

**LOW FLOW HYDRAULICS IN RIVERS FOR
ENVIRONMENTAL APPLICATIONS**

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LOW FLOW HYDRAULICS IN RIVERS FOR ENVIRONMENTAL APPLICATIONS

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by

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EXECUTIVE SUMMARY

BACKGROUND AND MOTIVATION

Effective management and utilization of water-linked ecosystems requires the ability to predict biological responses to management actions. In South Africa, management actions are usually manifest in rivers as changes to the hydrological regime, which is the fundamental driver of biological processes (Poff et al., 1997). The linkage between hydrology and biological response must be made through hydraulics, however, because it is local variables such as flow velocity and depth, rather than discharge, that define aquatic habitat. The movement, dispersion and dilution of pollutants are also determined by hydraulic conditions, as is the movement and distribution of sediment which, to a large degree, determine channel form. Hydraulic analysis is therefore a crucial component in the determination of the ecological Reserve in terms of both quantity and quality, and in any river rehabilitation measures.

Hydrology and aquatic ecology are both mature disciplines in South Africa, and techniques developed for ecosystem management (e.g. Hughes and Munster, 2000; Davies et al., 1993) are founded on well-established precedent. River hydraulics, on the other hand, is poorly developed and relies heavily on overseas experience that has limited applicability to South Africa's unique rivers and analysis needs. Further, most hydraulics methodology emanates from the engineering fraternity and is intended for flood, or at least relatively high flow applications. The hydraulics needs for environmental applications in South Africa are primarily for low flow analyses, and adequate and appropriate techniques still need to be developed. Vegetation and large substrate material are common features of South African rivers. Under low flow conditions vegetation patches and rocks become relatively large, and sometimes effectively discrete, roughness features. The controls on flow depth and velocity therefore become more localized than for high flows, requiring new and different analysis approaches.

Progress in understanding the phenomena involved and developing appropriate techniques has been made through previous Water Research Commission projects K5/856 (James et al., 2001) and K5/1174 (Jordanova et al., 2004). This project aimed to develop the provisional findings of the earlier projects into practically applicable, field verified methods and to address some remaining unresolved issues. The products are intended to provide practical ways for carrying out hydraulic analyses for Environmental Flow Assessment studies and to enhance capacity for implementing river conservation and rehabilitation measures.

OBJECTIVES

The aim of the project as specified in the contract is:

“Develop appropriate methods for describing the hydraulic characteristics of South African rivers under conditions of low discharge and the influence of vegetation and large bed roughness.”

The specific objectives for achieving this aim were outlined as methods for realising the following products:

1. Flow velocity and depth distribution prediction.
2. Large element resistance prediction method.
3. Vegetation resistance prediction method.

MAJOR RESULTS AND CONCLUSIONS

Flow velocity and depth distribution prediction (Chapters 5 and 6)

Local hydraulic conditions, especially flow depth and velocity, are important determinants of aquatic habitat, and their occurrence needs to be described quantitatively in environmental flow assessments and river conservation and rehabilitation studies. This entails prediction of their magnitudes, ranges and extents of occurrence in selected river reaches for specified discharges. The complexity of flow patterns under low flow conditions has led to the use of depth and velocity frequency functions, derived from field measurements. It is shown (Chapter 5) that velocity and (by implication) depth distributions are significantly different under multiple local control and resistance control conditions, both of which are common and can occur at the same site for different discharges. It is concluded, therefore, that the statistical approach is unreliable for practical applications under low flow conditions. The situation is too complex for conventional rapidly varied flow analysis, and computational modelling is considered to be the appropriate approach. A public domain 2-dimensional model, River2D, is identified as a suitable tool. It is shown in Chapter 6 to be able to predict velocity distributions reliably under large-scale roughness conditions, and especially for the trans-critical flows associated with multiple local controls. It is shown in Chapter 8 to predict velocity frequency distributions reliably in field applications.

Large element resistance prediction method (Chapter 4)

Conventionally, flow resistance in rivers is described using equations (such as those of Chézy, Darcy-Weisbach and Manning) that implicitly assume the dominant resistance phenomenon to be boundary shear stress. Such equations are inherently

unsatisfactory for low flow conditions, where the size of roughness elements is comparable to the flow depth and resistance is dominated by form drag. Two new methods are proposed for the conditions of large- and intermediate-scale roughness, i.e. when the flow depth (y) is less than the height of the roughness elements (h) and between one and four times the height of the roughness elements respectively.

