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The Practical Application of an Enhanced Conveyance Calculation in Flood Prediction

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Abstract

An enhanced one-dimensional mathematical model for simulating flood levels and calculating stage-discharge relationships is presented. Enhanced conveyance subroutines have been developed and incorporated into the commercially available river modelling software ISIS. The newly developed software has been verified using experimental and field data.

When a river overtops its banks there is a vigorous interaction between slow moving flood plain flow and faster moving main channel flow. This interaction mechanism has been the focus of intense research over the past forty years. A selective review of this research is detailed with particular attention to the case of meandering channels.

The Ackers Method and the James & Wark Method are two discharge capacity methods that have emanated from this recent research and are considered to be the most practically suitable methods and are indeed recommended by the Environment Agency of England and Wales. The methods account for interaction effects when flow is overbank in a straight and meandering channel respectively. It is these methods that have been incorporated into the commercially available and industry leading one-dimensional river model ISIS to enable an enhanced conveyance calculation.

The newly developed software has been tested against the Flood Channel Facility Series A and B experiments to a satisfactory level of accuracy. The testing included prediction of stage discharge relationships and water level prediction.

In addition it has been applied to the River Dane in Cheshire which is highly meandering and suited to the James and Wark methodology. This was intended to give practical advice concerning the use of the James and Wark Method and the degree of accuracy in estimating the 'channel parameters' which are required by this method. The results of this work showed that a significant rise in water level prediction is obtained when using the enhanced code. Also, it was clear that a high degree of accuracy was not required in estimating the 'channel parameters' with the possible exception of the sinuosity term.

The new software was also applied to the River Kelvin near Glasgow which is dissimilar to the Flood Channel Facility and the River Dane, however it is representative of many British rivers. The James and Wark Conveyance Method was applied to this 19 km reach and calibration results were compared using the current industry standard method, the Divided Channel Method, and the James and Wark Method. While improved calibration results were obtained, there were locations where significant adjustment of roughness coefficients was required. This application showed the significance of applying an enhanced conveyance calculation in a natural environment and the practicalities involved in doing so.

This research project has bridged the gap in knowledge between improved discharge capacity or conveyance methods and practical one-dimensional river modelling. The enhanced software that has been developed is shown to be more accurate than the current industry standard method.

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1.0 Introduction

The problem of flooding has existed since man chose to live alongside rivers. While a river can provide food, power and recreation it can also kill and devastate communities situated nearby. It has long been of interest to the public in general, not just engineers, how to predict the maximum flood levels that might occur, and how to protect against such events. This is especially of interest in modern times as there is a public perception that flooding is becoming more common.

As the modern world develops more and more land is being developed whether it be for housing or industry. Often, such developments are located beside or near rivers where they are obviously at risk from flooding. Developers need to realise that the flood plain, or land adjacent to the river channel, is an integral part of the river system. During high flows it is this land that will be inundated and any property built in this area is at extreme risk.

In order to assess flood flows the river engineer uses a one-dimensional river modelling tool which effectively creates a mathematical model of a river. This 'tool' can provide the relationship between stage and discharge and maximum flood level predictions. With this information a suitable flood protection scheme can then be designed.

One-dimensional river models are widely used despite a limited degree of accuracy. A major limitation of such models being that the only energy loss mechanism it assumes is that of boundary friction. i.e. surface roughness in a natural river. This thesis will detail the other energy loss mechanisms, and ways of modelling them, that do occur in river flow.

In particular, recent research (See Chapter 2) has focussed on the interaction of main channel and flood plain flow. This is where the flow in the main channel has exceeded the bankfull depth and flooded onto the flood plain. (see Figure 1.01) A channel that exhibits flow at two stages, similar to Figure 1.01, can be referred to as a compound channel.

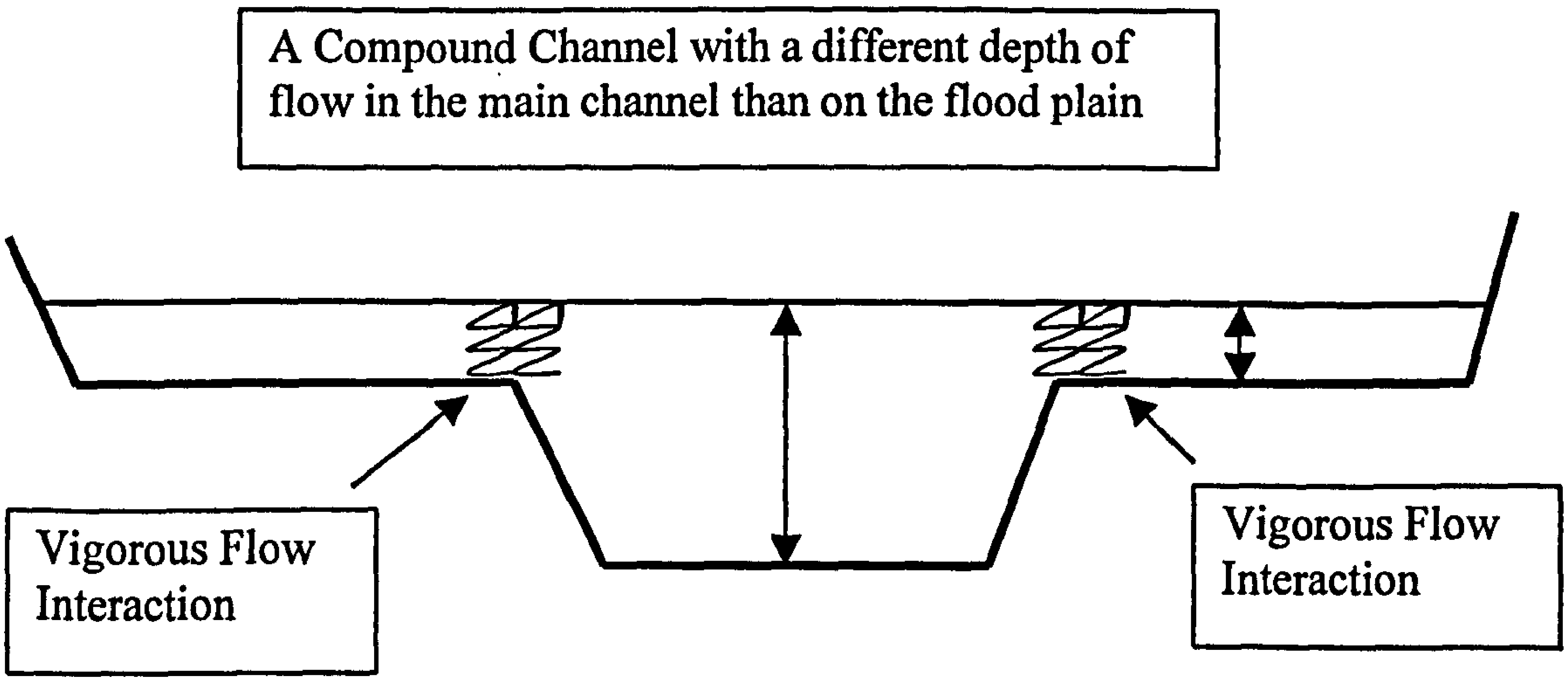


Figure 1.01 Overbank Flow (Compound Channel)

As the slow moving flow on the flood plain interacts with the faster moving flow in the main channel there is a resulting vigorous exchange of momentum which dissipates energy. This is not accounted for in current one-dimensional river models.

There are now several discharge capacity or conveyance methods available to model such losses yet none has so far been, utilised by the practising engineer. It is important to be confident of the discharge capacity of a river as it is fundamentally required in the following engineering applications, flood alleviation, drainage and water supply.

It would therefore seem reasonable to assess these new discharge capacity methods, that attempt to account for these interaction losses, and to incorporate them into a one-dimensional river model. Only when this has been done will the true merit of the various methods be realised in the most practically useful manner.

This research project has attempted to incorporate two new discharge capacity or stage-discharge calculation methods into the industry standard one-dimensional river modelling package ISIS. The result is an enhanced discharge capacity and flood prediction tool.

A review of relevant literature has been undertaken to highlight the key developments in research concerning overbank flow interaction. This body of research has followed two broad categories, namely that of Straight compound channels and Meandering compound channels. (i.e. channels with straight or meandering plan form)

While a meandering river will exhibit three-dimensional motion, a full 3D analysis of a natural river would not be feasible at present due to cost and computing time. In engineering practice it is the one-dimensional model that is widely used as it is extremely efficient in terms of ease of use, time and cost. The fundamental theory of one-dimensional river models will be reviewed including a derivation of the St Venant equations which form their base. The solution of these complex non-linear partial differential equations will also be detailed.

The incorporation and testing of two new discharge capacity methods to the commercially available and industry leading river modelling package ISIS will be presented in Chapter 4.

In Chapter 5 and 6 the newly developed software will be applied in a practical manner to both the River Dane and the River Kelvin to assess its use to the practising engineer. Industry has not utilised the body of research that is available concerning this subject and this thesis aims to address this. As a result the practicalities of this work are stressed at all times.

The Thesis essentially reports on the background, software development, testing and application of newly developed river modelling software and details the merits or otherwise of this work.

Chapter 2 Literature Review

2.0 Introduction

In river engineering the stage discharge relationship is an extremely important piece of information. Normally, this relationship is obtained from statistical analysis of data measured at a river gauging station. Due to the predominance of inbank flows there tends to be a high level of accuracy for stages up to bankfull. In 1964, Sellin observed an anomaly in the stage discharge relationship when water levels marginally exceed bankfull depth. The reason for this anomaly has been the subject of research since the early 1960's. The following section gives an overview of the work carried out to date.

2.1 Straight Compound Channel Research

Research concerning straight compound channels has tended to focus on discharge prediction, velocity distribution, boundary shear stress distribution and turbulence measurements. This vast body of research is not directly relevant to the research described in this thesis and as a result only a very brief discussion has been included here for the purposes of providing background information.

Amongst the earliest studies on straight channels with overbank flow, Sellin (1964) identified the anomaly in the stage discharge relationship as flow just exceeds bankfull. Point velocity and stage-discharge measurements were recorded in a variety of geometrical combinations. Of particular interest was the study of the surface flow which was sprinkled with aluminium powder and photographed. Figure 2.01 illustrates the vertical vortices that were observed along the main channel and flood plain interface. Sellin explained this phenomenon by momentum exchanged between the main channel and the flood plain. Zhelezneyakov (1965) and Imamoto et al (1991) also observed these secondary currents using photographic techniques.

Sellin (1964) noticed that at low flood plain depths the discharge falls below that of the bankfull discharge. As the flood plain depth increases then the discharge begins to increase again. He also showed that the discharge at each water level, above bankfull, was less than that calculated assuming bed friction as the only energy loss mechanism. This implied that there must be other energy loss mechanisms associated with overbank flow in straight channels.

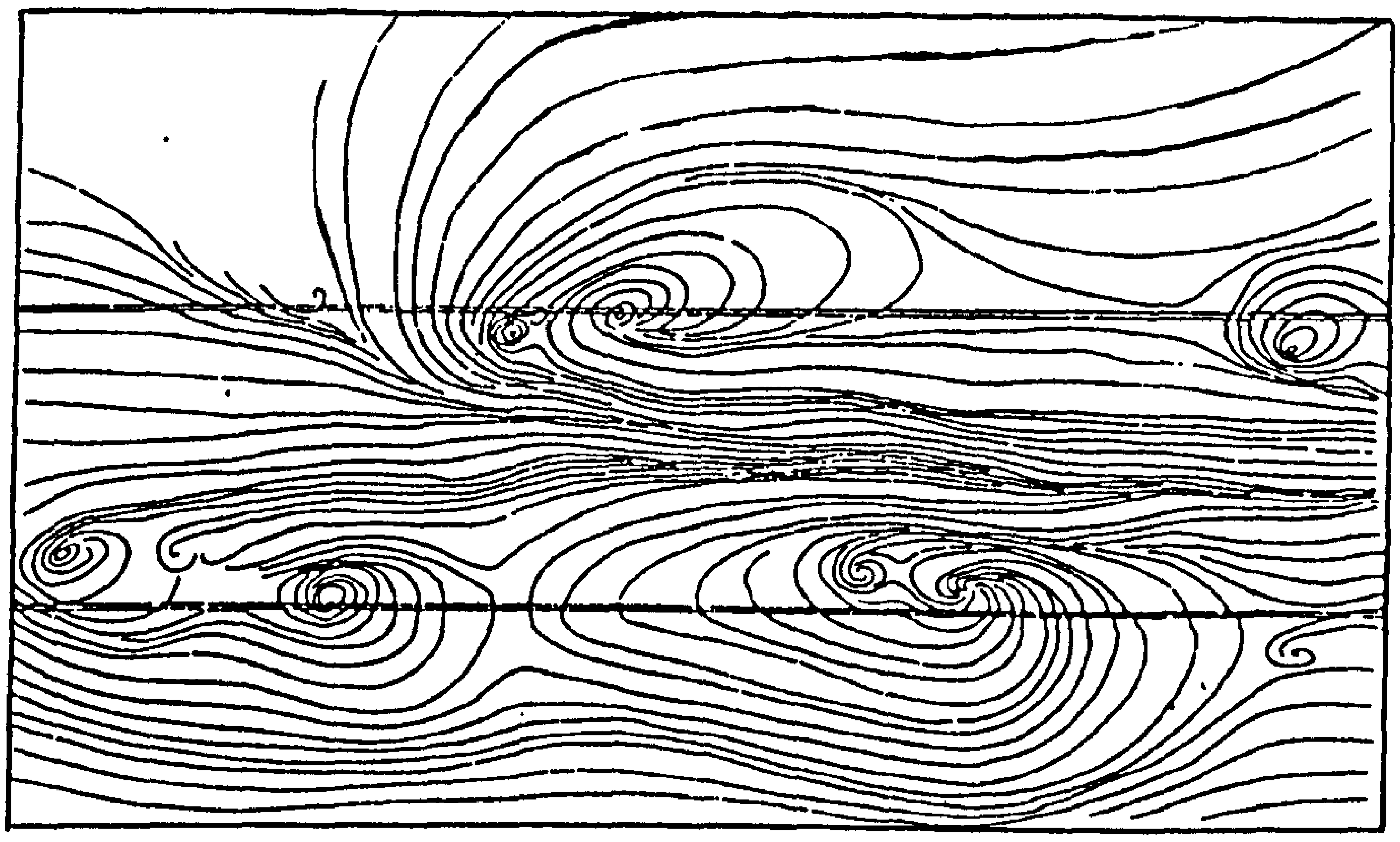


Figure 2.01 Secondary Currents Observed by Sellin (1964)

Importantly, this phenomenon is not limited to laboratory studies and was observed at field scale by Bhowmik and Demissie (1982) and Knight et al (1989) for the Salt Creek river in Illinois and the River Severn at Montford bridge respectively.

Perhaps the most significant experimental study performed on straight compound channels is the Series A experiments undertaken at the Flood Channel Facility at HR Wallingford (see Knight and Sellin (1987)). The apparatus itself is 50m long and 10m wide and between 1986-1989 a series of different models with a straight main channel were constructed. The aim of these experiments was to observe the various flow processes associated with overbank flow. Specifically, the following four parameters were tested to ascertain its influence:

- Relative flow depth
- Main channel side slope
- Channel width
- Relative roughness

As a result of these experiments a comprehensive stage discharge prediction method, the Ackers Method (1991), was developed. (See Section 2.2.3.5) The Ackers Method accounted for the various flow processes that were observed during the Series A experiments.

According to Wark, James and Ackers (1994), the important flow mechanisms that affect the conveyance of a straight compound channel are:

- The velocity differential between the main channel and flood plains which induces a lateral shear layer between those two regions
- 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 noticeable secondary circulations form vortices with vertical axes located along the main channel / flood plain interfaces.
- The secondary circulations and lateral shear effects cause the boundary shear stresses to be redistributed around the cross-section, with increased values at the edge of the flood plain close to the main channel.
- These mechanisms combine to reduce the discharge in the main channel and increase it on the flood plains.
- The secondary currents also affect the vertical and lateral distributions of longitudinal velocity, particularly in the main channel.
- The strength of the interaction depends on main channel / flood plain widths and side slopes; main channel / flood plain bed roughness and the velocity differential across the shear layer.
- The bed shear stress on the flood plains is increased by the interaction. In the main channel it is reduced.

The various flow processes observed, as proposed by Shiono and Knight (1991), in a straight compound channel are illustrated in Figure 2.02

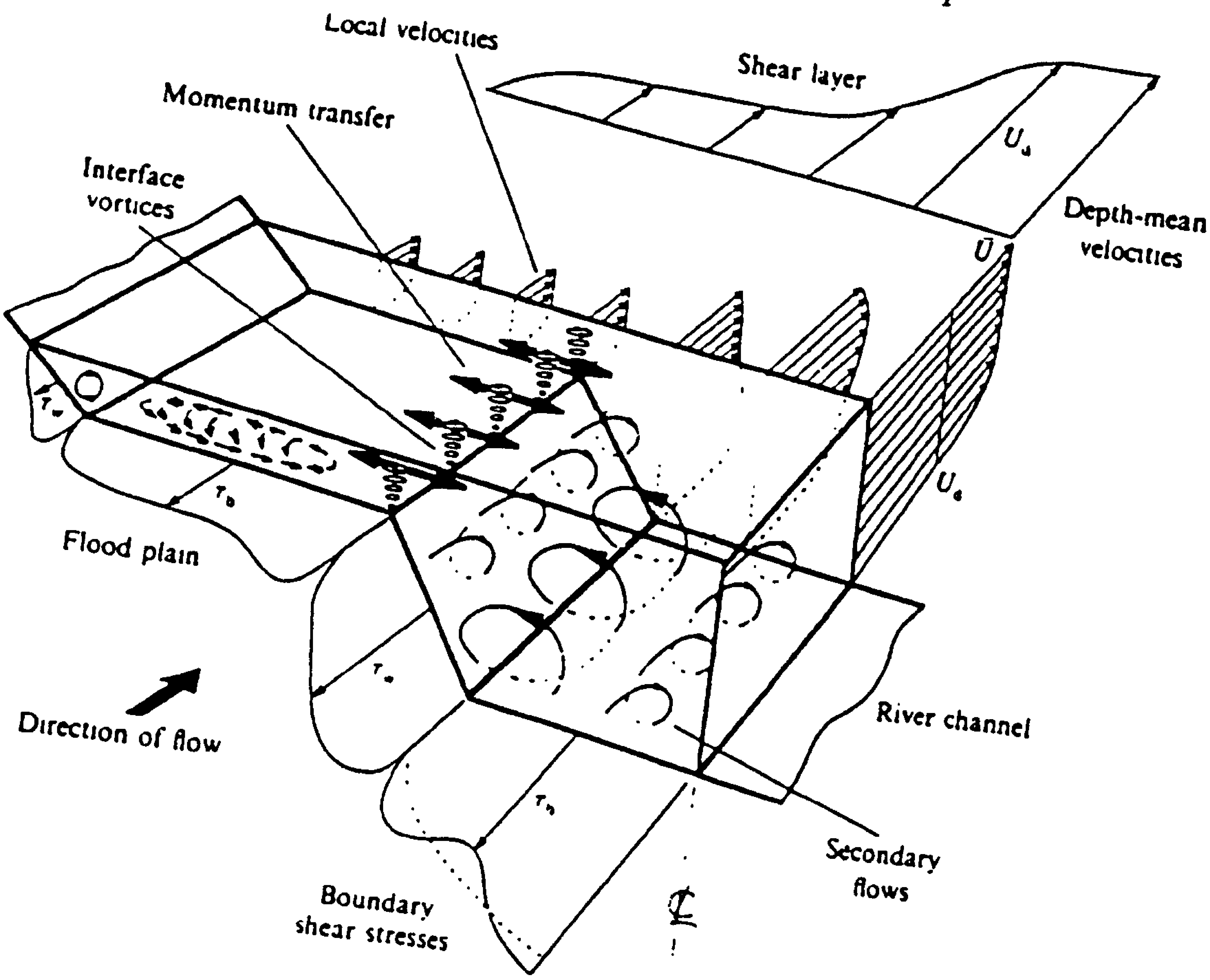


Figure 2.02 Flow Processes in a Straight Compound Channel
(Shiono and Knight (1991))

Further details of straight compound channel experiments can be found in Sellin (1964), Ervine and Baird (1982), Knight et al (1983), Myers (1978, 1984), James and Brown (1977), Rajaratnam and Ahmadi (1981), Wormleaton et al (1982), Wormleaton (1986), Ackers (1991), Wark (1993), Field, Lambert and Williams (1998) or Macleod (1998) for a detailed description of straight channel experiments.

2.2 Straight Compound Channel Modelling Techniques

At present Engineers use straight channel methods when calculating river stage discharge relationships or calculating conveyance. The conveyance calculation is usually performed within a one-dimensional river model and is calculated by either the Single Channel or the Divided Channel Methods.

2.2.1 Single Channel Method (SCM)

This method of modelling a compound channel involves, as the name suggests, a single channel of flow with no sub-divisions. It is practically undesirable as it does not allow for any variation in bed roughness across the channel. In addition, there is a significant flaw in its prediction for depths just above the bankfull depth. At these small overbank depths there is a significant increase in the wetted perimeter with a disproportionate increase in flow area. This leads to values of hydraulic radius that are artificially small.

Knight et al (1989) observed this phenomenon on the River Severn, where a back calculation of 'n' using Manning's equation resulted in a significant reduction in this term. This implies that the flow resistance would decrease when flow goes overbank. It has since been shown that there are additional energy losses when floodplain flow interacts with main channel flow which contradicts this finding. It is suggested that the hydraulic radius term is inappropriate for compound channels Mcleod (1998).

Wark (1993) has reviewed the historic development of this method.

2.2.2 Divided Channel Method (DCM)

In order to avoid the discontinuity at bankfull level the cross-section can be sub-divided into a main channel with floodplain zones and is referred to as the Divided Channel Method. This method was first proposed by Lotter (1933). Manning's equation is generally applied in each flow zone to obtain a zonal estimate of discharge. These are then summed to give a total discharge. Figure 2.03 illustrates some of the possible sub-divisions that could be used, Wormleaton and Merrett (1990).

Ramsbottom (1988) applied various divided channel methods to field data and concluded that the best results were obtained by including the vertical divisions of the wetted perimeter of the main channel but not the flood plains. The divided channel method is commonly used in one-dimensional river models, such as ISIS and MIKE 11, without the inclusion of division lines in the wetted perimeter. This can be considered the industry standard method at present.

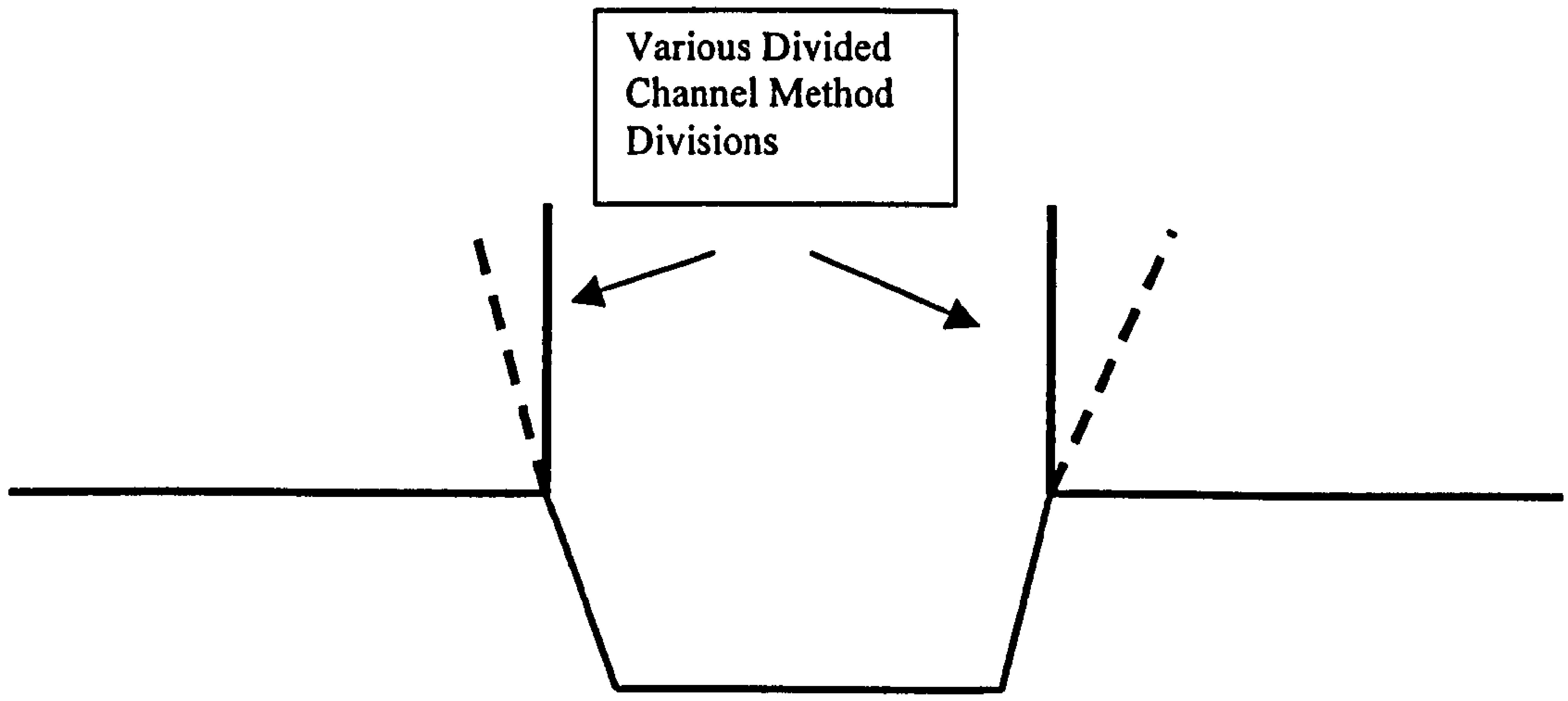


Figure 2.03 Divided Channel Method Divisions

The method assumes that all energy losses are due to bed friction and makes no allowance for interaction losses. Consequently, the DCM can be in error by as much as 30%, Myers and Brennan (1990).

2.2.3 New Methods

Research has tended to follow three distinct paths in modelling of compound channel flows. Specifically, these are Apparent shear stress methods, Adjustment factor methods and Lateral distribution methods.

2.2.3.1 Apparent Shear Methods

Apparent shear stress methods have been considered by authors Baird and Ervine (1982), Knight and Demetriou (1983), Knight and Hamed and Wormleaton and Merrett (1990). This being where the secondary losses are accounted for by including an apparent shear stress on the vertical division lines which separate the main channel from the flood plain. The methods proposed by the various authors are empirical in nature and were based on a limited range of experimental conditions.

2.2.3.2 Adjustment Factor Methods

These methods are generally based on a basic divided channel approach and then 'adjusted' to account for interaction losses. Baird and Ervine (1982), Wormleaton and Merret (1990) proposed adjustment factors that were related to the apparent shear stress while Ackers (1991) developed a method that simply corrects a 'basic discharge' calculated assuming only bed friction losses.

2.2.3.4 Lateral Distribution Methods (LDM)

The lateral distribution method (LDM) is based on estimating the distribution of flow across a section and then integrating this to obtain the total discharge. The starting point for the LDM is the full 3D Reynolds equations for turbulent flow. These are simplified by integrating in the vertical direction to produce the 2D shallow water equations. However, in the case of a straight channel, the shallow water equations can be simplified further to a one-dimensional equation which describes the lateral variation of depth averaged velocity and discharge across a channel, Wark et al (1990), Knight and Samuels (1989) and Shiono and Knight (1990).

The following equation describes the lateral distribution of depth-integrated flow in a channel.

$$\rho g H So - \frac{1}{8}\rho f U_d^2\left(1+\frac{1}{S^2}\right)^{0.5} + \frac{\partial}{\partial y}\left\{\rho \lambda H^2\left(\frac{f}{8}\right)^{0.5} U_d \frac{\partial U_d}{\partial y}\right\} = 0 \qquad (2.01)$$

where U_d is the depth averaged velocity, λ is the dimensionless eddy viscosity, f is the Darcy-Weisbach friction factor and s is the main channel lateral side slope, H is the water depth and ρ is the flow density.

The secondary flow term is set to zero in equation 1 by Shiono and Knight (1989) as they assumed this to have a negligible effect. This can be considered a limiting factor.

Wark et al (1990) used an alternative form of equation 1 i.e. discharge intensity

$$gDS - \frac{Bf|q|q}{8D^2} + \frac{\partial}{\partial y}\left[Vt\frac{\partial q}{\partial y}\right] = 0 \qquad (2.02)$$

where B is a factor relating stress on an inclined surface to stress on a horizontal surface, D is the local flow depth, f is the Darcy Weisbach friction factor, g is gravitational acceleration, q is the unit flow, S the surface slope and U is the depth averaged velocity. The variable q is continuous even across a vertical step in depth where as the depth averaged velocity U as used by Shiono and Knight (1989) will display large discontinuities in such situations.

In this formulation the secondary flow losses are again ignored. The dimensionless eddy viscosity parameter was introduced and used as a "catch-all" parameter for lateral eddy viscosity and secondary flow. The difficulty in applying this equation came from this parameter. Knight (1999) has made some recommendations in estimating this parameter for a range of channel geometries.

Shiono and Knight (1991) introduced a secondary current term to their previous 1989 method. This was based on experimental results, and assumed that the shear stress due to secondary flow decreases approximately linearly either side of a maximum value which occurs at the boundary between main channel and flood plain.

The application of these quasi-two-dimensional analytical solutions has produced good estimates of the lateral distribution of depth-averaged velocity for mostly laboratory data. The fundamental limitation of this method being that it is for near uniform overbank flows in straight channels. There is no account for river meandering. A recent paper by Ervine et al (2000) develops the basic technique of Shiono and Knight (1989, 1991) to be applicable to both straight and meandering channels.

Some field applications using these methods are considered later in this thesis.

2.2.3.5 Ackers Method (1991)

This method is fundamentally based on the Flood Channel Facility Series A experiments. It is used to estimate stage discharge relationships in straight compound channels.

The method follows a sub-division technique as shown in Figure 2.04.

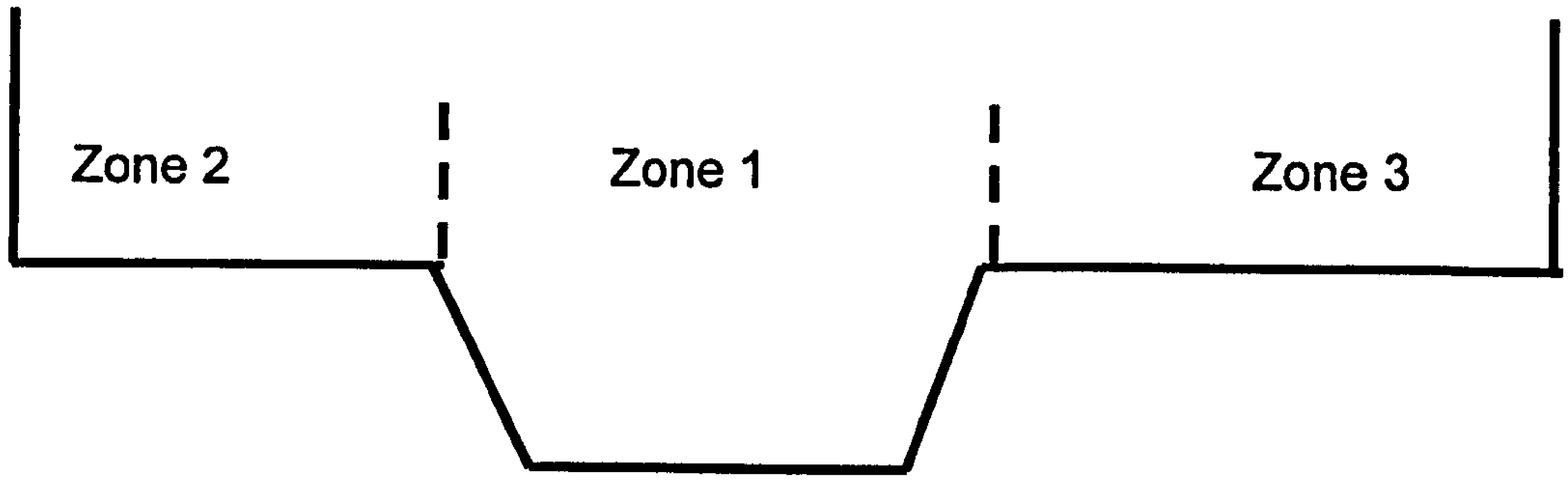


Figure 2.04 The Ackers Method cross-sectio

Zone 1 Main Channel
Zone 2 Left Flood Plain
Zone 3 Right Flood Plain

The cross-section is divided using vertical division lines which are not included as wetted perimeter values. A basic discharge 'Qbasic' is calculated for each zone assuming bed fiction to be the only source of energy loss. A range of adjustments are then made for a series of flow regions to account for interaction losses. The method calculates an estimate of discharge for each of the flow regions and selects the correct value subject to a series of rules.

The flow interaction process is very complex and, as can be seen in Figure 2.05, alternately increases and decreases with flow depth.

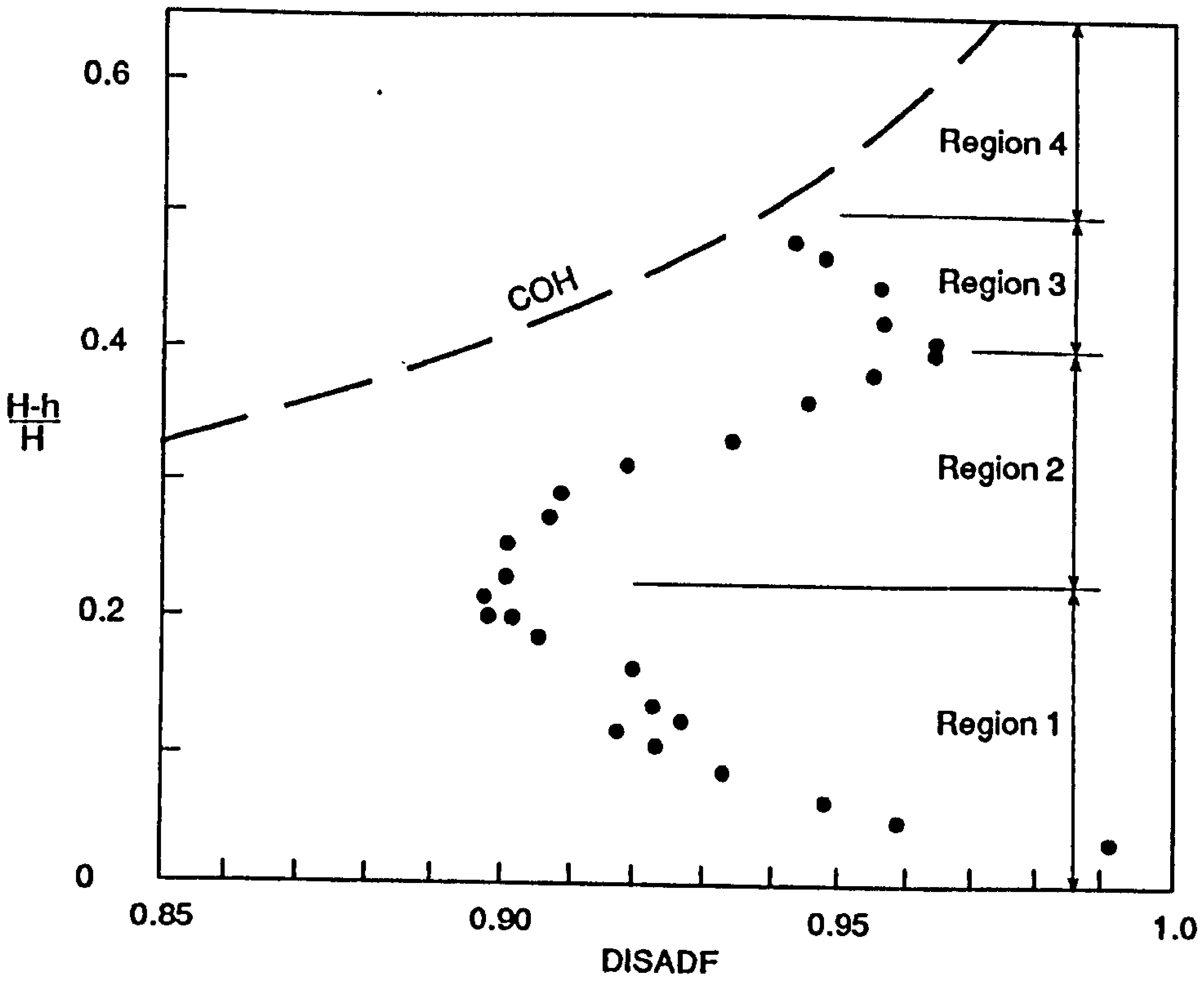


Figure 2.05 Four Regions of Flow Behaviour (Ackers (1991))

Also shown in Figure 2.05 is the channel coherence curve. This parameter is defined as the ratio of the conveyance calculated as a single cross-section to that calculated by summing the conveyance of the separate flow zones. The value of coherence is equal to unity or less and is a measure of the strength of interaction losses. A coherence value of 0.5 would imply 50% non-bed friction energy losses. As the channel depth increases COH tends towards a value of 1, implying that the compound channel behaviour is approaching that of a simple channel at high depths.

The method provides a different adjustment factor for each flow region. A logical process of selecting the correct discharge is then provided. Using the Ackers method additional corrections are available for skewed channels and for the full design of a compound channel. The various adjustment factors for the flow regions are as follows:

Region 1

This region of flow behaviour occurs at very low overbank stages

Q = Qbasic – DISDEF **(2.03)**

Where the correction factor DISDEF depends on the relative friction factor; velocity difference between main channel and flood plains; number of flood plains; flow depths in main channel and flood plains and the main channel aspect ratio. This was the only region where a subtractive correction factor was applied. In all other regions a multiplier correction factor was used. i.e.

$$\mathbf{Q_{2,3,4} = Q_{basic} * DISADF_{2,3,4}} \qquad \mathbf{(2.04)}$$

Region 2

At higher overbank stages the flow resistance in a straight compound channel reduces, illustrated by the turning point in Figure 2.05. Ackers (1991) observed that the laboratory results plotted on a line approximately parallel to but lower than the coherence curve. He decided to use as the model for $DISADF_2$ the value of COH at some "shifted stage" which is significantly larger than the actual stage.

Coherence depends on channel shape and roughness and 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. (Wark, James and Ackers (1994)).

Region 3

This flow region occurred at higher still stages and the resistance to flow increased. The adjustment factor $DISADF_3$ was expressed as a function of COH for the actual stage and depended on stage, cross-sectional shape and roughness.

Region 4

The data analysed by Ackers (1991) did not contain data at high enough stages to confirm the existence of region 4, where the flow resistance decreases with stage i.e. the adjustment factor $DISADF_4$ will increase with stage. It was proposed that the adjustment factor in this region should take the value of COH for the given stage.

Once the method has calculated the flow estimates for each flow region it selects the correct value from the following rules:

If QR1 ≥ QR2 then Q = QR1
If QR1 < QR2 and QR2 ≤ QR3 then Q = QR2
If QR1 < QR2 and QR3 < QR2 then Q = QR3 unless QR4 > QR3 then Q = Q4

A detailed description of the empirical equations used in this method can be found in Ackers (1991). The method has been applied to laboratory and field data with a reasonable level of accuracy and is currently recommended for use by the Environment Agency. A flow chart detailing the Ackers Method can be found in Chapter 3.

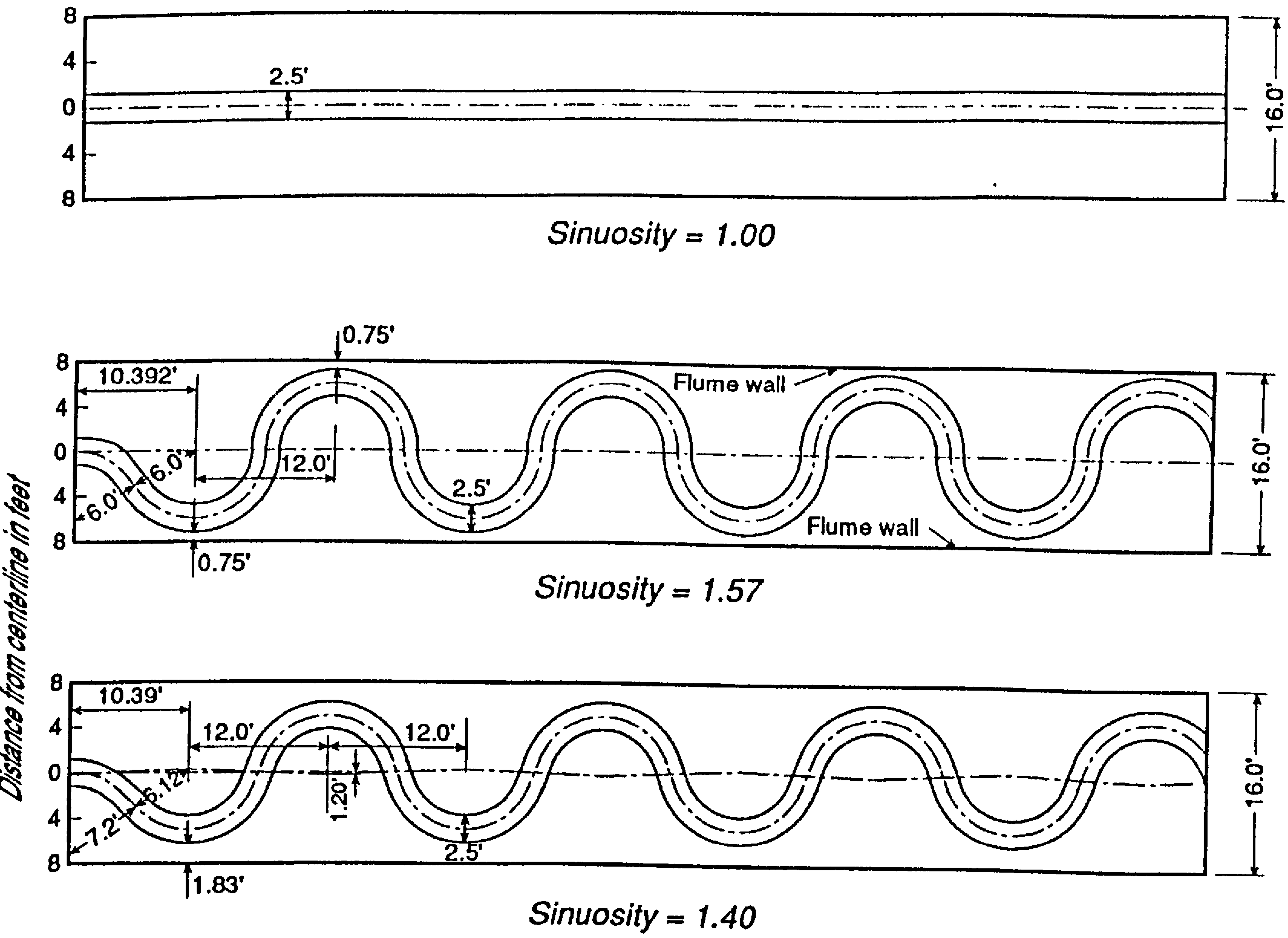
In recent times, research has moved on to the more complicated case of meandering compound channels. This indeed is of more practical interest as rivers tend to exhibit a meandering plan-form. The following section reviews the relevant work on meandering compound flow.

2.3 Meandering Compound Channel Research - *Flow Mechanisms*

The following section highlights some of the key experimental programs that have helped identify the flow processes occurring during overbank flow in a meandering compound channel. It is the findings of these researchers that have facilitated the development of models to account for the various flow processes.

2.3.1 United States Army Vicksburg (1956)

This early study was at large scale, 30.5m long by 9.2m wide, and was intended to observe how a range of geometrical parameters affected the discharge capacity. The parameters tested were radius of curvature of bends, sinuosity of main channel, depth of overbank flow, ratio of overbank area to main channel area and flood plain roughness. Figure 2.06 illustrates the various flumes modelled during this study.



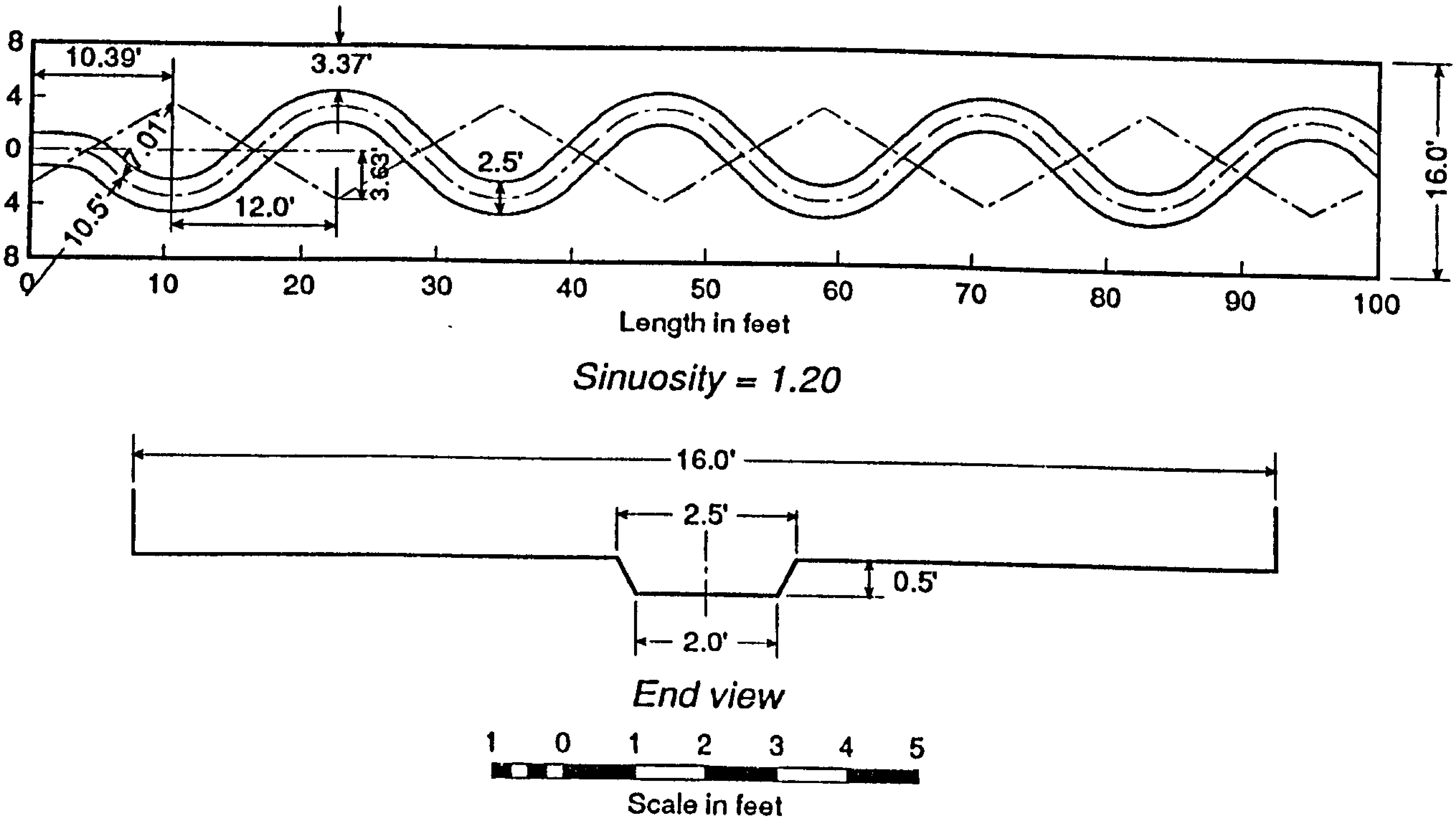


Figure 2.06 US Army Corps, Vicksburg (1956) Experimental Flumes

As can be seen from Figure 2.06 there were three different sinuosity's tested ranging from straight (sinuosity = 1.0) to medium-high (sinuosity = 1.57). The main channel for all experiments was trapezoidal and dimensions are shown on Figure 2.06. The first set of experiments carried out had smaller dimensions than that showed in Figure 2.06. These were a base width 1 foot and depth 0.5 feet. These were deemed to be unsatisfactory (inconclusive) and as a result the channel dimensions were increased to that shown in Figure 2.06 i.e. base width 2 feet and depth 0.5 feet. The experimental results were in terms of stage discharge relationships. For each experimental arrangement the discharge was measured at bankfull and three overbank stages.

The study concluded the following

- Where the main channel is narrow (and small) compared to the floodplain, the effect of channel sinuosity on the total discharge capacity is small.
- The effect of increased main channel sinuosity is to reduce the total discharge capacity.
- When the flood plain is more than three times the width of the meander belt the effect of the sinuosity on the total discharge capacity is small.
- The effect of increased flood plain roughness is to reduce the total discharge capacity.

Despite this study being over 40 years old it is arguably the only rival to the Flood Channel Facility experiments in terms of scale and findings relevant to practical use.

2.3.2 Toebes and Sooky (1967), Sooky (1964)

Toebes and Sooky (1967) performed a series of experiments to investigate the hydraulics of overbank flow in meandering channels with flood plains. The apparatus used was 7.3m long by 1.18m wide and of low sinuosity (1.09). The experimental arrangement consisted of a meandering channel of rectangular cross-section and is shown in Figure 2.07.

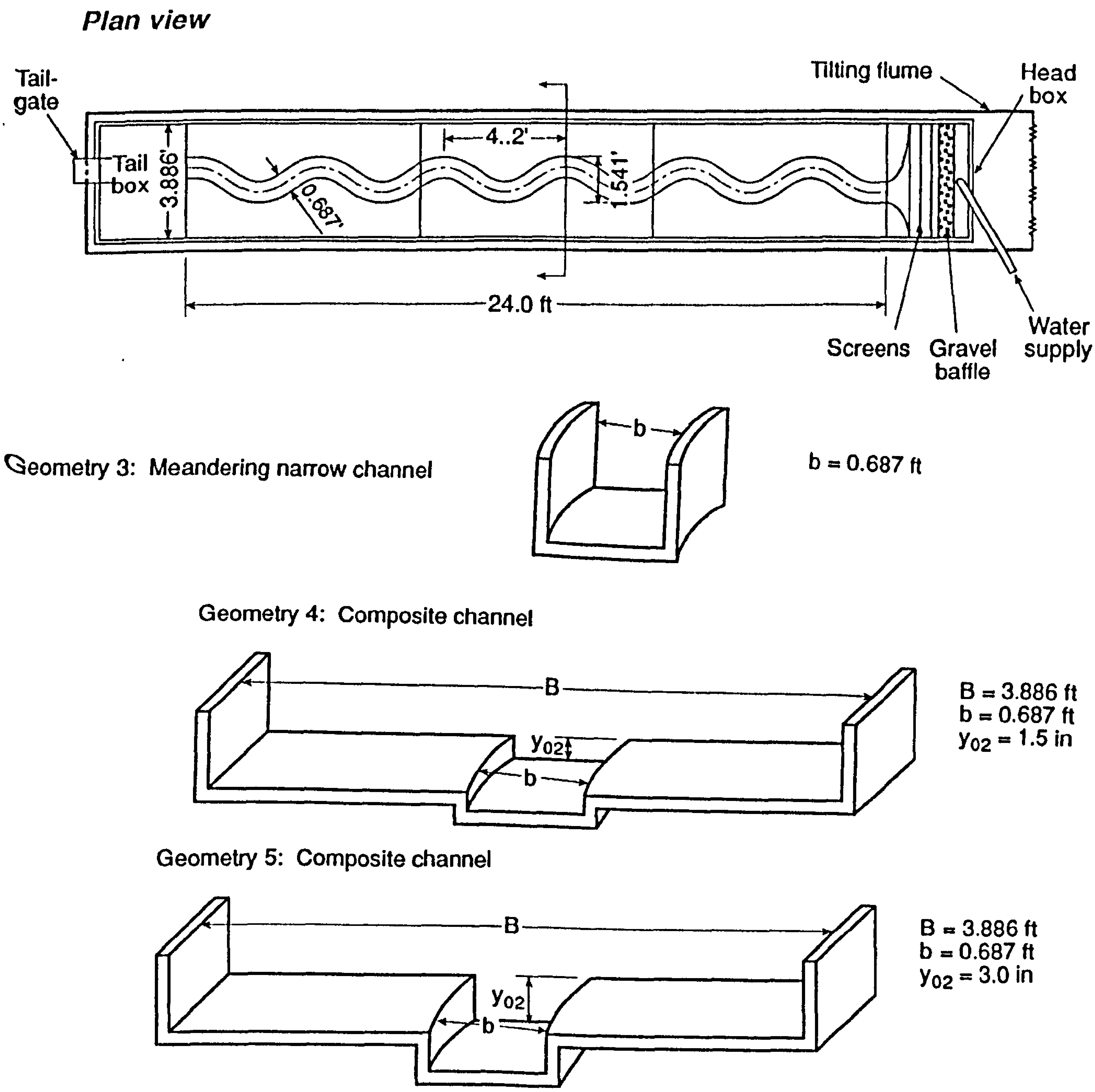


Figure 2.07 Experimental Apparatus of Toebes and Sooky (1967)

This study tested two different channel depths and seven longitudinal slopes and readings were taken concerning stage discharge and velocity variation over both main channel and flood plains. It was considered by these authors that, as of 1961, there was an almost complete lack of hydraulic data on meandering flood plain flow fields.

In order to test the accuracy of the stage discharge measurements the cross-section was divided into two separate regions by a horizontal line at bankfull. Then, discharge was calculated for each region, assuming only bed frictional losses, and summed to give a total discharge for each water level. Essentially, these authors discovered that this discharge, when calculated assuming only bed frictional losses, was over-predicted. This meant that all energy loss mechanisms were not being accounted for.

In an attempt to allow for additional energy losses the wetted perimeter term (T) was increased for both flow regions. This term was increased until there was agreement between the predicted and measured discharges. 'T' was considered to be a complicated function of overbank flow depth, mean velocities in the two zones and the longitudinal slope.

Another finding of this study being that during overbank flow the secondary currents, which are induced by channel bends, rotate in the opposite sense to inbank flow. During inbank flow the secondary currents are known to rotate with the surface currents directed towards the outside of the bend while this study observed, when flow was out of bank, the surface currents being directed toward the inside of the bend. This was an early observation of a phenomenon that has since been confirmed by recent studies by Stein et al (1988 & 1989) and Kiely (1989).

2.3.3 Kiely (1989 & 1990)

Kiely (1989) performed a series of experiments on both straight and meandering compound channels in order to determine the flow mechanisms during overbank flow. In Kiely's own words "this physical understanding is fundamental to any future numerical modelling". Kiely (1990) concentrated on meandering compound channels and undertook velocity and turbulence measurements, using a Laser Doppler Anemometer, for a range of geometries.

The experimental apparatus used in this study was 14.4m long by 1.2m wide and had a discharge capacity of 50l/s. A glass floor in the flume allowed uninterrupted access to an area 2.4m long by 1.2m wide for Laser Doppler Anemometry (LDA). Both the main channel and flood plains were constructed of smooth glass.

The study found that when flow is just out of bank the direction of flow is almost parallel to the main channel walls. However, when the flow is at highest depths, the flow direction is changed to being almost parallel with the outer flood plain walls. This indicates the existence of horizontal shearing at the junction of flood plain and main channel flows.

Kiely observed a reduction of 50% in the meandering main channel velocities compared with an equivalent straight channel. The velocity measurements also revealed that the maximum value, at all meander sections, was located on the flood plain outside the meander belt. The maximum velocities in the main channel, above and below bankfull, are close to the inner bend.

In addition, the following flow mechanisms were identified for the meandering geometry

- Secondary currents
- Horizontal shearing
- Flow expansion and contraction
- Downstream effects of cross-over flow

2.3.4 Willetts and Hardwick (1990)

Willetts and Hardwick (1990) performed a series of experiments of meandering plan form with the aim of identifying the key flow mechanisms associated with overbank flow. In addition, they were interested in the effect of channel geometry and sinuosity on the stage discharge relationship. The apparatus used in these experiments was 11m long by 1.2m wide and is shown below in Figure 2.08. Both trapezoidal and quasi-natural cross-section geometries were tested.

Figure 2.09 shows an illustration of some of the features observed in this study.

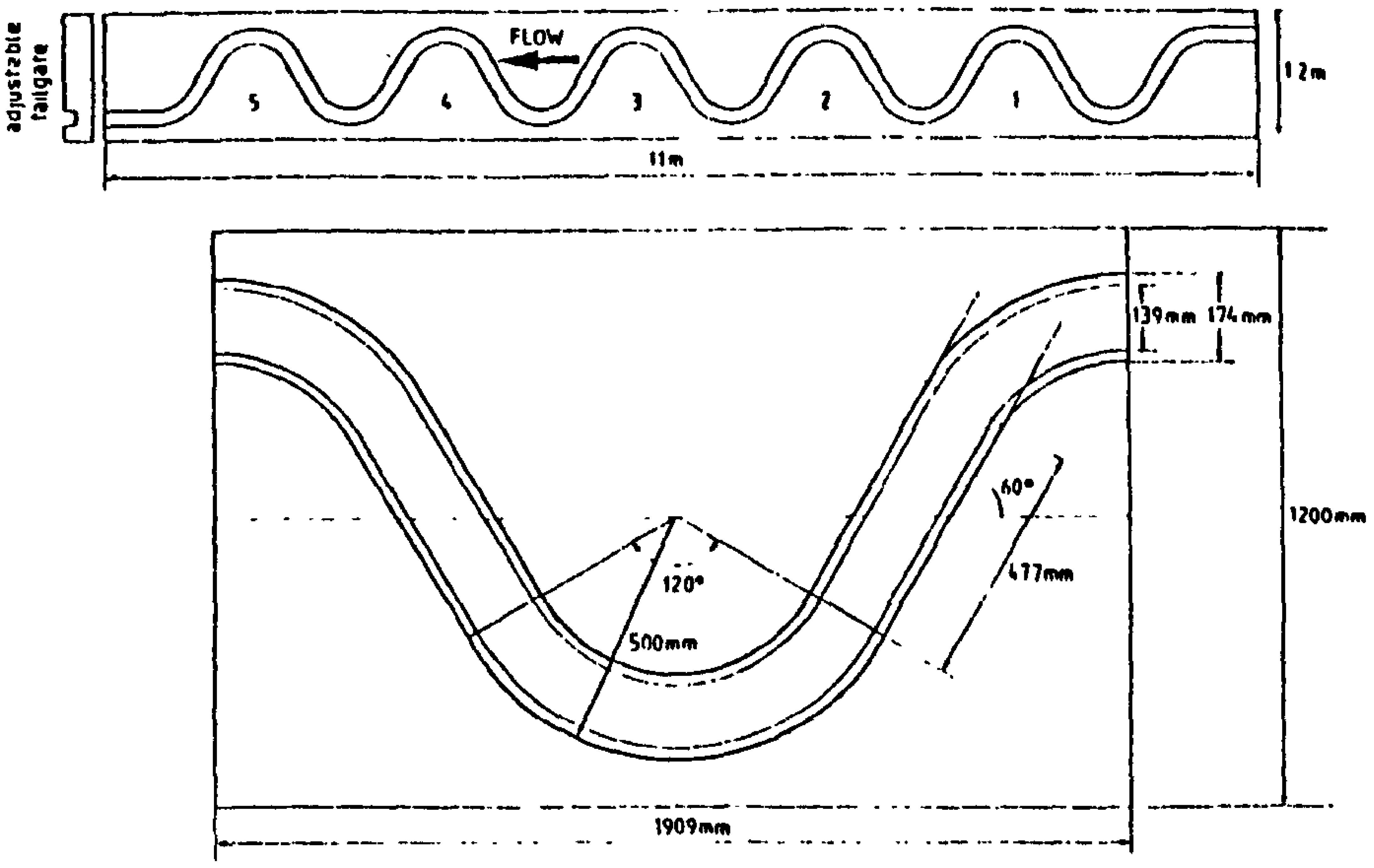


Figure 2.08 Experimental Apparatus of Willets and Hardwick (1990)

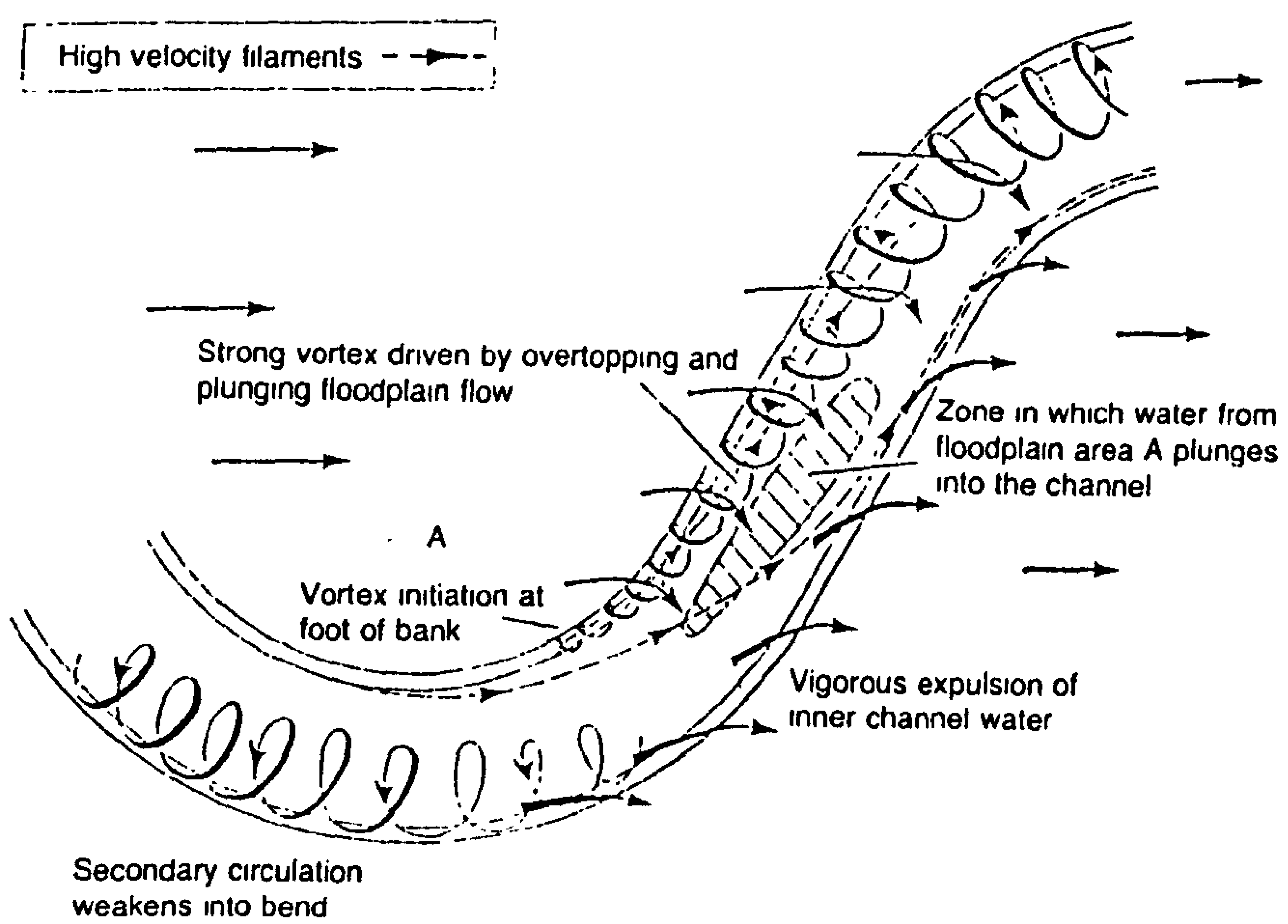


Figure 2.09 Flow Features within a meandering channel
(Willets and Hardwick (1993))

2.3.5 Lorena (1992) - Flood Channel Facility Experiments (1989-1991)

During the period 1989-1991, Lorena, carried out the Flood Channel Facility Series B experiments for meandering compound channels. This large scale experimental facility was 50m long by 10m wide and was constructed of smooth mortar. Two sinuosities were constructed i.e. 1.37 and 2.04 with two main channel geometries (trapezoidal and pseudo-natural). These experiments allowed the large-scale investigation of overbank flow processes. A more detailed description of these experiments is given in Chapter 4. A review of the main experimental findings is given in Ervine Willets Sellin and Lorena (1993).

2.3.6 Ervine Willets Sellin and Lorena (1993)

Ervine Willets Sellin and Lorena (1993) investigated 7 parameters that they thought would influence the flow interaction between the main channel and flood plain, in a meandering channel. The authors noted that when a river flows over-bank the sources of energy dissipation and flow resistance are much more difficult to determine. The reason for this being that there is extensive three-dimensional mixing of river and flood plain flows, especially in the case of meandering compound flows. In order to define some of these "sources of energy dissipation" the Flood Channel Facility Series B experiments were performed. The experimental apparatus was 50m long and 10m wide, and had a maximum flow rate of $1.1m^3/s$. The parameters tested were as follows:

- Sinuosity
- Relative Roughness of the flood plain with the main channel
- Aspect Ratio of the main channel
- Meander Belt Width relative to total floodway width
- Relative Depth of flow on flood plain compared with the main channel
- Cross-sectional shape and side slope of the banks of the main channel
- Flood plain topography

The results of these experiments detail the response of the discharge capacity to changes in the 7 parameters in terms of a non-dimensional correction factor F*. This term is defined in equation 2.05 and ranges between 0 and 1.

F* = actual measured discharge / theoretical discharge (i.e. bed friction only)
(2.05)

The theoretical discharge is calculated for each cross-section division and is the same as the Divided Channel Method. For each of the parameters tested (only 6 out of the 7 are discussed) the results are discussed in terms of F* and non-bed friction energy losses. For example, when a sinuosity of 2.0 was tested the value of F* was around 0.6 which implies 40% non-bed friction losses. This paper was important as it revealed the scale of non-bed friction losses in relation to a range of tests. The experiments were also carried out at a large scale and provide the raw data for the development of further modelling techniques. Results and discussion from the Flood Channel Facility Series B experiments can be found in Sellin et al (1993).

2.3.7 Liu and James (1997)

Liu and James (1997) carried out a series of experiments that focussed on the effects of flood plain geometry on the conveyance of meandering compound channels. Essentially, they constructed a 1:4 model of the SERC FCF 60 degree trapezoidal channel. Seven different geometrical arrangements were tested such as differing flood plain widths, sinuous flood plains and transversely sloping flood plains.

Of particular interest was the significance of having sinuous and transversely sloping flood plains.

The results of this work indicated the following

- Side slopes of the main channel banks increase the conveyance of a meandering compound channel, at low over bank stages, by reducing energy losses in the inner flood plain flow.
- The James and Wark Method overestimated the flow in the outer flood plain zones due to the assumption of bed friction only losses.
- Flow structure in compound channels with sinuous and laterally sloping flood plains is completely different, compared to straight flood plains.

- Flow separation from the convex bends induced reverse flows on the flood plains and secondary circulation in the main channel opposite in sense to that of with straight flood plains, similar to that of in-bank flow.
- When the flood plain is sinuous, flow separation is the dominant source of energy loss.
- The overall resistance of a sinuous flood plain is reduced by transversely sloping flood plains, although for the cases investigated, it was always substantially greater than for the straight flood plain cases.

It should be noted that due to the sharp bends used in this study the sinuosity effects discussed may not be applicable to less sinuous geometries.

2.3.8 Series B Extension Programme

A criticism of the FCF Series B Experiments was that they only considered a limited range of geometies and conditions. The FCF Series B Extension Programme was carried out to rectify this situation. The experiments were performed at the University of Glasgow in collaboration with the Universities of Bristol and Aberdeen. Essentially, this involved the construction and testing of small-scale flumes.

As already mentioned the main purpose of this study was to investigate several parameters, which may influence river-flood plain interaction, that were not included in the initial FCF Series B experiments. The physical model that was constructed at the University of Glasgow by Mcleod (1998) was 8m long by 1.65m wide and had a maximum discharge rate of 60l/s and shown in Figure 2.10. The following parameters were investigated:

- The main channel side slope was varied
- Main channel and Flood Plain Roughness
- Bankfull Depth and Main Channel Aspect Ratio
- Cross-sectional shape
- Model Scale

The physical measurements taken included stage and discharge, flow visualisation and velocity measurements. A total of 30 different geometries were tested, details of which can be found in Ervine and Macleod (1993).

The findings discussed in this paper reinforce what had been observed in the initial FCF Series B Experiments but over a wider range of conditions. The authors experimental findings have been used as the basis for an Artificial Neural Network (ANN) for predicting discharge capacity in a meandering compound channel.

The apparatus constructed at Aberdeen University was 11m long by 1.2m wide and had a maximum discharge rate of 30l/s. Rameshwaran and Willetts (1997) varied the following 8 parameters that were found by Ervine et al (1993) to influence flow behaviour. These were, Sinuosity, Aspect Ratio of main channel, main channel side slope, cross-sectional shape, relative roughness, flood plain slope, meander belt width relative to flood plain width and relative overbank flow depth. The results have also been used as the basis of a new design method for estimating overall flow resistance. (see Rameshwarran and Willets (1997)).

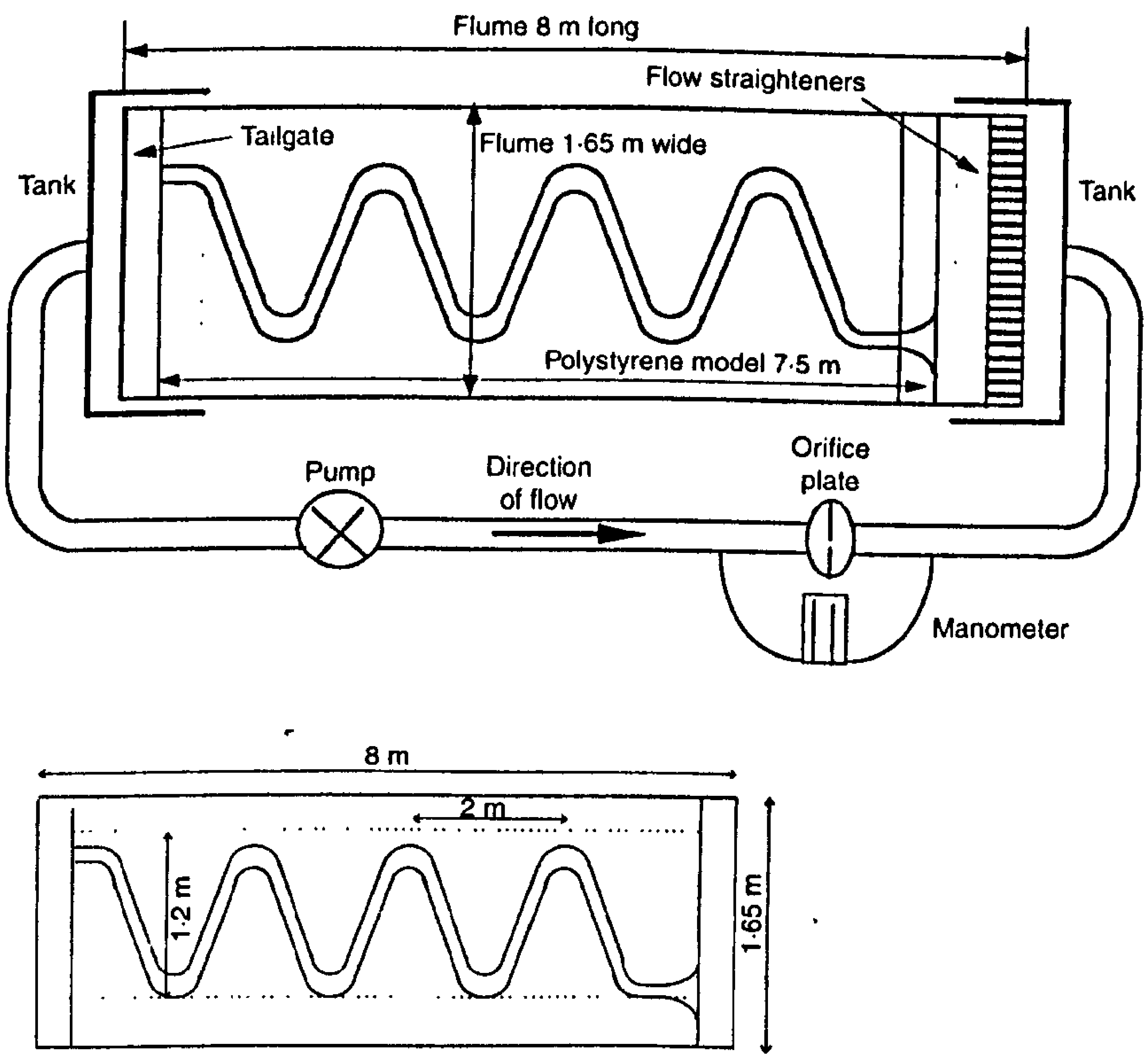


Figure 2.10 FCF Series B Extension Programme – Glasgow Flume

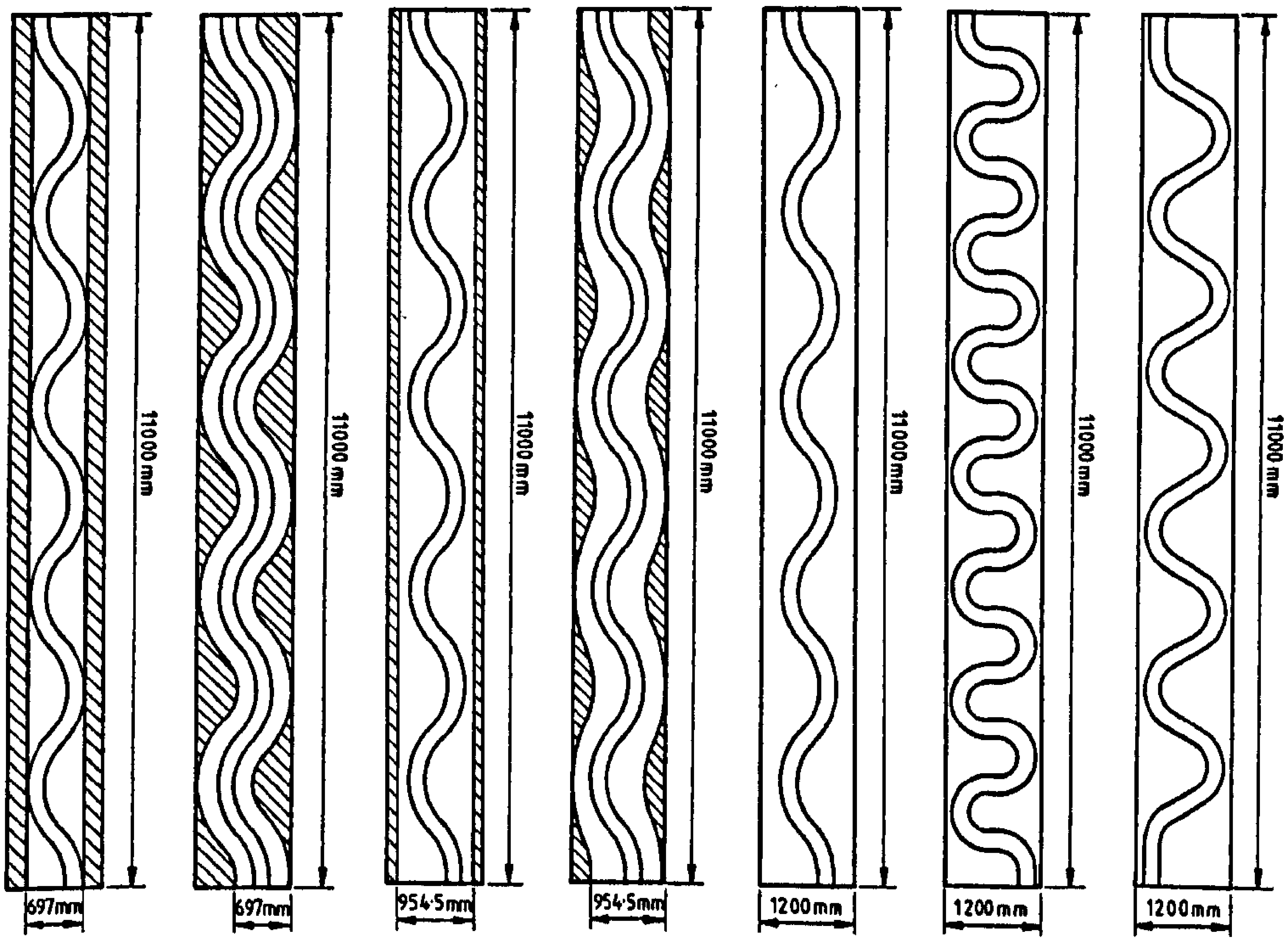


Figure 2.11 FCF Series B Extension Programme – Aberdeen Flume

For information on the Bristol Study see Wilson (1998). The main outcome of these experiments was the production of new discharge capacity methods which are reviewed in the following section.

2.4 Meandering Compound Channel Modelling Techniques

2.4.1 Toebes and Sooky (1967)

Essentially, these authors discovered that discharge, when calculated assuming only bed frictional losses, was over-predicted. This meant that all energy loss mechanisms were not being accounted for. In an attempt to allow for additional energy losses the wetted perimeter term (T) was increased for both flow regions. This term was increased until there was agreement between the predicted and measured discharges. 'T' was considered to be a complicated function of overbank flow depth, mean velocities in the two zones and the longitudinal slope.

2.4.2 James and Brown (1977)

James and Brown attempted to account for the additional energy losses associated with overbank flow, in both straight and meandering channels, by adjusting the Manning's 'n' parameter.

This meant that the value of 'n' accounted for both bed friction and secondary losses. The adjusted 'n' values were used in tandem with standard resistance formulae to obtain a value of discharge, for a given stage, assuming the cross-section were a single channel. The result was a formula that could be used to calculate a value of 'n' that would account for all losses and was dependent on relative flow depth and the ratio of floodplain width to main channel width. However, most of their experiments were concerned with straight compound channels with only a few focussed on meandering channels. As a result, it is unlikely that this method would be suited to a natural river application.

2.4.3 Yen and Yen (1983)

Yen and Yen (1983) also treated the cross-section as a single channel and the main channel was considered to be a resistance element. They proposed a Darcy-Weisbach type resistance coefficient to account for expansion and contraction losses induced by the main channel. The model did not account for flow in the main channel and is dependent on empirical information obtained for closed conduits which is unverified for open channels. This model would be unlikely to be suitable for incorporation to a one-dimensional model as it cannot account for main channel flow which is a significant proportion of natural river flow.

2.4.4 Ervine and Ellis (1987)

Ervine and Ellis (1987) produced a method for the prediction of stage discharge relationships where the cross-section is divided into three zones i.e.
Zone 1 : the main channel below bankfull, Zone 2 : the flood plain within the meander belt width and Zone 3 : the remaining area out with the meander belt. They identified the main sources of energy loss in each zone as follows:

Zone 1

- Friction on the wetted perimeter.
- Boundary resistance due to transverse shear and internal friction associated with secondary currents induced by the meander bends.
- The turbulent shear stress generated by the velocity difference between the main channel and the collinear component of the floodplain flow at the horizontal interface at bankfull level.

- Bed form resistance associated with the undulating riffle-pool sequence.

Zone 2

- Friction on the wetted perimeter
- expansion of flow as it enters the main channel
- contraction of flow as it re-enters the floodplain

Zone 3

- Bed Friction

Friction losses are estimated using the Darcy-Weisbach equation with the friction factor given by the Colebrook-White equation. Secondary Current losses are estimated using the method of Chang (1983) for fully developed circulation in wide, rectangular channels. Subsequent experimental observations have confirmed the early findings of Toebes and Sooky (1967) that the secondary circulation to be generally in opposite sense for overbank flows compared with inbank flows. This is because the horizontal shear layer at bankfull level, rather than centripetal acceleration drives it. Chang's method was derived for the inbank case and is therefore inappropriate for overbank cases. Ervine and Ellis account for the growth and decay of secondary currents by applying only half of the head loss predicted by Chang's 1983 model. Expansion losses for flood plain flow are determined by application of the force-momentum principle, and contraction losses by using loss coefficient values presented by Rouse (1950) and used by Yen and Yen (1983).

