Hydraulic Simulation in Instream Flow Studies: Theory and Techniques

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CORRIGENDUM

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The following corrections should be made in this publication:

Page 33, equation 18 reads,

$$\text{Error (\%)} = \frac{Q_m - Q_p}{Q_m} \times 100\%$$

It should read,

$$\text{Error (\%)} = \frac{|Q_m - Q_p|}{Q_m} \times 100\%$$

Page 39, equation 19 reads,

$$\text{Error (\%)} = \frac{V_m - V_p}{V_m} \times 100\%$$

It should read,

$$\text{Error (\%)} = \frac{|V_m - V_p|}{V_m} \times 100\%$$

Page 85, 3rd paragraph. The sentence reads,

Using equation 20 (page ____), the allowable error of closure is:

$$\text{Allowable error} = 0.05 \ \frac{660}{5280} = 0.018 \qquad (21)$$

It should read:

Using equation 20 (page 65), the allowable error of closure is:

$$\text{Allowable error} = 0.05 \sqrt{\frac{660}{5280}} = 0.018 \qquad (21)$$

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HYDRAULIC SIMULATION IN INSTREAM FLOW STUDIES: THEORY AND TECHNIQUES

Instream Flow Information Paper No. 5

by

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DISCLAIMER

The opinions, findings, conclusions, or recommendations expressed in this report are those of the author and do not necessarily reflect the views of the Office of Biological Services, Fish and Wildlife Service, U.S. Department of the Interior, nor does mention of trade names or commercial products constitute endorsement or recommendation for use by the federal government.

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ABSTRACT

Hydraulic simulation for instream flow studies is defined as the description of the changes in distribution of velocities, depths, and substrates as a function of discharge. These changes are described in mathematical terms to facilitate the quantification of change of a dependent variable such as velocity with an incremental change in discharge. The type of hydraulic simulation technique appropriate for a certain situation depends on the degree of resolution required for the study, the characteristics of the stream, and the limitations inherent in the technique.

Several types of techniques for the prediction of the stage-discharge relationship and the velocity distribution-discharge relationship are presented. General limitation and site imposed constraints pertaining to each type of simulation technique are also presented.

The type of study site selected depends on the assumption that the study site is critical to the species under study, or that the study site is representative of a larger reach of stream. Either assumption may be implied or expressed in an instream flow methodology, and the study site reflects one of these assumptions. Study site preparation involves the strategic placement of transects which describe certain types of conditions or habitat areas within the channel. Further, the characteristics of the study site may have profound influences on one's ability to simulate the hydraulics of the stream.

Data requirements and precision specifications are provided, as well as guidelines for efficient data collection. Several appendices are included which detail data collection procedures for each type of data required for hydraulic simulation. The appendices also detail methods for collecting hydraulic data in large rivers, as well as equipment needed for such data collection. Finally, a brief description of several computer programs for hydraulic simulation is given in the appendices.

PREFACE

This document discusses the theory and limitations of different techniques of hydraulic simulation as well as site selection considerations and constraints, and data collection requirements and techniques. In the preparation of this information paper we have attempted to answer most of the questions which might arise during the hydraulic simulation phase of an instream flow study. However, it is highly unlikely that all conceivable situations could have been anticipated and addressed by this paper. Therefore, it is important for the reader to obtain a good understanding of the theory and potential limitations of a particular simulation technique. Such an understanding will aid in the rational selection of a technique which is consistent with the level of detail of the study, and the vagaries of the study site.

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PURPOSE AND SCOPE

The purpose of this paper is to familiarize the field investigator with the theoretical aspects of hydraulic simulations and limitations to various approaches. Discussions of the data requirements, site limitations, and level of reliability of different approaches will aid the investigator in the selection of the appropriate technique to meet the level of accuracy and reliability required for individual situations. The approach utilized should have a level of reliability in output which is consistent with the overall approach to the determination of the value of instream flows.

Descriptions of various approaches are designed for the technical nonengineer. Therefore, descriptions and discussions of the theoretical aspects of hydraulic simulation have been generalized and simplified somewhat. Conversely, the discussions of site suitabilities and limitations to methods imposed by site selection are very practical matters for the field investigator, and are discussed in considerable detail. Finally, data requirements and specifications for various approaches are given to aid the investigator in establishing a work plan and to ensure maximum preservation of reliability for any given approach.

Theoretical considerations for hydraulic simulation are discussed as two separate processes: (1) the determination of the stage-discharge relationship, and (2) the determination of the velocity distribution-discharge relationship. The discussion of the theoretical approaches of determining these relationships is followed by a discussion of the potential limitations and relative errors associated with the different approaches. Methodological concepts and limitations to approaches associated with site selection are introduced in a subsequent discussion. A final section discusses the data requirements and accuracy specifications for different approaches. Data collection procedures and equipment requirements are contained in several appendices, designed to aid the field practitioner unfamiliar with the data collection techniques involved with the various approaches. More detailed expositions of these topics may be found by review of the references included with this paper.

INTRODUCTION

The purpose of this paper is to describe the theory and data requirements for hydraulic simulation models. However, before the "how" of modeling is discussed, it is appropriate to discuss modeling and its use. Whether we choose to admit it or not, we all model at one time or another. A biologist standing on the bank of a river may visually assess the fishery habitat of a river at a particular flow and form an opinion about the fish's response to some alternate stream discharge. Having never been a fish, the biologist translates the things he can see (depth of flow, cover, bottom type, water movement, turbulence, etc.) into things he cannot see, namely, the attractiveness of those parameters from a fish's viewpoint. Thus, a "mental model" of the stream reach has been constructed, requiring certain data inputs (visual stimuli), data manipulations (pretending one is a fish), and interpretations (whether alternate stream flows would provide good habitat or not). Mental models based upon an understanding of certain processes are very useful for formulating a professional opinion when predicting the outcome of alternative situations. They are not very useful for communicating flow assessments from one person to another, particularly between individuals of different disciplines. Nor are they very useful when extrapolated beyond the range of experience of the observer.

In order to increase its transferability and predictive ability, a mental model may be expressed in written narrative form or in mathematical terms. This process allows a large, complex system to be broken down into a sequence of smaller, more predictable parts, which are then connected by a train of logic.

The primary differences between the mathematical model and a word model are that the complex interactions and relationships have been expressed as a variety of implicit or explicit functions, preferably explicit, and the logic train is clearly stated and exposed for examination. If the logic of a model (i.e., assumptions required to connect the parts) does not reflect the real world situation, it can be replaced by logic that does, or at least with logic which is accepted as approximating the real world situation.

The mathematical simulation model serves as the laboratory world in which the scientist can conduct experiments to test various hypotheses. Once the model has been built and verified, an inexhaustible number of management alternatives can be simulated and the relative difference between project impacts determined. There are several advantages to conducting experiments in this way. For example, one might wish to determine the change in depth and velocity of a river with changes in discharge. These changes could be measured directly over many flows, and an empirical relationship made for each variable. This process might take as long as a year or two to complete, and there would still be some flows which lie above and below the end points of the observations. These same parameters could be modeled in a much shorter time period, and the results extrapolated beyond the range of observation. Thus, modeling has the advantages of <u>time efficiency</u> and <u>extrapolative capability</u>.

Perhaps a value or some relationship used in the model is not known with any certainty, or perhaps a relationship is well documented, but its importance to the system is uncertain. Various trials can be made with the model, using different values for an uncertain parameter. If the output of the model changes drastically, the uncertain parameter is important to the system. If the uncertain parameter is not important to the system, manipulation of that parameter will not greatly influence the model output. Thus, a model may be employed for sensitivity analysis, which is very useful in establishing research and data collection priorities.

Hydraulic simulation for instream flow investigations is defined, for the purposes of this paper, as the description of the physical characteristics of a stream reach in mathematical terms. In the simplest of examples, a series of measurements of depth, velocity, and width across a cross section describes the spatial distribution of depths and velocities within the cross section at the discharge (flow) which was measured. Repetition of the process at many discharges would result in a different description of these distributions at each discharge measured. Extension of the process to several cross sections, related to one another by measurement, allows the description of such distributions for a stream reach.

Approaches utilized in the determination of the distribution of hydraulic parameters at different discharges vary considerably in the level of field effort and reliability of the output (i.e. predicted values of hydraulic parameters as compared to measured values for the same flow). The selection of a particular approach may be constrained by limitations of cost and manpower, limitations imposed by on-site considerations, and the required level of accuracy of the predictions.

In order for the mathematical simulation model to be useful for planning purposes, it not only has to reflect existing conditions (employ a rational logic process); but the model must also be capable of extension over space and into time (have predictive abilities). In reference to hydraulic simulation models, a researcher should select a study area which is representative of a much larger segment of stream, and be cognizant of the base line equilibrium condition of the watershed.

Extension of the model over space can be quite satisfactorily handled by proper study site selection. If, in fact, the study reach modelled is representative of a larger system, then the logic follows that the response of the study reach is indicative of the response of the system. Study site selection and the representative reach concept are presented in detail in pages 44 to 51.

Extensions of the model over time is irrevocably tied to the watershed hydrology and geomorphology. Whether a hydraulic simulation model or an empirical assessment of stream flow characteristics is used, the general procedure is to examine historic stream flow records, measure the physical characteristics of the stream under present day conditions, and then predict relationships which will occur at some future date. Axiomatically it has been assumed that neither the physical characteristics of the stream nor the hydrologic characteristics of the watershed will change, or have changed, significantly with time. It is further assumed that the period of hydrologic record is sufficiently long to have incorporated most of the random variability in stream flow which will ever occur in the watershed.

These are large assumptions to make, but in many instances they are valid. However, at other times, changes must be anticipated and incorporated into the logic train of the model. Whenever a substantial land use or channel change is anticipated, the biologist should be prepared to query the engineering/hydrology community concerning the resulting shape and character of the stream channel.

In order to ask the right questions, the biologist should have an elementary understanding of three basic concepts of fluvial geomorphology: equilibrium, aggradation, and degradation.

Aggradation refers to a persistent rise in the elevation of the stream bed, whereas degradation is a persistent lowering. A stream which is neither aggrading nor degrading is said to be in equilibrium. This equilibrium condition is not static. It fluctuates from year to year about some average condition as the stream experiences some annual pattern of scour and fill. As long as the net change in stream bed elevation over a number of years is near zero, the stream is said to exist in a state of "dynamic equilibrium."

For a stream which is in a state of dynamic equilibrium and is expected to remain so, study results can be extended into time with relative certainty even though a particular cross section's shape is noticeably changed by scour or fill. A corollary to the definition of "dynamic equilibrium" is that a stable relationship exists between sediment yield and stream flows in the watershed. Therefore, if one cross section is scoured, another is filled. The character of a specific study site may change, but if the river is truly in a state of equilibrium, its overall character or condition will remain constant and flow recommendations will be applicable into time.

Aggradation and degradation are usually insidious, long term, processes reflecting a fundamental change in the relationship between sediment yield and stream discharge in the watershed. Over the long term, these processes may have more profound effect on a fishery resource than discharge itself.

While it is difficult to quantify the magnitude of channel changes, a compilation of case histories assembled by Lane (1955) gives considerable insight into the types of changes to anticipate under different sediment yield/stream discharge conditions.

A reduction of discharge with no corresponding reduction in sediment load promotes aggradation. This type of impact is often associated with flood control reservoirs and out-of-stream diversions. The result of this type of perturbation is a reduction in both width and depth, and an increase in the width to depth ratio. Because the stream has less energy than required to transport sediment, the particle size of the substrate is typically reduced. These streams often experience encroachment by riparian vegetation which tends to stabilize the new channel configuration (Fraser, 1971).

An increase in the sediment load with no change in discharge will also result in aggradation, but with width increasing. Land clearing and gravel washing operations often stimulate aggradation, acccompanied by a reduction in substrate size. Channel braiding is often symptomatic of this type of aggradation. A reduction in discharge coupled with an increased sediment load will amplify and aggravate the situation.

A reduction of sediment with no change in discharge promotes a gradual removal of fines from the stream, leaving behind only large substrate materials. This process is termed armoring and is very common below reservoirs or stilling basins.

An increase in the discharge coupled with a reduction in sediment load results in a marked increase in depth and reduction in the channel's width-to-depth ratio. Armoring may be so pronounced that spawning gravels are removed as in the tailrace area of many dams. If an increase in discharge and reduction in sediment load occurs in braided alluvial channels, the braided channel may well revert to a single channel, a process which is presently occurring on the Bighorn River, Montana (Montana Department of Natural Resources, 1977).

The preceding discussion of the processes of aggradation and degradation is designed to alert the reader to the types of changes which can be expected to occur in a river channel in response to an imbalance in the sediment-discharge ledger. It is important that the concepts of equilibrium, aggradation, and degradation are well understood; for an instream flow recommendation which is perfectly adequate from a biological aspect may be completely undone by geomorphic processes which were ignored during the development of the recommendation.

DEFINITIONS OF TERMS
USED IN THE ANALYSIS OF OPEN CHANNEL FLOWS

The terms of importance in the analysis of open channel flow are defined below, and illustrated in Figure 1.

Width (w) - the distance across a channel at the water surface measured normal to flow.

Depth (d) - the vertical distance from a point on the bed to the water surface.

Thalweg depth (y) - the vertical distance of the lowest point of a channel section to the water surface. Maximum depth of cross section.

Thalweg - the longitudinal line connecting points of minimum bed elevation along the streambed.

Hydraulic depth (d) - equivalent to mean depth. $d = A/w$.

Stage - the elevation, or vertical distance of the water surface above a datum (a plane of known or arbitrary elevation).

Cross-sectional area (A) - the area of the cross section containing water, normal to the direction of flow. Computed as width X mean depth of cross section.

Wetted perimeter (P) - the distance along the bottom and sides of a channel cross section, in contact with water. Roughly equal to the width + 2 times the mean depth.

Hydraulic radius (R) - the ratio of the cross sectional area to the wetted perimeter, $R = A/P$. For wide shallow channels, R approximates the hydraulic depth.

Hydraulic slope (S_h) - the change in elevation of the water surface between two cross sections, divided by the distance between the cross sections.

Bottom slope (S_o) - the change in the average elevations of the bed between two cross sections, divided by the distance between them.

Thalweg slope - the change in the elevation of the bed, measured at the point of maximum depth (y), divided by the distance between cross sections.

Energy slope (S_e) - change in total energy (potential and kinetic) available, divided by the distance between cross sections. See discussion of Bernoulli's equation in the next section.

Channel roughness (n) - a coefficient of resistance to flow caused by particle friction and channel features.

Uniform Flow and Varied Flow - <u>Uniform Flow</u> by definition means that the depth of flow is the same at every section of the channel. Thus, the hydraulic, energy, and bottom slopes are parallel. If the flow is <u>varied</u>, the depth of flow changes along the length of the channel. Varied

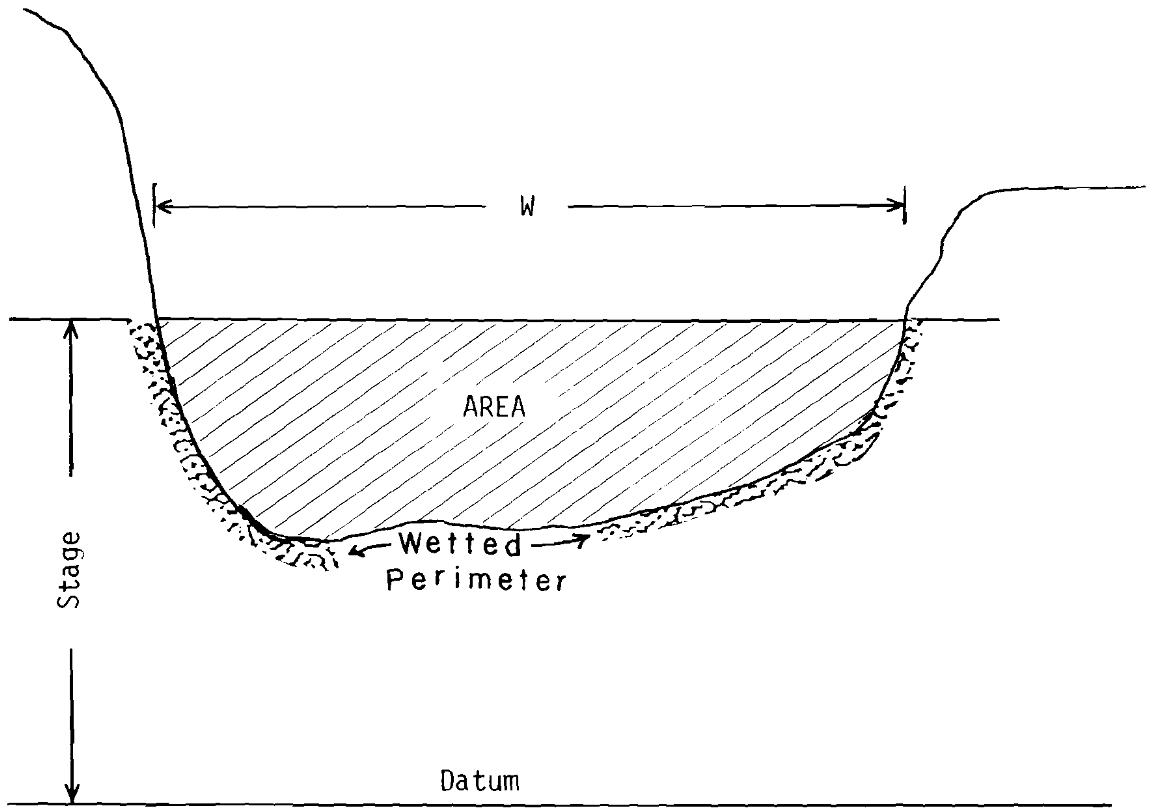


Figure 1: Channel geometry elements used in the analysis of open channel flow.

flow is classified as either rapidly or gradually varied, depending on the distance within which the change in depth occurs. Rapidly varied flow is manifest in an abrupt change in depth, resulting in hydraulic jumps, hydraulic drops, and related phenomena. The criterion for uniform or varied flow is change in depth with respect to space.

Steady Flow and Unsteady Flow - Flow in an open channel is said to be steady if the depth of flow does not change or can be assumed constant over a specified time interval. The flow is unsteady if the depth changes with time.

Sub-critical, Critical, and Super-critical Flow - In any body of moving water both inertial and gravity forces are acting on the water body. The effect of gravity on the state of flow is represented by the ratio between inertial and gravity forces. This ratio is given by the froude number, defined as

$$F = \frac{V}{\sqrt{gD}}$$

where,

V = mean velocity
g = the acceleration of gravity
D = the hydraulic depth

If F is less than unity, gravity forces predominate, so the flow has low velocity and is described as tranquil or streaming. If F is greater than unity, the effects of inertia are more pronounced, so the flow has high velocity and is described as shooting, rapid, or torrential. When F is equal to unity, flow is defined as critical.

Most instream flow studies are concerned primarily with the subcritical state of flow, although hydraulic simulations for certain recreational activities may deal with super-critical states of flow.

COMMONLY USED EQUATIONS FOR THE ANALYSIS OF OPEN CHANNEL FLOWS

The water surface elevation in a stream defines the cross-sectional area of flow. If the velocity is also known, the discharge can be calculated using the equation of continuity:

$$Q = AV \qquad (1)$$

where,

Q = discharge in cubic feet per second (cubic meters per second).
A = area of the cross section of flow in square feet (square meters).
V = average velocity of flow through the cross section in feet per second (meters per second).

Over the years, considerable empirical and theoretical research has been conducted on the relationship between channel features and the velocity in the channel. The first velocity equations were empirical relationships based on the observed behavior of flow in open channels. Around 1770, a French engineer, Antoine Chezy, developed the first velocity equation, which is now known as the Chezy equation:

$$V = C\ (RS)^{\frac{1}{2}} \qquad (2)$$

where,

V = average velocity in a channel
C = Chezy constant (empirically derived for each channel).
R = hydraulic radius
S = S_e, slope of the energy grade line

In 1889, an Irish engineer, Robert Manning presented a velocity equation, known in its present form as the Manning equation:

$$V = \frac{1.486}{n} R^{2/3} S_e^{\frac{1}{2}} \qquad (3)$$

where,

V = the mean velocity in the channel, in feet per second.
R = the hydraulic radius, in feet
S_e = slope of the energy grade line
n = coefficient of roughness, referred to as Manning's n.

The version of the Manning equation given as equation 3, is in English units. If metric measurements are input for R, the term 1.486 is omitted from the equation, and V will be given in metric equivalents to R.

The discharge may be calculated using either of the above velocity equations, and substituting for V in the continuity equation (equation 1). The Chezy equation for determining discharge becomes:

$$Q = C\ R^{\frac{1}{2}}\ S_e^{\frac{1}{2}}\ A \qquad (4)$$

and the Manning equation may be expressed as:

$$Q = \frac{1.486}{n} R^{2/3} S_e^{1/2} A \tag{5}$$

The range in Manning's n reported by Henderson (1966) are:

Man-made channels	
Concrete	0.012 - 0.014
Rubble set in concrete	0.017
Earth, smooth, no weeds	0.020
Earth, some stones and weeds	0.025
Natural river channels	
Clean and straight	0.025 - 0.030
Winding, with pools and shoals	0.033 - 0.040
Very weedy, winding and overgrown	0.075 - 0.150
Clean straight alluvial channels	$0.031\ K^{1/6}$

In the last expression, K is the size of bed material for which 75 percent of the bed material is smaller and 25 percent larger, measured by the median axis of the particle. The equation $0.031\ K^{1/6}$ is applicable to flows at high river stages and is not appropriate for low flows.

In both the Chezy and Manning formulae, the slope required as an input is the slope of the energy grade line. This slope is defined as the difference in total energy at two (or more) channel sections, divided by the distance between them. The total energy at a channel section is found with the open channel form of the Bernoulli equation:

$$H = z + d + \frac{V^2}{2g} \tag{6}$$

where,

H = total energy head, in feet (meters)
z = elevation of the bed, in feet (meters)
d = average depth for section, in feet (meters)
V = average velocity in feet per second (meters per second)
g = acceleration of gravity, 32.2 ft/sec^2 (9.8 m/sec).

For practical purposes, it can be seen (Figure 2) that the terms z + d equal the water surface elevation for a given cross section. Referring to Figure 2, the slope of the energy grade line is:

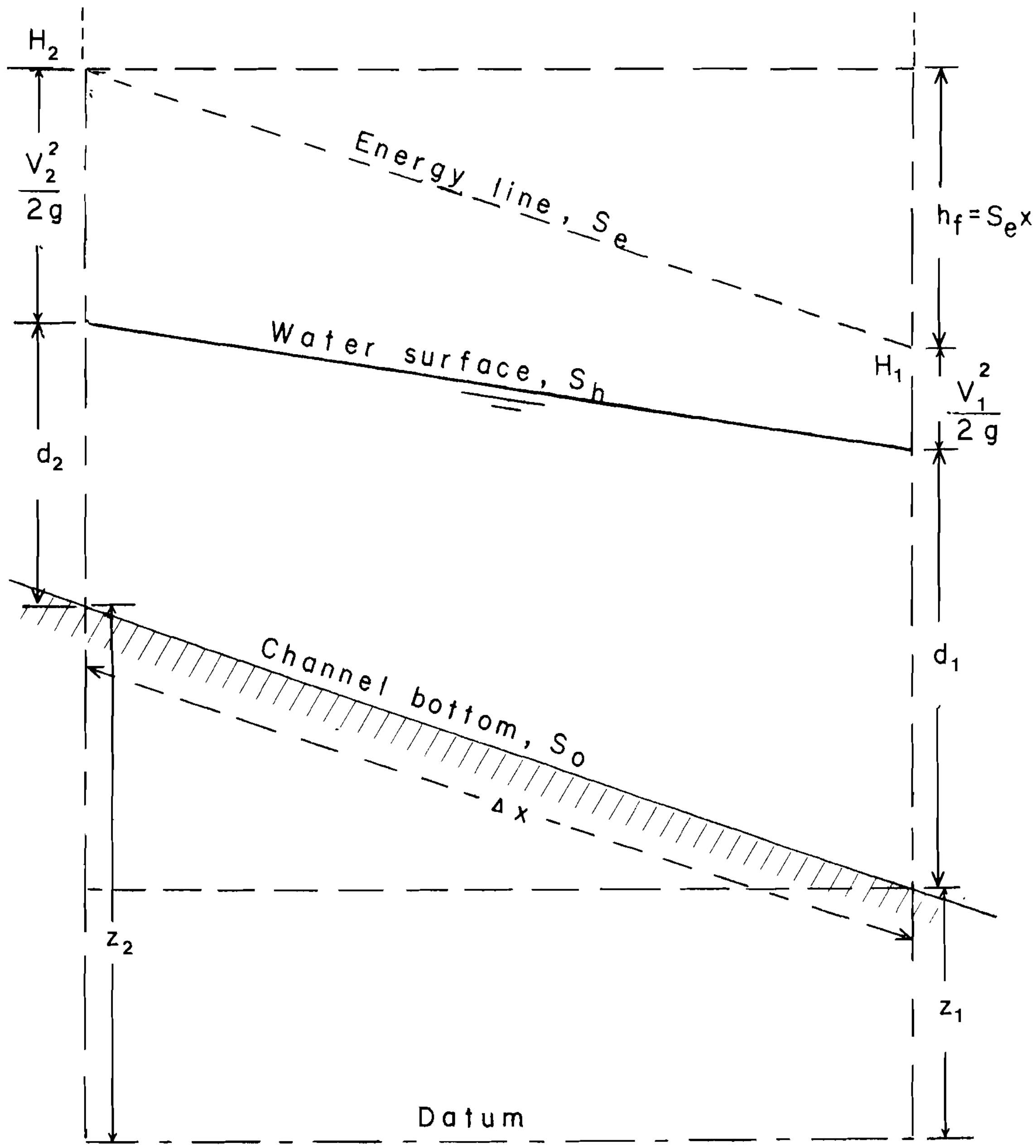


Figure 2: Energy elements used in the analysis of open channel flow.

$$S_e = \frac{H_2 - H^1}{\Delta x} \quad (7)$$

If the assumption is made that flow in the channel is uniform, then the bed slope, hydraulic slope, and energy slope are considered equal, $S_o = S_h = S_e$.

PREDICTING THE STAGE-DISCHARGE RELATIONSHIP

The determination of the relationship between the stage at a cross section and the discharge associated with that stage must be considered the initial step in the type of simulation used for instream flow assessments. Once the stage has been determined for a certain discharge, its elevation is used for simulation in two ways: (1) The depth distribution is found for each cross section by subtraction of bed elevations across the channel from the stage. Thus, if the stage and bed elevations are known, the depth may be determined at any location on the cross section; and (2) The stage identifies the location of the free surface, and is used to establish boundaries for some of the equations used to describe the velocity distribution.

Several approaches may be used in the prediction of the stage discharge relationship. Approaches described in this section include: (1) Use of Manning's equation, uniform flow assumed; (2) Calculation of water surface profile under conditions of gradually varied flow; and (3) Direct determination with varying numbers of measurements.

MANNING EQUATION, ASSUMING UNIFORM FLOW CONDITIONS

This approach can be used to determine the stage-discharge relationship for individual cross sections. The uniform flow assumption allows the use of the measured hydraulic slope instead of the energy slope, since by definition, they are equal. In addition, this approach assumes that flow variations caused by changes in channel configuration are negligible.

Generally, the more uniform the channel, the more reliable the results using this approach. As the channel becomes less uniform, the reliability of the results deteriorates.

Under this approach, the Manning equation is solved for n at one discharge, for which the following measurements must be made: (1) The water surface elevation (stage) and the discharge at the measured flow; (2) The hydraulic slope; and (3) The dimensions of the channel cross section.

The cross-sectional area and hydraulic radius are determined by the cross-sectional measurements and the stage. Manning's n may then be computed for the cross section by equation 5:

Solving for n,

$$n = \frac{1.486}{Q} R^{2/3} S^{\frac{1}{2}} A \qquad (8)$$

Manning's n is then assumed constant in subsequent calculations where new stages are calculated for different discharges, using equation 5.

WATER SURFACE PROFILES UNDER VARIED FLOW CONDITIONS

In most cases, the assumption of uniform flow cannot be made, either because of channel conditions or because of accuracy requirements of the instream flow study. The computation of the water surface profile is a means of more accurately determining the stage-discharge relationship with little more effort than the previous method. While the computations are considerably more complex, there are several computer programs available which are capable of rapid computation of the water surface profile. Program names and descriptions may be found in Appendix E, and may be used for this computation procedure.

The determination of the water surface profile requires essentially the same kind of data as the previous approach. However, the computation procedure is much different. This approach determines the energy losses between two cross sections under assumed conditions of depth and roughness. The following discussion of this method is very general. For specifics, the reader is referred to the discussion of gradually varied flow in Chow (1959). Given the discharge, the elevation of the bed and distance between cross sections, and an assumed value for Manning's n, the computations follow this general sequence.

1. Starting at the downstream-most cross section, a water surface elevation is assumed or given. For the next section upstream, an elevation is assumed; this elevation will be verified or rejected on the basis of subsequent calculations.

2. The depth of flow is computed for the corresponding water surface elevations.

3. The cross-sectional area is determined from the channel dimensions and assumed water surface elevation.

4. The mean velocity is calculated, using the continuity equation for the known discharge and cross-sectional area.

5. The velocity head ($V^2/2g$) is calculated, and the total head determined by addition to the starting water surface elevation.

A separate set of calculations is then made using the Manning equation:

6. The hydraulic radius is determined for the cross section, using the above assumed water surface elevation.

7. The energy slope between adjacent cross sections is determined by:

$$S_e = \frac{n^2 V^2}{2.22 R^{4/3}} \quad (9)$$

where,

n = the assumed value for Manning's n
V = the mean velocity calculated in step 4 above
R = the hydraulic radius from step 6 above.

8. The friction loss between the two adjacent cross sections is found by multiplying the average energy slope by the distance between stations.

9. This friction loss is added to the computed total head at the first station, to give the total energy head at the next upstream station. If the value obtained does not agree closely with that found in step 5, a new water surface elevation is assumed and the process repeated until agreement is obtained.

10. Even though internal agreement may be obtained within the computations, the computed water surface elevations may not agree with those measured in the field. In this case, the value of Manning's n is changed, and the process repeated until the energy-balanced water surface elevations "calibrate" with the observed water surface elevations.

11. Once calibration is achieved, Manning's n is assumed constant, and the flow profile computed for other discharges of interest.

If a range of flows very different from the field measured flows are of interest, the stage-discharge approach discussed in the following section is most appropriate.

DIRECT DETERMINATION OF STAGE-DISCHARGE RELATIONSHIP

The most accurate method of obtaining a relationship between stage and discharge is to measure the discharge at various stages and to develop an equation relating discharge to stage.

A stage-discharge relationship is influenced by a number of channel factors such as cross-sectional area, shape, slope, and roughness. The interaction of these factors "control" the stage-discharge relationship. If the stage-discharge relationship does not change with time, the control is stable and can be used without adjustment for changes over time.

The stage-discharge equation is of the form:

$$Q = a\,(s - ZF)^b \tag{10}$$

where,

- Q = discharge
- s = stage
- ZF = point of zero flow
- a and b = constants derived from measured values of discharge and stage.

The stage-discharge relationship for Oak Creek near Corvallis, Oregon, is given in Figure 3.

To determine the stage for any cross section, a least-squares equation is determined from a log-log plot of discharge against stage. For any interpolated or extrapolated discharge, the stage is calculated directly from this empirical equation.

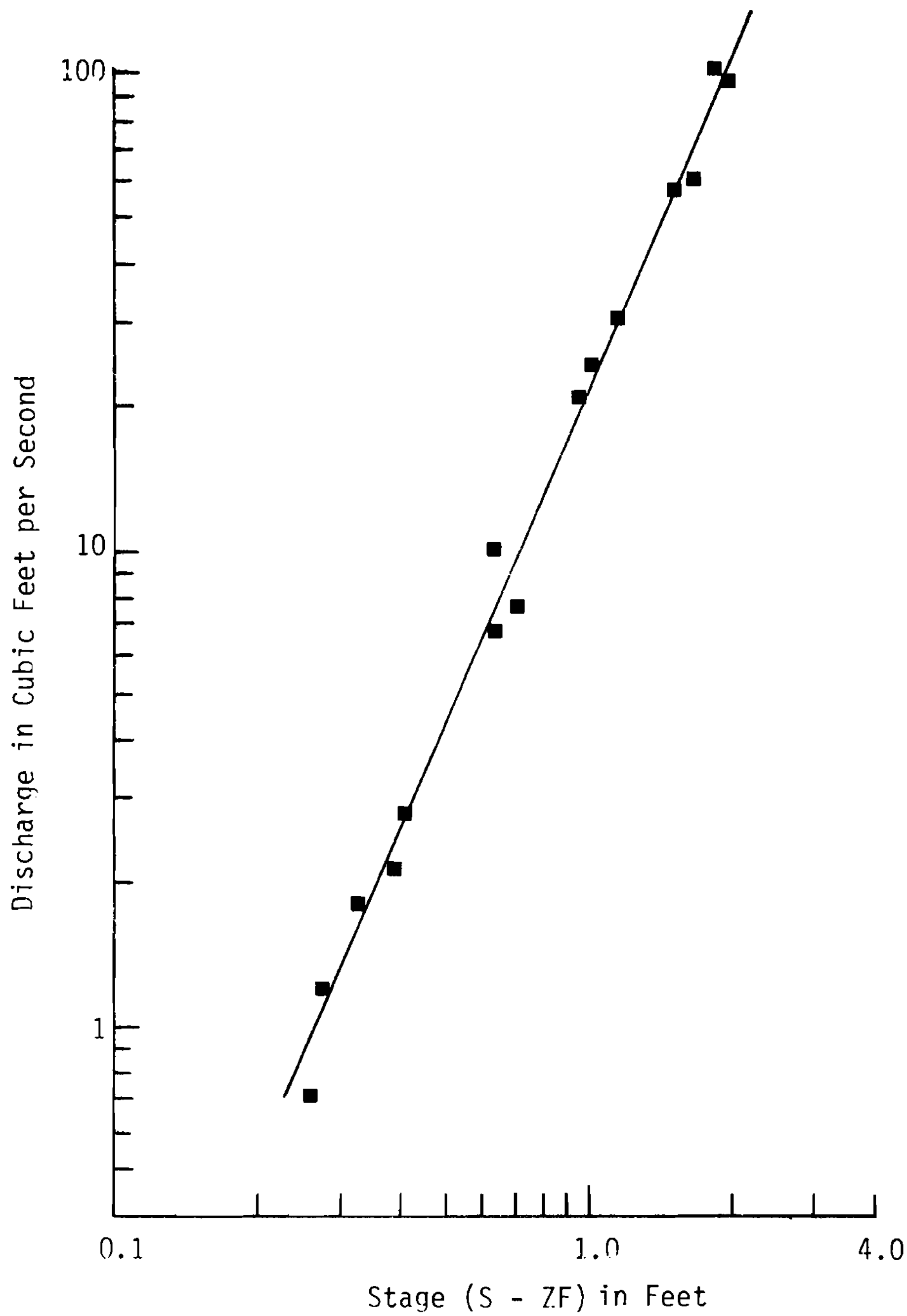


Figure 3: Stage-discharge rating curve for Oak Creek, near Corvallis, Oregon.

