Flood Discharge Assessment

An Engineering Guide

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HR Wallingford

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Summary

The Ministry of Agriculture, Fisheries and Food (MAFF) has funded a programme of research at HR Wallingford into methods for the assessment of flood discharge. This work was identified as of highest priority by the Research Consultative Committee on Flood Protection in 1985 and a long term research programme commenced in the following Financial Year. The research was carried out with the co-operation of the former Regional Water Authorities and more recently the National River Authorities in England and Wales.

This report is one of two contract completion reports commissioned by MAFF and it describes the research outputs which can be translated immediately into engineering hydrometric practice. The methods will support the use of

- the extension of rating curves,
- the slope area method for calculation of river flows - especially channel roughness estimation, and
- the velocity area method for calculation of river flows.

The report itself contains no new methods which have not already been published in interim reports to MAFF during the project. The value of this report is intended to lie in that it draws together all the practical advances into a single document. Thus the text has been compiled using substantial proportions of other documents together with examples of the application of the methods. The second contract completion report summarises all the research and develops the main conclusions and recommendations to be made from the project.

There are three main sections to the report. The selection of channel and flood plain roughness is of critical importance in the assessment of the stage-discharge relationship for an open channel. Section 2 of the report contains a series of photographs at UK gauging stations together with calculated roughness values, and, a regression method for assessing roughness from channel dimensions. Several calculation procedures for the stage-discharge rating equation for out-of-bank flows were examined during the research. Section 3 describes the best available methods identified in the research. Finally Section 4 describes instrumentation and survey for a flood flow measurement site, including a peak velocity recorder developed in the course of the research.

Contents

Page

Contents continued

1 Introduction

1.1 Background

The Water Resources Act of 1963 placed on the Water Resources Board the duty of collecting data relating to the demand for water and the actual and prospective water resources for England and Wales. Consequently, many gauging stations were primarily designed to establish the quantity of water available for the community. The provision of flood data was originally considered to be of secondary importance.

When a flow measurement structure or rated channel section is out-flanked by a flood flow the uncertainties associated with flow measurement rise from 3-10% for in-bank flow conditions to 30% or more for out-of-bank flood conditions. Uncertainties of this magnitude can have a profound impact on the retum period associated through standard statistical techniques with a particular discharge. They may also lead to the design of a flood protection scheme being conservative with associated economic losses, or altematively inadequate with the benefits of the proposed scheme not being achieved.

Reporting upon the errors in flood discharge measurement the "Wolf" report of the Research Consultative Committee on River Flood Protection (MAFF, 1985) stated :

> "A research programme should be set up to develop new methods for measuring or estimating flow particularly over a flood plain. The objective of the project should be to produce a method which is inexpensive and effective and can possibly be applied after the event."

The Wolf committee identified this project as of the highest priority and the recommendation formed the basis for the present research carried out by HR Wallingford (HR) for the Ministry of Agriculture, Fisheries and Food (MAFF). This report briefly summarises the research carried out for the MAFF as part of the commission on River Flood Protection. The MAFF programme title was Flood Discharge Assessment and the programme number was initially 13F but in 1992 the programme number was changed to the Assessment Unit number FD 0105.

The research started in 1986 and was completed in March 1994. It was recognised from the outset of the project that the research would be long-term in nature since it relied in part on the capture of information on actual river floods and thus depended upon the vagaries of the climate. MAFF reviewed the progress of the work regularly and adjusted, when necessary, the objectives of the work on an annual basis. Finally, MAFF requested two project completion reports in 1994/95, the present one as a guide for prospective users of the work and a separate report summarising the whole of the project with appropriate conclusions and recommendations.

1.2 Objectives

The primary objective of the research was to develop methods of estimating or assessing the discharge, particularly peak discharge, of a flood that can be used at typical lowland gauging sites in the UK. During the course of the research rivers with water surface slopes at the bankfull discharge ranging between 1/336 and 1/5376 were studied. The methods, preferably, were to be applicable after the event. This broad objective was then broken down into a series of approaches designed to tackle different parts of the overall task.

The approaches which were followed during the research programme included:

- a review of "current" practice (carried out in 1987);
- the collection and analysis of data from several existing gauging stations;
- laboratory and field experiments to investigate measures to improve flood flow estimation in co-operation with the National Rivers Authority; and
- the use of computational models of gauging sites to extend the rating curve.

1.3 Summary of research

The first step in the research programme was to write to all the former Regional Water Authorities in England and Wales and identify the current practices adopted to assess or measure flood discharges. The review of current practice at the start of the research is contained in Tagg and Hollinrake (1987). Resulting from the initial contacts data from existing gauging stations was collected and various simple methods developed to predict the stage discharge curves. The results were compared with observations and presented by Ramsbottom (1989) and it was evident that comparatively few data existed for measured flood flows. Laboratory and field experiments were instigated to develop site specific measures to improve flood flow estimation and resulted in the development and evaluation of a device for measuring flood plain flow velocity which could be interrogated after the passage of a flood event. The development of the instrument is reported in Hollinrake (1990, 1991).

The approaches to flood discharge estimation described in the above reports share the disadvantage that they depend upon the choice of site specific parameters, or empirical ways of dividing the flow area. Consequently attention was turned toward further understanding the physical processes involved in the propagation of flow in straight compound channels from the SERC FCF research programme. The lateral distribution method is one such approach based on estimating the distribution of flow across a section and then integrating to obtain the total discharge. The method was applied to experimental data from the straight channel work on the SERC Flood Channel Facility at HR and to gauging data for flood flows supplied by the NRA. A model based on the lateral distribution method is described by Wark et al (1991).

Following the development of a method suitable for estimating conveyance in straight compound channels it was seen to be important to carry out a similar exercise for meandering compound channels. An empirical procedure was developed under a separate MAFF and NRA R&D project based upon experimental data from the meandering channel work undertaken on the SERC

Flood Channel Facility and from other experimental and field information. The procedure is reported in HR (1992a).

All the methods described above for improving the estimation of flood discharge require a knowledge of the roughness characteristics of the river channel and flood plain. At present, assessment of the channel roughness coefficient is made from site inspection or reference books. The need to be able to establish a more accurate method of determining the channel roughness characteristics was identified. Field data for rivers with a range of channel characteristics and water surface slopes were analysed to provide a relationship for determining the Manning's roughness coefficient for bankfull flow conditions. The empirical method described for estimating bankfull channel roughness characteristics is described in Hollinrake (1993). The field estimation of Manning's n has been further explored through a set of photographs taken at UK gauging stations with corresponding values of roughness calculated from measured flows, section geometry and water surface slope (Hollinrake and Millington, 1994).

1.4 Outline of procedure and contents of the guide

The research has contributed to three broad areas of flood discharge assessment, (see Figure 1), namely:

- the development and use of rating curves,
- the slope area method, and
- the velocity area method.

At most sites routine conversion of water level into a flow rate is achieved through the use of a unique rating curve or stage-discharge relationship. This rating curve may be derived from plotting measured flows at the site (and thus links to the velocity-area method) or from hydraulic calculation (thus linking to the slope-area method). The following sections of this guide provide information on the use of rating curves:

- plotting procedures in Section 3.6
- hand calculation of out-of-bank raing curves in Sections 3.2, 3.3 and 3.5
- computational model method for out-of-bank rating curve in Section 3.4

The technical report by Hollinrake and Millington (1994), describes further the use of one and two dimensional computational models for extending rating curves. One dimensional models can be used provided that care is taken to represent all the hydraulic controls around the site within the structure of the standard modelling software. The conclusion on two-dimensional models was that these are not yet appropriate for occasional use within general hydrometric practice but require a high standard of modelling expertise for successful application.

The slope-area method of flood discharge estimation requires a knowledge of the conveyance of the river and flood plain at the site of interest. The assessment of conveyance depends upon the cross-section shape, the hydraulic resistance of the ground surface and the calculation procedure. The slope of the water surface is also required to compute the flood discharge. The following Sections of this guide provide information for use in the slope area method:

- channel roughness from photographs of UK gauging stations in Section 2.2

- channel roughness from regression against river geometry in Section 2.3

- flood plain roughness estimates (from Dutch research) in Section 2.4

- the calculation procedures in Sections 3.2,.3.3 and 3.5

- the use of maximum water level recorders to capture the water surface slope described in Section 4.3

The assessment of flood flows should, wherever possible, include measurement of the velocity or discharge at the site under some conditions to reduce the margin of uncertainty in the purely predictive procedures. Calculation of river discharge from velocity measurements is routine at any open channel gauging site which is equipped with a cableway or bridge. In many cases however, not all the flood flow passes through the gauged part of the river section and altemative means of estimating the bypassing flow are needed. Section 4.2 of this guide describes a peak velocity meter that can be installed prior to a flood to capture information on the velocity over the flood plain at the peak of the flood.

The research which has led to this guide used flow and other measurements at many gauging stations in the UK. When the procedures are applied to ungauged sites to provide estimates of flood flows, due account should be made on the restrictions on site selection outlined in Section 4.4. In general the gauging stations used in this study have been sited in straight river reaches and, for example, this may have influenced the local values of river roughness used in the methods in Section 2.

2 Roughness selection

2.1 Background

In river engineering Manning's equation is usually used to represent channel resistance and is given by the relationship :

$$n = R^{0.667} * S^{0.5} / V \qquad (1)$$

where

n = Manning's roughness parameter ($s/m^{0.333}$)
R = hydraulic radius (m)
S = water surface slope (m/m)
V = mean flow velocity (m/s)

The equation provides a means of estimating discharge for steady, uniform flow in open channels from :

$$Q = A * R^{0.667} * S^{0.5} / n \qquad (2)$$

where

Q = discharge (m^3/s)
A = channel cross sectional area (m^2)

The main difficulty in using Manning's equation is estimating accurately a value of the roughness coefficient. Existing methods include the procedure developed by Cowan (1956) also known as the Soil Conservation Service Method (SCS, 1963), see Appendix 1. The method involves the selection of a basic value of Manning's n for a uniform, straight, and regular natural channel. The basic value is then adjusted for the effects of surface irregularities, shape and size of channel cross section, obstructions, vegetation and flow conditions and meandering of the channel.

An alternative method of estimating Manning's n involves the use of tables, as presented by Chow (1959) in association with photographic documentation of representative channel forms, see Appendix 2. The photographic method of estimating Manning's n was adopted by the United States Geological Survey. Photographs of river channels of known resistance are presented by Barnes (1967), with a brief description and summary of the geometry and hydraulic parameters which define the channel. Additional information in this form is also given by and Hicks and Mason (1991).

The problem facing British river engineers lies essentially in the selection of the basic Manning's n value for a straight, uniform and regular channel. Assessment of the roughness coefficient either from site inspection or reference books is frequently at best a speculative guess. This shortcoming was recognised by the former Severn-Trent Water Authority (STWA). In 1984 STWA commissioned a study to produce a direct measure of the Manning's n for a range of flows up to bankfull at sixteen open channel sites in the Severn-Trent Region already functioning as accurate flow measurement sites.

Data for 66 measurements of bankfull discharge, water surface slope and channel characteristics at six of the sites were made available to HR by the Severn Trent Region of the NRA (1991). The sites had channel widths at bankfull ranging between 20m and 80m, bankfull hydraulic radii between 1m and 3.5m, water surface slopes between approximately 1 in 330 and 1 in 5400 and bankfull discharges from 20 cumecs to 350 cumecs. These data were used to develop the procedures for estimating the Manning's n for bankfull flow conditions described in sections 2.2 and 2.3 below.

2.2 Photographic method

Barnes (1967) in presenting colour photographs and descriptive data for the roughness characteristics of 50 stream channels in North America states "Familiarity with the appearance, geometry, and roughness characteristics .... will improve the engineer's ability to select roughness coefficients for other channels." Hicks and Mason (1991) similarly present "a reference dataset for use in visually estimating roughness coefficients" of 78 river reaches in New Zealand. The objective of this section of the report is to present representative data and photographs of eight rivers for which roughness coefficients have been determined and which will be useful in estimating roughness characteristics of similar channels, see Plates 1 to 8.

The bankfull roughness of channels is required for the recommended methods used in estimating the stage / discharge relationships for overbank flows detailed in Section 3.

clean or dirty, ie blocked by organic debris, and the values are shown in Table 2. For flood plains vegetated with crops, brush or trees it is suggested that the values presented by Chow (1959) are adopted, see Appendix 2.

Although this information is given in good faith as being the best currently available, it should be noted that

- it derives from a variety of sources.
- the channel roughness values in equation 4 were derived from rivers in the Severn Trent region of the UK.
- no field evaluation of the accuracy of the combination of this information has been possible within the research project.

3 Calculation procedure

3.1 Background

"Straight" channels within the context of this report are considered to be channels with a sinuosity between 1.00 and 1.02. Sinuosity is defined as the ratio of the length along the centreline of the main river channel (thalweg) to the length along the centreline of the river valley between the same end points. Meandering channels are considered to be those channels having a sinuosity greater than 1.02.

When the main channel sinuosity is less than 1.02 the Divided Channel Method (DCM) is proposed as the procedure for estimating the stage-discharge relationship for overbank flows for "straight" channels, with appropriate corrections for sinuosity and for cases with a main channel sinuosity greater or equal to 1.02 the James and Wark (1992) method is proposed.

A "straight" channel method may also be appropriate at higher sinuosity if the lateral slopes of the flood plains are steep enough to constrain the flow to being parallel to the main channel. There is an intuitive argument that in this case the interaction between channel and flood plain is similar to the "straight" channel situation. The nature of the energy losses depends on whether the flows are parallel and not on the channel and flood plains being straight. There is no evidence to verify this argument and this aspect is still open to conjecture but recent numerical experiments, Seed and Wark (1994) are consistent with this interpretation.

The Divided Channel Method uses vertical division lines which are included in the wetted perimeter of the main channel but omitted from the wetted perimeter of the flood plains see Figure 5. The wetted perimeter of the flood plain should include the bank of the flood plain

The Lateral Distribution Method (LDM) is based on the work of Wark et al (1990) and was developed from an analysis of the results obtained from Phase A of the SERC Flood Channel Facility (FCF) work. It is a fairly complex mathematical model of flow distributions in straight compound channels and is based on approximations to the physical processes rather than an empirical approach.

Table 2 Roughness coefficients for flood plains with hedges (Klaassen and van der Zwaard, 1974)

Hedgerow separation (m)	Manning's n roughness coefficient for clean or dirty hedgerows with varying flow depths									
	0.25 m		0.50 m		1.00m		1.50m		2.00m	
	Clean	Dirty	Clean	Dirty	Clean	Dirty	Clean	Dirty	Clean	Dirty
50	0.038	0.072	0.045	0.089	0.053	0.091	0.054	0.086	0.051	0.080
100	0.032	0.053	0.032	0.064	0.042	0.067	0.042	0.063	0.041	0.060
250	0.029	0.040	0.031	0.045	0.029	0.045	0.032	0.045	0.032	0.041
500	0.028	0.035	0.028	0.036	0.029	0.037	0.027	0.036	0.029	0.034
1000	0.027	0.031	0.027	0.032	0.027	0.031	0.025	0.029	0.027	0.027

The FCFA procedures are based on a detailed analysis of stage discharge data collected from the FCF during Phase A. The FCF data is the most comprehensive and accurate laboratory data set that exists. The FCFA procedures allow prediction of stage-discharge relationships, division of total discharge into main channel and flood plain components and estimation of boundary shear stresses.

3.2 Straight Channel - Divided Channel Method (DCM)

Introduction

The purpose of this report is to present a simple hand calculation method for estimating flood flows that can be easily implemented by river engineers. Ramsbottom (1989) and James and Wark (1992) found the DCM method to be a simple and accurate means of estimating out-of-bank flows. Consequently the the Divided Channel Method (DCM) is proposed as a procedure for estimating the stage-discharge relationship for overbank flood flows for "straight" channels.

The procedure for obtaining the data required in order to apply the DCM method and a worked example for the River Severn at Montford are given below. The estimated flows are compared with the flows derived from the rating curve established for the site.

The selection of the site and survey requirements where flow estimation is to be undertaken should be based upon the guidelines given in Section 4.4. The channel roughness parameter can be estimated using the formulae detailed in Section 2.3. **Ideally the slope of the river reach should be the water surface slope of the river in flood possibly measured by maximum level gauges (See Section 4.3).** The water surface slope, however, can be assumed to be equal to the bank slope. Methods of obtaining the bank slope are as follows :

- By measuring between the contours on OS maps where there are enough contours upstream and downstream of the site to define the slope adequately.

- Surveying bank levels upstream and downstream of the site when the cross section survey is carried out.

- From longitudinal sections of the river held by the River Authority (if available).

The water surface slope for a particular peak flow event can also be measured by surveying wrack marks deposited by flood flows. Caution must be adopted when using this approach to ensure that

- the wrack mark is representative of the flood event being studied.
- the level of wrack mark has not been influenced by wave action.
- the wrack mark has not subsequently been washed/blown down if formed of organic debris.

It is strongly recommended that the surveying of wrack marks to determine water surface slope is not used as a substitute for determining the water surface slope from peak water surface level gauges.

The flood plain roughness parameter can be estimated from Section 2.3 and Appendix 2.

Additionally it will also be necessary to estimate that width of the flood plain(s) which would be actively flowing and that which would be acting as storage. The assessment of flood plain flow activity requires considerable experience and professional judgement. The experience can be built up by visiting potential flood flow estimation sites during high flows.

Worked example

Introduction

The DCM method requires an estimate of the roughness coefficients to be used for the main river channel and the flood plain(s). It is proposed to adopt Manning's n coefficients in this worked example. For the main channel the Manning's n for bankfull flow is used in estimating the main channel flow component above bankfull. Research has shown (Pimperton and Karle, 1993), that Manning's n tends to reduce as discharge increases to bankfull, however, the relationship is essentially asymptotic above bankfull, see Figure 4. Estimates of flood plain roughness should be determined from site inspection, scaling from maps if information on separation of hedgerows is required and by reference to Section 2.3 and Appendix 2.

Problem definition

The discharge for bankfull flow and the corresponding Manning's n roughness coefficient is to be determined for the gauging section on the River Severn at Montford using the equations given in Section 2.3. The DCM method is then to be used to estimate the flood flow at four stages above bankfull and the estimates are to be compared with the discharge at each stage as determined from the rating curve for the site. The notation used with the DCM method is shown in Table 3.

Table 3 Notation for "straight" channel

		Units
A_b	cross-sectional area of main channel at bankfull	m^2
A_L	unsubscripted, cross-sectional area of left flood plain	m^2
A_{MC}	unsubscripted, cross-sectional area of main channel	m^2
A_R	unsubscripted, cross-sectional area of right flood plain	m^2
B_b	bankfull width of main channel	m
n	coefficient in Manning's equation	$m^{0.333}/s$
n_{pred}	predicted Manning's n coefficient from equation 4	$m^{0.333}/s$
P_b	wetted perimeter of main channel at bankfull	m
P_L	unsubscripted, wetted perimeter of left flood plain	m
P_{MC}	unsubscripted, wetted perimeter of main channel	m
P_R	unsubscripted, wetted perimeter of right flood plain	m
Q_L	unsubscripted, left flood plain discharge	m^3/s
Q_{MC}	unsubscripted, main channel discharge	m^3/s
Q_{pb}	predicted bankfull discharge from equation 3	m^3/s
Q_{pt}	predicted total discharge	m^3/s
Q_R	unsubscripted, right flood plain discharge	m^3/s
R_b	hydraulic radius of main channel at bankfull	m
R_L	unsubscripted, hydraulic radius of left flood plain	m
R_{MC}	unsubscripted, hydraulic radius of main channel	m
R_R	unsubscripted, hydraulic radius of right flood plain	m
S_b	main channel water surface slope at bankfull	
S_{fl}	main channel water surface slope under flood conditions	
s	sinousity	

Subscripts

1-4 Stages 1 to 4

Solution

1. Define bankfull flow cross-section and calculate necessary geometric parameters

The gauging section for the River Severn at Montford is shown in Figure 6. Wharton et al (1989) defined the bankfull level "as the level at which incipient flooding occurs." Natural channels rarely adopt a symmetry such that incipient flooding occurs at the same level. In respect of the River Severn at Montford, where the main channel is bounded by two flood plains, the level of incipient flooding, and consequently bankfull flow, was taken to be the lower of the river bank levels, in this case the left bank.

The following geometric characteristics are calculated for the water surface at bankfull at a specific cross-section at the gauging section.

A_b = 145 m^2

B_b = 36 m

P_b = 40 m

R_b = 3.625 m

It should be noted that the values given above for a single section differ from the reach average values used in Sections 2.2 and 2.3.

The water surface slope observed on 12 October 1987 when bankfull flow conditions occurred at the gauging section has been adopted in order to determine the roughness coefficient. The bankfull water surface slope for the event was

S_b = 0.000186

Using Equation 3 the predicted bankfull discharge is

Q_{pb} = 139 m^3/s

and using Equation 4 the predicted bankfull Manning's n is

n_{pred} = 0.033

Comparison with Section 2.2 shows that the bankfull discharge is underpredicted by 8% and the roughness overpredicted by 18%. However, Equations 3 and 4 were derived from the average channel cross-sectional characteristics from each study reach. Applying the formulae to a single section will give the bankfull flow and roughness coefficient for that section. This comparison indicates the magnitude of difference which can be attributed to the use of cross-section data for a specific section as opposed to the more lengthy computation of reach average properties.

2. <u>Define out-of-bank flow cross section and calculate necessary geometric parameters</u>

The out-of-bank flow cross section subdivisions are shown on Figure 7. The water surface slope used for predicting the out-of-bank flows was

$$S = 0.000212$$

and was derived by averaging thirteen observations of water surface slope for events at or above the predicted bankfull flow conditions. The Manning's n roughness coefficient applied to the main channel was 0.033 as determined for bankfull conditions. Site inspection showed the flood plains to be short grass pasture with field boundaries defined by incomplete hawthorn hedgerows set approximately 300m apart, see Plate 3. Consequently the roughness coefficient for the flood plains was set at 0.027 for shallow depths and 0.025 for the higher stages to represent the predominantly short grass covered flood plains, see Table 1.

The following geometric characteristics were calculated and discharges predicted for the main channel using Equation 2.

Main channel

Stage	A_{MC1-4}	P_{MC1-4}	R_{MC1-4}	$R_{MC1-4}^{0.667}$	S_n	$S_n^{0.5}$	n_{pred}	Q_{MC1-4}
(m)	(m^2)	(m)	(m)					(m^3/s)
4.5	160	41	3.90	2.48	2.12E-4	1.456E-2	0.033	175
5.0	179	42	4.26	2.63				208
5.5	196	43	4.56	2.75				238
6.0	213	44	4.84	2.86				269

Note: The vertical division line is included in the main channel wetted perimeter but not in the flood plain wetted perimeter, see Figure 5.

The geometric characteristics were calculated for the left and right flood plains and discharges calculated using Equation 2.

Left flood plain

Stage	A_{L1-4}	P_{L1-4}	R_{L1-4}	$R_{L1-4}^{0.667}$	S_n	$S_n^{0.5}$	n	Q_{L1-4}
(m)	(m^2)	(m)	(m)					(m^3/s)
4.5	1	12	0.10	0.19	2.12E-4	1.456E-2	0.027	0.1
5.0	18	36	0.50	0.63			0.026	6.4
5.5	38	63	0.60	0.71			0.026	15
6.0	73	65	1.12	1.08			0.025	46

and

Right flood plain

Stage	A_{B1-4}	P_{B1-4}	R_{B1-4}	$R_{B1-4}^{0.667}$	S_B	$S_B^{0.5}$	n	Q_{B1-4}
(m)	(m²)	(m)	(m)					(m³/s)
4.5	1	2.5	0.40	0.54	2.12E-4	1.456E-2	0.027	0.3
5.0	5	17	0.29	0.44			0.026	1.2
5.5	16	23	0.70	0.79			0.026	7
6.0	28	27	1.04	1.03			0.025	17

For a given stage the predicted total discharge is calculated as the sum of main channel and left and right flood plain discharges, ie.

$$Q_{PT} = Q_{MC} + Q_L + Q_R \tag{7}$$

The predicted total discharges were then compared with the discharges derived from the NRA-ST stage-discharge rating for the site.

Discharge

Stage (m)	Predicted total discharge Q_{PT} (m³/s)	Rated discharge (m³/s)	Difference (%)
4.5	175	177	-1
5.0	215	207	+4
5.5	260	248	+5
6.0	332	312	+6

The rated discharge was determined from the rating curve at Montford which was derived from a comprehensive set of current meter gaugings taken from a cableway at the site, which extends across the width of the channel and flood plain. Under flood conditions at Montford data from flow measurement exercises shows that flow exists across the full width of the section.

In estimating flood discharges at previously ungauged sites care needs to be exercised to ensure that an accurate assessment is made of that part of the flood plain which is actively flowing and that which is acting as storage. Assuming British river flood plains to be fully active across the wetted section under flood conditions will more likely than not lead to an overestimate in the discharge carried by the flood plains and consequently in the estimated total flow.

3.3 Meandering Channel - James and Wark method

Introduction

Following the development of a method suitable for estimating conveyance in straight compound channels it was seen to be important to carry out a similar exercise for meandering channels. The approach adopted used the results available from the SERC Flood Channel Facility Phase B data with the objective of presenting a hand calculation method that can be implemented by river engineers.

The equations used for estimating out-of-bank flows in meandering channels and the approach adopted in the development of the James and Wark method are summarised in Appendix 3. It is considered to be important and necessary that the approach adopted in the development of the method should be read and clearly understood before proceeding to the worked example. This worked example is also reproduced in the NRA R & D Report 13 (NRA, 1994).

Worked Example

Problem Definition

The conveyance of a two-stage meandering river channel is to be determined. The reach under consideration is shown in Figure 8 and is represented by the surveyed cross-section at the location indicated, which is presented in Figure 9. The slope of the flood plain is estimated as 0.0014.

Manning's n values for the main channel and flood plains of 0.025 and 0.045 respectively, were chosen. The method detailed in Section 2.3 could be applied to determine the roughness coefficient for a straight reach of main channel. This basic value should then be adjusted using the LSCS method to determine the roughness parameter for the meandering main channel to take account of meander losses.

The following are required.

- The capacity of the main channel at bankfull.
- The zonal and total discharges when the water level is 1.2m above bankfull level.

Solution

Step 1 Define cross-section zones and calculate the necessary geometric parameters

The zone subdivisions are shown in Figures 10 and 11. Because the geometry varies along the reach the positions of the subdivision planes are selected by judgement to represent average conditions for the reach. The notation used with the James and Wark method is shown in Table 4.

