Chapter 6
Steady Flow - Water Surface Profiles

Section I
Introduction

6-1. Scope

This chapter is limited to a discussion of calculating rigid boundary, steady-flow, water-surface profiles. The assumptions, equations, and general range of application are presented in this section; data requirements, model development, special problems, and an example calculation follow in subsequent sections.

6-2. Assumptions of the Method

Computer programs used to compute steady, gradually varied flow water surface profiles are based on a number of simplifying assumptions. A thorough understanding of these assumptions is required before an adequate model of a study reach can be developed. Considerable engineering judgment is required in locating cross sections and preparing input data. The assumptions and how they affect program application follow:

a. Steady flow. Depth and velocity at a given location do not vary with time. This assumption requires that the flow remain constant for the length of time being considered. Of course, for natural rivers this condition does not hold true precisely. However, it is usually acceptable for general rainfall and snowmelt floods in which discharge changes slowly with time. For such floods, a person standing on the bank of a stream during a flood would most likely not perceive the vertical movement or curvature of the water surface.

b. Gradually varied flow. The depth and velocity change gradually along the length of the watercourse. These conditions are valid for most river flows, including floods, and the assumption of a hydrostatic pressure distribution (associated with gradually varied flow) is reasonable as long as the flow changes are gradual enough so that the imaginary lines of flow are approximately parallel.

c. One-dimensional flow. Variation of flow characteristics other than in the direction of the main axis of flow may be neglected and a single elevation represents the water surface of a cross section perpendicular to the flow. Thus, velocities in directions other than the direction of the main axis of flow and effects due to centrifugal force at curves, are not computed. A correction factor is applied to account for the horizontal velocity distribution.

d. Small channel slope. The stream channel must have a slope of 1 in 10 or less. Small slopes are required because of the assumption that the hydrostatic pressure distribution is computed from the depth of water measured vertically. For a bed slope of 1:10, which is steep for a natural channel, measuring the depth vertically results in an error of only one percent. Most floodplain studies are performed on streams that meet this requirement.

e. Rigid boundary. The flow cross section does not change shape or roughness during the flood. While this assumption is generally used, many alluvial streams may undergo considerable change in the shape of the bed and banks during a major event.

f. Constant (averaged) friction slope between adjacent cross sections. Approximation of the friction loss between cross sections can be obtained by multiplying a representative friction slope by the reach length that separates them. Various approximating equations are used to determine the friction slope. For example, in HEC-2 four equations are available, designated as average conveyance, average friction slope, geometric mean friction slope, and harmonic mean friction slope (U.S. Army Corps of Engineers 1990b). This assumption requires that cross section spacing and the selection of an appropriate friction-slope equation for computing the loss be governed by conditions in the reach.

6-3. Standard-step Solution

In open channel flow, the potential energy, Z, is specified as the height of the solid boundary confining the flow above some datum. If the pressure distribution is hydrostatic, the pressure energy, P/γ, is the depth of water above the solid boundary. These two energy terms can be added to obtain

$$WS = P/\gamma + Z \tag{6-1}$$

where WS is the water surface elevation above the datum, as shown in Figure 6-1. The equation can then be rewritten

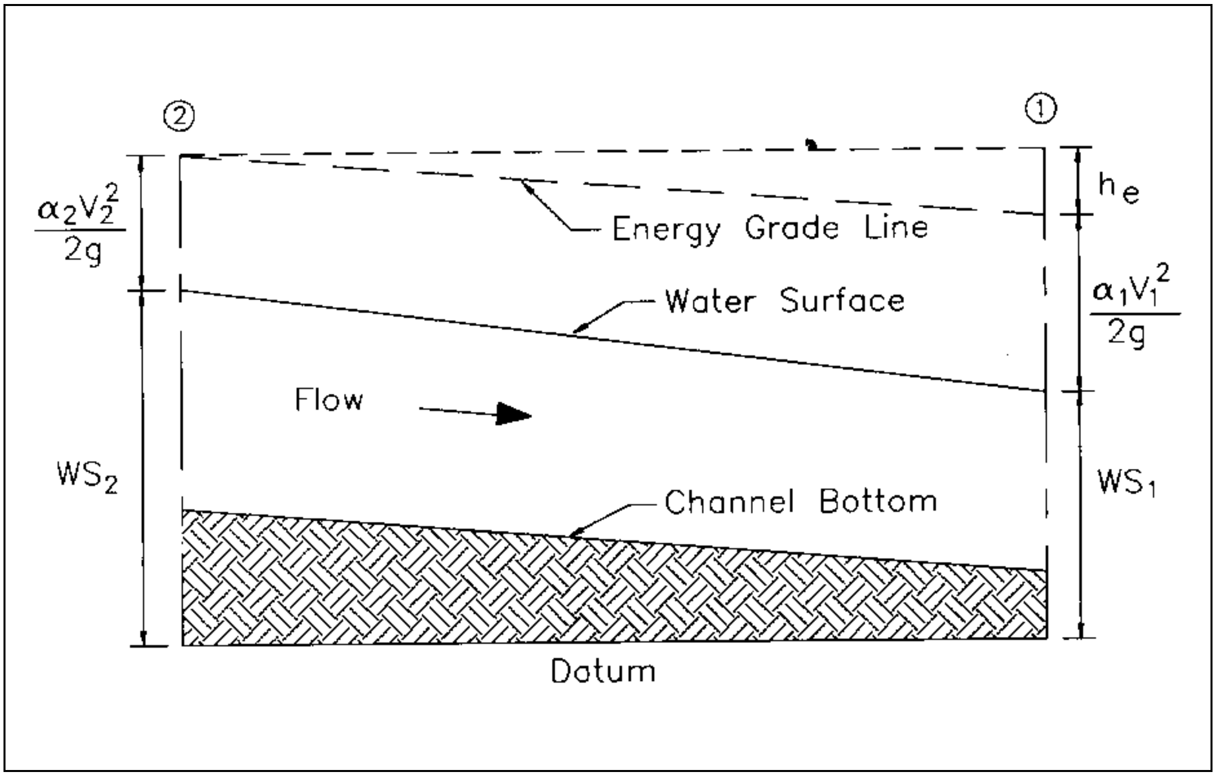

Figure 6-1. Open channel energy relationships

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \tag{6-2}$$

An equation for the energy head loss h_e can be written as follows

$$h_e = L\bar{S}_f + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \tag{6-3}$$

where

L = discharge weighted reach length
$\bar{S}_f$ = representative friction slope for reach
C = expansion or contraction loss coefficient

The solution of Equation 6-2 is the basis of water surface profile computations in programs such as HEC-2. The standard step method used to obtain a solution requires successive approximations. A trial value of WS_2 in Equation 6-2 is assumed, and values for h_e and change in velocity head are computed and summed to obtain ΔWS. This value is added to the known downstream water surface elevation to compute WS_2. The difference between trial and computed values converges with successive trials. The steps in this procedure are as follows:

a. Assume a water surface elevation at the upstream cross section (or downstream cross section if a supercritical profile is being calculated).

b. Based on the assumed water surface elevation, determine the corresponding total conveyance and velocity head.

c. With values from step 2, compute $\bar{S}_f$ and solve Equation 6-2 for h_e.

d. With values from steps 2 and 3, solve Equation 6-2 for WS_2.

e. Compare the computed value of WS_2 with the values assumed in step 1; repeat steps 1 through 5 until the values agree to within .01 feet (or .01 meters).

6-4. Range of Applicability

The assumptions of the method as described in section 6-2 are the basis for determining applicability. Their effects in modeling are as follows:

a. Steady flow. This assumption generally is not a significant problem. For most naturally occurring floods on major streams, flow changes slowly enough with time that steady flow is a fair assumption. Even when it is not, the assumption would seldom cause any computational problems. Three conditions under which a steady-flow model may not be applicable are:

(1) A rapidly moving flood wave, as from a dam breach, for which the time-dependent term of the full unsteady-flow Equation has a significant effect.

(2) Backwater effects from a downstream boundary condition, such as a tidal flow, are significant.

(3) A flat channel slope resulting in a pronounced loop effect in the relationship between discharge and elevation. See Chapter 5 for more information.

b. Gradually varied flow. This is a reasonable assumption in most river reaches that are free of structures and severe changes in channel geometry; however, this may not be a valid assumption in the vicinity of structures such as bridges and channel controls. The estimation of energy losses and the computation of water surface elevations in rapidly changing flow become more uncertain. Under these conditions, the estimated energy loss may be too high or too low, or the computational process may not be able to determine a water surface elevation based on computed energy losses, and a critical depth is assumed. For most floodplain studies, the critical depth solution is not valid. A critical depth solution at a cross section will not provide a basis for computing a floodway encroachment based on a change of water surface elevation.

c. One-dimensional flow. This may not always be a valid assumption. Two major problems that violate the assumption of one-dimensional flow are multiple water surface elevations and flow in multiple directions.

(1) Multiple water surface elevations within one cross section usually result from multiple flow paths. When the flow in each path is physically separated from the other paths, the distribution of flow in each path is a function of the conveyance (or energy loss) through the length of that path. Because the one-dimensional model distributes flow in each cross section based on the conveyance in that cross section, the flow distribution in the model is free to shift from cross section to cross section in the computational process. The traditional solution to the problem is to divide the model into the separate flow paths and compute a profile for each (see Chow 1959, Sec. 11-9).

(2) Flow in multiple directions cannot easily be modeled with a single cross section perpendicular to the flow. In cases where the flow is gradually expanding, contracting, or bending, a cross section generally can be defined that will reasonably meet the requirement, but it does take special care. When flow takes a separate path, as in the case of a levee overflow or a side diversion, the flow lost from the main channel must be separately estimated and subtracted from the main channel flow. The HEC-2 program has a split flow option to compute lateral flow losses and the resulting profile in the main channel (U.S. Army Corps of Engineers 1982a).

d. Small channel slope. This condition is common in natural streams. A slope less than 1 in 10 means that the pressure correction factor is close to 1 and not required. Also, the depth of flow is essentially the same whether measured vertically or perpendicular to the channel bottom (Chow 1959). For most valley streams where floodway computations are performed, a 1 in 10 slope would be considered steep. Channel slopes are usually less than 1 in 100.

e. Rigid boundaries. This assumption requires that the channel shape and alignment be considered constant for the period of analysis. The concern is not with long term changing boundaries, like those on meandering rivers, but with local scour and deposition that can occur in a stream during a flood event. The problem is more pronounced at major contractions, such as bridge crossings, because there is an increase in velocity with the potential for increased scour. Guidelines for determining critical scour velocities can be found in design criteria for stable channels of EM 1110-2-1610.

6-5. Example of Steady Flow Water Surface Profile Study

a. Study objective. The overall objective was a comprehensive reanalysis of water surface profiles for a reach of the Tug Fork River in the Williamson, West Virginia, flood protection project area (Williams 1988a, 1988c).

b. Description of the study reach. The Tug Fork River originates in the southern part of West Virginia and flows for 155 miles in a northeasterly direction to Louisa, Kentucky, where it joins the Big Sandy River.

(1) In the headwater regions the terrain is mountainous, but in the lower reaches, the valleys are wide and the hills gentle and rounded. Through most of the area, the river flows in deep, narrow, sinuous valleys between steep side ridges. Williamson is located in the lower third of the Tug River Basin, where the valley is 800 to 900 feet wide.

(2) The original water surface profile study reach extended from Kermit, West Virginia, to the central business district of Williamson, a distance of 20 miles. The general slope of this reach is about 2 feet per mile.

(3) The channel is alluvial with a bottom width of about 150 feet and stable banks with heights ranging up to 25 feet above low water. Bed sediments are sand and gravel. Vegetation, predominately conifer, lines both banks and covers the floodplain except where cleared for agricultural or industrial use.

c. Summary of water surface profile model and parameter evaluations. Refinements to the original HEC-2 data file included substituting field data at bridges, developing reach lengths, and assigning Manning's roughness coefficients by vegetation and land use. Channel bank limits were reestablished to better approximate the limits of bank vegetation.