PREDICTING THE VELOCITY DISTRIBUTION

If the velocity distribution is measured for each flow of interest, the data can be used directly and no analytical procedure is needed to estimate the velocity distribution. In most cases, only a limited amount of resources is available to do field work in any particular instream flow study; hence, estimates must be made of the velocity distribution of flows for which velocities were not measured.

Velocity predictions are made using techniques which are similar to those used to predict stage. However, for any discharge there is only one stage, whereas the velocity varies from place to place across the section. It is important here to define what is meant by a velocity distribution in instream flow studies. Figure 4 illustrates two ways of expressing the velocity distribution in a channel. Figure 4a shows the distribution as a series of contour lines connecting points of equal velocity. Figure 4b shows the velocity distribution as a series of mean velocities in a group of adjacent channel subdivisions. The conceptualization of the velocity distribution for most instream flow studies is the type shown in Figure 4b. Essentially, each subsection or channel segment is treated as a separate channel, with its own depth, substrate, and average velocity. Any number of subdivisions may be used to define the velocity distribution in this manner; the more channel segments, the more detailed the description of the velocity distribution. In actual practice, around 20 subdivisions are most commonly used, although there is no firm limitation to this number.

In the following discussions, approaches to estimating the velocity distribution in a cross section are described. The first section describes the use of the Manning equation where no velocity measurements are made to calibrate the equation. The second section discusses the calibration of Manning's n with a series of measured velocities at one flow. The third section describes a procedure using more than one set of measured velocities.

MANNING EQUATION WITH NO VELOCITY MEASUREMENTS

This approach requires the stage-discharge relationship to be known from the previous computation procedures. Other data requirements include the dimensions of the cross section and the slope (S_h if uniform flow assumption is made, S_e if gradually varied flow).

The computation procedure is started by subdividing the cross-section into a series of channel segments, as shown in Figure 5. Each channel segment has the geometric properties of cross-sectional area (a_i), hydraulic radius (r_i), and each has a roughness coefficient (n_i). The following assumptions are made to continue the computation procedure:

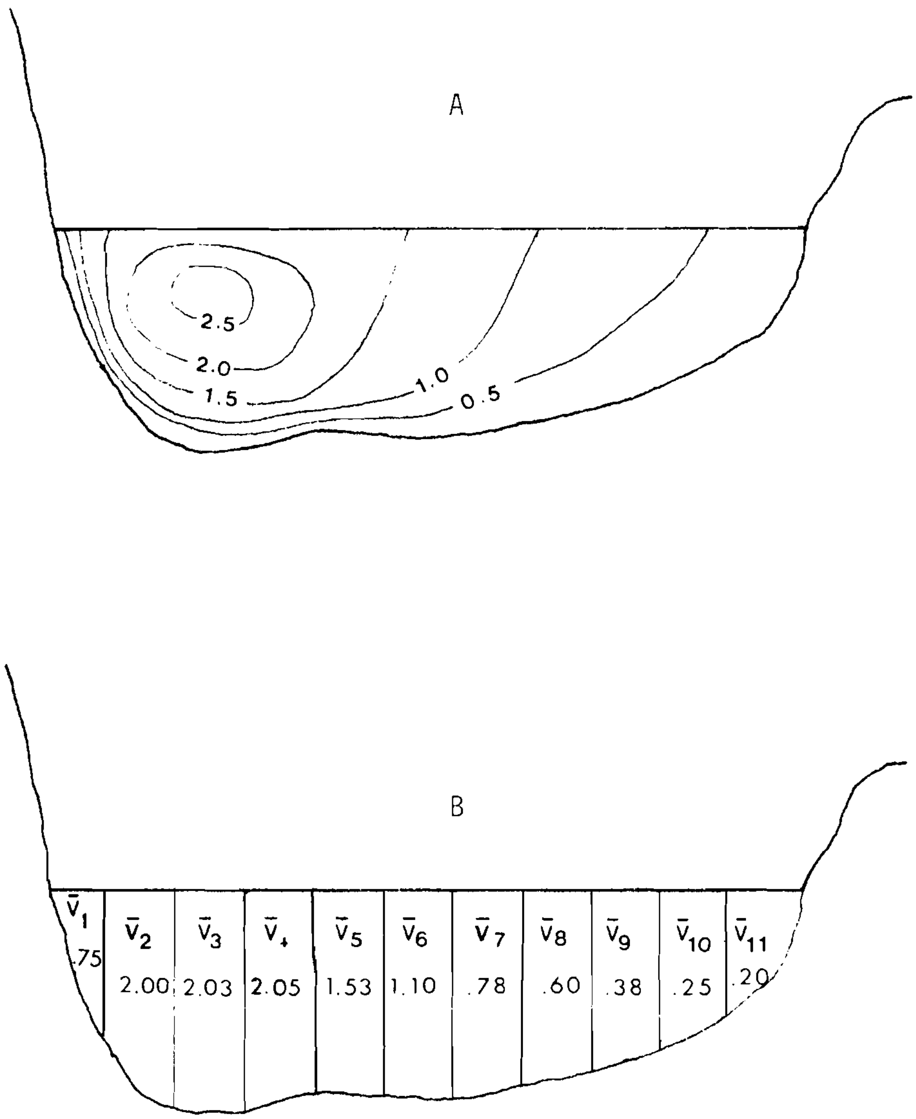


Figure 4: Conceptualized view of the velocity distribution in a natural channel: (A) as a series of contour lines connecting points of equal velocity, and (B) as a series of channel segments, each with its own mean velocity.

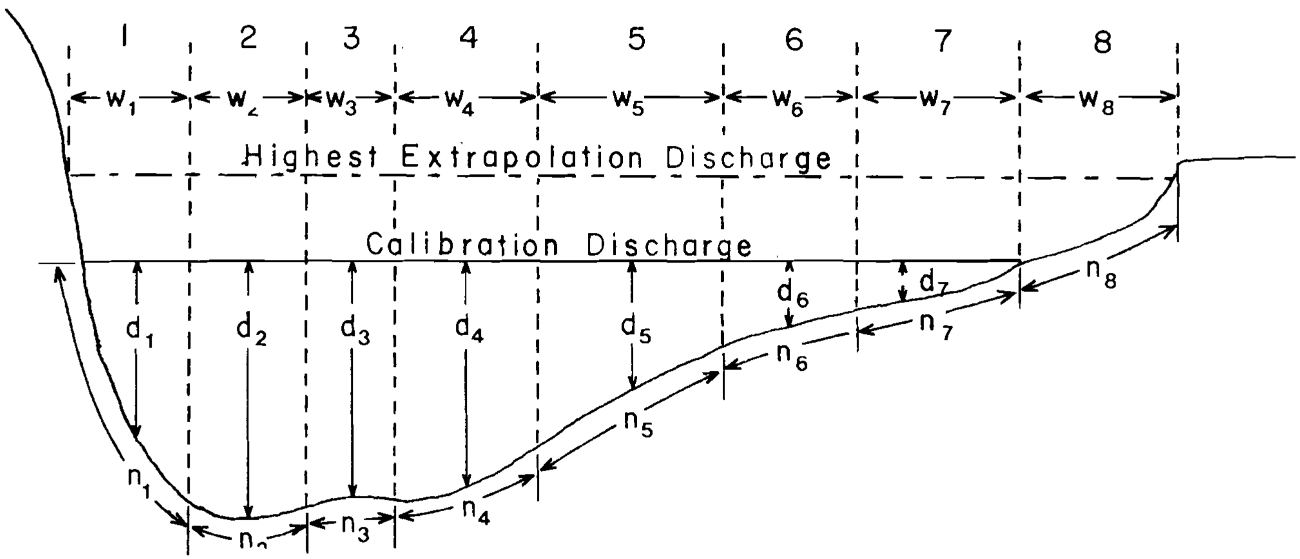


Figure 5: Subdivision of a cross section into a series of channel segments, each with geometric elements particular to the channel segment.

1. The slope is the same for all channel segments.
2. There is no slope of the water surface normal to the direction of flow.
3. Each channel segment is rectangular.

The mean velocity for each channel segment may be calculated from the Manning equation as follows:

$$v_i = \frac{1.486}{n_i} r_i^{2/3} S^{\frac{1}{2}} \qquad (11)$$

where,

v_i = the mean velocity of the channel segment
r_i = the hydraulic radius (a_i/w_i) for the channel segment, based on the stage as determined previously, and on the dimensions of the segment.
S = the slope, as previously described
n_i = the roughness coefficient for the channel segment.

The calibration of this equation could be simplified considerably by assuming that the roughness coefficient is the same for every channel segment (i.e. $n_1 = n_2 \ldots = n_i = n_o$), where n_o is the roughness coefficient for the whole channel as determined in the computation of the stage-discharge relationship. The validity of this assumption depends on the uniformity of the channel and channel materials, the roughness of the banks, and so forth. In some situations, it will be apparent that the assumption of constant roughness will not be true. In other cases, there will be channel segments which will be out of the water at the time the calibration measurements were made (for example, segment 8 in Figure 5). Either situation may require an estimation of Manning's n for a particular channel segment.

Where the roughness of a particular segment is unknown the relative roughness can be specified. For instance, the segment with the lowest roughness may have a specified value, n_o, and the remainder have a roughness related to n_o by a constant, c_i. The roughness of a particular segment is then expressed as:

$$n_i = c_i \, n_o \qquad (12)$$

The value of c_i may be estimated by comparison of the size of bed materials in adjacent channel segments, by the following expression:

$$c_i = \left(\frac{d_i}{d_o}\right)^{1/6} \qquad (13)$$

where,

c_i = roughness modifier for each channel segment

d_o = median diameter of the particle size which is larger than 75% of the bed material, in the segment where Manning's n is known. This may be related to the segment where n is lowest, or to the mean value of n for the whole cross section.

d_i = the median diameter of the particle size which is larger than 75% of the bed material, for the channel segment in question.

In order to simplify data collection to satisfy the changing nature of n across a channel, many investigators base their initial channel segmentation on substrate changes or "breaks" across the channel. A problem with equation 12 is that form roughness (resistance caused by bed configuration) and vegetation roughness are not included in the estimation. In these cases, estimates of roughness for different types of channels may be made using tables, diagrams, and pictures from Chow (1959) or Barnes (1967).

MANNING'S EQUATION WITH ONE SET OF VELOCITY MEASUREMENTS

Referring to the cross section shown in Figure 5, suppose that in addition to the slope, width, depth, and discharge, a measurement of the velocity was made at each vertical column separating each channel segment. Such velocity measurements would be repeated for each cross section. Each channel segment would then have an average velocity, calculated from the measured velocities on either side of the segment. In this case, the roughness for each channel segment may be calibrated using the Manning equation for each channel subdivision:

$$n_i = \frac{1.486}{V_i} d_i^{2/3} S^{\frac{1}{2}} \qquad (14)$$

This approach is applicable where the flow for which velocities are being predicted is wholly within the bounds of the channel segments containing the calibration flow. If the new flow is higher than the calibration flow, the roughness in an uncalibrated segment (i.e., segment 8 in Figure 5) may be estimated by comparison with an adjacent calibrated segment. This estimation may be made by visual observation and judgement, or by the method presented previously. In this case:

$$n_i = c_{i,m} n_m \qquad (15)$$

where,

n_i = the roughness of the uncalibrated channel segment

$c_{i,m}$ = a constant relating segment i to segment m, the calibrated segment

n_m = the roughness of the adjacent, calibrated segment

The value of $c_{i,m}$ may be estimated by similar logic to equation 12, where:

$$c_{i,m} = \left(\frac{d_i}{d_m}\right)^{1/6} \tag{16}$$

where,

d_i = the particle size in the uncalibrated segment

d_m = the particle size in the calibrated segment, with the particle size in both cases, the median diameter of the particle size which is larger than 75% of the bed materials.

As in the previous discussion, this approach is applicable when changes in roughness are caused primarily by changes in particle size. Roughness related to form and vegetation may be estimated using references such as Chow (1959) and Barnes (1967).

DIRECT DETERMINATION OF THE VELOCITY DISTRIBUTION

Figure 6 shows a cross section in which the velocity of each channel segment is determined for each of three different discharges. The average velocity for any channel segment where two or more such velocity measurements have been made, may be related to the total discharge

$$v_i = a_i \, Q^{b_i} \tag{17}$$

where, v_i is the mean velocity of the i-th channel segment when the total discharge of the stream is Q. The constants a_i and b_i are obtained by fitting a least squares regression to two or more velocity-discharge data pairs. For discharges not measured, v_i is found by applying the empirical constants a_i and b_i to the discharge for which an estimate of v_i is desired.

The concept that the average velocity in a cross section is related to the discharge by an equation $v = a\,Q^b$ appears to be well accepted in the literature (Park, 1977). The assumption is made that the average velocity in a channel segment is also related to the total stream discharge by an equation of the same form.

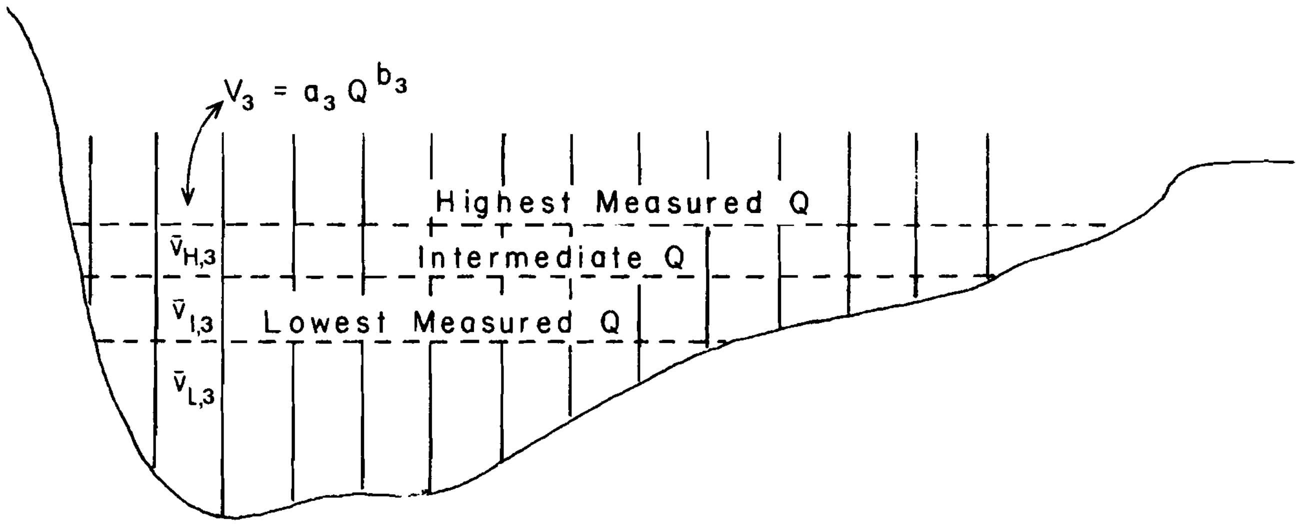


Figure 6: Segmented channel with the mean velocity of each segment measured at each of three different discharges to establish a velocity-discharge rating for the segment.

LIMITATIONS AND ERROR ANALYSIS OF APPROACHES

Some of the problems encountered in the application of these methods are more serious in the engineering sense than they are for instream flow applications. For example, use of the Manning equation, assuming uniform flow, or even the stage-discharge approach, can result in predicted water surface elevations higher at downstream stations than at upstream stations. This may be caused by errors introduced during data collection, or by failure to account for changes in the bed elevation adequately. This problem can result in the appearance that water is running uphill, cause for alarm in engineering work. This error is real and reflects a mistake, but in respect to depth, this error is usually small. If sections are 100' apart or more, an error of ±0.05 feet is acceptable; if outside this range, additional data collection may be warranted. The following section deals with the more serious limitations of the various approaches which can have an effect on the accuracy and reliability of the predicted hydraulic conditions and, therefore, on the instream flow recommendation itself.

GENERAL LIMITATIONS IN HYDRAULIC SIMULATION

The major problem encountered with the types of hydraulic simulations discussed in this paper, is that predictions of hydraulic conditions are made using as few discharge measurements as possible. Over a wide range of flows, certain processes occur in the natural channel which affect the relationship between stage and discharge, as well as the velocity distribution. These processes may introduce significant errors into any hydraulic prediction, regardless of the method used. Several characteristics of open channel flow and factors affecting the stage-discharge relationship are discussed below. Factors related to on-site channel conditions will be discussed in greater detail in the section on site selection.

Extended over a very wide range of flows, from essentially zero flow to overbank, many channels will exhibit an S-shaped rating curve such as that shown in Figure 7. As the channel fills from a zero flow stage, much of the discharge is accommodated by increasing the width of channel filled with water. After the channel width is essentially filled, increases in discharge are accompanied by well-behaved increases in stage. If the flows of interest lie along this straight portion of the rating curve, accurate predictions are possible with relatively few measured discharges. As the discharge overtops the banks, the additional flow is again reflected in an increase in width (and velocity), and the increase in stage is proportionately less. The degree to which changes in the stage-discharge relationship occur is primarily a function of channel shape. If the range of flows of interest for instream

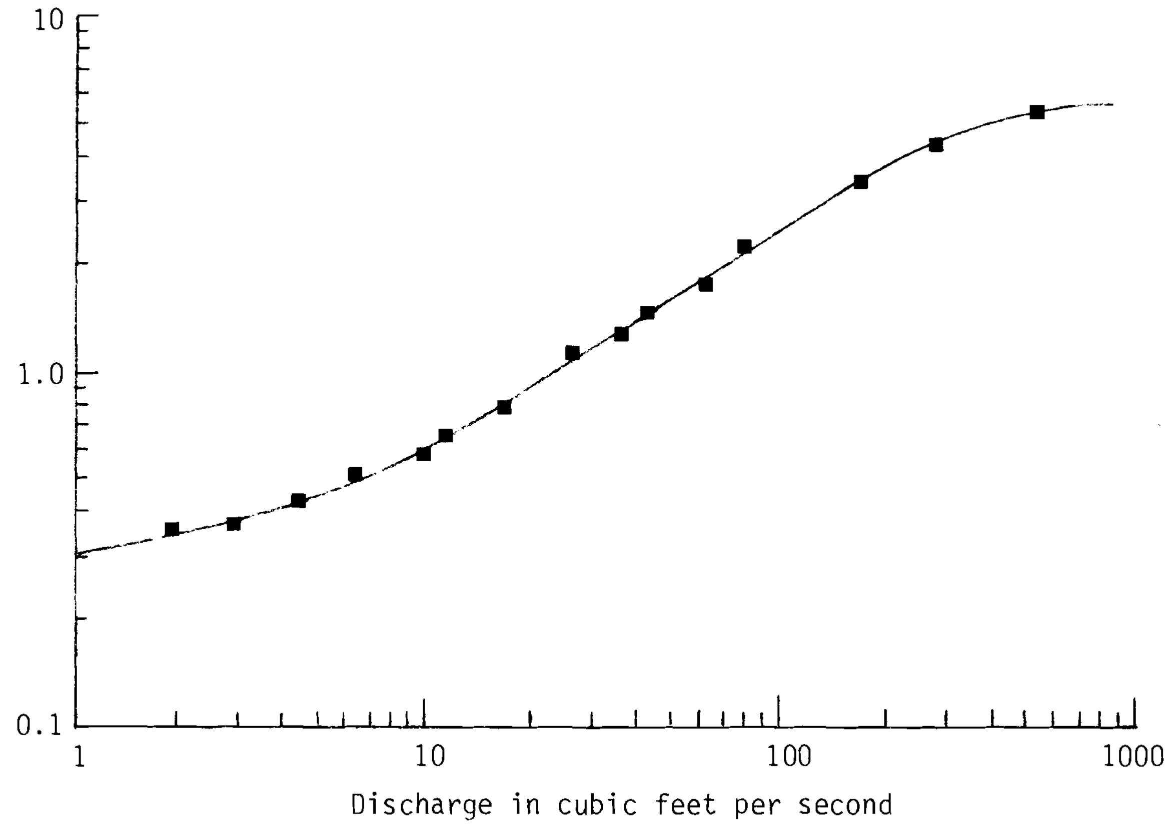


Figure 7: S-shaped rating curve resulting from plotting stage versus discharge over a very wide range of flows.

flow studies includes extremely low or high flow conditions, the investigator should be prepared to collect additional stage-discharge data as required to define the rating curve under those conditions.

A related phenomenon occurs in alluvial channels (a channel cutting through previously deposited materials; primarily, those streams having unconsolidated gravel, sand, or silt beds). In these streams the bed material is often moved by the stream, especially at the higher discharges. These streams often exhibit a stage-discharge relationship similar to the one shown in Figure 8. Such streams are usually relatively stable between periods of high flow. Care should be taken not to overlap the high flow period during the data collection process.

If a point of interest is located just upstream from its juncture with a larger stream, the flow in the larger stream may control the stage in the smaller stream. This phenomenon is called "variable backwater," because at some times of the year the backwater is present, and at other times it is either absent or of much reduced extent in the tributary stream. The simplest solution to the problem of variable backwater is to avoid areas where they occur. If this is not possible, a stage-discharge relationship can be established during the period of the year when the backwater is not present or is of reduced extent. Assessment of the backwater during high flow periods should be tied to the stage-discharge relationship of the main stem river.

Other factors commonly influencing the stage-discharge relationship include aquatic vegetation and ice. The effect of aquatic vegetation is to increase the roughness and to decrease the cross-sectional area of the channel. The result is an increase in stage for a given discharge in comparison to the channel without aquatic vegetation. In streams with an attached algae, such as *Cladophora*, the stage-discharge relationship when the algae is dormant may be much different than while it is growing rapidly. In this situation, it may be necessary to construct two rating curves, one for the growing season and one for the non-growing season.

Ice has a complex impact on the stage-discharge relationship. The presence of ice decreases the cross-sectional area, changes the wetted perimeter, and changes the roughness. The types of changes involved depend on whether the ice is surface-formed (sheet ice) or bottom-formed (anchor ice). It is generally advisable not to attempt to construct a rating curve, or use the Manning equation when ice is present in the stream.

For the most part, these generalized limitations should serve as warnings to the field practitioner, not prohibitions. A stage-discharge relationship may be developed under any of the conditions mentioned above. However, if these conditions exist, extra care in study planning and data collection are warranted.

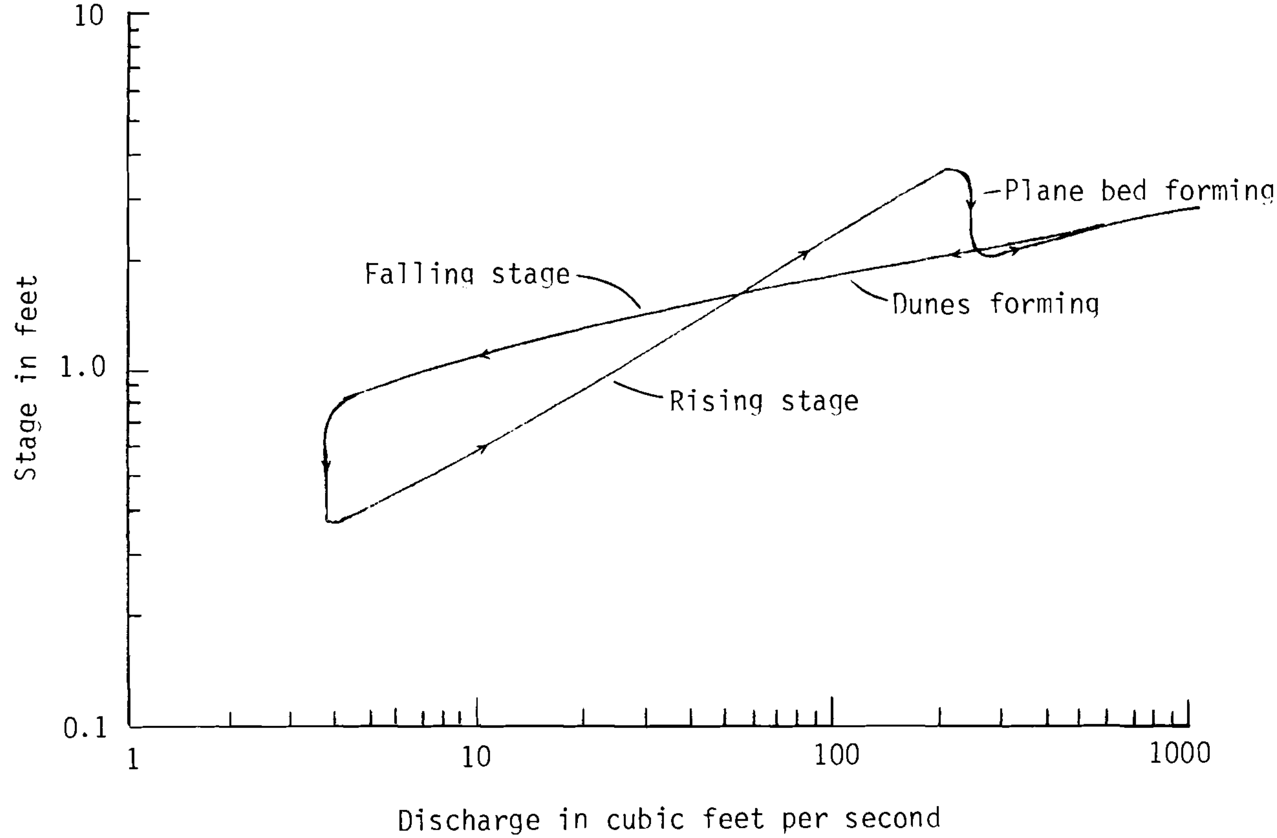


Figure 8: Rating curve loop typically resulting from the movement of bed materials in alluvial channels. After Simons (1976).

LIMITATIONS TO THE USE OF THE MANNING EQUATION

From the instream flow assessment standpoint, the greatest advantage in the use of Manning's equation is that only one set of measurements is needed to calibrate the equation. For an agency charged with determining instream flow requirements of many streams, frequently with little time or resources to accomplish the task, this advantage may be extremely attractive. Unfortunately, the same factors giving the advantage are also involved in the limitations to the approach.

As discussed previously, Manning's n may vary from place to place in the channel. However, both the energy slope and Manning's n also vary with discharge. An example of this variation is illustrated by data from Oak Creek in the Oregon Coast Range near Corvallis, Oregon. Figure 9 shows the variation in Manning's n over a wide range of discharges in Oak Creek. The variation in energy slope is shown in Figure 10.

The practice of taking only one set of calibration measurements means that the value of Manning's n and the energy slope are known with certainty for only one flow. In other calculations, both variables may be assumed constant, Manning's n alone held constant, or adjustments made to Manning's n based on an estimate of its value at other flows. However, unless several sets of measurements are available, the true value of n is not known with certainty for any but the calibration flow(s).

The relative importance of the energy slope and Manning's n in the introduction of error can be seen by comparing the range of values in Figures 7 and 8. The maximum range of the energy slope is from 0.009 to 0.012. When the square root is taken, this results in a range in $S_e^{1/2}$ of 0.095 to 0.109, which would result in a maximum error in predictions of about 13% if the slope were assumed constant, and estimated at one of the extremes of slope.

In contrast, the range of Manning's n is from about 0.075 to almost 0.5. If the range of flows of interest for Oak Creek were from 10 to 100 cfs, the variation in n would be from about 0.10 to 0.075, resulting in maximum potential error of about 133% if n is assumed constant. If the range of flows of interest were between 5 and 30 cfs, the range in n is from about 0.15 to 0.075 and represents a maximum potential error of 200% if n is assumed constant. Therefore, the reliability of the Manning equation for making hydraulic predictions from one set of calibration measurements is limited by the range of flows of interest and the extent of extrapolation from the calibration flow.

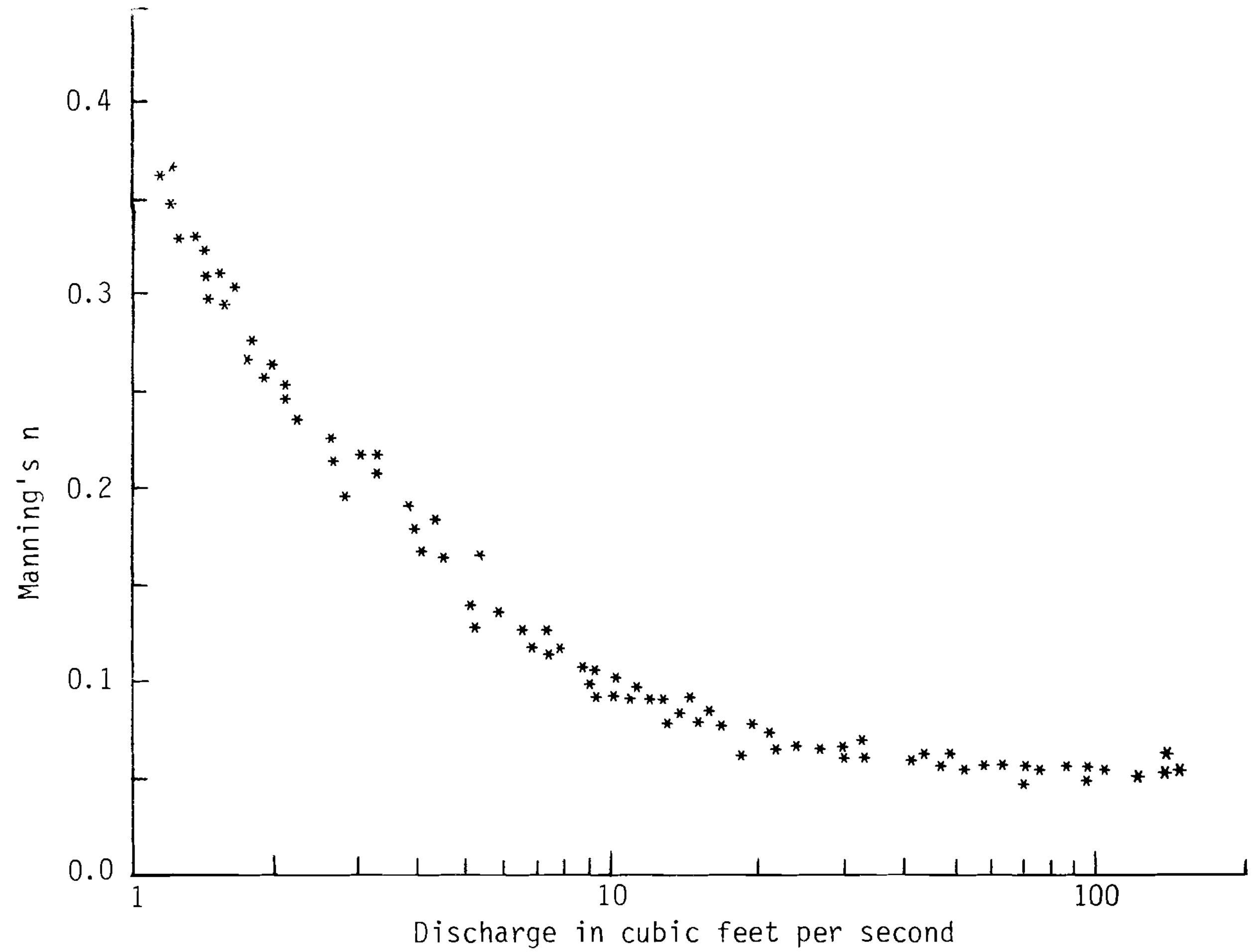


Figure 9: Relationship between Manning's n and river discharge for Oak Creek, near Corvallis, Oregon.

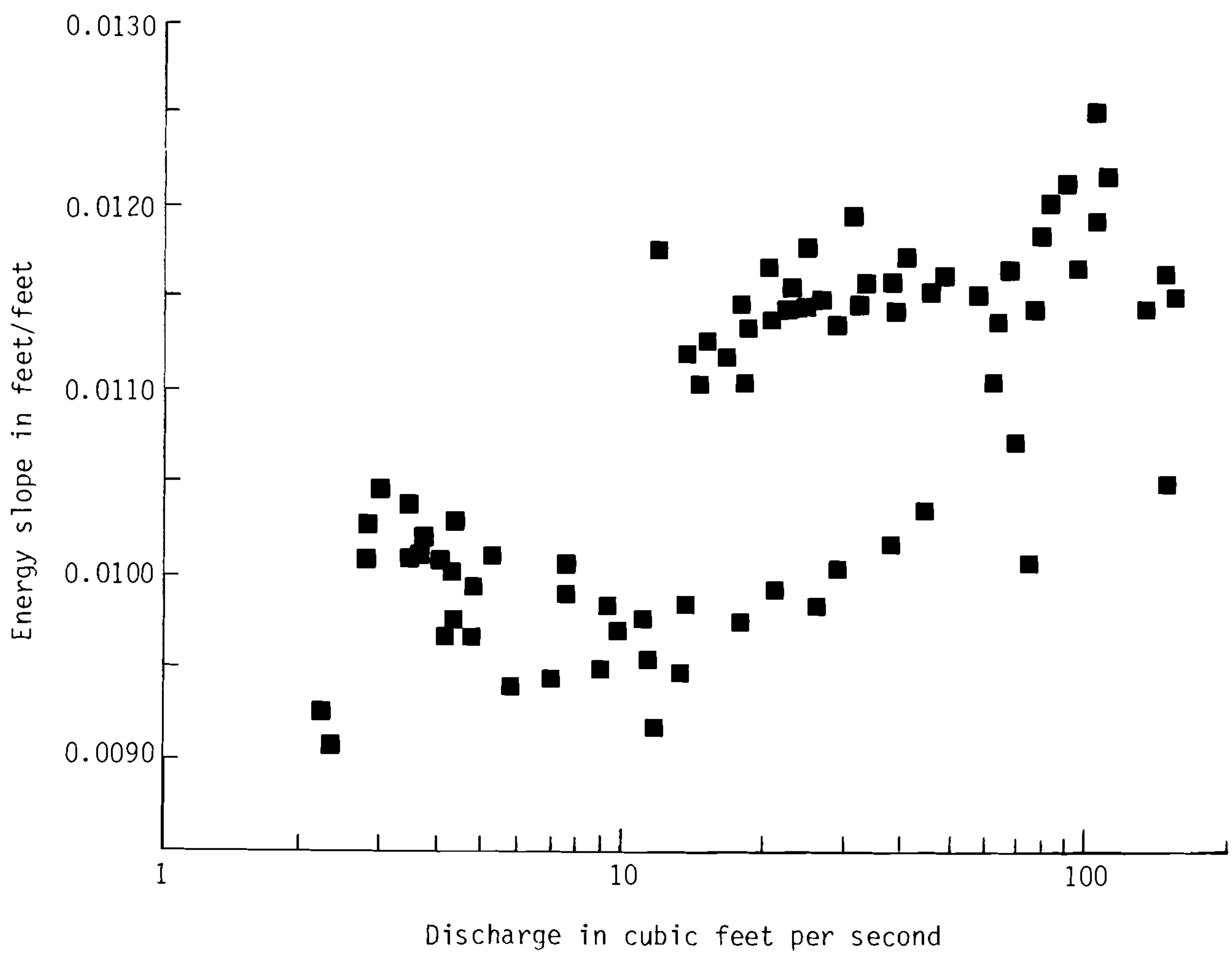


Figure 10: Relationship between energy slope and river discharge for Oak Creek, near Corvallis, Oregon.