The method was applied to the laboratory data of the US Army Corps of Engineers, Vicksburg (1956) and Toebes and Sooky (1967) with reasonable accuracy. (See Figure 2.12)

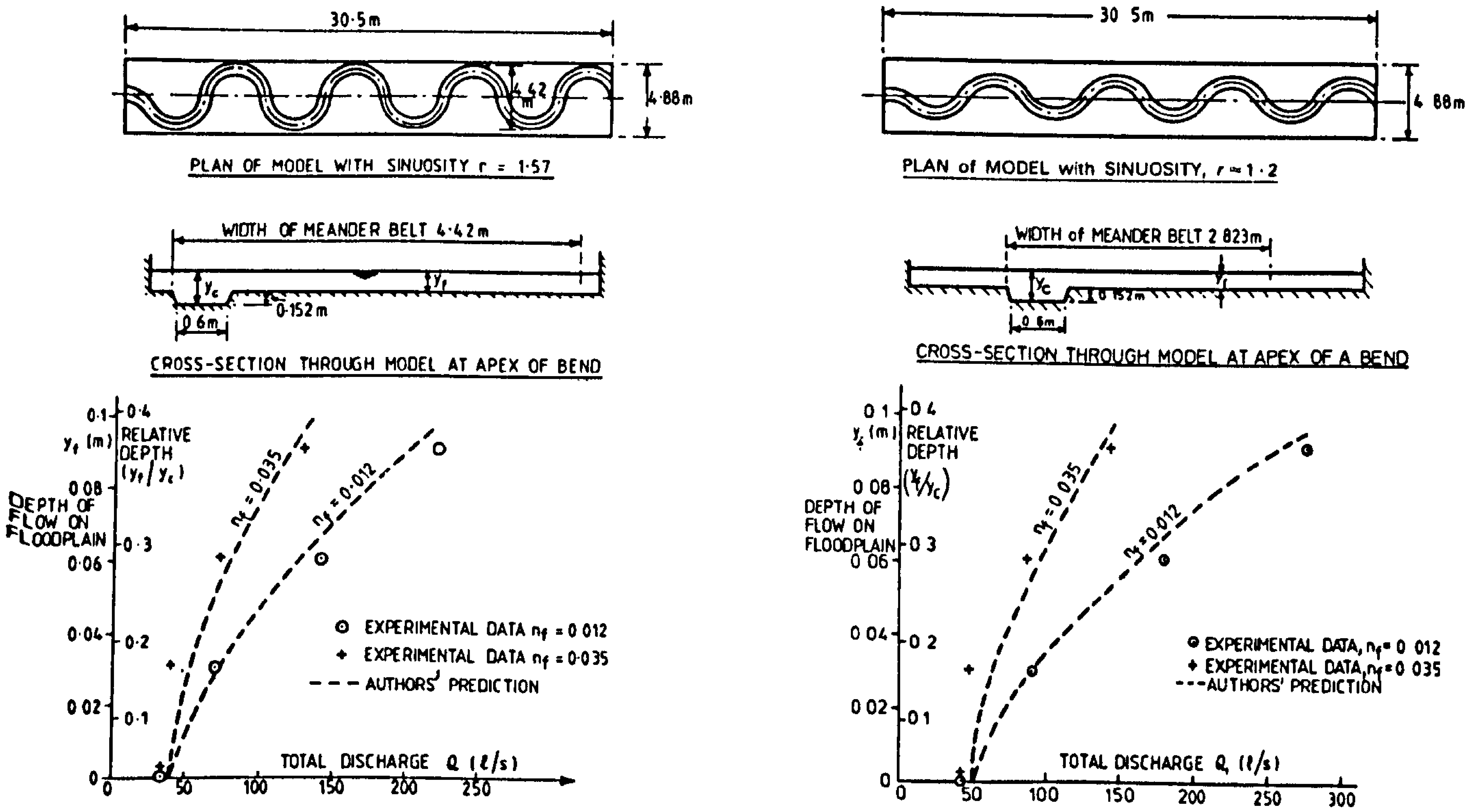


Figure 2.12 Comparison Between US Army Corps of Engineers, Vicksburg (1956) and Ervine and Ellis (1987)

2.4.5 James and Wark (1992)

In 1992, James and Wark developed this semi-physical / semi-empirical method for the calculation of stage discharge relationship. It was based on the Flood Channel Facility Series B experiments at HR Wallingford and can be considered a development of the Ervine and Ellis Method (1987).

The river cross-section is divided into four separate flow zones and there are empirical formulae to account for the various energy loss mechanisms in these zones. Figure 2.13 shows the James and Wark defined cross-section.

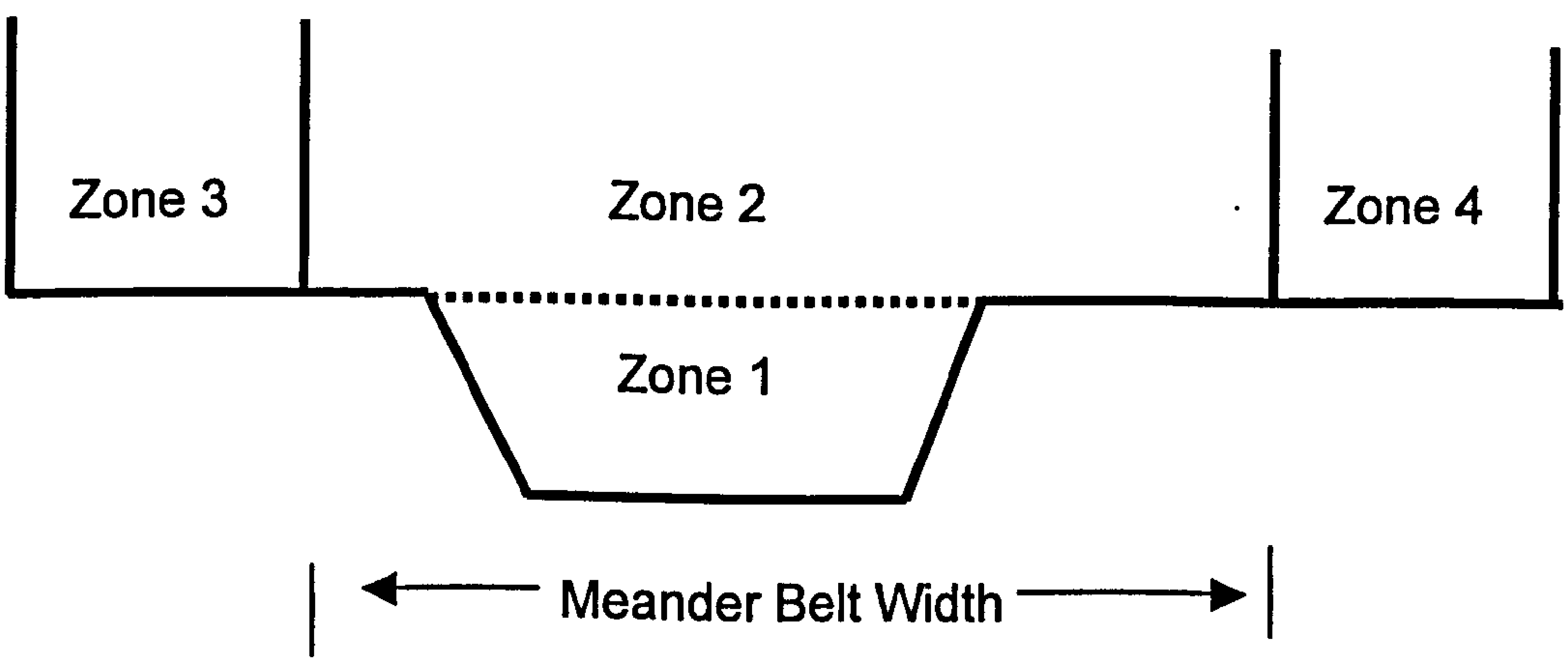


Figure 2.13 The James and Wark Method cross-section divisions

Zone 1 is the area up to bankfull
Zone 2 is the region above bankfull but within the meander belt width
Zone 3 is the region on the left outside the meander belt width
Zone 4 is the region on the right outside the meander belt width

The solution technique begins with a defined water level which is used to calculate zonal areas, wetted perimeters and Hydraulic Radii. For each zone a discharge is calculated and summed to give a total discharge for the defined water level. i.e.

$$Q_T = Q_1 + Q_2 + Q_3 + Q_4 \qquad (2.06)$$

Zone 1

In this zone, below bankfull, the sources of energy loss are bed friction, secondary circulations that are driven by the shear imposed by the flood plain flow and bulk exchange of water between the main channel and the flood plain. Due to the poor understanding of the flow mechanisms in this flow region an empirical approach has been used to calculate discharge. Essentially, the discharge in this zone is calculated using Manning's equation which includes meander bend losses in the term n'. This term is the basic Manning's 'n' adjusted using the Linearised Soil Conservation Service Method (LSCSM).

The LSCSM is used to adjust Manning's 'n' so that meander bend losses are accounted for. Having obtained this value of the bankfull discharge (Q_{bf}) it is then adjusted to account for the effects of overbank flow. The adjustment factor (Q_1') was derived from the FCF Series B Experiments and was found to depend on the following

- *The flood plain flow depth on the flood plain (Y2)*
- *The channel sinuosity*
- *The cross-section geometry*
- *Flood plain roughness*

After the adjustment is made using equation 2.07 the correct Zone 1 discharge with allowances for meander bends and overbank flow is obtained.

$$Q_1 = Q_{bf} * Q_1' \qquad (2.07)$$

Zone 2

The zone 2 adjusted discharge is calculated by the product of the area, above bankfull and within the meander belt width, and the velocity which is calculated equation 2.08.

$$V2 = \frac{(2gSoL)}{\frac{f2L}{(4R2)} + F1F2Ke} \qquad (2.08)$$

where g is gravitational acceleration, So is the flood plain gradient, L is the meander wavelength, f is the Darcy-Weisbach friction factor, R is the hydraulic radius, F1 is the factor for non-friction losses in zone 2 associated with main channel geometry and F2 is the factor for additional non-friction losses in zone 2 associated with main channel sinuosity. (see also Flow Chart 5 in Chapter 4)

It should be noted that the wetted perimeter term for this zone does not include the horizontal division at bankfull or the vertical divisions at the extremes of the meander belt width. The wetted perimeter for zone 2 is the total length of the wetted surface across the section less B(SIN-1). The empirical equations used in calculating V2 are required to account for flow expansion and contraction losses and other energy loss mechanisms.

Zones 3 and 4

Flow in the outer flood plain zones is assumed to be controlled by bed friction only. As a result the discharge in these areas can be calculated using Manning's equation. The James and Wark Method was applied to a range of experimental data which included the FCF Series B data which was used to derive the method. The results showed a significant improvement on the bed friction only method and other newly developed methods. It was also applied to field data from the River Roding and again showed a significant improvement in stage discharge prediction. The authors claim that in this application the bed friction only method over-predicted discharge by approximately 10% while the James and Wark method under-predicted discharge by 2%. This method has been adopted by the Environment Agency for England and Wales and is recommended for practical use.

2.4.6 Greenhill and Sellin (1993)

These authors set out to develop a "simple" method for predicting discharges in meandering compound channels. The study made use of the experimental results of the Flood Channel Facility Series B experiments as show in Figure 2.07. The dimensions are similar to those discussed earlier by Lorena (1992). Essentially, the method proposed was based on the Manning-Strickler equation and was applied to various cross-section sub-divisions. They began with the basic divided channel method of Lotter (1933) and gradually refined it until a method with suitable accuracy was derived.

The refinements were

- a horizontal division at bankfull to represent the shear layer caused by the movement of water leaving the flood plain and passing over the main channel.
- divisions to separate the meander belt width from the remainder of the flood plain
- Use of the main channel slope to calculate the discharge for the region of flow within the meander belt width
- Inclining the boundary between the inside and outside of the meander belt to account for the velocity difference

Five different models were tested and the results shown in Figure 2.14. Method 5 was the most accurate and had a percentage error of ±3.5% for a discharge of $1.1 m^3/s$.

At lower depths the accuracy, as shown in Figure 2.14, was very good. Method 5 applied Manning's equation in each zone using the main channel slope in zones 1 and 2 and the flood plain slope in zones 3 and 4. The division lines separating the meander belt and the rest of the flood plain were inclined at 45°. The method was applied to other data sets with variable success. However, the authors established that the model was inaccurate for low overbank depths and geometries with a very wide main channel. The method was found to be accurate to 2% on the FCF 60 degree meander geometry for discharge between 0.05m^3/s and 0.8m^3/s. It should be noted that it was applied to a limited range of conditions.

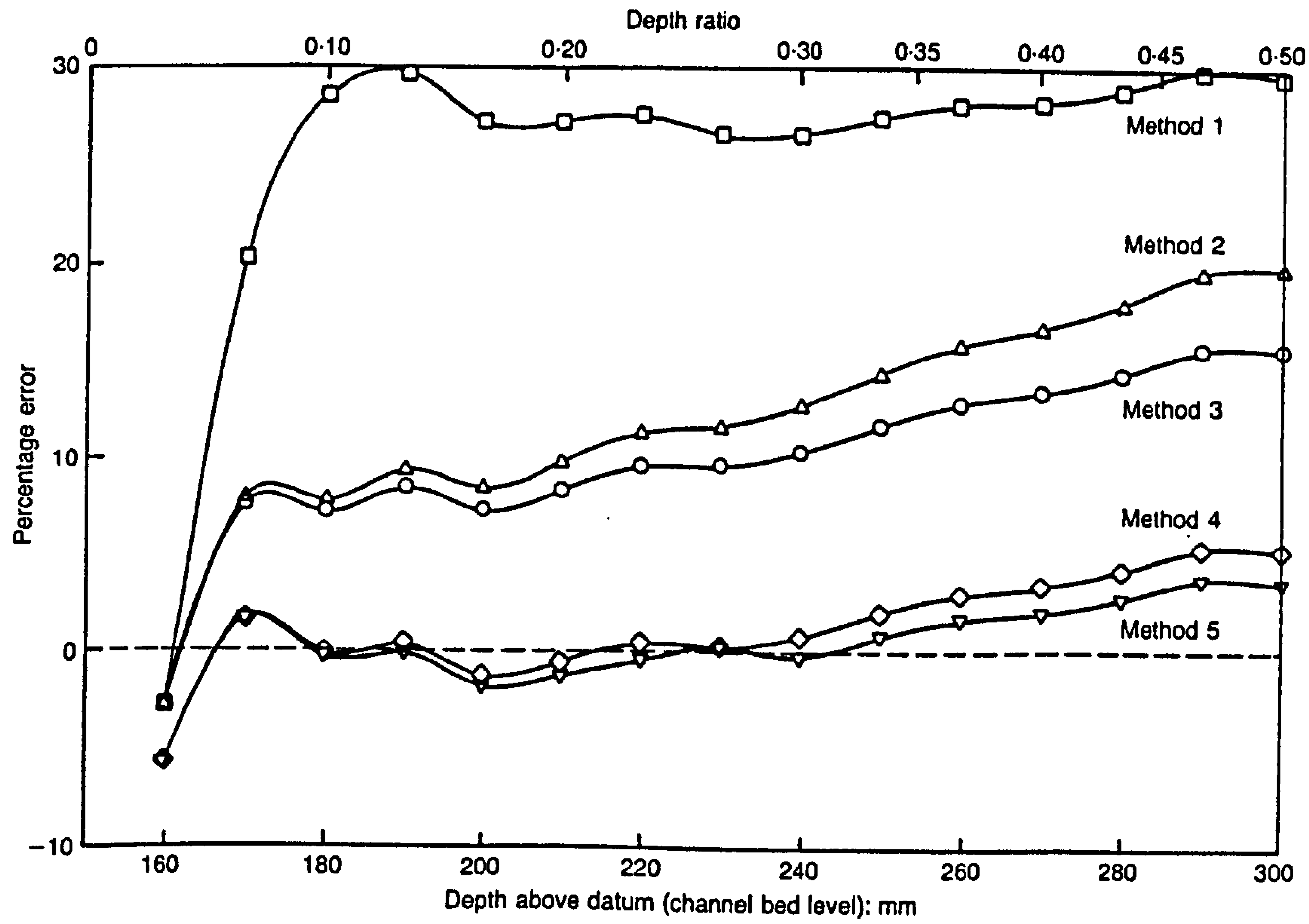


Figure 2.14 Percentage Error in Discharge Prediction (Greenhill and Sellin (1993))

The method developed was reasonably accurate, for the data that it was developed from, and was indeed simple to use and could be considered as the basis of a more theoretically correct method for use in a one-dimensional river model.

2.4.7 Muto (1997)

Having performed small scale laboratory experiments on meandering channels with sinuosities of 1.093, 1.370 and 1.571, Muto (1997) analysed three existing methods for stage-discharge prediction, namely, the Divided Channel Method, Ervine and Ellis (1987) and James and Wark (1992). Muto concluded that the James and Wark method was the most accurate for his experimental data. Muto also proposed a new method, based on Ervine and Ellis (1987), which introduced several new parameters and took the effects of secondary flow and turbulence into account.

It gave reasonable predictions of both zonal and total discharge for the geometries investigated in this study.

2.4.8 Willets and Rameshwarran (1998)

Willets and Rameshwarran (1998) developed a method for estimating the overall flow resistance based on the FCF Series B extension programme results. The method presented was based on the resistance coefficient relationship for a two-dimensional open channel.

$$\frac{1}{\sqrt{f}} = 2\log\frac{R}{Ks} + 2.23 \qquad \textbf{(2.09)}$$

where f is the Darcy-Weisbach friction factor, K_s is the equivalent roughness size and R is the hydraulic radius.

The approach accounted for many relevant geometrical parameters and scale effects and performed with a reasonable level of accuracy. The channel system was treated as a single channel. Domains were defined, in the first of which viscosity was found to be influential but not in the second.

Domain 2 was considered to be roughness dominated. The method calculated the flow resistance in each of these domains. The true potential of this method has not been practically demonstrated as it has only been applied to laboratory data.

2.4.9 Koopaei and Ervine (2000)

Koopaei and Ervine (2000) developed a method for the analysis and design of a compound channel and was applicable to both straight and meandering cases. This particular study had gathered together the best available laboratory and field data for both straight and meandering compound channels.

In addition, they assessed all the main analysis methods, such as Ackers Method, James and Wark Method and The Lateral Distribution Method. The aim of doing so was to produce a new method that combined the best attributes of the existing techniques yet improved the existing situation. It was also important that the new method was accurate at both laboratory and field scale.

The new method reported is based on the work of Shiono and Knight (1989) and Wark, Samuels and Ervine (1990), referred to earlier as the Lateral Distribution Method. The novelty of the method is that it includes the influence of secondary currents and is applicable to both straight and meandering channels. Of particular note is that the method has been applied to a broad range of small scale, large scale and field data.

The authors concluded that in situations where secondary currents are dominant the method will give improved predictions of depth-averaged velocity when compared with other methods.

The various methods that have been reviewed are either one-dimensional or quasi two-dimensional however, some recent work has focussed on full three-dimensional modelling, Manson and Pender (1994) and Morvan and Pender (2000).

Morvan and Pender (2000) presented a fully three-dimensional numerical model of the Flood Channel Facility Series B experiment B23. The predictions of the 3D model are compared with the observed velocity and turbulence measurements.
At the time of writing the authors were in the process of applying their 3D model to 1km reaches of the River Nith, River Severn and River Ribble which will be of significant interest to engineering practice.

Currently the practicalities of 3D modelling are not economic. For example, in order to model the 50m long FCF Series B experiments in full 3D, the run time was approximately 48 hours. For practical river modelling these methods are not currently applicable and are limited to 'special sites of interest'. (See Samuels, May and Spaliviero (1998))

2.5 Field Studies

The various discharge capacity methods that have been almost exclusively applied to laboratory data. The following section reviews the few field scale studies that have been reported. Interestingly, these applications comment on the need for such methods to be incorporated into a one-dimensional river modelling package. The dearth of field studies is due to the combination of expense in gathering data and the uncertainties that exist in the accuracy of field data.

2.5.1 River Severn

Gauging station data was gathered by Ramsbottom (1989) from a selection of UK rivers. One of the best sites used was on the River Severn at Montford Bridge. It should be noted that this gauging station site is of straight plan form. Wark (1993) has applied his version of the Lateral Distribution Method to this site (and the other sites identified by Ramsbottom (1989)) and compared it against other methods that were available.

2.5.2 River Main

Lynesss, Myers and Wark (1997) discussed the application of the Lateral Distribution Method to a reach of the River Main in County Antrim. The reach used was reconstructed as a two-stage compact compound channel comprising main channel, flood-plain berms and flood banks as shown in Figure 2.15.

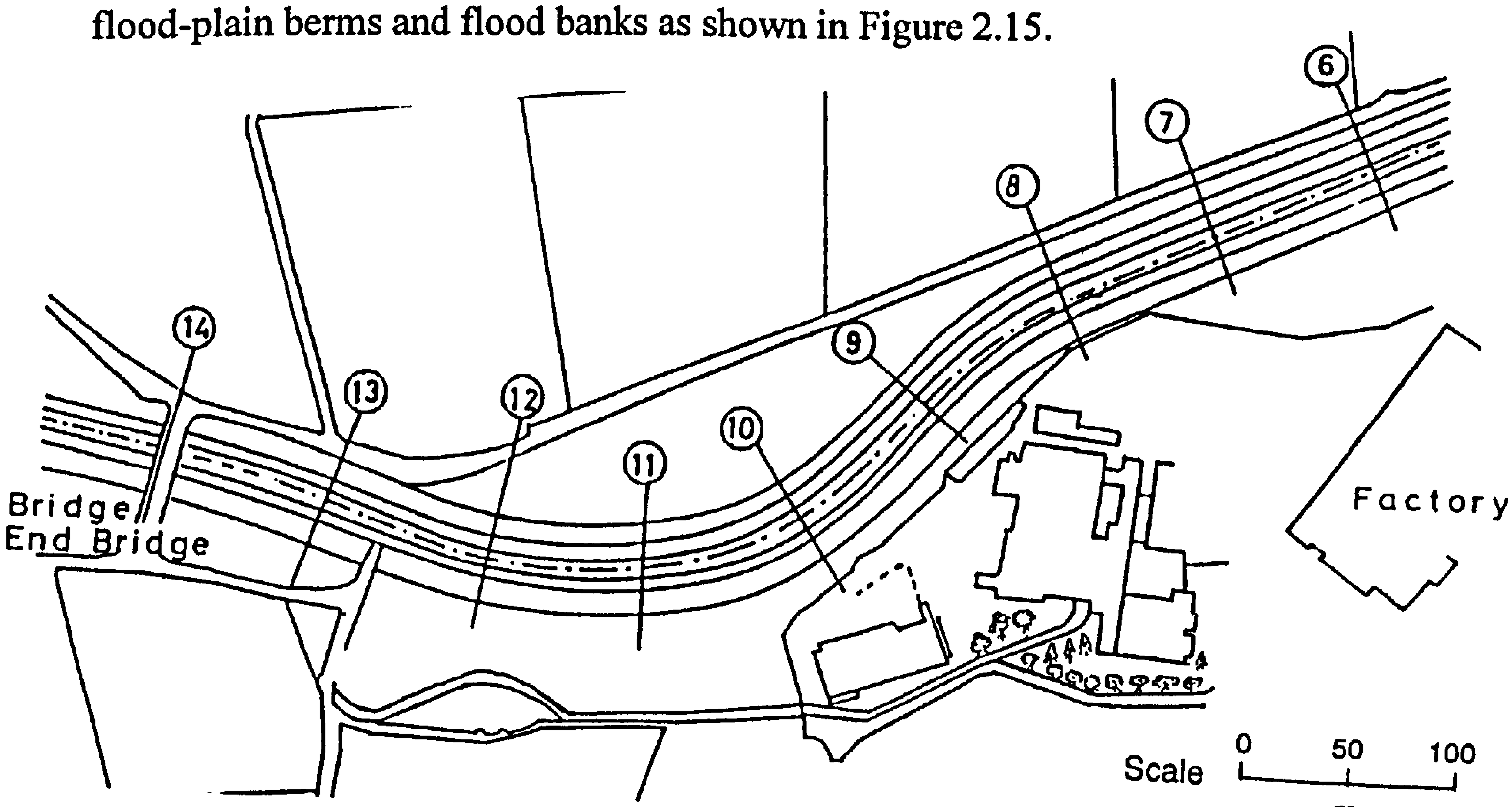


Figure 2.15 Plan view of the experimental reach of the River Main

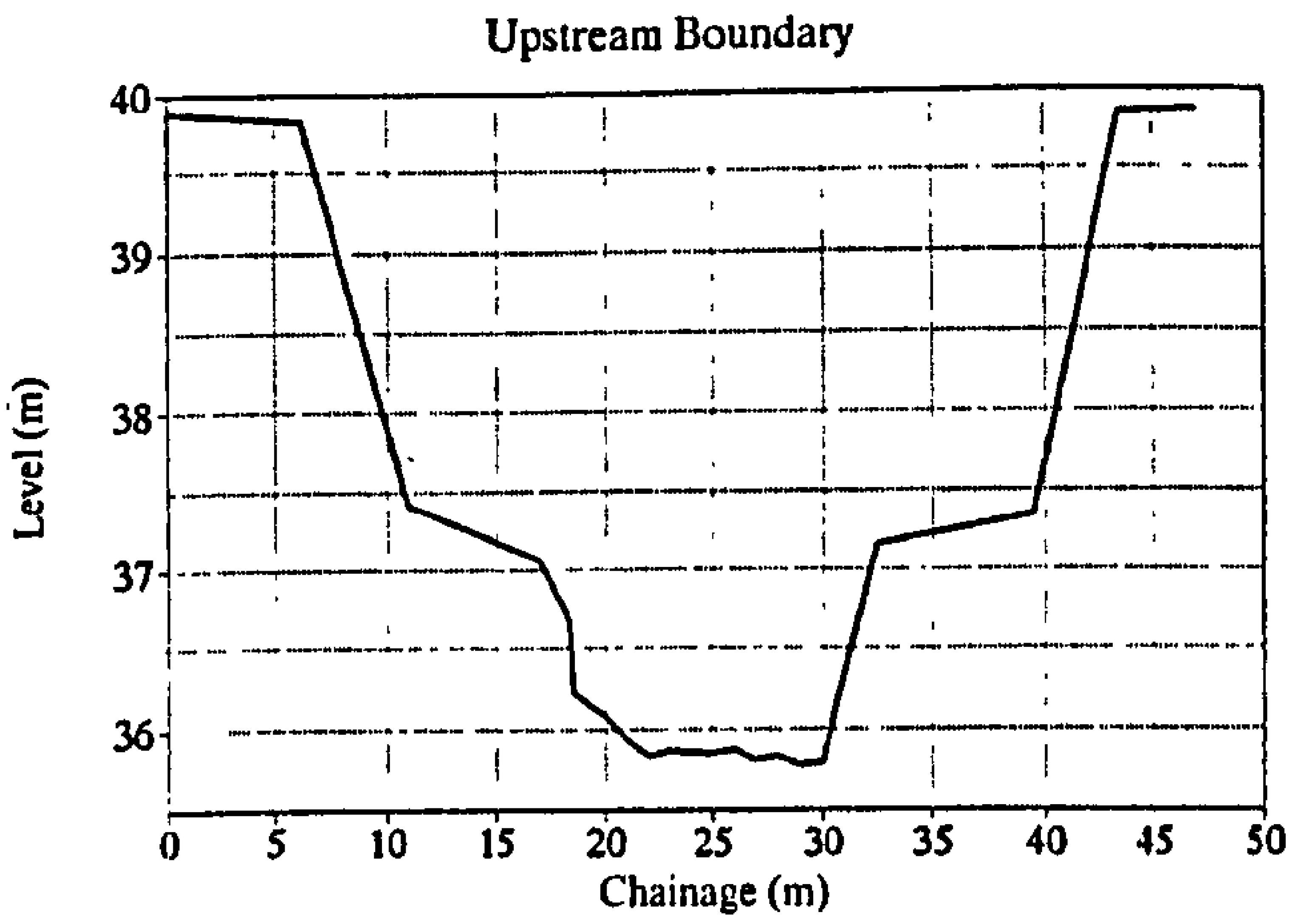


Figure 2.16 Typical cross-section of Section 14 the River Main

The reach as shown in Figure 2.15 is 800m long and has been the subject of detailed observations. The surveyed cross-sections were located at intervals of 100m. Flow gauging has allowed the computation of stage-discharge curves at the upstream end (section 14) and the downstream end (section 6) of the reach. For section 14 the gauging also produced the distribution of depth averaged velocities and unit width discharges for a range of overbank flow depths.

The authors then applied various conveyance calculations to model the observed data. Namely, the Single Channel Method (SCM), the Divided Channel Method (DCM) and the Lateral Distribution Method (LDM) as developed by Wark et al (1990). The authors showed that a reasonable level of accuracy can be obtained when using the LDM for the estimation of energy and momentum coefficients α and β respectively, and conveyance. The LDM was found to lie between the SCM and DCM conveyance estimates for relative depths greater than 0.3. At very high depths the LDM conveyance estimate tended to that of the SCM which is appropriate as the channel will start to act as a single flow.

It was suggested that this technique (LDM) could be used as a conveyance table pre-processor if incorporated in a one-dimensional river model. This is certainly plausible however it needs to be tested over a significantly longer reach that 800m. By incorporating this method into a one-dimensional model it could be observed how improved the water level prediction would be.

2.5.3 The River Blackwater

The River Blackwater is a doubly meandering channel consisting of a lower channel with a sinuosity of 1.18 and an upper channel with a sinuosity of 1.05. The study reach is 520 m long and has been gauged at the upstream end and also comprises five pressure transducers. River level and discharge are recorded continuously every 15 minutes. This reach has been specially constructed as a two-stage channel following the building of a new trunk road and consequent relocation of part of the river. The cross-sections are almost perfectly trapezoidal. Further information on this location can be found in Wilson (1998).

2.5.4 River Dane - *Ervine and Macleod (1999)*

These authors made an attempt to use the James and Wark method in tandem with a one dimensional steady state river model. Interestingly this tool was applied to a 5km reach of the River Dane in Cheshire. The reach of the River Dane used in this study is highly meandering and well suited to the James and Wark method. The newly developed model was a steady state one-dimensional river model combined with the James and Wark channel flood plain interaction methods.

A pre-processing software was used to calculate stage conveyance relationships at each surveyed cross-section. This information was then utilised in an explicit computation of water surface profile, based on the energy balance equation.

This new "tool" was then validated against Flood Channel Facility Series B Data and applied to two different natural flood events. The results of the field study were compared with the industry standard river modelling package MIKE 11. This comparison revealed that the new method, which accounts for interaction losses, under predicted water levels in 14 out of 30 cross-section locations.

In theory you would expect the water level using the new method to be higher than a method that simply applies bed friction. This implies that, at the cross-sections where the water level is under predicted, the stage conveyance relationship is incorrect. An additional limitation of this study was that there was only one location where observed data was available. For further information on the River Dane, see Chapter

5 of this thesis. A more robust 1D model containing the James and Wark Method is discussed in Forbes and Pender (2000).

2.5.5 River Roding

A comprehensive set of data was collected on the River Roding in Oxfordshire which is of meandering plan form. This data set, studied by Sellin et al (1985-89), can be considered among the best available field data for meandering compound channel flow. Full details of both the field and laboratory measurements taken in this study can be found in Sellin and Giles (1988) and Sellin et al (1990).

The field study involved monitoring a stretch of the River Roding which had been reformed as a two-stage channel as part of flood alleviation scheme. The existing flood plains were excavated to form berms while the main channel remained untouched with a bankfull capacity of $3m^3/s$. The resulting channel (shown in Figure 2.10) had a low flow channel which meanders within the berm limits with a sinuosity of 1.38. James and Wark (1992) applied their stage discharge method to this field data and predicted that the discharge would be over predicted by 9.5% if bed friction only was assumed.

The results shown in Table 2.01 show that the James and Wark method is performing accurately in a natural situation. However, it should be noted that Mcleod (1998) could not verify the Manning's 'n' value used by James and Wark (1992).

It is proposed that further analysis of this study should be carried out to ascertain the true performance of new conveyance calculation techniques.

Case	P2	P2	M2	M2
Method	Mean Error %	St. Deviation	Mean Error %	St. Deviation
Friction Only	9.5	9.0	7.3	8.6
James and Wark	-2.0	1.7	-2.2	3.2

Table 2.01 Errors in Predicting Overbank Discharges: Roding Study

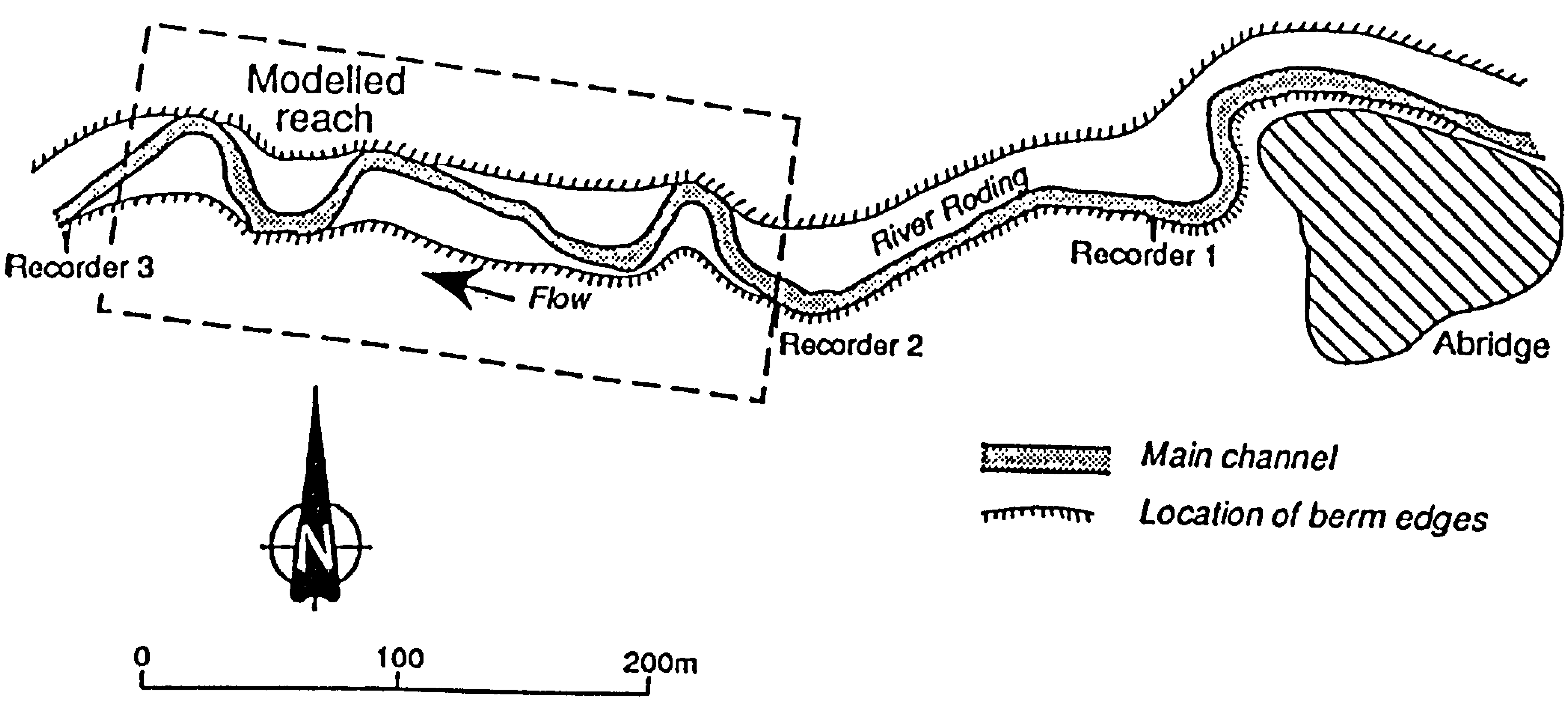


Figure 2.17 Study Reach of The River Roding

The aim of this review of literature has been to outline they key developments in the field of compound channel research. It is not intended to be an exhaustive review but rather to inform on current best practice. It is clear that there is still not a generally accepted or robust method for calculating the discharge capacity for the meandering case in particular. It is also evident that despite extensive research and modelling applications, they have tended to be at laboratory scale. There are only a few field studies reported in the literature and this is a feature that needs to be addressed. Also, the main use of a new conveyance method would be best utilised within a one-dimensional river model, where it would be used to calculate conveyance. This is beginning to occur, Ervine and Macleod (1999), and would be the most practically useful way of utilising the various methods.

As a consequence, it is the purpose of this thesis to develop a robust one-dimensional river model that includes an enhanced conveyance calculation.

Chapter 3 Numerical River Modelling Theory

3.0 Numerical River Modelling

In trying to simulate a river in flood the main aim is to accurately predict the changes in water level and discharge as a flood wave passes through the river channel. By simulating these changes we can then confidently design flood protection works or assess flood risk for flood plain development or construction of bridges.

In order to simulate a river in flood, an engineer's main tool is that of a one-dimensional model. These models essentially predict discharge and flood levels for given meteorological events, and can indicate the extent of flooding. The 1D model has been used since the late 1970's and are now commercially available and robust. They are essential tools in water resource management.

One of the current industry leading models is called ISIS and has been used extensively in this research project. ISIS was developed by Halcrow Consulting Engineers in partnership with HR Wallingford. The background to this modelling package and the information required to run it will be outlined in the following section. It should be noted that the following procedure is similar to that of other commercial models.

3.1 Model Data Requirements

In order to construct a numerical river model certain fundamental information is required. Firstly, the river should be surveyed at locations where there is geometrical change or a structure. This is normally decided by undertaking a walking tour of the river reach. The interval between cross-sections should not be excessive and as a general guide, no longer than 250m. Samuels (1989) provides some guidance on locating cross-sections on rivers where the hydraulic conditions are not interrupted by hydraulic structures such as bridges or weirs. The following equation gives a typical backwater length (L).

$$L = \frac{0.7D}{S_0} \qquad \textbf{(3.01)}$$

where, L = backwater length, D = water depth and S_0 = river bed slope

In order to obtain a suitable distance between cross-sections the following equation is used

$$\Delta X = \frac{0.15D}{S_0} \tag{3.02}$$

Having decided on the survey locations a full topographical survey should be carried out to provide cross-section information. An example cross-section is shown from the River Kelvin.

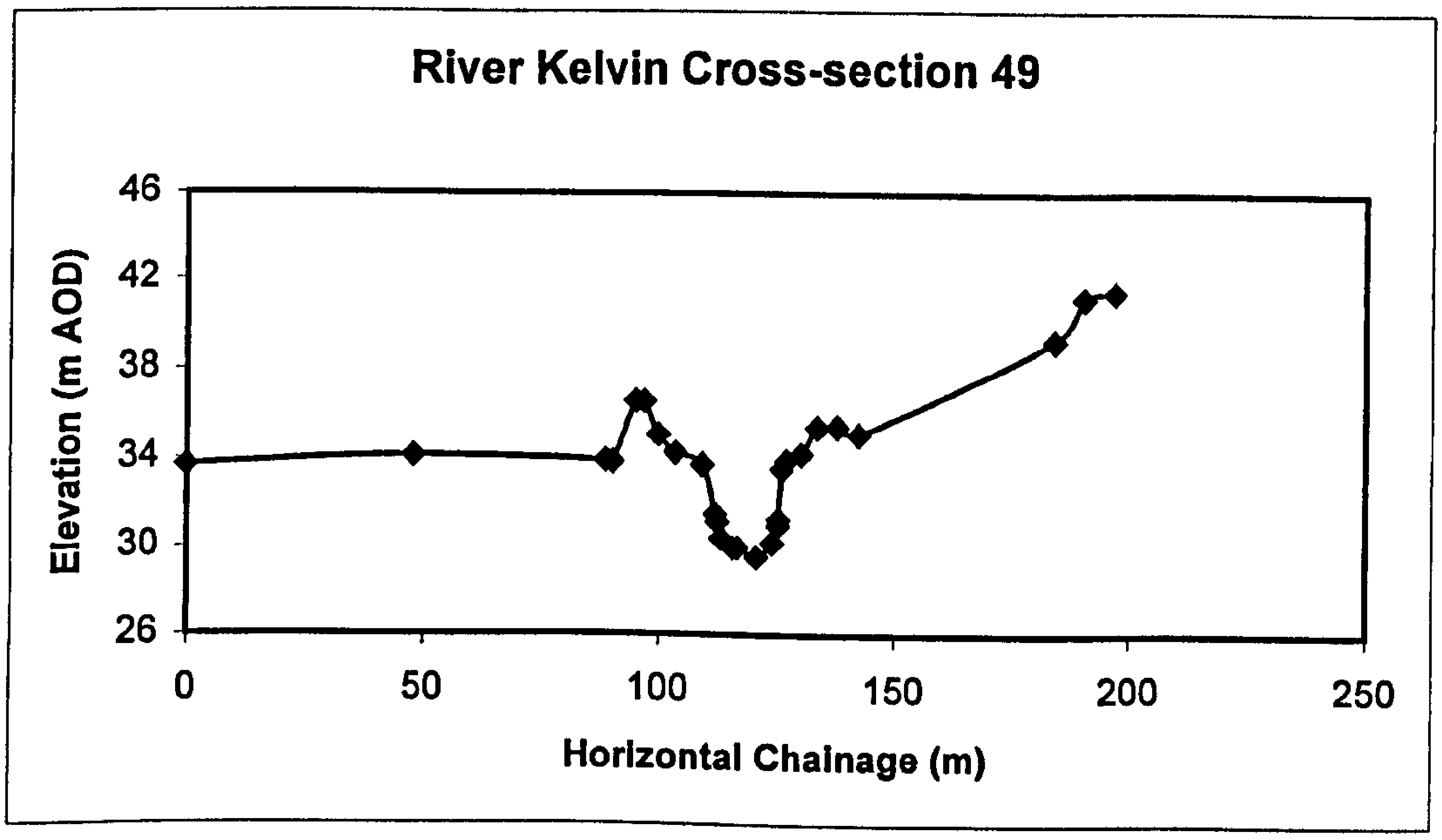


Figure 3.01 Example of a Surveyed Cross-Section

The data obtained from the survey is then used directly to construct the numerical model as shown in Figure 3.02.

The first stage in constructing a numerical model is to represent the river geometry with numbers. This is achieved by surveying the river cross-sections at selected locations, as shown in Figure 3.02.

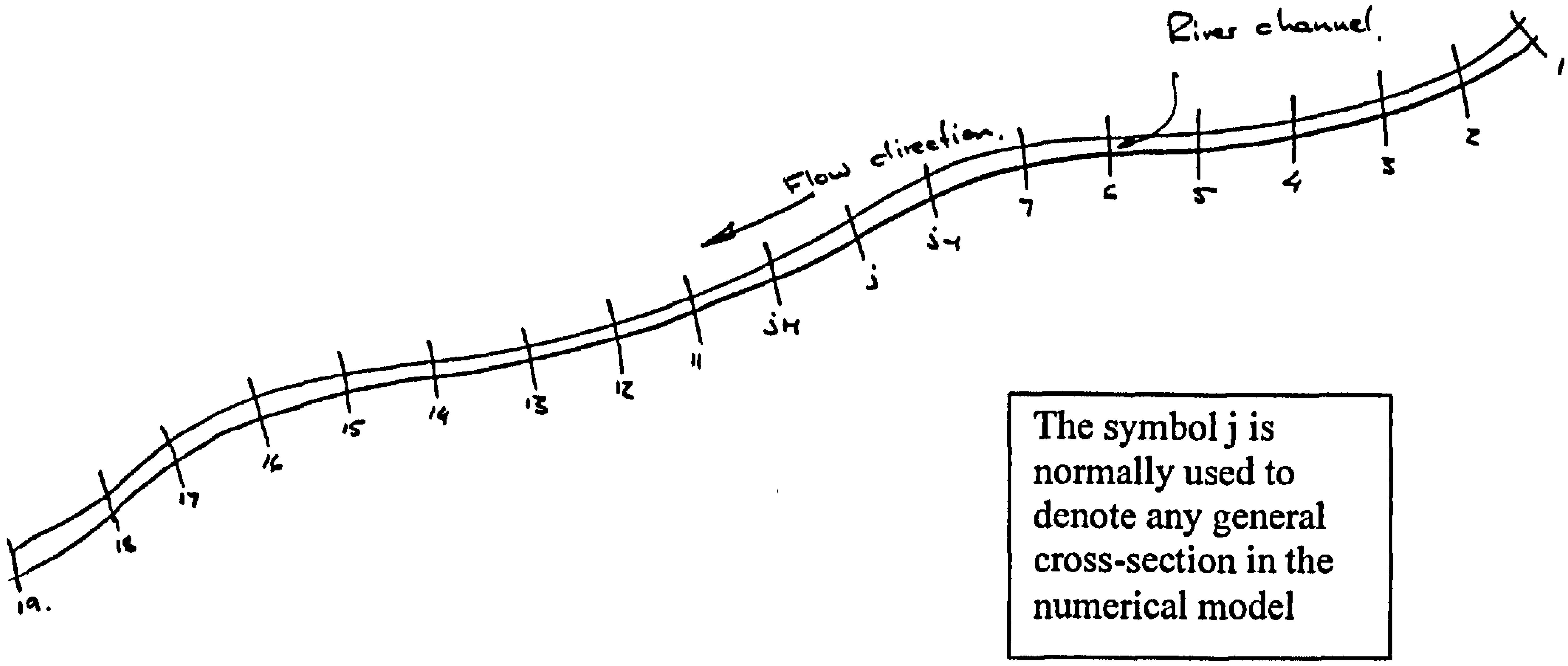


Figure 3.02 Numerical River Model

From the survey data it is possible to calculate the hydraulic properties of the river channel. Namely, Area, Wetted Perimeter, Hydraulic Radius, Top Breadth, Conveyance and Momentum correction coefficient.

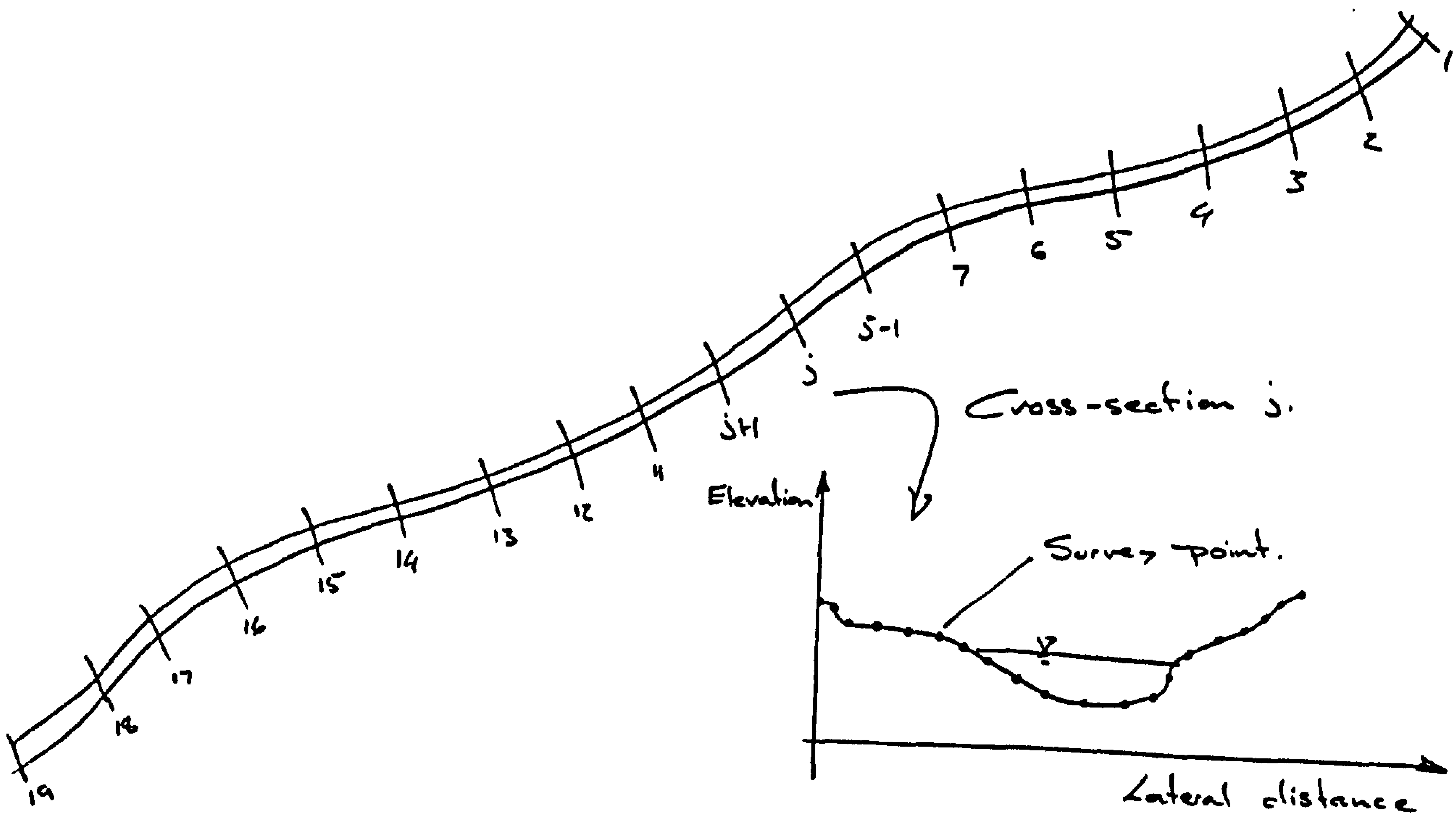


Figure 3.03 Numerical River Model

In addition to surveying the channel cross-sections it is important to survey the chainage of the cross-sections starting at section 1.

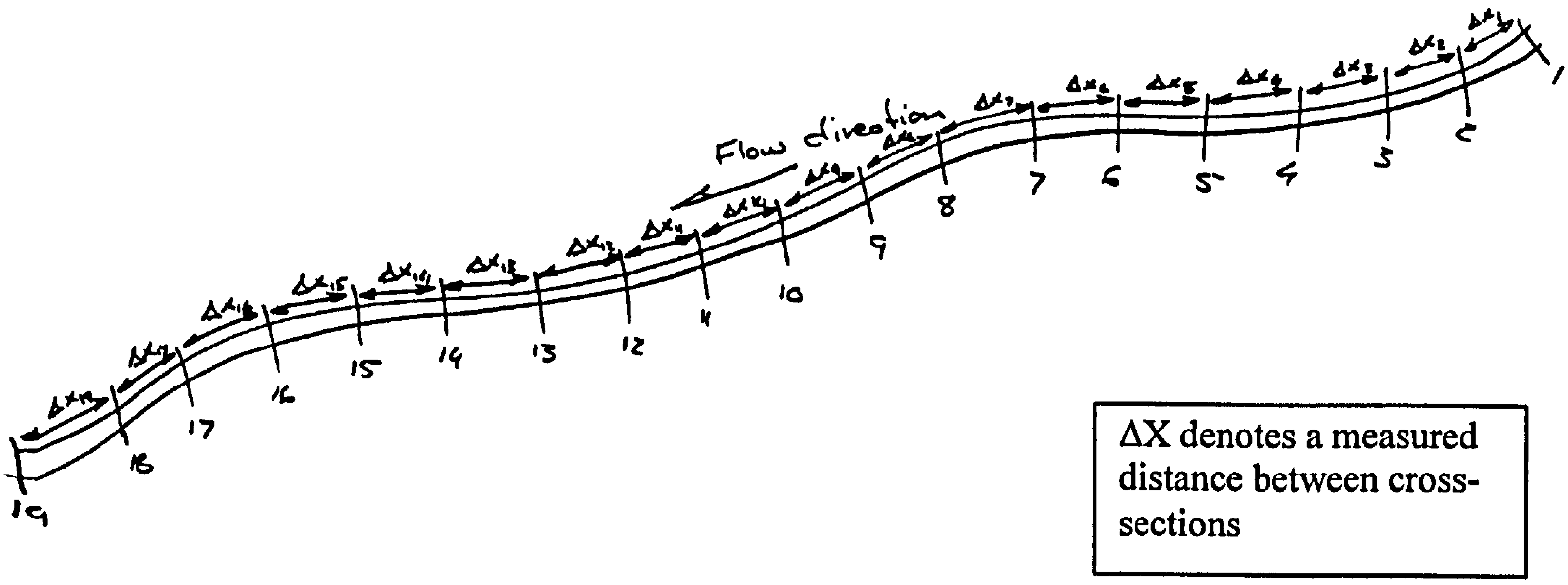


Figure 3.04 Numerical River Model

The physical river data is modelled by the computer as shown in Figure 3.05

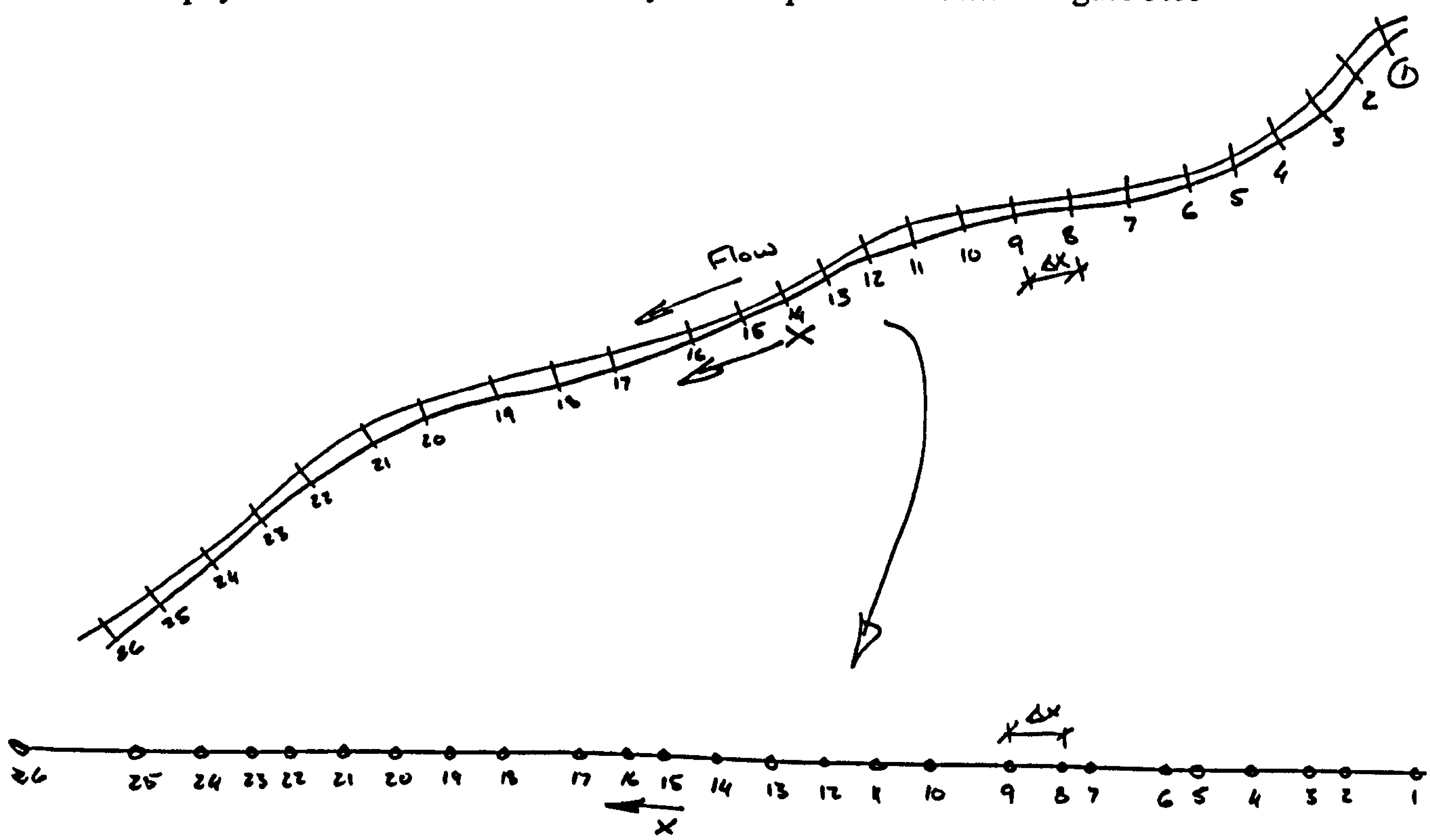


Figure 3.05 Numerical River Model

Figure 3.06 shows the solution technique of the numerical model, known as a finite difference solution. Figure 3.07 shows the outcome of the solution technique i.e. estimates of Q and H as each model cross-section.

Each horizontal line represents a time at which the flow and water level will be evaluated

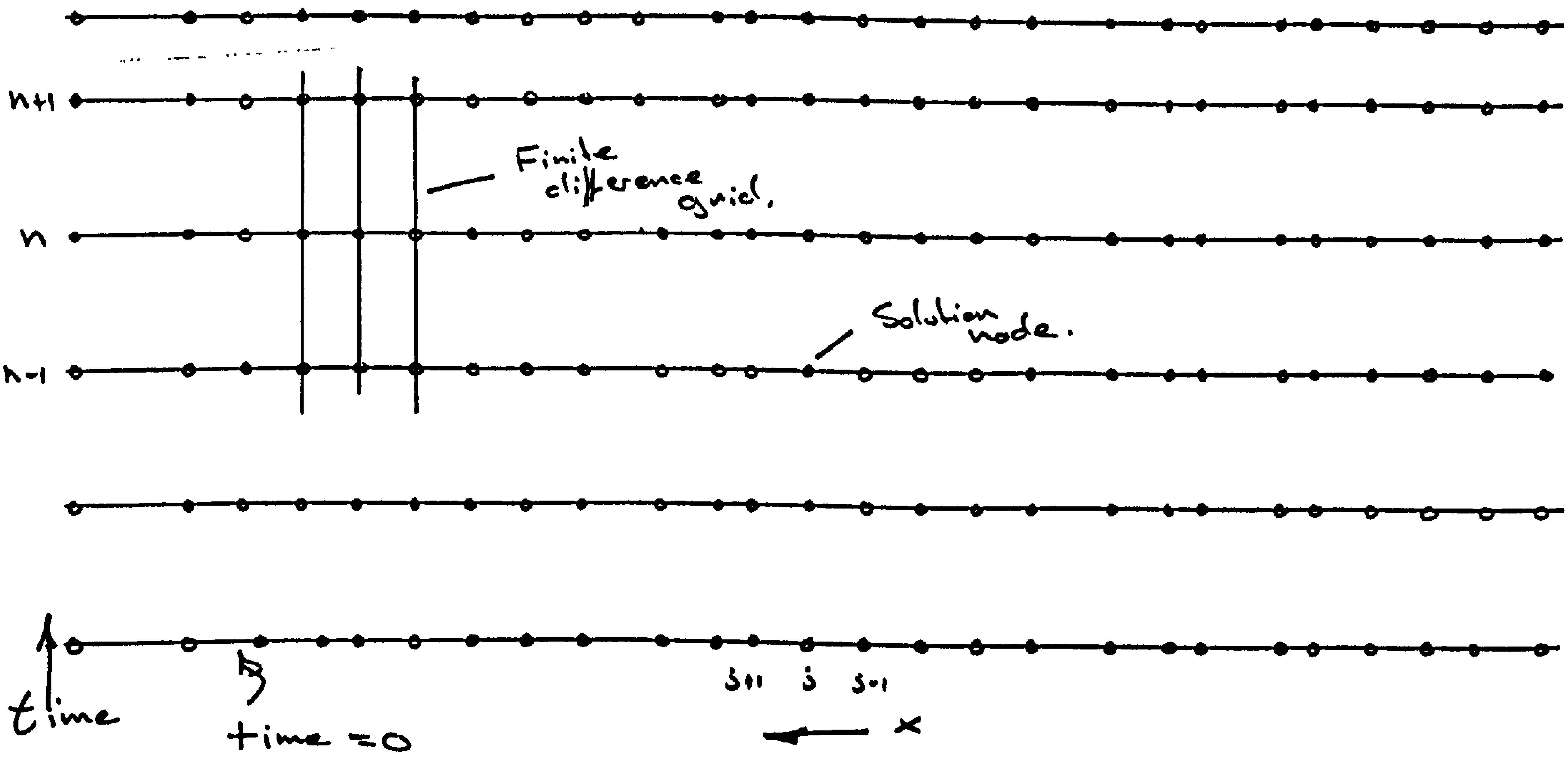
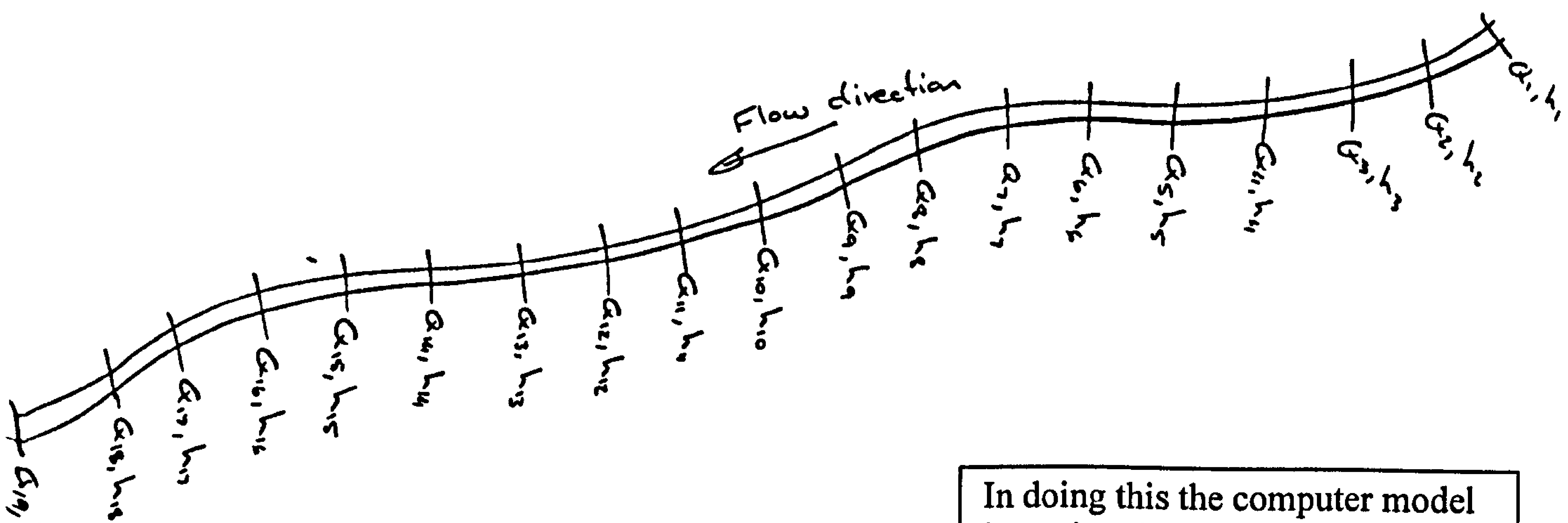


Figure 3.06 Numerical River Model



In doing this the computer model is tracing the evolution of water surface profile along the river length through time

Figure 3.07 Numerical River Model

3.1.1 Boundary Conditions

In order to calculate the flow and stage at each cross-section during the passage of a flood wave it is necessary to provide the computer model with information on conditions at the upstream and downstream boundaries. This information informs the model what is occurring outwith the model area.

At the upstream end the boundary conditions can be either an inflow hydrograph or a stage hydrograph. At the downstream end the possible boundary conditions can be an outflow hydrograph, a stage hydrograph or a rating curve. The boundary conditions mentioned are normally measured at river gauging stations.

Having obtained a detailed survey of the river and estimated the boundary conditions at the upstream and downstream end the computational analysis can proceed. Two different forms of analysis can be performed by a one-dimensional model namely a steady analysis and a n unsteady analysis.

3.1.2 Boundary Layer Roughness

An estimate of boundary roughness is required at each data line in the cross-sectional data file i.e. where there is a pair of co-ordinate points. The estimate takes the form of Manning's roughness coefficient 'n'. Chow (1959) and Henderson (1966) provide tables of estimates that are commonly used for reference.

The previous section has indicated the data that is required and how it is used by a numerical model. The following section derives the fundamental one-dimensional equations and discusses the finite difference solution scheme.

3.2 Steady Flow Analysis

A steady flow analysis is often carried out in engineering practice to predict maximum flood levels. These are of particular interest when designing flood protection works.

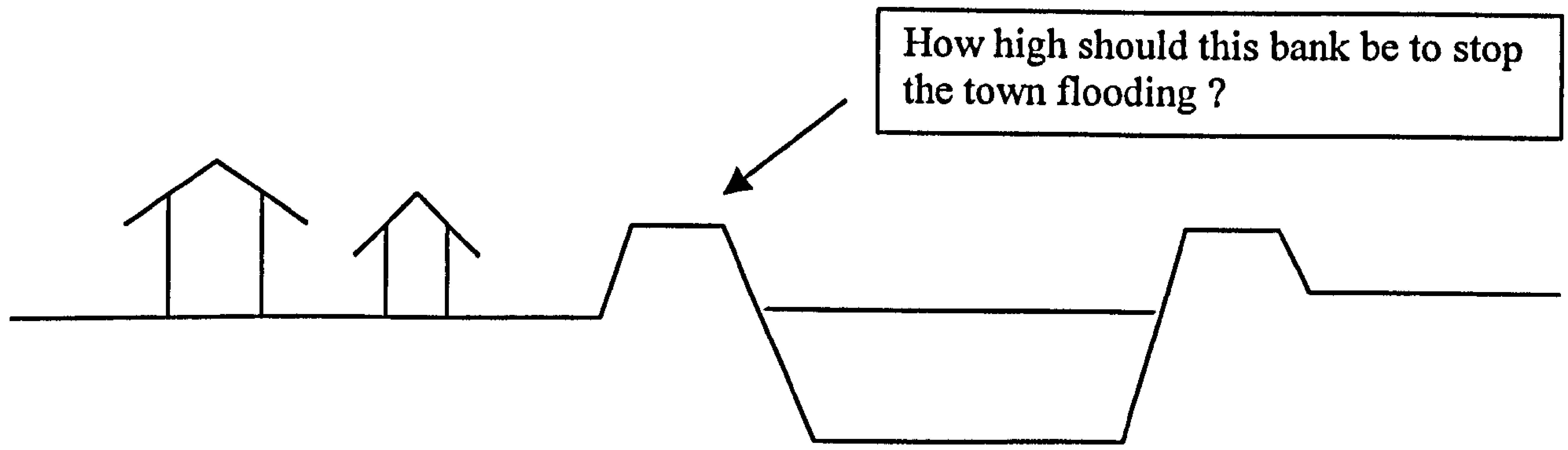


Figure 3.08 Example of Steady Flow Modelling

The difference between steady state and unsteady state in modelling terms is in the boundary conditions. A steady flow model requires an estimate of peak flow at the upstream boundary and an estimate of maximum water level at the downstream end. These values are normally related to a return period i.e. the 100 year return period flow and corresponding 100 year return period water level.

The value of flow is assumed to travel through each model cross-section, which his unrealistic. In reality, at any cross-section, the flow varies with time and in a steady analysis only the maximum value is used and applied for an infinite duration.

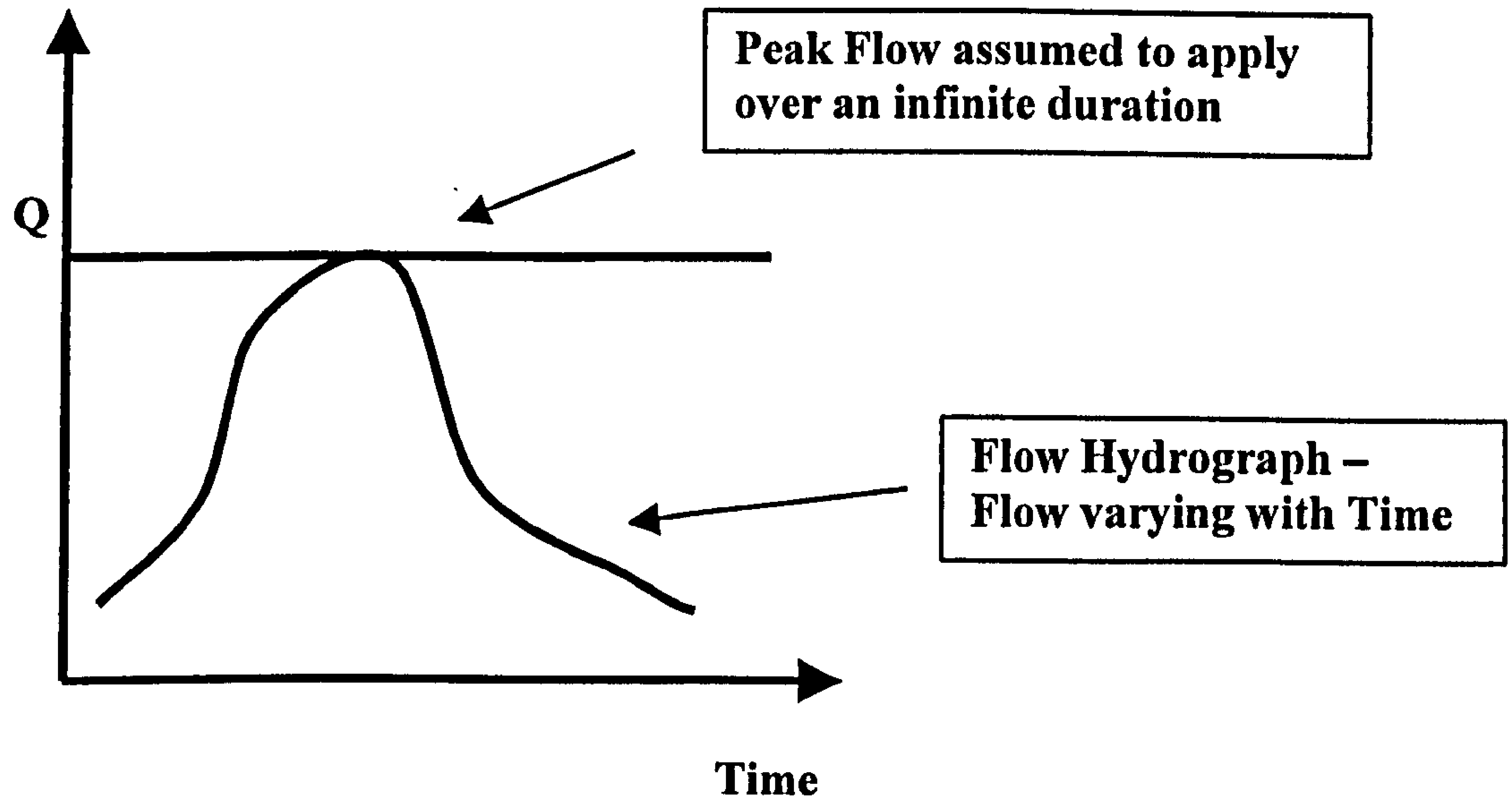


Figure 3.09 Flow Hydrographs for Steady and Unsteady Analysis

A steady flow analysis will result in a conservative approach as there is no variation in flow. In order to observe the variation of flow with time an unsteady analysis is required.

3.2.1 Unsteady Flow Analysis

An unsteady analysis requires information on the variation of flow with time, normally at the upstream end. This is in the form of a flow hydrograph and is shown in Figure 3.09. At the downstream boundary a rating curve or stage discharge relationship is desirable. This form of analysis is considered more accurate, than a steady state analysis, as it dynamic and simulates the actual passage of a flood wave.

3.3 Numerical Derivation of The Saint Venant Equations

Introduction

In order to derive the one-dimensional flow equations certain assumptions are made.

- Across the section, velocity is uniform and the water level is horizontal.
- Streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic.
- Effects due to boundary friction and turbulence can be accounted for through the application of resistance laws.
- The average slope of the channel bed is small enough such that the cosine of the angle it makes with the horizontal may be replaced by unity.

Assuming that density is constant, one-dimensional open channel flow may be described by two dependent variables: Flow (Q) and water level (h). The calculation of two unknowns requires two equations, each of which must represent a physical law. Fluid dynamics offers three such equations, namely: the conservation of mass, momentum and energy. The mass-momentum couple of conservation laws can be applied to both continuous and discontinuous flow variables Abbott (1970), and will therefore be the basis of the succeeding derivation.

3.3.1 St Venant Equations

Conservation of Mass

Consider an infinitesimal control element of unit width during a time *dt*, as shown in Figure 3.10.

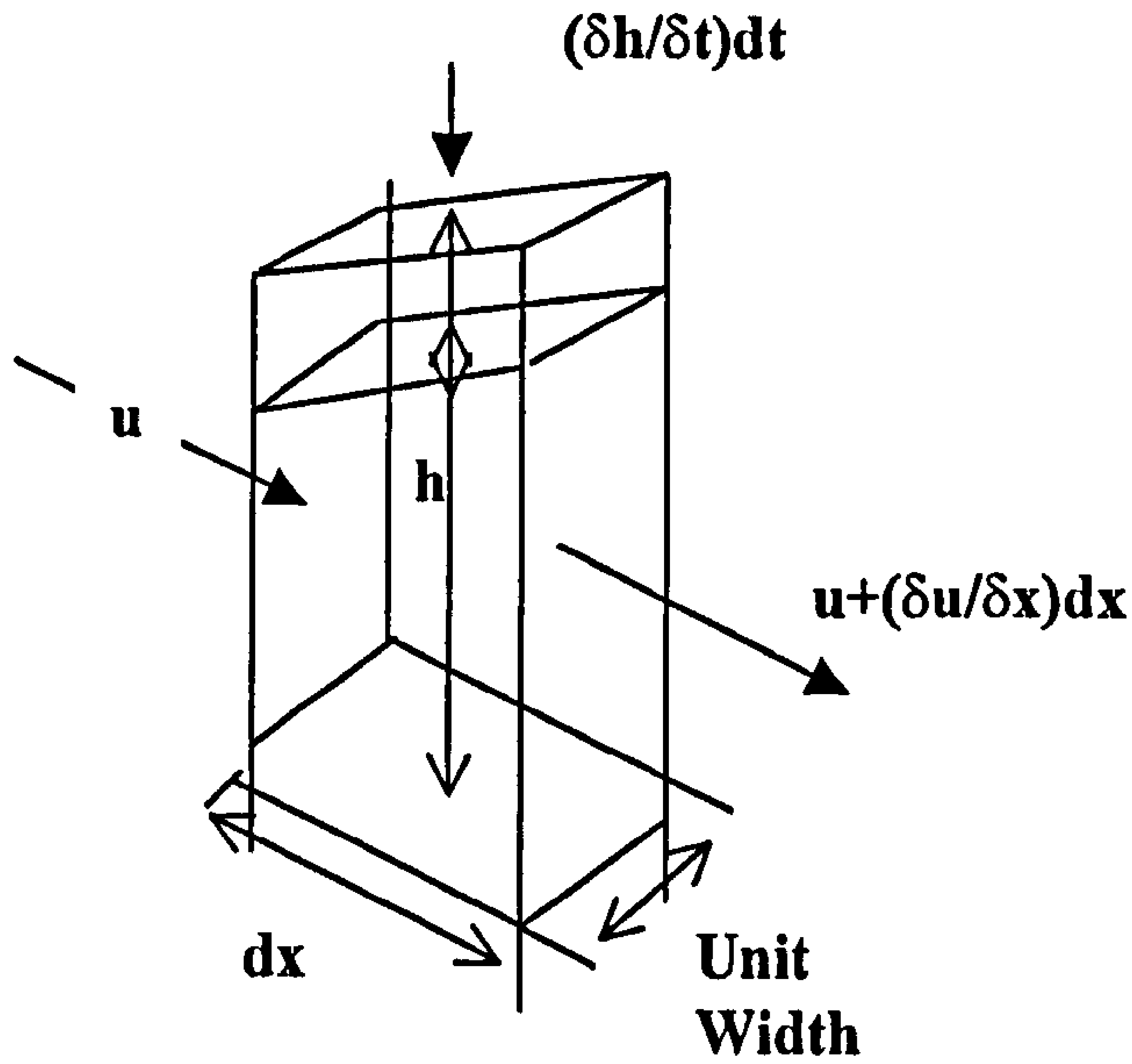


Figure 3.10 - Control Element

The accumulation of mass of the element during time *dt* is

$$dx\frac{\partial}{\partial t}(\rho h)dt$$

The mass inflow into the element in the time *dt* is

$$\rho uhdt$$

The mass outflow from the element in the same time is

$$\left[\rho uh+\frac{\partial}{\partial x}(\rho uh)dx\right]dt \qquad \textbf{(3.03)}$$

Hence,

$$dx\frac{\partial}{\partial t}(\rho h)dt = \rho uhdt - \left[\rho uh + \frac{\partial}{\partial x}(\rho uh)dx\right]dt \qquad \textbf{(3.04)}$$

Simplifying, to give the *mass* conservation law

$$\frac{\partial}{\partial x}(\rho uh) + \frac{\partial}{\partial t}(\rho h) = 0 \qquad \textbf{(3.05)}$$

For an incompressible fluid ρ is constant, hence this reduces to the *volume* conservation law

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x}(uh) = 0 \qquad \textbf{(3.06)}$$

3.3.2 Conservation of Momentum

The accumulation of momentum in the element over time *dt* is

$$dx\frac{\partial}{\partial t}(\rho uh)dt \qquad \textbf{(3.07)}$$

The impulse-momentum applied to and convected into the control element of Figure 2.01 in time *dt*, is the momentum flux density multiplied by *dt*

$$\rho\left(u^2h + \frac{gh^2}{2}\right)dt \qquad \textbf{(3.08)}$$

Convected *out* of the element in time *dt* is

$$\left[\rho\left(u^2h + \frac{gh^2}{2}\right) + \frac{\partial}{\partial x}\rho\left(u^2h + \frac{gh^2}{2}\right)dx\right]dt \qquad \textbf{(3.09)}$$

Equating the net impulse-momentum inflow to the momentum accumulation gives the momentum conservation law

$$\frac{\partial}{\partial t}(\rho uh)+\frac{\partial}{\partial x}\left[\rho\left(u^2h+\frac{gh^2}{2}\right)\right]=0 \qquad \textbf{(3.10)}$$

Again, for constant density (incompressible fluid)

$$\frac{\partial}{\partial t}(uh)+\frac{\partial}{\partial x}\left[u^2h+\frac{gh^2}{2}\right]=0 \qquad \textbf{(3.11)}$$

3.3.3 Bed Slope

For a very small bed slope (i < 1:1000), it is convenient to take an x-axis along the sloping bed and to measure water depth orthogonal to such as in Figure 3.11.

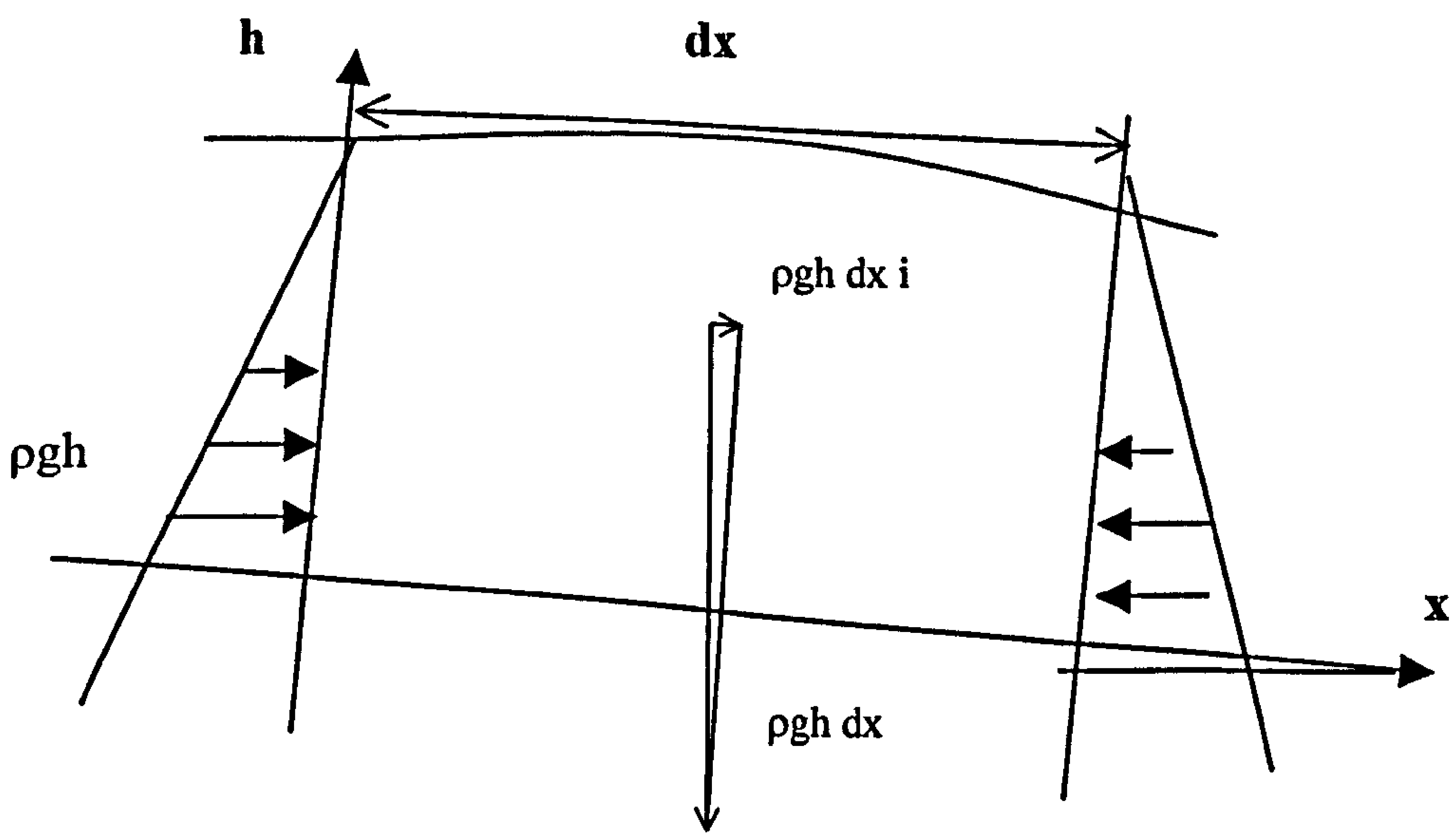


Figure 3.11 Influence of Bed Slope

Due to the small slope, the pressure exerted on the control element can be assumed to be hydrostatic with a maximum of ρgh at the channel bed. The mass equation remains unchanged while the momentum equation becomes

$$\frac{\partial}{\partial t}(uh) + \frac{\partial}{\partial x}\left(u^2 h + \frac{gh^2}{2}\right) - ghi = 0 \tag{3.12}$$

3.3.4 General Cross-section

The equations can be further modified so that they describe the flow in a natural river channel. That is, they may be extended to take account of variable cross-sectional geometry.