(1) Sensitivity of calculated profiles was evaluated to determine the significant hydraulic parameters. Superelevation, bed scour during floods, local inflows, overbank flows, relative roughness, and seasonal vegetation roughness were analyzed. Key sources of field data for these evaluations were high-water marks from 1984 and 1977 floods and USGS gage records at Williamson.

(2) Some of the results from these evaluations were bed scour during these events was found to be negligible, superelevation did not impact except to indicate that the calibration tolerance should be relaxed from 0.5 foot to 1 foot, and local inflow changes improved agreement between calculated and observed profiles between gages.

(3) The three most significant hydraulic parameters were the identification of significant overbank flow through the town of Williamson, changes in the values of roughness as rare flood events overtopped all trees, and seasonal changes in vegetative roughness.

(4) The maximum discharge during the 1977 event was so significant that two extrapolations were made, one for a 94,000 cfs event and one for a 117,000 cfs event. The procedure for extrapolating the rating curves followed EM 1110-2-1601 which utilizes "relative roughness" and uses observed data to calculate roughness height. The details of the extrapolation procedure and other details of the study are presented in Williams (1988a, 1988c). Calibration of the HEC-2 model to the two flood events is discussed in a later section under the heading "Model Calibration and Verification" (6-11).

Section II
Data Requirements

6-6. Introduction to Data Requirements

The time and effort required for completion of water surface profile studies depend upon the detail of the analysis required to secure the results desired. In some cases the character of available basic data and the time available impose practical limitations on the scope of the study. In preliminary investigations a rapid approximate method may give results fully as satisfactory for the purpose involved as a more accurate but time consuming computational procedure. In other cases, the utmost degree of accuracy possible by a detailed and thorough analysis may be profitable and essential for reliable engineering. Accordingly, profile computations should be initiated with a careful appraisal of the degree of accuracy necessary for satisfactory results, considering the purpose and character of the investigations involved, the detail and probable accuracy of basic data available, the complexity of flow conditions in the stream, and the budget and time limit for completion of the studies.

a. Theory. Hydraulic theory is well established for channels with rigid boundaries, and computer simulation models based on this theory produce consistent and accurate results if properly applied. Major sources of error are inaccuracies in data and improper modeling of flow conditions.

b. Categories of data. Basic data are grouped into five categories: cross sections, reach lengths, loss coefficients, flow regime, and starting condition. The accuracy required for this data depends upon the accuracy needed in the final results. At times, it seems most economical to compensate for inadequacy of data by using safety factors such as providing liberal amounts of freeboard. In rural areas such procedures may be acceptable, but in urban areas both property damage and loss of life can result from designs based on inadequate and inaccurate data. Cross-sectional data and loss coefficients are discussed in Appendix D.

6-7. Flow Regime

Water surface profile computations begin at a cross section with known or assumed starting conditions and proceed upstream for subcritical flow or downstream for supercritical flow. Subcritical profiles computed by a program such as HEC-2 are constrained to critical depth or above, and supercritical profiles are constrained to critical depth and below. The program will not allow profile computations to cross critical depth except for certain bridge-analysis problems. When flow passes from one flow regime to the other, it is necessary to compute the profile twice, alternately assuming subcritical and supercritical flow (U.S. Army Corps of Engineers 1990b).

6-8. Starting Conditions

If feasible, profile computations should be started at a point of control where the water surface elevation can be definitely determined. This may be at a gaging station, a dam, or a section where flow is at critical depth. However, for practical reasons, it is often necessary to start the computations at other locations.

a. Known elevation. When a profile computation begins at a dam or a gaging station on a river where the water-surface elevation versus discharge relationship is known and is applicable to the conditions for which a profile is desired, the starting elevation can be determined from a rating curve. A common situation of this type involves the computation of a water surface profile starting at a full-pool elevation of a reservoir with a specified discharge through or over the dam.

b. Critical depth. In certain instances it may be feasible to start computations from a point where it is known that critical depth will occur. Critical depth in rivers may occur where the channel slope steepens abruptly, or at a natural constriction in the channel. Critical depth may be produced artificially by structures that raise the channel bottom or constrict the channel width. If a critical depth location can be determined, the critical depth option for determining the starting elevation can be specified in input to a program like HEC-2, and it will compute the critical depth and use it.

c. Uniform flow. If the assumption of uniform flow is reasonable, the slope-area method may be used to find a starting elevation based on the computation of normal depth. If an estimate of the slope of the energy grade line and an initial estimate of the starting water surface elevation are input to HEC-2 at a given cross section, the program will do a normal-depth calculation automatically. It will compute the discharge for the initial conditions, and compare it with the given discharge. If there is a significant difference, it will adjust the depth and repeat the computation in a series of iterations until a 1 percent difference criterion is met for the computed and given discharges.

d. Estimated slope. When the starting elevation for a selected discharge cannot be determined readily, it is necessary to derive a starting elevation using available expedients. One method is to select a water-surface slope on a similar stream(s), and solve Manning's Equation by trial-and-error or graphically for the water-surface elevation necessary to give that slope.

e. Estimated stage. Another method is to begin profile computations using a trial starting elevation at a location some distance downstream from the reach for which the backwater curve is desired. The error resulting from an incorrectly assumed trial starting elevation will tend to diminish as the computation progresses upstream. The distance downstream can be estimated from the regression equations presented in "Accuracy of Computer Water Surface Profiles" (U.S. Army Corps of Engineers 1986). Equations are presented for both critical and normal depth starting assumptions. The impact of the starting depth assumption can be tested by computing a second profile beginning at the same downstream location but at a different trial starting elevation. The starting assumption is reasonable if the two corresponding backwater curves merge into one before the computations have progressed to the reach for which the backwater curve is desired. In selecting the trial starting elevations, one elevation should be below and the other above the true elevation.

f. Tidal conditions. When the profile computation begins at the outlet of a stream influenced by tidal fluctuations, the maximum predicted high tide, including wind-wave set up, is taken as the starting elevation at a station usually located at the mouth of the stream.

Section III
Model Development

6-9. Data Sources

Data requirements for water surface profile computations were discussed in the preceding section. To reiterate, the following data are required: discharge, flow regime, starting water surface elevation, roughness and other energy loss coefficients, and the geometric data--cross sections and reach lengths. Sources for geometric data and energy loss coefficients are discussed in Appendix D. Sources for the remaining items are discussed here.

a. Discharge. The discharge used in water surface profile computations is generally the peak discharge associated with a given frequency. For example, in a multiple-profile analysis for a floodplain-information study, peak discharges for the 10-, 50-, 100-, and 500-year events may be required. Peak discharges are generally obtained from flood-frequency analysis or from the application of historical or design storm precipitation data to rainfall-runoff models such as HEC-1.

b. Flow regime. Since water surface profile computations in a model such as HEC-2 do not cross critical depth, it is necessary at the outset of an analysis to decide whether to analyze the flow as subcritical or supercritical. The flow regime is subcritical in most rivers; however, if this assumption is used and is incorrect, program output will indicate that a wrong decision may have been made. Critical depth will be assumed and noted in the output for cross sections in the model where the regime is different from that assumed. For reaches in which flow actually passes from one regime to the other, it may be necessary to make a separate computation for each regime and combine the results for a complete analysis.

c. Starting water surface elevation. Alternative methods for determining the starting water surface elevation are discussed in the preceding section on data requirements.

6-10. Data and Profile Accuracy

It would seem, from the list of suggested cross-section locations in Appendix D, that the effects of most undesirable features of nonuniform, natural stream channels can be lessened by taking more cross sections. While this is generally true, time, cost, and effort to locate and survey cross sections must also be considered. A balance must be set between the desirable number of cross sections and the number that is practical. Accuracy of the data and the profiles should be part of the balance consideration.

a. Associated error. Errors associated with computing water surface profiles with the step-profile method can be classified as basic theory, computational, or data estimation (McBean and Pernel 1984).

(1) Minimizing error in the application of theory is the responsibility of the engineer conducting the study.

(2) Computation errors include numerical round-off and numerical solution errors. The former is negligible using today's modern computers and the latter can be minimized by employing readily available mathematical solution techniques.

(3) Data estimation errors may result from incomplete or inaccurate data collection and inaccurate data estimation. The sources of data estimation errors are the accuracy of the stream geometry and the accuracy of the method used and data needed for energy loss calculations. The accuracy in stream geometry as it affects accuracy of computed profiles is important. The accuracy of energy loss calculations depends on the validity of the energy loss Equation employed and the accuracy of the energy loss coefficients. The Manning Equation is the most commonly used open channel flow Equation and the coefficient measuring boundary friction is Manning's *n*-value.

b. Accuracy of data collection and estimation.

(1) Aerial survey and topographic map accuracy. Stream cross-sectional geometry obtained from aerial surveys (aerial spot elevations and topographic maps) that conform to mapping industry standards are more accurate than is often recognized. Cross-sectional geometry obtained from aerial spot elevation surveys is twice as accurate as cross-sectional geometry obtained from topographic maps derived from aerial surveys for the same contour interval.

(2) Profile accuracy prediction. The effect of aerial spot elevation survey or topographic mapping accuracy on the accuracy of computed water surface profiles can be predicted using the mapping industry accuracy standards, reliability of Manning's coefficient, and stream hydraulic variables.

(3) Manning's coefficient estimates. The reliability of the estimation of Manning's coefficient has a major impact on the accuracy of the computed water surface profile. Significant effort should be devoted to determining appropriate Manning's coefficients.

(4) Additional calculation steps. Significant computational errors can result from using cross-sectional spacings that are often considered to be adequate. The

errors are due to inaccurate integration of the energy loss-distance relationship that is the basis for profile computations. This error can be effectively eliminated by adding interpolated cross sections (more calculation steps) between surveyed sections.

(5) Aerial survey procedures. Aerial spot elevation survey methods are more cost effective than field surveys when more than 15 survey cross sections are required. Use of aerial spot elevation survey technology permits additional coordinate points and cross sections to be obtained at small incremental cost. The coordinate points may be formatted for direct input to commonly used water surface profile computation computer programs.

c. Errors in the data.

(1) Profile errors resulting from use of commonly applied field survey methods of obtaining cross-sectional coordinate data are a function only of Manning's coefficient of reliability "Nr" (U.S. Army Corps of Engineers 1986). Computed profile error resulting from survey error is small even for rough estimates of Manning's coefficient.

(2) Profile errors resulting from use of aerial spot elevation surveys for obtaining cross-sectional coordinate data vary with the contour interval and reliability of Manning's *n*-value.

(3) The small profile error for the aerial spot elevation survey method is due to the high accuracy of aerial spot elevation surveys and the randomness of the measurement errors at the individual coordinate points. The latter results in compensating errors along the cross-sectional alignment. For the error prediction determined from the regression Equations to be valid, eight or more cross-sectional coordinate points are needed to ensure that the randomness and thus compensatory error process has occurred.

(4) The error in computed water surface profiles increases significantly with decreased reliability of Manning's coefficient. The profile errors resulting from less reliable estimates of Manning's coefficient are several times those resulting from survey measurement errors alone.

(5) There is significantly greater error for larger contour intervals for topographic maps than for aerial spot elevation surveys. Data from topographic maps are simply less accurate. Also, topographic map cross-sectional elevations can only be obtained at the contour intervals. Significant mean profile errors (greater than 2 feet) may be expected for analyses involving steep streams, large contour intervals, and unreliable estimates of Manning's coefficients.

(6) The error prediction Equations in "Accuracy of Computed Water Surface Profiles" (U.S. Army Corps of Engineers 1986) may be used to determine the mapping required to achieve a desired computed profile accuracy.

6-11. Model Calibration and Verification

a. Calibration. The goal of calibration is to obtain a set of parameters for a model so that it will respond like the physical system it represents. A calibrated steady-flow water surface profile model should compute water surface elevations that are essentially the same as observed elevations (from high water marks or gage readings) not only for the set of conditions used in calibration but for others as well. This is accomplished with a trial-and-error procedure in which a water surface profile is computed with an initial set of parameters and compared to the observed data. The parameters are adjusted on the basis of the comparison, and the procedure is repeated until a suitable fit is obtained.

b. Verification. Verification is closely akin to calibration in that it, too, amounts to the comparison of computed model output to observed data. The distinction between the two procedures is usually made on the basis of timing and the different data sets involved. A model is first calibrated to one set of observed data and then verified with another set.

c. Factors in reconciling differences. Several factors that might be considered in reconciling differences between computed and observed data (Hoggan 1989) are as follows:

(1) There is usually some leeway in assigning *n* values, and these might be adjusted upward or downward slightly to achieve a better fit of computed and observed data.

