be encased with gravel fill or embedded in soil embankment. Generally, both ends of the pipes, upstream and downstream, are beveled for smooth passage of flow 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 may also be used to pave the concrete roadway. In Fig. 9, small vented fords of this type are presented. Corrugated metal pipes are most commonly employed for construction of vented fords. Other types of pipes which can be used, depending upon availability and relative cost, include iron pipes, PVC pipes, concrete pipes, etc. Some typical vented fords are shown in Figs. 9, 10, 11, 12, and 13.

Design considerations for a ford were discussed previously in Section 4.2.1. For a vented ford, considerations related to the vent should be added:




(a) GROUTED RIPRAP AND ASPHALT SECTION




(b) ALL ASPHALT COVER

FIG. 9 SIMPLE VENTED FORDS

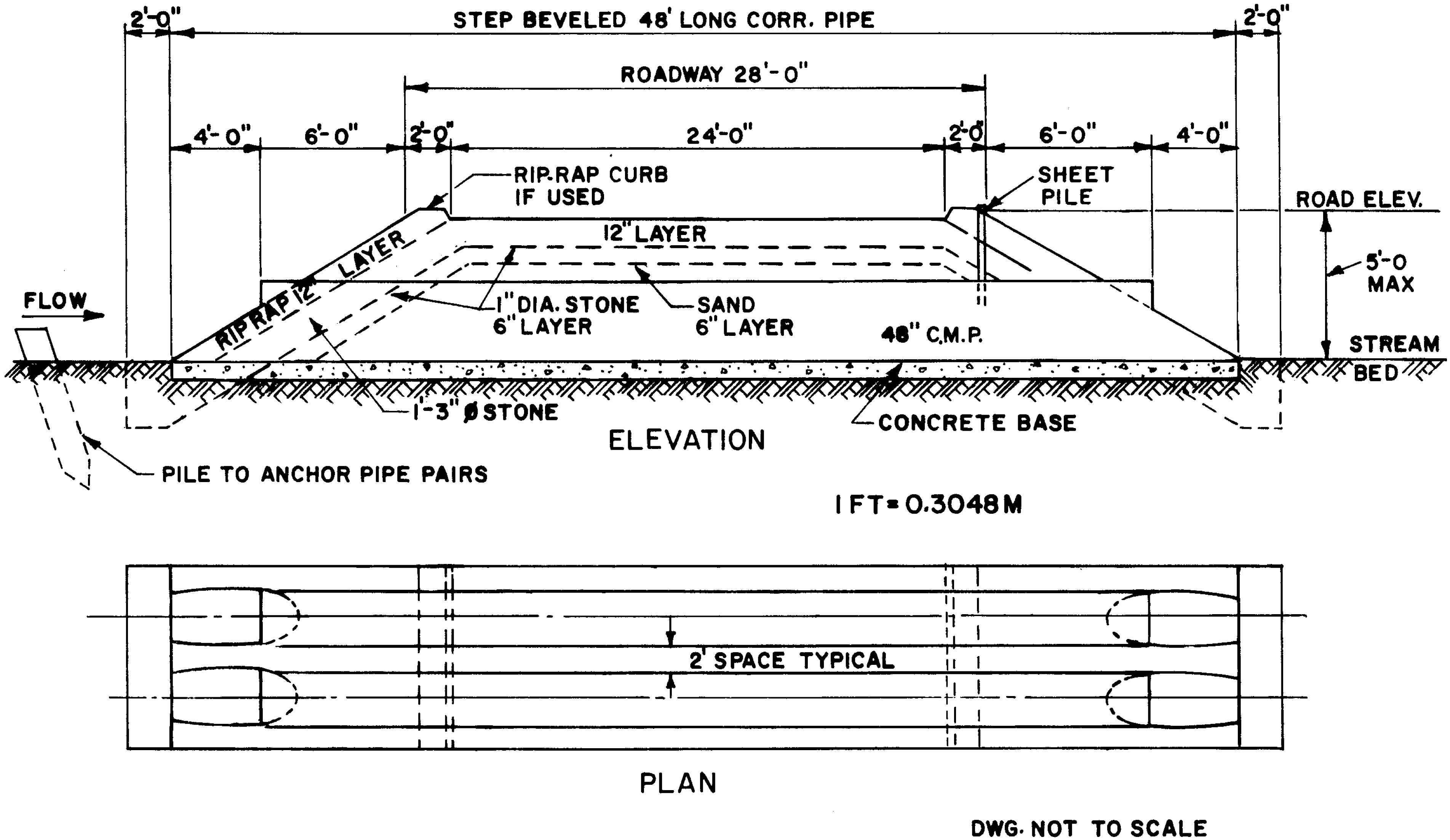
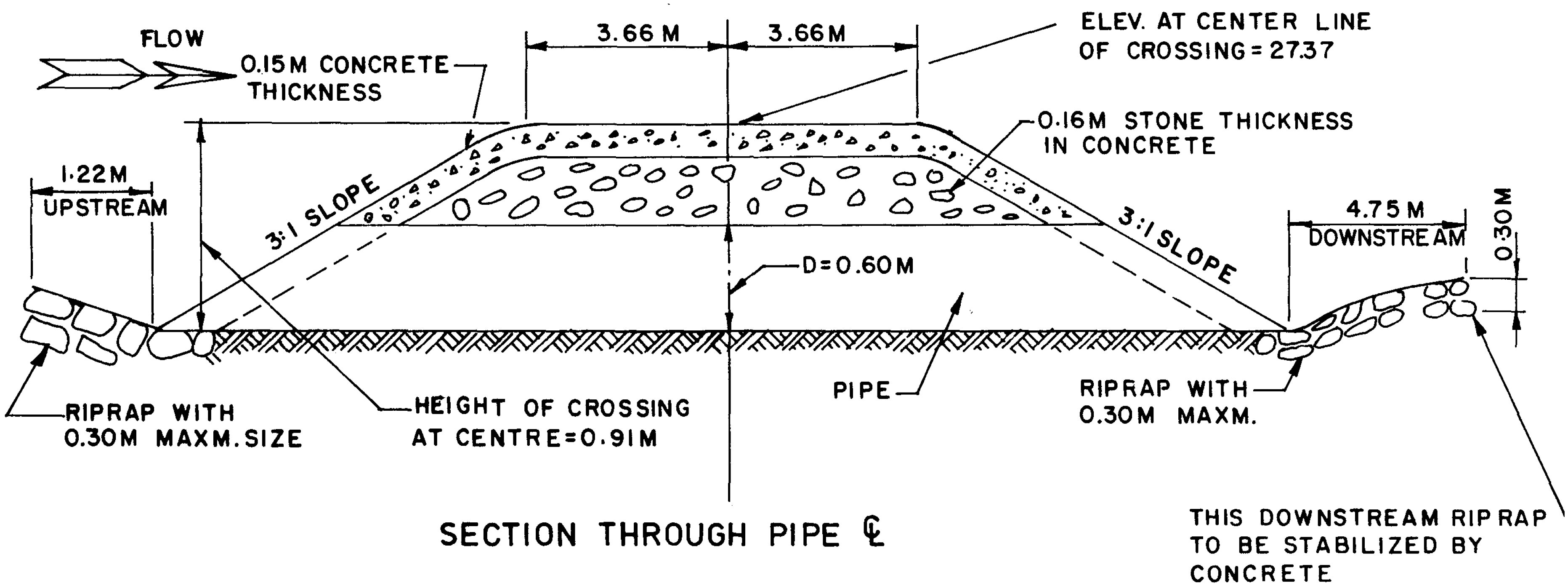



FIG. 10 VENTED FORD, GRUNDY COUNTY, IOWA




SECTION THROUGH PIPE ℄

DWG. NOT TO SCALE 1M=3.28 FT.

FIG. 11 VENTED FORD, SASKATCHEWAN: TYPE I

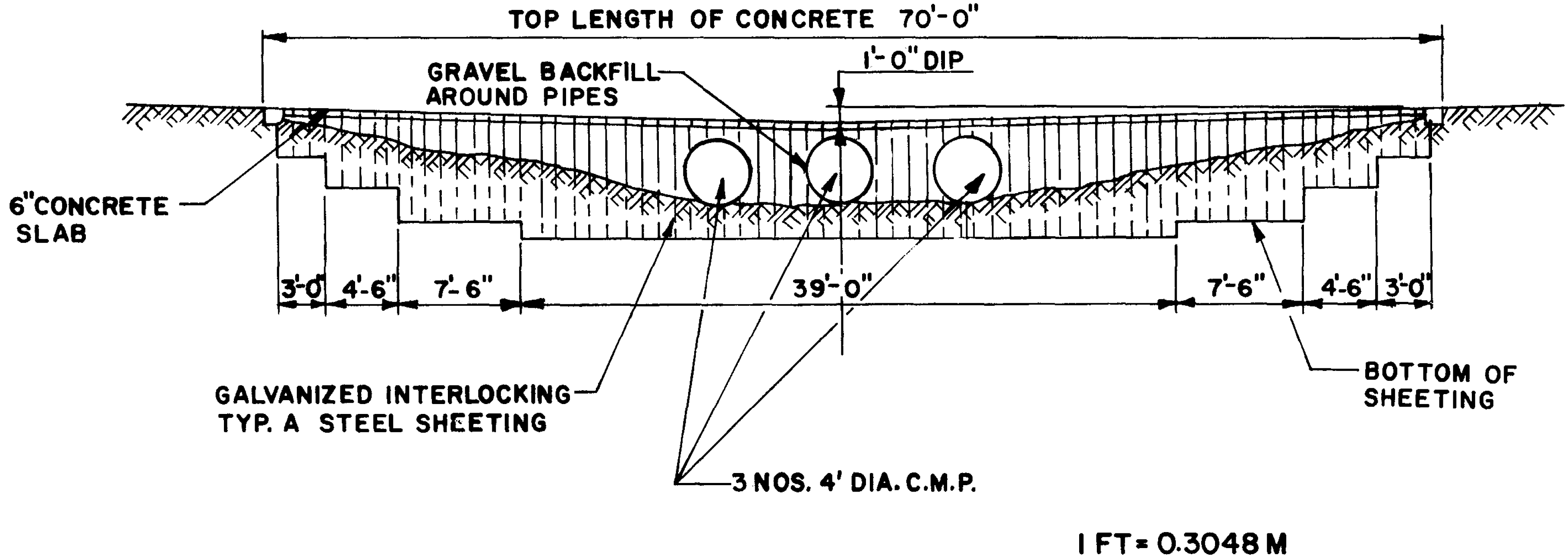



FIG. 12 VENTED FORD, SASKATCHEWAN: TYPE 2

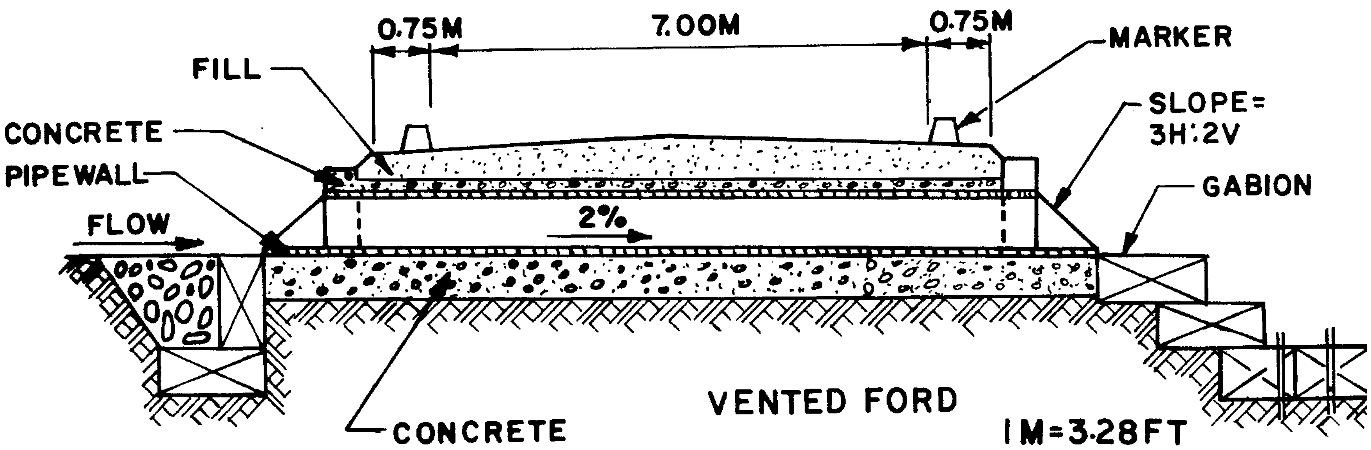



FIG. 13 FRENCH DESIGN OF LWSC IN AFRICA

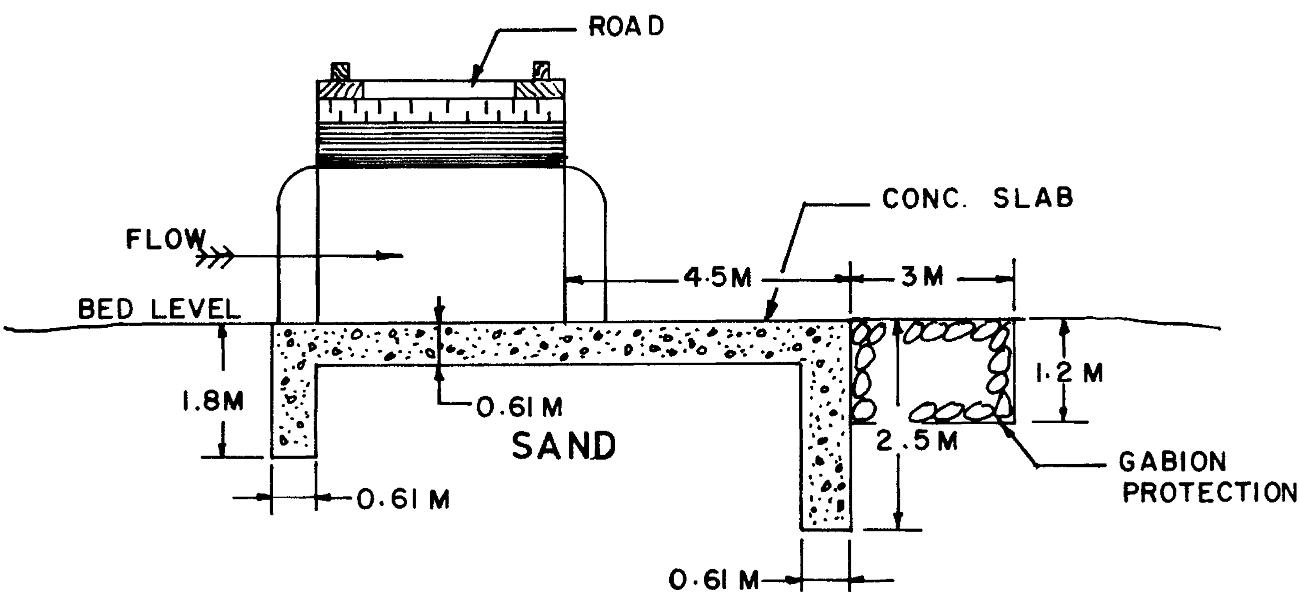



FIG. 14 A SKETCH OF EXTENDED CONCRETE SLAB

A vented ford constricts the stream flow into a narrow vent creating extremely high velocity and severe erosion at the vent outlet. A splash apron may be provided at the culvert outlet to protect the streambed immediately downstream so that it does not undermine the structure. It may be a solid concrete slab as shown in Fig. 14 or extended riprap or gabions which will move a scour hole further downstream, preventing undermining of the roadway and culverts.