LIMITATIONS TO THE USE OF THE DIRECT DETERMINATION APPROACH

Variations in Manning's n and energy slope result from the variable nature of flow resistance and energy expenditure in natural channels. The use of the direct determination approach directly or indirectly reflects these variations. Therefore, this technique eliminates many of the problems associated with Manning's equation over a wide range of flows.

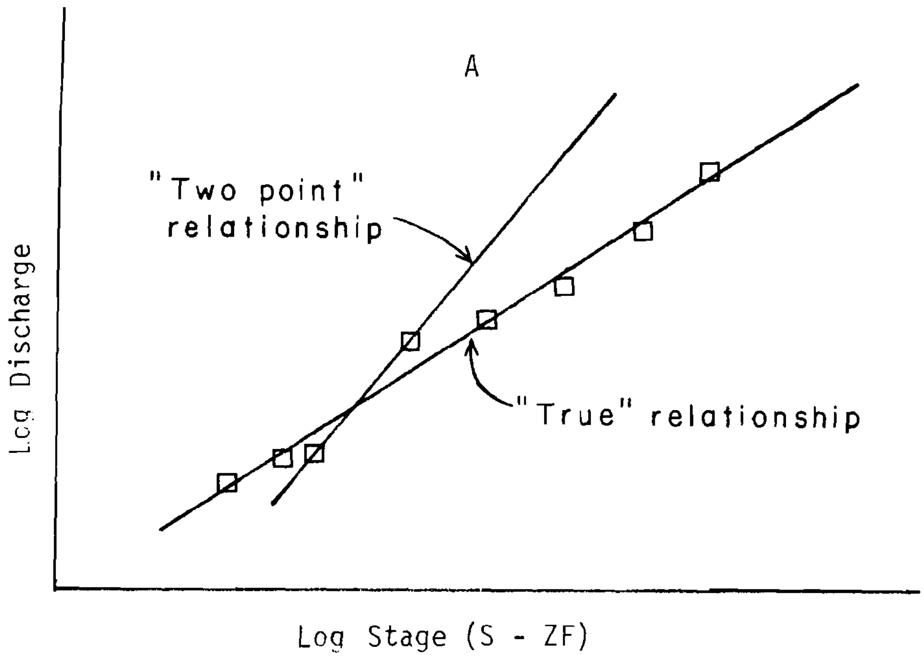
This approach is more time consuming than the use of the Manning equation because more than one trip to the field is required. However, proper study preplanning, and work scheduling can reduce travel and data collection time, and allow available resources to be used quite efficiently.

If only two measurements are used to construct the rating curve of stage versus discharge, significant problems may be encountered. Errors may be introduced by measuring two flows too close together. This may result in a two-point regression curve which is significantly different from a least squares curve using three or more points. This type of error is shown in Figure 11a.

A second source of error associated with this approach is an incorrect estimation of the point of zero flow. This error is illustrated in Figure 11b. The point of zero flow (ZF) can be thought of in two manners. First, as the stage at which no flow occurs in the stream. This elevation may be determined in the field as the highest elevation of the thalweg at a control section. A control is some feature of the channel causing a backwater condition upstream (see section on site selection for further discussion of control features).

The point of zero flow may also be considered as the value which causes the stage-discharge relationship to be linear in a log Q versus log S plot. This value may be obtained by collecting sufficient data that the term ZF can be determined by plotting points until the stage-discharge relationship on log-log paper is linear. If a small (not more than 4) number of points are being used, the determination will have to be made in the field.

At least two points (two discharges) are needed to establish a stage-discharge relationship, and two can be used effectively if done with care. The use of three or more points, in addition to the point of zero flow, is much preferred to two. Errors of the type shown in Figure 11b can be corrected by applying limits to the slope, b, of the regression equation. Fewer than 10% of the values of b for stage-discharge rating curves are greater than 4.0 or less than 1.4 (Log discharge on the ordinate, Log stage on the abscissa). If the slope is outside these bounds, additional points should be measured for the curve. A good



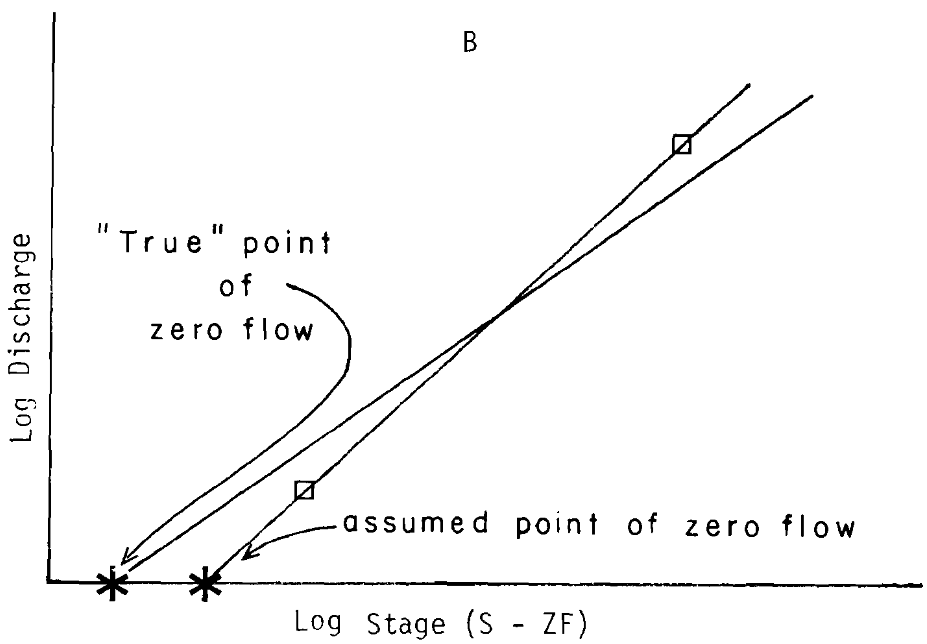


Figure 11: Two sources of error possible from using a two-point rating curve. A - error from discharge measurements too close together. B - error in estimating point of zero flow.

estimation of ZF will usually eliminate the need for this type of correction. A second technique which will improve the effectiveness of a two-point rating curve, is to ensure that the two points are sufficiently spread apart. Under the two-point system, the second discharge measurement should be at least 2 times the first flow if the second is higher. If the second is lower, it should be 0.5 times the first flow.

The use of the equation $v = a\ Q^b$ is well accepted in the literature for mean channel velocity, but the form $v_i = a_i\ Q^{b_i}$ is not generally accepted. The only use of the equation in the past has been by Bovee, et al. (1977). Information presently available suggests that the equation does predict subsection velocities, with at least 90% of the calculated velocities being within 10% of the measured velocities.

ERROR ANALYSIS FOR VARIOUS APPROACHES

Stage-Discharge Predictions

The relative amounts of error involved in using the Manning equation with one set of calibration measurements, and in using the rating curve system with various numbers of data points, was determined from data obtained from 11 streams. Rating curves for each stream were obtained from U.S. Geological Survey gaging stations. These curves served not only as potential data sources, but were also used to define the measured stage for a given discharge.

In the case of Manning's equation, one stage-discharge point was taken from a rating curve and used to calibrate the equation. Predictions of the stage at different discharges were then compared to the measured value at each of the other discharges on the rating curve. Then another point was taken from the rating curve, the equation recalibrated, and the process repeated. A similar technique was used in testing the direct determination approach, first using two points, then three, four, and so on. By selecting different points and combinations of points, it was possible to make several thousand comparisons between predicted and measured stages at different discharges over a wide range of hydrologic conditions.

For each stage-discharge prediction, the absolute value of the error was determined by:

$$\text{Error (\%)} = \frac{Q_m - Q_p}{Q_m} \times 100\% \qquad (18)$$

where,

Q_m = the measured discharge of the section
Q_p = the predicted discharge of the section

For each river, and for each approach used, the mean error was calculated as the arithmetic average of the absolute errors for each trial. Table 1 shows the mean error of the stage-discharge predictions, using all combinations and permutations of data points and no limits to the range of extrapolations made from measured calibration flows. Table 2 shows the mean error of the stage-discharge predictions, with the following limitations placed on selection of data pairs and range of extrapolations:

1. When Manning's equation was used, the range of extrapolations was limited to a minimum flow of 40% of the calibration flow; to a maximum flow of 250% of the calibration flow.

2. If a two-point rating curve were used:

   a. The second point in the rating curve must be outside the range of 0.8 times the first flow to 1.25 times the first flow.

   b. The slope of the regression line, $Q = a(S - ZF)^b$, must be between 1.4 and 4.0. If the slope of the regression line was outside this range, a third point was added by estimating the point of zero flow.

   c. The range of extrapolation was limited to a minimum flow of 0.77 times the minimum measured flow point, to a maximum flow of 1.30 times the maximum measured flow.

3. If three or more points were used to define the rating curve, the range of extrapolations was limited to a minimum flow which was 0.4 times the minimum flow used to define the curve, to a maximum flow of 2.5 times the maximum flow used in the rating curve.

Data which have been subjected to the preceding selection and limitation process will henceforth be termed refined data.

The error associated with the number of points used to establish a stage-discharge relationship is shown graphically in Figures 12 and 13 for Oak Creek in Oregon, and the Yakima River near Umtanum, Washington. Figure 12 shows the mean error of the stage-discharge predictions, using all combinations and permutations of data points. Figure 13 shows the errors associated with the number of points used to establish the stage-discharge relationship, using only refined data as defined previously.

Table 1. Mean percent error associated with the number of points used in establishing a stage-discharge relationship, with no limitations on the data used or range of extrapolations. Use of one point indicated application of Manning's equation. Limits on regression slope employed for 2-point rating curve.

Location of Gaging Station	Number of Points used in Calibration 1	2	3	4	5	6
Oak Creek near Corvallis, OR	56.8	22.2	22.8	12.5	11.3	10.8
Yakima River at Umtanum, WA	47.8	8.0	3.1	2.6	2.4	2.3
American River near Nile, WA	38.0	13.6	8.1	6.2	5.6	5.3
Yampa River at Maybelle, CO	22.3	21.6	6.9	6.0	5.6	5.5
Yampa River at Steamboat Springs, CO	61.2	5.4	9.2	6.8	6.0	5.7
Cache la Poudre River at mouth (CO)	50.6	11.1	11.0	7.3	6.6	6.2
South Platte River near Kersey, CO	41.0	12.0	10.7	9.4	--	--
North Branch, Elkhart River near Cusperville, IN	22.6	26.1	47.5	23.5	20.2	18.8
Elkhart River at Goshen, IN	22.9	11,944	10.1	7.6	6.8	6.4
St. Joseph River At Elkhart, IN	30.5	15.2	14.3	10.0	9.2	8.7

Table 2. Mean percent error associated with the number of points used in establishing a stage-discharge relationship, using refined data with limits placed on the data used and range of extrapolation. Use of one point indicates application of Manning's equation.

Location of Gaging Stations	Number of Points used in Calibration 1	2	3	4	5	6
Oak Creek near Corvallis, OR	14.8	10.2	9.5	9.2	8.9	8.7
Yakima River at Umtanum, WA	18.2	2.7	2.5	2.3	2.2	2.2
American River near Nile, WA	17.6	6.1	5.8	5.8	5.4	5.3
Yampa River at Maybelle, CO	12.6	5.8	5.3	5.1	5.1	5.2
Yampa River at Steamboat Springs, CO	14.1	5.4	5.5	5.3	5.1	4.9
Cache la Poudre River at mouth (CO)	24.4	5.2	4.9	4.7	--	--
South Plate River near Kersey, CO	26.2	5.3	4.9	4.7	--	--
North Branch, Elkhart River near Cusperville, IN	16.4	22.9	16.8	15.0	14.1	13.5
Elkhart River at Goshen, IN	20.6	7.1	6.6	6.2	6.0	5.9
St. Joesph River at Elkhart, IN	20.1	8.0	7.2	7.0	7.0	6.1

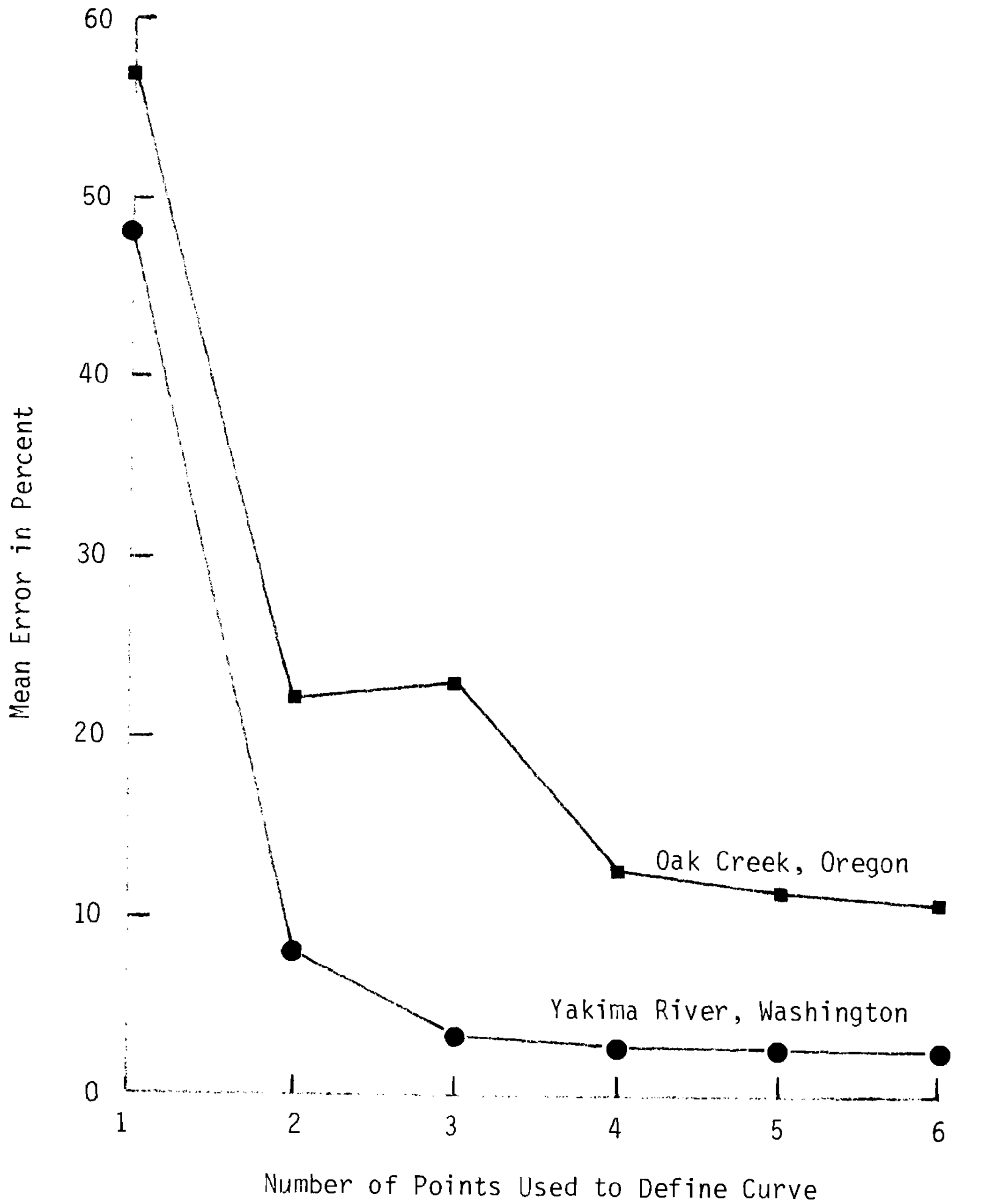


Figure 12: Mean error associated with the number of points used to define the stage-discharge relationship, for two streams, with no limits placed on the combinations of points used and no limit to the extent of extrapolation from measured flows.

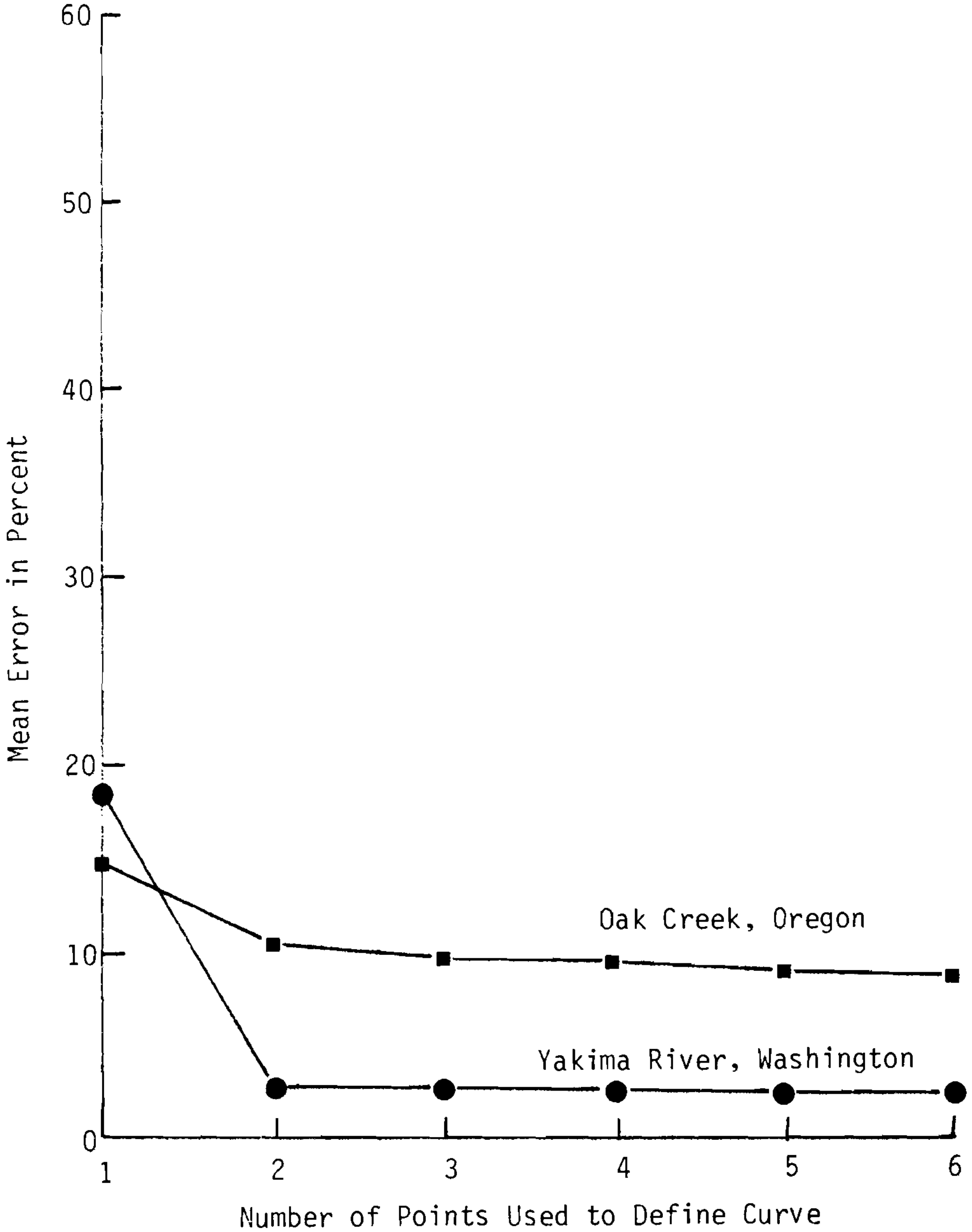


Figure 13: Mean error associated with the number of points used to define the stage-discharge relationship, for two streams, with limits placed on the combinations of points used and limits on the extent of extrapolation from measured flows (refined data).

Tables 1 and 2, and Figures 12 and 13 show some interesting trends. When no limitations are placed on the data used for calibration, nor on the extent of extrapolations, the use of a two-point rating curve may in fact be no better than using Manning's equation. However, the use of a three-point rating curve almost always produces more reliable results than either the two-point system or Manning's equation. Figures 12 and 13 show that at some number of points on the rating curve, there is little improvement in the accuracy of the predictions by adding more points. If unlimited or unbounded data, such as that used in Table 1, is used, this limit is approached with three or four stage-discharge points. If the suggested refinements are made when selecting data points and extrapolating the data, the limit of reliability is approached with only two data points. However, the useful range of extrapolated discharges is smaller when two points are used, so in most cases it may be desirable to use three, even if the reliability may not improve significantly.

Velocity Predictions

The methods presented for calculating the velocity distribution in a stream are based on equations which have been accepted by hydraulic engineers for calculating the mean velocity of the channel. These concepts have not been generally applied when the channel is subdivided into a series of channel segments. These methods have been used before by others, but the discussion of errors resulting from the use of a particular approach has been limited.

Elser (1976) conducted a brief field test of the Manning equation for predicting channel segment velocities in a large prairie river. Velocity measurements were made for five channel segments, at five discharges ranging from 100 cfs to around 400 cfs. Calibration measurements were made at about 150 cfs. Average velocities for each channel segment were calculated for each of the five flows and compared with the measured values. From this data an analysis of the frequency and magnitude of errors could be conducted. The percentage error was calculated as an absolute value by the following equation:

$$\text{Error (\%)} = \frac{V_m - V_p}{V_m} \times 100\% \qquad (19)$$

where,

V_m = the measured average velocity of the subsection
V_p = the predicted average velocity of the subsection

The number and percentage of predicted velocities within specified percentage error bounds are shown in Table 3. The small number of ob-

Table 3. Number and percentage of occurence of predicted velocities using the Manning equation, within specified intervals of percentage error based on measured velocities. From Elser, 1976.

Error in Percent	Number of Predicted Velocities in Error Bracket	Percent of Predicted Velocities in Error Bracket
0 - 10	9	36
10 - 20	6	24
20 - 30	5	20
30 - 40	3	12
40 - 50	0	0
50 - 60	1	4
60 - 70	1	4

servations used in this trial suggest that further field testing of this method is warranted.

Bovee, Gore, and Silverman (1977) conducted a limited field test of the equation $v_i = a_i Q^{b_i}$ in the same stream reach for which Elser's field test was conducted. In this sense, conditions for the two tests were identical and, therefore, results from the tests are comparable.

Velocity predictions were made using a two-point rating curve system and were applied to specific locations on a transect, rather than for average velocities for a channel segment. Velocity predictions were made for flows ranging from 140 cfs to about 400 cfs. A total of 71 velocity measurements and predictions are included in the analysis shown in Table 4. Error calculations and presentations were made by the same procedures used with Elser's data.

Both the above field tests were conducted during the developmental phase of both approaches for instream flow use. Velocity predictions for both studies were made in a highly turbulent, non-uniform riffle area. Results could be expected to give higher accuracy in pools and runs, although these areas have not been subjected to a field test.

RECOMMENDATIONS

The preferred approach in predicting either the stage-discharge relationship or the velocity distribution would be to use the rating curve approach for each type of prediction, using at least three points which span the flows of interest. Outwardly, it might seem that this approach requires an unreasonable amount of time and manpower. However, it is a rare instream flow study that is conducted with only one trip to the field. It is much more common to conduct numerous field trips to a particular stream, for one reason or another. In this case, collection of additional field data is a relatively small imposition. The cost of three sets of data is small compared to the cost of one set which may prove to be unreliable.

If only two sets of stage-discharge or velocity-discharge data are available, selection of data pairs for construction of the two-point rating curve should follow the following guidelines:

1. The higher discharge measurement should be at least twice as high as the lower discharge measured.

2. If the slope of the log-stage versus log-discharge curve is less than 1.4, or greater than 4.0, a third point should be added by adding the estimated point of zero flow.

Table 4. Number of percentage of occurrence of predicted velocities, using a two-point rating curve system, within specified intervals of percentage error based on measured velocities. From Bovee, Gore, Silverman, 1977.

Error in Percent	Number of Predicted Velocities in Error Bracket	Percent of Predicted Velocities in Error Bracket
0 - 10	35	49
10 - 20	16	23
20 - 30	10	14
30 - 40	4	6
40 - 50	1	1
50 - 60	2	3
60 - 70	0	0

3. Hydraulic predictions should not be made for flows which are less than 0.77 times the minimum measured flow, nor for flows higher than 1.3 times the maximum measured flow.

If the Manning equation is used, the gradually varied flow procedure is preferred to the assumption of uniform flow in the prediction of the stage-discharge relationship. When using the Manning equation to determine the velocity distribution, it should be calibrated from velocity measurements made across each cross section. The useful range of extrapolations, using the Manning equation with one set of calibration measurements is from 0.4 to 2.5 times the calibration discharge.

Table 5 lists the recommended approaches and useful range of flow extrapolations for different numbers of input data points.

Table 5. Recommended approaches using alternative numbers of stage-discharge and velocity-discharge measurements.

Number of Measurements	Approach	Useful Range of Extrapolation
1	Manning Equation	0.4 to 2.5 times calibration discharge
2	Two-point rating system, refined data (see text)	0.77 times the minimum discharge measured, to 1.30 times the maximum discharge measured.
3	Rating Curve system	0.4 times the minimum discharge measured, to 2.5 times the maximum discharge measured.

SITE SELECTION CONSIDERATIONS

In order for a simulation model to be useful in the planning process, its output must be capable of extrapolation into space and into time. Extensions into time were discussed in the Introduction. The study area is a sample of the conditions found in a larger stream reach, which allows the extension of results obtained from the study area over the larger reach. In turn, the characteristics of the study area can have a profound effect on the ability to adequately simulate the hydraulics of the stream. The selection of a hydraulic simulation technique may be largely dictated by limitations imposed by the study area. This section describes two subjects as related to study area characteristics:

1. Site selection extensions into space as a function of methodological approach.

2. Limitations of hydraulic simulation models imposed by on-site conditions.

EXTENSIONS INTO SPACE: METHODOLOGICAL APPROACH

Critical Reach Concept

Under the critical reach concept, the study site represents the type of area within the stream which is most sensitive to changes in flow and is assumed to be "critical" to the fish population by limiting the success of a particular life stage. This concept further assumes that if adequate flow is maintained through a critical reach, adequate habitat conditions will prevail for species throughout the entire stream.

The critical reach approach requires more than casual experience with the dynamics of the fish population in a given river. If the limited life stage or critical reach cannot be clearly identified, one's reasoning and analysis may result in a questionable flow recommendation. For example, low recruitment of young of the year might be known to be limiting to the population of adults. A logical assumption might be that the availability of spawning area is the limiting factor. The critical reach would be selected over a heavily utilized spawning area. If spawning area is in fact the limiting factor to recruitment, the examination of flows over the spawning grounds would be entirely appropriate. However, if the limiting factor for recruitment were inadequate habitat conditions for fry, a flow regime based on increasing spawning could actually increase fry mortality. This could result in an outcome opposite of that desired. In this case either the assumption concerning maintenance of "noncritical" habitat was not valid, or the wrong type of site was chosen as a critical area.

However, in many instances the field investigator will have sufficient information about a stream and its fish population to estimate the limiting factors and their associated critical reaches. For purposes of facilitating site selection and assessment, the critical reach should meet two basic criteria:

1. The reach should be highly sensitive to changes in stream flow. This means that the rate of change of width, depth, and velocity with respect to discharge, should be greater for the critical reach than for other portions of the stream. Generally, the most sensitive stream reaches with respect to discharge are elevated portions of the channel, such as riffles and gravel bars. However, the shape of the channel also affects sensitivity. Convex channel profiles exhibit a very rapid rate

of change of hydraulic variables with changes in discharge. Trapezoidal and rectangular channels are fairly sensitive at low flows but relatively insensitive to discharge at medium or high flows. Triangular or parabolic channels are relatively insensitive to flow changes and the rate of change is often difficult to interpret. Figure 14 shows several cross-sectional profiles and the associated rate of change of a simple hydraulic parameter (wetted perimeter) with discharge.

2. The critical reach must also act as a biological control. The species for which flow-related interpretations are made must be directly limited by the type of habitat present in the critical reach at some life stage. For example, if spawning is limiting for a trout population, then a convex gravel bar might best be selected as a critical reach. If invertebrate food production is considered limiting, a rocky riffle area would be an appropriate critical reach. In all cases, it is imperative that the linkage between the limited life stage and the critical reach is firmly established.

Representative Reach Concept

Application of the representative reach concept is appropriate when the limited life stage of a fish population is not known with any certainty. It is also a useful tool to analyze the interrelatedness of different species and/or life stages at different times of the year. Rather than assume a single type of habitat is controlling the population, the representative reach concept assumes that the importance of a particular area varies with time as well as with discharge. This approach utilizes a series of cross sections to sample the relationships among the flow regime and all the different types of habitat within a representative reach of stream. In theory, the variance among the hydraulic parameters of the study reach would be about the same as if another study reach within the same stream segment had been used. In other words, the variance among hydraulic parameters among study reaches within a river segment is assumed less than the variance among hydraulic parameters among river segments.

This assumption requires that the stream segment from which the study reach is selected is fairly homogenous. Classification of these rather large homogenous stream segments should consider topography, geology, gradient, stream flow, and biological communities. Acts of man, such as extensive channelization or diversions, may also delimit certain reaches of streams. This process of delimitation of large homogenous reaches is termed stratification, and is best completed in the office prior to any visit to the field. The purpose of this stratified sampling process is to determine, without the encumbrance of personal judgement, locations of study areas which by virtue of representation, can be extrapolated over larger areas.

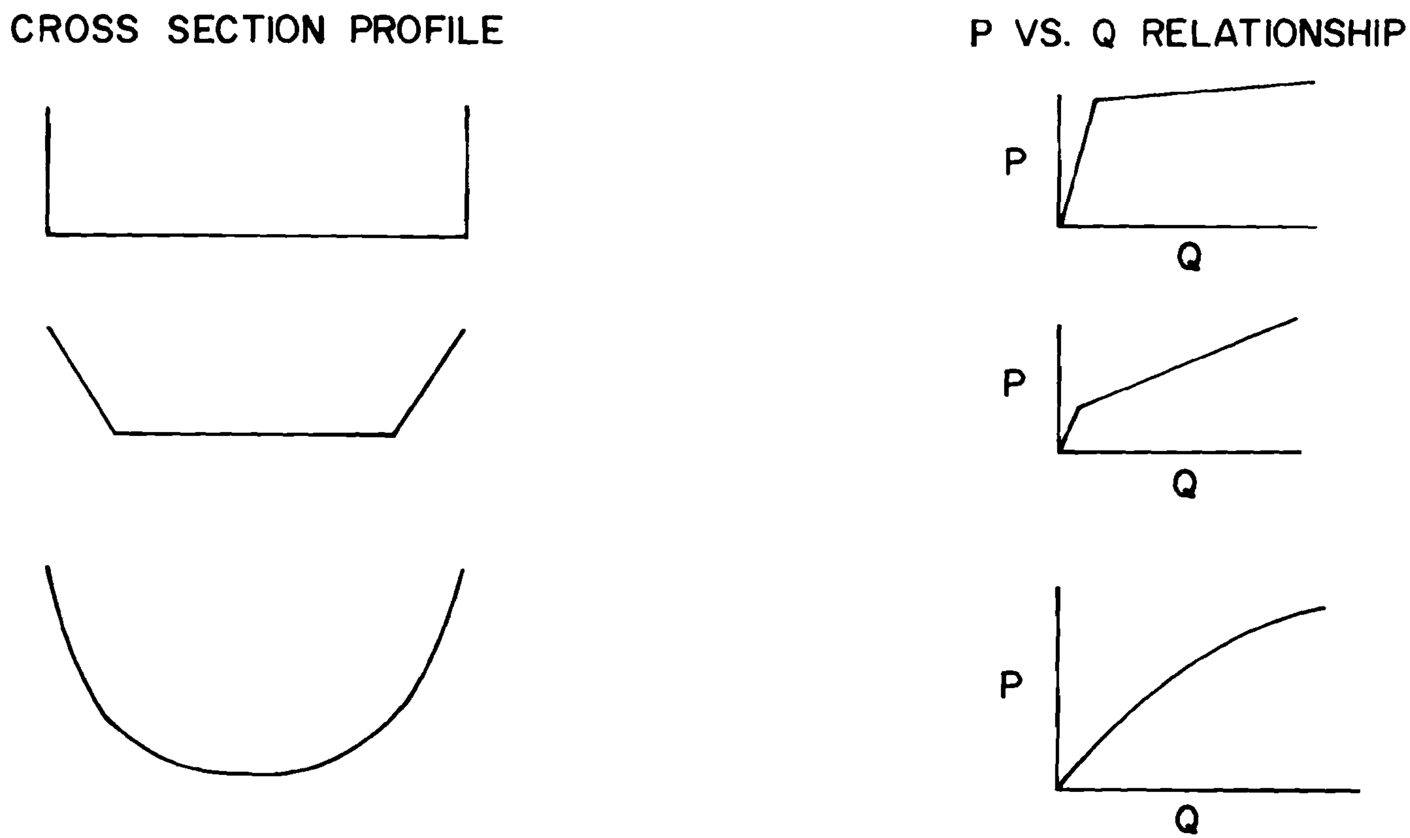


Figure 14: Rate of change of a simple hydraulic parameter (wetted perimeter) as a function of cross-sectional profile.

Topographic and geologic maps, stream flow and water quality records, along with species distribution maps and studies, and aerial photographs (where available) collectively provide raw data for the stratification phase. Field trips or over-flights are helpful in partitioning off rather homogenous stream segments but are often insufficient as the sole means of stratification.