The equation for average flow velocity, V , under large-scale roughness conditions is

$$V = \frac{1}{F} \sqrt{2gR_v} \sqrt{S} \quad 4.10$$

where g is gravitational acceleration, R_v is the volumetric hydraulic radius and S is the energy gradient. Based on laboratory data, the resistance coefficient, F , is given by

$$F = 0.16 Fr^{-0.51} Re^{0.004} \Lambda^{0.222} \quad 4.16$$

where Fr is the Froude number, Re is the roughness element Reynolds number and Λ is the areal concentration of roughness elements.

For greater convenience in field applications, equation (4.16) may be replaced by

$$F = 0.05 Fr^{-0.868} Re^{0.12} \sigma^{-0.228} \quad 4.20$$

where σ is given by

$$\sigma = \frac{D_{84}}{D_{50}} \quad 4.18$$

For intermediate-scale roughness, the resistance effect is a varying balance of boundary shear and form drag contributions. By combining flows in vertically separated zones dominated by these influences, the average flow velocity for this condition is given by

$$V = \sqrt{\frac{4}{f}} \sqrt{2g y_{eff2}} \sqrt{S} \quad 4.29$$

where y_{eff2} is the effective flow depth, accounting for the volume of the roughness elements, and f is an effective friction factor that accounts for bed element resistance, given by

$$\sqrt{\frac{4}{f}} = \left(\frac{y_{eff1}}{y_{eff2}} \right)^{\frac{3}{2}} \frac{1}{F} + \left(1 - \frac{y_{eff1}}{y_{eff2}} \right)^{\frac{3}{2}} \sqrt{\frac{4}{f_2}} \quad 4.28$$

where f_2 is the friction factor under small-scale roughness conditions, F is the large-scale resistance coefficient and y_{eff1} is the height of the tops of the roughness elements above the lowest bed level.

Flow resistance for intermediate-scale roughness may also be described as a combination of roughness element drag and boundary friction equations, as

$$V = \left(\frac{1}{F} \right)^a \left(\sqrt{\frac{4}{f}} \right)^{(1-a)} \sqrt{2gR_v} \sqrt{S} \quad 4.30$$

where a is a weighting factor, varying from 1 to 0, that depends on the relative submergence (y/h) according to

$$a = -0.67 \ln \left(\frac{y}{h} \right) + 0.992 \quad 4.31$$

Predictions by these equations are shown to be satisfactory by comparison with laboratory and field data for the large-roughness case and laboratory data for the intermediate-roughness case.

Vegetation resistance prediction method (Chapter 7)

Practical conveyance prediction methods are presented for three situations pertaining to the occurrence of vegetation in rivers and wetlands, viz. flow through emergent vegetation, flow in channels with emergent vegetation boundaries, and flow in channels with discrete vegetation patches.

Practical resistance estimation for flow through emergent vegetation. Basic research carried out in Project K5/856 (James et al., 2001) was used to develop a practically usable equation for describing the resistance of emergent, reed-type vegetation. A detailed and rigorous computational model had been developed in the earlier project which, while accounting reliably for the underlying physical processes influencing flow resistance, is too complex and cumbersome for routine practical applications and provides a level of detail and resolution beyond what is usually necessary for practical purposes. The model was therefore applied hypothetically to identify the variables most significant in determining vegetative resistance; these proved to be the vegetation stem diameter, the stem spacing, the stem drag coefficient and the channel slope. The rational equation form developed in Project K5/856 was adopted, i.e.

$$V = \frac{1}{F} \sqrt{S} \quad 7.8$$

where V is the flow velocity, S is the channel slope, and F is the vegetation resistance coefficient. By simulating a wide range of conditions and performing a regression analysis on the results, the following relationship for the discharge coefficient in terms of the important variables was derived.

$$F = 1.885 \left(\frac{s}{D} \right)^{-0.65} \left(\frac{D}{y} \right)^{0.07} C_D^{0.48} \quad 7.16$$

where s is the stem spacing, D is the stem diameter, y is flow depth and C_D is the stem drag coefficient. All available field and laboratory data were acquired and analysed to provide a way of estimating C_D from graphs or the relationship

$$C_D = \alpha Re^\beta \quad 7.15$$

where Re is the stem Reynolds number. Values of α and β are tabulated for different foliage conditions.

