Project 252

Design of Straight and Meandering Compound Channels

Interim Guidelines on Hand Calculation Methodology

SERC
SCIENCE AND ENGINEERING RESEARCH COUNCIL

MAFF ENVIRONMENT matters

HR Wallingford

HR Wallingford

NRA
National Rivers Authority

R&D Report 13

Design of Straight and Meandering Compound Channels

Interim Guidelines on Hand Calculation Methodology

J B Wark, C S James and P Ackers

Research Contractor:
HR Wallingford

National Rivers Authority
Rivers House
Waterside Drive
Aztec West
Bristol
BS12 4UD

R&D Report 13

Publisher:
National Rivers Authority
Rivers House
Waterside Drive
Aztec West
Bristol
BS12 4UD
Tel: 0454-624400
Fax: 0454-624409

First Published 1994

This report is the result of work jointly funded by the National Rivers Authority (NRA), the Ministry of Agriculture, Fisheries and Food (MAFF), the Science and Engineering Research Council (SERC) and HR Wallingford.

Dissemination Status

Internal: Released to Regions
External: Released to Public Domain

Statement of Use

R&D Report 13 is intended for use by river engineers to calculate steady discharges in rivers with flood plains or berms, by hand. Detailed calculation steps are given.

Research Contractor

This document was produced under R&D Project 252 by:

HR Wallingford Ltd
Howbery Park
Wallingford
Oxon
OX10 8BA
Tel: 0491-35381
Fax: 0491-32233

NRA Project Leader

The NRA's Project Leader for R&D Project 252 was:

Roger Pethick - NRA Thames Region.

CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

A compound channel consists of a main channel, which accommodates normal flows, flanked on one or both sides by a flood plain which is inundated during high flows. For water levels above the flood plain, the flow is strongly influenced by the interaction between the fast-flowing water in the main channel and the relatively slow-moving water over the flood plains. This significantly complicates the estimation of stage-discharge relationships. The extra turbulence generated by the flow interaction introduces energy loss over and above that associated with boundary resistance. This is not accounted for by the conventional resistance equations and their direct applications may result in significant error.

Various research projects have been conducted on compound channel behaviour, between the late 1960's and the early 1980's. In the late 1980's the Science and Engineering Research Council (SERC) along with HR Wallingford agreed to fund a series of experiments using a large scale model facility. The SERC Flood Channel facility (FCF) is a flume 50 m long, 10 m wide with a maximum discharge of 1.1 cumecs.

Phase A of the SERC FCF work investigated the detailed hydraulic behaviour of straight compound channels and Phase B of meandering compound channels. At the end of Phase A the NRA commissioned HR Wallingford to analyse the stage discharge relationships and develop a hand calculation procedure for estimating discharges in straight compound channels (Ackers 1991). MAFF assisted with the production of a detailed technical report (Ackers 1991). Similarly, after Phase B the NRA funded an analysis of stage discharge relationships for meandering compound channels. MAFF funded the production of a detailed technical report (James & Wark 1992).

This report brings together the two procedures for straight and meandering compound channels in one document, to facilitate their use in the design office. Chapter 2 summarises the important mechanisms which affect the discharge capacities of straight and meandering compound channels. Chapter 3 provides guidelines on the choice of the straight or meandering method. The details of the two procedures are given in Chapter 4, along with detailed worked examples. Annexes at the end of the report give summaries of the development and verification steps which were followed for the two procedures. Implications for software and a bibliography are also given.

Both the guidelines given in this report and the layout are regarded as interim. It is issued in the hope that users will provide feedback on the procedures and their use.

KEY WORDS

Hydraulic, Capacity, Design, Compound channels, Straight, Meanders, Flood plains, Stage-discharge, Bed shear stress, Worked examples, National Rivers Authority, HR Wallingford

1. INTRODUCTION

A "compound" channel consists of a main channel, which accommodates normal flows, flanked on one or both sides by a flood plain which is inundated during high flows. For water levels above the flood plain, the flow is strongly influenced by the interaction between the fast-flowing water in the main channel and the relatively slow-flowing water over the flood plains. This significantly complicates the estimation of stage-discharge relationships. The extra turbulence generated by the flow interaction introduces energy loss over and above that associated with boundary resistance. This is not accounted for by the conventional resistance equations (such as Chézy, Manning and Darcy-Weisbach), and their direct application may result in considerable error.

1.1 Background

It has become more generally known that traditional methods of calculating discharge can give rise to significant errors when applied to compound channels. Various university research projects were conducted between the late 1960's and early 1980's. It became increasingly obvious that the use of such very small scale laboratory model results to design or analyze prototype channels could introduce potentially large errors. The Science and Engineering Research Council (SERC) along with HR Wallingford agreed to fund a series of experiments using a very large scale model facility.

The SERC Flood Channel Facility (FCF) was constructed at HR Wallingford. It is a laboratory flume 50m long by 10m wide with maximum discharge of $1.1m^3/s$, this compares with typical university experiments with flumes of the order of 10m long by 1m to 2m wide.

Phase A of the SERC FCF programme investigated the detailed hydraulic behaviour of straight compound channels. The flood plains and main channel were constructed to be parallel to each other. A few experiments were also carried out with the main channel skewed to the flood plains at small angles. Phase B of the programme investigated the behaviour of meandering compound channels.

At the end of Phase A the National Rivers Authority and The Ministry of Agriculture, Fisheries and Food (MAFF) were concerned that the results of these important experiments should be reported to practising engineers in a useful form. The NRA commissioned HR Wallingford to analyze the stage discharge results and define a hand calculation procedure for estimating discharges in straight compound channels. The results of this project were reported in detail by Ackers (1991) and summarised in R&D Note 44. MAFF assisted with production of the detailed technical report, Ackers (1991).

During the course of the Phase B work it became apparent that the behaviour of meandering compound channels is quite different to straight compound channels.

Again the NRA commissioned HR Wallingford to analyze the Phase B data and provide a hand calculation procedure for estimating the discharge capacity of meandering compound channels. MAFF again provided assistance in publishing the detailed technical report, James and Wark (1992).

These two projects, sponsored by the NRA, complement the original laboratory work carried out for SERC and other research carried out by HR Wallingford for MAFF, under the Commission A on river flood protection. Phase C of the FCF work is due to commence in 1994 and will investigate sediment transport behaviour in straight and meandering compound channels. This last Phase is being funded by SERC, HR Wallingford, the NRA, MAFF and the European Commission.

This manual brings together the two procedures for straight and meandering compound channels in one document. Chapter 2 summarises the important mechanisms which affect the discharge capacities in straight and meandering compound channels. Chapter 3 provides guidelines on the choice of the straight or meandering method. The details of the two procedures are given in chapter 4, along with work examples of their application. Annexes at the end of the manual give summaries of the development and verification steps which were followed for the two procedures. A summary of implications arising from the use of the new procedures is given along with a bibliography.

1.2 Status of Guidelines

The procedures and guidelines given in this manual represent the best current (1994) state of knowledge and are an improvement on traditional methods. However, it must be recognised that these procedures have not been fully verified against field data. It is inevitable that practitioners will push the bounds of the advice and knowledge. Research is in progress on the behaviour of compound channels and it is expected that some of the procedures and guidelines given here may be superseded in the future.

1.3 Amendments to Guidelines

Both the guidelines given in this manual and the layout of the manual are regarded as interim. This manual is issued for engineers to use in the hope that users will provide comments and feed back on the procedures and their use. Comments are specifically invited on the useability of the manual and the procedures. Any comments relating to this manual should be sent to:

R&D Co-ordinator
NRA Thames Region
Kings Meadow House
Kings Meadow Road
Reading
Berks
RG1 8DQ

Comments must be received by January 1996. A revised manual will be prepared and issued in 1996.

2. PROCESSES WHICH AFFECT DISCHARGE CAPACITY OF COMPOUND CHANNELS

2.1 Processes In Straight Compound Channels

Much research has been carried out in recent years into the behaviour of flow in straight compound channels. There have been three basic aims of these projects:

- to collect stage-discharge data for overbank conditions in straight compound channels;
- to collect other hydraulic data such as bed shear stress and velocity distributions during overbank flow; and
- to identify the presence of complex structures which may affect the overall hydraulics.

Ackers (1991) and Wark (1993) have both carried out extensive literature surveys of experimental and theoretical research into straight compound channels. There is little doubt that the internal structure of flows in straight compound channels is complex. There is an interaction between the fast moving main channel and slower flood plain flows. In certain conditions this interaction can significantly affect the discharge capacity of a two stage channel.

Important mechanisms

The main mechanisms which affect the conveyance capacity of straight compound channels have been identified as follows:

1. The velocity differential between main channel and flood plain flows induces a lateral shear layer between these two regions.

2. Secondary circulations, both in plan and within the cross-section, carry fast moving fluid from the main channel to the flood plain and vice-versa. The relative strength of these secondary currents is reduced when the flood plain is rough and when the main channel side slope is slack. The most noticable secondary circulations form vortices with vertical axes located along the main channel / flood plain interfaces.

3. The secondary circulations and lateral shear effects cause the boundary shear stresses to be redistributed around the channel cross-section, with increased values at the edge of the flood plain close to the main channel.

4. These mechanisms combine to reduce the discharge in the main channel and increase it on the flood plains.

5. The secondary currents also affect the vertical and lateral distributions of longitudinal velocity, particularly in the main channel.

6. These interaction mechanisms are found to affect zones of the main channel and flood plain adjacent to the channel bank. In the case of narrow channels or flood plains these shear layers may extend across the whole channel or flood plain.

7. The approximate widths of the shear layers are proportional to the flow depth and turbulent viscosity and are inversely proportional to the bed friction factor.

8. The strength of the interaction depends on :

   main channel / flood plain widths, depths and side slopes;
   main channel / flood plain bed roughness and
   the velocity differential across the shear layer

9. The bed shear stress on the flood plains is increased by the interaction. In the main channel it is reduced.

10. The low depths and increased velocities on the flood plains close to the main channel may cause high (supercritical) values of the local Froude number. In these circumstances the local generation of surface waves will give rise to increased dissipation.

Figure 2.1 shows the important flow structures in a straight compound channel.

2.2 Processes in Meandering Compound Channels

Much less research has been carried out in to the behaviour of meandering compound channels than straight channels. The general view is that the mechanisms present are far more complex than in a straight channel and this has been reflected in slower progress in research, until recently. The SERC FCF Phase B work confirmed the presence of complex mechanisms and the method given in section 4.2 results directly from this research programme.

Methods of calculating discharge in straight compound channels have been applied to some of the data collected during phase B of the FCF work, see James and Wark (1992). The poor results obtained demonstrated that straight channel methods are not appropriate for use with meandering channels.

The task, therefore, was to develop a new procedure for the estimation of discharge in meandering channels. In order to carry out this work information on the behaviour of both inbank and out of bank meandering channels was required. Laboratory and field experiments were identified from the literature. Both the SERC FCF Phase B data and data from the University of Aberdeen were used in the development of the procedures. Other sets of laboratory data were used to verify the methods.

Important mechanisms

The internal structure of currents during overbank flows has been found to be highly complex. The available data has been reviewed by James and Wark (1992). The most important observations are:

1. The longitudinal velocities below bankfull tend to follow the main channel side walls while the floodplain velocities are generally in the valley direction. Thus the flood plain flows pass over the main channel and induce a horizontal shear layer.

2. In meandering compound channels the energy loss due to secondary currents in the main channel is greater than for an equivalent simple channel and the currents rotate in the opposite sense compared to inbank flows.

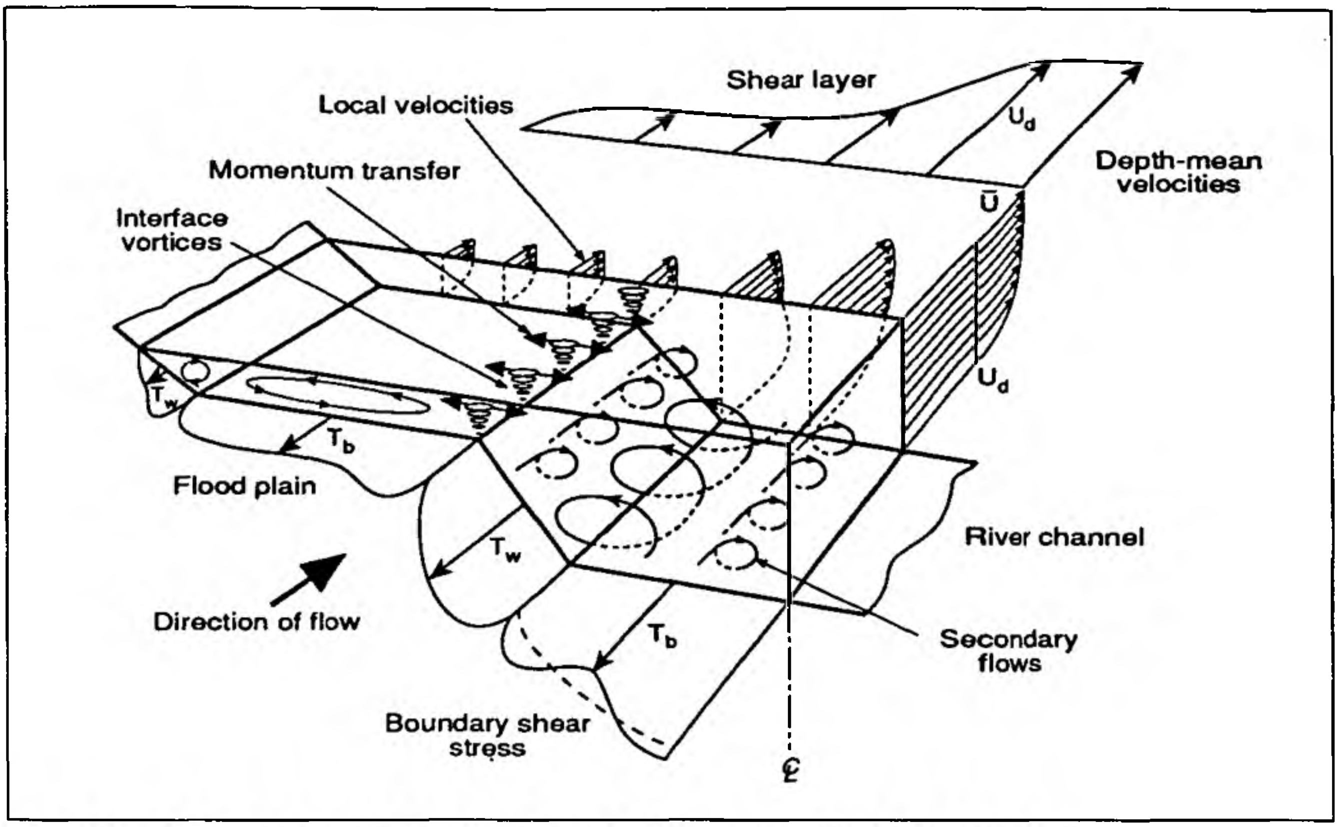
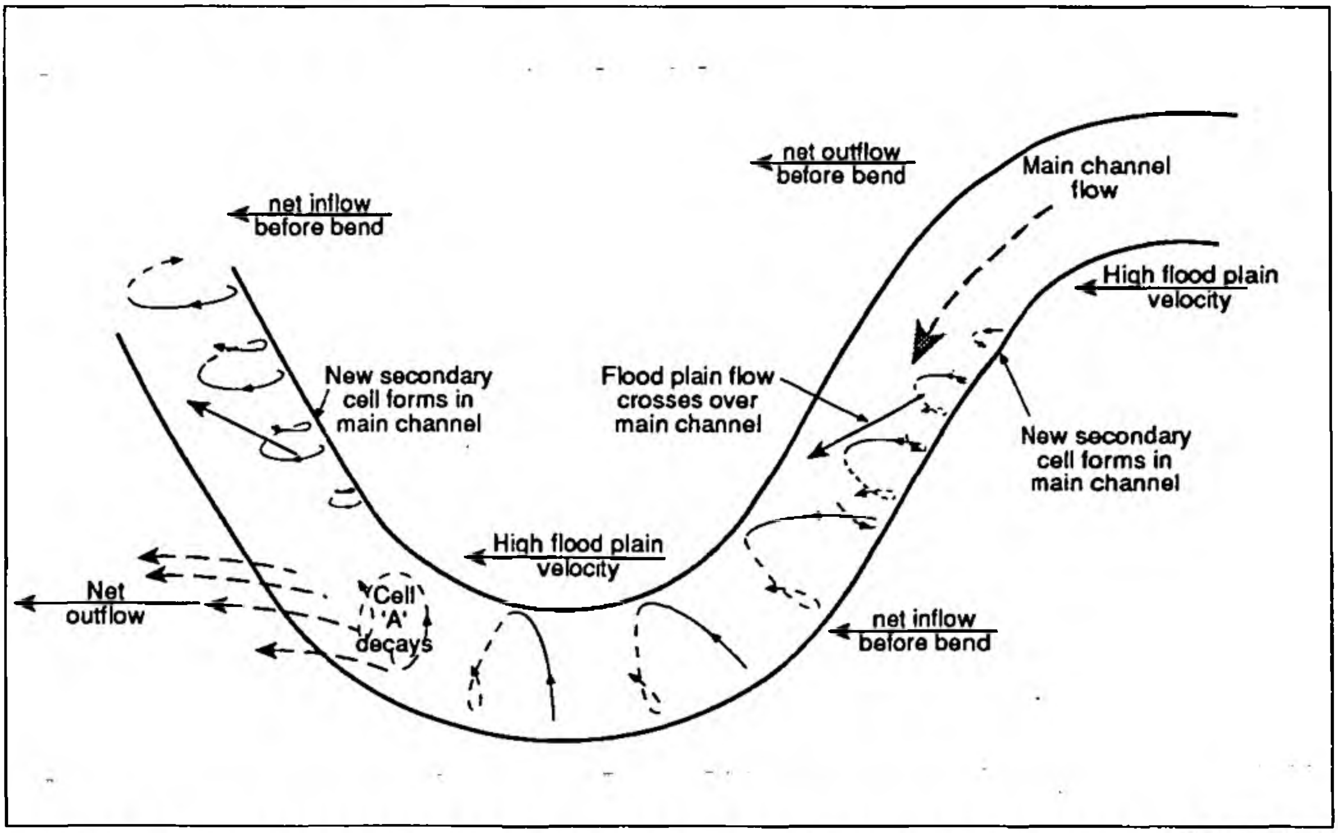


Fig 2.1 Flow processes in a straight compound channel (Shiono and Knight, 1991)



JW/2.1&2.2/9-93/DW

Fig 2.2 Flow processes in a meandering compound channel (Ervine and Jasem 1991).

3. Fluid passes from the main channel onto the flood plain and back into the main channel in the following meander bend. Hence the proportion of discharge passed by the main channel and flood plain varies along a meander wavelength. These bulk exchanges of fluid between slow and fast moving regions of flow introduce extra flow resistance.

4. Flows on the flood plain outside the meander belt are usually faster than those within the meander belt. It would appear that the extra flow resistance induced by the meandering main channel has a relatively small effect on the outer flood plain.

Figure 2.2 shows these mechanisms operating in a meandering compound channel.

3. GUIDELINES ON THE USE OF THIS MANUAL

3.1 General

The methods presented in this manual are based on laboratory data. There are at present insufficient data for real rivers to verify their generality. Natural rivers obviously have irregularities and features which have not been accounted for in the laboratory experiments, and it is not possible to make specific recommendations for all possible situations. The methods should not, therefore, be treated as rigorous, universal procedures; additional, unspecified decisions based on professional experience and judgement will be required in most applications. The following discussion deals with the problem of the transition from straight to meandering conditions.

3.2 Meandering Channels

The method presented here for meandering compound channels was developed and verified against laboratory data with main channel sinuosities in the range 1.09 to 2.04. The method was shown to perform well for all of this laboratory data and performed reasonably well for the limited amount of field data available, James and Wark (1992). However, applications to straight compound channel laboratory and field data show that the method developed for meandering compound channels under-predicts discharges by about 20% on average, with extreme cases being under-predicted by as much as 40%, James and Wark (1992). It was also shown that straight channel methods will over-predict discharges in meandering compound channels by as much as 50% to 60%, James and Wark (1992) or NRA R&D Project Record 252/2/T (1992).

3.3 Straight Channels

The James and Wark method for meandering compound channels has been verified successfully at sinuosities as low as 1.09 but does not accurately predict discharges in straight compound channels (sinuosity 1.00). The FCFA method, Ackers (1991), has been extended to cope with channels which are skewed to the flood plain by angles smaller than 10°, which corresponds to a sinuosity of 1.02. Unfortunately there is no available information on the behaviour of flow in channels with sinuosities between 1.09 and 1.02 on which to base recommendations as to when to switch from the meandering to a straight channel method. The recommendations given below are therefore not well documented and must be regarded as tentative.

3.4 Choice of Method

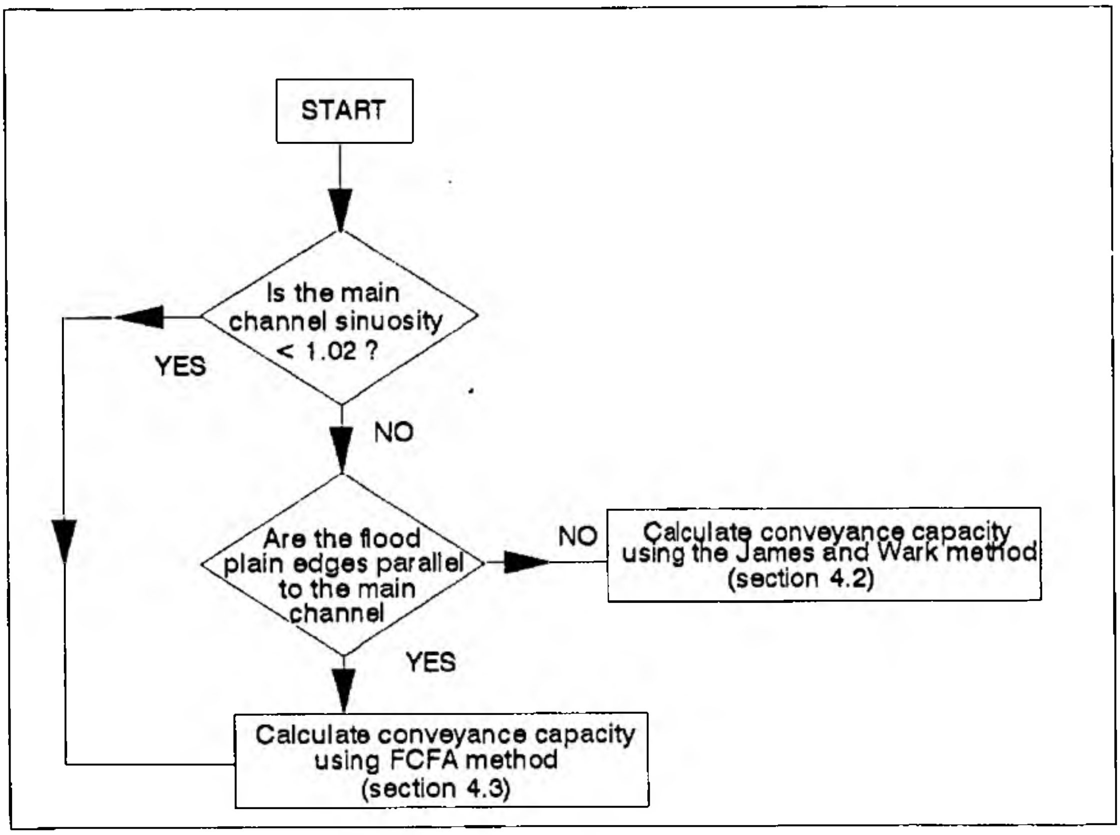
When the main channel sinuosity is less than 1.02 use the straight channel method, with appropriate corrections for sinuosity and for cases with main channel sinuosity greater than or equal to 1.02 use the James and Wark (1992) method. A straight channel method may also be appropriate if the lateral 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 laboratory or field evidence to verify this argument and this aspect of flows in meandering compound channels is still open to conjecture.