A stratified river segment may be thought of as a series of short reaches with a common morphology. Each of these short reaches is relatively similar to every other short reach within the stratified segment, with the frequency of dissimilar reaches decreasing as a normal distribution function. This means that if several random samples were taken within the population of representative reaches, the probability of obtaining a representative reach "typical" of the stratified segment would be greater than the probability of obtaining one which is "atypical" of the stratified segment. This process is termed "stratified random sampling" and follows accepted statistical sampling methods. Figure 15 illustrates the hierarchy of nomenclature used to focus activities down to a study area.

Even though a good job of stratification (identifying the relatively large homogenous segments) has been done, there will always be zones within the stratified reach which are obviously different from the rest of the reach. Consistent with probability theory and the random sampling process, the stratified segment may be simply dissected into a population of small reaches of similar length, and several samples drawn at random from the population. This implicit zonation recognizes that variance between candidate representative reaches does exist, and it is assumed that the different types of candidate reaches will be sampled in the proportion that they occur in the stratified segment. Specifically, it is assumed that the type of reach which occurs the most often is the most likely to be sampled, and the most unusual or atypical reach the least likely to be sampled.

The technique of explicit zonation may further be used to ensure that at least one sample is taken from each "different" type of candidate reach. Using the process of explicit zonation, subpopulations of candidate reaches are generated by a further classification of reach types within the stratified stream reach. For example, all riffle-pool sequences are clumped together as one discrete population, all meandering reaches as another population, and all braided reaches as yet another population. Explicit zonation essentially means the application of a second stratification process within the larger stratified segment. While this sampling technique ensures that at least one candidate from each reach type will be sampled, the investigator may be faced with the problem of degree of zonation (how different is different?). In the final selection of the representative reach to study, implicit zonation within the explicit zone is the least biased. Figure 16 illustrates the difference between implicit and explicit zonation.

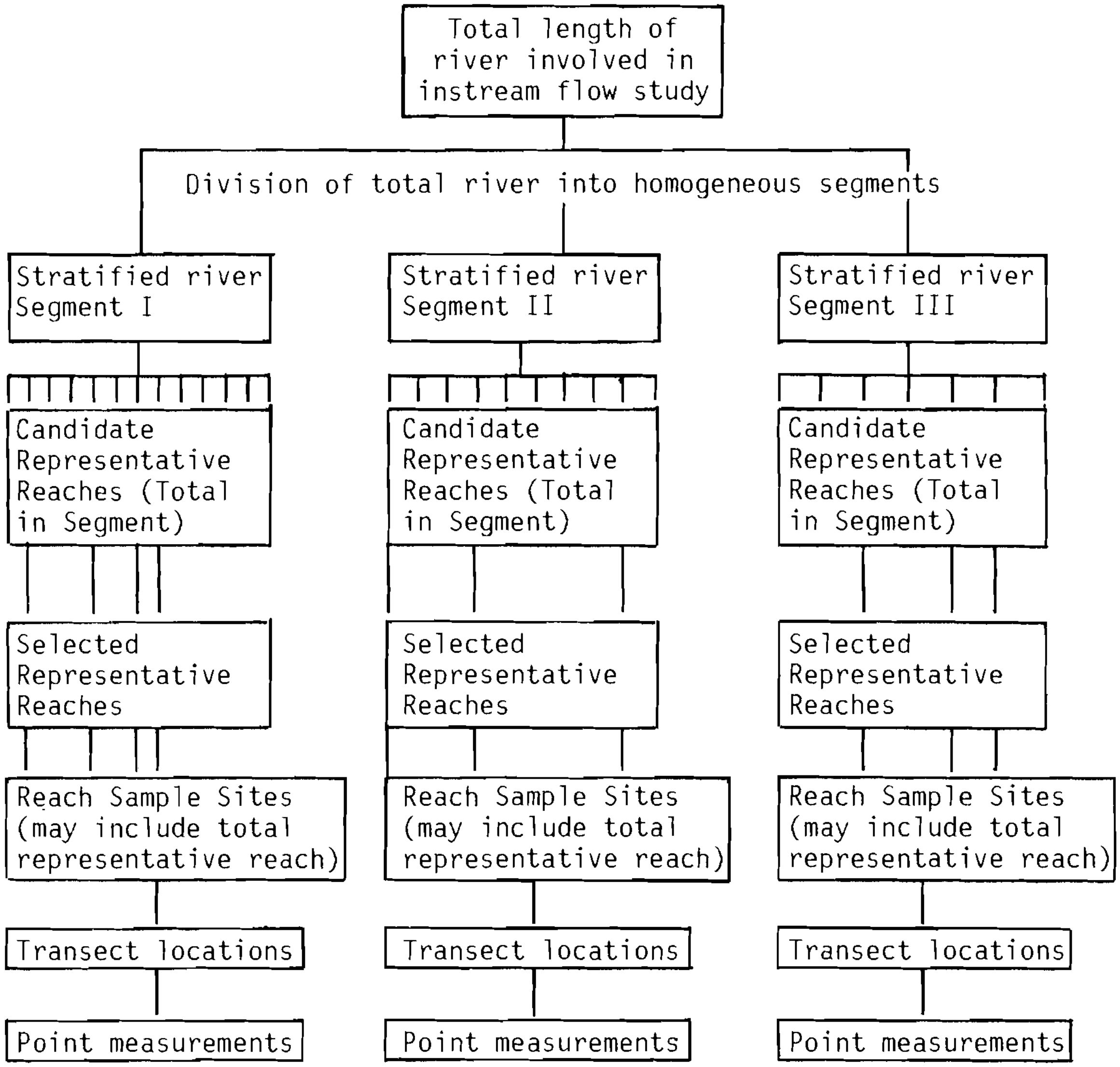


Figure 15. Hierarchy of nomenclature used to focus data collection activities at the study area.

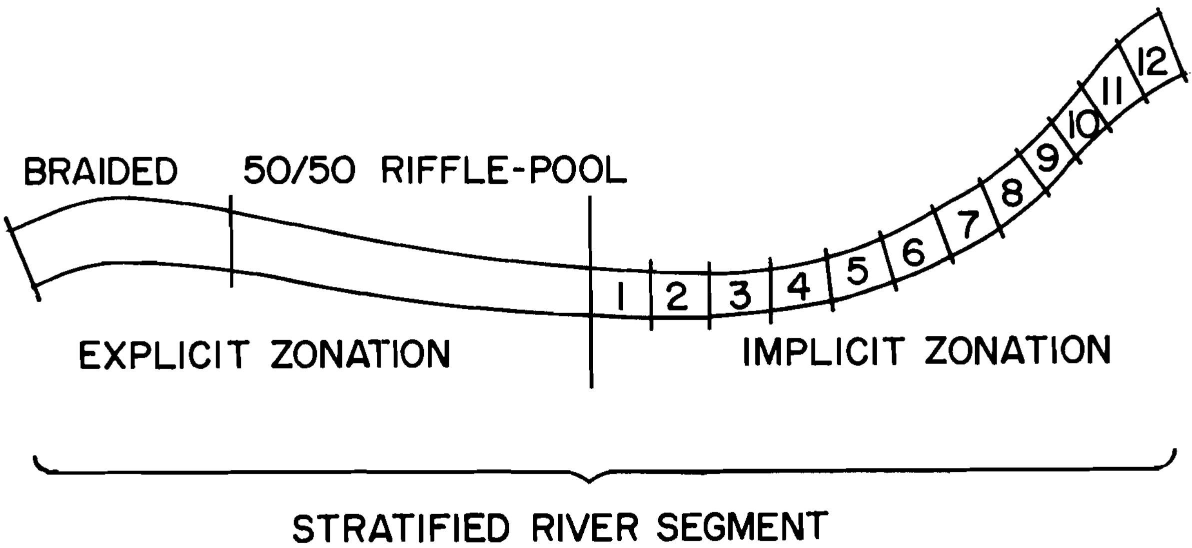


Figure 16. Implicit and explicit zonation of a stratified river segment to generate a population of candidate representative reaches.

If the technique of explicit zonation is used, the candidate reaches may or may not be of the same length. However, with implicit zonation each candidate reach must be of nearly equal length to avoid sampling bias. Candidate reaches should be large enough to encompass two riffle and pool sequences, or meander crossing-meander pool sequences. This distance is a function of the high discharges occurring in the stream, which also largely determine the channel width. As a general rule, the spacing of successive riffles in straight channels, and crossing bars in meandering channels, is about five to seven times the channel width (Leopold, Wolman, and Miller, 1964). Thus the length of candidate representative reaches encompassing two such features should be 10 to 14 times the average channel width.

Having dissected the stratified stream segment into candidate reaches by the explicit and/or implicit zonation processes, it is now necessary to obtain a sample or number of samples (representative reaches) from the population of candidate reaches. A random sampling of the candidate reaches is preferred, and several techniques may be used. Each candidate reach is numbered sequentially on a map through the entire population of candidate reaches. A random sample may be obtained through some type of random number generator; a table of random numbers, a deck of cards with a number of cards equal to the number of candidate reaches, the rolling of a single die if there are six or less candidates, etc. Eeeny, meeny, miney, mo is not considered a randomly generated number.

The number of samples taken is a function of the variance within the population to be sampled. If explicit zonation is used to generate the population, then only one sample from each population is needed, although more might be taken to satisfy any uncertainties concerning the zonation. If implicit zonation is used, at least three samples should be taken. The purpose of this exercise is to determine areas which are to be inspected on-site, without having to inspect the entire river. Therefore, the number of samples taken is not critical, but it should not be more than about half the total population of candidate reaches.

Now that a small number of candidate reaches have been selected, the investigator should arrange for an on-site inspection of each selected "representative reach." If, on inspection, all of the representative reaches are fairly similar, only one is required as a location for a study site. In this case, such considerations as access, logistics, and landowner permission may guide the selection of the study site. However, if on-site inspection reveals that all of the "representative reaches" are significantly different, it may be necessary to either set up study sites in each reach or revert to an explicit zonation and resample the population of candidate reaches.

At this time, it is necessary to determine the length of river within the stratified segment represented by each selected candidate reach. If explicit zonation is used, the represented reach length equals the river length in each explicitly defined zone. If implicit zonation is used, the length represented is proportional to the number of samples taken. If four samples are taken, and each is to be studied separately, then each selected representative reach represents 25% of the total stream length within the stratified segment. However, if after inspecting the four sites, three seem to be very similar and one different, then only two selected representative reaches are used, one representing 75% of the segment length and one representing 25%.

The study site itself is then set up within the representative reach. The study site may encompass the entire representative reach, or a portion of it. The guiding principle is that the study site should be a sample of most of the variance in habitat types, in the proportion that they occur, within the representative reach. If this variance can be described in only half of the representative reach, then the study area may encompass this area only.

ESTABLISHMENT OF STUDY AREAS

As previously mentioned, a study area may encompass the entirety of the selected representative reach, or only a portion of it, depending on the variation in habitat encountered within the representative reach. The purpose of the study area is to provide a sample of the variation in hydraulic conditions and habitat types, as well as the proportion occupied by each habitat type within the representative reach. The variation and proportionality of habitat types are determined through the use of strategically placed transects, delineating discrete areas of the stream. Each transect represents a time commitment of 1 to 2 man-hours of field work. Consequently, it is always advisable to describe the study area with as few transects as possible. However, too few or improperly placed transects will give a distorted view of the stream. Transect placement is a critical determinant in the reliability of an instream flow model, and is discussed in detail in this section.

When establishing a study area, the first task is the establishment of a permanent (relative to the duration of the study) benchmark. The benchmark serves as a reference point both for relocating transects and for referencing elevations. Typical benchmarks may be an X chipped in a rock outcrop, a large distinctive boulder or bridge abutment, an iron pin driven firmly into the ground, or a spike driven into a tree trunk. The two criteria that a benchmark must meet are ease of relocation and permanence. The investigator should be wary of establishing benchmarks near trails or game paths, or on unstable ground (i.e. in areas which might slump or slide). It is the responsibility of the notetaker to clearly document the location and description of the benchmark. A map sketch and photographs of the study area should be included in the description.

The ends of cross sections (transects) should likewise be marked by permanent markers, called headstakes. A headstake may be a length of pipe or concrete reinforcing bar (rebar) driven flush to the ground surface or buried. The headstake should be long enough to penetrate the frost zone so that frost heave does not change its elevation. Headstakes should also be placed in such a way that disturbance by cattle, wildlife, or humans is unlikely. Like the benchmark, the locations of headstakes must be thoroughly documented by the person keeping the field notes. If possible, the positions of headstakes should be referenced by a bearing and distance from the benchmark and photographed. Several techniques for locating headstakes are listed in Appendix A. Documentation of the study area is important even if only one trip to the area is planned. In the event a second trip is necessary, relocation of transects is difficult or impossible without documentation. It is a rare instream flow study that is completed with only one trip to the field sites.

The placement of transects is governed by two principles. Obviously, transects should be placed where they are sampling a discrete type of habitat: pools, runs, riffles, backwaters, etc. However, an overriding principle to be followed is that the downstream-most transect should be placed at a hydraulic control, and all hydraulic controls within the section should be crossed by a transect. A hydraulic control can be defined as a physical feature, natural or man-made, which indicates a stage-discharge relationship. The most notable attribute of a control is its influence on the hydraulic slope. Controls are reflected by a break or inflection in the slope of the water surface.

This rule is mandatory. Therefore, the first transects to be placed within the study area are those which cross hydraulic controls. Figure 17 illustrates a river reach with initial transect placement over the hydraulic controls. The placement of these transects ensures better hydraulic predictability. If such placement can be made to correspond to a major type of habitat in the stream, so much the better.

Having placed the control transects, the next step is to place transects which sample major habitat types in the study area which were missed in the initial placement. At this stage the field crew must resist the temptation of proliferating the area with cross sections. As a general rule, never use two cross sections where one would suffice. Figure 18 shows the hypothetical study area with the addition of transects delineating major habitat types.

Depending on the level of detail desired, additional transects may be added to define the transition zones from one type of habitat to another. During this phase of study area setup, the tendency to dot the landscape with headstakes becomes very strong and must be resisted with an iron will. Remember that the greater the detail with which a reach is modeled, the more dissimilar from adjacent reaches it becomes. In the hypothetical reach we have used as an example, two additional transects could be added to delineate the transitions around the pool, as shown in Figure 19. Additional cross sections could have been added, but would not have contributed much to the description of the reach.

The final suggestion concerning the placement of transects deals with location of transitional transects at the head of pools. It is recommended that these transects be placed well into the transition zone, toward the pool. At high flows this transition zone will be a distinctively pool-type habitat. At low flows, the transition zone will be more riffle-like. Thus, the "head" of the pool will migrate up and downstream in accordance to the discharge. Additionally, these zones are especially prone to the development of eddies. An eddy represents an area of negative velocity (i.e. the direction of flow is "upstream"). While these areas are of great biological importance in many streams, they are extremely difficult to represent adequately in a simulation model.

SITE IMPOSED LIMITATIONS TO HYDRAULIC SIMULATION

The discussion of the theory of hydraulic simulation mentioned several general limitations to various approaches. This section addresses limitations to the capabilities of hydraulic simulation

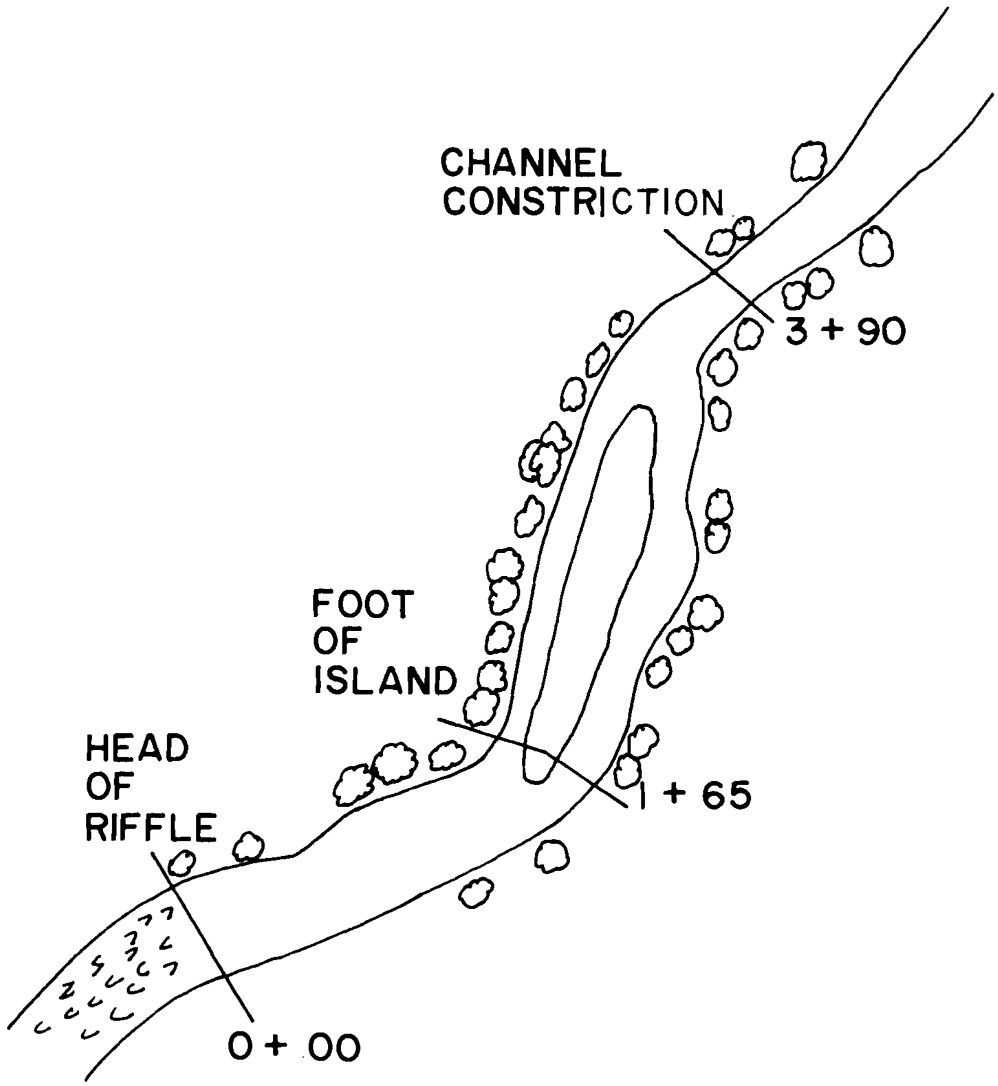


Figure 17: Initial transect placement over hydraulic controls in a study area.

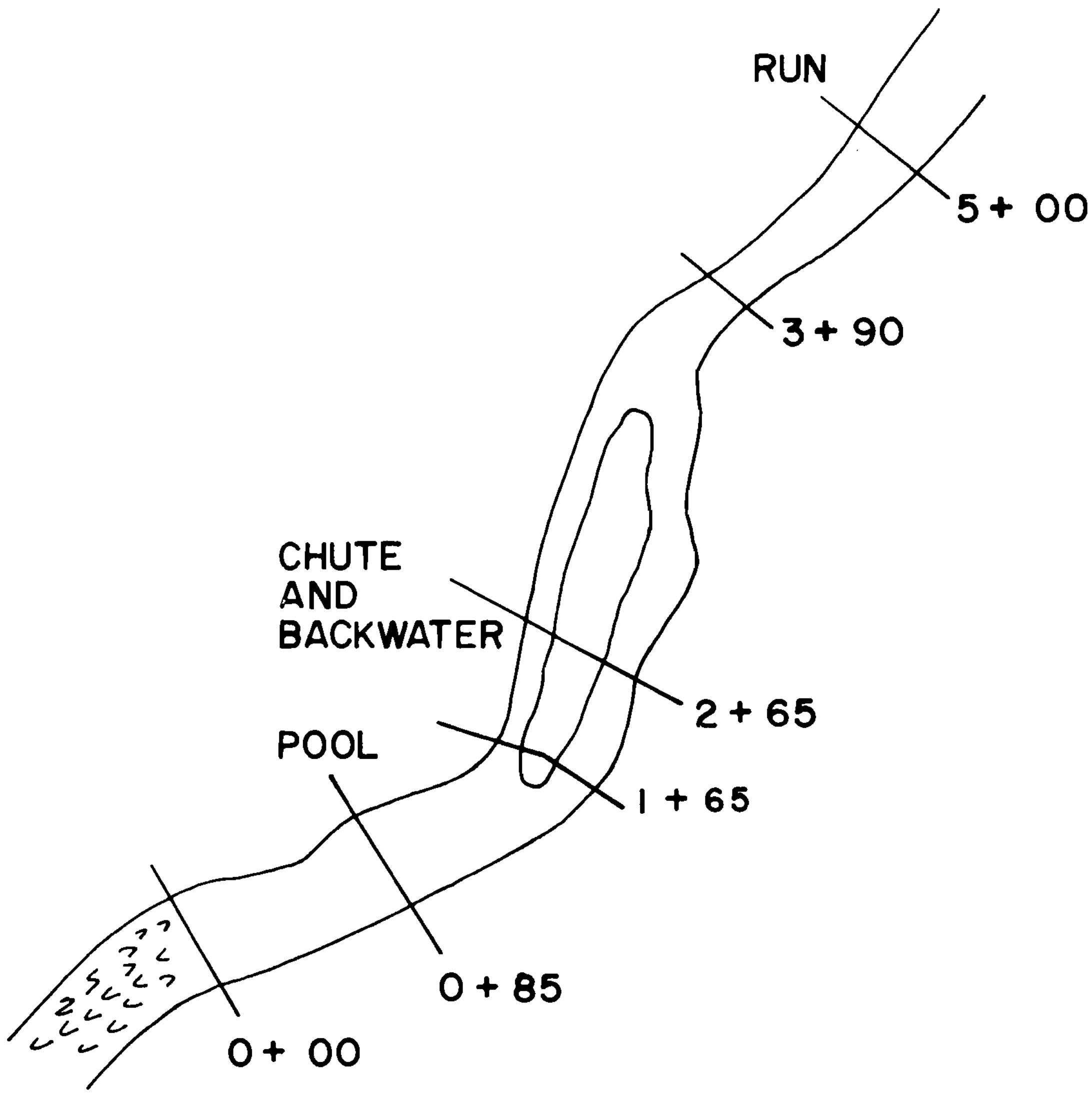


Figure 18: Addition of transects over major habitat areas in a study area.

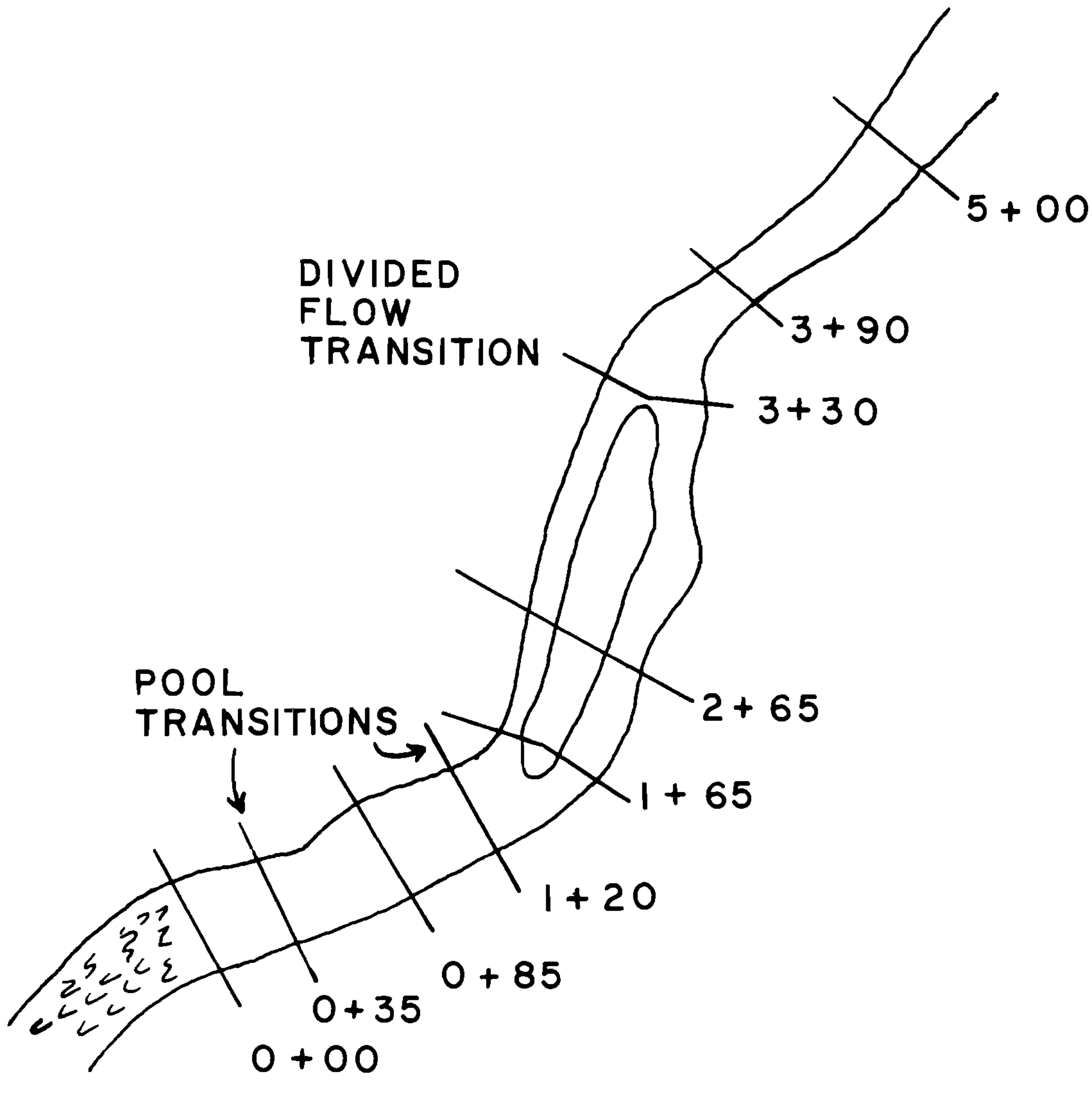


Figure 19: Final transect placement over transition areas in a study area.

approaches which are site-imposed. Initially, limitations have been classified as either "problems of complexity" or "problems of stability."

Complex Channels

For the purposes of this paper, a complex channel is defined as one of the following:

1. Divided Flow Problems: Braided streams with three or more channels; island complex in larger streams.

2. Channels with unstable or rapidly changing channel geometry. Fluid bed streams; irregular and rapidly varied cross sectional configuration.

3. Large bed element streams: Channels with large, irregularly spaced roughness elements.

Limitations to hydraulic simulations are related either to the number of cross sections needed to describe the reach, or to problems with calibrating the model to field data. The latter problem is associated primarily with the use of the Manning equation, while the former affects both types of simulations discussed in this paper.

Aside from the problem of a rapidly growing population of transects, simulation of complex channels by the rating curve approach is not difficult and results are quite reliable. However, due to the types of computations involved with Manning equation simulations, coupled with the channel segmentation process used to describe the velocity distributions, complex channels are difficult to model by this method.

In complex channels, hydraulic controls may be difficult to identify or be so numerous that the field team spends most of its time surveying controls. A stream reach with a complex of hydraulic controls is shown in Figure 20.

Additionally, many controls are not perpendicular to the channel, but lie diagonally across the channel.

Figure 21 illustrates an example of a diagonal control. Velocity is a vector having both magnitude and direction. At low flows, the direction of flow is perpendicular to the control and, therefore, is not parallel to the banks of the channel. At high flows, the direction of flow is more closely parallel to the channel banks. Intuitively one wishes to install transects at right angles to the channel. If a right angle transect is placed over a diagonal control, virtually all of the eleva-

Figure 20. Stream reach with complex hydraulic controls. Controls marked in white. Photo courtesy of Don Kelly, aquatic biologist, Sacramento, CA.

tions measured will be incorrect. Therefore, the control transects must be parallel to the control, even though such practice might be uncomfortable to the investigator.

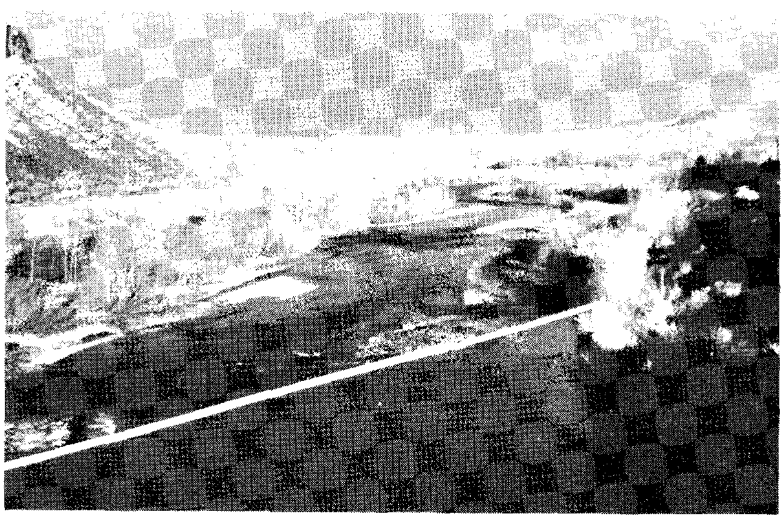


Figure 21. Stream reach with diagonal hydraulic control (marked in white). Photo courtesy of Tim Cochnauer, Idaho Dept. of Fish and Game.

Islands often provide unique transect placement problems because either the head or the foot of the island, or both, may act as a hydraulic control. If the flow around the island is subcritical, the flow profile is started at the foot of the island (Station 1+65, Figure 17). If the flow is supercritical, the flow control point would be at the head of the island (Station 3+30 in Figure 19).

Channels with large, irregularly spaced roughness elements are most commonly represented by streams flowing through boulder fields (Figure 22). In this type of stream, identification of controls is usually no problem. The problem arises from segmentation of the cross section to define the velocity distribution. Manning's n tends to fluctuate widely at specific locations in boulder strewn rivers. If only the mean cross-sectional velocity is desired, a single value of n may be used successfully for the entire cross section. However, when attempting to calibrate the model to measured velocity distributions, it becomes difficult to control the water surface elevation at the measured level. Thus, for boulder strewn rivers, Manning's equation may be calibrated to the water surface elevation or to the velocity distribution. Calibration to both parameters simultaneously proves to be quite difficult. Again, in using the rating curve approach, this problem is not hard to overcome.

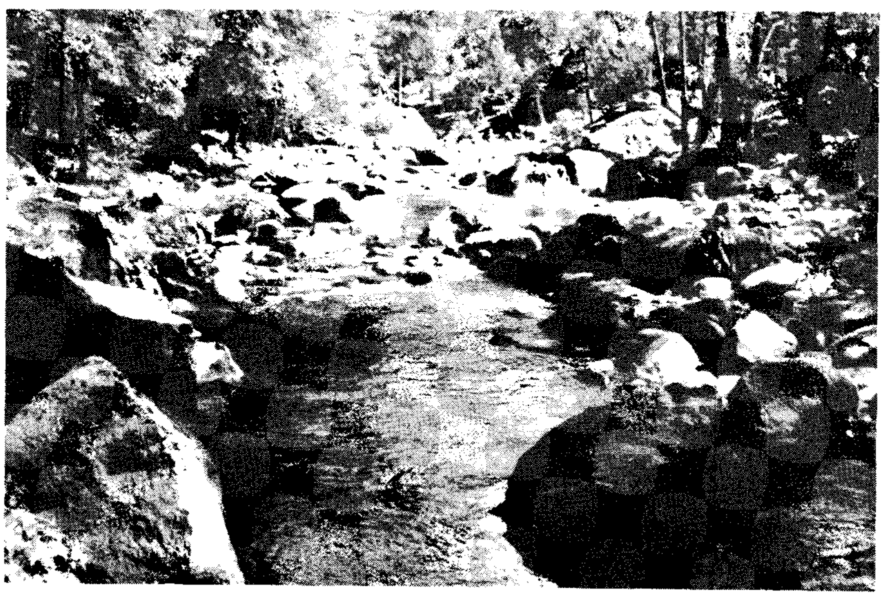


Figure 22. Stream reach with large, irregularly spaced bed elements. Photo courtesy of Don Kelly, aquatic biologist, Sacramento, CA.

Channel Stability

Changes in channel configuration occur regularly in nature in response to changes in flow regime, sediment yield, and chance events such as large storms or runoff from unusually deep snowpack. Short-term and nonpersistent channel changes are termed scour and fill.

Scour and fill occur with some regularity in all streams at one time or another. The primary factor influencing the technique selected for hydraulic simulation is the periodicity with which scour and fill occurs. This phenomenon is most active and the periodicity shortest in alluvial streams with sandy beds. Most alluvial streams experience at least one cycle of scour and fill each year, resulting in a rating curve loop (Figure 8). As long as the field measurements do not overlap the rising and falling limbs of the hydrograph, the rating curve approach works well in this type of stream. However, in streams which exhibit rapid periods of scour and fill, the rating curve approach will not work as well because the cross section itself will be different each time it is measured. What was a pool is now a sandbar. In this situation, the use of two or three replications of the Manning equation, with data collected at different flows, would be preferred for hydraulic simulation, each addressing the various channel configurations.

Under the condition of dynamic equilibrium, it is immaterial that pools are converted to sandbars. In order for this to happen, a sandbar somewhere was converted to a pool. Therefore, if a dynamic equilibrium has been assumed or determined, the relative proportion of pools and bars should remain fairly constant even though their positions change with time.

SUMMARY OF POTENTIAL SITUATIONS AND RECOMMENDED APPROACHES

The preceding discussions are designed to help the field investigator select a hydraulic simulation technique which is consistent with the type of study being conducted and the type of river being studied. Several different settings (type of study, type of river) which could confront the investigator, and the suggested hydraulic simulation approach for each situation are summarized in Table 6. Additionally, Table 6 includes the page numbers in the text which discuss a particular simulation approach or its limitations.

The approaches suggested in Table 6 are given with the caveat that the stream is in equilibrium, or very nearly so. If the stream is currently in dis-equilibrium, or a change in equilibrium status is anticipated due to some change in watershed characteristics, stream flow pattern, land use, etc., it is necessary to predict what the channel will

Table 6: Potential instream flow study situations and recommended hydraulic simulation approaches.

