Flow structures in a compound meandering channel with flat and natural bedforms

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Flow structures
in a compound meandering channel
with flat and natural bedforms

by

Jake Spooner

A Doctoral Thesis

Submitted in partial fulfilment of the requirements for the award of
Doctor of Philosophy of Loughborough University

September 2001
Abstract

Keywords: Experiments, Laser Doppler Anemometer, Meandering channel, Flooding, Velocity, Turbulence, Stage-discharge, Sediment, Flat bed, Natural bedforms

Detailed experiments were conducted on a meandering compound channel, with a sinuosity of 1.384, in a 13m long 2.4m wide flume. Two cases were examined, where the main channel contained flat and natural bedforms. Measurements recorded include stage-discharge, sediment transport and bed shear stress. A three-component Laser Doppler Anemometer measured the velocity and turbulence in the flow and the bedform was measured using Digital Photogrammetry.

It was found from the stage-discharge data that at most depths the effect of the bedforms is to reduce the discharge capacity of the channel. The maximum reduction in the discharge capacity was at the bankfull flow depth where the discharge was reduced by thirty percent. The sediment transport rate was found to decrease at relative overbank flow depths of 0.2-0.3. The velocity and turbulence measurements were used to examine the flow structure. It was found that the formation of bedforms in the main channel significantly affects the flow structure of the flow in the main channel, although the flow on the floodplain is similar. Significant secondary flow circulations were found in the natural bed case, particularly at higher flow depths. The secondary circulations are caused by centrifugal force, flow entering the main channel from the floodplain and reverse flows as the flow passes over ridges in the natural bed case.

A new method for predicting velocity and discharge in meandering channels has been introduced based on the two-dimensional curvilinear equations for streamwise motion. The turbulence terms were found to be insignificant and the method was applied to data sets at different scales.
Acknowledgements

I would like to offer my gratitude to all those friends and colleagues who have helped me during the course of my research.

Particular thanks must go to Prof. Koji Shiono, my supervisor, for his assistance, encouragement and guidance.

Thanks must also go to the other members of the department without whom the work would not have been possible, namely (in no particular order); Dr P. Rameshwaran, T. Chan, R. Siqueira, Dr T. Feng and Dr J. Chandler.

I am also grateful to the technical staff within the Department of Civil and Building Engineering for their expertise in the construction and maintenance of the facilities, particularly M. Barker and D. Sanham.

I would also like to thank my friends. In particular those who have been with me through my time in Loughborough; from Bill Mo - Keith, Dave and Cazza, Jon (who has been here as long as I have!) as well as Ian and Sarah.

Finally, thank you to my parents for their support and encouragement.
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<th>Symbol</th>
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</tr>
<tr>
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<td>Bankfull channel width</td>
</tr>
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<td>$Ch1$</td>
<td>Measured velocities</td>
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<td>Boundary layer depth</td>
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<td>Discharge adjustment factor</td>
</tr>
<tr>
<td>$DISDEF$</td>
<td>Discharge deficit function</td>
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<tr>
<td>$f$</td>
<td>Darcy-Weisbach friction factor, where $f = \left(\frac{8\tau_w}{\rho U_a^2}\right)$</td>
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<tr>
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<td>Non-dimensional discharge function</td>
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<td>Grams</td>
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<tr>
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<tr>
<td>$U^*$</td>
<td>Shear velocity</td>
</tr>
<tr>
<td>$\bar{U}$</td>
<td>Mean longitudinal velocity</td>
</tr>
<tr>
<td>$u'$</td>
<td>Longitudinal turbulence</td>
</tr>
<tr>
<td>$V$</td>
<td>Transverse velocity</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Sectional mean transverse velocity</td>
</tr>
</tbody>
</table>
List of symbols

\( \bar{V} \)  Mean transverse velocity
\( v' \)  Transverse turbulence
\( W \)  Vertical velocity
\( W_s \)  Sectional mean vertical velocity
\( \bar{W} \)  Mean vertical velocity
\( w' \)  Vertical turbulence
\( x \)  Longitudinal distance
\( x_{ld} \)  Distance from leading edge of boundary layer
\( x_h \)  Longitudinal distance in the horizontal plane
\( x^* \)  Bed shear stress coefficient
\( y \)  Transverse distance
\( y_{in} \)  Transverse distance from inside on channel bend
\( y^* \)  Bed shear stress coefficient
\( z \)  Vertical distance

\( \delta_b \)  Boundary layer thickness
\( \lambda \)  Wavelength / dimensionless eddy viscosity
\( \varepsilon_{xy} \)  Eddy viscosity
\( \mu \)  Fluid viscosity
\( \nu \)  Kinematic viscosity
\( \rho \)  Water density
\( \tau_{b*} \)  Bed shear stress
\( \tau \)  Reynolds shear stress

\( u'v' \)  Reynolds shear stresses
\( v'w' \)
\( u'w' \)

\( \theta \)  Meander arc angle / angle of skew
\( \phi \)  Angle of flow deviation
1 Introduction

Rivers are a natural feature of our landscape and form an integral part of the water cycle. As such they are at the mercy of the prevailing weather conditions. Under normal conditions flow in a river remains within the main channel. Occasionally they become inundated with excessive volumes of water and pass out onto the surrounding areas. Normally this is a harmless event as natural channels have a floodplain designed to cope with this increase in flow. However, flooding is classified as a natural disaster, and extreme events can be catastrophic.

In recent years the cost of flood damage has started to escalate. This is due to a variety of reasons; population increases, migration and economic expansion have meant the pressure for space and land has increased. In the UK there are currently 1.8 million homes in flood risk areas and new developments are encroaching further onto floodplain areas. These new developments are likely to increase the chances of flooding, as well as increase the consequences of such an event with property and lives at risk. In November 2000 the UK suffered severe flooding; two people died, 7406 properties were flooded, and the repair bill was estimated to be around £500 million (Environment Action Floods Special, December 2000).

Berz (2000) analysed all the natural disasters that occurred between 1988 and 1997. He found that nearly a third were floods, causing 58 percent of all the deaths (226,000) and accounted for a third of the overall economic loss (over US$ 233 billion). Mozambique suffered some of the worst flooding in recent times during 2000. An estimated 700 people died, 490 000 people were displaced and the infrastructure of the country was devastated (Mozambique News Agency, May 20001). Although it is unlikely that any river management or flood protection strategy could prevent extreme flooding as in Mozambique, most flood events can be managed safely or the consequences minimised. As the potential economic and social costs rise, the need for more reliable flood protection measures can only increase.

In order to instigate reliable flood protection measures we have to understand rivers and their flow processes. However, all the characteristics that make each river unique also make them difficult to understand. Rivers and floodplains form a very complex

1 http://www.poptel.org.uk/mozambique-news/newsletter/aim184.html
system. The simple solution in flood schemes used to be single straight channels, which are the most efficient channels at discharging water. However, it is now accepted that as well as being unaesthetic they also provide an unsatisfactory variation of natural habitats. For these reasons engineers have moved towards designing more natural channels, i.e. ones which tend to meander. A more detailed awareness of the complex flow patterns, and the channel characteristics which directly influence the flow mechanisms has generally led to improved accuracy in more recent stage-discharge prediction methods. As well as enabling more efficient channel designs, better prediction methods also enable the design and construction of more effective flood defences and improved flood prediction warnings. Although this may not necessarily prevent flooding, it shall help to minimise any consequences thereof.

There seems to be two, almost distinct, research groups working within rivers and open channel flow; those who study physical models in laboratories and those that work on real rivers. It is widely acknowledged that laboratory experiments are a long way removed from the variety of flow situations and characteristics found in real rivers. Work on real rivers normally concerns flood prediction and flood prevention, or modification to the path of the river. The use of computer modelling is now a standard procedure for these works. However, such models cannot be built without an understanding of flow characteristics or used without prior validation. River engineers themselves also require an understanding of the problems to apply the models. Taking the necessary readings, particularly during flooding (which is one the main interests for river engineers today), to gain such understanding is vast, expensive and dangerous in real rivers. This is where laboratory research has proved its worth. Whether research involves a simple uniform channel or a model of a real situation, controlled laboratory conditions can provide accurate and useful data in a safe environment. These data sets can then be used to understand the basic fundamental flow mechanisms and applied to models and real conditions.

One of the main focuses of flood research in the UK has been the Flood Channel Facility (FCF), at HR Wallingford; built in 1986 the FCF is a large-scale facility (60m long and 10m wide). A 3-phase programme has been conducted examining straight, skewed and meandering channels. This has been supplemented by further research conducted in smaller-scale facilities in universities. The relatively recent introduction of laser systems into hydraulic research has revolutionised our in-depth knowledge of open channel flow. Laser Doppler Anemometer (LDA) systems are able to measure
instantaneous velocities within a body of fluid without disturbing the flow. This means it is now possible to take highly detailed and more accurate readings than ever before. Unsurprisingly, this technology has been embraced by the different institutions studying flow, particularly universities. As a result there is now a mass of information and papers available about different kinds of flow.

To date, research in flooding has progressed from straight compound channels to skewed and meandering compound channels. The skewed and meandering channels have more complex flow patterns, with greater energy losses, and this makes the prediction of the stage-discharge increasingly complex. These greater energy losses mean that traditional straight channel prediction methods greatly under-predict stages for skewed and meandering channels. Consequently alternative prediction methods are needed for skew and meandering channels. Much of the research to date has involved rectangular, trapezoidal or naturally shaped cross-sectional main channels in straight or meandering channels. These tend to simplify the channel flow by neglecting sediment transport and bedform within the channel. Channels with a mobile bed and natural bedforms are expected to have even more complicated flow mechanisms with even greater energy losses. Alternative methods or modifications to existing methods are required to reflect this. The aim of this thesis is therefore to examine the effect of a mobile bed and self-forming bedforms in a meandering channel.

In order to examine the effects of natural bedforms in the main channel two flow cases were examined; a flat bed (rectangular main channel) and natural bed (self-formed bedforms). The stage-discharge rates were found for a range of depths varying from inbank flow up to deep overbank flow with sediment transport in the natural bed case. Four depths were chosen for further examination where detailed velocity and turbulence measurements were taken using a Laser Doppler Anenometer. The bedforms were recorded using digital photogrammetry. The research from the Loughborough flume, on which this thesis is based, therefore represents a more natural and realistic channel. It also provides an additional and unique data set for use by river engineers and researchers to apply and test computer and other prediction models.

This thesis consists of six chapters. In Chapter 2 the characteristics and flow mechanisms in compound channels are examined. Straight, skewed and meandering compound channels are reviewed, including a brief summary of some of the existing
prediction methods. Chapter 3 details the Loughborough flume, the experimental apparatus and the measurement procedures. Chapter 4 shows and discusses the measured data, with comparisons between the flow in the flat bed and natural bed cases. Chapter 5 shows the derivation of a new theoretically based prediction method for compound meandering channels with further verification on data from the FCF flume. Finally Chapter 6 gives the summary of the work, conclusions and recommendations for further work.
2 Literature review

When studying channel flow it is important to have an understanding of the influential factors and flow mechanisms. The literature review examines the different flow mechanisms that have been found to influence the flow and stage-discharge relationship of straight, skew and meandering channels.

From previous literature, including Ervine et al. (1993) and Rameshwaran (1997), the main influential factors for channels can be summarised as:

- Slope (S)
- Channel shape
- Boundary roughness
- Relative flow depth of the floodplain (DR)
- Sinuosity (sin)
- Aspect ratio (P)
- Meander belt width relative to the floodplain width (MBW:FW)
- System scale

2.1 Straight compound channels

There has been a great deal of research conducted on straight channels. Between 1987 and 1989, extensive and detailed research on straight channels was investigated in phase A of the research programme conducted at the FCF, which also incorporated skew channel investigations.

2.1.1 Longitudinal velocity

Longitudinal velocity is the speed of the flow in the longitudinal direction relative to each section. In the main channel this is typically the streamwise velocity aligned to the main channel, whilst on the floodplain it is the velocity in the direction of the floodplain. In straight channels the longitudinal velocity in the main channel is faster than that on the floodplain. This causes a shear layer between the flow in the two areas. This shear layer was originally discovered by G. V. Zheleznyakov in the 1950’s. Sellin (1964) first visualised that the shear causes vertical vortices and horizontal eddies at the main channel/floodplain interface (see Figure 2.1).

This has been further proved using flow visualisation by researchers such as Imamoto and Ishigaki (1983, 1990) and Pasche and Rouve (1985). Figure 2.2 shows how the
momentum is transferred between the floodplain and main channel, although Fukuoka and Fujita (1989) added that horizontal eddies only occur when a large velocity differential exists. Tokuyay (1994) stated that the interaction is increased for floodplains with a greater roughness, with Wormleaton (1996) confirming similar results. Tominaga and Nezu (1991) observed that increased roughness on the floodplain had an affect on the longitudinal velocity. With a rougher floodplain the velocity on the floodplain was reduced, and the maximum velocities in the main channel did not reach up to the free surface as it did for the smoother case.

Knight and Demetriou (1983) showed results that indicated the lateral retardation of the main channel flow from the shallow regions. Kiely (1990) described how this shear layer (or velocity differential) between the areas occurs along their interfaces and counteract each other i.e. the flow in the main channel will increase the flow on the floodplain and the flow on the floodplain will decrease the flow in the main channel. Wormleaton (1996) stated that momentum is exchanged between the two sections and found that the effects of this shear layer extended out across the width of the floodplain, decreasing to zero shear at the outer edges of the floodplain. Myers and Elsayy (1975) and Knight and Mohammed (1984) showed that in straight channels with single floodplains the interaction between the flow in the two sections actually reduces the discharge. Ackers (1991) claimed that the interaction is affected by the shape of the main channel. In channels with rectangular cross-sections the faster flow in the main channel is in closer proximity to the slower flow on the floodplain so the interaction effects are increased, whereas in trapezoidal channels there is a transition zone which limits the interaction effects.

Myers (1978) found that the turbulence and apparent shear stress at the vertical main channel/floodplain interface were greater at lower overbank flow depths and decreased as the flow depth increased. He tried to account for the interaction through an 'apparent shear force' (a product of the turbulence intensity and area of interface), which was a maximum at a flow depth over the minimum depth which corresponded to the greatest reduction in the velocity. Wormleaton et al. (1982) stated that the apparent shear stress increased with floodplain roughness. Kawahara and Tamai (1989) concluded that this increase with roughness was due to enhanced turbulent diffusion. Wormleaton et al. (1982) also found that the vertical shear stress was more significant than the shear stresses at the horizontal or diagonal interfaces, even with the low aspect ratio of the main channel (~2.4) in their experiment.
Kawahara and Tamai (1989) stated that the mechanisms of the lateral momentum transfer are advection due to secondary currents and turbulent diffusion. Modelling the flow with a Reynolds stress model they showed that advection dominated the transfer rate.

At relative depths near 0.5, Lai and Knight (1988) showed the effect of the interaction as the velocity near the main channel/floodplain junction reduced, with the maximum velocities on the floodplain. At higher depths the velocity across the main channel was constant with faster velocities on the floodplain. Tominaga et al. (1989) found similar results.

### 2.1.2 Secondary currents

Secondary currents are defined as flow normal to that in the longitudinal flow direction. They distort the longitudinal velocity pattern and boundary shear stress distribution and are therefore important as they affect the flow resistance, sediment transport, bed and bank erosion, and in turn influence the channel morphology (Bathurst et al., 1979). Secondary currents are induced in two ways: through Reynolds stress variations or centrifugal force. Reynolds stress induced currents occur in straight channels and are caused by anisotropic turbulence. In comparison to centrifugal currents, found in meandering channels, they are weak. Kiely (1990) found them to be about only 2 percent of the maximum longitudinal velocity.

Numerous researchers, including Shiono and Knight (1989, 1991), Tominaga et al. (1989) and Tominaga and Nezu (1991) have investigated secondary flows in compound channels. They all found two distinct main secondary flows (or cells) in the main channel/floodplain interface area with one large cell extending across the entire width of the floodplain. Kiely and McKeogh (1993) stated that the secondary flows in the main channel and on the floodplain all act so that the water near the water surface moves away from the main channel/floodplain interface. The flow pattern of secondary currents are influenced by the shape of the channel, Figure 2.3 shows an example of different flow patterns found in rectangular and trapezoidal channels by Shiono and Knight (1989). Steeper bank slopes create greater separation between the floodplain and the main channel flows. They found that the strength of the different cells was dependant upon the relative flow depth of the channel and that a roughened floodplain has only a limited affect on the secondary currents.
2.1.3 Bed shear stress

Rajaratnam and Ahmadi (1979, 1981) showed that the bed shear reduces from the centre of the main channel towards the edge of the main channel. Then the bed shear stress sharply increases at the floodplain interface, decreases and levels off for most of the width of the floodplain, and finally decreases near the floodplain wall. They also stated that, when compared to single straight channels, the effect of the floodplain is to reduce the bed shear stress in the main channel. This is consistent with a reduction of velocity in the main channel due to slower velocity on the floodplain.

Tominaga et al (1989) stated that the wall shear stress increased where the secondary currents flowed towards the wall, but decreased where the currents flowed away from the wall. Knight and Demetriou (1983) showed similar results occurred on the main channel bed. It has already been shown that the bank slope affects the flow patterns in the channel, in addition Rhodes and Knight (1994) stated that the bank slope had a significant effect on the distribution of shear stress at the main channel-floodplain interface, hence bank erosion will occur at different locations on the main channel wall. So the bed shear stress is important as it governs the sediment transport in the channel.

2.1.4 Overall flow mechanisms

The main characteristics of flow can be summarised as fast flow in the main channel and slower flow on the floodplains with momentum transferred between the two regions, and secondary flow vortices. The overall flow mechanisms were clearly illustrated in Shiono and Knight (1991), shown in Figure 2.4.

2.1.5 Mobile bed

Knight and Brown et al. (1999) reported on the response of straight mobile bed channels for inbank and overbank flows. During high flow the sand formed transverse dunes in the main channel. As a result they found that obtaining the required data, which included stage-discharge and velocity, in a mobile channel was far more difficult than for rigid boundary channels. It was not possible to measure the boundary shear stress in the main channel due to the bedforms, which they were unable to freeze sufficiently for them to remain stable. In some cases sediment was deposited on the floodplain next to the main channel, leading to embankments. The velocity in the main channel remained significantly faster compared with that on the floodplain, despite the increased roughness of the main channel bed. There was also a three-fold increase
with depth in Manning’s n, which highlighted that using a single value in computer models is inadvisable.

2.2 Skew compound channels

Skew compound channels have a straight main channel, which is angled across the floodplain. From the experimental data in James and Brown (1977), Elliott and Sellin (1990), Ervine and Jasem (1989, 1995) and Sellin (1995) it is clear that there is a definite reduction in the discharge capacity of a skewed channel compared to an equivalent straight channel. Ervine and Jasem (1989) presented results that quantified the discharge reduction to be 5-10 percent for equivalent straight channels. Elliott and Sellin (1990) found the decrease to be 2-12 percent, with the maximum reduction at a relative depth of 0.2 irrespective of the skew angle. Ervine and Jasem (1995) observed that increasing the skew angle increases the interaction of the flow and therefore the resistance of the channel.

2.2.1 Longitudinal velocity

All the published research, including Rajaratnam and Ahmadi (1983), Ervine and Jasem (1989), Elliott and Sellin (1990) and Sellin (1995), found that the longitudinal velocity is faster on the downstream floodplain, which diverges, than on the upstream floodplain, which converges. Ervine and Jasem (1995) valued this difference to be typically 20-30 percent, but sometimes up to 50 percent and claimed it was because of a momentum exchange caused by a net transfer of mass from the converging to the diverging floodplain.

Also, within the main channel itself the velocity is greater nearer the downstream edge than on the upstream edge. Elliott and Sellin (1990), Ervine and Jasem (1995) and Sellin (1995) all found that the location of the greatest velocity shifts from the middle of the main channel towards the diverging side and maybe even on to the floodplain for higher discharges, as is shown by the dashed line in Figure 2.5.