Care should be exercised in selecting the culvert so that the size is large enough to limit exit velocity of the flow not to exceed about 10 ft/sec (3 m/sec). Otherwise expensive outlet protection may be necessary.

Vents and the embankment should be sloped, since it is believed that the sloped entrance and embankment catch less debris and have a natural self-cleaning tendency during highwater, ensuring uninterrupted flow through pipes.

No guard rails are recommended for a vented ford to avoid catching debris and floating materials during a flood. If curbs or wheel guards are used, sloping them towards the downstream side would be desirable.

For economic reasons, earth embankment is often used for a vented ford. Particularly for a high and long embankment, it may otherwise become very expensive. Although expected flood damage due to overtopping of flow is rather high for earth embankment, the saving on the construction cost may outweigh the cost incurred from the damage. Protection of slopes, however, by riprap may be desirable.

For a vented ford across a very wide floodplain, overflow sections of the road embankment can be constructed away from the culverts so that damage to the structures and embankment on top and around them are reduced.

It is recommended that for any sizable structure, a systematic economic analysis be undertaken in order to obtain an LTEC design which may offer substantial cost savings. The following procedure is suggested:

1. Select an initial design which may seem most economical, and estimate its capital cost (costs of the main structure, culverts, roadway pavement, embankment, approach fills, and erosion protection).

2. Determine the flow condition and water surface elevations for floods of different magnitudes by hydrologic and hydraulic analyses. Annual risk costs of anticipated flood damage are then estimated (damage costs for the structure and properties in the vicinity, and traffic detour costs).

3. Determine the total annual cost of the crossing, approaches, anticipated flood damage and traffic detours or delays by adding annual capital costs to risk costs.

4. Alternative designs are then selected by changing various components of the initial design. By repeating steps 1-3, annual capital costs and risk costs for the new designs can be determined. An increase in capital cost due to improvement of structural design is usually associated with a reduction in risk cost. If the amount of increase in capital cost of the alternative design exceeds the reduction in risk cost, the total cost of the alternative design is greater, and the change in design is not economically sound. On the other hand, if the increase in capital cost is less than the reduction in risk cost, the alternative design is preferable and further improvement in the same direction can be made.

5. This process is repeated until the total cost of a design is least so that the increase in capital costs equals approximately the decrease in risk costs.

The following measures are suggested in selecting the initial design:

1. Set the roadway elevation at about the stage of 1-year flood.

2. Choose the largest allowable culvert size, considering the minimum cover required.

3. Fewer lines of larger culverts are preferable to many lines of smaller culverts. One line of culvert is not preferred because of possible blockage by debris.

4. For a low crossing in a wide stream, the depth of overtopping flow is usually shallow and resulting erosive action is not high. Use of expensive riprap for embankment protection may not be economically justifiable.

### 4.3.2 <u>Hydraulics of Vented Fords</u>

Two types of flow conditions occur at a vented ford: (1) when the water elevation is lower than the crest of the vented ford, all the water flows through the vent, constituting a culvert flow, and (2) when the flow depth in the stream is higher than the height of the crest, a portion of water overtops the crest, and along with the flows through the culvert, creates a weir flow condition.

#### 4.3.2.1 Discharge-Depth Relationship

The discharge and depth relationship for culvert flow can be determined from the charts developed by the Federal Highway Administration. These charts are readily available in many culvert handbooks, including Hydraulic Engineering Circular No. 5 (3) and therefore, they are not repeated here.

When a vented ford is overtopped, the discharges through the culverts and over the ford can be determined by using the 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. An example is presented in Section 5.2.2.3.

#### 4.3.2.2 Exit Velocity

Upon finding the overflow and the culvert discharges, the exit velocity of the overflow and the outlet velocity of the culvert flow can be computed separately. The exit velocity of the overflow was already discussed in Section 4.2.2.2. The outlet velocity of the culvert flow can be obtained by dividing the culvert discharge by the cross-sectional area of the culvert barrel. Based on these computed velocities, erosion protection measures for embankment and for culvert outlets can be designed.

## 4.4 DESIGN OF LOW BRIDGES

### 4.4.1 Design Considerations

A low-water bridge usually includes the following components:

- Foundation, resting on bedrock or hard soil; or raft type with cutoff walls and impervious aprons.
- Substructure to support the roadway slab and provide an adequate opening for passage of day-to-day flow and normal flood flow.
- Superstructure, consisting of roadway slab or deckslab, curbs or guiding stones.
- Approaches

When hard rock or a hard soil which is not erodible at flow velocity during flooding is available at shallow depths, footings may be used to transfer the load from the substructure to the rock or soil. Sandy strata is generally not suitable for low-water crossings but where there is no other choice, a raft type of foundation with cutoff walls and impervious aprons can be employed. The concrete raft serves to distribute the pressure evenly to the sand surface. The level of the raft floor is usually kept at some depth below the bed level to prevent the floor from acting as a weir when retrogression of bed levels takes place. The well foundation type is adopted when the bedrock is at a large depth and the raft type is not considered economical.

Piers and abutments of a low bridge require streamlining to reduce resistance to the stream flow. The abutments should not be projected excessively into the stream since this would constrict the flow path generally creating scour potential. Piers should be well anchored to the foundation and the upstream edge of the pier pointed so that accumulation of debris is minimized. For smaller low bridges, piles of various kinds have been substituted for regular piers in order to reduce construction cost. Piles are found particularly suitable and economical for weak soil such as silt.

Superstructures commonly used for support to roadway pavement include: arches, R.C. rigid frames, R.C. slabs, steel or concrete beams supporting wooden deck, steel beams supporting R.C. slab, etc. Arch-type low-water bridges have been adopted primarily for longer bridges for many river crossings in India. In the United States they are not common. The R.C. slabs with concrete or masonry piers and abutments have been most commonly used in the United States. For example, data obtained from the Virginia Department of Highways and Transportation reveal that most low-water bridges (approximately 60%) in Virginia are of this type, with a concrete deck slab resting over solid concrete piers and abutments.

The deck slab for low-water bridges may be as thick as for high-level bridges. However, upstream and downstream edges of the deck slab are usually smoothly rounded to enhance the efficiency of discharge over the slab during overtopping. In addition, the deck slab should be well anchored to the substructure to prevent its being carried away by flood water.

The low bridge is particularly susceptible to floating debris, which exert a horizontal force on the bridge whose impact may cause considerable damage and occasionally overturn it. Bridges with spans of 20 ft (6.1 m) to 25 ft (7.6 m) have been found sufficient to pass debris freely. Also, in order to reduce the risk of overturning, the height of the piers should be limited to about 10 ft. For the bridge deck, the general tendency is to discourage use of markers or guide posts on low-water bridges construct- across streams where frequent occurrence of floating trees and debris can result in substantial damages. Low solid curbs (about 6 in or 15 cm height) with drain slots may be used instead.

The risk analysis described in Chapter 3 may assist the decision maker in selecting the optimal design. The same procedure suggested in the previous section should be followed.

### 4.4.2 Hydraulics of Low Bridges

Basically, a low slab bridge can be of two types: closed or concrete bottom and open bottom. The concrete bottom bridge is hydraulically the same as a box culvert, which has been discussed earlier. The open bottom bridge is hydraulically the same as a high bridge.

#### 4.4.2.1 Discharge-Depth Relationship

For an open bottom bridge, the flow depth upstream of the bridge can be computed by adding the flow depth without the bridge and the height of the backwater created by the bridge. The accurate backwater computation, however, is time consuming and complex. The following equation may be used instead for an approximation:

$$h^* = C_D \, (A_s/A)(V^2/2g) \qquad (14)$$

where $C_D$ = drag coefficient, use $C_D$ = 2 for LWSC
$A_s$ = projected area of slab, pier, and abutments on a plane perpendicular to flow
$A$ = cross-sectional area of flow
$V$ = average velocity of flow = $Q/A$

4.4.2.2 Exit Velocity

The outlet velocity of the flow can be determined by dividing the discharge by the cross-sectional area of the flow at the bridge.

## 4.5 EROSION PROTECTION

The most common and devastating problem encountered with low-water stream crossings is erosion of the streambed, embankments, and foundations. The life of the structure is often dictated by the extent and behavior of protective measures employed. Covering the entire erodible surface with hard materials, such as asphalt, concrete or soil cement is often costly and may be impractical for low-water stream crossings where economy is the chief consideration. Riprap has been used by highway engineers for protecting embankments of low-cost bridges because of its economy and high degree of effectiveness.

Although the economic analysis will finally determine whether erosion protection measures are justified, the following general considerations may be used for the preliminary evaluation of the need for erosion protection.

Erosion protection for embankment slope or downstream bed may be desirable if:

-- Tailwater elevation is less than the elevation of overtopping flood.
-- Velocity over the roadway is more than 4 ft/sec (1.22 m/sec).
-- Duration of overflow is more than 10 hours.
-- Frequency of overtopping is more than 10 days per year.
-- Depth of overflow is more than 6 in (0.15 m).
-- Exit velocity is more than 10 ft/sec (3.05 m/sec).

### 4.5.1 Types of Riprap

The following riprap types can be used to protect embankment and streambeds:

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

The above riprap types have been listed in order of the construction costs from less expensive to more expensive. For low-water stream crossings where economical design is the main criterion, dumped riprap and hand-placed riprap may be preferred. Gabions should be used where erosive action is vigorous and the foundation especially vulnerable. For a small structure, concrete slab, though expensive, may be justified where erosion is a chronic problem. Concrete gravel or soil cement may be substituted for solid concrete to reduce cost. The selection of type and size of average, above all, must be commensurate with the funds available and the degree of protection desired.

### 4.5.2 Size of Stone

The size of riprap needed to protect an overflow embankment from erosion can be determined by using Fig. 15. Size $D_{50}$ is the diameter of a spherical stone that would have the same weight as the 50 percent size of the stones. Local mean velocity where riprap is to be placed should be used. The third variable in the figure is the slope of embankment or streambed. These curves have been developed for stones having a unit weight of 165 pounds per cu ft and with a 41° angle of repose. If the angle of repose of stone is other than 41°, the following correction should be made:

$$D_{50}' = D_{50} \cdot (\sin \emptyset / \sin 41^{\circ}) \cdot [\sin (41^{\circ} - \alpha) / \sin (\emptyset - \alpha)] \tag{15}$$

where $D_{50}'$ = corrected stone size
$\emptyset$ = angle of repose of stones
$\alpha$ = embankment slope

### 4.5.3 Bedding Stone and Filter Fabric

Stone riprap requires graded bedding of stone so that the foundation soil will not be siphoned out through the void of the stones. Foundation with fine and uniform soil should be covered with graded bedding stone before stone riprap is placed. Commercial filter fabrics may be used instead of graded gravel filter because they are economical, available, and durable. Filter fabrics are highly permeable to water yet keep the soil in place.

The user is cautioned to consult engineering references before selecting the type of riprap, bedding stone, or filter fabrics to be used at a particular installation. At some sites, it may be necessary to specifically design the type or size of rock that will be most satisfactory.

Where flow velocity is extremely high and erosion a chronic problem, more expensive but, in some cases, more effective erosion protection measures may need to be used. Such measures may include:

-- Projecting culvert far downstream from the structure
-- Constructing endwall and wingwall at inlet and outlet of culvert
-- Use of slab apron
-- Plunging pools to dissipate erosive energy
-- Impact structures at culvert outlet

Descriptions of these measures are beyond the scope of this guide. When these measures are to be used, the designer is advised to consult the appropriate manuals, such as the Hydraulic Engineering Circular No. 14, Hydraulic Design of Energy Dissipators for Culverts and Channels, Federal Highway Administration.

### 4.5.4 Erosion Protection of Approach Sections

Approaches at either end of a low bridge is normally protected by a wingwall abutment. It keeps the approach embankment fill from spilling into the channel and holds the roadway at the bridge surface elevation. The backfill soil pressure, particularly when it is saturated with water from overflow and when the flood recedes rapidly, is generally high. The abutment, therefore, should be bulky and rest on firm foundations to resist such pressure. A typical design is shown in Fig. 16. The angles of wingwall may vary depending on the direction of the approaching flow. They should be aligned so that water approaching and leaving the bridge flows smoothly around the abutments without much disturbance.