4. CALCULATION PROCEDURES

4.1 Guidelines for Use of Methods

Specific guidelines are given in sections 4.2 and 4.3 for the meandering and straight compound channel methods respectively. The main purpose of the guidlines given here is to assist the user in the choice of appropriate method. The method should be chosen by answering the two questions in the flow chart below.



Calculation of Darcy friction factor

The step by step procedures and worked examples given below have been drafted so that the user may use Manning's equation directly, rather than calculating a Darcy f value. This has been done because Manning's equation is the the most prevalent method used by practising engineers to calculate the effects of bed friction. This approach is entirely appropriate for the majority of field applications but other approaches, such as the rough or smooth turbulence versions of the Colebrook-White equation may be more appropriate in special circumstances. Annex E gives the relationships between the various common approaches to bed friction. In particular for Manning's Equation it is:

$f = (8\ g\ n^2) / R^{1/3}$

4.2 Design of Meandering Compound Channels

4.2.1 The James and Wark method

The philosophy behind the development of this method for meandering compound channels is summarised in annex A and James and Wark (1992). The approach is to divide the channel cross section into the four zones defined by Figure 4.1.

Zone	**Description**
1	Main channel below bankfull stage
2	Inner flood plain zone
3,4	Outer flood plain zones on left and right of the main channel.

The discharge associated with each channel zone is calculated according to the given procedures and summed to obtain the total discharge in the channels. The equations are summarised below and a step by step procedure for application is defined.

The zonal discharge equations

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

$$Q_T = Q_1 + Q_2 + Q_3 + Q_4 \tag{4.1}$$

Zone 1 : main channel

The correct flow in zone 1 is given by

$$Q_1 = Q_{bf}\, Q_1' \tag{4.2}$$

Where the bankfull discharge, Q_{bf}, is calculated using standard hydraulic formulae, section 4.2.2, step 2 and the adjustment factor (Q_1') is the **greater** of :

$$Q_1' = 1.0 - 1.69\, y' \tag{4.3}$$

or

$$Q_1' = m\, y' + K c \tag{4.4}$$

with

$$m = 0.0147\, B^2/A + 0.032\, f' + 0.169 \tag{4.5}$$

$$c = 0.0132\, B^2/A - 0.302\, s + 0.851 \tag{4.6}$$

$$K = 1.14 - 0.136\, f' \tag{4.7}$$

$$y' = y_2 / (A/B) \tag{4.8}$$

$$f' = f_2 / f_1 \tag{4.9}$$

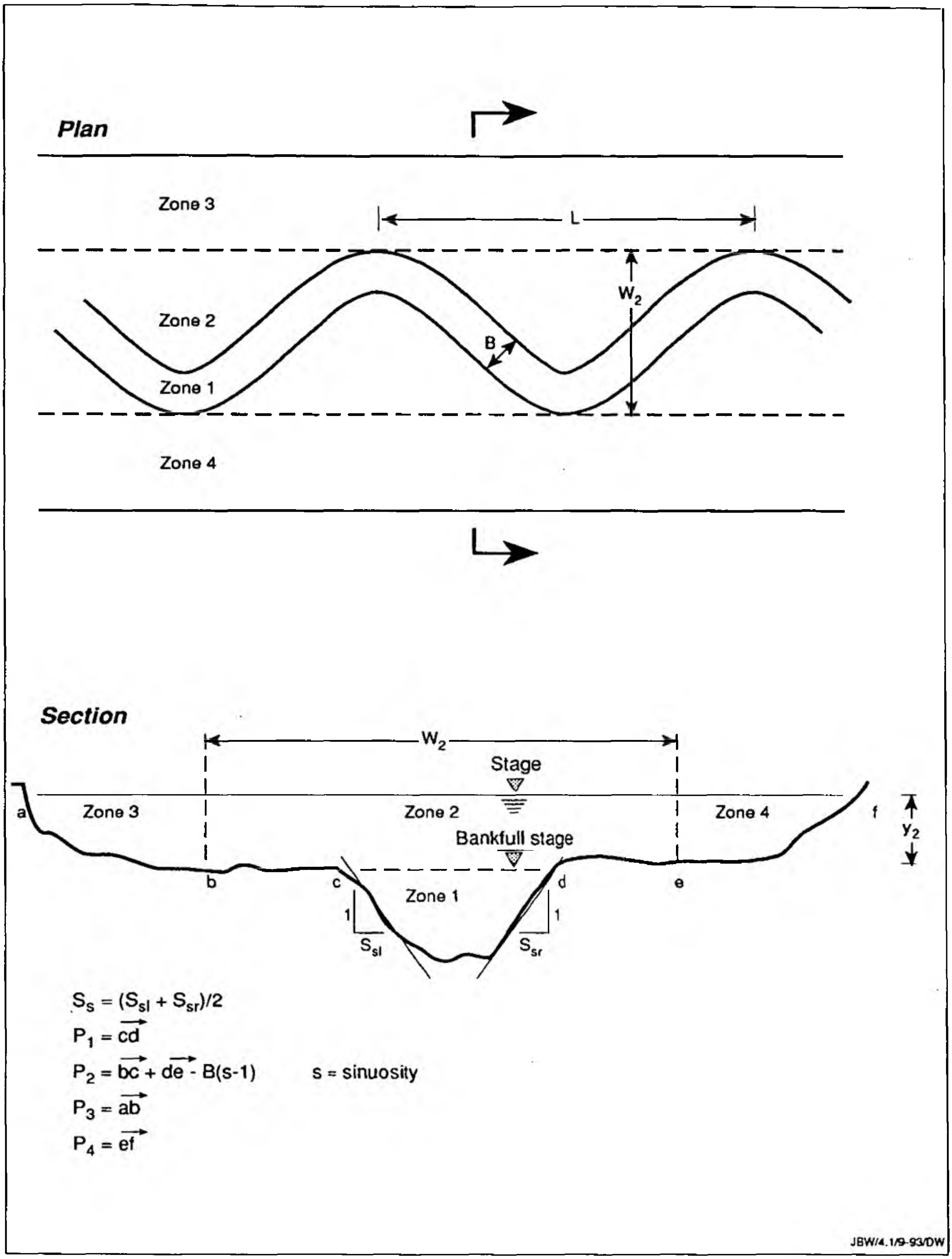


Figure 4.1 Cross-section subdivision of overbank flows, meandering channels

Zone 2 : inner flood plain

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

$$Q_2 = A_2 V_2 \qquad (4.10)$$

in which

$$V_2 = \left(\frac{2 g S_o L}{(f_2 L) / (4 R_2) + F_1 F_2 K_c} \right)^{1/2} \qquad (4.11)$$

with

$$F_1 = 0.1 B^2/A \qquad \text{for } B^2/A < 10$$
$$F_1 = 1.0 \qquad \text{for } B^2/A \geq 10 \qquad (4.12)$$

$$F_2 = s/1.4 \qquad (4.13)$$

$$K_c = C_{sl} C_{wd} (C_{suc} (1 - y_2/(y_2 + h))^2 + C_{sac} K_c') \qquad (4.14)$$

K_c' is the basic contraction coefficient, as given in Table 4.1.

Table 4.1 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

$$C_{sl} = 2(W_2 - B)/W_2 \qquad (4.15)$$

$$C_{wd} = 0.02 (B^2/A) + 0.69 \qquad (4.16)$$

$$C_{suc} = 1.0 - S_s / 5.7 \qquad \text{(but } C_{suc} \text{ not less than 0.1)} \qquad (4.17)$$

$$C_{sac} = 1.0 - S_s / 2.5 \qquad \text{(but } C_{sac} \text{ not less than 0.1)} \qquad (4.18)$$

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.

$$Q_3 = A_3 V_3$$
$$Q_4 = A_4 V_4 \qquad (4.19)$$

where

$$V_3 = (8 g R_3 S_o / f_3)^{1/2}$$
$$V_4 = (8 g R_4 S_o / f_4)^{1/2} \qquad (4.20)$$

Bed shear stresses

For the design of scour protection, it is recommended that boundary shear stresses be determined for the main channel bed and banks for the full range of inbank stages, using currently available methods. In addition, the banks should be able to resist stresses of

$$\tau = 1.6\,\gamma\,y_2\,S_o \qquad (4.21)$$

on the upstream side, and

$$\tau = 5\,\gamma\,y_2\,S_o \qquad (4.22)$$

on the downstream side.

Notation for James and Wark method

		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	m
K	coefficient in equation for zone 1 adjustment factor	-
K_e	factor for expansion and contraction losses in zone 2	-
K_c	contraction coefficient	-
L	meander wavelength	m
m	coefficient in equation for zone 1 adjustment factor	-
n	coefficient in Manning's equation	$m^{1/3}/s$
n′	coefficient in Manning's equation, including bend losses	$m^{1/3}/s$
P	wetted perimeter	m
P	unsubscripted, wetted perimeter of main channel at bankfull	m
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 (length along centreline of main channel / straight length)	-
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 at main channel bank	m
y'	dimensionless flow depth on flood plain = $y_2/(A/B)$	-
ρ	density of water (approximately 1000 kg/m^3)	kg/m^3
γ	unit weight of water (approximately 9.81 x 10^3 N/m^3)	N/m^3
τ	boundary shear stress	N/m^2

Subscripts

1-4 zones 1 to 4

Procedure for application of the James and Wark method

In a typical application the following are required:

- the capacity of the main channel at bankfull;
- the zonal and total discharges when the water level is above bankfull level; and
- values of boundary shear stress for designing scour protection of the main channel banks when the water surface is above bankfull level.

The worked example in section 4.2.2 follows the step by step procedure given below. The overall methods are relatively straight forward and detailed comments are included in the worked example.

Step	Description
1.	Define cross-section zones and calculate the necessary geometric parameters
1.1	Obtain or calculate main channel area, wetted perimeter, top width
1.2	Calculate main channel sinuosity
1.3	Calculate main channel longitudinal slope and side slope
1.4	Obtain or calculate inner flood plain area, wetted perimeter and width
1.5	Obtain or calculate outer flood plain areas and wetted perimeters
1.6	Obtain or calculate bed friction values from either existing guidelines or site data
2.	Calculate the capacity of the main channel at bankfull stage
3.	Calculate the discharge for chosen water level
3.1	Calculate zone 1 discharge
3.2	Calculate zone 2 discharge
3.3	Calculate zone 3 discharge
3.4	Calculate zone 4 discharge
3.5	Calculate total discharge
4.	Calculate maximum bank shear stresses
4.1	Calculate maximum shear stress on upstream banks
4.2	Calculate maximum shear stress on downstream banks

4.2.2 Worked example for James and Wark method

Problem definition

The conveyance of a two-stage river channel is to be determined. The reach under consideration is shown in Figure 4.2 and is represented by the surveyed cross-section at the location indicated, which is presented in Figure 4.3. The slope of the flood plain is estimated as 0.0014. Manning's n values for the main channel and flood plains are estimated as 0.025 and 0.045 respectively, based on the observed surface roughnesses.

The following are required:

- the capacity of the main channel at bankfull;
- the zonal and total discharges when the water level is 1.2 m above bankfull level; and
- values of boundary shear stress for designing scour protection of the main channel banks when the water surface is 1.2 m above bankfull level.

Solution

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

The zone subdivisions are shown in Figures 4.4 and 4.5. Because the geometry varies along the reach the positions of the subdivision planes are selected by judgement to represent average conditions over the reach. From the geometries defined by this subdivision, the following geometric characteristics are calculated for the water surface 1.2 m above bankfull.

Step 1.1 Obtain or calculate main channel area, wetted perimeter, top width

$A = 5.07\ m^2$
$P = 6.40$ m
$B = 6.10$ m *from survey*

Step 1.2 Calculate main channel sinuosity

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 4.4, 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: $1.0 \le s \le 1.02$ then we should use the straight channel method, section 4.3 with appropriate corrections for sinuosity.

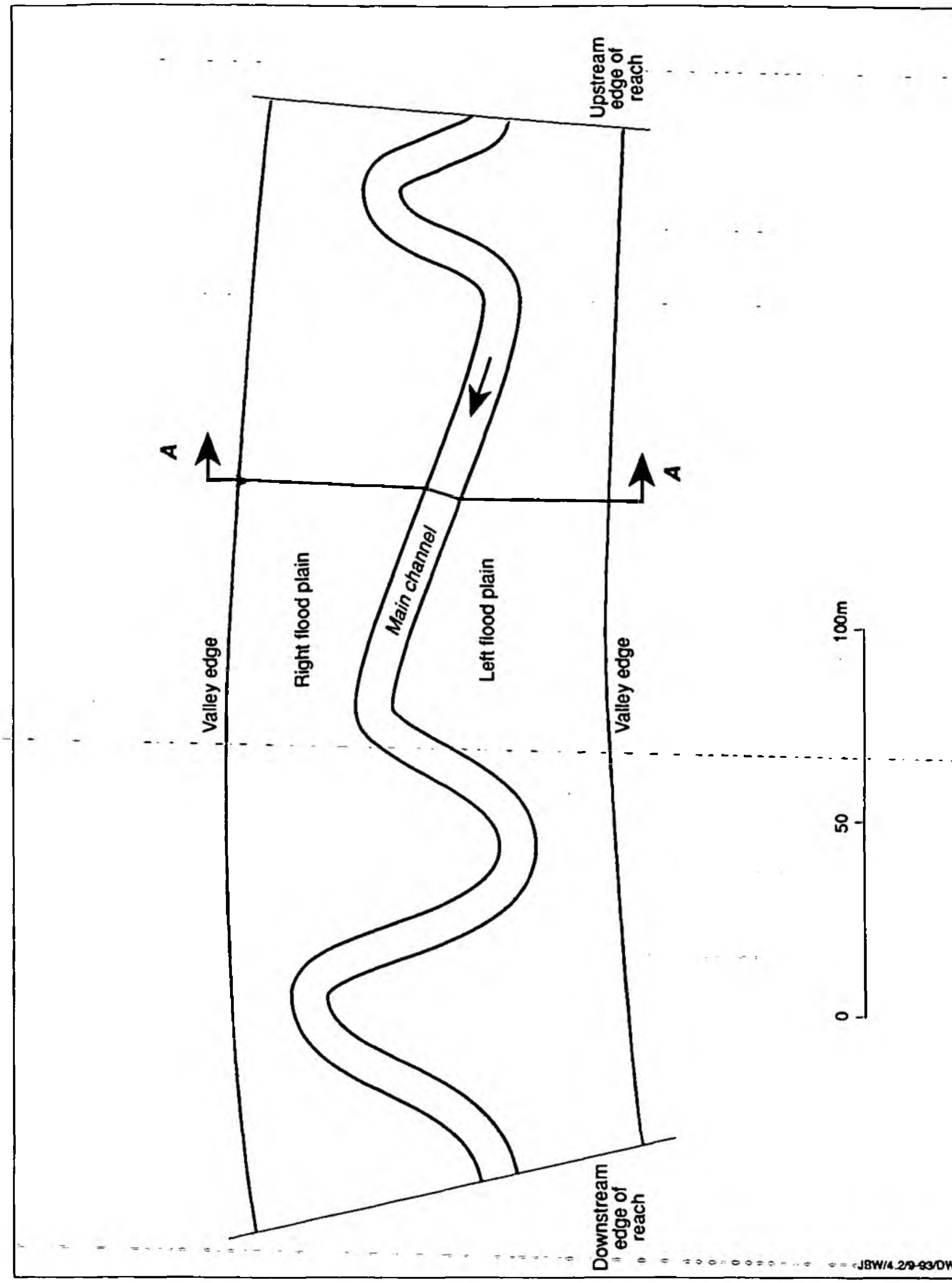


Figure 4.2 Plan of problem reach

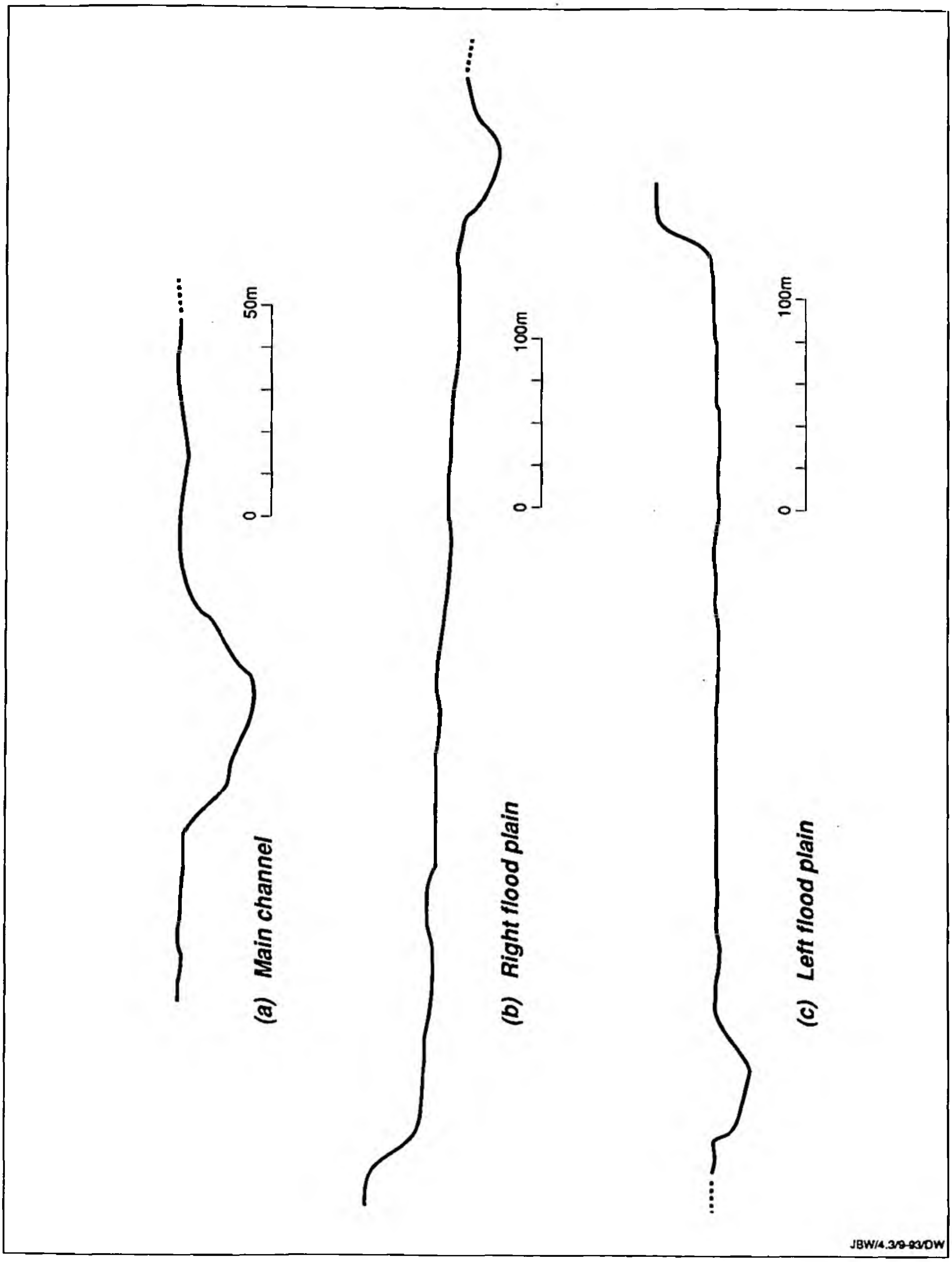


Figure 4.3 Cross-section (A-A) through problem reach (looking upstream)