Study Setting	Recommended[1] Approach	Advantages	Disadvantages	Pages
I. River basin or watershed overview studies	Manning equation, gradually varied flow computation procedure (step-backwater)	Input data may be estimated without actual field measurement.	Low accuracy. Limited range of extrapolation	13-15 17-21 28-31
II. River or site specific instream flow studies or environmental impact studies. Conflict with other water uses low.				
A. Simple, stable channel	Manning equation, step-backwater computation procedure, one set of field measurements.	Fair accuracy with minimal amount of field data collection	Accuracy depends on cross sectional and roughness variability. Limited range of extrapolation	13-15 21-22 28-31 51-56
B. Complex, stable channel; configuration changes rapidly spatially. Large bed streams.	Rating curve with two or more sets of field measurements.	Easier calibration of field data in model. Fair-to-high accuracy. Large range of flows may be calculated.	Subject to "two-point" errors, which may be substantial. Modest data collection requirement.	15-17 22-24 31-33 52-60

Table 6 (con't)

Study Setting	Recommended Approach	Advantages	Disadvantages	Pages
II. River or site specific instream flow studies or environmental impact studies. Conflict with other water uses low.				
C. Fluid bed stream. Channel configuration changes with time.	Manning equation, step-backwater computation procedure. One set of field measurements	Ability to simulate not dependent on "constant" channel configuration	Accuracy depends on complexity of stream. Limited range of extrapolations	13-15 21-22 28-31 52-60
III. River or site specific instream flow studies or EIS. Conflict with other water uses moderate or high.				
A. Simple, stable channel	Rating curve with three or more (preferably more) sets of field measurements.	High accuracy of predictions. Large range of flows may be simulated.	Considerable investment in data collection	15-16 22-24 31-33

Table 6 (con't)

Study Setting	Recommended Approach	Advantages	Disadvantages	Pages
III. River or site specific instream flow studies or EIS. Conflict with other water uses moderate or high.				
B. Complex, stable channel; channel configuration changes rapidly spatially. Large bed element streams	Rating curve with three or more (preferably more) sets of field measurements.	Easier calibration of field data in model. High accuracy of predictions. Large range of flows may be simulated.	Considerable investment in data collection.	15-16 22-24 31-33 52-59
C. Fluid bed stream. Cross sectional configuration changes rapidly with time.	Manning equation, step-backwater computation procedure. Calibration measurements made at several discharges.	Simulation capability not limited by "constant" channel. Large range of flows can be simulated if calibration flows properly spaced.	Accuracy depends on cross sectional and roughness variability. Considerable data collection requirement.	13-15 21-22 28-31 52-59

Table 6 (con't)

Study Setting	Recommended Approach	Advantages	Disadvantages	Pages
IV. Channel alteration impacts; channel modifications for habitat improvement.	Manning equation, step-backwater computation procedure. Calibration measurements from engineering design.	Simulations can be made "before-the-fact" from engineering designs. Modifications in design can be made prior to construction.	Simulation should include portions of stream above and below impacted area for calibration purposes. Manning's N must be estimated where substrate changes or rip-rapping are involved.	11-15 17-21 24-28

[1]Assumption that stream is in equilibrium can be accepted.

be like under new equilibrium conditions. Given this situation, the only way to simulate the hydraulic characteristics of the new channel is to use the Manning equation, preferably using the gradually varied flow (step-backwater) computation procedure.

Finally, the level of field intensity required depending on the potential for conflict with other water uses is considerably different. One would prefer to assume that the conflict with other water uses will be small, so that the cost of data collection could be minimized. However, unless there is clear evidence to the contrary, one should assume that there will be some conflict with other water uses, and that an instream flow recommendation will be contested. The approach used should be selected accordingly.

DATA REQUIREMENTS AND SPECIFICATIONS

Upon completion of study area setup and documentation, collection of data for hydraulic simulation may proceed. It is assumed herein that the reader has some experience in the basic techniques required for the collection of data for hydraulic simulations. This discussion deals primarly with what to measure, not how to measure. A more complete amplification of basic measurement techniques may be found in the appendices.

Several types of data are common to all hydraulic simulation techniques mentioned in this paper. Common data requirements include:

1. Stationing.
2. Headstake elevations.
3. Water surface elevations.
4. Bed elevations.
5. Discharge measurement.
6. Estimate of substrate size.

If the Manning equation with velocity calibration or rating curve approaches are to be used, another type of data must be added to the list:

7. Velocity distribution at each transect.

STATIONING

Stationing refers to the measurement of the distances between transects. These measurements should be made between transects on both sides of the stream, particularly around bends. For consistency, cross sections should orginate from the same side of the stream. Identify left or right stream bank according to their position when looking downstream. Station indexing should start (0+00) at the most downstream cross section. A station index should be the distance along the thalweg (deepest portion of the channel) between transects, but may be given as the average of the distances measured on both banks. If the latter technique is used, make sure that the headstakes for each transect are nearly equidistant from the banks. The precision for stationing should be to measure this distance to the nearest foot (30 cm).

HEADSTAKE ELEVATIONS

Headstake elevations are usually referenced to an arbitrary datum at the benchmark (usually 100.00 feet). If desired, the headstake elevations may also be referenced to elevation above mean sea level if a reference of known elevation is near the study area. Headstake elevations should be read to the nearest 0.01 foot if English units are used, or the nearest 0.5 cm if metric. Headstake elevations should be surveyed from downstream to upstream, and then re-surveyed back to the benchmark. This procedure is called re-check or level loop closure. To determine the allowable error of closure, equation 20 may be used:

$$\text{Maximum Error of Closure} = 0.05\sqrt{M} \qquad (20)$$

where M = length of level loop in miles (Bouchard and Moffitt, 1965).

WATER SURFACE ELEVATION

Measurement of the water surface elevation requires a special procedure for the rodman, and good coordination between rodman and levelman. The rodman should dip the rod to the water surface and lift it again as soon as the rod forms a meniscus with the water. The levelman should read the high rod reading repeated the most often. The rodman can help by loudly indicating "touch" when the rod touches the water surface. This technique has proven much more effective and accurate than standing the rod at water's edge.

The water surface elevation should be measured on both sides of the river at each transect. If the elevations from both sides of a transect are not equal, an average should be calculated for the transect. One should expect water surface elevations to be unequal at the inside and outside edges of meanders. The degree of inequality is a function of the radius of curvature of the meander and the velocity of the water. Water surface elevations should be measured to the nearest 0.01 foot or 0.5 cm. It is a good idea to survey water surface elevations at the start and end of the day to determine the amount of change in stage which has occurred during data collection. A temporary staff gage placed at water edge at the beginning of the day may be read to determine change in stage.

BED ELEVATIONS

A profile of each cross section should be measured from headstake to headstake. The profile is defined by a series of elevations and horizontal measurements, starting at some pre-defined zero point. (NOTE: when measuring horizontal distances across a transect, do not attach tag lines or chains to the headstake. This practice may change the headstake elevation. The headstake is normally used as the zero point, with the tagline anchor behind the headstake.) Horizontal distances should be measured to the nearest foot (30 cm). Elevations should be measured to the nearest 0.1 foot (5 cm if in metric). During the profiling, the rodman needs only to make measurements where there is an obvious break in slope of the bed, or where a change in substrate material is encountered. The substrate should be described at each measurement point. Substrate descriptors are based on the size classes listed in the modified Wentworth scale shown as Table 7. Estimated percentages of each substrate class should be included in the substrate descriptions. Bed elevations may be determined by sounding if use of a stadia rod is impractical.

DISCHARGE MEASUREMENTS

The discharge at the time of the field measurements may be obtained directly if a USGS gage house is nearby, and if one can be relatively certain that the flow in the study area is the same as at the gage (i.e., no significant inflow or outflow between the gage and the study area).

If access to a stream gage is not possible or the stream flow is not known to be the same between the gage and the study area, the discharge must be measured at the study area. Velocity measurements are likely to be made at all cross sections (with the exception of using Manning's equation with no velocity calibration), thus the discharge could be calculated for any of the cross sections. The accuracy of discharge measurements is greatly influenced by the amount of variation in the cross section. Thus, if discharge were calculated for all the cross sections, one might find large discrepancies in the discharge. Therefore, the discharge should be calculated from the most uniform cross section in the study area. If all cross sections are highly nonuniform, it may be desirable to find one outside the study area exclusively for discharge measurement. The discharge should be calculated to at least two significant figures, or should be consistent with the allowable number of significant figures based on the precision of the stream gaging measurements. Discharge measurement techniques are included in Appendix B.

Table 7. Modified Wentworth Particle Size Scale.

	Range (mm)	Approx. Median (mm)
Mammoth Boulder	4000	--
Very Large Boulder	3500 - 4000	3750
	3000 - 3500	3250
	2500 - 3000	2750
	2000 - 2500	2250
Large Boulder	1650 - 2000	1825
	1330 - 1650	1490
	1000 - 1330	1165
Medium Boulder	830 - 1000	915
	665 - 830	750
	500 - 665	580
Small Boulder	415 - 500	450
	335 - 415	375
	250 - 335	290
Large Cobble	190 - 250	220
	130 - 190	160
Small Cobble	100 - 130	115
	64 - 100	85
Very Coarse Gravel	50 - 64	57
	32 - 50	40
Coarse Gravel	16 - 32	24
Medium Gravel	8 - 16	12
Fine Gravel	4 - 8	6
Pea Gravel	2 - 4	3
Very Coarse Sand	1 - 2	1.5
Sand	.062 - 1	.5
Silt-Clay	.062	--

VELOCITY DISTRIBUTION

If only one set of field measurements will be used with the Manning equation, velocity measurements should be taken with the calibration of Manning's n in mind. This means that the velocity measurements should coincide with the major breaks in topography or substrate observed during the measurement of the cross sections. These breaks usually serve as the boundaries to segmentation of a cross section into subsections. A mean velocity for each segment is required for calibration.

If multiple sets of paired velocity-discharge measurements are to be used in the rating curve approach, measurements may be made at major breaks as above, or evenly spaced across the cross section. It is imperative that all subsequent measurements of velocity are made at the same locations as the initial measurements. <u>Therefore, field notes should locate the positions of velocity measurement points in terms of their distance from the headstake, not from water's edge</u>. Failure to follow this procedure may negate the use of this hydraulic prediction procedure, and has been a frequent source of error in the past.

The location of each velocity measurement is called a "vertical." The number of verticals used per transect depends on the complexity of the velocity distribution and the detail desired by the researcher. The number of verticals used will usually be relatively independent of stream size. For the determination of the velocity distribution, 10-20 verticals are typically used; whereas, the transect used for the discharge measurement should employ 20-30 verticals. In all cases, the velocity to be measured is the mean column velocity at the vertical. Velocity measurement techniques and rules governing measurement of mean column velocity may be found in Appendix B.

Measurement of the velocity distribution is usually the most time consuming portion of the data collection procedure. Therefore, it is strongly recommended that current meters be provided for more than one member of the field team. Ideally, each member of the team should be outfitted with a meter, and should know how to use it. The cost of additional equipment is minimal compared to the savings in time and manpower afforded by extra current meters.

SEQUENCING STUDY AREA SETUP AND DATA COLLECTION

Certain activities must be completed in study area setup before data collection can proceed. Additionally, certain types of data are closely related (such as stage and discharge) and it is advisable to measure these parameters as concurrently as possible. Therefore, the following sequence is suggested for establishing the study area and collecting the data.

Step 1. Locate cross sections.

Step 2. Establish benchmark and permanent reference points (headstakes) for each cross section. Take photographs as required.

Step 3. Measure distance between transects and assign stationing or station index.

Step 4. Survey headstake elevations in reference to benchmark. Check for closure (Appendix A).

Step 5. Survey water surface elevations at each transect.

Step 6. Measure discharge at selected transect.

Step 7. Measure velocity distribution at remaining transects.

Step 8. Measure cross section profile. If sounding is used to measure the profile, steps 7 and 8 may be combined. Also, if sounding is used, be sure to survey the portion of the profile between the headstake and water's edge. Describe substrate.

Step 9. Recheck water surface elevations. If significantly different, redo Step 6.

Step 10: Review field notes before leaving site.

These data are common to both Manning equation and rating curve approaches to hydraulic simulation (with exception of velocity, as noted). If the rating curve approach is used, only the stage-discharge and velocity distributions need to be measured on subsequent trips to the field.

GUIDELINES FOR EFFICIENT DATA COLLLECTION

Experiences within the Instream Flow Group (IFG) have shown that certain procedures and types of apparatus can greatly increase consistency and efficiency of data collection. Some of these are mentioned in the text, but are important enough to be repeated.

1. Once duties of members of the field crew have been assigned, they should not be changed. While a change in duty may reduce boredom, it also introduces minor discrepancies in field techniques which can accumulate to significant errors. A minimum of two people is required for a crew. Three or four people are desirable. Crews of more than four people tend to be rather inefficient.

2. Use a standard survey notebook for taking field notes. The rag paper will hold a written image even if dropped in the water, and may be used during rainstorms. The notebook design helps prevent the loss of pages. As a footnote, the person taking data ought to have legible handwriting and be able to add and subtract accurately. Be sure to take notes in pencil, and cross out mistakes - <u>do</u> <u>not</u> <u>erase</u>. Use 3H pencil or harder lead to avoid smudging.

3. There should be more than one current meter per field crew. This will alleviate the problem of having two-thirds of the crew sleeping under a tree while velocities are being measured.

4. Avoid holding conferences around the level. The more people near the tripod, the greater the probability that the levelman will have to re-level the instrument.

5. For large or noisy rivers, two small walkie-talkies greatly reduce the strain of communicating between river and bank. In most situations a citizen's band radio will suffice. However, near large population centers or heavily travelled highways, the volume of CB traffic may require the use of commercial band radios. Alternatively, a prearranged set of flag or hand signals can be used for long-range communication.

6. A small hand winch, or come-along, is extremely useful for stretching taglines across larger streams.

7. Adequate preventive maintenance of equipment is essential. Current meters should be cleaned and oiled each time they are used, and frequently during use if the stream is carrying much sediment. Also, current meters should be spin-tested prior to each use (see Appendix B). Levels should be calibrated once a year, and immediately if dropped or knocked over.

8. Under no circumstance should taglines or boats be anchored to headstakes.

9. It is always better to enter a river measurement exercise over-equipped than under-equipped. This is particularly appropriate when using a boat in the collection of data. If a stream section or river stage appears to be unsafe to work in, do not work in it. Even if the data can be collected without drowning anyone, it is likely to be somewhat sloppy and of limited value in the hydraulic simulation. Suggested types of equipment for work in various sizes of river are listed in Appendix D.

10. A good rule of thumb is to collect data in the upstream direction; however, when using a tagline, start measurements at the upstream cross section and move downstream. This way the tagline may be moved with, not against, the current.

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APPENDIX A

SURVEYING TECHNIQUES

This appendix discusses the basic equipment and surveying techniques commonly used for hydraulic simulations in instream flow studies. The techniques described are somewhat abbreviated, and confined to those techniques most often used for instream flow work. For a more complete description of surveying techniques, the reader is referred to a professional surveying course or to one of the references quoted in this text.

Two types of measurements are regularly used in instream flow studies. The first type is called leveling, which is the process of finding the difference in elevation between two points by measuring the vertical distance between the horizontal planes intersecting the two points. The second type is the measurement of linear distances between two points, which may be achieved by taping or use of surveying or electronic devices.

USE OF THE LEVEL

A level is an instrument combining a telescopic sight having both vertical and horizontal cross hairs, with a level vial which indicates when the instrument is in a level position. The types of levels most commonly used for instream flow work are the American type engineer's level and the self-leveling level. Occasionally, a transit is used as a level, but these are somewhat more difficult to use than a level, and are somewhat less precise for leveling. A line drawing of an engineer's level is shown in Figure A-1 to familiarize the reader with some of the principal components of the instrument. A self-leveling level has many components in common with the engineer's level.

Focusing

The process of focusing is the most important function to be performed with a telescopic sight. The telescope consists of an objective lens (mounted on a sliding tube inside the tube), a reticle (cross hairs), and an eyepiece. The purpose of the negative lens is to focus the object image on the reticle. The focusing pinion for the negative lens is a large knob on the top or side, near the center of the tube.

Since the reticle remains fixed in the telescope tube, the distance between it and the eyepiece must be adjusted to suit the eye of the observer. This is done by focusing the eyepiece on the cross hairs with the eyepiece focusing ring. After the eyepiece has been adjusted, objects are brought into focus with the objective focusing pinion. If the cross hairs appear to travel over the object sighted when the eye is shifted slightly in any direction, parallax exists. Further adjustment of either the objective lens system or the eyepiece is required to eliminate parallax.

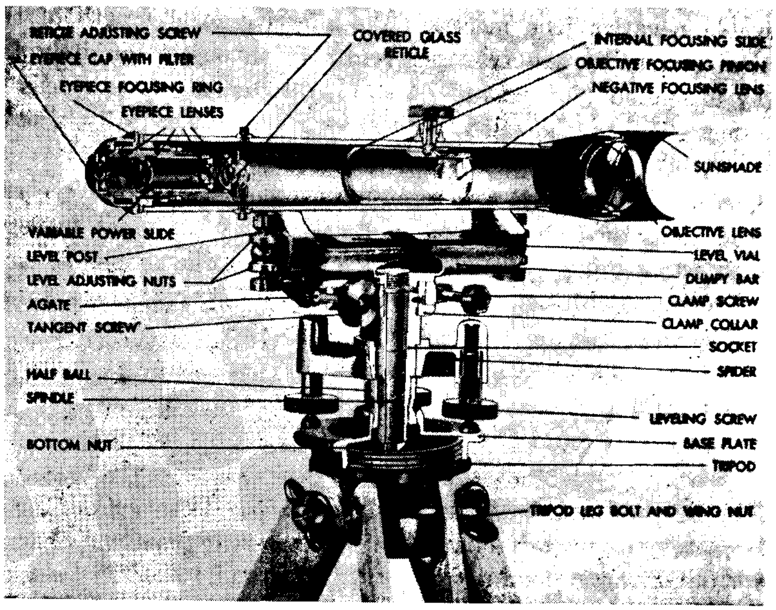


Figure A-1. Line Drawing of an engineer's level.

Setting Up the Level

The safest way of transporting a level is to keep it in its case. In the field, it is common practice to carry the tripod and level from place to place as a single unit. Special care should be exercised when moving the level in this fashion, particularly if the instrument is being carried across the river. It is a good idea for the rodman to accompany the levelman across the river, prepared to grab the levelman should the levelman lose his balance or step into a hole.

When mounting the level on the tripod, it is important that the level is screwed snugly to the tripod. If the level is mounted too loosely, the instrument is unstable and will be difficult or impossible to level. If the mount is too tight, the instrument may freeze to the tripod, or the threads may be stripped.

When setting up the tripod, the tripod leg bolts and wing nuts should be loose. The tripod should be set up in such a way that the platform is approximately level to start with. This will facilitiate leveling the instrument. If possible, determine the direction of sighting for most of the sights, and set one leg of the tripod in that direction. This will leave a space for the levelman to stand while sighting, without straddling one of the legs. On side hill setups, placing one leg on the uphill side and two on the downhill slope usually gives a stable setup. Once the tripod is adequately positioned, the legs may be firmly pressed into the ground by stepping on the tripod leg spurs near the bottoms of the legs. Before leveling the instrument, be sure to retighten the tripod leg wing nuts - <u>do</u> <u>not</u> <u>over-tighten</u>.

Leveling the Instrument

One of the most obvious differences between an engineer's level and a self-leveling level is the procedure used in leveling the instrument.

Engineer's levels typically have four leveling screws and a spirit level (a sealed vial partially filled with alcohol). A self-leveling level has three leveling screws and a bull's-eye (circular) spirit level.

In leveling the four-screw head, the telescope is turned until it is over two opposite (diagonally opposed) screws. The bubble is approximately centered in the level vial by turning both leveling screws in opposite directions, at the same speed, with both hands. A simple rule is that the bubble moves in the same direction as the left thumb. Failure to move both screws at the same speed will often leave the leveling head wobbly. The procedure is then repeated with the telescope over the other two leveling screws. It is impractical to attempt to exactly center the bubble on the first try, since it will be thrown off during the cross-leveling. Readjusting each pair of screws about three times is usually enough to complete the leveling process.

A self-leveling level has a three-screw head and a bull's-eye spirit level. These levels contain a system of internal prisms which allow a level line of sight even if the instrument itself is not exactly level. For a three-screw head, the telescope is aligned over one screw. The telescope is made level by alternately turning this screw, then the other two simultaneously, until the bubble lines up in the center of the bull's-eye. The telescope need not be rotated in the process.

Reading the Level Rod

The type of rod used in most instream flow work is the self-reading rod, which is read by the levelman as he sights through the telescope and notes the apparent intersection of the horizontal cross hair with the rod. The two most commonly used rods are the wooden, 13-foot Philadelphia rod, and the 25-foot telescoping fiberglass rod. For extended use, the Philadelphia type rod does not hold up too well under the rigors of river cross-sectioning. However, the wooden rod floats, which can be a definite advantage if the rodman loses his grip. The fiberglass rod does not float, and a few have been lost by dropping them into deep pools. The fiberglass rod is very durable, and the 25-foot length eliminates many problems when large variations in elevations are encountered. Regardless of the type of rod used, a standard convention is used in reading them.

The graduations on a stadia rod are accurately-painted alternate black and white spaces 0.01 ft (1 cm if metric) in width. The 0.1 and

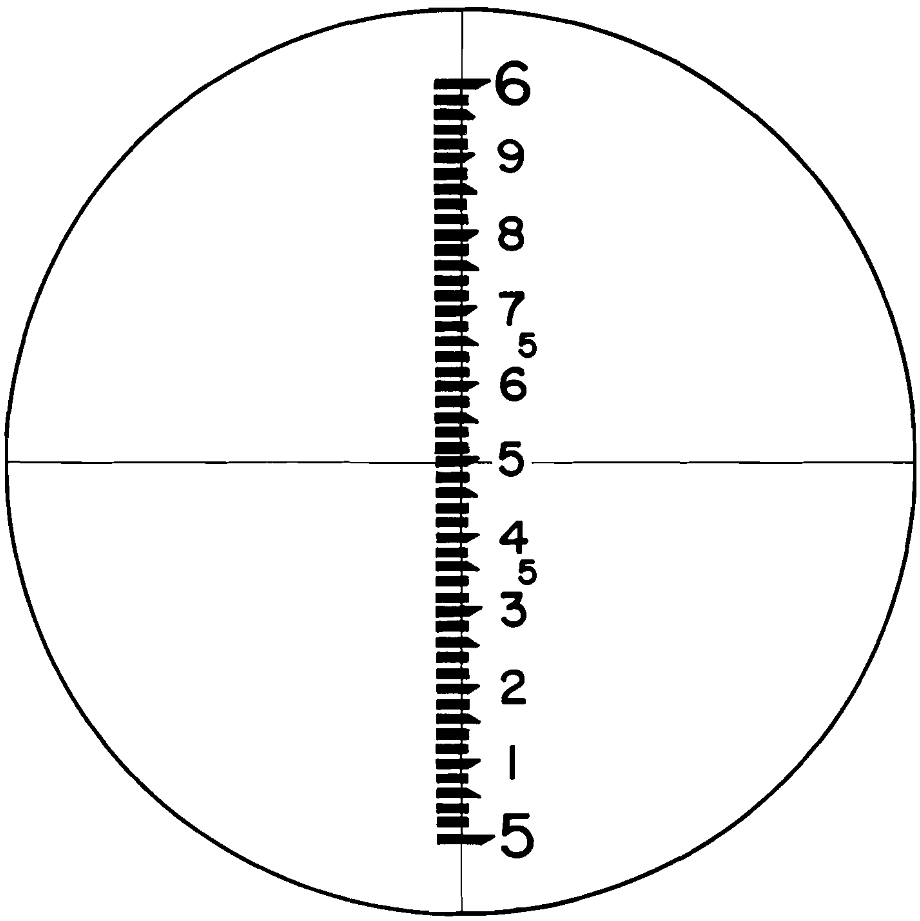


Figure A-2. View through a level on a rod reading of 5.495 feet.

0.05 ft graduations are emphasized by points or spurs extending the black markings. Tenths are designated by black numerals straddling the proper graduation, whereas whole feet or meters are marked by red numerals. On some rods, small red numerals are placed alongside the black numerals to facilitate reading the proper "whole foot" on short sights where only a small portion of the rod is visible through the level.

A typical view through a level would appear as in Figure A-2. If the instrument is properly leveled and the rod held plumb on the objects, the elevation of the object is 5.495 feet below the elevation of the instrument.

Many levels have short horizontal cross hairs above and below the center cross hair. These are called stadia hairs and are used in the measurement of distances. A discussion on the use of stadia will be given later in this appendix. Be especially careful not to read an elevation off a stadia hair instead of the central cross hair.

Holding the Rod

Normally, the duties of the rodman are a relatively simple routine, and border on being boring. However, when collecting data for hydraulic simulation the rodman's life is more exciting. Basically, the rodman must keep the rod plumb over the point to be measured. This task of handling a long rod in three feet of fast water is easier said than done.

The rodman's job can be made easier by equipping the rod with a rod level. A rod level is a small bull's eye spirit level, mounted on an L-shaped bracket which can be attached to the rod. When the bull's eye bubble is centered, the rod is plumb in both directions. This is especially important when dipping the water surface.

In the absence of a rod level, the levelman makes certain the rod is plumb in a lateral direction by its coincidence with the vertical cross hair. The rodman then rocks the rod forward and backward, through the plumb line. The levelman watches through the telescope, noting the minimum rod reading. The minimum rod reading will occur when the rod is standing plumb.

As a related issue, the rodman must be aware of tree branches which tend to deflect the top of the rod. This is particularly true when the long fiberglass rods are used. They may be perfectly plumb at the rodman's level, but bowed away from plumb up where the readings are being made. This is a point to keep in mind when placing headstakes. Errors can be reduced significantly by avoiding areas of low canopy. If these areas cannot be avoided, the rodman may have to serve as a part-time tree pruner.

LEVELING

Two different types of leveling are commonly used for collection of hydraulic simulation data. Differential leveling is the process of determining the difference in elevation between two points. Differential leveling is used for determining headstake and water surface elevations. Profile leveling, the process of determining the elevation of points at measured distances along a selected line, is used for measuring the cross section profiles, and determining the water surface profile.

Several definitions are commonly used in all types of leveling:

1. <u>Backsight</u>

   A backsight (BS) is a rod reading taken on a point of known elevation. It is the vertical distance between the line of sight and the point of known elevation on which the rod is held. The line of sight is always (except in extremely unusual cases) higher than the benchmark or turning point. Therefore, the algebraic sign of the backsight rod reading is positive (+). There is only one backsight for each setup of the instrument. Please note that the term "backsight" has nothing to do with the direction in which the instrument is pointed.

2. <u>Height of Instrument</u>

   The height of instrument (HI) is the elevation of the line of sight when the instrument is level. It is found by adding the backsight rod reading to the known elevation of the point on which the backsight was taken. Figure A-3 shows a backsight on a benchmark to determine height of instrument.

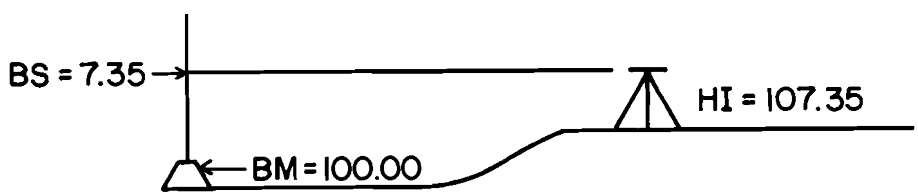


Figure A-3: Determination of the height of instrument by taking a backsight on a bench mark.

3. Foresight

A foresight (FS) is a rod reading taken on any point, the elevation of which is to be determined. The algebraic sign of the foresight is negative (-). The FS is subtracted from the HI to find the elevation of the point. Figure A-4 shows a foresight on a headstake to determine the elevation of the headstake.

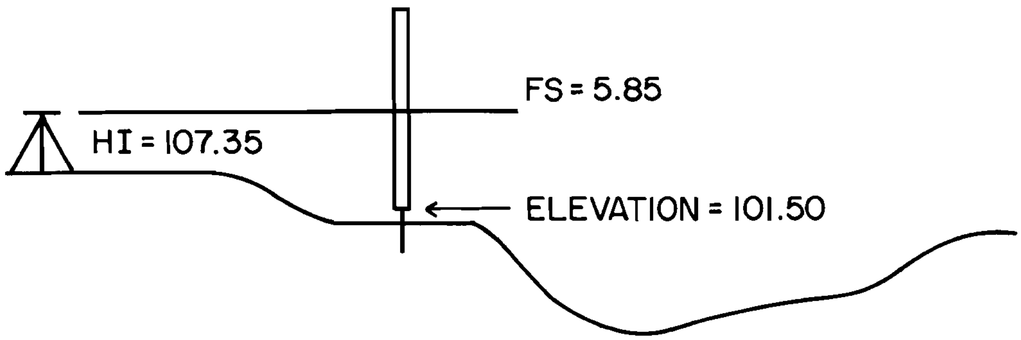


Figure A-4. Determination of an unknown headstake elevation by taking a foresight from a known height of instrument.

4. Turning Point

A turning point (TP) is a temporary benchmark upon which foresight and backsight readings are taken to continue the line of levels. Turning points should be solid and not likely to change elevation while the instrument is being moved. The rod should remain on the TP while the instrument is moved, unless the TP is a stake or some other object which would allow the rod to be reset at exactly the same place for both plus and minus sights. The rod should not be placed on the ground when making a turn, as it may settle before a new HI is determined. Figure A-5 shows the principle involved in using a turning point.

Differential Leveling

One of the most common applications of differential leveling is to run a circuit of levels to determine the elevations of cross section headstakes relative to a previously established benchmark. Unless

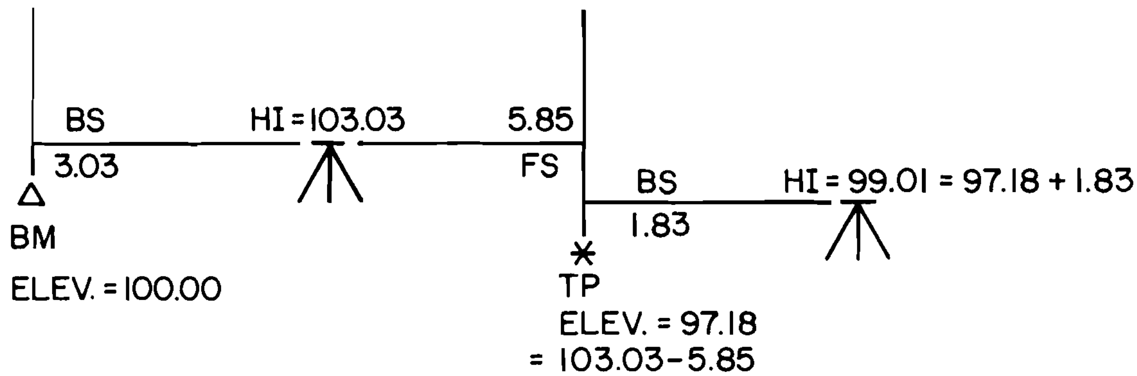


Figure A-5. Principle involved in using a turning point.

otherwise instructed, the elevation of only one headstake per transect is normally required. Once the elevations of transect headstakes is determined, the headstake acts as a benchmark for subsequent measurements.

The most important concept of differential leveling is level loop closure or "return check." Elevations are determined for each headstake on one side of the river, moving the level as required to make sights. After the last headstake has been measured, the instrument is moved, if only picked up and then releveled. Then the same headstakes are resurveyed on a return survey back to the benchmark. This procedure deviates somewhat from the classic procedures expounded in surveying texts. However, experience has shown that this procedure for closure gives adequate closure precision for the designated purpose. While it may be possible to sight all of the headstakes from one setup position, the instrument must be moved at least once during the level loop, to detect errors evolving from instrument setup. Another point to keep in mind is that error reduction can be significant if backsights and foresights are kept about the same length.

Figures A-6 and A-8 show a typical headstake level loop performed on the hypothetical stream reach presented in the section on transect placement. In Figures A-6 and A-8, backsights are indicated by plus signs (++++) while foresights are indicated by minus signs (----). The rod reading for each sight is written directly above the line of sight. Elevations of headstakes, turning points, and instrument heights are given for each location.

Figure A-6 shows the forward (usually upstream) survey of headstake elevations through the study reach. Field notes for the forward survey are shown in Figure A-7. The left hand sheets of surveying field notebooks contain 6 columns. Five columns are needed for leveling notes.

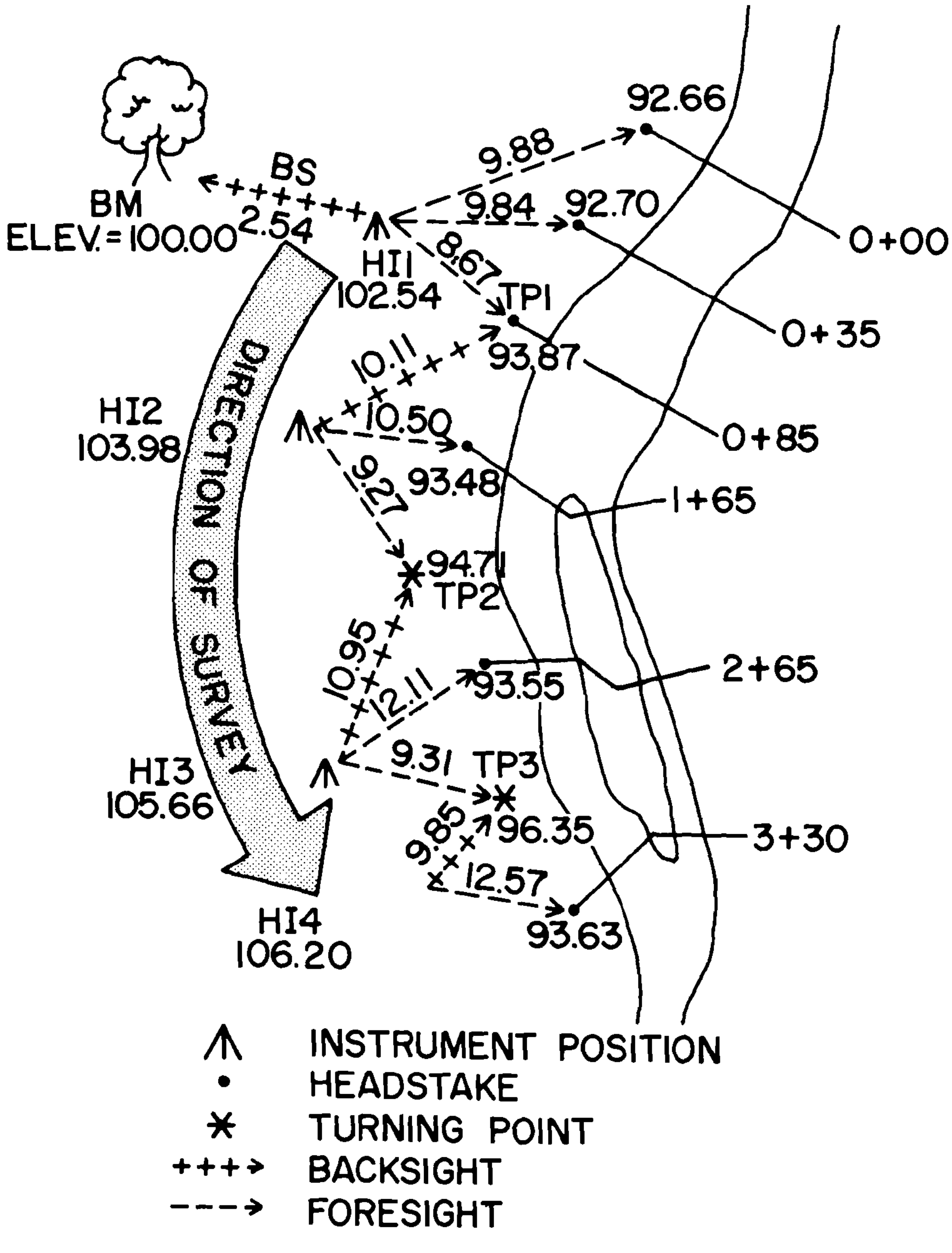


Figure A-6: Forward (upstream) survey of headstake elevations through the study area.