Table 4 Notation for meandering channel

		Units
A	cross-sectional area	m^2
A	unsubscripted, cross-sectional area of main channel	m^2
B	top width of main channel	m
C_{sl}	length coefficient for expansion and contraction losses, Zone 2	m
C_{ssc}	side slope coefficient for contraction loss, Zone 2	
C_{sse}	side slope coefficient for expansion loss, Zone 2	
C_{wd}	shape coefficient for expansion and contraction losses, Zone 2	

c	coefficient in equation for Zone 1 adjustment factor	
F_1	factor for non-friction losses in Zone 2 associated with main channel geometry	
F_2	factor for additional non-friction losses in Zone 2 associated with main channel sinuosity	
f	Darcy-Weisbach friction factor	
f'	ratio of flood plain and main channel Darcy-Weisbach friction factors	
g	gravitational acceleration	m/s^2
h	hydraulic mean depth of main channel = A/B	
K	coefficient in equation for Zone 1 adjustment factor	
K_c	contraction coefficient	
K_e	factor for expansion and contraction losses in Zone 2	
L	meander wavelength	m
m	coefficient in equation for Zone 1 adjustment factor	
n	coefficient in Manning's equation	$m^{0.333}/s$
n'	coefficient in Manning's equation, including bend losses	$m^{0.333}/s$
P	wetted perimeter	m
P	unsubscripted, wetted perimeter of main channel at bankfull	
Q	zonal discharge	m^3/s
Q_{bf}	main channel bankfull discharge	m^3/s
Q_{calc}	calculated discharge	m^3/s
Q_{meas}	measured discharge	m^3/s
Q_T	total discharge	m^3/s
Q_1	adjustment factor for Zone 1 discharge	
R	hydraulic radius	m
R	unsubscripted, hydraulic radius of main channel at bankfull	m
S	main channel gradient	
S_o	flood plain gradient	
S_s	cotangent of main channel side slope (Horizontal / Vertical)	
s	channel sinuosity	
V	mean flow velocity	m/s
V	unsubscripted, mean flow velocity in main channel at bankfull	m/s

W_2	width of Zone 2	m
y_2	flow depth on flood plain a main channel bank	m
y'	dimensionless flow depth on flood plain = y_2 / (A/B)	

Subscripts 1-4 Zones 1 to 4

From the geometries defined by this subdivision, the following geometric characteristics are calculated for the water surface 1.2m above bankfull.

Zone 1 : Main Channel

A = 5.07 m^2

P = 6.40 m

B = 6.10 m from survey

The main channel sinuosity is found from the plan of the reach. It is defined as the ratio of the length along the channel centre line (between two points) to the straight line distance between the points. Using points x and y on Figure 10, this gives a sinuosity of

s = 376 m / 275 m

= 1.37

Note : Since s > 1.02 we should use the method for meandering compound channels. If s had been 1.0 ≤ s ≤ 1.02 then we should use the "straight" compound channel method described in Section 3.2 with appropriate correction for sinuosity.

The main channel slope is obtained by dividing the flood plain slope by the sinuosity, ie.

S = 0.0014 / 1.37

= 0.00102

The main channel side slopes are measured on the cross-section reproduced in Figure 11. The average of the values for the right and left banks will be used in the calculations, ie.

S_s = (1.43 + 1.64) / 2

= 1.54

Note: The final solution is likely to be relatively insensitive to side slope, so great accuracy is unnecessary in estimating the value.

Zone 2 : Inner Flood Plain

A_2 = 47.77 m^2 from survey

The wetted perimeter is calculated excluding the division planes, ie.

P_2	=	Wetted surface +	Wetted surface - Channel top width (sinuosity - 1.0)
		to left of main channel	to right of main channel

P_2 = 22.48 + 17.72 - 6.10 × (1.37 - 1.00)

= 37.94 m

W_2 = 49.40 m   from survey

Zone 3 : Left Bank Outer Flood Plain

A_3 = 16.28 m^2

P_3 = 18.90 m   from survey

Zone 4 : Right Bank Outer Flood Plain

A_4 = 8.00 m^2

P_4 = 21.00 m   from survey

Step 2   Calculate the capacity of the main channel at bankfull.

Q_{bf} = A V

A = 5.07 m^2 from Step 1

V is calculated using Manning's equation,

$V = 1 / n\ R^{0.667}\ S^{0.5}$

The coefficient n is given as 0.025, based on surface roughness. This must be adjusted to account for meander losses, which can be done using the Linearized SCS Method, given by equation 5, ie.

$n' = n\ (0.43\ s + 0.57)$

$= 0.025 \times (0.43 \times 1.37 + 0.57)$

$= 0.029$

Note: If the given value of 0.025 had been obtained from a back calculation on measured discharges then this would already account for the influence of the meandering channel on inbank flow resistance and the adjustment above would be unnecessary.

The hydraulic radius is given by

$R = A / P$

$= 5.07 / 6.40$

$= 0.792$ m

Therefore

$V = (1 / 0.029) \times 0.792^{0.667} \times 0.00102^{0.5}$

$= 0.943$ m/s

Therefore the bankfull discharge is

$Q_{bf} = 5.07 \times 0.943$

$= 4.78$ m^3/s

Step 3 Calculate the discharge for water level 1.2m above bankfull

Step 3.1 Calculate Zone 1 discharge

$Q_1 = Q_1' \, Q_{bf}$ (equation 3.12, Appendix 3)

$Q_{bf} = 4.78$ m^3/s from Step 2

The Zone 1 adjustment factor, Q_1', is the greater of the values given by equations 3.10 and 3.11, Appendix 3.

$Q_1' = 1.0 - 1.69\,y'$ (equation 3.10, Appendix 3)

where

$y' = y_2 / (A / B)$

$= 1.20 / (5.07 / 6.10)$

$= 1.44$

Therefore

$Q_1' = 1.0 - 1.69 \times 1.44$
$= -1.43$

$Q_1' = m\,y' + K\,c$ (equation 3.11, Appendix 3)

where

$m = 0.0147\,B^2/A + 0.032\,f' + 0.169$

and

$B^2/A = 6.10^2 / 5.07$

$= 7.34$

$f' = (n_2 / n_1)^2 (R_1 / R_2)^{0.333}$ (equation 3.9, Appendix 3)

$R = 0.792$ m *from Step 2*

$R_2 = A_2 / P_2$

$= \quad 47.77 / 37.94$

$= \quad 1.259$ m

consequently

$f' \quad = \quad (0.045 / 0.025)^2 \times (0.792 / 1.259)^{0.333}$

$= \quad 2.78$

Therefore

$m \quad = \quad 0.0147 \times 7.34 + 0.032 \times 2.78 + 0.169$

$= \quad 0.366$

$K \quad = \quad 1.14 - 0.136 f'$

$= \quad 1.14 - 0.136 \times 2.78$

$= \quad 0.762$

$c \quad = \quad 0.0132 B^2/A - 0.302 s + 0.851$

$= \quad 0.0132 \times 7.34 - 0.302 \times 1.36 + 0.851$

$= \quad 0.534$

Therefore

$Q_1' \quad = \quad 0.366 \times 1.44 + 0.762 \times 0.534$

$= \quad 0.934$

which is greater than the value given by equation 3.10, Appendix 3.

Therefore the discharge in Zone 1 is

$Q_1 \quad = \quad 0.934 \times 4.78$

$= \quad 4.46$ m^3/s

In engineering applications the level of accuracy will be less than implied by quoting the answer to this precision hence Q_1 should be give as :

$Q_1 \quad = \quad 4.5$ m^3/s

Step 3.2 Calculate Zone 2 discharge

$Q_2 \quad = \quad A_2 V_2$ (equation 3.13, Appendix 3)

where

$A_2 \quad = \quad 47.77$ m^2 from Step 1

$$V_2 = \left(\frac{2\, g\, S_0\, L}{(f_2\, L) / (4\, R_2) + F_1\, F_2\, K_e} \right)^{0.5}$$ (Equation on 3.14, Appendix 3)

The average meander wavelength is estimated from Figure10 by dividing the flood plain length by the number of wavelengths over the reach, ie.

L = 275 / 3

= 91.7 m

R_2 = 1.259 m — from Step 3.1

f_2 = $(8\, g\, n_2^2) / R_2^{0.333}$ — (equation 3.8, Appendix 3)

= $(8 \times 9.81 \times 0.045^2) / 1.259^{0.333}$

= 0.147

F_1 = $0.1\, B^2/A$ — (equation 3.15, Appendix 3)

where

B^2/A = 7.34 — from Step 3.1

= 0.1×7.34

= 0.734

F_2 = s / 1.4 — (equation 3.16, Appendix 3)

= 1.37 / 1.4

= 0.979

K_e = $C_{sl}\, C_{wd}\, (C_{sse}(1 - y_2 / y_2 + h))^2 + C_{ssc}\, K_c)$ — (equation 3.17, Appendix 3)

where

C_{sl} = $2\,(W_2 - B) / W_2$ — (equation 3.18, Appendix 3)

= $2 \times (49.4 - 6.10) / 49.4$

= 1.753

C_{wd} = $0.02\, B^2/A + 0.69$ — (equation 3.19, Appendix 3)

and

B^2/A = 7.34 — *from Step 3.1*

Therefore

C_{wd} = $0.02 \times 7.34 + 0.69$

= 0.837

C_{sse} = $1.0 - S_s / 5.7$ (equation 3.20, Appendix 3)

= $1.0 - 1.54 / 5.7$

= 0.730

C_{ssc} = $1.0 - S_s / 2.5$ (equation 3.21, Appendix 3)

= $1.0 - 1.54 / 2.5$

= 0.384

h = A / B

= $5.07 / 6.10$

= 0.831 m

$y_2 / (y_2 + h)$ = $1.2 / (1.2 + 0.831)$

= 0.591

K_c = 0.217 from Figure A3.3

Therefore

K_e = $1.753 \times 0.837 \times (0.730 \times (1 - 0.591)^2 + 0.384 \times 0.217)$

= 0.301

Therefore

$$V_2 = \left(\frac{2 \times 9.81 \times 0.0014 \times 91.7}{((0.147 \times 91.7) / (4 \times 1.259) + 0.734 \times 0.979 \times 0.301)} \right)^{0.5}$$

= 0.933 m/s

Therefore the discharge in Zone 2 is

Q_2 = 47.77×0.933

= 44.57 m^3/s

= 44.6 m^3/s

Step 3.3 Calculate Zone 3 discharge

Q_3 = $A_3 V_3$

where

$A_3 \quad = \quad 16.28\ m^2$ from Step 1

V_3 is calculated using Manning's equation,

$V_3 \quad = \quad (1 / n_3)\ R_3^{0.667}\ S_o^{0.5}$

where

$n_3 \quad = \quad 0.045$

$R_3 \quad = \quad A_3 / P_3$

$\quad = \quad 16.28 / 18.90$

$\quad = \quad 0.861\ m$

Therefore

$V_3 \quad = \quad (1 / 0.045) \times 0.861^{0.667} \times 0.0014^{0.5}$

$\quad = \quad 0.753\ m/s$

Therefore the discharge in Zone 3 is

$Q_3 \quad = \quad 16.28 \times 0.753$

$\quad = \quad 12.26\ m^3/s$

$\quad = \quad 12.3\ m^3/s$

Step 3.4 Calculate Zone 4 discharge

$Q_4 \quad = \quad A_4\ V_4$

where

$A_4 \quad = \quad 8.00\ m^2$ from Step 1

V_4 is calculated using Manning's equation,

$V_4 \quad = \quad (1 / n_4)\ R_4^{0.667}\ S_o^{0.5}$

where

$n_4 \quad = \quad 0.045$

$R_4 \quad = \quad A_4 / P_4$

$\quad = \quad 8.00 / 21.00$

$\quad = \quad 0.381\ m$

Therefore

$V_4 \quad = \quad (1 / 0.045) \times 0.381^{0.667} \times 0.0014^{0.5}$

$\quad = \quad 0.438$ m/s

Therefore the discharge in Zone 4 is

$Q_4 \quad = \quad 8.00 \times 0.438$

$\quad = \quad 3.5$ m^3/s

Step 3.5 Calculate total discharge

$Q_T \quad = \quad Q_1 + Q_2 + Q_3 + Q_4$ (equation 3.3, Appendix 3)

$\quad = \quad 4.5 + 44.6 + 12.3 + 3.5$

$\quad = \quad 64.9$ m^3/s

Hence the total discharge in the channel is 65 m^3/s.

3.4 Lateral Distribution Method (LDM)

As an alternative to the simple hand calculation approach for "straight" channels the LDM method, which requires the solution of non-linear differential equations, is considered worthwhile as a practical method for estimating the discharge in a two stage channel. The Lateral Distribution Method (LDM) is based on estimating the distribution of flow across a section and then integrating to obtain the total discharge.

The governing equation, (either 8 or 9), is derived from the general 2-D shallow water equations. Two main assumptions are made in the derivation of these equations : flow is steady and uniform (in the longitudinal direction) and the water surface is horizontal across the channel.

$$g\,DS_{xf} - \frac{Bf\,|U|U}{8} + \frac{\partial}{\partial y}\left[\,\nu_t\,D\frac{\partial U}{\partial y}\,\right] = 0 \qquad (8)$$

$$g\,DS_{xf} - \frac{Bf\,|q|q}{8\,D^2} + \frac{\partial}{\partial y}\left[\,\nu_t\,\frac{\partial q}{\partial y}\,\right] = 0 \qquad (9)$$

Gravity Bed shear Lateral shear

where

B $= (1 + S_x^2 + S_y^2)^{0.5}$: a factor relating stress on an inclined surface to stress in the horizontal plane, see Wark et al (1990).

D = flow depth. (m)

f = Darcy friction factor.

g = Gravitational acceleration.

= 9.81 m/s^2

S_x	= Longitudinal slope of channel bed.	(m/m)
S_y	= Lateral slope of channel bed.	(m/m)
x	= Longitudinal coordinate direction.	(m)
y	= Lateral coordinate direction.	(m)
q	= Longitudinal unit flow (=UD).	(m^2/s)
U	= Longitudinal depth averaged velocity.	(m/s)
v_t	= Lateral eddy viscosity.	(m^2/s)

Given estimates of the bed shear and lateral shear term it is possible to solve Equations 8 or 9 for the distribution of flow across the channel and flood plain. This in turn may be integrated to provide the discharge or used to calculate the distribution of bed shear stress across the channel. Equation 9 is to be preferred on technical grounds.

The bed shear term is calculated by local application of 1-D theory. For example Manning's equation :

$$f = 8\,g\,n^2 / D^{0.333} \qquad (10)$$

n = Manning's n

The lateral shear term is more difficult to evaluate and various models for the lateral eddy viscosity have been proposed. An early example, Vreugdenhil and Wijbenga (1982), used a constant value of v_t but did not compare the solution with measured data. More physically realistic models may be obtained by dimensional analysis. The lateral eddy viscosity relating to bed roughness generated turbulence is given by equation 11.

$$v_t = \lambda\, U.\, D \qquad (12)$$

where

$U.$ = the shear velocity $(\tau_b / \rho)^{0.5}$

λ = the nondimensional eddy viscosity (NEV)

ρ = fluid density

τ_b = bed shear stress

Values of λ are usually quoted as being approximately 0.16 ± 50% in straight laboratory flumes increasing to between 0.6 and 2.0 in river channels, see Okoye (1970). Some authors, eg. Wormleaton (1988), suggest that shear layer driven turbulence may be an important source of lateral shear in compound channels. In this case it can be shown that the lateral eddy viscosity is given by an expression of the form :

$$v_t = C\, l_s\, \Delta U \qquad (12)$$

where

C = a constant

l_s = a length scale related to the width of shear layer

ΔU = velocity difference across the shear layer.

More sophisticated attempts have been made using a depth averaged form of the k-ε turbulence model, Keller and Rodi (1989). However the cost of the additional computation is large and k-ε models are unlikely to form the basis of practical design aids. Analytic solutions to equation 8 are available only for certain simplified cases, Samuels (1988) and Shiono and Knight (1988).

Wark et al (1991) developed a numerical solution and applied the method to discharge and velocity data available from the SERC FCF. Optimum values of the NEV were identified and comparisons made with other methods of calculating discharge.

River gauging data

Examples of the application of the LDM to data from the River Severn at Montford and River Penk at Penkridge is given below.

Figure 12 shows the stage discharges predicted with both single and multiple values of NEV for the River Severn at Montford. From the top plot it is clear that the fixed value of about 0.16 provides results which closely match those measured. A similar match can be achieved with variable values of NEV as shown in the lower plot. The predicted velocity profiles for both cases are shown in Figure 13 for three stages. The use of different NEV values across the channel appears to accentuate the 'peaks' at the main channel/flood plain boundaries, making them both narrower and larger. Overall little practical difference is evident in the distributions: they are very similar in the main channel and over most of the wide flood plain. The predicted discharges for the three velocity profiles are shown in the following table.

Stage (mAOD)	NEV		
	0.16	Varies	Difference %
	m^3/s	m^3/s	
6.087	346.0	344.3	0.5
5.200	235.2	235.2	0.0
4.730	197.3	197.3	0.0

Again there is no practical difference and the extra work required to identify the NEV values appropriate for each part of the channel does not result in improved prediction for either the velocity profile or the total discharge.

The four velocity profiles for the River Penk, Figure 14, show good agreement with the observations. The main channel velocities are underpredicted slightly, although this is only significant at the highest stage. The corresponding discharges are given below and it is clear that the LDM gave excellent results in this case.

Stage	Q_{obs}	Q_{calc}	Error
mAOD	m^3/s	m^3/s	%
1.94	32.8	29.4	-10.4
1.90	28.2	28.2	0.0
1.84	26.5	26.6	0.4
1.66	21.8	22.3	2.3

The results at 1.94m was obviously out of step with the other results and when the original current metering data, provided by the NRA, was checked it turned out that this one set of data was unreliable. **This highlights the problems in collecting accurate and consistent discharge data for real rivers.**

3.5 The FCFA method

The philosophy behind the development of this method for straight compound channels is summarised in Ackers (1991). It has the title FCFA method since it was derived from the Series 'A' data of the Flood channel facility (FCF). The approach is to divide the channel into three zones shown in Figure 15.

Zone	Description
1	Main channel.
2	Flood plain zone on the left of the main channel.
3	Flood plain zone on the right of the main channel.

Vertical division lines are used and these are not included in the wetted perimeters for any of the zones. The "basic" zonal discharges are calculated from standard friction equations (eg. Manning's) and added to obtain a "basic" discharge, which is then adjusted to account for the effects of the interaction between the main channel and flood plain flows. The adjustment required depends on the characteristics of the channel and also varies with stage. Four regions of flow behaviour are identified, as shown in Figure 16.

The effect of flow interaction is complex, alternately increasing and decreasing with flow depth through the different regions. Also shown on this diagram is the curve of the channel coherence, COH. Ackers (1991) introduced this new parameter and defined it as : the ratio of the conveyance calculated as a single cross-section to that calculated by summing the conveyances of the separate flow zones. The coherence of a compound channel provides a measure of the relative strength of the interaction effect between the zonal flows. As channel depth increases COH typically tends to a value of 1, indicating that the compound channel behaviour approaches that of a simple compact channel at larger depths.

Ackers (1991) provided a different adjustment function for each region, and a logical procedure for selecting the correct discharge value from those calculated assuming each adjustment function in turn. He provided additional corrections to account for the effect of deviations of up to 10° between the alignments of the main channel and the flood plains and a procedure for dividing the computed total discharge at any stage into main channel and flood plain components.

The correction factors vary strongly with stage; there are four equations which describe the variation of the correction factors with stage in the four regions. At any particular stage it is impossible to tell beforehand which region gives the true adjustment factor. The approach is to calculate adjusted discharges using the factors for the four regions. Once all four adjusted discharges have been obtained then it is simple to choose the correct value using the guidelines provided by Ackers.

The FCFA has been demonstrated as giving the most consistently accurate results of the hand calculation techniques available for straight compound channels for laboratory data, Ackers (1991). Only the LDM can match the FCFA method in terms of accuracy and consistency. However, extensive calculations are required to reach the final solution. The step-by-step calculation procedure for FCFAM is also included in the NRA R & D report 13 (NRA, 1994). James and Wark (1992) showed that the accuracy of the FCFA method for field data from UK gauging stations was significantly worse than for laboratory data, being about ±15% (mean plus two standard deviations) in the field compared with ±3% for the laboratory data.

3.6 Extrapolating rating curves - hand calculation methods

A rating curve is a (unique) relationship between water level and flow rate at a site. The curve is often assumed to be unique and single valued but there are many processes in flood propagation which may invalidate this assumption. These include:

- unsteadiness in the flood flow from either a rising or receding flood;
- unsteadiness caused by large physical variations at the just-out-of-bank condition;
- changes in channel dimensions or resistance during the flood flow due to sediment movement,
- backwater from tides, confluences or moving element structures, and
- seasonal variations in river roughness or bed level.

Nevertheless, rating curves together with their extrapolation probably still form the most widespread means of providing flood discharge assessments. Hence one approach to the research was to examine the most effective means of extending flood flow rating curves. Two types of extrapolation were reviewed in detail using information collected from the RWAs: extrapolation based upon a stage-discharge plot and extrapolation based upon the geometry of the section and hydraulic flow laws.

In the extrapolation by stage-discharge plotting it became evident that, at some of the sites tested, the out-of-bank flow conditions were represented better by allowing a discontinuity at the bankfull condition rather than assuming that the stage-discharge curve is continuous at this point (Ramsbottom, 1989). Figure 17 illustrates this for the River Culm at Wood Mill. This observation has important practical implications. If the discharge capacity of the channel and flood plains reduces as the flow exceeds the bank level, then the slope of the out-of-bank rating curve will be changed and this increases the discharge estimates at the highest flows from those made assuming continuity at the bankfull condition. On the other hand, if the best fit allows for a step increase in the flow as the stage increases above the bank level, then the change in the slope of the rating curve reduces the discharge values from those assuming continuity of the rating curve. For the experiments in the SERC Flood Channel Facility, the stage-discharge curves showed a step decrease in flow at the bankfull condition (Myers & Brennan, 1990)

The alternative means of extrapolating rating curves is to use gauged flow information to calibrate the conveyance of the site for known flood conditions and then to use a hydraulic calculation procedure to extend the rating curve to high depths. At the heart of this method is the estimation of the conveyance of the river section at the gauging site. Appropriate methods are described in Sections 3.2, 3.3 and 3.5 above.

4 *Equipment*

4.1 Background

The ability to estimate discharge in a channel depends upon a knowledge of certain hydraulic or geometric characteristics of the channel, in particular the cross sectional area of the flow, flow width and water surface slope. The cross section of a river channel and its flood plain can be readily determined by survey. The determination of the water surface slope of a flood event requires that water level gauges along the study reach record and store the maximum water level reached so that it can be read after the passage of the flood event. This section of the report presents a method for recording flood levels and so determining the water surface slope under flood conditions for both the main river channel and flood plains. The measurement of flood plain flows using a simple mechanical system, a peak velocity meter, is also described.

4.2 Peak velocity meter

Field measurement of flood plain velocity for flood events in the United Kingdom is very limited. During times of flood NRA and LA staff are generally more concerned with providing emergency response to the flooding rather than measuring the discharge, problems also arise concerning safety and access to a measuring site. Consequently methods of assessing flood plain flows that could be applied after the event were investigated.

The development of the peak velocity meter is described in Hollinrake (1990,1991) and consists of a deflecting vane with a clutch bearing which only allows rotation in the direction of flow, see Plate 9. Consequently after the passage of a flood the vane will retain the deflection due to the peak velocity. It should be noted that the maximum velocity may not necessarily coincide with the maximum flood depth. The meter can measure flow velocities over a range from 0.05 m/s to 0.75 m/s with an accuracy of ±10% for flows normal to or approaching at 20 degrees to the centreline of the meter, see Figure 18. Flood plain flows would be assessed based upon the velocity-area principle. A schematic representation of a flow measurement site is shown in Figure 19.

4.3 Maximum level gauges

A comparatively simple and inexpensive method of recording flood levels is the use of maximum water level gauges. These gauges record and store only the maximum level reached. Three basic methods are used :

a) a colour change on treated tape or rod when in contact with the water.

b) a float which leaves an indication of the highest position it has reached and which can only be lowered manually after the passage of a flood event.

c) capture of a column of water by a non-return valve.

The makes of gauge available for the above basic methods of recording flood levels are described fully in HR (1992b). The colour change type of gauge is the most economical and is shown on Plate 10 and in Figure 20.

Regular inspection of the gauge is suggested to ensure that there has been no deterioration of the colour tape, or that condensation has faded the tape. In the event of the passage of a flood the gauge should be surveyed at the earliest opportunity and the tape repositioned or replaced.

4.4 Survey requirements

The following guidelines are proposed for obtaining the survey data and are based mainly on the method adopted by NRA-ST for the roughness study carried out as part of the work for the River Information and Maintenance System :

Channel characteristics

Chose a reach of river where :

- it is essentially straight
- the cross sectional characteristics are uniform with the channel formed in alluvium, gravels etc. but not bedrock. The study reach is not affected by tides.
- the bank vegetation along the reach is uniform, ie composed entirely of grass, scrub or trees etc; the study reach is clear of aquatic weed growth; and
- the flood flows are contained within the channel and flood plains of each surveyed section along the study reach, ie there is no bypass flow at any of the sections.

Care should be taken in choosing the study reach to ensure that it is outside the backwater influence of structures downstream of the area of interest, particularly for determination of the roughness characteristic of the channel as the water surface slope could be influenced by variable backwater effects. In order to estimate the backwater length due to a structure the formula (Samuels, 1989) is

$$L = 0.7 * D_b / S_b \qquad (13)$$

where

D_b	=	bankfull depth.	(m)
S_b	=	bankfull water surface slope.	(m/m)

In the absence of a known water surface slope, the slope obtained from contour information on a 1:25000 Ordnance Survey map of the reach will allow an estimate of the backwater length to be determined.

Common sense is needed in interpreting the influence of structures. If the structure is gated with a high sill level, even though the gates may be open, or a weir with a high crest level then the backwater length will be significant. If the crest level of the weir is low relative to the bankfull level of the channel then the influence of the structure will be less under higher flows. Likewise with an arch bridge, a single span arch bridge with its springing point at bank top will have little backwater influence compared to that exerted by a medieval bridge with several arches in the main channel.

Survey data

Survey five river sections including the flood plains to the flooded limit.

The length of the survey reach is partially determined by the channel characteristic requirements detailed above and controlled to a degree by the natural river slope under study. It is considered that in order to minimise the effect of surveying tolerances the length of reach studied should have a minimum fall of 0.3m.

Maximum level gauges should be located at the upstream and downstream river cross sections, levelled in relative to the section at which they are located and levelled into each other with the separation of the gauges being accurately measured.