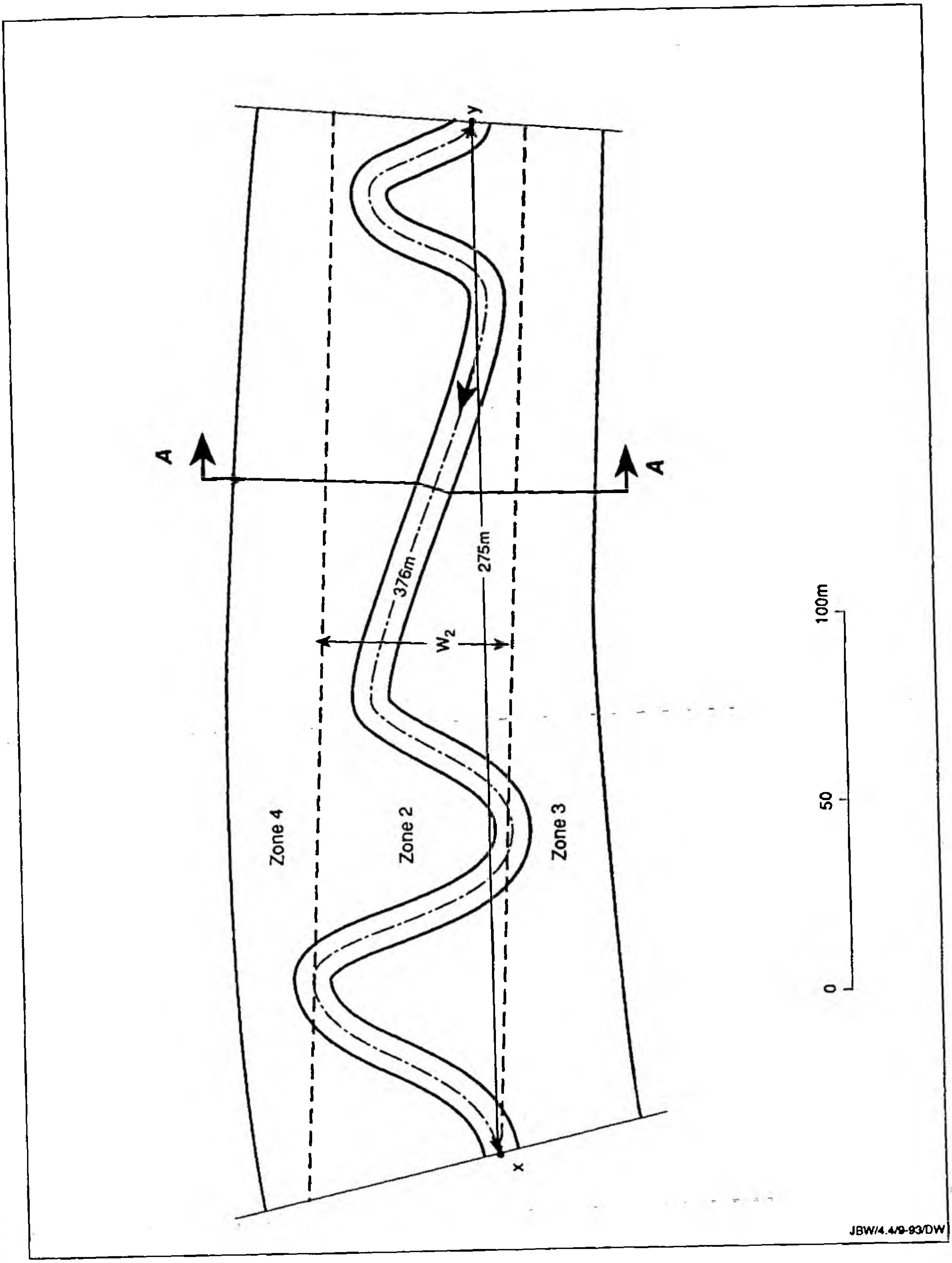


Figure 4.4 Plan of problem reach with zones shown

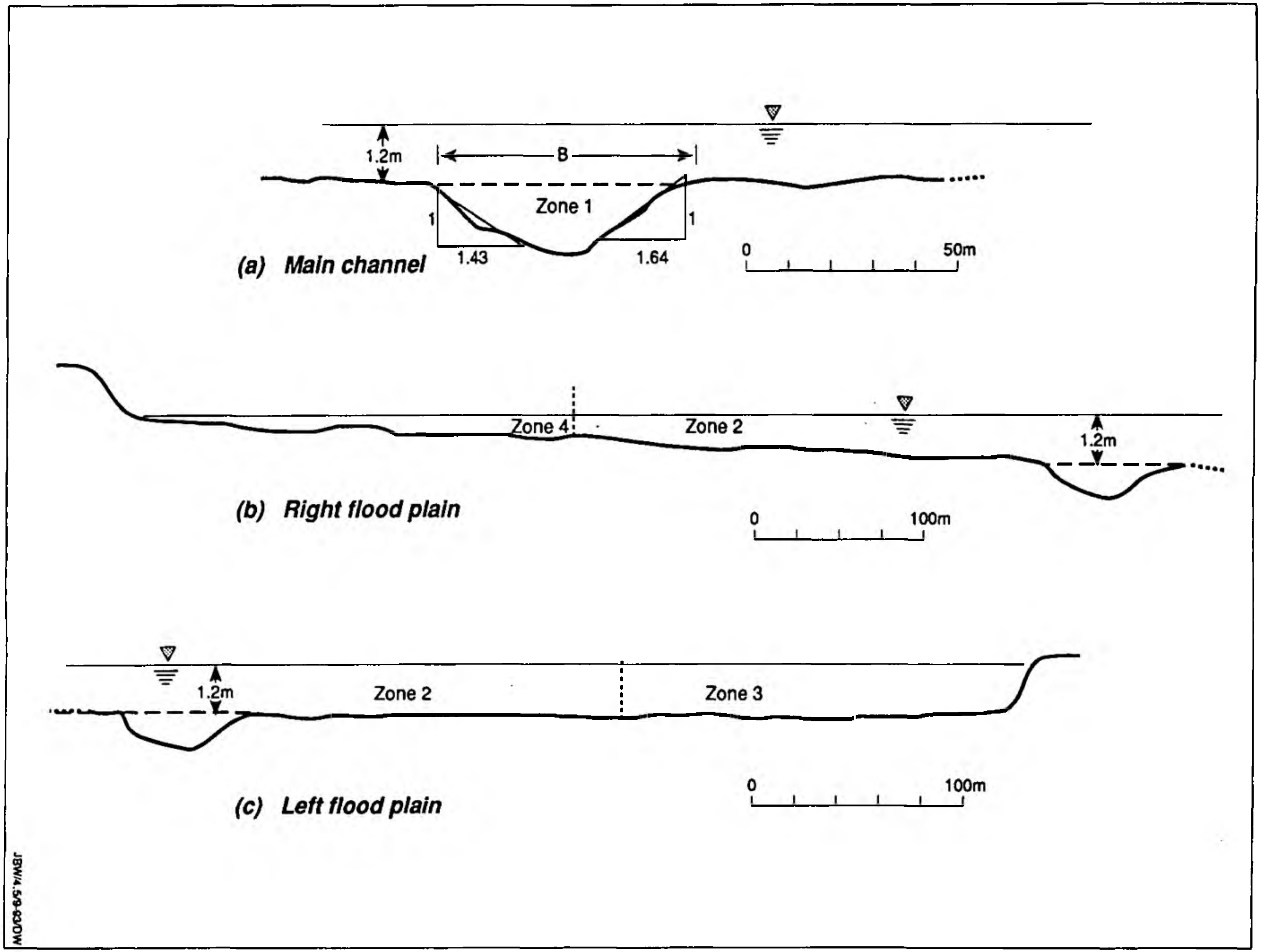


Figure 4.5 Cross-section through problem reach (looking upstream)

Step 1.3 Calculate main channel longitudinal slope and side slope

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

$S = 0.0014 / 1.37 = 0.00102$

The main channel side slopes are measured on the cross-section reproduced in Figure 4.5. The average of the values for the right and left banks will be used in the calculations, i.e.

$S_s = (1.43 + 1.64) / 2 = 1.54$

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

Step 1.4 Obtain or calculate inner flood plain area, wetted perimeter and width

$A_2 = 47.77\ m^2$ *from survey*

The wetted perimeter is calculated excluding the division planes, i.e.

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

$P_2 = 22.48 + 17.72 - 6.10 \times (1.37 - 1.00) = 37.94\ m$

$W_2 = 49.40\ m$ *from survey*

Step 1.5 Obtain or calculate outer flood plain areas and wetted perimeters

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 stage

$Q_{bf} = A\,V$

$A = 5.07\ m^2$ *from Step 1*

V is calculated using Manning's equation,

$V = (R^{2/3}\,S^{1/2})\ /\ n_1$

The coefficient n_1 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 A1.1 in Annex 1, i.e.

$n' = n_1\,(0.43\,s + 0.57)$

$= 0.025 \times (0.43 \times 1.37 + 0.57) = 0.029$

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

$V = (1 / 0.029) \times 0.792^{2/3} \times 0.00102^{1/2} = 0.943\ m/s$

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 1.2 m above bankfull

Step 3.1 Calculate zone 1 discharge

$Q_1 = Q_1'\,Q_{bf}$ *(eqn 4.2)*

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

The zone 1 adjustment factor, Q_1', is the greater of the values given by equations 4.3 and 4.4

$Q_1' = 1.0 - 1.69\,y'$ *(eqn 4.3)*

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

$= 1.20 / (5.07 / 6.10) = 1.44 \quad \therefore Q_1' = 1.0 - 1.69 \times 1.44 = -1.44$

$Q_1' = m\, y' + K\, c$ *(eqn 4.4)*

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

$B^2/A = 6.10^2 / 5.07 = 7.34$

$f' = (n_2 / n_1)^2\, (R / R_2)^{1/3}$ *(Annex A)*

$R = 0.792$ m *from Step 2*

$R_2 = A_2 / P_2 = 47.77 / 37.94 = 1.259$ m

$f' = (0.045 / 0.025)^2 \times (0.792 / 1.259)^{1/3} = 2.78$

Therefore

$m = 0.0147 \times 7.34 + 0.032 \times 2.78 + 0.169 = 0.366$

$K = 1.14 - 0.136\, f'$

$= 1.14 - 0.136 \times 2.78 = 0.762$

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

$= 0.0132 \times 7.34 - 0.302 \times 1.37 + 0.851 = 0.534$

Therefore

$Q_1' = 0.366 \times 1.44 + 0.762 \times 0.534 = 0.934$

which is greater than the value given by equation 4.3

Therefore the discharge in zone 1 is

$Q_1 = 0.934 \times 4.78 = 4.46\ m^3/s$

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

$Q_1 = 4.5\ m^3/s$

Step 3.2 Calculate zone 2 discharge

$Q_2 = A_2\, V_2$ *(eqn 4.10)*

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

$$V_2 = \left(\frac{2\, g\, S_o\, L}{(f_2\, L) / (4\, R_2) + F_1\, F_2\, K_e} \right)^{1/2}$$ *(eqn 4.11)*

The average meander wavelength is estimated from Figure 4.4 by dividing the flood plain length by the number of wavelengths over the reach, i.e.

$L = 275 / 3 = 91.7$ m

$R_2 = 1.259$ m *from Step 3.1*

$f_2 = (8\, g\, n_2^2) / R_2^{1/3}$ *(Annex A)*

$= (8 \times 9.81 \times 0.045^2) / 1.259^{1/3} = 0.147$

$F_1 = 0.1\, B^2/A$ *(eqn 4.12)*

$B^2/A = 7.34$ *from Step 3.1*

$= 0.1 \times 7.34 = 0.734$

$F_2 = s / 1.4$ *(eqn 4.13)*

$= 1.37 / 1.4 = 0.979$

$K_e = C_{sl}\, C_{wd}\, (C_{soe}(1 - y_2/(y_2 + h))^2 + C_{sac}\, K_c)$ *(eqn 4.14)*

$C_{sl} = 2\, (W_2 - B) / W_2$ *(eqn 4.15)*

$= 2 \times (49.4 - 6.10) / 49.4 = 1.753$

$C_{wd} = 0.02\, B^2/A + 0.69$ *(eqn 4.16)*

$B^2/A = 7.34$ *from Step 3.1*

$C_{wd} = 0.02 \times 7.34 + 0.69 = 0.837$

$C_{soe} = 1.0 - S_s / 5.7$ *(eqn 4.17)*

$= 1.0 - 1.54 / 5.7 = 0.730$

$C_{sac} = 1.0 - S_s / 2.5$ *(eqn 4.18)*

$= 1.0 - 1.54 / 2.5 = 0.384$

$h = A / B$

$= 5.07 / 6.10 = 0.831$ m

$y_2 / (y_2 + h) = 1.2 / (1.2 + 0.831) = 0.591$

$K_c = 0.217$ *Table 4.1*

Therefore

$K_e = 1.753 \times 0.837 \times (0.730 \times (1 - 0.591)^2 + 0.384 \times 0.217) = 0.301$

Therefore

$$V_2 = \left(\frac{(2 \times 9.81 \times 0.0014 \times 91.7)}{((0.147 \times 91.7) / (4 \times 1.259) + 0.734 \times 0.979 \times 0.301)} \right)^{1/2}$$

$= 0.933$ m/s

Therefore the discharge in zone 2 is

$Q_2 = 47.77 \times 0.933 = 44.57\ m^3/s = 44.6\ m^3/s$

Step 3.3 Calculate zone 3 discharge

$Q_3 = A_3 V_3$

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

V_3 is calculated using Manning's equation,

$V_3 = (1 / n_3)\ R_3^{2/3}\ S_o^{1/2}$

$n_3 = 0.045$

$R_3 = A_3 / P_3$

$= 16.28 / 18.90 = 0.861$ m

Therefore

$V_3 = (1 / 0.045) \times 0.861^{2/3} \times 0.0014^{1/2} = 0.753$ m/s

Therefore the discharge in zone 3 is

$Q_3 = 16.28 \times 0.753 = 12.26\ m^3/s = 12.3\ m^3/s$

Step 3.4 Calculate zone 4 discharge

$Q_4 = A_4 V_4$

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

V_4 is calculated using Manning's equation,

$V_4 = (1 / n_4)\ R_4^{2/3}\ S_o^{1/2}$

$n_4 = 0.045$

$R_4 = A_4 / P_4$

$= 8.00 / 21.00 = 0.381$ m

Therefore

$V_4 = (1 / 0.045) \times 0.381^{2/3} \times 0.0014^{1/2} = 0.437$ m/s

Therefore the discharge in zone 4 is

$Q_4 = 8.00 \times 0.437 = 3.5$ m³/s

Step 3.5 Calculate total discharge

$Q_T = Q_1 + Q_2 + Q_3 + Q4$ *(eqn 4.2)*

$= 4.5 + 44.6 + 12.3 + 3.5 = 64.9$ m³/s

Hence the total discharge in the channel is 65 m³/s

Step 4. Calculate maximum bank shear stresses

The distribution of boundary shear should be determined by simulation (or other appropriate methods) for all stages below bankfull to establish values for the design of bank protection. In addition, local concentrations during overbank flow events must be allowed for.

Step 4.1 Calculate maximum shear stress on upstream banks

Upstream banks must be able to resist

$\tau = 1.6\ \gamma\ y_2\ S_o$ *(eqn 4.19)*

$= 1.6 \times 9.81 \times 10^3 \times 1.20 \times 0.0014 = 26.4$ N/m²

Step 4.2 Calculate maximum shear stress on downstream banks

Downstream banks must be able to resist

$\tau = 5\ \gamma\ y_2\ S_o$ *(eqn 4.20)*

$= 5 \times 9.81 \times 10^3 \times 1.20 \times 0.0014 = 82.4$ N/m²

For engineering purposes these values should be rounded up to say 30 N/m² and 85 N/m². Because of the uncertainty of the locations of the shear stress concentrations, protection should extend over the regions shown on Figure 4.6.

4.2.3 General comments

1. The above example involved the estimation of some geometric parameters (L and S_v). The final solution is expected to be relatively insensitive to small variations in these parameters and great accuracy in their determination is not necessary.

2. For this example the bed roughness (Manning's n) values were given. In practice the engineer will have to estimate these values. Users should be aware that the estimation of bed roughness is not exact and represents a significant potential source of error when carrying out any hydraulic calculations.

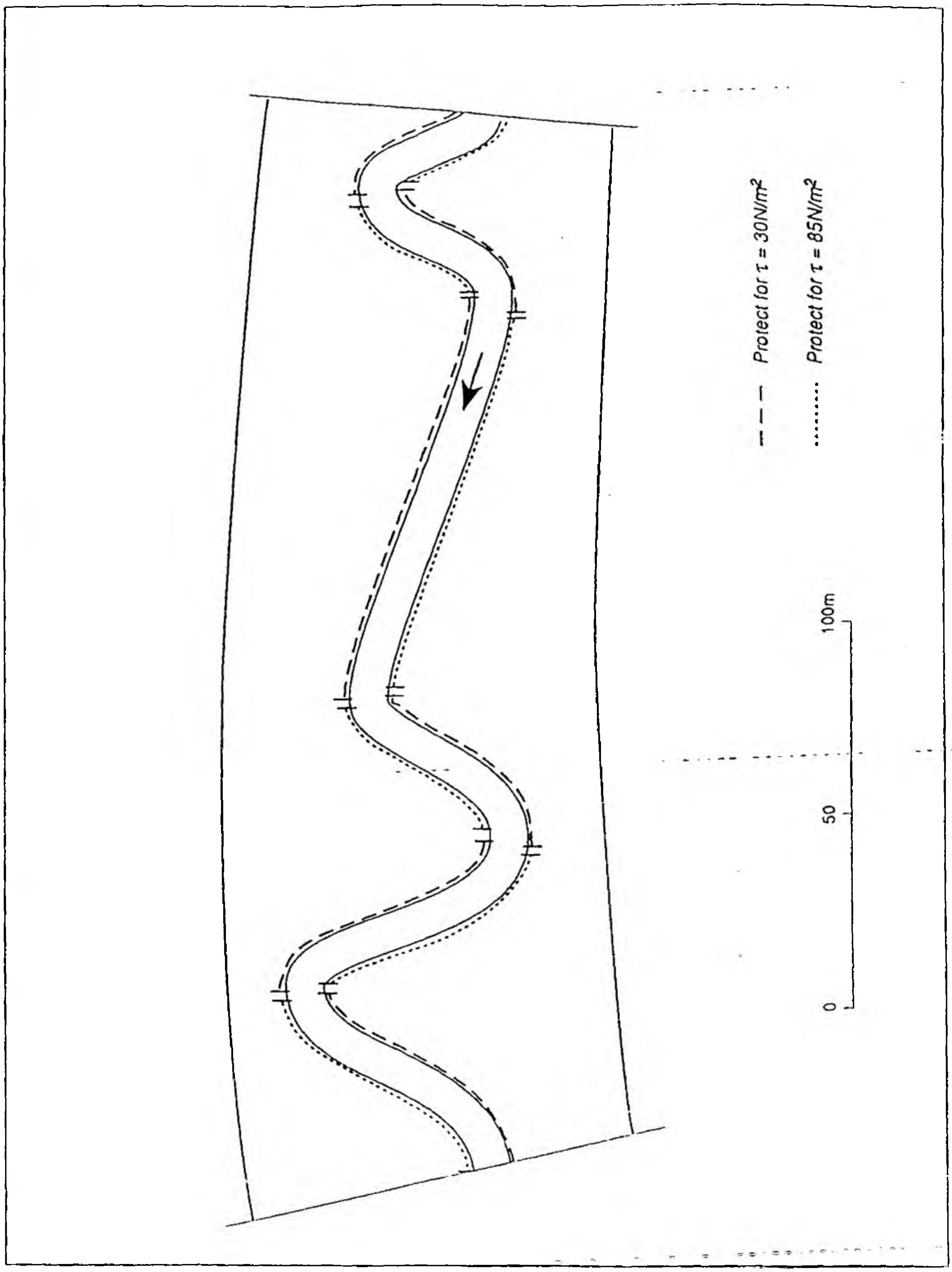


Figure 4.6 Bank scour protection

3. The method was developed using laboratory data. These laboratory channels were designed to have well defined meandering geometries, which were well suited to the method. Natural or man-made channels in the field are unlikely to match simplified laboratory channels in all respects. This means that the engineer will have to apply his own judgement in determining some of the geometric parameters such as: side slopes, width of zone 2, bankfull stage etc.

4. Although the calculations above have been quoted to two or three decimal places the user should be aware that the true level of accuracy of the calculations is much less and the final solution should be rounded off.

4.3 Design of Straight Compound Channels

4.3.1 The FCFA method

The philosophy behind the development of this method for straight compound channels is summarised in annex B and Ackers (1991). The approach is to divide the channel into the three zones shown in Figure 4.7.

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

The basic discharges for each zone are calculated and summed to obtain the total basic discharge. The final total discharge is obtained by correcting the basic discharge using the equations developed by Ackers (1991). Ackers identified four regions of flow behaviour where the correction functions differ, Figure 4.8. Extensive calculations are required to reach the final solution and the equations are presented below as part of a step by step procedure.

The zonal discharge equations

In a typical application the following are required:

- the capacity of the main channel at bankfull;
- the zonal and total discharges when the water level is above bankfull level; and
- values of boundary shear stress for designing scour protection of the main channel banks when the water surface is above bankfull level.

The steps which follow outline the procedure for computing discharge values corresponding to specified water levels.

Step	Description
1 to 3	Define the physical characteristics of the channel reach and cross-section in terms of the variables required for the subsequent calculations.
4 to 6	Compute the basic discharges for the main channel, flood plains, and the whole cross-section for a specified water level.

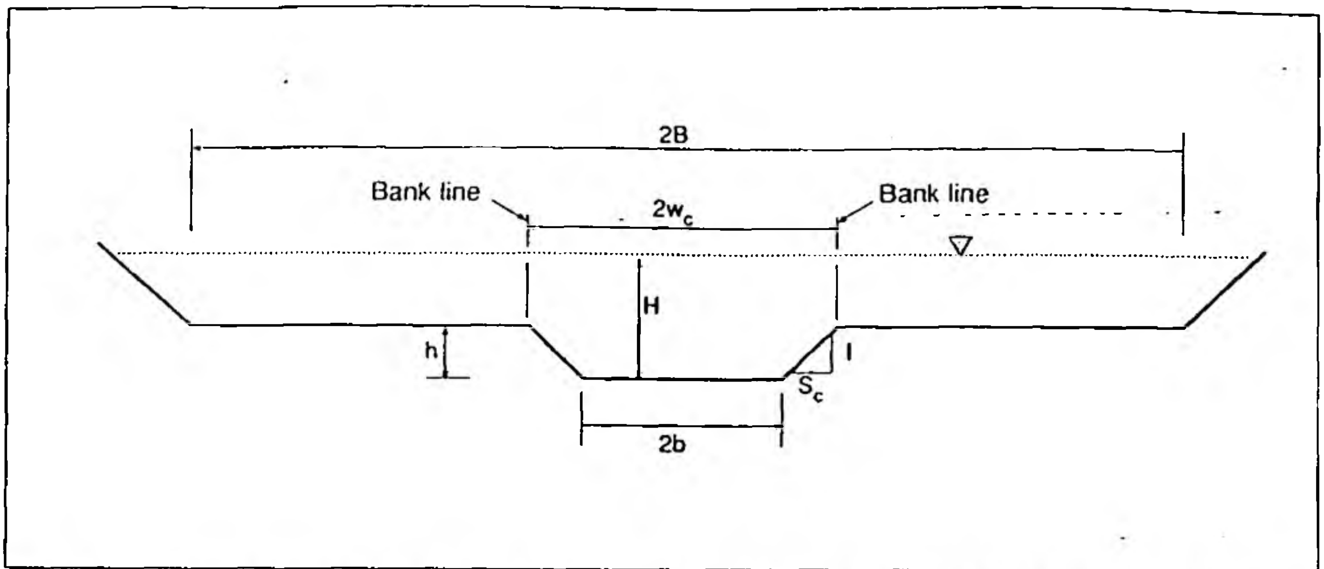


Fig 4.7 Compound channel cross-section with definition of variables

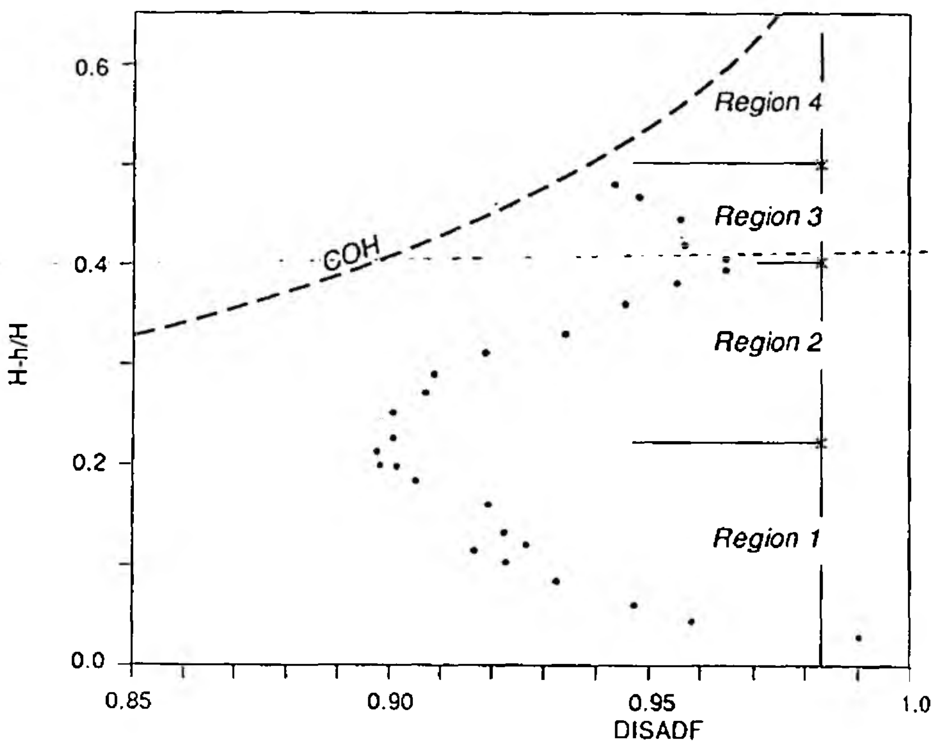


Fig 4.8 Regions of flow behaviour

7, 8 10 and 12	Adjust the basic total discharge to account for flow interaction between the main channel and flood plains, assuming the flow to be in Regions 1, 2, 3 and 4 respectively.
9, 11 and 13	Identifying the correct flow region and hence calculate the correct adjusted discharge. The adjustment and selection steps are interspersed so that the correct region is identified at the earliest opportunity, to avoid unnecessary calculations.
14	Applies the additional correction to account for deviation between the main channel and flood plain alignments.
15, 16	Calculate the adjusted zonal discharges.
17, 18	Calculate the maximum boundary shear stresses.