## 4.6 DEBRIS CONTROL STRUCTURES

Basically, low-water stream crossings are relatively cheap structures. Thus, construction of debris control structures should be avoided

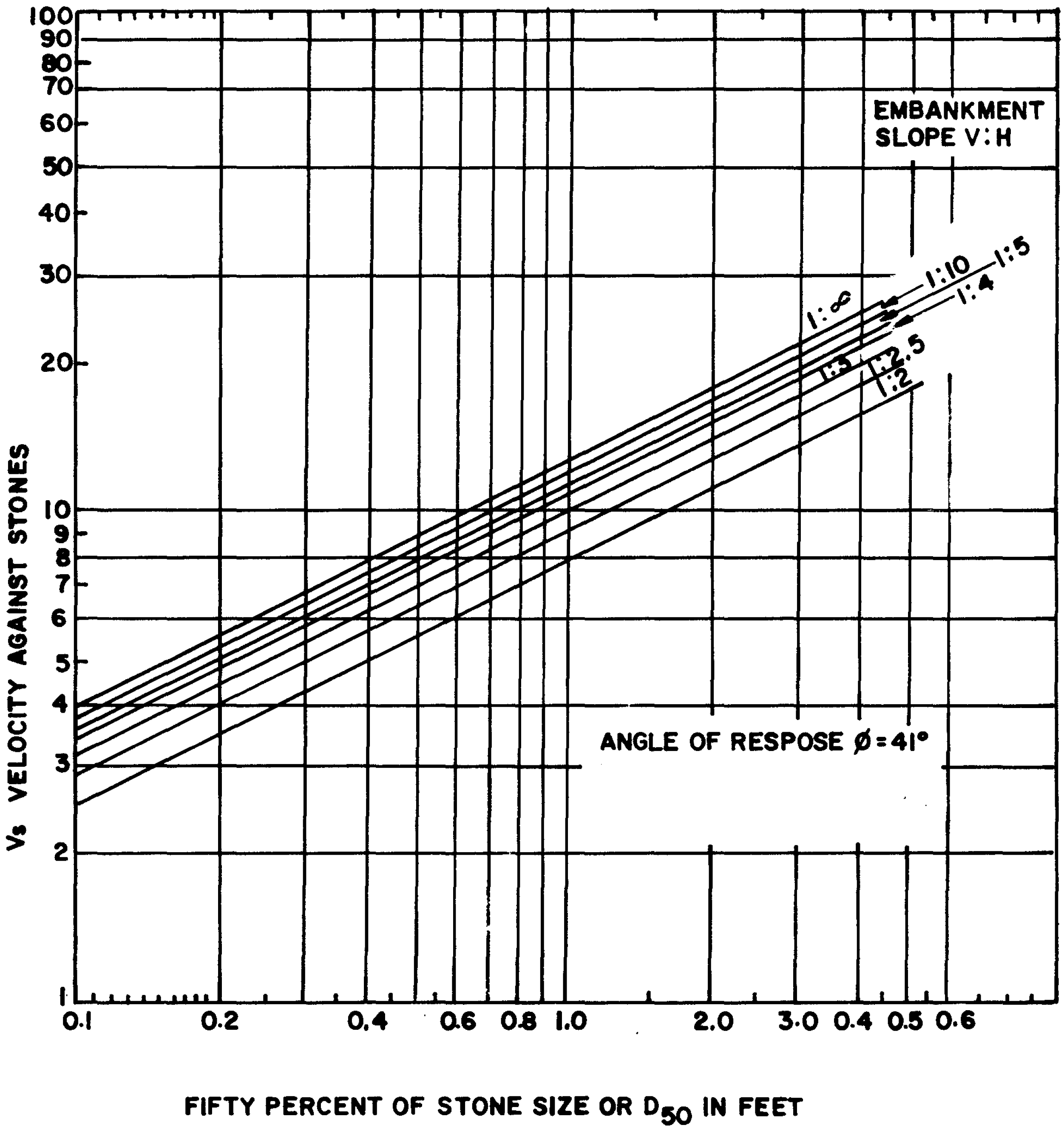



FIG. 15 LWSC RIPRAP SELECTION CURVE

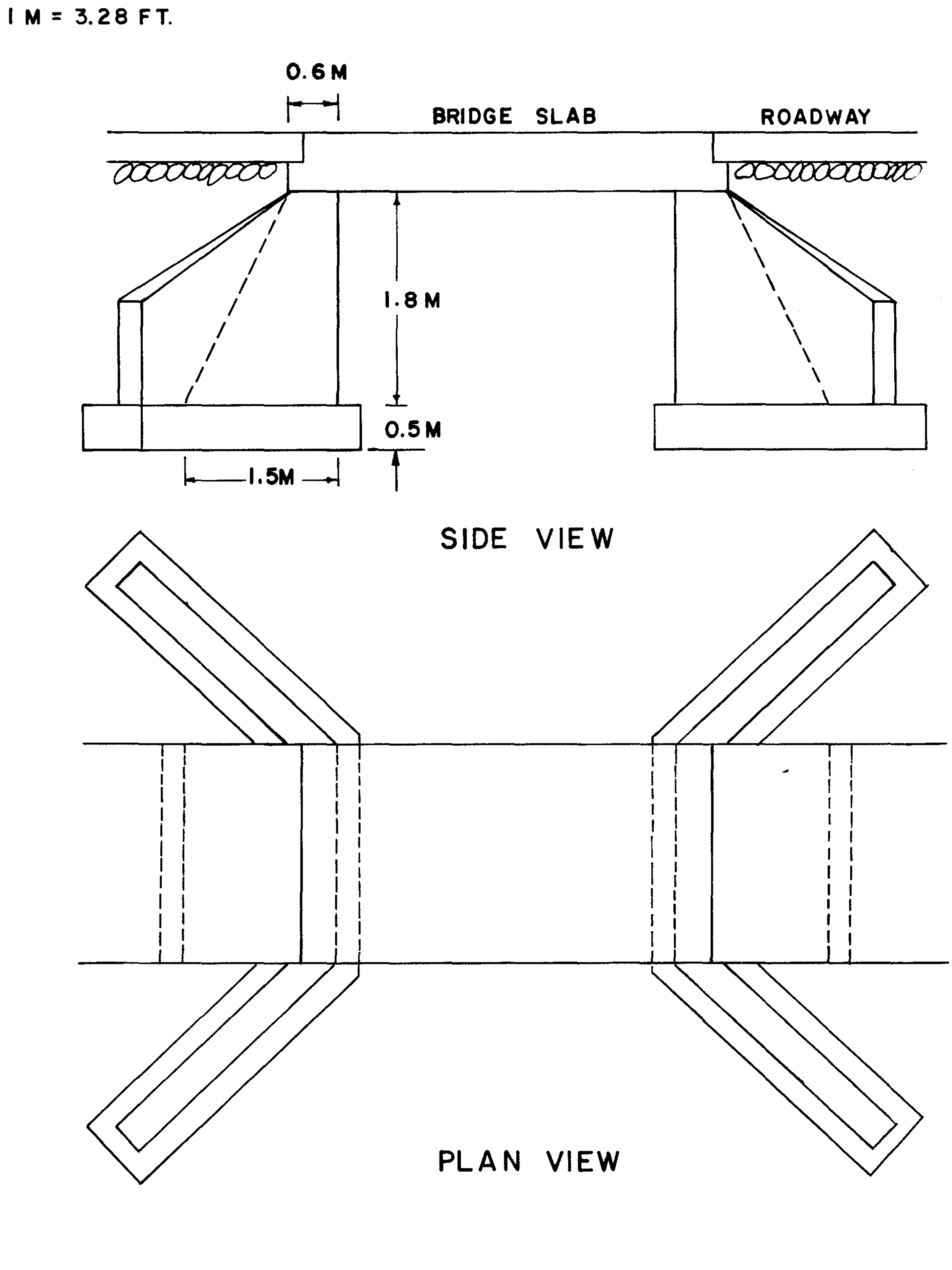



FIG. 16 WINGWALL ABUTMENT

with cost factor in mind. The dimensions of culvert or low-bridge openings should instead be designed to let debris pass with as little expected problem as possible.

If too much debris accumulates upstream of an LWSC structure, the result may be the failure of the structure. Some locations with wooded drainage areas have streams carrying considerable debris, where debris control structures are worthwhile.

Some practical debris control structures for culverts used by highway engineers are:

1. Debris deflector to deflect the major portion of the debris away from the culvert entrance.
2. Debris fin to align debris for passage through the culvert barrel without clogging the inlet.
3. Debris rack to collect the debris before it reaches the culvert entrance.
4. Floating drift boom to collect floating drift.

For details, one should consult "Debris-Control Structures", Hydraulic Engineering Circular No. 9, Federal Highway Administration. However, debris is difficult to control locally and a long-range management program to clear the stream may be emphasized instead.

## 4.7 STRUCTURAL DESIGN

### 4.7.1 Load Criteria

Once the hydraulic design is completed, the next step is structural design. The following loads should form the basis for structural design (1, 2):

#### 4.7.1.1 Dead Load ($W_D$)

Dead load is the constant load caused by the weight of the structure itself and some additional elements supported by the structure, such as earth loading, pavement, sidewalk, cables and other public utility services or hydrostatic pressure.

#### 4.7.1.2 Live Loads ($W_L$)

Live loads consist of loads applied intermittently to the structure. They may include people, vehicles or a combination of these loads. Live load criteria for structure and roadways should be in accordance with AASHTO Specifications for Bridges. For the design of LWSCs on a low-volume road, the H loadings should be used. The wheel load for H-20-44 loading is 16,000 lbs (7,264 kg).

For a vented ford where a pipe is embedded in the road subgrade, the live load on the pipe will be reduced, depending on the height of the fill. Highway live loads in relation to fill height for various types and sizes are readily available from design manuals for pipes (1, 2).

#### 4.7.1.3 Impact Factors ($I_f$) for Pipe

In designing pipe with less than 3 ft (0.91 m) of cover fill, the live load should be multiplied by the following factors to take into account the impact due to moving load.

| Height of Cover H, ft (1 ft = 0.305 m) | Impact Factor, $I_f$ |
|---|---|
| 0 - 1 | 1.3 |
| 1 - 2 | 1.2 |
| 2 - 3 | 1.1 |

4.7.1.4 Impact Fraction for Slab

In AASHTO Specifications, the impact fraction is expressed as a fraction of live load and can be determined by the formula,

$$I = \frac{50}{L + 125} \tag{16}$$

where

I = impact fraction (maximum 30 percent)

L = length of portion of span which is loaded to produce the maximum stress in the member (ft)

## 4.7.2 Vented Fords

Various types of pipes are used for vented fords. Circular concrete pipe is the most common and will be presented here to demonstrate design procedure. For the structural design of other types of pipes, the designer is advised to use appropriate manuals published by the manufacturers (1, 2).

4.7.2.1 Load Factor ($L_f$) for Concrete Pipes

Vented fords are comparable to buried pipes. The vertical load on a pipe is distributed over its width and the reaction is distributed in accordance with the bedding on which the pipe is placed. When the pipe strength in design has been determined by plant testing, load factors must be developed to relate the in-place strength to the plant test strength. The ratio of the in-place strength to the plant test strength is called load factor.

For a vented ford for which the pipe is installed in shallow bedding, with its top projecting above the surface of the adjacent natural ground and then covered with an embankment, the $L_f$ values of 2 to 3 are recommended.

#### 4.7.2.2 Factor of Safety (F.S.)

For reinforced concrete pipe, a factor of safety of 1.0 should be applied. For non-reinforced concrete pipe, a factor of safety of 1.25 to 1.5 is normally used.

#### 4.7.2.3 Selection of Pipe Strength--Concrete Pipes

After the size of the pipe is determined, the type of the pipe which meets the strength requirement should be selected. The pipe strengths are expressed by Three-Edge Bearing test (T.E.B.) and are often classified by D-loads. The D-load is the three-edge bearing test load in pounds per linear foot per foot of nominal inside diameter of the pipe in feet.

The required T.E.B. strength of circular non-reinforced concrete pipe of diameter less than 24 in (0.6 m) is expressed in pounds per linear foot, not as a D-load, and is computed by the equation:

$$\text{T.E.B.} = \left(\frac{W_L}{1.5} + \frac{W_D}{L_f}\right) \text{F.S.} \tag{17}$$

The required T.E.B. strength of circular reinforced concrete pipe is normally expressed as a D-load and is computed by the equation:

$$\text{D-load} = \left(\frac{W_L}{1.5} + \frac{W_D}{L_f}\right) \frac{\text{F.S.}}{D} \tag{18}$$

where D = Diameter of pipe (ft)

A pipe which would withstand the required three-edge bearing test load, T.E.B. or D-load computed, should be selected.

4.7.2.4 Example

Given: A 4-ft circular non-reinforced concrete culvert is to be installed for a vented ford. The culvert is to be laid directly on the original streambed and covered with 2 ft of 120 lb per cu ft backfill, and 8 in of 150 lb per cu ft pavement. Consider H-20 for live load. The wall thickness of the culvert is 5 in.

Find: The required pipe strength.

Solution:

1. Determination of dead load ($W_D$):

For a pipe installed with less than 3 ft of cover, the dead load is approximately the weight of the materials on top of the pipe.

$$W_D = [120 \times 2 + 150 \times (8/12)] \times 4.85 = 1649 \text{ lb/ft}$$

2. Determination of live load ($W_L$):

For D = 4 ft, H = 3 ft and H-20 loading, a live load of 1221 lb per lin ft is obtained (from appropriate tables in concrete pipe manuals). The impact load is included.

3. Selection of load factor ($L_f$) and factor of safety:

Values of 2.5 for the load factor and 1.4 for the factor of safety are selected.

4. Computation of pipe strength

$$T.E.B. = \left(\frac{1221}{1.5} + \frac{1649}{2.5}\right)(1.3) = 1916 \text{ lb/ft of inside diameter}$$

Answer: A pipe which would withstand a minimum three-edge bearing test load of 1916 lb. per lin. ft per ft of inside diameter would be required.

### 4.7.3 Slab Bridges

#### 4.7.3.1 Simply Supported Slab

The design of simply supported slabs is more or less standardized for specific uses. For R.C. slab bridges, the bottom and top reinforcement bars for a simply supported slab for spans equal to 10 to 30 ft are recommended in Table 5.

The positions of the bottom bars called $S_1$ and $S_2$ and the top bars $S_3$ and $S_4$ are shown in Fig. 17. See details A of Fig. 17 for slab thickness of 16 in and less, and detail B for more than 16 in of thickness. Typical slab and curb dimensions and the reinforcing steel are shown in Fig. 18 and Table 6.