STA	BS	HI	FS	Elev.
BM	2.54			100.00
		102.54		
0+00			9.88	92.66
0+35			9.84	92.70
0+85 (TPI)	10.11		8.67	93.87
		103.98		
1+65			10.50	93.48
TP2	10.95		9.27	94.71
		105.66		
2+65			12.11	93.55
TP3	9.85		9.31	96.35
		106.20		
3+30			12.57	93.63

Figure A-7: Field notes from forward (upstream) survey of headstake elevations at study site.

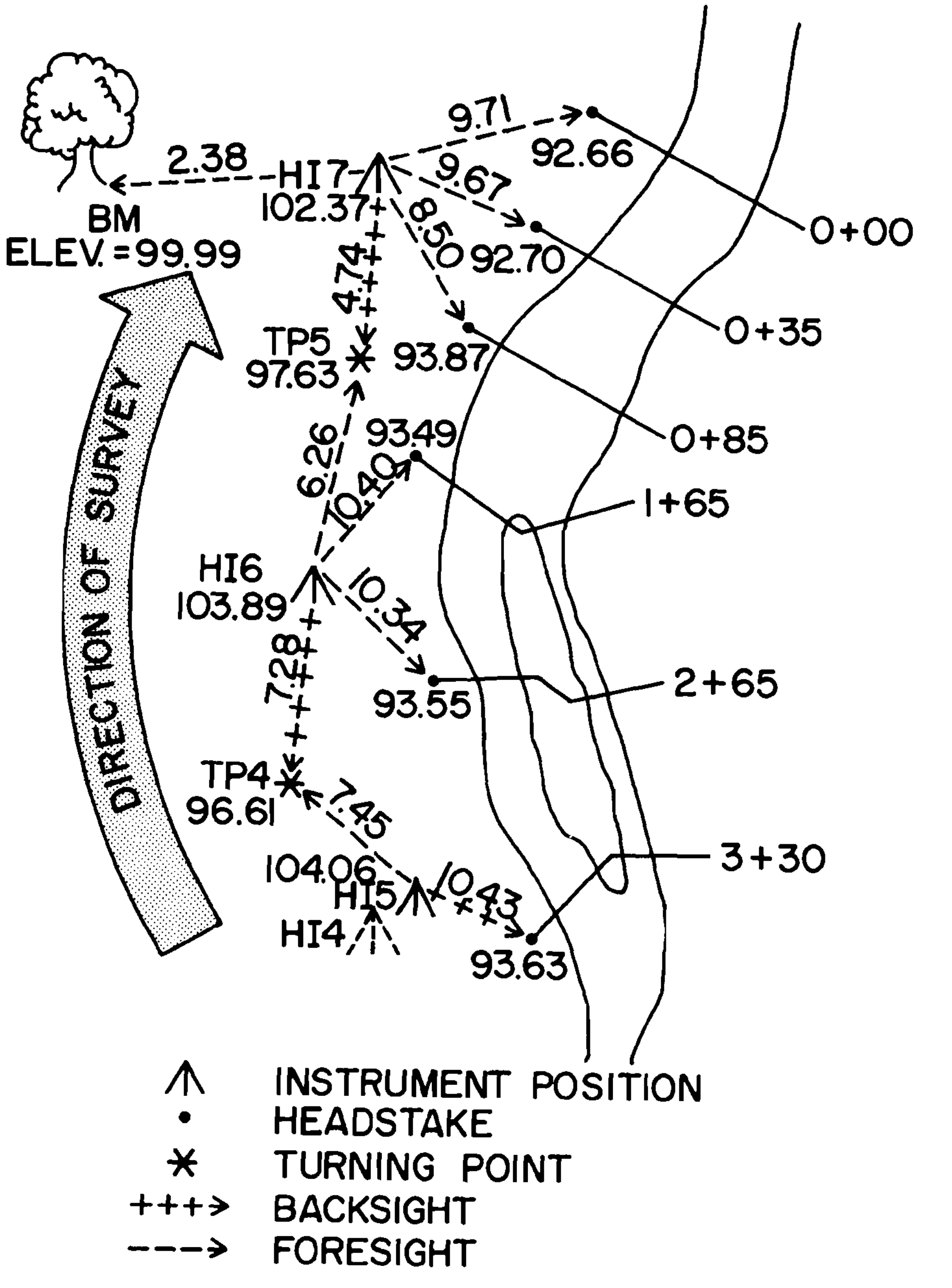


Figure A-8: Return check survey (downstream) of headstake elevations and benchmark through the study area.

From left to right, the column headings are station (STA), backsight (BS), instrument height (HI), foresight (FS), and elevation (Elev.). Elevations should be calculated for each headstake or turning point as the survey proceeds, to facilitate finding errors on the return survey.

Figure A-8 shows instrument setups, rod readings, and elevations for the return leg of the level loop. It is not unusual for the return leg to be a bit more organized than the forward leg, as the crew becomes more familiar with the study area. Figure A-9 shows the field notes for the return survey of the study area.

Note that the field notes in Figures A-7 and A-9 contain the same information as the schematics in Figures A-6 and A-8. It is somewhat confusing to the beginner to interpret two readings on the same line, such as the entries for turning points. This makes it especially important to calculate elevations as the survey proceeds. Simply remember that the elevation of a turning point must be calculated before a backsight on the turning point can be used to calculate the new HI.

Having completed the level loop, we may now determine whether or not we have "closed" within the allowable range of error. Using equation 20 (page __), the allowable error of closure is:

$$\text{Allowable error} = 0.05\ \frac{660}{5280} = 0.018 \qquad (21)$$

The error of closure at the benchmark for the sample level loop was 100.00 - 99.99 = 0.01 ft. Therefore, closure was obtained within allowable limits for third order precision.

Profile Leveling

Profile leveling requires the measurement of both horizontal distances and vertical elevations. When measuring either the cross-sectional or water surface profiles the headstakes for which elevations have been determined are used as benchmarks. The instrument is set up at each cross section and a backsight taken on the headstake to determine HI. When the height of instrument has been determined, elevations are measured at various distances across the transect (foresights). There is no way to check the foresights, other than to rerun the profile; therefore, extreme care must be taken to prevent mistakes in reading and recording the rod readings and in calculating the elevations at all stations.

Figure A-10 shows a typical instrument setup for obtaining the water surface elevation and cross sectional profile of transect 0+00 of Figures A-6 and A-8. Note that the precision of ground elevations

STA	BS	HI	FS	Elev.	
3+30	10.43			93.63	
		104.06			
TP4	7.28		7.45	96.61	
		103.89			
2+65			10.34	93.55	
1+65			10.40	93.49	
TP5	4.74		6.26	97.63	
		102.37			
0+85			8.50	93.87	
0+35			9.67	92.70	
0+00			9.71	92.66	
BM			2.38	99.99	

Figure A-9: Field notes from return-check survey of headstake and benchmark elevations at study site.

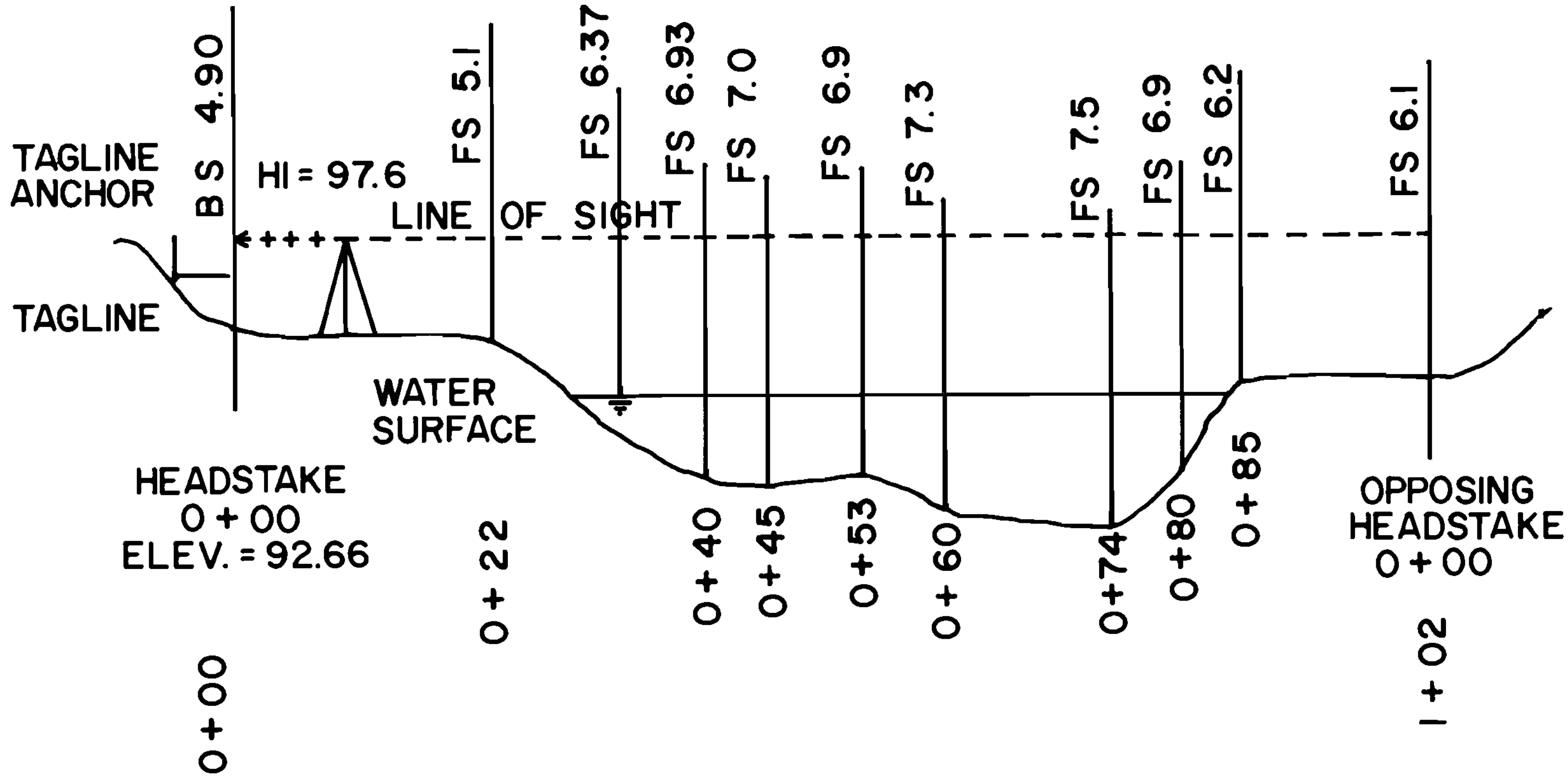


Figure A-10: Profile leveling a cross section in a study area.

across the transect is to the nearest 0.1 ft, but the water surface elevation is measured to 0.01 foot. A typical set of field notes for the profile is shown in Figure A-11. Note that the field notes for each profile contains a description of the substrate as provided by the rodman as he traverses the cross section.

MEASURING DISTANCES

Distances are measured at several times during a field survey; distances between headstakes, thalweg distances, and distances across the profiles of the cross sections. Distance measurements are also used for locating and re-locating the benchmark or headstakes. The most commonly used instruments for measuring distance are the tape, stadia, and electronic distance meters (EDM).

Taping

Taping is the most commonly used technique for measuring distance. Several types of tapes are available, but the steel surveyor's and the tagline are the most popular. The surveyor's tape is graduated in one-foot increments, with the last foot subdivided into tenths. Taglines may be small cable, cotton rope, or polypropylene, and are usually marked in one-foot increments, with special markings at all 10- and 100-foot marks. The only constraint in selecting a tape, besides personal preference, is that the tape should be incremented in the same units as the level rod. If the rod is metric, use a metric tape; avoid tapes marked in feet and inches. If measurement units are mixed, the field notes will be subject to a great deal of conversion, which is a potential source of error.

The distance to be measured is the horizontal distance between objects, whether headstakes, transects or positions on a transect. One advantage of using a tape is that it is possible to measure distances along a curve. This is particularly useful when measuring between transects along a meandering stream. This procedure introduces some error, and can be eliminated by placing transects close enough that the distance between can be approximated by a straight line. Greater errors are introduced by stringing the tape up, over, and around obstructions along the bank. One may avoid this problem by setting up the study area at a fairly low flow, so that the tape can be stretched along the streambed and not along the bank.

Some error may be introduced by sag in a tape when stretched across a transect. In many cases, the error is small enough that it can be neglected. However, for long transects, the magnitude of the sag error may be large enough to warrant correction. The correction factor for sag can be calculated by:

Cross Section Profile Station 0+00, Deer Creek Site A-1					
STA	BS	HI	FS	Elev.	Substrate
0+00	4.90			92.66	Left Bank Headstake
					Silt
		97.56			
0+22			5.1	92.5	50/50 sand, gravel
WS			6.37	91.19	water surface
0+40			6.9	90.7	25/25/50 gravel,
					small cobble, medium
					cobble
0+45			7.0	90.6	25/75 large cobble,
					small boulder
0+53			6.9	90.7	medium boulder
0+60			7.3	90.3	medium boulder
0+74			7.5	90.1	medium boulder
0+80			6.9	90.7	50/50 small boulder,
					medium cobble
0+85			6.2	91.4	50/50 sand, gravel
1+02			6.1	91.5	right bank headstake,
					silt

Figure A-11: Field notes for profile levelling, for cross section 0+00 from Figure A-10.

$$C = \frac{-W^2 L^3}{24\ P^2} \qquad (22)$$

where,

C = Difference between length of curve (due to sag) and straight line from one support to next, in feet
L = unsupported length of tape, in feet
W = weight of tape, in pounds per foot
P = pull on tape, in pounds

The correction factor, C, is always negative and is added algebraically to L to determine the corrected total distance across the channel. Errors due to sag are systematic in that they always cause the recorded distance to be greater than the true distance.

A correction for sag is necessary only if the magnitude of the error is significant to the level of precision with which the distance measurements are made. For example, a 250-foot cable weighing 0.032 pounds per foot, with a pull of 500 pounds, would have a correction of 1.33 feet. This would equate to an incremental adjustment of 0.04 feet every 10 feet. Thus, with a precision requirement of measuring to the nearest foot, the correction factor is negligible. However, if only 50 pounds of pull were applied, the correction factor would be -13.3 feet, or about 0.5 feet per 10 feet. In this case, correction would be justified. It should be obvious that one way of reducing errors due to sag would be to use a tightener to apply a large amount of pull to the tag line or tape. Using a tightener, such as a winch or come-along, it may be possible to tape across channels 500 to 1000 feet wide (the upper limit to successful taping has not been established). Without benefit of such equipment, the upper limit for taping across channels is about 400 feet.

Use of Stadia

Horizontal, straight line distances can be measured directly with many levels. These levels contain two small horizontal cross hairs above and below the main horizontal cross hair. The distance between the telescope and rod is found by subtracting the rod reading for the lower stadia cross hair from that of the upper stadia hair and multiplying by a constant (usually 100). Figure A-12 shows a view through the telescope on a stadia rod 93 feet away.

Using stadia, a precision of about 1/500 can be achieved with reasonable care. Errors in stadia work are usually the result of poor rod readings. With this precision, the upper limit for a single stadia-sight distance is about 500 feet.

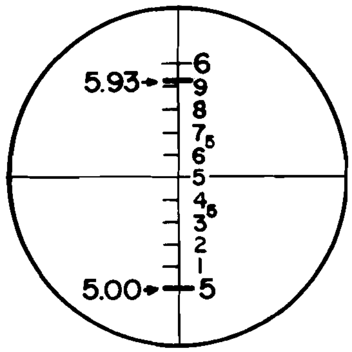


Figure A-12: View of stadia hairs through a level, on a stadia rod 93 feet away.

Electronic Distance Meters (EDM's)

An electronic distance meter works on the principle of determining the time required for an induced electromagnetic wave to reach a reflector and return to the sender. Automatic equipment converts this time to a distance. Two different types of EDM's are available. The first type is activated by an infrared frequency signal or laser beam. The signal is reflected from a bank of retroprisms, and the distance calculated by the time required for the signal to return to the source. Since this type of EDM is "light-frequency" activated, a clear line of sight to the target is required. Foliage, mist, and smoke can interfere with the signal. Additionally, the target and signal source must be essentially on-line, resulting in a narrow band within which the target must remain in order to obtain a reading.

The second type of EDM broadcasts a microwave to a receiver, which relays the signal back to the source. This type of EDM is unaffected by reduced visibility, but may be jammed by transmission lines or other high energy electromagnetic fields. They are not affected by normal radio transmissions. The feature which makes these meters particularly attractive for hydrographic work is that they can maintain signal contact when the source and receiver units are offset by as much as 45°. Thus, it is much easier to prevent loss of signal when moving the boat across the channel. An advantage of the "line of sight" meters is that if contact is broken, they do not need to be recalibrated in order to continue measurements. The price of electronic distance meters is quite high, ranging from about $3000 to $20,000 in 1978 (Appendix D). However, they are practically indispensable for working on large rivers. Even a fairly "inexpensive" EDM has an accuracy of about 0.04 feet at 1600 feet. The price is usually reflected in the range and beam width, not the accuracy.

LOCATING AND RELOCATING BENCHMARKS OR HEADSTAKES

One of the most immediate problems facing the field crew contemplating a return to a study area is their ability to relocate the benchmark and headstakes. If the study area is one of many, and/or heavily vegetated, documentation of benchmarks and headstakes locations is a great concern. It is amazing how quickly those things can disappear in the brush. Two fairly simple methods can be used to accomplish this relocation: the chord method or the bearing and range method.

Chord Method

The chord method utilizes a fixed distance from two or more easily recognized landmarks to a benchmark or headstake. This method may be used in conjunction with on-site photographs and the bearing and range method.

Figure A-13 shows the concept of the chord method for locating a benchmark. During the initial placement of the benchmark, the distance

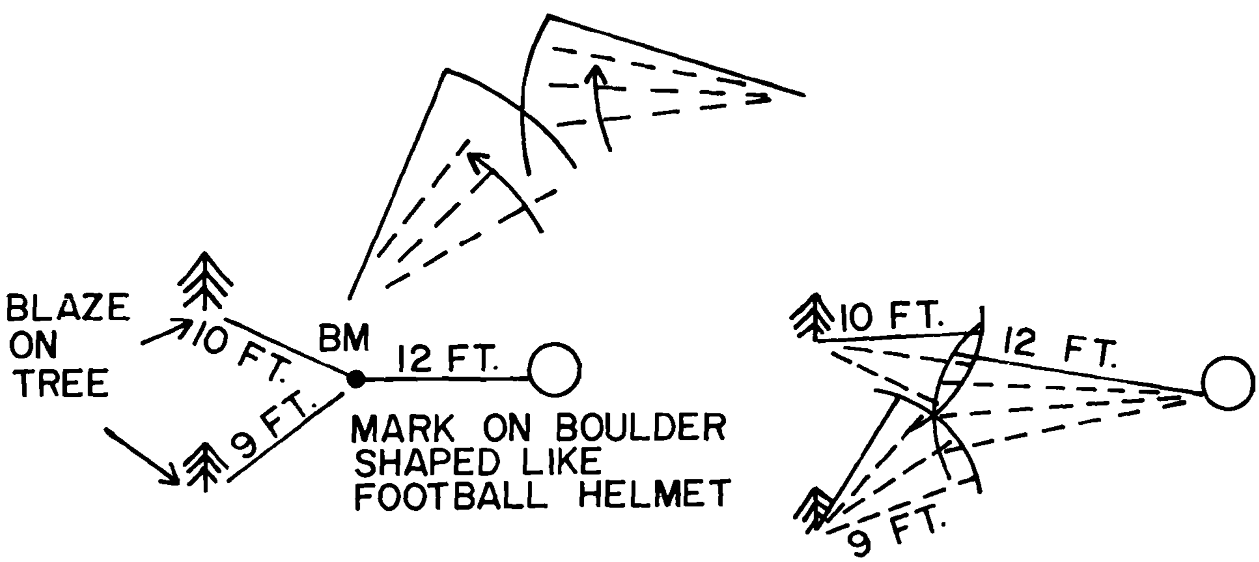


Figure A-13. Chord method of relocating a benchmark.

to a well defined landmark, is documented (description of the landmark, including photographs, Figure A-13a). On subsequent trips the landmarks are located, and an arc of the prescribed distance made from each landmark (Figure A-13b). The intersection of the chord traces marks the location of the benchmark.

Bearing and Range Method

The bearing and range method utilizes a horizontal angle and a distance from one point to another to describe the second point's position. Angles may be described by a bearing or an azimuth. A bearing is the angle measured from either the north or south, toward the east or west, such that the reading is less than 90 degrees. For example, a northeast bearing would be read as N 45° E. The proper quadrant is shown by the letter N or S preceding the angle, and the letter E or W following it. An azimuth is an angle measured from north and may range from 0 degrees to 359 degrees. They do not require letters to identify the quadrant. Bearings are usually read from an instrument such as a Brunton compass or a transit. Azimuths are usually given by common pocket compasses. The system used is irrelevant, and largely a matter of preference for the field crew. It is important, however, to obtain "true" bearings or azimuths. Therefore, it is essential that the declination of the compass is set correctly, and the declination used written in the field notes.

The bearing and range system is particularly useful for identifying headstake locations relative to the benchmark. Figure A-14 shows a sketch of our hypothetical stream reach as it might appear in the field notes, giving headstake locations by bearing and range.

SUGGESTED READING

Two references were heavily utilized in the preparation of this appendix. For additional details concerning surveying techniques the reader is referred to Brinker and Taylor (1963) and Roth, et al. (1977).[1]

[1]References cited in appendices are listed in references following main text.

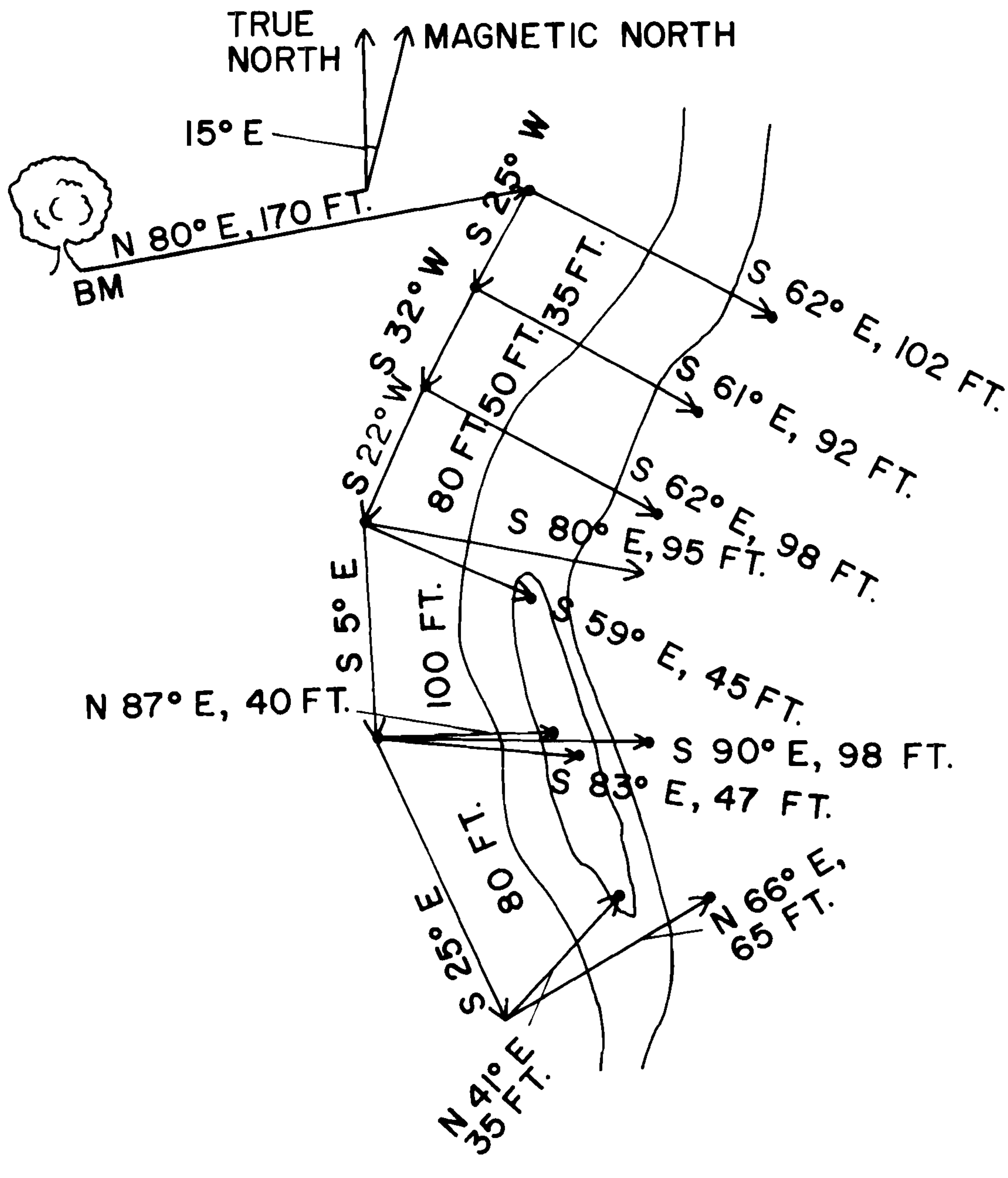


Figure A-14: Bearing and range method of locating headstakes from a known benchmark position.

APPENDIX B

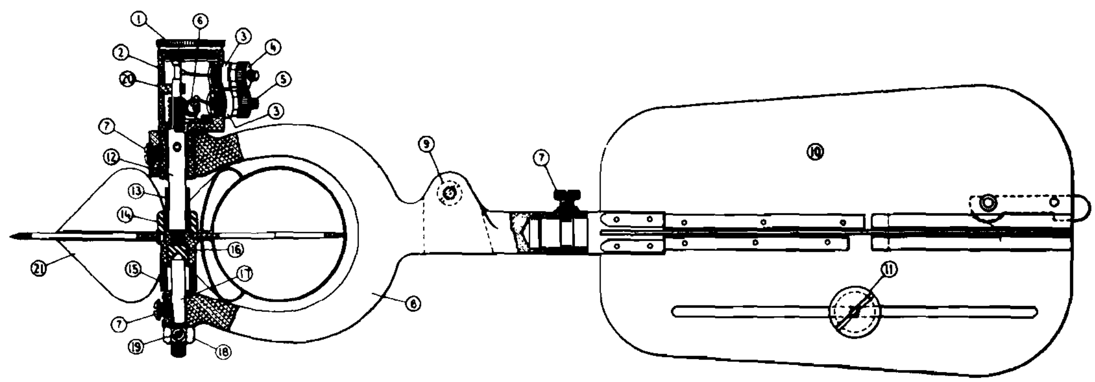
VELOCITY, DEPTH AND DISCHARGE MEASUREMENTS

Velocity measurements are made to determine the velocity distribution across a transect, and are also used in the calculation of the discharge. Depth measurements are also used in the calculation of the discharge; and on rivers where surveying the cross section is impractical, depth measurements can be substituted for rod readings to obtain the cross sectional profile.

EQUIPMENT

Current Meters

The most commonly used instrument for measuring the velocity is a current meter. The meter consists of a wheel which rotates in flowing water and a device for determining the number of revolutions. Figure B-1 shows a line drawing of a Price AA current meter, which is fairly typical of all "vane-type" meters. As the bucket wheel (21) rotates, an electrical contact is closed on either a single-contact cam, or a penta gear (6). If a headset or counter is attached to the single contact post (4), a signal is produced each time the bucket wheel completes a revolution. If the headset is connected to the penta-contact post (5), a



ASSEMBLY

LIST OF PARTS

1 Cap for contact chamber
2 Contact chamber
3 Insulating bushing for contact binding post
4 Single-contact binding post
5 Penta-contact binding post
6 Penta gear
7 Set screws
8 Yoke
9 Hole for hanger screw
10 Tailpiece
11 Balance weight
12 Shaft
13 Bucket-wheel hub
14 Bucket-wheel hub nut
15 Raising nut
16 Pivot bearing
17 Pivot
18. Pivot adjusting nut
19 Keeper screw for pivot adjusting nut
20. Bearing lug
21. Bucket wheel

Figure B-1. Line drawing of a Price AA current meter.

signal is produced once every five revolutions. The penta-contact is very useful in fast water.

The velocity at the point of the current meter is measured by counting the number of signals (revolutions in a specified time interval). Thus, a standard piece of equipment accompanying the use of a current meter is a stopwatch. Each meter is calibrated by the supplier and an equation for the relationship between velocity and revolutions per unit time derived. For most Price meters, the meter is supplied with a rating table, such as the one shown in Figure B-2, which shows the velocity for a given number of revolutions in a given time interval. From Figure B-2, 40 revolutions in 40 seconds equals a velocity of 2.17 feet per second. Notice that 40 seconds is the smallest time interval listed on the rating table. This time interval is required to obtain a time-averaged velocity at the point. The user would be well advised to memorize the "stop counts" in the columns of Figure B-2. Stopping the count at some intermediate number of revolutions (27, for example) negates the use of the table and requires the use of the equation to calculate the velocity.

DEPARTMENT OF INTERIOR - BUREAU OF RECLAMATION

RATING TABLE FOR TYPE ______ METER NR ______

SUSPENSION ______ RATED ______

EQUATIONS: $V = 2.14\,N + 0.03$ $N < 1.00$

$V = 2.19\,N - 0.01$ $N > 1.00$

Limits of Actual Rating ______ to ______ ft. per sec.
at Bureau of Standards, Washington, D.C.

Condition of Meter

Time, Seconds	VELOCITY IN FEET PER SECOND — Revolutions 3	5	7	10	15	20	25	30	40	Time, Seconds
40	0.190	0.297	0.404	0.565	0.832	1.10	1.37	1.63	2.17	40
41	0.186	0.291	0.395	0.552	0.813	1.07	1.34	1.60	2.12	41
42	0.181	1.284	0.387	0.539	0.793	1.05	1.30	1.56	2.07	42
43	0.179	0.278	0.378	0.528	0.776	1.03	1.27	1.52	2.02	43
44	0.175	0.273	0.370	0.515	0.759	1.00	1.25	1.49	1.98	44
45	0.173	0.267	0.363	0.505	0.742	0.980	1.22	1.46	1.93	45
46	0.169	0.263	0.355	0.494	0.727	0.960	1.18	1.43	1.89	46
47	0.166	0.256	0.348	0.485	0.712	0.941	1.17	1.40	1.85	47
48	0.162	0.252	0.342	0.475	0.697	0.922	1.14	1.37	1.81	48
49	0.160	0.248	0.336	0.466	0.684	0.903	1.12	1.34	1.78	49
50	0.158	0.244	0.329	0.458	0.672	0.886	1.10	1.31	1.74	50
51	0.156	0.239	0.323	0.449	0.659	0.868	1.08	1.29	1.71	51
52	0.154	0.235	0.318	0.440	0.646	0.853	1.06	1.26	1.68	52
53	0.151	0.231	0.312	0.434	0.635	0.836	1.04	1.24	1.65	53
54	0.149	0.229	0.308	0.425	0.624	0.821	1.02	1.22	1.62	54
55	0.147	0.224	0.301	0.419	0.614	0.808	1.00	1.20	1.59	55
56	0.145	0.220	0.297	0.413	0.603	0.793	0.984	1.18	1.56	56
57	0.143	0.218	0.293	0.404	0.592	0.781	0.969	1.16	1.53	57
58	0.141	0.214	0.288	0.398	0.584	0.768	0.952	1.14	1.51	58
59	0.139	0.211	0.284	0.391	0.573	0.755	0.937	1.12	1.48	59
60	0.137	0.207	0.280	0.387	0.565	0.742	0.922	1.10	1.46	60
61	0.134	0.205	0.276	0.380	0.556	0.731	0.907	1.08	1.43	61
62	0.132	0.203	0.271	0.374	0.547	0.721	0.892	1.07	1.41	62
63	0.132	0.199	0.267	0.370	0.539	0.708	0.879	1.05	1.39	63
64	0.130	0.196	0.263	0.363	0.530	0.697	0.866	1.03	1.37	64
65	0.128	0.194	0.261	0.359	0.524	0.689	0.853	1.02	1.35	65
66	0.126	0.192	0.256	0.355	0.515	0.678	0.841	1.00	1.33	66
67	0.126	0.190	0.252	0.348	0.509	0.669	0.828	0.988	1.31	67
68	0.124	0.188	0.250	0.344	0.502	0.659	0.817	0.973	1.29	68
69	0.122	0.184	0.246	0.340	0.494	0.650	0.804	0.960	1.27	69
70	0.122	0.181	0.234	0.336	0.487	0.642	0.793	0.948	1.25	70
	3	5	7	10	15	20	25	30	40	

Time, Seconds	VELOCITY IN FEET PER SECOND — Revolutions 50	60	80	100	150	200	250	300	350	Time, Seconds
40	2.73	3.27	4.37	5.47	8.20	10.94	13.68	16.42	19.15	40
41	2.66	3.19	4.26	5.33	8.00	10.67	13.34	16.01	18.69	41
42	2.60	3.12	4.16	5.20	7.81	10.42	13.02	15.63	18.24	42
43	2.54	3.05	4.06	5.08	7.63	10.18	12.72	15.27	17.82	43
44	2.48	2.98	3.97	4.97	7.46	9.94	12.43	14.92	17.41	44
45	2.42	2.91	3.88	4.86	7.29	9.72	12.15	14.58	17.02	45
46	2.37	2.85	3.80	4.75	7.13	9.51	11.89	14.27	16.65	46
47	2.32	2.79	3.72	4.65	6.98	9.31	11.64	13.94	16.30	47
48	2.27	2.73	3.64	4.55	6.83	9.11	11.40	13.68	15.96	48
49	2.22	2.67	3.57	4.46	6.69	8.93	11.16	13.40	15.63	49
50	2.17	2.62	3.49	4.37	6.56	8.75	10.94	13.12	15.32	50
51	2.13	2.57	3.43	4.28	6.43	8.58	10.73	12.87	15.02	51
52	2.09	2.52	3.36	4.20	6.31	8.41	10.52	12.63	14.73	52
53	2.05	2.47	3.30	4.12	6.19	8.25	10.32	12.39	14.45	53
54	2.01	2.42	3.23	4.05	6.07	8.10	10.13	12.16	14.18	54
55	1.98	2.38	3.18	3.97	5.96	7.95	9.94	11.94	12.93	55
56	1.94	2.34	3.12	3.90	5.86	7.81	9.77	11.72	13.68	56
57	1.91	2.30	3.06	3.83	5.75	7.67	9.60	11.52	13.44	57
58	1.87	2.26	3.01	3.77	5.65	7.54	9.43	11.32	13.20	58
59	1.84	2.22	2.96	3.70	5.56	7.41	9.27	11.13	12.98	59
60	1.81	2.17	2.91	3.64	5.47	7.29	9.12	10.94	12.76	60
61	1.78	2.13	2.86	3.58	5.38	7.17	8.96	10.76	12.56	61
62	1.76	2.10	2.82	3.52	5.29	7.05	8.82	10.59	12.35	62
63	1.73	2.07	2.77	3.47	5.20	6.93	8.68	10.42	12.16	63
64	1.70	2.04	2.73	3.41	5.12	6.83	8.54	10.26	11.97	64
65	1.68	2.01	2.69	3.36	5.04	6.73	8.41	10.10	11.78	65
66	1.65	1.98	2.64	3.31	4.97	6.63	8.29	9.94	11.60	66
67	1.63	1.95	2.60	3.26	4.89	6.53	8.16	9.80	11.43	67
68	1.60	1.92	2.57	3.21	4.82	6.43	8.04	9.65	11.26	68
69	1.58	1.89	2.53	3.16	4.75	6.34	7.92	9.51	11.10	69
70	1.56	1.86	2.49	3.12	4.68	6.25	7.81	9.38	10.94	70
	50	60	80	100	150	200	250	300	350	

Figure B-2. Rating table for a Price AA current meter.