Preliminary investigations suggest that most UK rivers with compound sections will flow in Regions 1 or 2 for floods with recurrence intervals up to about 20 years. Calculations should be carefully checked if higher regions are indicated. Artificial or modified channels may operate over a wider range of regions than natural ones.

Procedure for application of the FCFA method

Step 1. Determine the longitudinal gradient of the channel reach, S_o, from survey information

Step 2. Determine the geometric variables required to define the adjustment functions

The basic discharges for the main channel and flood plain zones can be computed using flow areas and wetted perimeters obtained directly from the appropriate surveyed cross-section. The discharge adjustment functions, however, include the geometric variables defined in Figure 4.7, and their estimation requires representation of the cross-section by a basic trapezoidal geometry. This is done using the following steps.

Step 2.1 Plot the surveyed cross-section, as illustrated by Figure 4.9, for example

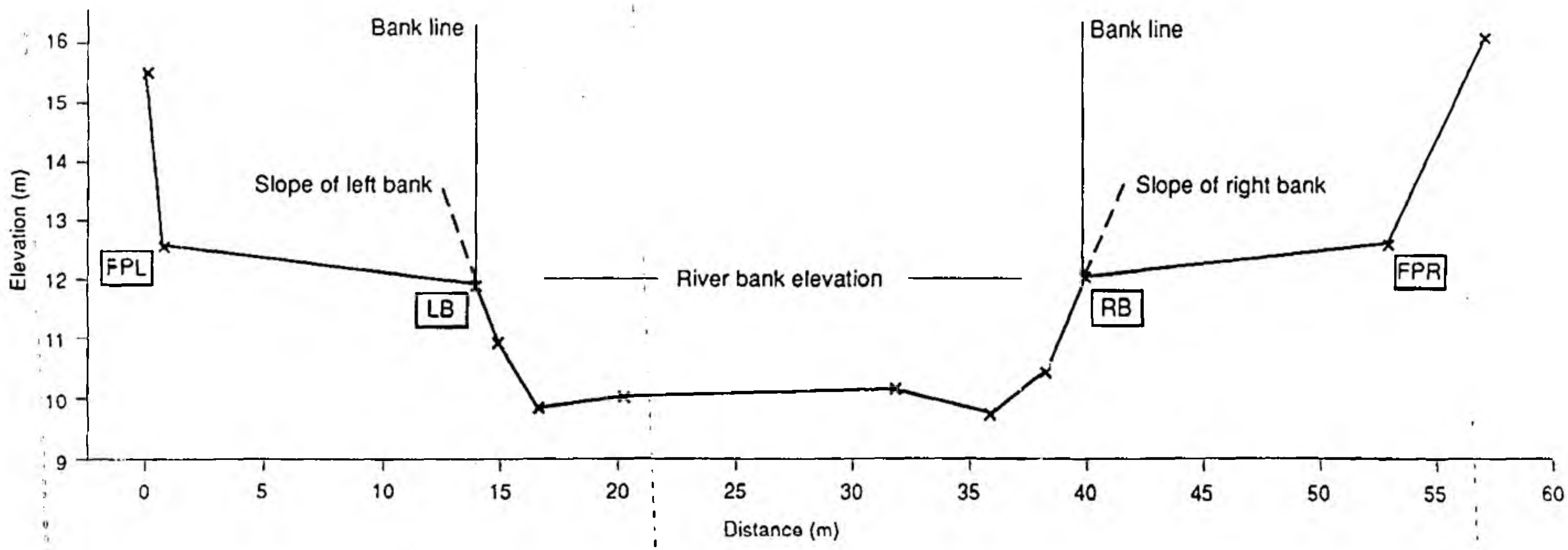
Step 2.2 Identify the points on the cross-section which mark the divisions between the main channel and the flood plains on both sides

Draw vertical lines through these points to define the bank lines separating the main channel and flood plain zones. The distance between the bank lines is $2w_C$. If there is a flood plain on one side of the main channel only, then just one bank line is defined and w_C is half the main channel width at the level of the division point.

Step 2.3 Determine the river bank elevation

This is defined by the bank elevations at the locations of the bank lines - one value if there is only one flood plain, and the average of the two values for two flood plains.

Fig 4.9 Surveyed cross-section



Step 2.4 Fit a uniform slope to the main channel bank on each side

If the banks are irregular and the actual slopes vary, fit the straight lines to the upper two thirds of the bank profiles. The average of these slopes, expressed as ratios of horizontal to vertical distances, defines s_C.

Step 2.5 Calculate the cross-sectional area of the main channel

Area below the river bank elevation (as determined in step 2.3 above) and between the bank lines, A_{Csurv}, from the surveyed cross-section.

Step 2.6 Determine the depth of the main channel, h

This is the distance below the river bank elevation of a horizontal channel bed located so that the area of the trapezium defined by the bed, the top width ($2w_C$) at the river bank elevation, and the side slopes (s_C), is the same as A_{Csurv}. It can be calculated as

$$h = [\, 2w_C \pm \{((2w_C)^2 - 4\, s_C\, A_{Csurv})\}^{0.5}\,] / 2s_C \qquad (4.23)$$

It will be obvious which of the two solutions of this equation is correct.

Step 2.7 Determine the bottom width of the main channel

$$2b = 2w_C - 2\, h\, s_C \qquad (4.24)$$

Step 2.8 Identify the positions of the backs of the flood plains

The distance between these defines the maximum total compound channel width, 2B, for two flood plains. For one flood plain the maximum value of B is the distance from the back of the flood plain to the bank line, plus w_C. Note that if the flood plains slope upwards and are not completely inundated, the total width (2B) is less than the maximum, with the dry part ignored (see Figure 4.9). The limits of the water surface can be determined from the surveyed cross-section.

Step 3. Estimate roughness coefficients for the main channel and flood plains

The resistance equation used is a matter of personal choice. Manning's equation (with corresponding n values) is probably the most widely accepted and will be used for describing the procedure, although this does not necessarily imply recommendation for its general use. If measured stage-discharge data are available, they should be used to estimate roughness coefficients. For the main channel, the value (n_C) adopted should correspond to near bank-full flows. It is not possible to infer the value for the flood plains (n_F) directly from measured data; a value must be assumed, which can be checked subsequently and refined. The slope used for calculating the n values should be the hydraulic gradient, but if reliable measurements of this are not available the surveyed channel gradient (S_o) can be used. If no measured data are available, n_C and n_F should be estimated in the usual way.

Step 4. Specify a value for H, the flow depth measured above the idealized bed of the main channel

The steps that follow lead to an estimate of the discharge for this water level. These steps should be repeated for the required range of H values to define the stage-discharge relationship.

Step 5. Calculate the basic discharges in the main channel and flood plain zones for the specified flow depth

In these calculations the bank lines between the zones should be excluded from the wetted perimeters. Areas and wetted perimeters should be measured from the surveyed cross-section, not the idealized trapezoidal section.

Step 6. Add the zonal basic discharges together to obtain Q_{basic}, the basic discharge for the whole cross-section

This must now be adjusted to account for flow interaction effects. The adjustment must be made using the adjustment function applicable in each of four possible flow regions; the correct value will be selected from these as calculations proceed.

Step 7. Adjust Q_{basic}, assuming flow is in Region 1

Step 7.1 Calculate H_*, the ratio of flow depths on the flood plains and in the main channel

$$H_* = (H-h) / H \tag{4.25}$$

Step 7.2 Calculate the Darcy-Weisbach friction factors for the main channel, F_c, and the flood plains, f_f, using the relationship

$$f = 8\,g\,R\,S / V^2 \tag{4.26}$$

in which

- g is the gravitational acceleration = 9.81 m/s^2,
- R is the appropriate hydraulic radius (= A/P, excluding the bank division lines from P) (m),
- S is the hydraulic gradient, equal to the channel gradient (S_o) for uniform flow, and
- V is the appropriate basic average flow velocity (m/s).

V_C and V_f can be calculated by dividing the basic zonal discharges (step 5) by the appropriate areas. If there are two flood plains a single value of f_f should be calculated by using the combined areas, wetted perimeters and basic discharges.

Step 7.3 Calculate the dimensionless flood plain discharge deficit

$$Q_{*2f} = -1.0\,H_*\,(f_C / f_f) \tag{4.27}$$

Step 7.4 Calculate the dimensionless main channel discharge deficit

$Q_{*2C} = -1.240 + 0.395 (B / w_C) + G H.$ for one flood plain (4.28)

or

$Q_{*2C} = -1.240 + 0.395 (2B / 2w_C) + G H.$ for two flood plains (4.29)

Where

$G = 10.42 + 0.17 (f_F / f_C)$ for $s_c \geq 1.0$ (4.30)

or

$G = 10.42 + 0.17 (s_c f_F / f_C) + 0.34 (1 - s_C)$ for $s_c < 1.0$ (4.31)

The value of Q_{*2C} should not be less than 0.5. If the calculated value is less than this, set it to 0.5 and set Q_{*2F} to zero.

Step 7.5 Calculate the aspect ratio adjustment factor

$ARF = 2b / 10h$ (4.32)

ARF should not exceed 2.0. If the calculated value is greater than this, set it to 2.0.

Step 7.6 Calculate the total discharge deficit, the difference between Q_{basic} and the actual discharge

$DISDEF = (Q_{*2C} + N_F Q_{*2F}) (V_C - V_F) H h ARF$ (4.33)

in which

N_F is the number of flood plains (1 or 2), and

V_c, V_F are the zonal main channel and flood plain average flow velocities respectively.

Step 7.7 Calculate the Region 1 adjusted discharge for the specified water level

$Q_{R1} = Q_{basic} - DISDEF$ (4.34)

Step 8. Adjust Q_{basic} assuming flow is in Region 2

The adjustment is defined by the channel coherence at a flow depth greater than that specified. (Channel coherence is the ratio of the conveyance calculated as a single cross-section to that calculated by summing the conveyances of the separate flow zones).

Step 8.1 Calculate the "shift" to be applied to the specified flow depth

$shift = 0.05 + 0.05 N_F$ for $s_c \geq 1.0$ (4.35)

or

$shift = -0.01 + 0.05 N_F + 0.06 s_C$ for $s_c < 1.0$ (4.36)

Step 8.2 Calculate the shifted flow depth

$H' = (H h) / (h - shift H)$ (4.37)

Step 8.3 Calculate the channel coherence for the shifted flow depth

$$COH = \frac{(1 + A_*) \{((1 + A_*) / (1 + f_* P_*))\}^{0.5}}{1 + A_* \{(A_* / (f_* P_*))\}^{0.5}} \quad (4.38)$$

in which

A_* $= A_F / A_C$,
A_F is the total flood plain flow area (i.e. for both sides if there are two flood plains),
A_C is the main channel flow area,
f_* $= f_F / f_C$,
f_F is the Darcy-Weisbach friction factor for the flood plains,
f_C is the Darcy-Weisbach friction factor for the main channel,
P_* $= P_F / P_C$,
P_F is the total flood plain wetted perimeter (i.e. for both sides if there are two flood plains), excluding the bank lines,
P_C is the main channel wetted perimeter, excluding the bank lines.

The areas and wetted perimeters should correspond to the required flow depth, i.e. H′ for this calculation. The friction factors should also be recalculated, as in Step 7.2, using H′. If the shifted flow depth is above the extreme lateral points of the surveyed cross-section, extend the cross-section vertically from these points to the required level to enable areas and wetted perimeters to be calculated.

Step 8.4 Define the Region 2 discharge adjustment factor

$$DISADF_2 = COH \quad (4.39)$$

Step 8.5 Calculate the Region 2 adjusted discharge for the specified water level

$$Q_{R2} = Q_{basic} \times DISADF_2 \quad (4.40)$$

Step 9. Determine if Q_{R1} is the actual discharge, Q

$$\text{If } Q_{R1} \geq Q_{R2} \text{ then } Q = Q_{R1} \quad (4.41)$$

If $Q = Q_{R1}$ the calculations are complete for the specified water level, unless a skew correction (step 14) is required. If $Q_{R1} < Q_{R2}$ the actual discharge is still unknown; in this case proceed with step 10.

Step 10. Adjust Q_{basic} assuming flow is in Region 3

Step 10.1 Calculate the channel coherence, COH

using the equation given for the Q_{R2} calculation, but for the specified flow depth, H, instead of H′.

Step 10.2 Calculate the Region 3 discharge adjustment factor

$$DISADF_3 = 1.567 - 0.667\, COH \quad (4.42)$$

Step 10.3 Calculate the Region 3 adjusted discharge for the specified water level

$Q_{R3} = Q_{basic} \times DISADF_3$ (4.43)

Step 11. Determine if Q_{R2} is the actual discharge

If $Q_{R2} \leq Q_{R3}$ then $Q = Q_{R2}$ (4.44)

If $Q = Q_{R2}$ the calculations are complete for the specified water level, unless a skew correction (step 14) is required. If $Q_{R2} > Q_{R3}$ the actual discharge is still unknown; in this case proceed with step 12.

Step 12. Adjust Q_{basic} assuming flow is in Region 4

Step 12.1 Define the Region 4 discharge adjustment factor

This is equal to the channel coherence for the specified flow depth, H, as calculated above for Region 3, i.e.

$DISADF_4 = COH$ (4.45)

Step 12.2 Calculate the Region 4 adjusted discharge for the specified water level

$Q_{R4} = Q_{basic} \times DISADF_4$ (4.46)

Step 13. Determine which of Q_{R3} and Q_{R4} is the actual discharge

If $Q_{R3} > Q_{R4}$ then $Q = Q_{R3}$ (4.47)
or
If $Q_{R3} < Q_{R4}$ then $Q = Q_{R4}$ (4.48)

Discharge calculations are now complete for the specified water level, unless a skew correction is required. If so, proceed with step 14.

Step 14. Apply the skew correction if the main channel is not aligned with the flood plains

This is done as follows and applies for angles of skew up to 10°.

Step 14.1 Measure the angle of skew (in degrees) between the main channel and the flood plain (Φ) on a suitable map

Step 14.2 Calculate the discharge deficit from the results already obtained

$DISDEF = Q_{basic} - Q$ (4.49)

Step 14.3 Correct the discharge deficit to account for skewness

$$DISDEF_{skew} = DISDEF \times (1.03 + 0.074\ \Phi) \qquad (4.50)$$

Step 14.4 Recalculate the actual discharge

$$Q = Q_{basic} - DISDEF_{skew} \qquad (4.51)$$

Q is the actual discharge for the specified flow depth, H.

Calculate main channel and flood plain discharges

If discharges for the main channel and flood plains are required separately, they can be estimated as follows. This will be necessary if f_F is to be estimated from measured data. The procedure has not been verified for skewed main channels and should be applied with caution for such cases.

Step 15. Determine the actual, adjusted, total discharge for the required water level, as described in section 2

Step 16. Identify the flow region and calculate the separate discharges

Step 16.1 Calculate zonal discharges for region 1

If the actual discharge is in Region 1, i.e. $Q = Q_{R1}$, determine the separate discharges using the results from the predictive method described in the section 2, i.e.

$$Q_C = Q_{Cbasic} - Q_{*2C}\ (V_C - V_F)\ H\ h\ ARF \qquad (4.52)$$

for the main channel, and

$$Q_F = Q_{Fbasic} - Q_{*2F}\ (V_C - V_F)\ H\ h\ ARF \qquad (4.53)$$

for each flood plain.

Step 16.2 Calculate zonal discharges for regions 2, 3 or 4

If the actual discharge is in one of Regions 2, 3 or 4, assume that the flood plain discharges are unaffected by the interaction, and allocate all the adjustment to the main channel discharge, i.e.

$$Q_C = Q_{Cbasic} - DISDEF \qquad (4.54)$$
$$Q_F = Q_{Fbasic} \qquad (4.55)$$

Calculate boundary shear stress

Boundary shear stresses are required for predicting locations of scour, designing scour protection, and estimating sediment transport rates. These issues will be addressed by future research. The following steps can be used for obtaining provisional estimates of the average shear stress on the main channel bed and the average and maximum shear stresses on the flood plains.

Step 17. Calculate the average shear stress on the bed in the main channel

Step 17.1 Calculate the average boundary shear stress, ignoring the interaction effects

$$\tau_{oC} = \rho\, g\, R_C\, S \tag{4.56}$$

in which

ρ is the density of water (1000 kg/m^3), and
R_C is the hydraulic radius of the main channel, excluding the bank division lines from the wetted perimeter.

Step 17.2 Calculate the discharge adjustment factor for the main channel

$$DISADF_C = Q_C / Q_{Cbasic} \tag{4.57}$$

in which

Q_C is the actual main channel discharge and
Q_{Cbasic} is the basic main channel discharge

Step 17.3 Calculate the corrected average boundary shear stress

$$\tau_{oC}' = \tau_{oC}\,(DISADF_C)^2 \tag{4.58}$$

Step 18. Calculate the average shear stress on the surface of the flood plain, ignoring the interaction effects

$$\tau_{oF} = \rho\, g\, (H - h)\, S \tag{4.59}$$

This will apply on the flood plain surface beyond the zone of interaction with the main channel flow. Allow for a maximum local value of 5 τ_{oF} within a distance of 3 h from the bank line.

Notation for FCFA method

		Units
A	cross-sectional area	m^2/s
A_{Csurv}	area of main channel below bank elevation, from surveyed cross-section	m^2/s
A_*	ratio A_F/A_C	-
B	half the total compound channel width. For partially inundated sloping flood plains this should be taken as half the water surface width.	m
b	half the bottom width of the main channel	m
COH	channel coherence	-
DISADF	adjustment factor applied to basic discharge to account for interaction effects; subscript will indicate appropriate region	-
DISDEF	discharge deficit, i.e. difference between actual and basic discharges	-
$DISDEF_{skew}$	discharge deficit, accounting for main channel skew	-
f	Darcy-Weisbach friction factor, = $8gRS/V^2$	-
f_*	ratio f_F/f_C	-

G	parameter in Region 1 discharge deficit prediction	-
g	gravitational acceleration	m/s^2
H	depth of flow in main channel	m
H_*	ratio of flow depths on flood plain and main channel	-
H'	shifted flow depth in main channel (for Region 2 prediction)	m
h	depth of main channel bed below river bank elevation	m
N_F	number of flood plains, 1 or 2	-
n	Manning's roughness coefficient	$m^{1/3}/s$
P	wetted perimeter	m
P_*	ratio P_F/P_C	m
Q	actual discharge, unsubscripted for whole compound channel	m^3/s
Q_{basic}	zonal discharge ignoring bank lines from wetted perimeter, unsubscripted for sum of main channel and flood plain values	m^3/s
Q_R	discharge as adjusted to account for interaction effects in region indicated by numerical subscript	m^3/s
Q_{*2}	discharge deficit normalized by $(V_C-V_F)Hh$	-
R	hydraulic radius, $= A/P$	m
S	hydraulic gradient of channel	-
S_o	surveyed channel gradient	-
s_C	side slope of main channel bank, horizontal/vertical	-
shift	addition to main channel flow depth in Region 2 adjustment prediction	m
V	average flow velocity	m/s
w_C	half width of main channel between bank lines	m
ρ	density of water (approx 1000 kg/m^3)	kg/m^3
τ_o	average bed shear stress	N/m^2
τ_{oC}'	average main channel bed shear stress adjusted for interaction effect	N/m^2
Φ	angle of skew between main channel and flood plains	°

Subscripts

C	main channel
F	flood plain
L	left bank
R	right bank
1,2,3,4	region of flow behaviour

4.3.2 Worked example for FCFA method

Problem definition

The straight reach of river has a gradient of 0.00047 and a cross-section defined by the surveyed coordinates listed below.

Distance (m)	Elevation (m)
0	15.50
0.60	12.58
13.86	11.93
15.01	10.60
16.80	9.86
20.30	10.09
31.75	10.20
35.93	9.85
38.30	10.51
39.75	12.07
52.80	12.64
57.00	16.11

Manning's n values for the main channel and flood are estimated as 0.025 and 0.030 respectively. The main channel is aligned at 4° to the axis of the flood plains.

The following are required:

- the discharge when the water level is 14.79 m;
- the discharges for the main channel and flood plains separately; and
- the boundary shear stresses for the main channel and flood plains.

Solution

Step 1. **Determine the longitudinal gradient of the channel reach, S_o, from survey information**

From given information, $S_o = 0.00047$

Step 2. **Determine the geometric variables required to define the adjustment functions**

Step 2.1 **The cross-section is plotted as Figure 4.9**

Step 2.2 Identify the points on the cross-section which mark the divisions between the main channel and the flood plains on both sides

The points most realistically marking the divisions between the main channel and flood plains are identified as LB and RB on Figure 4.9. The bank division lines are drawn vertically through LB and RB. The distance between the bank lines is therefore

$$2w_c = 39.75 - 13.86 = 25.89m$$

Step 2.3 Determine the river bank elevation

Bank elevation at LB = 11.93m
Bank elevation at RB = 12.07m

$$\therefore \text{River bank elevation} = \frac{11.93 + 12.07}{2} = 12.00m$$

Step 2.4 Fit a uniform slope to the main channel bank on each side

On the left bank the main channel bank slope is defined by point LB and the adjacent surveyed point to the right, therefore

$$s_{CL} = \frac{15.01 - 13.86}{11.93 - 10.60} = 0.865$$

On the right bank the main channel bank slope is defined by point RB and the adjacent surveyed point to the left, therefore

$$s_{CL} = \frac{39.75 - 38.30}{12.07 - 10.51} = 0.929$$

$$\therefore s_C = \frac{s_{CL} + s_{CR}}{2}$$

$$= \frac{0.865 + 0.929}{2} = 0.897$$

Step 2.5 Calculate the cross-sectional area of the main channel

Calculate area of main channel up to bank full level by adding areas of adjacent trapezia or triangles defined by the surveyed points.