5 Discussion on accuracy

The British Standard, BS 3680, Part 3A/ISO 748 (1993), Measurement of Liquid Flow in Open Channels - Velocity Area Methods states "No measurement of a physical quantity can be free from errors which may be associated with either systematic bias caused by errors in the standardizing equipment or a random scatter caused by lack of sensitivity of the measuring equipment". In a similar vein visual assessments of a quantity can be subject to larger errors. Estimates of the accuracy in selection of river channel roughness coefficient, calculation procedure and equipment are shown in Table 5.

Photographic methods of assessing roughness coefficients are subjective. Bailey and Ray (1966) indicate that trained engineers can select roughness coefficients with an accuracy of ±15%. However, the work by Burnham and Davis (1990) indicates a broader margin of uncertainty which increases with Manning's n. Participants at an introductory hydraulics and hydrology course at HR Wallingford, overestimated the measured Manning's roughness coefficient for river reaches on average by 35% using a combination of the photographs in Section 2.2 and Cowans method. Photographs of channels of known resistance are useful in estimating the roughness characteristics of similar channels. Familiarity with the geometry, appearance, and roughness characteristics of these channels will improve the river engineer's ability to select roughness coefficients for other channels.

Uncertainties in the predicted values of bankfull roughness coefficient and bankfull discharge from the channel properties method of flood estimation reflect the uncertainty in the data from which the method was derived. The uncertainties are dominated by the error in the discharge measurement which typically is of the order of ±8% to ±12% for a current meter gauging. The overall uncertainty contains the random and systematic uncertainties associated with measurements

Table 5 Summary of accuracy of roughness selection, calculation procedure and equipment

Method of flood estimation	Applicability	Method	For	Against	Uncertainty (%)
Roughness selection					
1. Photographic method	Estimating roughness characteristics of natural channels.	Guide book.	Quick reference.	Subjective. Limited value unless variation of roughness coefficient with stage documented.	±15.0%
2. Channel properties	Estimating bankfull roughness and bankfull discharge.	Hand calculation.	Objective.	Empirical relationship based on limited data set.	1.1%[1] n_{pred}[2] Std. Dev. 10.1% - 0.13%[1] Q_{pb}[2] Std. Dev. 9.8%
Calculation procedure					
1. Divided channel method	Derivation of rating curve for a straight channel at an ungauged site. Channel sinuosity < 1.02.	Hand calculation/Spreadsheet.	Accurate once calibrated. Over estimates shallow out-of-bank flows.	Requires values of channel and flood plain roughness coefficients. No firm conceptual foundation.	8.8%[3] Std. Dev. 10.2% (Lab.) - 1.0%[3] Std. Dev. 6.6% (Field)
2. Lateral Distribution Method	Derivation of rating curve for a straight channel at an ungauged site. Channel sinuosity < 1.02.	Computer based calculation.	Accurate once calibrated. Accurately estimates out-of-bank flows.	Requires values of channel and flood plain roughness coefficients and non dimensional eddy viscosity.	3.5%[3] Std. Dev. 4.1% (Lab.) 1.2%[3] Std. Dev. 6.7% (Field)
3. Flood Channel Facility Method	Derivation of rating curve for straight and skewed channels at an ungauged sites. Channel sinuosity < 1.02.	Hand calculation/Spreadsheet. Iterative calculation.	Accurate once calibrated. Accurately estimates out-of-bank flows.	Requires values of channel and flood plain roughness coefficients.	- 2.0%[3] Std. Dev. 3.8% (Lab.) - 2.8%[3] Std. Dev. 7.6% (Field)
4 James and Wark Method	Derivation of rating curve for a meandering channel at an ungauged site.	Hand calculation/Spreadsheet.	Accurate once calibrated. Accurately estimates out-of-bank flows. Data used to verify method covered limited range of conditions.	Requires values of channel and flood plain roughness coefficients.	- 2.1%[3] Std. Dev. 9.7% (Lab.) - 2.0%[3] Std. Dev. 1.7% (Field)
5. Rating curves		Extrapolation of in-bank rating to determine out-of-bank flows.			
Equipment					
1. Peak velocity meter	Measurement of flood plain flow velocity.	Vane meter with clutch bearing.	Read after passage of flood event. Calibration maintained for flows approaching meter at up to 20° to meter centreline.	Peak velocity may not necessarily coincide with maximum flood depth. Limited flow velocity range calibration (0.05 m/s to 0.75 m/s).	±10%
2. Maximum level gauges	Measurement of water surface level	Gauge with water sensitive tape.	Simple, inexpensive.	Deterioration of water sensitive tape with time.	± 0.004m Based on root sum square of ± 3mm uncertainty in water surface level and gauge datum.

Note : [1] Average error in predicted roughness coefficient and discharge, eg Error = average of 100 * $((n_{predicted} - n_{observed})/n_{observed}$. Standard deviation in average.
[2] n_{pred} = predicted bankfull roughness coefficient : Q_{pb} = predicted bankfull discharge.
[3] Mean error and standard deviation in mean.

of the individual parameters, eg width, depth, time of exposure of current meter and current meter calibration.

The calculation procedures for determining the flow in straight and meandering compound channels were developed using laboratory data from the SERC Flood Channel Facility at HR Wallingford. The average uncertainty in discharge calculated using the procedures is ±7%. The random and systematic uncertainties associated with measurements of the individual parameters are still present but the higher accuracy was achieved due to the controlled laboratory environment. The accuracy in predicting discharge at field sites using the calculation procedures was achieved by using measured data from the sites to calibrate the procedures. Maintaining the accuracy in calculated discharge when applying the procedures at an ungauged field site will require a knowledge of the roughness coefficients for the channel and flood plain at the site.

The example in Section 3.2 indicated that uncertainties of the order of 10 to 20% could be expected in the use of the regression equations of Section 2.3 with single section rather than reach average cross-section parameters.

Field measurement of flood plain velocity for flood events in the UK is very limited. The peak velocity meter provides a simple, inexpensive and safe means of obtaining data. The uncertainty in velocity reading of ±10% for flow velocities between 0.05m/s and 0.075m/s will enable flood plain flows to be predicted with a similar degree of certainty to that associated with a current metering exercise. The meter calibration holds for flow depths up to 0.3m and for flows aligned normal to and at up to 20° to the meter vane. The velocities associated with deeper flows can be measured by positioned several meters through the depth. Use of the meter also relies on the assumption that the maximum velocity on the flood plain coincides with the maximum depth and this assumption may contribute additional uncertainty in "flashy" rivers. However, the ability to obtain data enabling more accurate estimates of flood plain flow and an improvement in understanding of flood plain activity outweighs the uncertainty associated with the instrument readings.

6 Acknowledgements

The helpful co-operation of the ten NRA Regions in England and Wales is gratefully acknowledged. The help afforded and the excellent data provided by the staff of the Severn-Trent Region of the National Rivers Authority has been of particular value. Most of the Regions have been visited by members of the research team, and a considerable amount of information has been provided for use in this research project.

The series of colour photographs in Section 2.2 are published with the permission of the Severn-Trent region of the National Rivers Authority.

The helpful and constructive comments from the NRA regions in respect of the draft report are acknowledged.

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Figures

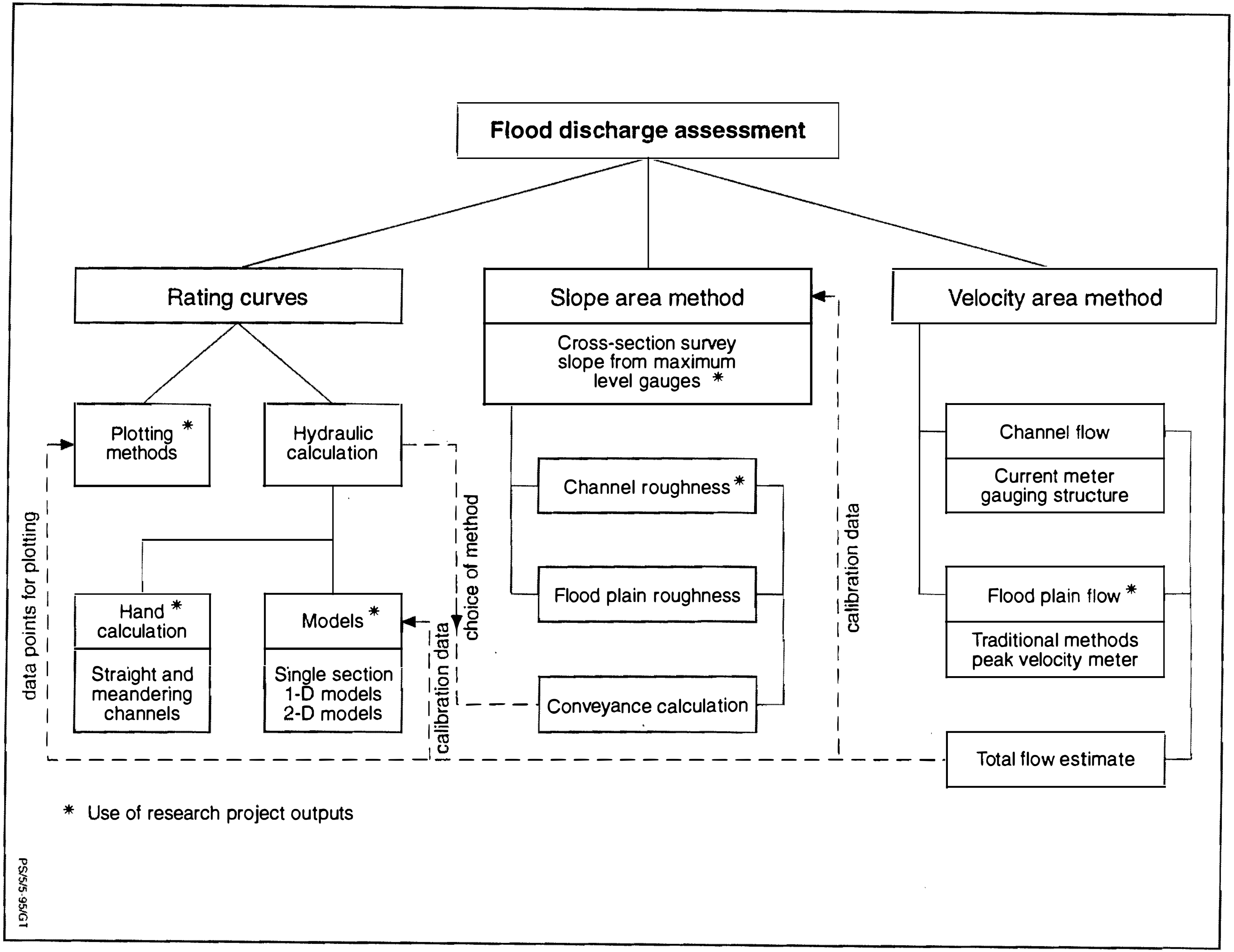


Figure 1 Research contribution to the assessment procedure.

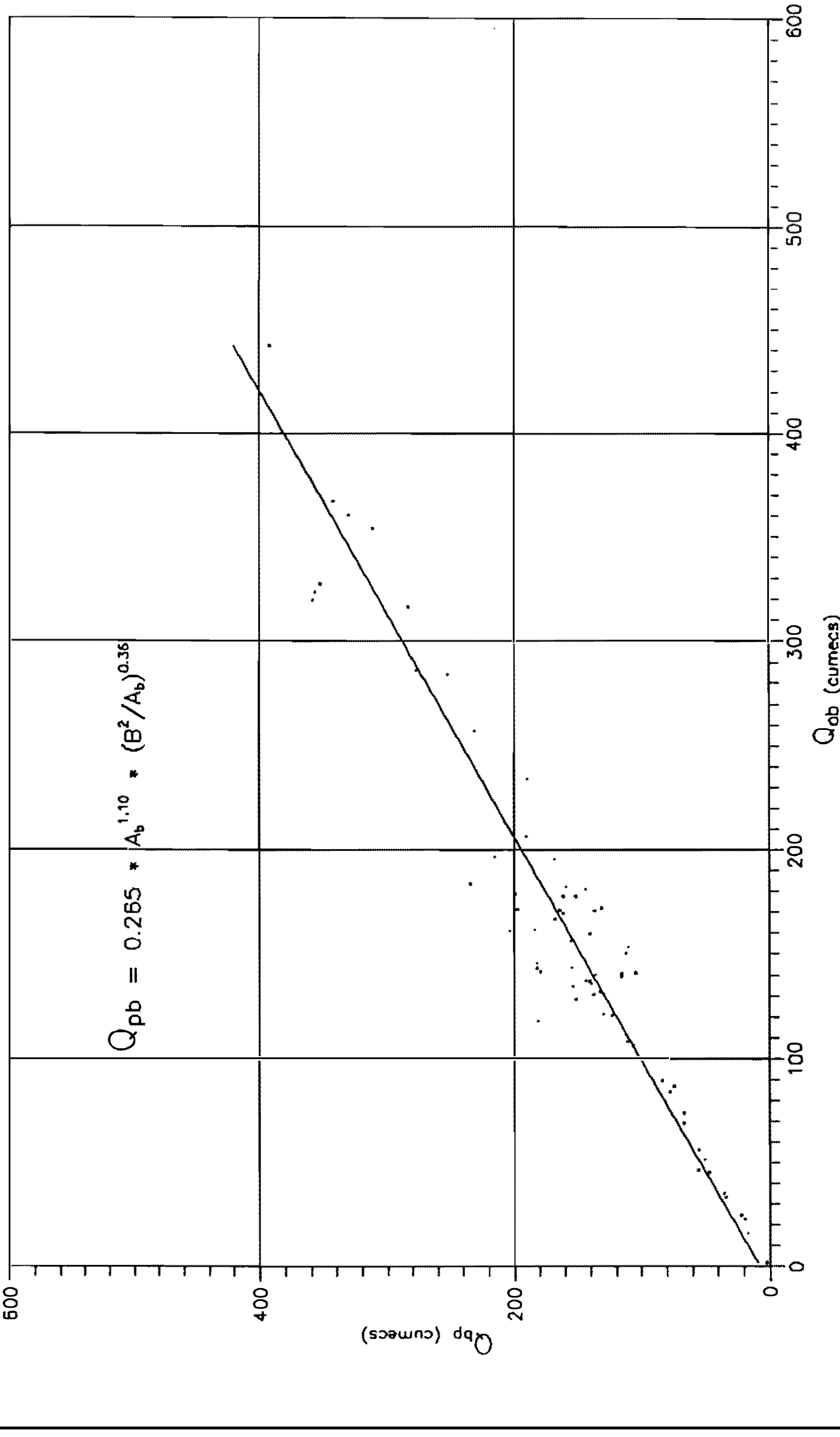


Figure 2 Predicted bankfull discharge. Equation 3.

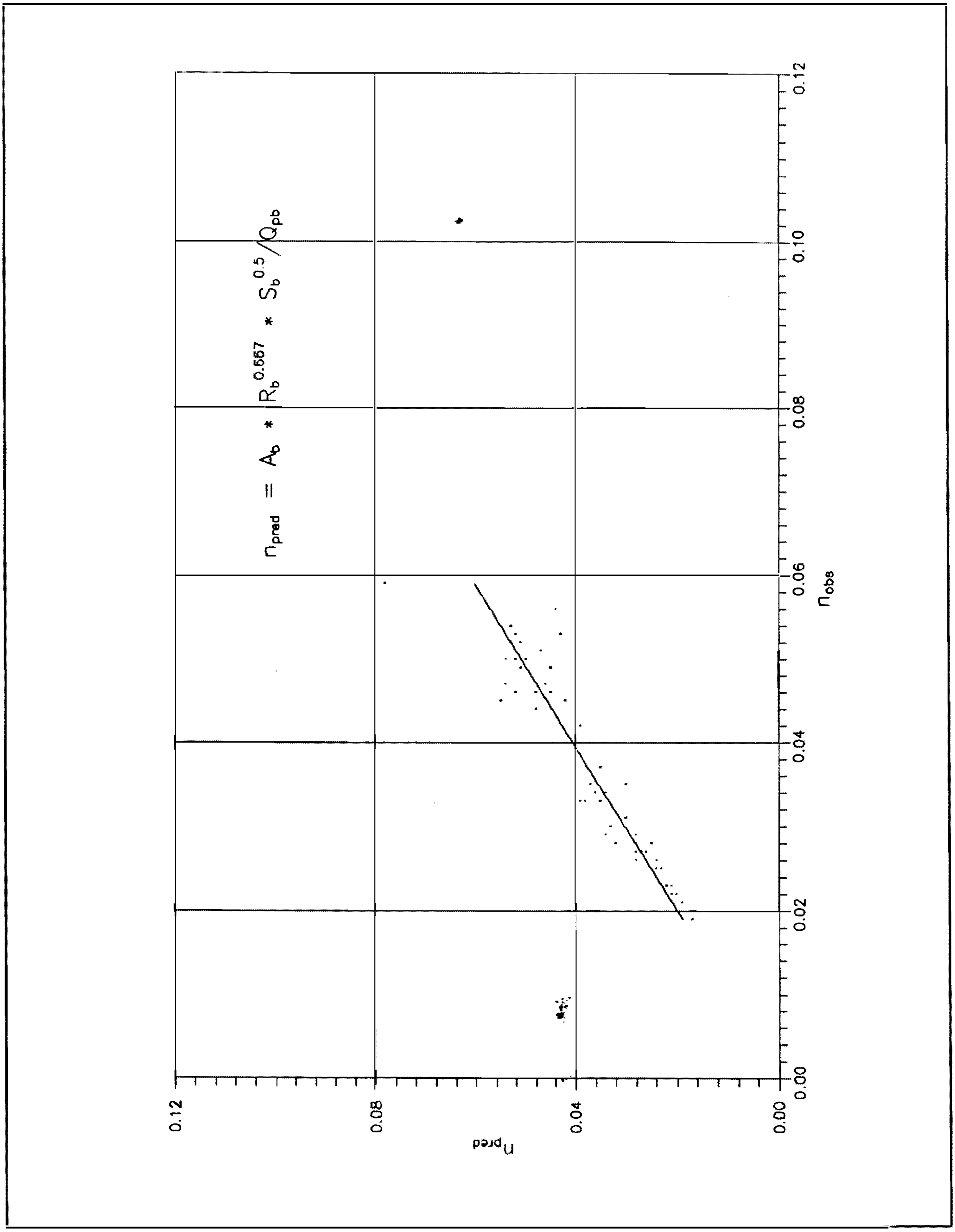


Figure 3 **Predicted bankfull roughness coefficient. Equation 4.**

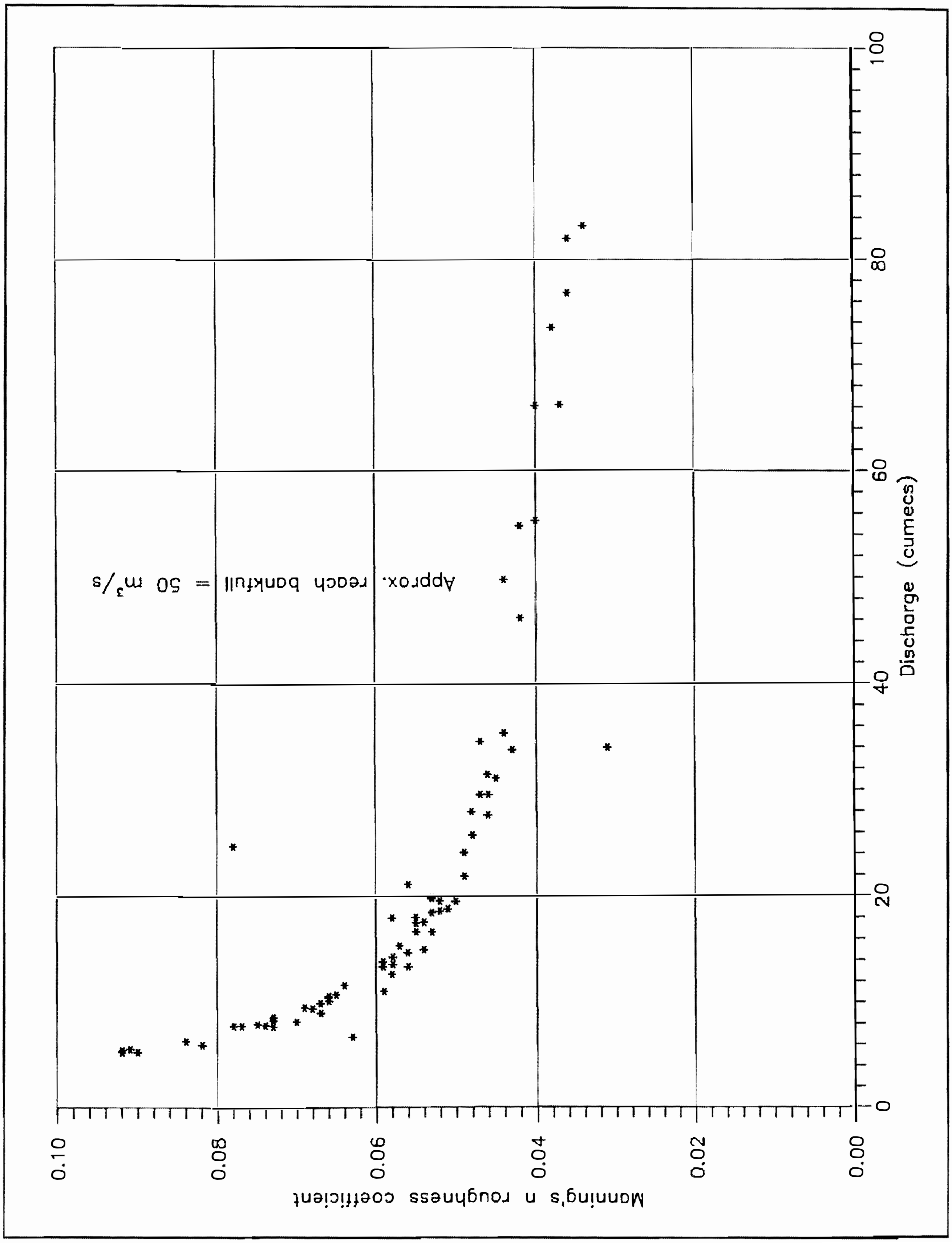


Figure 4 **River Tanat at Llanyblodwel. Manning's roughness coefficient (NRA-ST, 1993).**

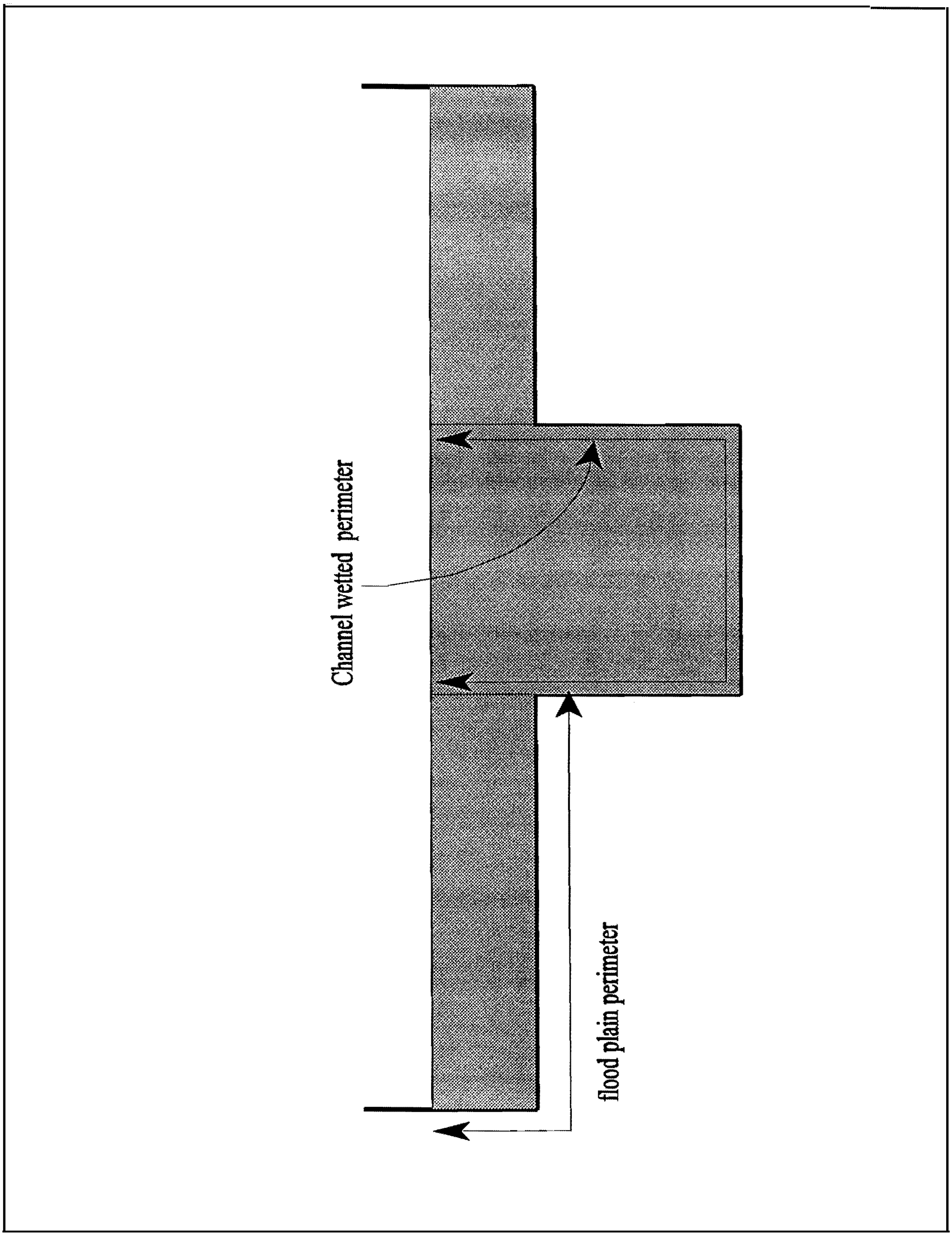


Figure 5 The Divided Channel Method. DCM2.