Flow in channels with emergent vegetation boundaries: The effect of strips of vegetation on channel conveyance was demonstrated and elucidated in Project K5/856 (James et al., 2001). This work has been extended and confirmed by field data from the main channel of the Okavango delta, and presented as a reliable and simple practical method for predicting conveyance for channels with strips of vegetation along their banks. If the contribution of flow within the vegetation is important, it can be estimated using the approach described above. The clear channel between the vegetation strips is considered as a channel of composite roughness, with different roughness on the bed and the vegetation boundaries. The discharge can then be estimated using Manning's equation, with the effective resistance coefficient (n_e) determined from the composite roughness equations of Horton (1933) (equation 7.17) or Pavlovski (1931) (equation 7.18):

$$n_e = \left(\frac{\sum_{i=1}^N (P_i n_i^{3/2})}{P} \right)^{2/3} \quad 7.17$$

$$n_e = \left(\frac{\sum_{i=1}^N (P_i n_i^2)}{P} \right)^{1/2} \quad 7.18$$

in which n_i and P_i are the resistance coefficient and wetted perimeter of the i^{th} of N channel subsections. The value of n for reed-type vegetation boundaries was shown by the field data to be about 0.13. It is also shown how resistance coefficient values for alluvial beds can be inferred from sediment transport measurements.

Flow in channels with discrete vegetation patches: Extensive laboratory data on flow in channels with fragmented vegetation patches were obtained in Project K5/856 (James et al., 2001), but no prediction method was developed. It is concluded in this report that the complexity of the situation precludes simple expressions of effective resistance and that computational modelling is necessary. The two-dimensional model River2D was tested against the laboratory data and found to successfully predict flow depths and effective Manning's n values. Vegetation patches can be represented either as areas of extreme roughness or zones of flow through a porous medium. Owing to limitations of the software in specification of roughness values, the porous medium representation was more successful in these applications although it is less effective in describing velocity distributions.

ATTAINMENT OF OBJECTIVES

Flow velocity and depth distribution prediction

The freely available River2D model has been shown to be an effective tool for predicting velocity (and, by implication, flow depth) distributions in rivers under low flow conditions, thereby meeting the contract objective. Recommendations and guidelines for the model's general use for South African rivers will be provided through the Water Research Commission project K5/1508.

Large element resistance prediction method

Equations have been presented for predicting resistance for large-scale and intermediate-scale conditions in rivers with large substrate material. These provide for analysis of the full range of low flow conditions in South African rivers, and meet the contract objectives fully. The equations provide the first explicit recognition of the different phenomena contributing to resistance in low flows, giving them a significantly better level of generality than existing methods, and providing a sounder basis for empirical calibration.

Vegetation resistance prediction method

Procedures have been presented for estimating conveyance in fully and partially vegetated channels. These cover the full range of vegetation conditions likely to be encountered in practical environmental flow assessment, river rehabilitation and river conservation studies, and therefore fully meet the contract objectives. The contribution to knowledge of this work is in providing accessible and practical methods for estimating conveyance under different conditions. The three approaches presented show the appropriateness of different treatments of different levels of system complexity: uniform vegetation resistance can be described by a single, simple equation; resistance estimation for channels with vegetated banks requires composite resistance coefficient determination as well; flow description in channels with fragmented vegetation patches requires computational modelling.

CAPACITY DEVELOPMENT

The project constituted the basis for the PhD thesis of Mrs Angelina Jordanova (white female). At the time of submission of this report, her thesis was in the final stages of preparation. Mrs Jordanova also developed her research management capacity by co-supervising the following MSc project.

The project provided the opportunity and topic for the MSc project report by Mr Mashau Mashau (black male) entitled "Flow Resistance in Open Channels with Intermediate-Scale Roughness", and completed in 2006. Mr Mashau received a grant-holder bursary from the National Research Foundation, but facilities and supervision were provided through this project.

The project provided resources and guidance for the following Investigational Projects for final year BSc(Eng) students:

- Mr T Goeieman (black male) and Mr D Kwele (black male) (2004): "Habitat Hydraulics – the effect that geomorphic features have on Hydraulic Habitats".
- Mr C Cilliers (white male) and Mr G Harli (white male) (2005): "Laboratory Investigation of Longitudinal and Transverse Fragmentation of bed Roughness on Channel Conveyance".
- Ms M Brinkley (white female) and Mr J Walls (white male) (2006): "The effect of Mixed-Size Roughness Elements on Flow Resistance".
- Mr S Bhembé (black male) and Mr V G Pandey (black male) (2006): "Composite Roughness in Open-Channel Flow".

BSc(Eng) student Mr J Walls (white male) was employed for vacation work in 2005/6 and gained experience in laboratory experimentation.

RECOMMENDATIONS FOR FURTHER RESEARCH

The project has produced methods for predicting low flow conditions in rivers that are complete and usable but, as with all methods that rely wholly or partly on empiricism, further strengthening of the data base and further field confirmation would be valuable.

Field-based investigations of flow with large roughness elements in riffle situations would be valuable to test and refine the approaches presented for predicting resistance under intermediate- and large-scale conditions. Recommendations for definition of the roughness size as a percentile bed material size are necessary. The contribution of the relatively small substrate material between the largest elements to overall resistance needs to be established. Resistance relationships for small-scale roughness conditions need to be reassessed and reliably quantified.