A_{Csurv} =

$$[(12.00 - 11.93) + \frac{(11.93 - 10.60)}{2}] [15.01 - 13.86]$$

$$+ [(12.00 - 10.60) + \frac{(10.60 - 9.86)}{2}] [16.80 - 15.01]$$

$$+ [(12.00 - 10.09) + \frac{(10.09 - 9.86)}{2}] [20.30 - 16.80]$$

$$+ [(12.00 - 10.20) + \frac{(10.20 - 10.09)}{2}] [31.75 - 20.30]$$

$$+ [(12.00 - 10.20) + \frac{(10.20 - 9.85)}{2}] [35.93 - 31.75]$$

$$+ [(12.00 - 10.51) + \frac{(10.51 - 9.85)}{2}] [38.30 - 35.93]$$

$$= [\frac{(12.00 - 10.51 + 0}{2}][(39.75 - 38.30)$$

$$\frac{(12.00 - 10.51)}{(12.07 - 10.51)}]$$

$$= 0.74 \; x \; 1.15 + 1.77 \; x \; 1.79 + 2.03 \; x \; 3.50$$

$$+ 1.86 \; x \; 11.45 + 1.98 \; x \; 4.18 + 1.82 \; x \; 2.37$$

$$+ 0.75 \; x \; 1.38$$

$$= 0.85 + 3.17 + 7.09 + 21.24 + 8.26 + 4.31 + 1.03$$

$$= 45.95 m^2$$

Step 2.6 Determine the depth of the main channel, h

$$h = \frac{2w_C \pm [((2w_C)^2 - 4S_C A_{Csurv})]^{0.5}}{2s_C}$$

$$= \frac{25.89 \pm [\,25.89^2 - 4 \times 0.897 \times 45.95]^{0.5}}{2 \times 0.897}$$

$$= \frac{25.89 \pm 22.48}{1.79}$$

$$= 27.02m \text{ or } 1.90m$$

Clearly $h = 1.90m$

Step 2.7 Determine bottom width of main channel

$$2b = 2w_C - 2hs_C$$

$$25.89 - 2 \times 1.90 \times 0.897$$

$$= 22.48m$$

Step 2.8 Identify the positions of the backs of the flood plains

The positions of the backs of the flood plains are identified as FPL and FPR on Figure 4.9.

$$2B = 52.80 - 0.60 = 52.20m$$

Step 3. Estimate roughness coefficients for the main channel and flood plains

From the given information $n_c = 0.025$ and $n_f = 0.030$

Step 4. Specify a value for H, the flow depth measured above the idealized bed of the main channel

Required water level is 14.79m

H = Water level - (river bank elevation - h)

$$= 14.79 - (12.00 - 1.90)$$

$$= 4.69m$$

Step 5. Calculate the basic discharges in the main channel and flood plain zones for the specified flow depth

Left flood plain:
Flow area is calculated by adding areas of adjacent trapezia or triangles defined by the water surface and the surveyed points.

$$A_{FL} = [\frac{(14.79 - 12.58) + 0}{2}][(0.60 - 0)\frac{(14.79 - 12.58)}{(15.50 - 12.58)}]$$

$$+ [\frac{(14.79 - 12.58) + (14.79 - 11.93)}{2}][13.86 - 0.60]$$

$$= 1.11 \; x \; 0.45 + 2.54 \; x \; 13.26$$

$$= 34.11 m^2$$

Wetted perimeter:

$$P_{FL} = [(14.79 - 12.58)^2 + [(0.60 - 0)\frac{(14.79 - 12.58)}{(15.50 - 12.58)}]^2]^{1/2}$$

$$+ [(12.58 - 11.93)^2 + (13.86 - 0.6)^2]^{1/2}$$

$$= 2.26 + 13.28$$

$$= 15.54 m$$

Hydraulic radius:

$$R_{FL} = \frac{A_{FL}}{P_{FL}} = \frac{34.11}{15.54} = 2.19 m$$

Basic discharge, from Manning's equation:

$$Q_{FLbasic} = \frac{A_{FL}}{n_F} R_{FL}^{2/3} S^{1/2}$$

$$= \frac{34.11}{0.03} 2.19^{2/3} \; 0.00047^{1/2}$$

$$= 41.63 \; m^3/s$$

Right flood plain:
Area:

$$A_{FR} = [\frac{(14.79 - 12.07) + (14.79 - 12.64)}{2}] [52.80 - 39.75]$$

$$+ [\frac{(14.79 - 12.64) + 0}{2}][(57.00 - 52.80)\frac{(14.79 - 12.64)}{(16.11 - 12.64)}]$$

$$= 2.55 \ x \ 13.05 + 1.08 \ x \ 2.60$$

$$= 34.58 m^2$$

Wetted perimeter:

$$P_{FR} = [(12.64 - 12.07)^2 + (52.80 - 39.75)^2]^{\frac{1}{2}}$$

$$+ [(14.79 - 12.64)^2$$

$$+ [(57.00 - 52.80) \frac{(14.79 - 12.64)}{(16.11 - 12.64)}]^2]^{\frac{1}{2}}$$

$$= 13.06 + 3.38$$

$$= 16.44 m$$

Hydraulic radius:

$$R_{FR} = \frac{A_{FR}}{P_{FR}} = \frac{34.58}{16.44} = 2.10 m$$

Basic discharge, from Manning's equation:

$$Q_{FRbasic} = \frac{A_{FR}}{n_F} R_{FR}^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$= \frac{34.58}{0.03} \ x \ 2.10^{\frac{2}{3}} \ x \ 0.00047^{\frac{1}{2}}$$

$$= 41.02 m^3/s$$

Basic discharge for both flood plains together,

$$Q_{Fbasic} = Q_{FLbasic} + Q_{FRbasic}$$

$$= 41.63 + 41.02$$

$$= 82.65 m^3/s$$

Main channel

$$A_C = [\frac{(14.79 - 11.93) + (14.79 - 10.60)}{2}] [15.01 - 13.86]$$

$$+ [\frac{(14.79 - 10.60) + (14.79 - 9.86)}{2}] [16.80 - 15.01]$$

$$+ [\frac{(14.79 - 9.86) + (14.79 - 10.09)}{2}] [20.30 - 16.80]$$

$$+ [\frac{(14.79 - 10.09) + (14.79 - 10.20)}{2}] [31.75 - 20.30]$$

$$+ [\frac{(14.79 - 10.20) + (14.79 - 9.85)}{2}] [35.93 - 31.75]$$

$$+ [\frac{(14.79 - 9.85) + (14.79 - 10.51)}{2}] [38.30 - 35.93]$$

$$+ [\frac{(14.79 - 10.51) + (14.79 - 12.07)}{2}] [39.75 - 38.30]$$

$$= 3.53 x 1.15 + 4.56 x 1.79 + 4.82 x 3.50$$

$$+ 4.65 x 11.45 + 4.77 x 4.18 + 4.61 x 2.37$$

$$+ 3.50 x 1.45$$

$$= 4.05 + 8.16 + 16.85 + 53.19 + 19.92 + 10.93 + 5.08$$

$$= 118.18 m^2$$

Wetted perimeter:

$$P_C \;=[(11.93 - 10.60)^2 + (15.01 - 13.86)^2]^{1/2}$$
$$+ [(10.60 - 9.86)^2 + (16.80 - 15.01)^2]^{1/2}$$
$$+ [(10.09 - 9.86)^2 + (20.30 - 16.80)^2]^{1/2}$$
$$+ [(10.20 - 10.09)^2 + (31.75 - 20.30)^2]^{1/2}$$
$$+ [(10.20 - 9.85)^2 + (35.93 - 31.75)^2]^{1/2}$$
$$+ [(10.51 - 9.85)^2 + (38.30 - 35.93)^2]^{1/2}$$
$$+ [(12.07 - 10.51)^2 + (39.75 - 38.30)^2]^{1/2}$$
$$= 1.76 + 1.94 + 3.51 + 11.45 + 4.19 + 2.46 + 2.13$$
$$= 27.44m$$

Hydraulic radius:

$$R_C = \frac{A_C}{P_C} = \frac{A_C}{P_C} = \frac{118.18}{27.44} = 4.31m$$

Basic discharge, from Manning equation:

$$Q_{Cbasic} \quad \frac{A_C}{n_C} R_C^{2/3} S^{1/2}$$
$$= \frac{118.18}{0.025} x\ 4.31^{2/3}\ x\ 0.00047^{1/2}$$
$$= 271.29m^3/s$$

Step 6. Add the zonal basic discharges together to obtain Q_{basic}, the basic discharge for the whole cross-section

$$Q_{basic} = Q_{Cbasic} + Q_{Fbasic}$$
$$= 271.29 + 82.65$$
$$= 353.94\ m^3/s$$

Step 7. Adjust Q_{basic} assuming flow is in Region 1

Step 7.1 **Calculate H_*, the ratio of flow depths on the flood plains and in the main channel**

$$H_* = \frac{(H-h)}{H}$$

$$= \frac{4.69 - 1.90}{4.69}$$

$$= 0.595$$

Step 7.2 **Calculate the Darcy-Weisbach friction factors for the main channel, f_{c}, and the flood plains, f_p, using the relationship**

$$f_C = \frac{8g\ R_C\ S}{V_C^2}$$

$$R_C = 4.31m\ (Step\ 5)$$

$$V_C = \frac{Q_C}{A_C}$$

$$= \frac{271.29}{118.18}$$

$$= 2.30\ m/s$$

$$f_C = \frac{8\ x\ 9.81\ x\ 4.31\ x\ 0.00047}{2.30^2}$$

$$= 0.0302$$

Flood plains:

$$f_F = \frac{8g\ R_F\ S}{V_F^2}$$

$$R_F\ \frac{A_F}{P_F} = \frac{34.11\ +\ 34.58}{15.54\ +\ 16.44} = 2.15m$$

$$V_F = \frac{Q_F}{A_F}$$

$$= \frac{82.65}{(34.11\ +\ 34.58)}$$

$$= 1.20\ m/s$$

$$f_F = \frac{8\ x\ 9.81\ x\ 2.15\ x\ 0.00047}{1.20^2}$$

$$= 0.0547$$

Step 7.3 Calculate the dimensionless flood plain discharge deficit

$$Q_{*2F} = -\ 1.0\ H_*\ \frac{f_C}{f_F}$$

$$= -\ 1.0\ x\ 0.595\ x\ \frac{0.0302}{0.0547}$$

$$= -0.3296$$

Step 7.4 Calculate the dimensionless main channel discharge deficit

$$Q_{*2C} = -1.240 + 0.395 \frac{2b}{2w_c} + G\ H_* \quad \text{for two flood plains}$$

$2B = 52.20m$ (*Step* 2.8)

$2w_c = 25.89m$ (*Step* 2.2)

$$G = 10.42 + 0.17 \frac{s_C f_F}{f_C} + 0.34\ (1 - s_C)\ for\ s_C < 1.0$$

$$= 10.42 + 0.17 \frac{0.897x0.0547}{0.0302} + 0.34\ (1 - 0.897)$$

$$= 10.73$$

$$= -1.240 + 0.395 \frac{52.20}{25.89} + 10.73\ x\ 0.595$$

$$= 5.941$$

Step 7.5 Calculate the aspect ratio adjustment factor

$$ARF = \frac{2b}{10h} \quad 2b = 22.48m\ (Step\ 2.7)$$

$h = 1.90m$ (*Step* 2.6)

$$= \frac{22.48}{10\ x\ 1.90}$$

$$= 1.18$$

Step 7.6 Calculate the total discharge deficit, the difference between Q_{basic} and the total discharge

$$DISDEF = (Q_{*2C} + N_F\ Q_{*2F})(V_C - V_F)H\ h\ ARF$$

$$= (5.941 + 2\ (-0.3296))\ (2.30 - 1.20)\ 4.69\ x\ 1.90\ x\ 1.18$$

$$= 61.09\ m^3/s$$

Step 7.7 **Calculate the Region 1 adjusted discharge for the specified water level**

$$Q_{R1} = Q_{basic} - DISDEF$$

$$= 353.94 - 61.09$$

$$= 292.85\ m^3/s$$

Step 8. **Adjust Q_{basic} assuming flow is in Region 2**

Step 8.1 **Calculate the "shift" to be applied to the specified flow depth**

$$Shift = -0.01 + 0.05\ N_F + 0.06\ s_C \ for\ s_C < 1.0$$

$$= -0.01 + 0.05 \times 2 + 0.06 \times 0.897$$

$$= 0.144$$

Step 8.2 **Calculate the shifted flow depth**

$$H' = \frac{Hh}{h - shiftH}$$

$$= \frac{4.69 \times 1.90}{1.90 - 0.144 \times 4.69}$$

$$= 7.28m$$

Step 8.3 **Calculate the channel coherence for the shifted flow depth**

H′ = 7.28m corresponds to a water level of 12.00 - 1.90 + 7.28 = 17.38m

Recalculate areas, wetted perimeters, basic discharges and f values.

Because level 17.38m is above the surveyed cross-section, the cross-section will be extended vertically from the first and last points for calculation of areas and wetted perimeters.

Left flood plain:

Shifted area = area for water level at 14.79m plus extra, i.e.

$$A'_{FL} = 34.11 + (17.38 - 14.79)(13.86 - (0.60 - x_L))$$

$$+ (\frac{(17.38 - 14.79) + (17.38 - 15.50)}{2})((0.60 - x_L)-0)$$

where

$$x_L = (0.60-0)\frac{(14.79 - 12.58)}{(15.50 - 12.58)} = 0.45m$$

$$A'_{FL} = 34.11 + 35.51 + 0.34$$

$$= 69.96m^2$$

Shifted wetted perimeter:

$$P'_{FL} = (17.38 - 15.50) + ((15.50 - 12.58)^2 + (0.60-0)^2)^{1/2}$$

$$+ ((12.58 - 11.93)^2 + (13.86 - 0.60)^2)^{1/2}$$

$$= 1.88 + 2.98 + 13.28$$

$$= 18.14m$$

Shifted hydraulic radius:

$$R'_{FL} = \frac{A'_{FL}}{P'_{FL}} = \frac{69.96}{18.14} = 3.86m$$

Shifted basic discharge:

$$Q'_{FLbasic} = \frac{A'FL}{n_F} R'^{2/3}_{FL} s^{1/2}$$

$$= \frac{69.96}{0.03} x\ 3.86^{2/3} x\ 0.00047^{1/2}$$

$$= 124.33m^3/s$$

Right flood plain:

Shifted area = area for water level at 14.79 m plus extra, i.e.

$$A'_{RL} = 34.58 + (17.38 - 14.79)\,(52.80 + x_R - 39.75) + \left(\frac{(17.38 - 14.79) + (17.38 - 16.11)}{2}\right)$$

$$(57.00 - (52.80 + x_R))$$

where

$$x_R = (0.60 - 0)\,\frac{(14.79 - 12.58)}{(15.50 - 12.58)} = 2.60m$$

$$A'_{RL} = 34.58 + 40.53 + 3.09$$

$$= 78.20\ m^2$$

Shifted wetted perimeter:

$$P'_{FR} = (17.38 - 16.11)$$

$$+ ((16.11 - 12.64)^2 + (57.00 - 52.80)^2)^{1/2}$$

$$+ ((12.64 - 12.07)^2 + (52.80 - 39.75)^2)^{1/2}$$

$$= 1.27 + 5.45 + 13.06$$

$$= 19.78\ m$$

Shifted hydraulic radius:

$$R'_{FR} = \frac{A'_{FR}}{P'_{FR}} = \frac{78.20}{19.78} = 3.95m$$

Shifted basic discharge:

$$Q'_{FRbasic} = \frac{A'_{FR}}{n_F} R'_{FR}{}^{2/3} s^{1/2}$$

$$= \frac{78.20}{0.03} \times 3.95^{2/3} \times 0.00047^{1/2}$$

$$= 141.29\ m^3/s$$

Main channel:

Shifted area:

A'_C = area for water level at 14.79m + (17.38 - 14.79) x 2 w_C

= 118.18 + (17.38 - 14.79) x 25.89

= 185.24²m

Wetted perimeter remains the same, i.e.

P'_C = 27.44m

Shifted hydraulics radius:

$$R'_C = \frac{A'_C}{P'_C} = \frac{185.24}{27.44} = 6.75m$$

Shifted basic discharge:

$$Q'_{Cbasic} = \frac{A'_C}{n_C} R'^{2/3}_C s^{1/2}$$

$$= \frac{185.24}{0.025} 6.75^{2/3} 0.00047^{1/2}$$

$$= 573.78 \; m^3/s$$

Shifted friction factor for flood plains:

$$f'_F = \frac{8\ g\ R'_F\ s}{V'^2_F}$$

$$A'_F = 69.96 + 78.20 = 148.16m^2$$

$$P'_F = 18.14 + 19.78 = 37.92m$$

$$R'_F = \frac{A'_F}{P'_F} = \frac{148.16}{37.92} = 3.91m$$

$$V'_F = \frac{Q'_F}{A'_F}$$

$$= \frac{124.33 + 141.29}{148.16}$$

$$= 1.79\ m/s$$

$$f'_F = \frac{8\ x\ 9.81\ x\ 3.91\ x\ 0.00047}{1.79^2}$$

$$= 0.0448$$

Shifted friction factor for main channel:

$$f_C = \frac{8\ g\ R'_C\ S}{V'^2_C}$$

$$R'_C = 6.75m$$

$$V'_C = \frac{Q'_C}{A'_C}$$

$$= \frac{573.78}{185.24}$$

$$= 3.10m/s$$

$$f_C = \frac{8\ x\ 9.81\ x\ 6.75\ x\ 0.00047}{3.10^2} = 0.0265$$

$$COH = \frac{(1 + A_*)\ [((1 + A_*)/(1 + f_* P_*))]^{0.5}}{1 + A_*\ [(A_*/(f_* P_*))]^{0.5}}$$

$$where\ A_* = \frac{A'_F}{A'_C} = \frac{148.16}{185.24} = 0.799$$

$$f_* = \frac{f_F}{f_C} = \frac{0.0448}{0.0265} = 1.691$$

$$P_* = \frac{P'_F}{P'_C} = \frac{37.92}{27.44} = 1.382$$

$$COH = \frac{(1 + 0.799)\ [(1 + 0.799)/(1 + 1.691\ x\ 1.382)]^{0.5}}{1 + 0.799\ [(0.799/(1.691\ x\ 1.382))]^{0.5}}$$

$$= 0.900$$

Step 8.4 Define the Region 2 discharge adjustment factor

$DISADF_2$ = COH = 0.900

Step 8.5 Calculate the Region 2 adjusted discharge for the specified water level

$$Q_{R2} = Q_{basic} \times DISADF_2$$

$$Q_{basic} = 353.94 \ m^3/s \ (Step\ 6)$$

$$= 353.94 \times 0.900$$

$$= 318.55 \ m^3/s$$

Step 9. Determine if Q_{R1} is the actual discharge, Q

$Q_{R1} = 292.85 \ m^3/s$

$Q_{R2} = 318.55 \ m^3/s$

$Q_{R1} < Q_{R2}$ ∴ actual discharge is still unknown.

Step 10. Calculate the channel coherence, COH

Adjust Q_{basic} assuming flow is in Region 3

$$COH = \frac{(1 + A_*) \ [((1 + A_*)/(1 + f_* P_*))]^{0.5}}{1 + A_*[A_*/(f_* P_*))]^{0.5}}$$

Areas, wetted perimeters and f values correspond to H = 4.69m, as determined in Steps 5 and 7.

$$A_* = \frac{A_F}{A_C} = \frac{68.69}{118.18} = 0.581$$

$$f_* = \frac{f_F}{f_C} = \frac{0.0547}{0.0302} = 1.811$$

$$P_* = \frac{P_F}{P_C} = \frac{31.98}{27.44} = 1.165$$

$$COH = \frac{(1 + 0.581) \ [((1 + 0.581)/(1 + 1.811 \times 1.165))]^{0.5}}{1 + 0.581 \ [(0.581/(1.811 \times 1.165))]^{0.5}}$$

$$= 0.864$$

Step 10.1 Calculate the Region 3 discharge adjustment factor

$$DISADF_3 = 1.567 - 0.667\ COH$$

$$= 1.567 - 0.667 \times 9.864$$

$$= 0.991$$

Step 10.2 Calculate the Region 3 adjusted discharge for the specified water level.

$$Q_{R3} = Q_{basic} \times DISADF_3$$

$$Q_{basic} = 353.94\ m^3/s\ (Step\ 6)$$

$$Q_{R3} = 353.94 \times 0.991$$

$$= 350.65 m^3/s$$

Step 11. Determine if Q_{R3} is the actual discharge

$$Q_{R2} = 318.55\ m^3/s$$

$$Q_{R3} = 350.65\ m^3/s$$

$$Q_{R2} < Q_{R3} \therefore \text{actual discharge is } Q_{R2}$$

$$i.e.\ Q = 318.55\ m^3/s$$

Note that in this case the flow is in region two and steps 12 and 13 are unnecessary:

Step 12. Adjust Q_{basic} assuming flow is in Region 4

Step 12.1 Define the Region 4 discharge adjustment factor

Step 12.2 Calculate the Region 4 adjusted discharge for the specified water level

Step 13. Determine which of Q_{R3} and Q_{R4} is the actual discharge

The discharge requires further adjustment to account for the skew alignment of the main channel.

Step 14. Apply the skew correction if the main channel is not aligned with the flood plains

Step 14.1 **Measure the angle of skew (in degrees) between the main channel and the flood plain (Φ) on a suitable map**

From the given information $\Phi = 4°$

Step 14.2 **Calculate the discharge deficit from the results already obtained**

$$DISDEF = Q_{basic} - Q$$
$$= 353.94 - 318.55$$
$$= 35.39 \ m^3/s$$

Step 14.3 **Correct the discharge deficit to account for skewness**

$$DISDEF_{skew} = DISDEF \ x \ (1.03 + 0.074 \ \Phi)$$
$$= 35.39 \ x \ (1.03 + 0.074 \ x \ 4)$$
$$= 46.93 \ m^3/s$$

Step 14.4 **Recalculate the actual discharge**

$$Q = Q_{basic} - DISDEF_{skew}$$
$$= 353.94 - 46.93$$
$$= 307.01 \ m^3/s$$

Step 15. **Determine the actual, adjusted, total discharge for the required water level, as described in section 2**

From previous analysis $Q = 307.01$ m^3/s

Step 16. **Identify the flow region and calculate the separate discharges**

Flow is in Region 2. Separate discharges are therefore calculated according to step 16.2, rather than step 16.1.

Step 16.1 **Calculate zonal discharges for region 1**

Step 16.2 Calculate zonal discharges for regions 2, 3 or 4

$$Q_C = Q_{Cbasic} - DISDEF$$

$$Q_{Cbasic} = 271.29\ m^3/s \quad (from\ previous\ Step\ 5)$$

$$DISDEF = DISDEF_{skew} = 46.93\ m^3/s\ (from\ previous\ step\ 14.3)$$

$$= 271.29 - 46.93$$

$$= 224.36\ m^3/s$$

$$Q_F = Q_{Fbasic}$$

$$= 82.65\ m^3/s\ (from\ previous\ Step\ 5)$$

i.e. 41.63 m^3/s on the left plain, and 41.02 m^3/s on the right plain.

Step 17. Calculate the average shear stress on the bed in the main channel

Step 17.1 Calculate the average boundary shear stress, ignoring the interaction effects

$$\tau_{oC} = \rho\ g\ R_C\ S$$

$$R_C = 4.3lm,\ (from\ Step\ 5\ of\ stage\text{-}discharge\ analysis)$$

$$S = 0.00047\ (from\ given\ information)$$

$$= 1000\ x\ 9.81\ x\ 4.31\ x\ 0.00047$$

$$= 19.87\ N/m^2$$

Step 17.2 Calculate the discharge adjustment factor for the main channel

$$DISADF_C = Q_C/Q_{Cbasic}$$

$$Q_C = 224.36\ m^3/s\ (from\ Step\ 2)$$

$$Q_{Cbasic} = 271.29\ m^3/s\ (from\ Step\ 5)$$

$$= 224.36/271.29$$

$$= 0.827$$

Step 17.3 Calculate the corrected average boundary shear stress

$$\tau_{oC} = \tau_{oC'} \, (DISADF_C)^2$$

$$= 19.87 \ (0.827)^2$$

$$= 13.59 \ N/m^2$$

Step 18. Calculate the average shear stress on the surface of the flood plain, ignoring the interaction effects

$$\tau_{oF} = \rho \ g \ (H-h)s$$

H = 4.69m (*Step 4 of the stage-discharge analysis*)

$$= 1000 \ x \ 9.81 \ x \ (4.69 \ - \ 1.90) \ x \ 0.00047$$

$$= 12.86 \ N/m^2$$

Allow for

$5 \times \tau_{oF} = 5 \times 12.86 = 64.32$ N/m^2

within a distance of

3h = 3 x 1.90 = 5.70m from the bank line.

