NASHVILLE STORMWATER MANAGEMENT MANUAL

VOLUME 3—THEORY

CHAPTER 7
Bridge Hydraulics

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Chapter 7
BRIDGE HYDRAULICS

SYNOPSIS

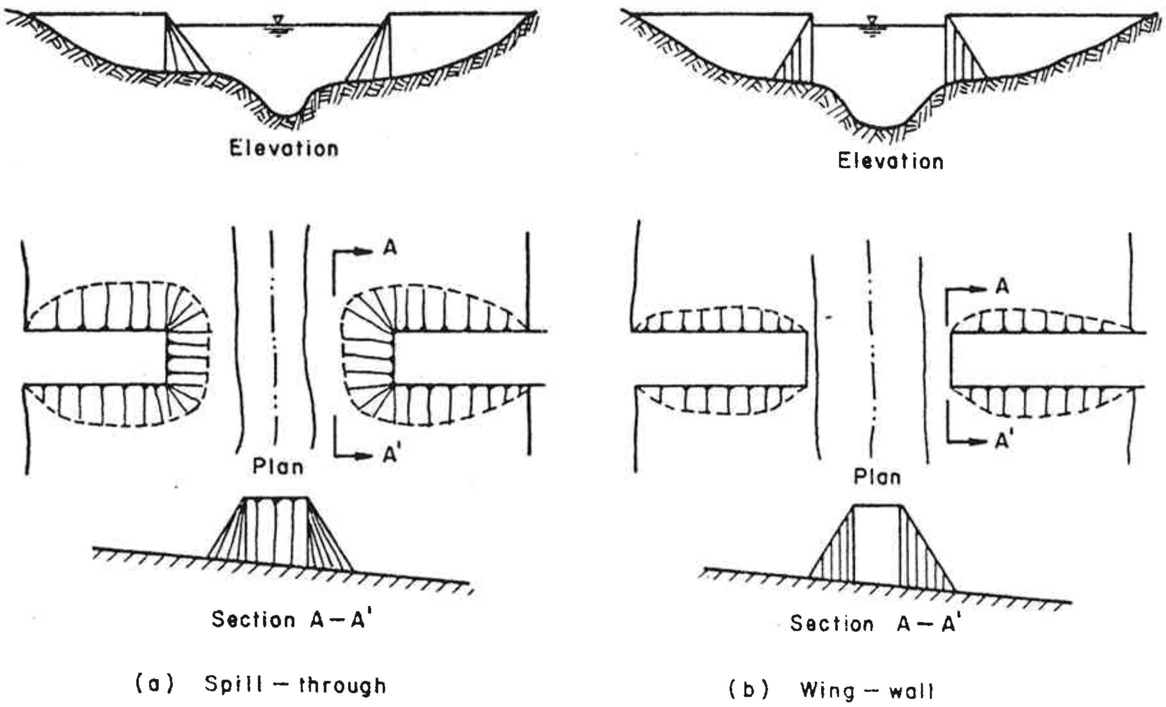
Bridge placement that constricts the normal passage of floodwaters can cause excessive backwater, which may result in flooding of upstream property, overtopping of roadways, excessive scour, or even the loss of a bridge. Bridge hydraulic computations are used to establish an appropriate hydraulic opening (length, vertical clearance, and location) within the acceptable limits of increased backwater.

Results of laboratory studies using physical models at Colorado State University (1957) and the Iowa Highway Research Board (1956 and 1958) have been combined with field data collected by the USGS to develop the bridge hydraulics procedures discussed in this chapter. Key references are HDS-1 (USDOT, FHWA, 1978) and WSPRO (HY-7) (USDOT, FHWA, 1986). Information on scour and degradation is summarized from the Training and Design Manual entitled Highways in the River Environment, Hydraulic and Environmental Design Considerations (USDOT, FHWA, 1975).

7.1 BRIDGE TYPES

River and stream highway crossings can sometimes be accomplished without encroachment into the flood plain by locating the bridge and its approaches far above and beyond a possible flood stage. However, since the cost of spanning an entire flood plain is usually prohibitive, some degree of encroachment is commonly imposed. An encroachment can occur when earth fill embankments are placed over the flood plain or into the channel itself, or when piers and abutments are placed in the main channel of a river.

A bridge is the most common type of facility used to minimize flood plain encroachment. The geometric properties of the bridge, illustrated in Figure 7-1, are commonly used, depending on the conditions at the site. The approach embankments may be skewed or normal (perpendicular) to the direction of flow. One approach may be longer than the other, producing an eccentric crossing. Abutments used for



Reference: USDOT, FHWA, Training and Design Manual (1975).

FIGURE 7-1
Geometric Properties of Bridge Crossings

the overbank-flow case may be set back from the low-flow channel banks to provide room to pass the floodflow; or the abutments may extend up to the banks or even protrude over the banks, constricting the base flow channel. Piers, dual bridges for multi-lane freeways, channel bed conditions, and spur dikes are additional factors to consider when selecting an appropriate type of bridge crossing.

7.2 RIVER DYNAMICS

In general, a river is dynamic, continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks. Changes may be slow or rapid, natural or artificially induced. When a project modifies a river channel locally, it frequently changes channel characteristics both up and down the river. The response of a river to man-induced changes often occurs in spite of attempts to keep the anticipated response under control.

An important component of river hydraulics at bridge crossings is the distribution and direction of flow velocities. The maximum longitudinal velocity is usually approximately 25 to 50 percent greater than the average velocity for the cross section. The instantaneous turbulent velocity in rivers can exceed the average velocity by as much as 70 percent or more.

When river flow encounters the change in direction caused by a bend, super-elevation of the channel bottom produces a transverse, or elliptical, velocity distribution component. Transverse velocities are caused by an imbalance of radial pressures on the flow as it travels around the bend. The magnitude of the transverse velocity component depends on the radius of curvature of the river and on the proximity of the flow to the banks. Immediately next to the banks, lateral velocity cannot occur if the river is narrow and deep.

7.3 FUNDAMENTALS

7.3.1 BACKWATER

It is seldom economically feasible or necessary to span the entire width of a flood plain. Where conditions permit, approach embankments can be extended onto the flood plain to reduce costs, recognizing that in so doing, the embankments will constrict the flow of the stream during flood stages. Normally, this is an acceptable practice provided water surface profile and scour conditions are properly evaluated.

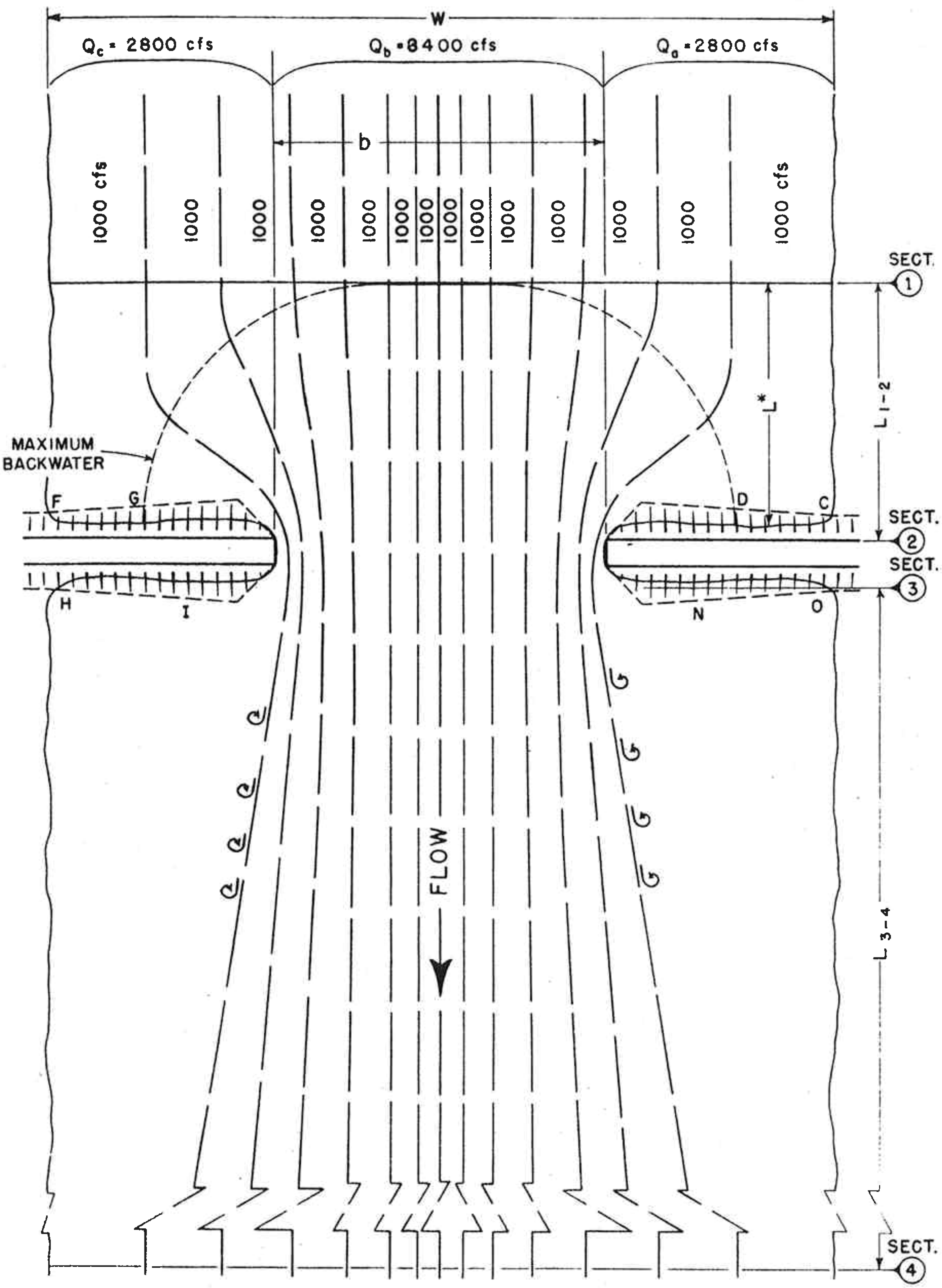
The manner in which flow is contracted in passing through a channel constriction is illustrated in Figure 7-2. For this example, the flow bounded by each adjacent pair of streamlines is the same (1,000 cfs). The channel constriction appears to produce practically no alteration in the shape of the streamlines near the center of the channel. A very marked change occurs near the abutments, however, since the momentum of the flow from both sides (or flood plains) must constrict the advancing central portion of the stream. Upon leaving the constriction, the flow gradually expands (5 to 6 degrees per side) until normal conditions are reestablished.