Elliott and Sellin (1990) found that the floodplain flow had a much greater retarding force at low depth ratios than it did for higher depth ratios. They also found that the momentum transfer process due to the vertical shear layer seen in straight channels is swamped by the strong cross-flow component, especially as the depth increases.
2.2.2 Flow separation

Ervine and Jasem (1995) illustrated that as the floodplain flow passes over the main channel flow the flow separates (see Figure 2.6). A layer of the floodplain flow next to the floodplain bed enters the main channel secondary flow cell. The flow in the upper layer of the floodplain flow is accelerated by the main channel flow and crosses over the main channel and flows into the opposite floodplain, although it is deviated by an angle, $\phi$, proportional to the angle of skew, $\theta$. This angle, $\phi$, is dependant on the relative flow depth and the floodplain roughness.

The channel's aspect ratio has a strong contribution to the interaction of the floodplain and main channel flows and this consequently affects the flow structure. Ervine and Jasem (1995) found that the degree of separation is dependant on the aspect ratio and the angle of the side slope of the main channel (see Figure 2.7). The larger the aspect ratio the deeper the floodplain flow is able to penetrate into the main channel and the steeper the bank slope the greater the degree of separation.

2.2.3 Secondary flow

Figure 2.6 shows how the secondary flows are driven by the floodplain flow, this was originally shown by Elliot and Sellin (1990). Ervine and Jasem (1990) found that for smooth floodplains the recirculating velocity reduced progressively downstream and it also varies significantly with the relative depth of the compound channel flow. The strongest recirculations were around 20-30 percent of the normal floodplain velocities and occurred at relative depths of 0.2-0.3. For rough floodplains the recirculating velocities reduced only slightly downstream (still 20-30 percent of the normal floodplain velocities), but were independent of the relative flow depth.

2.2.4 Bed shear stress

The bed shear stress was measured across a section in reports by Elliott and Sellin (1990) and Sellin (1995). They found that just inside the main channel next to the diverging floodplain a large peak occurs in the bed shear stress, which then decreases almost linearly to the edge of the floodplain. Whereas on the converging floodplain the bed shear stress remains almost constant.

2.2.5 Overall flow mechanisms

The main flow mechanisms in skew compound channels can be summarised as a velocity shift from the converging to the diverging floodplain, the main channel flow
diverting the overbank flow, and the overbank flow causing secondary circulations in the main channel.

2.3 Meandering channels

Many researchers, including Ervine et al. (1993), Kiely (1990) and Willetts and Hardwick (1990), have shown that meandering channels are not as efficient as straight or skewed channels. The additional flow mechanisms and energy losses in meandering channels reduce the efficiency of the channel, and ignoring additional losses in estimation methods can therefore lead to significant errors (James, 1994).

Non-bed friction losses can often be as great as the total bed losses from the main channel and floodplain, and these increase with sinuosity, smaller main channel aspect ratios and changes in the main channel from natural to trapezoidal or rectangular cross sections. Kiely (1990) and James and Wark (1992) identified the additional flow mechanisms in meandering channels to be secondary currents, horizontal shearing, flow expansion and contraction, downstream effects of cross-over flow and faster flow on the floodplain outside of the meander belt. These mechanisms are discussed below.

2.3.1 Inbank flow

For inbank flow secondary currents are introduced as the water passes round the bend. These are caused by the velocity variations at different depths and the resulting imbalance in the centrifugal force of the water; the faster velocity near the water surface drives the water to the outside of the bend and the lower flow towards the inside of the bend. Kiely (1990) found the magnitude of the secondary currents in meandering channels to be much higher than in straight channels, possibly up to 30 percent of the maximum longitudinal velocity, compared to 2 percent in straight channels. These currents affect both the lateral and vertical distribution of the longitudinal velocity, as well as the water surface profile. The water builds up at the outside of the bend, so the water profile slopes downwards from the outside of the bend to the inside, this effect is called superelevation. The increase in the length of the flow path and the increased turbulence both act to increase the energy loss of the channel (Chang, 1992). James and Wark (1992) found the non-friction energy losses accounted for between 15-40 percent of the total energy losses. Leeder and Bridges (1975) showed that a stagnant zone can appear in the flow around the bend. This is caused by the flow separating from the inside of the bend as it passes around the bend and is dependent upon the tightness of the bend and the Froude number.
2.3.2 Longitudinal velocity

Patra and Kar (2000) found, as have many researchers, that at low flood depths there are large differences between the average main channel and floodplain velocities, which confirms the momentum transfer between the two regions. As the flow depths increase the velocities become equal, indicating a reduction in the momentum transfer between the two areas. Kiely (1990) found that velocities in the main channel of meandering channels can be reduced by up to 50 percent of those in equivalent straight channels. Marriott (1999) found that with very sharp bends where flow separation occurred, the overbank flow increased the conveyance of the inbank zone, but for other cases the overbank flow reduced the velocity in the main channel below the bankfull level. Kiely (1990) also stated that the maximum velocities in the main channel, above and below bank level, are close to the inner bend. Sellin and Willetts (1996) showed that the maximum velocity remains close to the inner bend at the apex, but then weakens and moves across to the outside of the bend further downstream. Similarly Toebes and Sooky (1967) found that for high overbank flows the maximum velocity filament in the main channel moved from the outside of the meander to the inside, roughly following the inside of the bend (see Figure 2.8). This shows possible effects of long floods on the stability of the banks along the inside of the bend.

Kiely (1990) found that the highest velocity occurs on the floodplain outside of the meander belt. James and Wark (1992) stated that the flow outside the meander belt is relatively untouched by the flow in the main channel and within the meander belt. Kiely (1990) claimed that the area outside could be considered as a straight channel with minimum energy losses. Although Liu and James (1997) found accounting only for frictional losses outside the meander belt overestimated the conveyance of the section. Nevertheless, the meander belt width : total floodplain width ratio affects the interaction between the main channel and the floodplain. With a smaller meander belt width in relation to the overall floodplain width, the effect of the meandering channel on the overall discharge is reduced. The flow will also be effected by obstructions on the floodplain, such as vegetation or buildings. Liriano et al. (2001) investigated the effect of obstructions on the floodplain and found that the magnitude of the effect on water levels due to such obstructions is strongly dependant upon the location of the obstructions.
2.3.3 Secondary flow

With overbank flow the direction of the secondary currents is in the opposite sense to those for inbank flow (see Figure 2.8). Originating along the upstream edge of the crossover section the secondary current grows along the outside of the bend up to the apex and then rapidly decays. This effect has been shown amongst others by Willetts and Hardwick (1990), Kiely (1990), Willetts and Rameshwaran (1996), Shiono et al. (1993) and Ervine et al (1994). Kiely stated that this was due to the shear induced at the interface by the overbank flow and that the energy loss because of this shear is greater for meandering channels than for straight compound channels. Shiono and Muto (1998) found that the energy produced by the secondary currents was equivalent to that produced by the turbulence. In inbank flow the bed shear turbulence is the dominant feature, but in overbank flows the turbulence due to the mixing of the flow becomes dominant.

Kiely and McKeogh (1993) said this had implications for sediment movement and channel erosion. Willetts and Rameshwaran (1996) state that prolonged periods of overbank flow will lead to different characteristics in the topography of the channel bed compared to that for inbank flow. They also found that the strength of the secondary flows increase with floodplain roughness. Rameshwaran (1997) also stated that the extent of the secondary currents is determined by the planform geometry and the cross-sectional shape of the channel.

The sinuosity (or radius of curvature) directly influences the flow in a meandering channel, particularly the secondary currents. As the sinuosity of the main channel increases (or the radius of curvature decreases) the secondary currents increase. The flow resistance and the energy loss in the main channel significantly increases, this then reduces the discharge capacity of the channel (Chang, 1992). As overbank flow increases, the effects of the main channel's sinuosity on the overall flow of the channel reduce, although this reduction is more apparent in rough channels than smooth channels (James and Wark, 1992).

2.3.4 Horizontal shear

The flow in the main channel tends to run parallel to the edge of the sidewall. Kiely (1990) showed that low overbank flows run almost parallel to the main channel, but that as the overbank depth increases the flow tends to run parallel to the floodplain. Kiely concluded that this shows the existence of a horizontal shear layer along the main
channel/floodplain interface. Shiono and Muto (1998) investigated the magnitude of the horizontal shear through turbulence measurements. They concluded that for low floodwater depths there is significant deflection of the upper-layer flow caused by the interaction of the upper and lower layer flows. With deeper flow the interaction is less significant and the two layers are less dependent on each other. Shiono and Muto (1998) stated that the vertical shear layer caused by the floodplain flow crossing over the main channel is controlled by the angle of the meandering channel and the floodplain and the water depth.

2.3.5 Flow expansion and contraction

Kiely (1990) showed that as the water flows along the channel, water passes from the floodplain into the main channel, causing a flow expansion. The water in the main channel also flows into the floodplain, increasing its velocity and therefore causing a flow contraction. This can be seen in Figure 2.8. This means that the discharge in the main channel varies along the length of the meander, this is further supported by Ervine, Sellin and Willetts (1994) and Willetts and Rameshwaran (1996).

Sellin, Ervine and Willetts (1993) stated that it was the water passing out onto the floodplain downstream that draws the upstream floodplain flow into the main channel. They also found that the flow plumbs into the middle of the main channel rather than at the upstream bank, see Figure 2.8. These fluid exchanges between the two sections introduce extra flow resistance and energy losses and therefore increase the stage of the channel. Ervine and Ellis (1987) identified it to be one of the main causes of energy loss in the channel. Kiely (1990) also suggested that the energy loss due to the turbulence, evident as the flow re-enters the floodplain, creates a low velocity area downstream within the meander belt.

2.3.6 Overall flow mechanisms

The main flow mechanisms for meandering channels can be seen in Figure 2.8. They include the secondary currents, the shifting of the high velocity filament to the inner bank and the flow expansion and contraction as the flow exchanges between the main channel and the floodplain. Rameshwaran (1997) stated that numerous researchers investigating the stage-discharge relationship have found that as the flow changed from bankfull to overbank, the increased energy losses from the momentum transfer of the flow can actually reduce the discharge of the channel. Shiono et al. (1993) and Al-Romaiah (1996) found that this discharge reduction is more pronounced for meandering channels than for straight channels.
2.3.7 Mobile bed

Ackers (1970) conducted experiments on the development of initially straight channels carved into a sand bed. Providing that the conditions are such that sediment is transported, then meandering channels formed and the meanders migrated downstream. This was similar to the work carried out by Tyler and Friedkin (1945), who explained that the downstream migration of the meanders was due to the collapse of the banks whose material was then transported a small distance downstream before being deposited on a sandbar. Ackers (1970)b stated that the energy loss can be split into form losses and bed friction and that the form losses due to variations in the bend and cross-sections account for some 60 percent of the head loss in the channel. Kikkawa et al. (1976) noted that different size sand particles naturally collected together in certain parts of the channel. They called it Segregation Phenomenon where smaller particles collected near the inner bend and larger particles at the outside of the bend.

More recently compound meandering channels with a mobile main channel bed have been investigated by Lyness et al (1999). Their experiments were conducted as part of the phase C programme at the UK FCF at Wallingford. They investigated flow on a compound meandering channel with both smooth and rough floodplains and found that up to relative depths of 0.4 Manning’s n roughness in the main channel was greater than that on the floodplain, even for the rough floodplain experiment. At relative depths greater than 0.4 the main channel and floodplain Manning’s n values become similar. With a smooth floodplain the sediment formed bedforms between 40 – 80 mm in amplitude, approximately 450 – 700 mm apart in the main channel. With the rough floodplain the bedforms were more variable. Knight and Shiono (1996) hypothesised that the interaction effect of the main channel and floodplain flow may lead to a reduction in the sediment transport at low overbank depths.

Ishigaki and Muto (2001) showed that the secondary flows and the bedforms affect each other, where the secondary flows scour the bed and form sandbars. A preliminary report (Shiono et al., 2001) using the research presented here showed how the flow structure in the channel with a mobile bed is completely different from those in a rectangular channel. More secondary cells are generated with the mobile bed and the interaction between the upper- and lower-layer flows reduces.
2.3.8 Doubly meandering channels

Doubly meandering channels have meandering floodplain walls as well as meandering main channels. Naish and Sellin (1996) and Lambert and Sellin (2000) investigated the hydraulic performance of a doubly meandering compound channel by modelling a section of the river Blackwater in Hampshire. Naish and Sellin (1996) found that during overbank flow the secondary currents remained in a similar direction to the inbank flow and in the crossover region the secondary circulations prevented the floodplain flow from plunging below the bankfull level. They also discovered regions of stationary vortices behind the bends in the floodplain wall, which reduced the conveyance of the channel. The effect of these flow structures reduces as the flow depth increases. They also investigated the effect of inclined berms. Inclining the berms increased the strength of the stationary vortices, increased the longitudinal velocity in the main channel and reduced it on the floodplain. Lambert and Sellin (2000) found that up to overbank flow depths of 0.25 the mean velocity in the main channel actually decreased. At low overbank depths the velocity on the main channel and floodplain were different, but as the flow depth increased the difference between the two sections reduced.

2.4 Stage – discharge prediction methods

There have been numerous methods proposed to predict flow in all types of channels and flow, many with only slight variations between them. A brief overview of some of the basic and important methods and different approaches is presented here to give an idea of how stage-discharge prediction has been attempted. It is by no means a comprehensive or exhaustive review of all the methods available.

Many of the methods are based on simple formulae, such as Darcy-Weisbach, Chezy and Manning’s equations, which use roughness coefficients. It is generally accepted that they will provide reasonable results for straight single channels provided the flow is in the rough turbulent zone. None of these simple methods are designed to account for the interactions that occur in overbank channels or meandering flow, and their direct application leads to considerable error. The main approaches to account for the additional energy losses in straight or meandering overbank flows have been:

- Division channel methods
- Correction factor methods
- Two- and three-dimensional modelling
The division channel methods divide the channel into subsections, calculate the conveyance in each of the sections using equations such as Manning's equation, and then sum them to give a total discharge. There are many different divided channel methods, as they assume different positions and directions of the divisions, for example a horizontal split at the bankfull level, or vertical and angled splits originating from the main channel and floodplain junction. Within each of the section different factors can also be assumed, for example whether to include or exclude the dividing lines in the wetted perimeter or assign different roughness values to each section. In his investigations on a straight channel with one floodplain Myers (1978) showed that the momentum transfer was of significant magnitude, but decreased with depth so that at high flow depth the subdivisions may be justified.

Correction factor methods assign a correction factor to the value to give a total discharge for the channel. The various correction factors can either be applied to whole channels, segregated channels or the divided channel methods described above.

In two-dimensional modelling the depth-averaged equations of motion and continuity for steady uniform flow are used to calculate the depth-averaged velocity and bed shear stress in the streamwise direction. These are typically based on theoretical derivations of flow rather than empirical fitting of data. Three-dimensional models are used to predict the secondary flows in the channel as well as the longitudinal velocity. These tend to use turbulence and viscosity models and mainly involve straight channels due to the complex nature of the modelling.

2.4.1 Straight channel methods

Wark et al (1990) used a simple turbulence model, combined with accurate roughness values to predict the lateral velocity distribution across the channel, called the Lateral Distribution Method (LDM). This was applied to natural river channels and other scale model studies with reasonable success.

Ackers (1991, 1992, 1993) estimated the interaction between the floodplain and main channel flow from empirical adjustment factors. The flow is split into four regions based on the relative flow depth with 22 equations to estimate the discharge. Based on the FCF data set the method was also applied to natural rivers and other model studies.
Shiono and Knight (1989, 1990 and 1991) derived a two-dimensional model to predict the velocity distribution across the channel. Shiono and Knight (1989) combined the streamwise momentum on a fluid element with continuity. Using the eddy-viscosity approach and ignoring secondary currents to obtain the depth-averaged velocity equation. Later, Shiono and Knight (1990, 1991) found that secondary currents, particularly in compound channels, are a significant mechanism within the flow and cannot therefore be ignored, so a secondary flow term was introduced.

2.4.2 Skew channel methods

Ackers (1991) used a discharge deficit function to apply his straight channel method to skew channels. For skew channels of less than 10° and similar roughness on the floodplain and main channel:

\[
\text{DISDEF}_{(\text{skew})} = \text{DISDEF}_{(\text{straight})} (1.03 + 0.074 \theta)
\]  

(2.1)

where \( \theta \) is the angle of skew. Ackers concluded that in channels where the floodplains are rougher than the main channel equation (2.1) is too conservative.

For skew channels the Ervine et al. (1993) method used two zones (floodplain and main channel zones divided vertically), with a correction factor \( F^* \) and different roughness values for the different zones.

\[
F^* = \frac{\text{actual discharge in compound channel}}{\text{theoretical discharge based on bed friction}}
\]  

(2.2)

\( F^* \) indicates the interaction between the main channel and floodplains; as it decreases from unity the degree of interaction increases. Ervine and Jasem (1995) showed that the maximum interactions occurred at relative depths of 0.2 and increased with the skew angle, but warned that sub-division methods can grossly overestimate discharges.

2.4.3 Meandering channel methods

Toebes and Sooky (1967) divided the channel at the bankfull level and applied basic frictional losses to both sections. The additional energy losses (non-bed friction) were accounted for by increasing the wetted perimeter and found to be a function of the overbank flow depth, the mean velocities in the two zones and the longitudinal slope.
Greenhill and Sellin (1993) used Manning's equation to calculate flow in different regions of the channel, modifying the bed slope and the zonal areas to represent the additional energy losses. The results were within ± 3.5 percent for their data, but the application assumes a fully developed horizontal shear layer between the main channel and floodplain flow and so its applicability is limited as it is invalid for low overbank depths and wide channels where the main channel is dominant.

James and Wark (1992) used an empirical approach to calculate the discharges in different zones of the channel. Their method uses the bankfull discharge and then adjust it to the overbank flow. Based on the data set they used the results were typically accurate to five percent, but tended to underpredict the discharge and the available data only covered a limited range of conditions.

Al-Romaih (1995) did not use a friction factor. Instead, the parameters found to be influential in channel flow were placed into dimensionless groups and the relationship between them calibrated from reliable data.

Ervine and Ellis (1987) devised a method which accounted separately for energy losses due to the bed friction, the secondary flows, and the expansion and contraction in four zones; below bank, overbank within the meander belt width and two zones either side outside of the meander belt width. Although they stated that the turbulent shear between the main channel and the floodplain flow was important they were unable to quantify it. In many cases the floodplain friction was found to be dominant in the flow.

Shiono et al. (1999) developed the Ervine and Ellis (1987) method to account for the streamwise turbulent shear stresses at the bankfull level. They showed that the energy losses due to the secondary flow losses suggested by Chang (1983), part of the assumption in the method, varied from the measured values and therefore need to be re-considered. They stated this is due to the different secondary flow structure for inbank and overbank flows and the different originating mechanisms. The secondary flow losses were generally the largest losses, but the proportion of the secondary flow loss to the expansion loss decreases with sinuosity and relative flow depth. At large relative flow depths and high sinuosity the expansion loss can exceed the secondary loss. The losses due the interfacial bankfull shear were found to be significant and
need to be considered. They increased with sinuosity and reduced with relative flow depth. They concluded that a formula for the friction factor, $f$, more suited to natural channels is required similar to that presented in Muto (1997) for straight channels.

Ervine et al. (2000) developed Shiono and Knight's (1990, 1991) straight channel method for meandering channels. They replaced the secondary flow term $UV$ with a secondary flow coefficient $K$. Applying their method to various channel configurations they found $K < 0.5$ percent for straight compound channels and $2 < K < 5$ percent for meandering compound channels. This gives an indication of the greater significance of secondary flows in meandering channel compared to straight channels. However, having been derived directly from a straight channel method the model is mathematically incorrect when applied to meandering channels.