Investigations are necessary to relate vegetation stem drag coefficients directly to vegetation morphological characteristics, and to estimate the resistance coefficients of vegetated bank boundaries.

Existing guidelines for estimating resistance coefficients are in terms of Manning's equation. Appropriate reinterpretation of the underlying data in terms of resistance equation forms that account for form drag contributions would reduce the uncertainty of resistance prediction.

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The research was guided by a Reference Group comprising the following people:

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Other individuals made valuable contributions to this study, some directly to the research and others to its administration. The Centre for Water in the Environment at the University of the Witwatersrand provided continual support through provision of resources and stimulation by Professor Kevin Rogers and others. Prof. Thoms from the University of Canberra (Australia) provided the Cotter River field data. Mr Peter Hirschowitz, in particular, provided guidance on the use of River2D and links with the concurrent project K5/1508. Mrs Wendy Midgley handled the ever increasing administrative tasks and financial control with her usual cheerfulness, competence and efficiency. The School of Civil & Environmental Engineering at the University of the Witwatersrand provided the laboratory facilities, office space, computer facilities and technical assistance, as well as bursary support to finance Ms Jordanova's time for 9 months beyond the contract period. Research contributions were made through the

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1 INTRODUCTION

Effective management and utilization of water-linked ecosystems requires the ability to predict biological responses to management actions. In South Africa, management actions are usually manifest in rivers as changes to the hydrological regime, which is also the fundamental driver of biological processes (Poff et al., 1997). The growing need to predict the biological impacts related to water management activities demands further understanding of the relationships between hydrological variability and river ecosystem integrity (Richter et al., 1997). The linkage between hydrology and biological response must be made through hydraulics, however, because it is local variables such as flow velocity and depth, rather than discharge that define aquatic habitat. The movement, dispersion and dilution of pollutants are also determined by hydraulic conditions, as is the movement and distribution of sediment which, to a large degree, determine channel form.

Hydrology and aquatic ecology are both mature disciplines in South Africa, and techniques developed for ecosystem management (e.g. Hughes and Munster, 2000; Davies et al., 1993) are founded on well-established precedent. River hydraulics, on the other hand, is poorly developed, and relies heavily on overseas experience that has limited applicability to South Africa's unique rivers. Further, most hydraulics methodology emanates from the engineering fraternity and is intended for flood, or at least relatively high flow applications.

Vegetation and large substrate material are common features of South African rivers. Under low flow conditions vegetation patches and rocks become relatively large, and sometimes effectively discrete, roughness features. This kind of roughness features usually acts as obstacles to the flow. A drag force around such obstacles modified the velocity and velocity distributions. The controls on flow depth and velocity therefore become more localized than for high flows, requiring different analysis approaches.

In the new South African water law, the quantity of water required to maintain riverine functions is included in the ecological Reserve (Uys, 2001). Implementation of the new law requires that an ecological Reserve be determined for all the country's rivers, with those for which development is planned receiving priority attention. Statutory provisions are made for environmental flow requirement in the National Water Act (Act 36 of 1998) to determine the ecological component of the Reserve for all significant water resources. Hydraulic analysis is therefore a crucial component in the determination of the ecological Reserve in terms of both quantity and quality, as well as in any river rehabilitation measures.

Chapter 1: Introduction

This research is related to development of methods for hydraulics under low flow conditions required for environmental applications in South Africa.

The aim of the project is development of appropriate methods for describing the hydraulic characteristics of South African rivers under conditions of low discharge, and the influence of vegetation and large bed roughness.

The specific objectives for achieving this aim were outlined as methods for realising the following products:

- Flow velocity and depth distribution prediction.
- Large element resistance prediction method.
- Vegetation resistance prediction method.

The objectives have been addressed by undertaking literature survey, experimental investigation, data analyses, theoretical development and computer modelling.

Layout of the report

Chapter 2 reviews published information related to approaches and equations developed internationally for predicting flow resistance under large and intermediate scale roughness. **Chapter 3** includes experimental investigations related to overall flow resistance under large and intermediate scale roughness under different hydraulic conditions. **Chapter 4** presents development and verification of resistance equations for low flow conditions. **Chapter 5** describes experimental investigations of velocity distributions with large roughness. **Chapter 6** presents prediction methods for velocity distributions with large roughness. **Chapter 7** presents vegetation resistance prediction methods for fully and partially vegetated channels. **Chapter 8** addresses the modelling of river hydraulics using two-dimensional River2D software. **Chapter 9** presents conclusions of the project and recommendation regarding further research.