TABLE 5. REINFORCEMENT BARS FOR SIMPLE SPAN SLAB BRIDGE

| BRIDGE SPAN (FEET) | SLAB THICKNESS t (INCH) | SIZE AND SPACING (INCH) BOTTOM BARS | | SIZE AND SPACING (INCH) TOP BARS | |
|---|---|---|---|---|---|
| | | $S_1$ | $S_2$ | $S_3$ | $S_4$ |
| 10.0 | 11.0 | # 8 @ 8.5 | # 6 @ 12 | # 5 @ 17.0 | # 4 @ 12 |
| 12.0 | 12.0 | # 8 @ 8.0 | " | # 5 @ 16.0 | " |
| 14.0 | 12.5 | # 8 @ 7.0 | " | # 5 @ 12.0 | " |
| 16.0 | 13.5 | # 9 @ 8.0 | " | # 5 @ 16.0 | " |
| 18.0 | 14.5 | # 9 @ 7.5 | " | # 5 @ 15.0 | " |
| 20.0 | 15.0 | # 9 @ 7.0 | # 6 @ 12 | # 5 @ 14.0 | # 4 @ 12 |
| 22.0 | 16.0 | #10 @ 8.5 | # 7 @ 15 | # 5 @ 17.0 | # 4 @ 15 |
| 24.0 | 17.0 | #10 @ 8.0 | " | # 5 @ 16.0 | " |
| 26.0 | 18.0 | #10 @ 7.0 | " | # 5 @ 14.0 | " |
| 28.0 | 19.5 | #11 @ 8.5 | " | # 5 @ 17.0 | " |
| 30.0 | 21.0 | #11 @ 7.5 | # 7 @ 15 | # 5 @ 15.0 | # 4 @ 15 |

Note: 1 ft = 0.305 m

Source: Pennsylvania Department of Transportation - "Design Manual for Structures" Part 4,1966

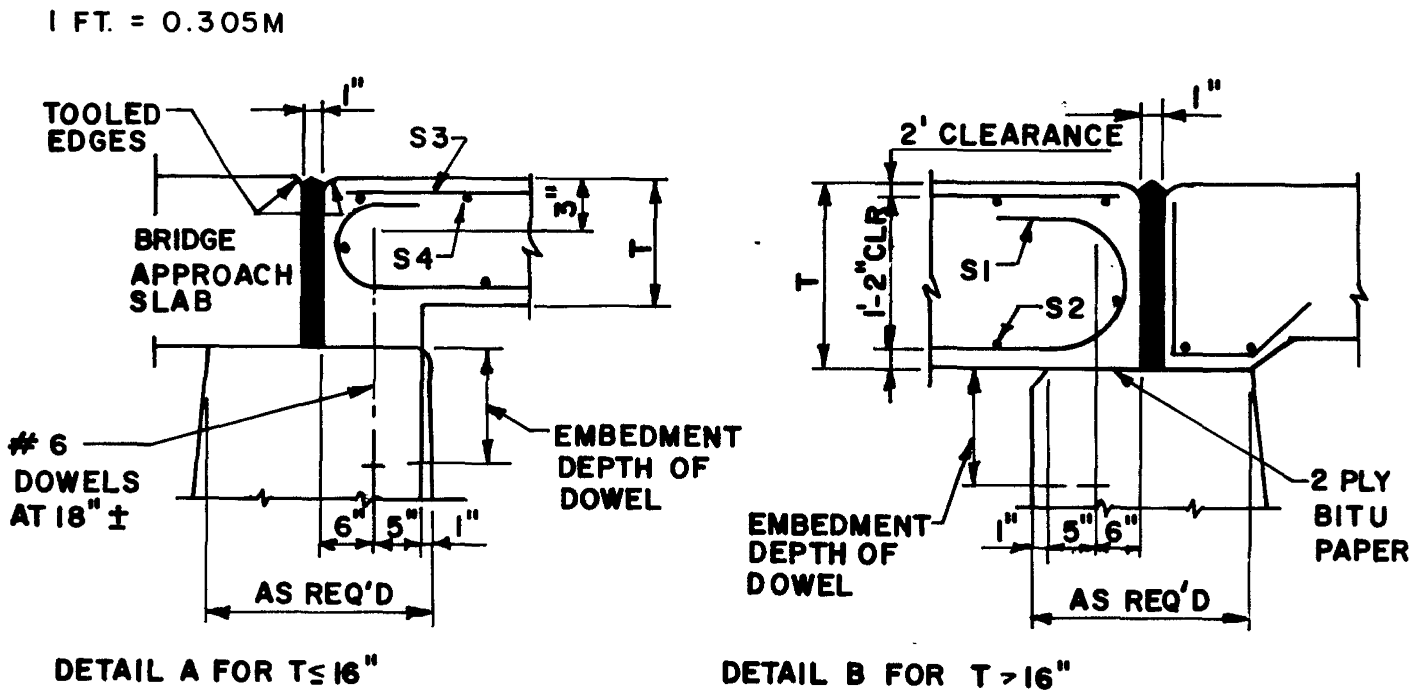
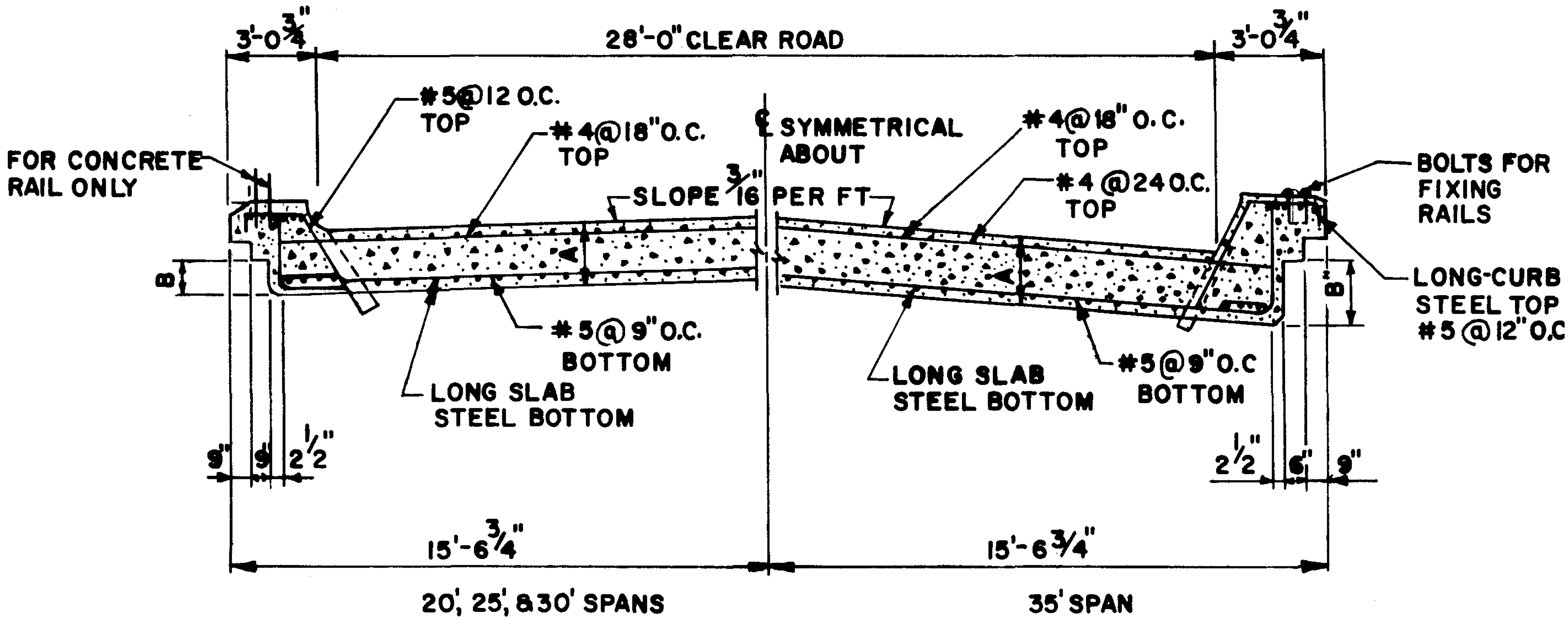



FIG. 17 SLAB BRIDGE: TYPICAL SECTIONS AT ABUTMENTS

1 FT. = 0.305 M




| TABLE 6 - SLAB & CURB DIMENSIONS & REINFORCING STEEL | | | | | |
|---|---|---|---|---|---|
| SPAN (FT) | H20-S16-44 LOADING | | | | |
| | DIMENSION A" | DIMENSION B" | LONG SLAB STEEL-BOT. | LONG CURB STEEL-TOP | LONG CURB STEEL-BOT. |
| 20 | 10 1/2" | 11 1/2" | #7@ 6 1/2" O.C. | #4@ 8" O.C. | #7@ 3" O.C. |
| 25 | 12 1/2" | 13 1/2" | #8@ 7" O.C. | #8@ 5" O.C. | #8@ 4" O.C. |
| 30 | 14 1/2" | 15 1/2" | #9@ 7" O.C. | #9@ 5" O.C. | #9@ 3 1/2" O.C. |
| 35 | 18 1/2" | 21 1/2" | #9@ 6" O.C. | #9@ 3 1/2" O.C | #9@ 3 3/4" O.C. |

FIG. 18 SLAB BRIDGE: TYPICAL SECTIONS

4.7.3.2 Continuous Slab

A standardized continuous slab design can be found in many reinforced concrete structure design handbooks (5). The designer is advised to use these sources for such a design. The reinforcements for a simple supported slab as presented in the last section should not be used for continuous slab.

## 4.8 WARNING SIGNS AND MARKINGS

The signs shown in Fig. 19 or similar signs should be used on each approach to an LWSC (19). The distances of the signs from the crossing are also given in Fig. 19. If desired, during the first year, red flags may be used for emphasis. These signs have to be approved later by the Federal Highway Administration.

If the location of the LWSC is not apparent, an additional advisory distance plate can be used 1000 ft from the crossing. This sign should be 250 ft before the "FLOOD AREA AHEAD" sign. It should have a black legend on a yellow background and would be 24 in X 18 in.

If the maximum recommended speed at an LWSC is less than the speed limit otherwise in effect, an additional advisory speed plate may be used.

Depth gage and roadway markers on two edges of LWSC are encouraged where debris poses no problem. They should be flexible so that they will bend without damage if debris lodges upon them.

1 FT. = 0.305 M

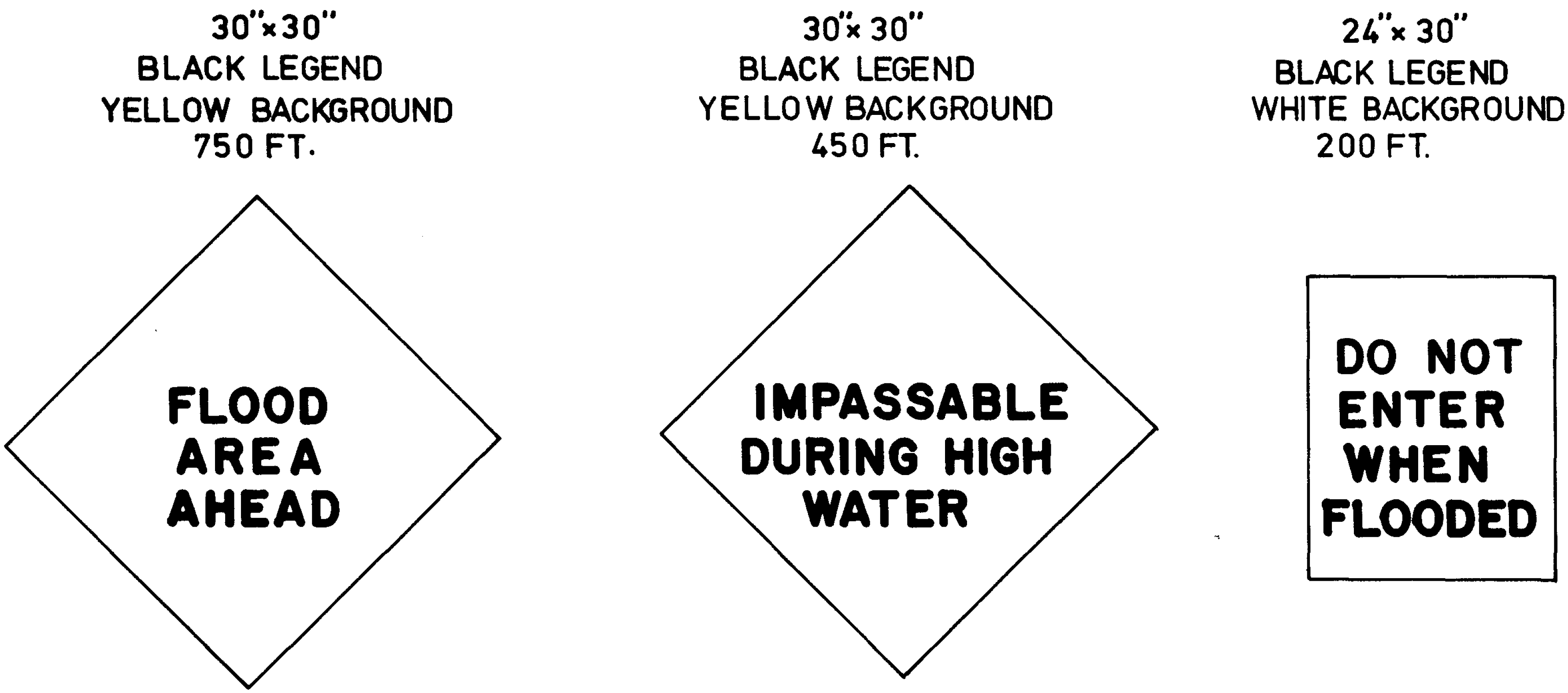



FIG. 19 SUGGESTED SIGNS AT DIFFERENT DISTANCES FROM LWSC.

# 5.0 DESIGN EXAMPLES

## 5.1 EXAMPLE 1: FORD

A 15-ft wide county road is to cross a dry creek and a ford is considered. The following data are collected:

ADT = 10
Channel slope, S = 0.003
Mannings coefficient, n = 0.02 (ordinary soil)
Flood mark is 4 ft above creekbed

The design of a ford does not require much engineering analysis. It is mostly based on experience and engineering intuition, and utilizes the general design considerations and criteria discussed earlier.

### 5.1.1 Roadway Pavement

A 15-ft wide, 8-in thick concrete slab will be placed over a 10-in thick gravel layer. The upper surface of the roadway is level with the original creekbed. If funds are sufficient, the concrete slab may be reinforced and the gravel layer replaced with a filter fabric to prevent subgrade soil from being siphoned out through the slab when cracks are developed later.

### 5.1.2 Cutoff Walls

An 8-in thick cutoff wall is placed on each side of the pavement to reduce water seepage through subgrade. It also supports the pavement when the creekbed is eroded. The upstream cutoff wall extends 3 ft below the roadway surface. An extra 1 ft is added to the downstream cutoff wall. Where the flow velocity is high or the creekbed is unstable, a deeper cutoff wall should be used. The roadway slab and the upstream cutoff wall should be bonded and made water tight to reduce uplift pressure of seeping water. The joint of the downstream cutoff wall should be left open so that water can be drained and the pressure reduced.