2.5 Summary

Skewed and meandering channels act to reduce the discharge capacity of compound channels compared to equivalent straight channels. This is caused by an increase in the interaction between the main channel and floodplain flows. In addition to bed friction the other main sources of energy loss and the main flow mechanisms can be summarised as:

- Secondary currents
- Momentum transfer
- Horizontal and vertical shear layers
- Flow expansion and contraction

As demonstrated by the previous research and investigations cited previously, these mechanisms affect the distribution of the flow. When the river is in flood they increase the flow in the floodplain and reduce it in the main channel.

The more we can understand the structure and the important processes of the flow in rivers, the greater our ability to account for the increased energy losses in the flow and the more accurately we will be able to predict the stage and discharge of channels.
Figure 2.1: Surface velocity patterns (after Sellin, 1964)

Figure 2.2: Large scale eddy structure (after Fukuoka and Fujita, 1989)
Figure 2.3: Illustration of secondary flows; a) rectangular channel, b) trapezoidal channel (after Shiono and Knight, 1989)

Figure 2.4: Mechanisms of overbank flow in a straight compound channel (after Shiono and Knight, 1991)
Figure 2.5: Schematic of depth averaged velocities along flume length (after Ervine and Jasem, 1995)

Figure 2.6: Floodplain flow bifurcates in region of skewed main channel (after Ervine and Jasem, 1995)
Chapter 2 – Literature Review

Figure 2.7: Effect of; a) main channel aspect ratio on cross-over region mixing, b) main channel bank slope on mixing in cross-over region (after Ervine and Jasem, 1995)

Figure 2.8: A representation of important flow mechanisms present within flooded meandering channels (after Willetts and Hardwick, 1993)
3 Methodology (Experimental set-up and data acquisition)

In this Chapter the experimental facilities, channel configurations and experimental procedures used throughout this study are explained. As stated before, the main aim of this study was to examine the effect of natural bedforms on the flow in a compound meandering channel. This involved examining changes in the characteristics of the flow and the bedforms in the main channel. In addition to a naturally formed bed a rectangular main channel, of equivalent depth to the natural formed bed case was also examined. The flat bed case was to enable comparisons between the flat bed and natural bed cases, to see how the flow mechanisms altered.

There are numerous other parameters that can be examined in this type of research. Time was the main limiting factor on the number of parameters that were investigated. However, the cost of modifications and the size of the flume also restricted the different parameters that could have been examined.

3.1 Experimental set-up

3.1.1 The Loughborough flume facility

The flume, as shown in Figure 3.1, was of rectangular cross-section 13m long, 2.4m wide and 0.3m deep with a fixed longitudinal slope of 0.002, constructed from glass and Perspex. Flow was controlled by a weir at the bottom of the flume (Figure 3.2) and three pumps. Figure 3.3 shows a plan of the main features of the flume.

The reservoir just upstream of the weir, actually situated in the flume, collected the sediment at the downstream end of the end of the channel (Figure 3.4). A sediment pump then circulated the sediment through a pipe system back to the upstream inlet. The maximum pump capacity was around 5 litres/sec and was measured by a 3100 Maxflo flow meter. The minimum flow rate used to ensure sediment re-circulation was 1.9 litres/second.

The bulk of the discharge was pumped from the main reservoir into the stilling pool at the top of the flume via either one or two pipes depending on the discharge. Each pipe contained an individual flow meter, which was calibrated prior to installation. Together the two pumps provided a maximum discharge of around 45 litres/sec. A wooden board
floating on the stilling pool helped to reduce the turbulent jet flow from the pipes, before the water flowed into the channel.

3.1.2 The Meandering channel

Within the flume a 0.4m wide meandering channel was formed from 150mm thick Styrofoam sections, which were cut and glued onto the flume basin. The joints were sanded and filled with Polylilla to ensure smooth connections between the Styrofoam sections and the foam was then painted to seal the channel. The Styrofoam at the test section was replaced with clear Perspex. This was to enable better visualisation in the channel and to allow possible laser measurements from beneath the flume. In the event this feature was not actually utilised.

A total of 3¼ meanders were constructed and the test section was half a wavelength long at meander number two. This allowed the flow to develop sufficiently at the test section. The channel's parameters are shown in Figure 3.5 and given in Table 3.1. A channel sinuosity of 1.384 was selected, as it was comparative to previous studies including the larger FCF channel. This enabled scale comparisons to be made.

<table>
<thead>
<tr>
<th>Table 3.1: Channel parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sinuosity, sin</td>
</tr>
<tr>
<td>1.3837</td>
</tr>
</tbody>
</table>

The main channel contained uniform sand with a mean size of 0.85mm. Initially the main channel depth was set to 75mm (aspect ratio 5.33), however at higher discharges the Perspex beneath the main channel became exposed. The range of depths over which the Perspex was not exposed did not provide a large enough range in the flow conditions so the main channel depth was decreased to 40mm (aspect ratio 10), see Figure 3.6. This aspect ratio was selected because the bed did not become exposed at higher discharges and it was also similar to previous research. The main channel parameters are shown in Table 3.2.

One of the other problems encountered during the initial experiments was sand at the inlet being washed away due the turbulent flow at the main channel inlet. To overcome
this, and ensure smoother flow into the channel, pebbles were added to the main channel bed at the inlet section.

Table 3.2: Main channel parameters

<table>
<thead>
<tr>
<th>Bankfull Width $B_{bf}$ (mm)</th>
<th>Bankfull Depth $h_{bf}$ (mm)</th>
<th>Depth of Sand $d_s$ (mm)</th>
<th>Bank Side Slope $SS_{mc}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>40</td>
<td>110</td>
<td>90°</td>
</tr>
</tbody>
</table>

3.2 Measurements

The data measured included stage-discharge, sediment transport, bedforms, bed shear stress, velocity and turbulence. Each of the measurement procedures are described in detail in subsequent sections. To aid some of the reasoning in later sections an overview of the order of experiments is described. Firstly the stage-discharge and sediment transport rates were established for the flume with a mobile bed. The order of flow depths investigated is shown in Table 3.3. The case number relates to the experiment number, where $g2$ refers to experiments on the natural bed. From examining the bedforms in all cases listed in Table 3.3, four flow depths (shown in Table 3.4) were chosen for further examination, including velocity and bed shear stress readings. The four depths selected, in actual order of experimentation, were $g2_1$, $g2_5$, $g2_3$ and $g2_1$, with approximate relative flow depths or depth ratios ($DR$) of 0.45, 0.3, 0.2 and 0.0 respectively. $DR$ is defined as:

For inbank flow, $-1 < DR < 0$,

$$DR = \frac{h_{mc} - h_{bf}}{h_{bf}}$$

For overbank flow, $DR > 0$,

$$DR = \frac{h_{fp}}{h_{fp} + h_{bf}}$$

where $h_{fp}$ is the depth of the flow relative to the floodplain level, $h_{mc}$ is the depth of the flow in the main channel, $h_{bf}$ is the bankfull depth of the main channel. Once the natural bed case was completed the stage-discharge relationship was then established for the
flat bed case, listed in Table 3.5, with further examination at similar relative depths to the mobile bed case i.e. g4_3, g4_5, g4_7 and g4_10 (listed in Table 3.6).

Table 3.3: Experiment order and details of natural bed case

<table>
<thead>
<tr>
<th>Experiment no.</th>
<th>Case no.</th>
<th>Relative flow depth (DR)</th>
<th>Depth, $h_p$ (m)</th>
<th>Discharge, $Q$ (m$^3$/s)</th>
<th>Velocity, $U$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>g2_1</td>
<td>-0.09</td>
<td>-0.0036</td>
<td>0.00356</td>
<td>0.245</td>
</tr>
<tr>
<td>2</td>
<td>g2_2</td>
<td>0.13</td>
<td>0.0062</td>
<td>0.00469</td>
<td>0.152</td>
</tr>
<tr>
<td>3</td>
<td>g2_3</td>
<td>0.21</td>
<td>0.0106</td>
<td>0.00628</td>
<td>0.151</td>
</tr>
<tr>
<td>4</td>
<td>g2_4</td>
<td>0.26</td>
<td>0.0140</td>
<td>0.00856</td>
<td>0.173</td>
</tr>
<tr>
<td>5</td>
<td>g2_5</td>
<td>0.31</td>
<td>0.0179</td>
<td>0.01106</td>
<td>0.188</td>
</tr>
<tr>
<td>6</td>
<td>g2_6</td>
<td>0.35</td>
<td>0.0219</td>
<td>0.01741</td>
<td>0.254</td>
</tr>
<tr>
<td>7</td>
<td>g2_7</td>
<td>0.40</td>
<td>0.0266</td>
<td>0.02297</td>
<td>0.288</td>
</tr>
<tr>
<td>8</td>
<td>g2_8</td>
<td>0.45</td>
<td>0.0324</td>
<td>0.03078</td>
<td>0.328</td>
</tr>
<tr>
<td>9</td>
<td>g2_9</td>
<td>0.50</td>
<td>0.0403</td>
<td>0.04251</td>
<td>0.378</td>
</tr>
<tr>
<td>10</td>
<td>g2_10</td>
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<td>0.0271</td>
<td>0.02297</td>
<td>0.283</td>
</tr>
<tr>
<td>11</td>
<td>g2_11</td>
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<td>0.0181</td>
<td>0.01108</td>
<td>0.186</td>
</tr>
<tr>
<td>12</td>
<td>g2_12</td>
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<td>0.0110</td>
<td>0.00624</td>
<td>0.147</td>
</tr>
<tr>
<td>13</td>
<td>g2_13</td>
<td>0.17</td>
<td>0.0084</td>
<td>0.00467</td>
<td>0.129</td>
</tr>
<tr>
<td>14</td>
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<td>0.0000</td>
<td>0.00349</td>
<td>0.218</td>
</tr>
<tr>
<td>15</td>
<td>g2_15</td>
<td>0.45</td>
<td>0.0326</td>
<td>0.03078</td>
<td>0.326</td>
</tr>
</tbody>
</table>

Table 3.4: Depth conditions of detailed measurement cases for natural bed case

<table>
<thead>
<tr>
<th>Depth Condition (DR)</th>
<th>Case no.</th>
<th>Relative flow depth, (DR)</th>
<th>Depth, $h_p$ (m)</th>
<th>Discharge, $Q$ (m$^3$/s)</th>
<th>Velocity, $U$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>g2_1</td>
<td>-0.09</td>
<td>-0.0036</td>
<td>0.00356</td>
<td>0.245</td>
</tr>
<tr>
<td>0.2</td>
<td>g2_3</td>
<td>0.21</td>
<td>0.0106</td>
<td>0.00628</td>
<td>0.151</td>
</tr>
<tr>
<td>0.3</td>
<td>g2_5</td>
<td>0.31</td>
<td>0.0179</td>
<td>0.01106</td>
<td>0.188</td>
</tr>
<tr>
<td>0.45</td>
<td>g2_15</td>
<td>0.45</td>
<td>0.0326</td>
<td>0.03078</td>
<td>0.326</td>
</tr>
</tbody>
</table>
Table 3.5: Experiment order and details of flat bed case

<table>
<thead>
<tr>
<th>Experiment no.</th>
<th>Case no.</th>
<th>Relative flow depth (DR)</th>
<th>Depth, $h_p$ (m)</th>
<th>Discharge, $Q$ ($m^3/s$)</th>
<th>Velocity, $U$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>g4_1</td>
<td>-0.52</td>
<td>-0.0208</td>
<td>0.00159</td>
<td>0.206</td>
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<tr>
<td>17</td>
<td>g4_2</td>
<td>-0.25</td>
<td>-0.0099</td>
<td>0.00317</td>
<td>0.263</td>
</tr>
<tr>
<td>18</td>
<td>g4_3</td>
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<td>-0.0001</td>
<td>0.00515</td>
<td>0.323</td>
</tr>
<tr>
<td>19</td>
<td>g4_4</td>
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<td>0.0059</td>
<td>0.00595</td>
<td>0.198</td>
</tr>
<tr>
<td>20</td>
<td>g4_5</td>
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<td>0.195</td>
</tr>
<tr>
<td>21</td>
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<td>0.00991</td>
<td>0.208</td>
</tr>
<tr>
<td>22</td>
<td>g4_7</td>
<td>0.29</td>
<td>0.0167</td>
<td>0.01348</td>
<td>0.240</td>
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<tr>
<td>23</td>
<td>g4_8</td>
<td>0.35</td>
<td>0.0218</td>
<td>0.01982</td>
<td>0.290</td>
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<tr>
<td>24</td>
<td>g4_9</td>
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<td>0.0270</td>
<td>0.02530</td>
<td>0.313</td>
</tr>
<tr>
<td>25</td>
<td>g4_10</td>
<td>0.46</td>
<td>0.0337</td>
<td>0.03312</td>
<td>0.342</td>
</tr>
<tr>
<td>26</td>
<td>g4_11</td>
<td>0.50</td>
<td>0.0395</td>
<td>0.04246</td>
<td>0.383</td>
</tr>
</tbody>
</table>

Table 3.6: Depth conditions of detailed measurement cases for flat bed case

<table>
<thead>
<tr>
<th>Depth Condition (DR)</th>
<th>Case no.</th>
<th>Relative flow depth (DR)</th>
<th>Depth, $h_p$ (m)</th>
<th>Discharge, $Q$ ($m^3/s$)</th>
<th>Velocity, $U$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>g4_3</td>
<td>0.0</td>
<td>-0.0001</td>
<td>0.00515</td>
<td>0.323</td>
</tr>
<tr>
<td>0.2</td>
<td>g4_5</td>
<td>0.2</td>
<td>0.0103</td>
<td>0.00793</td>
<td>0.195</td>
</tr>
<tr>
<td>0.3</td>
<td>g4_7</td>
<td>0.29</td>
<td>0.0167</td>
<td>0.01348</td>
<td>0.240</td>
</tr>
<tr>
<td>0.45</td>
<td>g4_10</td>
<td>0.46</td>
<td>0.0337</td>
<td>0.03312</td>
<td>0.342</td>
</tr>
</tbody>
</table>

3.2.1 Stage-discharge

To establish the stage-discharge curve of the flume, uniform flow was established for a full range of depths up to a relative flow depth of 0.5. Uniform flow is defined as $S_o = S_f$, where $S_o$ is bed slope and $S_f$ is friction slope. This is achieved by setting the slope of the water surface parallel to that of the bed slope. The water surface levels were measured using a pointer gauge, accurate to 0.1mm, which was mounted on a bridge across the flume (Figure 3.7). For inbank flow the water levels were measured at the
centre of the main channel at each crossover section. For overbank flow the water levels were measured on the floodplain, at the centre of the flume at each apex section. The water surface slope was calculated from the water surface levels measured along the flume. After each set of readings the discharge and weir settings were, if necessary, adjusted with the aim of establishing uniform flow. The flume was then allowed up to two hours to resettle and the water levels were rechecked until uniform flow conditions had been achieved.

With a mobile bed the equilibrium of the channel was subject to change as the bedforms altered. For this reason the flume was left to run continuously for approximately thirty-six hours per stage, until representative bedforms had developed and a stable water slope was achieved.

3.2.2 Sediment transport

For each of the measured stages the sediment transport rate was also measured. This involved diverting the sediment discharge through an alternative outlet and using a sieve to catch all the sediment that passed through the sediment pipe (Figure 3.8). Every twenty minutes the sieve would be removed, replaced with a second sieve, and then weighed to determine the amount of sediment being transported. After weighing the sediment was placed back into the main channel at the inlet. Altogether the sediment was collected over a period of six hours for each stage.

3.2.3 Bedform

The sand was levelled at the selected depth using a sand leveller (Figure 3.9), which as previously explained in Section 3.1.2 was 40mm. The flume was run at bankfull depth for thirty-six hours to establish inbank bedform, after which the flow was incrementally increased for overbank depths.

At each of the increments, uniform flow was established, as described in Section 3.2.1, and run for around thirty-six hours. This allowed sufficient time for the bedform to develop along the flume. The flume was then drained slowly at each depth and the bedform was mapped using automated digital photogrammetry. This produces a digital elevation model that was used to create elevation plots and extract cross-sectional data of the bedform. A more detailed description of the process and theory can be found in Chandler (1999) and Chandler et al. (2001). A Kodak digital camera was used to record the bed images.
The images from the photogrammetry at each of the depths can be seen in Figure 3.10 to Figure 3.18. The left-hand side is upstream and the bed height is indicated by the grey-scale; the darker the colour the deeper the bed. The images were used to select the flow conditions for further investigation. \( DR=0.45 \) was chosen due to the clear sand bars and bankfull \((DR)\) was chosen so that inbank and overbank flow conditions could be compared. Then \( DR=0.2 \) and \( DR=0.3 \) were chosen as suitable steps to show the development of the flow between the bankfull and \( DR=0.45 \) flow conditions. Depths lower than \( DR=0.2 \) would have been impractical due to the shallow flow on the floodplain making measurements on the floodplain difficult.

For each of the depth conditions the sand was re-levelled, run at bankfull flow for thirty-six hours to establish the inbank bedform, and then run at the chosen overbank flow depth for another thirty-six hours. Once the bedform had developed, the bed needed to be fixed so that detailed velocity measurements could be taken. This was so that the bedform remained unchanged during the velocity measurements. Otherwise the bedform would have either gradually changed over the duration of the measurements or instruments placed into the flow would have disturbed the bedform.

The procedure for fixing the bed was as follows:

- The flume was run at the required depth until the natural bedform had evolved.

- The flume was then carefully drained so as not to disturb the bedform. This was achieved by switching off the pumps and blocking the channel at the bottom of the flume so that the water in the flume drained away very slowly.

- Once all the pools of water in the main channel had disappeared a thin layer of cement was sieved over the sand. The water remaining in the sand acted as the moisturiser for the cement.

- The cement was then left to harden for 24 hours, with additional water being sprayed onto the cement throughout the period. After 24 hours the cement had hardened sufficiently for the flume to be run without damaging the surface.
It was assumed that as the bedform developed the average depth in the main channel remained constant. This was checked by calculating an average depth of the bedform from the digital elevation model. These can be seen in Table 3.7. The average depth was typically within 1.5 percent of the expected mean depth.

Table 3.7: Averaged depths of the bedforms

<table>
<thead>
<tr>
<th>Case</th>
<th>g2_1</th>
<th>g2_3</th>
<th>g2_5</th>
<th>g2_15</th>
</tr>
</thead>
<tbody>
<tr>
<td>Av. Depth (mm)</td>
<td>37.7</td>
<td>40.5</td>
<td>40.5</td>
<td>39.5</td>
</tr>
<tr>
<td>Error (%)</td>
<td>+5.7</td>
<td>-1.4</td>
<td>-1.3</td>
<td>+1.2</td>
</tr>
</tbody>
</table>

3.2.4 Bed shear stress

A 2.7mm Preston tube in conjunction with a 5mb-pressure transducer, shown in Figure 3.19, was used to measure the bed shear stress in the main channel and on the floodplain at the measurement sections. In the main channel the points were measured at four centimetre intervals along the bed and 1cm spaces up the wall. On the floodplain the stress was measured at approximately 15cm intervals which corresponded to the locations of the velocity measurements. At each point, except for measurements at the wall, the angle of the main flow component was measured using a flow vane indicator (see Figure 3.20). The Preston tube was then set to that angle at each of the points to measure the bed shear stress.