Taking out a small element from such a river section (Figure 3.12), a velocity distribution coefficient β may be applied to the depth-averaged velocity $\bar{u}$ to provide correction to the convected momentum mass.

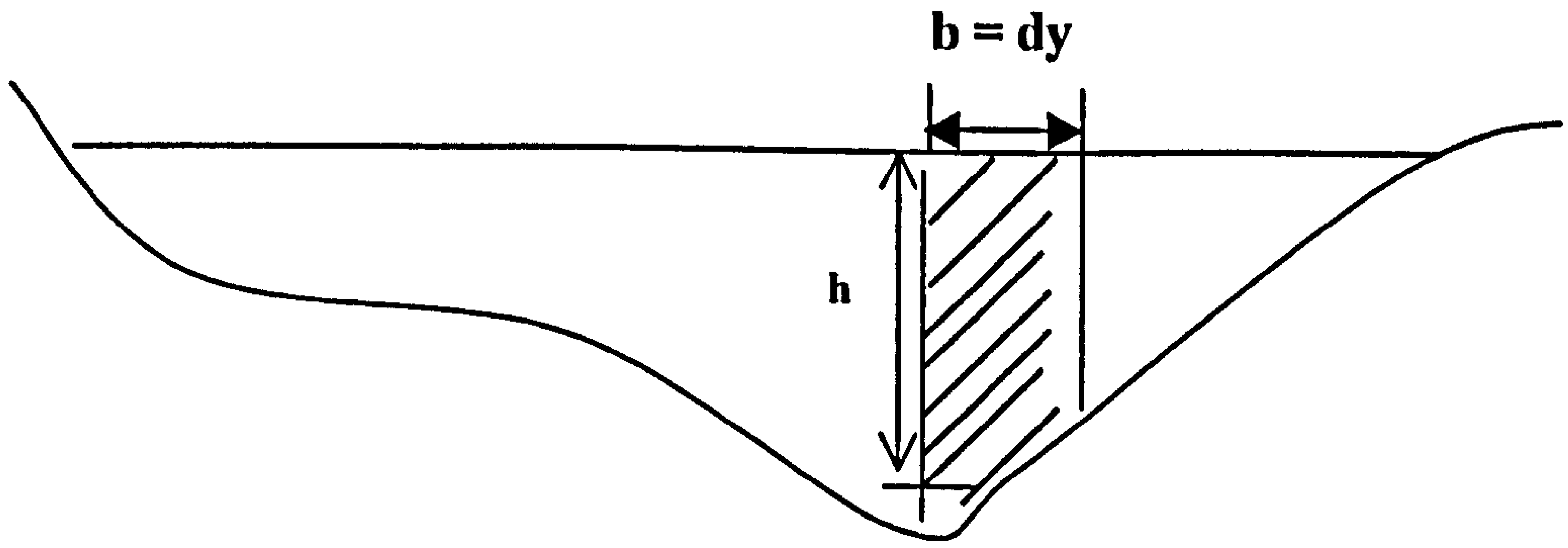


Figure 3.12 General Cross-section

Mass and momentum conservation laws for the above element are then

$$\frac{\partial}{\partial t}(h'b) + \frac{\partial}{\partial x}(h'b\bar{u}) = 0 \tag{3.13}$$

$$\frac{\partial}{\partial t}(h'b\bar{u}) + \frac{\partial}{\partial x}\left(\beta' h' b\bar{u}^2 + \frac{gb(h')^2}{2}\right) + \frac{\partial b}{\partial x}\left(\frac{g(h')^2}{2} - gh'bi_b\right) = 0 \tag{3.14}$$

Where h' is the depth and i_b is the local bed slope. Differentiating out in equation (3.13), it is found that the impulse terms with $\frac{\partial b}{\partial x}$ cancel out and equations (3.13) and (3.14) reduce to

$$\frac{\partial}{\partial t}(h'b)+\frac{\partial}{\partial x}(h'b\bar{u})=0 \tag{3.15}$$

$$\frac{\partial}{\partial t}(h'b\bar{u})+\frac{\partial}{\partial x}(\beta'h'b\bar{u}^2)+gh'b\frac{\partial h}{\partial x}=0 \tag{3.16}$$

Where h is now the surface level above some arbitrary horizontal datum.
If it is assumed that

- $\frac{\partial h}{\partial x}$ is constant across the width of the channel,

- there is no net loss or gain of mass or momentum from one element to another, and

then an integration can be carried out across the section giving

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0 \tag{3.17}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right)+gA\frac{\partial h}{\partial x}=0 \tag{3.18}$$

where

$A=\int h'dy$

$Q=\int h'\bar{u}dy=\bar{u}A$

$\beta=\frac{A}{Q^2}\int_A u^2dA$ (Boussinesq velocity distribution coefficient)

3.3.5 Bed Shear Stress

From figure 2.3, the bed force resisting the flow down the channel is

$$\tau_o P dx \qquad (3.19)$$

where P is the wetted perimeter. For *non-uniform* flow situations, Henderson and French prove that

$$\tau_o = \rho g \frac{A}{P} S_f \qquad (3.20)$$

Equating these two equations yields an expression for the bed force resisting flow down the channel

$$\rho g A S_f dx \qquad (3.21)$$

where S_f is the gradient of the total energy line also known as the friction slope. Inserting into equation 2.2.9

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gAS_f = 0 \qquad (3.22)$$

3.3.6 Evaluation of the Friction Slope

The friction slope S_f can be evaluated using any of the steady state friction laws

$$Q = K\sqrt{S_f} \qquad (3.23)$$

where K is the channel conveyance.

Rearranging

$$S_f = \frac{Q^2}{K^2} \qquad \textbf{(3.24)}$$

From Manning's equation

$$Q = \frac{AR^{2/3}}{n}\sqrt{S_f} \qquad \text{where } K = \frac{AR^{2/3}}{n} \qquad \textbf{(3.25)}$$

In any model based on the St Venant hypotheses, the energy slope is assumed to be representative of the reach between two computational points. However, as the conveyances K are properties of the cross-sections at either end of the reach, the problem arises as to how to interpolate between them in expressing S_f. Different methods of calculating the friction slope term can be found in Lyness and Myers (1994).

3.3.6.1 Conveyance

Conveyance is defined by Chow (1959) as a measure of the carrying capacity of a channel section, since it is directly proportional to Q. The estimate of conveyance is assumed to include account for energy losses that are occurring in a system. However, all energy losses are 'lumped' in to the be roughness parameter 'n'. This is generally accepted practice in industry despite being fundamentally flawed.

$$K = \frac{AR^{2/3}}{n}$$

3.3.6.2 Beta Parameter

Beta is used in the conservation of momentum equation of the St. Venant equations and as it is normally close to unity, it can be generally assumed that β=1 for practical situations, Lyness, Myers and Wark 1997. In fact, the ISIS Direct Steady Method assumes β=1 while the unsteady solution calculates Beta using the following relationship.

$$\beta = \frac{\sum_{i=1}^{J} A_i}{\left(\sum_{i=1}^{J} K_i\right)^2} \sum_{i=1}^{J}\left(\frac{K_i^2}{A_i}\right) \qquad \textbf{(3.27)}$$

As can be seen from Equation 3.27 the Beta parameter is calculated from the geometrical information.

3.3.6.3 Cross-Sections

Each model cross-section is assumed to be representative of the distance between three consecutive cross-sections.

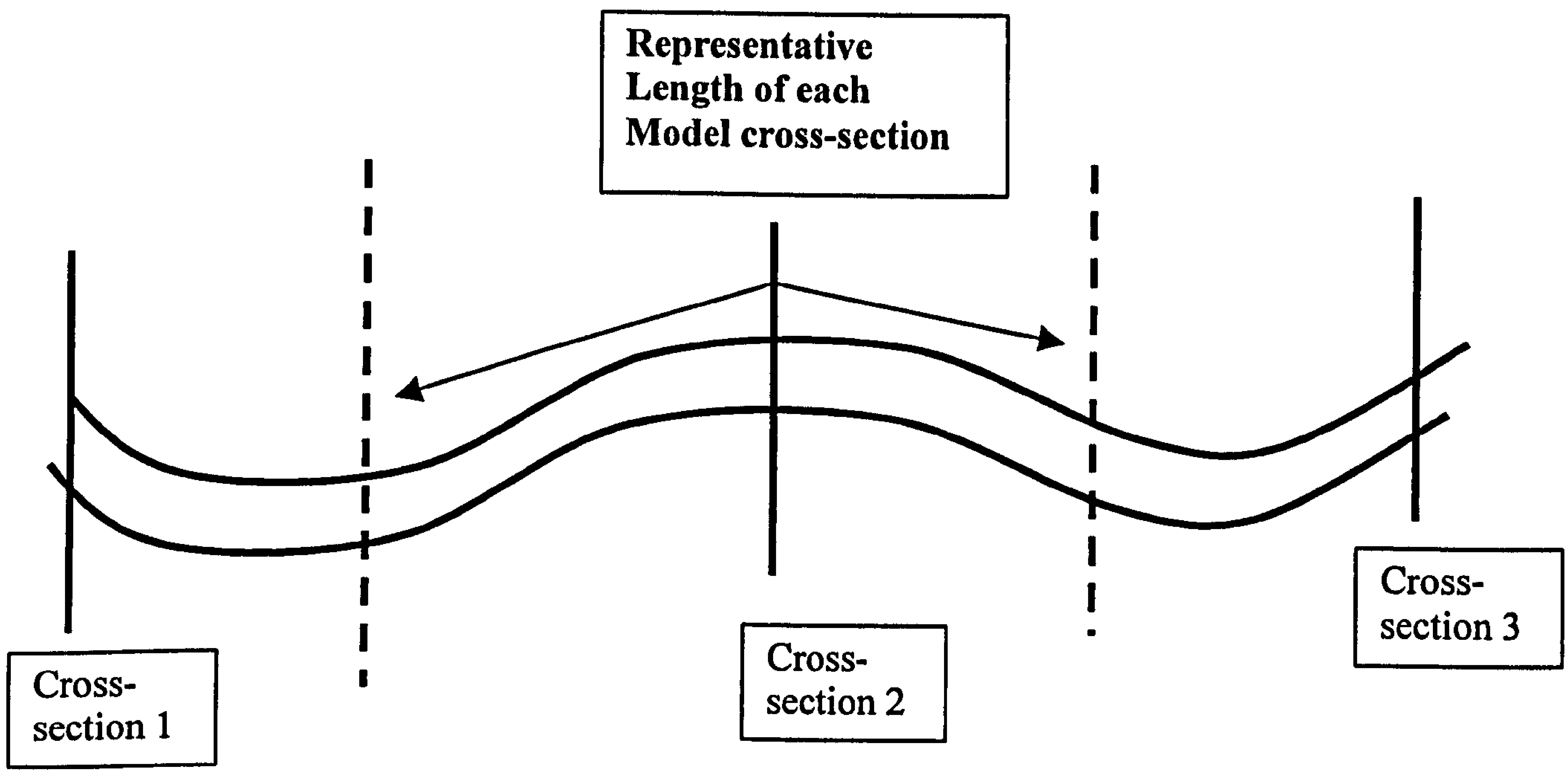


Figure 3.13 Representative Reach length of a River Model Cross-section

This representative reach length tends to be in the region of 150-300m in practical engineering studies.

3.3.7 Final Equations

Rewriting equation 3.17 in terms of Q(x,t) and h(x,t)

$$\frac{\partial A}{\partial t}(h) = \frac{\partial A}{\partial h}\frac{\partial h}{\partial t} = b\frac{\partial h}{\partial t} \tag{3.27}$$

and substituting expression for friction slope in equation 3.22 yields the St Venant Equations

$$\frac{\partial h}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = 0 \qquad \textit{Continuity} \tag{3.28}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gA\frac{Q|Q|}{K^2} = 0 \qquad \textit{Dynamic} \tag{3.29}$$

Where a lateral flow exists between the flood plains and the main channel, equations 3.28 and 3.29 become

$$\frac{\partial h}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = q \qquad \textit{Continuity} \tag{3.30}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\beta\frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gA\frac{Q|Q|}{K^2} = 0 \qquad \textit{Dynamic} \tag{3.31}$$

The inclusion of the lateral flow term in the dynamic equation had negligible effects on the predictions of flow and water level; therefore its contribution to momentum conservation has been ignored.

3.4 Numerical Solution - Preissmann Four-Point Implicit Scheme

Because analytical solutions are not available for the continuity and dynamic equations, the solution of such is normally undertaken through the use of finite differences. The basis for such a method is that the behaviour of the continuous variables, which describe the state of flow, can be evaluated at a discrete number of grid points within the space-time domain.

Several solution techniques exist whereby the differential equations are replaced by divide differences. Schemes developed by Preissmann, Delft Hydraulics Laboratory, Abbott-Ionescu, Vasiliev and Gunaratnam-Perkins are all detailed by Cunge et al. Only the Preissmann four-point implicit scheme will be detailed in this instance, as it is the solution technique that is used within the ISIS program.

Figure 3.14 shows four points in the x-t plane at distances x_j and x_{j+1} and times t^n and t^{n+1} at which the flow variables Q and h are to be determined.

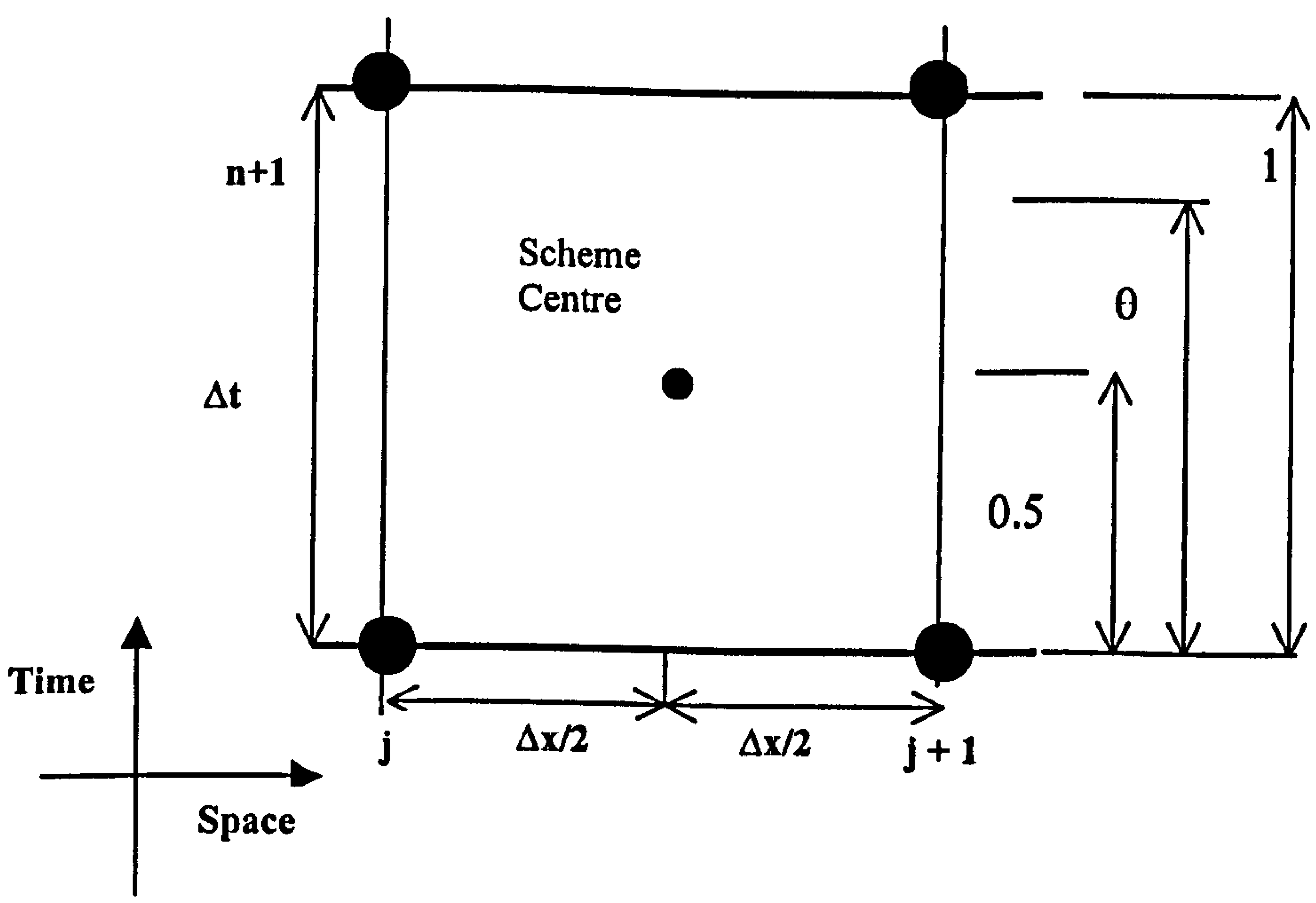


Figure 3.14 - Preissmann Four-Point Implicit Grid

In the Preissmann scheme, the space and time derivatives $\partial f/\partial x$ and $\partial f/\partial t$ (where the function f is usually flow (Q) and water level (h)) are represented by a weighted average of the values of f at four solution nodes, divided by the space and time increments respectively. For the space derivative, the weighting factor θ is a given value between 0.5 and 1.0, and for the time derivative the weighting factor is fixed at 0.5. Thus,

$$\frac{\partial f}{\partial x} \approx \theta \frac{\left(f_{j+1}^{n+1} - f_j^{n+1}\right)}{\Delta x} + (1-\theta)\frac{\left(f_{j+1}^{n} - f_j^{n}\right)}{\Delta x} \tag{3.32}$$

$$\frac{\partial f}{\partial t} \approx \frac{\left(f_j^{n+1} - f_j^{n}\right) + \left(f_{j+1}^{n+1} - f_{j+1}^{n}\right)}{2\Delta t} \tag{3.33}$$

As these equations have been written in general form, the interested reader is referred to Cunge et al (1980). It should be noted that the above equations contain four unknown quantities: stage and discharge at the time level $n+1$ and at space positions j and $j+1$. As a result, the equations cannot be solved explicitly. For any computational grid of N points, $2N-2$ equations with $2N$ unknowns (N values of Q, and N values of h) exist. Therefore two additional equations are necessary to solve the problem. These come from the boundary conditions.

Boundary conditions define the limits of the modelled river system. That is, they describe the characteristics of the flow at the upstream and downstream ends of the river reach. Boundary conditions that can be employed are as follows:

Upstream

- $Q_j^{n+1} = f(t)$ ~ *Flow as a function of time (Flow Hydrograph)*
- $h_j^{n+1} = f(t)$ ~ *Stage as a function of time (Stage Hydrograph)*

Downstream

- $Q_{jj}^{n+1} = f(t)$ ~ *Flow as a function of time (Flow Hydrograph)*
- $h_{jj}^{n+1} = f(t)$ ~ *Stage as a function of time (Stage Hydrograph)*
- $Q_{jj}^{n+1} = f(h_{jj}^{n+1})$ ~ *Relationship between stage and discharge (Rating Curve)*

For subcritical flow, typical boundary conditions that are used in river modelling are the upstream flow hydrograph and a downstream rating curve.

With these extra two equations, the *2N* unknowns can be solved simultaneously across all grid points at each succeeding time level. Due to the non-linear nature of these equations, some form of iteration technique must be employed (usually the Newton-Raphson Method (see Appendix 4)). The solution of the finite difference equations in their Newton-Raphson form is carried out using matrix methods. To solve for stage and discharge at the next time step requires a knowledge of cross-sectional area (A), top breadth (B), conveyance (K) and momentum correction coefficient (β) at the next time step.

These parameters are normally calculated at each cross-section for a number of different water levels, the values of which are held in a database. Once the data tables have been calculated for each cross-section, the numerical model can interpolate in these during the solution procedure to obtain satisfactory estimates of area, top breadth, and conveyance.

From this, the Preissmann four-point implicit technique may be summarised as follows

- Construct the system of *2N-2* continuity and dynamic equations in finite difference form
- To form the additional two equations, set up upstream and downstream boundary conditions
- Solve the system of *2N* equations using matrix methods and using current values of A, B and K as initial estimates of A ,B, K at the next time step
- Using the Newton-Raphson technique, repeat the solution of the *2N* equations with the computed values of A, B and K until convergence is achieved
- Repeat all of the above for each time step, for the duration of the unsteady flow event.

A more detailed description of the Preissmann scheme can be found in Cunge et al (1980), Preissmann and Cunge (1961) and Abbot (1970)

Chapter 4 Code Development and Testing

4.0 Incorporation of new Methods To ISIS

During the last decade, extensive research has been carried out on modelling the secondary losses resulting from overbank flow (See Chapter 2), with several new alternatives being proposed for the Single Channel Method and Divided Channel Method. Two of these methods, the Ackers (1991) and the James & Wark (1992) method, have been incorporated into the current ISIS software in an attempt to enhance the conveyance calculation in the computer model. When the water level is above bankfull level and river flow interacts with flood plain flow both methods account for energy losses other than bed friction. Essentially, these methods were chosen as they are recommended by the Environment Agency. Although more sophisticated methods are currently available they have not been proved to be, in practice, any better than those selected here. Further Details of the methods can be found in Ackers (1991), James and Wark (1992) and Forbes (1998).

4.1 Identification of Requirements For Code Modification

In order to discover how best to incorporate both the Ackers Method and the James and Wark Method into ISIS, a detailed examination of the existing ISIS source code was made. This code, previously known as ONDA, has been constantly developed over the past 26 years. The original ONDA software forms the basis of the current ISIS software.

All one-dimensional river models require to calculate cross-section properties, such as cross-sectional area, main channel top breadth, conveyance and the momentum correction coefficient. In fixed bed models this is normally undertaken as a pre-processing calculation where tables of water level versus cross-sectional area, top breadth, conveyance and momentum correction coefficient are computed for each cross-section prior to the start of the flood routing computations.

Once the section properties are calculated they are stored in an array which is often referred to as the "Conveyance Tables". In the existing ISIS source code this calculation was undertaken in a subroutine titled PRRVR. Developing an understanding of the existing PRRVR was difficult since no list of variables was available and, many years of development had resulted in many undocumented changes being made to this subroutine.

4.2 The Working of ISIS Subroutine PRRVR

PRRVR is the existing ISIS subroutine that calculates the geometrical properties for each cross-section. This subroutine was written in 1975 by the original authors of ONDA/ISIS and has experienced many changes over the years. The fundamental logic of the subroutine is illustrated in Flow Chart 1.

Essentially, PRRVR works by reading a user-defined data file and proceeds to loop through a series of water levels defining the shape of each surveyed channel section and calculating the required cross-section properties (See Flow Chart 1). The cross-section can be laterally sub-divided into a series of 'panels' (up to 50) which can exhibit different bed roughness. Figure 4.01 below shows a cross-section with three panels, namely, a main channel panel and a right and left flood plain panel. The dotted vertical lines indicate the panel divisions. Subroutine PRRVR calculates panel areas and wetted perimeters for each water level and stores them in an array.

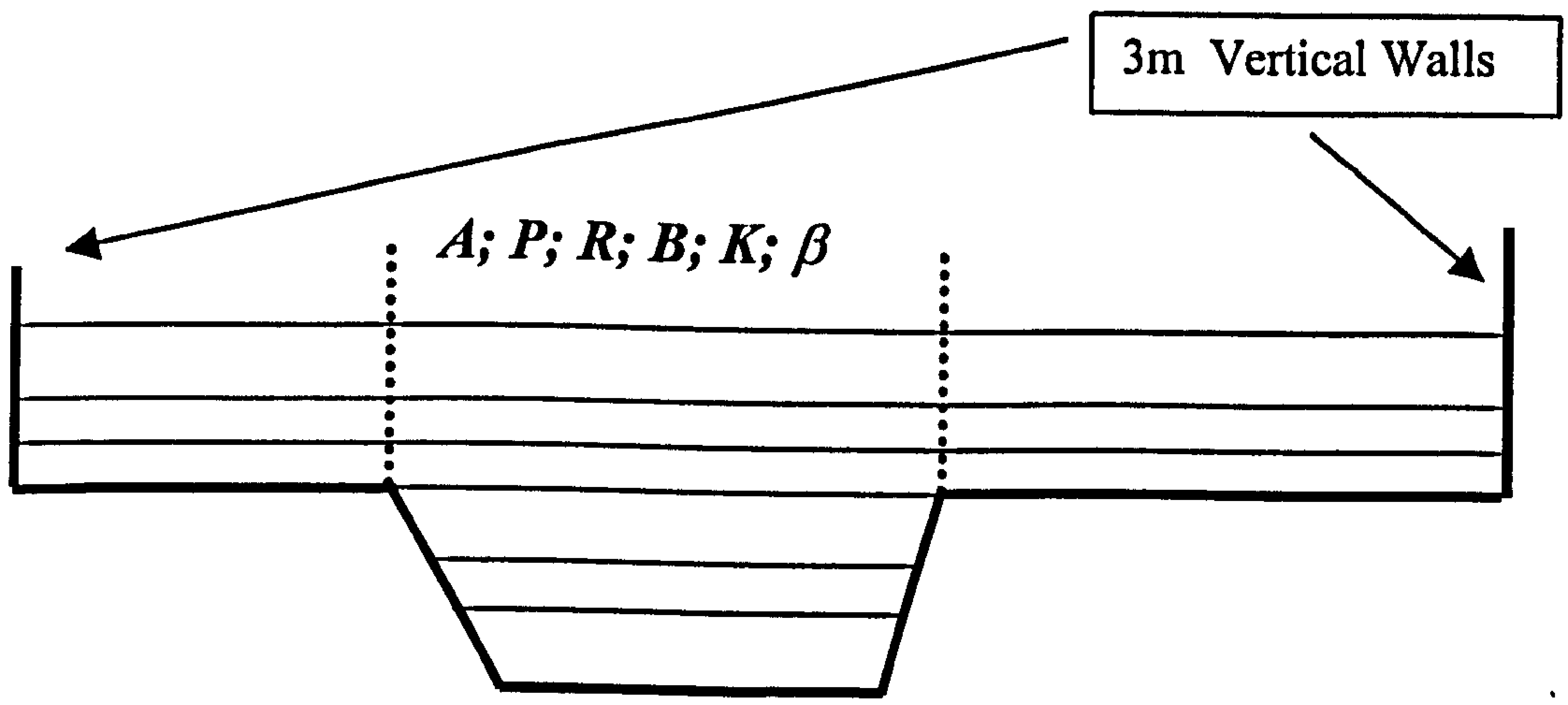
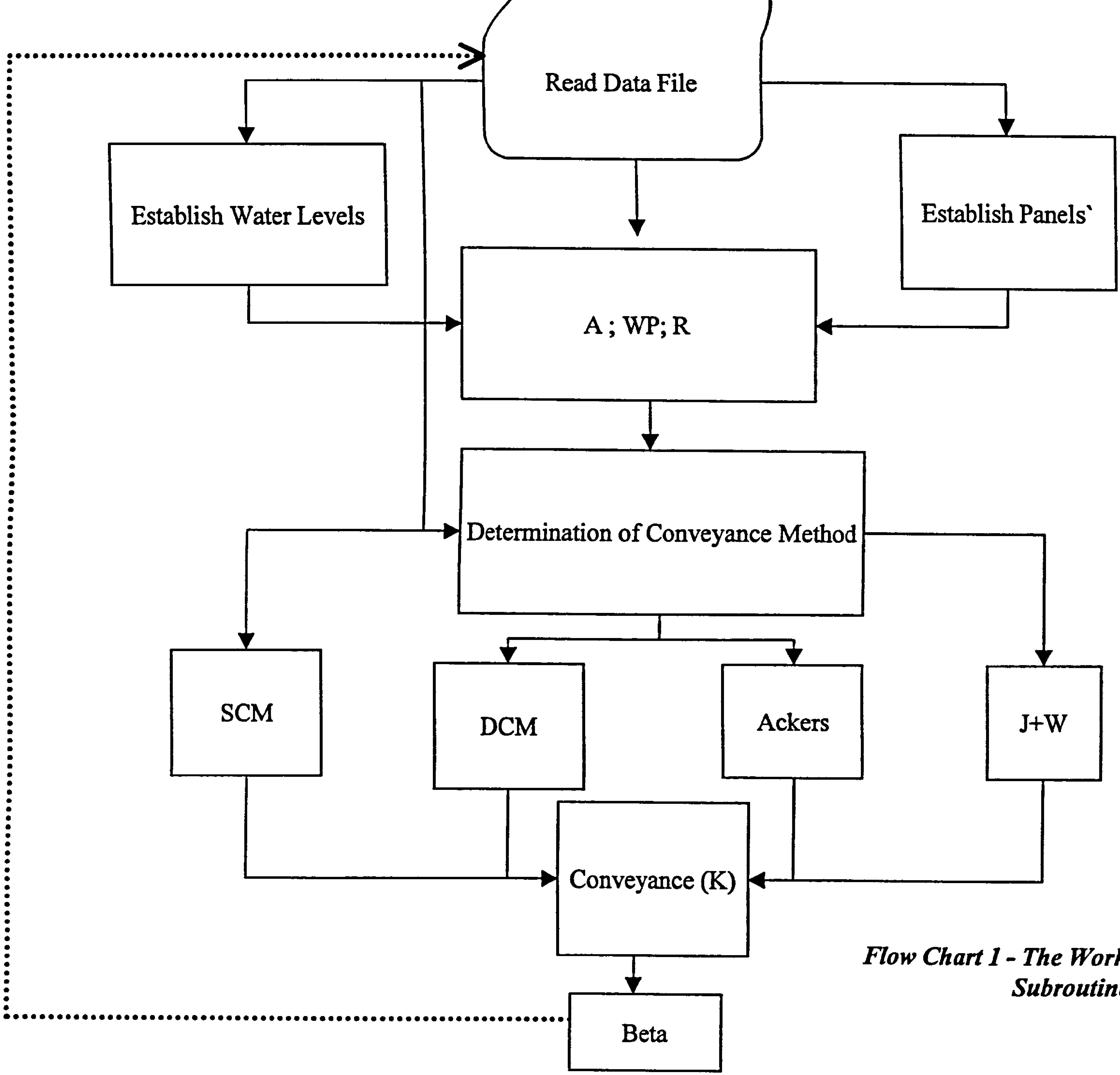


Figure 4.01 - The working of Subroutine PRRVR

The conventional conveyance method used in ISIS, and all other one-dimensional models, is the Divided Channel Method where the cross-section is split into main channel and flood plain zones i.e. 3 panels. The conveyance calculation requires knowledge of the channel shape and the following relationship:



Flow Chart 1 - The Working of ISIS Subroutine PRRVR

$$K = \frac{1}{n} A R^{2/3} \qquad (4.01)$$

Where

K = conveyance,

n = Manning's Roughness Coefficient,

A = Area

R = Hydraulic Radius. (A/P)

This method calculates conveyance for each water level in each panel and implicitly assumes that all sources of energy loss are due to bed roughness.

The final calculation in the PRRVR subroutine is that of the parameter β. The term Beta is used in the conservation of momentum equation of the St. Venant equations and as it is normally close to unity, it can be generally assumed that β=1 for practical situations, Lyness, Myers and Wark 1997. In fact, the ISIS Direct Steady Method assumes β=1 while the unsteady solution calculates Beta using the following relationship.

$$\beta = \frac{\sum_{i=1}^{J} A_i}{\left(\sum_{i=1}^{J} K_i\right)^2} \sum_{i=1}^{J} \left(\frac{K_i^2}{A_i}\right) \qquad (4.02)$$

It should be noted that PRRVR also calculates conveyance, and the other parameters, for a water level 3m above the highest surveyed level in the cross-section data. (See Figure 4.01 and 4.05) This 3m vertical wall is to ensure that the cross-sectional properties calculated cover a sufficient range to include the maximum water level, that may be computed during flood routing. The 3m default setting can be modified by the user if required.

Once the program has calculated the various parameters they are stored in an array to be used later during flood routing. When these 'conveyance tables' are complete, the numerical model can use them during the solution procedure to interpolate estimates of main channel top breadth, area and conveyance.

Having identified the existing conveyance calculation and reviewed its working it was evident that a different methodology would be required to incorporate the new methods into ISIS. The main difference being that the existing methods assume that the conveyance can be estimated using a uniform flow law while the new methods calculate a stage-discharge relationship. It is from this stage-discharge relationship that conveyance must be estimated using the relationship

$$K = \frac{Q}{S^{1/2}} \qquad \textbf{(4.03)}$$

where K is the conveyance, Q the discharge and S the main channel slope.

The new subroutines use the estimate of total discharge to calculate conveyance and do not make use of the sub-division estimates of Q. As a result, for any water level encountered, only the total estimate of conveyance will be calculated. This was deemed to be the most suitable way of incorporating the new subroutines within the existing ISIS framework.

It should also be noted that despite the Ackers and James & Wark Methods calculating panel discharges, recent research by Mcleod (1998) has indicated that, for the James and Wark Method, these may be in error. As a result, the estimate of total discharge was used and simply divided by the square root of a slope to obtain a value of conveyance.

The main channel rather than the flood plain slope is used here, as in one-dimensional river modelling Δx is specified as the distance along the centre-line of the main channel. This also means that the calculated value of conveyance, using Ackers or James and Wark Method, is independent of slope which is similar to the existing ISIS conveyance calculation. i.e. the calculation of discharge in the new subroutines is affected in equal proportion to changes in slope and consequently the conveyance does not change with changes in slope.

It should be remembered that this is a different process in obtaining the conveyance. In the existing ISIS software, the conveyance comes from the geometry and roughness parameters of a cross-section and is independent of slope. Indeed, the new methods have not been derived to enable the calculation of conveyance and the claim that they could be by their authors, James and Wark (1992), has been more an after-thought, than an intention in their formulation.

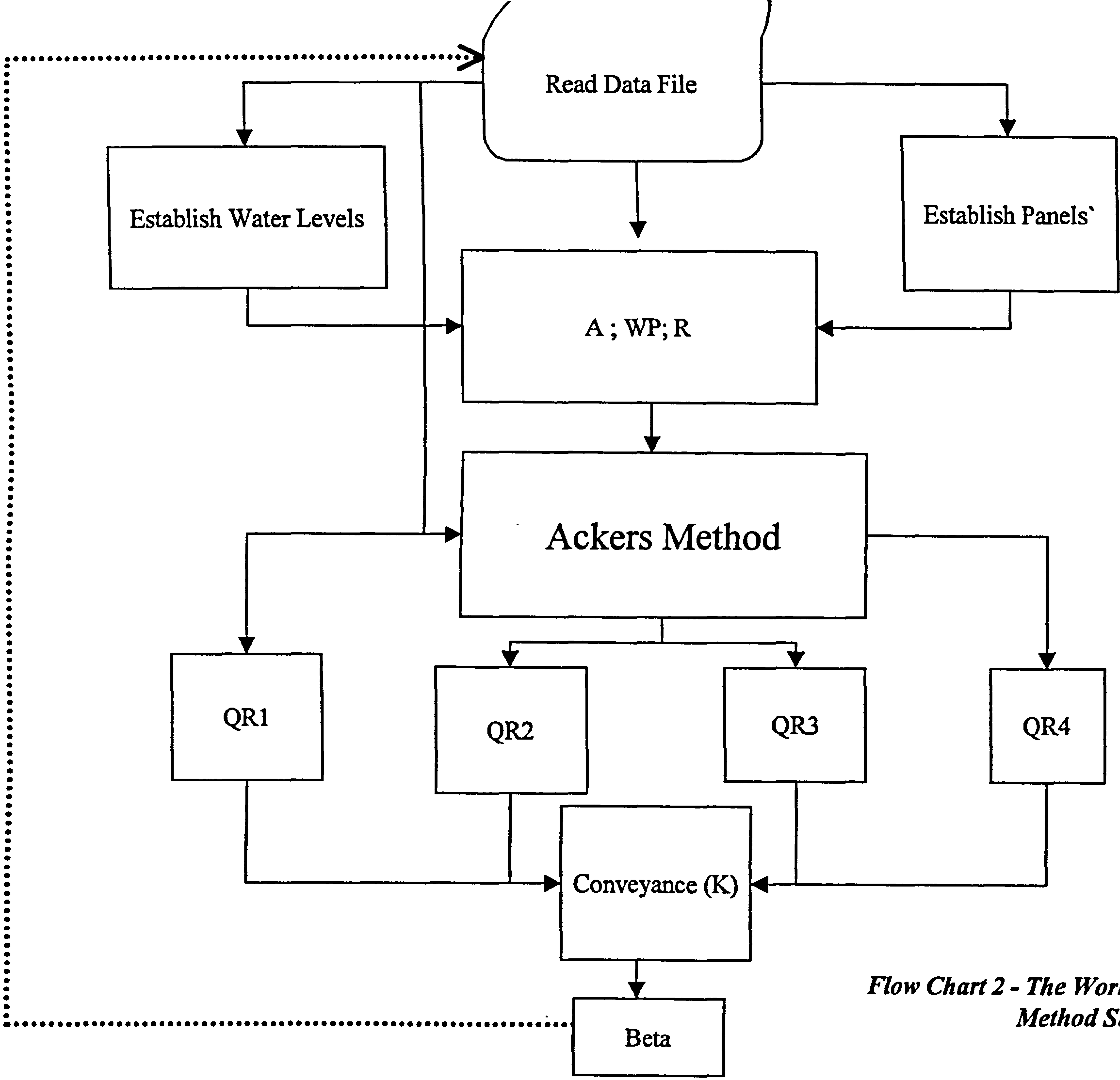
4.3 Coding of New Subroutines

The two new methods were coded separately using initially FORTRAN 90, and later translated back into FORTRAN 77 for compatibility with the existing ISIS source code. The following details the development of the new subroutines

4.3.1 The Ackers Method Subroutine

Flow chart 2 illustrates the computer coding of the Ackers Method. It follows the calculation procedure of the subroutine and illustrates locations where decisions are made.

The Ackers Method was originally intended for the design of a straight compound channel. It was also designed to be a hand calculation and generally required 17 pages of calculations for each water level. This discouraged potential users. The automation of this method will therefore be of significant benefit to those designing such channels. Although the procedure itself is aimed at producing a stage discharge relationship the subroutine can also be used to calculate conveyance, using equation 4.03 above.



Flow Chart 2 - The Working of The Ackers Method Subroutine

The Ackers Method should only be applied when the channel sinuosity is less than 1.02. This was a direct result of the fact that it was based on straight plan-form experiments at the FCF. This is a very limiting situation as it is rare to find a purely straight section in a natural river, which normally have extensive longitudinal variation. As a result this criterion has not been built into the coding and the user is responsible for the selection of conveyance method.

The initial steps of the subroutine are concerned with the reading in of the additional parameters, the longitudinal bed slope, sinuosity, meander wavelength and the main channel side slope. The first stage in the calculation is to translate the natural shape to an idealised geometry. This is required to define other parameters required by the Ackers Method.

The translation is initiated by defining a bankfull elevation which in this case was taken to be the average of the left and right bank elevations. Using this bankfull elevation a main channel area is calculated and additional parameters, such as depth and bottom width, that define the idealised representation of the natural cross-section are computed. An example of this is shown in Figure 4.02.

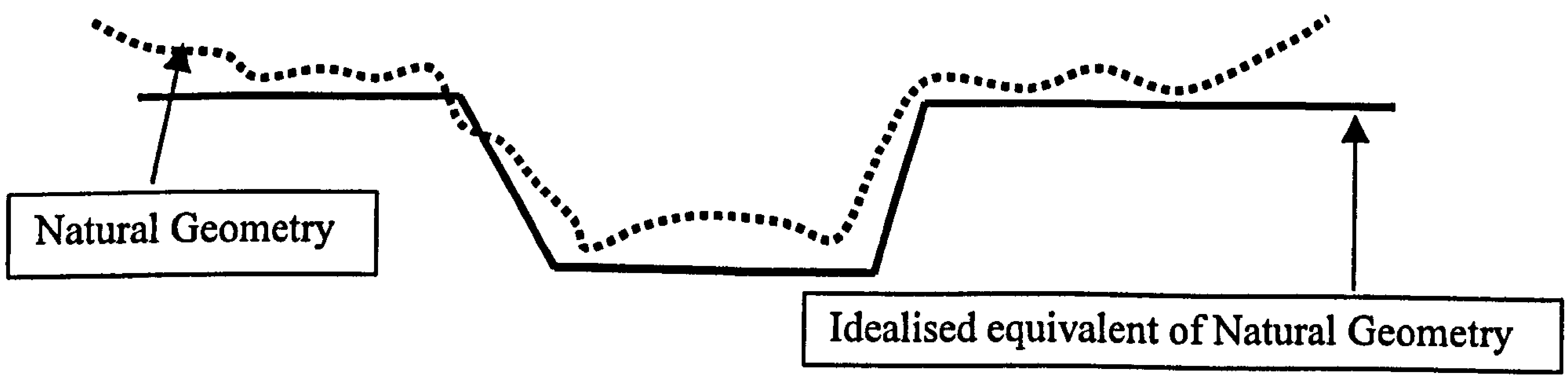


Fig 4.02 - Natural cross-section and Idealised Equivalent

Having defined the idealised geometry, the remainder of the calculation can proceed with flow depths being measured above the idealised bed of the main channel.

Figure 4.03 shows the definition of a "Panel" which is effectively the subdivisions of the cross-section. Panel 1 being the left hand side flood plain, Panel 2 being the main channel and Panel 3 being the right flood plain.

The subroutine continues by reading the geometrical parameters such as area, wetted perimeter and hydraulic radius for each panel, for the water level under consideration. These parameters are read and not calculated as the existing ISIS subroutine PRRVR already calculates and stores them in an array. The Ackers subroutine simply "picks" out the value it requires from this array.

When using the Ackers Method the cross-section is normally divided into a main channel zone and the flood plain zones on either side, as shown in Figure 4.03.
A basic discharge for each zone is calculated using Manning's equation and then summed to provide a total basic discharge 'Qbasic'. It is this basic discharge that is adjusted to account for secondary losses.

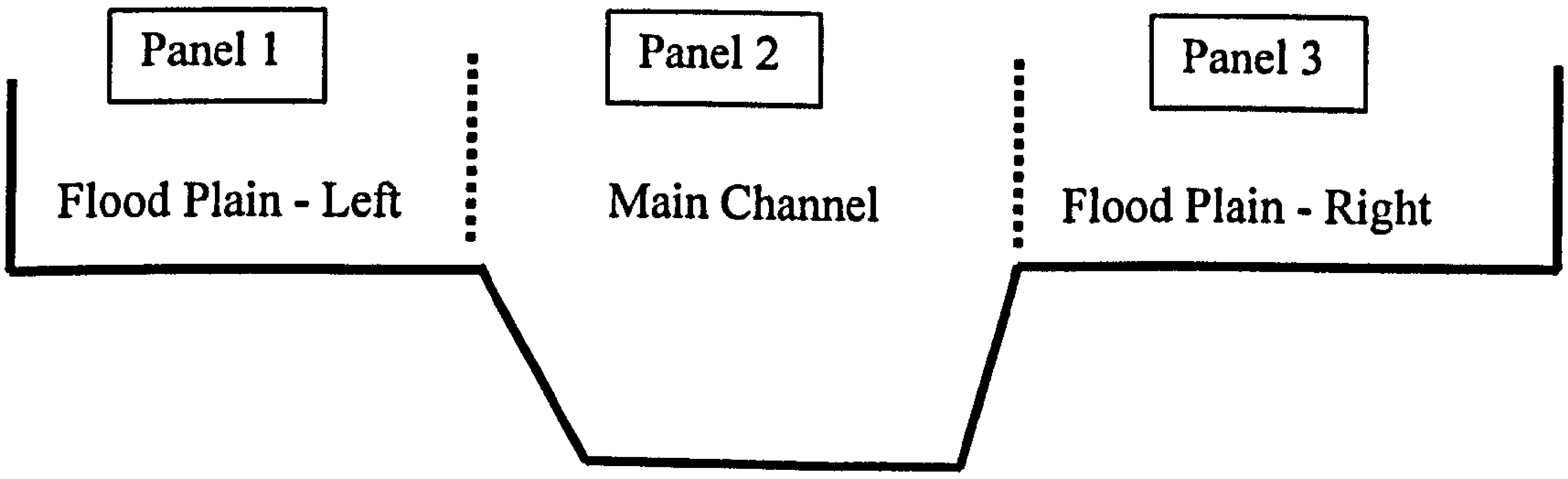


Figure 4.03 - Ackers Method Cross-Sectional Division

The adjustment is made by using formulae for each of the four possible flow regions, defined by Ackers (1991), the correct value being selected later in the calculation. Ackers (1991) proposed a different adjustment for each region.

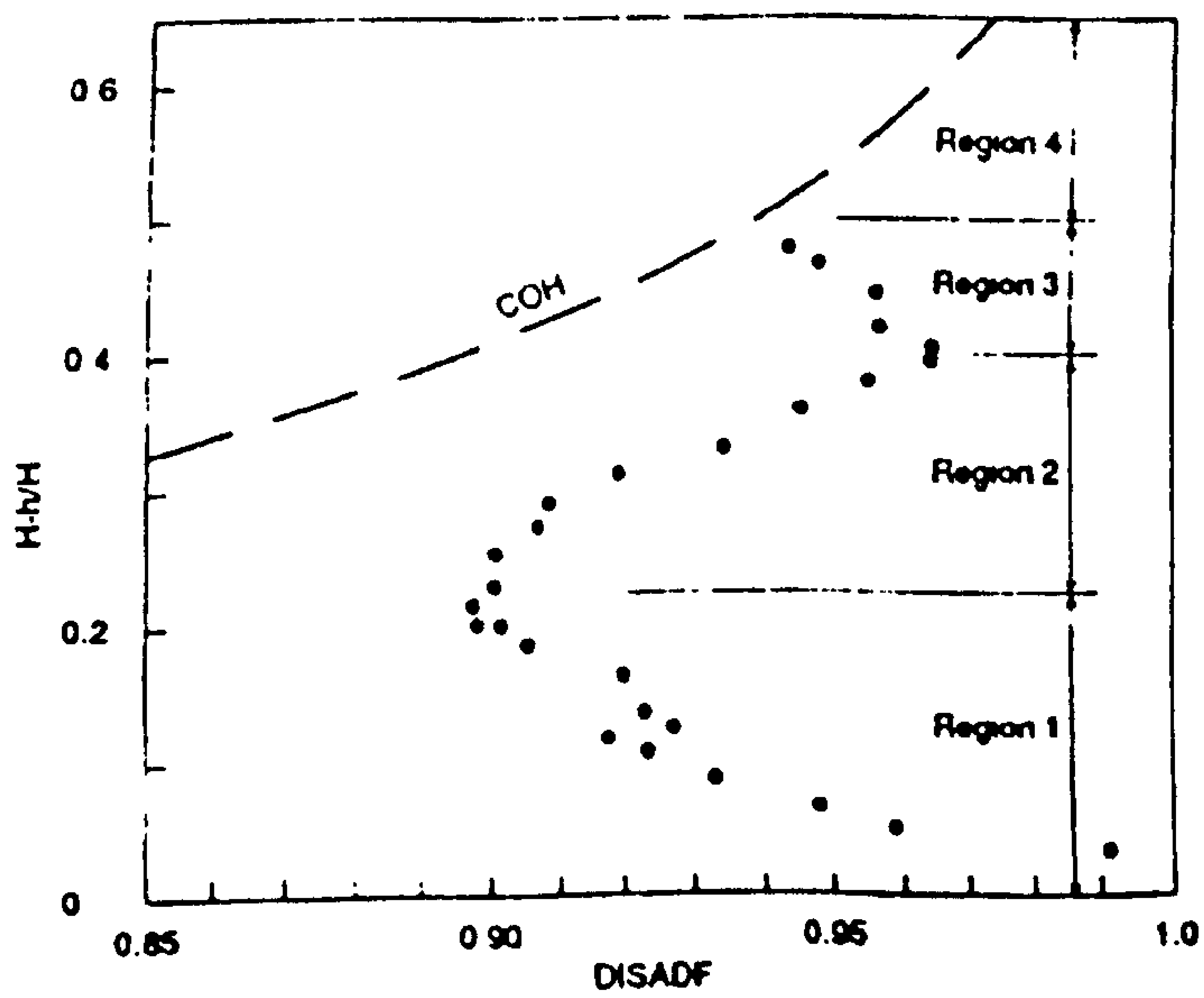


Figure 4.04 - Regions of Flow behaviour (Ackers 1991)

The region 1 adjustment involves the calculation of a discharge deficit (DISDEF) and is dependent of the relative friction factors, number of flood plains, velocity differentials and aspect ratio. Region 1 behaviour occurs at very low overbank stages. (i.e. Relative Depths of up to 0.2)

DISDEF is simply subtracted from Q_{basic}

$$\mathbf{Q = Q_{basic} - DISDEF} \qquad \mathbf{(4.04)}$$

For the other three zones the adjustment factor takes the form of a multiplier i.e.

$$\mathbf{Q = Q_{basic} * DISADF} \qquad \mathbf{(4.05)}$$

Region 2 behaviour occurs at slightly greater depths than region 1. The adjustment factor for this is more complicated than any of the other zones as it refers to a "shifted stage", which is larger than the actual stage. The reason for this being that Ackers (1991) observed that typical laboratory results coincided with a line approximately parallel to but lower than the coherence curve. (Coherence being defined as a measure of the relative strength of the interaction effects) This was an interesting coding problem as the shifted depth could be significantly higher than the actual depth thereby leading to a program crash. This occurred when the shifted depth was at a level that was higher than ISIS had calculated. This was solved by limiting the calculation to the last user-defined depth. The shifted depth corresponding to it can normally be catered for with in the 3m vertical wall.

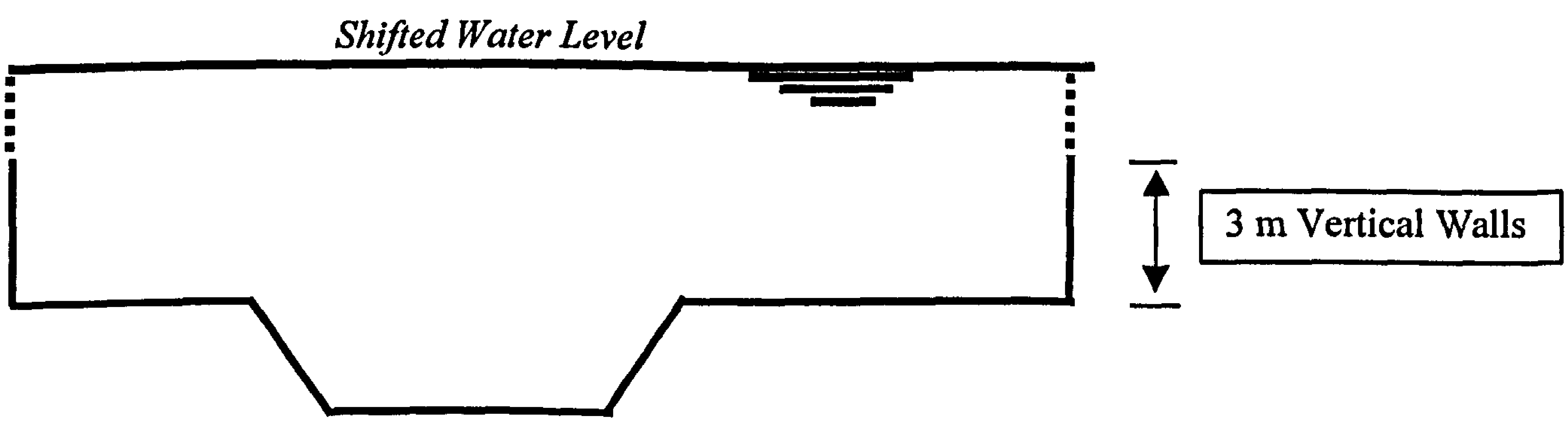


Figure 4.05 - Shifted Depth Exceeding 3m Vertical Walls

At larger stages, Region 3, the laboratory results decrease with stage and the adjustment factor is expressed as a function of coherence for the actual stage. Briefly the correction factor for region 3 is dependent on stage, geometrical shape and roughness.

The Ackers (1991) study did not include stages that were large enough to confirm the existence of Region 4, however, the discharge correction factor is expected to increase with increasing stage. Ackers provided a theoretical justification for assuming that DISADF4 should take the value of COH for the given stage.

Once the subroutine has gone through the calculations for the flow regions the correct flow is established and the final adjusted discharge obtained. This final discharge is then divided by the square root of the main channel slope to obtain a value of conveyance. The value of conveyance is the penultimate step in the subroutine. The momentum correction factor is then calculated using the following equation:

$$\beta = \frac{\sum_{i=1}^{j} A_i}{\left(\sum_{i=1}^{j} K_i\right)^2} \sum_{i=1}^{j} \left(\frac{K_i^2}{A_i}\right) \qquad (4.06)$$

As the Ackers Method subroutine uses the value of total discharge in the conveyance calculation, and not the zonal flows, the value of Beta must be 1. This is acceptable in practical river modelling, Lyness, Myers and Wark 1997.

After Beta is calculated the code returns to the start of the Loop and continues the process until no more water levels are encountered. Conveyance is calculated at every user-defined elevation and at intervals of 10% of total depth plus the additional 3m wall.

4.3.2 The James And Wark Method Subroutine

The following flow charts (3-8) illustrate the computer coding of the James and Wark Method. It illustrates the logic of the new subroutine and highlights the necessary decisions.

The initial step involves the reading of a user-defined data file to obtain values of sinuosity, side slope, meander wavelength and flood plain slope. Figure 4.06 illustrates an example data file for the James and Wark Method subroutine.

X and Y co-ordinates

Estimate of Manning's 'n'

Flood Plain Slope

Sinuosity

Meander Wavelength

Side Slope

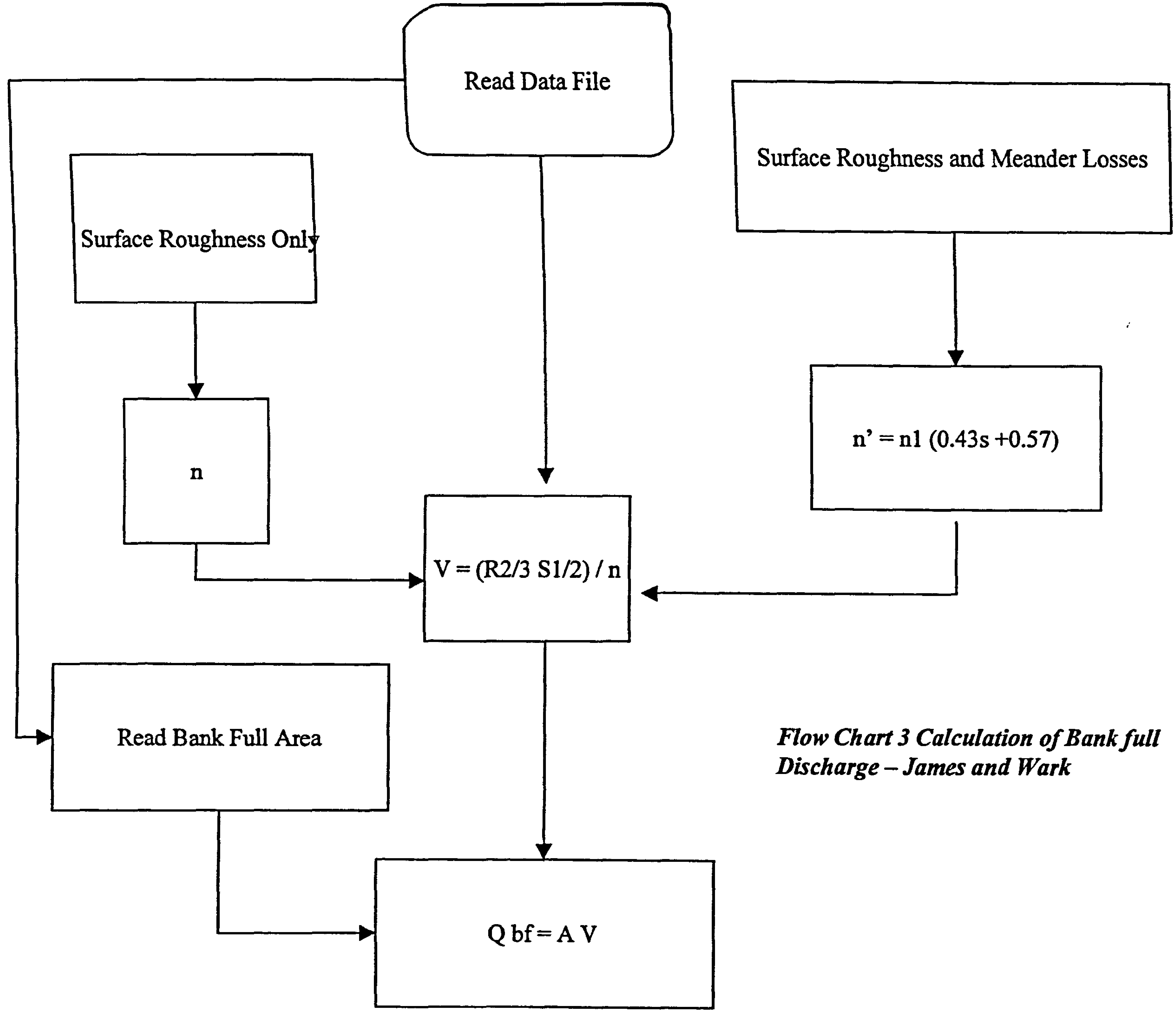
ross-ction abel

stance wnstream next odel oss-ction

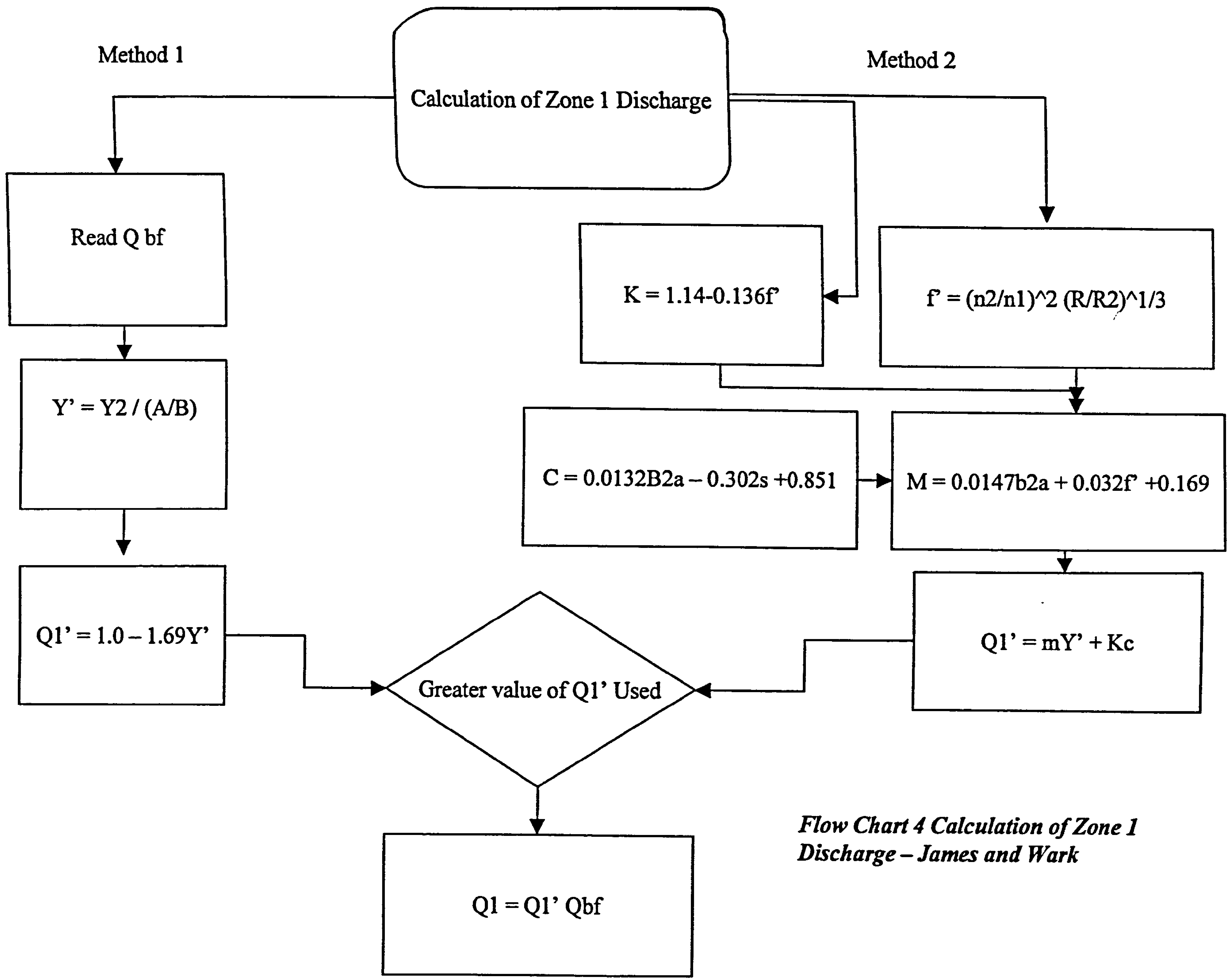
Number f lines f data in ross-ection

```
RIVER Section 1
SECTION
Sect1
16.490          jmswk                1.374   12.0   1.0   0.000996
16
0.000   0.272   0.010   1.000
0.000   0.260   0.010   1.000
0.000   0.255   0.010   1.000
0.000   0.238   0.010   1.000
0.000   0.225   0.010   1.000
0.000   0.198   0.010   1.000
1.890   0.198   0.010   *1.000   m
1.900   0.198   0.010   *1.000   p
1.950   0.153   0.010   1.000
2.320   0.153   0.010   1.000
2.800   0.048   0.010   1.000
2.950   0.048   0.010   1.000
3.100   0.198   0.010   *1.000   s
7.998   0.198   0.010   *1.000   n
10.00   0.198   0.010   1.000
10.00   0.272   0.010   1.000
```

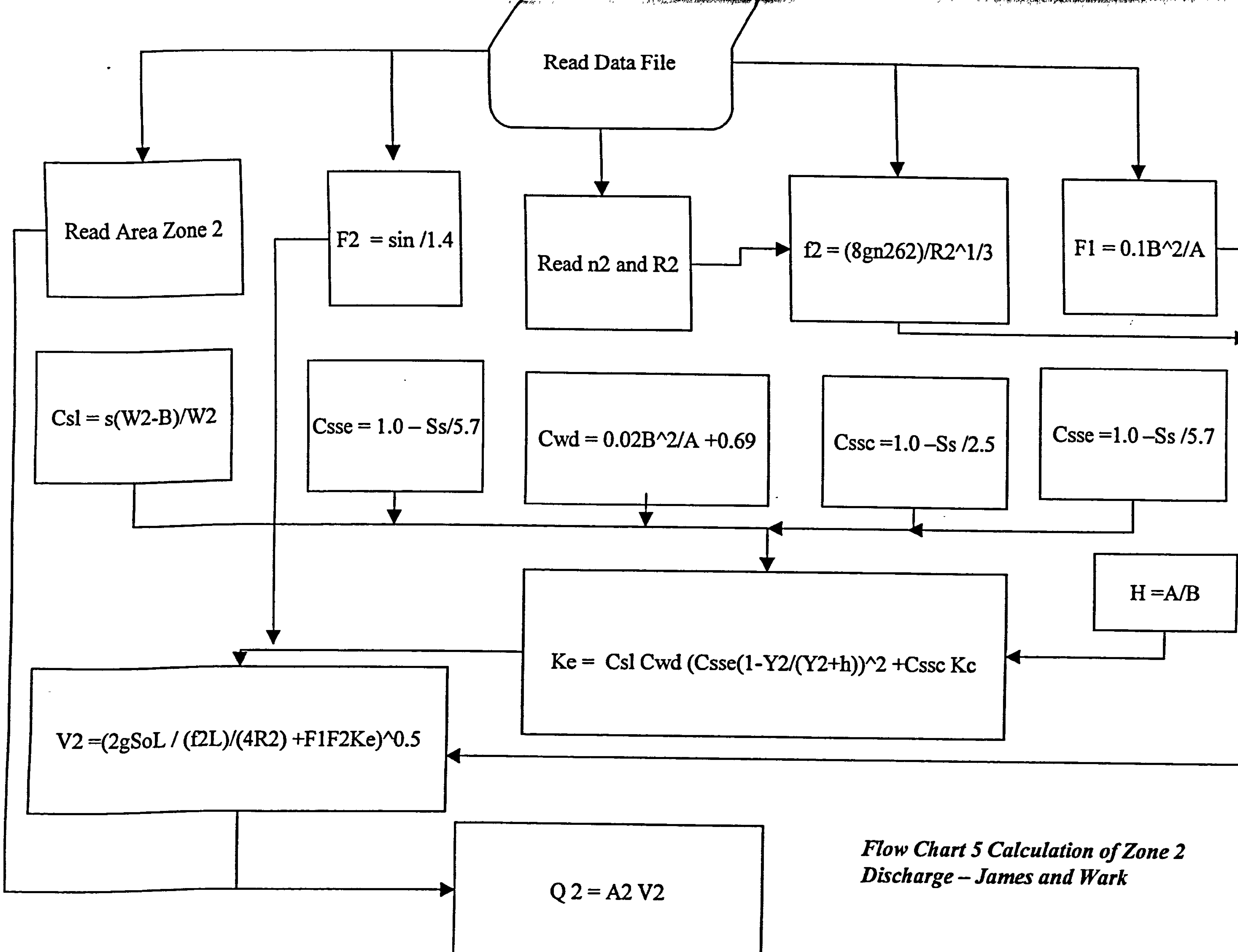
Figure 4.06 - Example James and Wark Method Data File



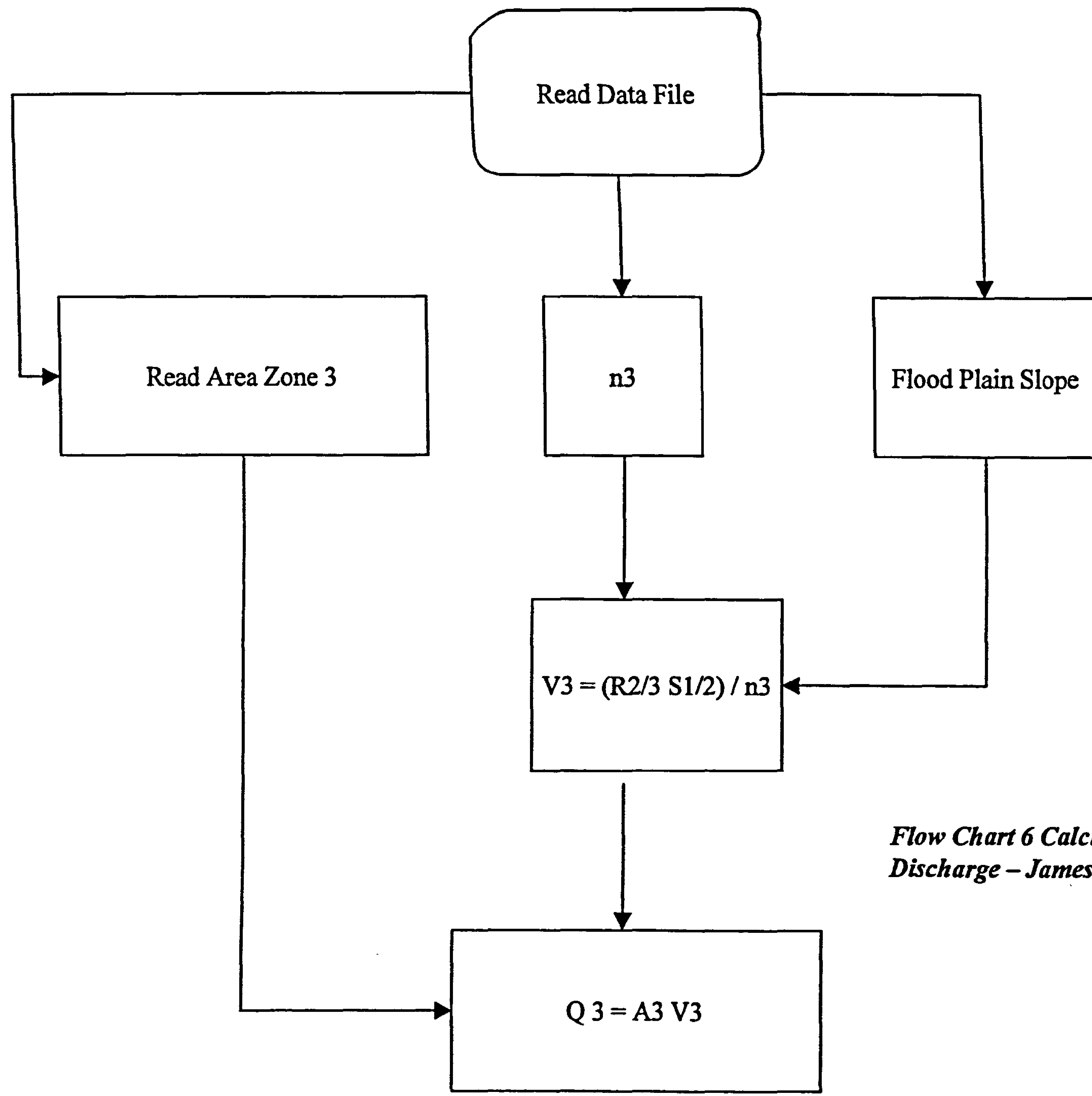
Flow Chart 3 Calculation of Bank full Discharge – James and Wark



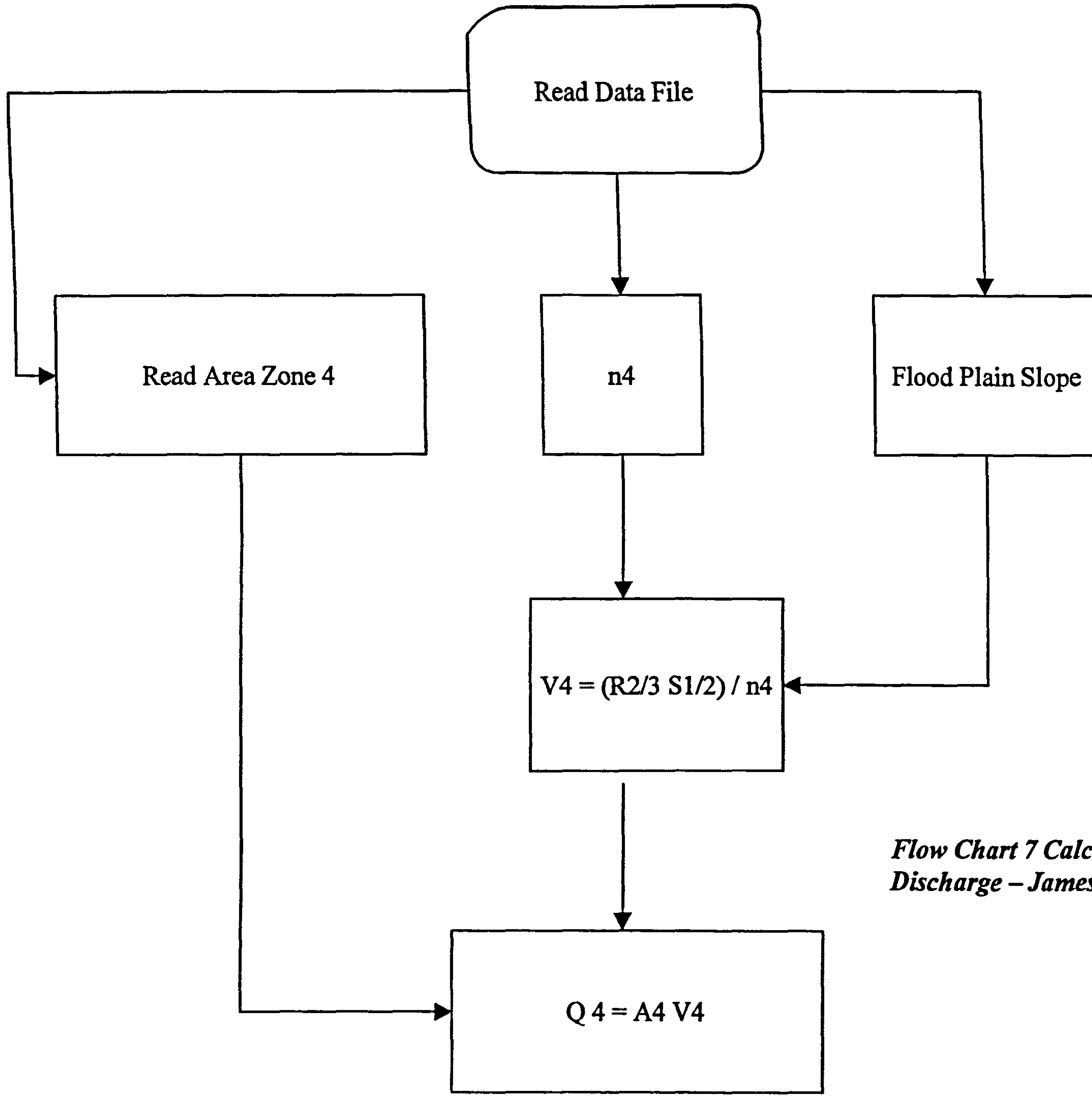
Flow Chart 4 Calculation of Zone 1 Discharge – James and Wark



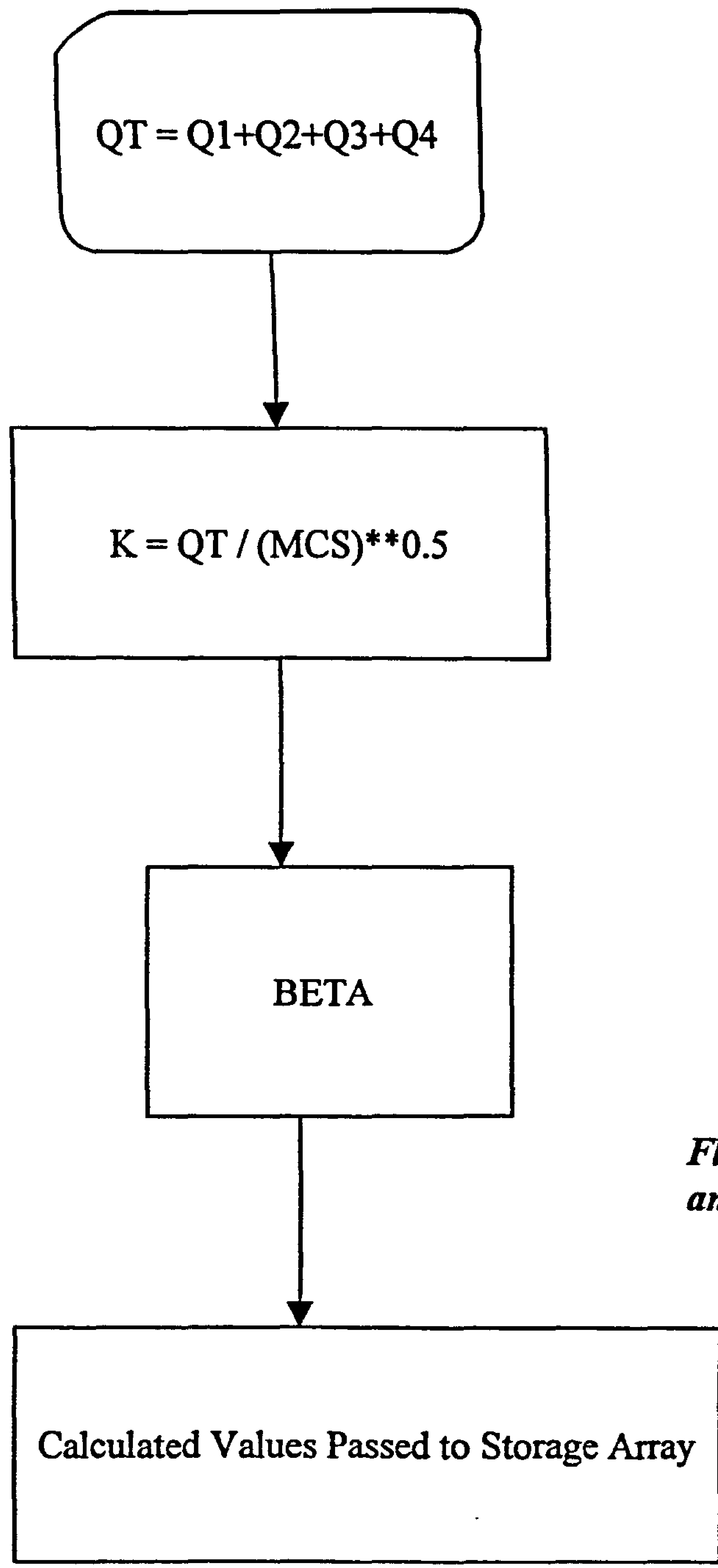
Flow Chart 5 Calculation of Zone 2 Discharge – James and Wark



Flow Chart 6 Calculation of Zone 3 Discharge – James and Wark



Flow Chart 7 Calculation of Zone 4 Discharge – James and Wark



Flow Chart 8 Calculation of Conveyance and Beta – James and Wark

At this stage it is necessary to identify the horizontal extent of both the main channel and the meander belt width. This is done by the addition of a '*' in the data line required. These are important markers as they also define the limits of the various flow zones that are used in the James and Wark Method. The terms 'p' and 's' refer to the left and right river banks respectively and 'm' and 'n' to the extents of the Meander belt width.

Figure 4.07 illustrates the flow zones which are defined as follows :

Zone 1 - area below bankfull
Zone 2 - area above bankfull within meander belt width
Zone 3 - area on LHS outside meander belt width
Zone 4 - area on RHS outside meander belt width

The first steps undertaken are to calculate the area and wetted perimeter for each water level encountered. This is essentially an exercise in data transfer from an array containing the values of A and P etc. These values are not directly calculated by the new subroutine as they are already available from the PRRVR.

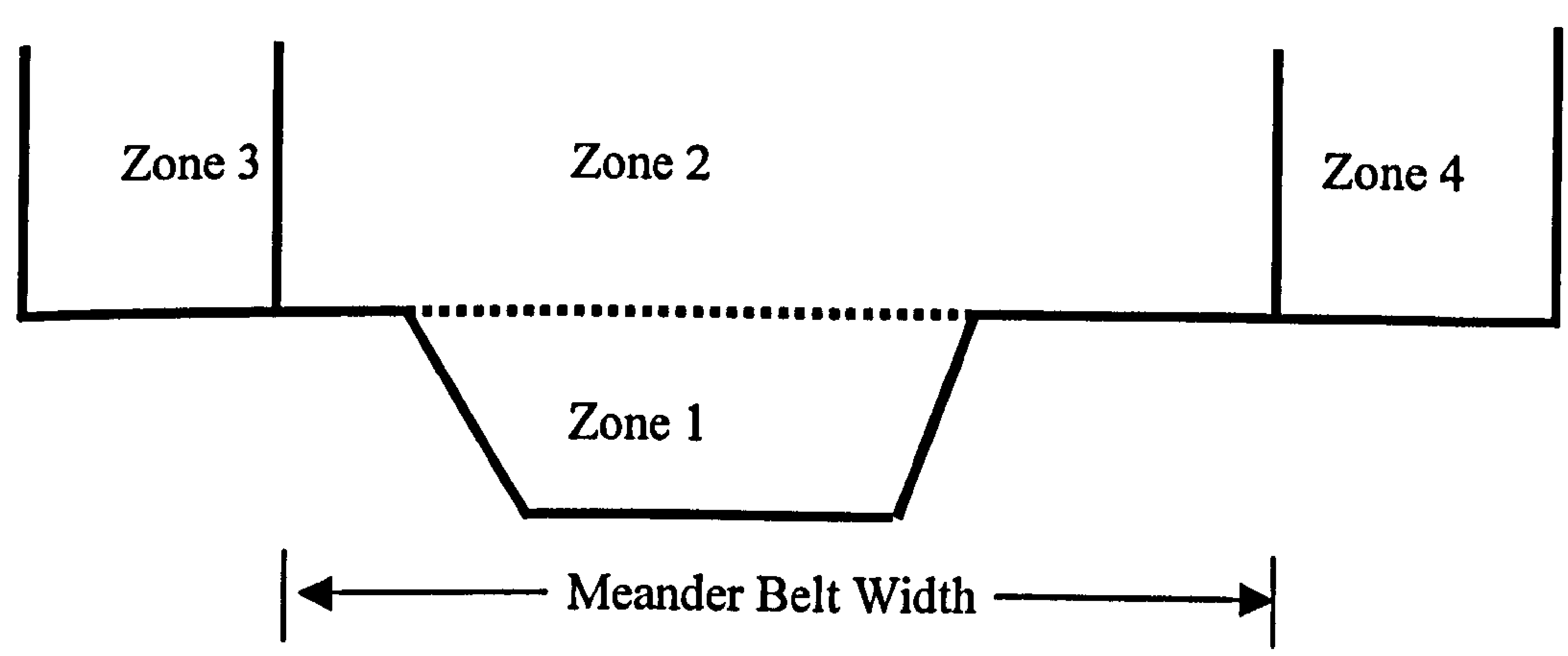


Figure 4.07 - Definition of Flow Zones For The James & Wark Method

Once the program has read in the additional data and obtained the values of area etc. it proceeds to calculate the bank-full discharge. This being obtained by the multiplication of area and mean velocity 'V' where the value of 'n' is adjusted to account for energy losses associated with river meandering.