(2) The reliability of the discharge values from a hydrologic model or other sources might be questioned. If differences in computed and observed profiles are great (a few feet or more), erroneous discharge values might be the problem, and this possibility should be investigated.

(3) Even though the precision of survey data is usually not a problem (as discussed in 6-10*c*), major

errors in survey data can occur, having significant impact on the accuracy of computed profiles, and may warrant checking.

(4) At some locations changing the bridge method used in the model may improve the computed profile.

(5) If a high water mark is unusually high at a bridge, it may have resulted from a snag or debris caught on the piers. A dam failure or diversion upstream can also abnormally affect high water marks.

(6) The replacement of a bridge, channel modifications, construction of encroachments, and development of adjacent land since the water marks were made would complicate calibration and verification.

(7) Questionable data are always a possibility. For example, inaccurate rainfall data could cause discharge values to be off, and information from local residents regarding high water marks may be in error.

d. Other considerations. Other considerations for the evaluation of the high water marks (Williams 1988b) are as follows:

(1) Looped rating curves. Some rivers exhibit a looped rating curve which indicates that for a given depth the discharge will be greater on the rising stage of a flood than on the falling stage. This leads to the maximum water surface elevation not corresponding to the peak discharge, and can result in calibrating a model to high water marks that are not consistent with the given discharge.

(2) Superelevation. Sometimes high water marks are taken at curves on a river in which the water surface is superelevated at the outside of a bend. Because a one-dimensional steady-flow model assumes a horizontal water surface, the computed elevation must be adjusted for this superelevation before it is compared with high water marks.

(3) Waves and "set up". If a debris line is used to determine high water marks, it may be higher than the actual water surface elevation because of the effect of waves. Errors can occur from water-momentum changes which result in a "set up" of the water surface elevation. This may occur if the debris line is not parallel to the flow, if the flow must make an abrupt change in direction, or at "dead end" areas.

(4) Backwater areas. If water surface elevations are affected by backwater, high water marks will be higher than normal-depth elevations. The effects of the backwater can be determined by varying the downstream control in the model. By using the downstream elevations required to match the high water marks, it can be determined if these elevations are within the expected downstream elevation range. This problem usually arises for a study reach on a tributary at a location near the confluence of the tributary with the main stream. If channel modifications on the tributary affect the downstream control, the calibrated *n* value for a given discharge may no longer be valid.

e. Adjusting n. Several suggestions for adjusting *n* values in the calibration process (Williams 1988a, 1988c) are as follows:

(1) Flow resistance caused by vegetation can vary due to the depth of flow, vegetative stand characteristics (see Figure 6-2), and amount of foliage. Differences in seasonal foliage may need to be considered when calibrating events that occur at different times of the year.

(2) Flow resistance is affected by bedforms and surface (or grain) resistance. Simons and Richardson (1966) describe the types of bedforms and their relative resistance (Figure 6-3). Brownlie (1981) has developed a flow resistance relationship which takes into account both the surface and the bedform. This should be used only in the alluvial portion of a river.

(3) A compound channel is one with laterally varying roughness and flow depth, as depicted in Figure 6-4. If compound channel subsections influence each other's flow by phenomenon such as momentum exchange between subsections, a composite *n* is recommended because each subsectional roughness height does not change appreciably with flow depth, but the composite height does (and so does the composite *n*). See EM 1110-2-1601, Appendix IV for details.

(4) The assignment of *n* values in water surface profile modeling should be done in a systematic and defensible manner by identifying the types of roughness encountered in the prototype along with a corresponding range of assigned *n* values. The reaches are then categorized by types of roughness and assigned *n* values within the established range. If this is done early in a study, it can be of value in establishing a good initial model and