The Preston tube measured the dynamic pressure, i.e. the difference between the Pitot and static pressures. The pressure transducer passed a voltage reading to a standard PC via an Analog/Digital interface card. For each data point the computer recorded 600 readings over a period of 600 seconds. The voltage was then turned into a bed shear stress. In order to calculate the bed shear stress the method established by Patel (1965) was used. This method based upon the assumption of a universal inner law calculates the bed shear stress from the dynamic pressure difference. Where:

\[ x^* = \log \left[ \frac{\Delta p g d_p^2}{4 \rho v^3} \right] \]  

and,
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\[ y^* = \log\left(\frac{\tau_b d_p^2}{4\rho v^2}\right) \text{ where } \tau \text{ is unknown} \]  \hspace{1cm} (3.2)

where \( p_p \) is the Preston tube reading, \( d_p \) is the diameter of the Preston tube, \( \rho \) is fluid density, \( v \) is kinematic viscosity and \( \tau_b \) is the bed shear stress.

for \( x^* \leq 2.9 \),

\[ y^* = x^* + 0.037 \]  \hspace{1cm} (3.3)

for \( 2.9 < x^* < 5.6 \),

\[ y^* = 0.8287 - 0.1381x^* + 0.1437x^2 - 0.006x^3 \]  \hspace{1cm} (3.4)

for \( x^* \geq 5.6 \),

\[ y^* = x^* - 2 \log(1.95y^* + 4.1) \]  \hspace{1cm} (3.5)

Substituting equation (2.1) and (2.2) into equation (3.3) can then be re-arranged to give the shear stress, \( \tau_b \).

3.2.5 LDA System

A three component Laser Doppler Anemometer (LDA) system was used to measure velocities and Reynolds stresses in the flow. The system works by aligning the laser beams at a precise location within the flow. By measuring the frequency shift of the lasers, as particles pass through the focal point, the velocity of the particle can be calculated. It is assumed that the particle is representative of the flow. The laser beams were emitted from submersible probes that were connected to the laser source via 10m fibre optic cables. Different measurements required different set-ups of the submersible probes and so are described later within each relevant section.

In addition to the probes and cables the LDA system comprised of a Spectra Physics Series 2000 power source, a TSI multicolour laser coupler device, FIND software and a Time and Precision traverser system. The back-scattered signals from the focal point are received by photomultipliers, passed onto amplifiers, frequency shifters and then TSI IFA 550’s (Intelligent Flow Analyser). The data was then recorded using the FIND Software Analysis package on a standard Pentium II personal computer. The
equipment can be seen in Figures 3.19 and 3.20. A plan of the system is shown in Figure 3.21.

The FIND programme controlled all the parameters and recording conditions of the laser system. Up to three velocities could be measured simultaneously with various modes of operation:

- **Coincident mode**, only takes readings when each channel registers a velocity simultaneously.
- **Even-time mode**, sets a time interval and takes one reading per channel per time interval.
- **Random mode**, takes readings in each of the channels independently.

Ideally coincident mode would have been used to collect all the data, as this would automatically provide the Reynolds shear stresses. For two-component measurements there were no problems as one of the probes emitted two sets of beams, which were aligned by the manufacturer. The three-component measurements required two probes and no precise alignment apparatus was available. With the measurement volume of the beams being only 25μm, it proved extremely difficult to align all six beams to obtain a reasonable data rate in coincident mode. As a result random mode had to be used to collect the three-component data. Even-time mode was found to be unsuitable for taking measurements, because if a value was not recorded during the time interval then the value from the previous time interval was used for extrapolation, which gave misleading values. Once the data was collected the FIND software analysed the data to give mean velocities and turbulence intensities.

### 3.2.5.1 Velocity measurements

The velocity was measured both in the main channel and on the floodplain. In the first experiment, the g2_15 case, the flow and bedforms were predominantly parallel to the floodplain rather than the main channel. So the velocities across the whole channel were measured at eleven cross-sections perpendicular to the floodplain (Figure 3.24). For all the subsequent experiments the velocities in the main channel were measured perpendicular to the main channel at seven cross-sections (Figure 3.25). This was due to the bedforms being more inline with the main channel at lower flow depths and to reduce the number of measurement points, so reducing the time required to record the
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data. The floodplain velocities were measured at each section perpendicular to the floodplain wall.

The bed level profiles of the main channel at each section were determined from digital photogrammetry. This was much easier and less time consuming than the traditional method of using a pointer gauge. It also eliminated any risk of a pointer gauge damaging the bedforms. Using the sectional bed level profiles, a grid of points, at which the velocities were to be measured, was created. The grids had spacings of 0.5cm in depth and 2cm in width, with widths of 1cm near the edge of the main channel. An example can be seen in Figure 3.26.

The laser beams, which measured the velocities, were shone down into the flow. This enabled measurements to be made close to the bed and also kept the probes clear of any protruding bedforms. This set-up had previously been used successfully by Knight and Shiono (1990), Tominaga and Nezu (1991) and Muto (1997). In their cases the water surface was very smooth, so the laser could pass through into the water with only minor distortions. However, due to the rough water surface found in this channel it was necessary to place the probes in a Perspex tank filled with water that rested on the water surface, shown in Figure 3.27. The tank was constructed from index matching Perspex which enabled the laser beams to pass into the flow undisturbed. For measurement points 4.5cm below the water surface the probes could be placed directly into the water without the tank.

The tank had a minimal affect on the flow. The theoretical boundary layer thickness, as calculated in Section 3.2.5.1.1, demonstrates the extent of the disturbance. The calculation shows that a majority of the readings occurred outside of the boundary layer and where this was not case the effect of the tank was negligible.

The collected data was analysed by the FIND programme to give average velocities. Due to the set-up of the probes these average values were then transformed into $\overline{U}$, $\overline{V}$ and $\overline{W}$, which are the longitudinal, transverse and vertical velocities relative to the measurement section using:
\[ \bar{U} = Ch1 \times \cos 45 + Ch2 \times \cos 45 \]
\[ \bar{V} = Ch3 \]
\[ \bar{W} = Ch2 \times \cos 45 - Ch1 \times \cos 45 \]

where Ch1, Ch2 and Ch3 were the velocities measured from each of the laser probes.

After each section was measured, the data was processed and plotted. If there were any discrepancies in the readings, or obvious errors, then they were retaken.

The velocity measurements were also analysed to give the turbulent kinetic energy, \( k \), from:

\[ k = \sqrt{Ch1^2 + Ch2^2 + Ch3^2} \]

where \( Ch1' \), \( Ch2' \) and \( Ch3' \) are the turbulence measured in each channel.

### 3.2.5.1.1 Boundary layer thickness of the Perspex tank

The terminology for calculating the boundary layer thickness is shown in Figure 3.28. For turbulent flow the general equation for the boundary layer thickness, \( \delta_{bl} \), is (Young 1989):

\[ \delta_{bl} = 0.37 \left( \frac{\nu x_{bl}^4}{U_{bl}} \right)^{\frac{1}{2}} \]  \hspace{1cm} (3.6)

\( x_{bl} \) is the displacement from the leading edge of the boundary layer
\( U_{bl} \) is the velocity of the flow outside the boundary layer

From equation (3.6) it is clear that the boundary layer thickness is inversely proportional to the velocity to the power \( 1/5 \).

The typical velocities for the different depth conditions ranged from 0.1-0.4 m/s and the maximum displacement, \( x_{oh} \) from the leading edge of the tank to the point of measurement was 14cm. Figure 3.29 shows the boundary layer thickness with velocity calculated for different \( x_{bl} \) displacements, which are listed in the legend (in m). The graph shows that the boundary layer thickness reduces with faster velocities and
smaller $x_{bl}$ displacements. Table 3.8 shows the results for a selection of velocities at the maximum $x_{bl}$ displacement (14cm). Table 3.9 shows the depth from the Perspex tank to the first row of LDA measurements for each of the flow cases. Comparing the values in Table 3.8 and Table 3.9 it is apparent that for the g2_5, g2_15 and g4_7 cases all the LDA measurements are outside of the boundary layer. However, for the other cases the top row of measurements were within the boundary layer and were therefore affected by the Perspex tank.

Table 3.8: Boundary layer thickness with velocity

<table>
<thead>
<tr>
<th>$U_{bl}$ (m/s)</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta_{bl}$ (mm)</td>
<td>7.69</td>
<td>6.69</td>
<td>6.17</td>
<td>5.83</td>
</tr>
</tbody>
</table>

Table 3.9: Depth from tank to first row of measurements

<table>
<thead>
<tr>
<th>Case</th>
<th>g2_1</th>
<th>g2_3</th>
<th>g2_5</th>
<th>g2_15</th>
<th>g4_3</th>
<th>g4_5</th>
<th>g4_7</th>
<th>g4_10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (mm)</td>
<td>3.56</td>
<td>5.6</td>
<td>7.87</td>
<td>7.78</td>
<td>4.9</td>
<td>5.3</td>
<td>6.7</td>
<td>3.7</td>
</tr>
</tbody>
</table>

The amount by which these readings are affected can be estimated. Figure 3.30 shows the typical velocity profile in a turbulent boundary layer. It can be seen that the velocity reduction is small at the top of the boundary layer, and it is only in the lower fifth of the boundary layer that the velocity decreases sharply. The equation for the velocity profile, as given in French (1985), for hydraulically smooth cases is:

$$U = 2.5 U^* \ln \left( \frac{9zU^*}{\nu} \right)$$

where $U^* = \sqrt{(gRS)}$ (shear velocity) and $z$ is the vertical distance above the bed.

For the Loughborough flume the largest boundary thickness calculated was 7.69mm with a velocity of 0.1m/s. These values were used in equation (3.7) to calculate a suitable $U^*$ value (0.0066m/s). Using this $U^*$ value the velocities were calculated from the channel bed up to the boundary layer limit. The nearest measurements to the tank were 3.56mm (for the g2_1 case). At that depth the worst possible scenario i.e. longest $x_{bl}$, slowest flow etc., the velocity reduction due to the tank was only twelve percent.
These calculations have shown that for a large majority of the measurements taken the Perspex tank had no affect on the velocity readings in the channel. Where the tank did have an affect it was minimal, with a maximum reduction in velocity of only twelve percent and in most cases significantly less. In any case, it was found that laser readings could not be taken within approximately 3mm of the boundary. This was due to the amount of noise in the LIDA readings created by the boundary. So, within the area where the Perspex tank would have had a significant affect on the readings, i.e. nearer the solid edge of the boundary layer, it was not possible to take readings.

3.2.5.2 Measurement duration

The accuracy of the LDA measurements is dependent upon the data rate and duration of the readings. The minimum duration necessary to achieve accurate and reliable readings for velocity, turbulence intensity and Reynolds stress was investigated. Numerous recordings of different duration, varying from 10 up to 300 seconds, were taken at a single point. During the recordings, the data rates were set between 40-50Hz as this was found to be the minimum obtainable data rate in the main experiments. (It should be noted that the typical data rate for most of the measured data points was much higher, over 100Hz.)

For each of these recordings the mean velocity, turbulence intensity and Reynolds shear stress were calculated along the time series and compared to the overall mean values (calculated from all the collected data). The results can be seen in Table 3.10. The deviation from the overall mean values for one of the data points is shown in Figure 3.31. The graphs show that the deviation is variable at the beginning of the recording and that the deviations of the different measurements vary. The mean velocity is relatively stable varying ±3 percent, turbulence intensity \( (k) \) varies by ±15 percent and the Reynolds stress \( (\tau_{uw}) \) is highly variable (±100 percent). From Table 3.10 it is clear that the time required to achieve an accurate mean value (within 1 percent of the total mean) is 10 seconds for velocity, 20 seconds for turbulence intensity and 40 seconds for Reynolds stress value (within 5 percent).

A measurement time of 180 seconds, which was used in the experiments, resulted in 7200 data points per channel (180 secs x 40Hz) at low data rates and 18000 data points per channel (180 secs x 100Hz) at typically achieved data rates. As a comparison, Muto (1997) required 5000 readings to obtain an accurate reading. The LDA measurements were therefore considered accurate and reliable.
Table 3.10: Laser data validation

<table>
<thead>
<tr>
<th>T (sec)</th>
<th>No. of samples</th>
<th>Velocity, U (m/s)</th>
<th>Velocity deviation, U (%)</th>
<th>Turbulence intensity, k (m/s)</th>
<th>Turb. int. deviation, k (%)</th>
<th>Reynolds stress, ( \tau_{uw} ) (N/m²)</th>
<th>( \tau_{uw} ) deviation, (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>8</td>
<td>0.1311</td>
<td>-0.29</td>
<td>0.00780</td>
<td>-1.17</td>
<td>0.000022</td>
<td>-22.61</td>
</tr>
<tr>
<td>20</td>
<td>8</td>
<td>0.1316</td>
<td>0.05</td>
<td>0.00790</td>
<td>0.15</td>
<td>0.000028</td>
<td>-5.22</td>
</tr>
<tr>
<td>40</td>
<td>8</td>
<td>0.1311</td>
<td>-0.27</td>
<td>0.00786</td>
<td>-0.40</td>
<td>0.000028</td>
<td>-3.56</td>
</tr>
<tr>
<td>60</td>
<td>8</td>
<td>0.1312</td>
<td>-0.25</td>
<td>0.00796</td>
<td>0.86</td>
<td>0.000029</td>
<td>-1.69</td>
</tr>
<tr>
<td>120</td>
<td>8</td>
<td>0.1314</td>
<td>-0.10</td>
<td>0.00792</td>
<td>0.42</td>
<td>0.000029</td>
<td>0.67</td>
</tr>
<tr>
<td>180</td>
<td>8</td>
<td>0.1314</td>
<td>-0.04</td>
<td>0.00789</td>
<td>-0.01</td>
<td>0.000029</td>
<td>-1.73</td>
</tr>
<tr>
<td>300</td>
<td>2</td>
<td>0.1312</td>
<td>-0.23</td>
<td>0.00782</td>
<td>-0.91</td>
<td>0.000027</td>
<td>-0.78</td>
</tr>
</tbody>
</table>
Figure 3.1: The Loughborough Flume

Figure 3.2: Weir Sections

Figure 3.3: Flume plan
Figure 3.4: Sediment collection

Figure 3.5: Geometric properties

Figure 3.6: Cross-section
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Figure 3.7: Pointer gauge

Figure 3.8: Sediment collection

Figure 3.9: Sand leveller
Figure 3.10: Bedform for g2_1, DR=-0.09

Figure 3.11: Bedform for g2_2, DR=0.13
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Figure 3.12: Bedform for g2_3, DR=0.21

Figure 3.13: Bedform for g2_4, DR=0.26
Flow direction

Figure 3.14: Bedform for g2_5, DR=0.31

Flow direction

Figure 3.15: Bedform for g2_6, DR=0.35
Figure 3.16: Bedform for g2_7, DR=0.40

Figure 3.17: Bedform for g2_8, DR=0.45
Figure 3.18: Bedform for g2_9, DR=0.50
Figure 3.19: Preston tube and pressure transducer

Figure 3.20: Flow vane indicator
Figure 3.21: LDA System

Figure 3.22: Traverser system
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Figure 3.23: LDA Plan

Figure 3.24: G2_15 measurement sections
Figure 3.25: Measurement sections

Figure 3.26: Typical measurement grid
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Figure 3.27: Perspex tank

Figure 3.28: Boundary layer terminology diagram
Figure 3.29: Boundary Layer Thickness with Velocity for a range of $x_{bl}$ displacements

Figure 3.30: Typical velocity profile in a turbulent boundary layer
Figure 3.31: Laser data validation:

a) Velocity $U$, b) Turbulence Intensity $u'$, c) Reynolds stress $\overline{u'w'}$

*Data rate ca. 40-50Hz*
4 Results

The results from the data are shown. The stage-discharge curves of the flat bed and natural bed cases are compared, as well as velocities and discharges within different sections of the channel. The sediment transport rate of the natural bed case is also discussed. The flow structure is also examined using longitudinal, transverse and vertical velocity plots, as well as the secondary currents, turbulent kinetic energy and layer-averaged velocities.

4.1 Stage-discharge

From examining the stage-discharge data, shown in Figure 4.1, it is possible to compare the efficiency of the natural and flat bed channels. From the figure it is clear that at low inbank and high overbank depths the discharges in both cases are similar. However, between the two extremes the flat bed channel is more efficient than the natural bed case, i.e. it has a higher discharge capacity for a given flow depth. Figure 4.2 shows the percentage difference between the natural bed and flat bed discharges. The largest difference is around 30% and occurs near the bankfull flow condition.

At low inbank depths, slow flows result in the minimal transportation of sediment (see Section 4.3). As a result, no prominent bedforms were formed, so they did not have a significant effect on the flow. The discharge in the natural bed case is therefore similar to the flat bed case. As the flow increases, nearing bankfull, the efficiency of the natural case decreases in comparison to the flat bed case. This reduction must be entirely due to the bedforms in the main channel and the resulting change in flow structure as no other parameter had been altered. At deeper overbank flow cases the difference between the two cases reduces as the efficiency of the two cases become similar. At a depth near $DR=0.5$, the difference between the two cases reduces to zero. This is due to the influence and significance of the main channel flow reducing as the flow on the floodplain becomes dominant.

4.2 Sectional discharges and velocities

The laser velocity measurements were used to calculate velocities and discharges within different sections of the channel. The below bank ($U_a$), overbank within the meander belt ($U_b$) and overbank outside the meander belt ($U_c$) velocities are listed in Table 4.1. The below bank and above bank velocities are shown in Figure 4.3. The sectional discharges ($Q_a$, $Q_b$, and $Q_c$) are listed in Table 4.2, and the proportion of the
flow carried below bank and overbank are shown in Figure 4.5 with a further breakdown shown in Figure 4.4. As a reminder, the g2 are natural bed cases and g4 are flat bed cases.

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth condition (DR)</th>
<th>Average velocity $U_a$ (m/s)</th>
<th>Belowbank $U_b$ (m/s)</th>
<th>Overbank within meander belt $U_c$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>g4_3</td>
<td>0.0</td>
<td>0.28545</td>
<td>0.28545</td>
<td>-</td>
</tr>
<tr>
<td>g4_5</td>
<td>0.2</td>
<td>0.17104</td>
<td>0.22592</td>
<td>0.12606</td>
</tr>
<tr>
<td>g4_7</td>
<td>0.3</td>
<td>0.22628</td>
<td>0.20912</td>
<td>0.22296</td>
</tr>
<tr>
<td>g4_10</td>
<td>0.45</td>
<td>0.33794</td>
<td>0.24319</td>
<td>0.33912</td>
</tr>
<tr>
<td>g2_1</td>
<td>0.0</td>
<td>0.17341</td>
<td>0.17341</td>
<td>-</td>
</tr>
<tr>
<td>g2_3</td>
<td>0.2</td>
<td>0.15156</td>
<td>0.17618</td>
<td>0.13055</td>
</tr>
<tr>
<td>g2_5</td>
<td>0.3</td>
<td>0.20473</td>
<td>0.15203</td>
<td>0.18866</td>
</tr>
<tr>
<td>g2_15</td>
<td>0.45</td>
<td>0.33634</td>
<td>0.24506</td>
<td>0.33680</td>
</tr>
</tbody>
</table>

The difference of the velocities below the bankfull level between the two cases is important. It shows significant similarities and differences between the two cases. The velocities below the bankfull level at the bankfull depth and shallow overbank flows are significantly slower in the natural bed case compared to the flat bed case. Figure 4.3 shows how with overbank flow the velocities within the main channel decrease to a minimum at a depth ratio of 0.3 in both the natural bed and flat bed cases. Even at this depth the velocity in the flat bed case of 0.2 m/s remains significantly faster than that of the natural bed case, around 0.15 m/s.

In the deepest case (DR=0.45) the velocity in the natural bed case increases rapidly, to a velocity far faster than seen that at any lower depth in the natural bed case. Importantly, the below bank velocity is the same as that in the flat bed case, hence the discharges at deeper depths in the two cases are similar. In the flat bed case the velocity also increases at DR=0.45, but the velocity remains slower than that at the bankfull depth. Above the bankfull level the velocities in both cases are very similar, especially at the 0.2 and 0.45 depth ratios. At DR=0.3 the velocity in the natural bed
case is slightly slower than that in the flat bed case by around 0.03 m/s, and must be linked to the slower flow below the bankfull level in the main channel.