In order to ensure consistent accuracy with a current meter, good preventive maintenance is a must. For all vane-type meters such as the Price AA, the pygmy, or the Gurley, a most important maintenance item is the protection of the pivot (Figure B-1, 17) and the pivot bearing (Figure B-1, 16). The pivot assembly provides a low-friction surface on which the bucket wheel is supported. If the pivot becomes blunted, or the pivot bearing damaged, the resistance increases and the meter will give low velocity readings. The greatest potential for damage occurs when a meter is transported with the pivot bearing and pivot in contact. On the Price AA meters, a raising nut (15) is provided. When screwed down, the raising nut lifts the pivot bearing off the pivot and prevents contact. Whenever a Price AA meter is transported, if only across the river, the raising nut should be screwed down. Pygmy meters and some Gurley meters are provided with a blunt brass travelling pivot. This pivot may be replaced for the operational pivot by loosening the set screw at the front of the yoke (7) and slipping one pivot out and the other in. Do not attempt to measure velocities with the travelling pivot in. Likewise, do not transport one of these meters with the operational pivot in.

Prior to and immediately following each use, the components of the meter should be cleaned and lubricated. A light, water-resistant oil should be used for a lubricant. Key oil for clarinets has been found to be a good, cheap lubricant. Oil should be applied to the pivot and pivot bearing, the penta gear and penta gear bushings (6), and the bearing lug (20). If measurements are made in silty or turbid water, the meter should be oiled frequently during its use.

The condition of the bearings should be checked prior to each use by a "spin test." With the shaft in a vertical position and cups protected from wind currents, the cups are given a quick spin. If the meter is in good condition, the cups should not stop spinning for at least three minutes. If the duration of spin is more than 1 minute, the meter may be used for all but very low velocities (less than 1 foot per second). A spin of less than 1 minute indicates that the instrument should be reconditioned. For pygmy meters, a spin of 30 seconds or less indicates the reconditioning is warranted.

For fairly deep or fast water, the Price AA type meter is usually the most practical instrument. For depths of less than about 0.5 feet (15 cm) the pygmy meter is more appropriate. Pygmy meters are essentially limited to velocities less than 3 feet per second (90 cm/sec). A third type of meter, the propeller type, is gaining quick popularity for many instream flow studies. The advantages of the propeller type current meters are that they are less sensitive to velocity components not parallel to the meter than are Price meters (Simmons and Peterka, 1967), and they are often equipped with a direct readout instrument. The direct reading provides considerable time savings to the field crew. The principal disadvantage of these meters is that it is very difficult to obtain a time-averaged velocity unless the flow past the meter is quite steady. (Note: Price meters may also be equipped for direct readout, but the same problem with time averaging still exists.)

While the propeller driven meter has considerable potential for instream flow use, its acceptance by the engineering community is not consistent. The precision of such instruments (0.1 foot per second) is certainly adequate for most instream flow models. However, most velocity measurements are precise to 0.01 foot per second, and this precision is usually preserved for discharge measurements. Four basic problems occur with the use of propeller models:

1. Precision of 0.1 foot per second.

2. Difficulty with time-averaging.

3. Turning resistance to the propellor may be affected by temperature.

4. Difficulty in accuracy recheck. The Price meter can be spin-tested for a quick check on the condition of the meter. A propeller model must be recalibrated frequently to test its accuracy.

Before using a propeller-type meter, the investigator should check with local engineers or water managers to ascertain their acceptance or rejection of results obtained with this type of meter. The IFG does not advocate the use of the propeller meter for the reason of potential problems with defending study results. However, under conditions of extremely limited resources, either time or manpower, the use of a propeller-type meter might be justified if the use of a vane-type meter is precluded.

Suspension Systems

Current meters are suspended by a sounding system which allows concurrent measurement of depth and velocity. For shallow, wadeable rivers, the most convenient system is a top-setting wading rod. The top-set rod has a main column, 1/2 inch, hexagonal stock which is graduated in 0.1 foot increments for measuring the depth. Interval markings follow the convention of a single mark every 0.1 foot, a double mark for each 0.5 foot increment, and a triple mark at each whole foot increment. Metric rods are usually singly marked at each centimeter and double marked at decimeters.

For unwadeable situations, the use of a boat-mounted cable suspension system is suggested. Such systems consist of a sounding reel (a hand winch with a depth gage precise to 0.1 foot), a length of small diameter cable, a hanger bar (to which the current meter is attached), and a sounding weight. Additionally, a boom assembly is required to extend the suspension system beyond the bow of the boat. Figure B-3 shows a typical small river, boat-mounted suspension system. Appendix C discusses boat-mounted systems in detail.

Figure B-3. Small river, boat-mounted suspension system.

VELOCITY MEASUREMENTS

In making a velocity distribution or discharge measurement, each cross section is divided into 10-30 partial sections depending on the level of detail desired. For discharge measurements, 20-30 partial sections are used. A partial section is a rectangle whose depth is equal to the sounded depth at a "vertical" and whose width is equal to the sum of half the distances to the adjacent verticals. At each vertical the following observations are made: (1) the distance to a reference point (zero point) on the bank, (2) the depth, and (3) the average velocity of the water column in the vertical.

The velocity in any vertical water column varies from zero at the bottom to a velocity at the surface about 1.15 times the average velocity in the column. Figure B-4 shows a typical vertical velocity distribution. The average velocity in the vertical may be approximated by one of the three following equations:

$$\bar{V} = V_{.6d} \tag{23}$$

$$\bar{V} = (V_{.2d} + V_{.8d})/2 \tag{24}$$

$$\bar{V} = (V_{.2d} + V_{.8d} + 2V_{.6d})/4 \tag{25}$$

Where V_x is measured at a fraction, x, of the depth <u>from the surface</u>.

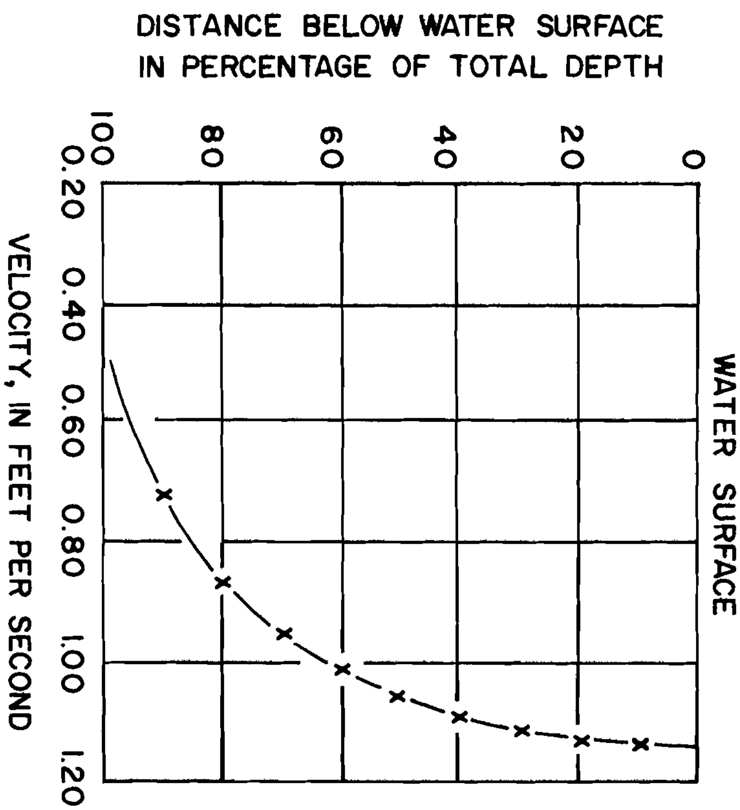


Figure B-4. Vertical velocity distribution typical of open channel flow.

For example, $V_{.6d}$ is the velocity measured at a depth of 0.6 times the total depth of the vertical. If the depth is 1.0 foot, the velocity is measured 0.6 foot below the surface. The use of equation 23 is termed the one-point method; equation 24, the two-point; and equation 25, the three point method. Equation 23 should be applied only where the depth of the vertical is less than 2.5 feet (about 75 cm). The two point method is usually applied only when the depth is greater than 2.5 feet. The three-point method is used when the velocities in the vertical are abnormally distributed, or when the 0.8 measurement is affected by bed turbulence or an obstruction.

For most suspension systems, the 0.2, 0.6, and 0.8 depths must be calculated from the total depth as determined by sounding. However, the top-set wading rod has a feature which allows the current meter to automatically be set at the 0.6 level. The head of a top-set rod is shown in Figure B-5. The depth of the vertical is read on the hexagonal sounding rod. Then the meter is placed in the 0.6 depth position appropriate for the measured depth by the meter positioning rod. If the depth is 1.4 feet, the "1" mark on the meter positioning rod is set even with the "4" mark on the grip of the wading rod (see Figure B-5). To move the meter positioning rod, the brake must be released by pushing the brake in toward the grip with the thumb. When releasing the brake, hold on to the positioning rod so that the meter does not slam into the ground. When the positioning rod is in this position, the meter will be suspended exactly 0.84 feet below the surface, which is 0.6 times 1.4, the depth of the vertical.

DISCHARGE CALCULATIONS

As previously defined, a vertical is measured at the center of each partial section. A partial section represents an approximate rectangle, the width of which is the sum of half the distances to the adjacent verticals. The mean depth for the partial section is very close to the depth measured at the vertical. These concepts are shown in Figure B-6. The cross-sectional area for each partial section, i, is equal to the width of the partial section, w_i, times the depth, d_i.

By the equation of continuity, the discharge through the partial section (called a partial discharge) is given as:

$$q_i = a_i \times v_i = w_i \times d_i \times v_i \qquad (26)$$

where v_i is the mean column velocity as measured at the vertical, and the other terms are previously defined.

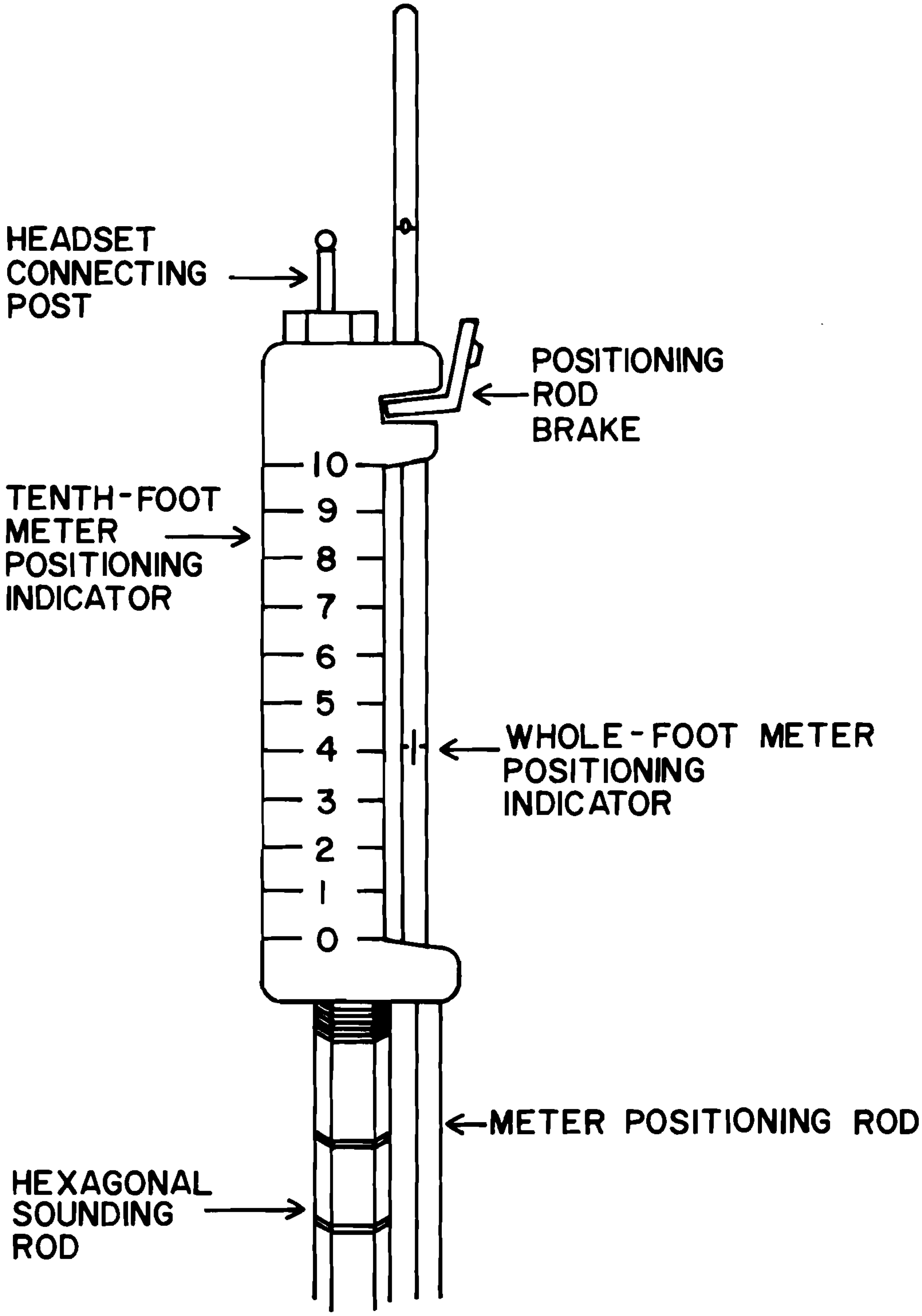


Figure B-5: Automatic six-tenths depth suspension of current meter on a top setting wading rod.

The total discharge for the cross section is the sum of all the partial discharges.

$$Q = \Sigma \quad q_i = \Sigma(\bar{V}_i \times w_i \times d_i) \qquad (27)$$

When measuring discharges, the U.S. Geological Survey recommends that no more than 10%, and preferably no more than 5%, of the total discharge flow through any partial section. For further details concerning discharge measurements, the reader is referred to Buchanan and Somers (1968) and Simmons and Peterka (1967).

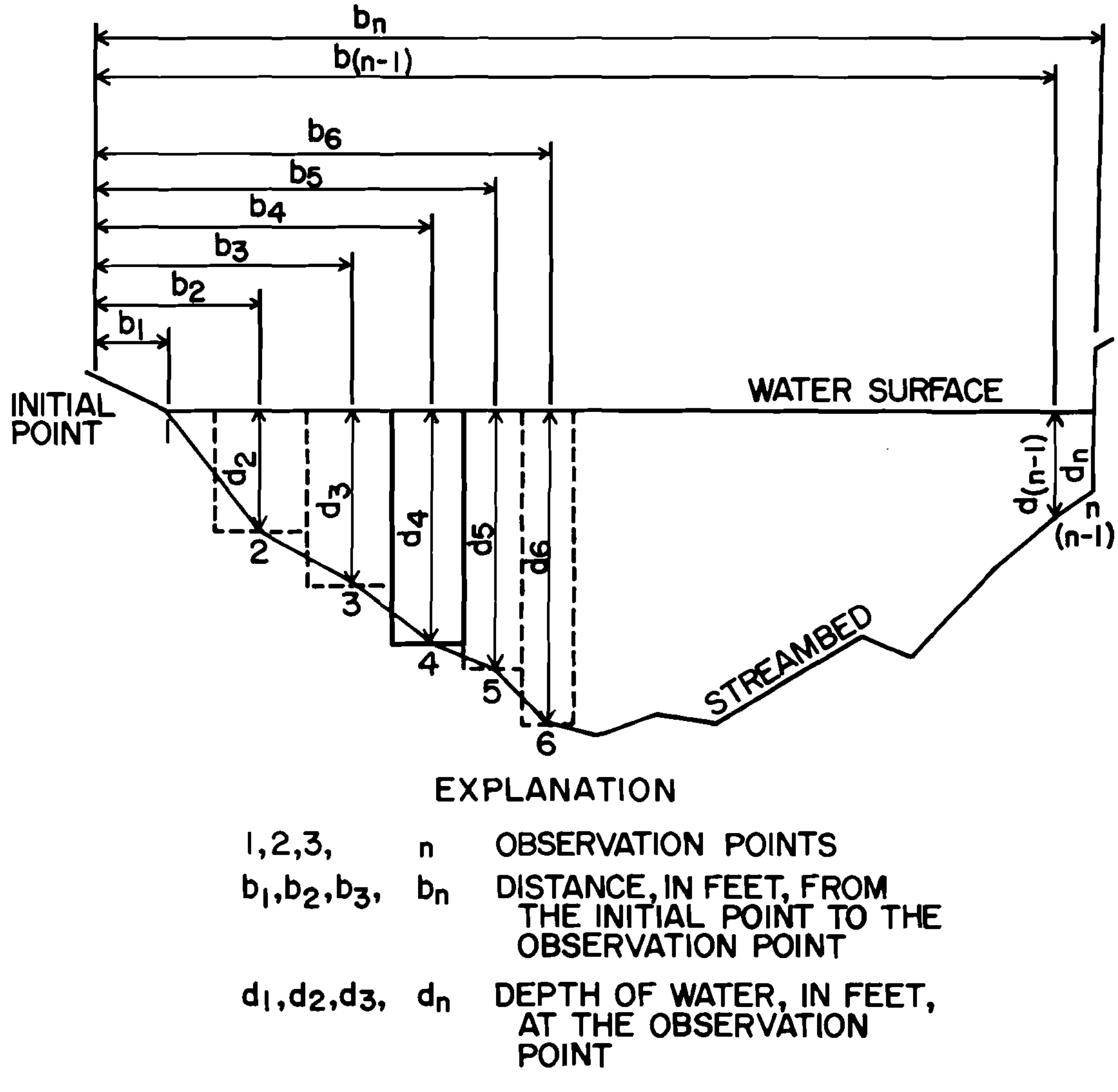


Figure B-6: Partial section concept used in measuring and calculating discharge.

APPENDIX C

FIELD TECHNIQUES FOR LARGE RIVERS

In theory, there is little difference between hydraulic simulations in small or large rivers. Hydraulic simulation models and data specifications are the same regardless of the size of the stream. Obviously, problems associated with large river data collection techniques are problems of scale. Often, problems of scale can be solved using equipment especially designed for the scale of the problem.

The principal difficulty encountered with large river data collection is in obtaining cross-sectional measurements. Specifically, cross section measurements may be broken down into three problem areas:

1. Maintaining a line of measurements on the transect (maintaining position).

2. Measuring distances from a reference point on the bank.

3. Measuring elevations, velocities, and substrate sizes at points on the transect.

MAINTAINING POSITION

Even if one could walk on water, he would have trouble maintaining position in a river. However, since most of us require the use of boats, the problems of maintaining a stable position relative to the bank can sometimes acquire massive proportions. The selection of a boat and power unit should be given careful consideration. Some researchers prefer to use a deep-keeled boat for its ease of handling in moving water. However, deep-keeled boats tend to be somewhat unstable if the crew has to move about in the boat. A flat-bottomed boat is a more stable platform for the crew, but may be difficult to hold on line in moving water. Perhaps a suitable compromise would be to use a boat of tri-hull design.

Three techniques may be used for maintaining position on-line:

1. Fixed line, fixed point system.

2. Fixed line, floating point system.

3. Floating line, floating point system.

With a fixed line, the transect is marked so that the boat can be placed on a semi-permanent line relative to the bank, and boat position is maintained by the boat crew. With a floating line, the transect is unmarked and the position of the boat is controlled by an observer on the bank. A fixed point system means that the boat is physically anchored on-line. Under a floating point system, the position of the boat would be controlled by a motor.

Fixed Line, Fixed Point System

For detailed hydraulic simulation models such as the rating curve approach, the fixed line, fixed point system is the most desirable. Any time a fixed position must be maintained for a relatively large time period, this system will give the most reproducible results.

The transect is marked by a 3/8-5/32 inch cable which is stretched across the stream, passed over the headstakes, and attached to a deadman on either side of the stream. The cable is then tightened by a hand- or battery-operated winch. The boat is then attached directly to the cable, (Figure C-1). For the greatest directional stability, the boat should be clipped to the cable at the gunwales, forward of amidships. Anchoring to the bow allows the boat to fishtail badly, making stationary measurements difficult.

The channel width which can be traversed successfully using this method depends on the scale of gear used. Given the size of boats normally available for this type of work (14 to 18 feet on the average--with 60 to 85 h.p. motors), the largest crossing practical would be on the order of 500 to 600 feet. Longer crossings would require a larger spool for carrying cable, and would require a larger, more powerful, craft to string the cable across the channel. Figure C-2 demonstrates the sequence involved in stretching a cable across a large channel. This sequence would apply only for the initial transect. Each end of the cable could then be moved to each subsequent transect. When using this approach, it is easiest to start at the upstream transect. In this way the drag on the cable will be toward the next transect to be worked instead of being against it.

With the boat anchored to the cable, the boat crew may pull the boat to measurement locations on the transect. If the cable is graduated in feet or meters, the distance from the reference point on the bank can be read directly from the cable by the boat crew.

Fixed Line, Floating Point System

The fixed line, floating point system utilizes a series of buoys to mark the line of sight of an observer on the bank. Markers can be placed at specified distances from a reference point on the bank through the use of stadia or an EDM setup at the headstake (Appendix A). The line of sight is set by an observer sighting through a level to the opposing bank headstake. The observer in the boat is responsible for setting out the markers according to directions supplied from the bank, and for holding the boat in place long enough for the distance measurement to be made. Distance measurements may be deferred until sounding measurements are made, if desired.

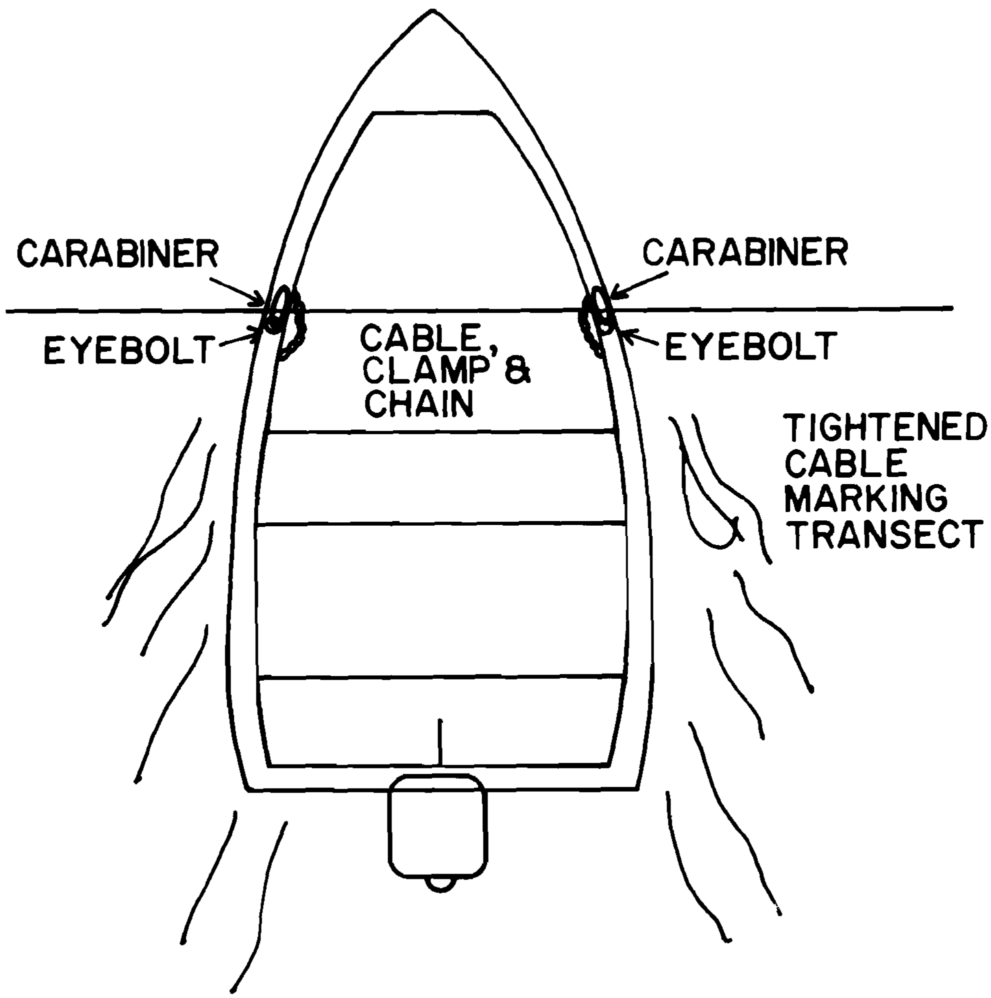


Figure C-1. Fixed line, fixed point method of maintaining longitudinal and lateral position in an unwadeable stream.

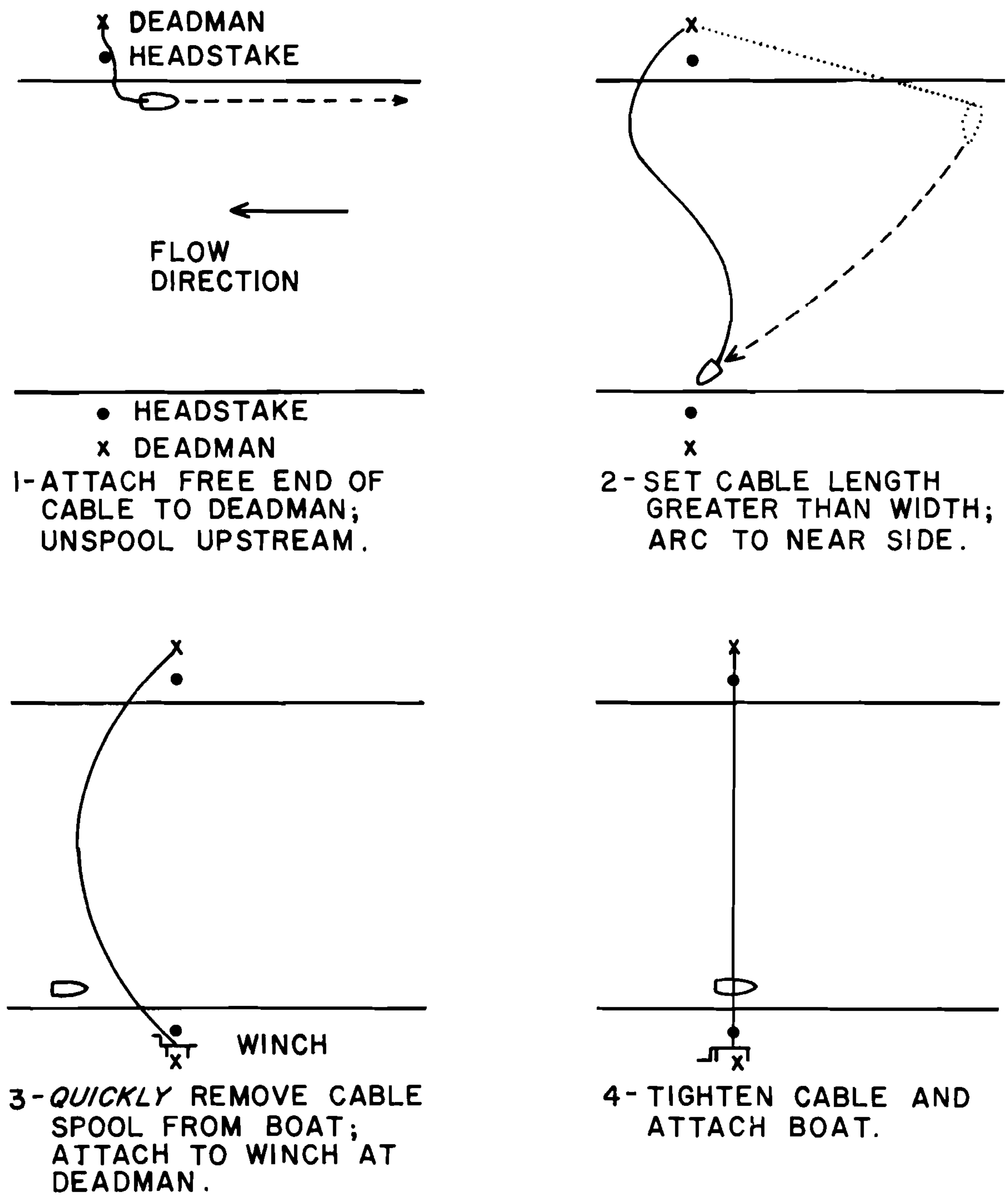


Figure C-2: Sequence involved in stretching cable or tagline across a large channel.

Because of the effect of different depths across the section, the initial placement of markers will result in the floats being on a crooked line across the river. Therefore, initial marker drops should be considerably upstream from the desired line of sight. They can then be towed downstream until the floats are on-line (Figure C-3). Option a of step 3 in Figure C-3 would be appropriate for either a water surface profile (WSP) model, or the initial data set for a rating curve model. Here, the target (either stadia rod or prism) mounted on the boat is held at the marker buoy by the helmsman, while the distance is read by the observer on shore. If the rating curve approach is used, second and third data sets must be taken at very nearly the same places as the initial data set. This may be accomplished as in option b of step 3, where the helmsman holds the line longitudinally, but is directed laterally to the right place by the observer on shore. A third option would be to reposition the buoys so that they mark the correct positions on the transect.

An alternative fixed line, floating point system might consist of a light tag line stretched across the channel in place of the cable used under the fixed point system. In this case, it may be possible to traverse a considerably wider river because of the smaller weight of the tag line. The boat would not be attached to the tag line, but free to be positioned at the appropriate place on the transect by the helmsman.

Floating Line, Floating Point System

Under this system, the location of the transect is "blind" to the helmsman. The boat is positioned both longitudinally and laterally at each measurement location by the bank observer either by radio contact or hand signals. All measurements, depth, velocity, substrate, and distance from the reference point, should be made concurrently at each position. This system is somewhat faster than either of the fixed line systems. However, the reproducibility of data collection points depends largely on the skill of the helmsman, and the coordination between the helmsman and the shore observer. Since the helmsman has no immediate reference point on the transect, great care must be taken to insure that the boat is not drifting while measurements are being made. This is the primary responsibility of the observer.