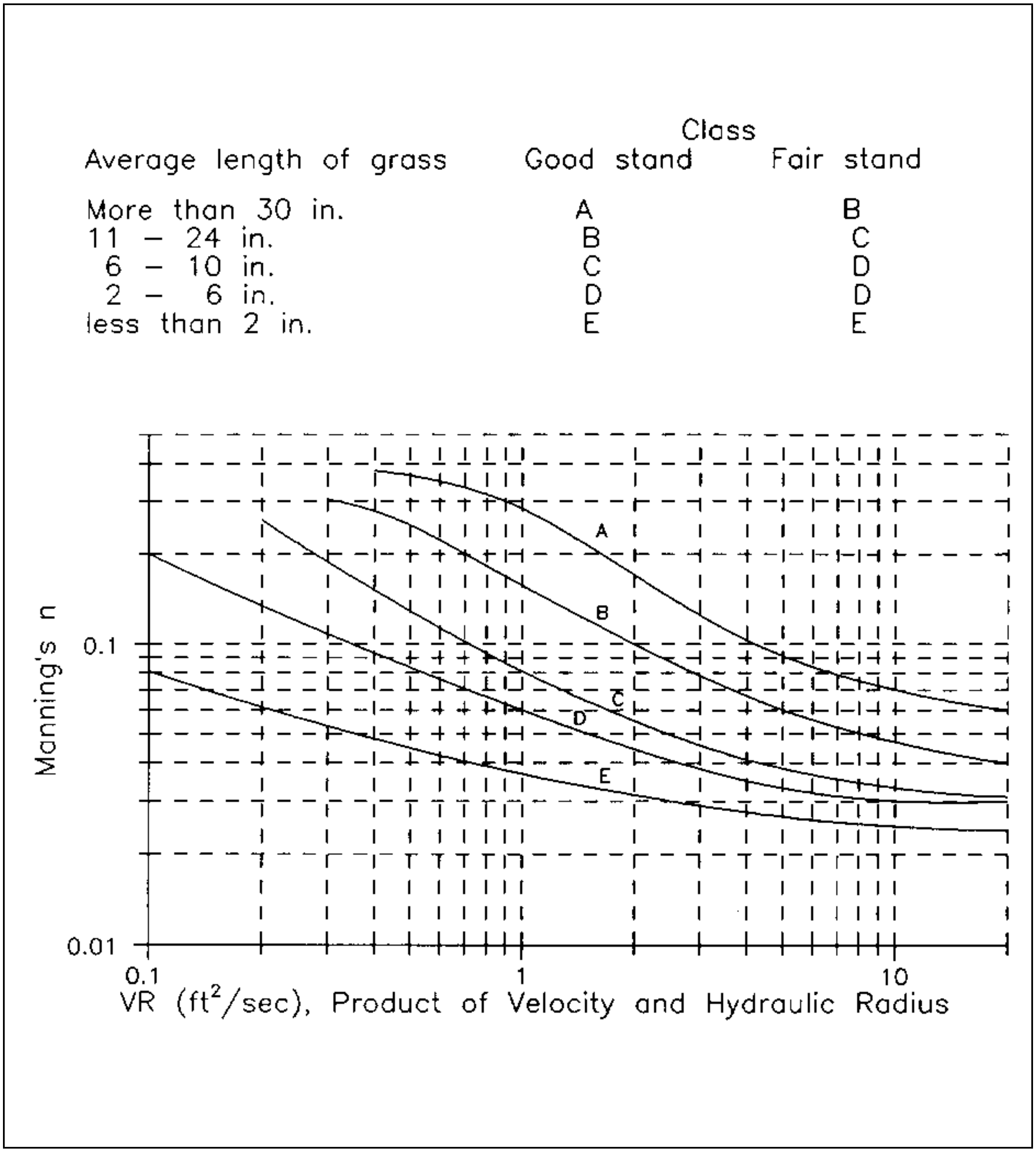

Figure 6-2. The behavior of Manning's *n* in grassed channels

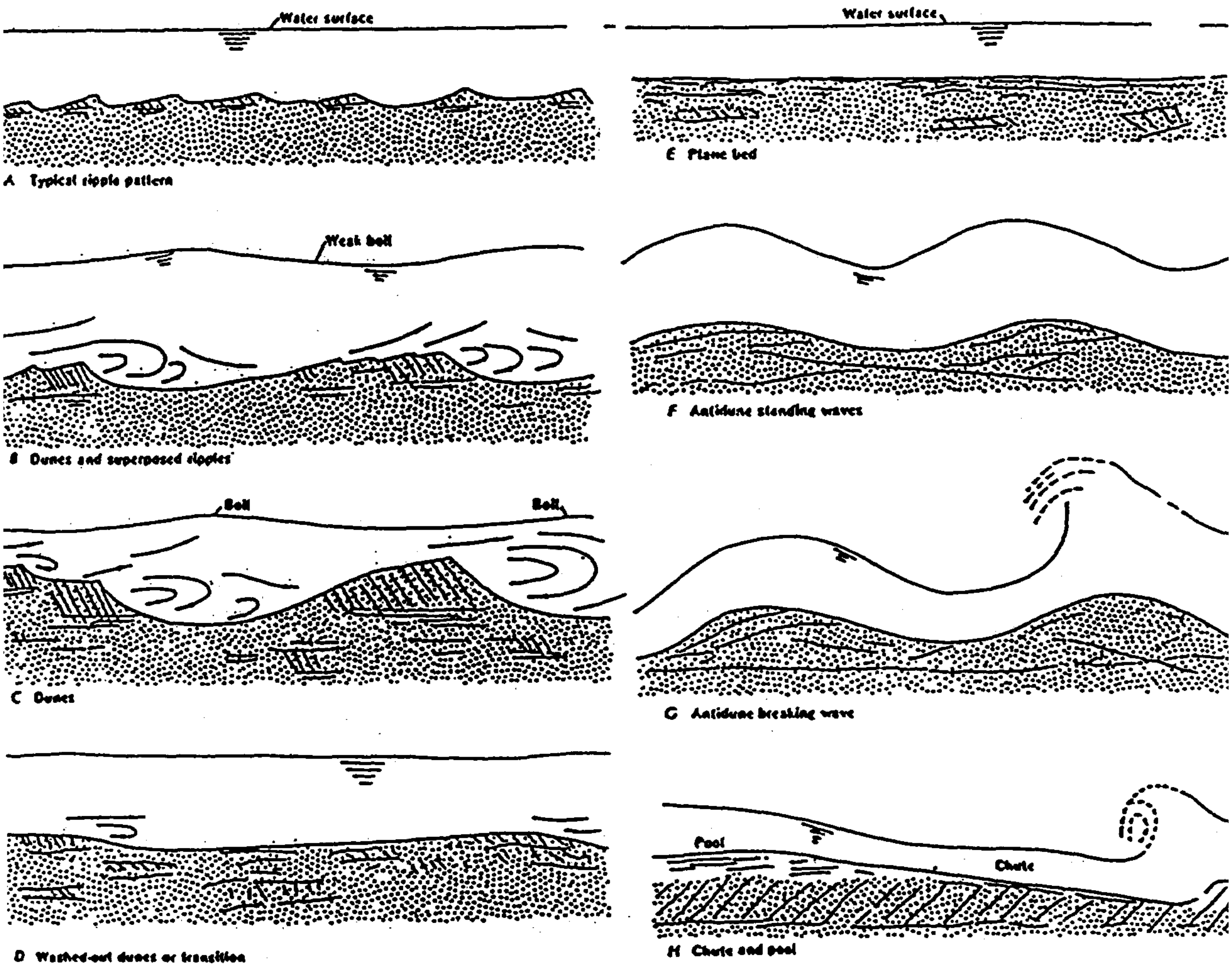

Figure 6-3. Types of bed forms and their relative resistance to flow

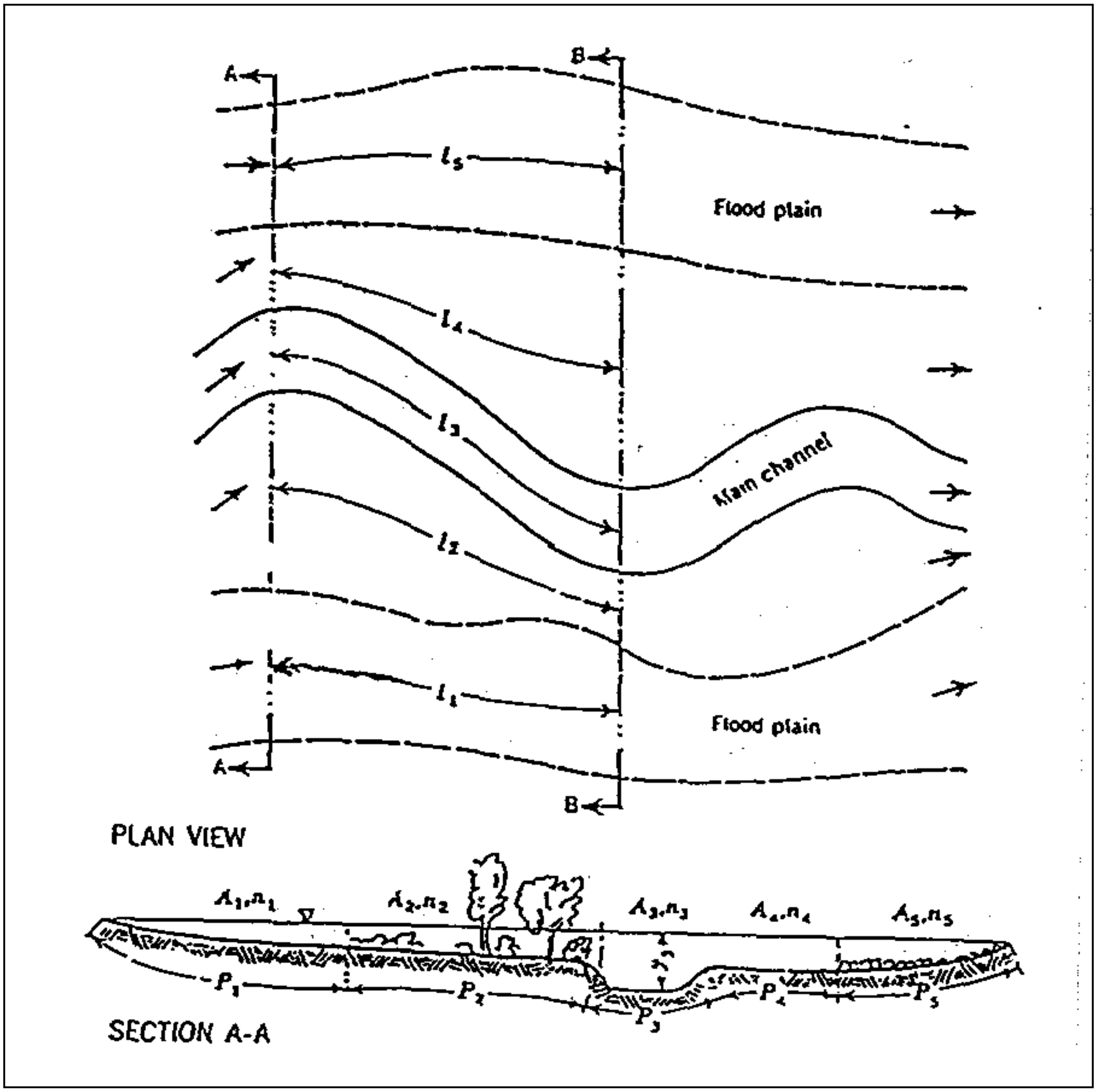

Figure 6-4. Compound channel with laterally varying Roughness and flow depth

become the basis for determining *n*-adjustment limits. An example of a table of *n* values used for model calibration in the Williamson, West Virginia, flood control project is presented in Table 6-1.

f. Example of HEC-2 calibration. A brief description of the calibration of an HEC-2 model used on 20-mile reach of the Tug River in West Virginia is presented in this section. The model was calibrated to floods that occurred in 1984 and 1977. For additional detail on the calibration, see Williams (1988a, 1988c).

(1) Using Chow (1959) as a guide, Manning's *n*-values were assigned to specific reaches of the river and put in the HEC-2 model. The initial *n*-values were adjusted to reproduce observed high water marks. These marks were reproduced within 0.5 foot except for three marks that were reproduced within 1.0 foot, attributable to superelevation "runup" at bends.

(2) Due to inconsistencies in the observed water-surface profiles for the 1984 flood, adjustments to the initial tributary discharges were made after the rainfall data were reexamined and the 1984 flood reconstituted. This changed the main stem discharge at the Kermit gage from 82,000 to 58,000 cfs for the 1984 flood.

(3) The calibration of the 1984 flood resulted in a channel Manning's *n* of 0.058 at the USGS gage in Williamson. The 1977 flood calibration produced channel *n*-values of 0.041 and 0.028 for the 94,000 and 117,000 cfs calibrations, respectively.

(4) Analyses of the detailed USGS discharge/velocity measurements from the 1984 flood indicated that significant flow through the Williamson central business district (CBD) occurred during the 1977 flood. To simulate this, the HEC-2 model was adjusted to reflect the geometry of the buildings and streets, and this overbank area was assigned a Manning's *n*-value of 0.020. Checks were made to assure that side flow over the existing floodwall was sufficient to meet the CBD conveyance potential.

Table 6-1
Roughness Description and Manning *n* Values for 1977 Flood Calibration

Reach River Miles	Left Overbank Description	n	Channel Description*	n	Right Overbank Description	n
38.4-43.86	Clearings of Grassy and Developed Areas to Light to Medium Dense Brush	0.01 to 0.069 Avg 0.057	Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush with Reaches of Medium Dense Brush	0.036	Clearings with Scattered Development and Road and Railroad ROW's	0.041 to 0.069 Avg 0.044
43.86-49.07	Generally Clearings of Grassy and Developed Areas with Intermediate Reaches of Light to Medium Dense Brush	0.041 to 0.069 Avg 0.045	Increased Sinuosity; Sharper Bendways; Typical Side Slopes of Medium to Heavy Dense Brush with Reaches of Light to Medium Dense Brush	0.041	Clearings with Reach of Development and Scattered Vegetation; Grassy Reach and a Reach of Light to Medium Dense Brush	0.041 to 0.048 Avg 0.045
49.07-53.86	Grassy Clearings with Some Development; Reaches of Light to Medium Dense Brush	0.044 to 0.051 Avg 0.045	Gradual Bendways; Typical Side Slopes of Light to Medium Dense Brush; A Reach of Medium to Dense Brush	0.036	Developed with Short Reaches of Grassy to Light to Medium Dense Brush	0.041 to 0.048 Avg 0.045

Section IV
Special Problems

6-12. Introduction to Special Problems

The nature of flow profiles and energy losses at natural or constructed channel features that cause increased energy losses or modified boundary conditions are discussed. Special modeling approaches are presented for various kinds of problems.

6-13. Bridge Hydraulics

a. Nature of flow through a bridge constriction. Flow through a bridge in a wide floodplain has been conceptualized as having four regions: accretion, contraction, expansion, and abstraction (Laursen 1970).

(1) The region of accretion begins upstream from the bridge, a distance just far enough so that the flow is not constricted by the influence of the bridge and the streamlines are parallel. This region extends downstream to a point close to the upstream face of the bridge. As the flow moves through this region towards the bridge, the flow in the overbanks of the floodplain must move laterally toward the channel so that it can pass through the bridge opening. Since the contraction takes place over a considerable distance, the type of flow is "gradually varied."

(2) The region of contraction begins immediately above the upstream face of the bridge where the first region ends and extends through the bridge. The flow contracts more severely in this region to pass through the bridge opening, and the geometry of the opening has a significant effect on the amount of energy loss. A jet is generally formed in the bridge opening, and extends into the region of expansion immediately downstream from the bridge, where it expands through turbulent diffusion and mixing. The type of flow is "rapidly varied" in these two regions of severe contraction and expansion, and the energy losses are relatively high compared to the other two regions.

(3) The region of abstraction extends downstream from the region of expansion to a point where the flow is fully expanded within the confines of the floodplain and the streamlines are again parallel. In this region the flow is "gradually varied" as it expands laterally away from the channel to fill the floodplain.

b. Backwater effects of bridges. Some of the findings of extensive studies on backwater effects of bridges (Bradley 1978) are depicted in Figures 6-5 and 6-6.

(1) The bridge constriction produces practically no alteration of the shape of the streamlines near the center of the channel (Figure 6-5); however, a very marked change is in evidence near the abutments. The momentum of the flow from both overbanks (or floodplain) must force the advancing central portion of the stream over to gain entry to the constriction. After leaving the constriction the flow gradually expands (5 to 6 degrees per side) until normal conditions in the stream are reestablished.

(2) Constriction of the flow causes a loss of energy, the greater portion occurring in the expansion downstream. In a subcritical flow regime, the effect of the constriction is reflected in a rise in water surface and energy grade line upstream from the bridge. This is illustrated with the centerline profile of the stream flow shown in Figure 6-6. The normal stage of the stream without the channel constriction is represented by the dashed line labeled N.W.S. (natural water surface). The water surface as affected by the bridge constriction is represented by the solid line and labeled W.S. The water surface is above the normal stage at cross section 1 by the amount of h1*, which is referred to as "bridge backwater." The flow crosses through normal stage close to cross section 2, reaches minimum depth near cross section 3, and returns to normal stage downstream at cross section 4.

c. Types of flow at bridges. One of several different types of flow may exist at a bridge depending upon the regime and the flow depth relative to key elevations of the bridge and approach structures. In addition to four different classes of low flow, pressure flow, weir flow, and combinations of weir and pressure or weir and low flow are possible. A typical discharge rating curve is shown in Figure 6-7.

d. Bridge loss calculations. The energy losses at a bridge can be divided into two categories: those that occur in the approach reaches immediately upstream and downstream from the bridge and those that occur through the structure. In computer programs such as HEC-2, the first category is computed with standard step profile calculations that use Manning's Equation to determine friction losses and apply contraction and expansion

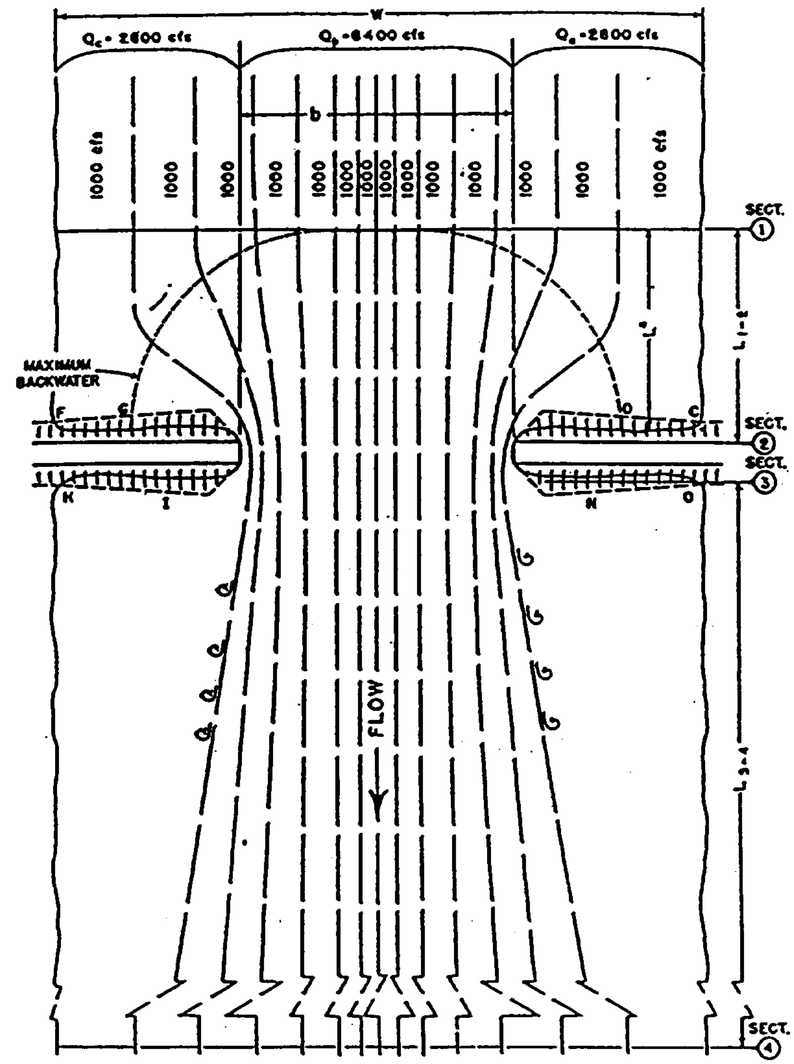

Figure 6-5. Flow lines for typical normal bridge crossing

coefficients to changes in velocity head between adjacent cross sections to determine other losses. The second category of losses, which occurs in the flow through the bridge structure, is determined by one of three different methods: the normal bridge method, the special bridge method, or by external hydraulic calculations input to the program. The special culvert method available for analyzing energy losses through culverts is covered in a subsequent section of this chapter.