2 LOW FLOW HYDRAULICS

2.1 Resistance Controlled Condition

2.1.1 Bed Roughness and Flow Resistance

Irregularities along the bed of a river under low flow conditions enhance diversity of physical habitat for aquatic animals and affect the velocity and depth distributions of flow within the channel by increasing resistance as well as causing flow separation and turbulent velocity fluctuations. The hydraulics of low flows, where the flow depth is of the same order of magnitude as the bed material size, is very complex, and current knowledge is limited. Understanding of flow regimes around large bed elements as well as the influence of vegetation on overall flow resistance is also essential for environmental studies such as Reserve determination, river restoration and rehabilitation.

The relative depth (y/h , where y is the flow depth, and h is the height of the bed roughness) describes the average degree of submergence of the surface roughness and is used to distinguish three flow regimes related to scale of bed roughness: large, intermediate and small (Bathurst et al., 1981). The roughness is referred to as large-scale if the roughness elements affect the free surface, a condition occurring when the relative depth (y/h) is less than 1. The roughness is intermediate-scale if the relative depth lies between 1 and 4. When the relative depth is higher than 4 the roughness is small-scale.

The flow regimes of these scales of roughness are characterized by very different functional dependencies of resistance origin. Well-inundated (small-scale roughness) flows are well understood, and various equations, based on the assumed proportionality between boundary shear and average flow velocity have been proposed. All of these account for the resistance processes with a single coefficient of resistance (Bathurst, 1982). The most commonly used equations are the following:

Darcy-Weisbach:

$$V = \sqrt{\frac{8g}{f}} \sqrt{RS} \quad 2.1$$

where f : Darcy-Weisbach friction factor.

Chézy:

$$V = C \sqrt{RS} \quad 2.2$$

where C : Chézy resistance coefficient.

Manning:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad 2.3$$

where n : Manning resistance coefficient.

In this regime, the roughness elements are very small relative the flow depth, and they do not significantly alter a one-dimensional flow field. When the roughness is of intermediate scale, the size of the roughness elements relative to the flow depth controls the degree of vertical mixing in the flow so that frictional resistance tends to decrease very rapidly with increasing depth of flow. Under the large scale condition, the flow depth is less than or equal to the height of the substrate and the drag force imposed by individual roughness elements causes most of the flow resistance.

Under low flow conditions the main source of energy loss in water flowing over a rough surface is the generation, spreading, and dissipation of vortices from the wake and separation zones behind each roughness element. Three basic flow types have been denoted as isolated-roughness flow, wake-interference flow, and quasi-smooth (or skimming) flow (Morris, 1954). Isolated-roughness flow occurs when roughness elements are far apart and the individual elements act as isolated bodies, developing drag forces by the flowing water. Under this condition, the wake zone and vortex-generating zone at each element are completed and dissipated before the next element is reached. Wake-interface flow appears with roughness elements placed sufficiently close together so that the zone of separation and vortex generation and dissipation associated with each element are not completed before the next element is encountered. The quasi-smooth or skimming flow results when the roughness elements are so close together that the flow skims the tops of the elements. Under this condition, there will be regions of dead water containing stable vortices between the elements.

A number of studies have been carried out to investigate flow resistance over rough surfaces with different relative submergences, and methods and equations for its prediction have been proposed (Dittrich and Koll, 1997; Lawrence, 1997; Lawrence, 2000; Nikora et al., 2001; Smart et al., 2002).

Lawrence (1997) strongly concluded: "a fundamental dimensionless parameter for evaluating overland flow hydraulics is a measure of the extent of the inundation of the surface roughness, as this parameter determines the dominant physical mechanism controlling the frictional resistance to flow." Three equations describing the dependence of the frictional resistance (f) on the relative submergence for each of the flow regimes were proposed as follows:

$y/h > 4$:

$$f^{-0.5} = 1.64 + (0.803) \ln(y/h) \quad 2.4$$

$1 < y/h < 4$:

$$f = 50 \left(\frac{h}{y} \right)^2 k^2 \quad 2.5$$

where k : von Karman's constant in turbulent velocity profile

$y/h < 1$:

$$f = 8nh^2 C_d \text{MIN} \left[\frac{\pi}{4}, \frac{y}{h} \right] \quad 2.6$$

where n : number of roughness elements per unit area,

C_d : drag coefficient of a roughness element,

$\text{MIN}[a,b]$: the minimum possible value taken on by either variable a or b .

Available published field and laboratory data were used for evaluation of the proposed equations. The frictional resistance (f) was plotted as a function of an inundation ratio, ($\Lambda = y/h$) in Fig. 2-1.