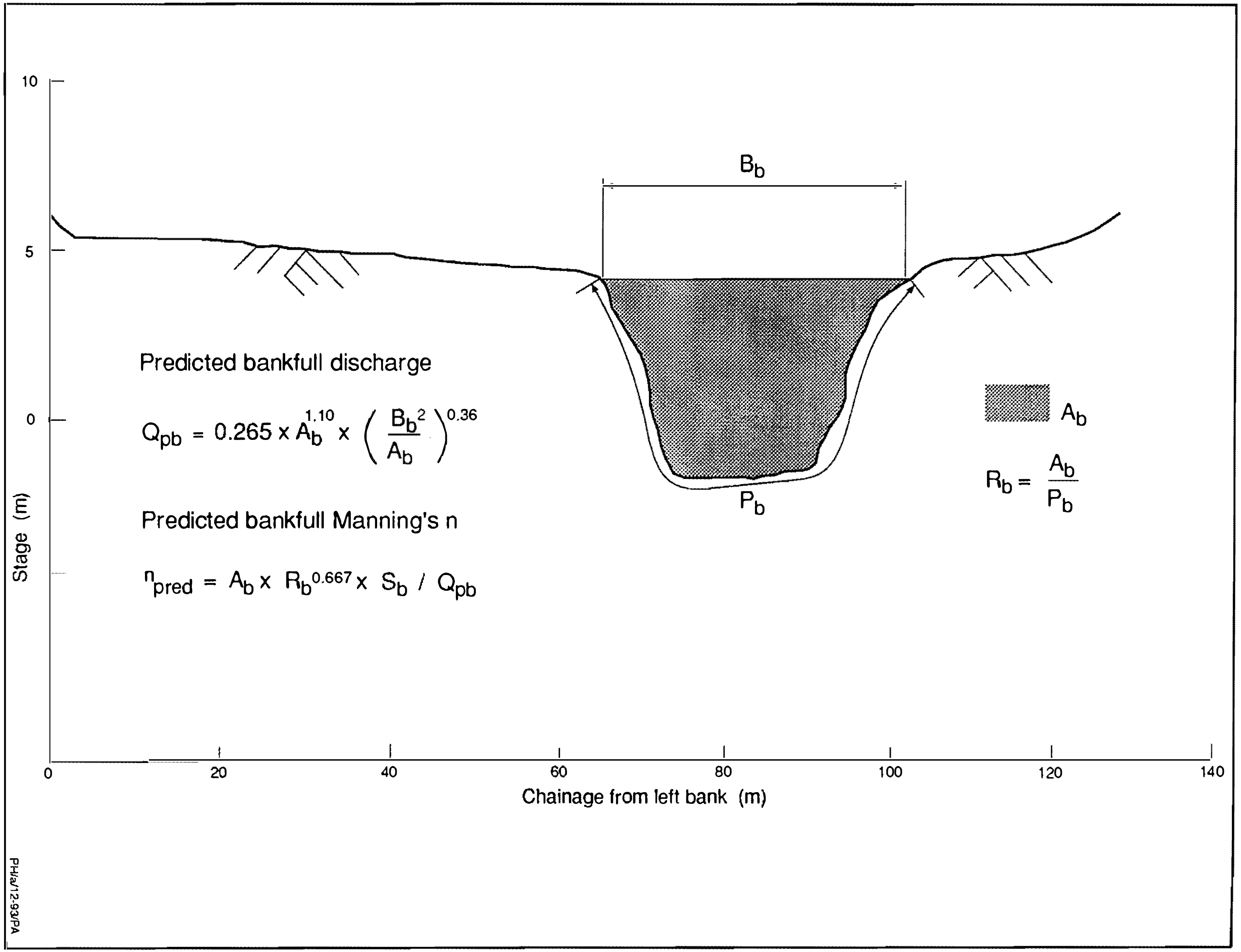
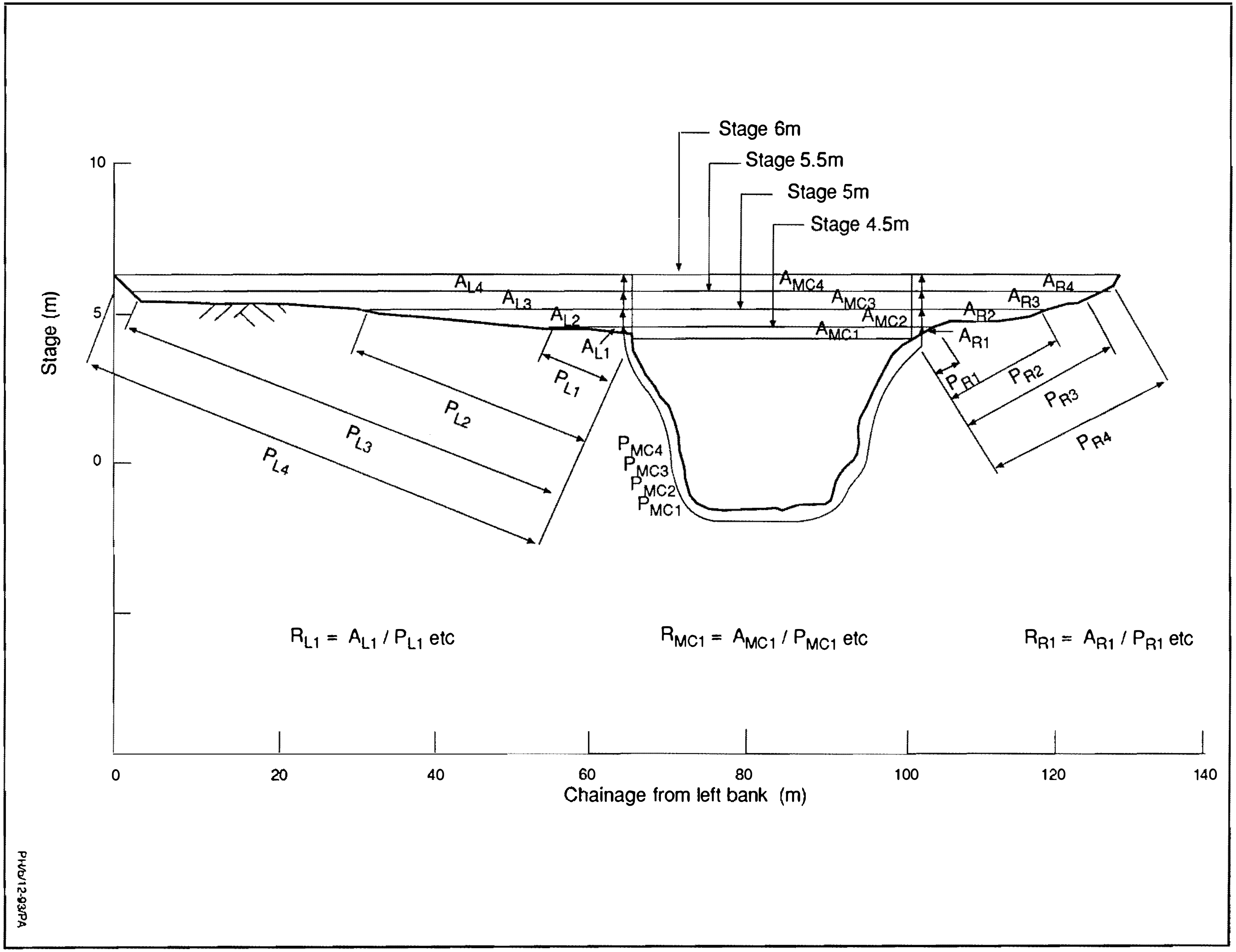


Figure 6 River Severn at Montford. Bankfull channel definition.

Figure 7 River Severn at Montford. Overbank flow channel definition.



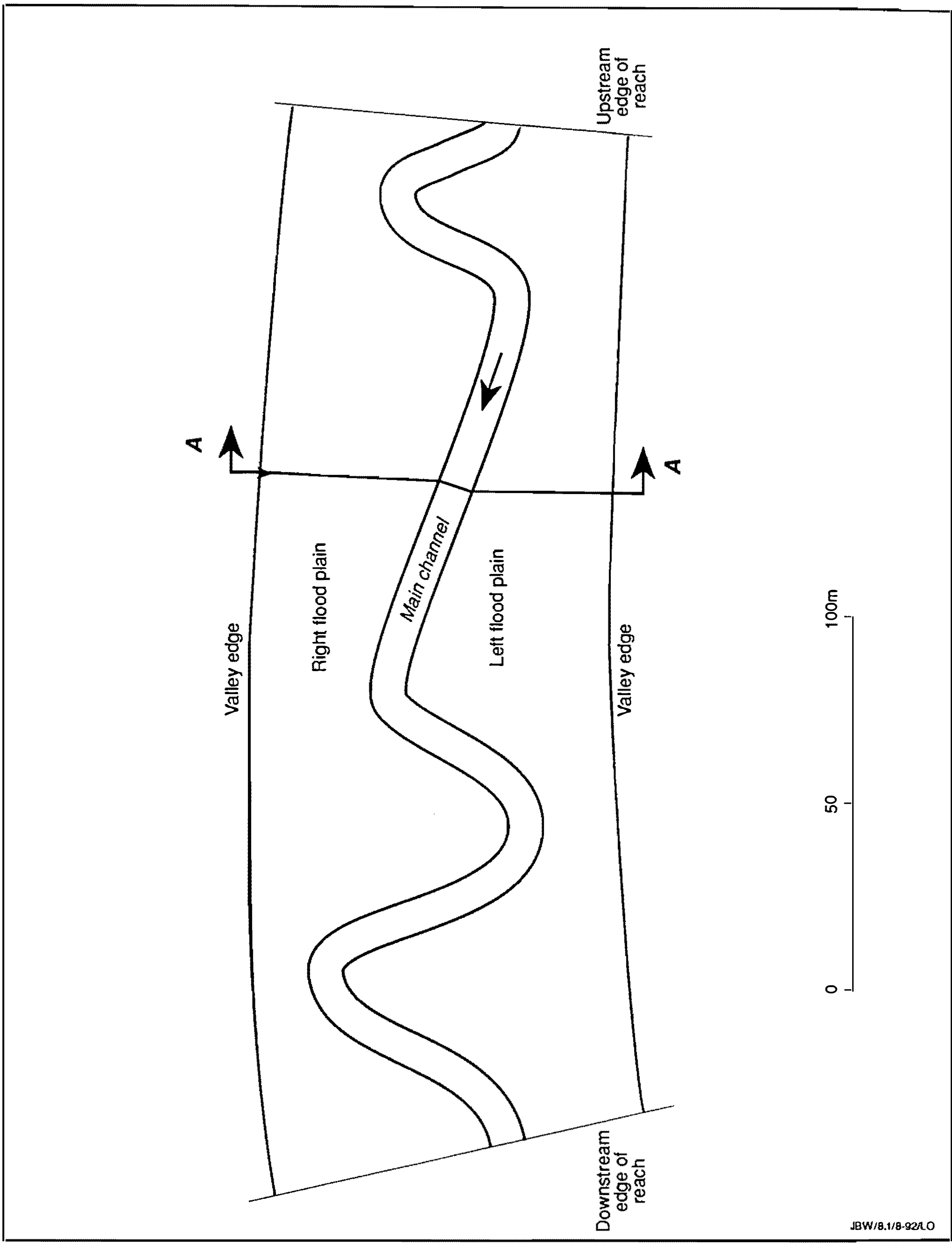


Figure 8 Plan of problem reach.

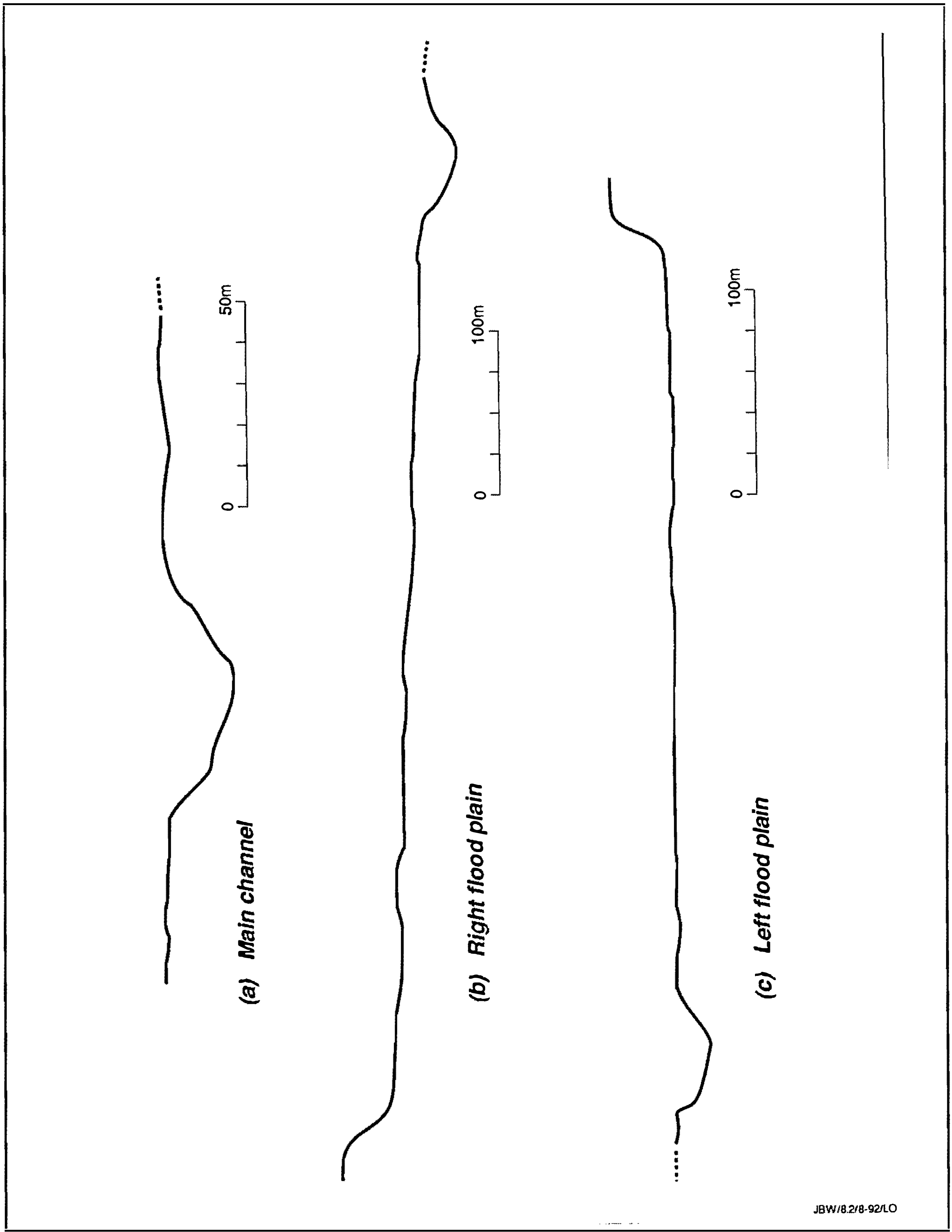


Figure 9 Cross section (A-A) through problem reach (looking upstream).

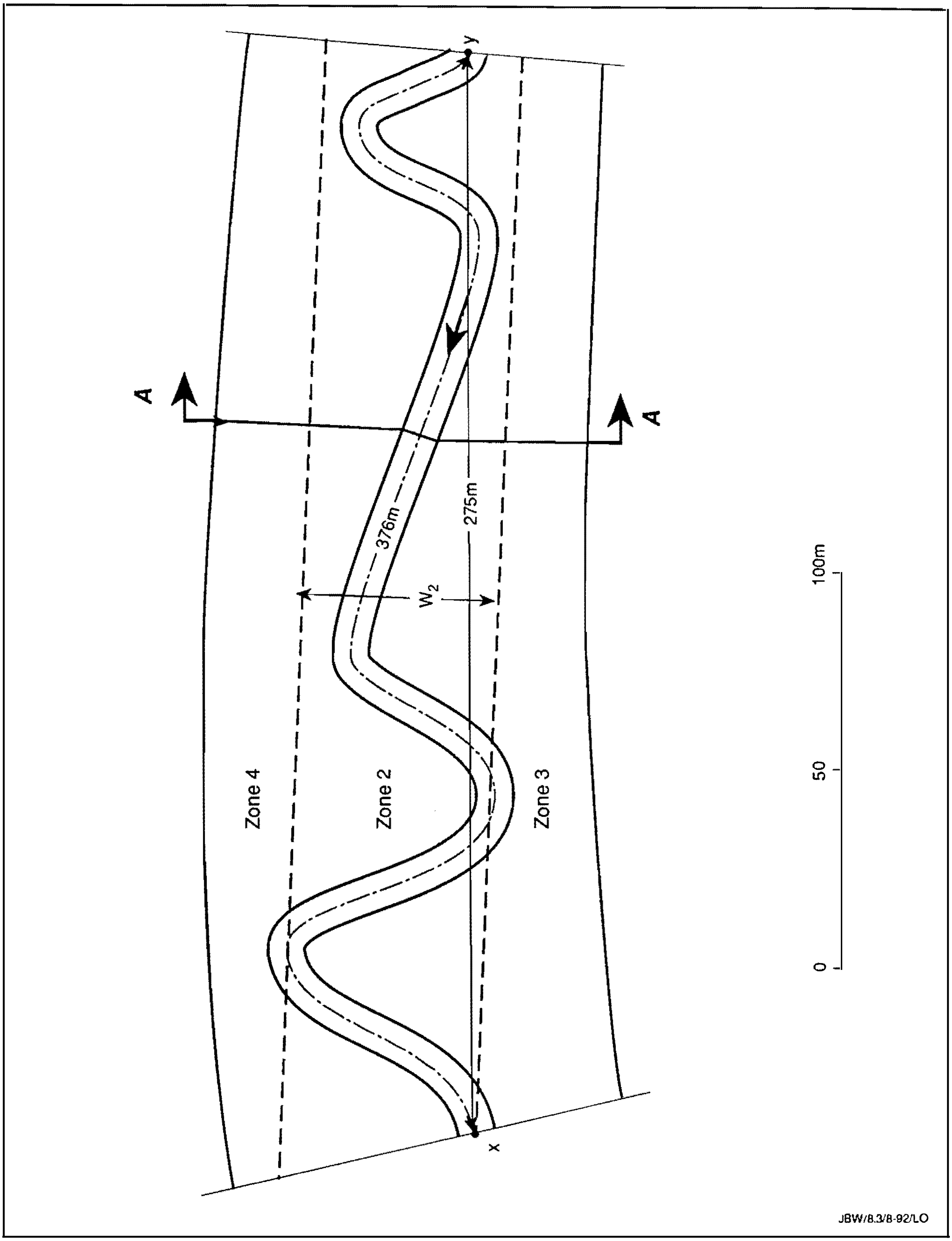


Figure 10 Plan of problem reach with zones shown

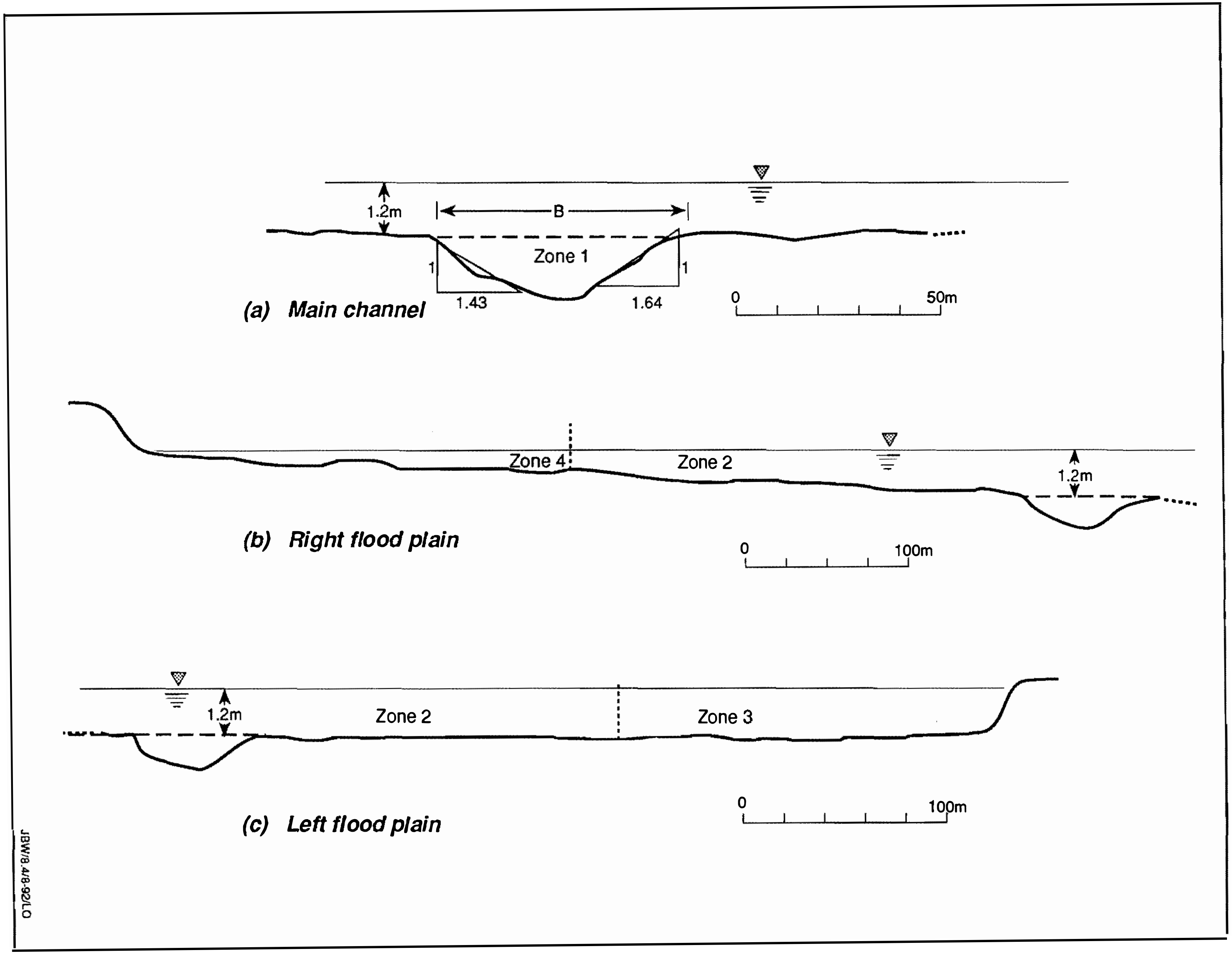
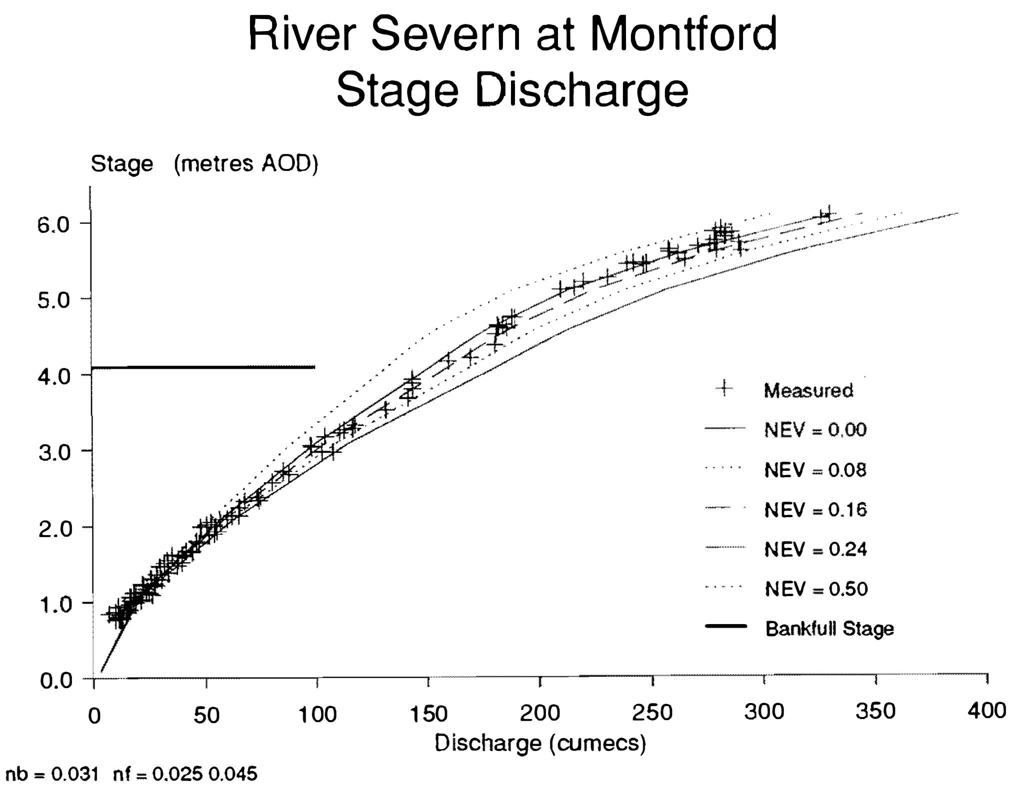


Figure 11 Cross section through problem reach (looking upstream).



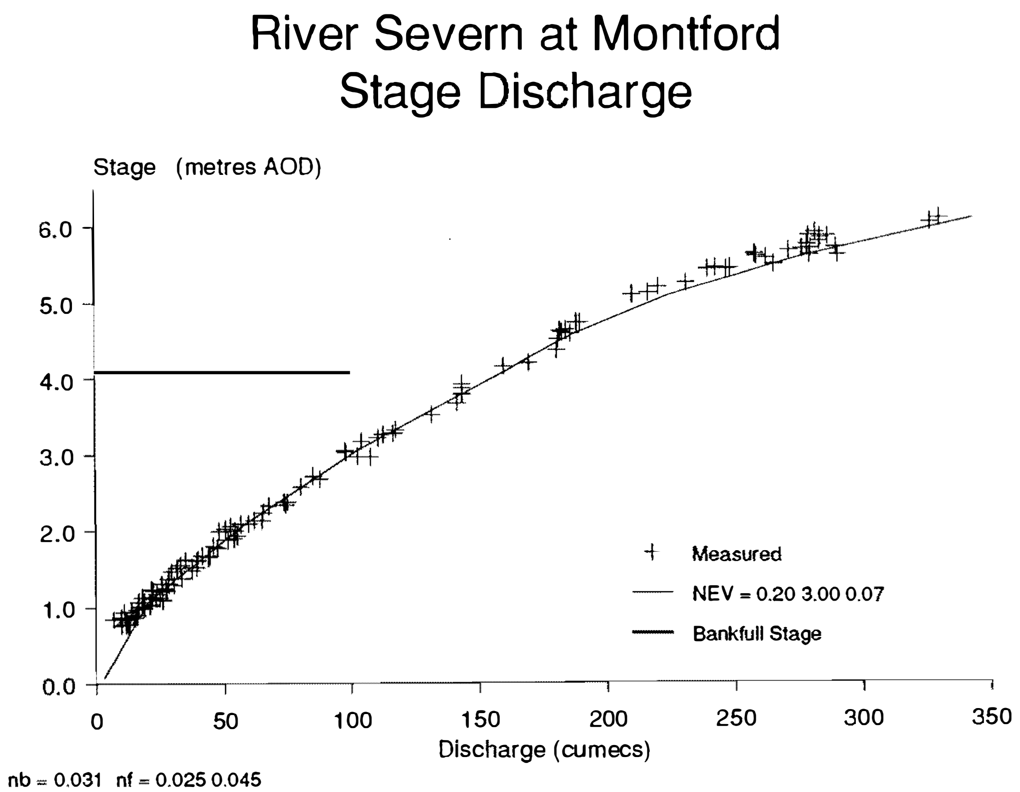


Figure 12 Stage discharges. River Severn at Montford.