This is achieved by use of the Linearised Soil Conservation Method (LSCS). From this a bank-full discharge is obtained which accounts for some of the effects of flow interaction.

The Zone 1 discharge is calculated by multiplying the bankfull discharge by an adjustment factor. This adjustment factor is calculated by two methods and the larger of the two values is selected.

Zone 2 is defined as the region above bankfull, but within the horizontal extent of the meander belt width. The discharge in this zone is also calculated by the multiplication of flow area (above bankfull only) and the mean flow velocity 'V2'. The term V2 contains many empirical parameters that are shown in flow chart 5.

These empirical terms are to account for the expansion and contraction of flow over the main channel. The term Kc is the flow contraction coefficient and is derived from a table published by Rouse (1950). The table is shown below as Table 4.01 with the correct value of Kc being interpolated relative to a value of Y2/(Y2+h).

The interpolation is facilitated in the code by a series of 'IF' statements.

y2/Y(2+h)	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Kc	0.5	0.48	0.45	0.41	0.36	0.29	0.21	0.13	0.07	0.01	0

Table 4.01 - Contraction Loss Coefficients (Rouse 1950)

The discharges in Zones 3 & 4 are obtained conventionally with bed friction being assumed as the only source of energy loss. The terms V3 and V4 are both calculated using Manning's equation and multiplied by the corresponding areas A3 and A4 respectively.

Having calculated the discharges in all four zones, they are finally summed to give a total discharge.

The penultimate calculation in this subroutine is to obtain a value of conveyance. To do this, the total discharge is divided by the square root of the longitudinal main channel slope.

The final calculation is that of Beta, the momentum correction factor. This is calculated by equation 4.06.

As the James and Wark subroutine only considers the total discharge, and not the individual zonal flows, the value of Beta must equal 1.

Once the subroutine completes its final computation the results are stored in the appropriate array for future use by the hydrodynamic calculations. On the completion of this the calculation moves to the next water level and begins again. This is repeated until all defined water levels, including a default vertical wall of 3m, have a corresponding value of conveyance.

4.3.3 Additional Adjustments To Existing ISIS Source Code

Alterations had to be made within subroutine PRRVR so that information could be transferred from it to the new subroutines ACKERS.F and JMSWK.F and vice versa.

Along with the coding of the new subroutines, changes needed to be made to the data entry unit, as some additional information had to be specified. For example, the J+W method requires meander wavelength, side slope, flood plain slope and sinuosity in addition to a surveyed cross-section and roughness estimate.

To account for these additional parameters they were given variable names and declared in the PRRVR subroutine. These parameters are only read if another variable name, relating to the new subroutines, is present in the data file.

The variable name 'CONVME' is used within ISIS to determine whether conveyance is to be calculated with or without panels. As a result, it seemed reasonable to add an 'IF' statement to PRRVR which would effectively mean the following :

If 'CONVME' = 1 THEN CONVME = SCM
If 'CONVME' = 2 THEN CONVME = DCM
If 'CONVME' = 3 THEN CONVME = ACKERS
If 'CONVME' = 4 THEN CONVME = J+W

where,
SCM = Single Channel Method
DCM = Divided Channel Method
ACKERS = The Ackers Method
J+W = The James and Wark Method

This was the methodology behind the application of the new methods within the existing ISIS framework.

The additional parameters had to fit into space left over in the existing data file. As a result, the data files that were used had to be of a very rigid format. This is a very onerous task when setting up a model containing a large number of cross-sections and could be avoided with the development of a proper "front-end" similar to the forms editor used in the full version of ISIS.

Having made the appropriate changes to the ISIS source code, and added the two new methods, the programs were compiled along with all the other ISIS subroutines and programs and new executable files produced. Two executable files were created the first to enable a water surface profile to be calculated and the second to produce the stage discharge relationship (ISIS Utility).

When use is made of the new ISIS utility (stage Discharge Relationship option within ISIS) it is clear why a single value of conveyance was used. This option calculates a value of conveyance then prompts the user for an estimate of slope, which it then multiplies by the conveyance to produce an estimate of discharge. i.e.

$$Q = KS^{1/2}$$

In the new James and Wark subroutine you are required to provide an estimate of slope which can then be used again in the utility program to obtain this estimate of discharge. If the new subroutine had made use of individual zonal conveyances there would have been confusion as to what slope to use in the utilities option.

Having developed two new conveyance calculation options within ISIS a series of tests need be carried out to assess ease of use and accuracy.

4.4 The Flood Channel Facility (FCF)

The following section outlines the data set that has been chosen to test the recently developed ISIS subroutines. The UK FCF experiments have been selected, as they provide a comprehensive data set collected using modern measurement techniques.

The modelling of the Flood Channel Facility experiments was to test the accuracy of the newly developed subroutines. James and Wark (1992) also modelled these experiments and it is expected that the same level of accuracy would be obtained. By doing so it could be confidently assumed that the coding was correct and that the method was being applied properly.

A description and the results of the FCF Series B experiments can be found in HR Wallingford Report SR2131 Sept 1993. It should be noted that these experiments were used to develop the James and Wark Method. The results of the Series A experiments have been published as HR Wallingford Report SR314 May 1992.

During the Series B experiments three channels were built and tested

1. a 60 Degree meandering channel with trapezoidal main channel,
2. a 60 Degree meandering channel with quasi-natural main channel,
3. a 110 Degree meandering channel with quasi-natural main channel.

Figure 4.08 - The Flood Channel Facility

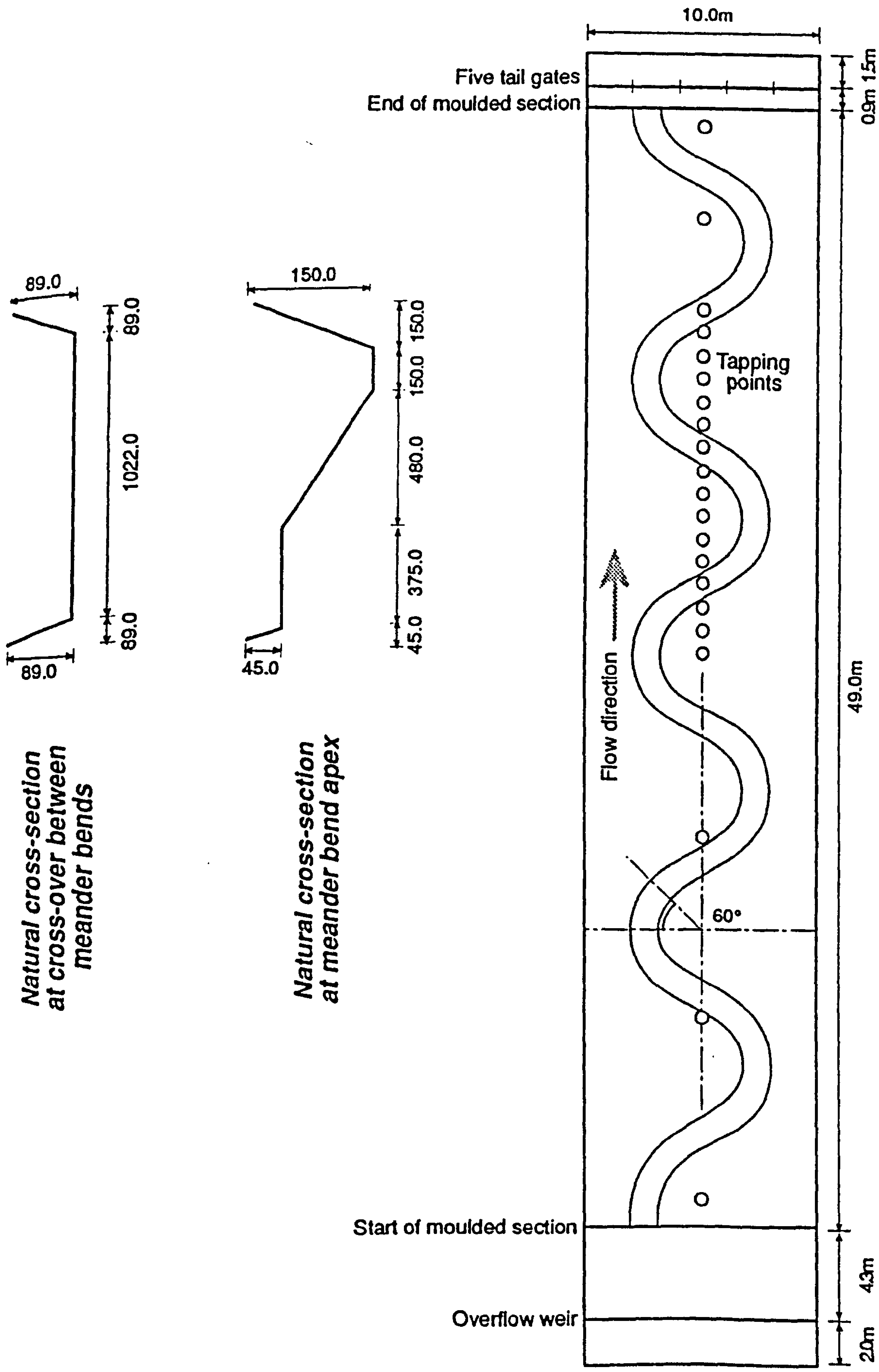


Figure 4.09 - The Flood Channel Facility

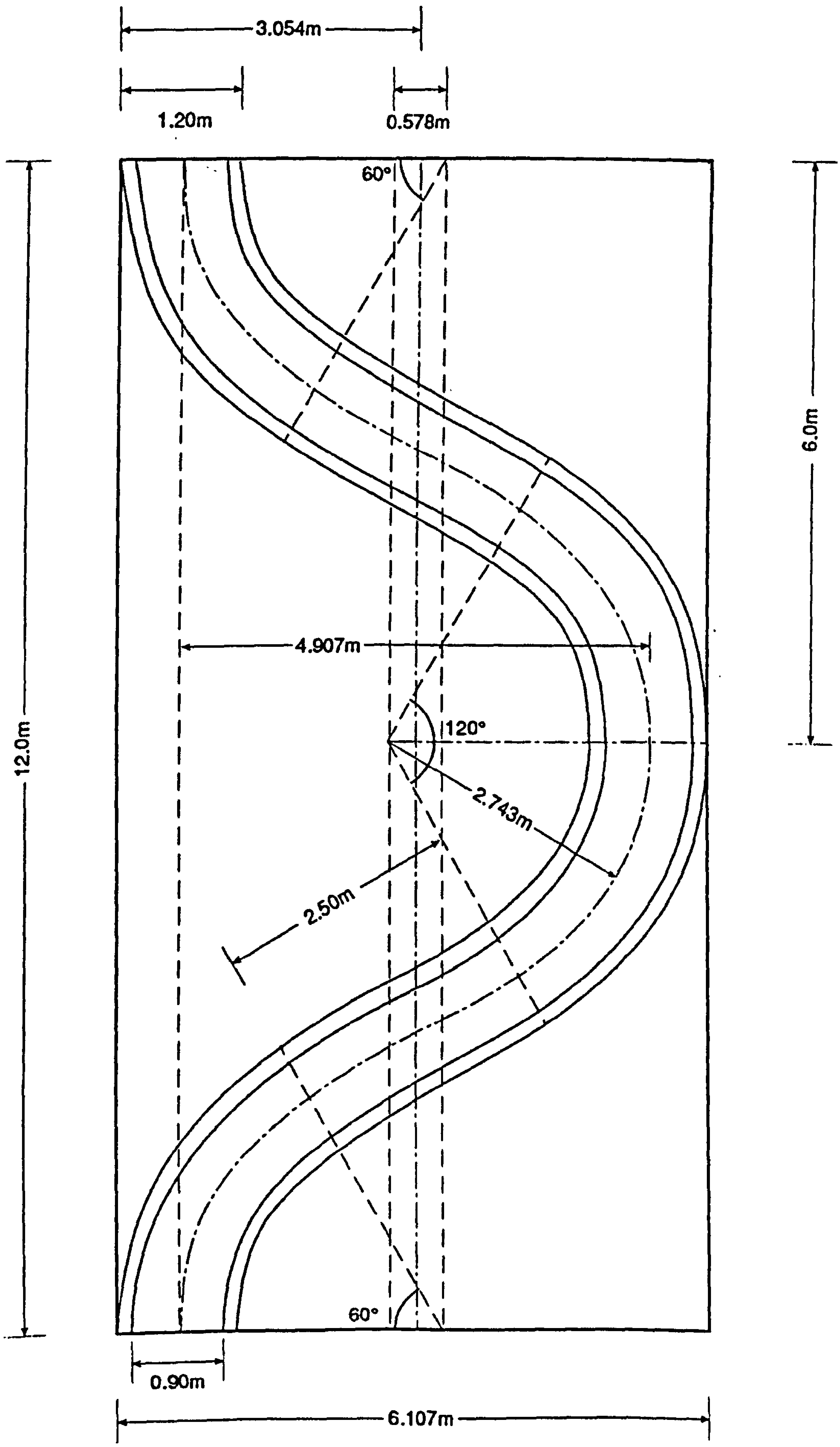


Figure 4.10 - The Flood Channel Facility

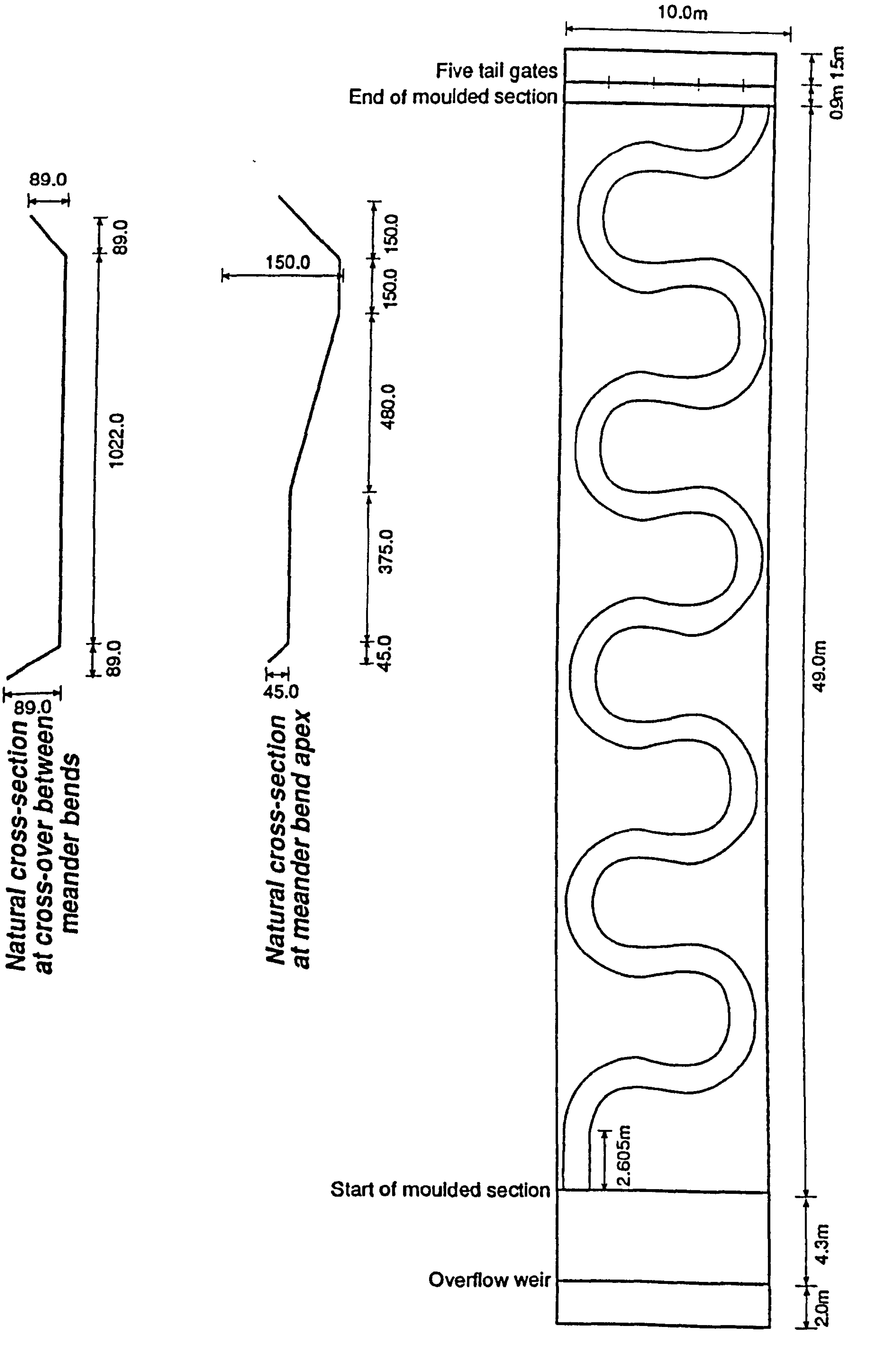


Figure 4.11 - The Flood Channel Facility

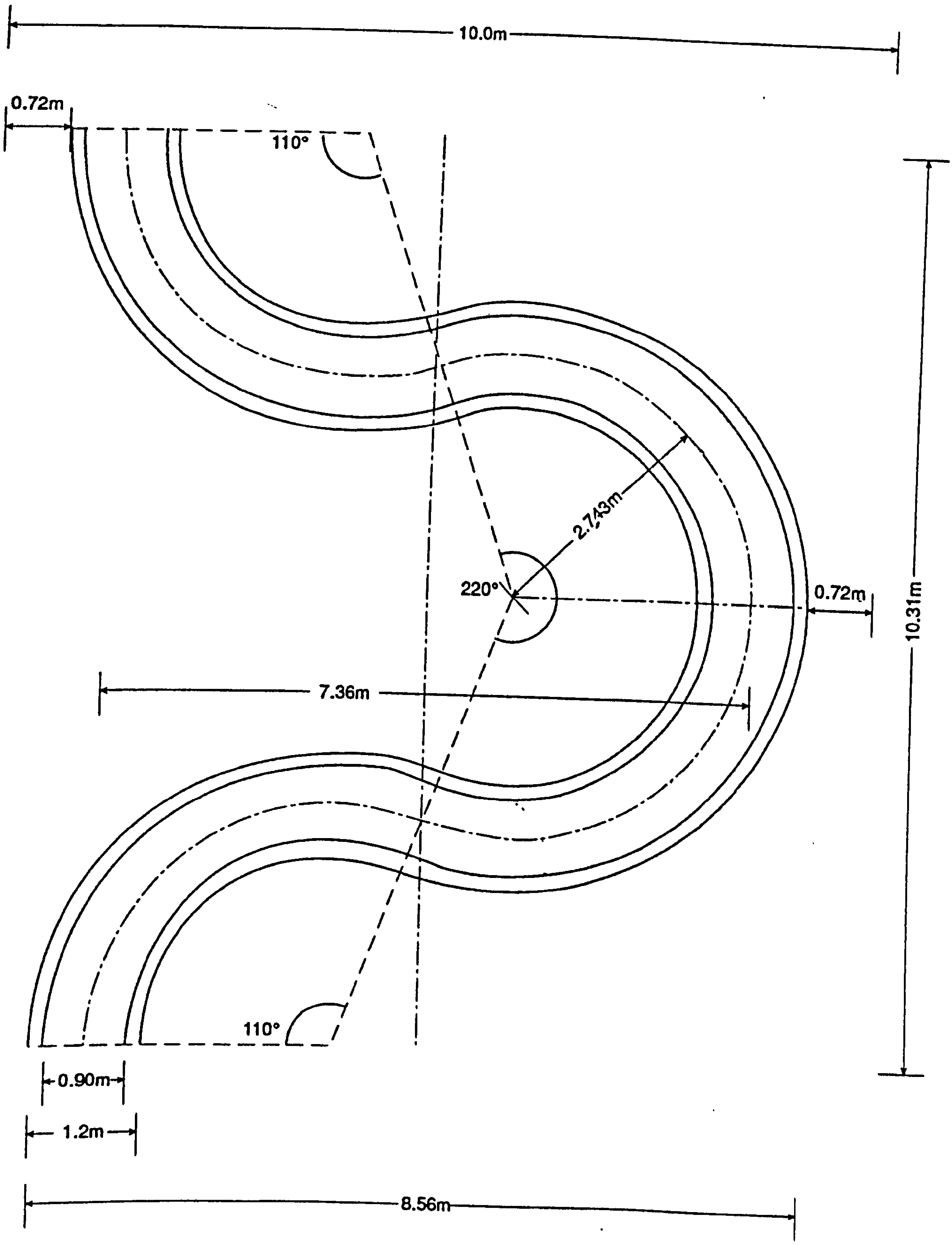


Figure 4.12 - The Flood Channel Facility

Of the geometries investigated it was considered that the quasi-natural would be of most practical interest to the current work. The quasi-natural channels, designed by Lorena (1992) at the University of Glasgow, were based on the average of 17 world rivers. Essentially, the bend apex cross-section of 17 real rivers had been surveyed and an average shape derived. This geometry was then scaled to the dimensions of the facility.

An example of this apex geometry is shown in Figure 4.13. This could be expected to give a reasonable representation of a natural geometry, however, it should be noted that longitudinal variation was not considered in these experiments. Although this was not considered directly, a small degree of variation is observed in the quasi-natural experiments. As the FCF channel moves downstream from a bend apex the cross-sectional shape changes linearly to become of trapezoidal shape, at the cross-over point. Beyond this point the channel changes linearly to again become of quasi-natural shape at the apex of the next bend.

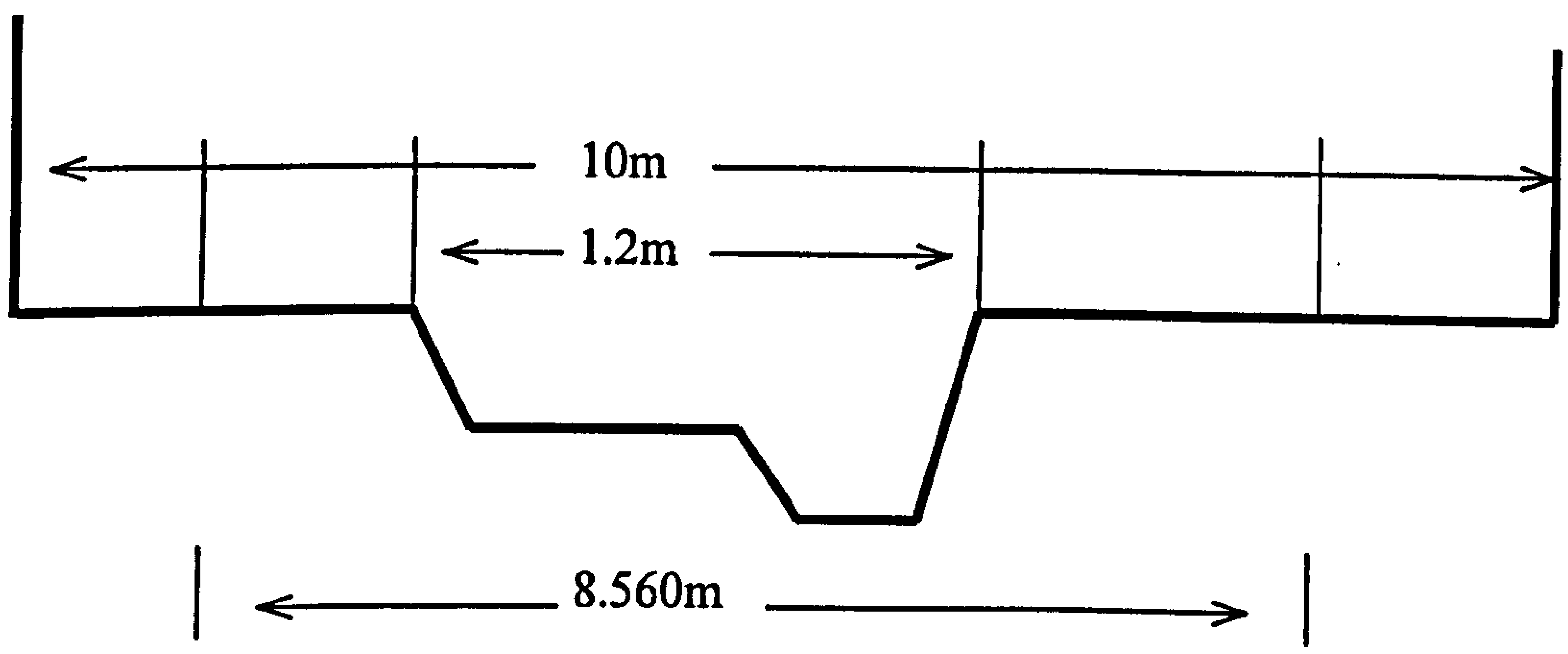


Figure 4.13 - FCF Quasi-Natural Apex Section Geometry 110 Degree Meander

Measurements were taken by suspending instruments from two moveable bridges. The first consisted of an I-beam and the second, a bridge consisting of two rigid trusses. The latter was designed to minimise the sag caused by large loads and to carry the two metre long automated carriage from which most of the measurements were collected. This automated carriage was built to enable movements in the vertical and horizontal directions and to rotate about the vertical axis.

The following measurements were taken

Velocity Data
Water Surface Data
Boundary Shear Stress Data
Turbulence Measurements
Discharge Measurements
Depth Measurement

It can be seen from the variety of readings taken that this has been a comprehensive study of meandering overbank flow. Of particular interest to the current study are the Discharge, Depth, and water surface profile measurements.

4.4.1 Potential Errors In FCF Data

The potential errors in discharge that might have resulted from the pumps are :

- the calibration of the orifice plate meters were only accurate to ±2%
- the calibrations assumed that no air was left in the pumps or the delivery lines
- it is claimed by Lorena(1992) that fluctuations in the water levels in the manometer tubes (especially for higher discharges) made accurate measurement of the difference in heads across the orifice plates very difficult.

The sources of error in water level that may have emanated from the stilling pots are :

- trapped air in the pipes linking them to the tapping points
- it is claimed by Lorena (1992) that the fluctuating water surface levels in the pots made their measurement with the point gauges difficult

These potential errors have been considered when using this data.
The aim of modelling the FCF Series B data has been to reproduce the observed measurements in terms of Stage Discharge Prediction and Water Surface Profile.

The FCF Series B experiments were carried out using the afore-mentioned geometries with additional features such as roughened flood plains, narrow flood plains and walls at each bend apex. A selection of these are used for testing of the newly developed software.

4.4.2 FCF Test Case

As mentioned in the previous section, a review of available data concluded that the Flood Channel Facility was the most useful and readily accessible data set. Full details of the FCF experimental arrangement and the proposed 10 year plan can be found in Knight and Sellin (1987).

It should be noted that the FCF Series B experiments were used in the derivation of the James and Wark Method, however, this does not discount it as a useful test case. Indeed, James and Wark (1992) tested their method against some of the Series B experiments and have published the results. It was intended that this study would verify the new subroutine by replicating these results.

Initial Problems

In attempting to model the FCF experiments using the James and Wark Method the following difficulties arose

1. interpretation of the published FCF data
2. determining correct information from the published J+W data
3. determining a reach average cross-section

The problems associated with 1 and 2 were mainly due to typographical errors or simply trying to interpret the correct information.

The problem in obtaining a reach average section is not so significant for idealised laboratory cases where there is little or no longitudinal variation. However, in a real river where there is extensive longitudinal variation, the surveyed cross-section cannot be directly applied. To the practising Engineer this can be confusing and if some approximate or average section has to be utilised it does indeed appear to be rather impractical. The problem of obtaining reach average geometries for natural rivers will be considered later in chapters 5 and 6.

4.4.3 Flood Channel Facility Series B Testing - Introduction

The previous section discussed the development of the new ISIS subroutines. The following will demonstrate their testing using selected data, from the Flood Channel Facility (FCF) experiments. The tests involve comparisons of Stage Discharge relationship and Water Surface profiles, for a range of hydraulic conditions.

As this approach is computational and certain decisions had to be made in terms of exact model dimensions, it is anticipated that the accuracy of predictions may not exactly match the observed measurements. However, an improvement on the conventional Divided Channel Method accuracy would certainly be expected. It should be remembered that many model users currently apply the Divided Channel Method which is based purely on bed friction. As discussed in Section 2.2.2 this method has been shown to be in error by +30% in some applications. (Myers & Brennan (1990))

During the FCF Series B experiments, two different sinuosities were investigated. Sinuosity is defined as the ratio of the distance along the centre line of the river to the straight line distance. The aim was to examine a mildly meandering experimental set-up with sinuosity 1.374 (60 Degree bend) and a highly sinuous case with a sinuosity of 2.043 (110 Degree bend). The bend angles refer to the cross-over section which is located between two bends. (Refer to Figures 4.09-4.12)

For testing purposes it was decided to use the quasi-natural geometry, which was derived by Lorena (1992).

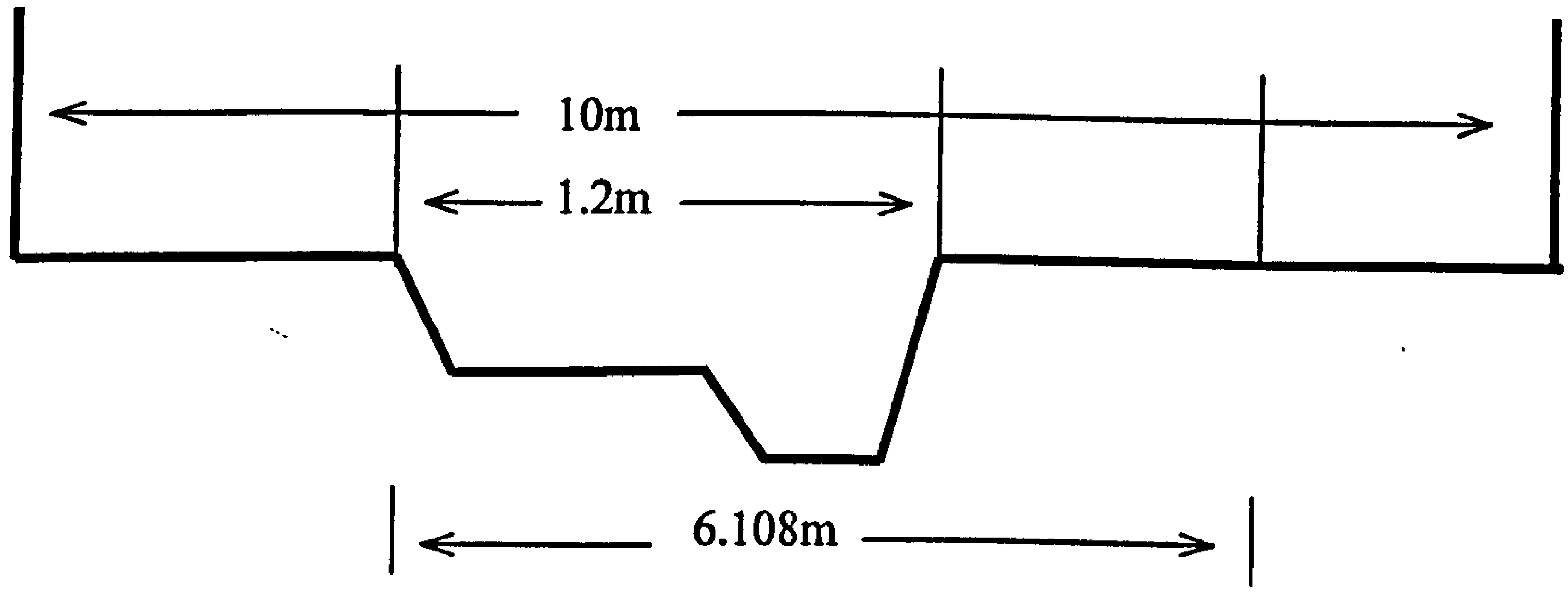


Figure 4.14 - FCF Quasi-Natural Apex Section Geometry 60 Degree Meander

Having decided on this geometry, individual experiments were selected for testing the JMSWK.F subroutine. For reference, the following experiments use the FCF Series B codes e.g B26. This simply implies that the experiment belonged to Series B and was the 26th experiment in the series.

4.4.4 Experiment B26 Stage Discharge Prediction

The FCF B26 experiment was selected to test the James and Wark Method's ability to reproduce the observed stage discharge relationship.

The experiment involved a quasi-natural main channel with smooth flood plains. A numerical model with the geometry shown in Fig 4.14 was set-up with the dimensions given in Table 2. Based on calibration data from the FCF experiment Lorena (1992) both the main channel and flood plain Manning's 'n' values were taken as 0.01.

$W_T = 10m$	$h = 0.150m$	$n_{mc} = 0.01$
$W_2 = 6.108m$	$B^2A = 14.57$	$n_{fp} = 0.01$

Table 4.02 - Model Dimensions

The terms in Table 4.02 are defined as follows, W_T is the total horizontal extent of the cross-section; W_2 is the width of the meander belt; h is the depth of the main channel; B^2A is the aspect ratio of the main channel; n_{mc} and n_{fp} are the respective main channel and flood plain Manning's roughness values.

It was shown by Lorena (1992) that more secondary energy losses would be present with smooth flood plains hence the choice of this application. If rough flood plains were chosen the testing of the James and Wark subroutine would have been less rigorous.

Figure 4.15 compares the stage discharge relationship obtained using JMSWK.F, the conventional Divided Channel Method and the experimental observations. It can be seen that the numerical JMSWK.F scheme over-predicts the observations by 2%. This level of agreement is considered reasonable. As expected, the Divided Channel Method consistently over-predicts discharge by around 15%.

Interestingly, James and Wark (1992) also used this experiment as a test case for their hand calculation procedure and concluded that their method would under predict the observed values by-2.7%. The difference between +2% and -2.7% is surprising as the stage discharge component JMSWK.F is simply a computerised version of the James and Wark hand calculation. Despite extensive testing and a series of hand calculations following the JMSWK.F procedure it has not been possible to reproduce the -2.7% error quoted in James and Wark (1992). It has therefore been concluded that the +2% over prediction is correct and that this degree of accuracy is acceptable in terms of practical river modelling.

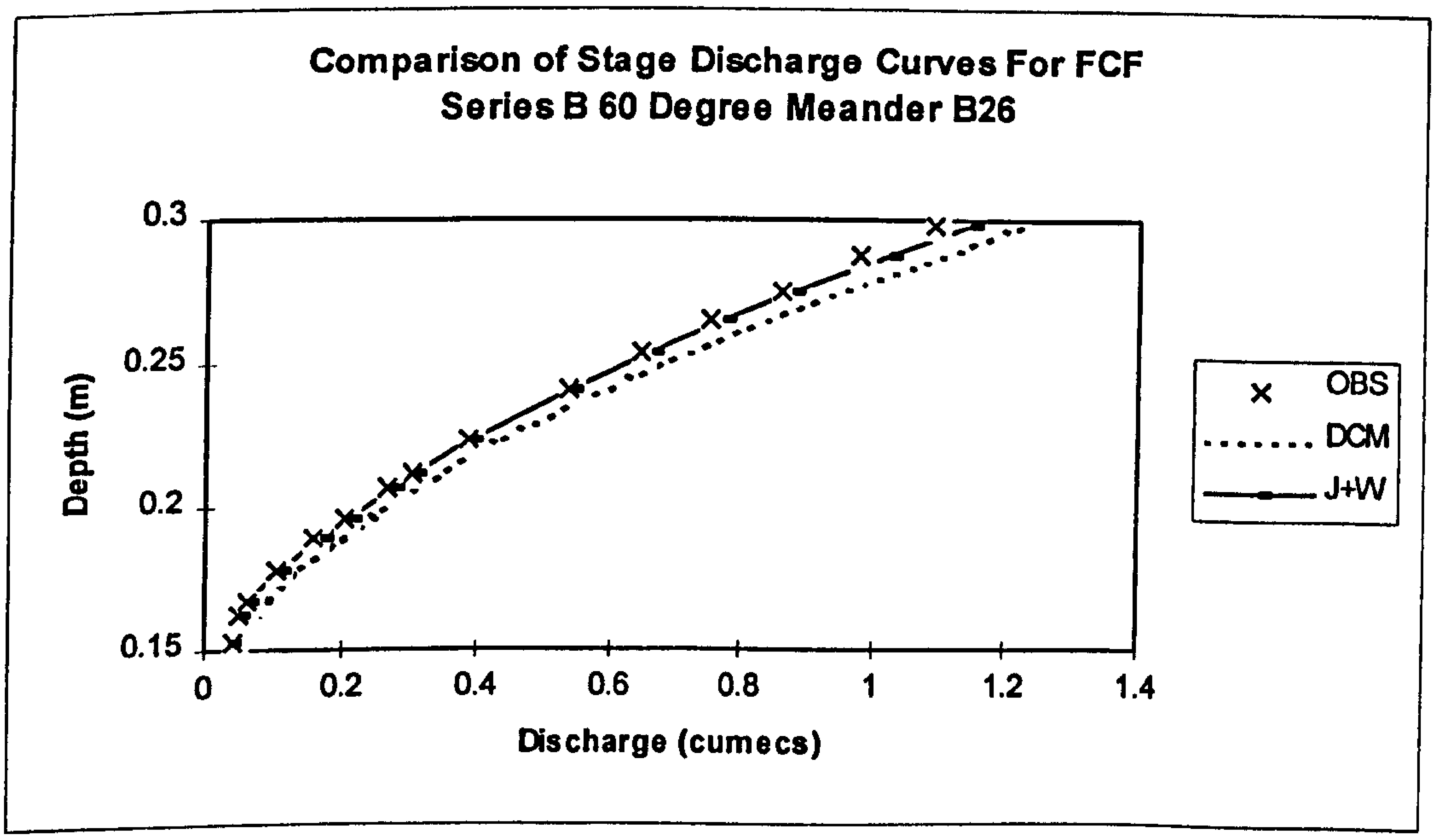


Figure 4.15 - Stage Discharge Curves For Experiment B26

4.4.5 Experiment B39 Stage Discharge Relationship

Experiment B39 refers to the 110 Degree Meander geometry with a sinuosity of 2.043. This was the only other geometry that was tested during the series B experiments. As a consequence of the increased sinuosity the meander bends take up more of the flood plain and also there is an additional meander wavelength longitudinally, compared with experiment B26. The experimental arrangement is shown in Figure 4.11 and a sample cross-section is shown in Figure 4.16.

This test was again aimed at reproducing the observed stage discharge relationship from this more sinuous geometry. It should be noted that the sinuosity of 2.043 is high when compared with what might be expected in the field.

$W_T = 10m$	$h = 0.150m$	$n_{mc} = 0.01$
$W_2 = 8.56m$	$B^2A = 14.64$	$n_{fp} = 0.01$

Table 4.03 - B39 Model Dimensions

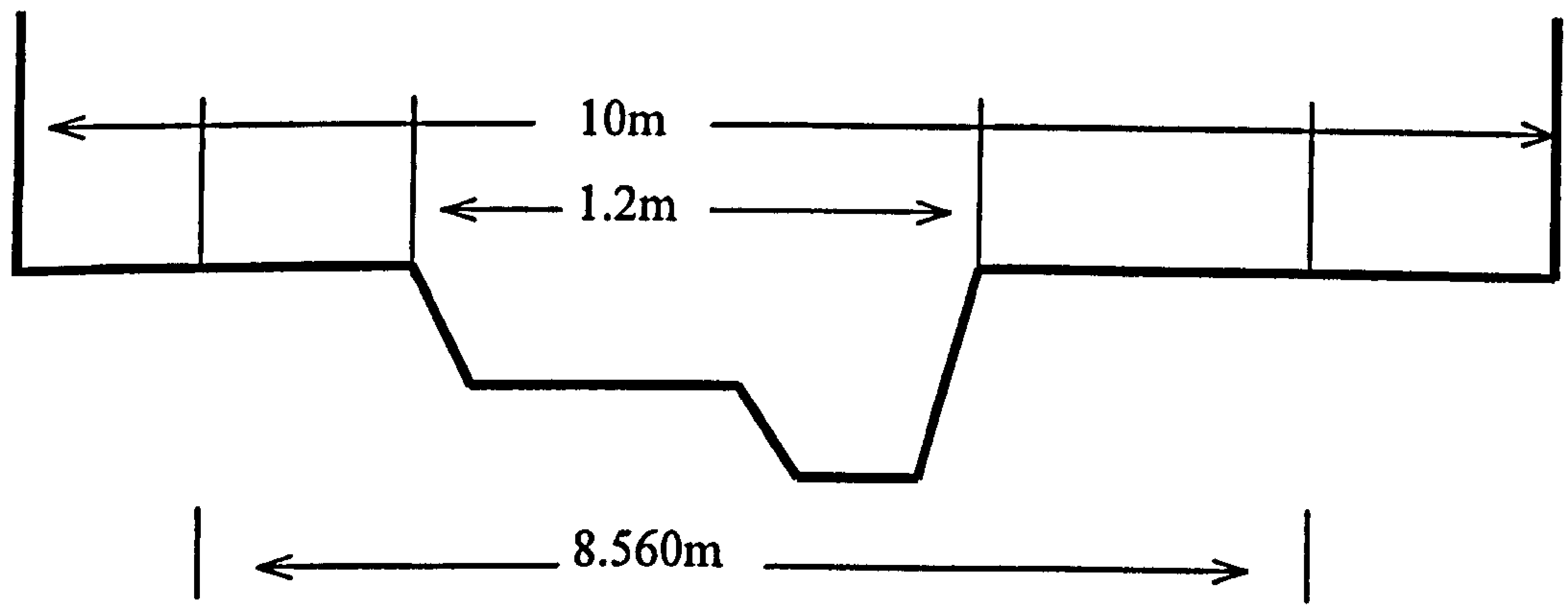


Figure 4.16 - FCF Pseudo-Natural Apex Section Geometry 110 Degree Meander

In order to model this experimental arrangement an ISIS model was constructed consisting of six identical cross-sections, each with the geometry shown in Figure 4.16, the flood plain slope being 1 in 979. It should be noted, however, that for a stage discharge analysis only a single cross-section is required.

As the cross-sectional properties are calculated before any hydrodynamic calculation, a full model is not required at this stage.

Figure 8 compares the stage discharge relationship, again obtained using JMSWK.F, with the experimental observations. The agreement is good with a maximum over prediction of +4%.

This degree of accuracy is acceptable in terms of practical river modelling.

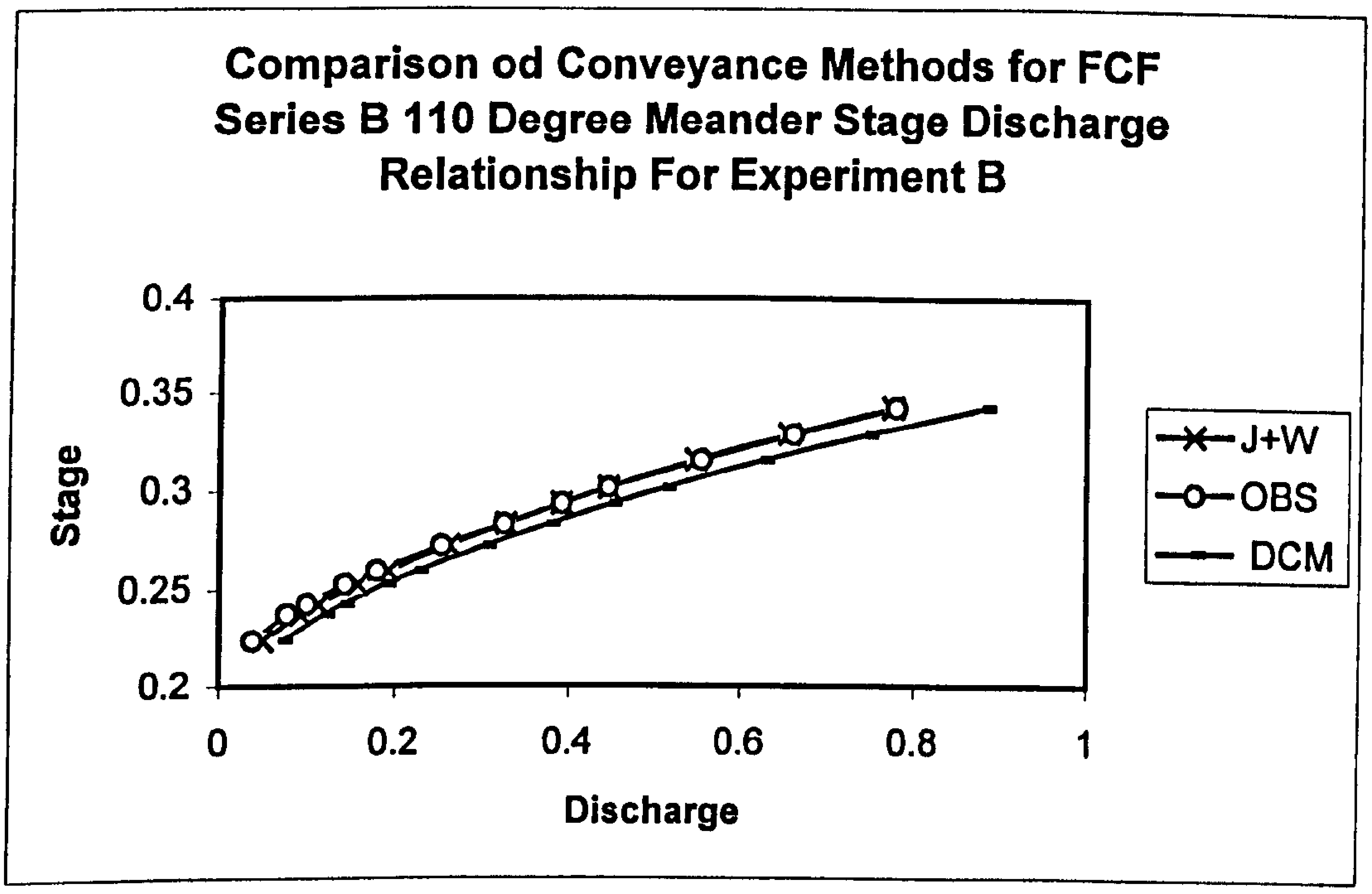


Figure 4.17 - Stage Discharge Curve For Experiment B39

4.4.6 Discussion of Stage Discharge Tests B26 and B39

James and Wark (1992) also used experiments B26 and B39 as test cases for their hand calculation procedure. They concluded that the method would under predict the observed discharge by -2.7% and -3.8% for B26 and B39 respectively. This differs from the computerised version of the method where, as reported previously, the differences were +2% and +4%. The discrepancy in results may be due to errors in the interpretation of the published James and Wark (1992) method, or errors in the published formulae or errors in the computer coding.

Extensive effort has been directed to check the code and confirming the computer results by a series of hand-calculations. These confirmed the coding to be correct. The problem must lie therefore in the interpretation of the published method or errors in published formulae.

It is interesting to note, that a previous application of the James and Wark Method to experiments B26 and B39, by Mcleod (1997), also produced results that were not in agreement with the published James and Wark (1992) results. Mcleod (1997) found that the discharge was underpredicted by -2.3% and -9.5% in B26 and B39 respectively. This reinforces the suggestion that the published method is open to different interpretation by individual users.

The ISIS analysis could only match the James and Wark (1992) prediction by increasing the measured Manning's 'n' value of 0.01. This could not be justified as the value of 0.01 has been published and adopted in other studies.

A potential source of the discrepancy between the ISIS study and the published James and Wark result is the value of bankfull area. This study has used the dimensions that were published by James and Wark (1992) as it was intended to reproduce their results. These dimensions are in fact marginally bigger than those used in the FCF. For example, the depth of main channel as built ranged from 146mm to 150mm. This study has used a constant depth of 150mm and it is plausible that this increased area is responsible for the small overpredictions in discharge. However, it has to be assumed that James and Wark (1992) also used a depth of 150mm in their study which would refute this argument.
It is reasonable to suggest that the disagreement in results are not significant and for practical application an error of +2-4%, in discharge, is acceptable.

It should also be remembered that many model users currently apply the Divided Channel Method which is based purely on bed friction. This method has been shown to be in error by +30% in some applications. Therefore, in relative terms the accuracy demonstrated in this section when using the James and Wark Method is reasonably good.

It has recently been discovered that there is indeed a discrepancy in the published value of Manning's 'n'. This study used a constant 'n' value of 0.010 which was indicated to be the actual value by Lorena (1992) and Crowder, Chen and Falconer (1997). However, it has been established that the true value of 'n' is actually more close to 0.0105 for the main channel and marginally higher on the flood plain. This value of Manning's 'n' is a minimum of 5% higher than that used by this author. This is undoubtedly the source of error in this study as an increase in roughness would further retard flow and cause a reduction in the predicted discharge.

The location of this discrepancy further reinforces the accuracy of the newly developed sub-routine. If the Manning's 'n' value is increased from 0.010 to 0.0105 then an accuracy in discharge of –1.6% is achieved which should be compared with the –2% suggested by James and Wark (1992) and –2.5 % by Mcleod (1998).

4.5 Water Level Prediction

The practising river engineer is regularly involved in flood studies. The outcome of such a study is the prediction of flood water levels at a series of locations, which would be used to design flood defences. In practical terms, this information is arguably more important than predicting a stage discharge relationship at a single cross-section. As a consequence, it is necessary to assess the enhanced ISIS subroutine's ability to perform as part of a full hydrodynamic model calculation.

As the FCF experiments used uniform flow as their hydraulic conditions then if ISIS were to replicate the observed measurements exactly the downstream boundary depth should be predicted at each cross-section.
The aim of testing these experiments being to reproduce the downstream boundary condition depth at each of the model cross-sections. For example, if the downstream boundary was a depth of 200mm then the hope was to observe 200mm at each model cross-section. The maximum difference, if any, would occur at Section 1 as it is furthest away from the controlling downstream boundary condition.

4.5.1 Experiment B26 Water Level Prediction

This experimental arrangement involved the quasi-natural geometry with the 60 Degree meander bend. In order to predict the water surface profile of the FCF a steady flow ISIS model was constructed using five cross-sections to describe the flume geometry. The upstream boundary condition was the measured experimental inflow and the downstream boundary the corresponding observed stage. The five section model had the quasi-natural geometry with a surface roughness of 0.01 in the main channel and the flood plains. The flood plain slope was 1/1004.

Figure 4.18 shows comparisons of computed and measured water levels at the upstream numerical model section, for all of the discharges used in the experimental programme.

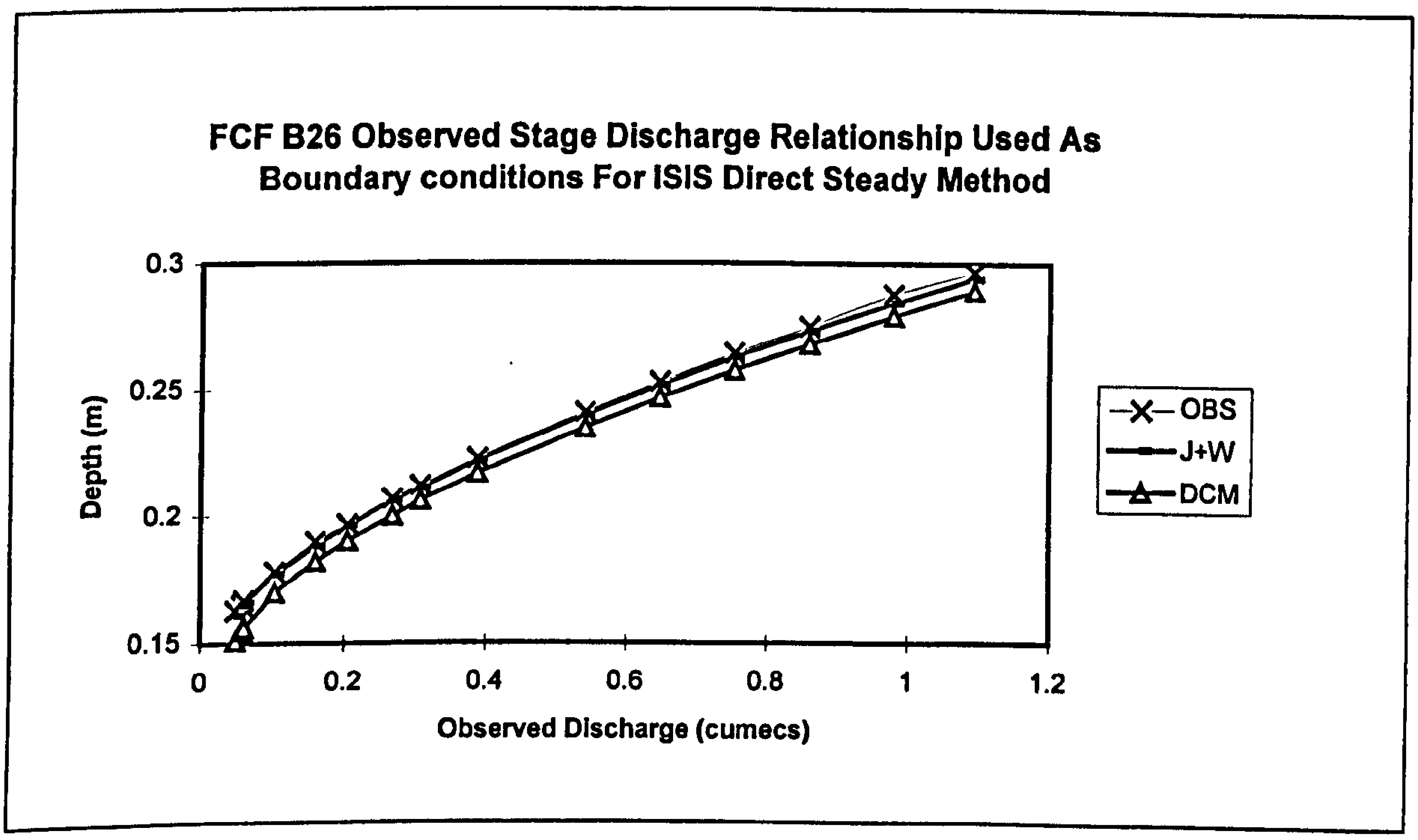


Figure 4.18 - Observed and Predicted Water Surface Profile

There is an almost perfect match over the majority of the depth range, with a slight under prediction in water level at depths above 0.274m. This is expected as the stage-discharge relationship is over-predicting at these depths.

When this flow over prediction is converted to conveyance using equation 4.03 a conveyance over prediction results, consequently a lower than observed water level results. On average the James and Wark Method under predicts the observed water level by 2mm. The Divided Channel Method under predicts the observed water level by, on average, 8mm.

This result clearly shows the improvement that can be obtained when using the James and Wark Method to calculate conveyance. The higher water level prediction is due to additional energy losses associated with overbank flow being correctly accounted for. The general agreement of the J+W method in Figure 4.18 is considered to be acceptable.

4.5.2 Experiment B39 Water Surface Profile

This experiment involved the quasi-natural geometry with the 110 Degree meander bend. A model was set up containing 6 cross-sections with a constant bed slope of 1 in 2000. In this case six cross-sections were required as there is an additional meander wavelength relative to B26. (See Fig. 4.10)

As with the B26 test the upstream and downstream boundary conditions were the constant observed inflow and downstream water level respectively. The dimensions are the same as that of experiment B39 which are shown in Table 3.

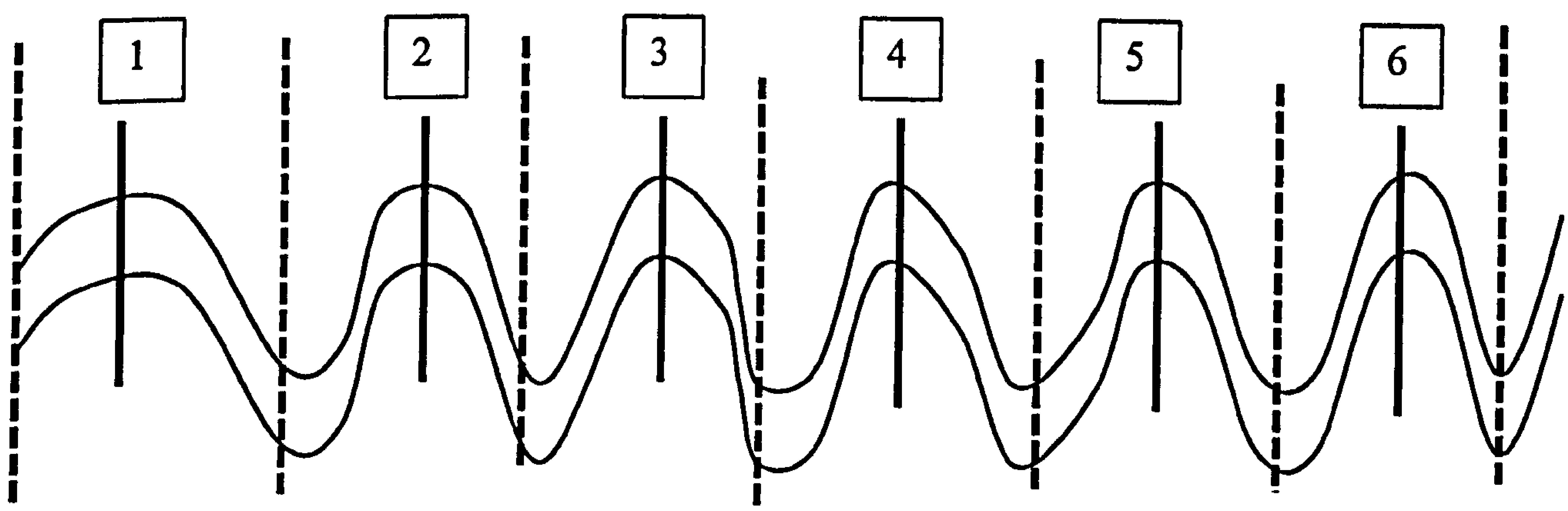


Figure 4.19 - Six Cross-Section Model with Representative Reaches

Figure 4.19 illustrates the model that has been set up to test this experiment and consists of six identical cross-sections. The distance between the cross-sections (ΔX) is equal to 16.49m and the global Manning's 'n' value is set at 0.01.

It should be noted that the resulting water level prediction would be expected to be marginally less than the observed. This is a direct result of the stage discharge relationship for this geometry (see B39) over-predicting by +4%.

If there is an over prediction in discharge then there will be a corresponding under-prediction of water level.

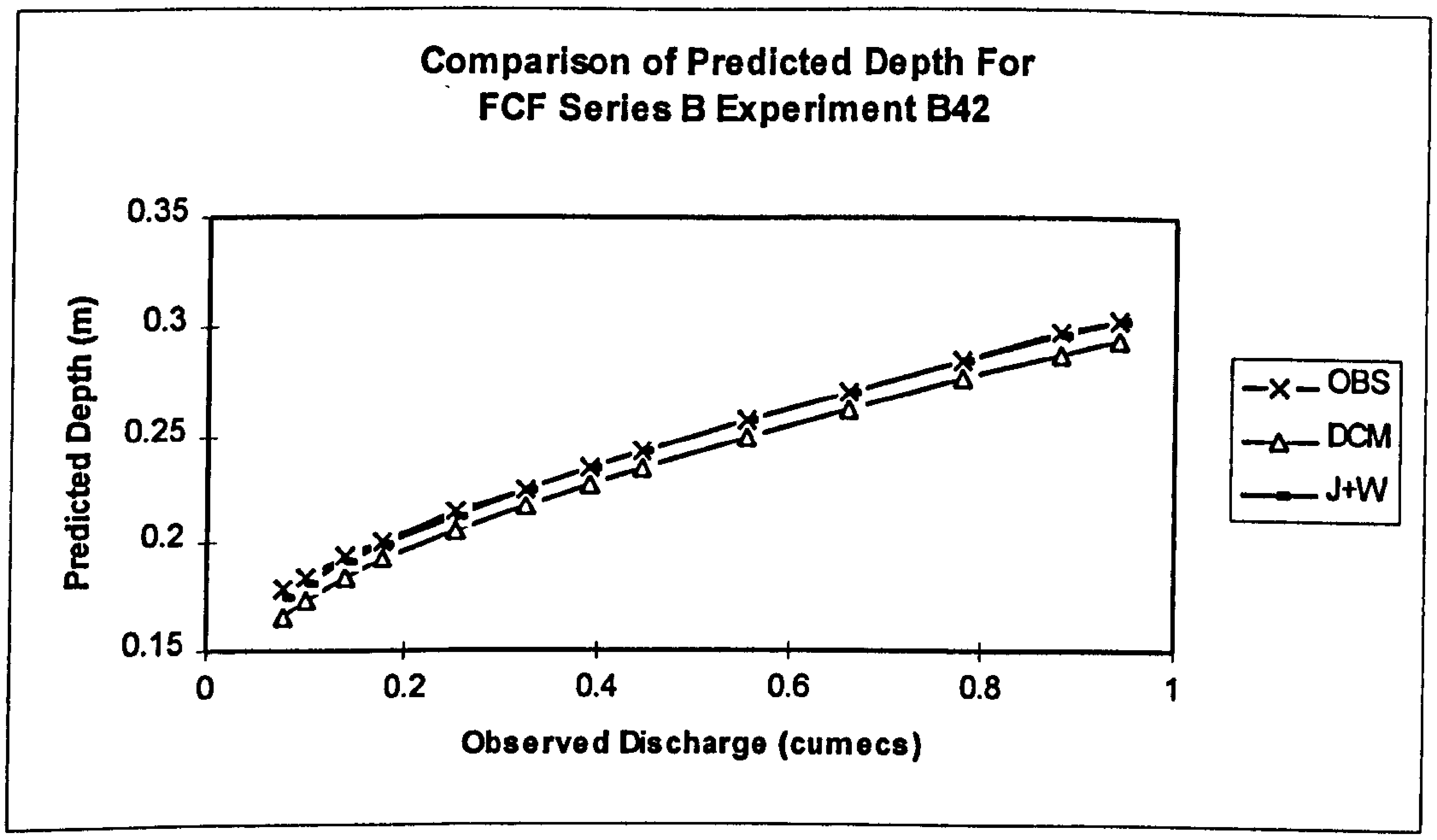


Figure 4.20 - Comparison of Water Level Predictions

Figure 4.20 shows the predicted depth compared with both the observed and bed friction only (DCM) predictions. It can be seen that the Divided Channel Method significantly under-predicts water level and that the James and Wark method gives better agreement with the observations. This is a direct result of the James and Wark Method accounting for energy losses over and above bed friction.

This result demonstrates that the James and Wark Method is providing sensible and accurate predictions. Using the James and Wark Method to calculate conveyance for one-dimensional river models results in better predictions of water level when applied to the Flood Channel Facility experiments.

4.5.3 Experiment B34 Water Surface Profile

Having restricted the testing to experiments which had smooth flood plains and obtained reasonable results, it is necessary to test experiments with roughened flood plains. The geometry for B34 was identical to B29, however, in this case the flood plains have been roughened using dowel rods. (see Figure 4.21)

The observed stage discharge relationship has been used as boundary conditions and the appropriate 'n' value calculated from a relationship proposed by Lambert and Sellin (1996) that related the Darcy-Weisbach friction factor to Manning's 'n', for the Dowel rods on the Flood Channel Facility. A graph by Lorena (1992) of roughness against relative depth illustrates that for smooth flood plains 'n' is approximately constant at all depths whereas, with rough flood plains, the 'n' value exhibits significant variation with depth. (See Fig. 4.22)

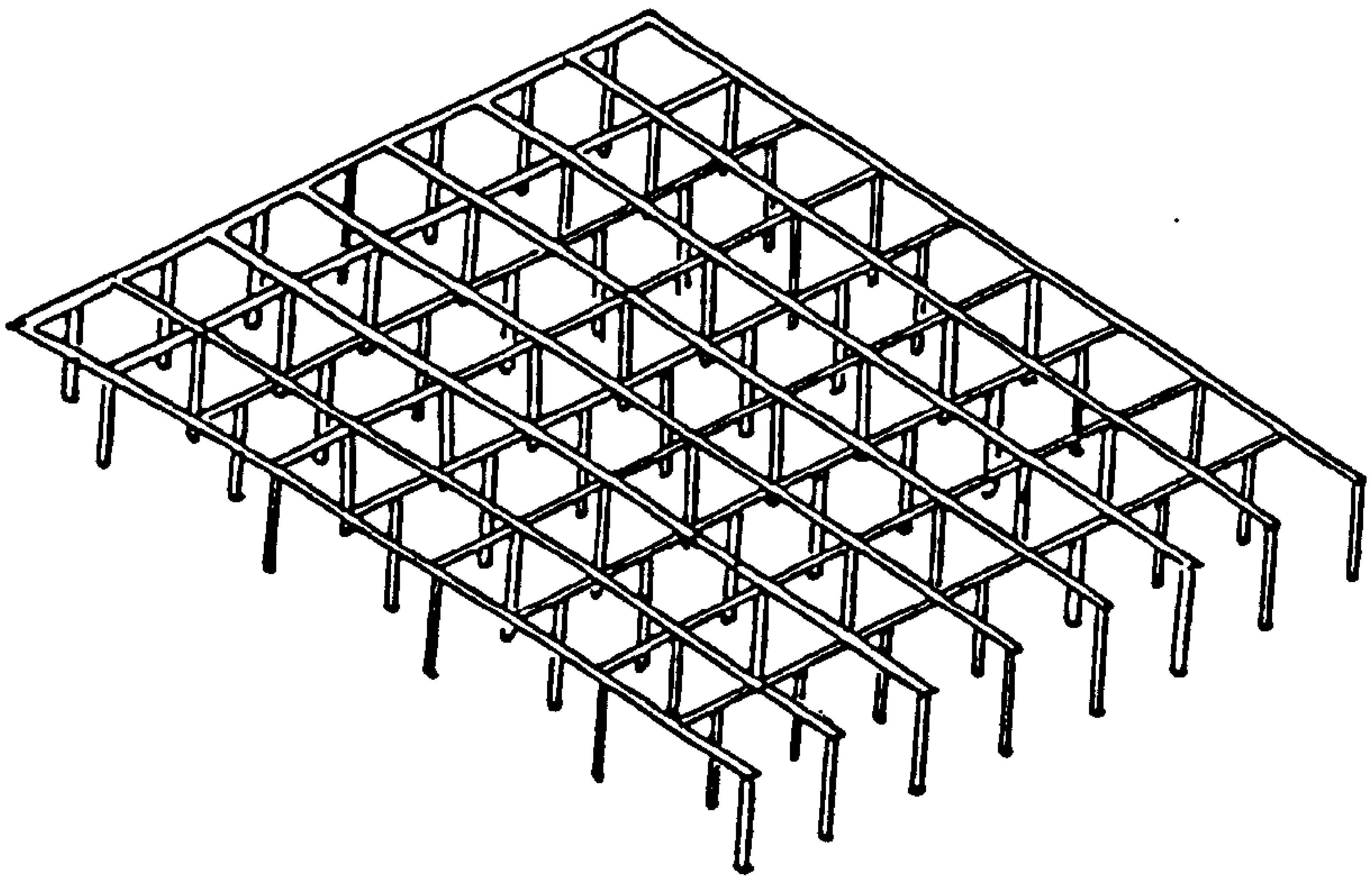


Figure 4.21 - Dowel Rod Roughness Frames

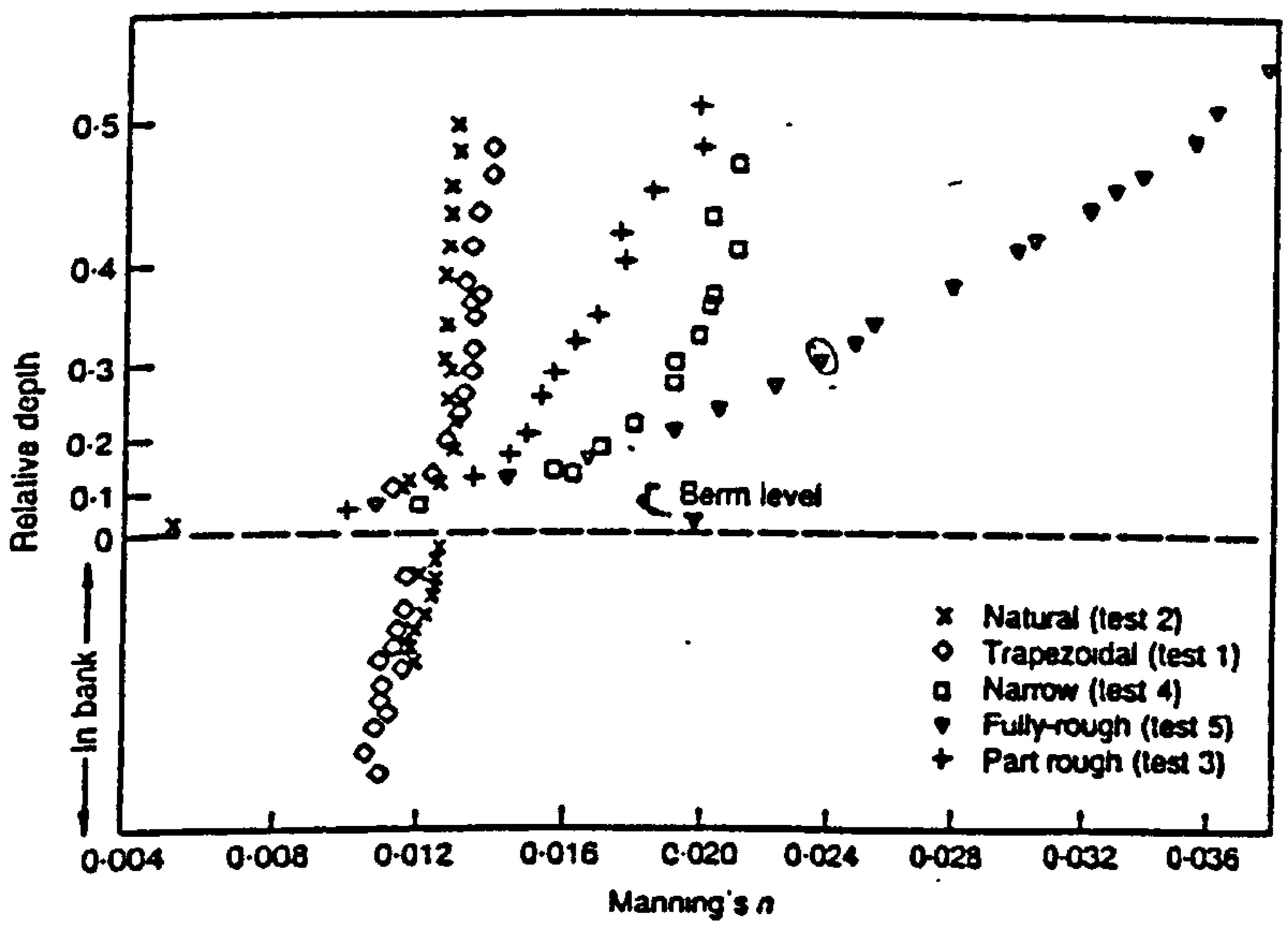


Figure 4.22 - Manning's 'n' Plotted Against Relative Depth (60 Degree Channel)

Figure 4.23 illustrates the predicted water surface profile using both the James and Wark Method and the Divided Channel Method. It can be seen that there are "non-smooth" points on the J+W and DCM curves. This is presumably due to inaccuracies in the estimate of 'n' obtained from the relationship proposed by Lambert and Sellin (1996).

The James and Wark Method under predicts the observed water levels by 1mm on average while the Divided Channel Method under predicts the water levels by 12mm on average. In general terms, however, Figure 4.23 again shows the deficiencies in the DCM. The ability of the J+W method to account for the additional energy losses associated with over bank flow is clear as predictions made using it follow the observed results.

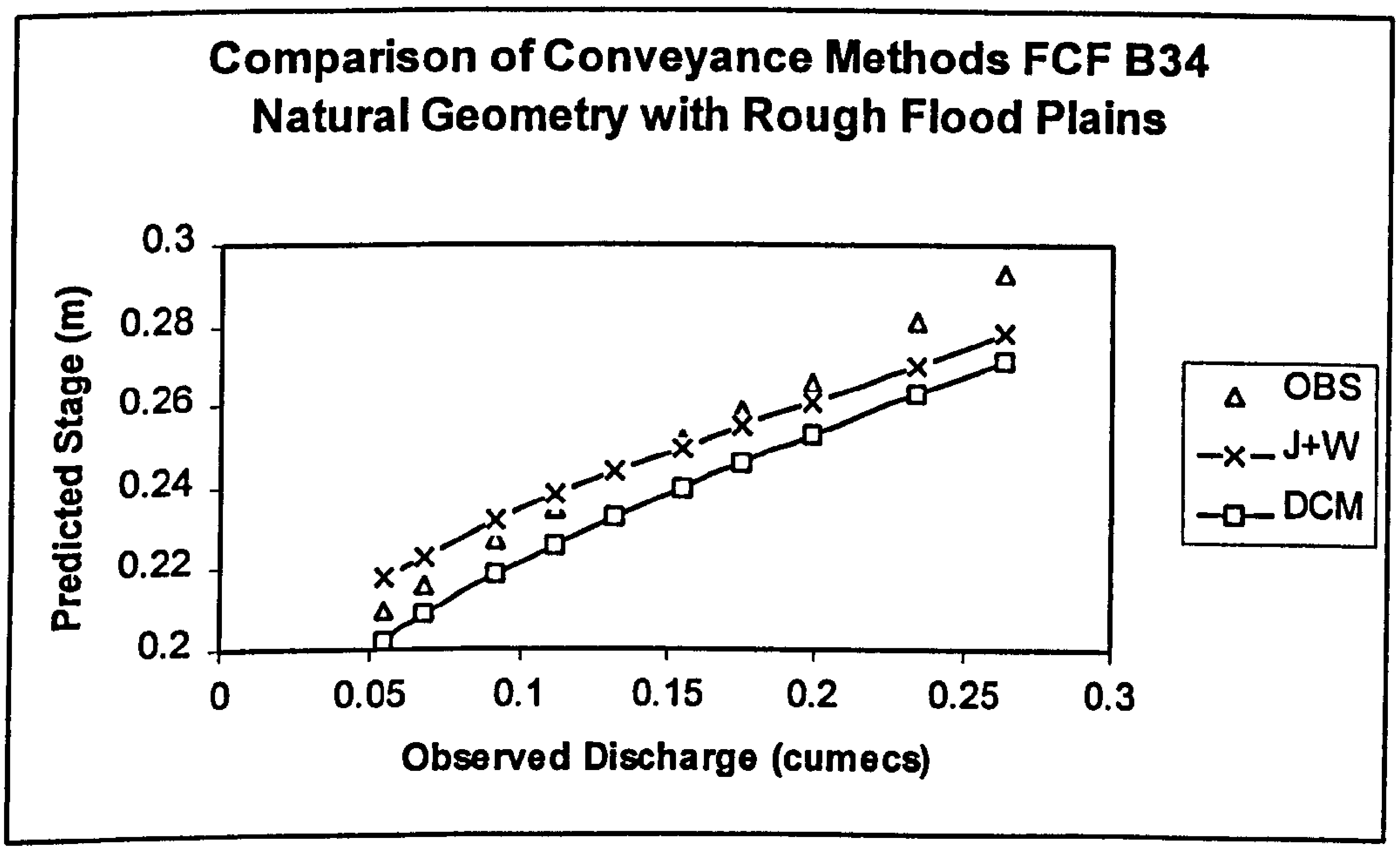


Figure 4.23 - Water Surface profiles Using Various Conveyance Methods

Despite there being an error in the 'n' value it clearly shows the difference in James and Wark predictions and divided channel method predictions. The error will be the same for both methods. The correct 'n' values are required to correctly model this experiment. This is a similar finding to that of Lambert and Sellin (2000).

4.5.4 Discussion of Water Surface Profile Tests For B26 and B39

It has been demonstrated through the previous tests that by using the James and Wark Method a full hydrodynamic calculation can predict water levels that are in close agreement with observed measurements for the FCF experiments. The stage discharge relationship over prediction for both B26 and B39 translates into a marginal under prediction in water level.

Again, it should be noted that it has recently been established that the value of Manning's 'n' used in this study was incorrect. Essentially the value of 'n' that was used should be increased by 20%. This would have the effect of raising the predicted water levels and consequently would be in closer agreement with the observed measurements.

Although the James and Wark Method under predicted the observed values by 2mm, in the case of B26, it was a considerable improvement on the existing Divided Channel Method which under predicted the observed water level by 8mm on average.

This may not seem greatly significant, however, when this discrepancy is scaled up to field dimensions the discrepancy may be very significant. For example, if the 50m long FCF flume were scaled up by a factor of 100 and the differences in water level prediction scaled up by the same factor, the James and Wark Method theoretically would predict a water level 0.6m higher than the Divided Channel Method.

It can also be seen from these tests that the James and Wark Method is a significant improvement on the conventional Divided Channel Method. This has interesting implications in terms of model calibration. The use of "lumped resistance coefficients" such as Manning's 'n' are common in engineering practise. This being where all potential sources of energy loss are added to the value of 'n', resulting in an inflated and unrealistic value.

As the James and Wark Method is directly accounting for additional energy losses it consequently predicts higher water levels which may result in better calibration. Essentially, this would mean the end of the lumped coefficient and the value of 'n' would only describe the bed roughness as intended.

However, it would perhaps be naive to expect this to occur in a natural river reach with extensive longitudinal and lateral variation. It is suspected that some calibration would still be required with the James and Wark Method in order to match with observed measurements.

It is concluded that the subroutine JMSWK.F is performing satisfactorily and shows an improvement over the conventional Divided Channel Method.

4.6 Testing of The Ackers Method Subroutine

To examine the correctness of the Ackers Method subroutine a limited selection of tests were carried out. The tests involved modelling theoretical and experimental data to predict stage discharge relationships and water surface profiles.

It should be noted that, in the long term, potential use of the Ackers Method within a one-dimensional river model may be limited in that it is designed for "straight" reaches. This study has deemed the Ackers Method to be less practically useful than, say, the James and Wark Method and this is reflected in the amount of testing that has been carried out.

The aim of this section was to confirm that the coding of the subroutine was correct and that it could be readily applied.

4.6.1 Test 1 - Hypothetical Test For Ackers Stage Discharge Relationship

The purpose of this test was to observe the difference in stage discharge prediction when using the Ackers method rather than the Divided Channel Method. It would be expected that the Ackers Method would predict smaller discharges, for each water level, than the Divided Channel Method as additional energy losses are being accounted for. Wark, James and Ackers (1994) provided an example calculation of the Ackers Method using the geometry shown in Figure 4.24.

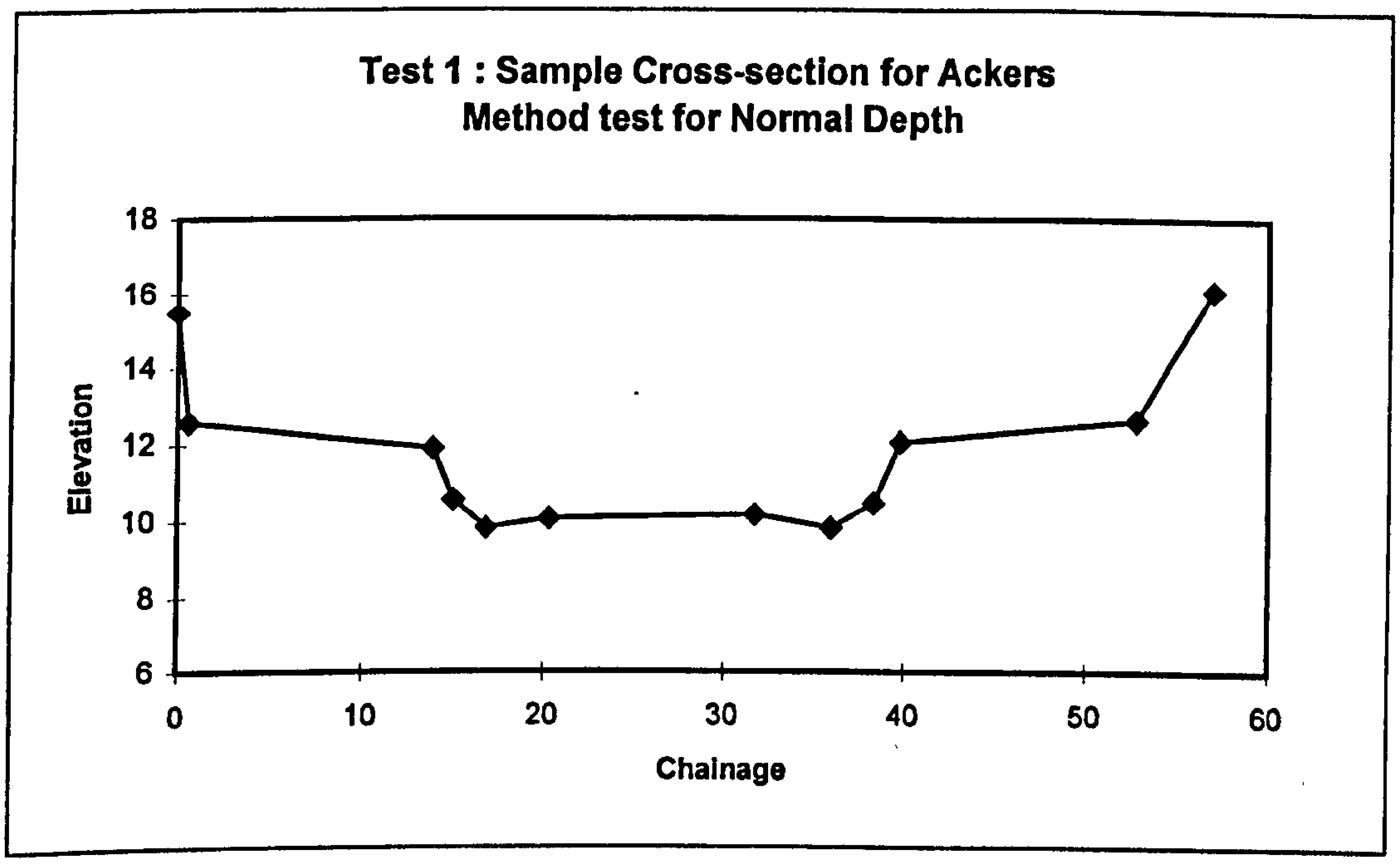


Figure 4.24 - Cross-section used for Ackers Hypothetical Test Case

The coding of the Ackers subroutine has been checked by reproducing the example calculation. The new ISIS subroutine matches every calculation for the given water level and extends the procedure to a range of higher and lower overbank water levels. The result being that for any user defined cross-section a stage discharge relationship, calculated using the Ackers Method, may be obtained.