Constriction of the flow causes a loss of energy, with the greater portion occurring in the reexpansion downstream. This loss of energy is reflected in a rise in the water surface and in the energy line upstream from the bridge. A profile along the center of the stream illustrates this energy loss, as shown in Figures 7-3 for wingwall abutments (Part A) and 7-4 for spillthrough abutments (Part A).

The normal stage of the stream for a given discharge, before constricting the channel, is represented by the dashed line labeled normal water surface. (Water surface is abbreviated as "W.S." in the figures.) The nature of the water surface after constriction of the channel is represented by the solid line "actual water surface." Note that the water surface starts out above normal stage at Section 1, passes through normal stage close to Section 2, reaches minimum depth in the vicinity of Section 3, and then returns to normal stage a considerable distance downstream, at Section 4. Determination of the rise in water surface at Section 1 is denoted by the symbol h_1* and is referred to as the <u>bridge backwater</u>.

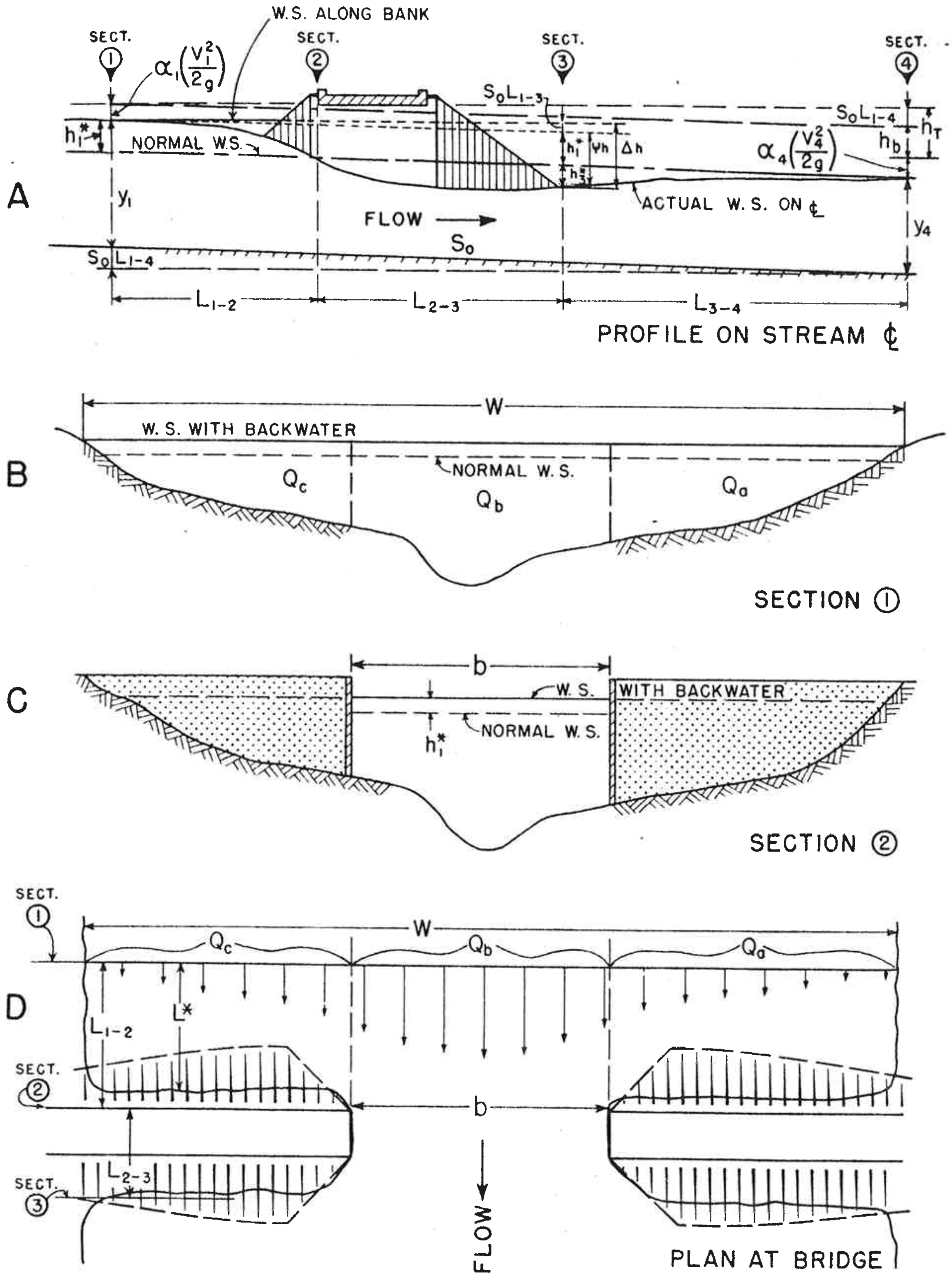
7.3.2 TYPES OF FLOW

The four types of flow encountered in bridge waterway design are labeled Types I, IIA, IIB, and III in Figure 7-5. The long dashed lines shown on each profile represent normal



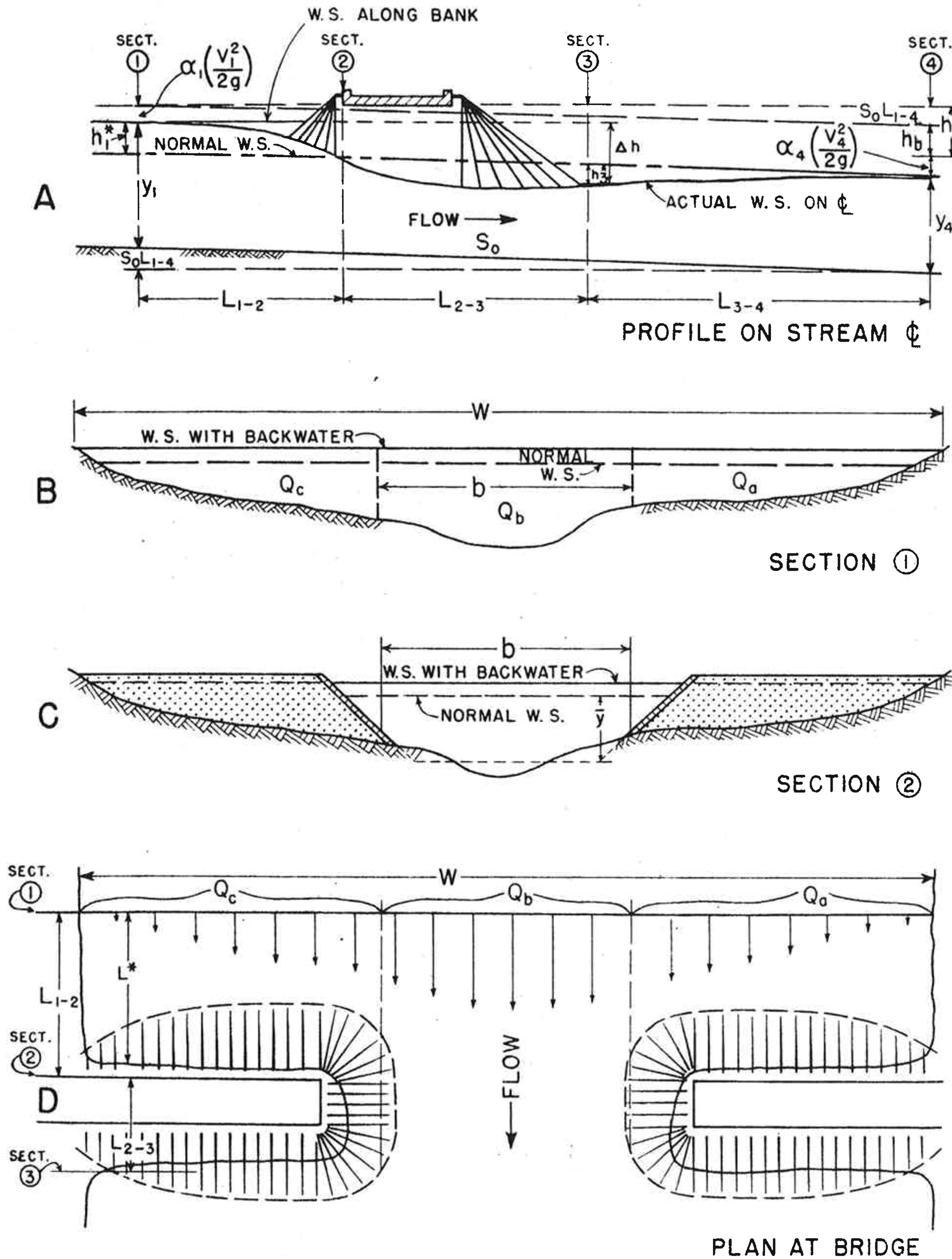
Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-2
Flow Lines for Typical Bridge Crossing