Figure 13 Unit flow profiles. River Severn at Montford.

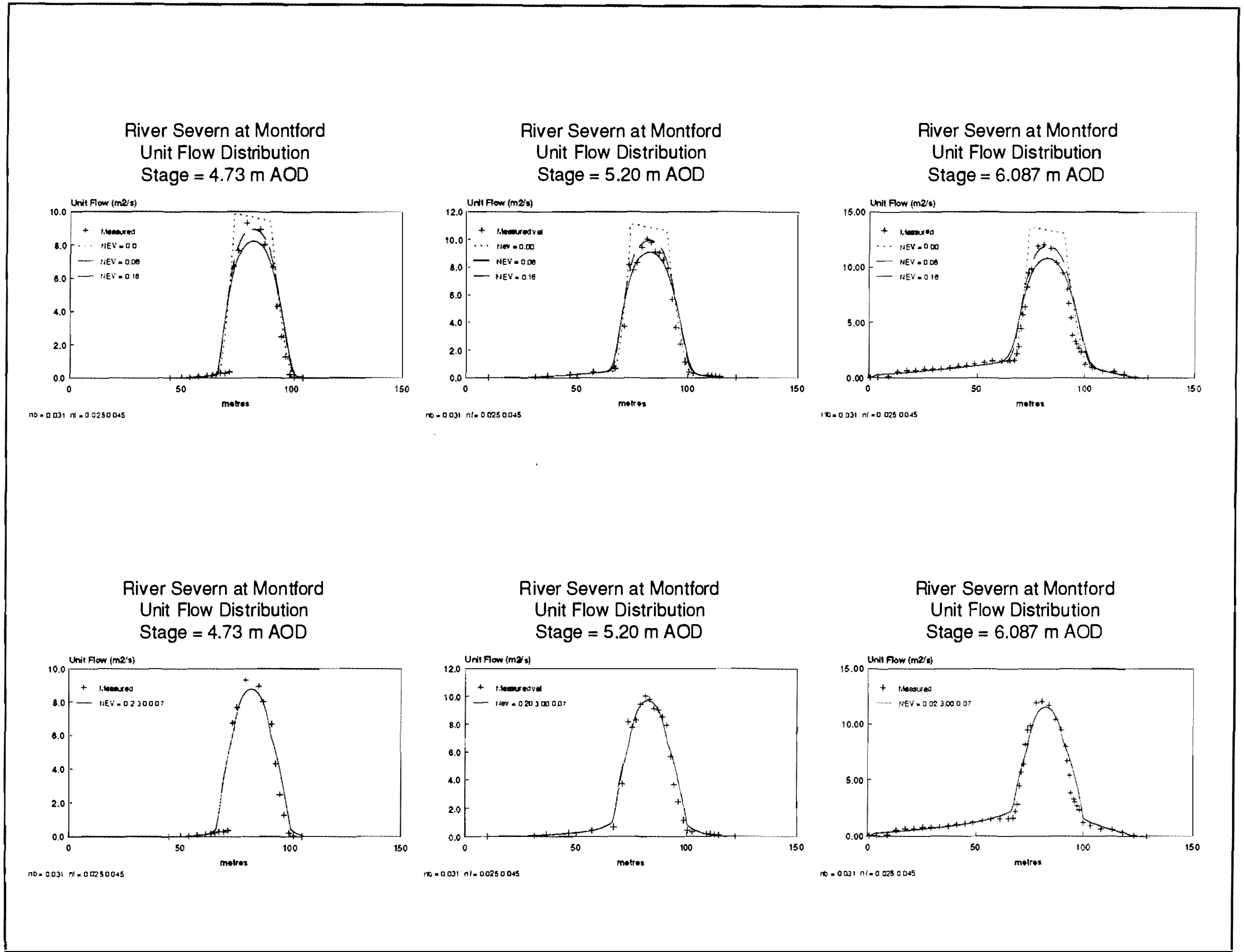
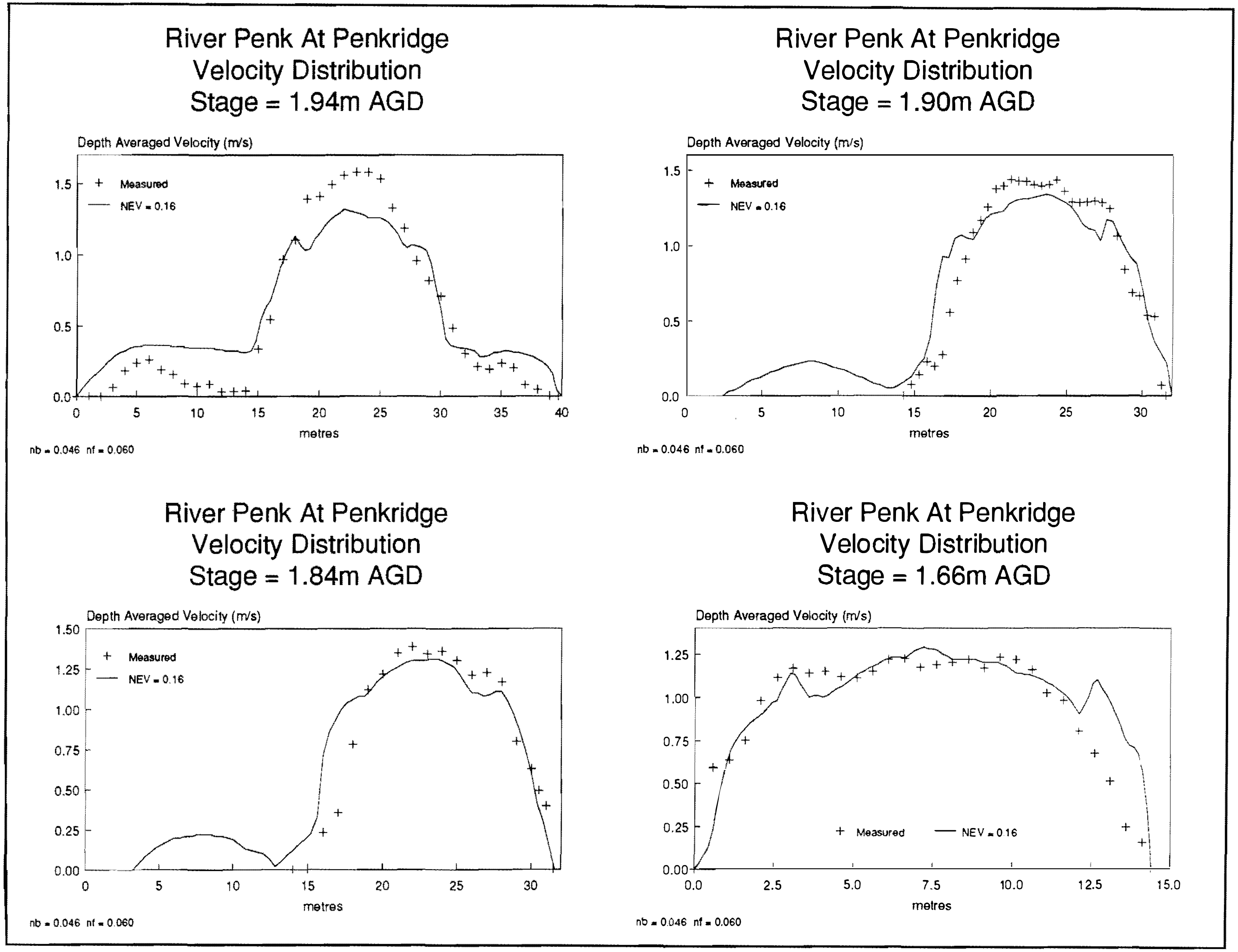


Figure 14 Velocity profiles. River Penk at Penkridge.



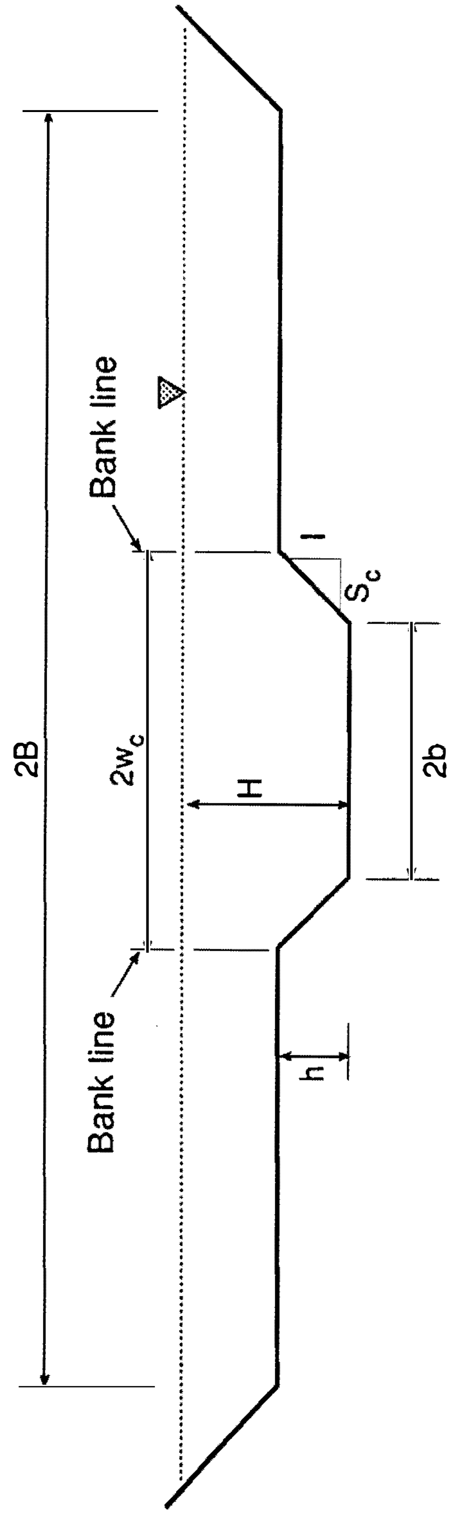


Figure 15 **Compound channel cross-section with definition of variables.**

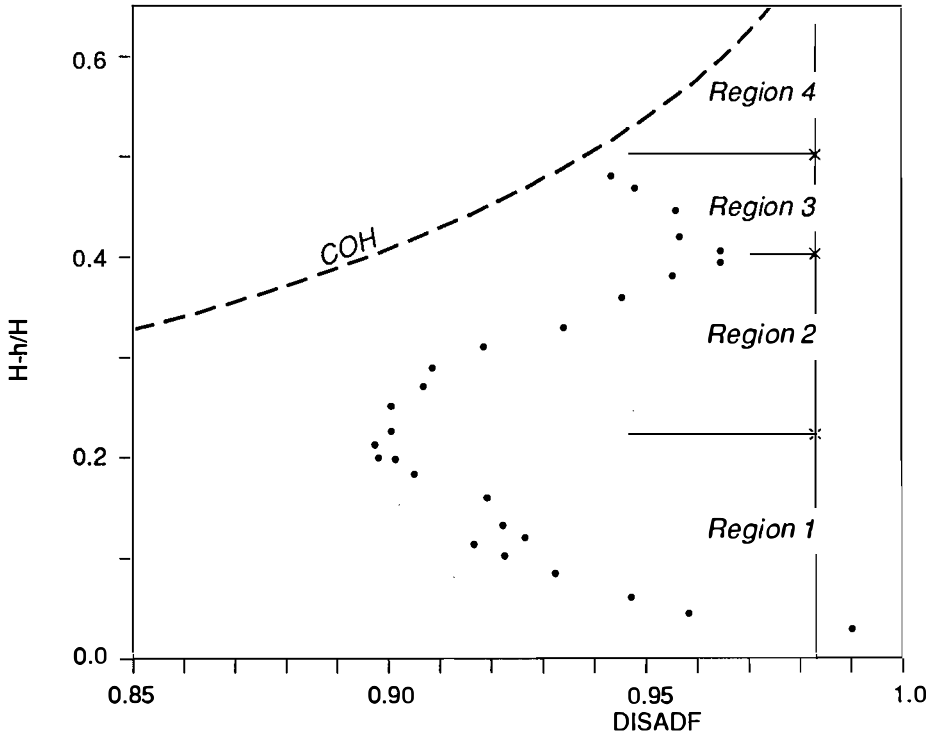


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Figure 16 Regions of flow behaviour.

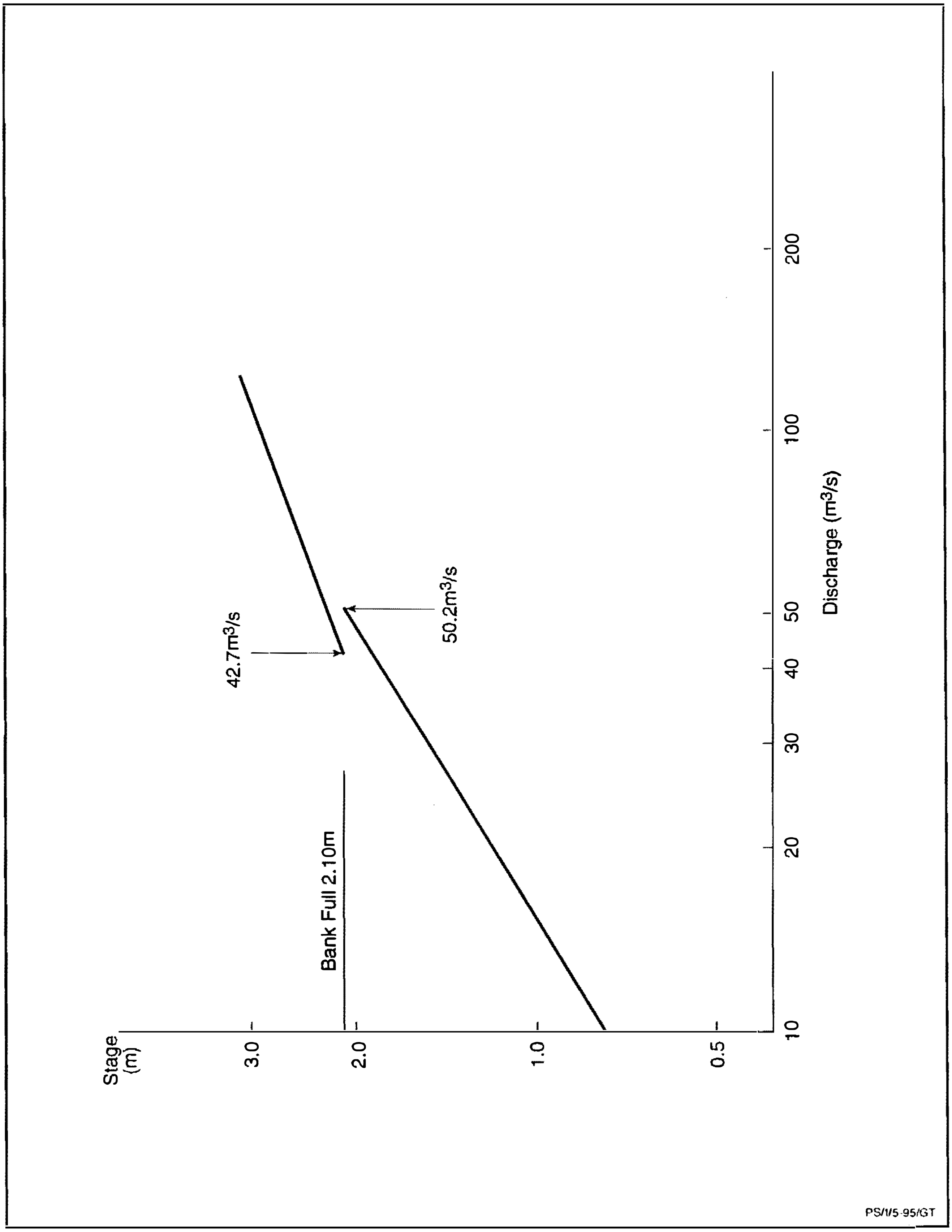


Figure 17 **Rating curve for the River Culm at Wood Mill.**

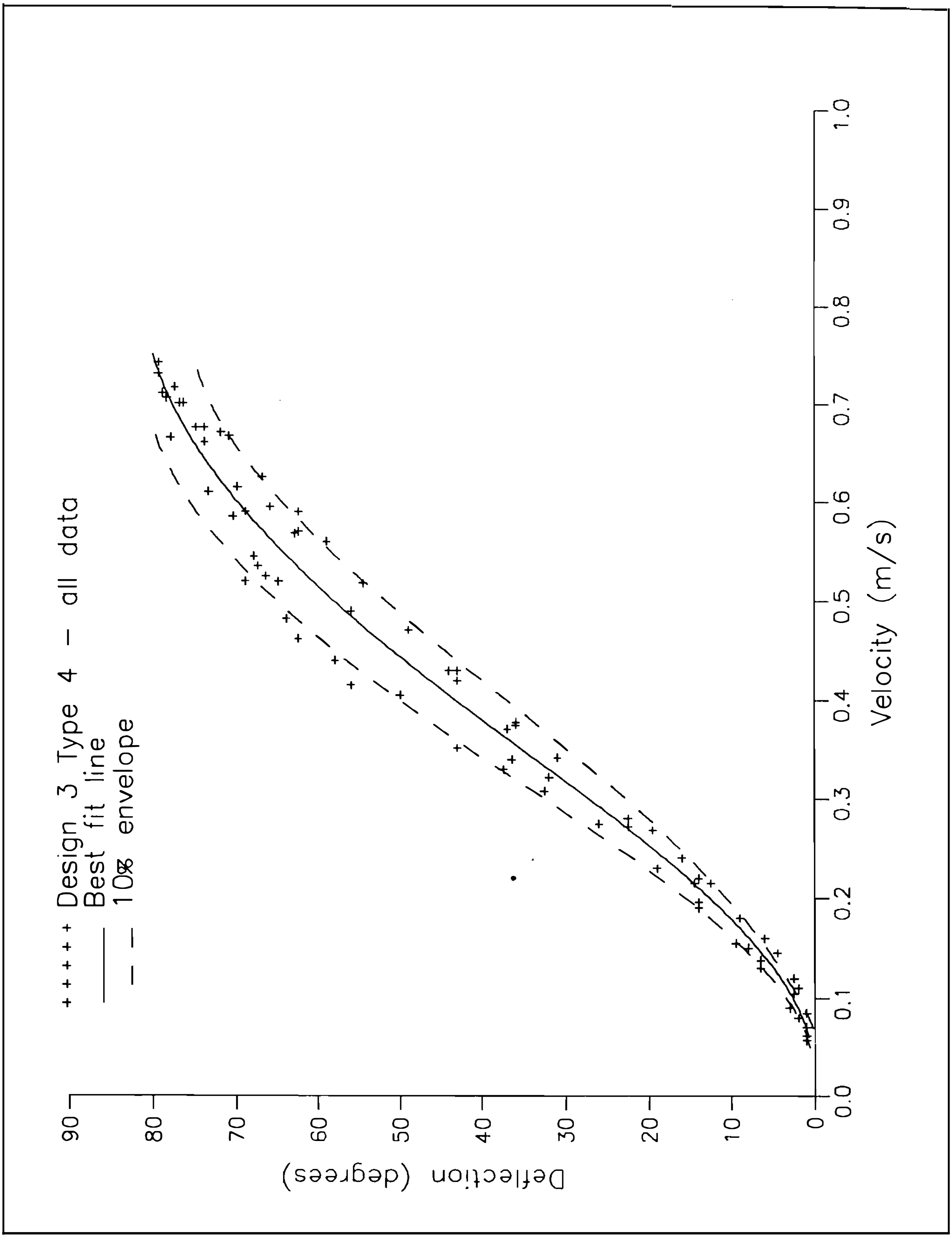
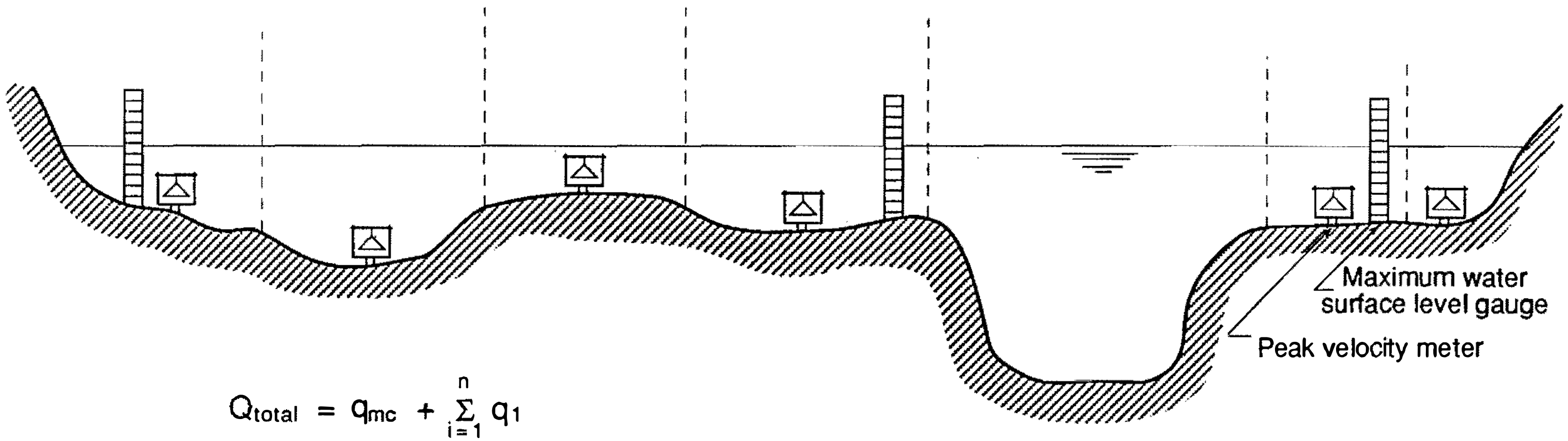


Figure 18 Design 3 - Type 4, meter calibration.

A_1	A_2	A_3	A_4	q_{mc} determined from flow measurement structure, current metering or computational analysis	A_5	A_n
Θ_1	Θ_2	Θ_3	Θ_4		Θ_5	Θ_n
$\bar{v}_1$	$\bar{v}_2$	$\bar{v}_3$	$\bar{v}_4$		$\bar{v}_5$	$\bar{v}_n$
$q_1 = A_1 v_1$	$q_2 = A_2 v_2$	$q_3 = A_3 v_3$	$q_4 = A_4 v_4$		$q_5 = A_5 v_5$	$q_n = A_n v_n$



$$Q_{total} = q_{mc} + \sum_{i=1}^{n} q_1$$

A = flow area (m^2)

Θ = vane deflection (degrees)

$\bar{v}$ = average flow velocity (m/s)

q = segment discharge (m^3/s)

Figure 19 Flow measurement site.