### 5.1.3 Erosion Protection

The creekbed on the downstream side of the ford should be protected against erosion. Stone riprap is the most economical. Riprap size can be determined according to flow velocity, obtained by using Manning's Formula. In this example, when flow depth y = 4 ft, flow velocity will be:

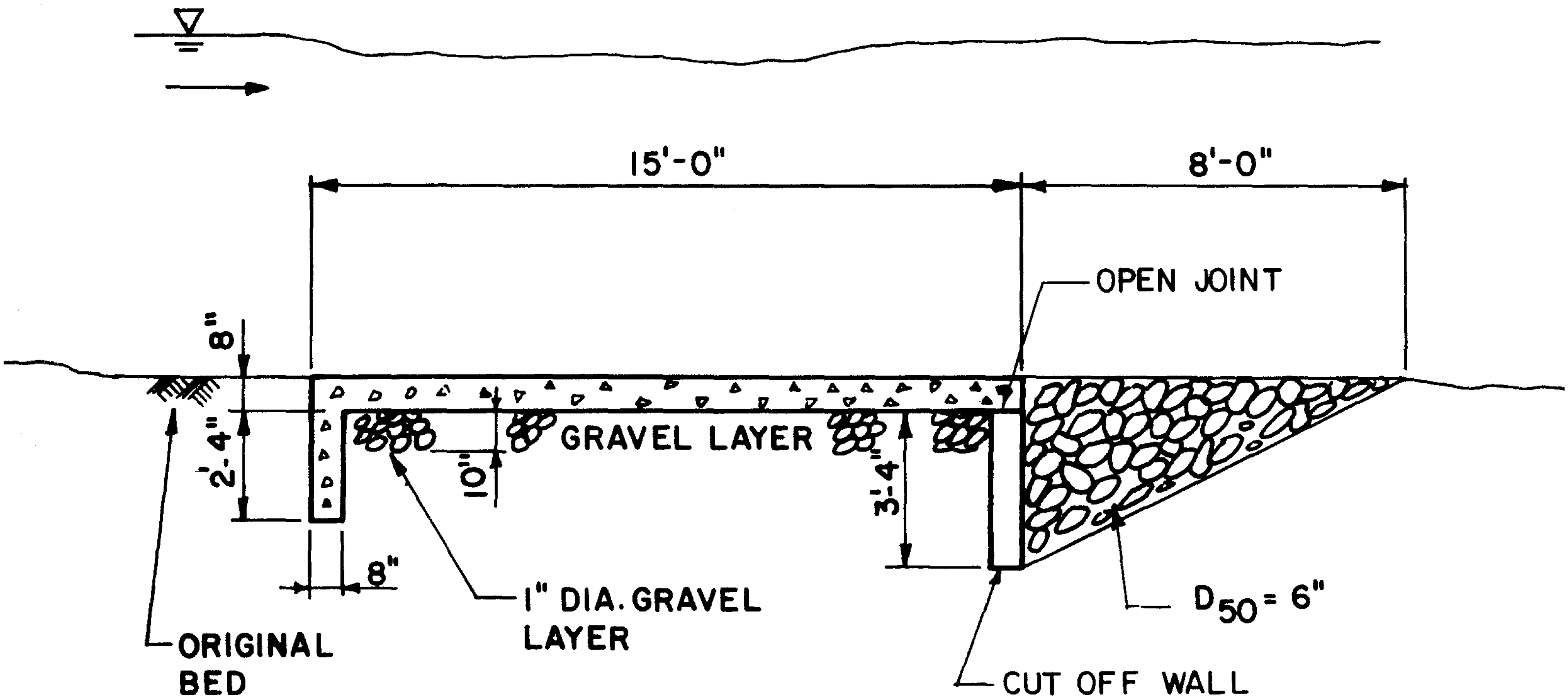
$$V = \frac{1.49}{n} y^{2/3} \quad S^{\frac{1}{2}} = \frac{1.49}{0.02} (4)^{2/3} \quad (0.0002)^{\frac{1}{2}} = 8.4 \text{ ft/sec}$$

For V = 8.4 ft/sec, the size of stone required can be determined from Fig. 15 as $D_{50}$ = 0.45 ft or 5.4 in. Graded stone, 50% of which is larger than 6 in diameter will be used to fill the downstream side, as shown in Fig. 20. The depth of riprap at the edge of the ford is 4 ft. (same as cutoff wall depth) and is gradually reduced downstream to zero, at twice the cutoff wall depth, 8 ft. Fig. 20 shows the details of the design.

For higher velocity, gabions may be used for erosion protection. The cost will be higher but life of the structure will be increased.

## 5.2 EXAMPLE 2: VENTED FORD

A 22 ft wide secondary road extends across a flood plain. Because of low daily traffic volume of 80, a low-water stream crossing is considered. Since it is a wide crossing and the construction cost is probably high, a risk analysis is required.




1 FT. = 0.305 M

FIG. 20 CROSS SECTION OF FORD

(EXAMPLE PROBLEM 1)

### 5.2.1 Data

#### 5.2.1.1 Unit Costs of Structure Components

The following unit costs are assumed for solving the example:

| | | |
|---|---|---|
| Roadway pavement | | \$ 17.20/sy |
| Embankment | | \$ 2.40/cy |
| Dumped riprap | | \$ 45.00/cy |
| Culvert: | 3 ft. diameter | \$ 32.00/ft |
| | 4 ft. diameter | \$ 48.00/ft |
| | 4.5 ft. diameter | \$ 56.00/ft |

#### 5.2.1.2 Stream Characteristics

Drainage area = 25 sq mi

It is a perennial stream, the main channel is meandering and the floodplain contains dense brush and vegetation. The Manning's roughness coefficient for the main channel is 0.045 and for overbanks 0.06. The profile of the crossing is shown in Fig. 21. The channel slope at the site is 0.0007, and the length of the stream is eight miles. The elevation difference between the farthest upstream point in the basin and the site is 32 ft. No hydrologic data are available except an estimate of the 10-year flood of 1320 cfs (from the highway drainage manual for the region).

#### 5.2.1.3 Land Use

Land use at the crossing is pasture land, and therefore the risk cost for the land may be disregarded. A farm house with a basement, worth $60,000, stands nearby with the first floor elevation at 86 ft above Mean Sea Level.

5.2.1.4 Traffic and Detour Data

| | | |
|---|---|---|
| ADT | = | 80 |
| Speed | = | 40 mph |
| Average occupancy in vehicle | = | 1.5 |
| Length of normal route | = | 4 mi |
| Length of detour | = | 12 mi |
| Unit cost of travel | = | $2.50/1000 vehicle miles |
| Value of lost time | = | $3.60/hr per occupant |

Increased accident cost due to detour may be assumed to be negligible.

5.2.1.5 Flood Loss Data

Embankment and pavement losses in terms of overflow depth and duration are adapted from HEC-17 and replotted in Fig. 22.

5.2.2 Design

The design will be presented in three parts:

(1) Hydraulic and Flood Frequency Analysis
(2) Selection of Designs
(3) Cost Analysis

5.2.2.1 Hydraulic and Flood Frequency Analysis

If neither hydraulic nor hydrologic data is available, the stage discharge relationship must be obtained analytically by using Manning's equation. For the flood frequency analysis, regression equations or charts available at the local highway agency may be used. For this example, the chart ($Q/Q_{10}$ vs recurrence interval) shown in Fig. 22 will be used.

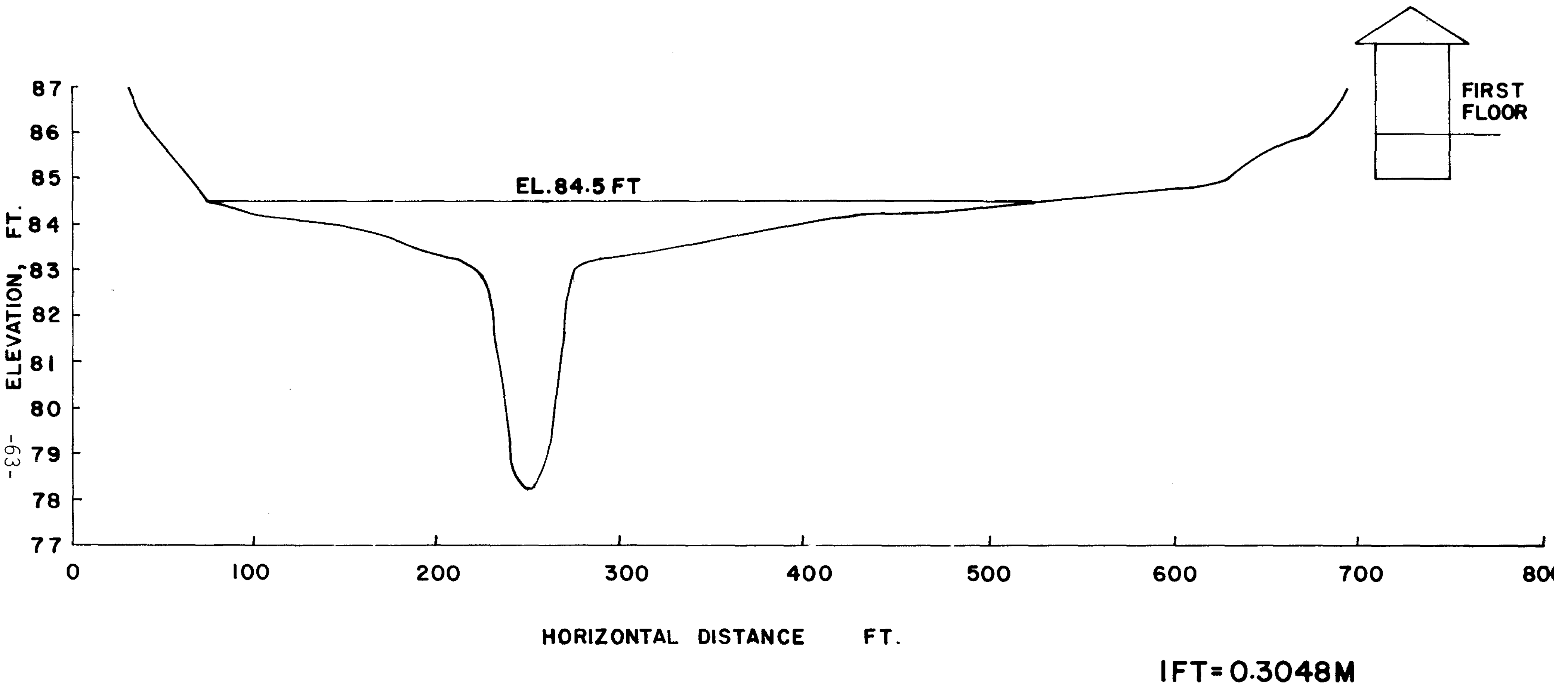
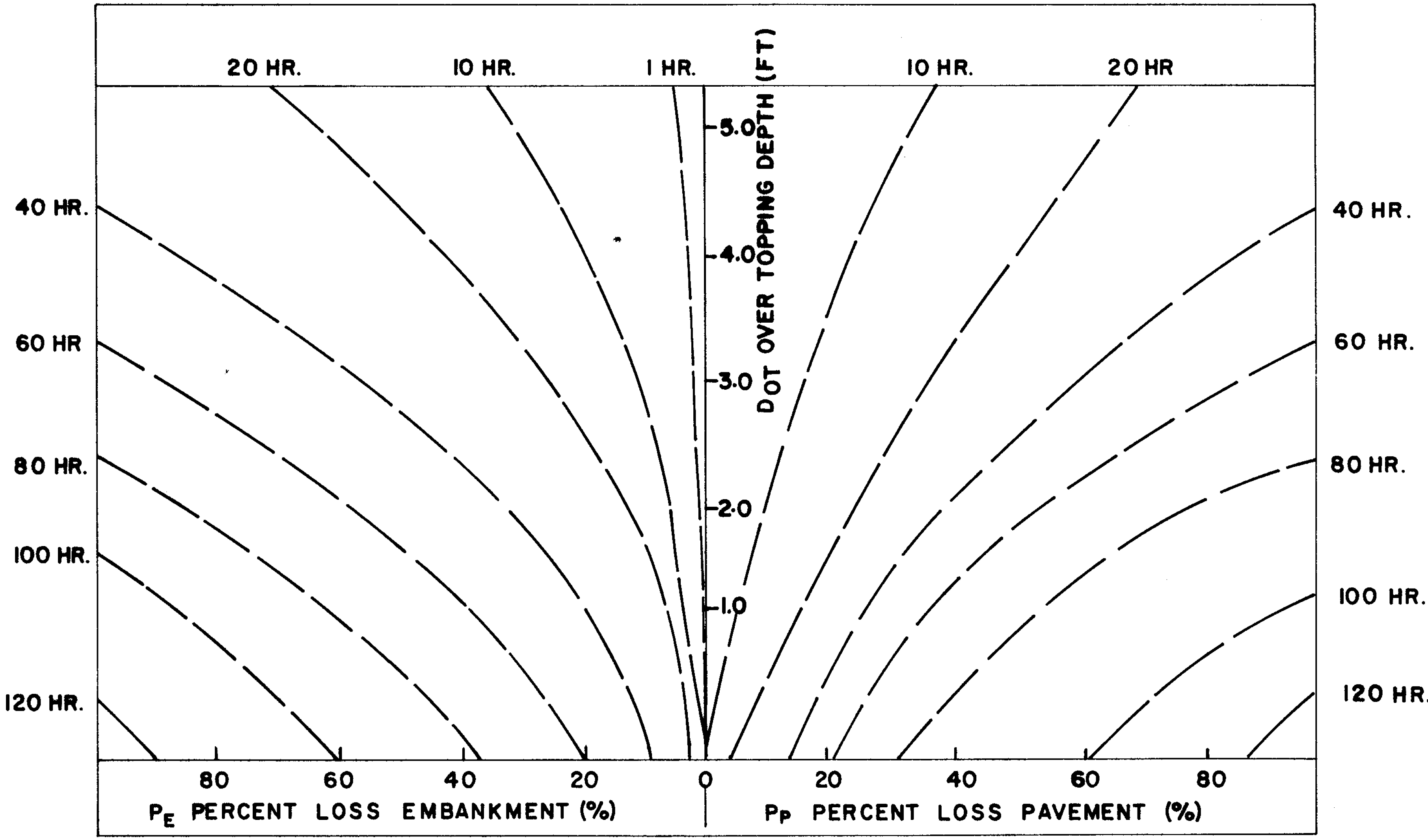