Figure 4.25 shows the difference in stage discharge predictions when using the existing ISIS divided channel method and Ackers. It illustrates clearly the significant differences that are calculated from each method. Compared to the Ackers curve the divided channel method over predicts discharge by 13% on average.

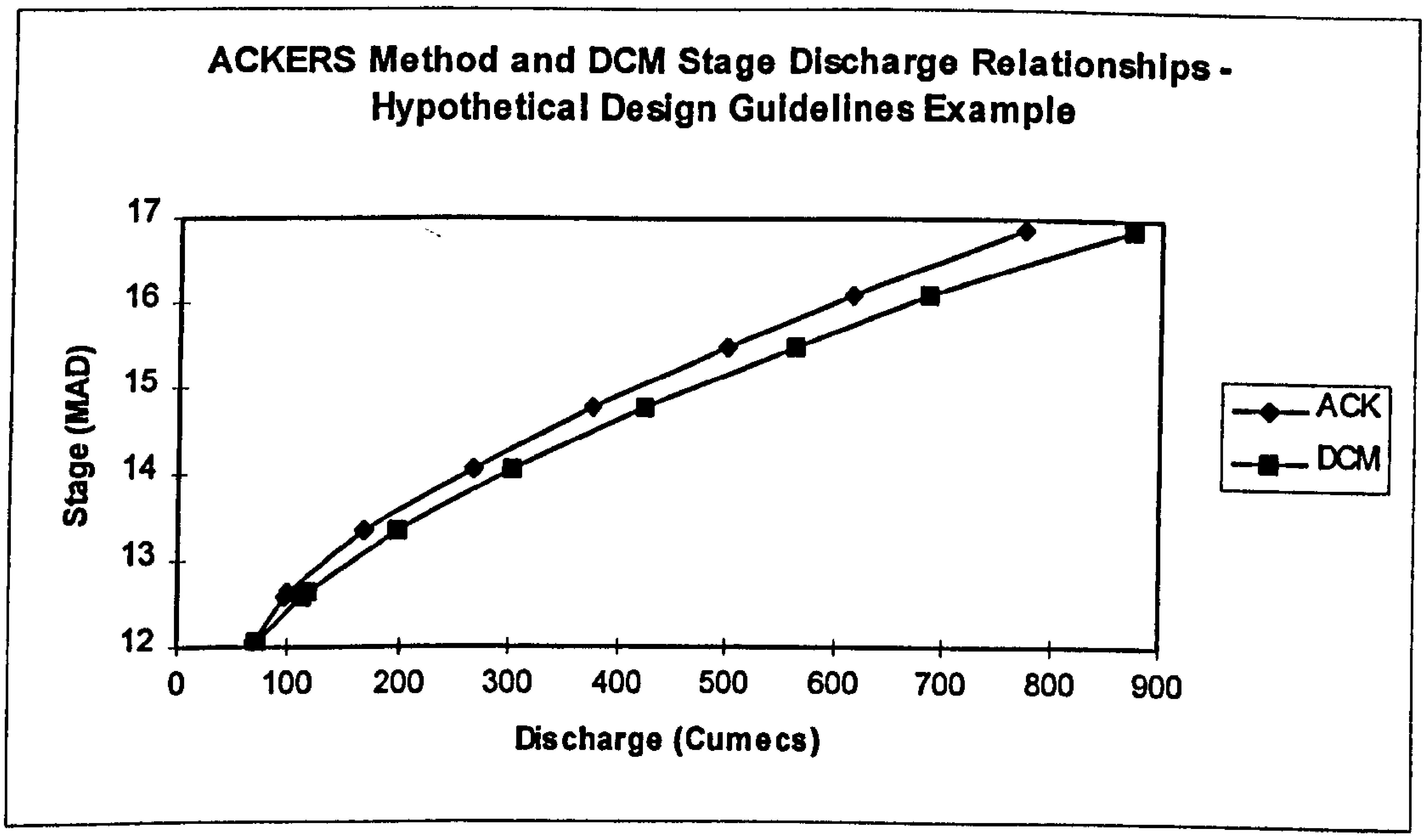


Figure 4.25 - Comparison Of Stage Discharge Relationships - Hypothetical Test 1

Figure 4.25 shows the expected result where for each water level encountered the Divided Channel Method over-predicts the Discharge.

4.6.2 Test 2 - Hypothetical Test Ackers Method Prediction of Normal Depth

The purpose of Test 2 was to verify the Ackers methods ability to predict normal depth for a quasi-natural reach of river. The data used for this example is again similar to the worked example contained in Wark, James and Ackers (1994).

The philosophy being that if the Ackers Method is performing accurately then it should predict a constant depth at each model section. An ISIS model was set up that consisted of 15 consecutive sections on a constant longitudinal bed slope of 1 in 2000m with the geometry shown in Figure 4.26.

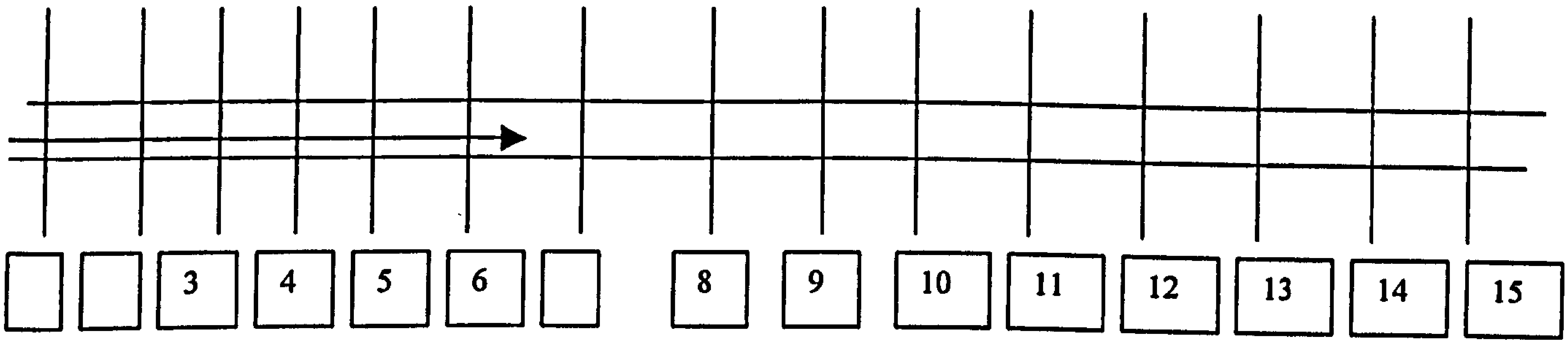


Figure 4.26 Set-up of 15 Section Ackers Test Model

The total longitudinal length of the model was 2800m with cross-sections at intervals of 200m. The boundary conditions were a steady inflow of 316.70m³/s and a known water level of 13.47m at the downstream end. The model was then run with the Ackers method option utilised and the following results were obtained.

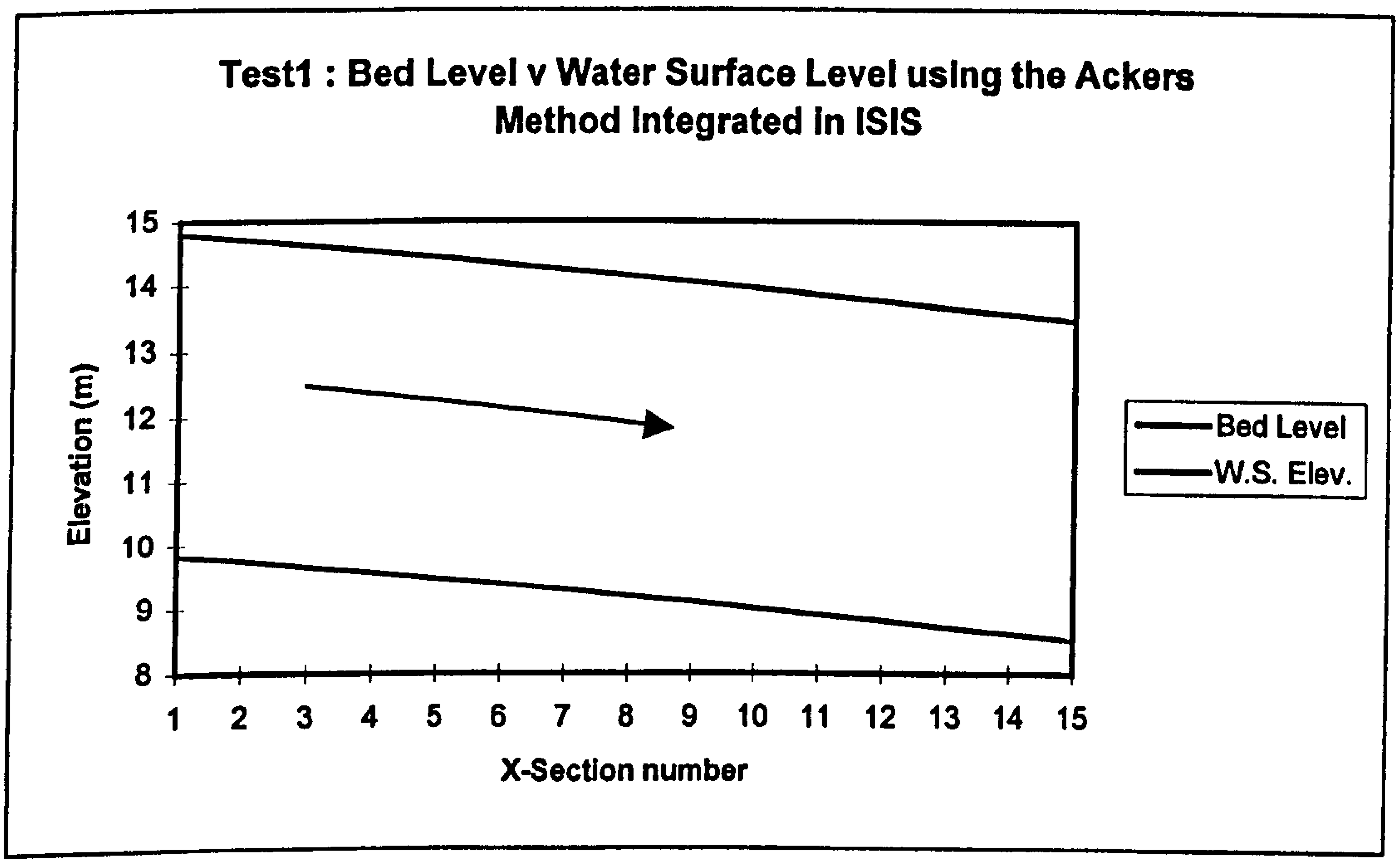


Figure 4.27 - Prediction of normal Depth For Hypothetical Test Case

The above Figure shows the expected normal depth profile where the bed slope is parallel to the water surface slope. This confirms the correct coding of the new subroutine.

This test case is quasi-natural in set-up as can be seen from the shape and number of cross-sections. The results clearly indicate that the ISIS Ackers method will predict normal depth for a 2800m long river model with only a maximum error in water level of 0.01m.

Sect. No.	Q	Bed Level	W.S. Elev.	Depth
1	316.7	9.85	14.803	4.953
2	316.7	9.76	14.708	4.948
3	316.7	9.66	14.613	4.953
4	316.7	9.57	14.518	4.948
5	316.7	9.47	14.424	4.954
6	316.7	9.38	14.329	4.949
7	316.7	9.29	14.234	4.944
8	316.7	9.19	14.139	4.949
9	316.7	9.1	14.043	4.943
10	316.7	9	13.949	4.949
11	316.7	8.91	13.853	4.943
12	316.7	8.82	13.757	4.937
13	316.7	8.72	13.662	4.942
14	316.7	8.63	13.565	4.935
15	316.7	8.53	13.47	4.94

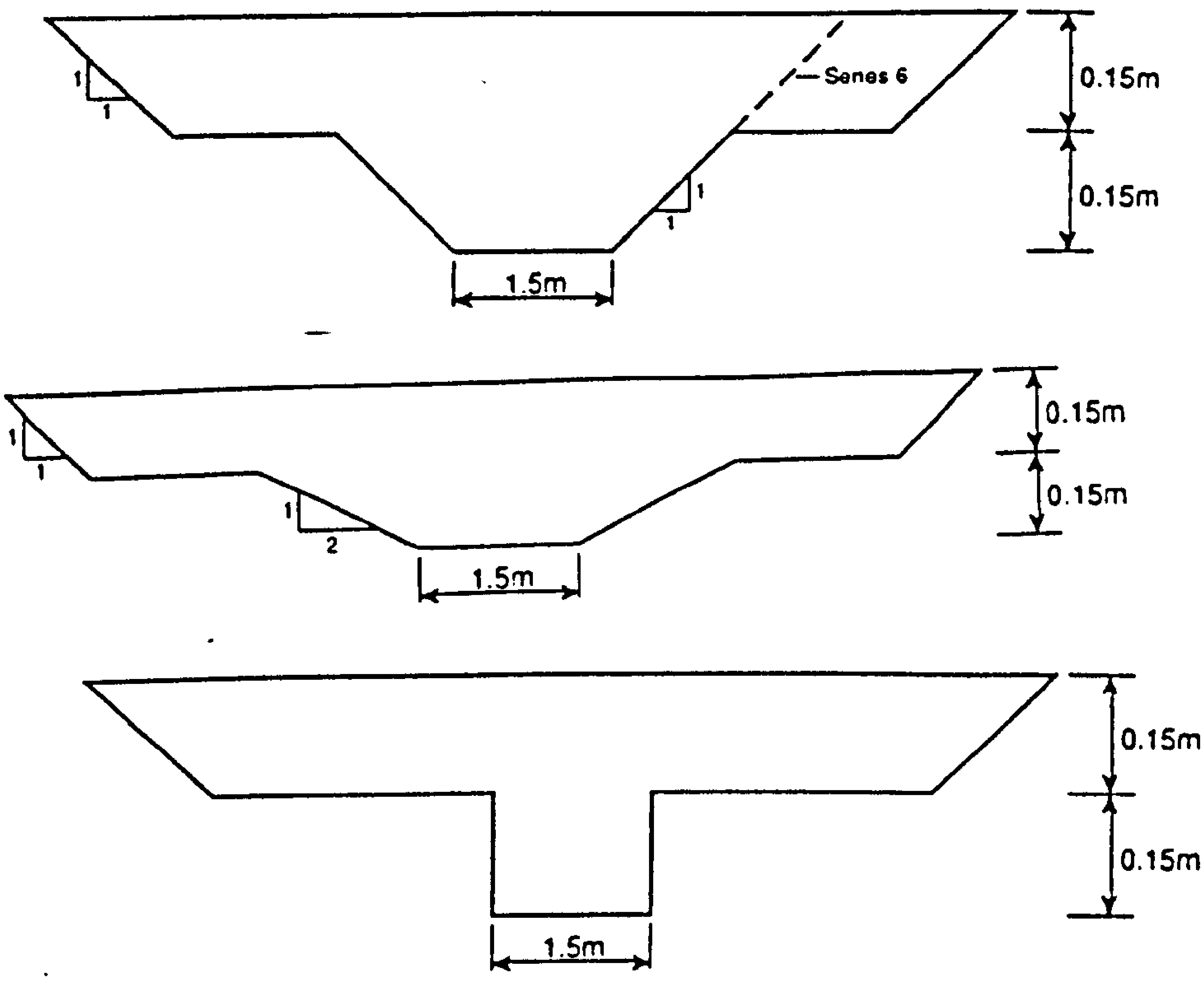
Table 4.04 - Water Level Predictions For Hypothetical Test Case

Table 4.04 indicates the similarity in predicted water levels at each of the fifteen cross-sections. The predicted depth at each cross-section in the model is approximately the same. The accuracy of the ISIS Direct Steady Method is ± 0.01m so the depth can be considered identical.

Although there is no observed data to compare results in this application it is of interest in that it shows the difference between the conventional industry method (the divided channel method) and the Ackers Method.

4.6.3 Test 3 - Ackers Method FCF Test Case – Water Level Prediction

The FCF Series A experiments were the first to be performed and were concentrated on channels that were straight in plan-form and exhibited the following geometries.



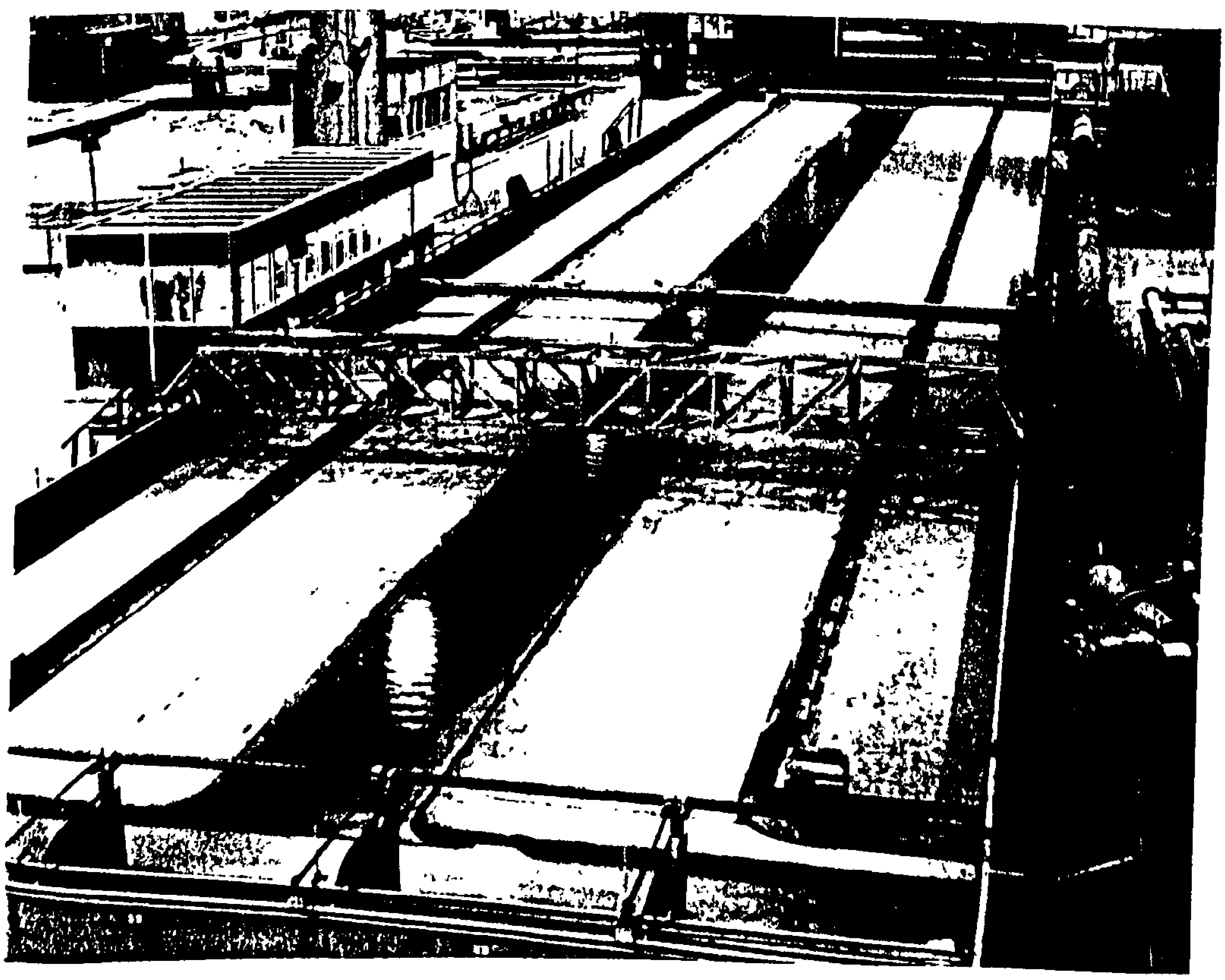


Figure 4.28 The FCF Series A Experimental Apparatus

For more detailed information on the FCF Series A experiments refer to Knight and Sellin (1987). The trapezoidal channel was selected as a sample test case and an ISIS model constructed consisting of 5 identical cross-sections.

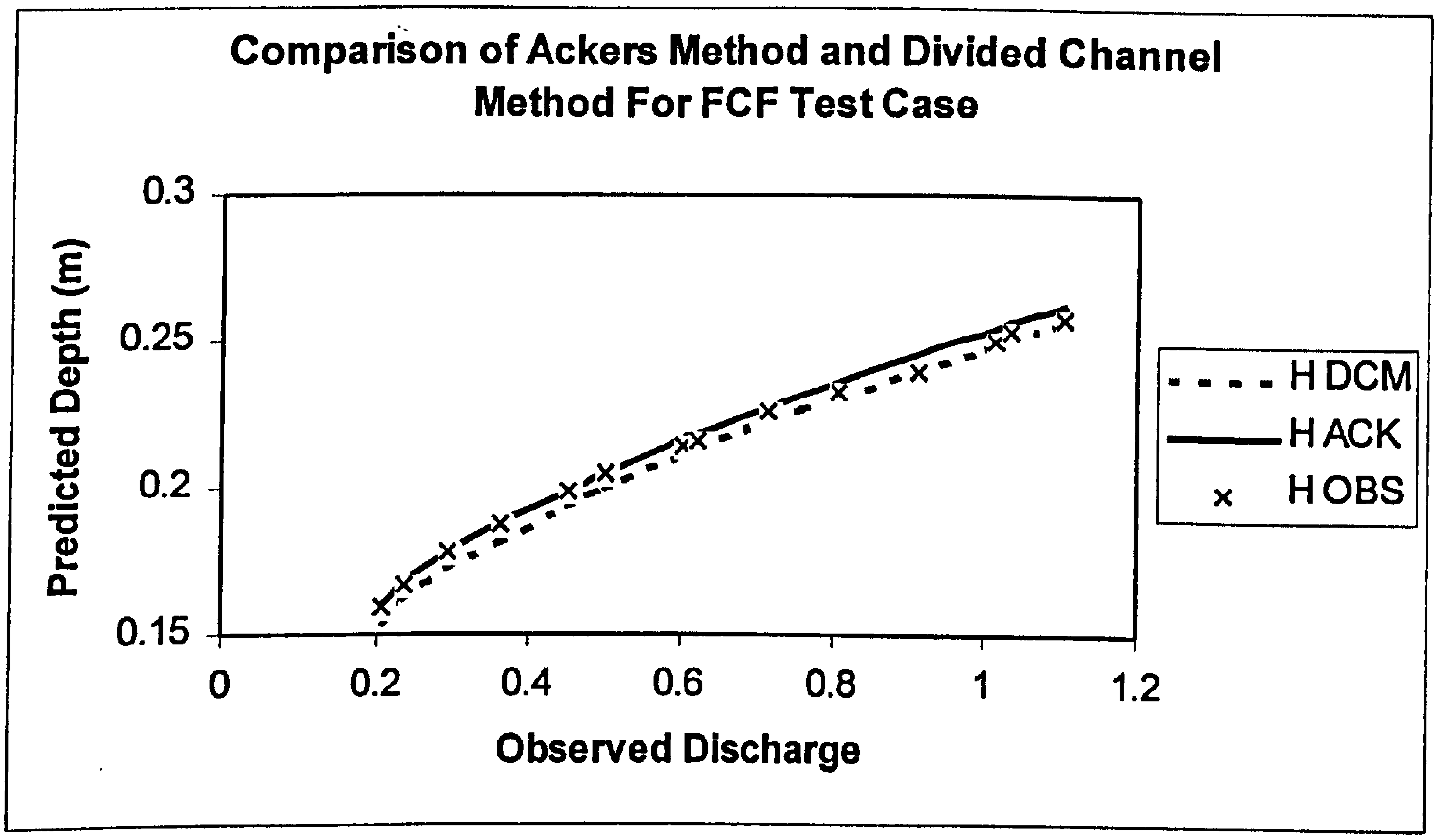


Figure 4.29 Water Level Predictions Test Using The FCF Series A Data

From Figure 4.29 above it is clear that the Ackers subroutine is in close agreement with the observed water levels from the FCF. The maximum difference between Ackers predictions and observed water levels is 6mm. This is an acceptable level of accuracy in modelling terms. The same level of difference is predicted between the Ackers and Divided Channel Method, with the Ackers Method predictions being higher.

Tests 1-3 indicate that the newly developed subroutine is performing correctly and has successfully modelled quasi-natural and experimental arrangements.

At this point a decision was taken to halt the Ackers testing as it was considered to be of limited practical value. The Ackers Method is an improvement on the Divided Channel Method, however it is of no use when modelling a naturally meandering river.

The method was originally chosen to be incorporated to ISIS in 1996 as it was then considered to be a desirable alternative to the Divided Channel Method. However after a detailed inspection and the development work it seemed a little outdated and less accurate than other computational methods such as the Lateral Distribution Method which may also have broader applications. Essentially, it is felt that the Ackers conveyance option will have limited application.

4.7 Reach Averaging

The James and Wark Method has been fundamentally based on the FCF Series B Experiments. These experiments exhibited little or no longitudinal variation in the model cross-sections. As a result, it begs the question, how will the James and Wark Method cope when applied to a river that has significant longitudinal variation ?

This has interesting implications for the survey information required for one-dimensional river models. It is common for the practising Engineer to specify cross-sections that should be surveyed having made an inspection or walking tour of the river. The choice of survey location usually being at a point of significant geometrical change e.g. expansion or contraction of the river cross-section.

However, in order to correctly use the James and Wark Method these surveyed cross-sections need be averaged over a defined reach. For this reach length additional parameters such as sinuosity and meander wavelength would have to be calculated.

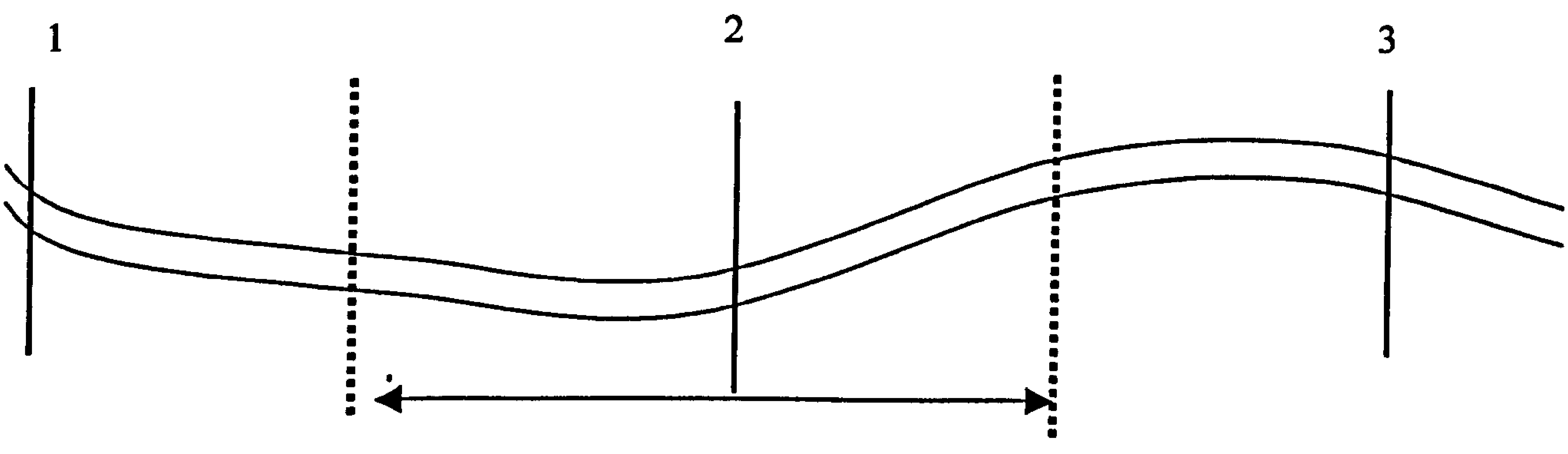


Figure 4.30 - The Representative Distance of a Cross-Section

Each cross-section in a numerical model is representative of half the distance upstream and downstream of its location. This is illustrated in Figure 4.30 and in a natural river this distance could be in the region of 150-300m. It would therefore seem practically reasonable that this could be defined as the reach and to simply apply the James and Wark Method over this distance.

The application of the method to the River Kelvin and River Dane will help to answer this question, however, the requirement for a reach average cross-section is a perceived limitation of the James and Wark Method and makes it less directly compatible with conventional one dimensional river modelling.

Initially it would be interesting to establish what can be used as a reach average cross-section. In order to answer this question it was decided to set up a variety of models of the Flood Channel Facility 60 Degree meander geometry and apply reach averaging with 5 cross-sections (Model 1), reach averaging with a cross-section at each bend apex i.e. 9 cross-sections (Model 2) and a model with 3 reach averaged cross-sections (Model 3).

The reason for five cross-sections in model 1 was to facilitate the correct modelling of the full length of the FCF. i.e. the Δx for the model matched the total FCF length. A reach was defined, in this case, as one complete meander wavelength. From Figure 4.31 it can be seen that there are 4 meander wavelengths and therefore 4 reaches.

Model 2 was given 9 cross-sections in order to simulate a cross-section at each bend apex. By doing so the Δx was reduced to a length of 8.25m however the value of meander wavelength was unchanged i.e. 12. The reason for this being that despite the reach length being only 6m (straight line length) the reach only includes half a meander wavelength. Therefore, the meander wavelength, which is defined as the straight line distance divided by the number of wavelengths, must retain a value of 12.

A similar situation is observed when using Model 3 with the meander wavelength remaining as 12. This model only has three cross-sections but again the geometrical shape and additionally defined parameters are identical to the other models.

However, it should also be remembered that when applying the James and Wark Method that additional reach averaged 'channel parameters' need be defined. This means that no matter how many cross-sections are specified each will have a representative reach length.

As long as the 'channel parameters' are relevant to this length then it is anticipated that reach averaging will be satisfied. To test this assumption three models were set up as shown on the following page.

Each model, as a direct result of having different numbers of cross-sections, has different lengths of representative reach. However, as the experiment being modelled has uniform meander wavelengths, side slopes and sinuosity the 'channel parameters' are in fact the same for each cross-section in each separate model. This is a result of these additional parameters being well defined in the FCF set-up

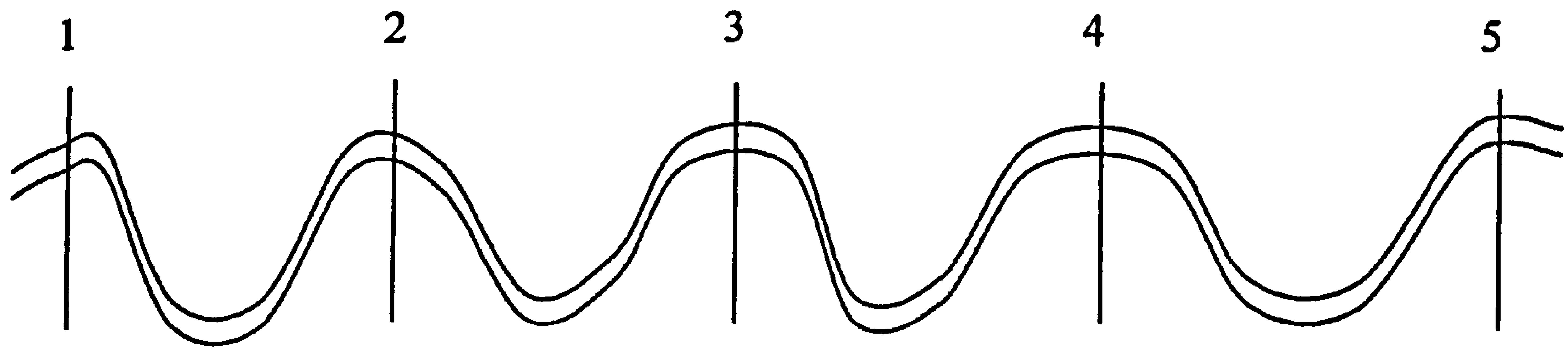


Figure 4.31 - Model 1 Reach Average Cross-Sections

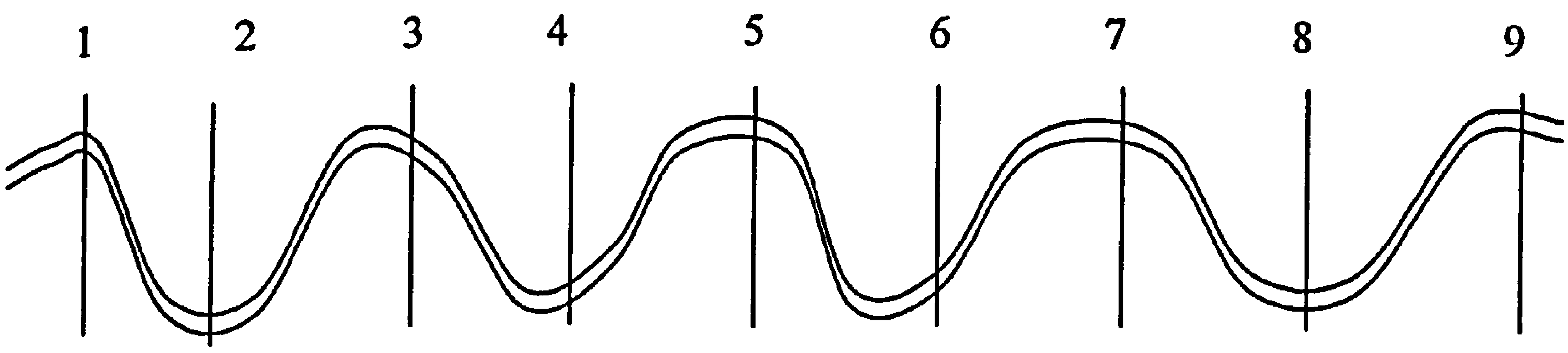


Figure 4.32 - Model 2 Cross-Section Located at Each Bend Apex

The first reach in Model 1 is between cross-sections 1 and 2 and it can be seen that in Model 2 there is an additional cross-section located over the first reach. i.e. 3 cross-sections. It is proposed that if this is assumed then the presence of an additional cross-section will account for greater energy losses than are actually present.

In the above case it would be expected that the predicted water level at cross-section 1 would be higher in Model 2 as more energy losses are being assumed than are occurring over the reach length.

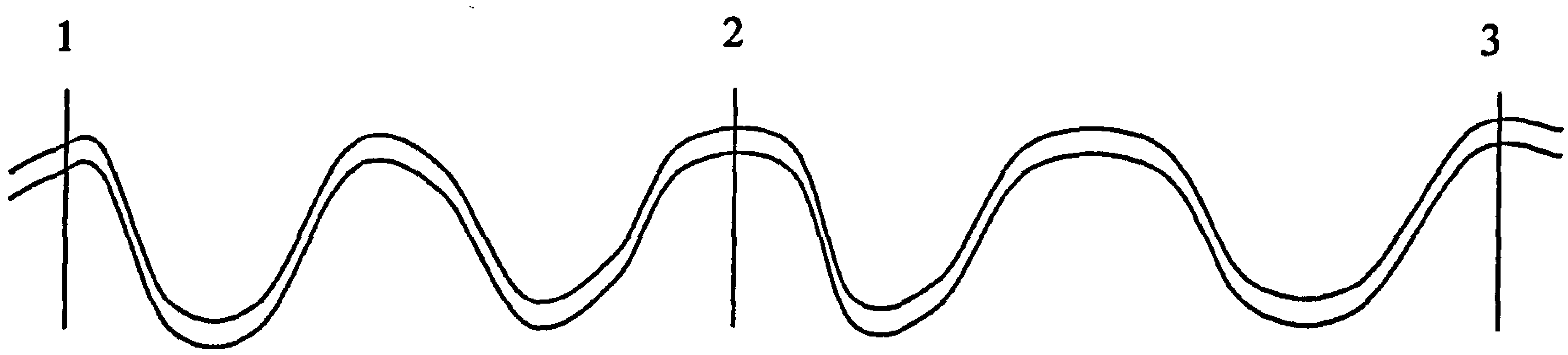


Figure 4.33 - Model 3 - Reach Average Cross-Sections (3 Cross-sections)

The results from each model are shown in Figure 4.34 and illustrate that the predicted water levels are identical. Thus proving that you can define any length as a reach as long as the additional parameters are defined for the same length of reach.

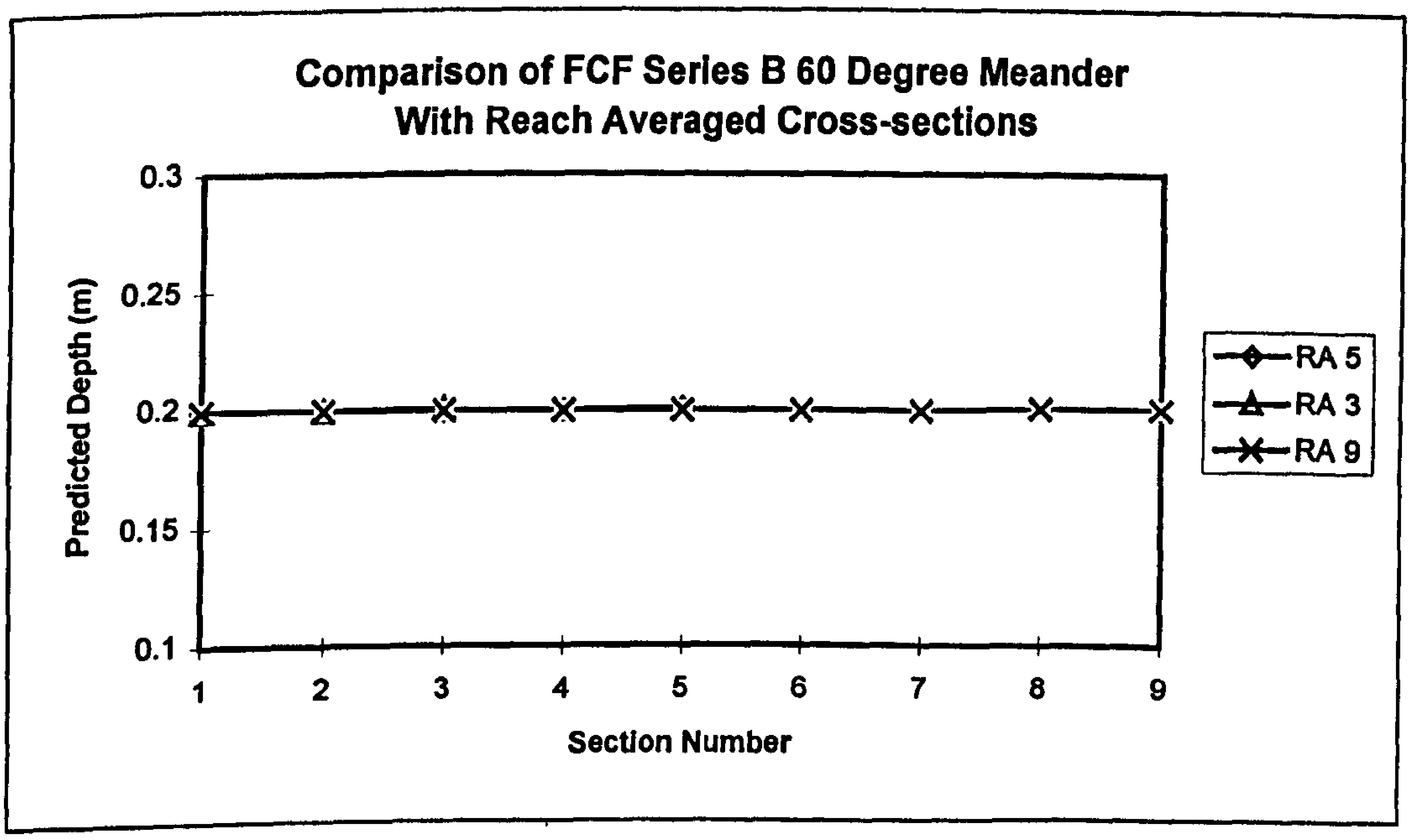


Figure 4.34 - Water Level Prediction Using Different Amounts of Cross-section

The additional cross-sections in a reach length do not increase water level predictions as each cross-section is defined over its representative reach length.

This is an interesting finding however it should be noted that the experiments being modelled were of uniform meander bends which exhibited no significant longitudinal variation. In a natural river, the meandering in non-uniform and cross-sections may vary extensively longitudinally and laterally.

It is concluded that as long as the 'channel parameters' are defined in relation to the reach length the results are the same no matter the reach length. As a result the application to real rivers can proceed in the same manner and it would appear reasonable to specify each cross-section's reach length as its representative length.

Chapter 5 The River Dane

5.0 Numerical Modelling of the River Dane

The aim of this section is to apply the ISIS Divided Channel Method and the ISIS James and Wark Method to the River Dane, Cheshire, England. The reach of interest flows between Rudheath and Northwich, is highly sinuous and therefore should be suited to the James and Wark methodology. For this reason a variety of tests have been performed including different methods of obtaining the reach averaged parameters and a sensitivity analysis.

Of particular interest will be the difference in water level prediction of the industry standard Divided Channel Method (DCM) and the James and Wark Method. Two different approaches of reach averaging will be considered and compared with the DCM results. This is intended to indicate the most practical approach and highlight any significant differences in water level prediction.

The difference between the Divided Channel Method and James and Wark results should illuminate the significance of secondary energy losses in one-dimensional river modelling. The James and Wark Method directly accounts for secondary energy losses and should be a significant improvement on the Divided Channel Method.

A sensitivity analysis will also be performed to provide information to the practising engineer concerning the degree of accuracy required in obtaining the 'channel parameters', that are fundamentally required by the James and Wark Method.

It should be noted that the test reach of the River Dane does not contain any bridges or tributary inflows which might obscure the true effect of the secondary losses.

5.1 Location and Features of The River Dane

The reach of interest on the River Dane extends from the confluence of the Weaver upstream to the bridge on the A556 southern by-pass. (See Figure 1) The reach is approximately 5000 m long and has been surveyed at intervals of 150-200m resulting in 30 cross-sections. (See Figure 5.01) The cross-sections extend on to the flood plain well beyond the location of distant flood banks and extend out as far as the limits of natural 1 in 100 year flooding. (Ervine & Macleod 1996) The hydrological catchment area of the study reach is shown in Figure 5.02.

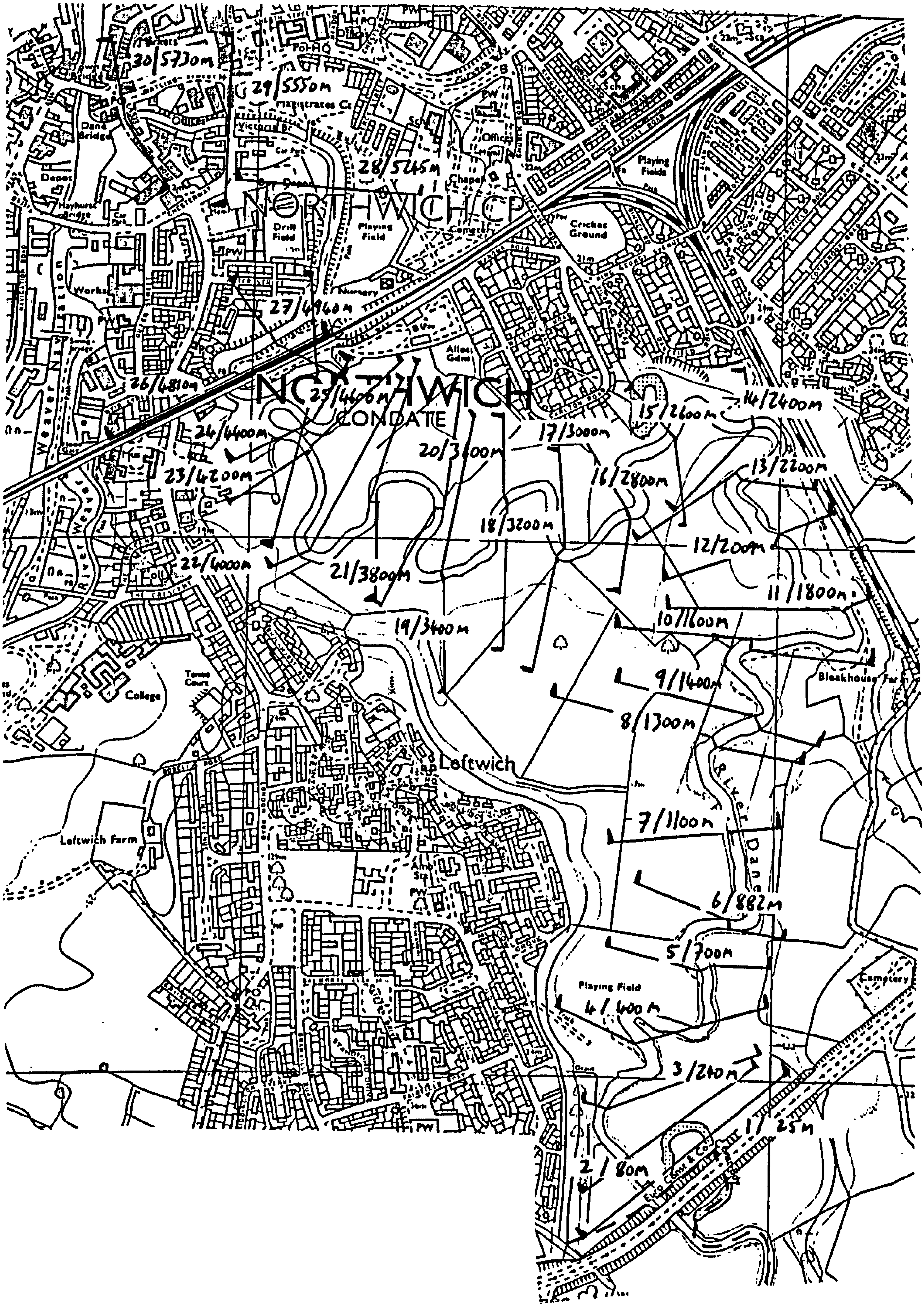


Figure 5.01 Location and Cross-Section Location of The River Dane

Figure 5.02 Catchment Area of The River Dane

The accuracy of river cross-sectional data is considered to be satisfactory despite flood plain level data being obtained from detailed contour maps rather than a detailed survey. This makes the flood plain levels less reliable but sufficiently accurate to give an indicative extent of natural flooding. This method of obtaining additional survey data is discussed further in Chapter 6, in relation to the River Kelvin, and is shown to be a practically reasonable method.

Picture 1 shows the main channel of the River Dane at chainage 3780m with extensive bank-side vegetation and relatively flat flood plains on either side of the main channel. Picture 2 shows a meander bend apex at chainage 3530m, again with extensive vegetation, where secondary losses should be observed.

Picture 1 River Dane at Chainage 3780m

Picture 2 River Dane at Chainage 3530m

Figure 5.02 shows the catchment area of the River Dane. The River Dane rises in the Peak District, flows west to join the River Wheelock, and from there north-west towards Northwich. The total catchment area is 407.1 km^2. River flows are gauged at Rudheath

5.1.1 Rudheath Gauging Station

At the upstream end of the study reach is Rudheath gauging station which has been in existence since 1949. (See Picture 3) Until 1981 measurements were made by wading. Accuracy of this data is questionable as a mobile sandy bed provided an uncertain base. In 1981, a new non-standard flat v notch weir was installed, although apparently has no theoretical equation for stage/discharge. (Ervine and Mcleod,1996)

Velocity-area measurements continue to be taken, with base level 13.19m OD. Confidence levels in the accuracy of the gauged discharges are reasonable, although doubts exist concerning higher flood levels. From Picture 3 it is clear that moderate flood levels will cause some by-passing of the main weir section.

Picture 3 Rudheath Gauging Station

In order to assess the performance of the numerical model, the data from Rudheath gauging station has been utilised. Specifically, the gauged floods of 1946 and 1995 were used. It should be noted that observed information is only known at Rudheath, cross-section 1 in the numerical model.

5.2 ISIS Modelling of The River Dane

The numerical modelling of the River Dane proceeded by initially constructing a steady state model. An unsteady analysis was deemed inappropriate due to the limited accuracy of the data available at Rudheath. The aim of the modelling work is to evaluate the best way of applying the James and Wark Method in the field and to assess the sensitivity of water level predictions to errors in the additional parameters. Comparisons will also be made with the existing ISIS Divided Channel Method.

An ISIS model was constructed using the 30 surveyed cross-sections, a value of peak inflow at the upstream boundary and a known water level at the downstream boundary. Some sample model cross-sections are shown in Figures 5.03, 5.04 and 5.05. A value of Manning's 'n' has been estimated after reference to a series of photographs, of the River Dane, and to Chow (1959). The River Dane was considered to be clean and winding with some pools and moderate vegetation. This compared with a value of 0.048 Chow (1959) and an earlier study, on the Dane, by Mcleod (1998).

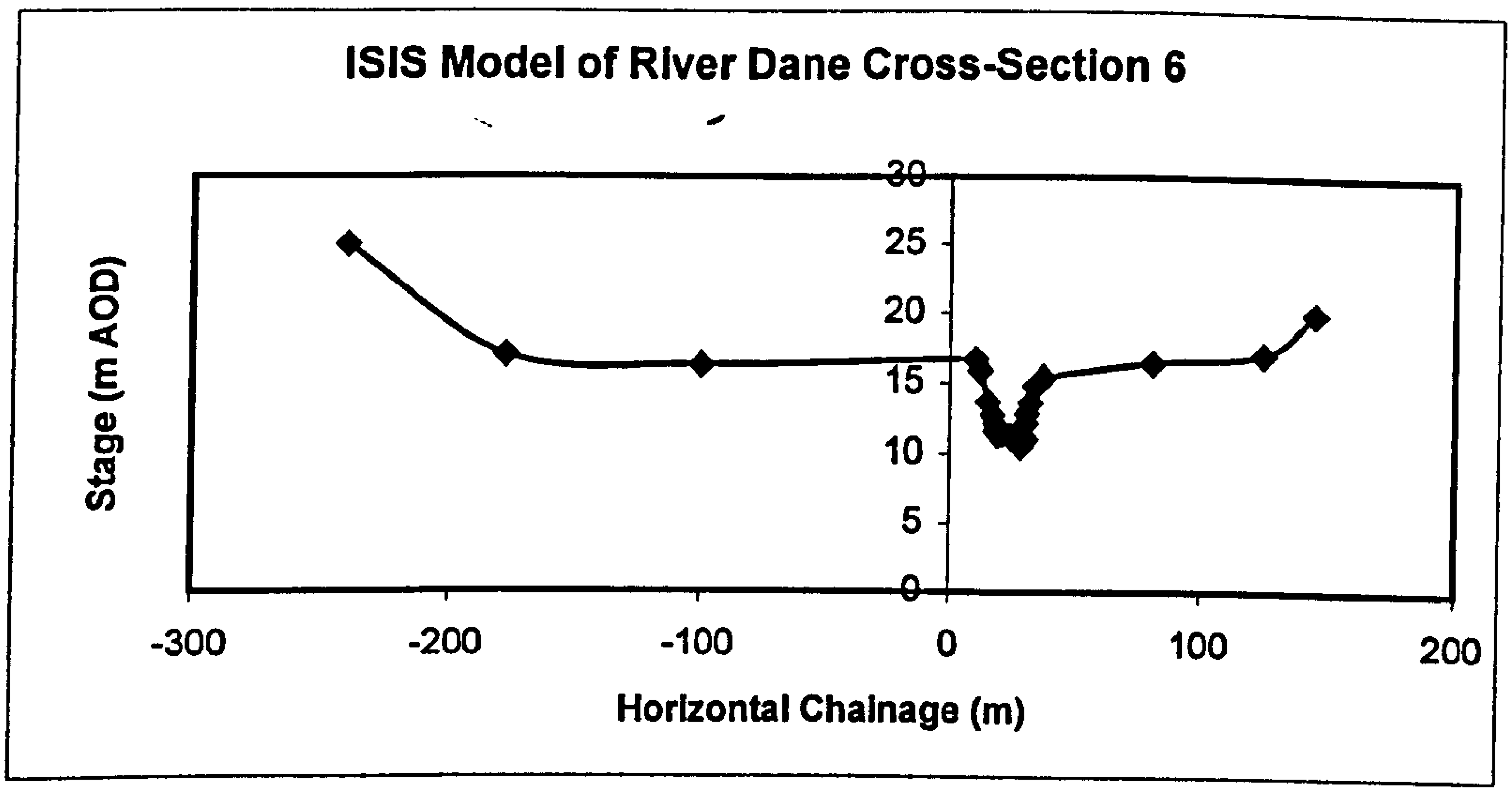


Figure 5.03 ISIS Model River Dane Cross-Section 6

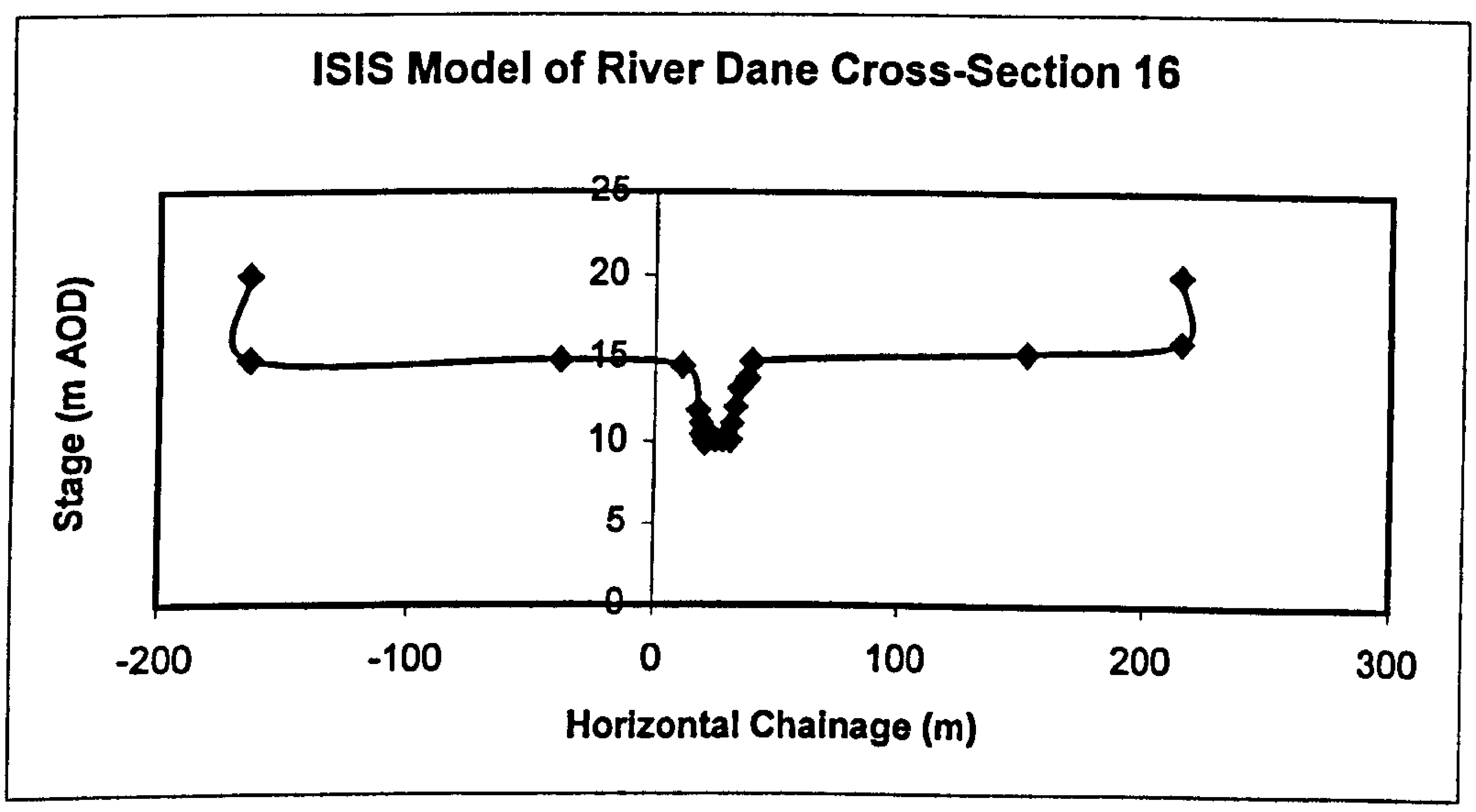


Figure 5.04 ISIS Model River Dane Cross-Section 16

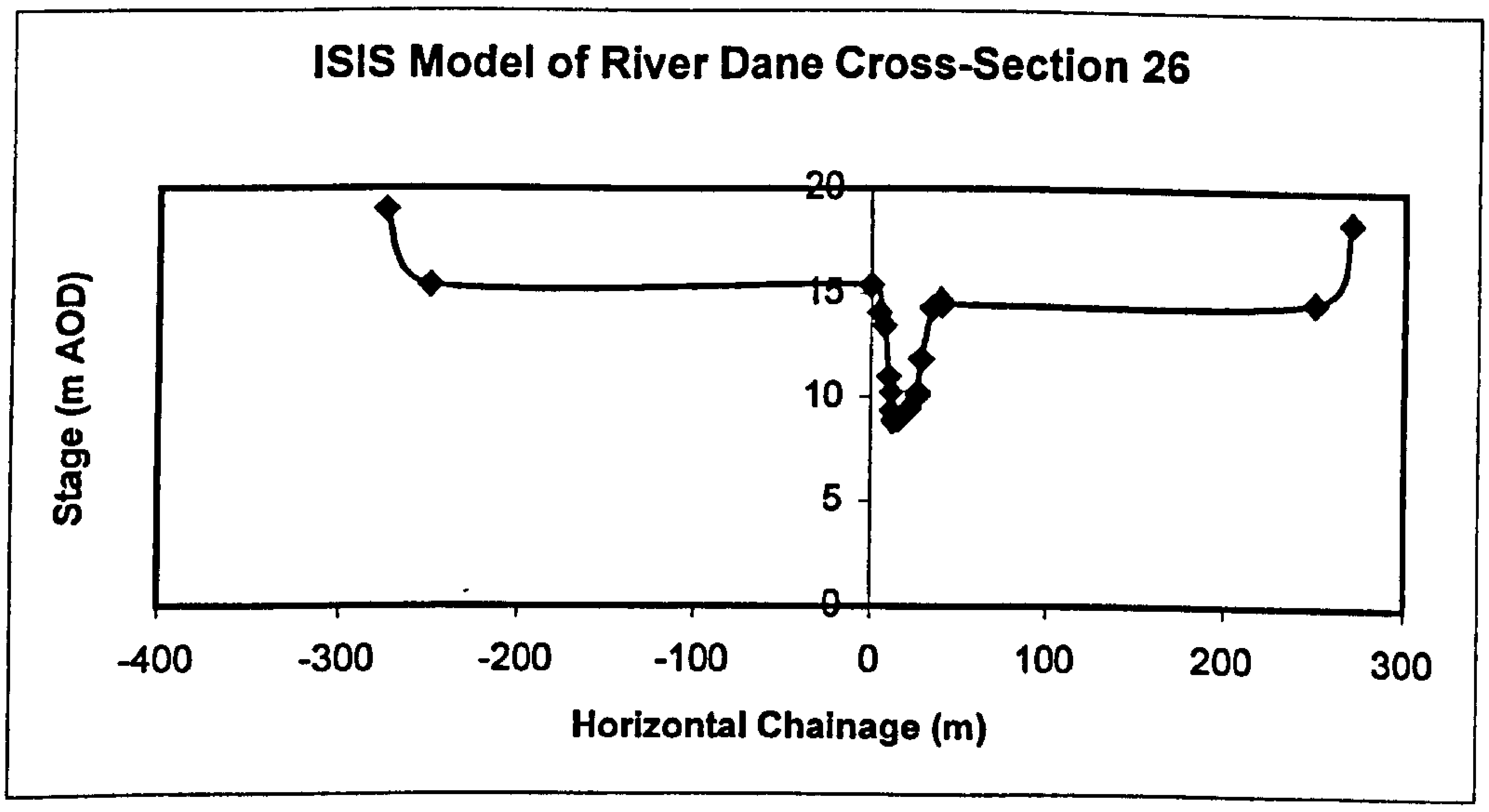


Figure 5.05 ISIS Model River Dane Cross-Section 26

As detailed in chapter 4 the application of the James and Wark Method requires additional channel parameters to those necessary for the Divided Channel Method. These parameters are estimates of sinuosity, meander wavelength, side slope and flood plain slope. In a natural river with extensive longitudinal variation, these can be difficult to assess, but with reasonable judgement an acceptable value can be obtained. The following section outlines the assumptions and methodology used in obtaining these values for the River Dane. It should be noted that two separate approaches have been considered and have been named Method 1 and Method 2.

5.3 Method 1

Assumed Reach Length

Wark (1998) suggested that a reach representative cross-section was required for the correct application of this method and that for every bend encountered a number of cross-sections were surveyed. However, this is not practically viable in terms of time and cost for most commercial modelling contracts. As a result, for Method 1, only the channel parameters sinuosity, meander wavelength, side slope and flood plain slope have been averaged and used along with the surveyed cross-sections.

For Method 1 the representative reach was assumed to be the entire length of the River Dane. This may be considered to be a valid assumption as the sinuosity and meander belt width of the study reach are approximately constant over the whole 5km. (See Figure 5.01)

It was decided that the 'channel parameters' would be calculated for the entire reach and used together with the 30 surveyed cross-sections. Effectively this meant that the additional parameters would be the same at each river cross-section.

Sinuosity

The sinuosity of the River Dane was calculated over the 5km reach by dividing the total centre line distance by an approximate straight-line distance. This resulted in a sinuosity estimate of 1.8 which is very high. This would seem to be a reasonable value, as can be seen from Figure 5.01, as the River Dane exhibits many tight bends.

Meander Wavelength

The meander wavelength was estimated from the 1:10000 plan map of the location and is defined as the straight line distance divided by the number of wavelengths in the reach length. In this case the total centre-line distance is 5000m and the number of wavelengths is 20. This results in a meander wavelength of 250m.

Side Slope

The side slope was simply measured from the upper two-thirds of the river-banks (see Figure 5.06). As the banks are generally irregular and the actual slopes vary, straight lines are fitted to the upper two-thirds of the bank profiles. The method is the same as that of Ackers (1991). The average of the right and left bank is taken and is expressed as the ratio of horizontal to vertical distances.

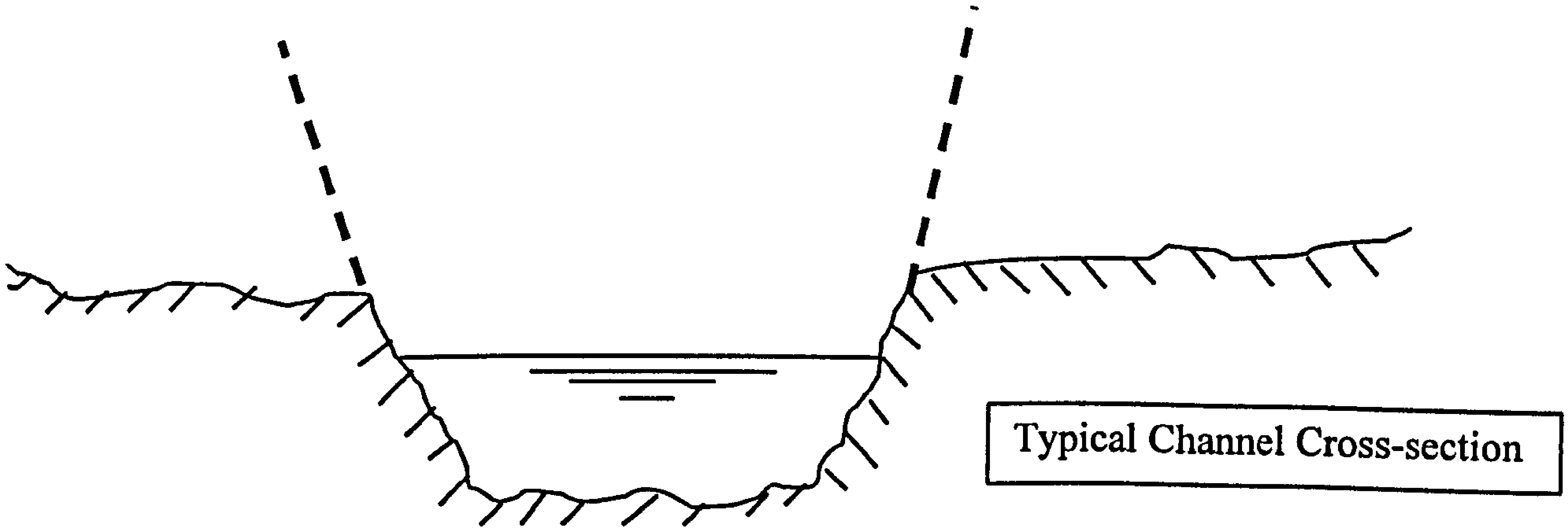


Figure 5.06 Extension of Upper Third of Main Channel Side Slope

Flood Plain Slope

The value of flood plain slope was assumed to be constant along the reach length and was taken from the cross-sectional information. In using this value the problem of adverse slopes is overcome.

Meander Belt Width

This parameter was estimated at each surveyed cross-section and in this application was approximately constant at 200m. It should be noted that the experiments that the James and Wark method were based on had horizontal flood plains which are not always the case in the field.

The fact that no account is made for sloping flood plains can be considered as a limitation of the method. Recent research by Liu and James (1997) has reported that sloping flood plains reduce flow resistance.

The technique of obtaining the 'channel parameters' by taking typical values over the whole river length is a reasonably straight-forward method. A possible flaw is that in places the value of sinuosity and meander wavelength obtained is not realistic, of local conditions. For example, at cross-sections 1 and 2 the sinuosity is low and would not agree with the specified value of 1.8.

Having obtained the 'additional parameters' a steady analysis can be performed. From this analysis it is intended to observe the significance of the new conveyance calculation in terms of water level prediction.

5.3.1 Performance against the January 1995 Flood

For the steady analysis of this flood event an estimate of the peak flow at the upstream end and the corresponding highest observed water level at the downstream end are required as boundary conditions. The details of the January 1995 flood event were provided by the Environment Agency, previously the National Rivers Authority. The flood peak at Rudheath was estimated as 107.64 m^3/s and the confluence level at the Weaver was approximately 10.7m OD.

Both James and Wark and Divided Channel Method analyses have been performed. As the James and Wark Method accounts for additional energy losses other than bed friction, it would be expected that the James and Wark predictions would result in a higher water level than predicted by the Divided Channel Method.

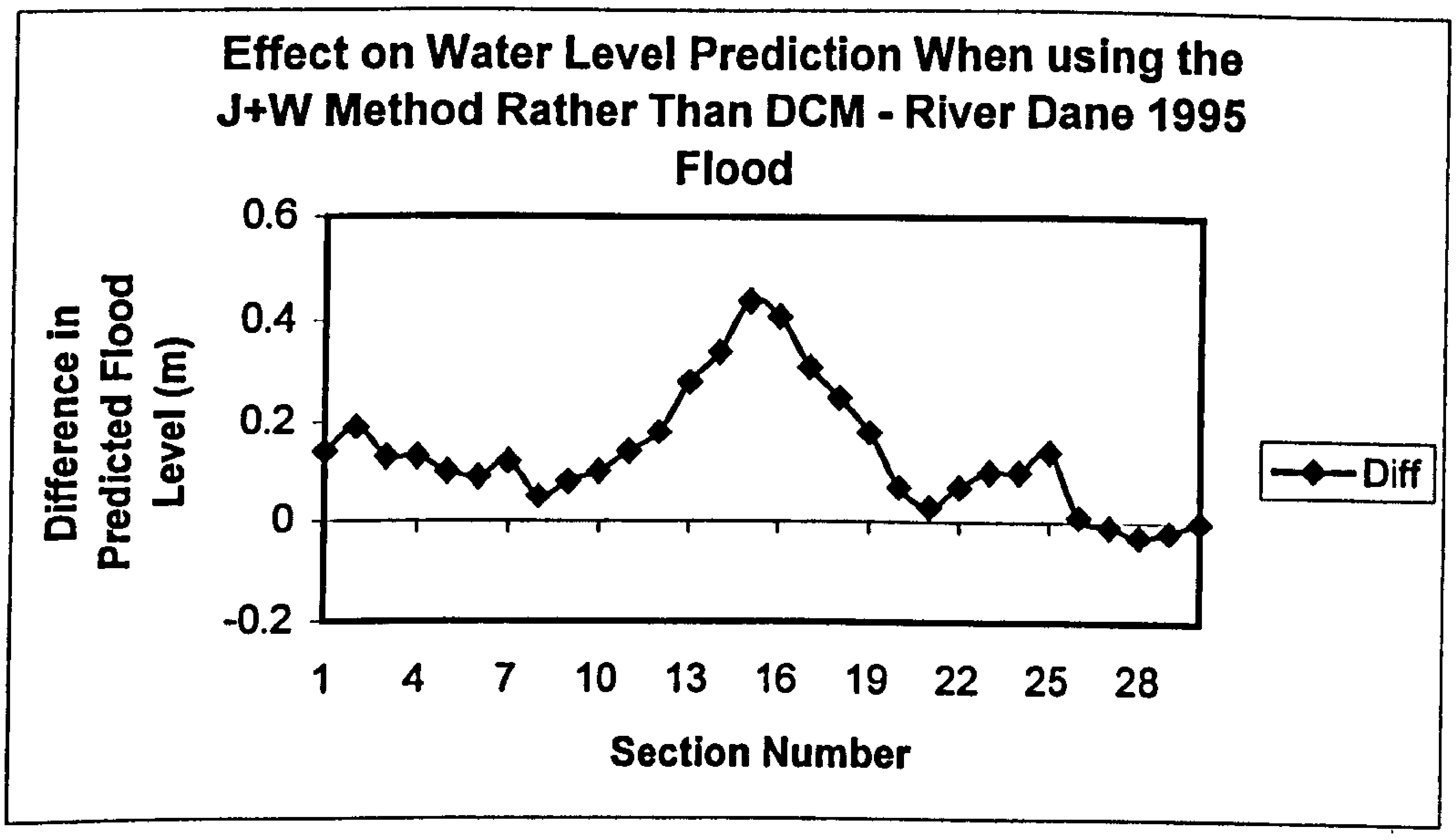


Figure 5.07 - Comparison of ISIS Steady State Conveyance Methods January 1995

From Figure 5.07 above it is clear that when the same Manning's 'n' is used that this indeed is the case.

The James and Wark method predicts water levels that are greater than the Divided Channel Method by 0.14m on average. Interestingly, there is a 0.44m at section 16 by 0.44m The water levels predicted at this location are just out of bank and would therefore expect a significant difference between the conveyance calculated at this location using the James and Wark and Divided Channel Methods.

The minor under predictions at sections 26-28 are due to the flood flow being contained within the main channel where the James and Wark Method in inappropriate.

Location	OBS WL (MAOD)	DCM WL(MAOD)	J+W WL (MAOD)
Rudheath	16.66	16.87	16.98

Table 5.01 - Results of January 1995 Flood Event Calibration Steady State

Both the Divided Channel Method and James and Wark methods over predict the flood level at Rudheath suggesting that the estimate of Manning's 'n' (0.048) is too high and could be reduced.

5.3.2 Performance against the 1946 Flood

Despite the evidence that the estimated 'n' value is too high it has been maintained at 0.048 for this simulation. The downstream boundary for this flood event was a water level of 13.5m and the corresponding flood flow was estimated to be around 170m³/s.

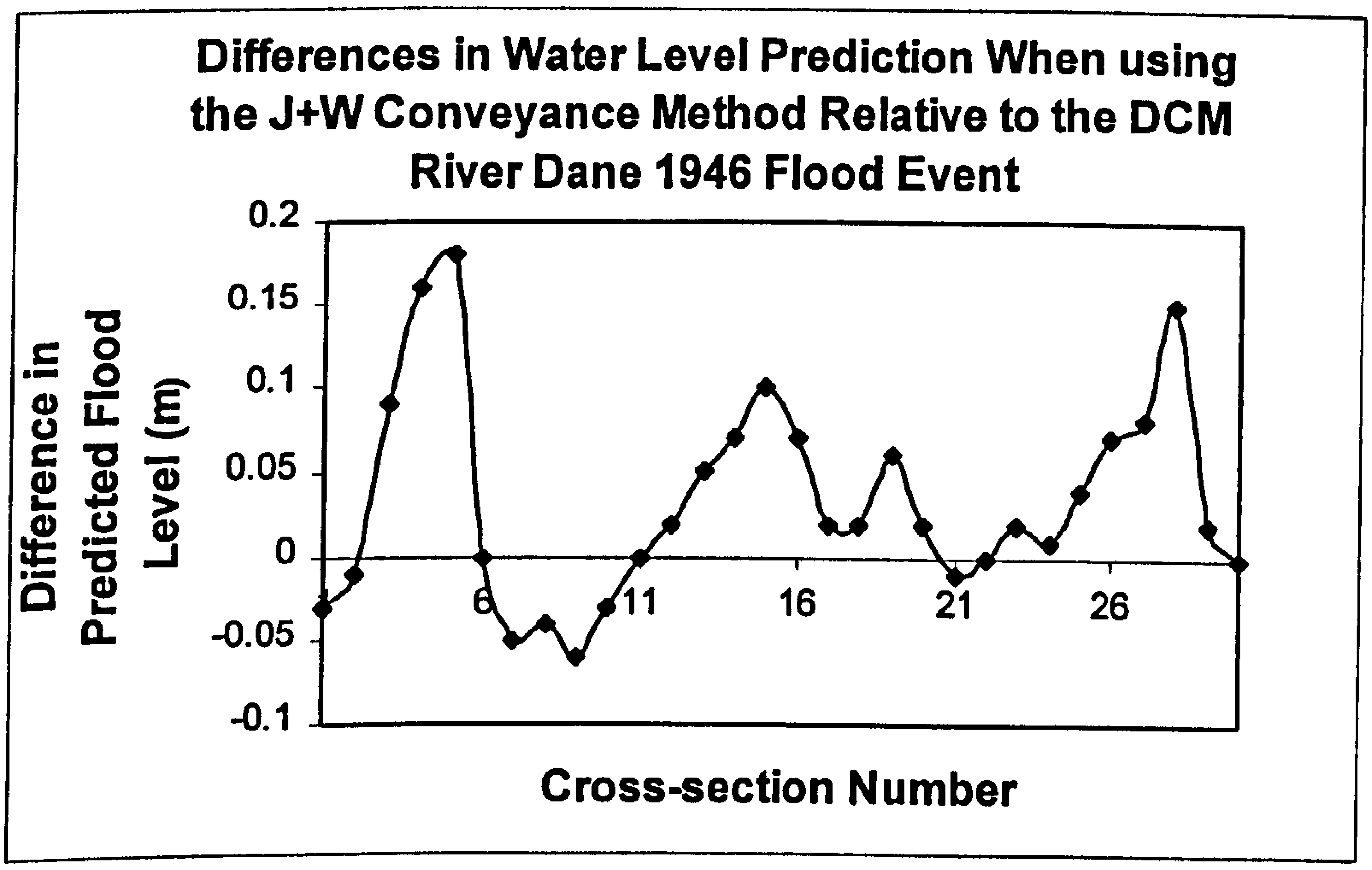


Figure 5.08 - Comparison of ISIS Steady State Conveyance Methods 1946 Flood

The predictions shown in Figure 5.08 illustrate the difference in water level prediction when using the James and Wark Method relative to the Divided Channel Method. It indicates a general increase in predicted flood level of around 0.1m. Notably, at cross-section 5 there is an increase of 0.18m which could be considered as practically significant. The under predictions at sections 6-10 are a result of these locations experiencing significant flood plain depth. At such levels of inundation the James and Wark method is not expected to perform accurately. **(See Section #)**

Table 5.02 shows the performance of both methods at Rudheath Gauging station.

Location	OBS WL (MAOD)	DCM WL(MAOD)	J+W WL (MAOD)
Rudheath	17.53	17.32	17.30

Table 5.02 - Results of 1946 Flood Event Calibration Steady State

For this considerable flood event the Divided Channel Method and James and Wark Method both under-predict water levels by approximately 200 mm. This under prediction at Rudheath was also observed by an earlier study of this river reach by Ervine and Macleod (1999) and may be a result of the straight plan form of sections 1 and 2 or that channel-flood plain interaction is less significant at this location.

The results of this application differ slightly to those of Ervine and Mcleod (1999). Their results had many locations where the DCM predicted a higher water level than the James and Wark Method. This result is unexpected as J+W accounts for additional energy loss mechanisms and should always predict a higher water level. It is suggested that there was an error in the conveyance calculation in the Ervine and Macleod (1999) application which has given misleading water level predictions. The current study shows variations in water level prediction between the Divided Channel and James and Wark Method that one would expect from the underlying theory.

Another factor which may have influenced the conveyance calculation and water level predictions in this practical application is the extensive longitudinal variation in cross-sections, which may be at too large a scale for the James and Wark method to cope with. Also, Mcleod (1998) concluded that when the James and Wark Method is applied to a geometry that is dissimilar to that on which it was based, then erroneous results can be obtained.

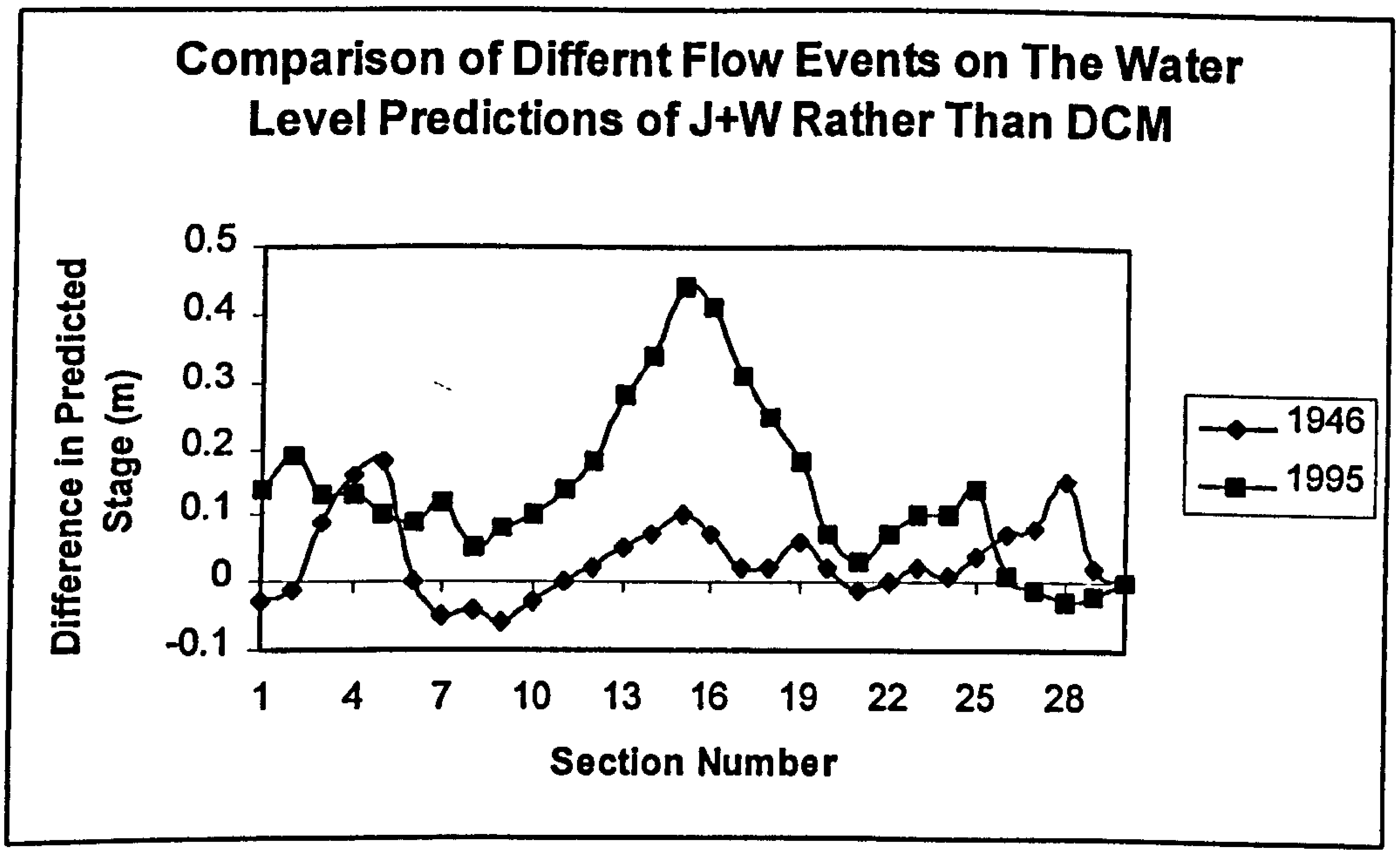


Figure 5.09 Differences Computed When Using The J+W

Figure 5.09 shows the predicted difference for each flood event on the same graph. It can be seen that the differences are much more significant when the flow is at small flood plain depths, as is the case with the 1995 flood. For the much bigger 1946 flood flow the predicted differences relative to the Divided Channel Method are considerably smaller. The maximum difference for this event was 180 mm which is significant in itself, but the general pattern shows an increase of around 0.06m. This would imply that the bigger the flood flow then the less significant the James and Wark Method predictions will be. This is in keeping with the theory of overbank flow research where the main region of interest is that of "just out of bank".