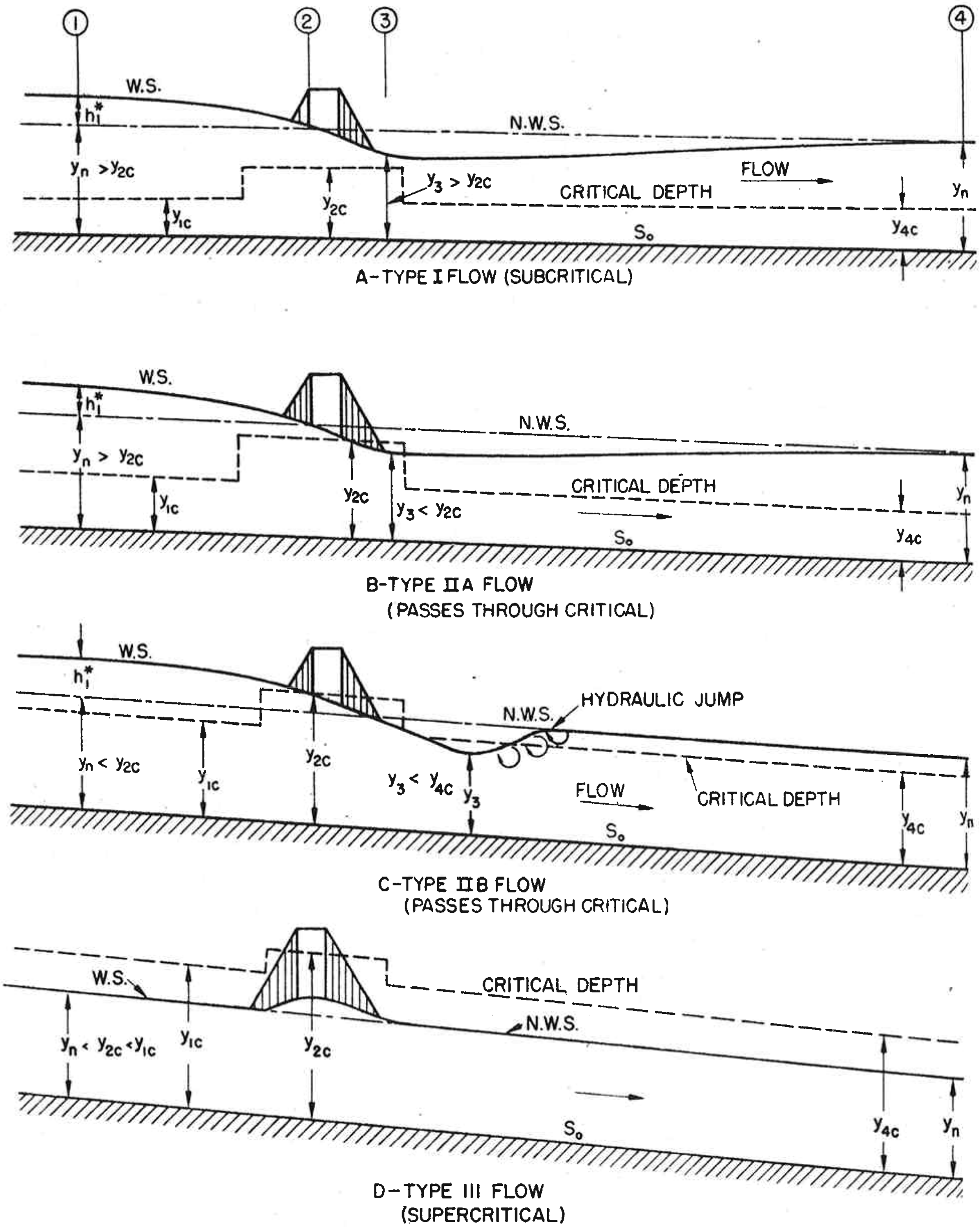
Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-3
Normal Crossings: Wingwall Abutments



Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-4
Normal Crossings: Spillthrough Abutments



Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-5
Types of Flow Encountered

water surface, or the stage the design flow would assume prior to placing a constriction in the channel. The solid lines represent the configuration of the water surface on the centerline of the channel after the bridge is in place. The short dashed lines represent critical depth, or critical stage, in the main channel, y_{1c} and y_{4c}, and critical depth within the constriction, y_{2c}, for the design discharge. Since normal depth is shown as essentially the same in the four profiles, the discharge, boundary roughness, and slope of the channel must all increase in passing from Type I to Type IIA, Type IIB, and Type III flow.

Type I Flow

In Figure 7-5, Part A, normal water surface is above critical depth at all times. This condition has been labeled Type I or subcritical flow, the type most often encountered in practice. The design information in HDS-1 (USDOT, FHWA, 1978) is limited primarily to Type I flow. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between Sections 1 and 4.

Type IIA Flow

There are at least two variations of Type II flow, described here as Types IIA and IIB. For Type IIA flow, shown in Figure 7-5, Part B, normal water surface again remains above critical depth in the unconstricted channel, but passes through critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at Section 4). Thus, the backwater expression for Type I flow is not valid for Type IIA flow.

Type IIB Flow

The water surface for Type IIB flow, shown in Figure 7-5, Part C, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction, dips below critical depth for the channel downstream from the constriction, and then returns to normal. The return to normal depth can be abrupt, as seen in Figure 7-5, Part C, where it takes place in the form of a poor hydraulic jump, since normal water surface in the stream is above critical depth. A backwater expression applicable to both Types IIA and IIB flow has been developed by equating the total energy between Section 1 and the point at which the water surface passes through critical stage in the constriction.

Type III Flow

In Type III flow, shown in Figure 7-5, Part D, the normal water surface is always below critical depth and the flow throughout is supercritical. This condition, which requires a steep gradient, occurs mainly in mountainous regions. Theoretically, backwater should not occur for this type, since flow throughout is supercritical, but an undulation of the water surface in the vicinity of the constriction is quite possible, as indicated in Figure 7-5, Part D.

7.3.3 TERMINOLOGY

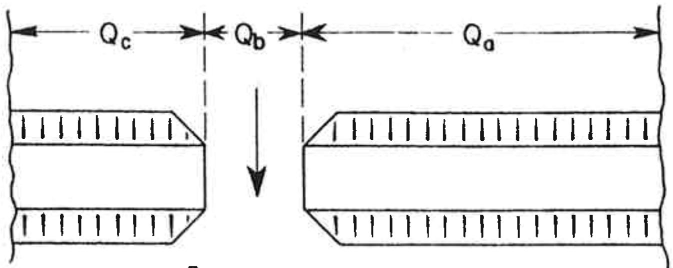
Terms and expressions frequently used in bridge hydraulics are briefly defined below.

Normal stage. Normal stage is the normal water surface elevation of a stream at a bridge site for a particular discharge, before stream constriction (see Figures 7-3, Part A, and 7-4, Part A). The profile of the water surface is essentially parallel to the bed of the stream.

Abnormal stage. When a bridge site is located upstream from, but relatively close to, the confluence of two streams, high water in one stream can produce a backwater effect extending for some distance up the other stream. This can cause the stage at a bridge site to be abnormal, i.e., higher than would exist for the tributary alone. An abnormal stage may also be caused by a dam, another bridge, or some other constriction downstream. The water surface with abnormal stage is not parallel to the bed.

Normal crossing. A crossing with alignment at approximately 90 degrees to the general direction of flow during high water (as shown in Figure 7-2).

Eccentric crossing. A crossing in which the main channel and the bridge are not in the middle of the flood plain (see Figure 7-6). Eccentricity is defined as one minus the ratio

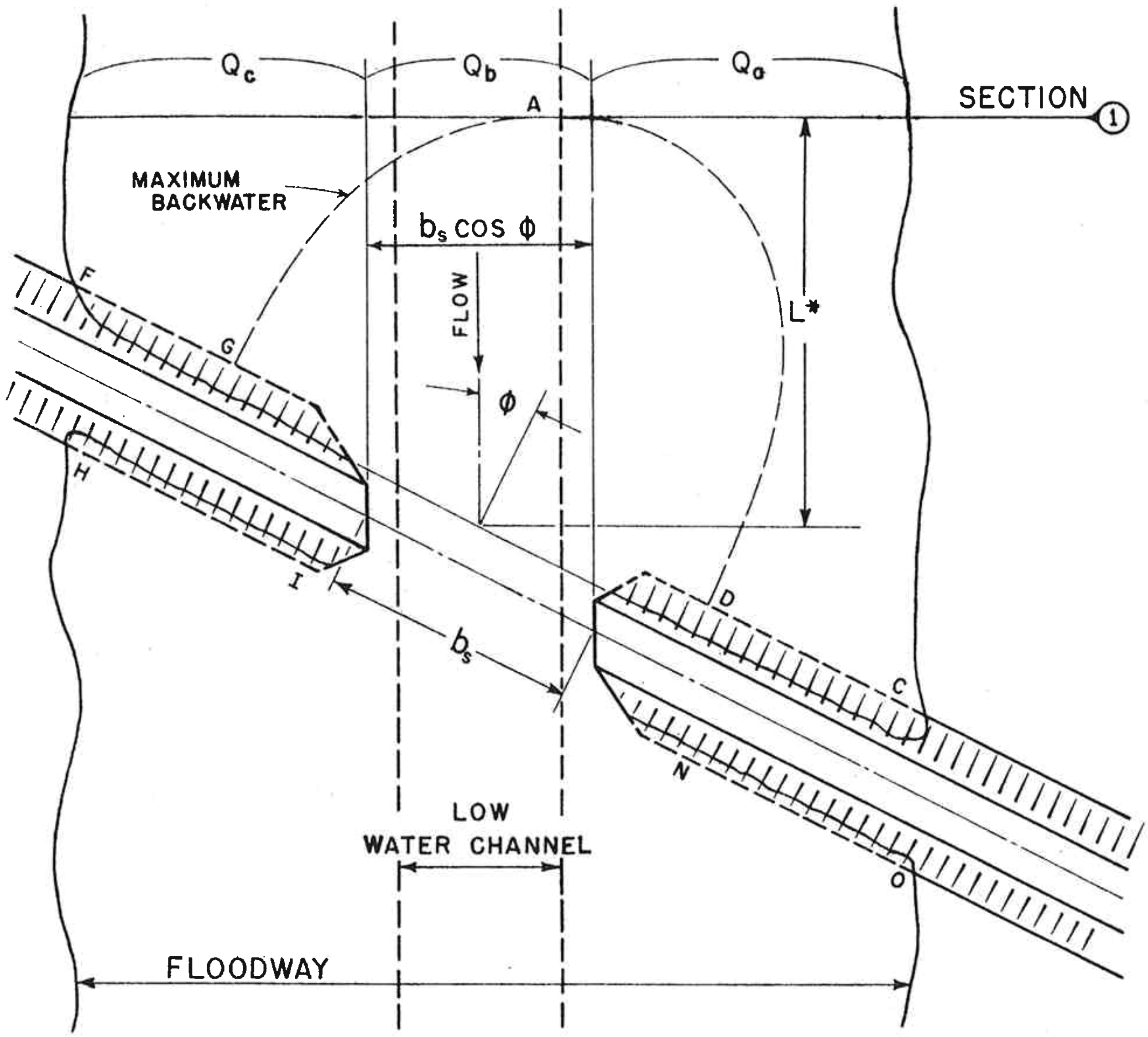


$$\text{eccentricity} = \left(1 - \frac{Q_c}{Q_a}\right) \quad \text{where} \quad Q_c < Q_a \quad \text{or}$$

$$\text{eccentricity} = \left(1 - \frac{Q_a}{Q_c}\right) \quad \text{where} \quad Q_a < Q_c$$

Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-6
Definition Sketch of Eccentricity at Bridge Crossings



Reference: USDOT, FHWA, HDS-1 (1978).