The percentage of the flow in different areas (below bank, overbank within the meander belt and outside the meander belt) is shown in Figure 4.4. Obviously at the bankfull flow depth (g4_3 and g2_1) all the flow is below the bankfull level. In the deepest cases (g4_10 and g2_15) the percentages of the flow are almost identical, with each of the three areas carrying similar percentages of flow. It is in the shallow overbank flow where the differences occur. At $DR=0.2$ (g4_5 and g2_3) the percentage of flow below the bankfull level in the natural bed case drops to 44.9 percent compared to 51.9 percent in the flat bed case. The percentage of flow outside the meander belt is similar in both cases, but within the meander belt the percentage of flow is greater

### Table 4.2: Sectional discharges

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth condition (<em>$DR</em>$)</th>
<th>Total discharge $Q$ (m$^3$/s)</th>
<th>Belowbank* $Q_a$ (m$^3$/s)</th>
<th>Overbank within meander belt* $Q_b$ (m$^3$/s)</th>
<th>Overbank outside meander belt* $Q_c$ (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>g4_3</td>
<td>0.0</td>
<td>0.00456</td>
<td>0.00456 (100)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>g4_5</td>
<td>0.2</td>
<td>0.00697</td>
<td>0.00362 (51.9)</td>
<td>0.00236 (33.8)</td>
<td>0.000995 (14.3)</td>
</tr>
<tr>
<td>g4_7</td>
<td>0.3</td>
<td>0.01269</td>
<td>0.00335 (26.4)</td>
<td>0.00676 (53.2)</td>
<td>0.002590 (20.4)</td>
</tr>
<tr>
<td>g4_10</td>
<td>0.45</td>
<td>0.03276</td>
<td>0.00389 (11.9)</td>
<td>0.02075 (63.3)</td>
<td>0.008123 (24.8)</td>
</tr>
<tr>
<td>g2_1</td>
<td>0.0</td>
<td>0.00253</td>
<td>0.00253 (100)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>g2_3</td>
<td>0.2</td>
<td>0.00628</td>
<td>0.00282 (44.9)</td>
<td>0.00251 (40.0)</td>
<td>0.000953 (15.2)</td>
</tr>
<tr>
<td>g2_5</td>
<td>0.3</td>
<td>0.01206</td>
<td>0.00269 (22.3)</td>
<td>0.00675 (56.0)</td>
<td>0.002616 (21.7)</td>
</tr>
<tr>
<td>g2_15</td>
<td>0.45</td>
<td>0.03173</td>
<td>0.00339 (12.4)</td>
<td>0.01994 (62.9)</td>
<td>0.007867 (24.8)</td>
</tr>
</tbody>
</table>

*percentage of total discharge shown in brackets below
percent) in the natural bed case compared to a percentage of 33.8 in the flat bed case. Figure 4.5 shows the proportion of the flow carried below and above the bankfull level. Where the lines cross in each case shows the depth at which 50 percent of the flow is carried on the floodplain and 50 percent within the main channel. The floodplain in the natural bed case, plotted with the full line, carries 50 percent of the flow at a shallower depth than the flat bed case, plotted with the dotted line. This demonstrates how at shallower depths there is a greater dependence upon the floodplain to carry the flow in the natural bed case.

4.3 Sediment transport
The sediment transport rate is shown in Figure 4.6. The figure shows the sediment rates at different scales. The figure clearly shows how at low inbank flow depths there is no sediment transportation, but it tends to increase up to overbank depths around \( DR=0.15 \), but the sediment transport rate then decreases until the rate reaches a minimum at \( DR=0.3 \). Above \( DR=0.3 \) the sediment transport rate increases again and at \( DR=0.45 \) the sediment transport rate has increased greatly, to nearly 2.4 grams per second. This sediment transport rate must be related to the velocity of the flow in the main channel, as it is the energy of the flow that determines the transportation of the sediment. Table 4.1 lists the velocities, and although the average velocity in the whole of the channel is lowest at the 0.2 relative depth condition, the lowest velocities below bank are at the 0.3 relative depth condition. Figure 4.3 shows the below bank velocity reduction at \( DR=0.3 \) and that the velocity increases significantly at \( DR=0.45 \), which correlates to the pattern of the sediment transport rate.

4.4 Flow structures
This section examines the flow structure of the flat and natural bed cases using the measured velocity and turbulence data from the 0.0, 0.2, 0.3 and 0.45 relative depth conditions. The flat bed case is shown in Figure 4.7 to Figure 4.30. The natural bed case is shown in Figure 4.31 to Figure 4.58. The features of the longitudinal velocities and secondary currents in the main channel are discussed using the cross-sectional plots, i.e. Sections el, e3, e5, e7, e9, e11 and f1 (as shown in Figure 4.7), or Sections 1-11 (as shown in Figure 4.31) for the g2_15 case. The figures showing cross-sectional plots are all viewed from downstream of the Section. For example, in the longitudinal velocity plots the water is flowing towards the viewer. The transverse and vertical velocities are each relative to their respective axes. All of the plots in this Chapter, except the layered depth-averaged plots, have been normalised. The velocities and
secondary currents are normalised by the average velocity ($U_d$) for each depth and the turbulent kinetic energy is normalised by the shear velocity squared ($U^2$, where $U^* = \sqrt{(gRS)}$). The values used for normalisation are listed in Table 4.3.

Table 4.3: Average velocities and shear velocities used for normalisation

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth condition ($DF$)</th>
<th>Average velocity ($U_d$)</th>
<th>Average shear velocity ($U^*$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>g4_3</td>
<td>0.0</td>
<td>0.28545</td>
<td>0.0255</td>
</tr>
<tr>
<td>g4_5</td>
<td>0.2</td>
<td>0.17104</td>
<td>0.0179</td>
</tr>
<tr>
<td>g4_7</td>
<td>0.3</td>
<td>0.22628</td>
<td>0.0209</td>
</tr>
<tr>
<td>g4_10</td>
<td>0.45</td>
<td>0.33794</td>
<td>0.0273</td>
</tr>
<tr>
<td>g2_1</td>
<td>0.0</td>
<td>0.17341</td>
<td>0.0246</td>
</tr>
<tr>
<td>g2_3</td>
<td>0.2</td>
<td>0.15156</td>
<td>0.0181</td>
</tr>
<tr>
<td>g2_5</td>
<td>0.3</td>
<td>0.20473</td>
<td>0.0214</td>
</tr>
<tr>
<td>g2_15</td>
<td>0.45</td>
<td>0.33634</td>
<td>0.0270</td>
</tr>
</tbody>
</table>

4.4.1 Flat bed case

The mean longitudinal ($\overline{U}$), transverse ($\overline{V}$) and vertical ($\overline{W}$) velocities and turbulent kinetic energy ($k$) were calculated at each of the measured sections. The figures are shown systematically with the longitudinal plots for each of the depth conditions together (Figure 4.7 to Figure 4.10), followed by the transverse plots for each depth condition (Figure 4.11 to Figure 4.14), the vertical velocities for each depth condition (Figure 4.15 to Figure 4.18), the secondary currents (Figure 4.19 to Figure 4.22), the turbulent kinetic energy (Figure 4.23 to Figure 4.26) and the layer-averaged velocity (Figure 4.27 to Figure 4.30).

4.4.1.1 Velocities

The longitudinal velocity for the bankfull case (g4_3) is shown in Figure 4.7. Along most of the channel the velocity maxima occurs at about mid-depth, although appears to be slightly nearer the water surface at the apex sections (Sections e1 and f1). At the apexes the maximum velocity is a quarter of the width from the inside of the meander, which is on the right at Section e1 and on the left at Section f1. The close contour lines at the inside of the bend at the apexes indicate there is a velocity reduction near the
water surface. Downstream of Section 1, the main velocity filament shifts across to the left of the channel, which is the outside of the bend. Simultaneously, the maximum longitudinal velocity reduces. After Section e5, the maximum velocity increases again, reaching a maximum relative velocity of 1.6 (around 0.5 m/s). The transverse velocities, shown in Figure 4.11, are a maximum at the apexes near the inside of the bend and the water surface, with relative velocities near 0.2. As the flow passed around the bend the maximum transverse velocity shifts across to the middle of the channel and reduces along the crossover section. This is typical of centrifugal force and inbank flow. The vertical velocities, shown in Figure 4.15, are comparatively small only reaching relative velocities of 0.04.

Figure 4.8 shows the longitudinal velocity for the DR=0.2 case. The maximum velocity behaves similarly to the inbank case, as described in the previous paragraph, i.e. it decreases downstream from Section 1 and increases from the crossover section to the downstream apex. The maximum relative velocity of 2.2 is much larger than the inbank case, but is actually slower, only around 0.42 m/s. At the apexes the highest velocity occurs near the inside of the bend, but below the bankfull level. This is due to the interaction of the flow in the main channel with the floodplain flow. The flow in the upper layer is slowed, as indicated by the dense contour lines at the inner bend near the bankfull level. At Section 1, the dense contour lines and the effect of the floodplain are restricted to the inside of the bend, but they progressively extend further across the channel downstream. At Section e9, in the crossover region, the influence of the flow from the floodplain extends to well beyond the middle of the channel. The bulging of the contours, towards the floodplain, at the outside of the apexes indicates the floodplain flow increasing the main channel velocity.

The longitudinal velocity for the DR=0.3 case is plotted in Figure 4.9. At Section e1 the floodplain flow on the right-hand side is beginning to show signs of increasing the velocity within the main channel. However, at Sections e3 to e9 the flow from the right-hand floodplain is significantly reducing the longitudinal velocity in the upper layer flow, typically down to relative velocities of 0.5. Along the right-hand edge of the channel there is a high velocity section below the bankfull level. In the crossover section the flow structure is similar to that at the 0.2 depth condition; the strong cross-flow from the right-hand floodplain reducing the longitudinal velocity near the right-hand edge with an area of faster flow in the left-and side of the channel. This is also reflected in the similar transverse velocities, shown in Figure 4.12 and Figure 4.13 for the two depths. The
difference being that in the 0.3 depth condition the transverse velocities are faster, up to 0.8 compared to 0.4, and the faster velocities extend further into the channel, as indicated by the dense contours.

For the $DR=0.45$ case the flow pattern seems to have changed drastically from the lower flow depths. The longitudinal, transverse and vertical velocities are shown in Figure 4.10, Figure 4.14 and Figure 4.18 respectively. The fastest velocities are now occurring on the floodplain, whereas the highest velocities in the lower flow conditions were all found within the main channel. At the apex sections, the highest velocities within the main channel are at the inner bank and in the upper layer. In the crossover region the longitudinal velocities have reduced right across the channel, where the relative velocities are typically around 0.5-0.7. This compares to the lower depth conditions where velocities less than one were seen only at the right-hand side with velocities greater than one on the left-hand side. This is also reflected with higher relative transverse velocities, which exceed 0.9 in some parts of the channel compared to 0.4-0.6 in the lower depth conditions. The relative vertical velocities are also greater, with relative values up to 0.2 compared to 0.15. This indicates that the secondary circulations are actually stronger and not just the cross-flow from the floodplain.

4.4.1.2 Secondary currents

The secondary currents for the bankfull case are plotted in Figure 4.19. Strong secondary currents appear at the apex sections, as would be expected for inbank flow due to centrifugal force. The anti-clockwise circulation at the upstream apex (Section e1) remains until Section e7. In the crossover region the secondary cell weakens and starts to circulate in the opposite direction.

Figure 4.20 shows the secondary currents for the $DR=0.2$ case. Even at low overbank flows the secondary flow structure has already altered in comparison to the bankfull case. At Section e1 there is still the anti-clockwise circulation at the right-hand side of the channel. However, this is much smaller in size and counteracted by a clockwise circulation on the outside of the bend, which extends across most of the channel. At Section e3 the clockwise circulation at the outside of the bend has disappeared. The floodplain flow is entering the channel from the right side of the channel, although at this point it is very weak and doesn't have an effect on the secondary circulations. At Section e5 the flow from the floodplain is stronger and causes an anti-clockwise circulation beneath the bankfull level. This circulation appears strongest at Section e7.
and then weakens. It is clear that it is the floodplain flow that causes the circulation at the outside of the apex.

The secondary currents for the $DR=0.3$ case can be seen in Figure 4.21. At the upstream apex the clockwise circulation at the left of the bend nearly extends all the way across the channel, with the exception of a very small anti-clockwise circulation next to the right-hand bank. Along the crossover section there is a small circulation in the lower corner at the downstream (left-hand) edge. Along the right-hand edge there is strong flow across the channel above the bankfull level. This is due to the floodplain flow flowing into the main channel. Beneath the cross-flow there is an anti-clockwise circulation. At Section e3 this separation is only small, but increases further across the channel downstream to Section e9.

The $DR=0.45$ case is shown in Figure 4.22. At the apex section there is no secondary circulation near the inner bank, as seen in the lower depth cases. Instead, the circulation generated by the floodplain flow crossing over the main channel dominates the entire channel. This flow consists of several smaller secondary circulations spread across the channel rather than just one large circulation. At Sections e5 to e11 there is very strong flow from the floodplain into the main channel. This cross-flow causes a very strong secondary circulation along the right-hand bank below the bankfull level. At Section e3 it is very small, but this circulation progressively grows in width downstream and at Section e11 extends across the entire width of the channel.

4.4.1.3 Turbulent kinetic energy

The normalised turbulent kinetic energy, $k$, for the flat bed case is plotted in Figure 4.23 to Figure 4.26. By comparing the turbulent kinetic energy with the longitudinal velocities described in Section 4.4.1.1 it is apparent that areas with high turbulence are related to areas with significant velocity gradients, as indicated by the dense contours in the velocity plots. These are typically near or slightly below the bankfull level along the upstream bank in the crossover section or across the channel where the cross-flow from the floodplain diminishes. Clear examples of the increase in turbulent kinetic energy are best seen at $DR=0.2$ in Figure 4.24. In Sections e5 to e9 the region of higher turbulence moves across the channel from the right-hand side of the channel to the middle. The highest relative turbulent kinetic energy values are generally found at $DR=0.2$ and progressively decrease at higher depths.
4.4.1.4 Layered-averaged velocities

For each column of measurements, layered average velocities, i.e. below and above bank, were calculated for the $U$ and $V$ velocity components. These were plotted on a plan view of the channel to provide an overall picture of the flow structure.

With bankfull flow, Figure 4.27, the direction of the flow is parallel with the main channel. At the apex sections the highest depth-averaged velocity occurs slightly inset from the inside of the bend. Downstream from Section 11 the point of highest velocity gradually shifts over to the left-hand side of the channel. The velocity profiles vary gradually across all the sections.

For the $DR=0.2$ depth condition, shown in Figure 4.28, the velocity profile across each section does not vary gradually. At the crossover section there is an abrupt reduction across the channel, with high velocities along the left-hand bank of the channel and slower flows on the other side. This is the effect of the flow crossing from the upstream floodplain into the main channel. The further downstream, the further away from the right-hand bank this abrupt reduction occurs. On the floodplain the discharge is highly susceptible to slight variations in the flow conditions, which is entirely due to the shallow depth of flow. These flow conditions can be either characteristics, such as standing waves (which were seen on the floodplain), or small discrepancies in the actual level of the floodplain. This results in variations in the floodplain flow with areas of low and high velocities. The direction of flow on the floodplain is also of interest. Near the floodplain walls, outside the meander belt, the flow runs parallel with the floodplain, whilst within the meander belt the flow is angled towards the inside of the bend.

For the deeper flow cases, $DR=0.3$ and $DR=0.45$ (Figure 4.29 and Figure 4.30 respectively), the below bank flows are quite similar. That is, the highest velocity point at the apex occurs at the inside of the bend and decreases almost uniformly to the outer bend, although for $DR=0.45$ there is a secondary peak near the outside of the bend. At the apexes, the overbank flow for the $DR=0.3$ case is similar to the below bank flow i.e. it decreases towards the outside of the bend. Whereas for the $DR=0.45$ case, the overbank flow is almost uniform across the channel at the apexes and is actually faster than the below bank flow. In these deeper cases the floodplain flow outside the meander belt remains parallel with the floodplain. The direction of the flow within the meander belt tends towards the inside of the bend. Although the deeper the
flow the weaker the flow across the floodplain, and in the $DR=0.45$ case it is nearly parallel with the floodplain.

### 4.4.2 Natural bed case

It should be noted that due to the mobile bed the bed characteristics vary along the channel. So the cross-sectional profile at the upstream apex (Section e1) will not necessarily be the same as the downstream apex (Section f1). This results in differences in the flow structure at either of the sections. As with the flat bed case the figures are shown systematically. Figure 4.31 to Figure 4.34 show the bedforms of the four depths that were examined. The longitudinal plots for each of the depth conditions are shown together (Figure 4.35 to Figure 4.38), followed by the transverse plots for each depth condition (Figure 4.39 to Figure 4.42), the vertical velocities for each depth condition (Figure 4.43 to Figure 4.46), the secondary currents (Figure 4.47 to Figure 4.50) and the layer-averaged velocities (Figure 4.51 to Figure 4.54).

#### 4.4.2.1 Bedforms

The left-hand side of the figures are upstream, the water flowed from left to right. The depth of the bedform is indicated by the colour, the darker the colour the deeper the section.

The bedform of the bankfull case, shown in Figure 4.31, conforms to the expected profile for inbank flow. A deeper section appears along the outside of the bend, with deposition of the sediment on the inside of the bend. The deposition forms a sand bar that extends along the middle of the channel to the crossover section. Between the sand bar and the left-hand bank there is a flatter section where no sediment transport has occurred. This is due to the slow velocities in that region (see Figure 4.35).

The $DR=0.2$ bedform is plotted in Figure 4.32. At the downstream apex the deeper section has moved from the outside to the middle of the channel and is deeper than the inbank flow condition. At the downstream apex the gully originates further upstream than the inbank bedform. There is an undisturbed region of sand along the outside of the meander between the crossover and downstream apex.

Figure 4.33 shows the $DR=0.3$ bedform. There is no clear structure to the bedform. It has become irregular along most of the length of the test section, even in the region
between the crossover section and the apex downstream, which remained undisturbed for lower flows.

The $DR=0.45$ bedform, shown in Figure 4.34, is very striking due to the series of ridges along the channel. It is clear that the ridges are created by the secondary flows, described in Section 4.4.2.3. At the crossover section the ridges extend directly across the channel from the upstream to the downstream bank, with small areas of scour next to the downstream bank as indicated by the dark areas. The ridges originating further downstream from the crossover section continue down to the apex, curving around the bend, meeting the downstream bank around the outside of the bend. The bed level directly downstream from the apex becomes shallow to meet the floodplain. Across the apex there are a few ridges with the deepest section at the centre of the channel.

### 4.4.2.2 Velocities

At the upstream apex of the bankfull case, shown in Figure 4.35, the fastest longitudinal velocity occurs in the centre of the channel, directly over the deepest part of the cross-section. The relative velocity across the section varies from 0.4 to 1.5, which is greater than in the flat bed case of 0.7 to 1.5, although the average velocity in the natural bed case is much slower as shown in Table 4.3. Downstream from the apex the maximum velocity reduces slightly, and there is a *dead zone* with minimal flow and sediment deposition in the shadow of the bend. The highest velocities occur between the crossover section and the downstream apex reaching up to relative velocities of 1.9. It is interesting that these high velocities occur over shallow areas. Normally high velocity results in bed scour, although this does not seem to be the case in this location. Both the relative transverse velocities and vertical velocities, shown in Figure 4.39 and Figure 4.43 respectively, are larger in the natural bed case than in the flat bed case. For example the largest relative transverse velocities extend up to 0.4, compared to 0.2 in the flat bed case.

The relative longitudinal velocity for $DR=0.2$ is presented in Figure 4.36. The area of highest velocity has moved across nearer to the inside of the bend compared to the bankfull case. The maximum velocity at the apexes has also increased compared to the flat bed case, reaching relative velocities of 1.7-1.8. As with the flat bed case the floodplain flow at the inner side of the apex slows the longitudinal velocity. The velocities decrease slightly at Section e3, but increase along the crossover section along the downstream (left) bank. While at the upstream edge, along the outside of the
bend between the crossover and downstream apex, there are comparatively slow flows. The transverse velocities are shown in Figure 4.40. They tend to be stronger than in the bankfull case, particularly at the apex sections and along the upstream bank in the crossover region. The vertical velocities are shown in Figure 4.44. The strongest velocities are found at the apex sections and are typically relative velocities of 0.2-0.3.