(1) The approach reach on each side of a bridge generally requires two cross sections: one next to the face of the bridge and one at the other end of the reach. On the upstream side of the bridge, the length of the approach for contraction of the flow is usually set at a distance equal to one times the average of the two abutment projections. On the downstream side, the length of the reach for expansion is usually set at a distance of four times the average of the abutment projections. See Figure 6-8.

(2) The normal bridge method computes losses through the bridge with the standard step method in the same manner the program computes losses between natural river cross sections. Two or more additional cross sections are located within the bridge opening to define the geometry of the bridge structure and changes in

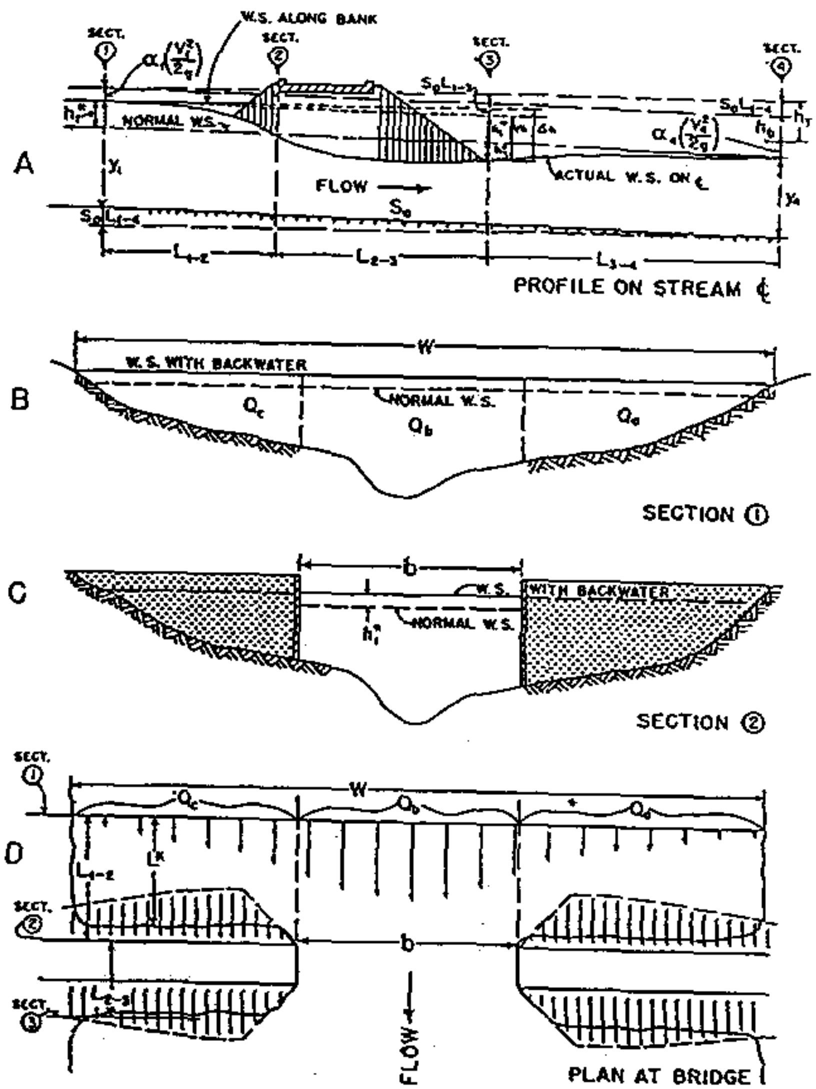

Figure 6-6. Stream profile and cross sections for normal bridge crossing, wingwall abutments

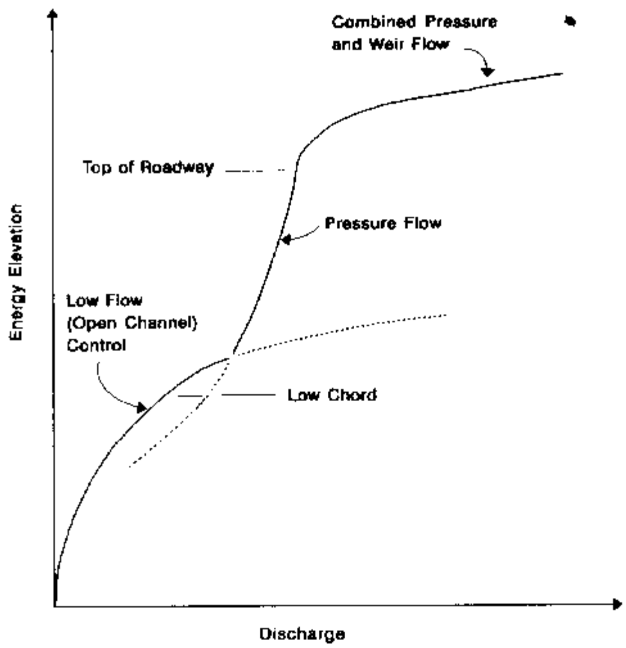

Figure 6-7. Typical discharge rating curve for bridge culvert

roughness for the bridge surfaces. In the computations, the area of the structure below the water surface is subtracted from the total flow area, and the wetted perimeter is increased where the water is in contact with the structure.

(3) The special bridge method computes the hydraulic losses through the bridge using hydraulic equations. The program determines whether the flow is low flow, pressure flow, weir flow, or a combination, and then applies the appropriate equations. Schematic flow diagrams and a description of the decision logic for this process, which is quite complex, are presented in the HEC-2 user's manual (U.S. Army Corps of Engineers 1990b).

(4) Externally computed bridge losses can be input to the program as computed changes in water surface elevations between cross sections located on opposite sides of the bridge.

(5) Guidelines for selecting a method for a particular bridge analysis are presented in the HEC-2 user's manual (U.S. Army Corps of Engineers 1990b). In general, the normal bridge method is most applicable when friction losses are the predominate consideration, or the conditions make it impractical to use the special bridge method. The special bridge method is most applicable for computing weir flow, pressure flow, low flow, or a combination of these that can be modeled effectively with the hydraulic equations available in the method. If the bridge acts as a hydraulic control and a rating curve is available, reading in the known water surface elevations would be the preferred method.

6-14. Culvert Hydraulics

a. Culvert loss calculations. Computation of the energy losses in the transition sections upstream and downstream from a culvert is almost the same as for a bridge. In the computation of the loss through the

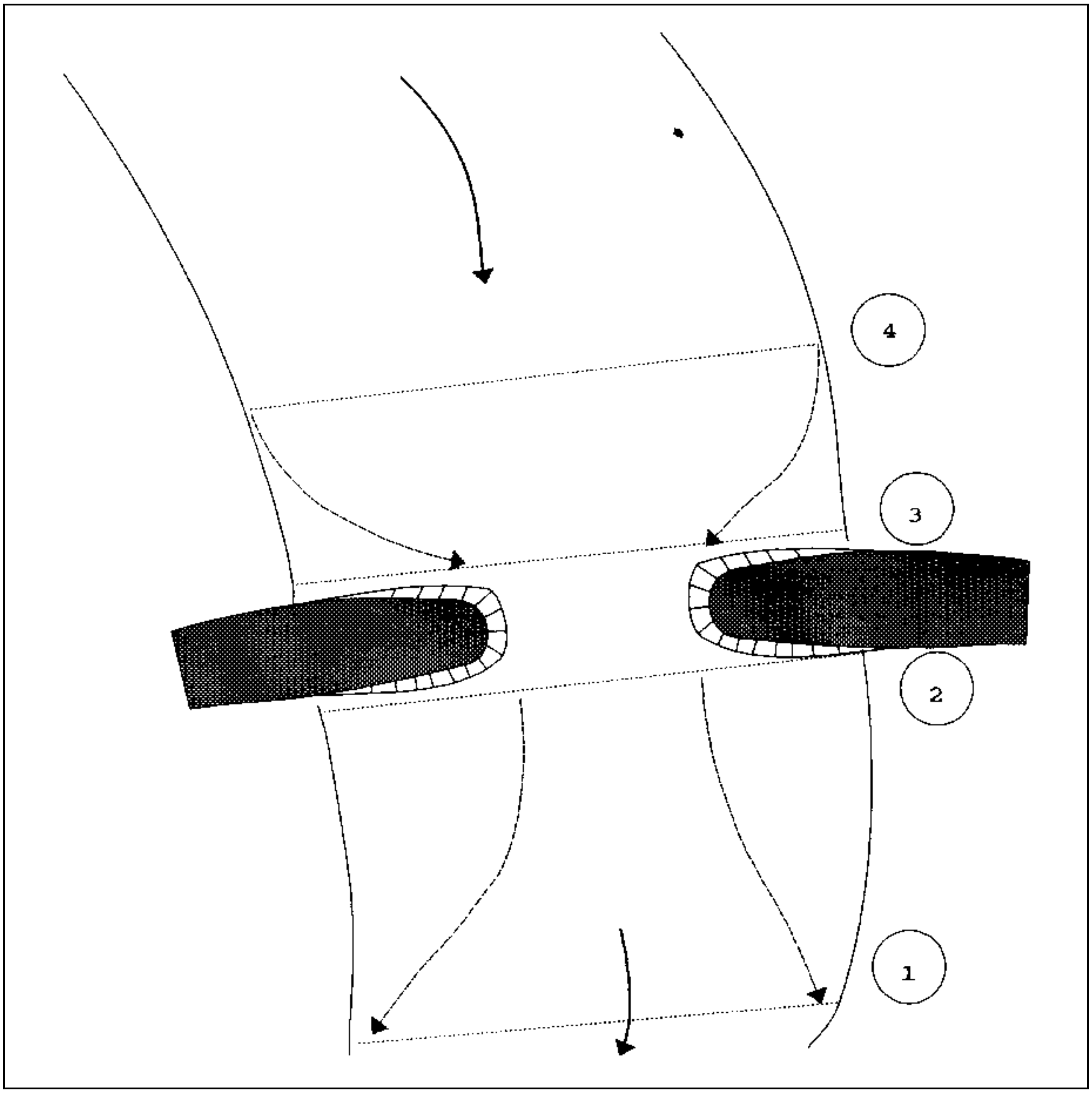

Figure 6-8. Cross section locations in the vicinity of bridges

culvert the concepts of "inlet control" and "outlet control" are used.

b. Inlet and outlet control. Inlet control of the flow occurs if the flow capacity of the culvert entrance is less than the flow capacity of the culvert barrel. Outlet control occurs if the culvert capacity is limited by downstream conditions or by the flow capacity of the culvert barrel. The headwater, which is the depth of water at the culvert entrance measured from the invert, is computed for a given flow rate under both inlet control and outlet control conditions. The higher value computed indicates which condition "controls," and it is this value that is used to determine the culvert loss.

(1) For inlet control, a series of equations that have been developed from extensive laboratory tests (U.S. Department of Transportation 1985) is used to calculate the headwater under various conditions. The headwater is computed assuming that the inlet acts as an orifice or a weir, and the capacity depends primarily on the geometry of the culvert entrance.