A possible implication of these small differences for high floods being that industrial users, who are primarily interested in estimating maximum water levels, would not see the benefit of using the James and Wark Method when it only changes the existing Divided Channel Method result, at Rudheath, by 20 mm. Method 1 assumed that additional parameters could be calculated over the full 5000m reach length. This was similar to the assumptions made in an earlier study by Mcleod (1998). It is now intended to use a different reach length and calculate all the 'channel parameters' in relation to it. This second approach, Method 2, will assume that each cross-section's representative length will be used as the reach length. This will mean that each reach will be in the region of 150m and all the 'channel parameters' will be calculated over this shorter reach length.

5.4 Method 2

The aim of this method of reach averaging is to attempt to refine the channel parameters and make them more locally relevant. This will inevitably result in smaller reach lengths and consequently make the estimation of the additional parameters more difficult. However, the 'channels parameters' should be described 'more correctly', using this technique rather than method 1.

Assumed Reach Length

In a one-dimensional model each user-defined cross-section is assumed to be representative of half the distance upstream and half the distance downstream from the surveyed location. In a natural river this 'representative distance' is generally in the region of 150-300m. As a result, it seemed reasonable to use this distance as the reach length and to average conditions over its length.

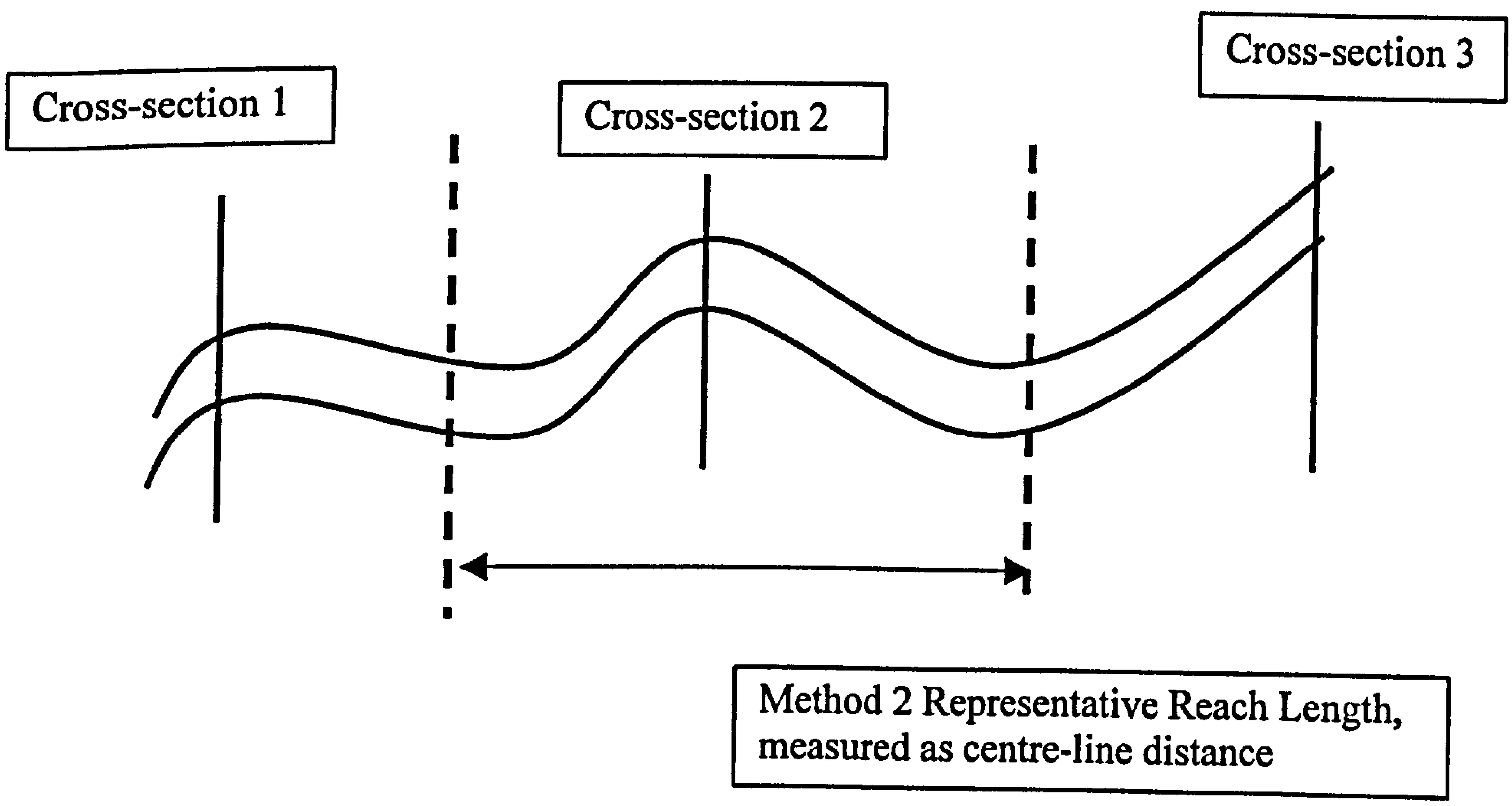


Figure 5.10 Representative reach Length for Method 2

Sinuosity

As the reach length for this model is significantly shorter than Method 1 the estimate of sinuosity is expected to be much smaller. This is mainly due to the limited meandering that can physically occur over a reach of say 200m. For this method of reach averaging the sinuosity ranged from 1.10-2.14.

Generally the sinuosity was around 1.45. As a consequence of reducing the reach length the sinuosity values must reduce.

Meander Wavelength

The meander wavelength for this reach length is again more difficult to define and is unlikely even to be one full wavelength. An estimate of 0.7 of the reach length, i.e. assuming each reach length consists of 0.7 of a meander wavelength has been used and is considered reasonable.

Side Slope

The estimate of side slope is obtained in a similar manner to that of Method 1. Essentially, straight lines are fitted to the upper two-thirds of the river-bank to obtain the slope estimate. (See Figure 5.06)

Flood Plain Slope

The average flood plain slope has been again used for this model and is the same as Method 1.

Having obtained the channel parameters for the shorter reach length, the January 1995 and 1946 flood events were again simulated using this data. A Manning's 'n' value of 0.048 was again used.

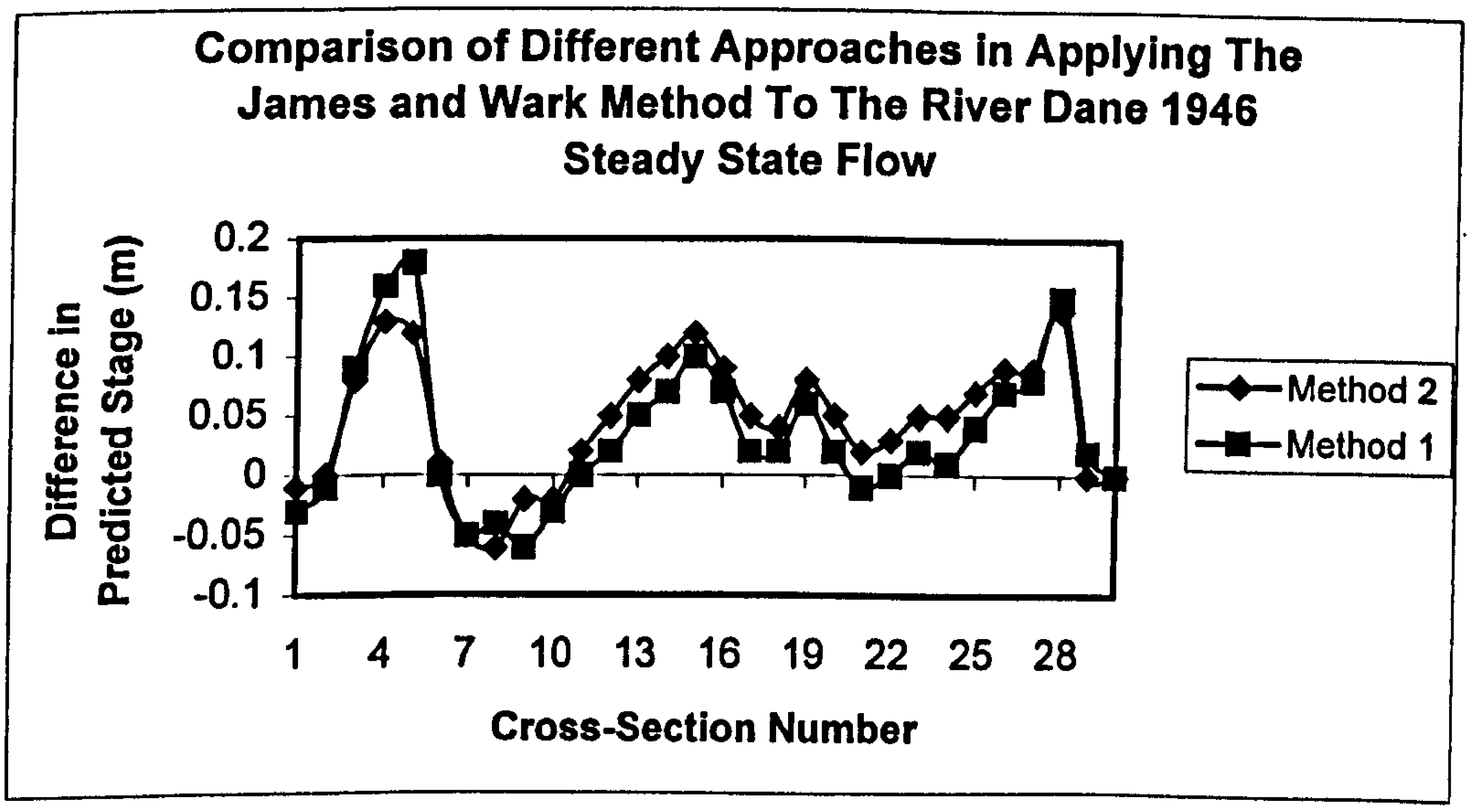


Figure 5.11 - Comparison of Reach Averaging Methods

Figure 5.11 illustrates the differences (i.e. between James and Wark predictions and Divided Channel Predictions) in water level prediction using various reach averaging assumptions together with the 1946 flood event data. Clearly, there is no significant difference in water level prediction despite the significant difference in reach length and additional parameters. This is similar to the findings of the FCF Tests (see Chapter 4) where no difference in water level predictions were observed, despite different reach lengths being used, as long as the 'channel parameters' were defined in relation to the reach.

For the Figure above it should be noted that only the reach length, sinuosity and meander wavelength are changed from Model 1 i.e. the side slope and flood plain slope are the same Generally, the James and Wark method will tend to over-predict the Divided Channel Method water levels.

Table 5.03 details the various model water level predictions for the Divided Channel Method and James and Wark Method for both methods. (M1 refers to Method 1 and M2 refers to Method 2)

Sect No	H_{DCM}	H_{J+W} M1	H_{J+W} M2	DCM/M1	DCM/M2	M1/M2
1	17.32	17.30	17.32	0.02	0	-0.02
2	17.25	17.25	17.26	0	-0.01	-0.01
3	17.04	17.14	17.13	-0.1	-0.09	0.01
4	16.78	16.92	16.89	-0.14	-0.11	0.03
5	16.63	16.75	16.71	-0.12	-0.08	0.04
6	16.39	16.36	16.38	0.03	0.01	-0.02
7	16.27	16.20	16.22	0.07	0.05	-0.02
8	16.08	15.99	16.00	0.09	0.08	-0.01
9	15.94	15.90	15.91	0.04	0.03	-0.01
10	15.81	15.79	15.80	0.02	0.01	-0.01
11	15.74	15.74	15.76	0	-0.02	-0.02
12	15.68	15.71	15.74	-0.03	-0.06	-0.03
13	15.58	15.64	15.67	-0.06	-0.09	-0.03
14	15.51	15.60	15.62	-0.09	-0.11	-0.02
15	15.41	15.53	15.54	-0.12	-0.13	-0.01
16	15.23	15.33	15.34	-0.1	-0.11	-0.03
17	15.02	15.05	15.08	-0.03	-0.06	-0.02
18	14.88	14.89	14.91	-0.01	-0.03	-0.02
19	14.69	14.72	14.74	-0.03	-0.05	-0.02
20	14.57	14.56	14.59	0.01	-0.02	-0.03
21	14.49	14.43	14.45	0.06	0.04	-0.02
22	14.43	14.38	14.40	0.05	0.03	-0.02
23	14.34	14.28	14.30	0.06	0.04	-0.02
24	14.26	14.18	14.20	0.08	0.06	-0.02
25	14.17	14.05	14.08	0.12	0.09	-0.03
26	13.92	13.76	13.78	0.16	0.14	-0.02
27	13.90	13.73	13.73	0.17	0.17	0
28	13.77	13.59	13.58	0.18	0.19	0.01
29	13.36	13.29	13.29	0.07	0.07	0
30	13.50	13.50	13.50	0	0	0

Table 5.03 Comparison of Water Level Predictions For Different Model Assumptions

5.5 Discussion

From this section it has been shown that the use of a reach average cross-section is not required. Table 3 shows the variation in water levels at each model cross-section. All that is required is the surveyed cross-section and estimates of the additional parameters averaged over a defined reach length. Essentially, the additional parameters should be reach averaged.

Model Performance

The models that have been tested by keeping Manning's 'n' constant and observing the difference between James and Wark Method and Divided Channel Method predicted water levels. It was initially expected that the James and Wark Method would over predict the Divided Channel Method levels at all cross-section locations. This has generally occurred. However, in an earlier study of the Dane by Ervine and Mcleod (1996) this was not the case and indeed the Divided Channel Method over predicted water levels at 16 out of the 30 cross-sections when using the significantly out of bank 1946 flood event. This is unexpected when using the James and Wark Method to calculate conveyance and it is probable that the conveyance calculation used by these authors was incorrect. It should be noted that although it may be incorrect the water level predictions are not significantly different to those obtained in this investigation.

Although the raw data is not ideal it has been of interest to observe the difference between current best practice (DCM) and the James and Wark Method. In terms of water level prediction there has not been a significant increase, in global terms, due to the inclusion of secondary losses. It is apparent that the James and Wark Method predictions are sensitive to just out of bank flows, similar to the 1995 event, and less sensitive to high flows which result in significant flood plain depths.

The James and Wark predictions may be limited by the extensive longitudinal variation in the surveyed cross-sections. It is possible that these are too much for the James and Wark Method to comfortably cope with. Mcleod (1998) postulated that the James and Wark method could give erroneous results for geometries dissimilar to that used in its development.

In terms of 'reach averaging', the reach length has been found to be unimportant as long as the additional parameters are calculated in relation to its length. This has been demonstrated both in the Flood Channel Facility and River Dane Models. The two different assumptions for representative reaches both proved to be similar in water level predictions. It is suggested that a modeller can define any length as a reach and, as long as the 'channel parameters' are estimated in relation to this, the predicted water levels will be similar.

5.6 Sensitivity Analysis – Steady State Modelling

It is not known what effect errors in the 'channel parameters' may induce in a field application. The following tests are intended to provide information on the required accuracy or sensitivity of these parameters and their consequent effect on water level predictions. It should be noted that the assumptions made in Method 1 have been used here. James and Wark (1992) recommended that sensitivity tests should be carried out in any practical application. The following tests are all carried out using an ISIS Steady State model with the 1946 flood used as boundary conditions. Manning's 'n' is also assumed to be 0.048. It should be noted that there could be inaccuracies in both the estimate of 'n' and the Flow, Ervine and Mcleod (1996), as is possible in any flood study, however, the following tests are intended to indicate the difference between the Divided Channel method and the James and Wark method water level predictions. The interested reader is referred to Ervine and Mcleod (1996) for sensitivity tests of these parameters.

The following parameters will be varied to observe their influence on predicted water levels.

- Sinuosity
- Meander Wavelength
- Meander Belt Width

From the additional parameters required only the flood plain slope and the side slope are not being investigated. The flood plain slope will not influence the calculations and the side slope is not deemed to be significant enough to merit further investigation. It is considered that no significant error would be made in estimating this term and James and Wark (1992) found that ±100% changes to the side slope only resulted in ±5% change in predicted discharge.

It should be noted that changing the slope makes no difference as the values of discharge in the James and Wark Method are being changed in direct proportion to the change in slope.

5.6.1 Sensitivity of Water Level Predictions to Estimate of Sinuosity

The following test is simply altering one parameter at a time, which is really a test of accuracy of data. In reality, if the sinuosity were to change then other parameters such as belt width, distance downstream and slope would all change. The aim of this test is to ascertain how accurate the estimate of channel sinuosity needs to be and to provide guidance to the practising engineer concerning the limits of acceptable accuracy.

A previous study of the River Dane defined the sinuosity as being 1.8, which represents a high sinuosity, and is considered to be accurate if the whole reach length is being considered. Figure 5.11 illustrates the difference obtained in water level predictions if sinuosity is varied as follows, 1.5, 1.8 and 2.1. This implies an error of approximately ± 20% in the sinuosity term.

Interestingly, the results show that, when the sinuosity is changed independently, the water levels reduce with increasing sinuosity. This is not what one would expect intuitively and is a direct result of the independent alteration to the sinuosity parameter and the 'make-up' of the James and Wark Method equations. The maximum difference in predicted water levels of -0.07m occurred when the sinuosity was increased to 2.1.

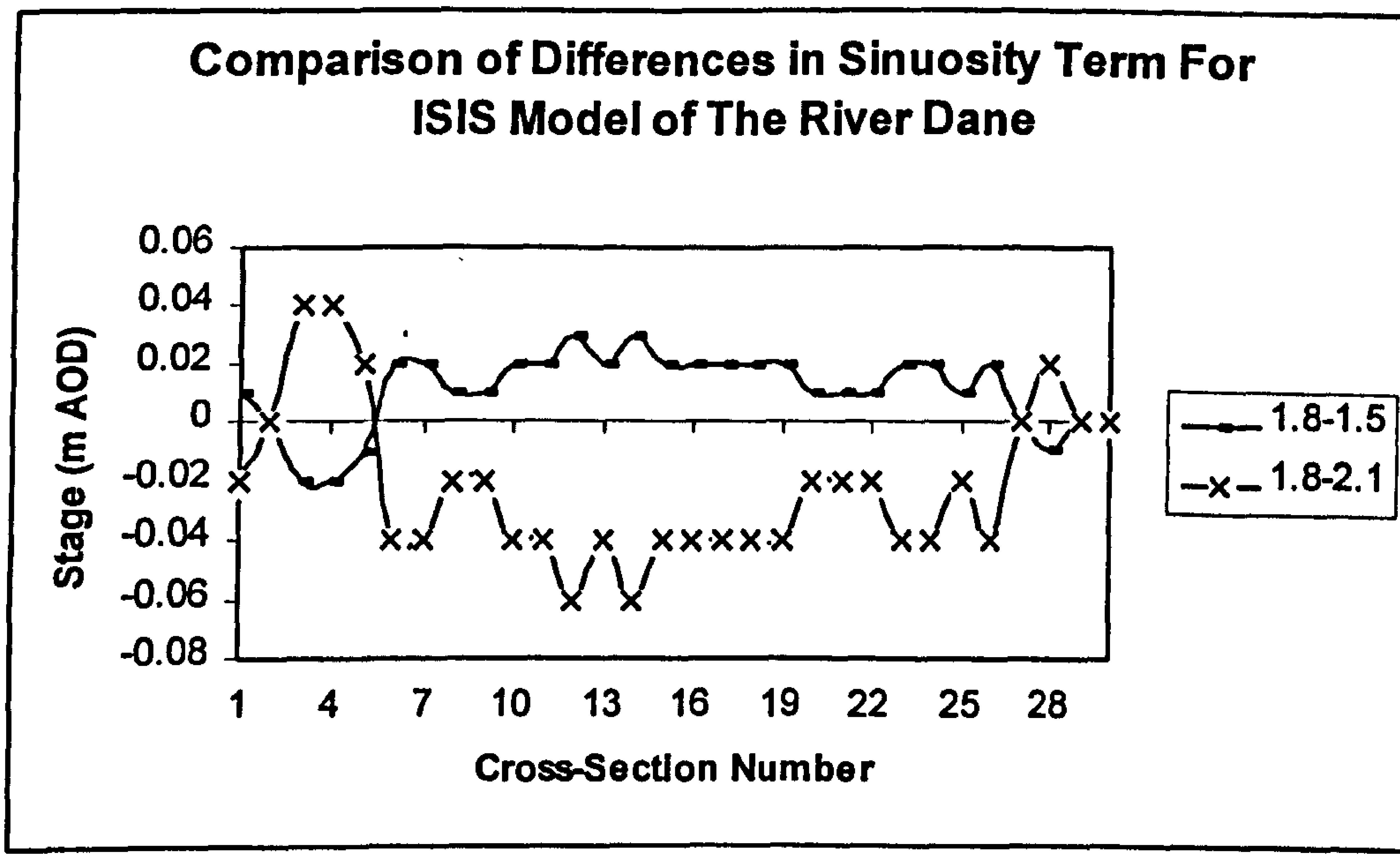


Figure 5.12 Graph of Differences predicted When Using Different Values of Sinuosity

As can be seen from Figure 5.12 when the sinuosity is either increased or decreased by 0.3 a similar pattern in water level prediction is observed. There are slight differences at some cross-sections. For example, at Cross-section 3, when the sinuosity is increased from 1.8-2.1, an over prediction of 17%, the water level rises by 0.04m but when the sinuosity is reduced from 1.8-1.5, an under prediction of 20%, the water levels are reduced by only 0.02m. This may be due to the relatively straight sections that are located at the upstream end of the model.

The general pattern shows that when the sinuosity is increased the predicted water levels will reduce by around 0.03m and when the sinuosity is reduced the predicted water levels will rise by 0.02-0.03m approximately

It has been found that an error of 17-20% in sinuosity will not have a significant effect on predicted water levels.

5.6.2 Sensitivity of Water Levels to Estimate of Meander Wavelength

This test was used to assess the required accuracy of the meander wavelength parameter. As mentioned earlier, it is possible that the estimate of this parameter for a natural river could be in error due to limited data or simply inaccurate measurement.

The actual meander wavelength for the River Dane, when assuming reach-averaging method 1, is 250m. The following test will maintain the other additional parameters and independently vary the meander wavelength parameter between 200m and 300m. This will effectively illuminate the difference in water level when the meander wavelength is in error by ± 50m.

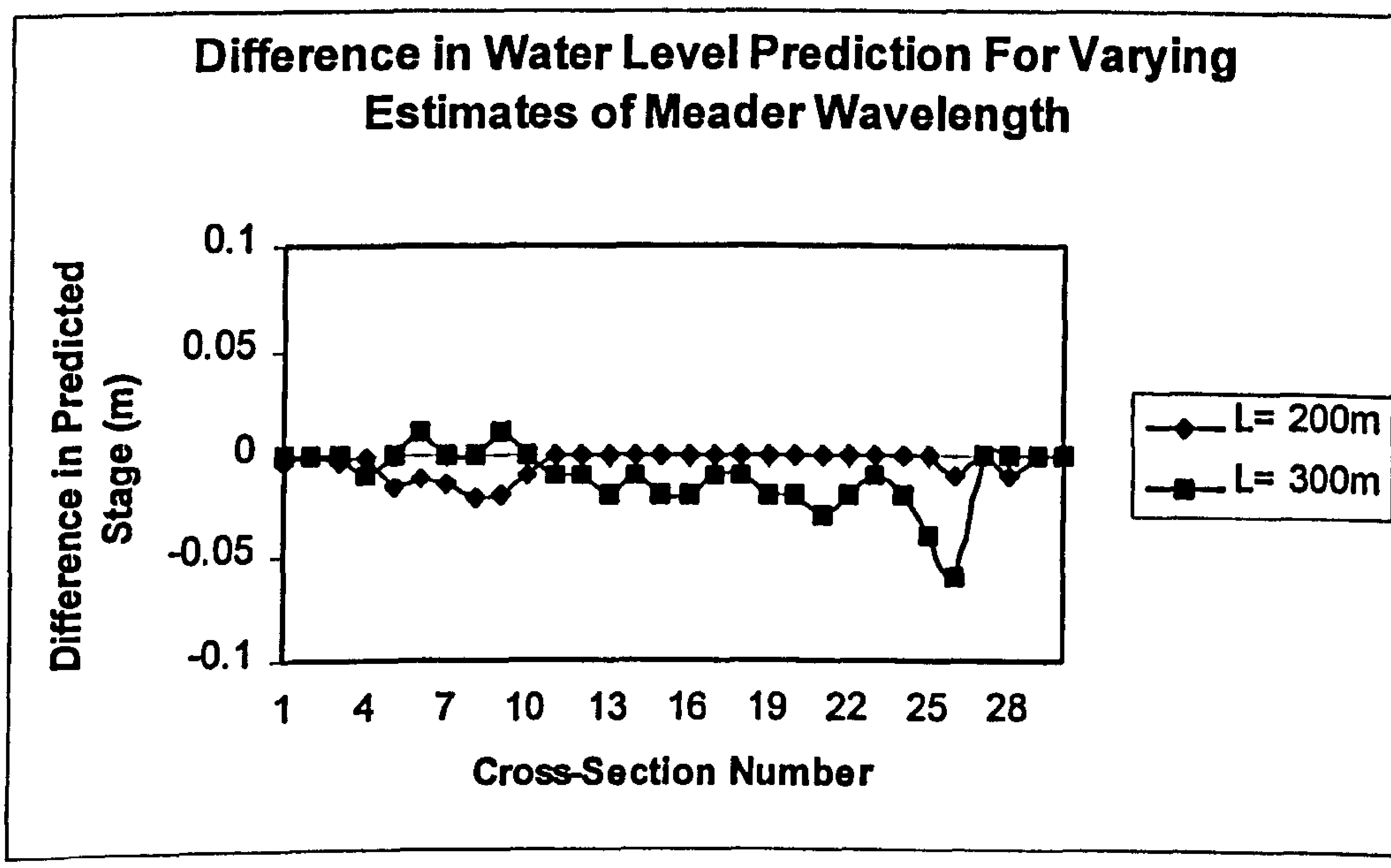


Figure 5.13 - Comparison of Water Level Predictions For Different 'L' values

Figure 5.13 above illustrates the practically negligible difference that an error of ± 50m in meander wavelength can have on water level prediction. The predicted water levels decreased as the meander wavelength increased, generally by about 0.02-0.03m. The maximum difference in water level was -0.06m when the meander wavelength was over predicted by 50m. This implies that a high level of accuracy is not required in estimating this parameter, for a natural river.

This supports the findings of James and Wark (1992) who found that an error in wavelength of ±50 % only resulted in a ±10% change in discharge. A 10% change in discharge would translate into a very small change in water level, similar to that observed above.

5.6.3 Sensitivity of Water Level Predictions to Estimate of Meander Belt Width

The estimate of meander belt width should be scaled from a plan view of the river being modelled and has been defined in Section 5.2. As this parameter is subject to interpolation, some error could be made in its estimation. It is possible that if this parameter is too big then more secondary losses are being included in the model than are present in reality and vice-versa. If more energy losses are assumed then one would expect an over prediction in water level to occur.

It should be noted however, that the changes in meander belt width will be made independently and consequently the effect on flood water levels may not in fact follow the theory. In reality if the meander belt width was greater, then the sinuosity and downstream length would also be greater. The combination of these parameters correctly defined would follow the theory. The purpose of the test is to observe what happens when the meander belt width parameter is incorrectly estimated.

It was decided to test an error in belt width of ± 30m which would be the maximum conceivable error that could be practically envisaged.

The results of this test are shown in Table 5.04 and the maximum difference is -0.05m at cross-section 5 when the meander belt width is over predicted by 30m. Figure 5.14 shows the predicted water levels for the various belt width estimates and clearly this parameter has no practically significant effect when changed independently. The magnitude of change in water level prediction is generally 0.01-0.03m. It appears that when the meander belt width is reduced then the predicted water levels rise marginally.

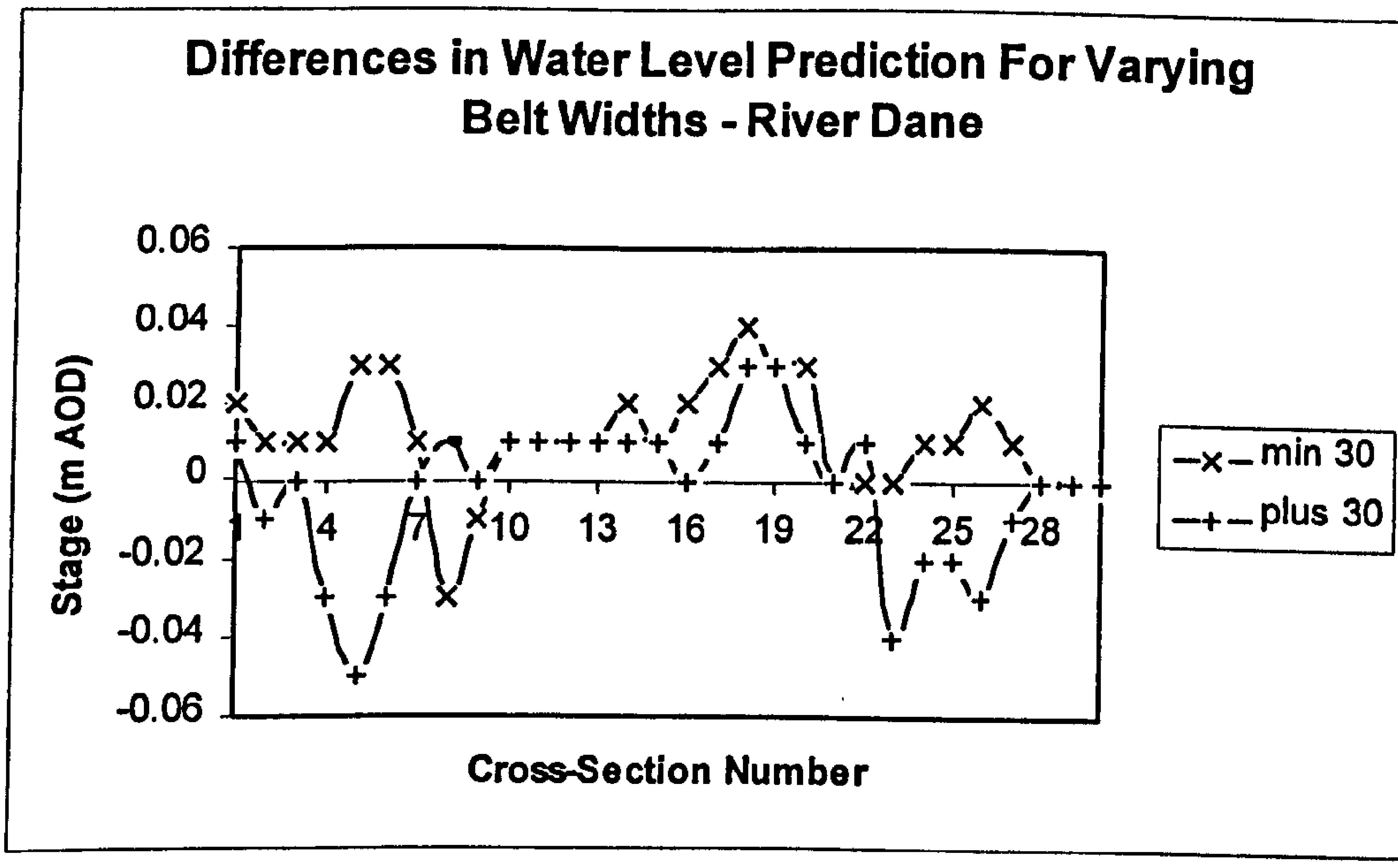


Figure 5.14 - Comparison of Water Level Predictions For Different Belt Widths

A similar pattern of results can be seen especially at cross-sections 1-10 and 22-30. The cross-sections remaining appear to suggest an increase in water level prediction whether the belt width is increased or decreased. It is suggested that these sections (11-21) are not very sensitive to alterations in this term and it can be seen that when the belt width is under predicted the water levels rise by more than if, the meander belt width, had been increased.

It is clear that potential users of the James and Wark conveyance method, when applied to a natural river like the River Dane, do not require a very accurate estimate of the meander belt width. An independent change in this term of ± 30m does not have a significant effect on predicted water levels.

In terms of applying this new conveyance method, it is clear that a high degree of accuracy is not required in estimating the 'channel parameters' and that perhaps more care should be taken in estimating correctly the flows and bed roughness parameters. The flow and roughness parameters have a limited degree of accuracy in practical river modelling and it is useful to learn that the new conveyance technique is not introducing any significant errors through the estimation of the 'channel parameters'.

Sect No	H_{-30}	H_{orig}	H_{+30}	orig −30	orig +30	Max Diff
1	17.34	17.32	17.33	-0.02	-0.01	0.01
2	17.30	17.29	17.28	-0.01	0.01	0.02
3	17.19	17.18	17.18	-0.01	0	0.01
4	17.00	16.99	16.96	-0.01	0.03	0.04
5	16.87	16.84	16.79	-0.03	0.05	0.08
6	16.44	16.41	16.38	-0.03	0.03	0.06
7	16.24	16.23	16.23	-0.01	0	0.01
8	15.99	16.02	16.03	0.03	-0.01	-0.04
9	15.92	15.93	15.93	0.01	0	-0.01
10	15.82	15.81	15.82	-0.01	-0.01	0
11	15.78	15.77	15.78	-0.01	-0.01	0
12	15.75	15.74	15.75	-0.01	-0.01	0
13	15.68	15.67	15.68	-0.01	-0.01	0
14	15.64	15.62	15.63	-0.02	-0.01	0.01
15	15.56	15.55	15.56	-0.01	-0.01	0
16	15.37	15.35	15.35	-0.02	0	0.02
17	15.11	15.08	15.09	-0.03	-0.01	0.02
18	14.96	14.92	14.95	-0.04	-0.03	0.01
19	14.79	14.76	14.79	-0.03	-0.03	0
20	14.63	14.60	14.61	-0.03	-0.01	0.02
21	14.46	14.46	14.46	0	0	0
22	14.41	14.41	14.42	0	-0.01	-0.01
23	14.31	14.31	14.27	0	0.04	0.04
24	14.22	14.21	14.19	-0.01	0.02	0.03
25	14.10	14.09	14.07	-0.01	0.02	0.03
26	13.80	13.78	13.75	-0.02	0.03	0.05
27	13.76	13.75	13.74	-0.01	0.01	0.02
28	13.60	13.60	13.60	0	0	0
29	13.27	13.27	13.27	0	0	0
30	13.50	13.50	13.50	0	0	0

Table5.04 Comparison of Water Level Predictions For Different Meander Belt Widths

5.6.4 Discussion

As mentioned earlier, other parameters that could be incorrectly defined by the practising engineer when using the James and Wark Method are the side slope and flood plain slope. The error in side slope should not be significant as any method of measuring this term should not provide a significantly different estimate. The flood plain slope can also usually be accurately measured during the topographical survey

From the previous tests it is clear that when the James and Wark Method is applied in a natural environment, the estimates of the additional parameters do not need to be to a high level of accuracy. Perhaps care needs be taken in defining the sinuosity estimate as it could lead to inaccurate water level predictions if significantly in error. For the case of the River Dane it is clear that the water level prediction is not sensitive to the meander wavelength or the meander belt width terms. It is also assumed that the side slope will not have a significant effect either as proposed by James and Wark (1992).

The tests undertaken have indicated the effects on water level prediction that inaccurate estimates of the additional parameters may cause.

The tests on reach length also proved to be insignificant as the difference in predicted water levels were practically negligible. The FCF and River Dane tests indicated that as long as the additional parameters were defined for the assumed reach then the reach length was not important. Wark (1998) indicated that reach averaged cross-sections and parameters were required, however this large-scale field study has proved otherwise.

The highly meandering nature of the River Dane has made it relatively easy to apply the James and Wark Method. The additional parameters were well defined and simple to estimate. However, most natural rivers could not be classified so easily and it is now intended to apply the James and Wark Method to the River Kelvin, in Glasgow, which is less well suited to the method as was the River Dane.

Chapter 6 The River Kelvin

6.0 Numerical Modelling Of The River Kelvin

The River Kelvin is a major river system draining the area to the northwest of the city of Glasgow. In December 1994, the river experienced a significant flood with a 1 in 200 year return-period. This resulted in the deaths of two people and millions of pounds of damage. As a direct result Halcrow Crouch Consulting Engineers were commissioned by East Dunbartonshire Council to assess the flood risk in the Kelvin valley and propose flood protection measures. As part of this study the Department of Civil Engineering at Glasgow University were employed to develop a computer model of the River Kelvin using in-house software, Pender (1985).

In this research project a new model of the River Kelvin has been developed using ISIS. The purpose of this was to utilise the available data to further test and evaluate the conveyance calculation routines developed in this project and described in chapter 4. The existence of the other model also provided the opportunity to compare model performance and predictions.

Of particular interest will be the comparison of a Divided Channel Method calibration with that obtained from a James and Wark Method calibration. This will be of significant practical interest as it would be expected that less adjustment of the Manning's 'n' term would be required with the James and Wark Method.

6.1 Catchment Area of The River Kelvin

The catchment of the River Kelvin, upstream of Killermont on the western outskirts of Glasgow, shown in Figure 6.02 extends to some 335 km^2, and ranges in elevation from 578m AOD at Earl's Seat to around 27m AOD at Killermont golf course. From here the River Kelvin flows through the more urbanised areas of Maryhill and Kelvinside before discharging into the River Clyde in the town of Partick.

A particular feature of the River Kelvin is that the ground level falls only 14m over the 20.5 km length between Kilsyth and Killermont in Glasgow. The average gradient of the River Kelvin channel over this reach is 1 in 1450, which is extremely flat, locally the gradient can vary between 1 in 1000 and 1 in 2500.

Its main tributaries are the Glazert Water, Luggie Water and Allander Water, these Rivers are gauged at Milton of Campsie, Oxgang and Milngavie respectively. The Glazert Water is the largest tributary measuring approximately 7.164km in length from Lennoxtown down to its confluence with the main reach of the Kelvin. The Glazert is approximately 12m wide and is steep in places, flowing mainly through agricultural land. The small urban areas of Lennoxtown and Milton of Campsie have only local areas that are vulnerable to flooding, however, other areas narrowly avoided inundation during the December 1994 flood event.

The Luggie water is 4.134km in length and flows through the town of Kirkintilloch and is approximately 10m wide. A large amount of vegetation and debris is present at the downstream end of this reach. Extensive flooding was observed on this tributary during the December 1994 event.

The Allander flows 4km from the town of Milngavie to its confluence with the River Kelvin and is generally quite clean and winding. The Allander is smaller in width than the other tributaries in that it is only 8m across. It flows through mainly agricultural land where there is extensive areas to attenuate flood flows.

There are also two significant ungauged burns that contribute to the flow in the Kelvin, namely the Garrel Burn at Kirkintilloch and the Park Burn at Hayston. They are both approximately 4m wide and are in close proximity to housing estates and are therefore of significant interest in assessing flood risk.

The main reach of the River Kelvin flows generally through agricultural land and the small towns of Kilsyth, Kirkintilloch, Torrance, Balmore and Bearsden. Extensive flooding has been observed in these towns and there is significant interest in flood models of these regions.

The flood event of $11^{th}/12^{th}$ December 1994 has been analysed in detail due to the widespread inundation and damage that occurred. Flooding occurred over the entire 20.5 km reach from Kilsyth to Bearsden. The town of Kirkintilloch experienced the worst effects of the flood not least because it is situated at the confluence of the Kelvin with the Glazert Water, Luggie Waters

For the purposes of this research project only the 20.5 km reach from Kilsyth to Bearsden has been modelled. (See Fig 6.01)

This reach has many complicated features that could prove difficult to model such as large Railway Embankments that restrict the movement of flood plain flow. Also, in many locations the River Kelvin has spoil banks (see Photograph 2) which restrain the main channel flows from spilling on to the flood plain. Further complications arise due to the development of housing estates and industrial units in the flood plains of the main Kelvin and its tributaries.

Figure 6.01 River Kelvin Location Map

KILSYTH
DUNBARTONSHIRE
KIRKINTILLOCH
GLASGOW
Milton of Campsie
Queenzieburn
Auchinstarry
Twechar
Craigend Muir
Craigmaddie Muir
Blairskaith Muir
Mugdock
Torrance
Balmore
Bishopbriggs
Lenzie
Auchinloch
Wester Auchinloch
Stepps
Muirhead
Chryston
Moodiesburn
Bridgend
Mollinsburn
Glenboig
Gartcosh
Garnkirk
Lochwood
Millerston
Craigend
Cadder
Springfield
Birdston
Westerton
Bardowie
Easterton
Drumbreck
Croy
Dullatur

Figure 6.02 Catchment Area Map

HYDROMETRIC NETWORK IN THE CATCHMENT UPSTREAM OF KILLERMONT

SCALE 1 : 50000

Photograph 1 The River Kelvin – Looking downstream from section 72 (Balmuildy)

Photograph 2 Glazert Water flowing into the River Kelvin (From the left)

Photograph 3 Railway Embankments on The River Kelvin

Photograph 4 Spoil Banks at Cross-section 63-64 Bardowie

Photograph 5 The Glazert Water

Photograph 6 The Luggie Water

Photograph 7 The Allander Water

6.2 Hydrology of The River Kelvin Catchment

December 1994 Flood Event - Meteorological Office - Precipitation

During the weekend of 10th-12th December 1994 extensive flooding was observed in the west of Scotland with the Kelvin catchment being one of the worst effected. The floods were due to prolonged rainfall of 170mm, or more in places, over 2 days when a belt of warm and moist air associated with a slow moving front was directed over Scotland. According to the Meteorological Office the prolonged rainfall event that occurred over the River Kelvin catchment on 10th and 11th of December 1994 has a return period of between 1 in 300 and 1 in 1000 years depending on the location and altitude of the rainfall gauge within the catchment. The duration led to the whole catchments of the principal rivers contributing to the run-off and, flows well in excess of any previous recorded peaks occurred. For example, Killermont Gauge on the main reach of the Kelvin recorded a peak flow of 265.70 m^3/s and was the highest recorded since records began in September 1979. It is estimated that the return period of this flow is 1 in 200 years. River flows are discussed in more detail in section 6.3.

It is important to distinguish between the return period for rainfall, which is a function of both intensity and duration of precipitation, and the flood return period, which is not only a function of the rainfall pattern, but also depends on the catchment characteristics.

The catchment characteristics comprise the catchment area, average annual rainfall, soil conditions, drainage channels serving the catchment, the slope of the channels and stream frequency within the catchment. The return periods for rainfall and flow are therefore unlikely to be the same. A detailed description of these parameters and their interaction can be found in "The Flood Studies Report", NERC (1975).

It should also be noted that heavy rain had been observed in the preceding days, which would have led to significant ground saturation. This being where the voids below ground level being full therefore any additional water attempting to infiltrate would simply run-off or pond on the surface. The combination of saturated soil and prolonged rainfall provides ideal conditions for a substantial flood event.

6.3 River Flow Information

In order to assess the flows in the Kelvin for the December 1994 event contact was made with the Scottish Environment Protection Agency (SEPA). It was intended to obtain information regarding gauged river flows and stage discharge relationships at gauging stations. SEPA currently operate six gauging stations within the River Kelvin catchment (Refer to Figure 6.02). The information obtained from SEPA included

- Peak stage / discharge for each year over the past ten years
- Current stage / discharge relationships and their upper limits of calibration
- Flood frequency curve over the full period on record and the past ten years
- Hourly flow data at each of the gauging stations

This information was used to provide boundary conditions for the simulation of the December 1994 flood event.

6.3.1 Gauging Stations Within The Kelvin Catchment Area

There are five gauging stations on the River Kelvin and its tributaries where a significant record of data was available, namely, Milton, Oxgang, Dryfield, Milngavie and Killermont (see Figure 6.02). Of the five gauging stations all except one recorded the December 1994 flood event as the highest on record. The two principal gauging stations on the main River Kelvin are located at Dryfield just downstream of Kirkintilloch, and at Killermont, near Glasgow, at the downstream limit of the study. The period of record extends over 36 years at Dryfield and 48 years at Killermont. The Gauging Stations that have been used in this study are all similar in that they use hydrostatic pressure measurements, see Photographs 8 and 9, to record the water depth

Photograph 8 Dryfield Gauging Station

Photograph 9 Dryfield Gauging Station

The Glazert Water is gauged at the downstream end of its reach at Milton of Campsie and has been in operation since September 1968. The peak flow observed in this tributary, since records began, was 87 m^3/s on the 11th of December 1994.

The Luggie water is gauged at Oxgang, which is some 2087m from the confluence with the Kelvin, and records date back to October 1974. The peak flow observed in this tributary, since records began, was 110 m^3/s again on the 11th of December 1994.

The Allander Water gauge is situated in Milngavie and was installed in November 1972. The peak flow observed in the Allander, since this time, is 49.75 m^3/s in March 1990. The Allander Water gauging station at Milngavie recorded the peak flow in December 1994 as a close second. The March 1990 event was coincident with a well-documented record flood level at Loch Lomond in a neighbouring catchment.

There are also two significant burns that contribute to the Kelvin, namely the Garrel Burn and the Park Burn. These Burns are not gauged and consequently there is no information regarding peak flows and water levels, however, as they are of a significant size (approx. 4m wide) they have been included as tributary inflows to the model.

The stage / discharge curves at the main gauging stations are presented in Figures 6.03, 6.04 and 6.05 (See Appendix 5). It is important to appreciate that the various relationships have limited degrees of accuracy, which are dependent upon the upper limit of calibration of the station. By inspection of Figures 6.03 to 6.05 it is apparent that the record peak flow has been obtained by extrapolating to at least twice the upper limit of calibration for each of the gauging stations.

In addition to the functioning gauging stations there is also a disused gauging station at Bridgend, upstream of the confluence with the Glazert Water. This gauge has been out of use for some thirteen years, however, the most recent stage / discharge relationship was obtained from SEPA to allow an estimate to be made of the peak flow in the River Kelvin at this location during the December 1994 flood event.

Picture 7 Flooding in Kirkintilloch December 1994

Picture 8 Flooding in Kirkintilloch December 1994

For unsteady flow simulations the computer model required inflow hydrographs at the upstream boundary, Garrel Burn, Glazert Water, Luggie Water, Park Burn and Allander Water. These were constructed by using the recorded flow data from the various gauging stations. For each flood event modelled, 5 days of flow measurements were extracted from each gauging station. Essentially, the day of highest measured flow was identified and two days of data either side of it were used to construct inflow hydrographs at the upstream end of the Kelvin and at all the tributary confluences.

6.4 Kelvin Model - Additional Flood Plain Data Included

The computer model includes 87 surveyed cross-sections of the main Kelvin, at intervals of 150-250m, over a 20.5 km reach. Among the Surveyed cross-sections there are two river gauging stations and six bridges. In addition, there are three main tributaries and two small burns contributing to the main river flow.

The bridges have been modelled using the techniques that are available within the ISIS software. These enable the modelling of arch and standard bridges. For flat bridges the USBPR method for calculating bridge afflux was used and the HR Wallingford Arch bridge routine used for the arch bridges.

The cross-sections used in the model combine the topographical survey data with additional data scaled from OS maps. The scaling was required in order to improve the flood plain resolution in the model, and ensure that each cross-section covered the full width inundated in December 1994.

6.4.1 Survey Information

A full topographical survey of the River Kelvin and its main tributaries was carried out by others and cross-sectional drawings produced. These drawings were directly used to obtain co-ordinate point data required by ISIS. Figures 6.06, 6.07 and 6.08 illustrate some typical cross-sections used in the River Kelvin model.

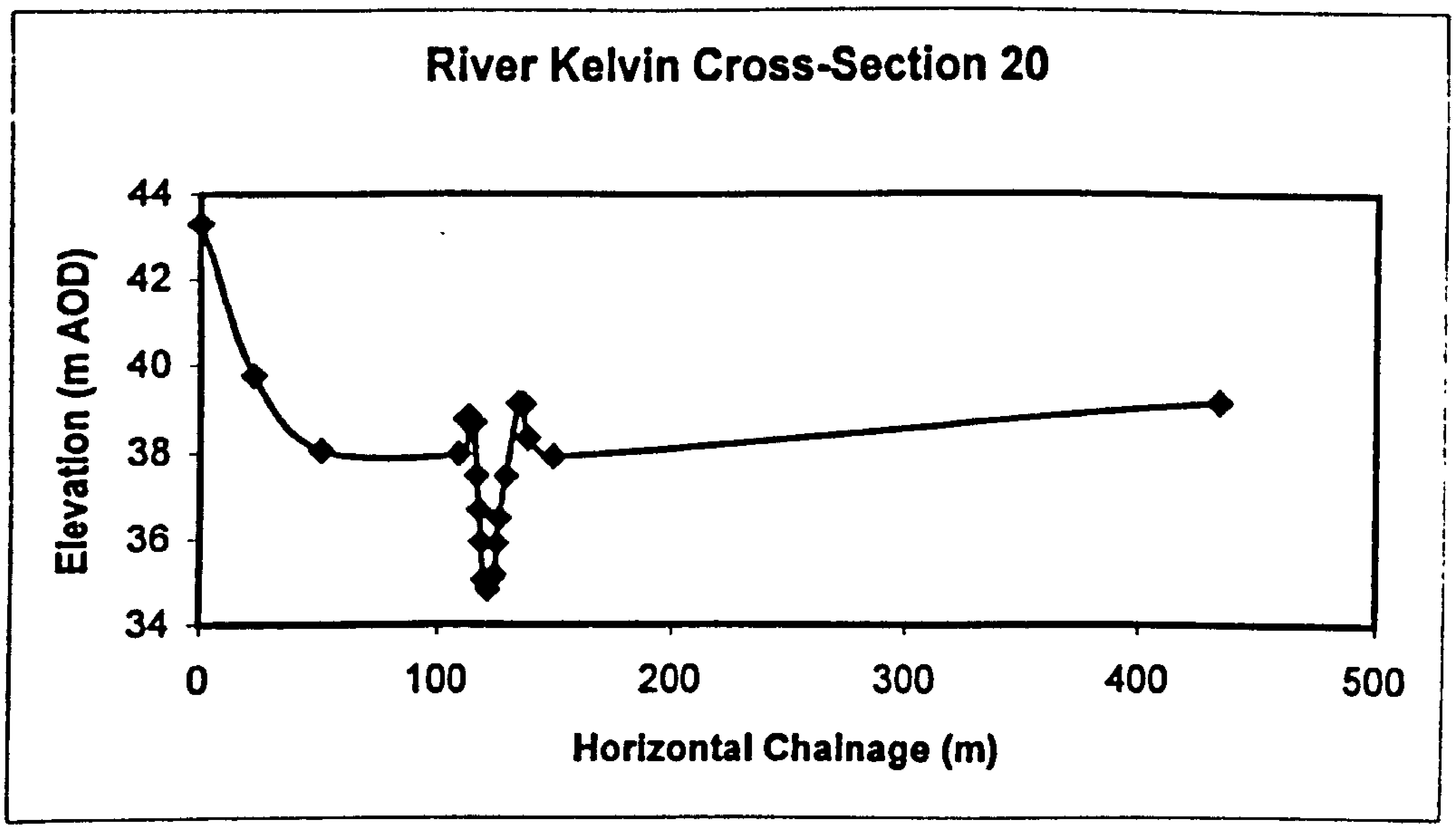


Figure 6.06 - River Kelvin Surveyed Cross-Section 20

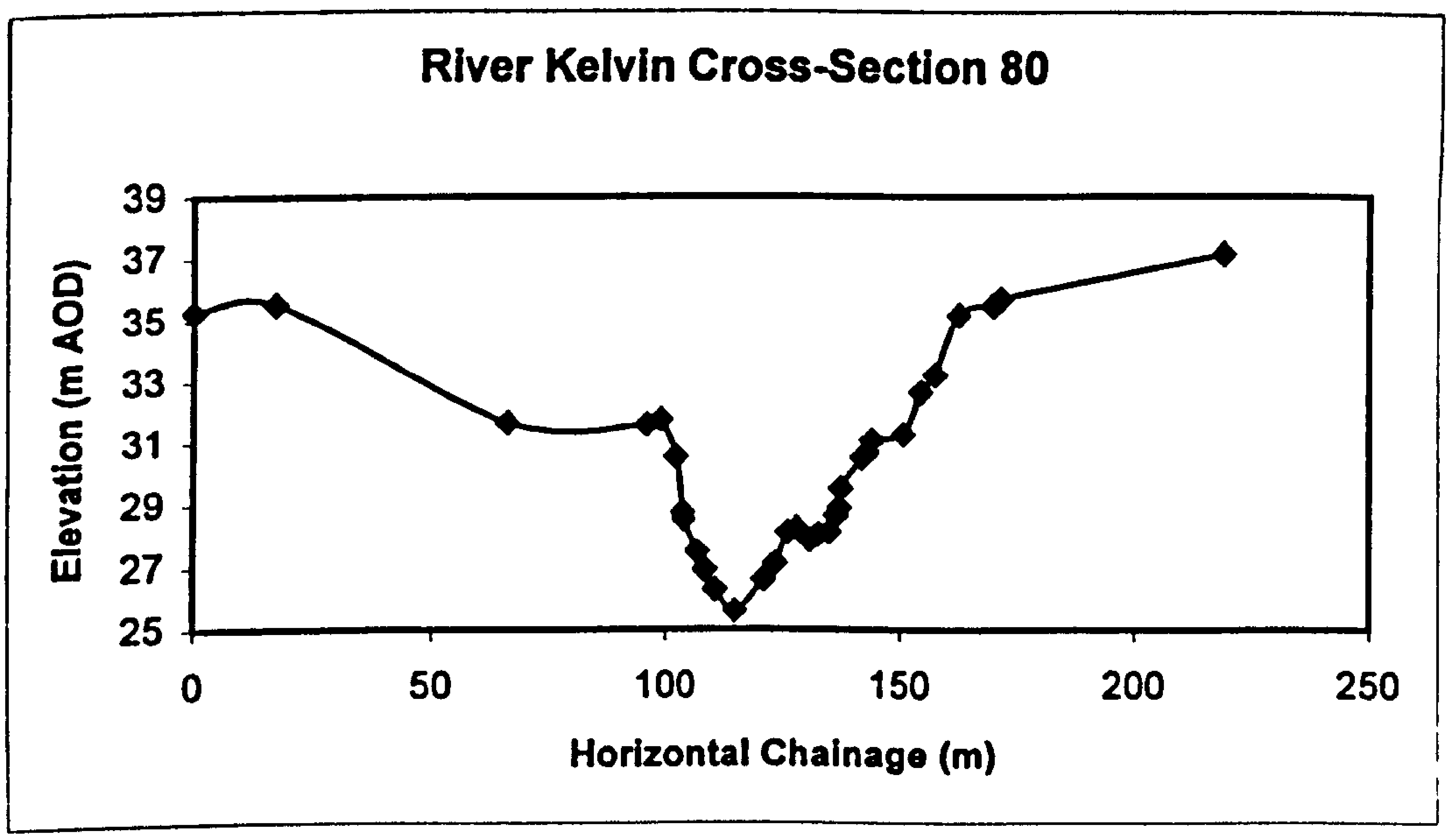


Figure 6.07 - River Kelvin Surveyed Cross-Section 49

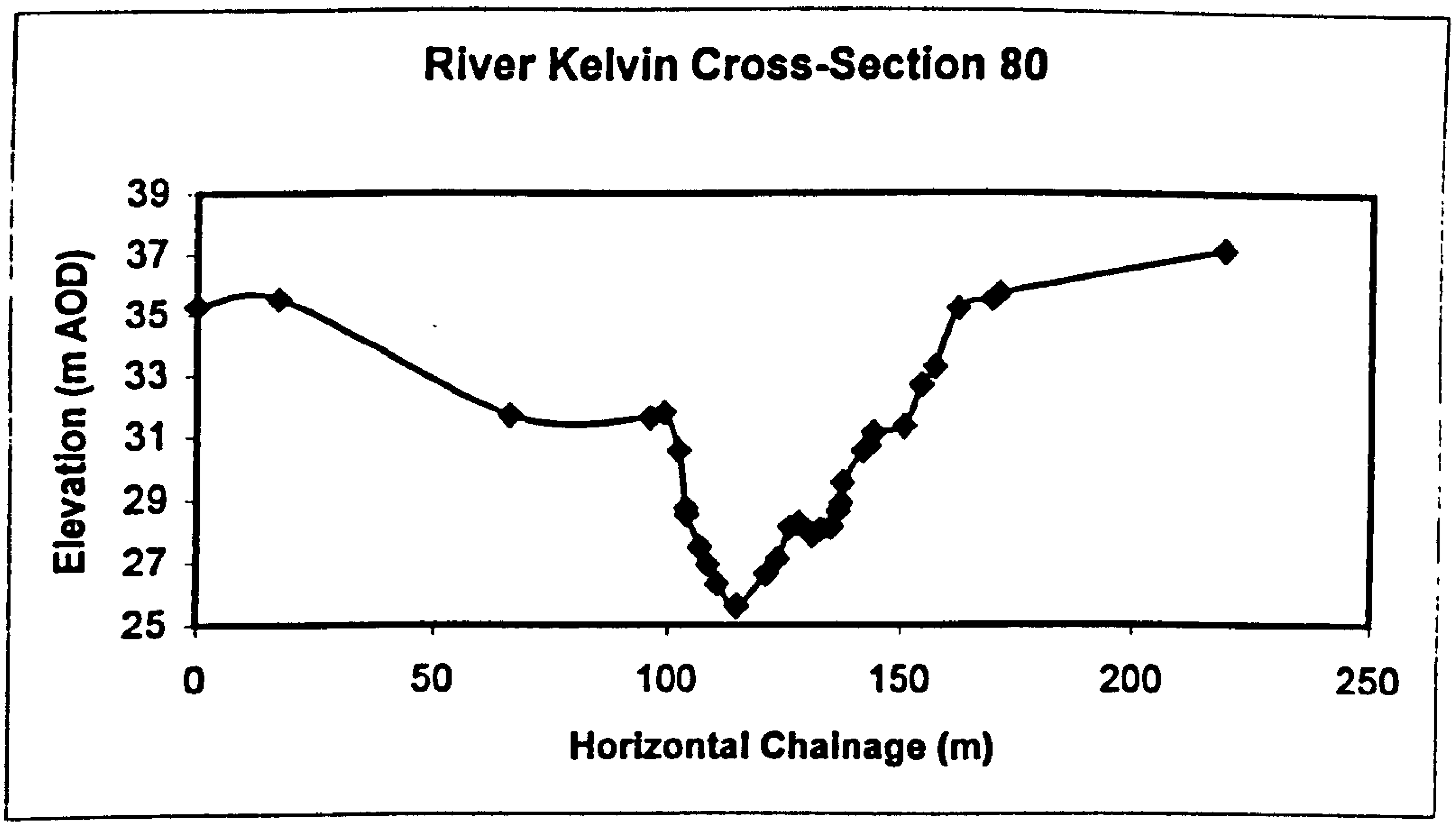


Figure 6.08 - River Kelvin Surveyed Cross-Section 80

6.4.2 Downstream Boundary

A rating curve is used as the downstream boundary condition in the model. Inspection of the equation provided by SEPA for the Killermont Gauge suggested that it had significantly overestimated the peak flow at this location for the December 1994 flood event. The equation used in the model was therefore a modified version.

6.5 Calibration

Dryfield and Killermont Gauging stations (see Fig 5.02) were predominately used to calibrate the model along with some observed water levels that were obtained for the December 1994 flood event. Specifically the following flood events were considered

24th-28th	October 1995	
N/A	February 1998	
18th-22nd	September 1985	
9th-13th	December 1994	(Simulated Event)

The February 1998 event was evaluated but not used in detail as it was very similar to the October 1995 event and it was considered that its use would not improve the calibration of the model.

For each of the above flood events, 5 days of flow measurements were extracted from the SEPA records. Essentially, the day of highest measured flow was identified and two days of data either side of it were used to construct inflow hydrographs at the upstream end of the Kelvin and at all the tributary confluences.

Figure 6.09-6.11 shows the Allander Water, Luggie Water and Glazert Water inflow hydrographs respectively, that were used in the model calibration, along with the December 1994 hydrograph.

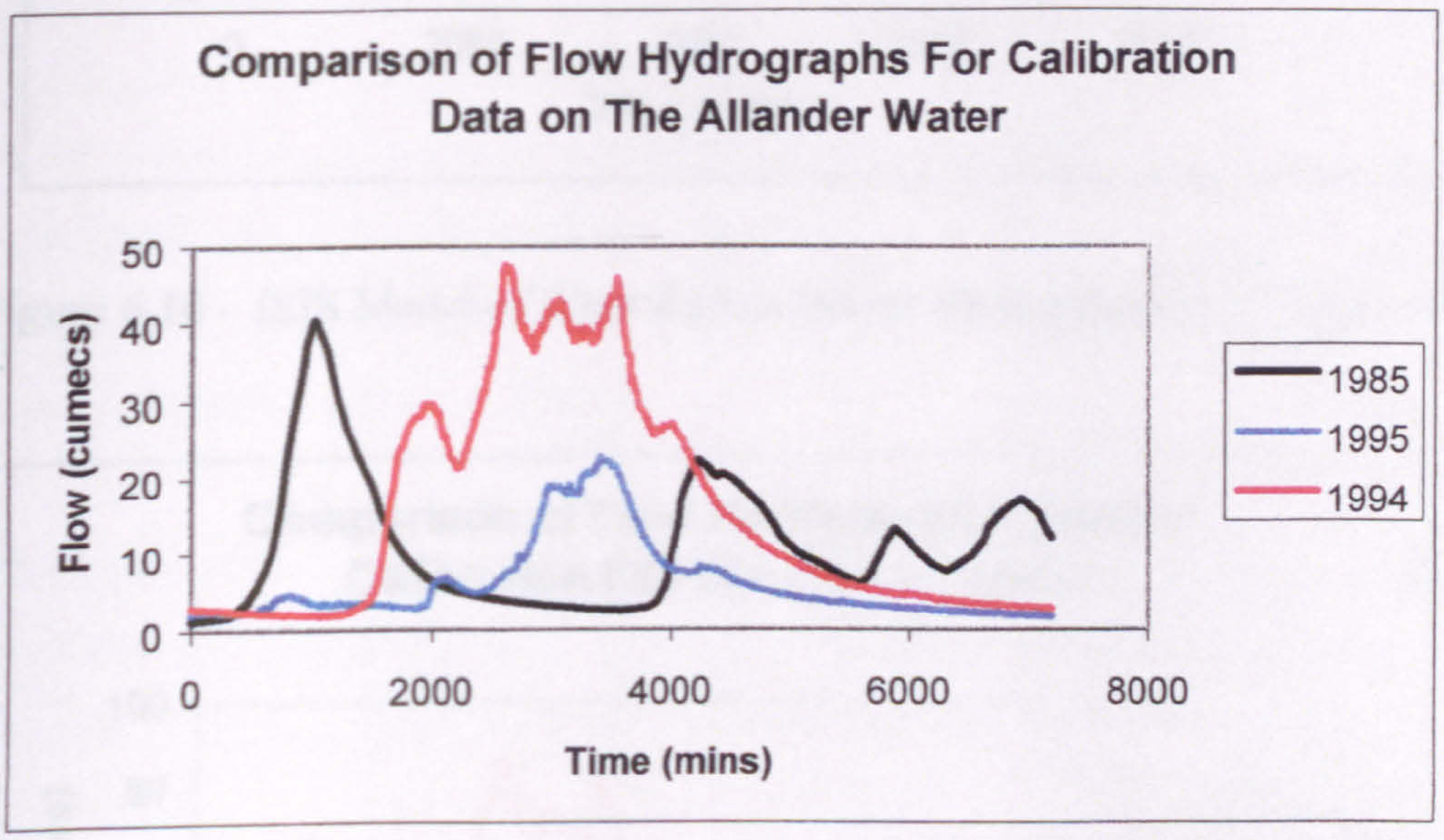


Figure 6.09 – ISIS Model of River Kelvin Inflow Hydrographs For Allander Water

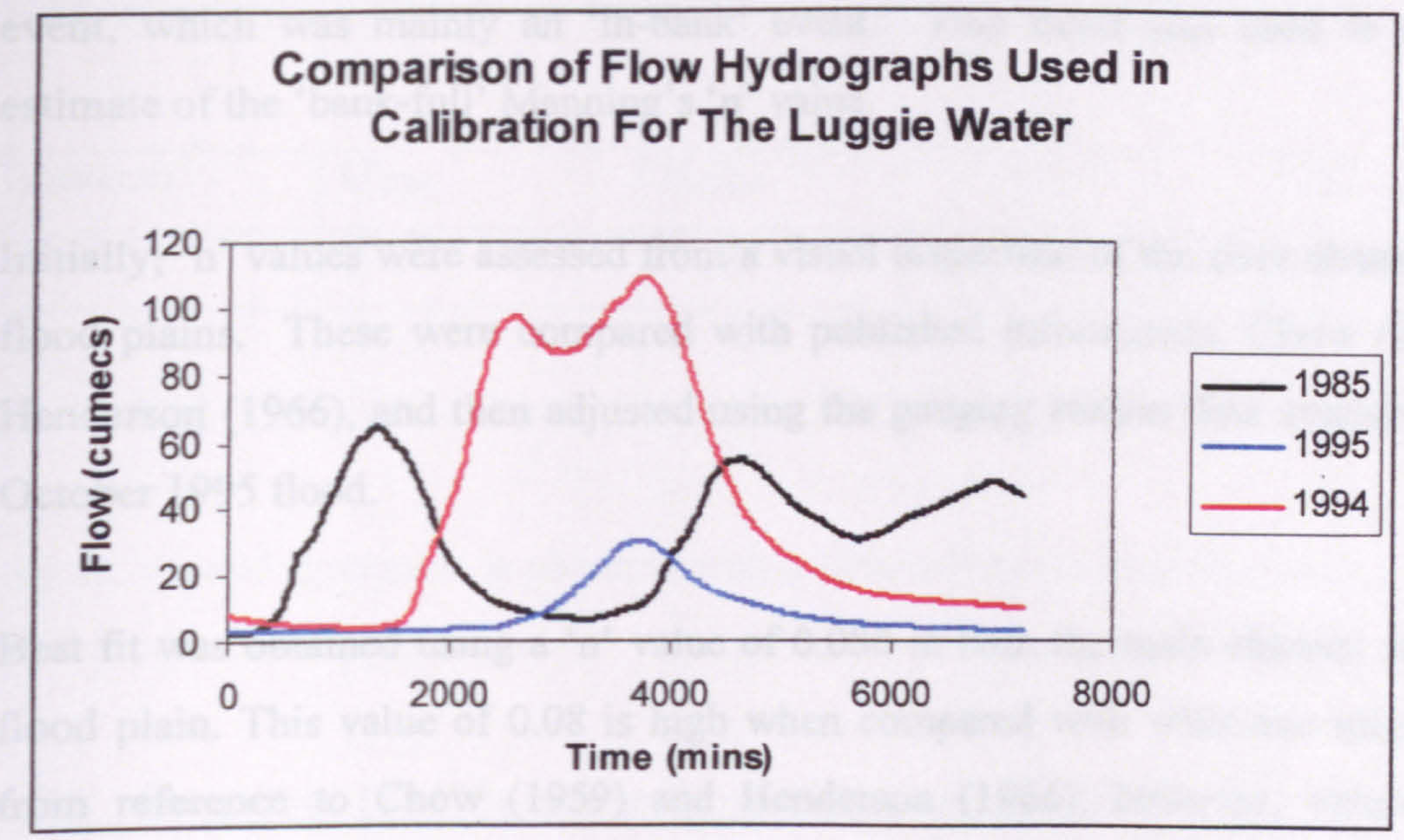


Figure 6.10 – ISIS Model of River Kelvin Inflow Hydrographs For Luggie Water

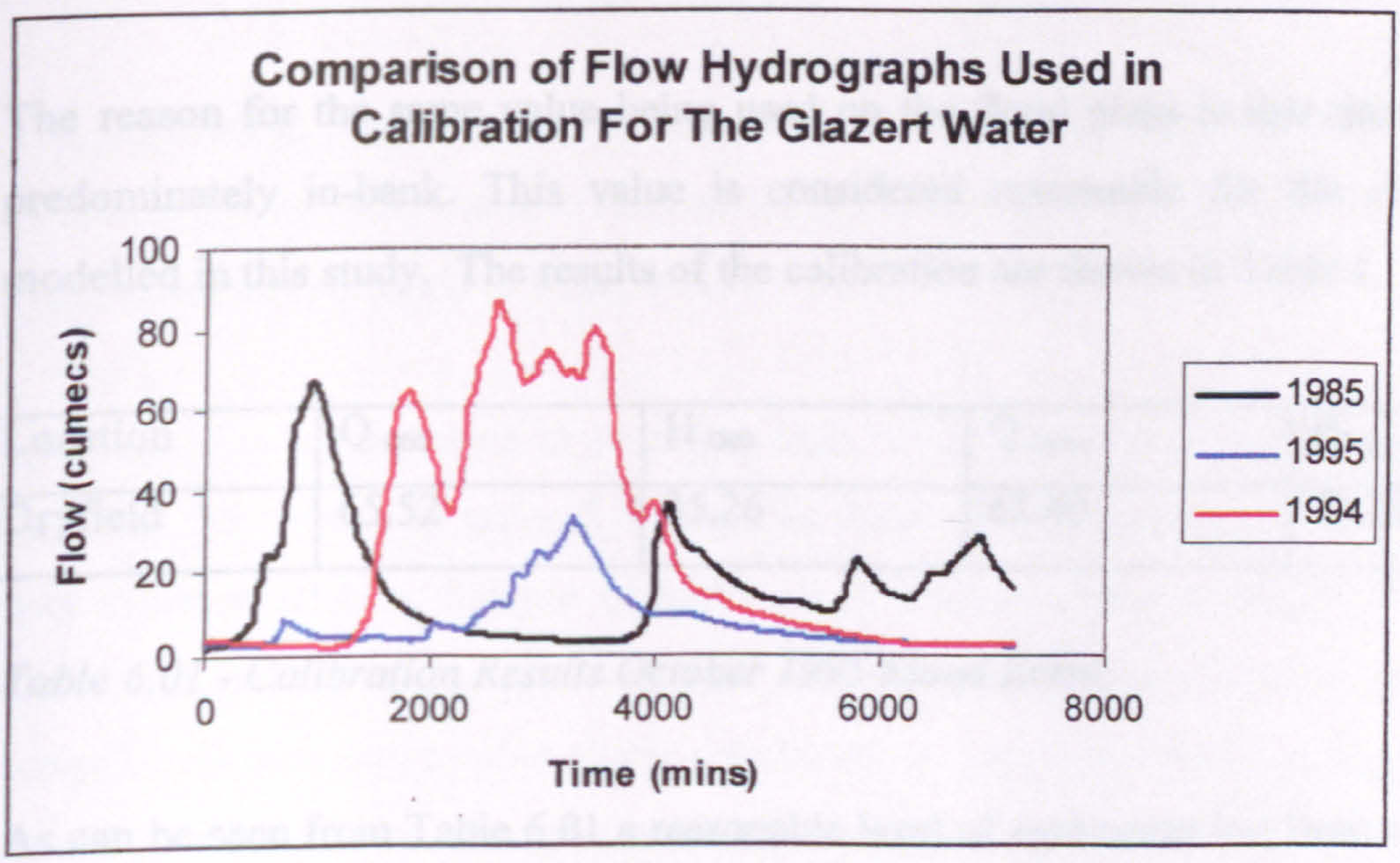


Figure 6.11 – ISIS Model of River Kelvin Inflow Hydrographs For Glazert Water

The calibration of a numerical model involves the systematic adjustment of channel roughness to alter predicted water levels until a reasonable agreement is obtained with observed water levels. To obtain a good calibration there should be a significant amount of observed flow and water level information. Essentially, the calibration can only be as good as the observed information.

The calibration of the River Kelvin model proceeded by using the October 1995 flood event, which was mainly an 'in-bank' event. This flood was used to obtain an estimate of the 'bank-full' Manning's 'n' value.

Initially, 'n' values were assessed from a visual inspection of the river channel and its flood plains. These were compared with published information, Chow (1959) and Henderson (1966), and then adjusted using the gauging station data available for the October 1995 flood.

Best fit was obtained using a 'n' value of 0.080 in both the main channel and on the flood plain. This value of 0.08 is high when compared with what one might expect from reference to Chow (1959) and Henderson (1966), however, values of this magnitude are not unknown in numerical models. In addition, Wilson (1998) obtained 'n' values in the region 0.08-0.1 on the River Blackwater by back calculation from flow and stage observations

The reason for the same value being used on the flood plain is that this flood was predominately in-bank. This value is considered reasonable for the rivers being modelled in this study. The results of the calibration are shown in Table 1.

Location	Q_{OBS}	H_{OBS}	Q_{DCM}	H_{DCM}
Dryfield	65.52	35.26	63.40	35.20

Table 6.01 - Calibration Results October 1995 Flood Event

As can be seen from Table 6.01 a reasonable level of agreement has been achieved at Dryfield, with a 3% difference in peak flow and a –0.06m difference in peak water level.

6.5.1 September 1985 Flood Event

The flood event of September 1985 was a significant "out of bank flood" and is used to verify the main channel roughness and calibrate the flood plain. Again, conditions at Dryfield Gauge are compared to assess the quality of the calibration. The main channel 'n' value of 0.080 and flood plain 'n' value of 0.10 was used for this analysis.

Location	Q_{OBS}	H_{OBS}	Q_{DCM}	H_{DCM}
Dryfield	95.00	36.00	108.62	35.98

Table 6.02 - Calibration Results September 1985 Flood Event

Again, a good agreement is obtained with a 14% difference in peak flow and a -0.02m difference in peak water level.

6.5.2 December 1994 Flood Event - *Verification Results*

For this Flood event the calibration process indicated a main channel n_c = **0.08** would be sufficient however the flood plain roughness would have to increase significantly in order to match with the observed water levels.

The following results show a reasonable level of accuracy despite using the Manning's n value as the sole lumped energy loss / resistance parameter. The flood plain value was assumed to be $\mathbf{n_{fp} = 0.35}$

Although this value seems to be rather high it is required due to the large areas of flooding that were encountered during this flood event. In some locations flooding was experienced to a distance of 800m away from the main channel. It should be noted that not all of the observed data was recorded during the flood event, some water levels were surveyed at a later date based on guidance from local residents. The accuracy is considered to be reasonable.

Sect No.	Observed	ISIS 94	Difference	
	m AOD	m AOD	OBS/ISIS	
26	38.90	38.71	-0.19	
28	38.50	38.69	0.19	
29	38.50	38.68	0.18	
32	38.00	37.95	-0.05	
34	37.70	37.83	0.13	
36	37.50	37.53	0.03	
38	37.10	37.05	-0.05	
41	36.65	36.69	0.04	*Dryfield*
49	35.85	36.06	0.21	
57	35.00	34.96	-0.04	
64	34.90	34.76	-0.14	
74	32.80	32.99	0.19	
87	30.82	30.72	-0.10	

Table 6.03 – Maximum Flood Levels For December 1994 Flood Event

From the results above it appears that there are five locations where the calibration is not within 200 mm (approximately) of the observed value. Namely, cross-sections 26, 28, 29, 49 and 74. However, it should be noted that given the accuracy of the data the calibration is reasonable. At cross-section 26 there is an under-prediction of 190 mm which although reasonably close is not as accurate as other locations. However, the calibration at this location is hindered by sections 28 and 29 being in close proximity. When reasonable agreement is obtained at 28 and 29 an unacceptable level of accuracy for cross-section 26 is obtained. As a result a balance has been found that allows a reasonable level of accuracy at these locations. In physical terms the poor calibration at section 26 could be due to the limitations of the ISIS Arch bridge routine that was fundamentally developed for small-scale prototype bridges and may be limited in its practical application.

Cross-section 49 is just upstream of Torrance bridge and it is conceivable that this is influencing conditions in this location. The over prediction of 210 mm at least suggests the model is conservative and could be improved if further survey work was carried out in this area.

At section 64 there is an under-prediction of 140 mm, this is undoubtedly due to the misrepresentation of a bridge at this location. This bridge was missed in the original 1996 survey and is only approximated in the model. It is recommended that a survey of this bridge is undertaken to improve the model data. The discrepancy of 190 mm at section 74 may be due to the large areas of inundation at this location.

Picture 9 Missing Bridge at Cross-Section 64

6.6 Calibration of The River Kelvin Using The James and Wark Method

The ISIS model of the River Kelvin was discussed in the previous sections. This section discusses the calibration necessary when using the James and Wark Method. This means that instead of bed friction being used as the only source of energy loss, secondary losses associated with flow interaction will also be accounted for. One would therefore expect the value of Manning's 'n' required to obtain agreement with observed overbank water levels will be less than that required for the divided channel method.

In order to apply the James and Wark method within a one-dimensional modelling tool, the following data is required:

1. a reach averaged cross-section
2. estimate of reach sinuosity
3. estimate of meander belt width
4. estimate of cross-section side slope
5. estimate of flood plain slope
6. estimate of meander wavelength

It should be noted that for a natural river with extensive longitudinal variation, obtaining these values requires one to exercise considerable engineering judgement.

The methods used to obtain these parameters for the River Kelvin are detailed in the following.

6.6.1 Reach Average Cross-Section

A reach-averaged cross-section is one that is representative of a given reach of river. The difficulty associated with obtaining this parameter is that a natural river is constantly varying in geometry and consequently, defining a single representative cross-section for a reach of river is problematic.

As has been discussed in Chapter 4 section 7, a length of reach has to be defined. In a one-dimensional model each user-defined cross-section is assumed to be representative of half the distance up and downstream from its location, Cunge (1980). For the purposes of the Kelvin model this 'representative length' has been chosen as the reach length. This has the advantage of using the surveyed cross-section without having to produce some average version. Indeed, it would be impractical to use anything other than what has been surveyed.

In addition, if a longer reach length had been selected then it would be harder to justify the use of a single representative cross-section due to possible variation of parameters with the reach length.

The use of the 'representative length' as the reach seems reasonable and fits comfortably in the framework of a one-dimensional model. Wark (1997) implied that several cross-sections were required to model any single bend which is likely to be difficult to achieve in practice as the cost of conducting a topographical survey can be the most expensive aspect of any numerical modelling exercise.

As this study is the first to apply the James and Wark Method, within an industry standard one-dimensional river model, over a significant length of a natural-river, it is considered appropriate to apply this practical approach.

Reach Sinuosity

The sinuosity of a reach of river is defined as the ratio of the channel thalweg distance to the straight-line distance. A value of sinuosity of close to 1 is representative of an almost straight or low sinuosity reach whereas a value of 2 represents a high sinuosity.

The sinuosity of each model section has been calculated in this manner for the River Kelvin, however, the exercise is complicated as each reach tends to be relatively short i.e. 200-300m. The straight and centre-line distances were scaled off a plan drawing of the River Kelvin and the resulting sinuosities were generally low i.e. 1.12-1.17

The sinuosity of the upper River Kelvin (Sections 1-49) can be considered low and generally speaking the river does not exhibit long meanders. The lower section of the Kelvin (Sections 50-87) does exhibit significant meandering and has an estimated sinuosity of 1.30.