Figure 4.37 shows the $DR=0.3$ longitudinal velocity. The variation in the relative velocities in the apex sections has reduced compared to both the bankfull and $DR=0.2$ cases, typically between 0.7-1.2. At the crossover section there are high velocities along the left-hand edge, up to 1.3-1.4, with lower longitudinal velocities at the right-hand side. The reduction is caused by the flow entering the main channel from the floodplain as the transverse velocities, shown in Figure 4.41, increase in this region. Across the width of the channel, the velocity reduces more gradually than for the lower depth flows. At the right-hand edge of the crossover section, the area of slower velocity below the bankfull level has resulted in the bed remaining unchanged from the initial bedform. This area of unchanged bedform increases in width downstream and indicates that no sediment transport has taken place.

The velocities for $DR=0.45$ are shown in Figure 4.38. It should be highlighted again that these velocities in this case were measured at different sections to those in the other cases, which are all normal to the floodplain. At the apex sections the relative velocities generally range from 0.7-1.1, with the some of the largest velocities on the floodplain. Other velocity peaks occur next to the inner bank at the apex, at the edges on the floodplain and above troughs in the bedforms. The flow above the bankfull level is faster than the below bank flow. The transverse velocities are shown in Figure 4.42, the pattern of flow is quite complicated, but the there is a noticeable pattern of higher transverse velocities above the ridges with regions of slower relative velocities near the water surface above the troughs. The vertical velocities are shown in Figure 4.46, with maximum velocities over the troughs and minimum velocities over the ridges.

4.4.2.3 Secondary currents

The bankfull case is plotted in Figure 4.47. At the apexes there is a singular secondary circulation with the top layer of flow flowing towards the outside of the bend. At Section e5 the main circulation is still in the anti-clockwise direction, but has moved across to the left-hand side of the channel. In the centre of the channel the flow is moving across
to the right-hand side of the channel. At Section e11 the secondary flows due to the centrifugal flow begin to increase again.

Figure 4.48 shows the secondary currents for $DR=0.2$. At the apexes there are strong flows at the inside of the bend towards the outside of the bend, which feeds into a singular secondary circulation in the direction expected for inbank flow. This circulation is restricted to the inner half of the bend by a secondary circulation at the outside of the bend in the opposite sense. At Section e3 there is strong cross-flow towards the outside of the channel, with a small circulation at the left-hand bank near the channel bed. Along the crossover section this circulation increases and the flow across the channel emanates from the right-hand floodplain, which is in the different direction to the cross-flow in the bankfull case. At Section e7 there is a small circulation below the bankfull level caused by the flow entering the main channel from the floodplain. This circulation grows progressively downstream to the apex section.

Figure 4.50 shows secondary currents for the $DR=0.3$ case. There is a centrifugal secondary circulation at the inside of each apex. This is opposed with another secondary cell at the outside of the bend, which extends across most of the channel. At Section e3 the cross-flow extends across the whole of the channel, including from the floodplain on the right-hand side. The flow from the floodplain increases along the crossover section, decreasing again around the downstream bend. It is clear that the cross-flow causes the secondary circulation beneath the bankfull level. There is also a rotation in the bottom of the deep section along the downstream edge at the crossover section, but this is smaller than the similar cell in the $DR=0.2$ case.

$DR=0.45$ is shown in Figure 4.50. The main secondary currents are caused by flow entering the main channel from the floodplain. Rather than there being one large cell across parts or the entire width of the channel, there is a series of circulations. This is similar to findings in Knight and Shiono (1996) and Shiono and Muto (1998). The troughs and crests are clearly formed by secondary circulations. Adjacent to some of the more pronounced peaks in the bedform there are smaller circulations. An example of these circulations can be seen in Section 11, just to the right of the bedform peak near the centre of the main channel. These are in the opposite direction to the main circulations and are caused by back flow as the water flows over the peaks. They help to maintain the sharp incline of the peaks. These smaller cells and the cross-flow offset
the main circulations from the centre of each bedform trough. The smaller cell near the inside of the bend increases as it passes around the bend.

4.4.2.4 Turbulent kinetic energy

The turbulent kinetic energy, \( k \), is shown in Figure 4.51 to Figure 4.54. The turbulent structure in all the cases is unsurprisingly much more complex than in the flat bed case. However, as with the flat bed case the largest turbulent kinetic energy is found in the \( DR=0.2 \) depth case. As with the flat bed case the larger turbulence values occur where there are steep gradients in the velocity. A good example of this in the \( DR=0.2 \) case (Figure 4.52) at Section e3 where there are values up to 9.0 in the centre of the channel.

4.4.2.5 Layer-averaged velocities

The averaged velocities for the bankfull case are plotted in Figure 4.55. The flow is not as parallel to the main channel and the velocity profile across the channel is not as smooth as in the flat bed case. At the apexes the outward flow is stronger, and the highest velocity filament is nearer the centre of the channel than the flat bed case. The slower flow is on the inside of the bend rather than on the outside. On the downstream apex there are few data points missing. This was due to the deposition of sediment at on the inside of the bend resulting in a very shallow bed, where no readings were possible (see Section 4.4.2.1).

\( DR=0.2 \) is presented in Figure 4.56. In the middle of the main channel at Sections e5 and e7, the upper layer flow is actually faster than the lower layer flow. This is opposite to normal, but is due to the very shallow water depth at those points. At Sections e9 and e11 there are sharp velocity reductions across the main channel due to the low flow from the floodplain. This characteristic is the same as for the flat bed case at the same depth.

For \( DR=0.3 \), shown in Figure 4.57, near the inside of the bend at Section e3 there are missing velocities below the bankfull level. This was due to a sandbar, which resulted in no velocity readings below the bankfull level (see Section 4.4.2.1 for more details about the bedforms). Except for Section e3, the below bank flow is parallel with the main channel.
Figure 4.58 shows the average velocities for DR=0.45. Along each section there are areas of converging and diverging averaged flows. This is due to the secondary circulations (Figure 4.50) and the series of ridges in the bedform (see Figure 4.34). The flow in the main channel is more in line with the floodplain rather than the main channel, especially the overbank flow. This is particularly enhanced due to the longitudinal nature of the bedforms. The upper layer flow is generally faster than the lower layer flow, this is opposite to the lower flow depth conditions.

4.4.3 Summary

The results show important differences between flow in the flat bed case and natural bed case. The most significant and important of which is that the velocity and conveyance of a channel with natural bedform is reduced compared to that of a flat bed case. At the bankfull flow depth the discharge capacity is reduced by 30 percent, a very significant proportion of the flow. This has important significance to prediction methods, particularly those derived from and based on research in channel with idealised cross-sections, i.e. rectangular channels. The sediment transport increases as the inbank flow depth increases up to the bankfull level, but then a decrease in the sediment transport rate was found as the overbank flow reaches relative depths around 0.2-0.3 before increasing again at the deeper overbank flows.

The main features in the channel are acceleration of the flow along the channel, secondary currents and cross flow from the floodplain. More specific features of the flow include:

- At the apex section the floodplain flow reduces the velocity at the edges of the main channel at low flow depths. As the flow depth increases the floodplain flow increases the velocity of the flow in the main channel.

- The secondary flows were found to be different in the inbank and overbank flow cases. There are three mechanisms that cause the secondary flows; centrifugal force, flow entering into the main channel from the floodplain and reverse flows as the flow passes over ridges in the natural bed case.

- As the flow depth increases the cross-flow from the floodplain along the cross-over section increases, becoming more dominant across the channel.
• In addition to the increase in the transverse flow the vertical velocities also increase indicating that the strength of the secondary circulations have increased, not just the cross-flow.

• In the overbank flow cases below relative depths of 0.3 the secondary circulation caused by the floodplain flow is similar to those found by many researchers as discussed in Section 2.3.3.

• At the 0.45 relative depth there are several circulations across the apex section in both the flat bed and natural bed cases. The 0.45 depth condition was particularly interesting in the natural bed case due to the bedforms in the main channel. The ridges along the channel, particularly the ridges extending directly across the crossover section are a feature of bedforms that have not been shown before. At the crossover section the circulation flows directly across the main channel from the upstream floodplain onto the downstream floodplain. The floodplain flow drives series of vortices, which separate from the upstream bank. There are also smaller reverse circulations where the flow crosses over the ridges. Around the bend these small reverse flows increase and displace the main circulations, which originated from upstream. The main characteristics of the flow at the 0.45 depth condition in the natural bed case are summarised in Figure 4.59.

• The layer-averaged velocities have shown that flows in the lower-layer tend to be closely aligned to the main channel and the upper-layer flows more closely aligned with the floodplain, but as the overbank flow increases both the upper and lower-layer flows become more parallel with the floodplain.

• The average velocity profiles across the channel have also been shown to be less uniform in the natural bed case than the flat bed case. This is due to the non-uniform nature of the bedforms and the more complex flow structures.

• The floodplain flow pattern in the flat bed and natural bed cases are almost identical, with the fastest flow occurring outside the meander belt. The flow within the meander belt width flows towards the inside of the bend, although this angle reduces as the flow depth increases.
Figure 4.1: Stage-discharge relationship for the natural and flat bed cases

Figure 4.2: Comparison of the natural bed discharge to the flat bed discharge
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Figure 4.3: Sectional average velocities

Figure 4.4: Percentages of sectional discharges

Figure 4.5: Proportion of flow below and overbank
Figure 4.6: Sediment transport rate plotted at different scales
a) 0-2.5 grms/s b) 0-0.1 grms/s
Figure 4.7: Longitudinal velocity $U/U_e$ for the flat bed case, DR=0.0
Figure 4.8: Longitudinal velocity $U/U_s$ for the flat bed case, $DR=0.2$
Figure 4.9: Longitudinal velocity $U/U_0$ for the flat bed case, DR=0.3
Figure 4.10: Longitudinal velocity $U/U_c$ for the flat bed case, DR=0.45
Figure 4.11: Transverse velocity $V/U_s$ for the flat bed case, $DR=0.0$
Figure 4.12: Transverse velocity V/\textit{U}_s for flat bed case, DR=0.2
Figure 4.13: Transverse velocity $V/U_0$ for flat bed case, $DR=0.3$
Figure 4.14: Transverse velocity $V/U_s$ for flat bed case, DR=0.45
Figure 4.15: Vertical velocity $W/U_s$ for flat bed case, $DR=0.0$
Figure 4.16: Vertical velocity $W/U_s$ for flat bed case, DR=0.2
Figure 4.17: Vertical velocity \( W/U_0 \) for flat bed case, DR=0.3
Figure 4.18: Vertical velocity $W/U_0$ for flat bed case, DR=0.45
Figure 4.19: Secondary currents for the flat bed case, DR=0.0
Figure 4.20: Secondary currents for the flat bed case, DR=0.2
Figure 4.21: Secondary currents for the flat bed case

DR=0.3
Figure 4.22: Secondary currents for the flat bed case, DR=0.45
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Growth of reverse flow into large circulation, causes reduction in the main circulation.

Series of vortices driven by floodplain flow separate from main channel bank.

Flow across ridges into neighbouring gulley, with reverse vortices.

Transfer of main channel flow onto the floodplain along downstream edge.

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Chapter 5 – Proposed method

In order to fulfil the aim of the thesis the data described in Chapter 4 has been used to derive a new method for predicting velocity and discharge in compound meandering channels. Application of the derived method is described together with application to previous data from the FCF and Muto (1997).

5.1 Background

Shiono and Knight (1989) developed a method to predict depth-averaged velocity in straight compound channels using the equations of motion and continuity. They combined the streamwise momentum on a fluid element with continuity for steady uniform flow, to give:

\[
\rho \left[ \frac{\partial(\bar{U}V)}{\partial y} + \frac{\partial(\bar{U}W)}{\partial z} \right] = \rho g S_0 + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} \tag{5.1}
\]

where \(\bar{U}, \bar{V}\) and \(\bar{W}\) are longitudinal, transverse and vertical mean velocities components relative to the main channel in the \(x, y\) and \(z\) direction, \(g\) is gravitational acceleration, \(S_0\) is the bed slope and \(\tau\) are shear stresses in the specified planes.

Using the eddy-viscosity approach and ignoring secondary currents, the depth-averaged velocity equation was obtained:

\[
\rho g H S_0 \cdot \frac{1}{8} f \bar{U}_d^2 \left(1 + \frac{1}{s^2}\right)^{\frac{1}{2}} + \frac{\partial}{\partial y} \left( \rho \lambda H^2 \left(\frac{f}{8}\right)^{\frac{1}{2}} \bar{U}_d \frac{\partial \bar{U}_d}{\partial y} \right) = 0 \tag{5.2}
\]

where \(H\) is water depth, \(f\) is Darcy-Weisbach friction factor, \(\bar{U}_d\) is depth-averaged streamwise velocity, \(s\) is main channel lateral side slope and \(\lambda\) is dimensionless eddy-viscosity.

Later, Shiono and Knight (1990, 1991) found that secondary currents, particularly in compound channels, are a significant mechanism within the flow and cannot therefore be ignored. A secondary flow term, \(\partial(\bar{U}\bar{V})/\partial y\), was introduced to equation (5.2) and the equation becomes:

\[
\rho g H S_0 \cdot \frac{1}{8} f \bar{U}_d^2 \left(1 + \frac{1}{s^2}\right)^{\frac{1}{2}} + \frac{\partial}{\partial y} \left( \rho \lambda H^2 \left(\frac{f}{8}\right)^{\frac{1}{2}} \bar{U}_d \frac{\partial \bar{U}_d}{\partial y} \right) = 0 \tag{5.2}
\]
Ervine et al. (2000) applied equation (5.3) to meandering channels by introducing a secondary flow coefficient, $K$, to represent the secondary flow term, $\overline{UV}$. They assumed that $\overline{U}$, $\overline{V}$ were both proportional to the depth-averaged velocity, so $\overline{UV} = K\overline{U}^2$. Equation (5.3) becomes:

$$\rho g HS_0 - \frac{1}{8} \rho f \overline{U}^2 \left(1 + \frac{1}{s^2}\right)^{1/2} + \frac{\partial}{\partial y} \left\{ \rho \lambda H^2 \left(\frac{f}{8}\right)^{1/2} \overline{U} \frac{\partial \overline{U}}{\partial y} \right\} = \frac{\partial}{\partial y} \left\{ H (\rho \overline{UV})_d \right\} \quad (5.4)$$

Equation (5.4) is derived from straight channel theory, but applying this equation to meandering channels is fundamentally incorrect. Using the concepts of the methods and theory from Shiono and Knight (1989, 1990, 1991) an alternative method for meandering channels is derived in the following section.

### 5.2 Developed Theory

The proposed method is based on the Navier-Stokes equations with curvilinear co-ordinates. The two-dimensional curvilinear equations were originally derived by Tollmien (1931). The derived equations can be found in Schlichtling (1968). With curvilinear co-ordinates the $x$-axis is parallel to the meandering channel and the $y$-axis is normal to the meandering channel. The orientation is similar to that shown in Figure 5.1, which shows the orientation for flow around a two-dimensional boundary layer along a curved wall. The equation of motion in the streamwise direction and the continuity equation for laminar flow are shown below:

$$\frac{\partial U}{\partial t} + \frac{r_m}{r_m + y_m} U \frac{\partial U}{\partial x} + V \frac{\partial U}{\partial y} + W \frac{\partial U}{\partial z} + \frac{UV}{r_m + y_m} =$$

$$- \frac{r_m}{r_m + y_m} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms} \quad (5.5)$$
where $U$, $V$ and $W$ are the instantaneous velocity components relative to the main channel in the $x$, $y$ and $z$ direction, $t$ is time, $r_i$ is the radius of curvature to the inside of the bend, $y_i$ is the distance across the channel from the inside of the bend, $\rho$ is the density of water and $\partial P/\partial x_h$ is the change in the hydrostatic pressure along the channel and $\partial x_h$ is the displacement in the horizontal plane.

For steady flow $\frac{\partial U}{\partial t} = 0$, equation (5.5) can be rearranged to:

\[
\frac{r_i}{r_i + y_i} \left( \frac{\partial U^2}{\partial x} - U \frac{\partial U}{\partial x} \right) + \left( \frac{\partial UV}{\partial y} - U \frac{\partial V}{\partial y} \right) + \left( \frac{\partial UW}{\partial z} - U \frac{\partial W}{\partial z} \right) + \frac{UV}{r_i + y_i}
\]

\[
= - \frac{r_i}{r_i + y_i} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms}
\]

Factorising the $U$ terms:

\[
\frac{r_i}{r_i + y_i} \left( \frac{\partial U^2}{\partial x} + \frac{\partial UV}{\partial y} + \frac{\partial UW}{\partial z} - U \left( \frac{\partial U}{\partial x} + \frac{\partial V}{\partial y} + \frac{\partial W}{\partial z} \right) + \frac{UV}{r_i + y_i} \right)
\]

\[
= - \frac{r_i}{r_i + y_i} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms}
\]

\[
\frac{r_i}{r_i + y_i} \left( \frac{\partial U^2}{\partial x} + \frac{\partial UV}{\partial y} + \frac{\partial UW}{\partial z} - U \left( \frac{\partial U}{\partial x} + \frac{\partial V}{\partial y} + \frac{\partial W}{\partial z} + \frac{V}{r_i + y_i} \right) + \frac{2UV}{r_i + y_i} \right)
\]

\[
= - \frac{r_i}{r_i + y_i} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms}
\]

From continuity equation, defined in equation (5.6), the equation becomes:
\[
\frac{r_{in}}{r_{in} + y_{in}} \frac{\partial U^2}{\partial x} + \frac{\partial UV}{\partial y} + \frac{\partial UW}{\partial z} + \frac{2UV}{r_{in} + y_{in}} = -\frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms} \quad (5.7)
\]

For turbulent flow, introduce turbulent fluctuations into equation (5.7), i.e. \( U = \overline{U} + u' \), \( V = \overline{V} + v' \) and \( W = \overline{W} + w' \) where the overbar signifies a time-averaged value, and \( u' \), \( v' \) and \( w' \) are fluctuations around the mean value. Equation (5.7) becomes:

\[
\frac{r_{in}}{r_{in} + y_{in}} \frac{\partial (\overline{U} + u') (\overline{U} + u')}{\partial x} + \frac{\partial (\overline{U} + u') (\overline{V} + v')}{\partial y} + \frac{\partial (\overline{U} + u') (\overline{W} + w')}{\partial z} + \frac{2(\overline{U} + u') (\overline{V} + v')}{r_{in} + y_{in}}
\]

\[
= -\frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms}
\]

Time-averaged results mean \( \overline{UU'} = 0, \overline{UV'} = 0, \overline{Vu} = 0, \overline{Uw'} = 0 \) and \( \overline{Wu'} = 0 \). Therefore:

\[
\frac{r_{in}}{r_{in} + y_{in}} \frac{\partial (\overline{UU} + uu')}{\partial x} + \frac{\partial (\overline{UV} + u'u')}{\partial y} + \frac{\partial (\overline{UW} + u'w')}{\partial z} + \frac{2(\overline{UV} + u'v')}{r_{in} + y_{in}}
\]

\[
= -\frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms} \quad (5.8)
\]

Separating equation (5.8) into velocity and turbulence terms, gives:

\[
\frac{r_{in}}{r_{in} + y_{in}} \frac{\partial U^2}{\partial x} + \frac{\partial UV}{\partial y} + \frac{\partial UW}{\partial z} + \frac{2UV}{r_{in} + y_{in}} \quad \text{(velocity terms)}
\]

\[
\frac{r_{in}}{r_{in} + y_{in}} \frac{\partial u'^2}{\partial x} + \frac{\partial uu'}{\partial y} + \frac{\partial uu'}{\partial z} + \frac{2uu'}{r_{in} + y_{in}} \quad \text{(Reynolds stresses)}
\]

\[
= -\frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial P}{\partial x_h} + \text{viscous terms}
\]