FIG. 21 CROSS SECTION OF CROSSING FOR EXAMPLE PROBLEM 2




1FT=0.3048M

FIG. 22 EMBANKMENT – PAVEMENT LOSSES

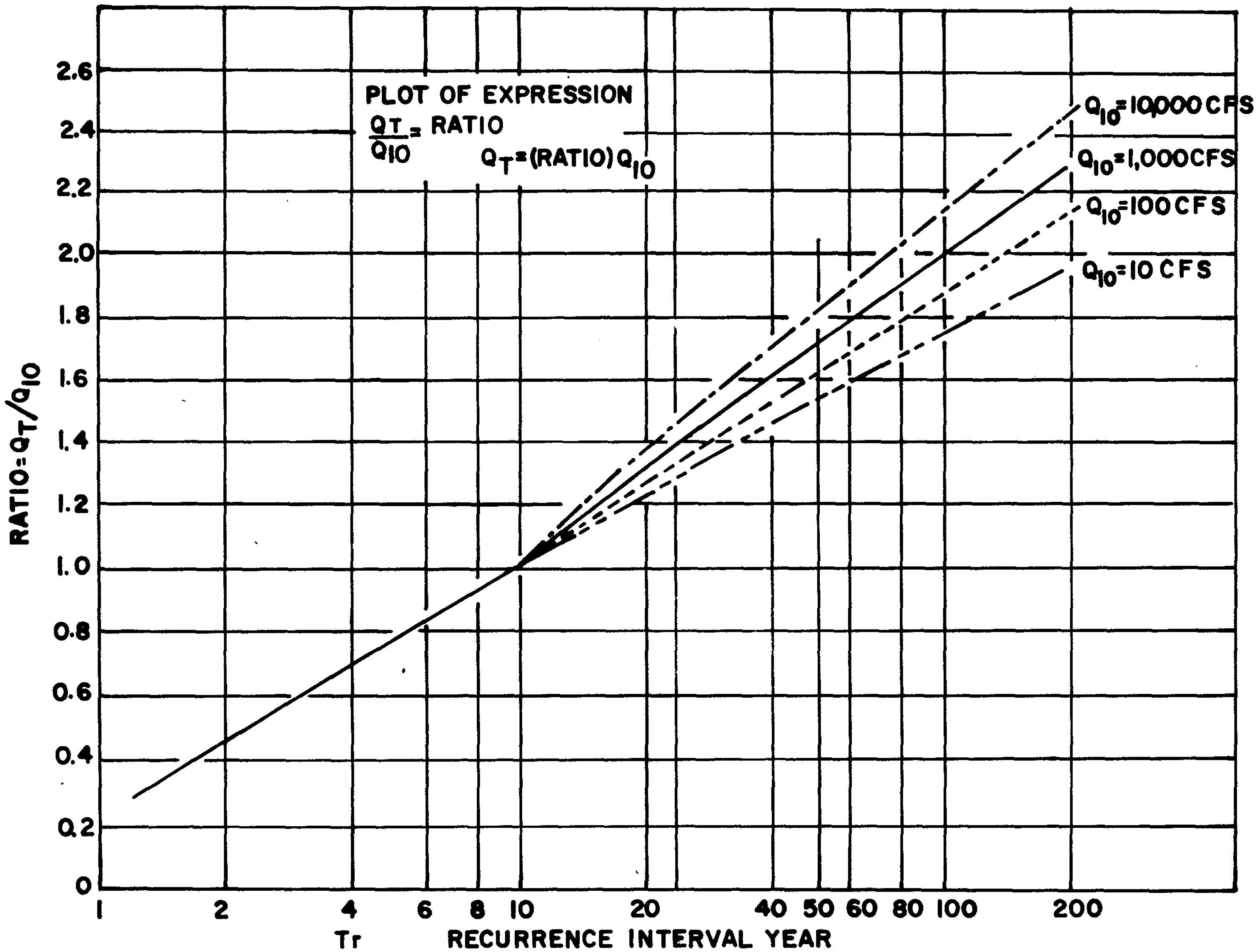



FIG. 23 PLOT OF $Q_T/Q_{10}$ VS. FREQUENY YEARS

A. Stage-Discharge Relationship

In using Manning's equation to develop stage-discharge relationships, a proper selection of Manning's roughness coefficient becomes important. Manning's coefficients for natural stream channels are generally given in various publications and hydraulics textbooks. It is recommended, however, that the designer consult the local district office of the U.S. Geological Survey to obtain appropriate values applicable to streams of any specific region. For a cross-section of irregular shape, the cross-section should be subdivided into several sections and Manning's equation applied to each subsection.

Based on the Manning's equation below, the discharge can be estimated.

$$Q = \frac{1.49}{n} R^{2/3} S^{\frac{1}{2}} A \tag{19}$$

where
- $Q$ = discharge (cfs)
- $n$ = Manning's roughness coefficient
- $R$ = hydraulic radius $R=A/P$, (ft)
- $S$ = energy gradient, or channel slope for a uniform flow
- $P$ = wetted perimeter (ft)
- $A$ = cross-sectional area ($ft^2$)

The stage-discharge relationship is developed in the following steps and the results are summarized in Table 7:

1. Since the stream is in a flood plain, the flow is subdivided into two portions, main channel and floodplain portions.

2. From Fig. 21, obtain wetted perimeters and cross-sectional areas for both the main channel and overbank section for various stages.

These are shown in Columns 2, 3, 5, and 6, respectively in Table 7.

3. Using Manning's equation, compute discharges separately for the channel section and over the floodplain (Columns 4 and 7, respectively).

4. The sum of Columns 4 and 7 is the total discharge for the specified stage. The resulting stage-discharge relationship is presented in Fig. 24. Plotted also in Fig. 24 are the stage-area and the stage-width relationships for later use in estimating mean velocity of flow and average width of overtopping flow.

TABLE 7. STAGE-DISCHARGE RELATIONSHIP

| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) |
|---|---|---|---|---|---|---|---|
| Stage (ft) | Main Channel (n=0.045) | | | Overbank (n=0.06) | | | Total Discharge |
| | P | A | Q | P | A | Q | |
| 83.00 | 51 | 165 | 3163 | --- | 0 | --- | 316 |
| 84.00 | 51 | 215 | 492 | 200 | 100 | 41 | 533 |
| 85.00 | 51 | 260 | 675 | 500 | 580 | 421 | 1096 |
| 86.00 | 51 | 305 | 881 | 580 | 1120 | 1141 | 2022 |
| 87.00 | 51 | 360 | 1161 | 615 | 1780 | 2376 | 3537 |

B. Flood Frequency Analysis

In this example, the magnitude of a 10-year flood, $Q_{10}$, is

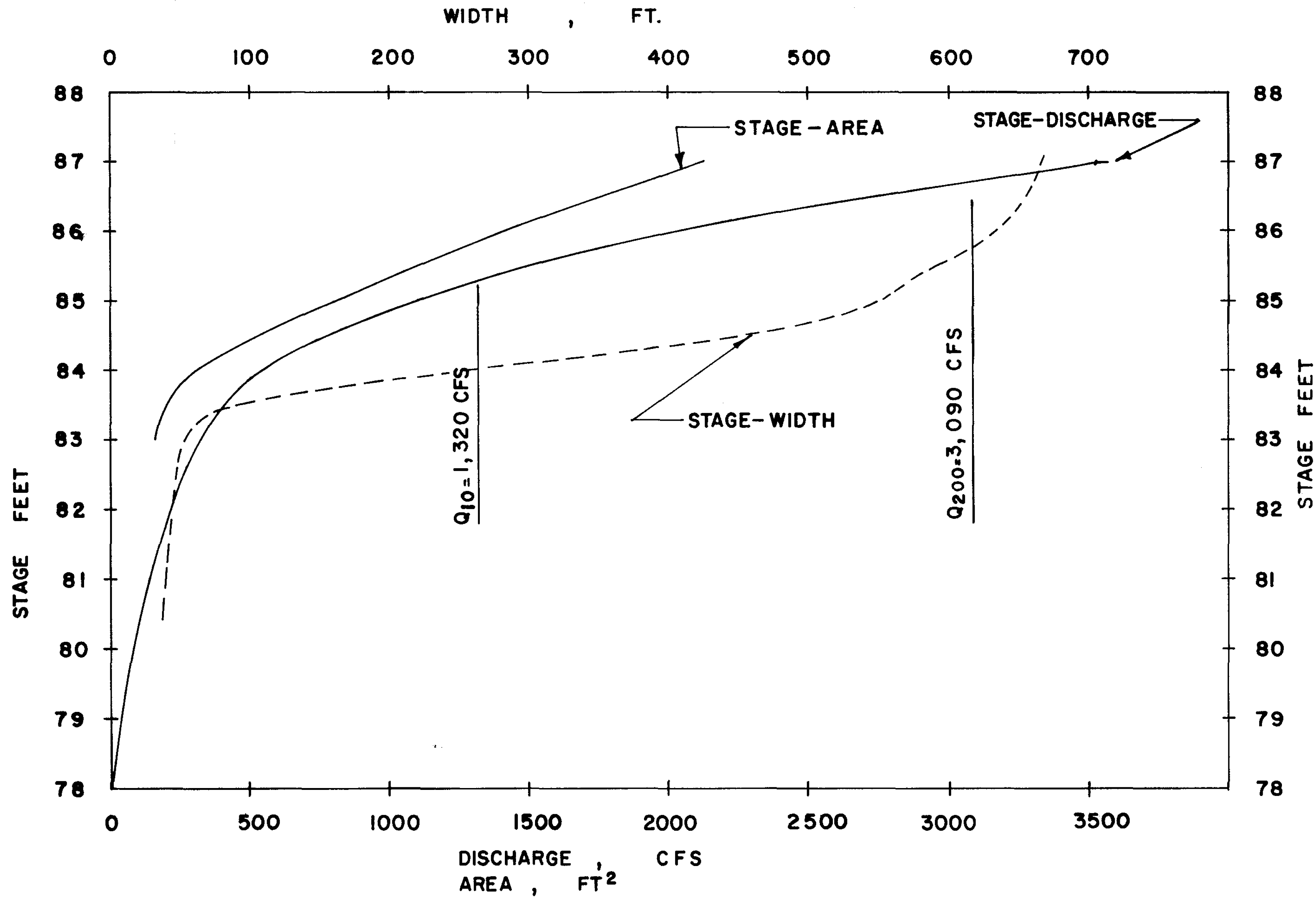



FIG. 24 STAGE-DISCHARGE RELATION AND CHARACTERISTICS OF STREAM CROSS SECTION

given as 1,320 cfs. (Normally, the flood of either a 10-year or 100-year recurrence interval can be obtained from the highway agency's drainage manual). Using this value, floods of other frequencies are computed by using Fig. 23 as shown in Table 8.

TABLE 8. FLOODS OF VARIOUS RECURRENCE INTERVALS

| Recurrence Interval (Years) | $Q/Q_{10}$ | Discharge, Q cfs |
|---|---|---|
| 1 | 0.24 | 317 |
| 2 | 0.46 | 607 |
| 4 | 0.70 | 924 |
| 10 | 1.00 | 1,320 |
| 50 | 1.74 | 2,300 |
| 200 | 2.34 | 3,090 |

5.2.2.2 Selection of Designs

The roadway elevation is first selected at 84.5 ft, which is about the stage of a 2-year flood and about 1 ft higher than the main channel banks. Assuming that the streambed is 6 ft below this roadway elevation, and assuming about 2 ft of soil fill over the drainage structure, 4-ft circular culverts are selected so that no excavation is needed to lay the culverts on the streambed. For the 50-ft wide main channel, the number of culverts could be up to eight. Four barrels are chosen as the initial design and the detailed hydraulic analysis and total cost estimate for this design alternative are presented.

The dimension of embankment at the main channel is shown in Fig. 25. For its slope protection 10-in diameter rock riprap is considered, assuming that the embankment soil satisfies the filter requirement. Rock will be dumped in place with an overall thickness of 1 ft.

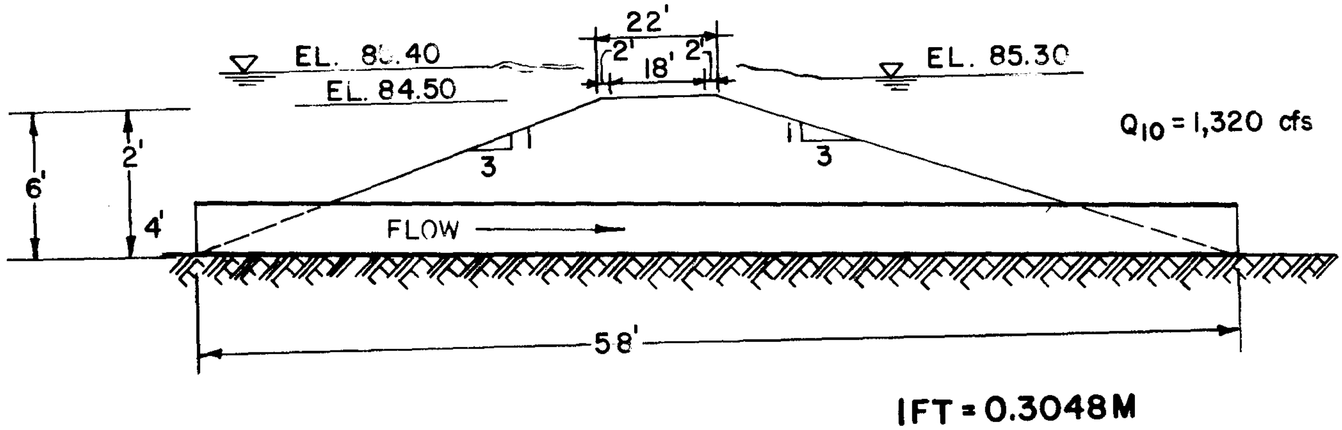



FIG. 25 DIMENSIONS OF CULVERT

5.2.2.3 Cost Analysis

A. Capital Costs

Annual capital costs can be determined by multiplying the capital recovery factor by total capital costs.

For a 25-year old service life and 8% interest rate, the capital recovery factor (CRF) becomes:

$$CRF = \frac{i}{1 - (1-i)^{-N}} = \frac{.08}{1 - (1-.08)^{-25}} = 0.0937$$

The capital costs, including the costs of roadway pavement, embankment, ripraps, and culverts, are summarized in the following table:

TABLE 9. SUMMARY OF CAPITAL COSTS

| Components | Unit Cost | Quantity | Cost | Remark |
|---|---|---|---|---|
| Pavement | $17.20/sy | 1,260 sy | $21,670 | Length=630 ft<br>Width = 18 ft |
| Embankment | $ 2.40/cy | 815 cy | $ 1,960 | 50 ft section, A=240 ft.<br>400 ft section, A=25 ft (avg) |
| Riprap | $45.00/cy | 164 cy | $ 7,380 | Main channel sec = 70.5 cy<br>Flood plain sec = 93.4 cy |
| Culvert | $48.00/ft | 232 ft | $11,140 | For 4 barrels (58 ft. length) |
| TOTAL | | | $42,150 | |

The annual capital cost will be 0.0937 x 42,150 = $3,949 per year.

B. Flood Damage

Since the extent of property loss and traffic delay loss are closely related to flood stage, backwater at the crossing must be determined in order to estimate properly the flood-related losses. Here our particular concerns are backwater elevations for floods of recurrence intervals of 1, 2, 4, 10, 50 and 200 years. Flow through a vented ford consists of culvert flow and road overflow at higher stages. Determination of the backwater for such a complex flow can be made by trial-and-error. For low-stage flow when no overtopping occurs, flow can be treated as a culvert flow and the computation becomes simple. Following sample calculations will explain such trial and error procedures.

Sample Computation of Backwater Elevation and its Damage Costs

| | | |
|---|---|---|
| Given: | 10-year flood | $Q_{10}$ = 1320 cfs |
| | Tailwater elevation | 85.3 ft. (from Fig. 24) |
| | Roadway elevation | 84.5 ft. |

Dimensions of embankment and culverts are as shown in Fig. 25.

Required: (a) Backwater elevation
(b) Property damage costs due to backwater
(c) Traffic delay costs
(d) Total damage costs

Solution:

(a) Backwater elevations:

1. Assume a total head H=0.9 ft above roadway. Backwater elevation becomes 84.5 + 0.9 = 85.4 ft. From Fig. 24, the channel width at 84.5 ft elevation is 450 ft and that at 85.4 ft is 580 ft. Hence, average width of overflow will be (450 + 580)/2 = 515 ft.

2. Compute the overflow rate by using equation 12,

$$Q = C_f L H^{3/2} \, (C_s/C_f)$$

From Figure 5: for H = 0.9 ft., $C_f$ = 3.03, and

for $Y_2/H = 0.8/0.9 = 0.89$, $C_s/C_f = 0.93$

The overflow becomes:

$$Q_1 = 3.03 \, (515) \, (0.9)^{3/2} \, (0.93) = 1{,}239 \text{ cfs}$$

3. Compute the flow through the culverts. The length of the culvert is 58 ft. The head difference is H = 85.4 - 85.3 = 0.1 ft. Using Nomographs in HEC No. 5 ($n = 0.012$, and $k_e = 0.5$), the discharge through one culvert becomes 24 cfs.