MEASURING DISTANCES

The preceding discussions should give the reader a fair idea of the options available for measuring distances across large rivers. Rivers up to about 600 feet in width can be measured using an incremented cable to which the boat is anchored. Substituting a light polypropylene tag line for the cable and maintaining boat position by motor, it might be feasible to measure as far as 800 to 1000 feet. The traversable distance depends on the adequacy of the deadmen, the strength and weight of

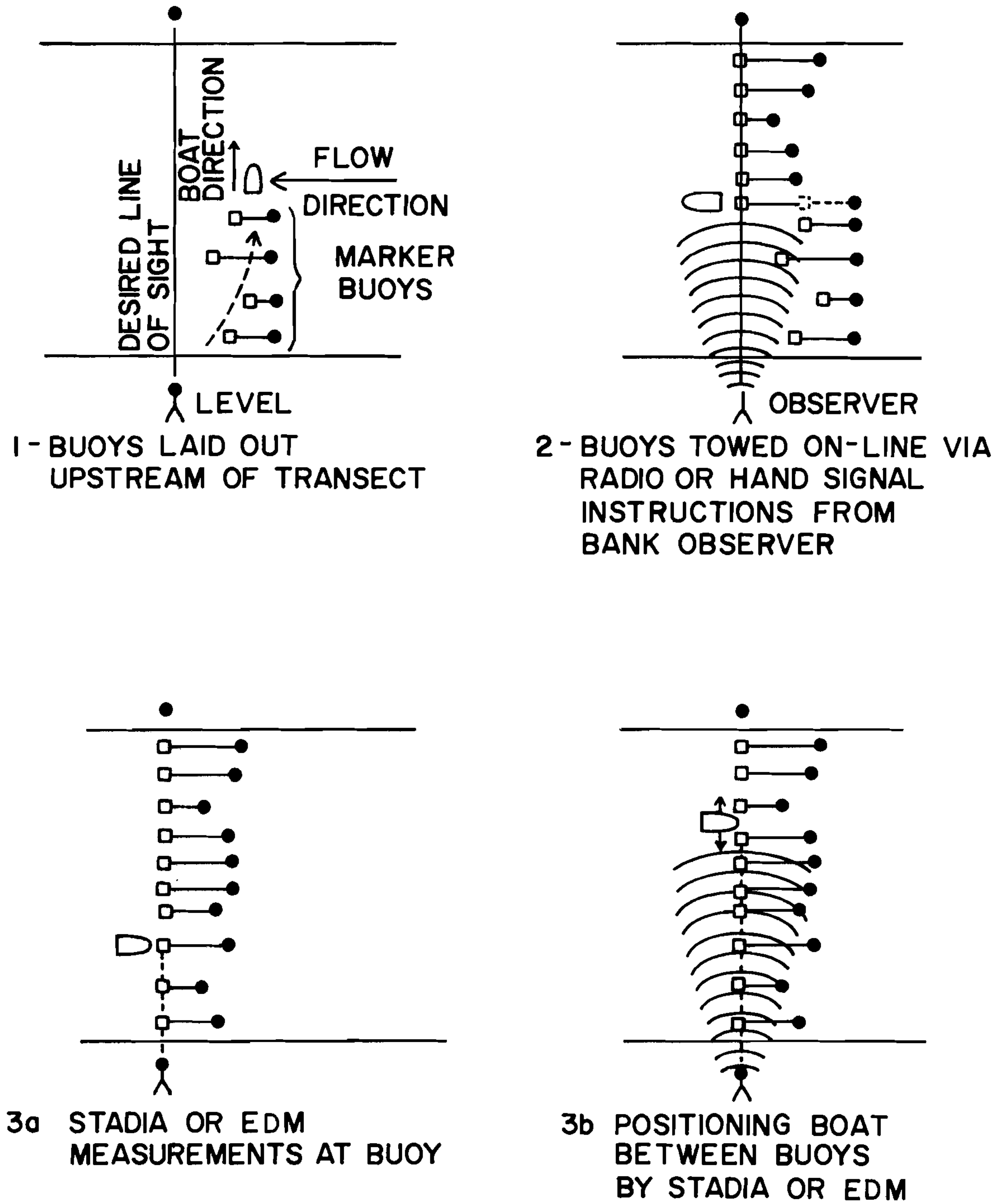


Figure C-3: Marking the line of sight by a series of buoys.

the line, and the pull capacity of the winch. Given heavy enough deadmen, a long enough line and a huge winch, it would be possible to traverse the Amazon, although it would not be very practical.

When attaching a cable to a deadman, it is important not to kink the cable. A cable with a tensile strength of 2 tons will break under 200 pounds of pull if it is kinked. The free end of the cable should be equipped with either a hook or a clevis for attachment to the deadman. The deadman might be a tree, where the cable is attached by means of a chain thrown around the trunk, with the cable clevis attached to the chain. In this case, an old tire or other protection feature should be placed between the chain and the tree to prevent stripping off the bark. An alternative would be to use a steel fence post anchored in cement, or a concrete filled drum with a chain set in the cement, buried in the bank. The practicality of any of these solutions depends largely on local conditions and the magnitude of the study.

Use of either stadia or an EDM requires that a target be mounted in the boat. With stadia, either a rodman or a bracket is needed in the boat to hold the rod. Under reasonable conditions, errors in stadia distances should not exceed 1 foot in a 500 foot sight; nor 10 feet in a 1000 foot sight. If an EDM is used, a retroprism target should be securely mounted in the boat. A cheap EDM target can be constructed from a piece of plywood covered with bicycle reflectors. An EDM is accurate to about 0.04 foot for distances from 1600 feet to 3 miles, depending on the instrument.

SOUNDING

Sounding is a method of determining depths or bed elevations, and for measuring velocities. In some cases, an estimate of the substrate may also be determined by sounding.

If only a stream bed profile is desired, an electronic depth sounder may be used. These units are quite useful even if a more precise set of measurements are desired, as the boat crew can traverse the channel several times and determine major features beneath the surface. The precision of depth sounders is around 0.5 to 1.0 foot, and many exhibit feedback (read secondary echoes reflecting off the bottom of the boat) in water less than about 6 feet deep. These problems can be mitigated somewhat by using narrow-beam instruments.

In most cases, velocity measurements are required along with depth measurements. Electronic depth sounders cannot be used to measure velocity, so an additional piece of equipment is needed. The most practical suspension system when both depths and velocities are required, is the sounding reel cable suspension system described in Appendix B.

The most pertinent features of a cable sounding system are: (1) the sounding reel, (2) the boat boom, (3) cable, (4) hanger bar, (5) current meter, and (6) sounding weight.

Sounding Reels

Sounding reels are compact, level wind, hand-operated winches equipped with a length (50 to 100 feet) of 1/8 inch cable. They are also equipped with a gage which measures the amount of cable unwound from a zero point.

This gage can be "zeroed" at the point that the sounding weight just touches the surface. When the weight is lowered to the bottom, the gage measures the distance to the nearest 0.1 foot.

Boat Boom

Boat booms must usually be fabricated to the specifications of the boat. However, they have several common design features. First, the cable boom must extend beyond the bow. If a heavy (75 to 100 pound) sounding weight is to be used, the boom should go straight off the bow. Skewing the boom to one side will cause the boat to list badly.

The boom must be firmly anchored to solid structural supports of the boat. Many booms are constructed with an A-frame design, with a mounting plate for the sounding reel and a cable pulley at the end. Crew members making the soundings will appreciate boom construction with sufficient length that the sounding reel can be operated from one of the seats in the boat.

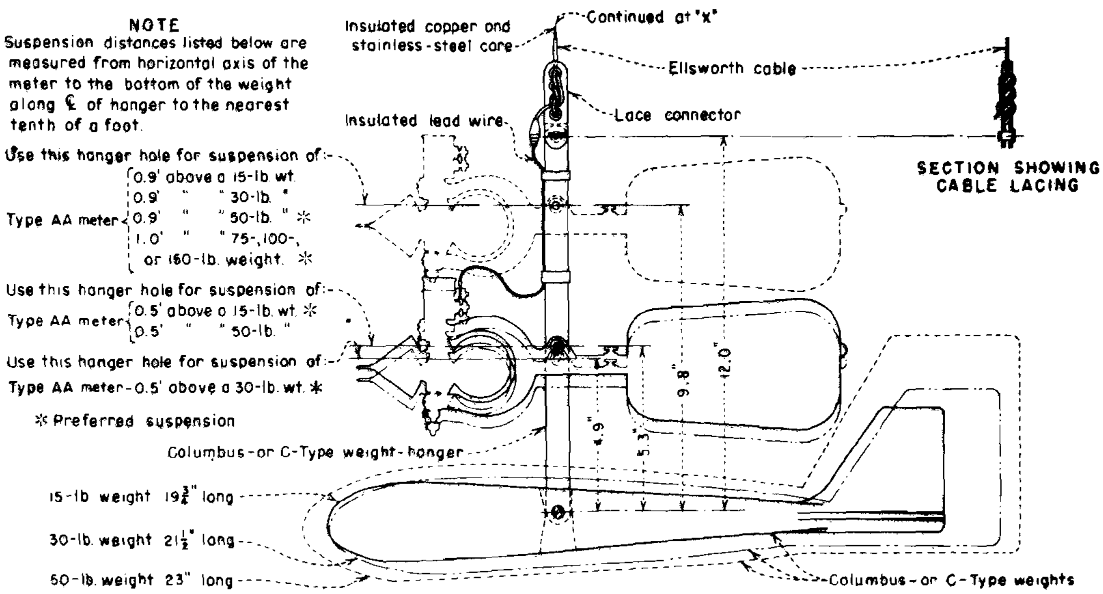
Suspension System

In addition to the boom and sounding reel, the suspension system consists of the cable, hanger bar, and sounding weight. The general assembly of a Price AA current meter on a cable suspension system is given in Figure C-4.

Sounding weights come in various sizes ranging from 15 to 150 pounds. Generally, the heavier weights are for use in faster water. High water velocities can cause the sounding line to deviate from a true vertical position, as shown in Figure C-5. An approximation of the true depth below the boom may be determined by using a heavier weight and/or making an angle correction. In Figure C-5, assuming the depth at points A and B are equal, the measured line AC is too long. If the angle of deflection is known, the line BC can be calculated by:

$$BC = AC \cos \alpha \qquad (28)$$

where α is the angle of deflection. This can be measured by attaching a protractor to the beam and noting the angle of intersection of the cable.



CABLE SUSPENSION

Figure C-4. General assembly of a Price AA current meter and sounding weight on a cable suspension system.

Figure C-6 shows the layout for a typical large river boat, rigged for use with a cable sounding system and EDM distance measurement. The boat is equipped with two outboard motors. The primary motor should be powerful enough for the basic transportation duties of the boat, or towing taglines across the stream. For floating point systems, a small trolling motor (5 to 7 h.p.) is suggested for holding position. The trolling motor has a larger arc and is thus more responsive for lateral movements. However, it is not powerful enough to produce the acceleration to affect current meter measurements.

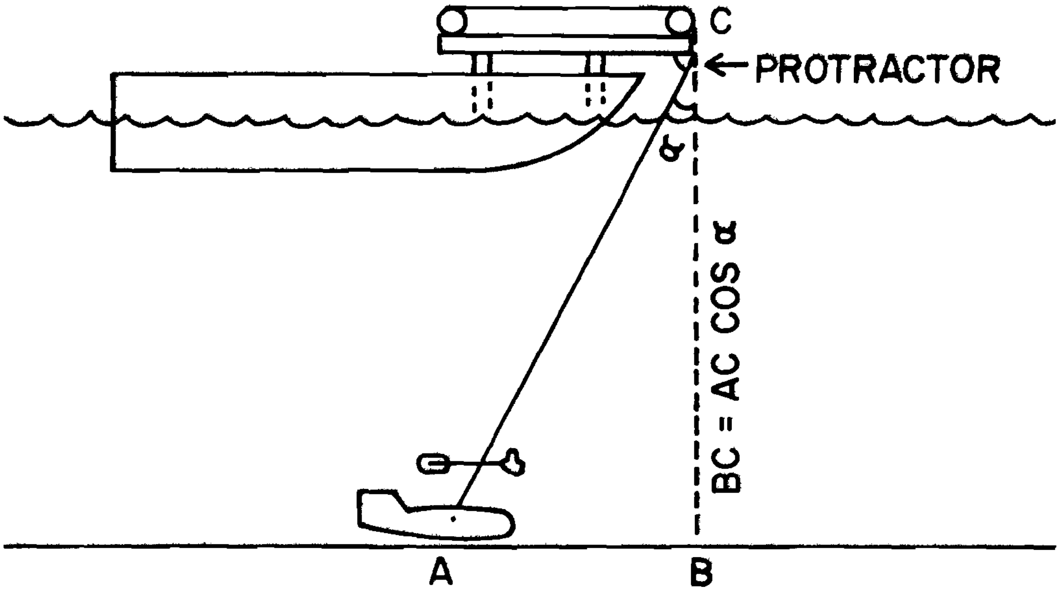
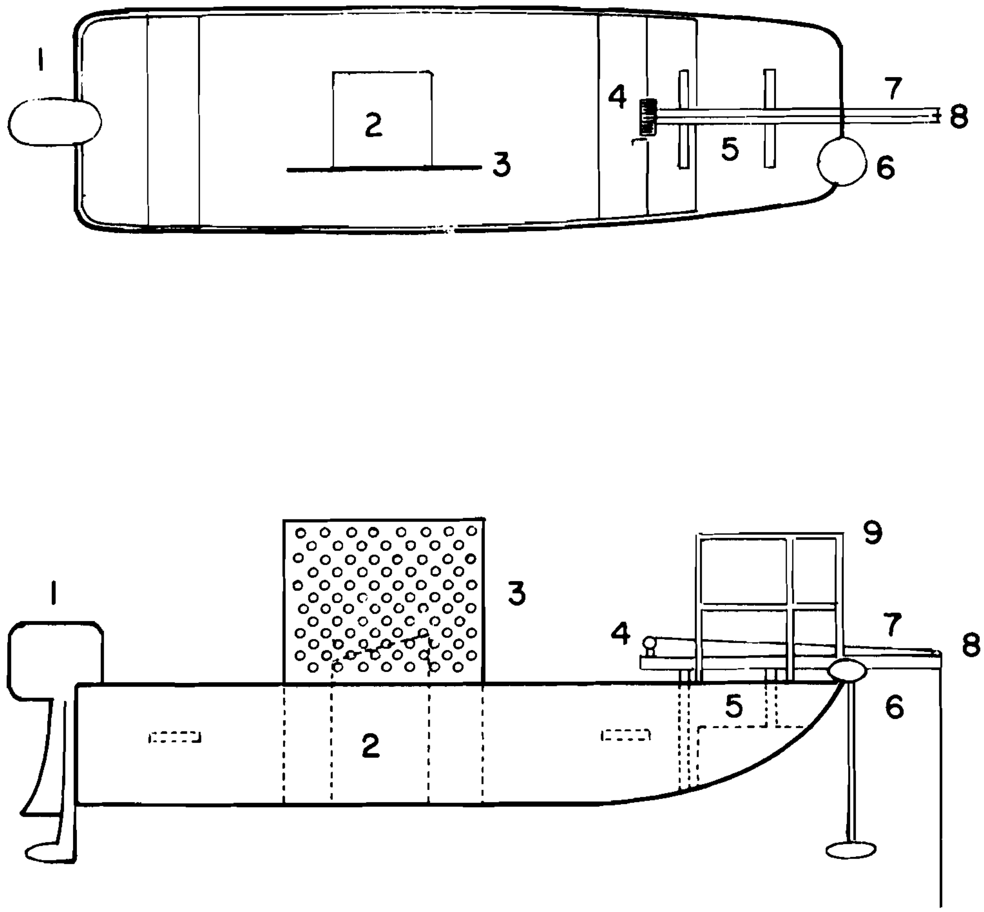


Figure C-5. Angle correction for sounding in high velocity water.



1- PRIMARY MOTOR, 25 - 120 h.p., as required.
2- CONTROL CONSOLE
3- EDM TARGET; PLYWOOD AND BICYCLE REFLECTORS
4- SOUNDING REEL AND CABLE
5- A-FRAME STRUTS ON SOUNDING BOOM
6- TROLLING MOTOR, 5 - 7 h.p.
7- SOUNDING BOOM
8- PULLEY, SEALED BEARING OR BRASS BUSHING
9- SAFETY RAILING

Figure C-6: Layout for a typical large river boat, rigged for cable sounding and EDM distance measurement.

APPENDIX D

EQUIPMENT LIST FOR FIELD DATA COLLECTION
IFG INCREMENTAL METHOD

Item Description		Approximate Unit Cost	Supplier(s)
Surveying Equipment			
Automatic levels--preferred for ease of leveling, superior optics, and internal level compensation. For large river work the magnification should be at least 30x. For rivers under 300 ft in width, 20-25x will suffice.			Most Engineering Supply Houses
Brand Names & Numbers*			
Lietz B-1 (32x)	without tripod	$ 1,050	
Nikon AE (32x)	"	875	
Geotec AL2 (32x)	"	795	
Lietz B-2 (25x)	"	775	
Lietz C-3 (25x)	"	550	
Geotec AL3 (25x)	"	495	

*Listing of a brand name does not constitute endorsement.

Item Description		Approximate Unit Cost	Supplier(s)
Automatic Levels (cont.)			
Model S-201 (30x)	without tripod	$ 505	Forestry
Cat No. 43724			Suppliers, Inc.
Model S-302 (20x)	"	382	
Cat No. 43722			Jackson, MS 39204
Builders Levels--more difficult to level than automatics, but comparable in optical properties; may be more durable and stand up better under abuse.			
Dumpy Level			
Berger 450 (33x)	without tripod	425	Forestry Suppliers
(20 to 44x zoom)		460	(as above)
Berger 110B (20x)	with tripod	170	or
Lietz Engineers Level (33x)	without tripod	395	Local Engineering
			Supply Houses
Tripod		115	Engineering
			Supply Houses
Level Rod			
Philadelphia type			"
(10 ft) graduated in 0.1 and 0.01		45	
(13 ft) " "		72	
Fiberglass telescoping (preferred--saves setups and turns)			"
(20 ft)		60	
(25 ft)		80	
(30 ft)		95	
Survey stakes			
½" x 30" concrete reinforcement bar (rebar)		15¢/lb	Scrap Metal Dealers

Item Description	Approximate Unit Cost	Supplier(s)
Distance Metering Equipment		
Steel tape (stainless or chrome plated) with reel		Forestry
(200 ft)	$ 185	Suppliers
(300 ft)	260	or
Fiberglass tape (hand looped)*		Engineering
(200 ft)	15	Supply Houses
(300 ft)	24	
1/8" cable tagline - 300 ft	130	
Incremental tagline--constructed from ¼" rope and marked (painted) at regular intervals. Virtually indestructible.		Surplus Stores or
Nylon	10¢/ft	Farm & Ranch
Polyester	3¢/ft	Suppliers
Electronic distance meters--applicable to rivers larger than about 600 ft in width.		
Geonometer (range to 2 miles, price includes a theodolite so level is not needed) Laser activated	$ 7,500	Engineering Suppliers
Beetle 500--infrared light source. Range = 1600 ft. Accuracy = $\pm$ .03 ft.	3,000	Forestry Suppliers, Inc.
Retroprisms and Traverse equipment.**	450	
Battery pack and charger	150	
Adaptor for mounting on level	170	

*May break if tape is allowed to drag in current and is then tightened.

***For our accuracy requirements, plus the fact that the target will be mounted on a boat, the traverse equipment can be replaced by a large target made of plywood covered with highway reflectors.

Item Description	Approximate Unit Cost	Supplier(s)
Field Support Equipment		
Large, unwadeable rivers		
Boat, 16' flat bottom or tri-hull	$1000-2000	
18 h.p. outboard motor	$ 1,000	
Small, (less than 400 ft wide) unwadeable rivers		
Boat, 14' flat bottom or tri-hull	$500-1500	
7½ h.p. outboard motor	$ 500	
Small, partially wadeable rivers		
Boat, 14' flat bottom or tri-hull	$500-1500	
or		
Raft, 5-man inflatable	$150-300	
Anchor rope, ½"		
Nylon	25¢/ft	
Polyester	10¢/ft	
Bank anchors, ½" x 48" rebar (2 for each transect)	15¢/lb	Scrap metal dealers
Quick release carabiners (2) for attaching boat to anchor rope	$ 7.00	Mountaineering shops
Hand-held 2-way radios (walkie-talkie) Business freqency, not CB — Standard	$500-700	Radio Equipment Shops
GE	$850-1,100	
Compass, magnetic, for stream mapping, site identification and transect relocation		
Brunton type	$ 85	Forestry Suppliers
"Boy Scout" type	$ 20	

Item Description		Approximate Unit Cost	Supplier(s)
Distance Metering Equipment (continued)			
Hewlett Packard			
3805 mounts on Tribrach--glass extra		$ 5,000	Hewlett Packard, Inc.
1 mile range with triple prism. Infrared.	to	5,650	
Total Station 3810		$ 9,250	
1 mile range, slope-distance vertical 20", sensitive to cold, not super accurate, not read electronic. Infrared.	to	10,500	
ETS 3820		$ 18,000	
3 mile range, horizontal and vertical angles all electronic.	59	20,000	
Range Finders			
Lietz - lens displacing type. Range 29 - 1000 ft Accuracy 1 percent up to 300 ft 2 percent 300 to 500 ft 5 percent 500 to 1000 ft		$ 300	Forestry Suppliers or Engineering Suppliers
Stream Gaging Equipment			
Current Meters			Scientific
Pygmy - used for shallow areas with velocities less than about 3 ft/second		$ 180	Instruments of Wisconsin (414-263-1600) or
Price AA - used for water greater than 0.4 feet in-depth, with velocities measurable up to about 10' per second		$ 240	Contracts & Procurement Section USGS Reston, VA (FTS 928-7271)

Item Description	Approximate Unit Cost	Supplier(s)
Sounding Systems		
Topsetting wading rod - may be used in streams up to 4 ft in depth	$ 135	
Suspended Systems		
Hand-held, with hanger bar; 15 pound sounding weight; 35 ft plastic coated cable, attached to 15 ft bare cable; with current meter	$ 400	
Boat-mounted, Type A crane (3 wheel); with B56 sounding reel, including 144' of cable	1,400	Scientific Instruments, Inc.
B56, reel with cable, only	900	
Immovable, hand built crane, materials		John Deere
3 - 4" pulleys w/sealed bearings	$ 25	Piper
¼" x 2" angle iron (24' @ 3.2 lbs/ft)		Cessna
Used	15	
New	19	Scrap Iron
¼" x 2" strap iron (6')	5	Dealers
Labor (cutting & drilling)	50	
	$ 114	
Sonar		
Raytheon Model 719-B depth sounder with narrow beam transducer; 2-6 ft dead space (i.e., useful only in water more than 6 ft deep)	$ 2,500	
Headset and Battery	$ 22	Scientific Instruments
Headset and Battery	10	Patch together from cheap electronics shop materials. Will work better than the above types.

APPENDIX E

COMPUTER SOFTWARE FOR HYDRAULIC SIMULATIONS IN INSTREAM FLOW ASSESSMENTS

At the outset, it must be emphasized that a methodology for assessing instream flow requirements is not the same as a hydraulic simulation model. All references to the R-2 Cross program, WSP program, stage-discharge approach, etc., refer to methods of predicting hydraulic variables at different discharges. An instream flow assessment method interfaces these measured or predicted hydraulic variables with some type of implied or expressed biological criteria to measure the habitat available for a given species at each discharge investigated. Therefore, the type of model used to predict hydraulic conditions in a stream is interchangeable in many methodologies.

However, if the IFG incremental method is to be used, only a select few computer programs are currently compatible for use with the IFG-3 (HABTAT) program. These hydraulic simulation programs will be pointed out during the discussion.

Three basic types of hydraulic simulation software are currently available from various sources. Input requirements are functions of the assumptions according to by the approach. Output varies by program.

TYPE 1 - MANNING EQUATION ASSUMING UNIFORM FLOW

R-2 Cross (U.S.Forest Service)

IFG-1 (IFG, USFWS) (IFG, U.S. Fish and Wildlife Service)

The IFG-1 program is a modified version of the U.S. Forest Service R-2 Cross program. The depth and velocity distributions may be predicted using the Manning equation, with input data either from sagtape or level measurements. Program outputs are:

1. Distance to channel edge (ft)

2. Channel width (ft)

3. Cross sectional area (ft^2)

4. Wetted perimeter (ft)

5. Surface width (ft)

6. Hydraulic radius (ft)

7. Average Depth (ft)

8. Discharge (cfs)

9. Average velocity (fps)

The primary difference between IFG-1 and R-2 Cross is that IFG-1 outputs widths of stream having specified depths. Both R-2 Cross and IFG-1 are to be used for single cross section methods only.

TYPE 2 - MANNING/BERNOULLI EQUATIONS ASSUMING GRADUALLY VARIED FLOW

Three "step-backwater" programs are available, and all are somewhat similar. However, at this time only the PSEUDO program of the U.S. Bureau of Reclamation is compatible with the IFG-3 program.

PSEUDO (Bureau of Reclamation)

This program utilizes an energy balance model, using the Manning equation and one set of calibration measurements which require level-surveying precision. The program has been modified to produce outputs compatible as inputs to the IFG-3 (HABTAT) program, which is described below. The PSEUDO program requires data inputs as described under the section concerning data collection. Program outputs include for up to 9 cross section subdivisions:

1. Station index which indicates distance upstream from initial cross section

2. Thalweg elevation at cross section

3. Thalweg slope

4. Centroid length - average distance between a cross section and the next downstream cross section

5. Conveyance (cross sectional) areas (ft^2)

6. Top widths (ft)

7. Hydraulic radii (ft)

8. Roughness coefficients

9. Velocities (mean for subsection) in ft/sec

10. Discharge in cfs

11. Computed water surface elevation in ft

This program is calibrated by adjusting Manning's n until the water surface elevations and velocities approximate those measured in the field at the calibration flow. Documentation for this program is only fair, and it is often difficult to calibrate. However, proficiency increases rapidly with practice.

Documentation is available from:

Office of Chief Engineer, Hydrology Branch
Sedimentation Section
U.S. Bureau of Reclamation
Denver Federal Center
Denver, Colorado

HEC-2 (Corps of Engineers)

The HEC-2 program is very similar to PSEUDO, although up to 20 cross section subdivisions for the velocity distribution may be used with HEC-2. Documentation is quite good, and may be obtained from:

The Hydrologic Engineering Center
U.S. Army Corps of Engineers
609 Second Street
Davis, California

WSP-2 (Soil Conservation Service)

The WSP-2 program is also quite similar to other step-backwater programs. With WSP-2 the user is limited to six cross section subdivisions to describe the velocity distribution, which may limit the WSP-2 program for use with instream flow studies. Documentation for WSP-2 is quite good, and available from:

Engineering Division
Soil Conservation Service
U.S. Department of Agriculture
Washington, D.C.

TYPE 3 - RATING CURVE APPROACH

IFG-4 (IFG, USFWS)

The IFG-4 program utilizes two or more sets of stage and velocity measurements taken at different discharges to establish a least-squares fit of log stage versus log discharge, and log velocity vs log discharge for each measurement point on the cross section. Input to the program may be taken directly from the field notes. Required inputs are:

1. Water surface elevation at each cross section.

2. Velocities at specified intervals across section.

3. Ground elevation (cross sectional profile).

4. Distance between cross sections.

5. Estimate of substrate composition at each velocity measurement point.

Given these inputs, the program computes the discharge for each set of calibration measurements. Outputs from the program include:

1. Station indexing

2. Distance across transect from zero point

3. Average depth of channel subsection

4. Average velocity of channel subsection

5. Substrate of channel subsection

These parameters may be obtained for up to 100 channel cross section subdivisions.

For each discharge simulated at each cross section the program also outputs an "adjustment factor." For a given discharge, the depths and velocities across the section are calculated independently. If the predicted depths and velocities are accurate, a discharge calculated from these variables should equal the discharge originally requested. The "adjustment factor" is a ratio between the discharge calculated from these simulated parameters and the discharge requested. This factor can be used as an indicator of the accuracy of the predictions; the closer to 1.0 the ratio is, the better the predictions. If the adjustment factor deviates significantly from 1.00 ±10% it indicates that some change has occurred on the stage-discharge relationship, and either more measurements are needed, or some manipulation of the data is needed to calibrate the model. This most frequently occurs at low flow extrapolations, and overbank, high flows.

The IFG-4 program has recently been completed. Documentation for this program should be available in August, 1978, from the Instream Flow Group.

IFG-3 (IFG, USFWS)

The IFG-3, or HABTAT program, is the core to the IFG incremental method. This program uses hydraulic input data from either the PSEUDO or IFG-4 hydraulic simulation programs or direct measurements. These hydraulic data are interfaced with probability criteria for specified life stages of different species. An adjunct to the HABTAT program is a curve maintenance program (CRVMNT) which contains digitized versions of probability-of-use curves for each life stage and species for which criteria have been developed. The appropriate curve sets are accessed by means of a catalog number, which is input to the program in the control deck. Catalog identifiers are six-digit numbers, with the first two digits identifying the family; the middle two, the species; and the last two digits, the life stage. Number 011300 refers to brown trout fry, as shown below:

01	13	00
Family Salmonidae	Species *Salmo trutta*	Life-stage fry

A listing of catalog numbers, by family and species, are included in Table E-1. Not all of these curves are on file at this time.

Having accessed the appropriate curve sets for the desired life stages and species, the HABTAT program computes the weighted usable area for the stream reach at each discharge simulated with the hydraulic model. For each species, life stage, and discharge, two-way matrix tables may be obtained as output (velocity versus depth, velocity versus substrate, or depth versus substrate). It is recommended that one copy of the complete output be produced for each study site for examination of the detailed distribution of hydraulic parameters within the reach. However, in those cases when all of the matrix tables are not needed or desired, a change of control cards gives a printout of a summary table of weighted usable area for each life stage of a species, by month (if flow duration curve used) and by discharge. Documentation for this program will be available in August, 1978.

Table E-1. Listing of Family and Species Identifiers for IFG Curve Maintenance Files.

01 - Salmonidae
- 00 - Coho Salmon
- 01 - Chinook Salmon
- 02 - Kokanee Salmon
- 03 - Sockeye Salmon
- 10 - Steelhead
- 11 - Rainbow Trout
- 12 - Cutthroat Trout
- 13 - Brown Trout
- 14 - Brook Trout
- 15 - Dolly Varden
- 20 - Mountain Whitefish
    - 00 Fry
    - 01 - Juvenile
    - 02 - Adult
    - 10 - Spawning General
    - 11 - Spring Spawning
    - 12 - Fall Spawning
    - 13 - Winter Spawning
    - 20 - Egg Incubation
    - 21 - Clear w/ S = 0.001
    - 22 - Turbid S = 0.001
    - 23 - Clear S = 0.0025
    - 24 - Turbid S = 0.0025
    - 25 - Clear S = 0.004
    - 26 - Turbid S = 0.004

02 - Centrarchidae
- 00 - Rock Bass
- 01 - Smallmouth Bass
- 02 - Spotted Bass
- 03 - Largemouth Bass (Northern)
- 04 - Bluegill
- 05 - Green Sunfish
- 06 - Orangespotted Sunfish
- 07 - Longear Sunfish
- 08 - White Crappie
- 09 - Black Crappie
- 13 - Largemouth Bass (Southern)
    - 00 - Fry
    - 01 - Juvenile
    - 02 - Adult
    - 03 - Spawning (Clear Water)

- 04 - Spawning (Turbid Water)
- 05 - Egg Incubation (clear Water)
- 06 - Egg Incubation (Turbid Water)

03 - Ictaluridae
- 00 - Stonecat
- 01 - Channel Catfish
- 02 - Flathead Catfish
  - 00 - Fry
  - 01 - Juvenile
  - 02 - Adult
  - 03 - Spawning
  - 04 - Egg Incubation

04 - Esocidae
- 00 - Grass Pickerel
- 01 - Chain Pickerel
- 02 - Northern Pike
- 03 - Ohio Muskellunge
  - 00 - Fry
  - 01 - Juvenile
  - 02 - Adult
  - 03 - Spawning
  - 04 - Incubation

05 - Percidae
- 00 - Walleye
- 01 - Sauger
- 02 - Yellow Perch
- 03 - Log Perch
- 04 - River Darter
- 05 - Channel Darter
- 10 - Banded Darter
- 11 - Rainbow Darter
- 12 - Greenside Darter
- 13 - Fantail Darter
- 14 - Orangethroat Darter
- 15 - Arkansas Darter
- 16 - Orangebelly Darter

06 - Clupeidae and Hiodontidae
- 00 - American Shad
- 01 - Atlantic Herring
- 02 - Gizzard Shad
- 10 - Goldeye
  - 00 - Fry
  - 01 - Juvenile
  - 02 - Adult
  - 03 - Spawning

07 - Acipenseridae and Polyodontidae
- 00 - White Sturgeon
- 01 - Shovelnose Sturgeon
- 02 - Paddlefish
  - 00 - Fry
  - 01 - Juvenile
  - 02 - Adult
  - 03 - Spawning

08 - Percicthyidae
- 00 - White Bass
- 01 - Striped Bass
  - 00 - Fry
  - 01 - Juvenile
  - 02 - Adult
  - 03 - Spawning

09 - Cyprinidae
- 00 - Carp
- 01 - Sturgeon Chub
- 02 - Emerald Shiner
- 03 - Red Shiner
- 04 - Sand Shiner
- 10 - Colorado Squawfish
- 15 - Longnose Dace
- 16 - Blacknose Dace
- 17 - Speckled Dace
- 20 - Stoneroller

10 - Catostomidae
- 00 - River Carpsucker
- 01 - Longnose Sucker
- 02 - White (common) Sucker
- 03 - Bluehead Sucker
- 04 - Mountain Sucker
- 05 - Blue Sucker
- 06 - Northern Hog Sucker
- 07 - Smallmouth Buffalo
- 08 - Bigmouth Buffalo
- 09 - Black Redhorse
- 10 - Golden Redhorse
- 11 - Shorthead Redhorse

11 - Sciaenidae
- 00 - Freshwater Drum

12 - Cottidae
- 00 - Mottled Sculpin

INSTREAM FLOW INFORMATION PAPERS ISSUED

1. Lamb, Berton Lee, Editor. Guidelines for Preparing Expert Testimony in Water Management Decisions Related to Instream Flow Issues. Fort Collins, Colorado, Cooperative Instream Flow Service Group, July 1977, 30 pages. (NTIS Accession Number: PB 268 597; Library of Congress Catalog Card No. 77-83281).

2. Lamb, Berton Lee, Editor. Protecting Instream Flows Under Western Water Law: Selected Papers. Fort Collins, Colorado, Cooperative Instream Flow Service Group, September 1977, 60 pages. (NTIS Accession Number: PB 272 993; Library of Congress Catalog Card No. 77-15286).

3. Bovee, Ken D., and Cochnauer, Tim. Development and Evaluation of Weighted Criteria , Probability- of -Use Curves for Instream Flow Assessments; Fisheries. Fort Collins, Colorado, Cooperative Instream Flow Service Group, December 1977, 49 pages. (NTIS Accession Number: PB ; Library of Congress Catalog Card No. -).

4. Bovee, Ken D. Probability-of-Use Criteria for the Family Salmonidae. Fort Collins, Colorado, Cooperative Instream Flow Service Group, January 1978, 88 pages. (NTIS Accession Number: PB : Library of Congress Catalog Card No. -).

5. Ken D. Bovee and Milhous, Robert T. Hydraulic Simulation in Instream Flow Studies: Theory and Techniques. Fort Collins, Colorado, Cooperative Instream Flow Service Group, May 1978, pages. (NTIS Accession Number: PB ; Library of Congress Catalog Card No. -).

6. Hyra, Ronald. Methods of Assessing Instream Flows for Recreation. Fort Collins, Colorado, Cooperative Instream Flow Service Group, June 1978, 52 pages. (NTIS Accession Number: PB ; Library of Congress Catalog No. -).

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16. Abstract (Limit: 200 words)
Several types of techniques for the prediction of the stage-discharge relationship and the velocity distribution-discharge relationship are presented, along with general limitations and site imposed constraints pertaining to each type of simulation technique.

Data requirements and precision specifications are provided, as well as guidelines for efficient data collection. The appendices include data collection procedures, methods for collecting hydraulic data in large rivers, the needed equipment, and information on several computer programs for hydraulic simulation.

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Instream flows
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Stage-discharge-velocity predictions
Stream survey techniques

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