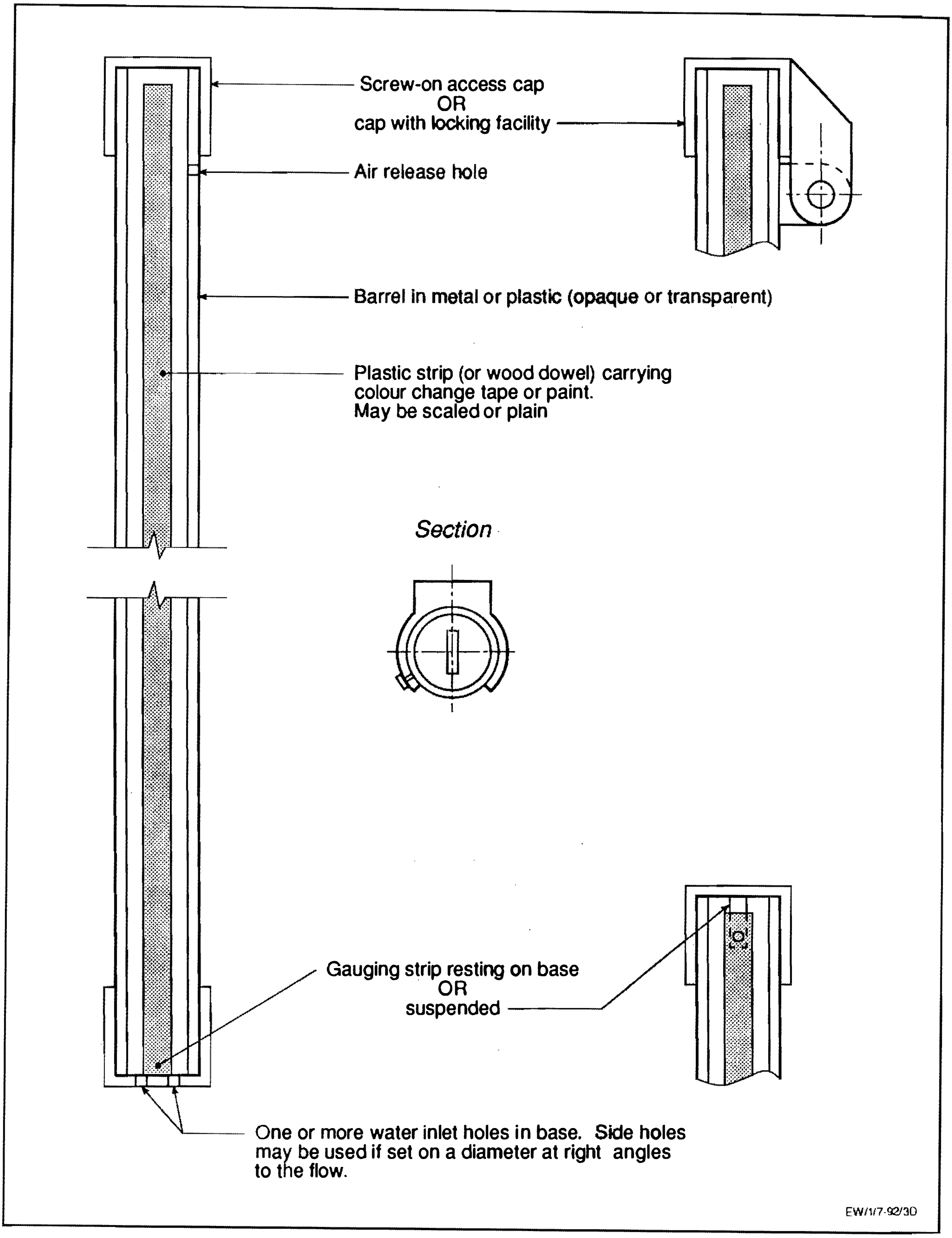
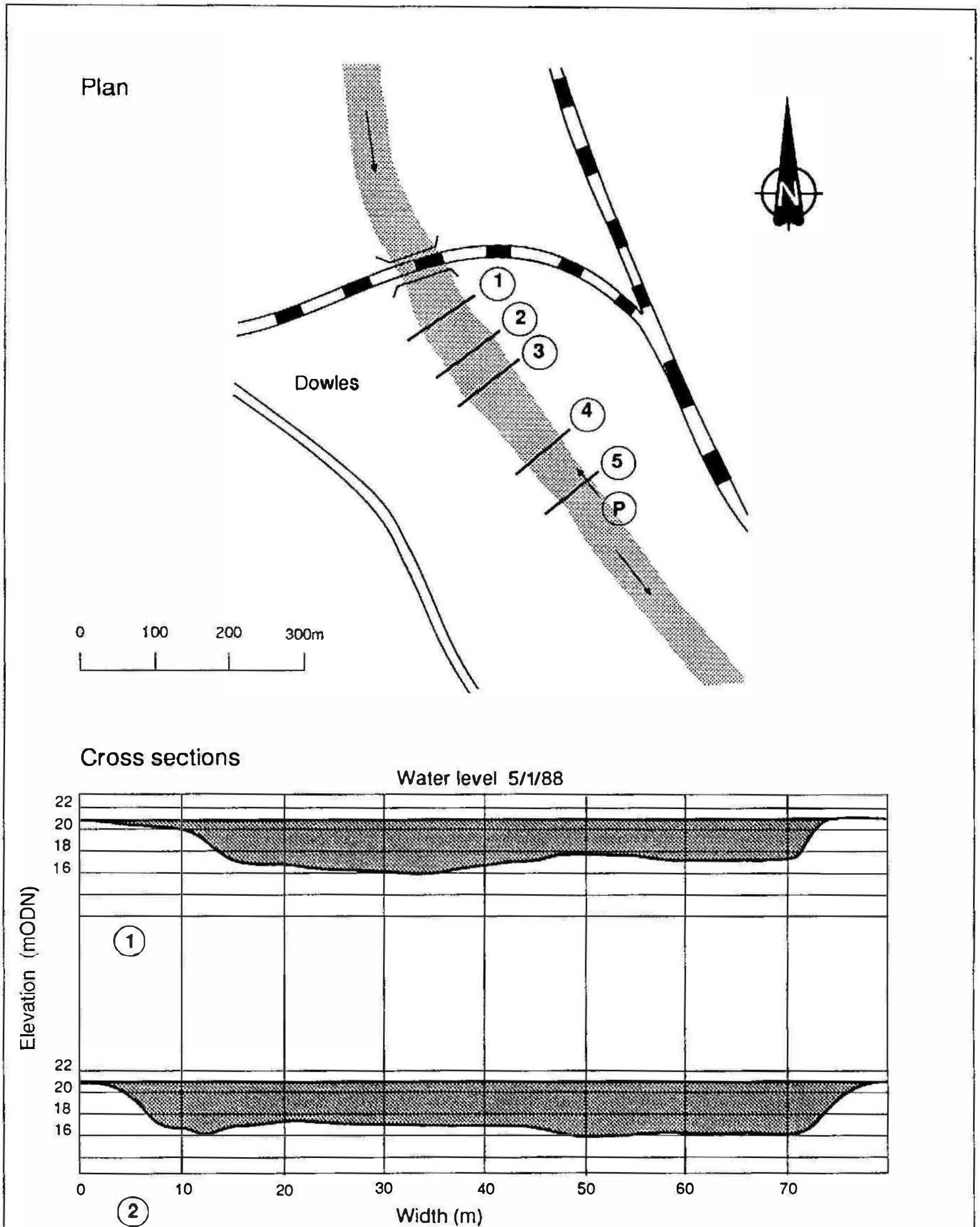


Figure 20 **Typical layout of a colour-change gauge.**

Plates

Plate 1

River Severn at Bewdley



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Plan and cross sections, River Severn at Bewdley

n =

Bankfull hydraulic and geometric characteristics 5/1/88

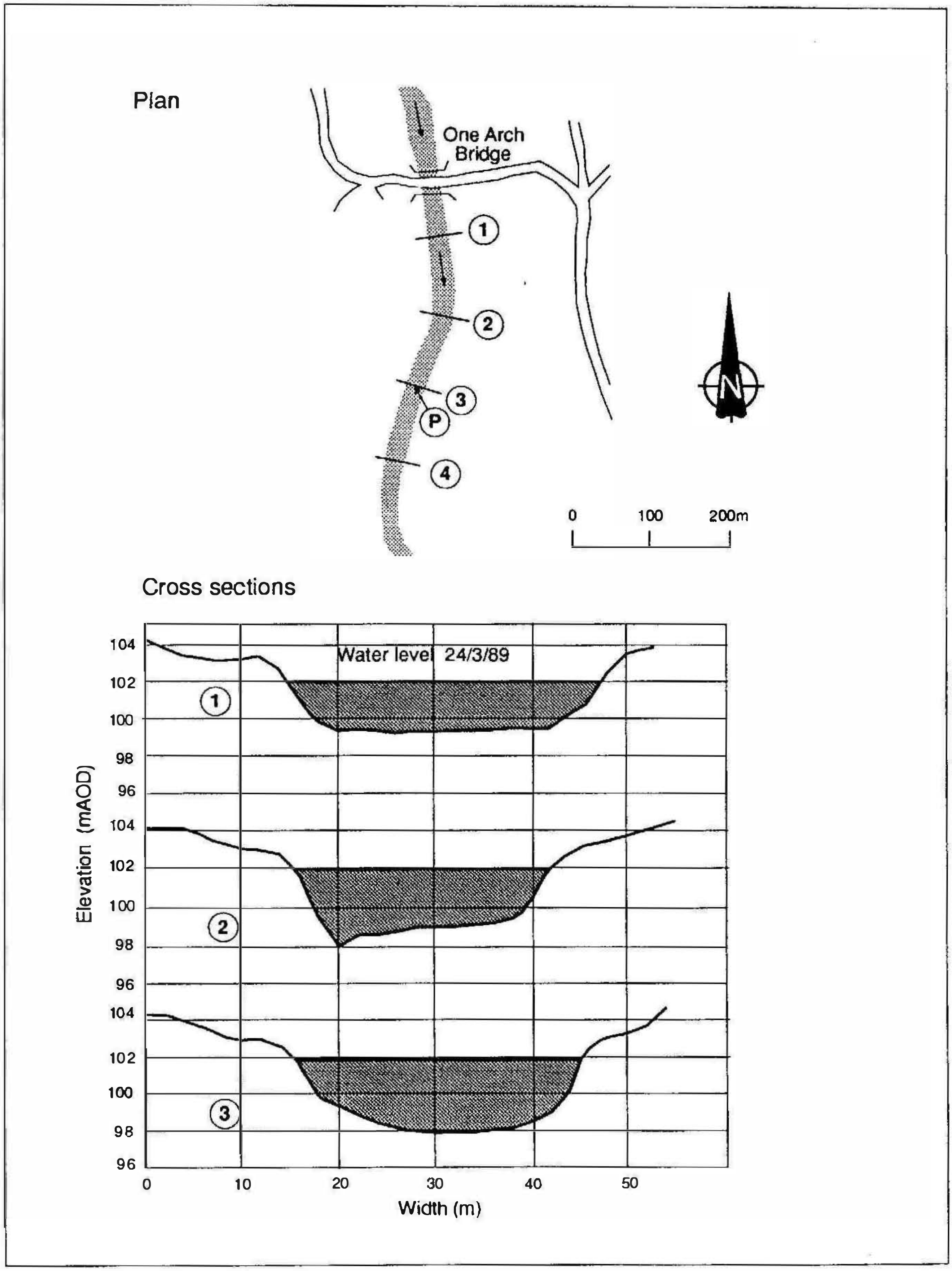
Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000203 (1:4926)
Average cross sectional area	=	249 m^2
Average flow width	=	78m (information available for xs 1 and 2)
Average hydraulic radius	=	3.24 m

Description of channel

Bed material gravel at upstream end, otherwise unknown. Banks grass with scattered willow, alder and hawthorn trees; undergrowth of nettle and brambles. Left and right flood plains are short grass pasture.

Plate 2

River Vyrnwy at Llanymynech



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Plan and cross sections, River Derwent at Chatsworth

n =

Bankfull hydraulic and geometric characteristics 24/3/89

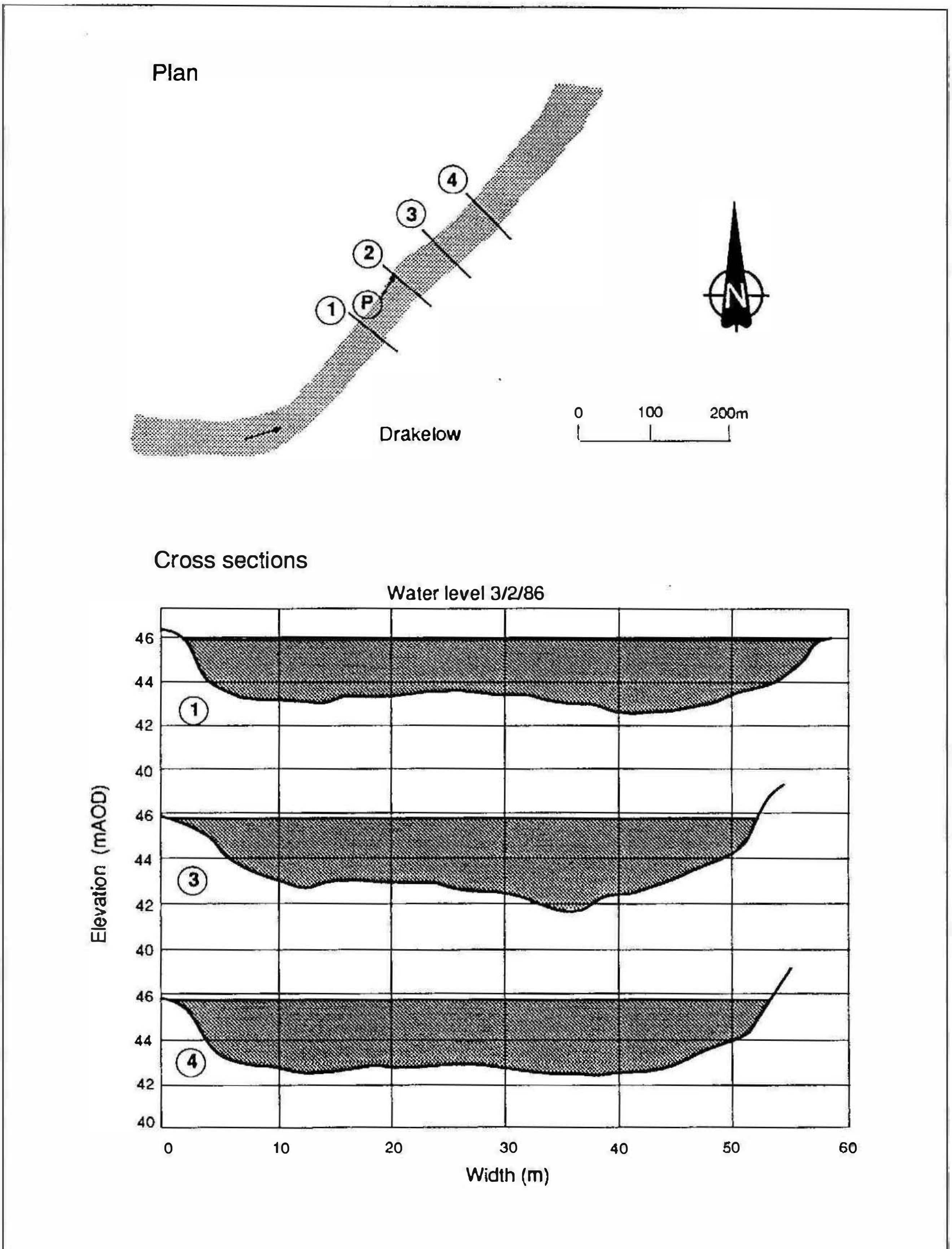
Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000695 (1:1439)
Average cross sectional area	=	77.59 m^2
Average flow width	=	29 m
Average hydraulic radius	=	2.42 m

Description of channel

Bed material gravel with shallow rock step. Banks grass with mature alder, sycamore and ash trees. Left and right flood plains are short grass pasture.

Plate 3

River Severn at Montford



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Plan and cross sections, River Trent at Drakelow

n =

Bankfull hydraulic and geometric characteristics 3/2/86

Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000437 (1:2288)
Average cross sectional area	=	142.5 m^2
Average flow width	=	54 m
Average hydraulic radius	=	2.62 m

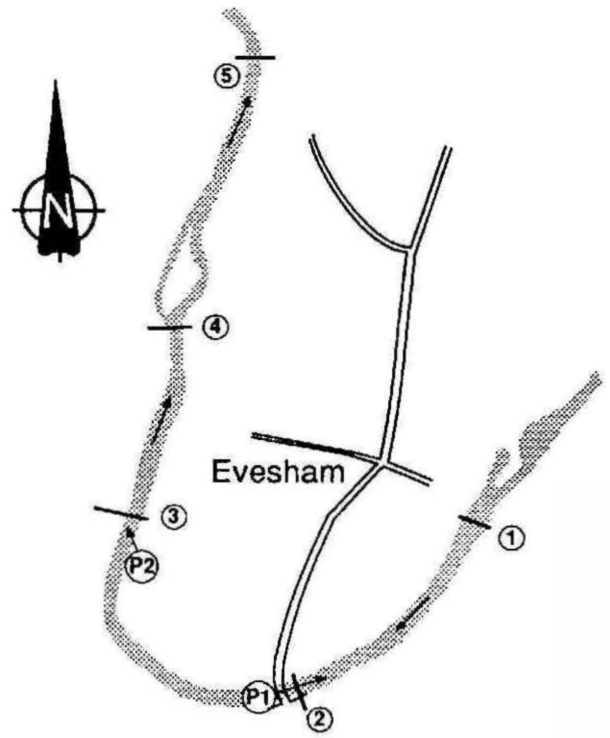
Description of channel

Bed material unknown. Left bank grass with horse chestnut trees at downstream limit of reach. Right bank with horse chestnut and beech trees. Left flood plain is a golf course with occasional oak, yew, hawthorn, horse chestnut and poplar tree coppice. Right bank formed by steep wooded bank lowering to flood plain at downstream limit with beech and chestnut trees with bank grass undergrowth.

Plate 4

River Trent at Drakelow

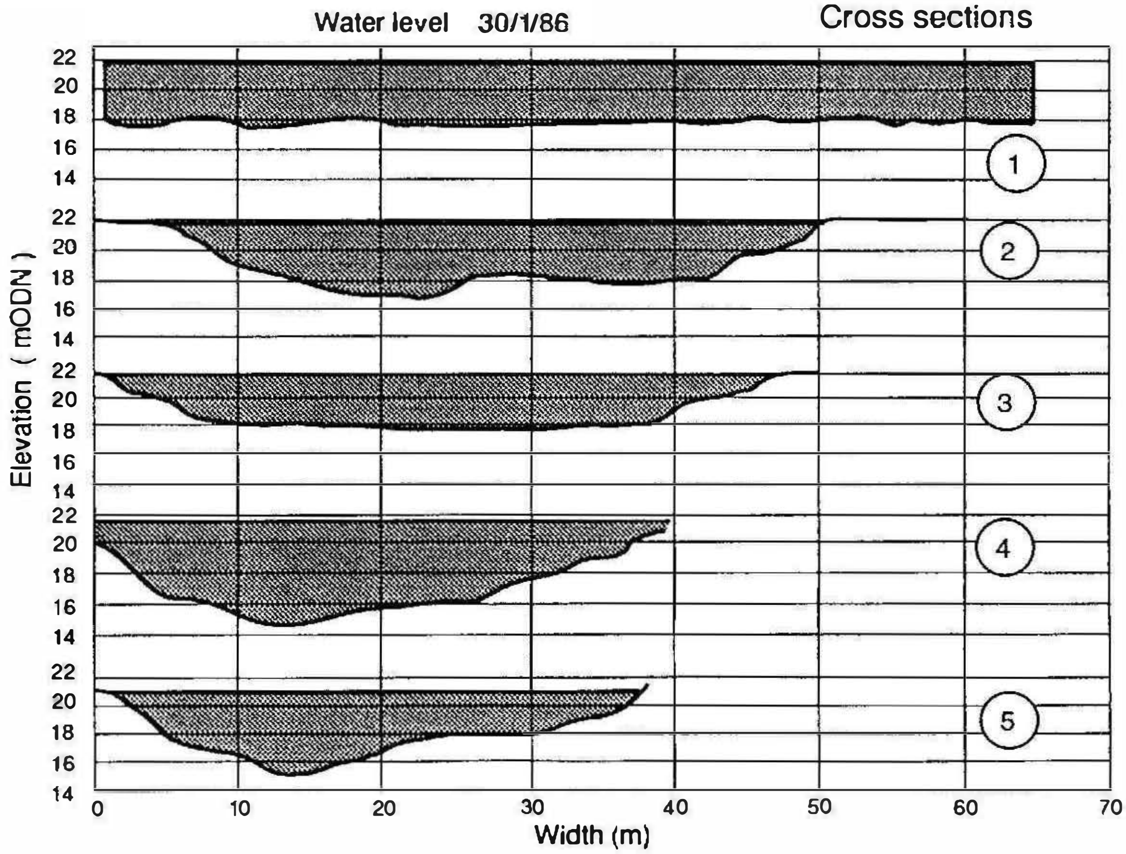
Plan



Description of channel

Meander. Bed material unknown. Bank lining variable; concrete wall, boat moorings, willow, hawthorne, ash, sycamore, holly, alder and horse chestnut with undergrowth of nettles and bramble.
Left flood plain parkland with mature elm trees, gardens with fences, rough pasture.
Right flood plain is parkland with mature elms and limes, market gardening.

0 400 800m



PH/4/12-95/ f.line

Plan and cross sections, River Avon at Evesham

n =

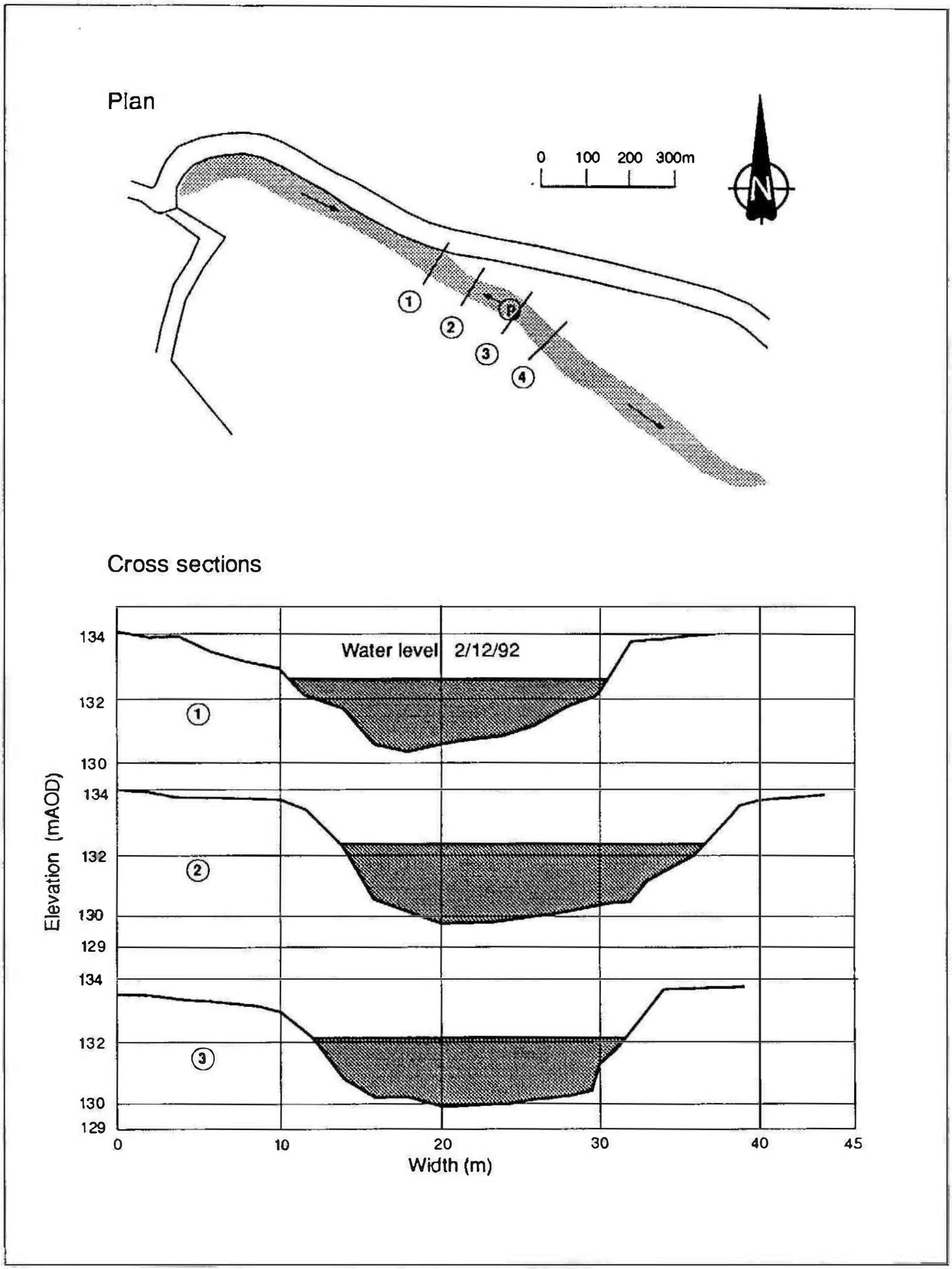
Bankfull hydraulic and geometric characteristics 30/1/86

Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000234 (1:4274)
Average cross sectional area	=	147.9 m^2
Average flow width	=	45 m
Average hydraulic radius	=	3.11 m

n =

Plate 5

River Derwent at Chatsworth



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Plan and cross sections, River Manifold at Ilam

River Manifold at Ilam

Bankfull hydraulic and geometric characteristics 2/12/92 see Figure A3.6

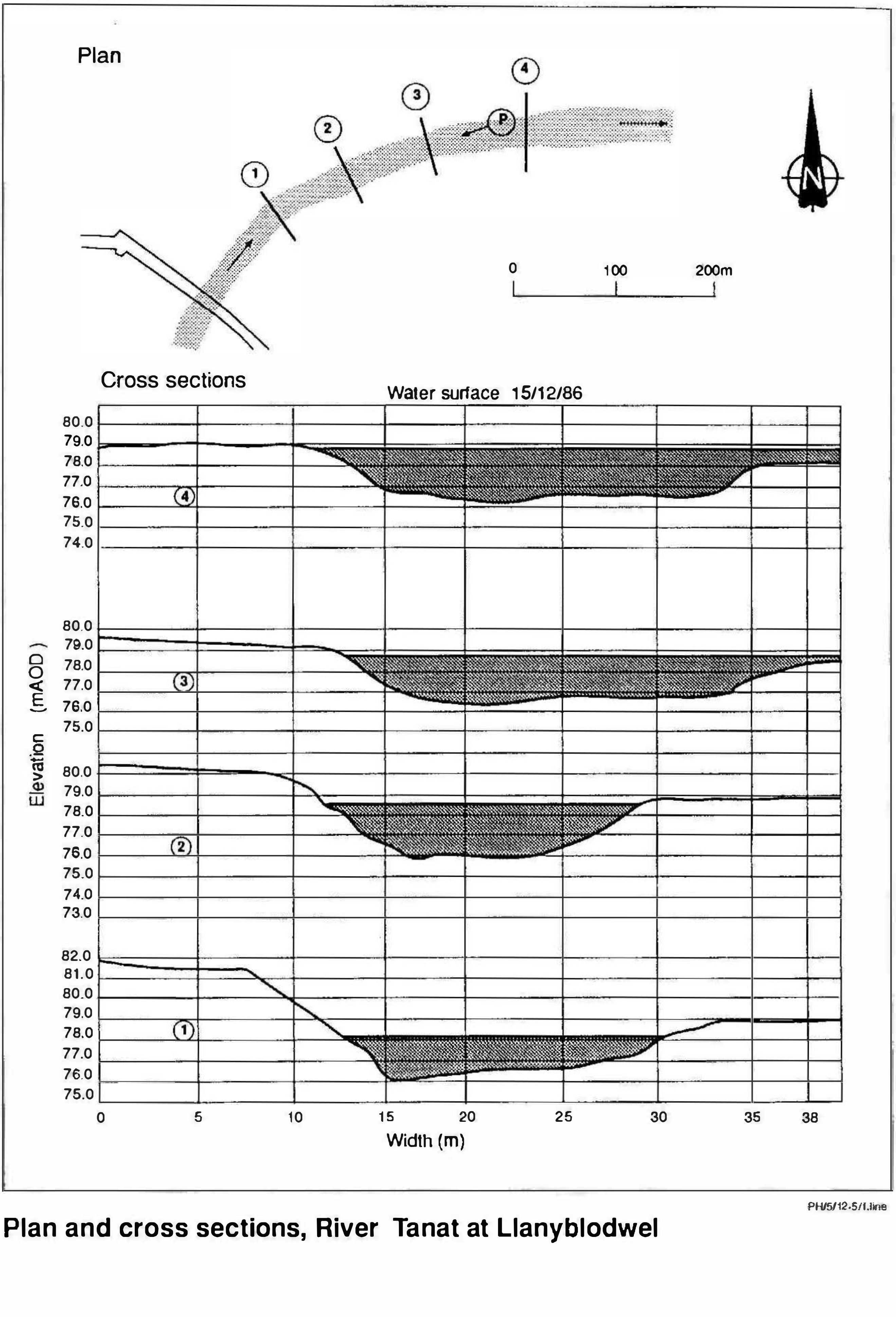
Manning's n roughness coefficient	=	0.042
Discharge	=	52.8m^3/s
Water surface slope	=	0.001977 (1:506)
Average cross sectional area	=	35.6m^2
Average flow width	=	21m
Average hydraulic radius	=	1.64m

Description of channel

Bed material is gravel and boulders. Bank vegetation of alder, ash, hazel, beech, sycamore and hawthorn traces with grass, scattered undergrowth of bramble. Flood plains of short grass pasture with hedgerows and wire fencing.