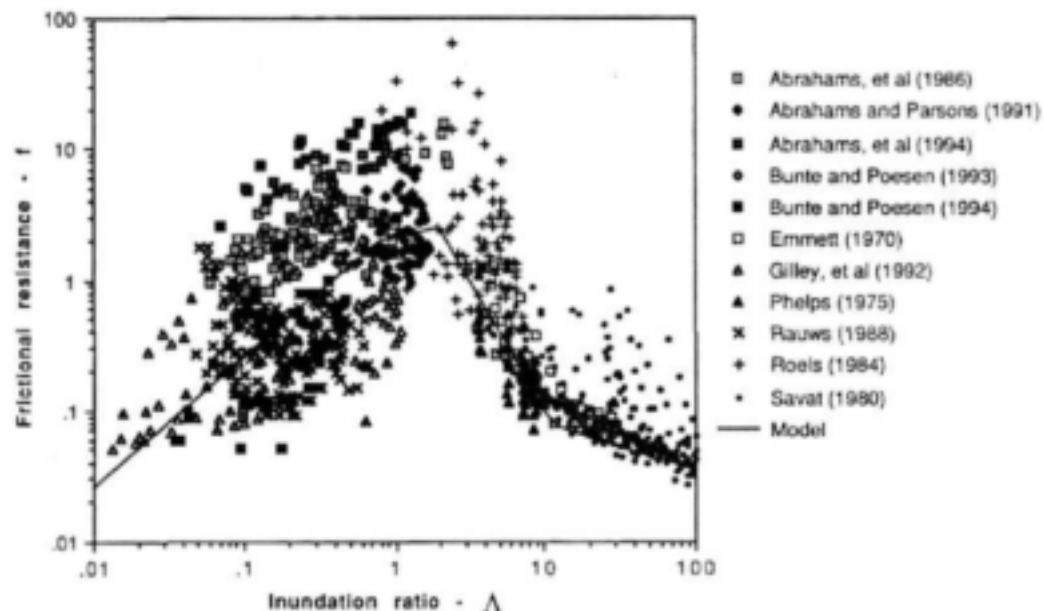


Figure 2-1 Frictional resistance as a function of flow inundation (Lawrence, 1997)

The proposed models relied on the following assumptions:

- A single length scale can be used to characterize the size of the roughness elements.

- The drag coefficient for a roughness element is constant.
- The geometry of the elements during partial inundation ($y/h \leq 1$) is approximately cylindrical.
- The reduction of the cross sectional flow area by the roughness elements is negligible.

Further investigation and laboratory experiments were performed (Lawrence, 2000), and two models, for large and intermediate scales, were proposed. A drag model was developed that accounts for the contributions of individual roughness elements to total flow resistance under large scale conditions. A proposed equation for calculation of flow resistance in term of friction factor f is:

$$f = \frac{2PA(y_{eff}/h)C_d}{[1 - PV(y_{eff}/h)]^3} \quad 2.7$$

where P : fraction of surface covered by roughness elements,
 C_d : drag coefficient,
 $A(y_{eff}/h)$: factor accounts for the change in frontal area with inundation,
 $V(y_{eff}/h)$: factor to account for volume of roughness elements.

The drag coefficient (C_d) drops off rapidly once the roughness elements become submerged. Based on experimental data with 18 % areal coverage, the following relationship was proposed:

$$C_d = A + B \tanh[a(y_{eff}/h - 1)] \quad 2.8$$

where A , B and a are fitted parameters.

Flow resistance for the large scale roughness condition is associated with high values of effective frictional factor. These high resistance values may result from increased friction due to the effects of hydrostatic wave drag around elements associated with deformation of the free surface on protruding or marginally inundated roughness elements, and the drag force derived from individual roughness elements. This regime is very complex and therefore cannot be explained using simple drag models only (Lawrence, 2000). The mixing length model (following) that was developed for intermediate scale roughness was extended into the range of large scale roughness for moderate Reynolds numbers (Lawrence, 2000).

Lawrence (2000) proposed the following mixing length model for the intermediate scale, which combines the turbulent mixing length scale with the height of the roughness elements.

$$f = \frac{c}{(y_{eff}/h)} \quad 2.9$$

in which c is a constant depending on the shear stress distribution. For the case of a linear shear stress distribution, c is about 10.

Ferro (2003) presented an attempt to modify the mixing length and drag models proposed by Lawrence (1997, 2000) and a quasi-theoretical model that was calibrated by available experimental gravel bed data. Using experimental data (Ferro and Giordano, 1991; Baiaumont and Ferro, 1997; Ferro, 1999) the drag coefficient (C_d) values were calculated by equation 2.8 and the following equation was proposed:

$$C_d = \frac{1}{2\Gamma} \left[1 - \frac{2\Gamma}{3(y/d_{50})} \right] \quad 2.10$$

where Γ is the concentration of coarse elements.