   For four culverts, the total discharge becomes:
   $Q_2 = 4 \times 24 = 96$ cfs

4. The total discharge will then be,

   $Q = Q_1 + Q_2 = 1239 + 96 = 1335$ cfs, which is close to 1320 cfs

   The assumed backwater elevation of 85.4 ft (see Fig. 20) is satisfactory.

5. If not, another value of H should be assumed and the procedure repeated.

The total heads for other discharges are computed and summarized in Table 10.

(b) Property damage costs due to backwater:

The damage costs due to backwater may be estimated by mutliplying the percent of damage by costs of structure and property. The percent of damage to roadway pavement and embankment due to overtopping flow is shown in Fig. 22. The damage is expressed in terms of backwater depth and overtopping time, $t_{ot}$.

For $Q_{10} = 1320$ cfs, backwater depth, H, was previously found to be 0.9 ft. The value of overtopping time can be obtained first by finding the discharge $Q_{ot}$ when the overtopping flow first occurs. It is

TABLE 10. HYDRAULIC COMPUTATION FOR EXAMPLE PROBLEM 2

No. of Barrels 4
Length 58

| Recurrence Interval Yrs. | Back-water depth H, ft | Head-water elev. ft | Tail-water elev. ft | Overflow | | | Culvert Flow n=0.012, K=0.5 | | | Total Discharge cfs | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Ave. Width ft | D/h | $Q_1$ cfs | Total Head H, ft | Discharge in 1 culvert cfs | $Q_2$ cfs | | |
| 1.2 | 0.0 | 84.5 | 83.2 | 0 | 0 | 0 | 1.3 | 86 | 344 | 344 | $Q_{ot}$ = discharge larger than this value will overtop roadway |
| 2 | 0.38 | 84.88 | 84.2 | 490 | 0 | 348 | 0.68 | 65 | 260 | 608 | $Q_2$ = 607cfs |
| 4 | 0.62 | 85.12 | 84.7 | 505 | 0.32 | 742 | 0.42 | 50 | 200 | 942 | $Q_4$ = 924 cfs |
| 10 | 0.90 | 85.40 | 85.3 | 515 | 0.89 | 1239 | 0.10 | 24 | 92 | 1335 | $Q_{10}$ = 1320 cfs |
| 50 | 1.74 | 86.24 | 86.2 | 545 | 0.98 | 2235 | 0.04 | 16 | 64 | 2299 | $Q_{50}$ = 2300 cfs |
| 200 | 2.24 | 86.74 | 86.7 | 555 | 0.982 | 2997 | 0.04 | 16 | 72 | 3069 | $Q_{200}$ = 3090 |

the discharge when headwater is at the road elevation of 84.5 ft and all water flows through the culverts. To determine its value, again a trial-and- error method may be used. First, the tailwater elevation is assumed to be 83.2 ft and the total head difference then becomes 1.3 ft. The discharge through the four culverts becomes 4 x 86 = 344 cfs. This matches with the discharge of 350 cfs read from Fig. 24 (the stage-discharge curve) when the tailwater stage is 83.2 ft.

The overtopping time $t_{ot}$ for discharges larger than the overtopping discharge can be obtained from:

$$t_{ot} = \frac{Q - Q_{ot}}{Q} \cdot t_b \tag{20}$$

The base time, $t_b$, for this drainage basin is:

$$\begin{aligned} t_b &= 6.92\ L^{1.6}/(DH)^{0.385} \\ &= 6.92\ (8)^{1.16}/(32)^{0.385} = 20.3 \text{ hrs} \end{aligned} \tag{21}$$

where

L = length of longest water course, mi
DH = elevation difference between farthest upstream to the site, ft

The overtopping time for 10-year flood $Q_{10}$ = 1,320 becomes:

$$t_{ot} = \frac{1320 - 350}{1320}\ (20.3) = 14.9 \text{ hrs}$$

For an overtopping depth of 0.9 ft and overtopping time of 14.9 hr (from Fig. 22) damages to the pavement and embankment are expected to be about four percent and two percent respectively, if no riprap is placed.

With 10-in diameter riprap*, no damage to the embankment and culverts is anticipated. Therefore, the total property damage cost will be:

pavement loss = 21,670 x 0.04 = $867

The first level of the farmhouse is at 86 ft elevation, which is above the backwater elevation of 85.4 ft. Therefore, no damage to the house is expected. If the backwater is above the first level of the house, the percent damage may be estimated by Equation 10. We must bear in mind that damage cost to be considered here should be only what is incurred from construction of the crossing--the damage due to the backwater minus damage cost under natural conditions.

(c) Traffic Delay Costs

Restoration time, $T_r$, is the sum of overtopping time and traffic restoration time. For this example, traffic restoration time is assumed to be two days for the cases in when the pavement damage is more than four percent. If pavement damage is three percent or less, it is assumed that traffic may resume without repairment.

For $Q_{10}$ = 1320 cfs, the flood overtops the roadway for a period of 14.9 hours (0.62 day) and the damage to the pavement is expected to be more than about four percent. Therefore, traffic restoration time becomes $T_r$ = 2.62 days.

*Check riprap safety by the method presented in the text, Section 4.4.2.

The increased running cost can be obtained by using Equation 6:

$$L_{rc} = (2.62)(80)(8)(250)/1,000 = \$419$$

The cost for lost time of vehicle occupants can be estimated from Equation 7:

$$L_{tc} = (2.62)(1.5)(80)\left(\frac{8}{50}\right)(3.6) = \$181$$

The total traffic delay cost will be:

$$L_{tr} = L_{rc} + L_{tc} = 419 + 181 = \$600$$

(d) Total Damage Costs

The total expected damage costs from the 10-year flood can be summed up:

| | |
|---|---|
| Roadway structural damage | $ 867.00 |
| Farmhouse damage | 0.00 |
| Traffic delay costs | 600.00 |
| Total | $ 1,467.00 |

To obtain annual risk cost, the total expected damage costs from floods of other recurrence intervals should be estimated. In this example, computations of damage costs from floods with recurrence intervals of 1, 2, 4, 10, 50 and 200 years are made and are summarized in Table 11.

TABLE 11. PAVEMENT DAMAGE AND TRAFFIC DELAY COSTS FOR VARIOUS FLOODS FOR EXAMPLE PROBLEM 2

| Recurrence Interval year | Discharge cfs | Backwater Elevation H ft. | Over-topping days | Pavement Damage Percent % | Pavement Damage Cost $ | Traffic Delay Time Days | Traffic Delay Cost $ | Total Damage Costs, $ $L(Q_i)$ 11 |
|---|---|---|---|---|---|---|---|---|
| 1 | 317 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 2 | 607 | 0.38 | 0.35 | 0 | 0 | 0.36 | 82 | 82 |
| 4 | 924 | 0.62 | 0.53 | 2 | 433 | 0.53 | 121 | 554 |
| 10 | 1,320 | 0.90 | 0.62 | 4 | 867 | 2.62 | 600 | 1,467 |
| 50 | 2,300 | 1.75 | 0.72 | 13 | 2,817 | 2.72 | 623 | 3,440 |
| 200 | 3,090 | 2.24 | 0.75 | 18 | 3,901 | 2.75 | 630 | 4,531 |

TOTAL ANNUAL COST

The annual risk cost can be determined by using Equation 3:

$$\text{Risk costs} = \sum_{i=1}^{n} (P_i - P_{i+1})\left[\frac{L(Q_i) + L(Q_{i+1})}{2}\right]$$

The detailed computation is shown in Table 12.

TABLE 12. ANNUAL RISK COST FOR EXAMPLE PROBLEM 2

| (1) Recurrence Interval Yr. | (2) Probability P | (3) Damage Cost L (Q) | (4) P - P | (5) Average Damage Cost | (6) Columns (4)x(5) |
|---|---|---|---|---|---|
| 1 | 1 | 0 | 0.5 | 41 | 21 |
| 2 | 0.5 | 82 | 0.25 | 318 | 80 |
| 4 | 0.25 | 554 | 0.15 | 1,011 | 152 |
| 10 | 0.10 | 1,467 | 0.08 | 2,454 | 196 |
| 50 | 0.02 | 3,440 | 0.015 | 3,986 | 60 |
| 200 | 0.005 | 4,531 | | | |
| | | | | Total | $509 |

The annual risk cost becomes $509.

C. Annual Total Expected Cost (TEC)

The annual total expected cost for the selected design alternative is the summation of the capital cost and the risk cost, which is:

3,949 + 509 = $4,458

D. Least Total Expected Cost (LTEC)

For other design alternatives, a similar analysis can be made. The design resulting in minimum total expected cost should be the preferred choice. A system analysis was made for the design alternative with various combinations of the following components:

| | | |
|---|---|---|
| (1) | Embankment elevations: | 83.5, 84.5, 85 and 86 ft. |
| (2) | Culvert diameters: | 3, 4, and 4.5 ft. |
| (3) | Number of culverts: | 1, 2, 4, 6, and 8 |
| (4) | Embankment protection: | with and without riprap |
| (4) | Traffic related costs: | Traffic delay time = overtopping time + two days (for pavement damage exceeding four percent) |

The results of the analysis indicate that the optimum design appears to be:

| | |
|---|---|
| Roadway elevation | = 85.0 ft. |
| One barrel of 4.5 ft culvert | |
| No riprap over embankment | |
| Construction cost | = $27,535 |
| Total annual expected cost | = $ 3,223 |

A vented ford with this design, however, is expected to be overtopped by a 1-year flood. If this is regarded as undesirable, then two barrels of 4.5 ft culverts can be used, instead. The total expected cost for this design then rises to $3,382.

A thorough system analysis is often time-consuming and tedious. Instead, the designer may choose only a few alternative design features to simplify the selection process.

As suggested in Section 4.3.1, the initial selection starts with low cost and thus high risk design. Then, for each new design, the increase in capital cost is compared with the decrease in risk cost. If the decrease in risk cost exceeds the increased capital cost, improvement of the new design is economically justifiable. Furthermore, improvement should be made in the same direction until the increase in capital cost is approximately the same as the decrease.

## 5.3 EXAMPLE 3: LOW BRIDGE

### 5.3.1 Data

#### 5.3.1.1 Unit Cost of Structure Components

| | |
|---|---|
| Structural concrete | \$ 470/cu.yd. |
| Reinforcing steel | \$ 0.75/lb |
| Loose riprap | \$ 45/cu.yd. |
| Embankment soil | \$ 2.4/cu.yd. |
| Pavement | \$ 22/sq.yd. |

#### 5.3.1.2 Hydrologic and Hydraulic Data

The stage-discharge curve obtained from a nearby gaging station is given in Fig. 25. Also plotted in this figure are the stage-area and stage-width curves for the crossing obtained from a survey of stream cross-section (Fig. 27). The watershed parameters are, drainage area = 19 sq mi; stream length - 7 mi; elevation difference = 200 ft:

50-year flood, $Q_{50}$ = 11,400 cfs

5.3.1.3 Land Use

There are no buildings in the vicinity of the crossing; floodplain upstream of the crossing is pasture land.

5.3.1.4 Traffic Data

Design ADT = 40; vehicle running cost = \$300/1000 vehicles miles;
Average occupancy = 1.5 person/vehicle;
Value of time = \$3.50/hr/person
Length of normal route = 2 mi; length of detour = 11 mi
Average speed of traffic = 45 mi/hr

5.3.1.5 Flood Loss Data

No structural loss due to flood will be considered. For embankment loss, the data in Section 5.2.1.5 (Fig. 22) will be used.

5.3.2 Design

5.3.2.1 Initial Design Feature

A 15-ft wide concrete slab bridge with the roadsurface at 6 ft elevation (1-year flood stage) is chosen as the initial design. The stream width at this elevation is about 90 ft, which is too wide for a single span slab bridge. Thus, a two-span slab bridge is considered. Two 22.5 ft slabs with a pier at the center and a pair of abutments on the stream bank are considered. The thickness of the slab is 16 in, with reinforcing steels aligned in accordance wtih the specification given in the table of Fig. 18. The detail of the initial design is shown in Fig. 27 with dotted lines.