ANNEX A: DERIVATION OF THE JAMES AND WARK METHOD

A.1 Introduction

The material presented in this annex is intended to give the user an insight into the development of the James and Wark method. Information and advice, which could not be included in the main part of the manual are presented here. Much of this material appeared in the Project Record (R&D Project Record 252/2/T). A more complete discussion of all aspects of flow in meandering channels is presented by James and Wark (1992).

The layout of this Annex reflects the approach taken when developing the conveyance estimation method. Initially the available laboratory information was collated and sumarised. Methods of computing inbank discharge in meandering channels were identified and evaluated against the available data. The data from the SERC FCF and the flume at the University of Aberdeen were used to develop the method for conveyance estimation in meandering compound channel flows . The method was then evaluated against other laboratory and field data. Finally bed shear stress data from the SERC FCF were evaluated to provide guidelines for the calculation of bed shear stresses.

A.2 Stage-Discharge Prediction

A.2.1 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 and boundary shear measurements were taken for inbank and overbank 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, 1991, 1993) were also used in the development and evaluation 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 for 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).

A.2.2 Inbank flows

Various methods were identified in the literature for accounting for the additional resistance to flow induced by channel curvature. The methods proposed by the following authors were selected as being potentially suitable for practical application.

- Soil Conservation Service (SCS) (1963)
- Toebes and Sooky (1967)
- Leopold *et al.* (1960)
- Mockmore (1944)
- Agarwal *et al.* (1984)
- Chang (1983)

In addition, modifications to two of these methods were 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

$$n'/n = 0.43\,s + 0.57 \qquad \text{for } s < 1.7$$
$$n'/n = 1.30 \qquad \text{for } s \geq 1.7 \qquad \text{(A1.1)}$$

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

Chang's (1983) method predicts the energy gradient associated with secondary circulation assuming that the circulation is fully developed. In fact, the circulation takes considerable distance to develop through a bend and begins to decay once the channel straightens out. For meanders, the circulation must reverse between successive bends and the associated energy gradient must drop to zero at two points over each wavelength. The average energy gradient associated with secondary circulation along the channel must therefore be substantially less than predicted assuming full development. Chang's (1984) approach for nonuniform flow through bends accounts for this and was simplified to apply to uniform flow. This enabled a correction factor to be computed which could be applied to the energy gradient predicted by his 1983 method, to account for growth and decay of circulation. Application of this correction is referred to here as the Modified Chang Method.

The selected prediction methods were applied to the SERC FCF trapezoidal channel data, the Aberdeen trapezoidal channel data, and the Vicksburg data. The average errors and standard deviations in estimating discharge by each method, for all the in bank data (62 measurements), are listed in Table A1.1. The upper value in each column is the average error in per cent; the lower value is the standard deviation. Two sets of results are presented for the SCS Method. In the first the adjustment factor was assumed to be on the higher side of the discontinuity where there was ambiguity, and in the second (marked *) the lower value was used. %Error as defined here is a skewed function and this biases the definition of Standard Deviation to positive %Error values.

Clearly, ignoring the energy loss induced by meandering introduces significant errors in the prediction of discharge for inbank flows. On the basis of simplicity and overall performance, it is recommended that the Linearized SCS method (equation A1.1) 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.

Table A1.1 Errors (%) in bend loss predictions

Method	Friction loss only	SCS Method (1963)	Toebes Sooky (1967)	Leopold *et al.* (1960)	Agarwal *et al.* (1983)	Mock-more (1944)	Chang Rect. (1983)	Mod. Chang	LSCS Method
Mean	16.14	-3.46	-1.02	-7.68	-22.80	-39.40	19.03	-1.76	-1.45
S - D	9.86	7.74	12.06	9.36	11.48	11.12	12.33	7.35	9.84
Mean		-2.76*							
S - D		8.48*							

Notes: %Error = 100 $(Q_{calc} - Q_{meas})/Q_{meas}$

S - D: Standard Deviation of the % error

* Denotes Mean % error and Standard deviation in the mean for the SCS method assuming the low side of the discontinuity in choice of correction factor values.

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 characteristics are followed. 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. A more detailed treatment is given by James and Wark (1992, 1993) and Wark and James (1993).

A2.3 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. This was demonstrated by applying the following widely used methods to the stage-discharge data obtained in the SERC FCF Phase B experiments.

The Divided Channel Method (DCM) separates the main channel and flood plain flows by vertical divisions. Discharges are calculated separately for the main channel and flood plain zones and then added. Zonal discharges are calculated using a friction equation with the vertical division lines included in the wetted perimeter for the main channel, but not for the flood plains. A variation of this method (DCM2) omits the vertical division lines from the main channel wetted perimeter as well.

In the Sum of Segments Method (SSGM) a vertical division line is located at each surveyed point defining the cross-section. Discharges are calculated in each of the resulting segments separately, using a friction equation and excluding the division lines from the wetted perimeters. The component discharges are then added.

In the FCFA Method (FCFAM) the basic zonal discharges as calculated by DCM2 are adjusted using empirical factors based on the SERC FCF Phase A data. The factors and their derivation are described by Ackers (1991) and summarized in section 4.3 above.

Because a hand calculation method was being sought, no computational methods were considered in this analysis although some are known to give good results for straight channels (for example, the Lateral Distribution Method, Wark *et al.*, 1991). The mean errors produced by these methods when applied to the SERC FCF Phase B stage-discharge data are presented in Table A1.2.

Table A1.2 Errors in discharge estimation by straight compound channel methods

Method	Mean Error (%)	Standard Deviation (%)
DCM	38.5	17.8
DCM2	41.6	16.8
SSGM	70.1	30.6
FCFAM	24.8	26.0

Note: %Error = 100 $(Q_{calc} - Q_{meas})/Q_{meas}$

The errors produced by straight channel methods when applied to meandering channels are clearly unacceptably large. The large standard deviations result mainly from trends in the predicted errors, which either increase or decrease strongly with increasing stage. These trends indicate that the methods do not account for all of the flow processes correctly.

A new method for predicting discharges in compound meandering channels was developed using a divided channel approach. The compound cross-section is divided into four zones, as shown in Figure 4.1. 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.

$$Q_T = Q_1 + Q_2 + Q_3 + Q_4 \qquad \text{(A1.2)}$$

The SERC FCF Phase B data were used to derive procedures for calculating the zonal discharges. The development of the procedures for each zone is described in the following paragraphs and the resulting equations are given in section 4.2 above.

Zone 1: main channel

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. Figure 2.2 summarises the main mechanisms present in overbank flow in a meandering compound channel.

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 inbank flow methods or obtained by measurement, if possible. The hydraulic slope which controls the flow in the main channel zone (S) is related to the flood plain or valley hydraulic slope by the channel sinuosity, (ie $S = S_o / s$). It should be noted that S_o can either be a ground slope if uniform flow is assumed or a water surface slope.

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 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 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.

The relationship between the adjustment factor (Q_1') and these variables is shown schematically in Figure A1.1. This shows that the main channel discharge is initially reduced as 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'.

Zone 2: inner flood plain

The method for predicting the inner flood plain discharge is based on quantitative descriptions of major loss mechanisms identified in the literature (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 the main channel) minus the top area of the main channel. The relative 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.

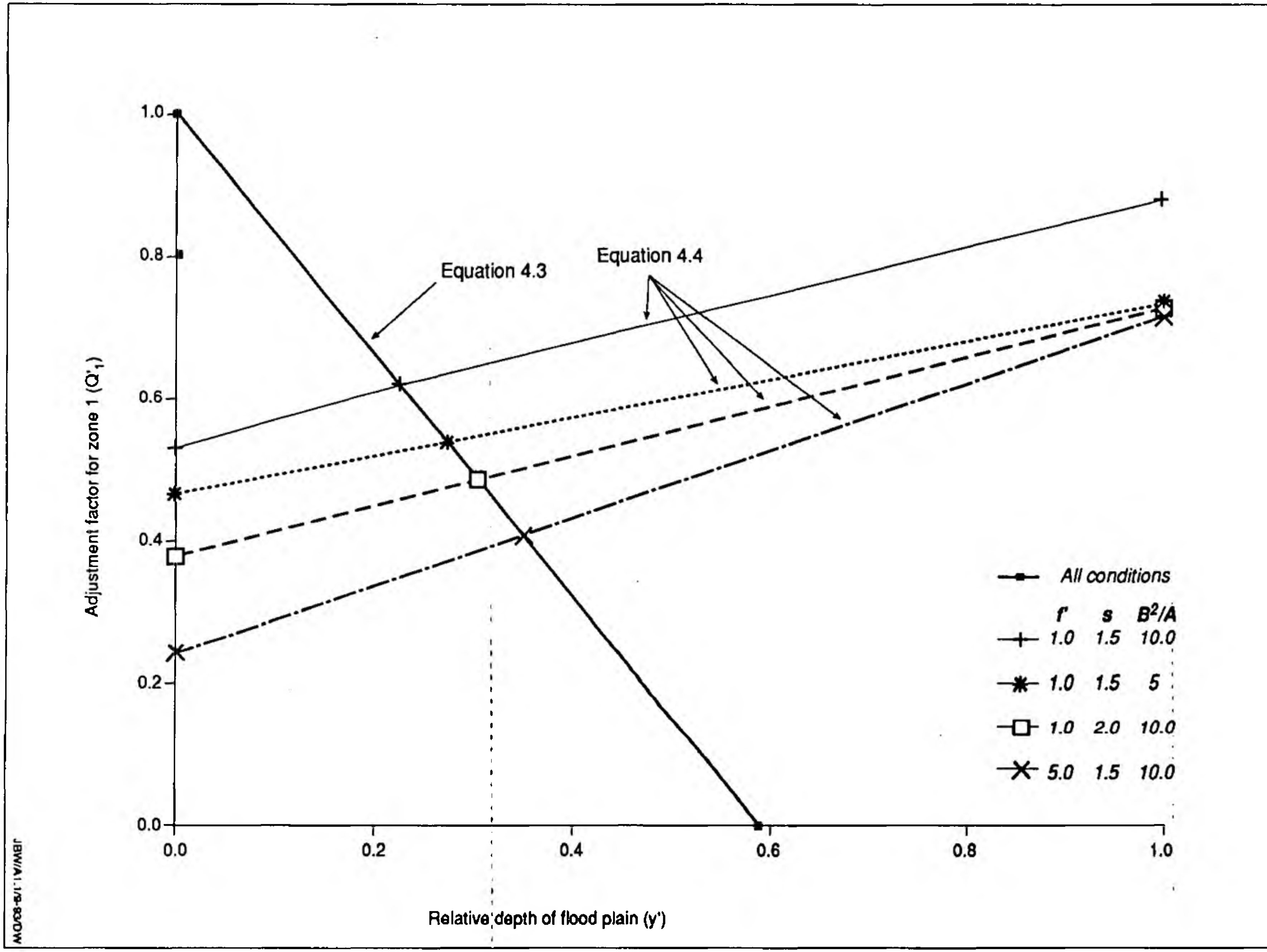


Figure A1.1 Adjustment factor for zone 1 discharge

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.

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.

A2.4 Evaluation of overbank flow models

Laboratory data

The methods proposed for calculating discharge for overbank flows in meandering channels have been evaluated by applying them to stage-discharge data measured in laboratory channels. Data were used from the SERC FCF Phase B experiments, the Aberdeen experiments, the Vicksburg experiments, and experiments performed by Kiely (1990) and Sooky (1964). The method proposed by Ervine and Ellis (1987) and two methods proposed by Greenhill (1992) were also evaluated. Calculations were also done with the same zonal subdivision as proposed here, but ignoring non-friction losses in all zones, (Friction only). The errors in reproducing the stage-discharge data by the different methods are summarized in Table A1.3. In this table James and Wark refers to the method presented in this report.

The results presented in Table A1.3 are the mean errors in predicted discharge calculated over 279 data points from all of the above sources. They show that considerable errors can be expected if non-friction losses are omitted. The method presented in this report performs the best. This method has the advantage over the others that it is based on measured discharges for zone 1, and should be more reliable if zonal conveyances are required separately. It is worth noting that the large standard deviations shown in Table A1.3 are caused by strong trends, over the ranges of stage considered, in the calculated errors and not random scatter.

Table A1.3 Errors in predicting overbank discharges: laboratory data

Method	Mean Error (%)	Standard Deviation (%)
Friction only	34.1	23.2
James and Wark	-2.1	9.7
Ervine and Ellis	5.3	18.3
Greenhill 4	11.5	19.3
Greenhill 5	7.6	14.7

Note: %Error = 100 $(Q_{calc} - Q_{meas})/Q_{meas}$

Distribution of discharge

The results above demonstrate the overall accuracy of the new method. The procedure is to calculate the discharges in the various parts or zones of the channel separately and to sum them together to obtain the total discharge. Hence the method gives the distribution of flow between the zones in addition to the total discharge.

There is very little independent information available on the distribution of discharge in meandering overbank flow. Sooky (1966) carried out detailed velocity measurements in shallow (Geometry 1) and deep (Geometry 2) meandering channels which were otherwise identical. These experiments were carried out in a channel which was built at a scale approximately 8-9 times smaller than the SERC FCF Phase B geometries. Sooky integrated these velocity measurements to obtain the proportion of the total discharge within each zone. Table A1.4 shows comparisons between the measured and calculated discharges in the main channel (zone 1) for the two cases Sooky considered. The method presented in this report has given the main channel discharges to an accuracy of about 5% to 7%, which is of the same order as the error in the measured values.

Table A1.4 Comparison between discharges in zone 1 (Sooky, 1966)

	$\%(Q_1/Q_T)$	
Case	Measured (Sooky, 1966)	Calculated (James and Wark)
Geometry 1	26.5	19.0
Geometry 2	38.2	33.5

Note:
The values above are the zone 1 discharges expressed as a percentage of the total discharge.

Sensitivity analysis

Meander wavelength and main channel side slopes (required for the zone 2 model) are not well defined in natural channels. However, sensitivity analyses have shown that predicted discharges are quite insensitive to these parameters and great accuracy in their estimation is not necessary. For example, for all of the laboratory data used above errors of ±50% in the wavelength gave mean errors of less than ±10% in the calculated discharge, similarly changes of ±100% in the main channel side slope gave mean errors of less than ±5% in the discharges.

Field data

The method presented in this manual was developed and verified using laboratory model data. There is very little field information available regarding the performance of full scale meandering channels with flood plains. The only detailed field investigation known at present was carried out on the River Roding in Essex, see Sellin and Giles (1989) or Sellin *et al.* (1990).

A combined laboratory and field monitoring research programme to study the behaviour of a stretch of the River Blackwater in Hampshire has recently been initiated. Laboratory work is currently under way on a large model (at 1:5 scale) of a 250 metre length of the proposed channel, which has been constructed in the SERC FCF at HR Wallingford.

The prototype channel has been constructed in the field to match the laboratory channel and a programme of field measurements is scheduled. Unfortunately the Blackwater project has not produced enough information to date for verification of the methods proposed in this report. However it is expected that eventually the laboratory and field information will form the basis of a verification. In the absence of the Blackwater data, the method has been applied to a selection of the information available from the Roding study.

The Roding study

Full details of the field and laboratory measurements carried out on this site are available in Sellin and Giles (1988) and Sellin *et al.* (1990). The study reach lies downstream of Abridge and as part of a flood alleviation scheme a two stage channel was formed by excavating berms on either side of the main channel (Figure A1.2). The original channel was untouched and remained in the natural state with a bankfull capacity of approximately 3 cumecs. The resulting flood channel has a low flow channel which meanders within the berm limits with a sinuosity of 1.38.

The project investigated the effects of different maintenance policies on the channel capacity. Most of the conditions investigated in the field and laboratory were with the flood berms covered, totally or partially, with extremely dense vegetation and verification of calibrated bed roughness values was not possible. The method was applied to the stage-discharge data from the following two cases.

P2 The berm growth was cut immediately after the summer growing season and so the berms were covered in short grass.

M2 The laboratory model data corresponding to the smooth berm case (P2 on the prototype).

In order to apply the method to these stage-discharge measurements the seven available sections were used to provided reach averaged areas, widths etc for all four flow zones at stages up to 1.0m above the berm level. The information provided by Sellin and Giles (1988) and Sellin *et al.* (1990) combined with widely accepted guidelines, Chow (1959) and Henderson (1966) allowed the berm Manning's n values for the two cases, P2 and M2 to be estimated as 0.050.

The measured and predicted stage-discharge curves for these two cases are shown in Figure A1.3 and the mean errors in Table A1.5. It is obvious from Table A1.5 that the present method improves the overall accuracy of the predicted discharges and that by ignoring the non-friction head losses the discharge will be over predicted by about 10%.

Table A1.5 Errors in predicting overbank discharges: Roding study

Case	P2		M2	
Method	Mean Error (%)	Standard Deviation (%)	Mean Error (%)	Standard Deviation (%)
Friction only	9.5	9.0	7.3	8.6
James and Wark	-2.0	1.7	-2.2	3.2

Note: %Error = $100*(Q_{calc} - Q_{meas})/Q_{meas}$

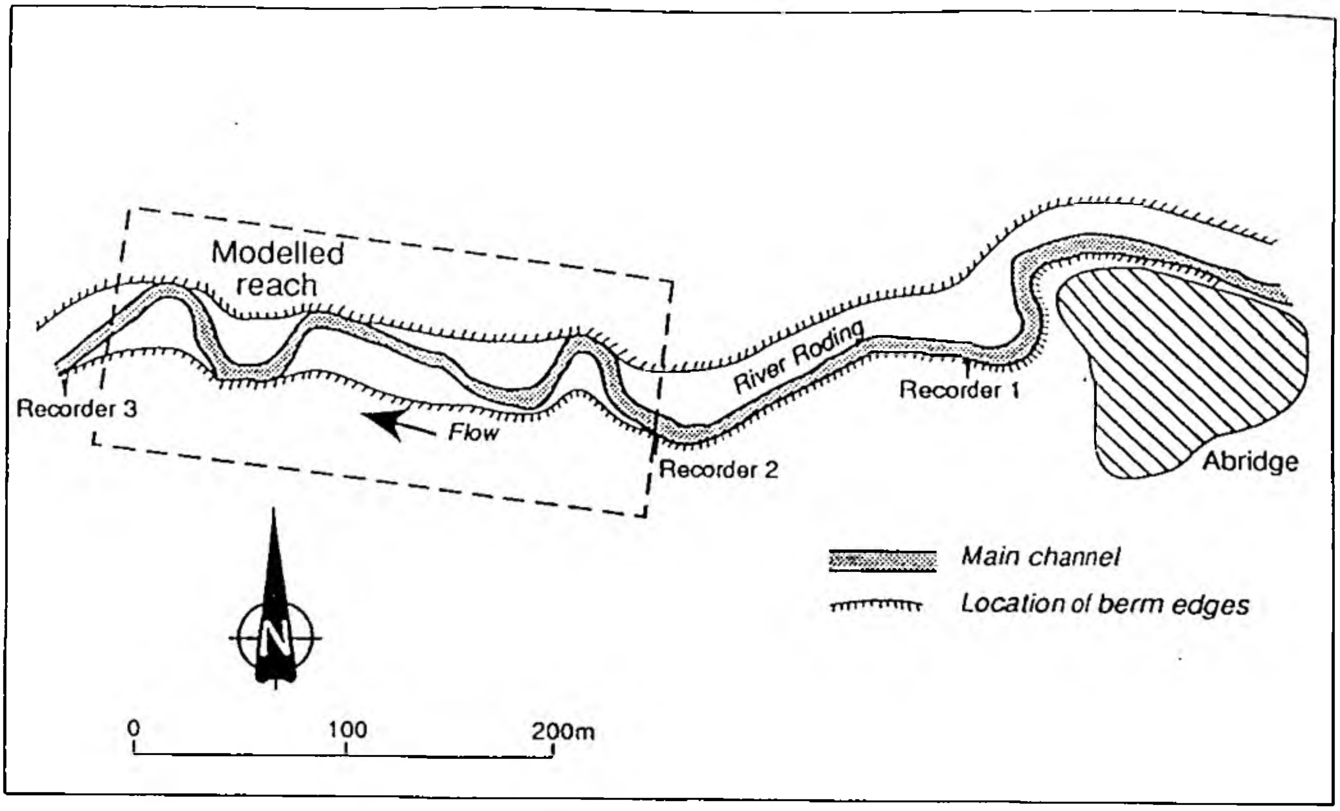


Figure A1.2 Location plan of study area on River Roding (after Sellin et al)

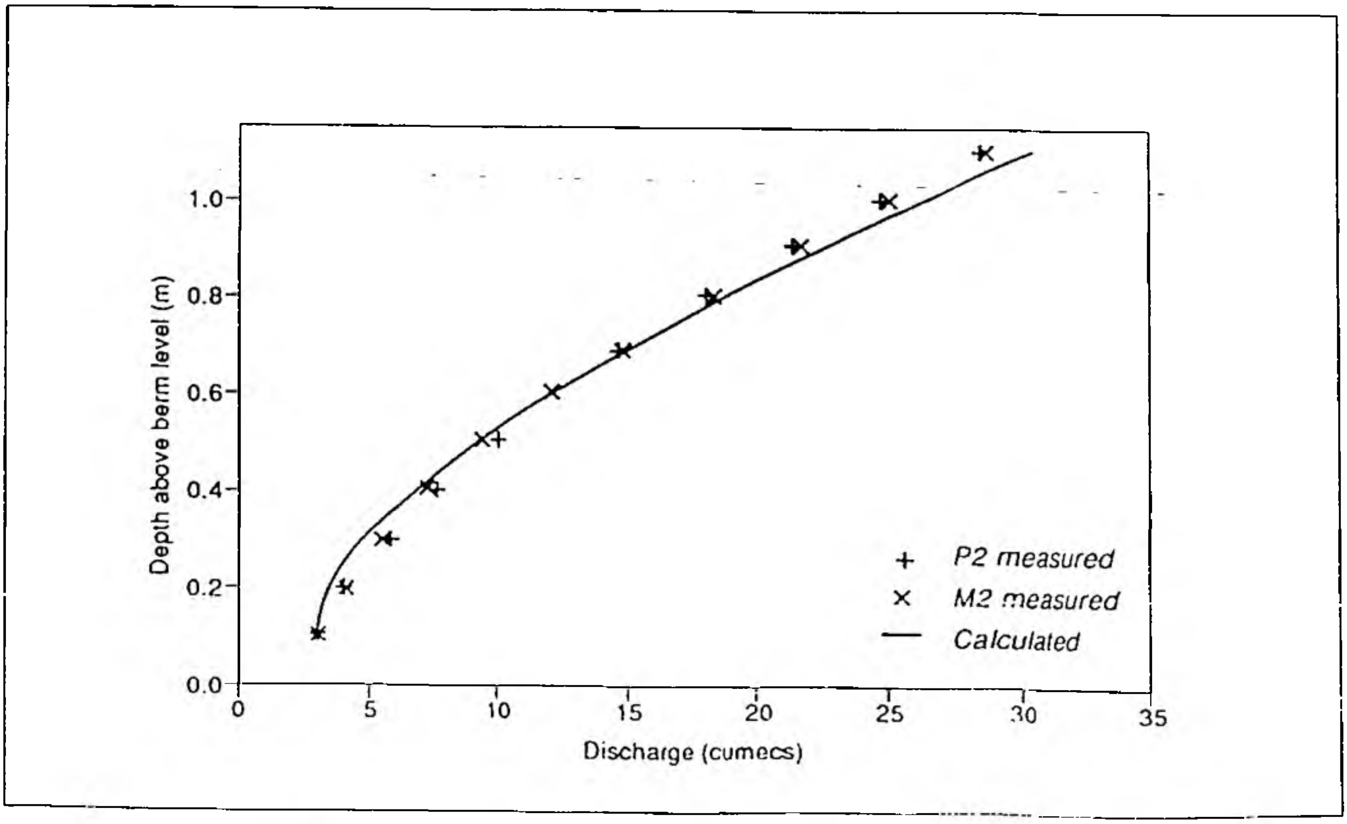


Figure A1.3 Stage discharges for the River Roding

Some sensitivity tests were carried out and they showed that as the flood plain was allowed to become smoother the two methods diverged more. Thus the effect of increased flood plain roughness is to make the non-friction head losses less important. It is not possible yet to give general guidelines as site specific aspects are likely to govern the relative importance of the various loss mechanisms. Sensitivity tests should be carried out for each application.