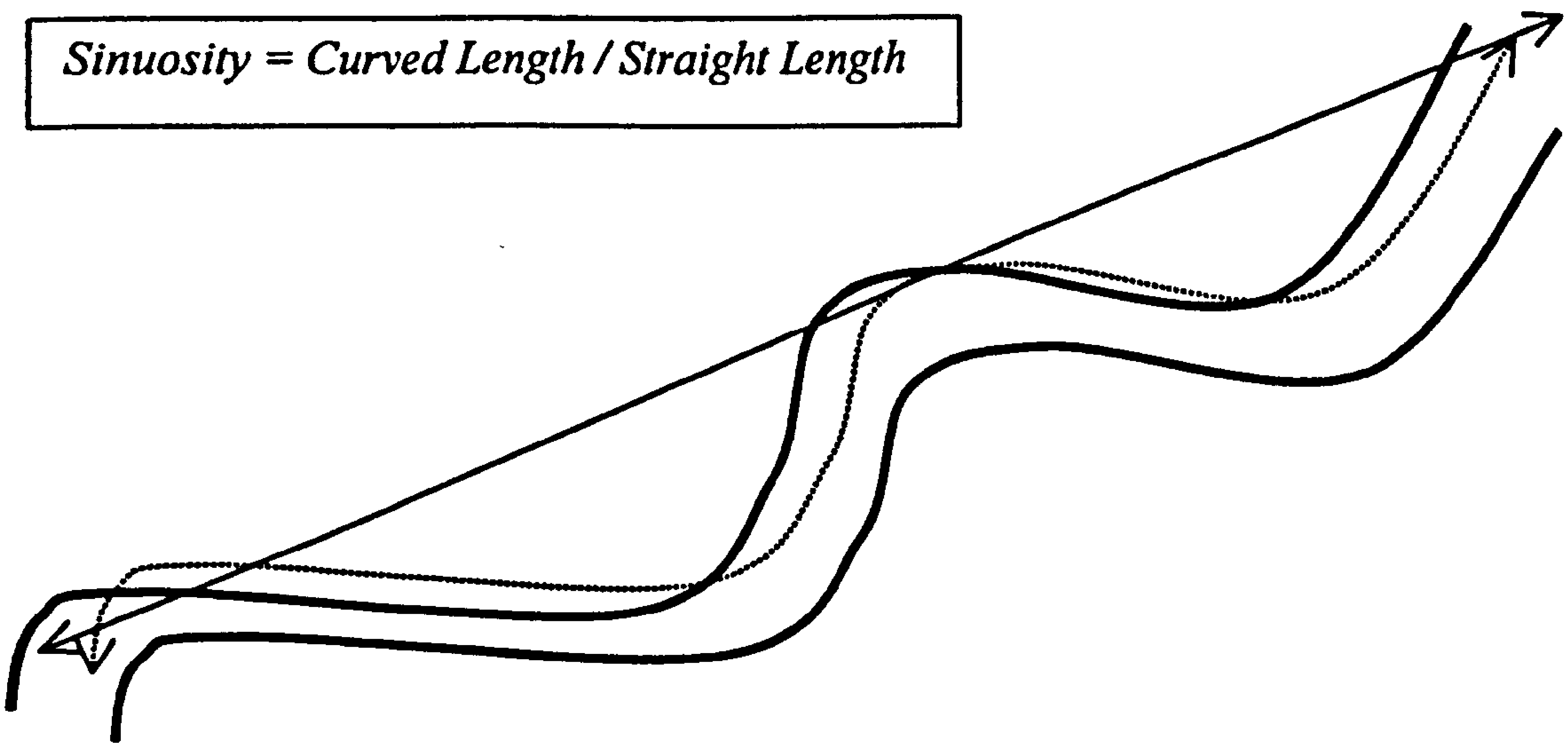


Figure 6.12 - Estimation of The Meander Wavelength Term

Meander-Belt Width

The meander belt width is illustrated in Figure 6.13 and is defined as the horizontal width between meander bend apexes. This parameter is estimated from a plan view of the river and once again engineering judgement is required in its estimation. Figure 6.13 illustrates how the parameter is calculated. For the River Kelvin the meander belt widths were relatively small, 50m around sections 60 to 87 decreasing to 20m where the river is almost straight.

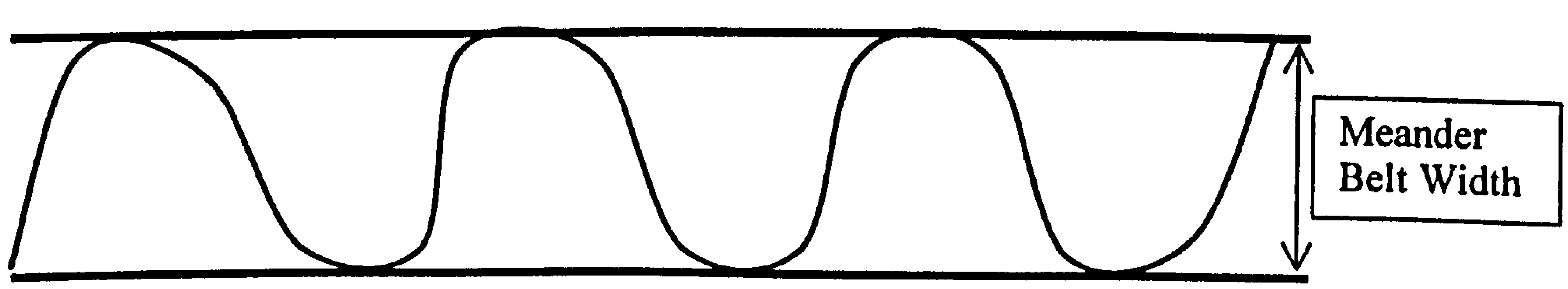


Figure 6.13 - Diagram showing Meander Belt Width – Plan view

Cross-Section Side Slope

The cross-section side slope is estimated using the upper two-thirds of the river-bank slope. This is a consequence of the probable irregularity in natural river- banks.

The values of side-slope for the River Kelvin were obtained using the 87 surveyed cross-sections for left and right-bank. The average of the two bank slopes has been used for each cross-section. The estimate of side-slope for each of the 87 cross-sections can be found in Appendix #.

Flood Plain Slope

The flood plain slope is the valley slope and is required in preference to the more commonly used Main Channel Slope (MCS). The James and Wark Method subroutine calculates a value for MCS by dividing the FPS by the sinuosity. This maybe a potential flaw in the James and Wark Method as, in practice, the main channel slope is measured and may not in general be the same as simply the FPS divided by the sinuosity which can only be estimated approximately in real rivers.
The estimation of this parameter is complicated as the slope is that of the 'representative length' and therefore is taken as the average of slopes between three consecutive cross-sections. A possible complication of doing this arises when an adverse slope is encountered. If this situation is encountered the average FPS of the River Kelvin is used.

Meander Wavelength

The average meander wavelength is defined as the number of wavelengths that occur in a reach length. Therefore, the reach length is divided by the number of wavelengths to obtain the estimate of this parameter. For the River Kelvin model, due to the short reach lengths, one wavelength per reach has been assumed.

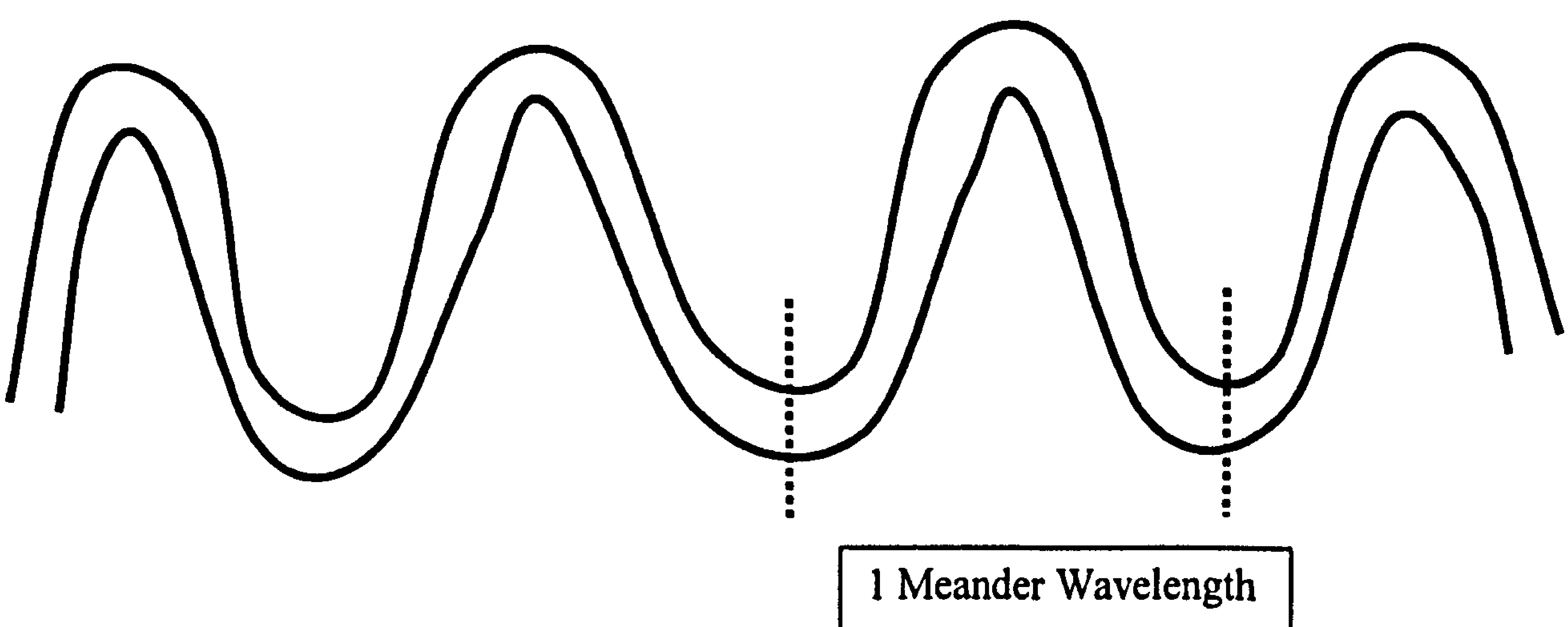


Figure 6.14 Estimate of Meander Wavelength

The above parameters were all scaled off a combination of 1:10000, 1: 15000 plan views of the Kelvin valley and 1: 500 cross-section drawings with interpolation where required. Once all the additional information has been obtained it is added to the data file in the appropriate locations and the model can be run.

6.6.2 October 1995 Flood Event

The October 1995 flood event was essentially an in-bank flood and so there would have been little or no flow interaction with the flood plain. This is reflected in the value of 'n' obtained through calibration which was 'n' = 0.080, the same as that estimated by the divided channel method. This is expected due to the lack of flow interaction and secondary losses.

Location	Q_{OBS}	H_{OBS}	Q_{DCM}	H_{DCM}	Q_{J+W}	H_{J+W}
Dryfield	65.52	35.26	63.40	35.20	63.04	35.13

Table 6.04 - James and Wark Calibration Results October 1995

An acceptable level of agreement has been observed with the flood plain 'n' value estimated at 0.080. Although this is essentially an in-bank flood some locations are experiencing low overbank flows and secondary interaction losses. This explains the 70mm difference between the Divided Channel Method and James and Wark Method shown in Table 6.04.

6.6.3 September 1985 Flood Event

The September 1985 flood event was used to verify the chosen Manning's 'n' values. The main channel 'n' value was again taken as 0.080 and the flood plain value was 0.10. This, however, led to a significant over prediction in water level at Dryfield of 160 mm. As a result the flood plain 'n' value was reduced to 0.085 and the following results obtained.

Location	Q_{OBS}	H_{OBS}	Q_{DCM}	H_{DCM}	Q_{J+W}	H_{J+W}
Dryfield	95.00	36.00	108.62	35.98	108.62	36.09

Table 6.05 - James and Wark Calibration Results September 1985

The calibration is again reasonably good and as expected a lower value of flood plain 'n' is used in the James and Wark method i.e. 0.1 for the Divided Channel Method and 0.085 for James and Wark Method.

6.6.4 December 1994 Flood Event

For the December 1994 Flood event a significant number of observed water levels were recorded, along the length of the study reach. Table 6.06 shows the observed values and the results of both the ISIS (DCM) and ISIS (J+W) calibrations. The main channel 'n' was kept constant at 0.080 and the flood plain 'n' varied between 0.08 and 0.35.

Sect No	OBS	ISIS DCM	ISIS J+W	Diff J+W v OBS	Diff J+W v DCM	Diff DCM v OBS
	MAOD	MAOD	MAOD	m	m	m
26	38.9	38.71	38.56	-0.34	-0.15	-0.19
28	38.5	38.69	38.39	-0.11	-0.31	0.19
29	38.5	38.68	38.38	-0.12	-0.30	0.18
32	38.0	37.95	37.98	-0.02	0.03	-0.05
34	37.7	37.83	37.87	0.17	0.04	0.13
36	37.5	37.53	37.60	0.10	0.07	0.03
38	37.1	37.05	37.13	0.03	0.08	-0.05
41 Dryfield	36.65	36.69	36.70	0.05	0.01	0.04
49	35.85	36.06	36.03	0.18	-0.03	0.21
57	35	34.96	34.87	-0.13	-0.09	-0.04
64	34.9	34.76	34.78	-0.12	0.02	-0.14
74	32.8	32.99	32.82	0.02	-0.17	0.19
87	30.82	30.72	30.75	-0.07	0.03	-0.10

Table 6.06 Results of December 1994 Simulation

It can be seen that a reasonable level of agreement between the observed and James and Wark Calibration has been obtained. By using the James and Wark Method to re-calibrate this model it was found that, in some locations, a 'n' value as low as 0.08 could be used on the flood plain while in other locations a value as high as 0.35 was required. This was probably a consequence of other factors such as Bridges, Railway Embankments inhibiting flow down the flood plain.

Locations where a good agreement is not observed, i.e. within approximately 200 mm of the observed value, are at cross-sections 26, 34 and 49. At section 26 this could be due to the limitations of the ISIS Arch bridge routine that was developed for small-scale prototype bridges and arguably may have limited practical value.

At cross-section 34 an overestimation of 170 mm is calculated and may be occurring for two reasons. Firstly the confluence of the River Kelvin and the Luggie Water is at this location which may be forcing up water levels and secondly there is a significant extent of the flood plain being inundated. As ISIS assumes a horizontal water surface an over prediction of water level could be plausible. The over prediction at section 49 is probably happening for the same reasons.

It should be noted that a significant reduction in the value of Manning's 'n' used was achieved with this calibration.

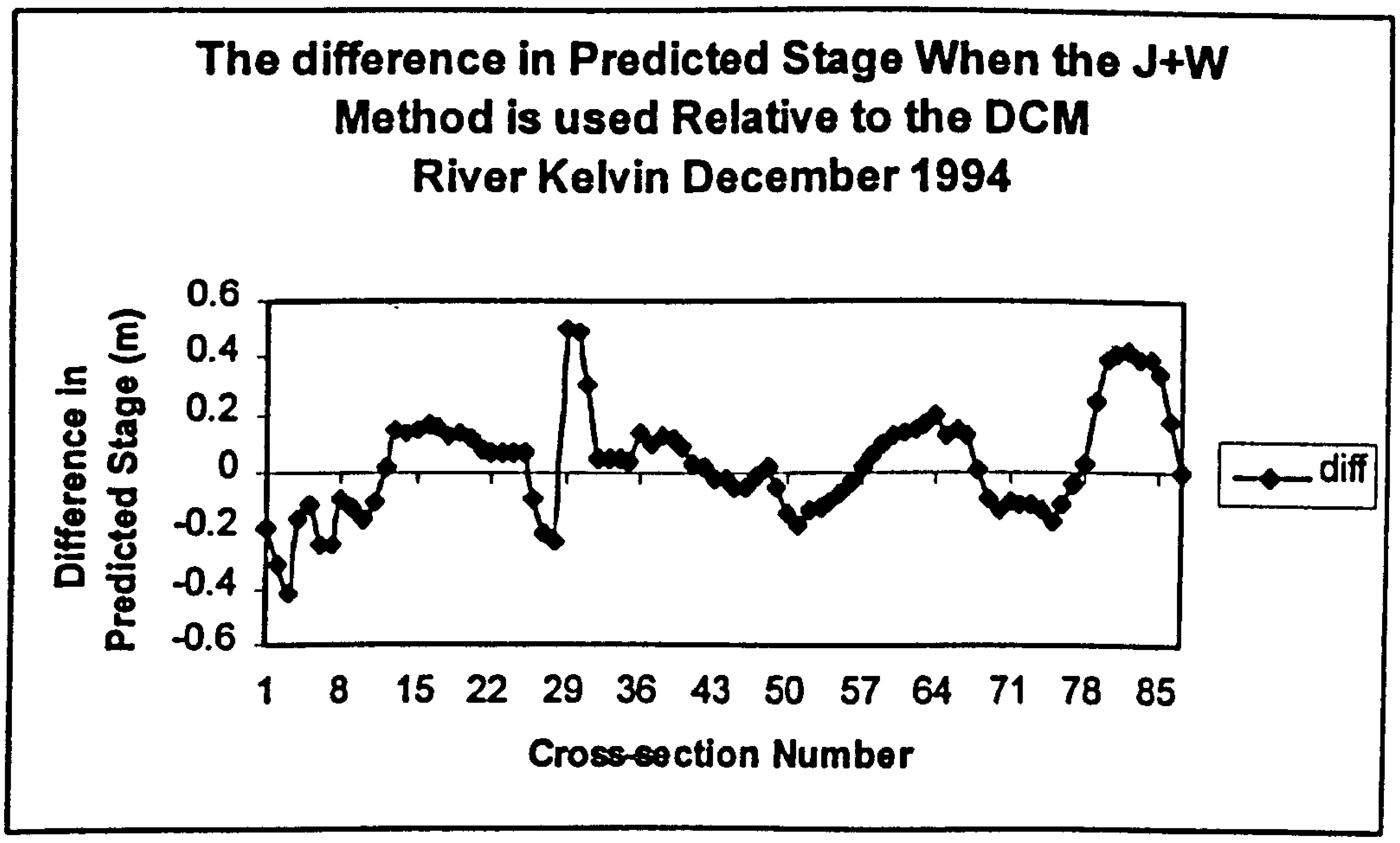


Figure 6.15 illustrates the differences in water level prediction for the River Kelvin December 1994 flood event using fully calibrated models ISIS (DCM) and ISIS (J+W)

6.7 Bridges on The Main Reach of The River Kelvin

On the main reach of the River Kelvin there are eight bridges of varying size and shape. The original flood study of the River Kelvin, carried out by the Department of Civil Engineering at Glasgow University, used river modelling software that approximated bridge effects. The current study has transferred this model to ISIS, which accounts for energy losses at bridges explicitly, and the significance of doing so is shown in Table 6.07. The ISIS software uses the USBPR method for flat soffit bridges and the HR Wallingford method for arched soffit bridges. Further details can be found in the ISIS Flow User Manual (1997).

Section Number	Channel Dec 1994 Flood Water Level (m O.D.)	ISIS Dec 1994 DCM Flood Water Level (m O.D.)	ISIS Dec 1994 J+W Flood Water Level (m O.D.)	Bridge Soffit Level (m O.D.)
12	39.57	39.59	39.57	39.73
26	38.84	38.70	38.56	39.54
33	37.87	37.87	37.90	36.34
36	37.34	37.52	37.60	36.24
39	37.08	36.93	37.01	36.50
50	35.61	35.69	35.51	34.73
64	N/A	34.76	34.78	35.50 approx
72	33.80	33.16	33.00	33.92

Table 6.07 - Water Level Prediction at Bridges on the River Kelvin December 1994

Note:

Channel Software – Original Glasgow University 'in-house' model that uses the Divided Channel Method to calculate conveyance

ISIS DCM – Existing Commercially Available ISIS Software that uses the Divided Channel Method to calculate conveyance

ISIS J+W – Recently Developed ISIS Software That uses the James and Wark Method to Calculate conveyance.

The results show that the Channel software has, in practical terms, made a reasonably accurate assessment of the energy losses that have occurred at the bridges on the Kelvin. The prediction at cross-section 72 is poor but this may be due to the missing bridge in the Channel model, at cross-section 64, which is upstream of this location.

After calibration the difference in predictions between the Divided Channel Method and the James and Wark Method, at bridge locations is not practically significant. The maximum difference between the two methods is 180 mm at cross-section 50.

6.8 Accuracy of Survey Data

In keeping with the practical theme of this research project a test regarding the amount of survey data at each cross-section was undertaken. For any flood study one of the major expenses involved is the topographical survey. Normally a walking tour is made of the reach in question and a decision made as to the location and number of survey cross-sections.

The survey of the River Kelvin was carried out following the December 1994 Flood event. Unfortunately, the survey work commissioned did not extend far enough onto the flood plains to include the full width inundated. (See Figure 6.14)

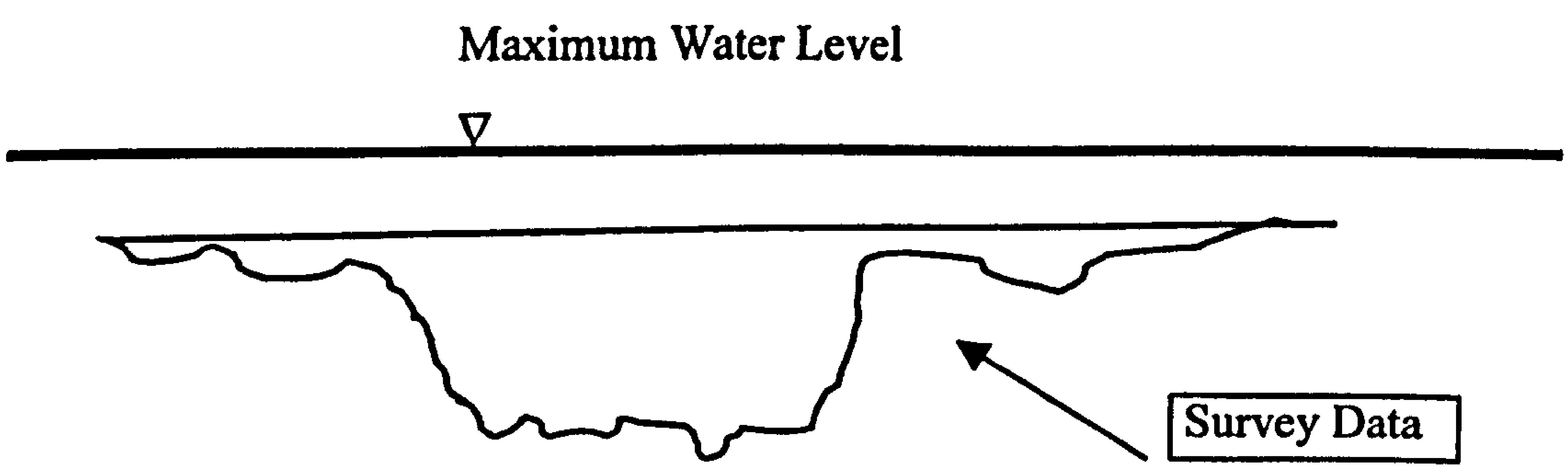


Figure 6.14 - Extent of Existing River Kelvin Survey Data

To enhance the ISIS model of the River Kelvin the cross-sectional data was extended laterally by use of the December 1994 flood inundation envelope.
As a result an improved model of the 1994 event was constructed. This provided an interesting investigation as it assesses how much fundamental survey data is required. This has implications in terms of time and cost to the practising Engineer.

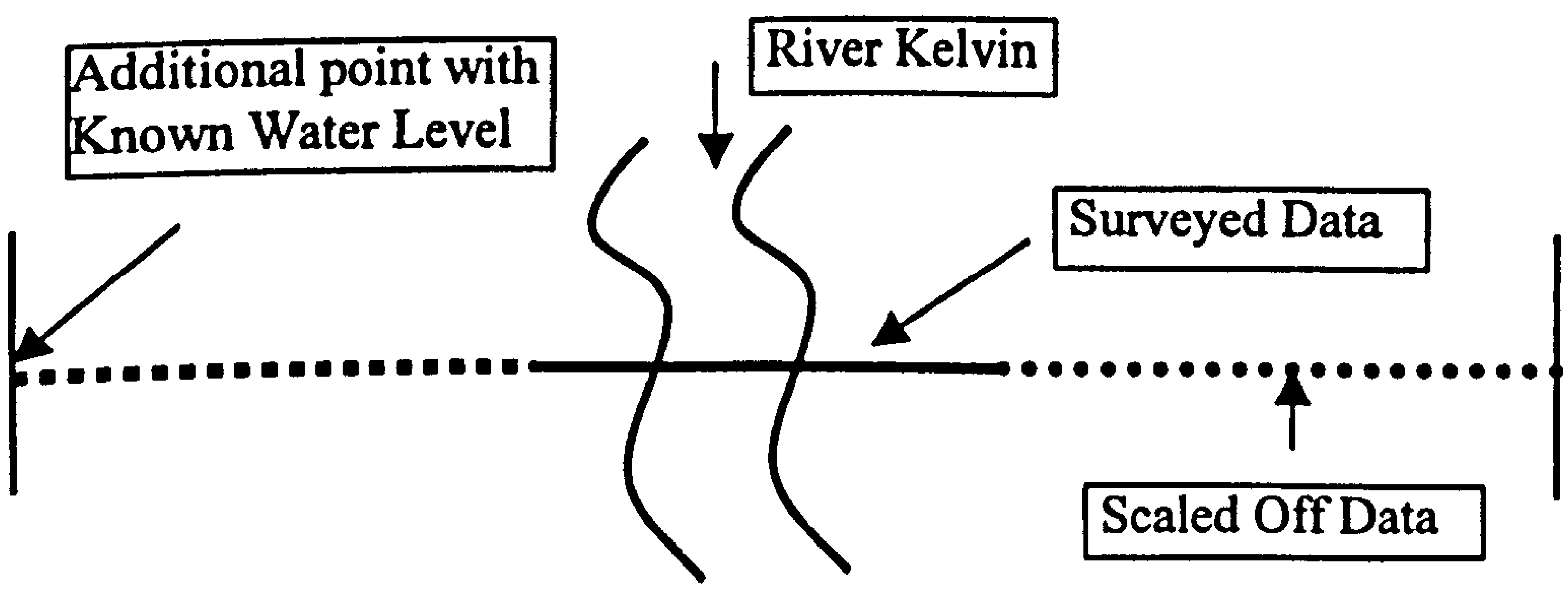


Figure 6.15 - Plan View of Extended Cross-sections

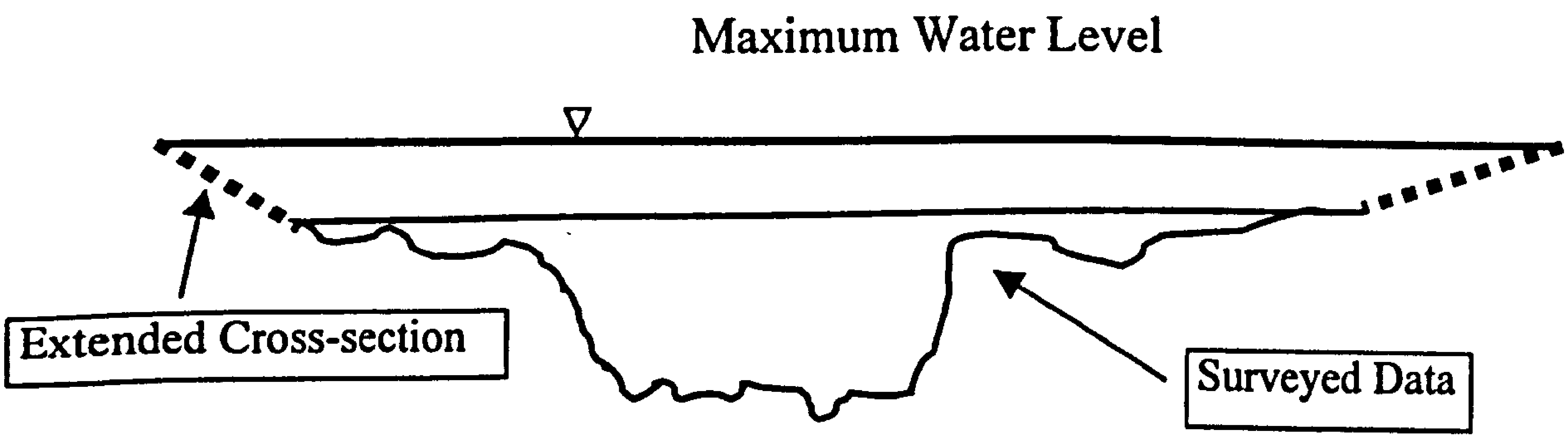


Figure 6.16 - Final Cross-section used in ISIS Model of The River Kelvin

From consultation with the flood inundation drawing an estimate of the ground level at the extreme point of the envelope was made. This allowed the extreme points of the observed flood envelope to be connected to the surveyed data. As illustrated in Figure 6.16 the connection was made by assuming a straight line. By doing so additional areas of flood plain were included in the model and in some locations it could amount to a width increase of 800m. However, due to the possible errors associated with the estimations of these extreme points, it was decided to examine the influence of the River Kelvin flood plain representation on the quality of numerical model predictions. In order to test the sensitivity of this assumption the following 3 models were constructed:

Model 1 - Original CHW Survey Data From December 1994 Flood Study
Model 2 – As Model 1 with extreme flood plain points reduced vertically by 0.5m
Model 3 – As Model 1 with extreme flood plain points raised vertically 0.5m

Table 6.08 shows a selection of model cross-sections between Kirkintilloch and Bearsden where significant horizontal additions have been made to the cross-sectional data and the effects of raising or reducing the extreme points. The aim is simply to observe the effect of including approximate survey points to the data set.

Sect No	*Model 1*	*Model 2*	*Model 3*	*Diff 2-1*	*Diff 2-3*	*Diff 3-1*
20	39.15	39.01	39.28	-0.14	-0.27	0.13
25	39.07	38.93	39.19	-0.14	-0.26	0.12
30	38.04	37.93	38.14	-0.11	-0.21	0.1
35	37.79	37.71	37.85	-0.08	-0.14	0.06
40	36.81	36.73	36.90	-0.08	-0.17	0.09
45	36.25	36.17	36.31	-0.08	-0.14	0.06
50	35.61	35.50	35.70	-0.11	-0.2	0.09
55	35.02	34.89	35.13	-0.13	-0.24	0.11
60	34.74	34.61	34.88	-0.13	-0.27	0.14
65	34.04	33.99	34.09	-0.05	-0.10	0.05
70	33.35	33.29	33.40	-0.06	-0.11	0.05
75	32.93	32.87	33.00	-0.06	-0.13	0.07
80	32.21	32.16	32.26	-0.05	-0.1	0.05
85	31.63	31.61	31.66	-0.02	-0.05	0.03

Table 6.08 - Effect on Water Levels of Differences in Elevation of Extreme Points on The River Kelvin December 1994 Flood Event (Sections 20-87 only)

The results shown in Table 6.08 suggest that significant differences in the extremes of flood plain levels result in relatively small changes to the predicted maximum water levels. Given that most flood protection schemes will be designed with a free-board of +0.5m, the technique of extending the flood plain width is considered acceptable.

6.9 River Kelvin - Discussion of Results

The modelling work described in this chapter has been carried out to the standard performed by the practising engineer. At times the data available has been less than ideal and reasoned judgement is required to advance a solution. The following discussion outlines aspects of this application that has proved complicated, problematic or required judgement, as well as, the significance of results.

6.9.1 Basic Model

The construction of the basic ISIS model was reasonably straight forward as the Kelvin had been surveyed for a previous flood study (CHW 1996). As a result the raw model data was readily available and had to be typed into the ISIS workbench. However, after a walking tour of the River Kelvin it became apparent that some of the data was erroneous or was in the wrong location. For example, at cross-section 64 a bridge was discovered that had not been included as part of the original survey work.

It was also concluded, between cross-sections 50 and 87, that the flood plain had not been surveyed in enough detail. Essentially, the survey had not gone far enough out from the river-banks and additional data had to be scaled off contour maps of this location and estimated using the technique mentioned in section 6.4. This was a direct result of an earlier flood study on the Kelvin where poor calibration was observed. The addition of the scaled off flood plain levels improved the calibration as there was additional area for flood water to flow on to which in-turn leads to a reduction in predicted water levels.

The basic model of the Kelvin was prone to unstable behaviour and also difficult to run unless a small time step was specified. Even although, ISIS employs the implicit Preismann finite difference scheme which is theoretically unconditionally stable for $\theta \geq 0.5$. This was due to a combination of extreme changes in cross-sectional data, bridges and rapidly increasing tributary inflows.

6.9.2 Divided Channel Method Calibration

The calibration of the original ISIS model was complicated in that there were 5 tributary inflows and 6 bridges, in addition, a high value of both main channel and flood plain Manning's 'n'. These factors combined to make the model unstable at times and careful adjustment of initial conditions and boundary conditions had to be undertaken. A common problem was that if the Manning's 'n'' was too high the model would crash due to instabilities.

The main channel 'n' value of 0.080 could be considered quite high and has been required in-order to match with observed conditions. The description of a Manning's 'n' value of 0.080 is that of "a natural stream with sluggish reaches, weedy with deep pools" Chow (1959), this does not sound very similar to the Kelvin which is more similar to "Clean, winding, some pools and shoals with some weeds and stones" Chow (1959), which has a maximum 'n' value of 0.050. The value of 0.050 was initially used in the model, however, a significant under prediction in water level was observed at Dryfield. It should be noted that there are significant amounts of tees and bushes on the river banks, as can be seen in Pictures 5 and 6, which may account for the higher 'n' value.

This is a common scenario for the practicing Engineer and commonly the only solution is to artificially inflate the book value of 'n' until the predicted water level is in close agreement with the observed.

A similar situation can be seen regarding the flood plain 'n'. The flood plain is generally assumed to be rougher than the main channel and a reasonable book value for the River Kelvin would be 0.070 i.e. "Scattered brush with heavy weeds" Chow (1959). The value eventually used in the calibration was 0.350 which has significantly inflated the book value. This value of 'n' has been used at the majority of the 87 cross-sections, some minor differences are required in certain areas. The inflation has been required due to a combination of secondary flow losses and the sizeable horizontal extents of the model cross-sections. The total horizontal extent of some cross-sections can be 1000m and in these locations significant adjustment of 'n' is required to aid calibration.

However, as the Kelvin has spoil banks training the main channel, these may be responsible for the high 'n' values. If a situation arises where the flood plain flow is blocked, for example, by trees, walls, railway embankments etc, then the roughness must tend to infinity.

Lorena (1992) performed experiments that had zero flood plain flow i.e. flow was stationary and acknowledged that where there was a major obstruction to the flow then the roughness must be infinite.

Recent research at the University of Bristol by Wilson (1998) has also indicated the estimation of high Manning's 'n' values. When considering the findings of the

Kelvin, Wilson (1998) and many engineering practioners you conclude that perhaps the book values proposed by Chow (1959) and Henderson (1966) need to be revised.

6.9.3 James and Wark Method Calibration

The fundamental test that was of interest during this exercise was to observe the difference in Manning's roughness coefficient that could be used for the different conveyance calculations. In practical river engineering it is the 'n' value that is used in calibrating a model. As the James and Wark Method accounts for additional energy losses other than bed friction it is instructive to observe how this influences calibration in a real river.

The Kelvin may not be similar to the Flood Channel Facility but, if the James and Wark Method is to be widely used it has to be capable of modelling any given river geometry. The following discussion outlines the relevant issues concerning the application of the James and Wark Method to a natural river.

Before a discussion of the James and Wark calibration is embarked upon, it is important to note that the River Kelvin may not be ideally suited to the application of this new method. A possible reason for this being that a particular feature of the River Kelvin between Kilsyth and Glasgow is that it is trained by spoil banks that rise generally 1.5m above the flood plain affording some level of protection against inundation of the agricultural land.

The spoil banks were constructed from the dredged material excavated from the solum of the river channel during the late 1930's as part of the "River Kelvin Statutory Maintenance Scheme". The maintenance scheme required that the centre line river bed level be maintained at or below a specified limit, a limit that is checked every few years by carrying out a survey of the river bed.

It should be noted that this scheme was discontinued a few years ago and the spoil berms have experienced significant erosion and that the spoil banks are currently significantly lower than 1.5m. The result being that the depth of the Kelvin has reduced due to sedimentation and many 'low spots' are visible in the spoil banks which allow earlier flooding during high flows.

These natural defences, where intact, are effectively restraining the main channel flow from the flood plains. Consequently flow interaction is not possible until a reasonable flood plain depth is encountered. However, during the September 1985 and December 1994 flood the embankments were overtopped by a considerable margin and flow interaction would have taken place.

The spoil banks contain the October 1995 flood at most cross-sections and this is probably the reason for a similar 'n' value when using the conventional Divided Channel Method or the James and Wark Method. i.e. 0.080 Generally speaking, it would not be appropriate to use the James and Wark Method to calibrate an in-bank flood event, however the Kelvin had many locations where interaction could have taken place during this flood event.

Despite the same Manning's 'n' value being used for both calibrations a different predicted water level was obtained at Dryfield. The Divided Channel Method produced a water level that is 70 mm higher than the James and Wark Method (see Table 4) and is in practical terms almost identical.

Again the September 1985 flood was used to verify the main channel 'n' value and calibrate the flood plain. Interestingly, the 'n' value required to enable good agreement at Dryfield was 0.085. This value should be compared with the 0.10 that was used in the Divided Channel Method Calibration.

This represents an 18% reduction in Manning's 'n' when using the James and Wark Method. This result is expected as the James and Wark Method accounts for energy losses in addition to bed friction. The influential secondary losses are being accounted for and as a result the flood plain 'n' value can be reduced.

The December 1994 flood has many more observed flood levels where comparison may be made and these are shown in Table 6. In terms of water level predictions the comparison between the ISIS James and Wark Method and the observed levels are good.

In general, the predictions are within 150mm of the observed which is reasonably in an application of this nature. The locations that do not fall into this criteria are cross-sections 26, 34 and 49. As mentioned earlier it is suspected that this is due to the limitations of the ISIS Arch Bridge option that was based on small-scale laboratory studies.

The flood plain 'n' value that has been required to produce this calibration is high in places. A range of 'n' values have been required to get good agreement with minimum 'n' values being 0.08 and maximum 'n' values being 0.35. These values should be compared with the Divided Channel Method 'n' value of 0.35 which was used at almost all cross-sections. The extremely high values of 'n' are required in locations where the flood plain flow is severely obstructed, if not halted, by embankments. Essentially the James and Wark Method calibration has required smaller 'n' values but not at every location. In some places the James and Wark 'n' value has been the same as the Divided Channel Method.

6.9.4 Ease of Using the James and Wark Method

It has to be noted that the estimation of the additional parameters required by the James and Wark Method has been problematic. It was thought that a reach averaged cross-section had to be employed to enable the correct working of this method Wark (1998), this study has used the surveyed cross-sections as it is considered the most practically advantageous solution. The use of anything other than the surveyed cross-sections seems impractical and pointless.

The data required for a one-dimensional model is intended to be straightforward, easy to use and relatively in-expensive. It has been with this in mind that the application of the James and Wark Method, within a one-dimensional model, has been attempted.

The values adopted for sinuosity, meander wavelength and meander belt width are all somewhat subject to interpolation. However, the results of Chapter 5 (River Dane) have indicated that a high level of accuracy is not actually required in the estimation of these parameters, for this practical situation.

This study has assumed a meander wavelength of 1 for each representative reach length. That is one wavelength occurs per surveyed reach length. This was an approximation as it was noticed that the actual meander wavelengths for each reach length tended to be in the region 0.6-1.0. The difference in water level prediction when assuming 1.0, compared to the actual value, was negligible and therefore it seems reasonable to assume that this parameter is unity.

6.9.5 Bridges

The modelling of bridges by use of the USBPR and HR Arch bridge Routine in ISIS has been compared with a previous model of the Kelvin which made no attempt to model bridges. The results of this are shown in Table 6.07 and indicate that simply approximating energy losses at bridge locations can provide reasonable water level predictions. The difference in predictions between the Divided Channel Method and the James and Wark Method were not practically significant.

The maximum difference is 180mm and it is suspected that no significant difference is predicted as the bridges tend to be located on straight reaches of river or the bridge modelling programs are not well suited to the bridges being modelled. On a straight reach of river the James and Wark Method would not predict significant amounts of energy loss and perhaps close agreement is obtained, with the Divided Channel Method, as high 'n' values are used in both calibrations.

6.9.6 Additional Survey Data

The addition of an extreme point has improved the model in that it allowed better modelling of the December 1994 flood. The sensitivity test, concerning the accuracy of this point, showed an error of 0.5m in the level of the extreme edge of the flood plain may be made without any practically significant effect on the predicted flood level.

6.9.7 Estimates of Manning's 'n' used in River Kelvin Calibration

As defined and indicated in sections 6.9.2 and 6.9.3 the calibrated 'n' values used in the River Kelvin model are very high. The high estimates of 'n' were required to match with observed flood levels from the various flood events used in the calibration process, including the December 1994 event. It is probable that the high flood plain values are a consequence of "ponding". This ponding is a feature of the river and is caused by the many obstructions to the flow, such as railway embankments, which effectively halt the flood water flowing down the flood plains. This effect does not explain the main channel value of 0.08.

It is possible that the high values of main channel 'n' are a result of inaccuracies in the measured flows. For this study the flows measured at the gauging stations were reduced to more realistic values (see Appendix 5), however, it is plausible that they are still high.

It is also plausible that a better method of modelling the River Kelvin would have been to assume the flood plains were acting like storage ponds. This however is not appropriate for a James and Wark analysis. It may be that an improvement in calibration may have been obtained if this approach had been adopted.

Chapter 7 Conclusions and Recommendations

7 Conclusions

7.1 Chapter 4 Code Development and Testing

The Ackers Method and the James & Wark Method were chosen at the beginning of this research project as at that time, they were considered to be the most likely to be adopted by industry and indeed were recommended by the Environment Agency for England and Wales. They are fundamentally methods for determining stage-discharge relationships for the design and analysis of two-stage channels.

The Ackers Method and The James & Wark Method have been coded in FORTRAN and successfully incorporated into the commercially available ISIS software, both Methods have been tested by comparing model results with FCF data. The level of agreement was considered to be acceptable.

The James and Wark Method over-predicted the observed Flood Channel Facility discharge by 2% on average for Experiment B26. Experiment B26 consisted of a quasi-natural main channel with a 60 degree meander bend. The Divided Channel Method was found to over-predict the observed Flood Channel Facility discharge by 22% on average for Experiment B26.

The James and Wark Method over-predicted discharge by 4% for Flood Channel Facility Experiment B39. Experiment B39 consisted of a quasi-natural main channel with a 110 Degree meander bend. The Divided Channel Method over-predicted by 28%. The improvement obtained by using the James and Wark Method is clear.

The James and Wark Method was found to under-predict the observed water level by 2 mm, on average, for Flood Channel Facility Experiment B26. The Divided Channel Method was found to under-predict the observed water level by 8 mm, on average.

The James and Wark Method under-predicted the observed water level by 1 mm, on average, for Flood Channel Facility Experiment B39. The Divided Channel Method was found to under-predict the observed water level by 9 mm, on average. Again the improvement obtained by using James and Wark over the Divided Channel Method is clear.

The published value of Manning's 'n', used in the Flood Channel Facility Experiments, of 0.010 has since been found to be less than that of the constructed channel. It appears that the true value of 'n' should have been around 0.0105. This difference accounts for the difference between this study and that of James and Wark (1992) when applied to the Flood Channel Facility Experiments.

James and Wark (1992) and Wark and James (1994) have stated the requirement of a 'reach-averaged cross-section' when applying the James and Wark Method. The requirement of a reach averaged cross-section is considered to be impractical and unnecessary. The reach length is not so important as long as the 'channel parameters' are defined in relation to it. The suggestion from Wark (1998) that a reach-averaged cross-section is a fundamental requirement does not appear to be valid.

The Ackers Method Conveyance Method was verified using Hypothetical data and Flood Channel Facility Data to an acceptable level of accuracy. The Ackers Method is considered to have a limited degree of practical application.

7.2 Chapter 5 The River Dane

The River Dane, although being a natural river, exhibits strong meandering characteristics and can be considered similar in many ways to the Flood Channel Facility data. Two different reach averaging assumptions have been tested on the River Dane with negligible differences in water level predictions. (see Figure 5.09) The first method of reach averaging assumed that the reach length was the entire study reach while Method 2 assumed only a typical cross-section's representative reach was used as its reach length. (See Section 5.2)

The use of both of these methods on the River Dane gave acceptable results as long as the 'channel parameters' are calculated in relation to it. Thus confirming the finding on the Flood Channel Facility.

The James and Wark Conveyance method predicts higher water levels than the Divided Channel Method for both the 1946 and 1995 flood events on the River Dane. The James and Wark Conveyance Method is more sensitive to flows that are 'just out of bank' than high flow events. For example, for the 'just out of bank' 1995 event there was a maximum increase in water level of 0.42m when using the James and Wark method relative to the Divided Channel Method. However, for the very high flow event of 1946 the maximum increase in water level was 0.18m. It should be noted that for the 1995 event most of the water level increases that resulted from using the James and Wark Method were approximately 0.20m.

The James and Wark Conveyance Method will result in the prediction of higher flood levels than the existing standard Divided Channel Method, when used within a one-dimensional river model. The predicted increases are considered practically significant and it is recommended that modelling of secondary losses is attempted.

As the James and Wark Method requires the estimation of 'channel parameters' a sensitivity analysis of the 'channel parameters', has been investigated for reach averaging Method 1. (See Section 5.6) This indicated the accuracy required in estimating these parameters in a natural environment.

The sensitivity of water level prediction to an independent change in sinuosity has been tested. When this term is independently increased from 1.8 to 2.1 the predicted water levels decrease by a maximum of 0.07m.

When the sinuosity term is reduced from 1.8-1.5 the predicted water levels increase by a maximum of 0.04m. Thus an error in this term of 15-20 % will not have a significant effect on predicted water levels.

The effect of an error in the meander wavelength term of ±50m has been investigated. In general, the predicted water levels decreased as the meander wavelength increased, with the maximum difference in water level being 0.06m. This implies that a high level of accuracy is not required in estimating this parameter, for a natural river. This supported the findings of James and Wark (1992).

The effect of an error in the meander belt width term of ± 30m has been investigated. This test showed that the maximum difference in predicted water level was -0.05m when the meander belt width was 30m bigger than it should be. When the meander belt width is reduced the predicted water levels rise marginally.

The side-slope term was not tested as it was not considered to be a parameter that could be severely miscalculated. In addition, James and Wark (1992) tested this parameter and established that ± 100% changes to the side slope only resulted in ± 5% changes in predicted discharge.

It is concluded that a high degree of accuracy is not required in estimating the 'channel parameters' in a natural environment similar to the River Dane.

For the case of the River Dane it is concluded that the water level predictions, by the James and Wark Conveyance Method, are not sensitive to the meander wavelength or meander belt width terms, however, the predictions are more sensitive to a significant error in the sinuosity term. In terms of consequences for modellers it means that care needs to be taken in estimating the channel sinuosity while a reasonable estimate will suffice for meander wavelength and meander belt width.

The James and Wark conveyance method can be used in natural rivers similar to the River Dane and an increase in predicted flood level would be expected, relative to standard industry methods (DCM).

7.3 Chapter 6 The River Kelvin

The River Kelvin is typical of many UK rivers however it is very different from the Flood Channel Facility and River Dane Geometries. A fully calibrated ISIS model of the River Kelvin has been constructed. The study reach is 20.5 Km long and has six bridges and five tributary inflows. The initial calibration of this model was performed using the current best industry practice and the Divided Channel Method for calculating conveyance.

The inflows, bridges and obstructions to flood plain flow complicated the calibration of this model. Three flood events were used in the calibration of this model, specifically that of October 1995, September 1985 and December 1994. The calibration process should only have used the first two events and then been used to predict the December 1994 event. However, due to the magnitude of this flood event and the longitudinal and lateral variation within this model, the December 1994 flood has also been used to refine the calibration of the model. This is considered reasonable practice in the absence of additional calibration data.

The Divided Channel Method Calibration resulted in a main channel 'n' value of 0.08 and a flood plain 'n' value of 0.35. These estimates are high relative to the book value estimates proposed by Chow (1959) and Henderson (1966). However, adjustment of the book values has been required to match predicted water levels with observed water levels. It is also proposed that since many of the cross-sections are 1000m in width it takes a very significant increase in Manning's 'n' to dramatically improve model calibration.

It is concluded that the flood plain 'n' is very high as the River Kelvin has many flood plain obstructions, i.e. spoil banks and railway embankments, which inhibit flood plain flow. This effectively means that the flood plains are so rough there is little flow down the flood plain and leads to ponding.

It is concluded that as similar estimates of high Manning's 'n' have been reported on the River Kelvin, the River Blackwater by Wilson (1998) and in many studies by engineering practitioners, the book values of Manning's 'n' proposed by Chow (1959) and Henderson (1966) need to be revised.

The River Kelvin has been re-calibrated using the James and Wark Conveyance Method. It should be noted that the 'channel parameters' were calculated in relation to each model cross-section's representative reach length, similar to Method 2 in Chapter 5. This calibration resulted in a main channel 'n' value of 0.08 and a flood plain estimate that ranged between 0.080 and 0.35. Overall, a reduction in the required 'n' value was observed. Again in places the calibration was affected by the flood plain obstructions which forced up the value of 'n'.

The ease of using the new software is complicated by the need for estimates of the 'channel parameters' which in a natural environment need a degree of judgement and can be time consuming.

The surveyed cross-sectional data for the River Kelvin between cross-sections 50 and 87 was not sufficiently detailed. The addition of extreme data points in the model data has been shown to be a practically reasonable method of enhancing the model data.

When applying new conveyance techniques to real river situations there are more unknowns to contend with such as flood plain obstructions and degree of accuracy of flows.

Finally, it is concluded that improved conveyance calculations using techniques such as James and Wark method are of limited value when applied to a River such as the Kelvin. The differences are more significant when applied to a meandering river such as the Dane, however, realisation of the benefit depends on an accurate assessment of Manning's 'n'. Where this is not possible then the analysis described here suggests there is little advantage in applying an improved conveyance calculation technique. The optimum natural application of the James and Wark method would be in analysing a constructed two-stage channel, such as the River Blackwater.

Despite some improvements in water level prediction and calibration this study has provided no clear evidence that the more sophisticated techniques for energy loss computation are useful for real rivers, such as the River Kelvin.

7.4 Future Recommendations

It is recommended that future work is carries out to establish the optimum conveyance method that accounts for secondary energy losses. Ervine and Koopaei (2000) are working towards this at present but the true benefits or otherwise will only be realised with incorporation to a one-dimensional river model, such as ISIS. Once a suitable method is chosen, a sensitivity analysis should be performed to determine important parameters and an optimum method of imputing data developed. A method that required fewer additional parameters than the James and Wark method would be advantageous. This could possibly be achieved through use of digital terrain models or GIS that could presumably generate this data automatically.

Any new method that is to be developed should be derived with incorporation to a one-dimensional model in mind. To date, this does not appear to be the case. Any new method should also be, as widely as possible, be verified against field data. More quality field data need be taken to help modellers verify potential conveyance methods.

It is recommended that the tables of Manning's 'n' proposed by Chow (1959) and Henderson (1966) are revised as they are often inappropriate in large scale, natural environment, flood studies.

Chapter 8 References

8.0 References

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Chapter 9 Appendices

Appendix 1

```
C       Last change:  CE   13 Aug 98    9:44 am
        subroutine ackers(panel,ACSURV,py,sy,so,sc,px,sx,fx,gx,nhts,
     +                    hyt,npanel,G,CCHNL,cfile,label,ymax,dflood)
C
C Program for design of straight compound channels using the ACKERS METHO
D
C
C        IMPLICIT NONE
        REAL x(1:30), y(1:30), WC2, RBL, SCR, SCL, SC, ACSURV,
     +ARE(1:40), AF, HLT(1:2)
        REAL B2, BP2, NP, NC, KWL, HFD, AFL, PFL, RFL, QFLB, SO, AFR, PF
     +     R, RFR, QFRB, QFB
        REAL AC(1:30), ACH, PCH, PC(1:30), RC, QCB, QB, HSTR, VC, FC, VF
     +     , RF, FF, G, QSTR2F
        REAL QSTR2C, G1, ARF, DISDEF, QR1, SHFT, HSH, HZH1, ASHFL, PSHFL
     +     , RSHFL, QSHFLB, XL
        REAL XR, ASHFR, PSHFR, RSHFR, QSHFRB, ASHC, RSHC, QSHCB, ASHF, P
     +     SHF, VSHF, FSHF, RSHF
        REAL VSHC1, FSHC, FZZ, PZZ, AZZ, COH, DISADF2, QR2, AZ3, FZ3,
     +        PZ3, COH3, DISADF3
        REAL DISADF4, QR3, QR4, ANGSKW, Q, DISDEF4, DISDEFSKW, QF, QC,
     +     QFP, DISADFC, TOCD
        REAL TOC, TOF, TOF5, DIST, AFLSURV, PFLSURV, P(1:40), AFRSURV,
     +      PFRSURV, HA, K, QACK, sy, py, sx, px, fx, gx, AREA, PERIM
        REAL HFINAL, ABL, HYT(110), HYRAD, RGHNS, CONVY, LEVEL, NPL,
     +      NPR, ACHNL, AREAZ(90), PERIMZ(90), HYRADZ(90), PSHC,
     +      NPZ(90), YMAX, YMAX1, DFLOOD, BETA(NHTS), ATOT, BETA
        INTEGER n, i, lb, rb, M, CHO, NF, RHO, ILBP1, ILBP, IRBP1,
     +      IRBP, FL, FR, J, k1, NHTS, IP, IHT, IZZ, IPTR, NPANEL, cfile
        CHARACTER*12 label
C
        include 'wincom.inc'
C
C!!!!!!!!!!!!!!!! Start of Ackers Conveyance calculation loop !!!!!!!!
C
C
        real panel(90,15,5)
        real cchnl(nhts)
C          WRITE(*,99999)((i,j,(PANEL(I,J,K1),K1=1,5),J=1,3),I=1,20)
C99999     format(2i3,1x,5e12.4)
C         write(*,*) 'The interpolated bankfull area is = ', ACSURV
C          PAUSE
C
C Find locations of left and right banks
        wc2 = sx-px
C          PRINT*, 'The main channel width is, 2Wc  =  ', wc2
C          PAUSE
C
        YMAX1=YMAX-DFLOOD
C         PRINT* ,'The maximum y Co-ord in the section data is', YMAX1
C
C
C Determine river bank elevation
C          PRINT* , py, sy
        rbl=(py+sy)/2
C          PRINT* ,'The river bank elevation = ', rbl
C          PAUSE
C
C
```

```
C        PRINT* , 'Main channel uniform bank slope is equal to ', sc
C        PRINT* ,'The longitudinal gradient of the main channel = ',so
C        PAUSE
C
C
C
C  Step 2.6 Determine the main channel depth
C
C
      HLT(1)=(WC2+(((WC2)**2-(4*SC)*ACSURV))**0.5)/(2*SC)
C        PRINT* , 'The main channel depth is', HLT(1)
C        PAUSE
      HLT(2)=(WC2-(((WC2)**2-(4*SC)*ACSURV))**0.5)/(2*SC)
C        PRINT* , 'The main channel depth is', HLT(2)
C        PAUSE
      HA=ACSURV/WC2
C        PRINT* ,'The approximate main channel depth is ', HA
C        PAUSE
C
C
        IF (ABS(HA-HLT(1)).LT.ABS(HA-HLT(2))) THEN
         HFINAL=HLT(1)
      ELSE
         HFINAL=HLT(2)
        END IF
C        PRINT* , 'The actual main channel depth is ', HFINAL
C        PAUSE
C
C
C
C!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
C
C
C       So and Sc to be input by user
C
C       Flags defining LB RB FPL and FPR required
C
C       Bankfull level to be defined
C
C
C       A, P, R, Q for each zone required
C
C       Shifted A, P, R, Q for each zone required
C
C
C       Determine the bottom width of the main channel
      B2=WC2-2*HFINAL*SC
C       PRINT*, 'The bottom width of the main channel , 2b is', B2
C
C       Identify the positions of the backs of the flood plains
C
      BP2=gx-fx
C        PRINT* , 'The location of the backs of'
C        PRINT*, 'the flood plains ,2B is', BP2
C        PAUSE
C
C  Loop to pick out wl,a,p,r,n from holding arrays
C
C      DO IP = 1,1
      areaz(90)=0
```

```
      FC=0
      FF=0
      HSTR=0
      G1=0
      QB=0
      QCB=0
      QFB=0
      VC=0
      ACH=0
      VF=0
      ARF=0
      QSTR2C=0
      QSTR2F=0
      NF=0
      HFD=0
      DISDEF=0
      QR1=0
      SHFT=0
      HSH=0
      QSHFLB=0
      ASHF=0
      PSHF=0
      RSHF=0
      VSHF=0
      FZZ=0
      PZZ=0
      AZZ=0
      FSHF=0
      COH=0
      COH3=0
      QACK=0
C      WC2=0
      RSHC=0
      VSHC1=0
      FSHC=0
      AZ3=0
      FZ3=0
      PZ3=0
C
      DO IHT= 1, NHTS
      LEVEL=HYT(IHT)
C      AREA=PANEL(IHT,IP,1)
C      PERIM=PANEL(IHT,IP,2)
C      CONVY=PANEL(IHT,IP,3)
C      HYRAD=PANEL(IHT,IP,4)
C      RGHNS=PANEL(IHT,IP,5)
C      write(*,99999) 'area,convy,hyrad,rghns,level'
C      write(*,99998) area,convy,hyrad,rghns,level
C99998 format(5e12.4)
C      END DO
C      END DO
C      WRITE(*,99999)((I,J,HYT(I),(PANEL(I,J,K1),K1=1,5),J=1,3),I=1,20)

C
C       Calculate HFD, the flow depth
C
      KWL=LEVEL
C        PRINT*,'The water level of calculation is ', KWL
```

```
      HFD=KWL-(RBL-HFINAL)
C        PRINT* , 'The flow depth of the main channel, H is', HFD

C
CCalculate the basic discharges
C
CFirst step is to calculate flood plain areas
C
C
       AFLSURV=PANEL(IHT,1,1)
C         PRINT* ,'The left flood plain area is ', AFLSURV
       PFLSURV=PANEL(IHT,1,2)
C         PRINT* ,'The wetted perimeter of the left flood plain is',
C     +         PFLSURV
      RFL=PANEL(IHT,1,4)
C        PRINT*,'The hydraulic radius of the left flood plain R = ',
C     +  PRINT* ,'RFL
      NPL=PANEL(IHT,1,5)
C        PRINT*,'The left flood plain roughness is = ', NPL
C Calculate the basic discharge for the left flood plain
C
         IF(RFL.GT.0)THEN
      QFLB=(AFLSURV/NPL)*(RFL**0.667)*(SO**0.50)
C        PRINT*, 'The basic discharge for the left flood plain is', QFLB
         END IF
C
C Right Flood Plain Properties
      AFRSURV=PANEL(IHT,3,1)
C        PRINT* ,'The right flood plain area =', AFRSURV
      PFRSURV=PANEL(IHT,3,2)
C         PRINT*,'The right flood plain Perimeter =',PFRSURV
      RFR=PANEL(IHT,3,4)
C        PRINT*,'The right flood plain Hydraulic Radius =',RFR
      NPR=PANEL(IHT,3,5)
C        PRINT*,'The right flood plain roughness =', NPR
C Calculate the basic discharge for the right flood plain
         IF(RFR.GT.0)THEN
      QFRB=(AFRSURV/NPR)*(RFR**0.667)*(SO**0.5)
C        PRINT*,'The basic discharge for the right flood plain is',
C     +  QFRB
      NP=(NPL+NPR)/2
C Calculate the total basic discharge for the flood plains
      QFB=QFLB+QFRB
C        PRINT*,'The total basic discharge for the flood plains is',
C     +     QFB
        END IF
C
      ACH=PANEL(IHT,2,1)
C        PRINT*,'The main channel area is = ', ACH
      PCH=PANEL(IHT,2,2)
C        PRINT* ,'The main channel Preimeter =', PCH
      RC=PANEL(IHT,2,4)
C        PRINT*,'The hydraulic radius of the main channel =', RC
      NC=PANEL(IHT,2,5)
C        PRINT*,'The main channel roughness = ' , NC
C
C Calculate the basic discharge of the main channel
C
        IF(RC.GT.0.AND.NC.GT.0)THEN
```

```
      QCB=(ACH/NC)*(RC**0.667)*(SO**0.5)
C        PRINT* ,'The basic discharge for the main channel is', QCB
      QB=QCB+QFB
C        PRINT* ,'The combined zonal discharge for the whole x-section'
C        PRINT* ,'is', QB
       END IF
C        PAUSE
C
C       The following are Ackers Method adjustments
C
C       Step 7.0
C
C       Step 7.1 Adjust QB assuming flow is in Region 1
C
         IF(HFD.GT.0)THEN
      HSTR=(HFD-HFINAL)/HFD
C         PRINT* ,'The ratio of flow depths on FPs and'
C         PRINT* ,'main channel is', HSTR
          END IF
C
C       Step 7.2 Calculate the Darcy Weisbach friction factors
C
         IF(ACH.GT.0)THEN
      VC=QCB/ACH
C         PRINT* ,'Vc is equal to', VC
          END IF
C
C       G presumably is already defined within ISIS
C       G=9.81
C         IF(VC.GT.0)THEN

      FC=((8*G)*RC*SO)/VC**2
C         PRINT* ,'The main channel darcy weisbach friction factor, fc is'
C    +    FC
          END IF

C         IF(AFLSURV.GT.0.OR.AFRSURV.GT.0)THEN
      VF=QFB/(AFLSURV+AFRSURV)
C         PRINT* ,'Vf is equal to', VF
          END IF
C
          IF(PFLSURV.GT.0.OR.PFRSURV.GT.0)THEN
      RF=(AFLSURV+AFRSURV)/(PFLSURV+PFRSURV)
C         PRINT* ,'The combined flood plain Rf is', RF
          END IF
C
          IF(VF.GT.0)THEN
      FF=(8*G)*(RF)*(SO)/VF**2
C         PRINT* ,'The flood plain darcy-weisbach'
C         PRINT*,'friction factor, Ff is', FF
          END IF
C
C       Step 7.3 Calculate the dimensionless flood plain discharge defici
t
C
          IF(FF.GT.0)THEN
      QSTR2F=-1.0*HSTR*(FC/FF)
```

```
C      PRINT* ,'The dimensionless flood plain discharge'
C      PRINT* ,'deficit is', QSTR2F
        END IF
C
C      Step 7.4 Calculate the dimensionless main channel discharge defec
it
C
C      There are 2 formulas for G1 depending on Sc
C      See page 32 of manual
C
C  How many flood plains are there? 1 or 2
C
C
       IF(NPANEL.GE.3)THEN
     NF=2
     ELSE IF(NPANEL.LT.3)THEN
     NF=1
       END IF
C       PRINT*, 'The number of flood plains is equal to ', NF
C
        IF (SC.GE.1.0) THEN
         IF(FC.GT.0)THEN
     G1=10.42+0.17*(FF/FC)
         END IF
        ELSE IF (SC.LT.1.0) THEN
      IF(FC.GT.0)THEN
    G1=10.42+0.17*(SC*FF/FC)+0.34*(1.0-SC)
      END IF
      END IF
C      PRINT* ,'G is equal to', G1
C
        IF (NF.EQ.1) THEN
         IF(WC2.GT.0)THEN
       QSTR2C=-1.240+0.395*((BP2/2)/(WC2/2))+G1*HSTR
         END IF
        ELSE IF(NF.EQ.2)THEN
         IF(WC2.GT.0)THEN
       QSTR2C=-1.240+0.395*(BP2/WC2)+G1*HSTR
         END IF
        END IF
C      PRINT*, 'The dimensionless main channel'
C      PRINT*, 'discharge deficit is ', QSTR2C
C
      IF(QSTR2C.LT.0.5)THEN
    QSTR2C=0.5
    QSTR2F=0
      END IF
C
C
C     Step 7.5 Calculate the aspect ratio adjustment factor
C
C     ARF should not exceed 2.0
C     If the calculated value is greater than this set it to 2.0
C
      IF(HFINAL.GT.0)THEN
    ARF=B2/(10*HFINAL)
      END IF
C
      IF(ARF.GT.2.0)THEN
```

```
      ARF=2.0
        END IF
C
C        PRINT* ,'ARF is equal to ', ARF
C        PAUSE
C       Calculate the total discharge deficit
C
C      Step 7.6 Calculate the region 1 discharge defecit
C
      DISDEF=(QSTR2C+(NF*QSTR2F))*(VC-VF)*HFD*HFINAL*ARF
C        PRINT* ,'The total discharge deficit, DISDEF is', DISDEF
C
C
C       Step 7.7 Calculate the region 1 adjusted discharge
C
        IF(QB.GT.0)THEN
      QR1=QB-DISDEF
C        PRINT* ,'The Region 1 adjusted discharge is', QR1
        END IF
C        PAUSE
C
C       Step 8 Adjust Qbasic assuming flow is in region 2
C
C       Step 8.1 Calculate the shift
C
C       There are 2 formulas for calculating the shift depending on Sc
C See page 32 of manual
        IF (SC.GE.1.0) THEN
         SHFT=0.05+(0.05*NF)
        ELSE IF (SC.LT.1.0) THEN
         SHFT=-0.01+(0.05*NF)+(0.06*SC)
        END IF
C
C        PRINT* ,'The shift to be applied is ', SHFT
C        PAUSE
C
C
C       Step 8.2 Calculate shifted flow depth
C
C
         HSH=(HFD*HFINAL)/(HFINAL-(SHFT*HFD))
C        PRINT* ,'The shifted flow depth is', HSH
C
C
C
C Step 8.3 Calculate the channel coherence for the shifted flow depth
C
C
         HZH1=RBL-HFINAL+HSH
C        PRINT* ,'H corresponds to a water level of', HZH1
C        PAUSE
C
C Add code that limits HZH1 to highest y Co-ord (YMAX1)
C
      IF(HZH1.GT.YMAX1)THEN
        HZH1=YMAX1
      END IF
```

```
      DO IZZ= 1, NHTS
      AREAZ(IZZ)=PANEL(IZZ,1,1)
      END DO
        IF(HZH1.GE.HYT(1).AND.HZH1.LE.HYT(NHTS))THEN
      IPTR=1
      CALL LINTRP(HYT,AREAZ,NHTS,HZH1,ASHFL,IPTR)
C        PRINT* , 'The interpolated Shifted left FP  area = ', ASHFL
        ELSE
          CALL PERROR(2700,'ACKERS',label)
      WRITE(ERRMSG,'(''Shft H too high, Try increasing DFlood.'')')
      CALL WRERR(NSTDER, WINFLG, CFILE, ERRMSG)
        END IF
C        PAUSE
      DO IZZ=1, NHTS
      PERIMZ(IZZ)=PANEL(IZZ,1,2)
      END DO
      IPTR=1
      CALL LINTRP(HYT,PERIMZ,NHTS,HZH1,PSHFL,IPTR)
C        PRINT* ,'The shifted left FP perim = ', pshfl
C        PAUSE
      DO IZZ=1, NHTS
      HYRADZ(IZZ)=PANEL(IZZ,1,4)
      END DO
      IPTR=1
      CALL LINTRP(HYT,HYRADZ,NHTS,HZH1,RSHFL,IPTR)
C        PRINT* ,'The shifted left FP R = ', RSHFL
C        PAUSE
      DO IZZ=1, NHTS
      NPZ(IZZ)=PANEL(IZZ,1,5)
      END DO
      IPTR=1
      CALL LINTRP(HYT,NPZ,NHTS,HZH1,NP,IPTR)
C        PRINT* ,'The mannings n value is = ' , NP
C        PAUSE
C      Calculate the shifted basic discharge
C
        IF(NP.GT.0)THEN
      QSHFLB=(ASHFL/NP)*(RSHFL**0.667)*(SO**0.5)
C        PRINT* ,'The shifted basic discharge for the left '
C        PRINT* ,'flood plain is', QSHFLB
        END IF
C        PAUSE
      DO IZZ= 1, NHTS
      AREAZ(IZZ)=PANEL(IZZ,3,1)
      END DO
      IPTR=1
      CALL LINTRP(HYT,AREAZ,NHTS,HZH1,ASHFR,IPTR)
C        PRINT*, 'The interpolated right FP  area = ', ASHFR
C        PAUSE
      DO IZZ=1, NHTS
      PERIMZ(IZZ)=PANEL(IZZ,3,2)
      END DO
      IPTR=1
      CALL LINTRP(HYT,PERIMZ,NHTS,HZH1,PSHFR,IPTR)
C        PRINT* ,'The shifted right FP perim = ', PSHFR
C        PAUSE
      DO IZZ=1, NHTS
      HYRADZ(IZZ)=PANEL(IZZ,3,4)
      END DO
```

```
      IPTR=1
      CALL LINTRP(HYT,HYRADZ,NHTS,HZH1,RSHFR,IPTR)
C        PRINT* ,'The shifted right FP R = ', RSHFR
      DO IZZ=1, NHTS
      NPZ(IZZ)=PANEL(IZZ,3,5)
      END DO
      IPTR=1
      CALL LINTRP(HYT,NPZ,NHTS,HZH1,NP,IPTR)
C        PRINT* ,'The mannings n value is = ' , NP
C        PAUSE
C        Calculate the shifted basic discharge
         IF(NP.GT.0)THEN
      QSHFRB=(ASHFR/NP)*(RSHFR**0.667)*(SO**0.5)
C        PRINT* ,'The shifted basic discharge for the right'
C        PRINT* ,'flood plain is', QSHFRB
         END IF
      DO IZZ=1, NHTS
      AREAZ(IZZ)=PANEL(IZZ,2,1)
      END DO
      IPTR=1
      CALL LINTRP(HYT,AREAZ,NHTS,HZH1,ASHC,IPTR)
C        PRINT* , 'The interpolated main channel area = ', ASHC
C        PAUSE
      PSHC=PCH
C        PRINT*,'The shifted P is the same as before ie = ', PSHC
C        PAUSE
      DO IZZ=1,NHTS
      HYRADZ(IZZ)=PANEL(IZZ,2,4)
      END DO
      IPTR=1
      CALL LINTRP(HYT,HYRADZ,NHTS,HZH1,RSHC,IPTR)
C        PRINT*,'The shifted main channel R = ', RSHC
C        PAUSE
      DO IZZ=1, NHTS
      NPZ(IZZ)=PANEL(IZZ,2,5)
      END DO
      IPTR=1
      CALL LINTRP(HYT,NPZ,NHTS,HZH1,NC,IPTR)
C        PRINT* ,'The mannings n value is = ' , NC
C        PAUSE
C        Calculate the shifted basic discharge
        IF(NC.GT.0)THEN
      QSHCB=(ASHC/NC)*(RSHC**0.667)*(SO**0.5)
C        PRINT* ,'The shifted basic discharge for the main'
C        PRINT* ,'channel is', QSHCB
        END IF
C
C       Calculate the shifted friction factor for the flood plains
      ASHF=ASHFL+ASHFR
C        PRINT* ,'ASHF is equal to', ASHF
C        PAUSE
      PSHF=PSHFL+PSHFR
C        PRINT* ,'PSHF is equal to', PSHF
C        PAUSE
        IF(PSHF.GT.0)THEN
      RSHF=ASHF/PSHF
C        PRINT*,'RSHF is equal to', RSHF
        END IF
C        PAUSE
```

```
            IF(ASHF.GT.0)THEN
          VSHF=(QSHFLB+QSHFRB)/ASHF
C             PRINT* ,'VSHF is equal to', VSHF
            END IF
C           PAUSE
            IF(VSHF.GT.0)THEN
          FSHF=(8*G*RSHF*SO)/(VSHF**2)
C             PRINT* ,'The shifted friction factor for the flood '
C             PRINT* ,'plains is', FSHF
            END IF
C           PAUSE
C           Calculate the shifted friction factor for the main channel
            IF(ASHC.GT.0)THEN
          VSHC1=QSHCB/ASHC
C             PRINT* ,'VSHC1 is equal to', VSHC1
            END IF
C           PAUSE
            IF(VSHC1.GT.0)THEN
          FSHC=(8.0*G*RSHC*SO)/(VSHC1**2)
C             PRINT* ,'The main channel shifted friction factor '
C             PRINT* ,'is equal to', FSHC
            END IF
C           PAUSE
C
C           Calculate the channel coherence
            IF(FSHC.GT.0)THEN
          FZZ=FSHF/FSHC
C             PRINT* ,'F* is equal to', FZZ
            END IF
C           PAUSE
            IF(PCH.GT.0)THEN
          PZZ=PSHF/PCH
C             PRINT* ,'P* is equal to', PZZ
            END IF
C           PAUSE
            IF(ASHC.GT.0)THEN
          AZZ=ASHF/ASHC
C             PRINT* ,'A* is equal to', AZZ
            END IF
C           PAUSE
            IF(AZZ.GT.0.AND.FZZ.GT.0.AND.PZZ.GT.0)THEN
          COH=((1+AZZ)*((1+AZZ)/(1+FZZ*PZZ))**0.5)/(1+AZZ*
     +          (AZZ/(FZZ*PZZ))**0.5)
C             PRINT* ,'The channel coherence for the shifted flow '
C             PRINT* ,'depth is', COH
            END IF
C           Step 8.4 Define the region 2 discharge adjustment factor
C
          DISADF2=COH
C             PRINT* ,'The Region 2 discharge adjustment factor is', COH
C             PAUSE
C           Step 8.5 Calculate the region 2 adjusted discharge
C
          QR2=QB*DISADF2
C             PRINT* ,'The region 2 adjusted discharge is', QR2
C
C           Step 9 Determine if QR1 is the actual discharge Q
C
C           PRINT* ,'QR1 is equal to', QR1
```

```
C          PRINT* ,'QR2 is equal to', QR2
C          PAUSE
           IF(QR1.GE.QR2)THEN
         Q=QR1
C          PRINT* ,'Q=QR1=', QR1
C          PAUSE
            END IF
C          PRINT* , 'If QR1 < QR2 then the actual discharge'
C          PRINT*, 'is still unknown'
C          PAUSE
C          Step 10.1 Calculate the Coherence assuming region three flow
C
C           PRINT* ,'The following are calculated assuming region 3 flow'
C           PAUSE
C
          IF(ACH.GT.0)THEN
        AZ3=(AFLSURV+AFRSURV)/ACH
C           PRINT* ,'A* is equal to', AZ3
          END IF
C          PAUSE
          IF(FC.GT.0)THEN
        FZ3=FF/FC
C           PRINT* ,'F* is equal to', FZ3
          END IF
C           PAUSE
          IF(PCH.GT.0)THEN
        PZ3=(PFLSURV+PFRSURV)/PCH
C          PRINT* ,'P* is equal to', PZ3
          END IF
C          PAUSE
          IF(AZ3.GT.0.AND.FZ3.GT.0.AND.PZ3.GT.0)THEN
        COH3=((1+AZ3)*((1+AZ3)/(1+FZ3*PZ3))**0.5)/(1+AZ3*(AZ3/
     +         (FZ3*PZ3))**0.5)
C          PRINT* ,'The Coherence assuming region 3 flow is', COH3
          END IF
C          PAUSE
C         Step 10.1 Calculate the region 3 adjustment factor
C
        DISADF3=1.567-(0.667*COH3)
C          PRINT*,'The Region 3 adjustment factor is', DISADF3
C          PAUSE
C
C Step 10.2 Calculate the region 3 adjusted discharge for specified water
 level
C
        QR3=QB*DISADF3
C          PRINT* ,'The Region 3 adjusted discharge for the '
C          PRINT* ,'specified water level is', QR3
C          PAUSE
C Step 11 Determine if QR3 is the actual discharge
C
C         PRINT* ,'QR2 is equal to', QR2
C         PRINT*,'QR3 is equal to', QR3
           IF(QR2.LE.QR3)THEN
         Q=QR2
C         PRINT* ,'As QR2 is less than or equal to QR3 then Q is', QR2
C         PAUSE
           END IF
C         PRINT* ,'If QR2 < QR3 then the actual discharge is QR2'
```

```
C         PAUSE
C
C         Calculate the region 4 flow
C         Step 12.1 Adjust QB assuming flow in region 4
C
        DISADF4=COH3
C          PRINT* ,'DISADF4 is equal to', DISADF4
C          PAUSE
C         Step 12.2 Calculate the region 4 adjusted discharge
C
        QR4=QB*DISADF4
C          PRINT* ,'The adjusted region 4 discharge is equal to', QR4
C          PAUSE
C          PRINT* ,'QR1 is equal to ', QR1
C          PRINT* ,'QR2 is equal to ', QR2
C          PRINT* ,'QR3 is equal to ', QR3
C          PRINT* ,'QR4 is equal to ', QR4
C          PAUSE
C
C Select the Correct discharge
C
        IF (QR1.GE.QR2)THEN
           QACK=QR1
        ELSE IF (QR1.LT.QR2.AND.QR2.LE.QR3)THEN
           QACK=QR2
        ELSE IF (QR1.LT.QR2.AND.QR3.LT.QR2)THEN
           QACK=QR3
        ELSE IF (QR4.GT.QR3)THEN
           QACK=QR4
        END IF
C
C          PRINT* ,'The Ackers method discharge Qack is ', QACK
C          PAUSE
C
C         Calculate the Ackers adjusted conveyance ie K=Qackers/(so**0.5)
           IF(SO.GT.0)THEN
        K=QACK/(SO**0.5)
C        PRINT* ,'The Ackers method Conveyance is ', K
           END IF
          CCHNL(IHT)=K
C          PAUSE
C
C
C CALCULATION OF BETA PARAMETER

         ATOT=ACSURV+AFLSURV+AFRSURV
         PRINT* , 'Total Area is ', ATOT
          IF(K.GT.0.AND.AT.GT.0)THEN
         BETA = ((ATOT/K**2))*((K**2)/(ATOT))
         PRINT* , 'BETA is equal to ', BETA
          END IF

           IF(HYT(IHT).GT.RBL)THEN
             BETA(IHT)=BETA
           END IF
C
C
        END DO
C
```

```
C         PAUSE
C
CC!!!!!!!This will be the end of the conveyance loop!!!!!!!!!!!!!!!!!!!!!
C
C
      return

      END
```

Appendix 2

```fortran
C      Last change:  CE   24 Aug 98    9:09 am
       subroutine Jmswk(panel,abl,py,sy,px,sx,nhts,hyt,sin,ss,l,mx,
     +          nx,FPS,rbl,G,CCHNL,BETA)
C
C Program uses the James and Wark Method to calculate the
C Conveyance of Meandering Channels with Overbank Flow
C
       IMPLICIT NONE
       REAL A, PERIM(90), TW, LCL, SLD, SIN, FPS, S, LBMCS, RBMCS,
     +         A2, P2, W2, A3, P1, N1I(90)
       REAL P3, A4, P4, N1, NDSH, R, V, QBF, YDSH, Y2, QDSH1, B2A, FDSH,
     +  R2, QDSH2
       REAL K, C, QDSH, SS, N2, M, Q1, L, SF2, BF1, BF2, CSL, CWD, CSSE,
     +CSSC, H, ZED, BETA, AT
       REAL KC, KE, V2, G, Q2, N3, R3, V3, Q3, N4, R4, V4, Q4, QT, GAM,
     +TU, TD, NWL, KON, ABL, PY, SY, PANEL(90,15,5),AREA, ACHNL,
     +PERIMS, CONVY, HYRAD, RGHNS, LEVEL, HYT(110), SX, PX, A2MID,
     + A2LFT, A2RGT, P2LFT, P2MID, P2RGT, NX, MX, NL2, NR2, RBL,
     +   Q2RGT, Q2MID, Q2LFT, R2LFT, R2MID, R2RGT, V2LFT, R2TOTAL,
     +V2RGT, V2MID, CCHNL(NHTS), KON1, KON2, KON3, KON4, BETA(NHTS)
       INTEGER I, J, K1, NHTS, IP, IHT, IPTR
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

       DO IHT=1, NHTS
       PERIM(IHT)=PANEL(IHT,3,2)
       END DO
       IPTR=1
       CALL LINTRP(HYT,PERIM,NHTS,RBL,P1,IPTR)
       PRINT* ,'Main Channel p1 = ', P1

       DO IHT=1, NHTS
       N1I(IHT)=PANEL(IHT,3,5)
       END DO
       IPTR=1
       CALL LINTRP(HYT,N1I,NHTS,RBL,N1,IPTR)
       PRINT* ,'N1 =', N1

       WRITE(*,*) 'The interpolated zone 1 area is = ', ABL
C       PRINT* ,'The main channel area = ', ABL
C       PAUSE
        IF(P1.GT.0)THEN
       R=ABL/P1
       PRINT* ,'R1 =', R
        END IF


       TW=SX-PX
       PRINT* ,'The main channel top width = ', TW

        PRINT* ,'FPS =', FPS
        PRINT* ,'SIN =', SIN


C Changing Flood Plain Slope to main channel slope