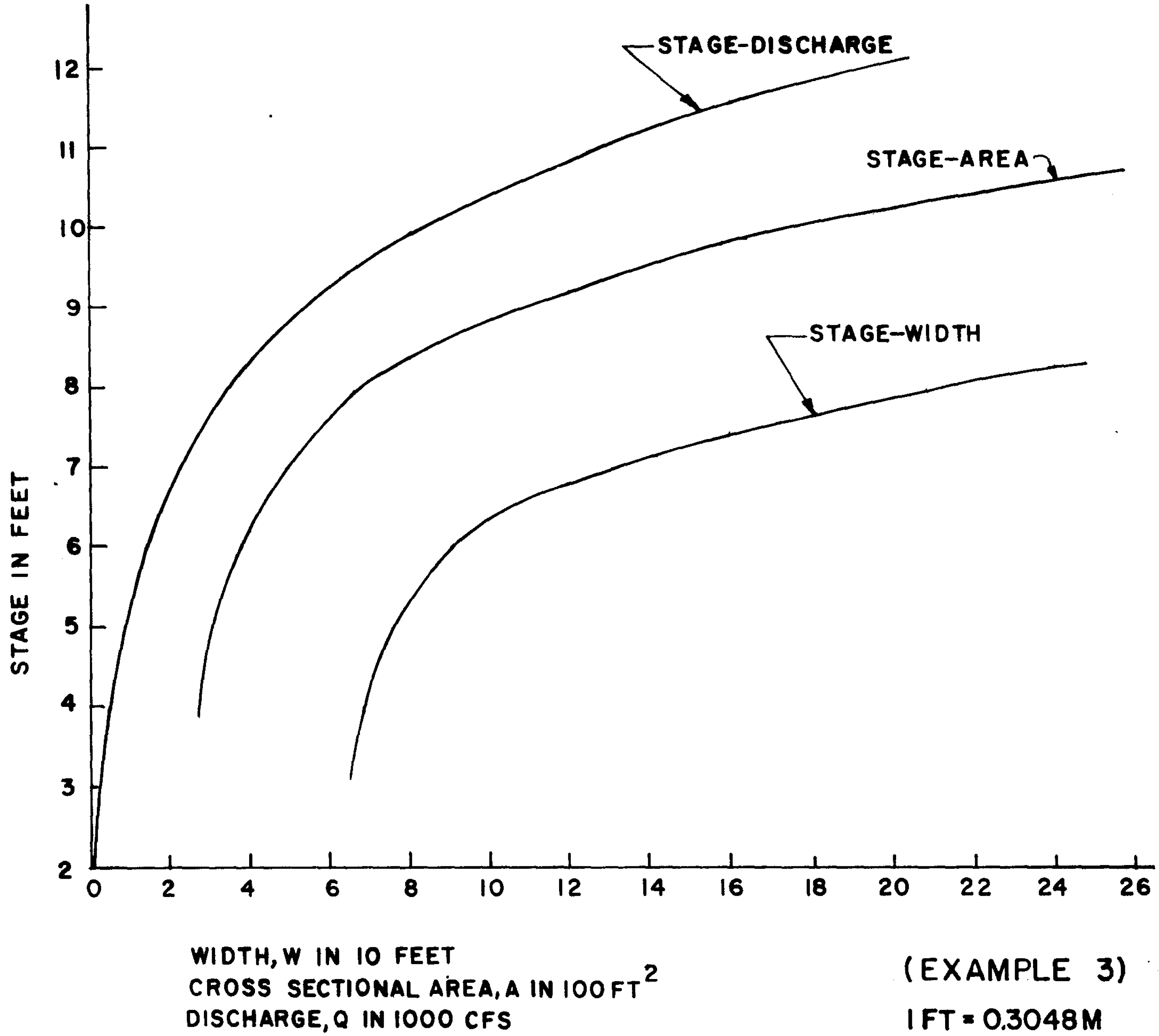



FIG. 26 STAGE-DISCHARGE, STAGE-AREA, STAGE WIDTH RELATIONSHIP

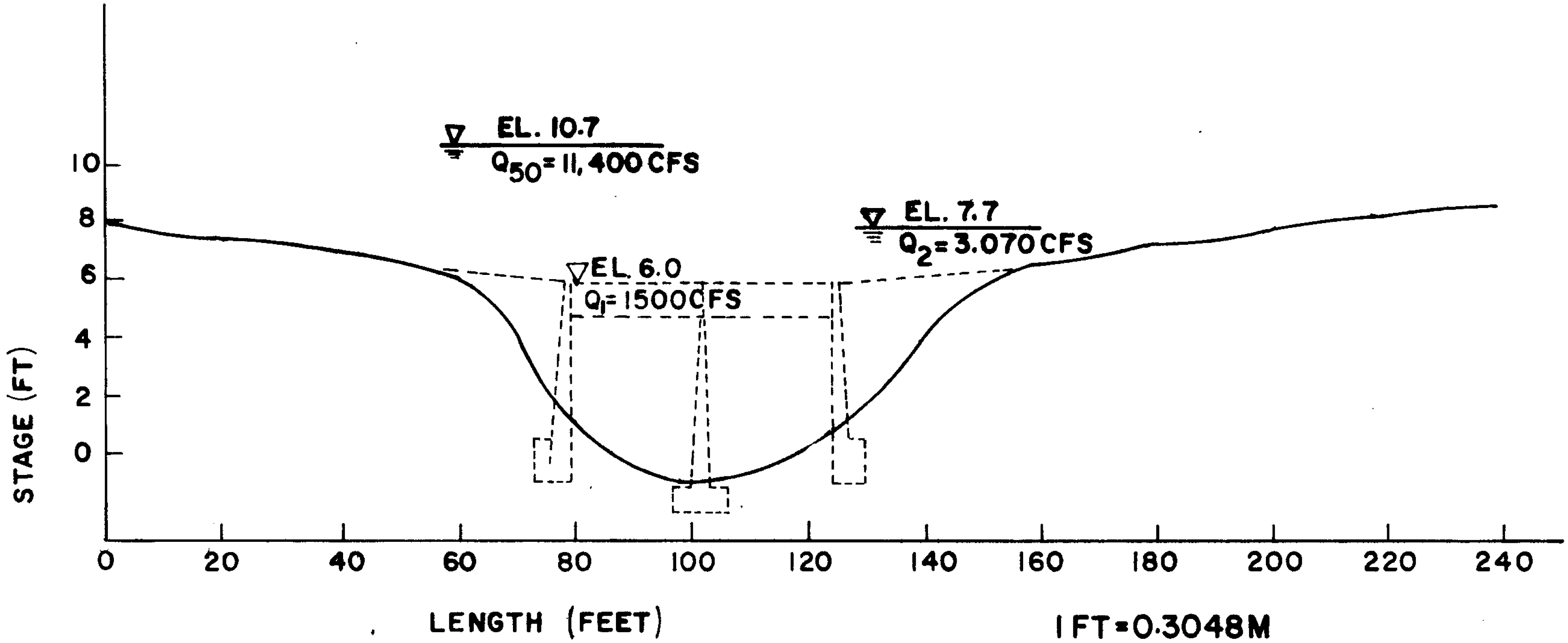



FIG. 27 STAGE DISCHARGE RELATIONSHIP
FOR LOW BRIDGE EXAMPLE

5.3.2.2 Hydrologic and Hydraulic Analysis

First, the discharge of a 10-year flood is determined from the 50-year flood by using Fig. 23.

Then, based on $Q_{10}$, the discharges of other floods are determined. The discharge of 2-year flood, for example, would be:

$$Q_2 = Q_{10} \times (0.45) = 3{,}070 \text{ cfs.}$$

The computed discharges of floods with recurrence intervals of 1-, 2-, 4-, 10-, 50- and 200-years are shown in Column 2 of Table 12.

Since the roadway elevation is set at the elevation of annual peak flood, the bridge will be submerged by a flood exceeding the annual peak flood. The backwater elevation h* can be approximated by using equation 14:

$$h^* = C_D \frac{A_s}{A} \cdot V^2/2g$$

For $Q_2$ = 3,070 cfs and for the cross-sectional area A = 620 $ft^2$, the mean velocity becomes V = 3,070/620 = 4.95 ft/sec.

The total projected area blocked by slab, pier and abutments, $A_S$, is approximately 210 sq ft. Therefore, the backwater is expected to be about:

$$h^* = 2 \cdot \frac{210}{620} \cdot \frac{4.95^2}{2 \times 32.2} = 0.26 \text{ ft}$$

The flow depth over the deck will be 7.7 + 0.26 - 6 = 1.96 ft. For other floods, overflow depths are computed and shown in Column 3 of Table 12.

### 5.3.2.3 Cost Analysis

#### A. Capital Costs

Capital cost of the bridge is estimated to be $40,015. The detailed computation is shown below:

TABLE 13. SUMMARY OF CAPITAL COSTS FOR EXAMPLE PROBLEM 3

| Item | Quantity | Unit Cost | Cost |
|---|---|---|---|
| Concrete slabs | 34 cu. yd. | $470/cu. yd. | 15,980 |
| Pier | 11 cu. yd. | $470/cu. yd. | 5,170 |
| Abutments | 50 cu. yd. | $235/cu. yd. | 11,750 |
| Reinforcement steel | 6700 lbs | $0.75/lb | 5,025 |
| Approach Embankment | 110 cu. yd. | $240/cu. yd. | 264 |
| Pavement | 83 sq. yd. | $ 22/sq. yd. | 1,826 |
| | | TOTAL COST | $40,015 |

Assuming that the interest rate is eight percent and service life of the bridge is 35 years, the capital recovery factor, CRF, becomes:

$$CRF = 0.8/[1 - (1 + 0.08)^{-35}] = 0.0858$$

Hence, the annual capital cost amounts to 0.0858 X 40,015 = $3,433.

B. Flood Damage Costs

The damage costs from flooding include those for the embankment, pavement, and traffic delay. Structural damage is considered minor in this analysis. Computations of damage costs due to the 2-year flood will be demonstrated.

(a) Overflow Damage Costs

In estimating damage cost of embankment, curves (Fig. 22), are used. The overflow duration for $Q_2$ = 3070 cfs can be obtained first by determining the overflow discharge, when the upstream water surface just reaches the top of the deck. Assuming a backwater of 0.25 ft, the overtopping discharge, $Q_{ot}$, can be read from the discharge-stage relationships (Fig. 26) for the stage of 5.75 ( = 6.00 - 0.25) to be 1,300 cfs. The base time, $t_b$, of the hydrograph for this basin is computed by Equation 21:

$$t_b = 6.92 \quad \frac{L^{1.16}}{(DH)^{0.385}} = 6.92\ (7)^{1.6}/(200)^{.385} = 8.6 \text{ hrs}$$

The overflow duration can then be computed by using Equation 20:

$$t_{ot} = \frac{3070 - 1300}{3070} (8.6) = 4.96 \text{ hrs, or } 0.2 \text{ days}$$

From Fig. 22, for an overflow depth of 1.96 ft and for a duration of five hr, four percent loss of pavement and three percent loss of embankment are expected. Damage cost of the pavement and embankment will be $73 ( = 0.04 X 1,826) and $8 (= 0.03 X 264) respectively.

(b) Traffic Related Cost

The crossing is at a remote area and the restoration time, Tr, is normally dictated by the waiting time for repair works. For this example, 3 days are assumed for cases involving damage of more than three percent of the pavement. For damages of less than 2 percent it is assumed that emergency repair work can be made and traffic resumed immediately after the flood has subsided. Under this assumption, the traffic delay time becomes:

$$t_r = 3 + 0.2 = 3.2 \text{ days for this example.}$$

The increased running cost can be computed from Equation 6:

$$L_{rc} = (3.2)(40)(9)\ 300/1,000 = \$346$$

Cost for lost time of vehicle occupants can be estimated from Equation 7:

$$L_{tc} = (3.2)(1.5)(40)(9/45)(3.5) = \$134$$

The total traffic delay cost will be:

$$L_{tr} = 346 + 134 = \$480$$

(c) Total Flood Damage Costs

The total flood damage costs due to the 2-year flood is the sum of the overflow cost and the traffic delay cost:

$$(73 + 8) + 480 = \$561.$$

To determine the annual risk cost, damage costs incurred due to $Q_1$, $Q_4$, $Q_{10}$, $Q_{50}$, and $Q_{200}$ must be determined similarly. They are tabulated in Table 14.

TABLE 14. DISCHARGE vs TOTAL DAMAGE COSTS FOR EXAMPLE PROBLEM 3

| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) |
|---|---|---|---|---|---|---|---|---|---|---|
| Recurrence Interval of Flood | Q cfs | Overflow depth ft* | Overtopping time days | Percent Damage | | Traffic Restoration Time Days | Damage Cost | | | Total Damage Costs $ |
| | | | | Pavement | Embankment | | Pavement | Embankment | Traffic Related | |
| 1 | 1,500 | 0.25 | 0.05 | 0 | 0 | 0.05 | 0 | 0 | 8 | 8 |
| 2 | 3,070 | 1.95 | 0.20 | 4 | 3 | 3.20 | 73 | 8 | 480 | 561 |
| 4 | 4,770 | 2.95 | 0.26 | 8 | 6 | 3.26 | 146 | 16 | 489 | 651 |
| 10 | 6,820 | 3.85 | 0.29 | 14 | 10 | 3.29 | 256 | 26 | 494 | 776 |
| 50 | 11,400 | 4.95 | 0.32 | 22 | 18 | 3.32 | 402 | 48 | 498 | 948 |
| 200 | 15,140 | 5.75 | 0.33 | 26 | 24 | 3.33 | 475 | 63 | 500 | 1,038 |

*Stage + 0.25 - 6.00 (backwater assumed about 0.25 ft. in all cases)

The annual risk cost for this design alternative can be computed from Equation 3. Details are presented in Table 15.

TABLE 15. ANNUAL RISK COST FOR EXAMPLE PROBLEM 3

| Recurrence Interval | P Probability | Damage Cost $ | P = | Average Damage Cost | Average PX Damage Cost ($) |
|---|---|---|---|---|---|
| 1 | 1 | 8 | | | |
| | | | 0.5 | 285 | 143 |
| 2 | 0.5 | 561 | | | |
| | | | 0.25 | 606 | 152 |
| 4 | 0.25 | 651 | | | |
| | | | 0.15 | 714 | 107 |
| 10 | 0.10 | 776 | | | |
| | | | 0.08 | 862 | 69 |
| 50 | 0.02 | 948 | | | |
| | | | 0.015 | 993 | 15 |
| 200 | 0.005 | 1,038 | | | |
| | Annual Risk Cost | | | | $ 486 |

C. Annual Total Expected Cost (TEC)

The annual estimated cost, which is the summation of the annual capital cost and risk cost, becomes $3,919.

Similar analysis should be carried out for alternative designs and all the annual estimated costs should be compared. The design resulting in the least cost should be the final choice. This procedure was presented in the last example and is not repeated here.

# 6.0 CONSTRUCTION AND INSTALLATION

## 6.1 ALIGNMENT OF STRUCTURE

An LWSC should be considered only in a stable stream reach. Any history of degradation or aggradation of the bed or movement of channel at the crossing site should be carefully reviewed. Natural stream meanders should be studied and, where feasible, low cost river training works may be considered. Precaution should be taken to prevent the stream from changing its course, particularly near the ends of the vent. For LWSCs used in wide flood plains, the lowering of approach fills is necessary to reduce the possibility of loss of structure. Approaches should be aligned to avoid undue scour or changes in main stream channel course. The first principle of LWSC location is to provide the stream with a direct entrance and exit. Any abrupt change in direction at either end will retard the flow and make the structure hydraulically inefficient.

## 6.2 FOUNDATIONS

Good foundations are as important for LWSC as for a high-level bridge. Foundation type and details will be dependent on site conditions and obviously vary from site to site, but typical foundation types are discussed in this section.

### 6.2.1 Foundation - Fords

A layer of gravels is normally used to support a concrete pavement. A thickness of about 8 to 12 in (20 to 30 cm) is considered sufficient.

### 6.2.2 Foundation - Vented Fords

6.2.2.1 Foundation for Pipes

The foundation will be as follows:

1. Soft foundation--the pipe foundation should have a width of three times nominal pipe diameter (D) up to a maximum of (D+4)' and depth of approximately 2 ft (0.61 m), and should be filled with suitable granular material, uniformly compacted.

2 Unyielding foundation--the pipe foundation should have a width of nominal pipe diameter plus 12" (0.305 m) and depth of 12" (0.305 m) minimum, up to 0.75D maximum and should be filled with lightly compacted material.

6.2.2.2 Sidefill

Sidefill material within one pipe diameter of the sides of pipe and not less than 1 ft (0.305 m) over the pipe should be readily compactible soil or granular fill material. Beyond these limits, the sidefills may have regular embankment fill. Sidefill material should be placed in layers not exceeding 6 in in compacted thickness. The materials should have proper density and optimum moisture content. Beyond 3 ft (0.914 m) from sides of pipes and 12 ft (0.305 m) from top of the pipe, it can have normal embankment fill.

6.2.3 Foundation - Low Bridges

When a low bridge is overtopped by flood, it receives a strong overturning moment. Thus, a good foundation is extremely important for a low bridge. Pier and abutment footings should be extended beyond the possible scour depth. For soft streambed, to ensure the foundation against the effect of scour, piles may be driven to a hard pan and the footings placed immediately over them. This type of foundation is dependable, can be rapidly constructed without deep excavation, and saves greatly on dewatering.

# 7.0 REFERENCES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