FIGURE 7-7
Skewed Bridge Crossings

of the lesser to the greater discharge outside the projected length of the bridge.

Skewed crossing. A crossing that is aligned other than 90 degrees to the general direction of flow during flood stage (see Figure 7-7).

Dual crossing. A pair of parallel bridges, such as for a divided highway.

Multiple bridges. Usually consisting of a main channel bridge and one or more relief bridges.

Width of constriction, b. For abutments with vertical faces, the construction width, b, is simply the horizontal distance between abutment faces. In the more usual case involving spillthrough abutments, where the cross section of the constriction is irregular, it is suggested that the irregular cross section be converted to a regular trapezoid of equivalent area, as shown in Figure 7-4, Part C. Then the length of bridge opening can be interpreted as:

$$b = \frac{A_{n2}}{\bar{y}} \qquad (7-1)$$

where:

b = Width of constriction, in feet

A_{n2} = Gross area of constriction below normal water surface at Section 2, in square feet

$\bar{y}$ = Mean depth of flow, referenced to normal stage, in feet

Width to depth ratio. Defined as the ratio of flood plain width to mean constriction depth:

$$\frac{A_1}{\bar{y}_2} \quad \text{(for irregular cross sections)} \qquad (7-2)$$

where:

A_1 = Area of flow, including backwater at Section 1, in square feet

$\bar{y}_2$ = Mean depth of flow at Section 2, in feet

7.3.4 CONVEYANCE

Conveyance is a measure of the ability of a channel to transport flow. In streams of irregular cross section, it is necessary to divide the water area into smaller but more or less regular subsections, assigning an appropriate roughness coefficient to each and computing the discharge for each subsection separately. By rearranging Manning's Equation as presented in Chapter 3, the following relationship is derived:

$$k = \frac{q}{\sqrt{S}} = \frac{1.49}{n} AR^{2/3} \tag{7-3}$$

where:

k = Channel subsection conveyance

q = Subsection discharge, in cfs

S = Channel bottom slope, in feet/foot

n = Manning's roughness coefficient

A = Subsection cross-sectional area, in square feet

R = Subsection hydraulic radius, in feet

Conveyance can, therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway computations, conveyance is used as a means of approximating the distribution of flow in the natural river channel upstream from a bridge. Total conveyance, K, is the summation of the subsection conveyances constituting the particular section.

7.3.5 BRIDGE OPENING RATIO

The bridge opening ratio, M, defines the degree of stream constriction involved, expressed as the ratio of the flow that can pass unimpeded through the bridge constriction to the total flow of the river. Referring to Figure 7-2:

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \tag{7-4}$$

where:

M = Bridge opening ratio

Q_b = Unimpeded flow, in cfs

Q_a and Q_c = Flow impeded by bridge obstruction, in cfs

Q = Total flow, in cfs

As shown in Figure 7-2, if Q_b = 8,400 cfs and Q = 14,000 cfs, M = 0.60.

The irregular cross section common in natural streams and the variation in boundary roughness within any cross section cause the velocity to vary across a river, as indicated by the stream tubes in Figure 7-2. The bridge opening ratio, M, is most easily explained in terms of discharges but is usually determined from conveyance relations. Since conveyance is proportional to discharge, assuming all subsections to have the same slope, M can also be expressed as:

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K} \tag{7-5}$$

where:

M = Bridge opening ratio

K_b = Conveyance of unimpeded flow, in cfs

K_a and K_c = Conveyance of flow impeded by bridge obstruction, in cfs

K = Total conveyance of flow, in cfs

7.3.6 KINETIC ENERGY COEFFICIENT

As the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A_1)^2/2g$ for the stream at Section 1, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head by a kinetic energy coefficient, α, defined as:

$$\alpha = \frac{\Sigma(k^3/a^2)}{K^3/A^2} \qquad (7\text{-}6)$$

where:

α = Kinetic energy coefficient for nonuniform velocity distribution

k = Conveyance of subsection (see Section 7.3.4)

a = Flow area of subsection, in square feet

K = Conveyance of total cross section (see Section 7.3.4)

A = Flow area of total cross section, in square feet

7.3.7 EFFECTIVE FLOW LENGTH

Because friction losses are directly proportional to flow length, it is important to obtain the best possible estimate of flow length. This is especially true for those cases where the friction loss is a significant component of the energy balance between two sections. The straight-line distance between sections is typically used to estimate the approach reach friction loss. For minor degrees of constriction, this is usually adequate. However, for significant constrictions, the straight-line distance is representative of only that portion of the flow that is generally in direct line with the opening. Flow farther from the opening must flow downstream and across the valley to reach the opening, thus traveling much farther than the straight-line distance. A technique has been developed to calculate an effective flow length that accounts for this increase. The optimum location of the approach section is estimated using the following equations:

$$L_{opt} = \left[\frac{b}{\pi(1 - m')}\right] \Phi \quad (7\text{-}8)$$

$$m' = 1 - \frac{b}{B} \quad (7\text{-}9)$$

$$\Phi = \frac{1}{2} \ln \left[\left(\sqrt{\frac{8}{\varepsilon^2} + 8} - \frac{3}{\varepsilon} - \varepsilon\right)\left(-\sqrt{8 + 8\varepsilon^2} - 3\varepsilon - \frac{1}{\varepsilon}\right)\right] - \ln\left(\varepsilon - \frac{1}{\varepsilon}\right) \quad (7\text{-}10)$$

$$\varepsilon = 1 + \delta + \sqrt{\delta^2 + 2\delta} \quad (7\text{-}11)$$

$$\delta = \frac{2}{\tan^2\left[1 - \left(\frac{b}{2B}\right)\pi\right]} \quad (7\text{-}12)$$

where:

L_{opt} = Optimum distance between the approach section and the upstream face of the bridge opening, in feet

b = Bridge opening length, in feet

m' = Geometric contraction ratio

B = Top width of the approach section flow area, in feet

Φ, ε, and δ = Terms for computing the optimum location for the approach section

By definition, the optimum location for the approach section, L_{opt}, is in a zone of nearly one-dimensional flow, to satisfy the basic requirements of the one-dimensional energy equation. However, as long as the effective flow length is used in the computations, the approach section can be placed almost anywhere. As a convention, the approach section is generally placed one bridge-opening length upstream from the bridge opening.

The computational technique above varies depending on the relative magnitudes of L_{opt} and b. For the ideal situation of a symmetric constriction with uniform, homogeneous conveyance, only one-half of the valley cross section is required. This one-half section is generally divided into ten equal-conveyance stream tubes between the edge of the water and the centerline at both the L_{opt} location and the upstream face of the bridge. Equal-conveyance stream tubes are equivalent to equal-flow stream tubes for one-dimensional flow.

Figure 7-8 illustrates a case with a small geometric contraction ratio. L_{opt} is less than b for lesser degrees of constriction. Since L_{opt} is located in a zone of nearly one-dimensional flow, the streamlines are essentially parallel between the approach section and the L_{opt} location. Between L_{opt} and the bridge opening, the corresponding flow division points are connected with straight lines. In this case, the effective flow length is the average length of the ten equal-flow stream tubes, computed as:

$$L_{av} = 1/10 \left[\sum_{i=2}^{10} s_i + (s_1 + s_{11})/2 \right] \qquad (7\text{-}13)$$

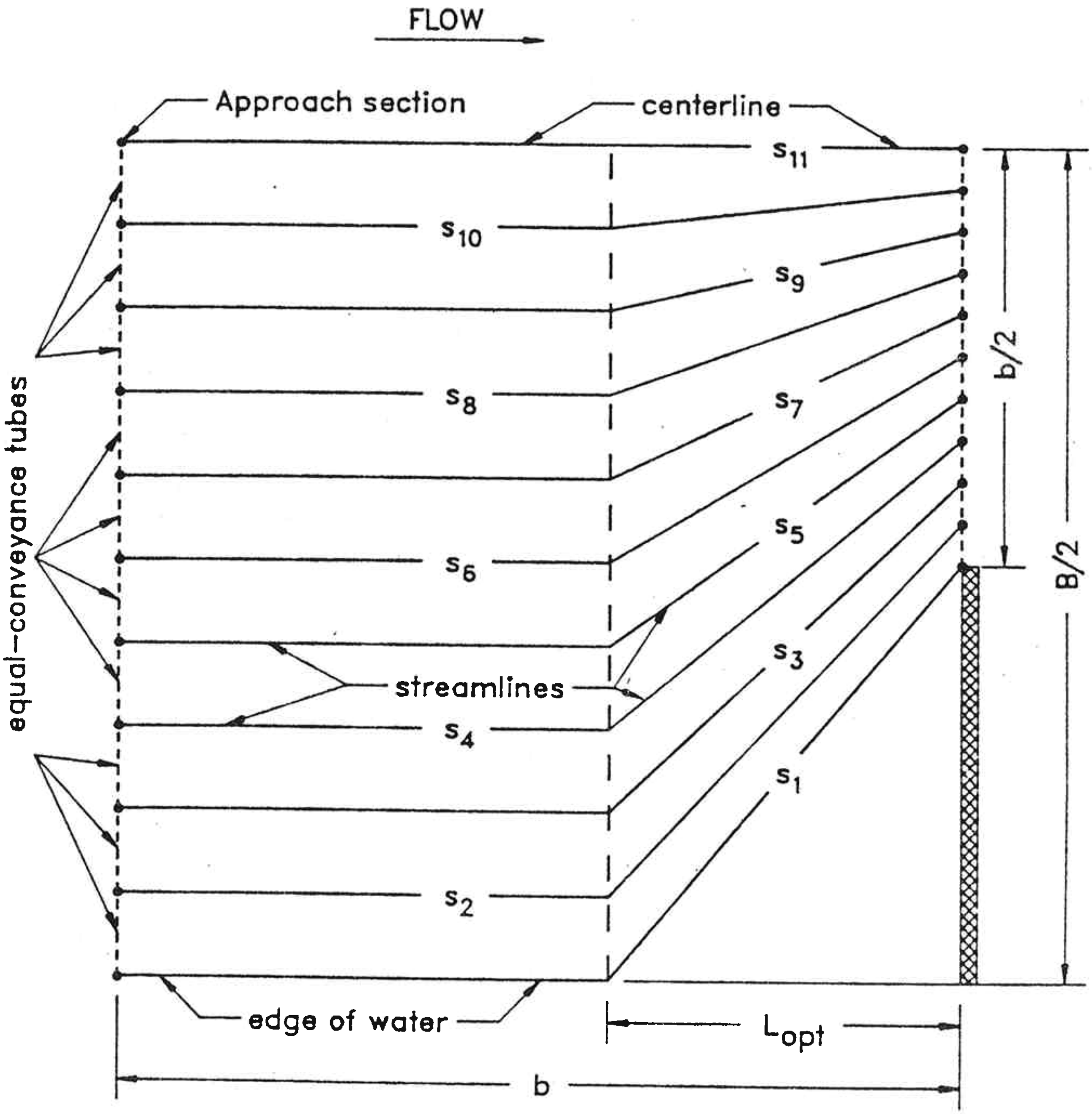
where:

L_{av} = Effective flow length for a symmetric constriction with uniform, homogeneous conveyance, in feet

s_i = Length of streamline, i, in feet

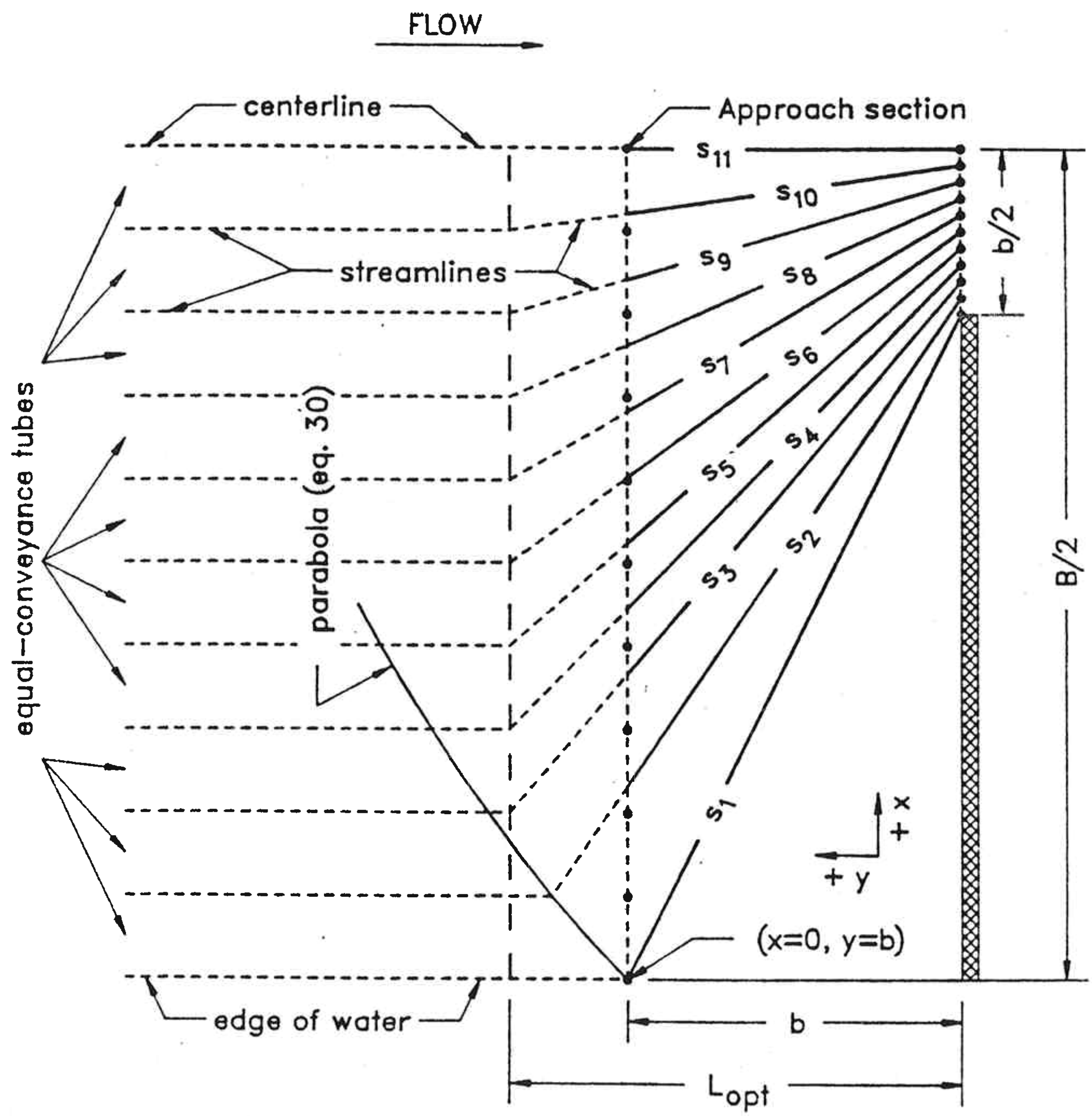
Although the straight-line pattern is a simplification of the actual curvilinear streamlines, the computed L_{av} values are less than 2 percent greater than the exact solution for small geometric contraction ratios.

Figure 7-9 illustrates a relatively high degree of geometric contraction. Connecting the flow division points of the L_{opt} and bridge sections does not result in representative



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-8
Effective Flow Length Configuration for Relatively Small Contractions



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-9
Effective Flow Length Configuration for a Relatively High Degree of Contraction

lengths for those streamlines farthest away from the opening. Therefore, a parabola is computed as follows:

$$y^2 = 2b\left(x + \frac{b}{2}\right) \tag{7-14}$$

where:

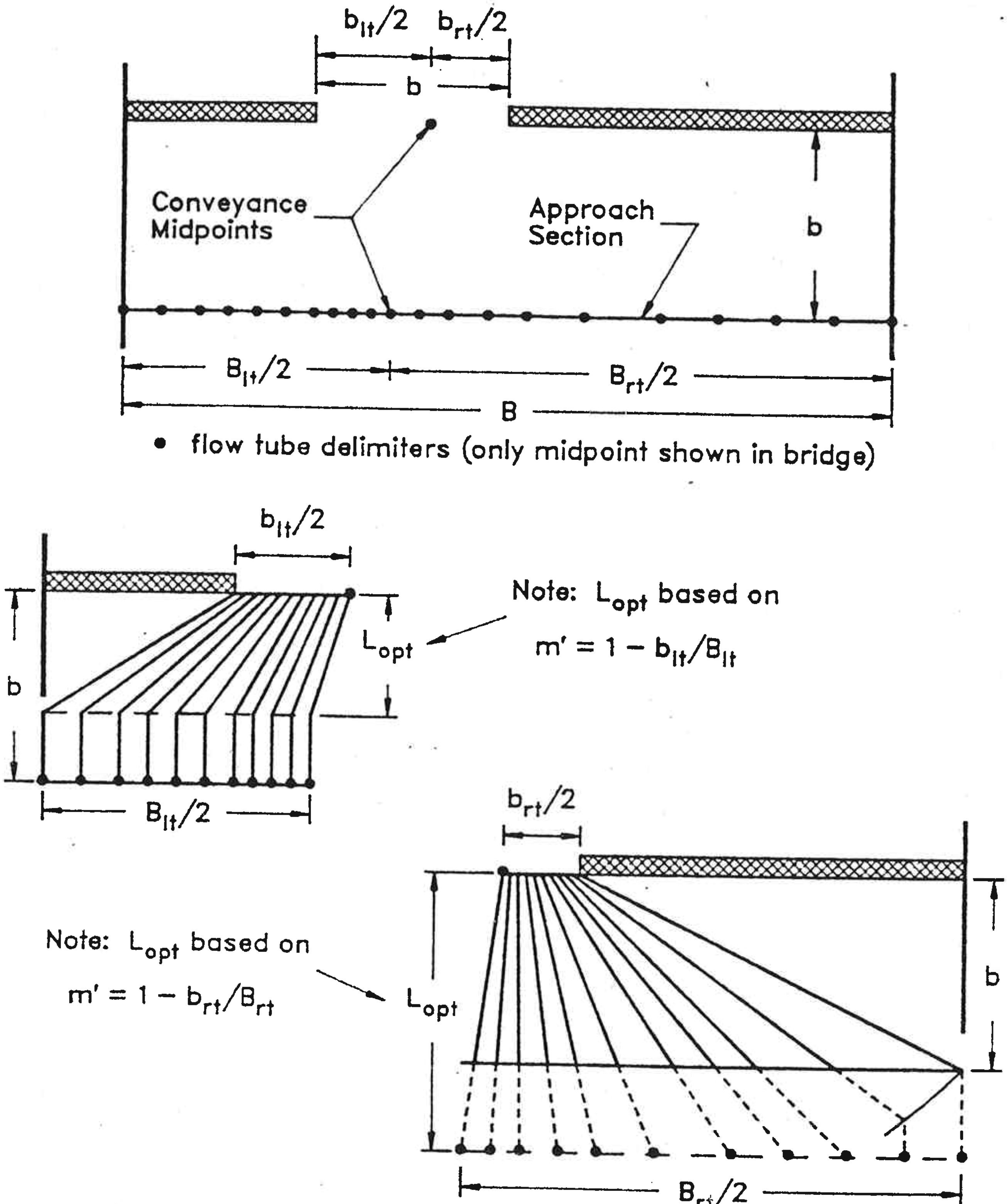
y = Parabola y coordinate, in feet

x = Parabola x coordinate, in feet

b = Bridge opening length, in feet

This parabola has its focus at the edge of the water and its axis in the plane of the upstream face of the bridge. Positive x and y distances are measured from the edge of the water toward the stream centerline and upstream from the plane of the bridge, respectively. For portions of the section where L_{opt} is upstream from this parabola, the parallel streamlines are projected to the parabola. A straight line then connects this projected point with the corresponding flow division point in the bridge opening. Flow division points of the L_{opt} section at or downstream from the parabola are connected directly to their corresponding flow division point for the bridge opening. Only the distances between the approach and bridge-opening sections are used to compute the effective flow length, L_{av}, with Equation 7-13. This process generally produces results that are within 5 percent of the exact solution. For very severe constrictions (e.g., m' = 0.95), the differences are closer to 10 percent.