Plate 6

River Manifold at Ilam



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Plan and cross sections, River Tanat at Llanyblodwel

n =

Bankfull hydraulic and geometric characteristics 15/12/86

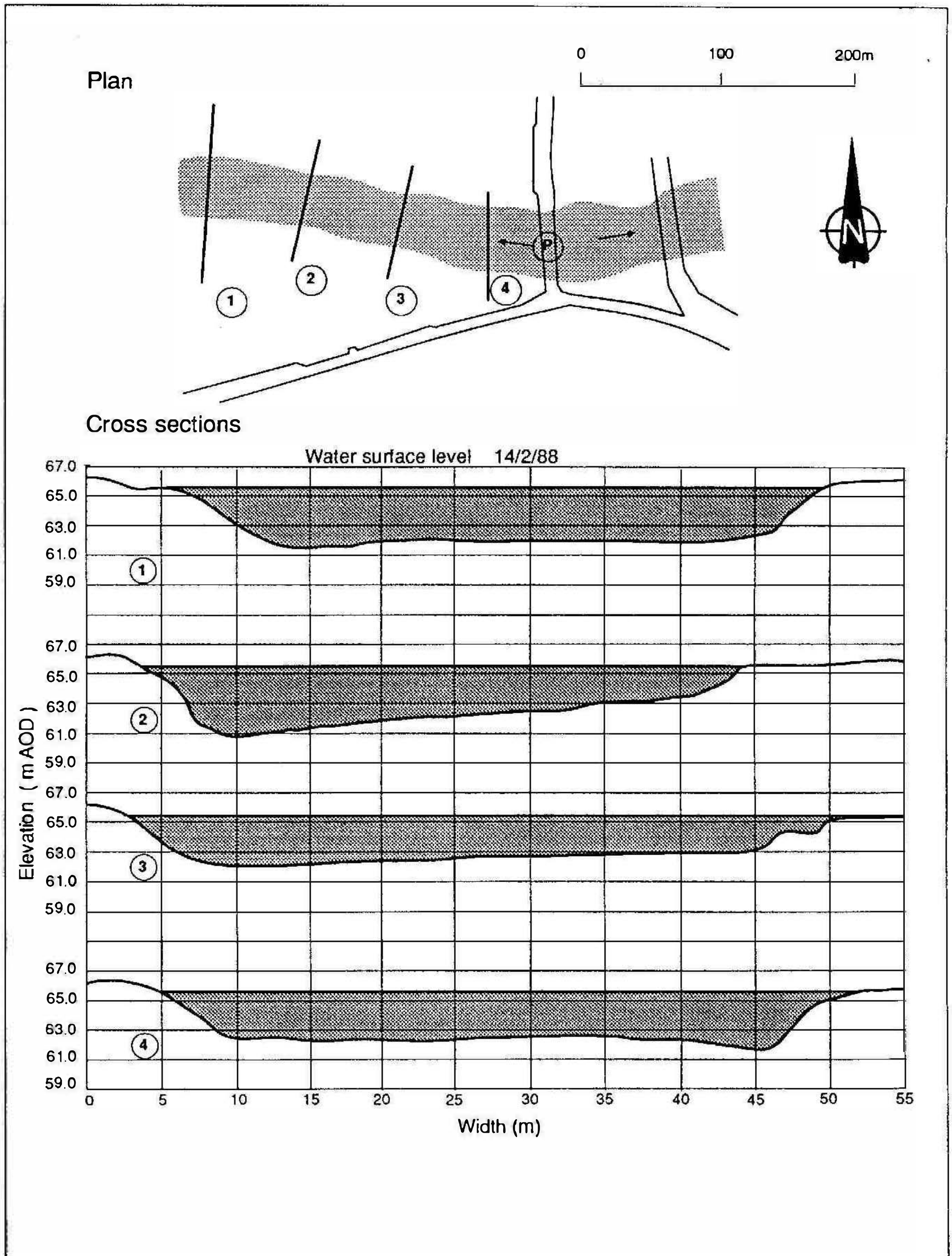
Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000298 (1:336)
Average cross sectional area	=	40.4 m^2
Average flow width	=	26.7 m
Average hydraulic radius	=	1.45 m

Description of channel

Bed is gravel and boulders. Banks lined with mature alders, ash and willow, undergrowth of bramble, nettle, wild rose and rank grass. Left flood plain is sown with crops, right flood plain is short grass pasture. Field boundaries delimited by hedgerows.

Plate 7

River Avon at Evesham



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Plan and cross section, River Vyrnwy at Llanymynech

n =

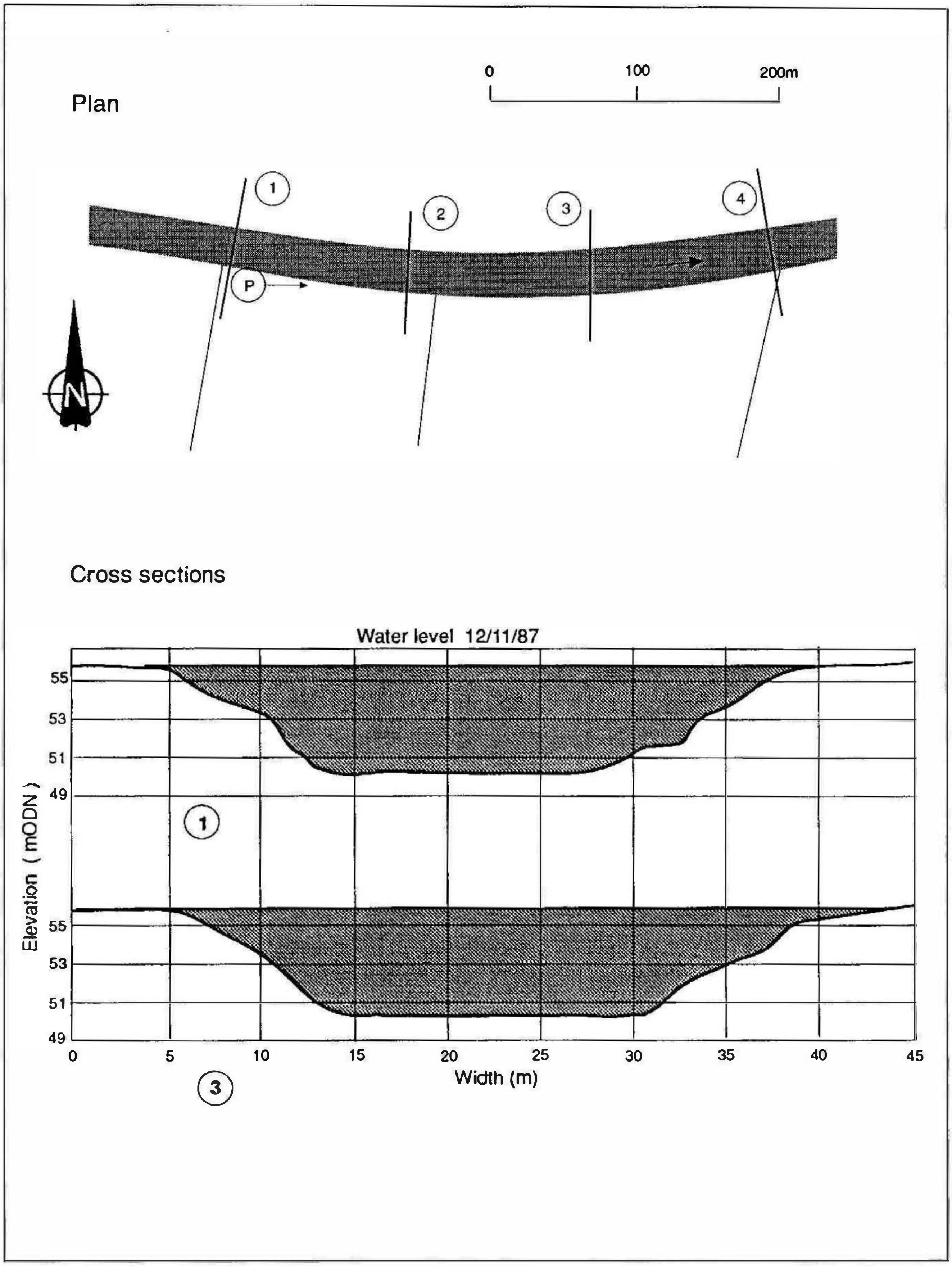
Bankfull hydraulic and geometric characteristics		14/2/88
Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000372 (1:2688)
Average cross sectional area	=	131.6 m^2
Average flow width	=	46.4 m
Average hydraulic radius	=	2.25 m

Description of channel

Bed material unknown. Right bank grass covered with scattered mature alders. Left bank lined with alder and willow. Left flood plain sown with crops, right flood plain is short grass pasture.

Plate 8

River Tanat at Llanyblodwel



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Plan and cross sections, River Severn at Montford

n =

Bankfull hydraulic and geometric characteristics 12/11/87

Manning's n roughness coefficient	=	
Discharge	=	
Water surface slope	=	0.000186 (1:5376)
Average cross sectional area	=	139 m^2
Average flow width	=	39.9 m
Average hydraulic radius	=	3.31 m

Description of channel

Bed material unknown. Left and right banks grass with occasional small willow trees. Flood plains of short grass pasture with hawthorn hedgerows and fences.

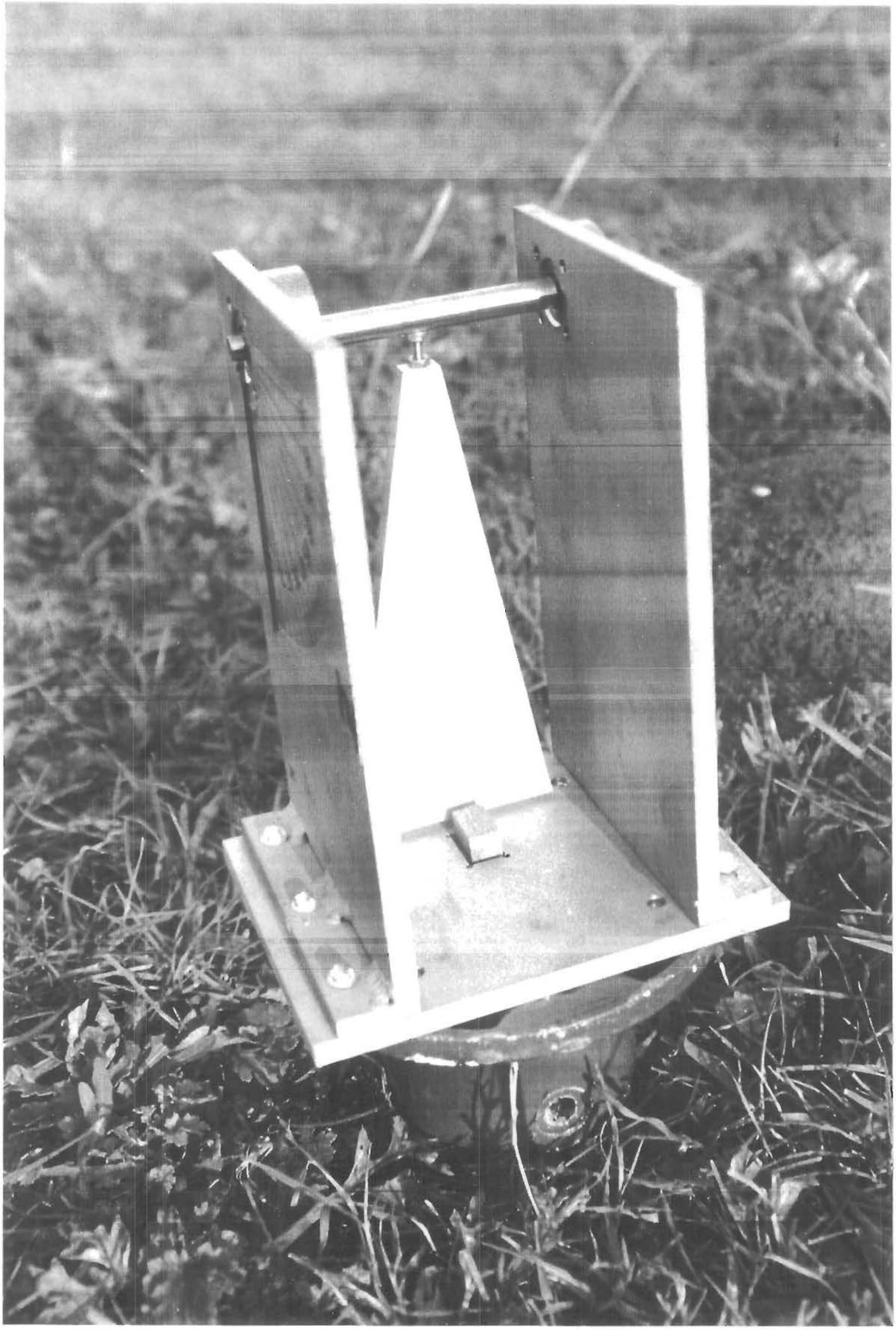


Plate 9 Peak velocity meter - Design 3

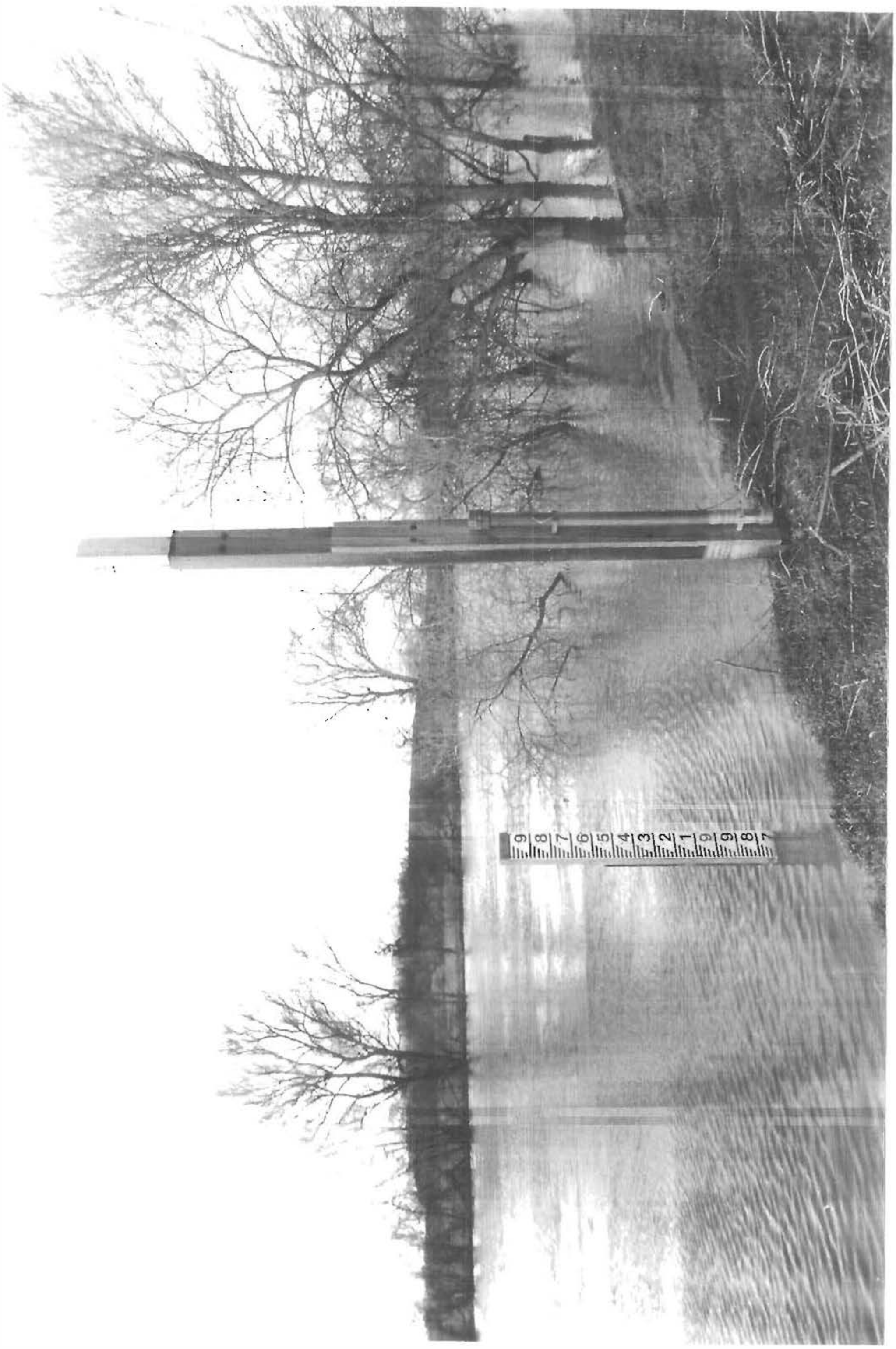


Plate 10   Two NRA Severn-Trent maximum water level gauges mounted in echelon

Appendices

Appendix 1

Cowan's method for estimating Manning's n

Appendix 1 Cowan's method for estimating Manning's n

Cowan (1956) developed a procedure for the US Soil Conservation service for estimating the value of Manning's n. By this procedure, the value of n may be computed by

$$n = (n_b + n_c + n_o + n_s + n_v)\, m_s$$

where

n_b is a basic n value for a straight, uniform, smooth channel in the natural materials involved.

n_c is a value for the channel condition to correct for the effect of surface irregularities.

n_s is a value for variations in shape and size of the channel cross section.

n_o is a value for the obstructions.

n_v is a value for vegetation and flow conditions.

m_s is a correction factor for the sinuosity of a meandering channel.

Base Value n_b

Channel character	**Basic n**
Channels in earth	0.020
Channels in fine gravel	0.024
Channels cut into rock	0.025
Channels in coarse gravel	0.028

Addition n_s for streamwise variation

Character of variations in size and shape of cross sections	**n_s**
Changes in size or shape occurring gradually	0.000
Large and small sections alternating occasionally or shape changes causing occasional shifting of main flow from side to side	0.005
Large and small sections alternating frequently or shape to changes causing frequent shifting of main flow from side to side.	0.010 to 0.015

Addition n_o for obstructions in the watercourse

Character of obstructions	n_o
Negligible	0.000
Minor	0.010 to 0.015
Appreciable	0.020 to 0.030
Severe	0.040 to 0.060

Obstructions may include debris deposits, exposed roots, fallen trees, boulders, rocks etc. In assessing their effect the following factors should be considered - reduction in flow area at various depths, angularity of the obstructions, position and spacing of the obstructions

Addition n_v for vegetation

Low influence n_v **= 0.005 to 0.010**

Dense growths of flexible turf grasses or weeds, of which Bermuda grass and blue grass are examples, where the average depth of flow is 2 to 3 times the height of vegetation. Supple seedling tree switches such as willow, cottonwood, or salt cedar where the average depth of flow is 3 to 4 times the height of the vegetation.

Moderate influence n_v **= 0.010 to 0.025**

Brushy growths, moderately dense, similar to willows 1 to 2 years old, dormant season, along side slopes of channel with no significant vegetation along the channel bottom, where the hydraulic radius is greater than 2 ft (0.6m).
Turf grasses where the average depth of flow is 1 to 2 times the height of vegetation.

Stemmy grasses, weeds, or tree seedlings with moderate cover where the average depth of flow is 2 to 3 times the height of vegetation.

High influence n_v **= 0.025 to 0.050**

Dormant season, willow or cottonwood trees 8 to 10 years old, intergrown with some weeds and brush, none of the vegetation in foliage, where the hydraulic radius is greater than 2 ft (0.6m). Growing season, bushy willows about 1 year old intergrown with some weeds in full foliage along side slopes, no significant vegetation along channel bottom, where hydraulic radius is greater than 2 ft (0.6m).

Very high influence n_v **= 0.050 to 0.100**

Turf grasses where the average depth of flow is less than one half the height of vegetation. Growing season, trees intergrown with weeds and brush, all in full foliage; any value of hydraulic radius up to 10 or 15 ft (3 to 4.6m). Growing season, bushy willows about 1 year old, intergrown with weeds in full foliage along side slopes; dense growth
of cat tails along channel bottom; any value of hydraulic radius up to 10 or 15 ft (3 to 4.6m).

Addition n_c for channel condition

Degree of irregularity	Surfaces comparable with	nc
Smooth	The best obtainable for the materials involved	0.000
Minor	Good dredged channels; slightly eroded or scoured side slopes of canals or drainage channels	0.005
Moderate	Fair to poor dredged channels; moderately sloughed or eroded side slopes of canals or drainage channels	0.010
Severe	Badly sloughed banks of natural channels; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged and irregular surfaces of channels excavated in rock.	0.020

Multiplier m_s for sinuosity

m_s = 1.0 $\quad s = 1$

m_s = 0.57 + 0.43s $\quad 1 < s < 1.7$

m_s = 1.30 $\quad s > 1.7$

Appendix 2

Guidelines for estimating Manning's n (Chow, 1959)

Appendix 2 Guidelines for estimating Manning's n (Chow, 1959)

		Min	Norm	Max
D	Natural streams			
D-1	Minor streams (top width at flood stage < 100ft)	Min	Norm	Max
(a)	Streams on plain			
	1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
	2. Same as above, but more stones and weeds	0.030	0.035	0.040
	3. Celan, winding, some pools and shoals	0.033	0.040	0.045
	4. Same as above, but some weeds and stones	0.035	0.045	0.050
	5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
	6. Same as 4, but more stones	0.045	0.050	0.060
	7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
	8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
(b)	Mountain streams, no vegetation in channel, banks usually steep, tress and brush along banks submerged at high stages			
	1. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
	2. Bottom: Cobbles with large boulders	0.040	0.050	0.070
D-2	Flood plains			
(a)	Pasture, no brush			
	1. Short grass	0.025	0.030	0.035
	2. High grass	0.030	0.035	0.050
(b)	Cultivated areas			
	1. No crop	0.020	0.030	0.040
	2. Mature row crop	0.025	0.035	0.045
	3. Mature field crops	0.030	0.040	0.050

(c)	Brush			
	1. Scattered brush, heavy weeds	0.035	0.050	0.070
	2. Light brush and trees, in winter	0.035	0.050	0.060
	3. Light brush and trees, in summer	0.040	0.060	0.080
	4. Medium to dense brush, in winter	0.045	0.070	0.110
	5. Medium to dense brush, in summer	0.070	0.100	0.160
(d)	Trees			
	1. Dense willows, summer, straight	0.110	0.150	0.200
	2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
	3. Same as above, but with heavy ?	0.050	0.060	0.080
	4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
	5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
D-3	Major streams (top width at flood stage > 100ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance			
(a)	Regular section with no boulders or brush	0.025	0.060	-
(b)	Irregular and rough section	0.035	0.100	-

Appendix 3

James and Wark Method for Meandering Channels

Appendix 3 James and Wark Method for Meandering Channels

Available Data

The SERC FCF Phase B experiments were limited to two sinuosities (1.37 and 2.04) and two main channel geometries (trapezoidal and pseudo-natural). Stage-discharge were taken for inbank and out-of-bank flows with smooth and rod roughened flood plains. Details of the Phase B experiments are described by James and Wark (1992).

Data from a series of experiments performed at the University of Aberdeen (Willetts et al, 1990 and Willetts, personal communication) were also used in the development of the methods. These experiments covered a wider range of sinuosities (1.2,1.4 and 2.04) than the Phase B experiments, and the main channel had a considerably smaller width-depth ratio.

Several other data sets were used in for the evaluation of the proposed methods. These were obtained from the experimental work of Kiely (1990), Toebes and Sooky (1967) and the US Army Corps of Engineers, Vicksburg (1956), and also the field and model test data for the River Roding presented by Sellin and Giles (1988) and Sellin et al (1990).

Inbank Flows

Various methods were identified in the literature for accounting for the additional resistance to flow induced by channel curvature, eg Mockmore (1944), Leopold et al (1960), Soil Conservation Service (SCS, 1963) and Chang (1983) to name a few. The method proposed by the SCS (1963) was selected as being suitable for practical application and in addition a modification to the method was formulated.

The SCS Method involves increasing the basic value of Manning's n to account for meander losses. An adjustment factor is defined for each of three ranges of sinuosity. The step nature of this recommendation introduces discontinuities at the limits of the sinuosity ranges, with consequent ambiguity and uncertainty. To overcome this problem the relationship was linearized and can be expressed as

$$n'/n = 0.43s + 0.57 \qquad \text{for } s < 1.7 \qquad (3.1)$$

$$n'/n = 1.30 \qquad \text{for } s \geq 1.7 \qquad (3.2)$$

in which n' is the value of Manning's n including bend loss effects, n is the basic value as determined by surface roughness, and s is sinuosity. This extension will be referred to as the Linearized SCS (LSCS) Method.

Ignoring the energy loss induced by meandering introduces fairly large errors in the prediction of discharge for inbank flows. On the basis of simplicity it is recommended that the Linearized SCS method be used for inbank discharge prediction in meandering channels. If the resistance is to be described by the Darcy-Weisbach f, the adjustment factor should be squared before it is applied to the basic value.

The resistance coefficient should be adjusted only if the basic value does not already account for meander losses. This would be the case if recommendations based on surface roughness are followed or if the roughness coefficient has been derived for a straight channel using Equation 5. If a value is determined from flow data measured at the site in question by slope-area calculation, it will already incorporate meander effects and should not be adjusted further.

Overbank Flows

Various methods have been proposed in the past for estimating discharges in straight compound channels, Wark et al (1991). Application of these to meandering channels results in unacceptable errors because they do not account for all of the important energy loss mechanisms present in meandering flows.

A new method for predicting discharges in compound meandering channels was developed using a divided channel approach. The notation used in the James and Wark method is shown in Table 4, Section 3.3.

The compound cross-section is divided into four zones, as shown in Figure 3.1. Zone 1 is the main channel below bankfull level, Zone 2 is the flood plain within the meander belt, and Zones 3 and 4 are the flood plains on either side of the main channel beyond the meander belt. For a given stage the discharge is calculated as the sum of the zonal discharges, calculated separately, i.e.