(2) For outlet control, the headwater is computed by taking the depth of flow at the culvert outlet, adding all head losses, and subtracting the change in the flow line (invert) elevation from the upstream to the downstream end. This is a complex process that must consider several conditions within the culvert and downstream of the culvert. A flow chart and description of the equations used in the computations are presented in the HEC-2 user's manual (U.S. Army Corps of Enigneers 1990b).

6-15. Limits of Effective Flow

Irregularities in the natural topography or the introduction of structures such as bridges or levees into a watercourse may require that field topographic data be modified to depict the effective flow areas through the channel irregularities or structures. Numerical models such as HEC-2 contain capabilities to restrict flow to the effective flow areas of cross sections. Among these capabilities are options to simulate sediment deposition, to confine flows to leveed channels, to block out road fills and bridge decks, and to analyze floodplain encroachments. Figure 6-9 illustrates these effective flow area modifications. In modeling it is important to study carefully the flow pattern of rivers being analyzed to determine effects of levees, bridges, and other obstructions to natural flow patterns. Appendix 4 of the HEC-2 user's manual provides guidance for modeling effective flow areas.

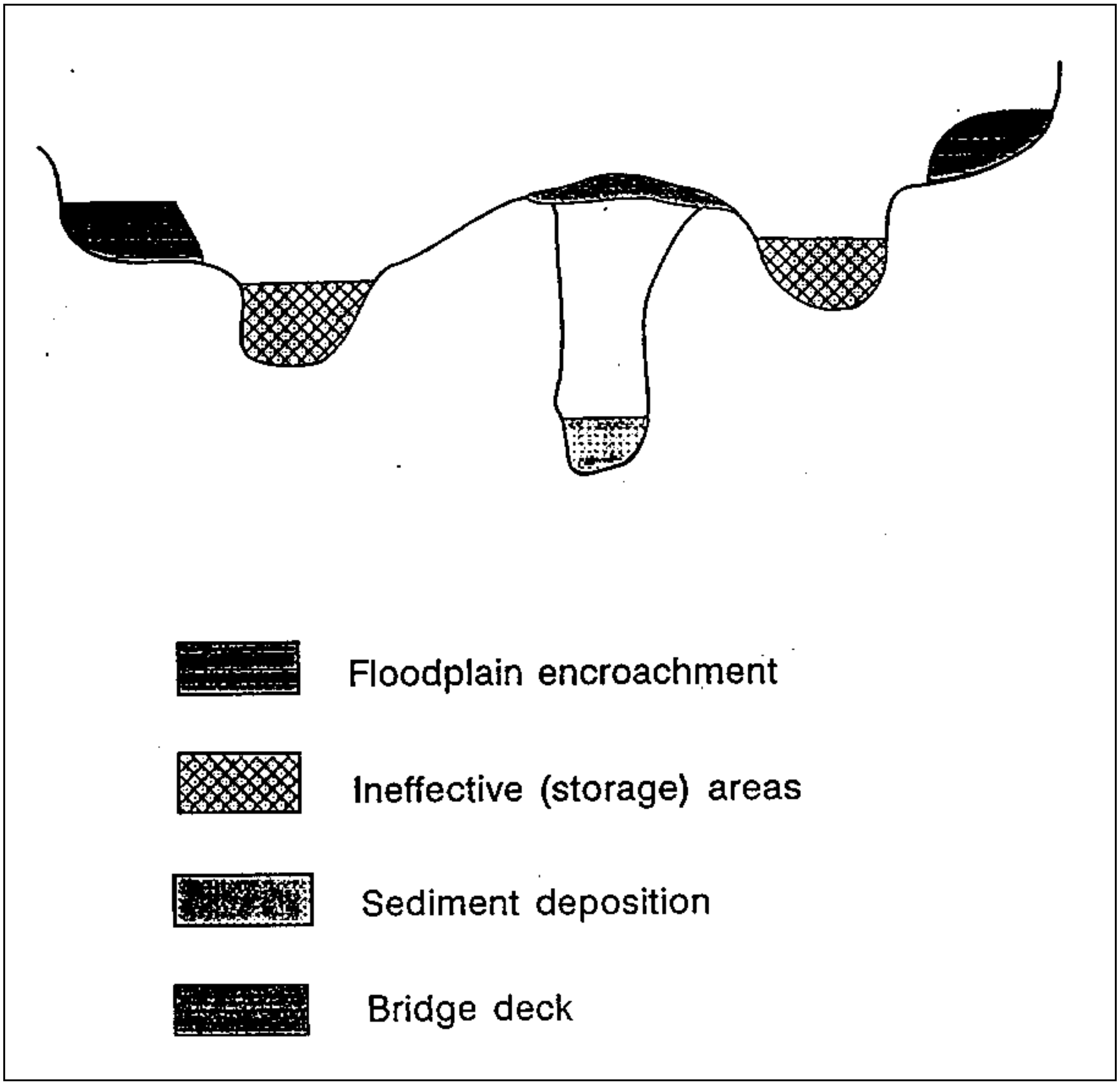

Figure 6-9. Types of effective flow options

6-16. Channel Controls

Any constriction in a channel that backs up water is a "control," and if the bed and banks of the channel at a control do not change, a constant relationship between discharge and water surface elevation will be maintained. The location of a control in a channel is called a "control section." And a control section controls the flow in such a way as to restrict the transmission of the effect of changes in flow condition either in an upstream direction or a downstream direction depending on the state of flow in the channel (Chow 1959). Streams are commonly made up of alternate reaches of slack water and rapids, and the head of a rapids being necessarily of a permanent nature is a control that tends to back water upstream.

a. Critical depth. The condition of critical depth implies a unique relationship between depth and discharge that can only occur at a control. The flow regime can pass from subcritical to supercritical, or vice versa, only if the flow passes through critical depth. Critical depth occurs when subcritical flow passes over a weir or free outfall. It may occur if the channel bottom is abruptly elevated or the side walls are contracted. In fact, measuring flumes are designed to force flow through critical depth by raising the bottom and narrowing the width of the channel. The discharge is determined by simply measuring the depth in the flume (Bedient and Huber 1988).

b. Importance of controls in computing water surface profiles. Since a control section holds a definitive stage-discharge relationship, it is a suitable location for developing discharge rating curves for water surface profile analysis. It is common practice to obtain starting water surface elevations from rating curves or conditions of critical depth at control sections. High water marks and gage readings at control sections are useful data in model calibration and verification.

6-17. River Confluences

a. Confluence of a river. At the confluence of a river and one of its tributaries, the determination of the water surface elevation of each stream immediately upstream from the confluence is necessary to continue the backwater computations up the main river or the tributary.

b. Example. The procedure in solving this problem at the confluence of the Missouri and Kansas Rivers is shown by example (EM 1110-2-1409) in Table 6-2. A discharge of 81,000 cfs from the Kansas River combines with 350,000 cfs from the Missouri River to give a total discharge of 431,000 cfs immediately below the confluence. Cross sections 1K and 6 are located immediately upstream from the confluence of the two streams, as shown in Figure 6-10. The hydraulic elements of cross sections 5, 6, 7 and 1K are shown in Table 6-3.

(1) The friction slope for each cross section is computed for the discharge of 81,000 cfs, at cross section 1K and 350,000 cfs at cross section 6. The friction-head loss h_f is then computed, using the average friction slope from cross sections 5 to 1K on the Kansas River and from 5 to 6 on the Missouri River.

(2) The velocity head for cross section 5 is computed at a discharge of 431,000 cfs, and the velocity head for cross sections 1K and 6 is taken as the weighted average velocity head for the discharge of 431,000 cfs through the combined area of the two cross sections. The total V^2Q value is determined for the combined area and divided by 431,000 to obtain the average V^2.

(3) The resulting change of 0.28 feet (h) between cross sections 5 and the combined area is added to the h_f of 0.10 feet to obtain the total rise in water surface of 0.38 feet between cross sections 5 and 1K. Likewise, the same change is added to h_f of 0.16 feet between cross sections 5 and 6 to obtain the total rise in water surface of 0.44 feet between backwater elevations.

(4) The method as described in the preceding paragraphs should be applied only to channels having low velocities not exceeding about 10 feet per second.

(5) Computer programs such as HEC-2 can compute water surface profiles for tributaries together with profiles for the main stream in a single execution of the program (U.S. Army Corps of Engineers 1990b).

6-18. Changing Flow Regime

a. Steady-state water. Most commercially available steady-state water surface profile programs such as HEC-2, can only simulate one regime of flow for a single profile computation. Whenever the calculated flow profile would cross critical depth from either the subcritical or supercritical regimes, or whenever the simulation cannot converge to a solution, critical depth at that location is assumed. For the majority of subcritical flow situations critical depth is a good assumption. However, in supercritical reaches in particular, the critical depth assumption may not be satisfactory.

Table 6-2
Backwater Computations by Method 1, Missouri and Kansas Rivers at Kansas City

Sec. No.	River Mile	Reach L	Area A	P	R	$R^{2/3}$	n	$S^{1/2}=.01$ V	$S^{1/2}=.01$ Q	S	Mean S	h_f	V	Q	V^2Q	V^2 avg.	h_v	$h_{v_1}-h_{v_2}$	Total H	W.S. Elev.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1	377.28		6,080	497	12.2		0.03	1.58	9,600				2.36	14,400	80,300					
732.2			38,600	910	42.4		0.025	7.22	278,700				10.81	416,600	48,700,000					
									288,300	0.000224				431,000	48,780,300	112.2	1.76			732.23
2	377.78	3080	2,200	210	12.3		0.03	1.86	4,100				2.70	7,000	51,000					
732.5			41,600	1310	31.6		0.025	5.97	248,300				10.30	424,000	44,100,000					
									252,300	0.000292	0.000258	0.27		431,000	44,151,000	108.4	1.68	0.17	0.44	732.68
3	377.94	845	1,400	123	11.4		0.05	1.51	2,100				2.15	3,000	13,800					
733.3			65,300	1970	33.0		0.025	4.61	301,200				6.51	428,000	31,800,000					
									303,300	0.000202	0.000247	0.21		431,000	31,813,800	72.2	1.12	0.47	0.68	733.37
4	378.33	2080	86,400	2170	30.2		0.025	4.37	377,300	0.000131	0.000167	0.34	4.99	431,000		43.6	0.68	0.44	0.78	734.15
734.0																				
5	378.65	1660	9,040	754	12.0		0.05	1.56	14,100				1.73	15,700	47,000					
734.5			64,800	2100	30.6		0.025	5.62	373,800				6.41	415,300	17,380,000					
									387,900	0.000123	0.000127	0.21		431,000	17,427,000	40.4	0.63	0.05	0.26	734.41
		BALANCE SECTION					350,000 c.f.s. from Upper Missouri					81,000 c.f.s. from Kansas River								
IK	North Kansas River																			
734.0		1430	27,700	844	32.8		0.025	6.08	168,600	0.000023	0.000073	0.10	2.93	81,000	693,000			0.28	0.38	734.79
6	Missouri River																			
734.6	378.86	1380	9,800	736	13.4		0.05	1.50	14,900				1.41	13,800	26,200					
			64,800	2100	30.9		0.025	5.84	368,300				5.19	336,000	9,070,000					
									383,200	0.000076	0.000101	0.14		350,000						
														431,000	9,791,200	22.7	0.35	0.30	0.44	734.85
	Proceeding up the Missouri River																			
7	379.41	2430	15,400	914	16.8		0.04	2.45	37,800				2.68	41,400	297,000					
734.8			42,300	1330	34.0		0.025	6.24	281,900				6.84	288,600	14,440,000					
									319,300	0.000120	0.000099	0.24		330,000	14,737,000	42.1	0.65	−0.30	−0.06	734.79
10	383.34		22,600	1800	12.6		0.050	2.11	48,000				2.06	46,700	198,000					
738.6			45,300	1310	34.6		0.025	6.31	285,800				6.13	257,800	10,440,000					
			9,800	519	18.0		0.040	2.68	26,200				2.58	25,500	170,000					
									360,000	0.00084				330,000	10,808,000	30.9	0.48			736.77
	Fairfax Bridge − 30' (See Section No. 10)																			
		2980'																		
738.2			22,800	1800	12.5		0.050	2.16	50,700				2.09	47,700	197,000					
			46,100	1310	35.2		0.025	6.38	283,800				6.00	276,400	9,950,000					
			10,800	821	13.6		0.040	2.70	27,800				2.54	25,900	167,000					
									372,100	0.000088	0.000081	0.22		330,000	10,314,000	29.3	0.46	0.02	0.24	736.01
	Fairfax Bridge																			
		80'																		
	384.0		9,910	730	13.6		0.030	1.70	16,800				1.99	19,600	78,000					
738.0			48,800	1640	29.7		0.025	5.70	278,300				6.67	325,400	14,480,000					
			2,340	168	14.2		0.030	1.78	4,300				2.06	4,800	20,000					
									299,100	0.000127	0.000113	0.01		330,000	14,578,000	41.7	0.65	−0.18	−0.16	736.83
	Fairfax Bridge ≠ 80' (See Section No. 10)																			
		80'																		
738.2			22,800	1800	12.5		0.050	2.16	50,700				2.08	47,700	207,000					
			46,100	1310	35.2		0.025	6.38	283,800				6.00	276,400	9,930,000					
			10,800	821	13.6		0.040	2.70	27,800				2.54	25,900	167,000					
									372,100	0.000088	0.000113	0.01		330,000	10,314,000	29.3	0.46	0.18	0.20	736.03
		2640'																		
11	388.27		28,100	2180	14.7		0.050	1.78	57,400				2.80	63,800	564,000					
740.0			28,200	852	34.3		0.025	8.28	383,300				9.13	266,800	22,300,000					
									440,700	0.000211	0.000130	1.24		330,000	22,764,000	69.0	1.07	−0.86	0.88	738.72