Moving the turbulence terms across to the right-hand side:
Replacing the turbulence terms with equivalent shear stress terms:

\[ \overline{u'^2} = -\frac{1}{\rho} \tau_{xx} ; \quad \overline{v'u'} = -\frac{1}{\rho} \tau_{xy} \quad \text{and} \quad \overline{w'u'} = -\frac{1}{\rho} \tau_{xz} \]

The right-hand side of the equation becomes:

\[ = -\frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial P}{\partial x_h} - \frac{r_{in}}{r_{in} + y_{in}} \frac{1}{\rho} \frac{\partial \tau_{xx}}{\partial x} - \frac{1}{\rho} \frac{\partial \tau_{xy}}{\partial y} - \frac{1}{\rho} \frac{\partial \tau_{xz}}{\partial z} + \frac{2}{\rho} \frac{\tau_{xy}}{r_{in} + y_{in}} \]

Using the following results to depth-average the left-hand side:

\[ \frac{1}{H} \int_0^H \overline{U} \, dz = \overline{U} \quad ; \quad \frac{1}{H} \int_0^H \overline{UV} \, dz = \overline{(UV)} \quad \text{and} \quad \frac{1}{H} \int_0^H \overline{\frac{\partial \overline{UW}}{\partial z}} \, dz = \overline{[\overline{UW}']_0} = 0 \]

And these results to depth-average the right-hand side:

\[ \frac{1}{H} \int_0^H \overline{\tau_{xx}} \, dz = \overline{(\tau_{xx})} \quad ; \quad \frac{1}{H} \int_0^H \overline{\tau_{xy}} \, dz = \overline{(\tau_{xy})} \quad ; \quad \frac{1}{H} \int_0^H \overline{\tau_{xz}} \, dz = \frac{1}{H} \overline{[\tau_{xz}]}_0 \quad = \frac{1}{H} (\tau_0 - \tau_b) = -\frac{\tau_b}{H} \]

Assuming the hydrostatic pressure, \( P = \rho g H \), and \( S_o = -\frac{\partial H}{\partial x_h} \), gives:

\[ \frac{\partial P}{\partial x_h} = \rho g \frac{\partial H}{\partial x_h} = -\rho g S_o \]

Using the above results and multiplying by density, \( \rho \), and depth, \( H \), the depth-averaged equation is:
Chapter 5 – Proposed method

5.2.1 Velocity terms

Introducing a $K$ coefficient similar to that introduced by Ervine et al. (2000), so that

\[
\overline{UV} = K \overline{U_d^2},
\]

equation (5.9) becomes:

\[
\rho H \left[ \frac{r_m + y_m}{r_m + y_m} \frac{\partial \overline{U_d^2}}{\partial x} + \frac{\partial \overline{U_d^2}}{\partial y} \right] = \frac{r_m}{r_m + y_m} \rho g H S_0 + \text{stress terms} (5.10)
\]

Dividing by \( \frac{r_m}{r_m + y_m} \), gives:

\[
\rho H \left[ \frac{\partial \overline{U_d^2}}{\partial x} + \frac{r_m + y_m}{r_m} \frac{\partial \overline{U_d^2}}{\partial y} + 2K \overline{U_d^2} \right] = \rho g H S_0 + \text{stress terms} (5.10)
\]

In order to be able to solve the equation, coefficients are introduced to the left-hand side of equation (5.10) to make a simple expression with only \( \overline{U_d^2} \), such as:

\[
\frac{\partial \overline{U_d^2}}{\partial x} = K' \overline{U_d^2} \quad \text{and} \quad \frac{\partial \overline{U_d^2}}{\partial y} = K'' \overline{U_d^2}
\]

Equation (5.10) becomes:

\[
\rho K' H \overline{U_d^2} = \rho g H S_0 + \text{turbulence terms} (5.11)
\]

Where:
\[ K_s = \left[ K' + \frac{r_i + y_i}{r_i} K'' + \frac{2K}{r_i} \right] \]  

(5.12)

\( K_s \) is not a constant, it is dependent upon the radius of curvature, and so it varies across the width of the channel. To aid description in later Sections the \( K' \) term is called an acceleration term, \( \frac{r_i + y_i}{r_i} K'' \) is called the secondary flow term and \( \frac{2K}{r_i} \) the centrifugal term as it would disappear in straight channels as \( r_i \) becomes infinite. Prior to investigating \( K_s \) using measured data the turbulence terms in the right-hand side of equation (5.11) are derived.

### 5.2.2 Turbulence terms

Solution of the pressure and turbulence components of equation (5.9):

velocity terms =

\[ \frac{r_i}{r_i + y_i} \rho g S_0 + \frac{r_i}{r_i + y_i} H \frac{\partial (\tau_{xx})_d}{\partial x} + H \frac{\partial (\tau_{xy})_d}{\partial y} - \tau_b + H \frac{2(\tau_{xy})_d}{r_i + y_i} \]  

As with the velocity terms, dividing by \( \frac{r_i}{r_i + y_i} \), gives:

velocity terms = \( \rho g S_0 + H \frac{\partial (\tau_{xx})_d}{\partial x} + \frac{r_i + y_i}{r_i} H \frac{\partial (\tau_{xy})_d}{\partial y} - \frac{r_i + y_i}{r_i} \tau_b + H \frac{2(\tau_{xy})_d}{r_i} \)

Using the eddy-viscosity approach from Shiono and Knight (1989), where the transverse shear stress, \( \tau_{xy} \), is expressed in terms of the transverse depth-averaged velocity gradient:

\[ \tau_{xy} = \rho \varepsilon_{xy} \frac{\partial U_d}{\partial y} \quad \text{and} \quad \varepsilon_{xy} = \lambda U^* = \lambda H \left( \frac{f}{8} \right)^{1/2} U_d \]

where \( \varepsilon_{xy} \) is the eddy viscosity and \( U^* \) is the shear velocity.

Therefore:
\( \tau_{xy} = \rho \lambda H \left( \frac{f}{8} \right)^{1/2} \bar{U}_d \frac{\partial \bar{U}_d}{\partial y} = \rho \lambda H \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} \frac{\partial \bar{U}_d^2}{\partial y} \)

In addition:

\( \frac{\partial (\tau_{ix})_d}{\partial x} \ll 1 \) and is therefore ignored and \( \tau_b = \frac{1}{8} \rho \bar{U}_d^2 \)

Substituting into (5.13) and substituting the same coefficients as for the velocity derivation to give a simple expression with only \( \bar{U}_d^2 \) terms;

velocity terms =

\[
\rho g H S_0 + \frac{r_m + y_m}{r_m} \frac{\partial}{\partial y} \left( \rho \lambda H^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} K'' \bar{U}_d \right) - \frac{r_m + y_m}{r_m} \frac{1}{8} \rho \bar{U}_d^2 + \frac{2}{r_m} \left( \rho \lambda H^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} K'' \bar{U}_d \right)
\]

(5.14)

5.2.3 Numerical solution

Combining the velocity equation (5.11) and the turbulence equation (5.14) the equation of motion in the streamwise direction is;

\[
0 = g H S_0 - \frac{r_m + y_m}{r_m} \frac{f}{8} \bar{U}_d^2 - H K_s \bar{U}_d^2 + \frac{\partial}{\partial y} \left( \frac{r_m + y_m}{r_m} \lambda H^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} K'' \bar{U}_d \right) + \frac{2}{r_m} \left( \lambda H^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} K'' \bar{U}_d \right)
\]

(5.15)

Equation (5.15) can be solved numerically for two-dimensional turbulent flow in meandering channels when \( K_s, \lambda \) and \( f \) are set. The depth, \( H \), is assumed to be constant across the channel.
An investigation of $K_a$ is carried out using the measured data. $K_a$ is the only unknown and can be calculated from equation (5.15) as all the other variables can be calculated from the measured data with given $\lambda(=0.07)$. The friction values across the channel were calculated from the measured bed shear stress using equation (5.16). The method was applied to the apex section as it is the most common location for prediction methods to be applied.

$$f = \frac{8r_b}{\rho U_a^2}$$

(5.16)

The $K_a$ term contains three main components $K$, $K'$ and $K''$, as shown in equation (5.12). $K$ and $K''$ can be calculated directly from the measured velocity data at the apex. $K'$ cannot, because $\partial \bar{U}_a^2/\partial x$ cannot be calculated from the velocity data across the apex section. So having calculated $K_a$ from equation (5.15) $K'$ can be calculated from equation (5.12). The components of $K_a$ are shown for each flow condition in the flat bed case in Figure 5.2 to Figure 5.5. The figures of the components give an indication of the flow structure at the apex section. It is clear that in all the flow conditions $K_a$ increases across the channel as $y$ increases. $K_a$ is larger in the shallow overbank case than for the inbank flow, but then decreases as the overbank depth increases. This is consistent with the two regimes of flow behaviour described in Rameshwaran (1997).

- Regime 1 is at shallow relative flow depths, in which non-bed friction losses increase with relative flow depth.
- Regime 2 involves higher relative flow depths, in which non-bed friction losses decrease with relative flow depth.

The figures also indicate the relative significance of the components of $K_a$ i.e. the acceleration term, secondary flow term and centrifugal term. At each of the depth conditions the largest component is the secondary flow term. In the two lower flow conditions, $DR=0.0$ and $DR=0.2$, the second largest component is the acceleration term and the smallest is the centrifugal term. In the two deeper flow conditions, $DR=0.3$ and $DR=0.45$, the centrifugal term is more significant than the acceleration term.
As \( K' \) was the only value not calculated directly from measured data at the apex it was compared to the value of \( K' (=dU'/dx) \) calculated from Sections e11 and f1, equivalent to just upstream of the apex. The two sets of \( K' \) values are shown in Figure 5.6 to Figure 5.9. The figures are not expected to show that the values match exactly as they are at different sections along the flume, but are used just to indicate that the \( K' \) values calculated at the apex are justifiable and of the correct order of magnitude. Examining the four figures it is clear that the values at the apex section (f1) are of a similar magnitude and have similar trends to the e11-f1 values, i.e. the values are negative near the inside of the bend and become positive a short distance inside the channel. The values of \( K' \) can therefore be considered reasonable, as can \( K_a \) in the numerical solution.

The figures of the terms in equation (5.15) are shown in Figure 5.10 to Figure 5.13. The figures show the relative significance of each of the terms. As the flow depth increases the hydrostatic pressure term \((gHS)\) increases, as do the magnitude and significance of the \( HK_aU_d^2 \) term. The increase in the \( HK_aU_d^2 \) term is due to the increase in the flow depth and velocity, as \( K_a \) remains similar between the 0.2 and 0.3 relative depth conditions and decreases between the 0.3 and 0.45 relative depth conditions. In the bankfull case (Figure 5.10) the bed shear stress term is more significant than the \( HK_aU_d^2 \) term, but as the flow depth increases the bed shear stress term significance reduces. It is also of interest to note that in all the cases the turbulence terms (shown below) are very small and insignificant.

\[
\frac{\partial}{\partial y} \left( \frac{r_m + y_m}{r_m} \lambda h^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} \frac{\partial U_d^2}{\partial y} \right) \quad \text{and} \quad \frac{2}{r_m} \left( \lambda h^2 \left( \frac{f}{8} \right)^{1/2} \frac{1}{2} \frac{\partial U_d^2}{\partial y} \right)
\]

As a result the turbulence terms at the apex could be ignored or eliminated from equation (5.15) without having a great effect on the overall results.

5.2.4 Analytical solution

As previously shown in Section 5.2.3 the turbulence terms in the apex in equation (5.15) are insignificant and can be ignored. Equation (5.15) therefore becomes simpler:
Rearranging equation (5.17), the equation for depth-averaged velocity in meandering channels becomes:

\[
\overline{U_d} = \sqrt{\frac{gHs}{HK_a + \frac{r_m + y_m}{r_m} \left(\frac{f}{8}\right)}}
\]  

(5.18)

This is the result for depth-averaged velocity in meandering channels where \( y, f \) and \( K_a \) vary across the width of the channel. The depth, \( H \), is assumed to be constant across the channel. The variable \( y_m \) is part of the geometry of the channel and is known, so in order to make the application of the method easier, the use of constant variables across the channel for \( f \) and \( K_a \) is investigated in the next section.

The average of the measured friction values and measured \( K_a \) values were used to calculate the depth-averaged velocity across the channel using equation (5.18). The average of measured \( K_a \) values with measured \( f \) values were also used for a similar calculation of the velocity across the channel. The two sets of calculated velocities are shown in Figure 5.14 and Figure 5.15 respectively and compared to the velocity calculated using the measured friction and \( K_a \) values. The figures show that neither the velocity profile using a constant friction factor or constant \( K_a \) match the velocity profile with both measured values exactly. However, the velocity profile using a constant friction factor is reasonable, indicating that the variation of \( K_a \) across the channel is particularly significant.

5.2.5 Depth-averaged velocity profile prediction

In order to predict the depth-averaged velocity across the channel, equations could be fitted for the variation of the friction and \( K_a \). However, as the variation of \( K_a \) is more significant and to simplify the method a constant friction factor is first assumed and the distribution of \( K_a \) fitted to give an accurate velocity profile. The normalised \( K_a \) values (\( K_a/K_a(\text{mean}) \), where \( K_a(\text{mean}) \) is the average value across the section) are shown in Figure 5.16. There are two distinct normalised \( K_a \) profiles across the channel; the bankfull and \( DR=0.2 \) cases, and the two deeper cases (\( DR=0.3 \) and \( DR=0.45 \)). The two groups
were used to calculate a normalised $K_a$ profile across the channel proportional to the width of the channel. These are shown in Figure 5.17 and Figure 5.18 respectively and the equations are given below:

For $0.0 < DR \leq 0.2$

$$K_{a\text{ (normalised)}} = -7.8689 \left(\frac{y_v}{b}\right)^3 + 19.32 \left(\frac{y_v}{b}\right)^2 - 8.9134 \left(\frac{y_v}{b}\right) + 0.962$$  \hspace{1cm} (5.19)

For $DR > 0.2$

$$K_{a\text{ (normalised)}} = -2.6277 \left(\frac{y_v}{b}\right)^3 + 4.0249 \left(\frac{y_v}{b}\right)^2 - 0.9003 \left(\frac{y_v}{b}\right) + 0.7746$$  \hspace{1cm} (5.20)

$$K_a = K_{a\text{ (mean)}} \times K_{a\text{ (normalised)}}$$  \hspace{1cm} (5.21)

The velocity in the main channel of each depth condition was calculated using the constant $f$ and the $K_a$ profiles from equations (5.19) and (5.20) and $K_{a\text{ (mean)}}$. The calculated velocity in the bankfull and $DR=0.2$ cases are shown in Figure 5.19, the two deeper cases are shown in Figure 5.20. As these figures show, all the cases give good approximations to the actual velocities, and the predicted sectional average velocities are generally within ±5 percent and only ±1.5 percent at higher flow conditions, as shown in Table 5.1. Using a constant friction factor therefore gives good enough results.

<table>
<thead>
<tr>
<th>Case</th>
<th>Measured average velocity (m/s)</th>
<th>Predicted average velocity (m/s)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>g4_3</td>
<td>0.2878</td>
<td>0.2817</td>
<td>-2.13</td>
</tr>
<tr>
<td>g4_5</td>
<td>0.2352</td>
<td>0.2464</td>
<td>4.79</td>
</tr>
<tr>
<td>g4_7</td>
<td>0.2247</td>
<td>0.2214</td>
<td>-1.48</td>
</tr>
<tr>
<td>g4_10</td>
<td>0.2982</td>
<td>0.2986</td>
<td>0.14</td>
</tr>
</tbody>
</table>

### 5.2.6 Discharge prediction

Using $K_{a\text{ (mean)}}$ values, equation (5.18) can also be used to calculate an average velocity and therefore discharge ($Q=VA$) in each section of the channel. Applying this to both the main channel and the floodplain the total discharge for the whole channel can be calculated. The $K_{a\text{ (mean)}}$ values in the main channel and on the floodplain are shown in
Figure 5.21. The discharges in the main channel and on the floodplain calculated using the $K_a$(mean) values are shown in Table 5.2 and Table 5.3 respectively. The tables also show the sectional discharges calculated from the velocity data and the comparative error of the predicted discharge. The discharges within the main channel are within 6.5 percent and on the floodplain the errors reduce to within 3.5 percent. The overall discharges are shown in Figure 5.22 and listed in Table 5.4. They show that the prediction method gives excellent agreement with the measured data. Combined, the error is 6.29 percent for the bankfull case and less in the overbank flow cases, within ±1.7 percent.

Table 5.2: Predicted discharges and errors of the flow in the main channel

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth ratio (DR)</th>
<th>Predicted discharge (m³/s)</th>
<th>Measured discharge (m³/s)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$g_4$ 3</td>
<td>0.00</td>
<td>0.00431</td>
<td>0.00459</td>
<td>6.29</td>
</tr>
<tr>
<td>$g_4$ 5</td>
<td>0.20</td>
<td>0.00414</td>
<td>0.00435</td>
<td>4.98</td>
</tr>
<tr>
<td>$g_4$ 7</td>
<td>0.30</td>
<td>0.00500</td>
<td>0.00510</td>
<td>1.80</td>
</tr>
<tr>
<td>$g_4$ 10</td>
<td>0.45</td>
<td>0.00880</td>
<td>0.00888</td>
<td>0.91</td>
</tr>
</tbody>
</table>

Table 5.3: Predicted discharges and errors of the flow on the floodplain

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth ratio (DR)</th>
<th>Predicted discharge (m³/s)</th>
<th>Measured discharge (m³/s)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$g_4$ 5</td>
<td>0.20</td>
<td>0.00293</td>
<td>0.00283</td>
<td>-3.47</td>
</tr>
<tr>
<td>$g_4$ 7</td>
<td>0.30</td>
<td>0.00896</td>
<td>0.00869</td>
<td>-3.10</td>
</tr>
<tr>
<td>$g_4$ 10</td>
<td>0.45</td>
<td>0.02296</td>
<td>0.02294</td>
<td>-0.10</td>
</tr>
</tbody>
</table>

Table 5.4: Predicted discharges and errors of the total flow

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth ratio (DR)</th>
<th>Predicted discharge (m³/s)</th>
<th>Measured discharge (m³/s)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$g_4$ 3</td>
<td>0.00</td>
<td>0.00431</td>
<td>0.00459</td>
<td>6.29</td>
</tr>
<tr>
<td>$g_4$ 5</td>
<td>0.20</td>
<td>0.01968</td>
<td>0.00718</td>
<td>1.65</td>
</tr>
<tr>
<td>$g_4$ 7</td>
<td>0.30</td>
<td>0.03176</td>
<td>0.01378</td>
<td>-1.29</td>
</tr>
<tr>
<td>$g_4$ 10</td>
<td>0.45</td>
<td>0.07041</td>
<td>0.03182</td>
<td>0.19</td>
</tr>
</tbody>
</table>
It was noticed in Figure 5.21 that for the deeper flow cases, where the floodplain flow forms a significant proportion of the flow, the floodplain \( K_a_{(mean)} \) values are similar to the main channel values. The method was therefore tested using the main channel \( K_a \) values on the floodplain as being able to use a single value for the whole channel will make the method much easier to apply. At DR=0.2 Figure 5.21 shows that the floodplain \( K_a_{(mean)} \) value is larger than the main channel. However, because the floodplain flow is less significant at the lower flow depths assuming a \( K_a_{(mean)} \) value on the floodplain similar to the main channel should still give reasonable results. The calculated discharges from assuming separate and single \( K_a_{(mean)} \) values are shown in Figure 5.23. From this figure it is clear that assuming a single value for both the floodplain and main channel gives similar results to separate \( K_a_{(mean)} \) values, except at the 0.2 relative depth condition which slightly overpredicts the discharge as expected since the used \( K_a_{(mean)} \) is smaller.