```

```
C          IF(SIN.GT.0)THEN
C          S=FPS/SIN
             PRINT* ,'The main channel slope is equal to ', S
           END IF

C          R=0
C          V=0

C The following calculation adjusts manning's n to account for
C meander losses, it uses the Linearised SCS method

C
            IF(SIN.LT.1.7)THEN
          NDSH=N1*((0.43*SIN)+0.57)
          PRINT* ,'The corrected Coefficient n''is', NDSH
            ELSE IF(SIN.GE.1.7) then
          NDSH=N1*1.30
          PRINT* ,'The corrected Coefficient n''is', NDSH
            END IF

          IF (NDSH.GT.0)THEN
          V=(1/NDSH)*(R**0.667)*(S**0.5)
          PRINT* ,'V is equal to  ', V
          END IF
C          PAUSE

          QBF=ABL*V
C          PRINT* ,'The bankfull discharge is ', QBF
C THE FOLLOWING SHOULD BE COMMENTED OUT FOR NATURAL RIVERS
            IF(QBF.GT.0.0385)THEN
              QBF=0.0385
            ELSE IF(QBF.LT.0.0310)THEN
              QBF=0.02970
           PRINT*,'The bankfull discharge is ', QBF
            END IF

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C
C          PRINT*,'This program is for Conveyance of Meandering Channels'
C          PRINT*,'with overbank flow'
C
C Areas and perimeters will now be entered or calculated
C
C          REAL PANEL(90,15,5)
C          WRITE(*,99999)((I,J,(PANEL(I,J,K1),K1=1,5),J=1,3),I=1,20)
C99999 FORMAT(2I3,1X,5E12.4)

C
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC


C Loop to pick out wl,a,p,r,n from holding arrays


C          DO IP = 1,3

            DO IHT= 1, NHTS

C           IF(HYT(IHT).EQ.RBL)THEN
C          PRINT* ,'R =', R
```

```
 C        PRINT*,'A=', A
 C        PRINT*, 'LEVEL=',LEVEL
 C        R=0
 C        V=0
 C       QBF=0
 C       A2=0
         P2=0
         R2=0
         FDSH=0
         ZED=0
 C        KE=0
         V2=0
         R3=0
         A3=0
         P3=0
         R3=0
         V3=0
 C        FPS=0
         R4=0
         V4=0
 C        SF2=0
 C       BF1=0
 C       BF2=0
 C       KE=0
 C       Q2LFT=0
 C        A2LFT=0
 C        R2LFT=0
 C       V2LFT=0
 C        Q2RGT=0
 C        A2RGT=0
 C        R2RGT=0
 C        V2RGT=0
 C        Q2MID=0
 C        A2MID=0
 C        V2MID=0
 C        P2LFT=0
 C        P2RGT=0

 C        L=0
 C        KON=0
 C        QT=0
 C
 C        LEVEL=HYT(IHT)
 C        AREA=PANEL(IHT,IP,1)
 C        PERIMS=PANEL(IHT,IP,2)
 C        CONVY=PANEL(IHT,IP,3)
 C        HYRAD=PANEL(IHT,IP,4)
 C        RGHNS=PANEL(IHT,IP,5)
 C
 C        WRITE(*,99997) 'AREA,CONVY,HYRAD,RGHNS,LEVEL'
 C        WRITE(*,99997) AREA,CONVY,HYRAD,RGHNS,LEVEL
 C99997 FORMAT(5e12.4)
 C
        LEVEL=HYT(IHT)
        PRINT*,'The water level of calculation is', LEVEL
```

```
         P2=P2LFT+P2RGT
        END IF

C       ELSE IF(LEVEL.LT.RBL)THEN

C      P2=P2LFT+P2RGT

C       END IF

      PRINT* ,'The wetted perimeter of the inner flood plain is', P2

       IF(P2.GT.0)THEN
      R2=A2/P2
       END IF
      PRINT* ,'The hydraulic radius of zone 2 is ', R2

C      PAUSE

      W2=NX-MX
      PRINT* ,'The width of the inner flood plain is equal to ', W2



C Obtain or calculate outer flood plain areas and wetted perimeters
C Zone 3


      A3=PANEL(IHT,1,1)
C      PRINT* ,'The area of the left outer'
C      PRINT* ,'flood plain, zone 3, is ', A3

      P3=PANEL(IHT,1,2)
C      PRINT* ,'The left outer, ZONE 3, wetted perimeter is ', P3
C      PAUSE

      A4=PANEL(IHT,5,1)
C      PRINT* ,'The right outer flood plain, zone 4, area is ', A4

      P4=PANEL(IHT,5,2)
C      PRINT* ,'The right outer wetted perimeter, zone 4, is ', P4


C      N1=PANEL(IHT,3,5)
C      PRINT* ,'The main channel roughness N1 is', N1

      NL2=PANEL(IHT,2,5)
      PRINT* ,'Zone 2 left n = ', NL2

      NR2=PANEL(IHT,4,5)
      PRINT* , 'Zone 2 right n = ', NR2

      N3=PANEL(IHT,1,5)
      PRINT* ,'Zone 3 n = ', N3

      N4=PANEL(IHT,5,5)
      PRINT* , 'Zone 4 n = ', N4

      N2=(NL2+NR2)/2
```

```
      IF(NL2.EQ.0)THEN
         N2=NR2
      ELSE IF(NR2.EQ.0)THEN
         N2=NL2
      END IF
      PRINT* ,'Therefore the zone 2 n =', N2
C     PAUSE

C Calculate the discharge for depth above bankfull
C Calculate ZONE 1 DISCHARGE

C The zone 1 adjustment factor QDSH1 is the greater of the values given
C by two separate equations (see design manual)

      Y2=LEVEL-RBL
      PRINT* ,'The flow depth on the flood plain Y2'
      PRINT*,'at main channel bank is ', Y2

      PRINT* ,'ABL =', ABL
      PRINT*,'TW =' , TW

      YDSH=Y2/(ABL/TW)
      PRINT* ,'Y''is equal to', YDSH
C      PAUSE

      QDSH1=1.0-(1.69*YDSH)
      PRINT* ,'The first method of calculating Q1''is equal to', QDSH1

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC


      G=9.81
      PRINT* ,'L = ', L
      PRINT* ,'S = ', S
      PRINT*,'G =' , G
      PRINT*, 'SS = ', SS

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

C Use second equation to calculate Q1'

      B2A=(TW**2)/ABL
      PRINT* ,'B2A is equal to ', B2A


      IF(N1.GT.0.AND.R2.GT.0)THEN
      FDSH=((N2/N1)**2)*((R/R2)**0.333)
      PRINT* ,'f''is equal to',FDSH
      END IF

      M=(0.0147*B2A)+(0.0320*FDSH)+0.169
      PRINT* ,'M is equal to',M
C
      K=1.14-(0.136*FDSH)
```

```
      PRINT* ,'K is equal to ', K

      C=(0.0132*B2A)-(0.302*SIN)+0.851
      PRINT* ,'C is equal to ', C

      QDSH2=(M*YDSH)+(K*C)
      PRINT* ,'The second method of calculating Q1''is equal to', QDSH2
C       PAUSE


C       PRINT* ,'The bigger value of Q1''is used'

       IF(QDSH1.GT.QDSH2)THEN
          QDSH=QDSH1
          PRINT* ,'Q1''is equal to', QDSH
       ELSEIF(QDSH1.LT.QDSH2)THEN

            QDSH=QDSH2
          PRINT* ,'Q1''is equal to', QDSH
C
          PAUSE
       ENDIF

C Therefore the discharge in zone 1 can be calculated

      Q1=QDSH*QBF
      PRINT*,'Therefore the discharge in zone 1 is', Q1


C
C Calculate ZONE 2 DISCHARGE
C
C The average meander wavelength is estimated by dividing the
C flood plain length by the number of wavelengths over the reach
C
C
      IF(W2.GT.0)THEN
      CSL=(2*(W2-TW))/W2
      PRINT* ,'Csl is equal to', CSL
      END IF

C
      CWD=(0.02*B2A)+0.69
      PRINT* ,'Cwd is equal to', CWD


C
      CSSE=1.0-(SS/5.7)
      PRINT* ,'Csse is equal to', CSSE

        IF(CSSE.LT.0.1)THEN
           CSSE=0.1
        END IF

C
      CSSC=1.0-(SS/2.5)
      PRINT* ,'Cssc is equal to', CSSC
```

```
      IF(CSSC.LT.0.1)THEN
        CSSC=0.1
        END IF
      PAUSE
C
      IF(TW.GT.0)THEN
      H=ABL/TW
      PRINT* ,'h is equal to', H
      END IF

C
      IF(Y2.GT.0.AND.H.GT.0)THEN
      ZED=Y2/(Y2+H)
      PRINT* ,'Y2/(Y2+h) is equal to', ZED
      END IF
      PAUSE


CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

C      KC= 0.217

C  The following is for the selection of kc

      IF(ZED.EQ.0)THEN
       KC=0.5
      ELSE IF(ZED.GT.0.AND.ZED.LT.0.1)THEN
       KC=0.5-((ZED/0.1)*0.02)
      ELSE IF(ZED.EQ.0.1)THEN
       KC=0.48
      ELSE IF(ZED.GT.0.1.AND.ZED.LT.0.2)THEN
       KC=0.48-(((ZED-0.1)/0.1)*0.03)
      ELSE IF(ZED.EQ.0.2)THEN
       KC=0.45
      ELSE IF(ZED.GT.0.2.AND.ZED.LT.0.3)THEN
       KC=0.45-(((ZED-0.2)/0.1)*0.04)
      ELSE IF(ZED.EQ.0.3)THEN
       KC=0.41
      ELSE IF(ZED.GT.0.3.AND.ZED.LT.0.4)THEN
       KC=0.41-(((ZED-0.3)/0.1)*0.05)
      ELSE IF(ZED.EQ.0.4)THEN
       KC=0.36
      ELSE IF(ZED.GT.0.4.AND.ZED.LT.0.5)THEN
       KC=0.36-(((ZED-0.4)/0.1)*0.07)
      ELSE IF(ZED.EQ.0.5)THEN
       KC=0.29
      ELSE IF(ZED.GT.0.5.AND.ZED.LT.0.6)THEN
       KC=0.29-(((ZED-0.5)/0.1)*0.08)
      ELSE IF(ZED.EQ.0.6)THEN
       KC=0.21
      ELSE IF(ZED.GT.0.6.AND.ZED.LT.0.7)THEN
       KC=0.21-(((ZED-0.6)/0.1)*0.08)
      ELSE IF(ZED.EQ.0.7)THEN
       KC=0.13
      ELSE IF(ZED.GT.0.7.AND.ZED.LT.0.8)THEN
       KC=0.13-(((ZED-0.7)/0.1)*0.06)
      ELSE IF(ZED.EQ.0.8)THEN
       KC=0.7
```

```
      ELSE IF(ZED.GT.0.8.AND.ZED.LT.0.9)THEN
       KC=0.07-(((ZED-0.8)/0.1)*0.06)
      ELSE IF(ZED.EQ.0.9)THEN
       KC=0.01
      ELSE IF(ZED.GT.0.9.AND.ZED.LT.1)THEN
       KC=0.01-(((ZED-0.9)/0.1)*0.01)
      ELSE IF(ZED.EQ.1.0)THEN
       KC=0
      END IF

      IF(KC.LT.0)THEN
        KC=0
      END IF

      PRINT* , 'KC IS EQUAL TO ', KC


      KE=CSL*CWD*(CSSE*(1-ZED)**2+CSSC*KC)
      PRINT* ,'Ke is equal to', KE

      IF(R2.GT.0)THEN
       SF2=(8*G*(N2**2))/(R2**0.33333)
       PRINT* ,'f2 is equal to', SF2
      END IF

C
       IF(B2A.LT.10.0)THEN
      BF1=0.1*B2A
      PRINT* ,'F1 is equal to', BF1
       ELSE IF(B2A.GE.10.0)THEN
      BF1=1.0
      PRINT* ,'F1 is equal to', BF1
       END IF

C

      BF2=SIN/1.4
      PRINT* ,'F2 is equal to', BF2

        IF(LEVEL.GT.RBL)THEN

       IF(SF2.NE.0.AND.R2.NE.0.AND.BF1.NE.0.AND.BF2.NE.0.
     +AND.KE.NE.0.AND.SF2.GT.0.AND.R2.GT.0.AND.BF1.GT.0.
     +AND.BF2.GT.0.AND.KE.GT.0)THEN

       V2=((2*G*FPS*L)/(((SF2*L)/(4*R2))+(BF1*BF2*KE)))**0.5

       END IF

       PRINT* ,'V2 is equal to', V2
        END IF
C
C
C

       Q2=A2*V2
       PRINT* ,'Therefore the discharge in zone 2 is', Q2
```

```
C
C Calculate ZONE 3 DISCHARGE
C
      PRINT*,'The zone 3 mannings n is', N3
      PRINT* ,'The Zone 3 area is ', A3
      PRINT*,'The Zone 3 wetted Perimeter is ', P3

       IF(P3.GT.0)THEN
       R3=A3/P3
      PRINT* ,'R3 is equal to', R3
        END IF
C
       IF(N3.GT.0)THEN
         V3=(1/N3)*(R3**0.6667)*(FPS**0.5)
        PRINT* ,'V3 is equal to', V3
       END IF
C
      Q3=A3*V3
      PRINT* ,'Therefore the zone three discharge is equal to', Q3

C
C Calculate ZONE 4 DISCHARGE
C
      PRINT* ,'The zone 4 mannings n is ', N4
      PRINT* ,'The zone 4 area is ', A4
      PRINT* ,'The zone 4 wetted perimeter is ', P4

      IF(P4.GT.0)THEN
      R4=A4/P4
      PRINT* ,'R4 is equal to', R4
C      PAUSE
      END IF

C
      IF(N4.GT.0)THEN
      V4=(1/N4)*(R4**0.667)*(FPS**0.5)
      PRINT* ,'V4 is equal to', V4
      END IF
      PAUSE

      Q4=A4*V4
      PRINT* ,'Therefore the zone 4 discharge is equal to', Q4

C
C Calculate TOTAL DISCHARGE
C
      PRINT* ,'Q1 = ', Q1
      PRINT* ,'Q2 = ', Q2
      PRINT* ,'Q3 = ', Q3
      PRINT* ,'Q4 = ', Q4

      QT=Q1+Q2+Q3+Q4

      PRINT* ,'Therefore the total discharge is ', QT
C
C The conveyance for each zone needs to be calculated

C      KON1= Q1/(S**0.5)
C      PRINT*, 'The zone 1 conveyance is ', KON1
```

```
C       KON2= Q2/(FPS**0.5)
C       PRINT*, 'The zone 2 conveyance is ', KON2
C       KON3= Q3/(FPS**0.5)
C       PRINT*, 'The zone 3 conveyance is ', KON3
C       KON4= Q4/(FPS**0.5)
C       PRINT*, 'The zone 4 conveyance is ', KON4

C       KON= KON1+KON2+KON3+KON4
C       PRINT* ,'The James and Wark Method Conveyance = ', KON


C Calculate the James and Wark Method Conveyance
C
        IF(S.GT.0)THEN
       KON=QT/(S**0.5)
       PRINT* ,'The James and Wark Method Conveyance = ', KON
        END IF

          IF(HYT(IHT).GT.RBL)THEN
        CCHNL(IHT)=KON
          END IF

C       END IF

C Calculation of Beta parameter, should equal 1

          AT=ABL+A2+A3+A4
          PRINT* ,'AT is equal to ', AT

             IF(KON.GT.0.AND.AT.GT.0)THEN
          BETA=((AT)/(KON**2))*((KON**2)/(AT))
          PRINT*,'BETA is equal to ', BETA
             END IF

          IF(HYT(IHT).GT.RBL)THEN
             BETA(IHT)=BETA
          END IF

         END DO

C       PAUSE
C
C
        return

        END
```

Appendix 3 Channel Parameters

River Dane -Estimates of Channel Parameters				
Reach Averaging Method 1				
	SIN	L	S.S.	FPS
Sect1	1.8	250	2.89	0.0078
Sect2	1.8	250	2.89	0.0078
Sect3	1.8	250	2.89	0.0078
Sect4	1.8	250	2.89	0.0078
Sect5	1.8	250	2.89	0.0078
Sect6	1.8	250	2.89	0.0078
Sect7	1.8	250	2.89	0.0078
Sect8	1.8	250	2.89	0.0078
Sect9	1.8	250	2.89	0.0078
Sect10	1.8	250	2.89	0.0078
Sect11	1.8	250	2.89	0.0078
Sect12	1.8	250	2.89	0.0078
Sect13	1.8	250	2.89	0.0078
Sect14	1.8	250	2.89	0.0078
Sect15	1.8	250	2.89	0.0078
Sect16	1.8	250	2.89	0.0078
Sect17	1.8	250	2.89	0.0078
Sect18	1.8	250	2.89	0.0078
Sect19	1.8	250	2.89	0.0078
Sect20	1.8	250	2.89	0.0078
Sect21	1.8	250	2.89	0.0078
Sect22	1.8	250	2.89	0.0078
Sect23	1.8	250	2.89	0.0078
Sect24	1.8	250	2.89	0.0078
Sect25	1.8	250	2.89	0.0078
Sect26	1.8	250	2.89	0.0078
Sect27	1.8	250	2.89	0.0078
Sect28	1.8	250	2.89	0.0078
Sect29	1.8	250	2.89	0.0078
Sect30	1.8	250	2.89	0.0078

River Dane Estimate of Channel Parameters				
Reach Averaging Method 2				
	SIN	L	S.S.	FPS
Sect1	1.1	28	2.89	0.0078
Sect2	1.1	75.25	2.89	0.0078
Sect3	1.33	112	2.89	0.0078
Sect4	1.64	161	2.89	0.0078
Sect5	1.5	168.7	2.89	0.0078
Sect6	1.25	140	2.89	0.0078
Sect7	1.25	146	2.89	0.0078
Sect8	1.25	105	2.89	0.0078
Sect9	1.5	105	2.89	0.0078
Sect10	1.25	140	2.89	0.0078
Sect11	1.25	140	2.89	0.0078
Sect12	1.25	140	2.89	0.0078
Sect13	1.42	140	2.89	0.0078
Sect14	1.33	140	2.89	0.0078
Sect15	1.25	140	2.89	0.0078
Sect16	1.67	210	2.89	0.0078
Sect17	1.67	210	2.89	0.0078
Sect18	1.25	140	2.89	0.0078
Sect19	2.14	210	2.89	0.0078
Sect20	1.43	140	2.89	0.0078
Sect21	1.25	140	2.89	0.0078
Sect22	1.25	140	2.89	0.0078
Sect23	1.67	140	2.89	0.0078
Sect24	1.67	140	2.89	0.0078
Sect25	1.3	144	2.89	0.0078
Sect26	1.41	119	2.89	0.0078
Sect27	1.27	152	2.89	0.0078
Sect28	1.6	214	2.89	0.0078
Sect29	1.4	170	2.89	0.0078
Sect30	1.4	126	2.89	0.0078

River Kelvin - Estimates of Channel Parameters					
Reach Averaging Method 2					
	SIN	L	S.S.	FPS	
Sect 1	1.12	134.5	0.658	0.00935	
Sect 2	1.12	176.9	0.658	0.00605	
Sect 3	1.12	204.73	0.591	0.00172	
Sect 4	1.12	230.53	2.798	0.00225	
Sect 5	1.12	235.13	0.98319	0.00235	
Sect 6	1.12	231.43	2.417	0.00073	
Sect 7	1.12	222	1.3734	0.00334	
Sect 8	1.12	165.04	1.015	0.00341	
Sect 9	1.12	109	1.015	0.00119	
Sect 10	1.12	168.62	1.964	0.00069	
Sect 11	1.12	170.13	1.302	0.00283	
Sect 12	1.12	111.2	1	0.00285	
Sect 13	1.12	165.63	0.75515	0.00172	
Sect 14	1.12	212.7	0.745	0.00224	
Sect 15	1.12	221.7	1.9807	0.00312	
Sect 16	1.12	236.03	1.362	0.00146	
Sect 17	1.12	215.36	0.5036	0.00057	
Sect 18	1.12	202.77	2.206	0.00094	
Sect 19	1.12	222.63	1.509	0.0008	
Sect 20	1.12	229.38	1.432	0.00055	
Sect 21	1.12	227	0.6646	0.0005	
Sect 22	1.12	227.1	0.775	0.00045	
Sect 23	1.12	224.8	1.44	0.00113	
Sect 24	1.12	214.55	0.6722	0.00043	
Sect 25	1.12	234.64	1.518	0.00112	
Sect 26	1.12	238.6	1	0.00134	
Sect 27	1.12	218.97	0.65756	0.00091	
Sect 28	1.12	217.14	0.8044	0.00045	
Sect 29	1.12	219.15	1.194	0.00164	
Sect 30	1.12	240.8	0.7935	0.00605	
Sect 31	1.12	241.2	1.5686	0.0069	
Sect 32	1.12	166.03	1	0.00241	
Sect 33	1.12	103.53	1	0.00321	
Sect 34	1.12	166.34	0.964	0.00226	

River Kelvin - Estimates of Channel Parameters					
Reach Averaging Method 2					
	SIN	L	S.S.	FPS	
Sect 35	1.12	207.63	1.206	0.00146	
Sect 36	1.12	197.81	1	0.00155	
Sect 37	1.12	225.09	1.14	0.00342	
Sect 38	1.12	237.1	0.8319	0.00257	
Sect 39	1.12	231.21	1.408	0.00096	
Sect 40	1.12	268.97	0.9702	0.00113	
Sect 41	1.12	230.04	1.25	0.00197	
Sect 42	1.12	190.8	0.9278	0.0018	
Sect 43	1.12	222.81	1.069	0.00156	
Sect 44	1.12	214.82	0.798	0.00226	
Sect 45	1.12	221.92	0.5216	0.0014	
Sect 46	1.12	220.71	0.7611	0.00209	
Sect 47	1.12	206.29	1.25	0.00209	
Sect 48	1.12	217.14	1.13	0.00152	
Sect 49	1.12	220.17	0.77	0.00152	
Sect 50	1.15	218.04	1	0.00165	
Sect 51	1.15	230.96	1.42	0.00395	
Sect 52	1.15	218.78	1.54	0.00299	
Sect 53	1.15	218.17	1.703	0.00092	
Sect 54	1.15	336.83	1.358	0.00069	
Sect 55	1.15	342.26	0.9319	0.00075	
Sect 56	1.15	236.52	0.8939	0.00098	
Sect 57	1.15	210.7	0.7967	0.00062	
Sect 58	1.15	213.87	1.1	0.00111	
Sect 59	1.15	238.61	1.85	0.00223	
Sect 60	1.15	204.38	1.78	0.00182	
Sect 61	1.3	197.65	1.594	0.00077	
Sect 62	1.3	183	1.51	0.00055	
Sect 63	1.3	198.62	1.1	0.0038	
Sect 64	1.3	225.42	2.03	0.00394	
Sect 65	1.3	213.96	1	0.00497	
Sect 66	1.3	182.08	0.7763	0.00497	
Sect 67	1.3	162.08	1.44	0.00069	
Sect 68	1.3	191.3	1.28	0.00069	

River Kelvin - Estimates of Channel Parameters					
Reach Averaging Method 2					
	SIN	L	S.S.	FPS	
Sect 69	1.3	208.31	1.132	0.00069	
Sect 70	1.3	196.04	1.49	0.00402	
Sect 71	1.3	51.38	1	0.00069	
Sect 72	1.3	81.35	1	0.00172	
Sect 73	1.3	182.96	1.37	0.00184	
Sect 74	1.3	241.15	1.025	0.00081	
Sect 75	1.3	229.42	2.1435	0.00069	
Sect 76	1.3	175.65	1.384	0.00174	
Sect 77	1.3	167.27	1.245	0.00274	
Sect 78	1.3	191.96	2.23	0.00169	
Sect 79	1.3	191.92	1.32	0.00534	
Sect 80	1.3	195.54	0.6845	0.00534	
Sect 81	1.3	198.15	0.8979	0.00171	
Sect 82	1.3	178.15	1.0832	0.00171	
Sect 83	1.3	70.46	0.7144	0.00069	
Sect 84	1.3	112.27	0.7144	0.00069	
Sect 85	1.3	148.88	1.57	0.00069	
Sect 86	1.3	177.85	1.43	0.00069	
Sect 87	1.3	177.31	1.38	0.00069	

Appendix 4 Newton Raphson Method

Appendix 4 Newton Raphson Iteration Technique

A common method used by Engineers for solving non-linear equations is the Newton-Raphson Method. If it is assumed that x_0 is an approximation to the root $x = \alpha$ of the equation $f(x) = 0$ then a closer approximation will be given by the point $x = x_1$ where the tangent to the graph at $x = x_0$ cuts the x axis as show in Figure A4 below.

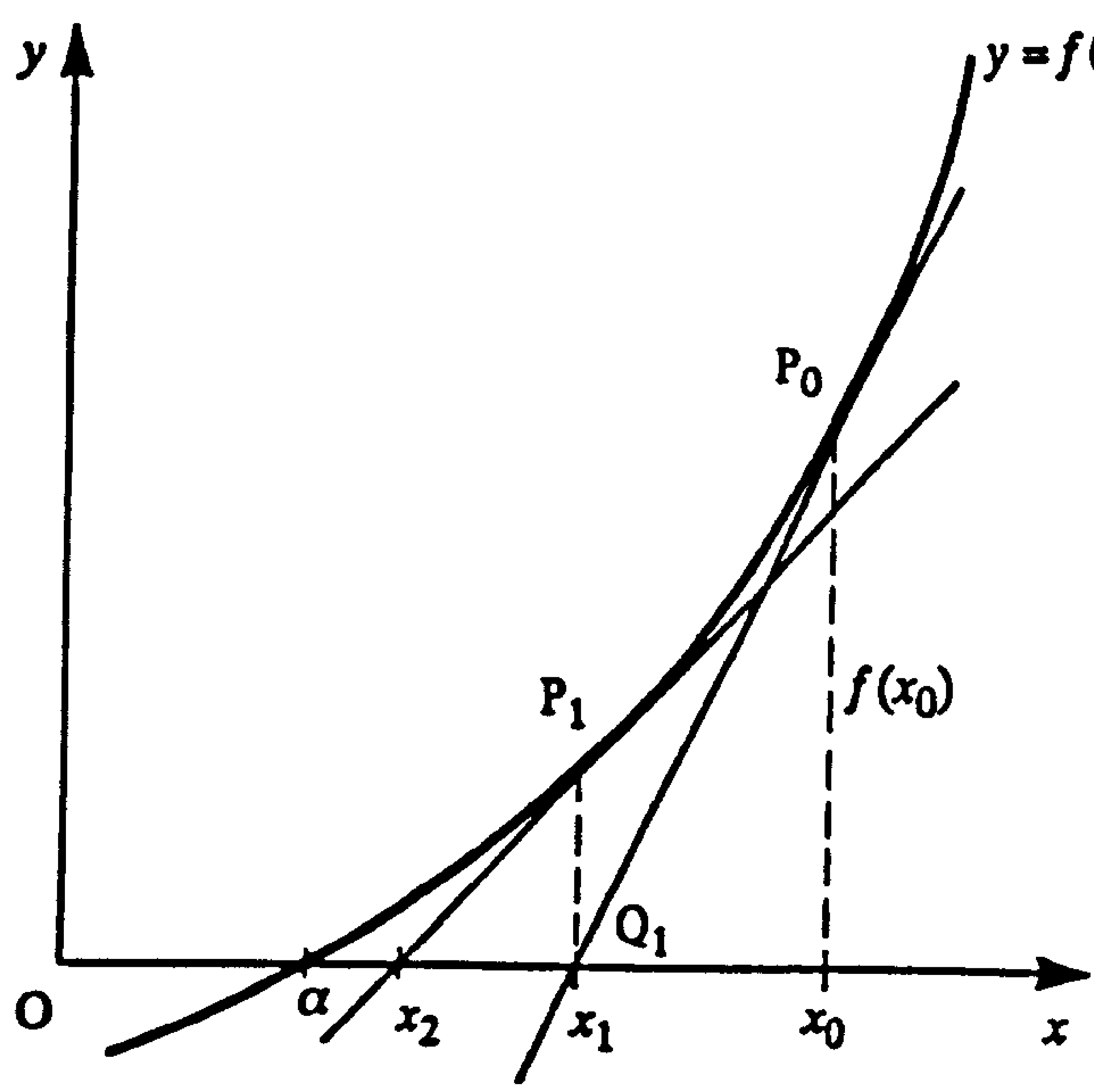


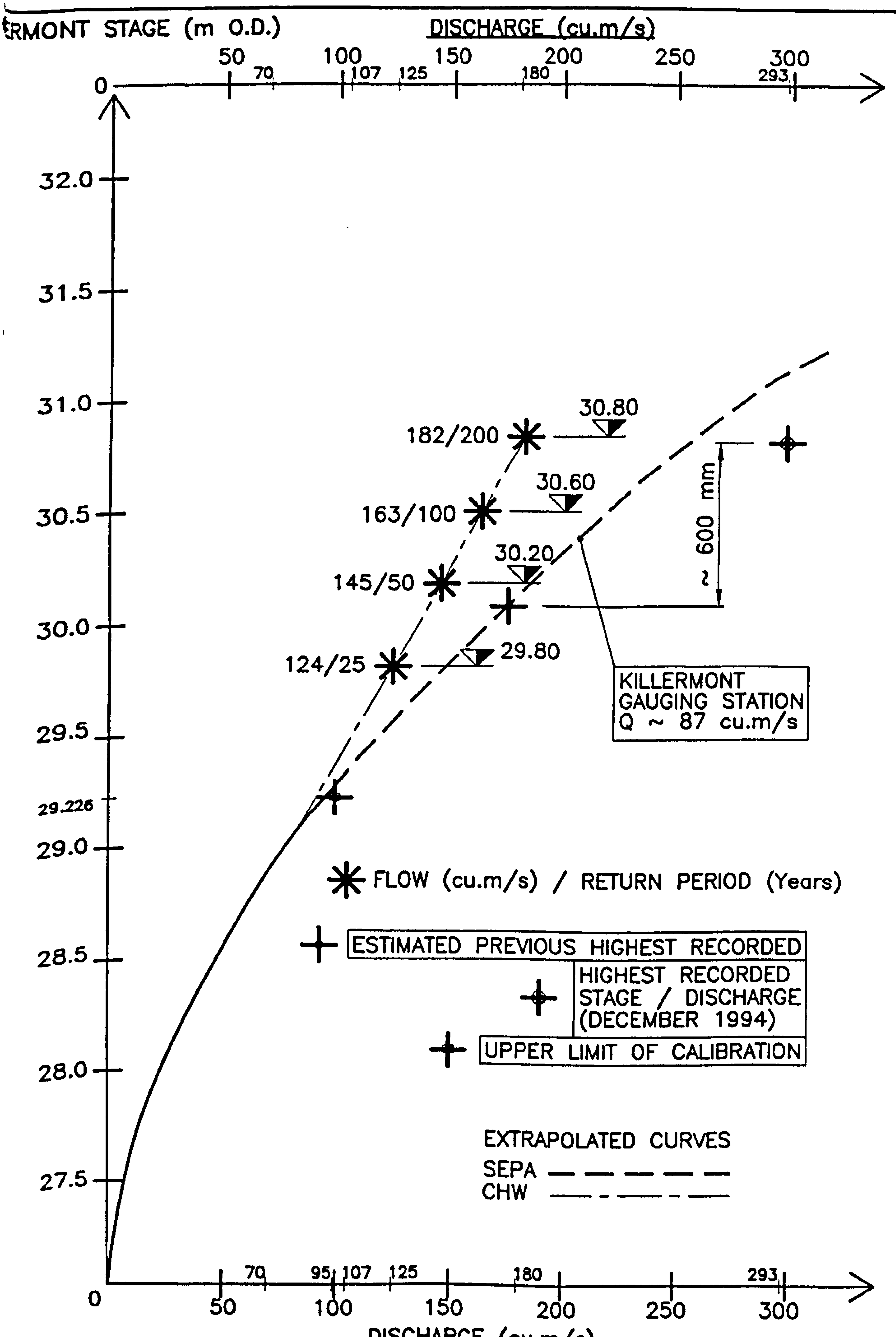
Figure A4 The Newton Raphson Root Finding Method

$f'(x_0)$ = slope of $P_0 Q_1 = \dfrac{f(x_0)}{x_0 - x_1}$ which can be rearranged to give

$$x_1 = x_0 - \frac{f(x_0)}{f'(x_0)}$$

By taking x_1 as the new approximation to the root $x = \alpha$ and repeating the procedure, as shown in Figure A4, a closer approximation is obtained. For examples of the Newton Raphson Method refer to James (1992).

Appendix 5 Stage Discharge Curves For The River Kelvin



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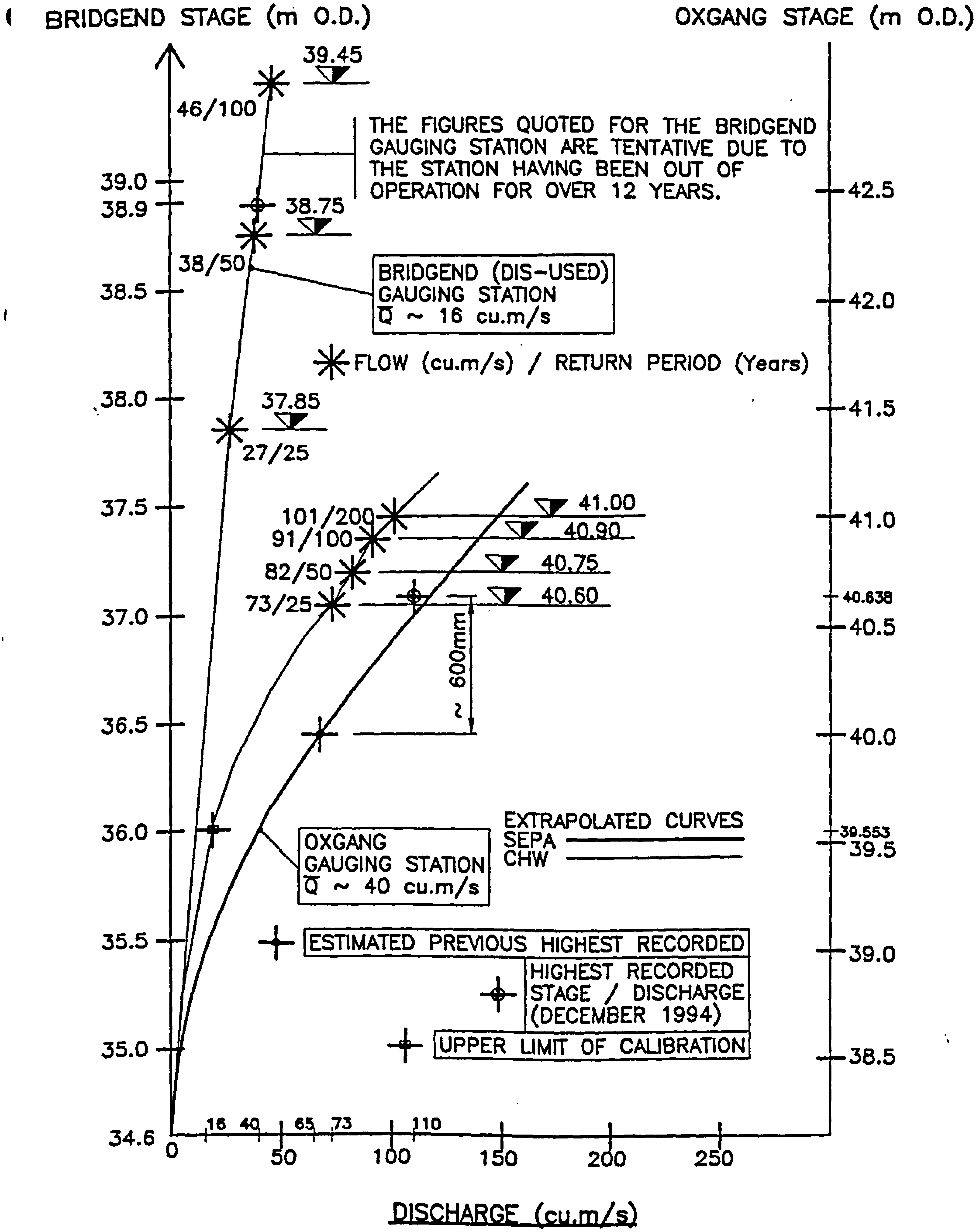

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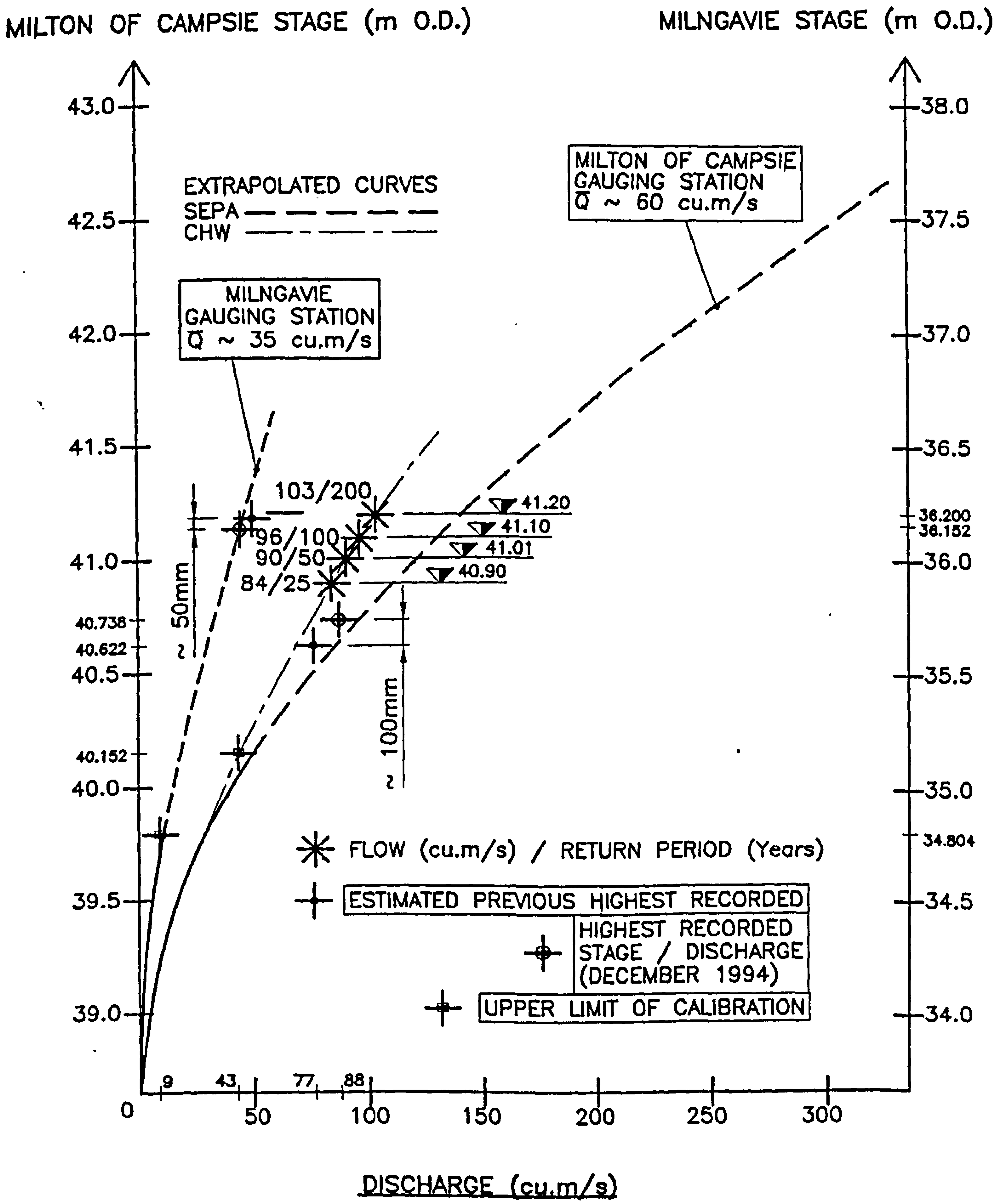

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REVISION	SCALE N/A	STAGE REPORT
PROJECT RIVER KELVIN CATCHMENT STUDY	DRAWING No. REPORT DRG.NO.2	

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Appendix 6 Published Work

THE APPLICATION OF ENHANCED CONVEYANCE CALCULATIONS IN FLOOD PREDICTION

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Abstract

Over the past twenty years extensive research has been conducted on overbank flow behaviour during river floods. When the main channel flow interacts with flood plain flow, secondary losses other than bed friction act to retard the flow. Traditional one-dimensional modelling tools commonly used in the UK, such as ISIS or HEC-RAS, currently take no account of these secondary losses

In an attempt to establish the nature and significance of secondary losses the Flood Channel Facility (FCF) was constructed at HR Wallingford in 1987. As a direct result of the meandering channel Series B experiments the James and Wark Method (1992) was developed to predict stage discharge relationships. For a given water level, this method will calculate a value of discharge taking into account the secondary losses. The paper will report on the modification of the method to fit into the river modelling software ISIS. Within the ISIS framework the James and Wark Method is used to calculate conveyance. The aim is to produce a more accurate flood prediction tool than currently exists.

The newly developed software has been tested on laboratory data and shown to be highly accurate in both stage discharge and water level prediction. The software has since been applied to natural rivers that have experienced significant flood events.

The paper will illustrate the significance of applying flume based conveyance calculation methods at the field scale.

Keywords
Meandering Channels, Overbank Flow, ISIS

Introduction

The main flood analysis tool of the river engineer is the one-dimensional river model. By using this tool flood levels and the extent of flooding can be established for various flow return periods. Predictions from these models are used as the basis for designing flood protection works and consequently need to be as robust and accurate as practically feasible. Recent research by Ervine et al (1993) has reported on the existence of secondary energy loss mechanisms associated with main channel and flood plain flows. Existing industry standard river models currently make no account of these new findings which if included will result in an increase in predicted flood levels. Obviously this has significant implications for design and this paper attempts to both incorporate these new findings into an industry standard model and report on the significance of the results.

Code Development and The James And Wark Method Subroutine

The James and Wark Method (1992) is a hand calculation that calculates stage discharge relationships for meandering compound channels and directly accounts for energy losses associated with over bank flow. This has been modified to calculate conveyance and been incorporated into the one-dimensional model ISIS. This was done by the addition of a new subroutine to the ISIS source code and a new data entry system. It is normal for a river modeller using ISIS or a similar package to enter surveyed cross-sectional data, however, the new software also requires estimates of sinuosity, side slope, meander wavelength and flood plain slope. It is also necessary to identify the horizontal extent of both the main channel and the meander belt width. This is done by the addition of a '*' in the data file where required. These are important markers as they also define the limits of the various flow zones (See Figure 1) that are used in the James and Wark Method. The terms 'p' and 's' refer to the left and right river bank boundaries and 'm' and 'n' to the extents of the meander belt width, which is the plan area within which the meandering main channel is contained.

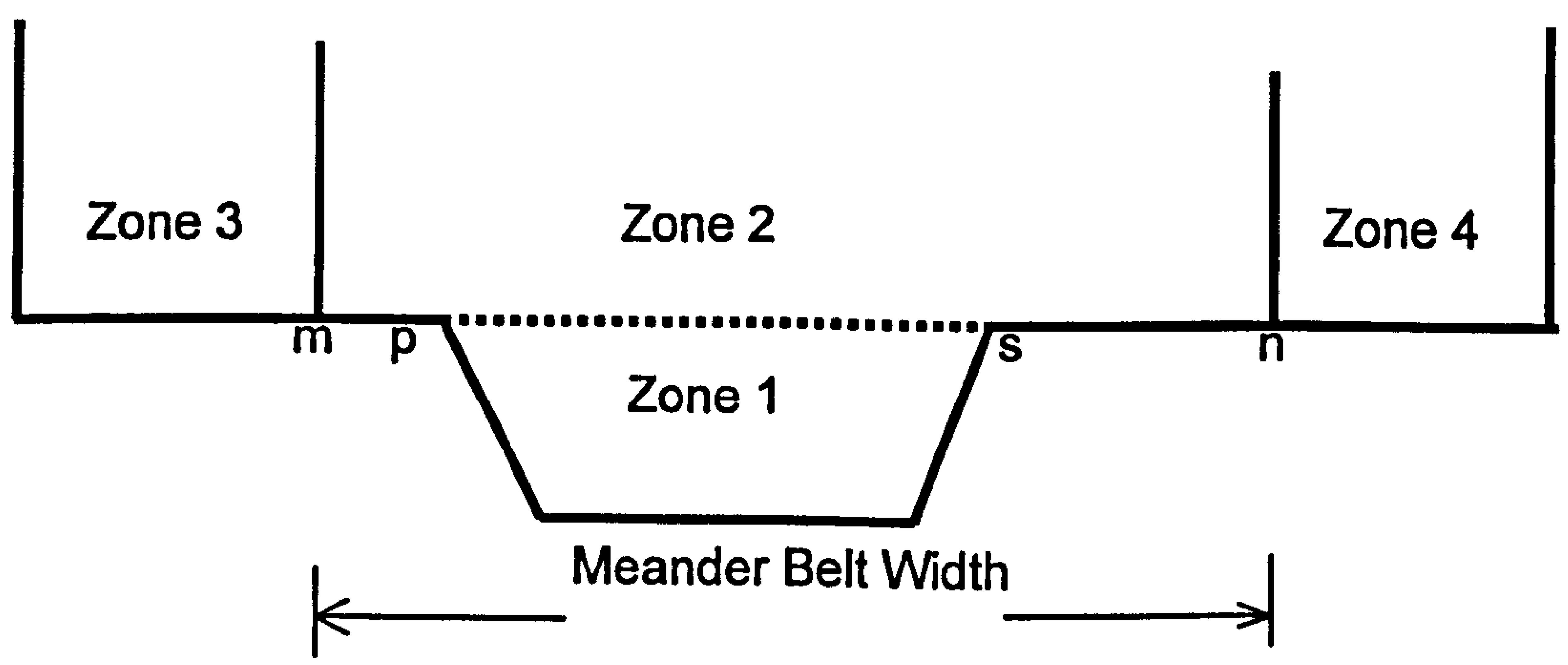

Fig. 1: Definition of Flow Zones For The James & Wark Method

Once the program has read in the additional data and calculated values of area etc. it proceeds to calculate a bank-full discharge. This being obtained by the multiplication of area and mean velocity (V) where the value of Manning's 'n' is adjusted to account for meander losses. This is achieved by use of the Linearised Soil Conservation Method (LSCS). From this a bank-full discharge is obtained which accounts for some of the effects of flow interaction.

The Zone 1 discharge is calculated by multiplying the bankfull discharge by an adjustment factor. This adjustment factor is calculated by two methods and the larger of the two values is selected. Zone 2 is defined as the region above bankfull but within the horizontal extent of the meander belt width. The discharge in this Zone is also calculated by the multiplication of flow area (above bankfull only) and the mean flow velocity (V2). V2 includes empirical terms to account for the expansion and contraction of flow over the main channel. The discharges in Zones 3 & 4 are obtained conventionally with bed friction being assumed as the only source of energy loss. Having calculated the discharges in all four zones, they are finally summed to give a total discharge. The final calculation in this subroutine is to obtain a value of conveyance. To do this, the total discharge is divided by the square root of the longitudinal main channel slope. (See Equation 1)

$$K = \frac{Q}{S^{1/2}} \tag{1}$$

where K is the conveyance, Q the total discharge and S the main channel slope.

Once the subroutine completes its final computation the results are stored in the appropriate array for future use by the hydrodynamic calculations. (The term Beta is also calculated but shall not be considered in this paper) On the completion of this the calculation moves to the next water level and begins again. This is repeated until all defined water levels, including a default vertical wall of 3m, have a corresponding value of conveyance. Further information regarding the James and Wark method can be found in James and Wark (1992), Wark et al (1994) and Forbes (2000).

Flood Channel Facility Series B Testing – Introduction

The aim of simulating these experiments is to verify the new computer code and assess the accuracy with which the James and Wark Method, as implemented in ISIS, can replicate the observed experimental measurements. An improvement on the conventional Divided Channel Method accuracy would certainly be expected. It should be remembered that many model users currently apply the Divided Channel Method which is based purely on bed friction. This method has been shown to be in error by 30% in some applications. (Wark et al (1994)) For testing purposes it was decided to use the quasi-natural geometry, which was derived by Lorena (1992). Figure 2 depicts the classical apex geometry and clearly shows the deeper section normally found on the outer side of a bend. In the region between the bends the cross-sectional geometry changes linearly from quasi-natural to trapezoidal and then back to quasi-natural at the next bend apex.

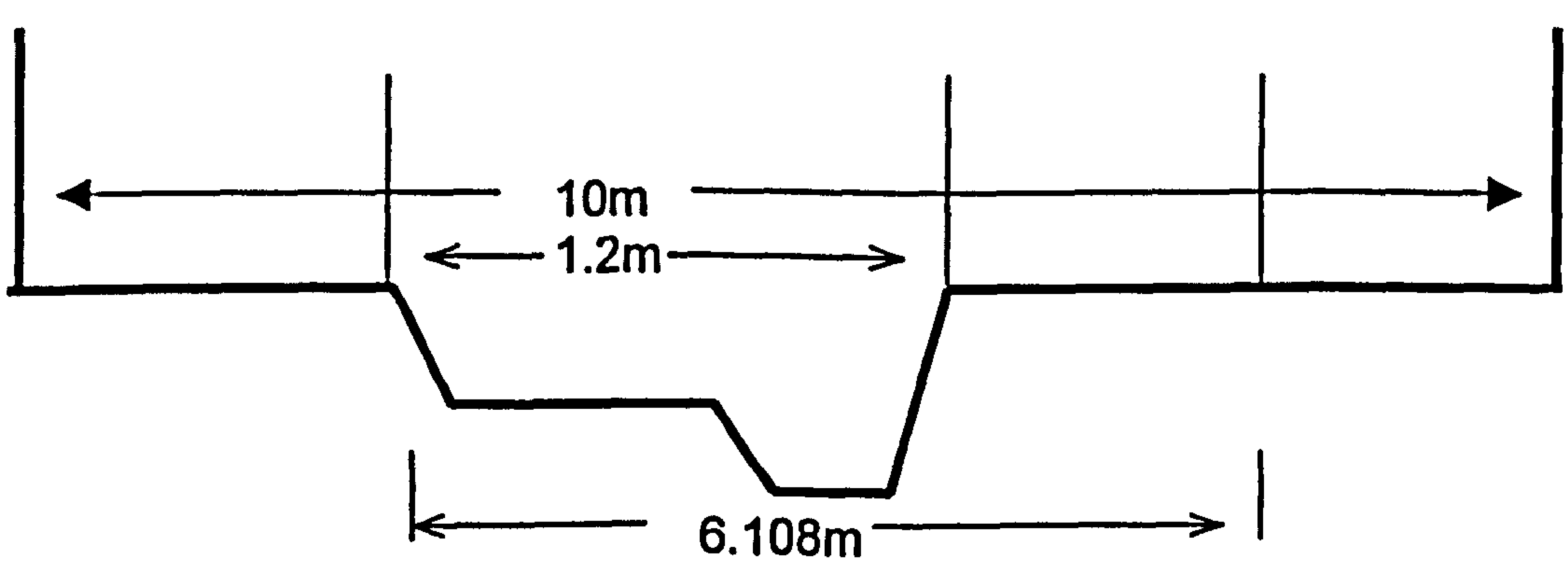

Fig. 2: FCF Quasi-Natural Apex Section Geometry 60 Degree Meander

Experiment B26 Stage Discharge Prediction

The FCF B26 experiment was selected to test the James and Wark Method's ability to reproduce the observed stage discharge relationship. The experiment involved a quasi-natural main channel with smooth flood plains. It was shown by Lorena (1992) that more secondary energy losses would be present with smooth flood plains hence the choice of this application. A numerical model with the geometry shown in Figure 2 was set-up. Based on calibration data from the FCF experiment both the main channel and flood plain Manning's 'n' values were taken as 0.01. If rough flood plains were initially chosen the applicability of using the James and Wark Method would have been limited. Figure 3 compares the stage discharge relationship obtained using the new software, the conventional Divided Channel Method and the experimental observations. It can be seen that the numerical James and Wark scheme over-predicts the observations by 2%. This level of agreement is considered reasonable. As expected the Divided Channel Method consistently over-predicts discharge by around 15%. This reinforces the need for more accurate methods for calculating stage discharge relationships.

Interestingly, James and Wark (1992) also used this experiment as a test case for their hand calculation procedure and concluded that their method would under predict the observed values by-2.7%. The difference between +2% and -2.7% is surprising as this stage discharge component is simply a computerised version of the James and Wark hand calculation. Despite extensive testing and a series of hand calculations following the procedure it has not been possible to reproduce the -2.7% error quoted in James and Wark (1992). It has therefore been concluded that the +2% over prediction is correct and that this degree of accuracy is acceptable in terms of practical river modelling.

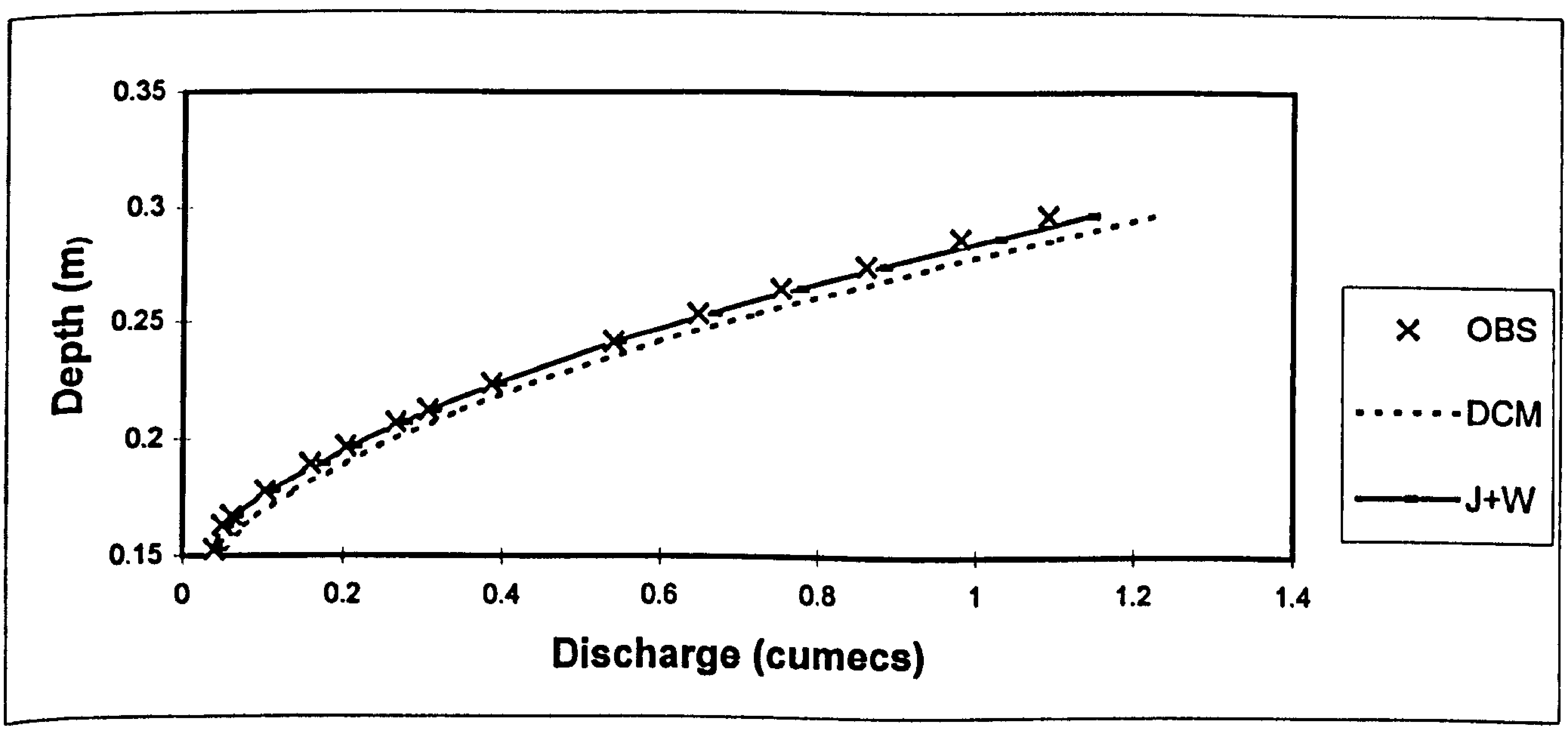

Fig. 3: Stage Discharge Curves For Experiment B26

Experiment B26 Water Level Prediction

Figure 4 shows comparisons of computed and measured water levels at the upstream numerical model section, for all of the discharges used in the experimental programme. The required 'additional parameters' were exactly known for the FCF. There is an almost perfect match over the majority of the depth range, with a slight under prediction in water level at depths above 0.274m. This is expected as the stage-discharge relationship is over-predicting at these depths. When this flow over prediction is converted to conveyance using Equation 1 a conveyance over prediction results, consequently a lower than observed water level results. On average the James and Wark Method under predicts the observed water level by 2mm. The Divided Channel Method under predicts the observed water level by, on average, 8mm. This result clearly shows the improvement that can be obtained when using the James and Wark Method to calculate conveyance. The higher water level prediction is due to additional energy losses associated with overbank flow being correctly accounted for. The general trend of the J+W method in Figure 4 is considered to be acceptable.

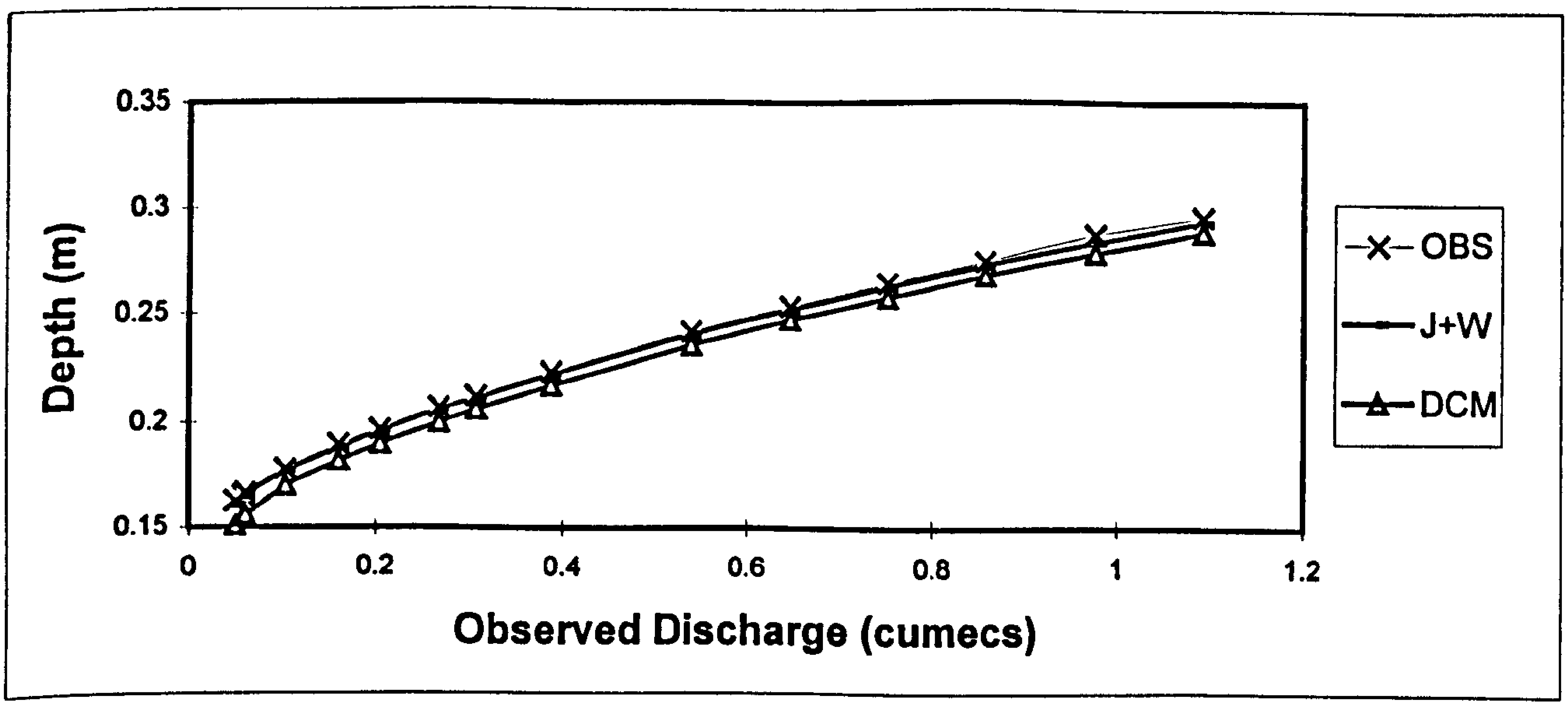

Figure 4: Observed and Predicted Water Surface Profile

Practical ISIS Modelling of a UK River

Data from a typical UK river has been obtained and used to further test the new software. The study reach is 5km long and highly meandering. The aim of this test is to observe the significance of the new software in terms of flood level prediction and to assess the sensitivity of the additionally required parameters to errors in their estimation. Comparisons will also be made with the existing ISIS Divided Channel Method.

An ISIS model was constructed using 30 surveyed cross-sections, a value of peak inflow of 170m^3/s at the upstream boundary and a known water level of 13.5m (AOD) at the downstream boundary. A value of Manning's 'n' has been estimated after reference to a series of photographs, of the River, and to Chow (1959). A value of 0.048 was estimated from Chow (1959) and confirmed by an earlier study, on the reach, by Ervine and Mcleod (1999). To enable the correct running of the James and Wark Method certain parameters are required in addition to that required for the Divided Channel Method. These parameters are estimates of sinuosity, meander wavelength, side slope and flood plain slope. In a natural river with extensive longitudinal variation, these can be difficult to assess but with reasonable judgement an acceptable value can be obtained.

Wark (1998) suggested that a reach representative cross-section was required for the correct working of this method and that for every bend encountered that a number of cross-sections were surveyed. However, this is not practically viable for most river modelling projects due to the cost associated with collecting the survey data. As a result, only the additional parameters have been averaged and used together with the surveyed cross-sections. The representative reach was assumed to be the entire study length of the River. This may be considered a valid assumption, in this case, as the sinuosity and meander belt width of the study reach are approximately constant over the whole 5km. It was decided that the 'additional parameters' would be calculated for the entire reach and used together with the 30 surveyed cross-sections. Effectively this meant that the additional parameters would be the same at each river cross-section. This was similar to the methodology of Ervine and MacLeod (1999).

The sinuosity parameter was calculated over the 5km reach by dividing the total centre line distance by an approximate straight-line distance. This resulted in a sinuosity estimate of 1.8 which is very high. The meander wavelength was estimated from a 1:10000 plan map of the location and is defined as the centre line distance divided by the number of wavelengths in the reach length. In this case the total centre-line distance is 5000m and the number of wavelengths 20. This resulted in a meander wavelength of 250m. The side slope was simply measured from the upper two-thirds of the river-banks. As the banks are generally irregular and the actual slopes vary, straight lines are fitted to the upper two-thirds of the bank profiles. The average of the right and left bank is taken and is expressed as the ratio of horizontal to vertical distances. The value of flood plain slope was assumed to be constant along the reach length and was taken from the cross-sectional information.

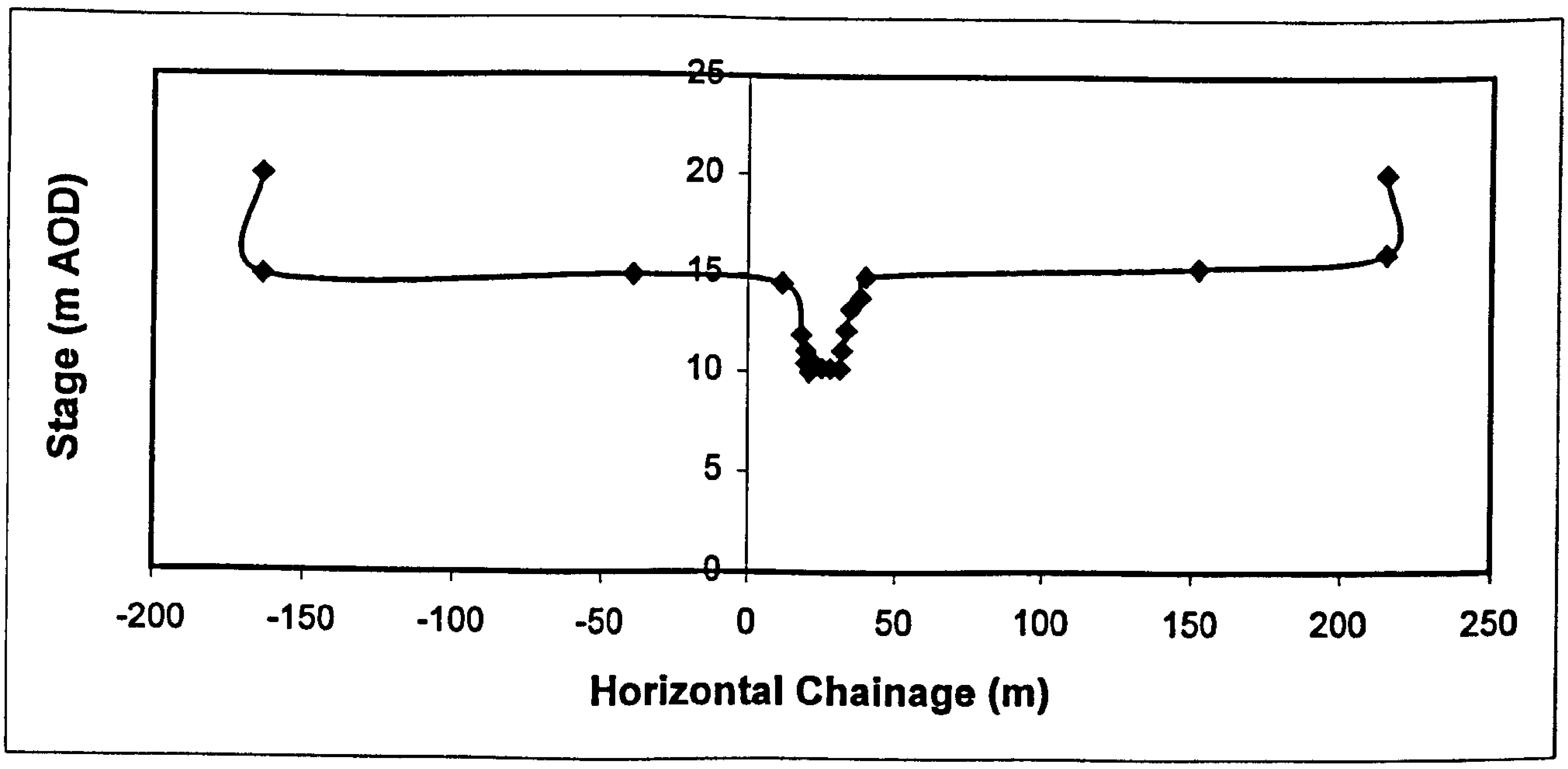

Figure 5: Typical ISIS Model River Cross-Section 16

This method of obtaining the 'additional parameters' has been selected as being a reasonably straight forward and rapid method. A possible flaw is that in places the value of sinuosity and meander wavelength is not realistic. For example, at cross-sections 1 and 2 the sinuosity is low and would not agree with the specified value of 1.8. It is also difficult in a naturally varying river to define reaches of significant longitudinal length that are geometrically similar. Having obtained the 'additional parameters' a steady analysis can be ran. From this analysis it is intended to observe the significance of the new conveyance calculation in terms of water level prediction.

Figure 6 shows the difference in water level prediction that can be expected when the James and Wark conveyance method is used rather than the bed friction only method. As can be seen the predicted water levels rise at the majority of cross-section locations with the maximum being +180mm at cross-section 5. The only significant discrepency is at cross-sections 6-10 which are experiencing significant flood plain depth where the James and Wark conveyance method will perform poorly.

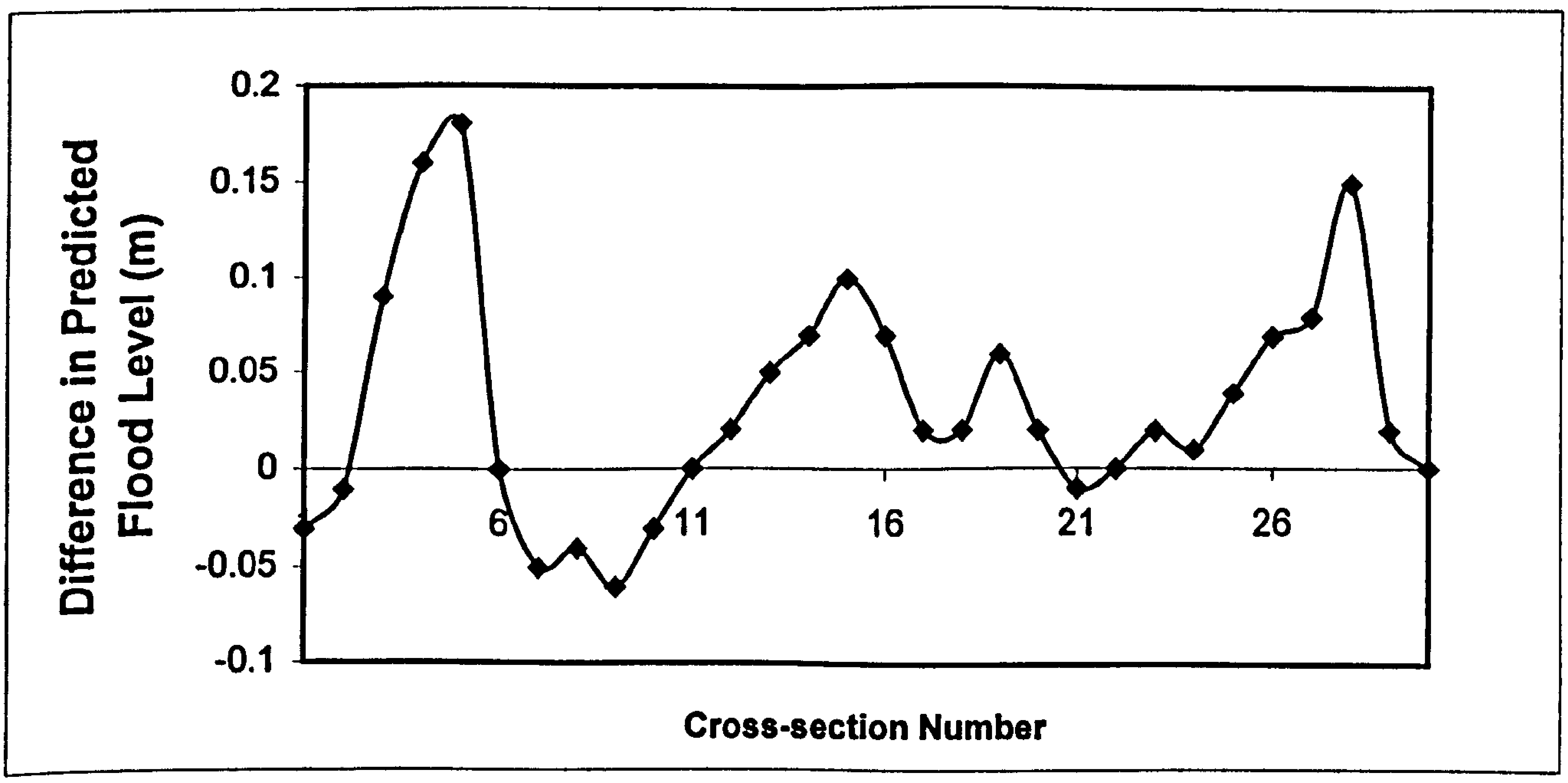

Figure 6: Difference in Using The J+W Conveyance method rather than DCM

Sensitivity Analysis – Steady State Modelling

It is not known what effect errors in the 'additional parameters' may induce in a field application. During the Flood Channel Facility tests these parameters were exactly known. This is not the case in a field study. The following tests are intended to provide information on the required accuracy or sensitivity of the additional parameters and their consequent effect on water level predictions.

Sensitivity of Water Level Predictions to Estimate of Sinuosity

In theory, if the sinuosity increases then an increase in secondary energy losses would be expected, Ervine et al (1993). This would in turn lead to an increase in predicted water levels. In reality, if the sinuosity were to change then other parameters such as belt width, distance down stream and slope would all change. The aim of this test is to ascertain how accurate the estimate of channel sinuosity needs to be and to provide guidance to the practising Engineer concerning the limits of acceptable accuracy. However, it should be noted that the following test is simply altering one parameter at a time, which is really a test of accuracy of data. The study reach was defined as having a sinuosity of 1.8 and is considered to be accurate if the whole reach length is being considered. Figure 7 illustrates the difference in water level predictions by using sinuosities of 1.5, 1.8 and 2.1. Interestingly, the results show that, when the sinuosity in changed independently, the water levels reduce with increasing sinuosity. This is a direct result of the independent alterations of the sinuosity parameter and the 'make-up' of the James and Wark Method equations. The maximum difference in predicted water levels of -0.08m occurred when the sinuosity was increased to 2.1.

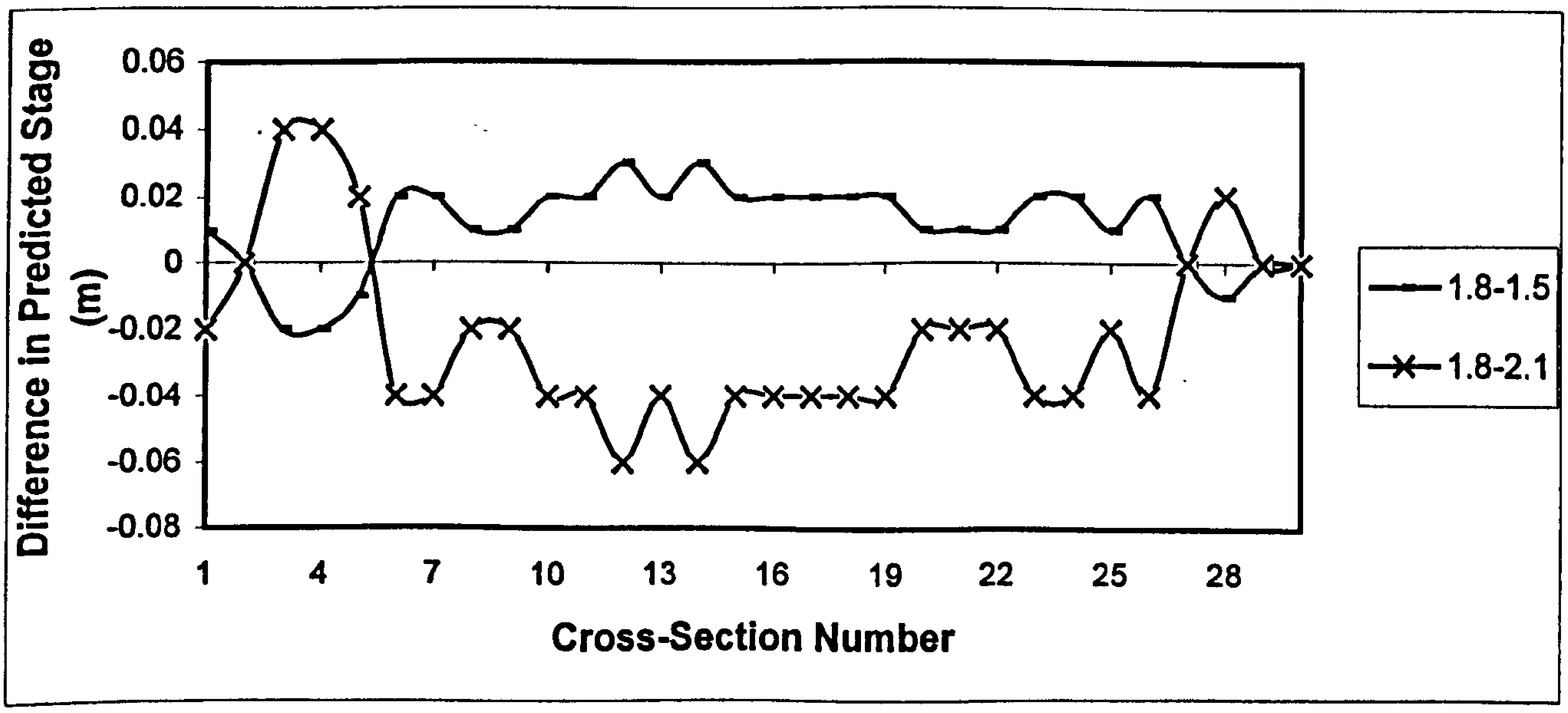

Figure 7: Graph of Differences predicted When Using Different Values of Sinuosity

As can be seen from Figure 7 when the sinuosity is either increased or decreased by 0.3 a similar pattern in water level prediction is observed. There are slight differences at some cross-sections. For example, at Cross-section 3, when the sinuosity is increased from 1.8-2.1 the water level increases by 0.04m but when the sinuosity is reduced from 1.8-1.5 the water levels are reduced by only 0.02m. This may be due to the relatively straight sections that are located at the upstream end of the model.

The general pattern shows that when the sinuosity is increased the predicted water levels will reduce and when the sinuosity is reduced the predicted water levels will rise. An error of 15-20% in sinuosity will not have a significant effect on predicted water levels.

Sensitivity of Water Level Predictions to Estimate of Meander Belt Width

The meander belt width term requires estimation and could easily be incorrectly measured. As a result, it was decided to test an error in belt width of ± 30m which would be the maximum conceivable error that could be practically envisaged. The measured meander belt width of this river was approximately 200m. The results of this test are shown in Figure 8 and the maximum difference is 0.05m at cross-section 5 when the meander belt width is increased by 30m. Figure 8 shows the predicted water levels for the various belt width estimates and clearly this parameter has no significant effect when changed independently. It appears that when the meander belt width is reduced then the predicted water levels rise marginally.

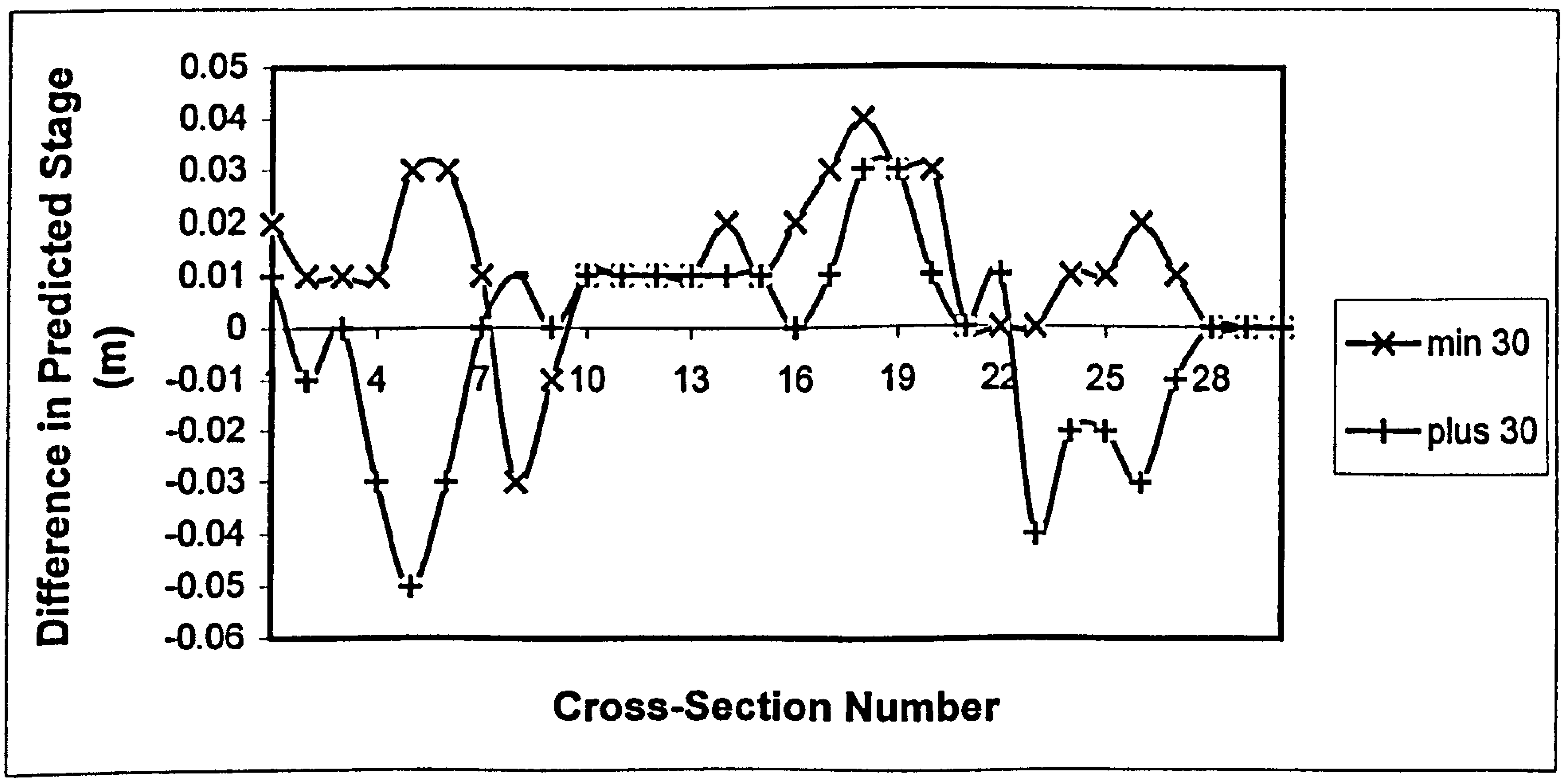

Figure 8: Differences in Water Level For a Range of Belt Widths

Again the important finding here being that a significant error in the meander belt width will not result in significant errors in flood level prediction. Further sensitivity tests and a more detailed discussion of the above tests can be found in Forbes (2000).

Discussion

It should be noted that the Manning's 'n' of 0.01 used in the Flood Channel Facility tests has since been found to be inaccurate despite it also being used by Crowder et al in their benchmarking study of 1997. Recent work has used a value of 0.0105 based on information from Lorena (1992) which, if used, would improve the accuracy of the FCF results presented here. An increase in roughness would act to further retard the flow and may result in better agreement with the James and Wark (1992) tests (–2.7%). The field study reported was complicated by the requirement of the 'additional parameters' which are not always simple to calculate where there is extensive longitudinal variation. A practical and simple approach has been adopted for this study which could be replicated on other river reaches. The newly developed software predicts higher flood levels than the more conventional method which is based solely on bed friction, when using the same value of Manning's 'n'. This will allow more representative values of 'n' to be used in calibration and avoid the use of 'lumped' energy loss coefficients. The prediction of higher water levels than current industry standard methods, when using steady state conditions, by as much as 180mm should be of interest and concern to the practising Engineer. The sensitivity tests performed indicated that a high degree of accuracy is not required in estimating the additional parameters. Further testing and application is required as the James and Wark method has limitations, mainly due to the limited geometries on which it is based, as described by Lambert and Sellin (1996). This may limit accuracy, but the existing results show that it is an improvement on conventional bed friction only conveyance techniques. Forbes (2000) reports on the extension of this application to other UK rivers for both steady and unsteady flows and the practicalities of doing so.

Conclusions

1. An enhanced flood prediction tool that accounts for energy losses associated with overbank flow has been presented.
2. The James and Wark conveyance method has been shown to predict higher flood levels, in a natural river application, than the existing bed friction only methods by as much as +180mm
3. An error in the sinuosity term of 15-20% will not have a significant affect on predicted flood levels
4. An error in the meander belt width of ± 30m will change predicted flood levels by a maximum of 0.05m

Acknowledgements

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