An attempt to extend the application of equation 2.9 for intermediate scale conditions resulted in the following general form of the power flow resistance law equation:

$$f = A + \frac{C}{(y/h)^2} \quad 2.11$$

in which A and C coefficients depend on the grain-size distribution of the bed particles and Froude number. Experimental testing of equation 2.11 showed that for Froude numbers greater than 0.5, the constant A is zero.

Furthermore, by the statistical analysis of the available experimental data, a power equation for f was proposed (Ferro, 2003):

$$f = 10^{b_0} \left(\frac{y}{D_{84}} \right)^{b_1} Fr^{b_2} \quad 2.12$$

where b_0 , b_1 and b_2 are numerical coefficients.

Relationships between the numerical coefficients (b_0 , b_1 and b_2) and the concentration (Γ) for a given ratio of median size of a coarse to a median size of fine components were proposed.

Various attempts at prediction of flow resistance under large-scale roughness conditions have resulted in a number of different equations that can be classified into three forms (Bray and Davar, 1987):

- Nondimensional semi logarithmic equation
- Dimensional power equation
- Nondimensional power equation.

2.1.1.1 Nondimensional Semi logarithmic Approaches

Semi-logarithmical approaches have been based on boundary layer theory, and these are of the form:

$$\frac{1}{\sqrt{f}} = c_0 \log\left(\frac{R}{c_2 D_x}\right) + c_1 \quad 2.13$$

where f : Darcy-Weisbach friction factor,
 R : hydraulic radius,
 D_x : x percentile particle grain size,
 c_0, c_1, c_2 : constants.

A number of equations have been developed for gravel-bed channels that take into account the effect of cross-sectional shape, differences between bed and bank roughness, and the effect of nonuniform sediment on flow resistance. For example, Hey (1979) proposed

$$\left(\frac{1}{\sqrt{f}}\right) = 2.03 \log\left(\frac{aR'}{3.5 D_{84}}\right) \quad 2.14$$

where R' : effective hydraulic radius.

The validity of the semi logarithmic resistance equation (equation (2.14)) for application to large-scale roughness conditions was tested by Bathurst (1985). He concluded that equation 2.14 under-predicts flow resistance, and proposed

$$\left(\frac{8}{f}\right)^{1/2} = 5.62 \log\left(\frac{y}{D_{84}}\right) + 4 \quad 2.15$$

where y : flow depth.

When bed roughness elements are large relative to flow depth, determination of the hydraulic radius as the ratio of flow area to wetted perimeter becomes ambiguous. The volumetric hydraulic radius (R_v) is therefore proposed by some researchers to be used instead of the conventional hydraulic radius (e.g., Bathurst et al., 1981; Smart et al., 2002). The volumetric hydraulic radius is defined as the volume of overlying water per unit plan area of bed.

Smart et al. (2002) proposed a flow resistance equation in terms of the volumetric hydraulic radius given by

$$\frac{U}{U_*} = 2.5 \ln\left(\frac{R_v}{d}\right) + B \quad 2.16$$

where d : representative roughness length characterizing the bed material,
 B : general coefficient,
 U : mean flow velocity, and
 U^* : bed shear velocity.

2.1.1.2 Nondimensional Power Approaches

Several investigators (Bray, 1979; Griffiths, 1981; Bathurst, 2002) have developed nondimensional power equations based on field data for gravel rivers. A general form of the nondimensional power equation is:

$$\frac{1}{\sqrt{f}} = b_0 \left(\frac{R}{b_2 D_s} \right)^{b_1} \quad 2.17$$

where b_0, b_1, b_2 are constants.

Bathurst (2002) analysed 27 field data sets to derive separate at-a-site relationships and investigated how they could be collapsed into a single formula. It was suggested that flow resistance can be more accurately described by a power law than by a semi-logarithmic law. Relative submergence based on D_{84} was found to be the primary predictor of the flow resistance. Two power law relations related to channel slope were identified, and two equations were therefore proposed.

Channel slope, $S < 0.8\%$:

$$(8/f)^{1/2} = 3.84 (y/D_{84})^{0.547} \quad 2.18$$

Channel slope, $S > 0.8\%$:

$$(8/f)^{1/2} = 3.10 (y/D_{84})^{0.93} \quad 2.19$$

2.1.1.3 Dimensional Power Approaches

Another empirical approach to derive a relationship for flow resistance is by means of power laws. The relationship between the mean velocity and the discharge can be formulated as:

$$V = c Q^m \quad 2.20$$

where c and m are regression coefficients.