A.3 Boundary Shear Stress Prediction

Very little information is available on which to base methods for predicting boundary shear stresses in meandering compound channels. The data obtained from the SERC FCF Phase B experiments have been analyzed by Knight *et al.* (1992) and Lorena (1992) and form the basis of the provisional recommendations presented here. There is no simple, general method for predicting boundary shear for inbank flows in meandering channels, but several simulation models have been developed which can be used for this purpose (for example, by Bridge, 1992, and Nelson and Smith, 1989).

For overbank flows, Knight *et al.* (1992) have shown that the sectional average boundary shear stress in the main channel is less than would occur at bankfull stage at all cross-sections through a meander wavelength. Sectional average values are insufficient for designing scour protection, however, because the distributions of boundary shear across the sections are not uniform and vary with flow condition. The measured distributions suggest that during overbank flows the shear stress on the main channel banks may be higher than for inbank flows at some locations through the meander. The shear stress on the bed, however, is less than for inbank flows. Design shear stresses for scour protection should therefore be based on inbank flows for the bed and on overbank flows for the banks.

Under overbank flow conditions the bank shear stress on the upstream bank does not exceed $1.6 \gamma y_2 S_o$ in any of the measured distributions, where γ is the unit weight of water defined by ρg (9.81×10^3 N/m^3). On the downstream bank a high, localised stress concentration was observed downstream of each bend apex, associated with the expulsion of water from the main channel to the flood plain (see Figure 2.1). This concentration is shown in Figure A1.4, which presents Lorena's plot of contours of shear stress for the 2.04 sinuosity channel with a flow depth on the flood plain of 50 mm. The concentrations were centred at points between 60° and 70° downstream of the apex section for all the experimental conditions. The maximum observed shear stresses in the concentrations approached $5 \gamma y_2 S_o$. The stress concentrations are very localised and decrease rapidly with distance but, because of the limited experimental conditions and consequent uncertainty regarding locations, they should be assumed to be more extensive when designing scour protection. The enhanced shear stresses also extend for some distance over the flood plain on the downstream side of the channel.

The observed shear stress distributions suggest that the sediment transport capacity in the main channel will be lower for overbank flows than for inbank flows. Net deposition of sediment may therefore occur in the main channel during prolonged flood events. The shear concentrations on the downstream banks during overbank flows suggest enhancement of meander migration in the valley direction during prolonged flood flows, and also corroborate the mechanism of meander cutoff by opening chutes across point bars.

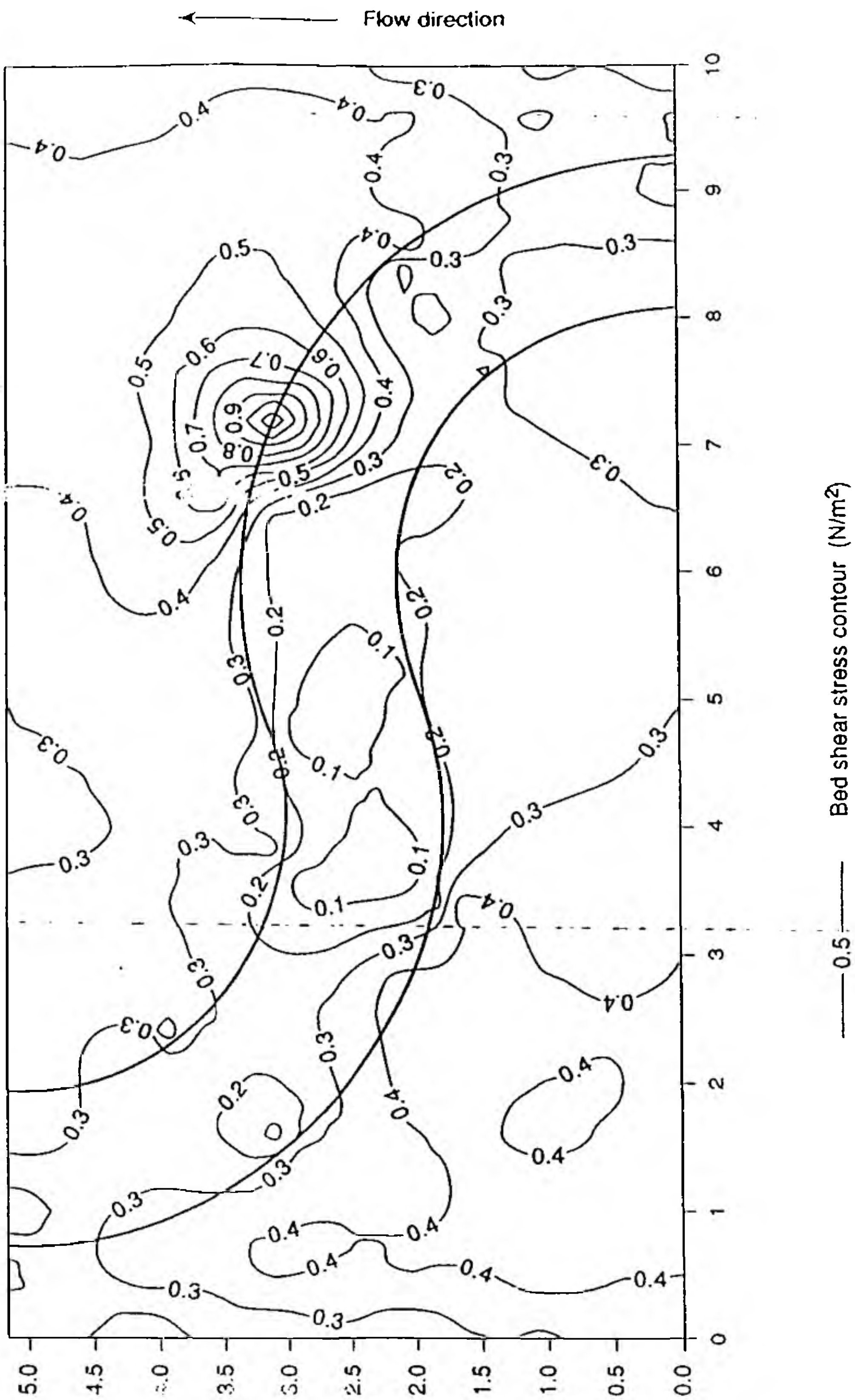


Figure A1.4 Example of boundary shear stress distribution in a meandering compound channel (after Lorena)

ANNEX B: SUMMARY OF THE FCFA METHOD

B.1 Introduction

The material presented in this annex is intended to give the user an insight into the development of the FCFA method. Information and advice, which could not be included in the main part of the manual are presented here. Much of this material appeared in the technical report (Ackers, 1991) and James and Wark (1993) and these references provide a more complete discussion of all aspects of flow in straight compound channels.

The layout of this annex reflects the approach taken when developing the conveyance estimation method. Initially the available laboratory information was collated and summarized. The data from the SERC FCF were used to develop methods for conveyance estimation in straight compound channel flows . The methods were then evaluated against other laboratory data. On the basis of these evaluations the methods were then modified and re applied to data. The most accurate methods were chosen for each flow region and finally bed shear stress data from the SERC FCF were evaluated to provide guidelines for the calculation of bed shear stresses.

B.2 Stage-Discharge Prediction

B.2.1 Available data

The adjustment functions were derived in the first instance from experimental results collected during Phase A of the SERC FCF other data sets were then used to validate and modify the various equations. Laboratory data sets which were available to Ackers (1991) include: SERC FCF Phase A; Asano *et al.* (1985); US Army, Vicksburg (1956); Kiely (1991); Knight *et al.* (1983, 1984); Myers (1978, 1984, 1985); Wormleaton (1982); Prinos and Townsend (1983, 1984) and Ervine and Jasem (1991). Details of the SERC FCF Phase A work are available in Ackers (1991), James and Wark (1993) and the fourteen volumes of data, SR 314, published by HR Wallingford.

The Phase A data were collected in compound channels with a fixed value of aspect ratio (b/h). Application of the original versions of the equations showed that for small over bank depths (Region 1) the aspect ratio of the main channel affects the channel discharge. Data collected by: Asano *et al.* (1985); US Army, Vicksburg (1956); Kiely (1991); Knight *et al.* (1983, 1984); Myers (1978, 1984, 1985); Wormleaton (1982) and Prinos and Townsend (1983, 1984) was used to derive an equation for an aspect ratio correction factor (ARF). The skew channel data collected by Ervine and Jasem (1991) was used to confirm that the same aspect ratio correction factor should be applied to skewed compound channels. The data collected by Myers (1984) was used to choose the best of three possible equations for Region 3.

B.2.2 Overbank flows

The procedure developed by Ackers (1991) follows a channel subdivision approach. The compound channel is divided into three zones:

Zone	Description
1	Main channel
2 and 3	Left and right flood plain areas

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 4.8.

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 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 10^0 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 detailed equations and procedures are presented in section 4.3.1, a brief description of the regions and the correction factors identified is given below, followed by a summary of the results obtained in applying the method to various data sets.

Region 1

Region 1 behaviour occurs at very low overbank stages. Ackers (1991) found that a subtractive correction factor provided the best model:

$$Q = Q_{basic} - DISDEF \qquad (A2.1)$$

where the correction DISDEF depends on: main channel/ flood plain friction factor ratio; the velocity difference between main channel and flood plains; the number of flood plains; flow depths in main channel and flood plains and the aspect ratio of the main channel. In all three other regions Ackers (1991) found that a multiplier correction factor was more appropriate:

$$Q = Q_{basic} \times DISADF \qquad (A2.2)$$

Region 2

At slightly greater stages the flow in the compound channel starts to increase. Ackers (1991) noticed that typical laboratory results plotted on a line approximately parallel to but lower than the coherence curve. He found that the best model for $DISADF_2$ is the value of COH at some "shifted stage", which is larger than the actual stage.

Coherence depends on channel shape and roughness and Ackers (1991) found that the shift required to obtain the shifted stage from the actual stage depends on the main channel side slope and the number of flood plains. Thus the correction factor for region 2 depends on all of these parameters.

Region 3

At larger stages again the laboratory results decrease with stage. Ackers (1991) found that $DISADF_3$ could be expressed as a function of COH for the actual stage. Thus in region 3 the correction factor depends only on the stage, channel cross-section shape and roughness.

Region 4

None of the laboratory data analyzed by Ackers (1991) contained data at large enough stages to confirm the existence of region 4, where the discharge correction factor is expected to increase with increasing stage. Ackers provided a theoretical justification for assuming that $DISADF_4$ should take the value of COH for the given stage.

Skewed compound channels

Having analyzed the straight channel data available from laboratory results Ackers (1991) compared measured and calculated results from three tests with angles of skew of 2.1°, 5.1° and 9.2°. He found that the effect of skewing the main channel to the flood plain is to reduce the discharge capacity further. For the range of skew angles investigated he found that the discharge deficit (DISDEF) for the skewed channel could be related to the DISDEF value for a comparable straight channel by a linear function of skew angle. If the flow is in regions 2, 3 or 4 then the straight channel DISDEF value can be calculated from the final and basic discharges.

General comments

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 (1991).

Ackers (1991) showed in preliminary investigations that most UK rivers with compound sections will flow in Regions 1 or 2 for floods with recurrence intervals up to about 20 years. Calculations should be carefully checked if higher regions are indicated. Artificial or modified channels may operate over a wider range of regions than natural ones.

B.2.3 Evaluation of overbank flow models

Laboratory data

Ackers (1991) provides detailed error statistics from application of the FCFA method to only the Phase A FCF data. He reports a mean error 0.001% with a standard deviation of 0.801% over all of this data with the FCFA method. Wark (1993) carried out an independent verification exercise by applying several methods to a wider range of data. The results given below summarize the analysis carried out. The methods used included the following:

- The Divided Channel Method (DCM) separates the main channel and flood plain flows by vertical divisions. Discharges are calculated separately for the main channel and flood plain zones and then added. Zonal discharges are calculated using a friction equation with the vertical division lines included in the wetted perimeter for the main channel, but not for the flood plains. A variation of this method (DCM2) omits the vertical division lines from the main channel wetted perimeter as well.

- The Single Channel Method (SCM) is the straight forward application of the basic friction law to the complete channel.

- In the FCFA Method (FCFA) the basic zonal discharges as calculated by DCM2 are adjusted using empirical factors based on the SERC FCF Phase A data. The factors and their derivation are described by Ackers (1991) and summarized in section 4.3 above.

- The Lateral Distribution Method, (Wark *et al.*, 1991) is a computational method. The equation describing the lateral distribution of depth integrated flow across the channel is solved numerically. The non-dimensional form of the lateral eddy viscosity (NEV) is used with an average value of 0.16. The distribution of flow is then integrated to provide the zonal and total discharges.

In addition to the FCF Phase A data set these methods were applied to data from laboratory flumes in the University of Ulster (Myers 1990) and the University of New South Wales (Lambert 1993). The mean errors and standard deviations obtained for each method are listed in Table A2.1. The methods were applied to all 197 data points from the FCF work, 20 data points from two of Myers tests and 85 data points from six geometries tested by Lambert.

These results show that of the hand calculation techniques the FCFA method gives the most consistently accurate results over each data set. Only the LDM can match the FCFA method in terms of accuracy and consistency. The large standard deviations are not caused by random error but rather by consistent trends of the errors with stage. James and Wark (1993) discusses this more fully.

Table A2.1 Errors in discharge estimation

Data set Method	FCF A ACC	SD	Myers ACC	SD	Lambert ACC	SD
LDM	3.5	4.1	0.2	7.2	-0.5	16.2
DCM	8.8	10.2	-1.0	6.8	23.5	13.1
DCM2	7.1	3.9	1.7	7.7	31.8	22.8
SCM	N/A		-13.2	6.3	55.0	45.8
FCFA	-2.0	3.8	-5.2	5.2	9.4	14.9

Note: ACC is mean %Error = mean of $100*(Q_{calc} - Q_{meas})/Q_{meas}$
SD is the Standard deviation in the mean value.

Field data

Ackers (1991) applied the FCFA method to a range of field data collected from river gauging sites in England and Wales. Wark *et al.* (1991) and James and Wark (1993) widened this validation to include the methods listed above. The results summarized below are taken from James and Wark (1993). The results differ slightly from those quoted by Ackers (1991), these differences arising from slightly different procedures for idealizing the main channel.

The results quoted are for the River Severn at Montford Bridge and the River Trent at Low Moor. These being the gauging sites with the two most extensive data records for overbank flow. The measured stage discharges were smoothed by taking running averages over three consecutive points. The Manning's n values used in the calculations are those quoted by Ackers (1991). He derived these values from a trial and error application of the FCFA method and hence they are optimised for the FCFA method. James and Wark (1993) also present results with Manning's n values optimised for the LDM. However these are broadly similar to those quoted in Table A2.2.

Table A2.2 Errors in discharge estimation, river gauging data

Data set	Severn		Trent	
Method	ACC	SD	ACC	SD
LDM	3.8	2.7	5.6	8.4
DCM	1.8	2.7	3.0	8.2
DCM2	5.4	3.5	3.4	8.2
SCM	-25.4	5.8	-16.4	7.0
FCFA	-4.1	3.3	-2.2	8.4

Note: ACC is mean %Error = mean of $100*(Q_{calc} - Q_{meas})/Q_{meas}$
SD is the standard deviation in the mean value.

The results for these field gauging sites are less conclusive than those for the laboratory data. Only the SCM gives totally unacceptable results and the other four methods give similar overall accuracies. The majority of the measured data fell in region 1 for the FCFA method. At higher stages The FCFA method is expected to give more accurate predictions.

B.3 Boundary Shear Stress Prediction

The interaction between main channel and flood plain flows also affects the magnitude and distribution of boundary shear stress, and will therefore influence: scour patterns; requirements for scour protection and estimating sediment transport rates.

Calculations for a hypothetical case have suggested that total bed material discharge could be reduced by a factor of two or three. Detailed assessment of these effects will be the subject of future research, Phase C of the FCF. Until new results are available, the effects should be accounted for by using conventional methods with the relevant hydraulic parameters, such as flow velocity and boundary shear stress, determined according to the procedures given in section 4.3. As a provisional measure the bed shear stress data collected during Phase A was analyzed by Ackers (1991) who presented the following conclusions.

(1) Local shear stresses on the flood plain close to the main channel may be five times greater than the value calculated from flow depth and channel gradient.

(2) The peak shear stress on the floodplain occurs at approximately three times the channel depth from the main channel bank.

(3) The average boundary shear stress within the main channel is reduced.

Ackers (1991) provided a simple equation to correct the shear stress calculated from the channel cross-section properties.

ANNEX C: IMPLICATIONS FOR 1-D RIVER MODELLING

C1 1-D River Models in Use within the NRA

Within the NRA a number of standard river modelling packages are used for undertaking both steady and unsteady modelling of rivers. In 1993, the more widespread packages in use were identified as:

	Package	Originator
•	FLUCOMP	(HR Wallingford)
•	SALMON-F	(HR Wallingford)
•	ONDA	(Halcrow)
•	HYDRO	(Mott MacDonald)
•	MIKE11	(DHI)
•	HEC2	(US Army Corps of Engineers)
•	FLOODTIDE	(Babtie-Dobbie)
•	Backwater	(NRA - Thames)

Each of the modelling software packages above have different originators and while they vary in detail, they do have a common purpose in that they are intended to approximate the St Venant equations of 1-D flow. They all use the computational technique of finite differences to solve the St Venant equations and so display some basic similarities to each other.

C2 Existing Methods Used to Calculate Conveyance

All of the packages above require channel cross-sections to be supplied at locations along the river. These cross-sections and other data describing the bed roughness of the channel are then used to calculate the conveyance of each cross-section within the model. Conveyance is a convenient measure of a rivers' capacity to pass discharge. The various 1-D models above all use slightly different methods of calculating conveyance. Typically the methods are based on variants of the divided channel or sum of segment methods. In summary, the following approach to conveyance calculations used in the some of the models are as follows :

Model	Method
FLUCOMP	sum of segments method with option for variable roughness in segments, plus a toggle to the divided channel method if required
SALMON-F	sum of segments method with option for variable roughness in segments
ONDA	sum of large segments / divided channel (or 'panels') method
HYDRO	horizontal/vertical subdivision of the section, treated as a single unit
MIKE11	uses a modified hydraulic radius based on Engelund's method and allows variable roughness with depth
HEC2	sum of segments method with option for variable roughness in segments

C3 Inclusion of the New Methods in 1-D Models

The new methods for calculating stage-discharges in compound channels have a number of implications regarding their use in 1-D river models. Primarily these are changes to the data specification for the cross-sectional data (ie additional data items) and changes to the conveyance calculation procedures.

C3.1 Data requirements

The data requirements for the new methods are slightly greater than those that would currently be specified in existing packages. Modifications to the cross-sectional data inputs would be required to account for additional items such as :

FCFA method

- idealised main channel and floodplain side slopes
- idealised main channel bed and bankfull elevations
- idealised main channel bed and top widths

These items are required for each cross-section in the model.

James and Wark method

- sinuosity of the channel
- meander wavelength
- main channel side slope
- pointers to indicate the limits of the inner flood plain meander belt

Where possible reach average values, based on sub-reaches of the model, should be used to specify these additional data items. The sub-reaches are likely to cover a number of cross-section locations in the model and should be selected such that the geometric parameters (main channel side slope, sinuosity and width of meander belt) remain approximately constant throughout the sub-reach. These data items are readily available from a combination of cross-section and plan surveys of the river reach and would not require any additional resources when undertaking a model study.

C3.2 Conveyance calculations

In general, 1-D models pre-calculate conveyance values at a range of depths and store them in tabular form prior to the backwater or time stepping calculations. In principle, therefore, the major changes to be incorporated into the models are for the existing conveyance calculations to be replaced by the new methods for compound channels.

In unsteady flood modelling, storage on the flood plains can play an important role in the attenuation of flood peaks. In a highly meandering river specifying the flood plain length equivalent to the river length between adjacent cross-section locations may have a tendency to over-estimate the storage area available on the flood plains. This may then lead to errors in the attenuation of a flood wave. It is important therefore to specify the river length and flood plain lengths separately, as some of the above models do.

C.4 Implication for 1-D River Models

There are a number of other issues to be considered when using a package with the new methods of calculating conveyances. The usual procedure when modelling compound channels is to calibrate firstly for the inbank roughness and then proceed to calibrate the overbank roughness. Analysis of the SERC FCF data has shown that the inbank or main channel discharge falls as the water level moves from inbank to overbank conditions. In existing methods this may lead to large errors in the flood plain roughness as the calibration procedure implicitly assumes that the main channel discharge either remains constant at overbank stages or that it increases with increasing depth. This implies that the calculated main channel flows and velocities will be too high at overbank stages and those on the flood plain will be too low. This results in incorrect values for the energy and momentum coefficients, which in turn leads to errors in :

- afflux calculations at structures,
- shear stress and sediment transport properties and
- the effective flood wave speed.

A major factor to be considered, should the new conveyance calculation methods for compound channels be incorporated in existing modelling packages, is that the calibration coefficients obtained from earlier model studies may no longer be applicable in the revised versions of the modelling software. The calibrated roughness coefficients (Manning's n, Colebrook-White k_s or Chezy C) would be effectively compound roughness coefficients which take account of surface and form roughness, vegetation and "resistance" losses due to interaction effects in compound channels. The latter of these is included explicitly in the new hydraulic methods and should therefore not be included in the roughness estimates for the channel or flood plain in any revised model. Considerable effort may therefore be required in re-calibrating existing models if further studies were to be undertaken using a revised modelling package.

C.5 Recommendations

Due to the lack of field data for compound channels it has been impossible to verify fully the new hydraulic methods and it is suggested that the methods only be included in 1-D modelling packages for development purposes at this point in time. The most appropriate development path to follow would be to include the methods in a single 'trial' package so that an assessment and evaluation of the methods could be made. For ease of application and interpretation of results, it would be desirable for this to be a steady-state backwater package (or steady-state module of an unsteady modelling package) with a switch to enable the method of conveyance calculation to be selected using a number of alternative calculation procedures including the newly proposed hydraulic methods.