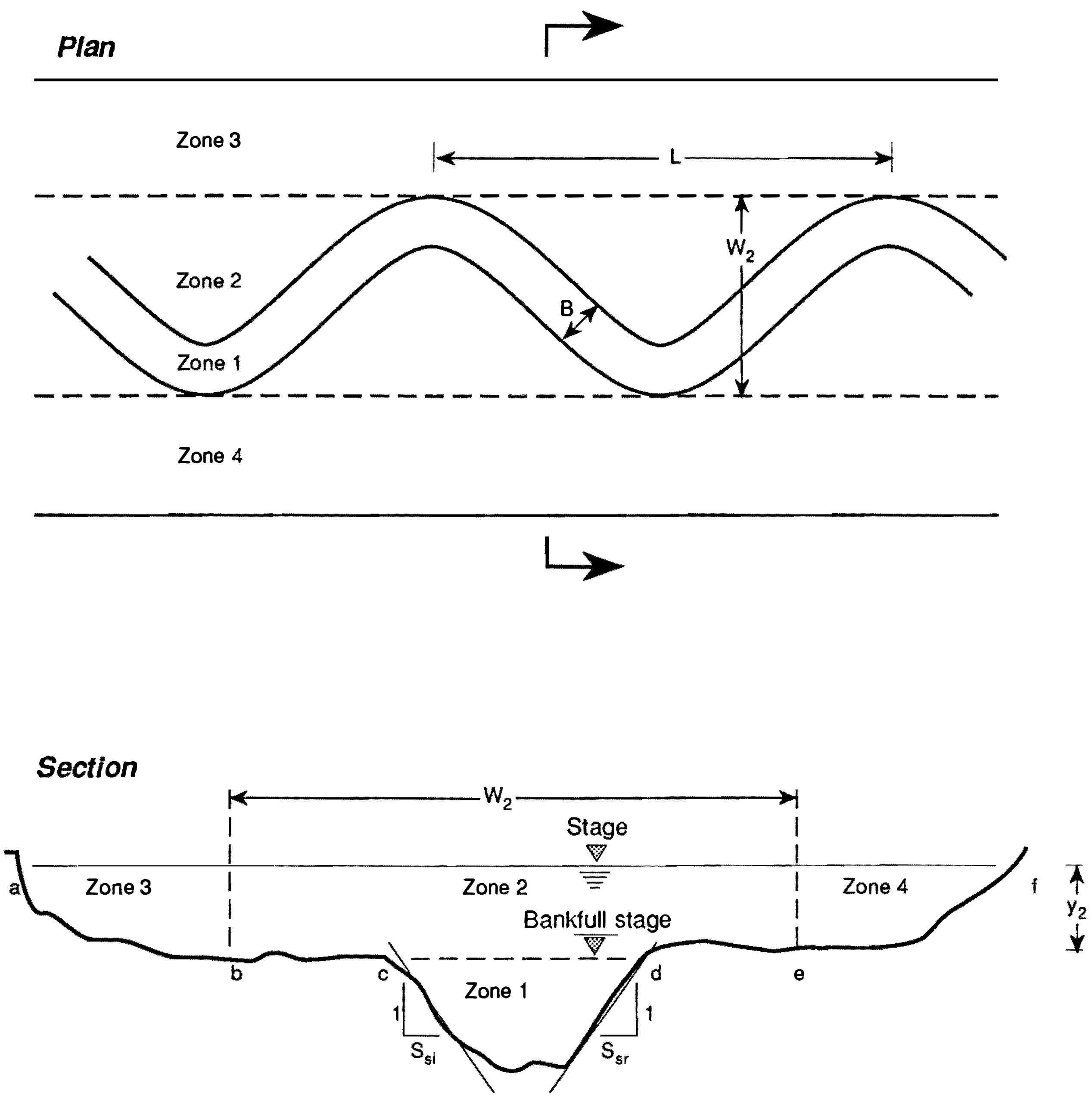
$$Q_T = Q_1 + Q_2 + Q_3 + Q_4 \tag{3.3}$$

The SERC FCF Phase B data were used to derive procedures for calculating the zonal discharges.

<u>Zone 1 : main channel</u>

The flow mechanisms in this zone are complex and not well understood. In addition to friction, energy is lost through secondary circulation driven by the shear imposed by the flood plain flow, which is radically different in character from the inbank secondary circulation. There is also considerable bulk exchange of water between the main channel and flood plain and so the discharge in this zone will vary over a wavelength, being maximum at a bend apex and minimum at some point between bends.

Because of the poor current understanding of the flow mechanisms, an empirical approach has been followed for predicting discharge. The variation of discharge along the channel is ignored. Hence for the purposes of stage-discharge estimation the flow in Zone 1 is assumed to be constant along the reach considered. The procedure is to calculate the bankfull discharge (Q_{bf}), and then to adjust this to account for the effects of overbank flow. The bankfull discharge can be estimated using Equation 4, Section 3.2 or obtained by measurement, if possible. The hydraulic slope (S) which controls the flow in the main channel zone is related to the flood plain or valley hydraulic slope by the channel sinuosity, (ie $S = S_0 / s$). It should be noted that S_0 can either be a ground slope if uniform flow is assumed or a water surface slope.



$S_S = (S_{sl} + S_{sr})/2$

$P_1 = \overrightarrow{cd}$

$P_2 = \overrightarrow{bc} + \overrightarrow{de} - B(s-1)$ s = sinuosity

$P_3 = \overrightarrow{ab}$

$P_4 = \overrightarrow{ef}$

JBW/2.1/8-92/LO

Figure 3.1 Cross-section subdivision of overbank flows

The adjustment factor was determined from the SERC FCF Phase B data. Actual discharges in this zone were obtained by integrating the velocity magnitude and direction measurements taken in some of the experiments. Bankfull discharges were estimated using the Modified Chang Method (1984) for the trapezoidal channel, and by extrapolating the inbank stage-discharge curves for the pseudo-natural channels. The ratio of actual to bankfull discharge defines the adjustment factor, Q_1'.

Q_1' was found to depend on :

- the flood plain flow depth at the edge of the main channel (y_2);
- the channel sinuosity (s);
- the cross section geometry and
- flood plain roughness.

These characteristics are represented by dimensionless parameters which were chosen as being both meaningful and easy to measure. The flow depth is normalized by the hydraulic depth of the main channel at bankfull, equation 3.4, where A is the cross sectional area and B the surface width of the main channel at bankfull.

$$y' = y_2 / (A/B) \qquad (3.4)$$

The cross section geometry is characterized by B^2/A. The flood plain roughness is expressed as the ratio of flood plain and main channel Darcy-Weisbach friction factors, i.e.

$$f' = f_2 / f_1 \qquad (3.5)$$

The Darcy-Weisbach friction factor can be calculated using the Colebrook-White equation :

$$\frac{1}{f^{0.5}} = \frac{V^2}{8\ g\ R\ S} = -2\ \log_{10} \left(\frac{k_s}{14.8R} + \frac{2.51}{Re\ f^{0.5}}\right) \qquad (3.6)$$

where

Re is the Reynolds Number

and V the mean flow velocity can be calculated from

$$V = -(32\ g\ R\ S)^{0.5} \log \left[\frac{k_s}{14.8R} + \frac{1.255\nu}{R\ (32\ g\ R\ S)^{0.5}}\right] \qquad (3.7)$$

where

k_s is a linear measure of effective roughness
ν is the kinematic viscosity of water

If Manning's n is used then f is related to n by

$$f = 8\ g\ n^2 / R^{0.333} \tag{3.8}$$

The ratio f′ can therefore also be expressed in terms of Manning's n

$$f' = (n_2 / n_1)^2\ (R_1 / R_2)^{0.333} \tag{3.9}$$

The relationship between the adjustment factor and these variables is shown schematically in Figure 3.2. This shows that the main channel discharge is initially reduced as a stage rises above bankfull, and that this reduction is independent of channel characteristics. At higher stages the discharge increases with stage at a rate which depends strongly on B^2/A, s and f′. This variation can be accounted for by choosing the adjustment factor to be the **greater** of :

$$Q_1' = 1.0 - 1.69\ y' \tag{3.10}$$

or

$$Q_1' = m\ y' + K\ c$$

with

$$m = 0.0147\ B^2/A + 0.032\ f' + 0.169$$

$$c = 0.0132\ B^2/A - 0.302\ s + 0.851$$

$$K = 1.14 - 0.136\ f' \tag{3.11}$$

Hence the correct flow in Zone 1 is given by

$$Q_1 = Q_{bf}\ Q_1' \tag{3.12}$$

Zone 2 : Inner flood plain

The method for predicting the inner flood plain discharge is based on quantitative descriptions of major loss mechanisms identified by other researchers (for example, Ervine and Ellis, 1987). These are

- friction on the wetted perimeter
- expansion of the flow as it enters the main channel, and
- contraction of the flow as it re-enters the flood plain

Additional losses associated with the bulk exchange of water between the main channel and flood plains are also likely to occur. However, due to the lack of any theoretical model which would account for this, for the purposes of stage-discharge estimation it is assumed that the discharge in Zone 2 is constant along the reach of valley considered.

Friction losses can be estimated using the Darcy-Weisbach equation. In this case the wetted perimeter does not include the vertical planes separating Zone 2 from Zones 3 and 4, or the horizontal plane separating Zones 1 and 2. It should be estimated as the total length of the flood plain surfaces across the section less B(s-1). This approximation is arrived at by considering that the total area over which bed friction acts is given by total area of flood plain (including main channel) minus the top area of the main channel. The relative

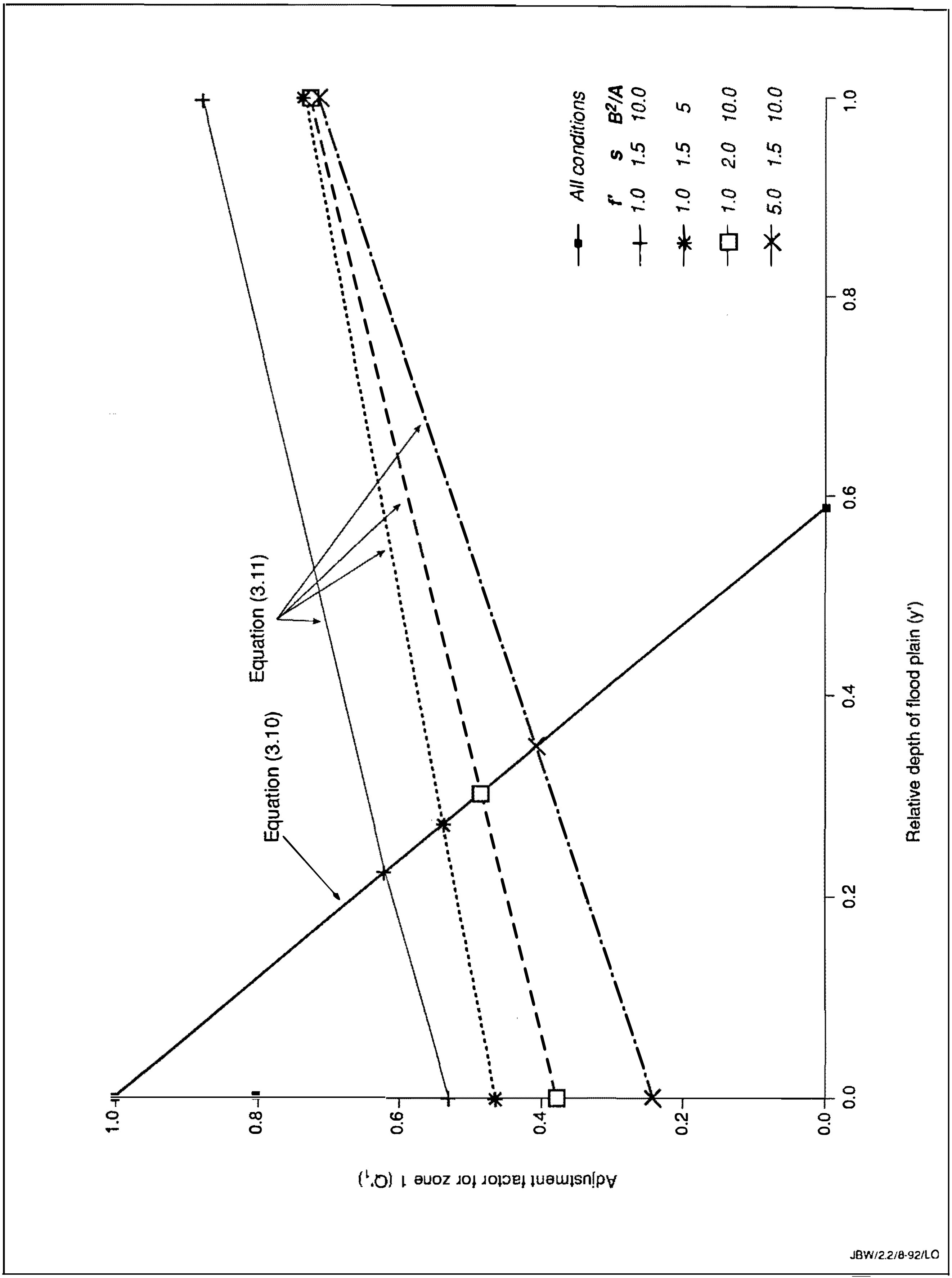


Figure 3.2 Adjustment factor for Zone 1 discharge

length of the main channel is the sinuosity. If Zones 3 and 4 do not exist, ie the main channel meanders across the full valley width, the flood plain surfaces up to the water surface should be included.

A basic description of the expansion and contraction losses was derived by analyzing the flow over a simple slot. The expansion loss was estimated by application of the energy and momentum equations, and the contraction loss using an empirical loss coefficient, as suggested by Yen and Yen (1983). An adjustment for width to depth ratio of the main channel was derived from data presented by Jasem (1990), and adjustments to account for the effect of main channel side slopes were derived from the results of Formica (1955), as presented by Chow (1959). The total loss over a meander wavelength was assumed to be proportional to the width over which expansion and contraction take place.

The SERC FCF Phase B and Aberdeen data showed that the non-friction losses were not wholly accounted for by the expansion-contraction model, and that there were additional effects associated with the main channel sinuosity and cross-sectional geometry. Empirical correction factors were introduced to account for these effects.

According to this model, the discharge for Zone 2 is given by

$$Q_2 = A_2 V_2 \qquad (3.13)$$

in which

A_2 is the cross-sectional area of Zone 2, and

V_2 is the flow velocity in Zone 2, given by

$$V_2 = \left(\frac{2 \; g \; S_o \; L}{(f_2 \; L) \; / \; (4 \; R_2) \; + \; F_1 \; F_2 \; K_e} \right)^{0.5} \qquad (3.14)$$

in which

g is the acceleration due to gravity

S_o is the flood plain or valley hydraulic gradient and may be either the ground slope if uniform flow is assumed or the water surface slope.

L is the meander wavelength, (Figure 3.1).

f_2 is the Darcy-Weisbach friction factor for the inner flood plain.

P_2 is the wetted perimeter for the inner flood plain. It is defined as the total wetted surface of the inner flood plain minus the term B(s-1), (Figure 3.1).

R_2 is the inner flood plain hydraulic radius (A_2/P_2), with the area and wetted perimeter as defined above.

F_1 is a factor to account for variations of non-friction energy loss associated with the main channel cross-section geometry, given by

$$F_1 = 0.1\ B^2/A \qquad \text{for } B^2/A < 10$$

$$F_1 = 1.0 \qquad \text{for } B^2/A \geq 10 \qquad (3.15)$$

F_2 is a factor to account for variations of non-friction energy loss associated with the main channel cross-section sinuosity, given by

$$F_2 = s/1.4 \qquad (3.16)$$

K_e is a factor to account for expansion and contraction losses, given by

$$K_e = C_{sl}\ C_{wd}\ (C_{sse}\ (1 - y_2\ /\ (y_2 + h))^2 + C_{ssc}\ K_c) \qquad (3.17)$$

y_2 is the flow depth on the flood plain, measured at the edge of the main channel, (Figure 3.1).

h is the step height for expansion and contraction, and can be approximated by the hydraulic mean depth of the main channel, (A/B).

C_{sl} defines the length over which expansion and contraction occur in one meander wavelength, and is given by

$$C_{sl} = 2(W_2 - B)\ /\ W_2 \qquad (3.18)$$

W_2 is the total width of the inner flood plain.

C_{wd} accounts for the effect of cross-section shape on expansion and contraction loss, and is given by

$$C_{wd} = 0.02\ (B^2/A) + 0.69 \qquad (3.19)$$

C_{sse} accounts for the effect of the main channel side slope on expansion loss, and is given by

$$C_{sse} = 1.0 - S_s\ /\ 5.7 \qquad \text{(but } C_{sse} \text{ not less than 0.1)} \qquad (3.20)$$

C_{ssc} accounts for the effect of the main channel side slope on contraction loss, and is given by

$$C_{ssc} = 1.0 - S_s\ /\ 2.5 \qquad \text{(but } C_{ssc} \text{ not less than 0.1)} \qquad (3.21)$$

S_s is the cotangent of the main channel side slope, (Figure 3.1).

K_c is the basic contraction coefficient, as given in Table 3.1, and by Figure 3.3

Table 3.1 Contraction loss coefficients (Rouse, 1950)

$y_2/(y_2+h)$	0.00	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
K_c	0.50	0.48	0.45	0.41	0.36	0.29	0.21	0.13	0.07	0.01	0.00

Zones 3 and 4 : outer flood plains

Flow in the outer flood plain zones is assumed to be solely controlled by friction. The zonal discharges are calculated using an appropriate friction equation with the division lines separating these zones from Zone 2 excluded from the wetted perimeter, ie.

$$Q_3 = A_3 V_3$$

$$Q_4 = A_4 V_4 \qquad (3.22)$$

where

$$V_3 = \left(\frac{8 \ g \ R_3 \ S_o}{f_3}\right)^{0.5}$$

$$V_4 = \left(\frac{8 \ g \ R_4 \ S_o}{f_4}\right)^{0.5} \qquad (3.23)$$

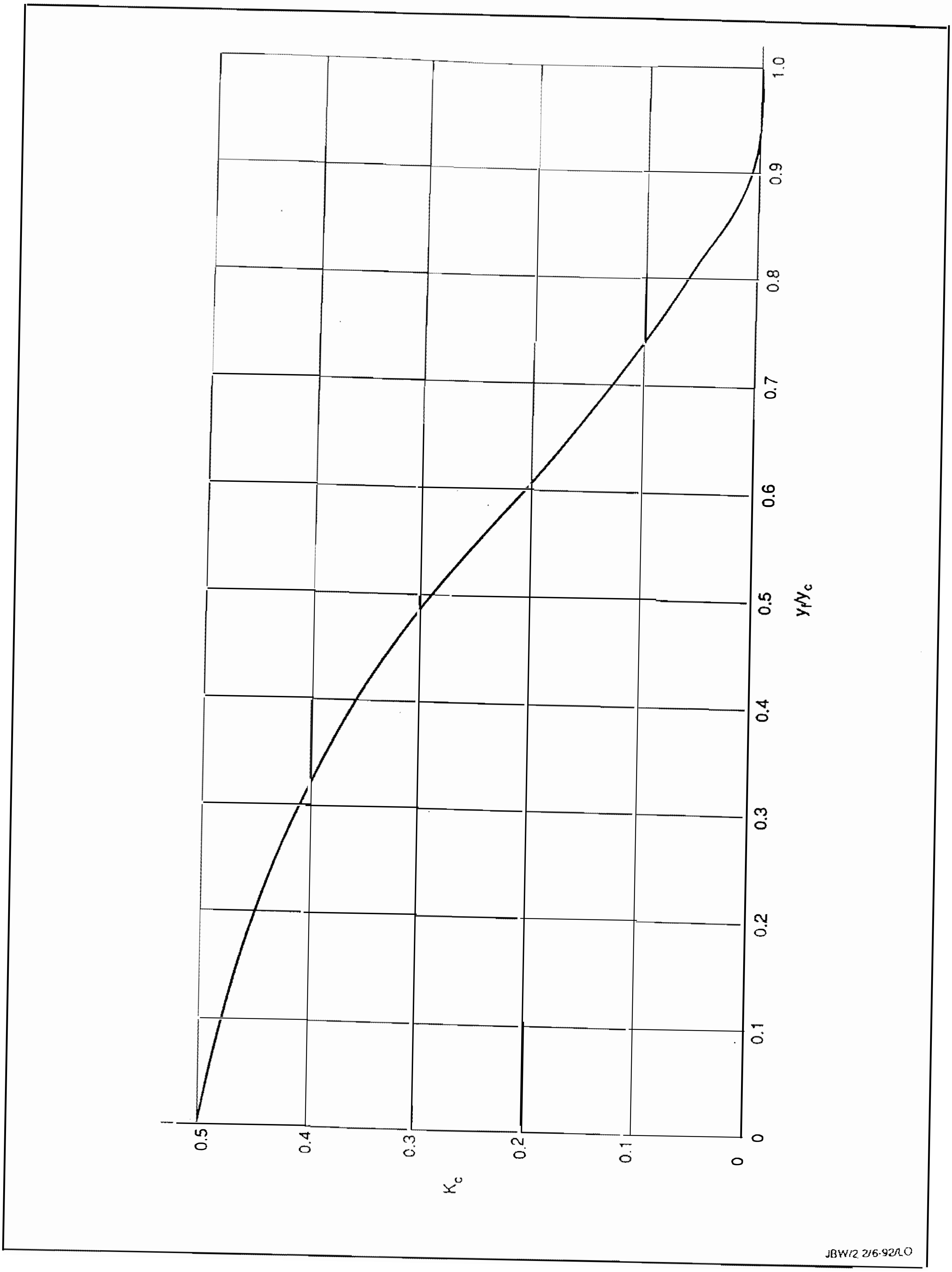


Figure 3.3 Contraction loss coefficient

Summary of Equations

The Zonal Discharge Equation

For a given stage the discharge is calculated as the sum of the zonal discharges, ie.

$$Q_T = Q_1 + Q_2 + Q_3 + Q_4 \qquad (3.3)$$

Zone 1 : Main Channel

The correct flow in Zone 1 is given by

$$Q_1 = Q_{bf}\, Q_1' \qquad (3.12)$$

The adjustment factor (Q_1') is the **greater** of :

$$Q_1' = 1.0 - 1.69\, y' \qquad (3.10)$$

or

$$Q_1' = m\, y' + K\, c$$

with

$$m = 0.0147\, B^2/A + 0.032\, f' + 0.169$$

$$c = 0.0132\, B^2/A - 0.302\, s + 0.851$$

$$K = 1.14 - 0.136\, f' \qquad (3.11)$$

$$y' = y_2 / (A/B) \qquad (3.4)$$

$$f' = f_2 / f_1 \qquad (3.5)$$

Zone 2 : Inner Flood Plain

According to this model, the discharge for Zone 2 is given by

$$Q_2 = A_2\, V_2 \qquad (3.13)$$

in which

$$V_2 = \left(\frac{2\, g\, S_o\, L}{(f_2\, L) / (4\, R_2) + F_1\, F_2\, K_e} \right)^{0.5} \qquad (3.14)$$

with

$$F_1 = 0.1\, B^2/A \qquad \text{for } B^2/A < 10$$

$$F_1 = 1.0 \qquad \text{for } B^2/A \geq 10 \qquad (3.15)$$

$$F_2 = s / 1.4 \qquad (3.16)$$

$$K_e = C_{sl}\, C_{wd}\, (\, C_{sse}\, (1 - y_2 / (y_2 + h))^2 + C_{ssc}\, K_c) \qquad (3.17)$$

with

$$C_{sl} = 2(W_2 - B) / W_2 \qquad (3.18)$$

$$C_{wd} = 0.02\, (B^2/A) + 0.69 \qquad (3.19)$$

$$C_{sse} = 1.0 - S_s / 5.7 \qquad \text{(but } C_{sse} \text{ not less than 0.1)} \qquad (3.20)$$

$$C_{ssc} = 1.0 - S_s / 2.5 \qquad \text{(but } C_{ssc} \text{ not less than 0.1)} \qquad (3.21)$$

Zones 3 and 4 : Outer Flood Plains

Flow in the outer flood plain zones is assumed to be solely controlled by friction. The zonal discharges are calculated using an appropriate friction equation with the division lines separating these zones from Zone 2 excluded from the wetted perimeter.

$$Q_3 = A_3\, V_3$$

$$Q_4 = A_4\, V_4 \qquad (3.22)$$

where

$$V_3 = \left(\frac{8\ g\ R_3\ S_o}{f_3} \right)^{0.5}$$

$$V_4 = \left(\frac{8\ g\ R_4\ S_o}{f_4} \right)^{0.5} \qquad (3.23)$$

Appendix 4

Conclusions and recommendations from The Research Summary Report SR 431

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Conclusions

The principal conclusions and recommendations for flow measurement practice are as follows.

1 When extending rating curves using gauged flows, an assessment should be made of the inbank and out-of-bank ratings separately without the assumption that the curve is continuous at the bankfull level. If the assessed flow decreases as stage rises past bankfull, this procedure will tend to <u>increase</u> the flow estimates at high river stages.

2 A version of the divided channel method is the best simple hand calculation procedure for estimating the extension of rating curves based upon cross-section properties. However, new conveyance calculation methods from the Flood Channel Facility data have been developed by Ackers (1991) (straight channels) and James & Wark (1992) (meandering channels). These methods should provide better estimates than those available previously (including the Divided Channel Method). Experience in the use of the new methods should be collated and evaluated after a period of pilot testing.

3 Computational hydraulic models (both 1-D and 2-D) have been demonstrated as offering means of extending the rating curve at gauging stations. It is unlikely that the 2-D software currently available commercially will be suitable for occasional use in hydrometric offices as they require specialist expertise to construct and operate models of specific sites.

4 A peak velocity meter for flood plain flow measurement has been developed and evaluated in field trials. This meter is inexpensive to construct and will allow estimates of flow velocity to be made after the passage of a flood.

5 The water surface slope should be monitored at all gauging stations used for flood flow measurement. This information may be captured at modest cost through the use of peak water level gauges installed so that the water level drop is at least 0.3 m between the gauges. The slope measurements will allow average roughness coefficients to be calculated for flood conditions and will support the use of slope-area flow estimation.

6 New information has been published for the estimation of river roughness. A regression equation relates the river roughness at a site to the channel dimensions, it has good accuracy for data from rivers in the Severn Trent region but is of lower accuracy in other areas of the UK. A series of photographs has been produced for rivers in the NRA Severn Trent region with a known bankfull discharge and Manning's n roughness coefficient.

Research and Development Needs

In reviewing the research carried out on Flood Discharge Assessment several further lines of research and development appear to have potential. The topics listed below, however, have not been prioritised.

1 The use of 2-D and 3-D modelling could be explored further through demonstration and benchmark testing of commercial software.

2 An analysis of flow measurements in all UK regions could be undertaken to assess whether a geomorphic approach to flood rating curve assessment is possible through regional regression equations.

3 The peak velocity meter requires further field evaluation and comparison with alternative means of flow measurement.

4 The photographic catalogue of river roughness conditions at UK gauging sites could be extended to cover all regions in the UK.

5 An easy-to-use computer package based on the Lateral Distribution Method could facilitate the estimation of out-of-bank rating of many gauging stations.

6 Field measurements to establish the roughness of British rivers under flood conditions should be encouraged. The information may be relatively inexpensive to collect if peak velocity meters are installed at some key sites and water surface slope measurements made.