In order to apply the method to further flow depths, an equation was fitted to the main channel mean \( K_a \) values. The fitted equation is shown in Figure 5.24. To test the equations they were applied to the stage-discharge data measured from the flow meters and depth gauges over the full range of overbank depths. The calculated discharges and actual discharges are both shown in Figure 5.25, with the errors shown in Figure 5.26. The errors vary from \(-5\) to \(-25\) percent, but overall the results are reasonable, especially at higher flow depths.

### 5.2.7 Friction factor

Having fitted an equation for \( K_a \) the remaining unknown with the method is how to calculate the average friction factor. The friction factor used in the calculations was calculated from the measured bed shear stress using equation (5.16). As Manning's coefficient, \( n \), is popularly used by engineers the average friction factor, \( f \), was used to calculate a local Manning's \( n \) coefficient using equation (5.22) below. As it is a local friction factor the hydraulic radius, \( R \), should be replaced with the water depth, \( H \). The Manning's \( n \) values are shown in Table 5.5.

\[
n = R^{\frac{1}{6}} \frac{f}{\sqrt{8g}} \tag{5.22}
\]
Table 5.5: Friction factor values

<table>
<thead>
<tr>
<th>Case</th>
<th>Darcy-Weisbach's f</th>
<th>Manning's n</th>
</tr>
</thead>
<tbody>
<tr>
<td>g4_3</td>
<td>0.0325</td>
<td>0.0119</td>
</tr>
<tr>
<td>g4_5</td>
<td>0.0427</td>
<td>0.0142</td>
</tr>
<tr>
<td>g4_7</td>
<td>0.0322</td>
<td>0.0126</td>
</tr>
<tr>
<td>g4_10</td>
<td>0.0236</td>
<td>0.0112</td>
</tr>
</tbody>
</table>

The Manning's values are similar to those quoted for Mortar in Chadwick and Morfett (1993) i.e. 0.011-0.013. So if the bed shear stress is unknown, a suitable Manning's \( n \) value can be estimated, and \( f \) calculated using equation (5.22). Equation (5.22) can be substituted into equation (5.18) so the Manning's \( n \) value can be used directly, as shown in equation (5.23).

\[
\bar{U}_d = \frac{gHS}{\sqrt{HK_a + \frac{r_{in} + y_{in}}{r_{in}} \left( \frac{gn^2}{H^{3/2}} \right)}}
\]  

(5.23)

5.2.8 Natural bed case

Having been verified on the flat bed case the method was applied to the natural bed case. The bed shear stress was not measured in the natural bed case due to difficulties with the method caused by the bedforms, but the bedforms in the main channel increase the roughness of the bed compared to the flat bed case. Consequently there are two unknowns in equation (5.18). Therefore to be able to calculate the effect of the natural bed \( K_a \) was assumed to be the same as the flat bed case and an additional friction factor, \( f' \), is introduced to vary the total friction so the predicted average velocity was equal to the actual average velocity, as shown in equation (5.24). This was to show the effect of the bedform in the main channel.

\[
\bar{U}_d = \frac{gHS}{\sqrt{HK_a + \frac{r_{in} + y_{in}}{r_{in}} \left( \frac{f + f'}{8} \right)}}
\]  

(5.24)
The predicted velocities for all of the depth conditions are shown in Figure 5.27 to Figure 5.30. The figures show clear differences in the velocity profiles across the channel, implying that the bedforms alter the flow structure. This is unsurprising as the flow structure between the two cases was shown to be different in Chapter 4. The $K_a$ distribution assumptions for the flat bed case therefore do not apply to the natural bed case and alternative derivations for the distribution of $K_a$ are required.

### Table 5.6: Additional friction factor for natural bed case

<table>
<thead>
<tr>
<th>Case</th>
<th>Depth condition $DR$</th>
<th>Darcy-Weisbach's $f$</th>
<th>Darcy-Weisbach's $f'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>g2_1</td>
<td>0.0</td>
<td>0.0325</td>
<td>0.038</td>
</tr>
<tr>
<td>g2_3</td>
<td>0.2</td>
<td>0.0427</td>
<td>0.065</td>
</tr>
<tr>
<td>g2_5</td>
<td>0.3</td>
<td>0.0322</td>
<td>0.047</td>
</tr>
<tr>
<td>g2_15</td>
<td>0.45</td>
<td>0.02364</td>
<td>-0.002</td>
</tr>
</tbody>
</table>

The additional friction factors, $f'$, are shown in Table 5.6. They clearly show a large increase in the resistance of the channel, with the values for $f'$ exceeding the $f$ values by 60 percent in the bankfull case and up to 45 percent in the 0.2 and 0.3 overbank depth conditions. At $DR=0.45$, where the discharge in the natural bed and flat bed cases is similar, $f'$ has reduced to zero. This would imply that the roughness of the bedforms in the main channel does not affect the overall flow in the channel, in fact the stage-discharge data in Section 4.1 show that the discharge in both cases are very similar.

Without knowing the bed shear stress in the natural bed case it is not possible to calculate the values of the different terms. As such, the calculations shown here are not strictly correct, they only act to highlight that there are differences between the flat bed and natural bed cases. The natural bed case therefore requires further consideration, which is one of the recommendations for further work in Section 6.1.

### 5.3 Further validation

The method is applied to data from the Flood Channel Facility at Wallingford and from Muto (1997). These represent larger and smaller scale flumes with similar sinuosities to the Loughborough flume i.e. $\approx 1.37$. 
5.3.1 Application to FCF data

Details of the flat bed case from the FCF data used to here for validation can be found in Ervine et al. (1993) and Sellin et al. (1993). The predicted velocity using the equations for $K_a$ from the Loughborough flume are shown in Figure 5.31. The figure shows the measured velocities, the predicted velocities using $K_a$ (mean) calculated from the Loughborough flume and the predicted velocities using a $K_a$ value calculated directly from the FCF cross-sectional data. The figure clearly shows that using the $K_a$ (mean) value from the Loughborough flume greatly underpredicts the velocity across the channel. However, the mean $K_a$ value calculated from the FCF data gives excellent representation of the velocities. The fact that the velocity profiles across the channels are similar between the Loughborough and FCF flumes indicates that although there is a difference in the scale of the flumes the flow structures are similar.

Only one data set of the velocity was available for the FCF flume from which $K_a$ (mean) could be calculated. So $K_a$ (mean) was also calculated from the whole channel characteristics. These values are shown in Figure 5.32. The values calculated from the main channel data and the whole channel are very similar. So if necessary, the $K_a$ (mean) can be calculated for the whole channel and still give good results of velocity and discharge. $K_a$ (mean) was calculated from the whole channel for four other depths, all of these are significantly smaller than the values calculated for the flume from the Loughborough formulae. From examining the formula for $K_a$ (5.12) the reduction in $K_a$ (mean) is a scale effect. In the larger FCF flume $r_{in}$ is larger and $K_a$ therefore reduces.

5.3.2 Application to Bradford data

The method was also applied to data from Muto (1997), who conducted investigations on meandering channels at a smaller scale than the Loughborough flume. In these cases the $K_a$ (mean) values predicted using the Loughborough data were underpredicted, which is consistent with the scale effects shown between the Loughborough flume and the FCF data. These $K_a$ (mean) values fitted to the Muto data are shown in Figure 5.33, along with the $K_a$ (mean) values calculated for the other flumes. The data was applied to the three difference sinuosities at the relative depths of 0.5. The sinuosities are 1.093, 1.370 and 1.571, which had crossover angles of 30°, 60° and 90° respectively. The predictions of the velocities for each case are shown in Figure 5.34, Figure 5.35 and Figure 5.36 with good approximations to the actual velocity.
5.4 Natural channels

Natural channels are typically much larger in scale than the flume on which the data has been tested and verified. Consequently if we were to extend the trend of the $K_a$ term to reduce as the scale increases, as shown in Figure 5.33, then it is possible that in natural channel the value of $K_a$ reduces to become insignificant and only the bed roughness is important. The $K_a$ term could then be removed from equation (5.23) so the prediction for the velocity becomes:

$$\overline{U}_d = \sqrt{\frac{SH^{\frac{3}{2}}}{\left(\frac{r_{in} + y_{in}}{r_{in}}\right)^n}}$$  \hspace{1cm} (5.25)

As $K_a$ has disappeared then the velocity distribution across the channel depends upon the local depth and roughness. It is reassuring to note that when equation (5.25) is applied to straight channels it results in the Mannings equation, as shown in equation (5.26), the most common formula used by river engineers to date.

$$\overline{U}_d = \frac{1}{n} H^{\frac{3}{2}} S^{\frac{1}{2}}$$  \hspace{1cm} (5.26)

5.5 Summary

A new method for the prediction of velocity and discharge in meandering channels has been theoretically derived. Using the data from the Loughborough flume the turbulence terms were found to be insignificant. The method was applied to the flow in the flat bed case at different depths giving good results of predicting the velocity profile across the channel as well as average velocities and discharges. Applying the method to the natural channel case emphasised the different flow structure between the two cases as the velocity profiles across the channel differ.

The method was applied to two additional data sets for further verification. The $K_a$ values were found to vary with scale, increasing as the scale of the flume decreased. This is consistent with the formula for $K_a$ as shown in equation (5.12), where $K_a$ will decrease as $r_{in}$ increases as it does with scale. The velocity profile between the three different scale experiments remained consistent, showing that the flow structure within
the flumes is similar despite the differences between the experiments. Further work is needed to clarify the scale effect in more detail as well as the effect of other parameters, which have not been investigated here. Once completed this further work should provide results which enable the method to be satisfactorily applied to further configurations.

It has been implied that due to the scale effect, the value for $K_a$ in natural rivers will reduce and only the bed friction remains important. A formula for this scenario has been proposed, which when applied to straight channels is the Mannings equation.
Figure 5.1: Two dimensional curvilinear co-ordinate system around a curved wall (from Schlichting 1968)
Figure 5.2: $K_a$ components for DR=0.0

Figure 5.3: $K_a$ components for DR=0.2

Figure 5.4: $K_a$ components for DR=0.3

Figure 5.5: $K_a$ components for DR=0.45
Figure 5.6: Comparison of $K'$ at apex ($f_1$) and $K'$ calculated from $e11-f1$ sections, \( DR=0.0 \)

Figure 5.7: Comparison of $K'$ at apex ($f_1$) and $K'$ calculated from $e11-f1$ sections, \( DR=0.2 \)

Figure 5.8: Comparison of $K'$ at apex ($f_1$) and $K'$ calculated from $e11-f1$ sections, \( DR=0.3 \)

Figure 5.9: Comparison of $K'$ at apex ($f_1$) and $K'$ calculated from $e11-f1$ sections, \( DR=0.45 \)
Figure 5.10: Components of the numerical solution for DR=0.0

Figure 5.11: Components of the numerical solution for DR=0.2

Figure 5.12: Components of the numerical solution for DR=0.3

Figure 5.13: Components of the numerical solution for DR=0.45
Chapter 5 - Proposed method

Figure 5.14: Comparison of the velocity profiles calculated using measured values and a constant friction value, DR=0.0

Figure 5.15: Comparison of the velocity profiles calculated using measured values and a constant $K_a$ value, DR=0.0
Figure 5.16: Normalised $K_a$ values across the apex section of the channel for each of the depth cases
Chapter 5 – Proposed method

Figure 5.17: Normalised $K_a$ profile for the shallower cases (0.0 < DR ≤ 0.2)

\[ y = -7.8689x^3 + 19.32x^2 - 8.9134x + 0.962 \]

Figure 5.18: Normalised $K_a$ profile for the deeper cases (DR > 0.2)

\[ y = -2.6277x^3 + 4.0249x^2 - 0.9003x + 0.7746 \]
Chapter 5 – Proposed method

Figure 5.19: Comparison of the predicted velocities using fitted $K_a$ with the actual velocities in the shallower cases ($0.0 < DR \leq 0.2$)

Figure 5.20: Comparison of the predicted velocities using fitted $K_a$ with the actual velocities in the deeper cases ($DR > 0.2$)
Figure 5.21: $K_a$ (mean) values in the main channel (mc) and on the floodplain (fp)

Figure 5.22: Comparison of the predicted total discharge with the actual total discharge
Figure 5.23: Comparison of the predicted total discharge using the fitted mean $K_a$ values with the actual total discharge.

Figure 5.24: Fitted values for main channel $K_a$. 

\[ y = -0.3813x + 0.2899 \]
Figure 5.25: Measured and predicted discharges for the overbank cases

Figure 5.26: Error between the measured and predicted discharges for the overbank cases
Chapter 5 - Proposed method

Figure 5.27: Predicted velocity for natural bed case, DR=0.0

Figure 5.28: Predicted velocity for natural bed case, DR=0.2

Figure 5.29: Predicted velocity for natural bed case, DR=0.3

Figure 5.30: Predicted velocity for natural bed case, DR=0.45
Chapter 5 – Proposed method

Figure 5.31: Predicted velocities in the FCF data

Figure 5.32: Calculated $K_a$ values for the FCF flume
Chapter 5 – Proposed method

Figure 5.33: $K_a(\text{mean})$ values for different flumes

Figure 5.34: Bradford data, $30^\circ$ crossover (SIN=1.093), DR=0.5

Figure 5.35: Bradford data, $60^\circ$ crossover (SIN=1.37), DR=0.5
Figure 5.36: Bradford data, 60° crossover (SIN=1.57), DR=0.5
6 Conclusions

An experimental programme was conducted on a meandering channel built in a 13m long, 2.4m wide flume. Measurements taken include stage-discharge, sediment transport, bedforms, velocity, turbulence and boundary shear stress. Two cases were examined with flat bed and natural bed main channels. The results form a useful data set showing the effect of natural bedforms in the main channel on the flow for inbank and overbank flows. The following conclusions have been drawn from this study:

- It was found that at most depths the discharge in the main channel with a natural bed was reduced compared to that in the flat bed case. The reduction was a maximum of 30 percent at the bankfull depth and decreased as the overbank flow depth increased. At a relative depth around 0.5 the discharge in both cases became similar. This reduction with natural bedforms has implications for engineers in modelling and design purposes. Either the reduction needs to be taken into account or channels need to be built with uniform cross-sections. Although, the latter option may have environmental issues, and it is for environmental reasons that meandering channels are used in flood alleviation schemes.

- The interaction between the floodplain and main channel flows was found to reduce the velocity in the main channel at certain depths. In the flat bed case the velocity was reduced in all the overbank flows as the fastest flow occurred at the bankfull flow depth. In the natural bed case the reduction in the velocity was found to be a maximum at the 0.3 relative depth, which also corresponded to a minimum sediment transport rate.

- The bedforms were recorded for each depth. In shallow overbank depths there was an area undisturbed by the sediment transport, around the outside of the bend upstream of the apex, with gullies formed at the apex section and running along the outside of the bend downstream of the apex. At $DR=0.3$ the bedforms were mixed with no clear bedforms formed. $DR=0.45$ showed significant bedforms with numerous ridges formed along the channel by the secondary circulations, the main features of the flow at this depth are shown in Figure 4.59.
The flow structures were examined from the measured velocity and turbulence data. The main features of the flow found can be summarised as follows:

- At the apex section the floodplain flow reduces the velocity at the edges of the main channel at low flow depths. As the flow depth increases the floodplain flow increases the velocity of the flow in the main channel.

- The secondary flows were found to be different in the inbank and overbank flow cases. There are three mechanisms that cause the secondary flows; centrifugal force, flow entering into the main channel from the floodplain and reverse flows as the flow passes over ridges in the natural bed case. As the flow depth increases the cross-flow from the floodplain along the crossover section increases, becoming more dominant across the channel. In addition to the increase in the transverse flow the vertical velocities also increase indicating that the strength of the secondary circulations has increased.

- At the 0.45 relative depth there were several circulations across the apex section in both the flat bed and natural bed cases. The 0.45 depth condition was particularly interesting in the natural bed case due to the bedforms in the main channel. The ridges along the channel, particularly the ridges extending directly across the crossover section are a feature of bedforms that have not been shown before. At the crossover section the circulation flows directly across the main channel from the upstream floodplain onto the downstream floodplain. The floodplain flow drives series of vortices, which separate from the upstream bank. There are also smaller reverse circulation flows where the flow crosses over the ridges. Around the bend these small reverse flows increase and displace the main circulations which originated from upstream. The main characteristics of the flow at the 0.45 depth condition in the natural bed case are summarised in Figure 4.59.

- The layer-averaged velocities have shown that flows in the lower-layer tend to be closely aligned to the main channel and the upper-layer flows more closely aligned with the floodplain, but as the overbank flow increases both the upper and lower-layer flows become more parallel with the floodplain. The average velocity profiles across the channel have also been shown to be less uniform in the natural bed case than the flat bed case. This is due to the non-uniform nature of the bedforms.
and the more complex flow structures. The floodplain flow pattern in the flat bed and natural bed cases are almost identical, with the fastest flow occurring outside the meander belt. The flow within the meander belt width flows towards the inside of the bend, although this angle reduces as the flow depth increases.

A new method, based on the two-dimensional curvilinear equations for streamwise motion, was introduced for predicting velocity and discharge in a meandering channel. With this method, the following flow characteristics were found for inbank and overbank flows:

- The turbulent energy losses are not significant at any depth.

- At bankfull and shallow overbank flow depths the bed friction is dominant and the non-bed friction losses increase with relative flow depth (i.e. $K_a$). Within these non-bed friction losses, as defined in Chapter 5, the secondary flow is the dominant mechanism with acceleration and centrifugal each less significant.

- As the flow depth increases the significance of the bed friction decreases and the non-bed friction losses become dominant. The secondary flow remains the dominant mechanism of the non-bed friction losses, but the centrifugal losses have become more significant than the acceleration losses.

The resulting equation is:

$$
\overline{U}_d = \sqrt{\frac{gHS}{HK_a + \frac{r_in + y_in}{r_in} \left( \frac{f}{8} \right)}}
$$

where $K_a$ is a parameter incorporating energy losses due to acceleration, secondary and centrifugal flows. Having examined the properties of $K_a$ at an apex section in the flat bed case a distribution of $K_a$ was shown. Differences were found between the flat bed and natural bed cases, indicating that specific prediction methods are required for channels with natural bedforms.
Applying the method to different scale data from the Flood Channel Facility at Wallingford and Muto (1997) the velocity profile across the main channel was found to be similar, indicating the existence of similar flow structures. $K_a$ was found to decrease with scale. Extending this to natural channels, i.e. large scale, $K_a$ would become insignificant. Removing the $K_a$ term and substituting Mannings coefficient, $n$, for Darcy-Weisbach's friction factor, $f$, results in a modified form of the Mannings equation for large natural meandering channels:

$$\overline{U_d} = \sqrt{\frac{SH^{\frac{1}{3}}}{\left(\frac{r_m + y_m}{r_m}\right)n^2}}$$

Applying this to straight channels leads to Mannings equation:

$$\overline{U_d} = \frac{1}{n}H^{\frac{1}{3}}S^{\frac{1}{2}}$$

6.1 Future work

From the work described here there are several issues which need to be investigated further.

- Sediment and natural bedforms have been shown to have a significant effect on the flow. These need further investigation, as the cases in which bedforms have been studied remain limited. In most methods developed to date, bedforms have not been taken into account. Therefore the effect of bedforms also needs consideration in prediction methods and flow models if natural channels are to be modelled more accurately.

- An energy coefficient, $K_a$, was used to account for the energy losses due to acceleration, secondary and centrifugal flows. $K_a$ has been shown to vary with depth, sinuosity and scale. Further work is needed to verify the dependence of this term on these and other parameters so that the method can be applied confidently to further flow conditions.
• The profile of $K_a$ across the channel was shown to vary between the flat bed and natural bed cases. It was not possible to directly quantify values for $K_a$ in the natural bed case due to the unknown bed shear stress. The bed shear stress in the natural bed case therefore needs further investigation so that the terms in the proposed method can be calculated and direct valid comparisons made between the flat bed and natural bed cases.

• A modified Mannings equation has been derived for meandering channels. Although the original Mannings equation has been used extensively this modified version needs to be applied and tested to see what effect the changes make in practice.
References


References


References


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