Tests could then be carried out to find the most appropriate method of specifying the data requirements and to make comparisons with measured field data over river reaches with known or observed stage and discharge information.

C.6 Outline Software Specification

C.6.1 Objectives

Part of the research project commissioned by the NRA included drafting an outline software specification for a professional software package intended to assist engineers in the analysis of compound channels. The software will predict stage discharge relationships for given straight and meandering compound geometries and also analyze available data in order to provide a sound basis for the extrapolation of the methods to higher stages. The specification appears in R&D Project Record 252/2/T, NRA (1993).

It is understood that discussions are being held to decide on the best strategy for producing this software package. It is likely that such a package will be developed in over the next year or so, should the decision to proceed be made.

C.6.2 Background of specification

This specification includes details of:

1) Minimum Hardware
2) Method of use (batch driven, Menu driven, Graphical interface)
3) Identification of appropriate source coding.
4) Data requirements
5) Calculation procedures
6) Format of presentation of results
7) Proposed Menu structure.

The detailed specification presented in R&D Project Record 252/2/T addresses each of these issues and will form the basis of any package developed.

ANNEX D: BIBLIOGRAPHY

Ackers P (1991) The hydraulic design of straight compound channels, Report SR 281, HR Wallingford, UK.

Ackers P (1993) Hydraulic design of two-stage channels, Proc. Instn. Civ. Engnrs Wat, Marit. & Energy, 1992,96, Dec., 247-257, Paper 9988.

Agarwal V C, Garde R J and Ranga Raju K G (1984) Resistance to flow and sediment transport in curved alluvial channels, Fourth Congress, Asian and Pacific Division, IAHR, Thailand, 11-13 September, pp 207-218.

Asano T, Hashimoto H and Fujita K (1985) Characteristics of variation of Manning's roughness coefficient in a compound cross section, IAHR, Proc. 21st Congress, Melbourne, Vol 6, August, pp 30-34.

Bridge J S (1992) A revised model for water flow, sediment transport, bed topography and grain size sorting in natural river bends, Water Resources Research, Vol 28, No 4, pp 999-1013.

Carling P A (1992) Personal communication.

Chang H H (1983) Energy expenditure in curved open channels, Journal of Hydraulic Engineering, Vol 109, No 7, pp 1012-1022.

Chang H H (1984) Variation of flow resistance through curved channels, Journal of Hydraulic Engineering, Vol 110, No 12, pp 1772-1782.

Chang H H (1988) Fluvial processes in river engineering, John Wiley & Sons, ISBN 0-471-631396-6.

Chow V T (1959) Open-Channel Hydraulics, International Student Edition, McGraw-Hill.

Corps of Engineers US Army, Waterways Experiment Station, Vicksburg, Mississippi (1956) Hydraulic capacity of meandering channels in straight floodways.

Cowan W L (1956) Estimating hydraulic roughness, Agricultural Engineering, Vol 37, No 7, pp 474-475.

Einstein H A and Banks R B, (1950). "Fluid resistance of composite roughness, Transactions, American Geophysical Union, Vol 31, No 4, pp 606-610.

Ervine D A and Ellis J (1987) Experimental and computational aspects of overbank floodplain flow, Transactions of the Royal Society of Edinburgh : Earth Science, Vol 78, pp 315-325.

Ervine D A and Jasem H K (1991) Personal communication

Greenhill R (1992) An investigation into compound meandering channel flow, PhD thesis, Department of Civil Engineering, University of Bristol.

Greenhill R K and Sellin R H J (1993) Development of a simple method to predict discharges in compound meandering channels, Proc. Instn. Civ. Engnrs Wat, Marit. & Energy, 1993, 101, Mar., 37-44, Paper 10012.

Hayat S (1965) The variation of loss coefficient with Froude number in an open channel bend. Thesis presented to the University of Iowa in partial fulfilment of the requirements for the degree of Master of Science.

Henderson F M (1966) Open Channel Flow, Macmillan.

Holden A P and James C S (1988) Discharge computation for compound channels, The Civil Engineer in South Africa, Vol 30, No 8, pp 371-376.

Horton R E, (1933). "Separate roughness coefficients for channel bottom and sides". Engineering News-Record, Vol 111, No 22, pp 652-653.

HR Wallingford (1988) Assessing the hydraulic performance of environmentally acceptable channels, Report EX 1799, HR Wallingford, September.

HR Wallingford (1991) Phase A & B Flood Channel Facility Manual, Inception Report, HR Wallingford, Report EX 2485 (November)

HR Wallingford (1992) Phase A & B Flood Channel Facility Manual, Interim Report, HR Wallingford, Report EX 2548 (March)

James M and Brown B J (1977) Geometric parameters that influence floodplain flow, Corps of Engineers US Army, Waterways Experiment Station, Vicksburg, Mississippi.

James C S and Wark J B (1992) Conveyance estimation for meandering channels, SR 329, HR Wallingford, December.

James C S and Wark J B (1993) Conveyance estimation for meandering compound channels, Part I. Development., Submitted to Proc. Instn. Civ. Engnrs.Wat, Marit. & Energy.

James C S (1994) Evaluation of methods for predicting bend losses in meandering channels, Technical note, Journal of Hydraulic Engineering, ASCE, Vol 120, No 2, February.

Jasem H K (1990) Flow in two stage channels with the main channel skewed to the flood plain direction, PhD thesis, University of Glasgow.

Kazemipour A K and Apelt C J (1979) Shape effects on resistance to uniform flow in open channels, Journal of Hydraulic Research, Vol 17, No 2, pp 129-148.

Kazemipour A K and Apelt C J (1983) Energy losses in irregular channels, Journal of Hydraulic Engineering, Vol 109, No 10, pp 1374-1379.

Kiely G, Javan M and McKeogh E J (1989) A comparison of turbulence measurements in straight, single meander and multiple meander channels, Int'l. Confr. on Channel Flow and Catchment Runoff, Centennial of Manning's and Kuichling's Rational Formula, University of Virginia, May.

Kiely G (1989) Experimental study of the mechanisms of flood flow in meandering channels, Proc. of the 23rd IAHR Congress, Ottawa, Canada, August.

Kiely G (1990) Overbank flow in meandering channels - the important mechanisms, Proceedings of the International Conference on River Flood Hydraulics, Wallingford, 17 - 20 September, pp 207-217.

Kikkewa H, Ikeda S and Kitagawa A (1976) Flow and bed topography in curved open channels, Journal of the Hydraulics Division, ASCE, Vol 102, No HY9, pp 1327-1342.

Knight D W and Demetriou J D (1983) Flood plain and main channel interaction, Proc ASCE, Jrnl Hydr. Eng, Vol 109, 8, August.

Knight D W, Demetriou J D and Hamed M E (1984) Stage discharge relations for compound channels, Proc 1st Intl Conf channels and channel control structures, April, ed Smith K V H, Springer Verlag.

Knight D W and Sellin R H J (1987) The SERC Flood Channel Facility, Journal of the Institution of Water and Environmental Management, Vol 1, No 2, pp 198-204.

Knight D W Samuels P G and Shiono K (1990) River flow simulation: Research and developments., IWEM 89 Conference paper, Jrnl. IWEM,4, April.

Knight D W, Yuan Y M and Fares Y R (1992) Boundary shear in meandering channels, Proc. Int. Symp. on Hydraulic Research in Nature and Laboratory, Wuhan, China, November, 1992.

Krisnamurthy M and Christensen B A, (1972). "Equivalent roughness for shallow channels". Journal of Hydraulics Div, ASCE, Vol 98, HY12, pp 2257-2262

Lambert M L (1993) Personal communication

Leopold L B, Bagnold R A, Wolman M G and Brush L M (1960) Flow resistance in sinuous or irregular channels, USGS Professional Paper 282-D, Washington D C, pp 111-134.

Lorena M L (1991) Personal communication

Lotter G K, (1933). "Considerations on hydraulic design of channels with different roughness of walls", Transations, All-Union Scientific Research Institute, Leningrad, Vol 9, pp 238-241.

McKeogh E J and Kiely G (1989) A comparison of velocity measurements in straight, single meander and multiple meander channels, Int'l. Confr. on Channel Flow and Catchment Runoff, Centennial of Manning's and Kuichling's Rational Formula, University of Virginia, May.

Mockmore C A (1944) Flow around bends in stable channels, Transactions, ASCE, Vol 109, pp 593-618.

Myers (1978) Momentum transfer in a compound channel, J Hydr. Resch., vol 16, IAHR.

Myers (1984) Frictional resistance in channels with flood plains, 1st Intl Conf. on channels and channel control structures, April, ed Smith K V, Springer Verlag.

Myers (1985) Flow resistance in smooth compound channels, experimental data, University of Ulster, March.

Myers W R C (1991) Personal communication.

Nelson J M and Smith J D (1989) Flow in meandering channels with natural topography, in River Meandering, Ikeda S and Parker G (Eds), American Geophysical Union, Water Resources Monograph 12, pp 69-102.

National Rivers Authority (1991) Design method for straight compound channels, R&D Note 44, NRA.

National Rivers Authority (1993) Assessment of flows in meandering compound channels, R&D Project Record 252/2/T, NRA.

Onishi Y, Jain S C and Kennedy J F (1976) Effects of meandering in alluvial streams, Journal of the Hydraulics Division, ASCE, Vol 102, No HY7, pp 899-917.

Posey C J (1976) Computation of discharge including over-bank flow, Civil Engineering, ASCE, pp 62-63.

Prinos P and Townsend R D (1983) Estimating discharge in compound open channels, in Proceedings of the 6th Canadian Hydrotechnical Conference, Ottawa, Canada. R D Townsend (Ed), vol 1, pp 129-146.

Prinos P and Townsend R D (1984) Comparison of methods for predicting discharge in compound open channels, Advances in water resources, vol 7, Dec.

Ranga Raju, K. G. (1970). Resistance relations for alluvial streams, La Houille Blanche, No 1.

Ramsbottom D M (1989) Flood discharge assessment, interim report, Report SR 195, HR Wallingford, March.

Rouse H (Ed) (1950) Engineering Hydraulics, John Wiley and Sons, New York.

Rozovskii I L (1957) Flow of water in bends of open channels, The Academy of Science of the Ukraine SSR, translated from the Russian by the Israel Program for Scientific Translations, Jerusalem, Israel, 1961.

Samuels P G and James C S (1992) Assessing flow in straight and meandering compound channels - using results from the SERC facility, Presented at MAFF Conference of river and coastal engineers, Universiyty of Loughborough, 6-8th July.

Scobey F C (1983) The flow of water in flumes, US Department of Agriculture, Tech Bull No 393.

Sellin R H J and Giles A (1988) Two stage channel flow, Final Report for Thames Water Authority, Department of Civil Engineering, University of Bristol.

Sellin R H J and Giles A (1989) Flow mechanisms in spilling meander channels, Proceedings, 23rd Congress, 1AHR, Ottawa, August 1989.

Sellin R H J, Giles A and van Beeston D P (1990) Two stage channel flow, Post-Implementation of a two-stage channel in the River Roding, Essex, J IWEM, 1990, Vol 4, pp 119, 130.

Smith C D (1978) Effect of channel meanders on flood stage in valley, Jrnl Hydr Div, ASCE, Vol 104, HY1, pp 49-58.

Soil Conservation Service (1963) Guide for selecting roughness coefficient "n" values for channels, US Department of Agriculture, Washington D C.

Sooky A A (1964) The flow through a meander - flood plain geometry, PhD thesis, Purdue University.

Stein C J and Rouve G (1988) 2D-LDV-Technique for measuring flow in a meandering channel with wetted flood plains - a new application and first results, Proc Intl Conf on fluvial hydr, Budapest, June 1988.

Stein C J and Rouve G (1989) 2D depth-averaged numerical predictions of the flow in a meandering channel with compound cross section, Hydrosoft, 1989, Vol 2, no. 1.

Toebes G H and Sooky A A (1967) Hydraulics of meandering rivers with flood plains, Journal of the Waterways and Harbors Division, ASCE, Vol 93, No WW2, pp 213-236.

Wark J B, Samuels P G and Ervine D A (1990) A practical method of estimating velocity and discharge in compound channels, Proc of Intl Conf on River Flood Hydraulics, Wallingford, 17-20 September.

Wark J B, Ramsbottom D M and Slade J E (1991) Flood discharge assessment by the lateral distribution method, Report SR277, HR Wallingford.

Wark J B (1993) Overbank flow in straight and meandering compound channels, Ph.d Thesis, Dept. of Civil Engineering, The University of Glasgow, July.

Wark J B and James C S (1993) Conveyance estimation for meandering compound channels, Part II Application and Verification., submitted to Proc. Instn. Civ. Engrs, Wat, Marit. & Energy.

Willetts B B, Hardwick R I and MacLean A G (1990) Model studies of overbank flow from a meandering channel, Proc of Intl Conf on River Flood Hydraulics, Wallingford, 17 - 20 September.

Willetts B B and Hardwick R I (1991) Personal communication.

Willetts B B (1992) The hydraulics of overbank flow in meandering rivers, Seminar on Multiple Stage Channels, HR Wallingford Ltd., Wallingford, UK.

Willetts B B and Hardwick R I (1993) Stage dependency for overbank flow in meandering channels, Proc. Instn. Civ. Engnrs Wat, Marit. & Energy, 101, Mar., 45-54, Paper 10049.

Wormleaton P R, Allen J and Hadjipanos P (1982) Discharge assessment in compound channel flow, Journal of the Hydraulics Division, ASCE, Vol 108, No HY9, pp975-994.

Wormleaton P R and Merrett D J (1990) An improved method of calculation for steady uniform flow in prismatic main channel/flood sections, Journal of Hydraulic Research, Vol 28, No 2, pp157-174.

Yen B C and Yen C L (1983) Flood flow over meandering channels, Proceedings, Rivers '83, River Meandering, ASCE, pp 554-561.

ANNEX E: CALCULATION OF DARCY FRICTION FACTOR

1) From channel geometry and measured discharge

$$V = Q / A$$

$$R = A / P$$

$$f = (8 g R S) / V^2$$

2) Manning's equation

$$f = (8 g n^2) / R^{1/3}$$

3) Rough turbulent law

$$f = [2 \log_{10} (14.8 R / k_s)]^{-2}$$

4) Smooth turbulent law

$$R_e = 4 V R / \nu$$

$$f = [2 \log_{10}((R_e f^{1/2}) / 2.51)]^{-2}$$

5) Colebrook-White transition law

$$f = [-2 \log_{10} (k_s / 14.8 R + 2.51 / (R_e f^{1/2}))]^{-2}$$

Notes:

1) If the smooth or transitional turbulence laws are used then the friction factor depends on the velocity as well as the bed roughness. Therefore an iterative procedure must be adopted when applying these formulae to the FCFA method and the James and Wark method.

2) The variables and symbols used above have their normal meanings.

Kikkewa H, Ikeda S and Kitagawa A (1976) Flow and bed topography in curved open channels, Journal of the Hydraulics Division, ASCE, Vol 102, No HY9, pp 1327-1342.

Knight D W and Demetriou J D (1983) Flood plain and main channel interaction, Proc ASCE, Jrnl Hydr. Eng, Vol 109, 8, August.

Knight D W, Demetriou J D and Hamed M E (1984) Stage discharge relations for compound channels, Proc 1st Intl Conf channels and channel control structures, April, ed Smith K V H, Springer Verlag.

Knight D W and Sellin R H J (1987) The SERC Flood Channel Facility, Journal of the Institution of Water and Environmental Management, Vol 1, No 2, pp 198-204.

Knight D W Samuels P G and Shiono K (1990) River flow simulation: Research and developments., IWEM 89 Conference paper, Jrnl. IWEM,4, April.

Knight D W, Yuan Y M and Fares Y R (1992) Boundary shear in meandering channels, Proc. Int. Symp. on Hydraulic Research in Nature and Laboratory, Wuhan, China, November, 1992.

Krisnamurthy M and Christensen B A, (1972). "Equivalent roughness for shallow channels". Journal of Hydraulics Div, ASCE, Vol 98, HY12, pp 2257-2262

Lambert M L (1993) Personal communication

Leopold L B, Bagnold R A, Wolman M G and Brush L M (1960) Flow resistance in sinuous or irregular channels, USGS Professional Paper 282-D, Washington D C, pp 111-134.

Lorena M L (1991) Personal communication

Lotter G K, (1933). "Considerations on hydraulic design of channels with different roughness of walls", Transations, All-Union Scientific Research Institute, Leningrad, Vol 9, pp 238-241.

McKeogh E J and Kiely G (1989) A comparison of velocity measurements in straight, single meander and multiple meander channels, Int'l. Confr. on Channel Flow and Catchment Runoff, Centennial of Manning's and Kuichling's Rational Formula, University of Virginia, May.

Mockmore C A (1944) Flow around bends in stable channels, Transactions, ASCE, Vol 109, pp 593-618.

Myers (1978) Momentum transfer in a compound channel, J Hydr. Resch., vol 16, IAHR.

Myers (1984) Frictional resistance in channels with flood plains, 1st Intl Conf. on channels and channel control structures, April, ed Smith K V, Springer Verlag.

Myers (1985) Flow resistance in smooth compound channels, experimental data, University of Ulster, March.

Myers W R C (1991) Personal communication.

Nelson J M and Smith J D (1989) Flow in meandering channels with natural topography, in River Meandering, Ikeda S and Parker G (Eds), American Geophysical Union, Water Resources Monograph 12, pp 69-102.

National Rivers Authority (1991) Design method for straight compound channels, R&D Note 44, NRA.

National Rivers Authority (1993) Assessment of flows in meandering compound channels, R&D Project Record 252/2/T, NRA.

Onishi Y, Jain S C and Kennedy J F (1976) Effects of meandering in alluvial streams, Journal of the Hydraulics Division, ASCE, Vol 102, No HY7, pp 899-917.

Posey C J (1976) Computation of discharge including over-bank flow, Civil Engineering, ASCE, pp 62-63.

Prinos P and Townsend R D (1983) Estimating discharge in compound open channels, in Proceedings of the 6th Canadian Hydrotechnical Conference, Ottawa, Canada. R D Townsend (Ed), vol 1, pp 129-146.

Prinos P and Townsend R D (1984) Comparison of methods for predicting discharge in compound open channels, Advances in water resources, vol 7, Dec.

Ranga Raju, K. G. (1970). Resistance relations for alluvial streams, La Houille Blanche, No 1.

Ramsbottom D M (1989) Flood discharge assessment, interim report, Report SR 195, HR Wallingford, March.

Rouse H (Ed) (1950) Engineering Hydraulics, John Wiley and Sons, New York.

Rozovskii I L (1957) Flow of water in bends of open channels, The Academy of Science of the Ukraine SSR, translated from the Russian by the Israel Program for Scientific Translations, Jerusalem, Israel, 1961.

Samuels P G and James C S (1992) Assessing flow in straight and meandering compound channels - using results from the SERC facility, Presented at MAFF Conference of river and coastal engineers, Universiyty of Loughborough, 6-8th July.

Scobey F C (1983) The flow of water in flumes, US Department of Agriculture, Tech Bull No 393.

Sellin R H J and Giles A (1988) Two stage channel flow, Final Report for Thames Water Authority, Department of Civil Engineering, University of Bristol.

Sellin R H J and Giles A (1989) Flow mechanisms in spilling meander channels, Proceedings, 23rd Congress, IAHR, Ottawa, August 1989.

Sellin R H J, Giles A and van Beeston D P (1990) Two stage channel flow, Post-Implementation of a two-stage channel in the River Roding, Essex, J IWEM, 1990, Vol 4, pp 119, 130.

Smith C D (1978) Effect of channel meanders on flood stage in valley, Jrnl Hydr Div, ASCE, Vol 104, HY1, pp 49-58.

Soil Conservation Service (1963) Guide for selecting roughness coefficient "n" values for channels, US Department of Agriculture, Washington D C.

Sooky A A (1964) The flow through a meander - flood plain geometry, PhD thesis, Purdue University.

Stein C J and Rouve G (1988) 2D-LDV-Technique for measuring flow in a meandering channel with wetted flood plains - a new application and first results, Proc Intl Conf on fluvial hydr, Budapest, June 1988.

Stein C J and Rouve G (1989) 2D depth-averaged numerical predictions of the flow in a meandering channel with compound cross section, Hydrosoft, 1989, Vol 2, no. 1.

Toebes G H and Sooky A A (1967) Hydraulics of meandering rivers with flood plains, Journal of the Waterways and Harbors Division, ASCE, Vol 93, No WW2, pp 213-236.

Wark J B, Samuels P G and Ervine D A (1990) A practical method of estimating velocity and discharge in compound channels, Proc of Intl Conf on River Flood Hydraulics, Wallingford, 17-20 September.

Wark J B, Ramsbottom D M and Slade J E (1991) Flood discharge assessment by the lateral distribution method, Report SR277, HR Wallingford.

Wark J B (1993) Overbank flow in straight and meandering compound channels, Ph.d Thesis, Dept. of Civil Engineering, The University of Glasgow, July.

Wark J B and James C S (1993) Conveyance estimation for meandering compound channels, Part II Application and Verification., submitted to Proc. Instn. Civ. Engrs, Wat, Marit. & Energy.

Willetts B B, Hardwick R I and MacLean A G (1990) Model studies of overbank flow from a meandering channel, Proc of Intl Conf on River Flood Hydraulics, Wallingford, 17 - 20 September.

Willetts B B and Hardwick R I (1991) Personal communication.

Willetts B B (1992) The hydraulics of overbank flow in meandering rivers, Seminar on Multiple Stage Channels, HR Wallingford Ltd., Wallingford, UK.

Willetts B B and Hardwick R I (1993) Stage dependency for overbank flow in meandering channels, Proc. Instn. Civ. Engnrs Wat, Marit. & Energy, 101, Mar., 45-54, Paper 10049.

Wormleaton P R, Allen J and Hadjipanos P (1982) Discharge assessment in compound channel flow, Journal of the Hydraulics Division, ASCE, Vol 108, No HY9, pp975-994.

Wormleaton P R and Merrett D J (1990) An improved method of calculation for steady uniform flow in prismatic main channel/flood sections, Journal of Hydraulic Research, Vol 28, No 2, pp157-174.

Yen B C and Yen C L (1983) Flood flow over meandering channels, Proceedings, Rivers '83, River Meandering, ASCE, pp 554-561.

ANNEX E: CALCULATION OF DARCY FRICTION FACTOR

1) From channel geometry and measured discharge

$V = Q / A$

$R = A / P$

$f = (8 g R S) / V^2$

2) Manning's equation

$f = (8 g n^2) / R^{1/3}$

3) Rough turbulent law

$f = [2 \log_{10} (14.8 R / k_s)]^{-2}$

4) Smooth turbulent law

$R_e = 4 V R / \nu$

$f = [2 \log_{10}((R_e f^{1/2}) / 2.51)]^{-2}$

5) Colebrook-White transition law

$f = [-2 \log_{10} (k_s / 14.8 R + 2.51 / (R_e f^{1/2}))]^{-2}$

Notes:

1) If the smooth or transitional turbulence laws are used then the friction factor depends on the velocity as well as the bed roughness. Therefore an iterative procedure must be adopted when applying these formulae to the FCFA method and the James and Wark method.

2) The variables and symbols used above have their normal meanings.