A nonuniform conveyance distribution in the approach reach is represented by defining the stream tubes on a conveyance basis. Asymmetric constrictions with nonuniform conveyances can be analyzed by treating each half of the reach on either side of the conveyance midpoints separately, then averaging the results. The effective flow length, L_{av}, for each side is the conveyance-weighted average streamline length. A typical asymmetric, nonuniform conveyance situation is illustrated in Figure 7-10.



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-10
Assumed Flow Pattern for a Nonsymmetric Constriction with a Nonhomogeneous Roughness Distribution

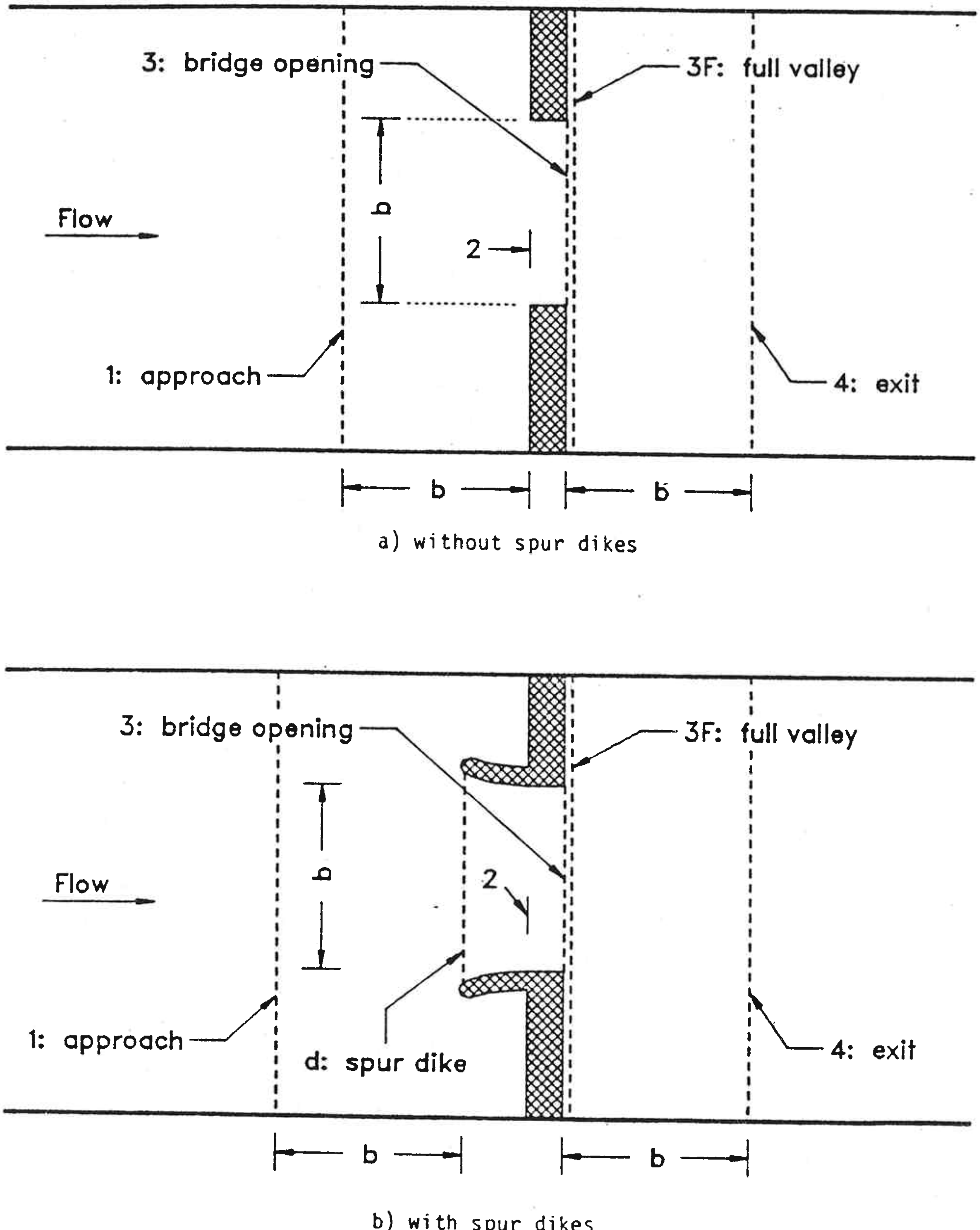
7.4 WATER SURFACE PROFILES

Computation of the water surface profile through a bridge having a single waterway opening requires definition of a minimum of four cross sections. The cross sections numbered 4, 3F, and 1 in Figure 7-11 are unconstricted valley sections and are referred to as exit, full-valley, and approach sections, respectively. These three sections, along with the bridge-opening section (Figure 7-11, Section 3), represent the minimum definition of a stream crossing.

The bridge-opening section is located at the downstream face of the bridge. Another location (Figure 7-11, Section 2) at the upstream face of the bridge is a control point in some cases, but requires no input data. If spur dikes are present, a dike section located at the toe of the dikes must be defined (Figure 7-11, Section d). Also, if flow over the embankment might occur, a road-grade section (not shown in Figure 7-11) is required to define the top of the embankment, which would serve as the crest of a weir.

The flow situation that exists at a single bridge opening depends on the relative elevations of the water surface both upstream and downstream of the bridge with respect to the elevations of the top of the bridge opening (referred to as low steel) and the top of the road grade. Free-surface flow exists when there is no contact or insignificant contact of water surface and low steel. Pressure flow through the bridge opening occurs as either (1) submerged orifice flow (when the water surface is in contact with low steel for the full flow length through the bridge) or (2) orifice flow (when only the upstream water surface is in contact with low steel). Any of these flows through the bridge opening can occur in conjunction with road overflow.

The type of flow depends on the elevation of the water surface relative to (1) the elevation of low steel in the bridge, which determines whether there will be free-surface or pressure flow through the opening, and (2) the minimum elevation along the top of the embankment, which determines whether there will be road overflow. Table 7-1 summarizes the flow classifications and the governing elevation relationships. The symbols used are defined as follows: h_{ds} and h_{us} are the water surface elevations immediately downstream and immediately upstream of the bridge,



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-11
Cross Section Locations for Water Surface Profile Computations at Single-Opening Bridges

Table 7-1
SUMMARY OF FLOW CLASSIFICATIONS FOR A SINGLE BRIDGE OPENING

a) Flow only through the bridge opening.

<table>
<tr><th>Class</th><th>Flow Class</th><th colspan="3">Relative Elevations</th></tr>
<tr><td>1</td><td>Free surface</td><td rowspan="2">$h_{ds} < Y_{ls}$</td><td>$h_{us} < Y_{ls}$</td><td rowspan="3">$h_{us} < Y_{min}$</td></tr>
<tr><td>2</td><td>Orifice</td><td rowspan="2">$h_{us} > Y_{ls}$</td></tr>
<tr><td>3</td><td>Submerged orifice</td><td>$h_{ds} > Y_{ls}$</td></tr>
</table>

b) Combination of flow through the bridge opening and weir flow over the road grade.

<table>
<tr><th>Class</th><th>Flow Class</th><th colspan="3">Relative Elevations</th></tr>
<tr><td>4</td><td>Free surface</td><td rowspan="2">$h_{ds} < Y_{ls}$</td><td>$h_{us} < Y_{ls}$</td><td rowspan="3">$h_{us} < Y_{min}$</td></tr>
<tr><td>5</td><td>Orifice</td><td rowspan="2">$h_{us} > Y_{ls}$</td></tr>
<tr><td>6</td><td>Submerged orifice</td><td>$h_{ds} > Y_{ls}$</td></tr>
</table>

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h_{ds} = Water surface elevation downstream of the bridge, in feet

h_{us} = Water surface elevation upstream of the bridge, in feet

Y_{ls} = Low steel elevation, in feet

Y_{min} = Minimum embankment elevation, in feet respectively; Y_{ls} is a low steel elevation; and Y_{min} is the minimum embankment elevation.

The computation of natural profile elevations, subscripted as h_{in} where i is the section number, should precede bridge hydraulic computations. Such data permit determination of the amount of backwater caused by the constriction and also provide an initial trial elevation in the iterative solution for the water surface profile through the bridge.

7.4.1 FREE-SURFACE FLOW

The total energy equation between the exit and approach sections (Sections 4 and 1), assuming natural profile elevation at Section 4, can be written as:

$$h_1 + h_{v1} = h_{4n} + h_{v4n} + \text{Losses}_{(1-4)} \qquad (7\text{-}15)$$

where:

h_1 = Water surface elevation at Section 1, in feet

h_{v1} = Velocity head at Section 1, in feet

h_{4n} = Natural profile elevation at Section 4, in feet

h_{v4n} = Natural profile velocity head at Section 4, in feet

$\text{Losses}_{(1-4)}$ = Summation of friction losses in the subreaches between Sections 1 and 4 plus an expansion loss between Sections 3 and 4

Friction loss in a subreach is computed as:

$$h_f = L_{av} S_f \qquad (7\text{-}16)$$

$$S_f = \frac{\left[(Q_0 + Q_1)/2\right]^2}{K_0 K_1} \qquad (7\text{-}17)$$

where:

h_f = Friction loss, in feet

L_{av} = Effective flow length, in feet (see Section 7.3.7)

S_f = Geometric mean friction slope, in feet/foot

Q_0 = Downstream discharge, in cfs

Q_1 = Upstream discharge, in cfs

K_0 = Downstream total cross-sectional conveyance (see Equation 7-3)

K_1 = Upstream total cross-sectional conveyance (see Equation 7-3)

The expansion loss in a subreach is computed as:

$$h_e = K_e \, (h_{v1} - h_{v0}) \tag{7-18}$$

where:

h_e = Expansion loss, in feet

K_e = Expansion coefficient (Range 0 to 1.0)

h_{v1} = Velocity head at Section 1, in feet

h_{v0} = Velocity head downstream, in feet

For contracting flow, the loss in a subreach is computed as:

$$h_c = K_c \, (h_{v0} - h_{v1}) \tag{7-19}$$

where:

h_c = Contraction loss, in feet

K_c = Contraction coefficient (Range 0 to 0.5)

h_{v0} = Velocity head downstream, in feet

h_{v1} = Velocity head at Section 1, in feet

Without spur dikes, the friction losses between the approach section and the upstream face of the bridge (Figure 7-11a, Sections 1 and 2) are expressed as:

$$h_{f(1-2)} = \frac{L_{av} Q^2}{K_1 K_c} \quad (7\text{-}20)$$

where:

$h_{f(1-2)}$ = Friction losses between Sections 1 and 2, in feet

L_{av} = Effective flow length, in feet (see Section 7.3.7)

Q = Design discharge, in cfs

K_1 = Conveyance at Section 1 (see Equation 7-3)

K_c = Control conveyance (see discussion below)

The value used for K_c is the minimum of the following conveyances: K_3, K_d (if dikes exist), and K_q. K_q is the conveyance of the K_q section, which is defined as that segment of the approach section that conveys discharge that can flow through the bridge opening without contraction. The horizontal limits of the K_q section are determined by projecting the bridge opening to the approach section with the projection lines oriented parallel to the general direction of flow.