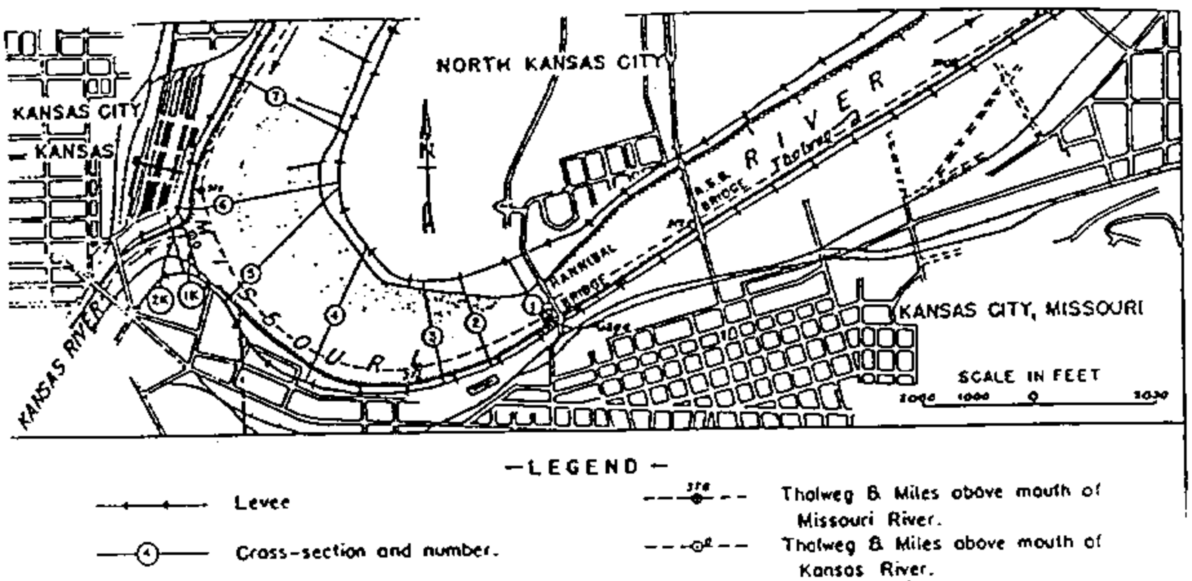

Figure 6-10. Index map, Missouri and Kansas Rivers at Kansas City, Missouri

Table 6-3
Tabulation of Hydraulic Elements, Missouri and Kansas Rivers at Kansas City

Section No.	River Mile	W.S. Elev.	A	P	R	n	$K' \times 10^{-6}$	$\frac{(K')^3}{(\Sigma K')^2}$	$A^{-2} \times 10^{10}$	$(9) \times (10) \times 10^{10}$	$F \times 10^{12}$	$K \times 10^{15}$	L_w	$F' \times 10^{12}$	L_1	$F'' \times 10^{12}$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1	377.58	752	6,000	496	12.1	.05	.9	.0000325	278	.01			1040			
			38,100	910	41.9	.025	27.3	.907	6.89	6.25						
							28.2			6.26	9.74	1.26		10.41		
		753	6,500	499	13.0	.05	1.1	.000052	237	.01						
			39,000	911	42.8	.025	28.3	.892	6.57	5.86						
							29.4			5.87	9.13	1.16		9.74		
		754	7,300	502	13.9	.05	1.2	.000060	204	.01						
			39,900	912	43.7	.025	29.4	.885	6.28	5.56						
							30.6			5.57	8.67	1.07		9.24		
2	377.78	752	2,500	208	12.0	.05	.4	.0000044	1800	.01			845		1060	
			41,000	1,320	31.0	.025	24.0	.951	5.95	5.66						
							24.4			5.67	8.82	1.68		9.53		7.93
		753	2,700	213	12.4	.05	.4	.0000038	1370	.01						
			42,300	1,325	32.0	.025	25.3	.954	5.59	5.33						
							25.7			5.34	8.31	1.50		8.94		7.51
		754	2,900	217	13.4	.05	.5	.0000063	1190	.01						
			43,600	1,330	32.8	.025	26.6	.946	5.26	4.98						
							27.1			4.99	7.76	1.36		8.33		7.04
3	377.94	752	1,200	120	10.0	.05	.2	.0000004	6940	.00			2000		845	
			48,300	1,577	30.6	.025	28.0	.979	4.29	4.20						
							28.2			4.20	6.53	1.26		7.83		6.00
		753	1,300	124	10.4	.05	.2	.0000003	5920	.00						
			49,900	1,581	31.5	.025	29.6	.980	4.02	3.94						
							29.8			3.94	6.13	1.12		7.28		5.66
		754	1,400	128	11.0	.05	.2	.0000003	5100	.00						
			51,400	1,585	32.5	.025	31.0	.981	3.79	3.72						
							31.2			3.72	5.79	1.03		6.85		5.35
4	378.33	752	61,400	2,163	28.4	.025	34.2	1	2.65	2.65	4.12	.86	1690	4.85	2040	3.23
		753	63,500	2,172	29.2	.025	35.8	1	2.48	2.48	3.86	.78		4.52		3.06
		754	65,700	2,181	30.1	.025	37.8	1	2.32	2.32	3.61	.70		4.20		2.89
5	378.65	752	7,200	745	9.8	.05	1.0	.000026	193	.01					1690	
			59,100	2,084	28.5	.025	32.9	.914	2.86	2.61						
							33.9			2.62	4.08	.88				3.34
		753	7,900	749	10.6	.05	1.1	.000029	160	.00						
			61,200	2,085	29.3	.025	34.6	.910	2.67	2.43						
							35.7			2.43	3.78	.78				3.12
		754	8,700	753	11.6	.05	1.2	.000032	132	.00						
			63,300	2,087	30.3	.025	36.6	.908	2.50	2.27						
							37.8			2.27	3.53	.70				2.94

b. Mixed flow regimes. It is unusual to find a reach where the flow is consistently supercritical. Constrictions and local reductions in cross-sectional area in a stream having an overall slope approaching critical slope can cause the flow regime to oscillate back and forth from supercritical to subcritical. Molinas and Trent (1991) have developed a backwater model which locates changes in flow regime and performs the water surface profile calculations once the regime transition points have been identified.

6-19. Ice-covered Streams

a. Ice stability. Ice stability analysis by Canadian and American researchers has shown that ice covers and the formation of ice jams are a complex process that is a function of relative stream dimensions, ice properties, and the velocity of flow. Various researchers have categorized ice-covered streams as narrow, wide, deep, and shallow in accordance with criteria that includes velocity, width, depth, and ice thickness.

(1) Pariset et al. (1966) present an ice stability criterion which is suitable for analysis of cohesionless-ice-covered wide rivers. Spring breakup ice is considered to possess negligible cohesion, and is approximately analyzed by Pariset's criterion. Calkins (1978) indicates that Pariset's Equations are appropriate for deep streams. He suggests that, as a rule of thumb, a river can be considered to be deep if the depth of flow is greater than 12 feet.

(2) Pariset's 1966 paper presents the following dimensionless stability criteria "X" for analyzing the ratio of the thickness "h" of ice to the upstream open water depth "H." (This is shown graphically in Figure 6-11.)

$$X = \frac{Q^2}{C^2 B H^4} \tag{6-4}$$

where

X = ice stability indicator
Q = discharge
C = Chezy coefficient
B = stream width
H = upstream depth

b. Ice-covered streams. Ice cover occurring on a small stream may have sufficient strength to completely bridge the stream during low flow, creating an approximate closed conduit condition. During high flows ice may be held in place by rocks or trees, and as flow rises, open channel conditions may occur above the ice, and pressure flow may occur beneath the ice. Ice covers wide stream floats, and is free to rise and fall with changing discharge.

(1) Profiles may be computed for ice-covered streams by normal standard-step backwater calculations if allowance is made for the flow area blocked by the ice, and if the increased wetted perimeter is accounted for. Hydraulic roughness values must also be adjusted to account for differences in roughness between the ice and the stream bed. The position of the floating ice relative to the free water surface (piezometric head) is determined by the specific gravity of the ice; a typical value is approximately 0.92. Figure 6-12 shows pertinent hydraulic parameters of an ice-covered stream.

A = open flow area under the ice
P_b = wetted perimeter of the channel
B = wetted perimeter of the ice cover
n_b = Manning's n value for the stream bed
n_i = Manning's n value for the ice cover
R = hydraulic radius

$$R = \frac{A}{P_b} \quad \text{(open channel)} \tag{6-5}$$

$$R = \frac{A}{P_b + B} \quad \text{(ice-covered channel)} \tag{6-6}$$

(2) For wide ice-covered channels, the total wetted perimeter ($W_p + B$) is double the wetted perimeter for the same flow area of an open channel. Thus, the resulting hydraulic radius is half that for an open channel. The increased wetted perimeter is the principal reason that an ice-covered stream requires a greater depth topass an equivalent discharge when compared to a stream flowing under open channel conditions.

$$n_c = \frac{(n_i^{3/2} + n_b^{3/2})^{2/3}}{2} \tag{6-7}$$

where

n_c = composite Manning's n value
n_b = stream bed Manning's n value
n_i = ice Manning's value

c. Ice jams. A number of researchers have classified ice jams with the different classification schemes depending on the season, ice type, and river width. The