When spur dikes are present, friction losses upstream from the bridge are computed separately for the two subreaches (Figure 7-11b, Sections 1 to d and d to 2) upstream from the the bridge by the equations:

$$h_{f(1-d)} = \frac{L_{av} Q^2}{K_1 K_c} \quad (7\text{-}21)$$

and

$$h_{f(d-2)} = \frac{L_{(d-2)}\ Q^2}{K_d\ K_c} \tag{7-22}$$

where:

$h_{f(1-d)}$ = Friction loss between Sections 1 and d, in feet

L_{av} = Effective flow length, in feet (see Section 7.3.7)

Q = Design discharge, in cfs

K_1 = Conveyance at Section 1 (see Equation 7-3)

K_c = Control conveyance (see discussion above)

$h_{f(d-2)}$ = Friction loss between Sections d and 2, in feet

K_d = Conveyance with spur dikes (see Equation 7-3)

$L_{(d-2)}$ = Straight-line flow distance from d to 2, in feet

The friction loss through the bridge is expressed as:

$$h_{f(2-3)} = L_{(2-3)}\left(\frac{Q}{K_3}\right)^2 \tag{7-23}$$

where:

$h_{f(2-3)}$ = Friction loss between Sections 2 and 3, in feet

$L_{(2-3)}$ = Straight-line flow distance from 2-3, in feet

Q = Design discharge, in cfs

K_3 = Conveyance at Section 3 (see Equation 7-3)

The friction loss in the flow expansion reach is computed as:

$$h_{f(3-4)} = \frac{b\ Q^2}{K_c\ K_{4n}} \quad (7\text{-}24)$$

where:

$h_{f(3-4)}$ = Friction loss between Sections 3 and 4, in feet

b = Bridge-opening length, in feet

Q = Design discharge, in cfs

K_c = Control conveyance (see discussion above)

K_{4n} = Natural profile conveyance at Section 4 (see Equation 7-3)

The expansion loss from Section 3 to 4 is computed by the equation:

$$h_e = \frac{Q^2}{2g\ A_4^2}\left[2\beta_4 - \alpha_4 - 2\beta_3\left(\frac{A_4}{A_3}\right) + \alpha_3\left(\frac{A_4}{A_3}\right)^2\right] \quad (7\text{-}25)$$

where β is a momentum correction factor for nonuniform flow distribution. α_4 and β_4 are computed as:

$$\alpha_4 = \frac{\Sigma\ (K^3/a^2)}{K^3\ A^2} \quad (7\text{-}26)$$

$$\beta_4 = \frac{\Sigma\ (k^2/a)}{K^2/A} \quad (7\text{-}27)$$

where lower case and upper case indicate subsection and total section properties. α_3 and β_e are related to bridge geometry and are computed as:

$$\alpha_3 = \frac{1}{C^2} \quad (7\text{-}28)$$

$$\beta_3 = \frac{1}{C} \quad (7\text{-}29)$$

where:

C = The coefficient of discharge for the bridge

7.4.2 ORIFICE FLOW

If the water surface is in contact only with the upstream girders, the water surface profile through the bridge can be determined with an orifice flow equation expressed as:

$$QBO = C_D A_{3net} \sqrt{2g\ (Y_u - Z/2 + h_{v1})} \tag{7-30}$$

where:

QBO = Orifice flow through the bridge opening, in cfs

C_D = Discharge coefficient

A_{3net} = Net area (total minus area of piers or piles) in the bridge opening for an elevation of h_{us} (Equation 7-31), in square feet

Y_u = Average upstream depth above the streambed, in feet

Z = Height of the bridge opening above the streambed, in feet

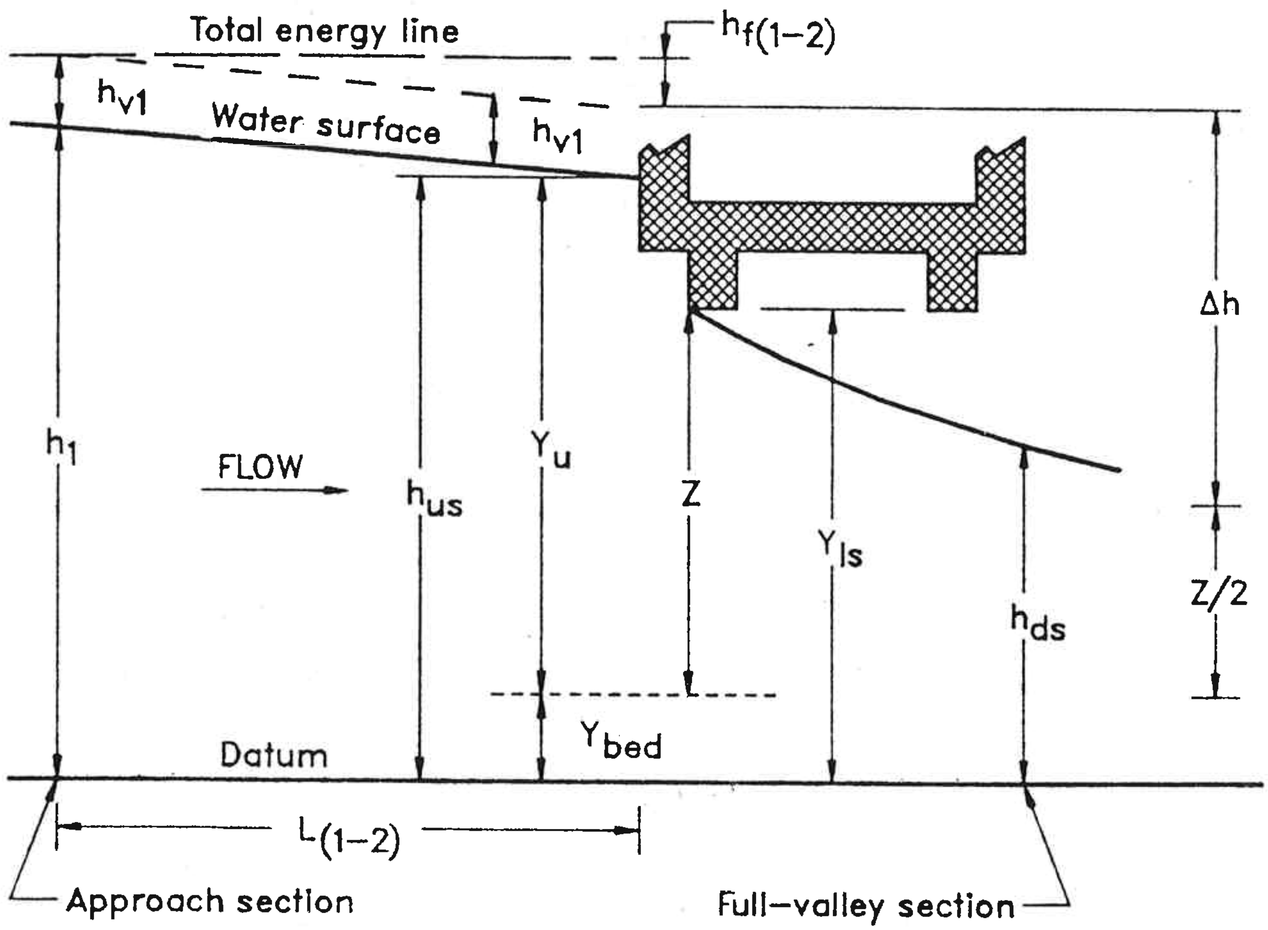
h_{v1} = Velocity head at the approach Section 1, in feet

g = Acceleration due to gravity, 32.2 feet/second^2

Figure 7-12 illustrates the definition of the variables involved in this computation. The water surface elevation immediately upstream from the bridge, h_{us}, is computed as:

$$h_{us} = h_1 - h_{f(1-2)} \tag{7-31}$$

where:



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-12
Definition Sketch for Orifice Flow Computations

h_{us} = Water surface elevation immediately upstream from the bridge, in feet

h_1 = Water surface elevation at the approach Section 1, in feet

$h_{f(1-2)}$ = Friction loss in the approach reach, in feet

The friction-loss term $h_{f(1-2)}$ is computed using Equation 7-20 with K_c set to equal the bridge-opening conveyance, K_3. The velocity head at the approach section, h_{v1}, is assumed to be applicable throughout the approach reach. The submergence elevation, Y_{1s}, is either computed from input data defining bridge deck elevation and girder depth or specified by the user. The hydraulic depth, Z, within the bridge opening is computed as:

$$Z = A_{3net}/b \tag{7-32}$$

and the reference bed elevation (Y_{bed}) is estimated as:

$$Y_{bed} = Y_{1s} - Z \tag{7-33}$$

and the average upstream depth is:

$$Y_u = h_{us} - Y_{bed} \tag{7-34}$$

where:

Z = Height of the bridge opening above the streambed, in feet

A_{3net} = Net area (total minus area of piers or piles) in the bridge opening for an elevation of h_{us} (Equation 7-31), in feet

b = Bridge-opening length, in feet

Y_{bed} = Reference streambed elevation, in feet

Y_{ls} = Elevation of low steel for the bridge, in feet

Y_u = Average upstream depth above the streambed, in feet

7.4.3 SUBMERGED ORIFICE FLOW

Flow through the bridge is handled as submerged orifice flow when the water surface is in contact with the girders for the entire flow length through the bridge (see Figure 7-13). Discharge through the opening for such a case is computed as:

$$QBO = C_D \, A_{3net} \sqrt{2g\Delta h} \qquad (7\text{-}35)$$

$$\Delta h = h_{us} + h_{v1} + h_{3n} \qquad (7\text{-}36)$$

where:

QBO = Submerged bridge opening discharge, in cfs

C_D = Discharge coefficient

A_{3net} = Total net flow area in the bridge opening, in square feet

Δh = Head differential, in feet

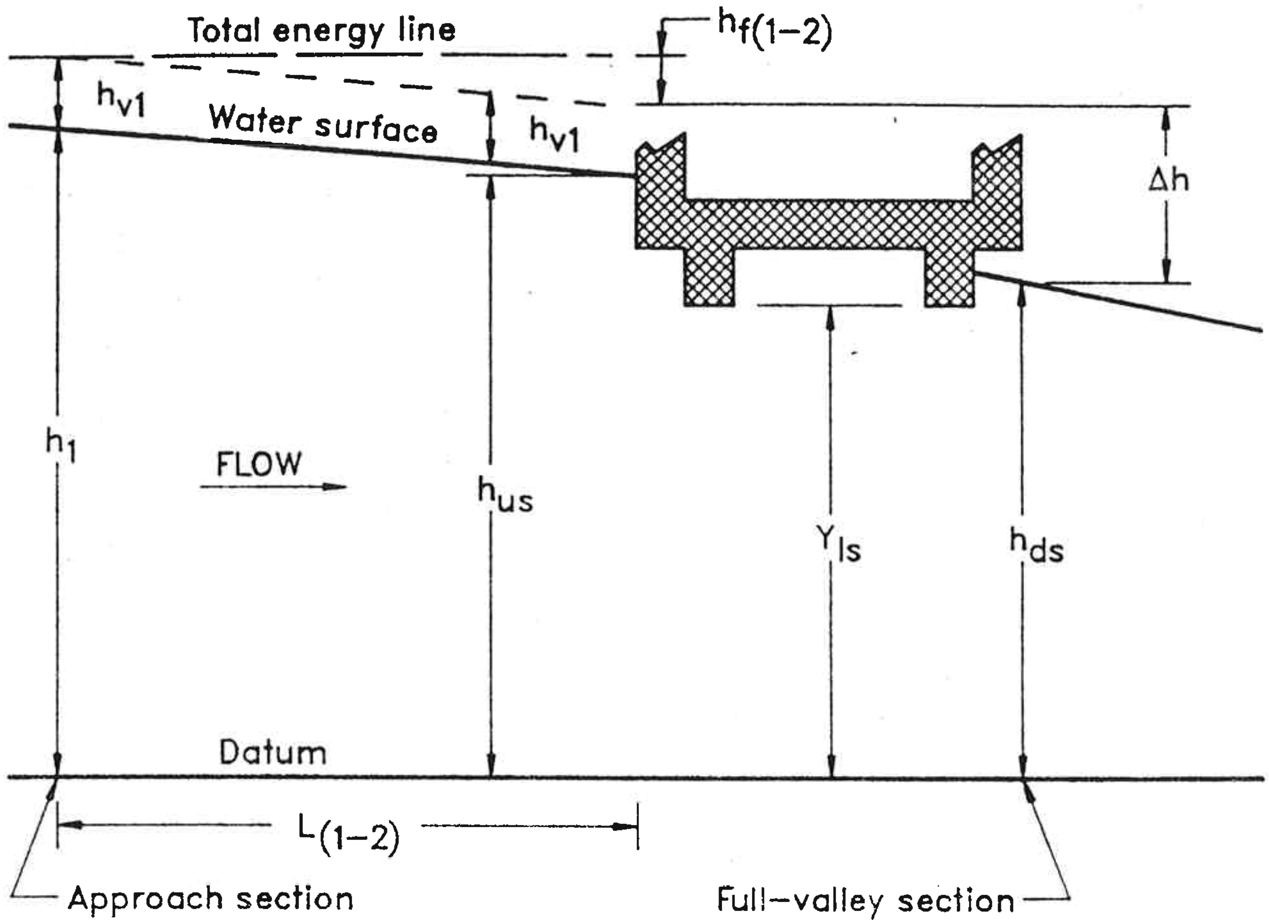
h_{us} = Upstream elevation, in feet

h_{v1} = Velocity head at Section 1, in feet

h_{3n} = Natural profile elevation at Section 3, in feet

Based on laboratory data, a constant value of C_D equal to 0.8 can be used to give the following form of Equation 7-35:

$$QBO = 6.42 \, A_{3net} \sqrt{\Delta h} \qquad (7\text{-}37)$$



Reference: USDOT, FHWA, HY-7 (1986).

FIGURE 7-13
Definition Sketch for Submerged Orifice Flow Computations

7.4.4 ROADWAY OVERTOPPING

When the water surface elevation immediately upstream from the embankment exceeds the minimum elevation along the top of the embankment, the embankment begins to function as a broad-crested weir. Fundamentals of this situation are covered in Chapter 5 on culvert hydraulics.

7.5 SCOUR AND DEGRADATION

Three types of interrelated phenomena should be considered when evaluating changes to bed level related to bridges: general scour, local scour, and degradation or aggradation.

7.5.1 GENERAL SCOUR

Scour occurs at contractions because the flow area becomes smaller than the normal stream and the average velocity and bed shear stress increase. As a result, the transport capacity of the stream increases at the contraction and more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, the velocity and shear stress decrease and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate.

General scour can be evaluated by considering the sediment transport and hydraulic properties in the vicinity of the contraction. The magnitude of general scour should be quantified before performing local scour calculations to establish an appropriate reference elevation.

Ignoring scour in water surface profile computations will give conservative backwater elevations, because scour can increase the waterway area of a bridge. A detailed procedure is presented in HDS-1 (USDOT, FHWA, 1978) for evaluating backwater elevations when the potential for scour exists. Although the scour-related reduction of backwater can be estimated, a design should not depend on scour as a means of enlarging the bridge waterway area and thereby lowering backwater.

An important consideration when the potential for scour is significant is the stability of bridge abutments and piers. A careful evaluation of velocity distributions and the stability of the streambed and embankment materials should be performed to verify that bridge abutments and piers are safe for selected design conditions. Spur dikes can be sized to provide additional stability.

7.5.2 LOCAL SCOUR

Local scour occurs in the bed of the channel around piers and embankments and is caused by the action of vortex systems induced by obstructions to flow. Local scour generally occurs independent of degradation, aggradation, and scour caused by contractions.

The basic mechanism causing local scour is the vortex of fluid resulting from the pileup of water on the upstream edge and subsequent acceleration of flow around the nose of the pier or embankment. The vortex erodes bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scourhole develops. As the depth is increased, the strength of the vortex is reduced, the transport rate is reduced, equilibrium is reestablished, and scouring ceases.

The presence or absence of sand bars can affect the depth of local scour. Because the time required for sand bar motion is much longer than the time required for local scour, even if steady-state conditions exist, the depth of scour is likely to fluctuate with time when there are sand bars traveling on the channel bed. When the crest of the sand bar reaches the local scour area, the transport rate into the hole increases, the scourhole fills, and the scour depth temporarily decreases. When a trough approaches, there is less sediment supply and the scour depth increases to try to reestablish equilibrium in sediment transport rates. A mean scour depth between these oscillations is referred to as <u>equilibrium scour depth</u>. It is not uncommon (as determined in laboratory tests) to find maximum depths to be 30 percent greater than equilibrium scour depths. The depth that would be reached if no sediment was transported into the scourhole is the <u>clear water scour depth</u>.

The horseshoe vortex system is dominant at piers, causing deepest scour at the nose of the pier. The axis of this vortex, or the vortex line, is horizontal and wraps around the base of the pier in the shape of a horseshoe. The high velocities scour the bed.

A wake vortex system has vertical axes and develops because of blockage of the flow by the pier. The wake vortices are commonly seen as eddies. This vortex system suspends the scoured material and carries it downstream with the flow. Downstream of embankments, large wake vortices or eddies are set up that scour the downstream sides of the embankments, the river bank, and the streambed. Wake vortices downstream of piers may create sufficient velocities to cause bed scour if the piers are wide. For most piers, however, very little additional scouring is caused by wake vortices.

The shape of the pier is very significant with respect to scour depth because it reflects the strength of the horseshoe vortex at the base of the pier. A blunt-nose pier causes the greatest scour depth. Streamlining the front end of the pier reduces the strength of the horseshoe vortex and thus reduces the scour. Streamlining the downstream end of piers reduces the strength of wake vortices.

Detailed studies of scour around embankments have been made mainly in laboratories. There are very few case studies for scour at field installations. Empirical procedures based on the average velocity and local depth of flow can be used to estimate local scour.

7.5.3 DEGRADATION AND AGGRADATION

Many rivers have achieved a state of practical equilibrium throughout long reaches that can be considered stable; these rivers are known as graded streams by geologists and as poised streams by engineers. However, this does not preclude significant changes over a short period of time or over a period of years. Conversely, many streams contain long reaches that are actively aggrading and degrading. These aggrading and degrading channels may pose a definite hazard to any highway crossing or encroachment.

Regardless of natural channel stability conditions, man's local activities may produce major changes in river characteristics locally and throughout the entire reach. All too frequently, the result of a river improvement is a greater departure from equilibrium than that which originally prevailed. As an example, the channel downstream from an impoundment is likely to undergo degradation from sediment removal by storage, while the channel upstream from an impoundment is likely to experience aggradation caused by reduced velocities. These potential changes in the channel profile should be accounted for when new facilities are designed.

Good design should enhance the natural tendency of the stream toward poised conditions. To do so, an understanding of the direction and magnitude of change in channel characteristics caused by the actions of man and nature is required. This understanding can be obtained by taking the following actions:

1. Studying the river in a natural condition

2. Having knowledge of the sediment and water discharge

3. Being able to predict the effects and magnitude of man's future activities

4. Applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers

Aggradation and degradation processes related to channel improvements are also discussed in Chapter 3. More detailed information can be found in the Training and Design Manual by the USDOT, FHWA (1975).