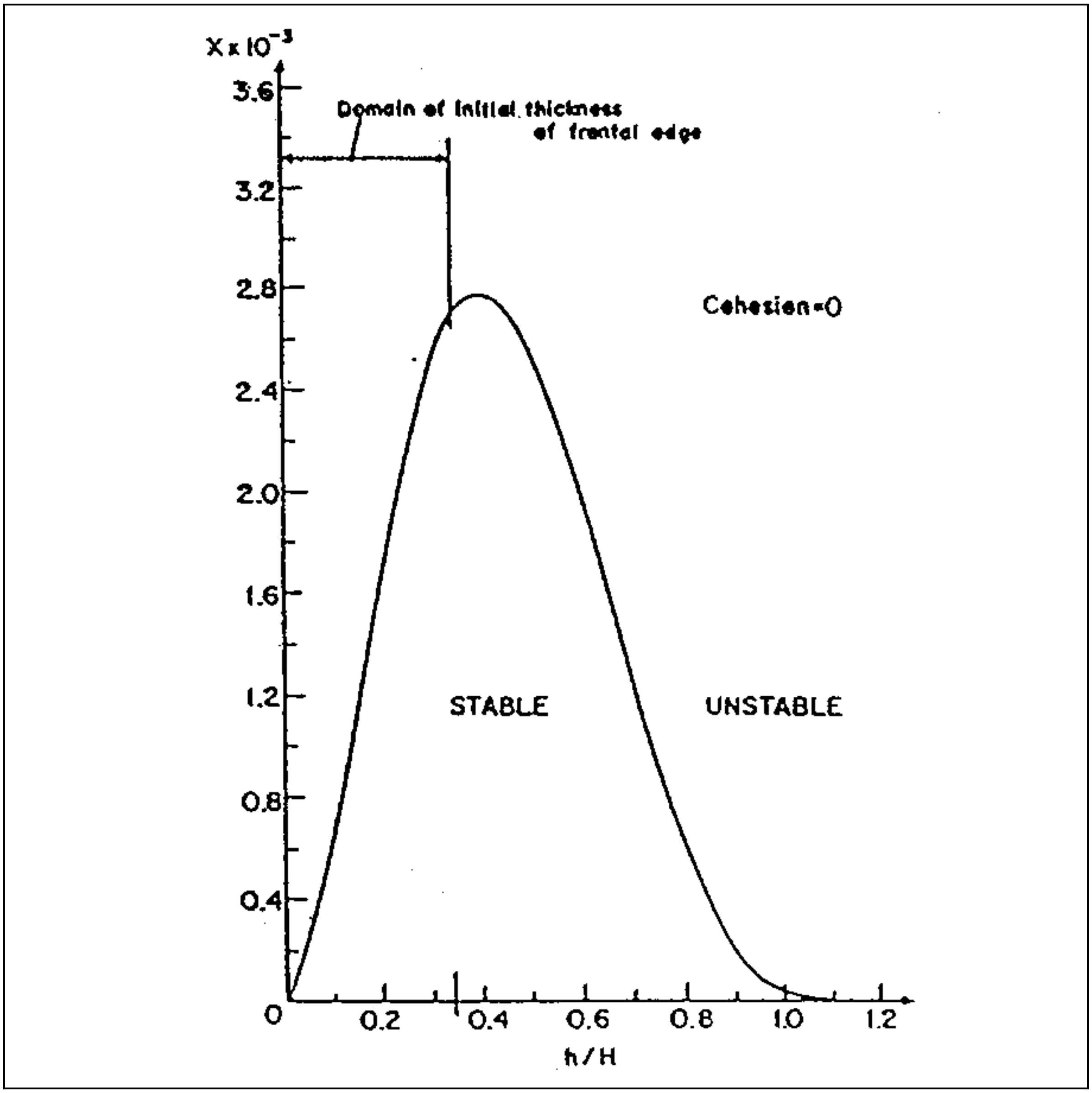

Figure 6-11. Stability function of ice cover for deep, wide channels

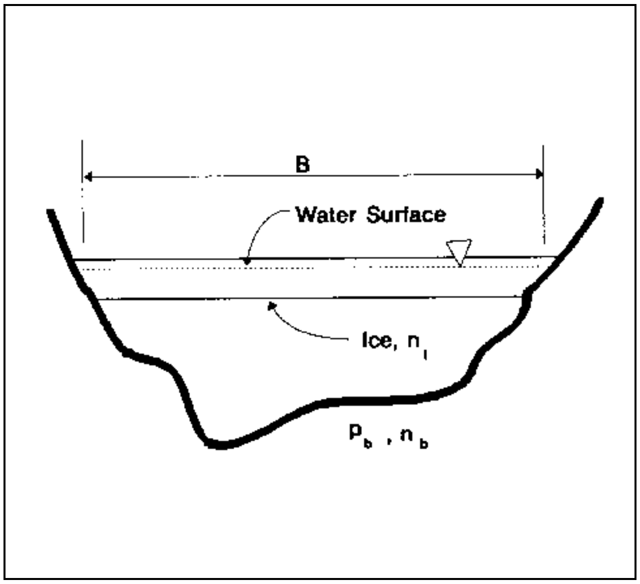

Figure 6-12. Hydraulic parameters of an ice-covered stream

primary objective of an ice jam analysis is to predict location, expected thickness and length, associated water levels, and duration.

(1) The locations of ice jams have been identified in the past by prior occurrences at a particular site. Out of a listing of 20 ice jam sites in Vermont, the one common feature that stands out at 14 of the sites is the presence of a relatively long backwater condition. At five sites, two or more streams form junctions; three of these sites are also at the end of a backwater section. Two sites have no structures influencing the jams, but have an almost annual occurrence. At one site, jams form at an obvious channel enlargement, and at the other jams form at an exposed ledge that crops out just upstream of a island. Two ice jam sites have no noticeable physical irregularities in the stream channel geometries, but appear to have relatively mild slopes.

(2) The length and thickness of an ice jam is governed by many factors. One study of ice jam lengths and volumes for streams in the northeastern U.S. showed that the ice jam length did not exceed 10 percent of the upstream river length which contributed ice to the jam.

(3) An estimate for volume of ice in an ice jam can be expressed as

$$V = (1 - C_i)L_r h \tag{6-8}$$

where

V = ice volume in the jam
C_i = coefficient of ice loss
L_r = length of river contributing ice
h = ice cover thickness at breakup

The ice loss coefficient has been computed for some streams in northern New England as ranging from 0.95 to 0.1. The high ice loss coefficient of 0.95 reflected a long river reach with many tributaries and a significant loss of ice to the river banks. The lower ice loss coefficient is for an ice jam in a short river length. Each ice jam site will have a different ice loss coefficient that will be consistent from year to year.

(4) Figure 6-13 shows the average jam depth h_j as a function of position within the normalized jam length L_r for two jams on narrow, steep rivers. The ice jam depth is expressed in multiples of the ice cover thickness prior to breakup, i.e., h_j/h. If the initial ice cover is 2 feet, then the ice thickness at the toe of the jam would be roughly 8 feet.

(5) The length of the ice jam L_j can be computed if no records are available by making an assumption about the ice thickness distribution and the volume of ice reaching the site. Using a very simple ice jam length thickness distribution as constant over the length of the jam of $h_j = 2h$, the ice jam length can be computed by dividing the expected volume of ice by the thickness distribution function, yielding

$$L_j = \frac{(1 - C_i)B}{2} \tag{6-9}$$

(6) Figure 6-14 shows the type of variation one can expect in ice jam thickness measurements in one cross section.

(7) The first calculation made in any analysis of an ice jam is to determine the ice volume expected to reach the jam location. The volume can be calculated by measuring river mileage from a USGS topographic map, calculating the expected ice thickness, and determining the average river top width. Once a volume has been calculated, engineering judgment must be used to determine the actual amount of ice reaching the site. A good first approximation is 10 percent.

Location	L_j		h_j	
	ft	m	ft	m
(O) Tunbridge	3000	915	1.5	0.46
(●) Woodstock	9200	2804	1.7	0.52

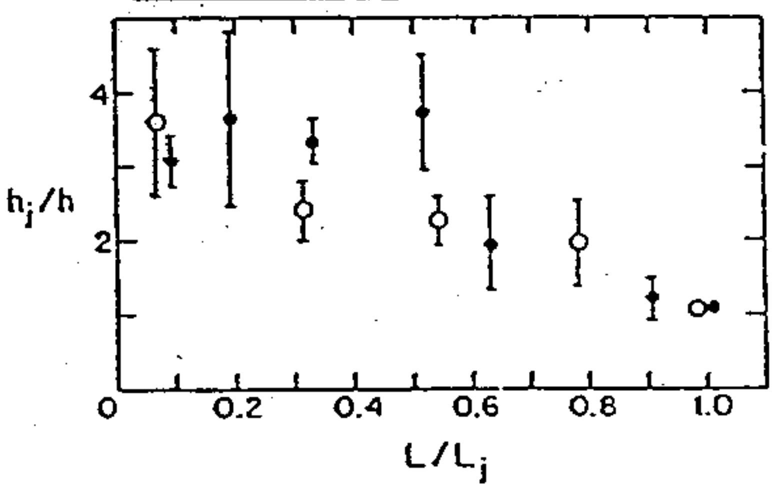

Figure 6-13. Nondimensional ice jam thickness versus its relative length (narrow, steep rivers)

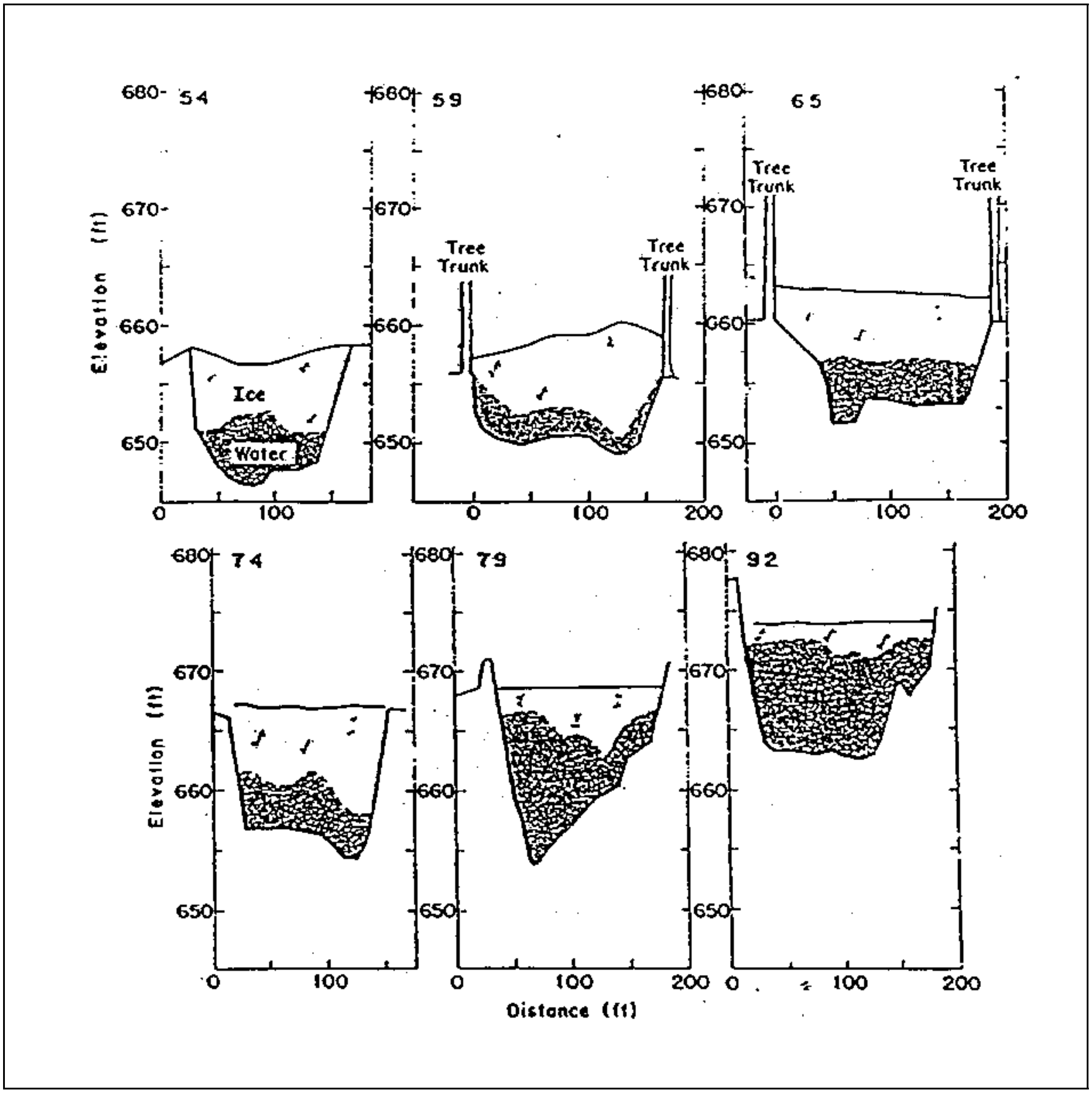

Figure 6-14. Typical ice jam sections on a shallow stream