Some of the existing power approaches for resistance prediction under low flow conditions introduce additional variables such as the channel slope and the roughness parameter (D_s). Rickenmann (1994, 1996) proposed the following equations for different slopes:

Channel slope, $S > 0.6\%$:

$$V = 0.37 g^{0.33} Q^{0.34} S^{0.20} D_{90}^{-0.35} \quad 2.21$$

Channel slope, $S < 1\%$:

$$V = 0.96 g^{0.36} Q^{0.29} S^{0.35} D_{90}^{-0.23} \quad 2.22$$

Based on field data from 21 streams in the Rocky Mountains of Colorado, multiple-regression analyses yielded an equation for predicting Manning's resistance coefficient (n) as a power function of the hydraulic radius and the friction slope (Jarrett, 1984):

$$n = 0.39 R^{0.38} S^{-0.16} \quad 2.23$$

As flow resistance under low flow conditions depends on the roughness geometry, its determination requires definition of a roughness parameter. It can be seen that a characteristic index of grain roughness (D_{90}) is incorporated into Eqs 2.21 and 2.22, while the hydraulic radius is used in Equation 2.23. A single grain size gives a coarse description of the roughness geometry, while calculation of the hydraulic radius based on the wetted perimeter is problematic when flow depth and height of substrate are of the same order of magnitude (Aberle and Smart, 2003). The standard deviation of the bed elevation as characteristic of roughness structure of rough beds was therefore introduced. Based on laboratory experiments the following equation was proposed:

$$V = 0.96 g^{0.20} S^{0.20} q^{0.60} s^{-0.40} \quad 2.24$$

where g : acceleration due to gravity,
 q : specific discharge per unit width,
 s : standard deviation of the bed elevations.

Flow resistance depends strongly on the bed roughness geometry; however, determination and application of a roughness parameter to be incorporated into resistance prediction is still problematic. On the one hand, it should be measurable in the field but at the same time it should give a realistic description of the bed geometry. Bed roughness characterization is therefore discussed below.

Aberle and Smart (2003) investigated the statistical properties of a series of bed profiles in order to quantify the effect of bed roughness on flow resistance. Analysis of longitudinal bed profiles from laboratory experiments identified the standard deviation (s) of the bed elevations as an appropriate roughness parameter. The following equation for prediction of flow velocity over rough bed was proposed:

$$\bar{u} = 0.96 g^{0.20} S^{0.20} q^{0.60} s^{-0.40} \quad 2.25$$

where g : acceleration due to gravity,
 S : slope,
 q : discharge per unit width,
 s : standard deviation of the bed elevation.

It was concluded that the use of the standard deviation as characteristic roughness leads to an improvement in estimation of flow resistance compared to the results obtained by using only characteristic of sediment size (D_n) as a roughness parameter.

2.1.1.4 Deterministic Approaches

The bed roughness configuration affects the overall velocity distribution. Prediction of cross-sectional averaged velocity as well as the velocity distribution over a rough river bed requires partitioning of total shear component into a fluid component and a form-drag component associated with flow around bed roughness elements (Wiberg and Smith, 1991). In a natural stream with bed roughness elements that are comparable in size to flow depth, large elements act as obstacles to the flow. When flow is forced around the large elements, drag forces are exerted on those elements and the momentum of flow is locally reduced, this modifying the velocity distribution. For an isolated roughness element skin drag is not significant, and the drag force (F_d) causing the flow resistance can be described by

$$F_d = \frac{1}{2} C_d \rho V_x^2 A_d \quad 2.26$$

where C_d : drag coefficient,
 ρ : density of water,
 V_x : approach velocity, and
 A_d : protruding cross-sectional area of a roughness element.

The protruding cross-sectional area (A_d) is a function of the roughness element shape and size,

$$A_d = s_1 d_y d_z \quad 2.27$$

where s_1 : shape related factor,
 d_y : transverse dimension of a roughness element, and
 d_z : submerged height of a roughness element.

If a roughness element has semi-elliptical shape then the shape related factor (s_1) is $s_1 = \pi/4$.

The drag force is transferred to the bed as a shear stress (τ_s) acting on area of bed (A_b) i.e.

$$F_d = \tau_s A_s = \tau_s s_2 d_y d_x \quad 2.28$$

where s_2 : another shape related factor, and
 d_x : longitudinal dimension of a roughness element.

For a semi-elliptical shape of roughness element, the shape related factor $s_2 = s_1 = \pi/4$.

If the shear force is transferred only by the roughness elements, then the average shear stress over a particular area of the bed is

$$\tau_0 = \tau_s p \quad 2.29$$

where p is a ratio of basal area of a roughness element to the area of bed considered.

Combining equations (2.26 – 2.29) an approach velocity can be calculated as (Smart et al., 2002)

$$V_s = \sqrt{\frac{2\tau_0 s_2 d_x}{C_d \rho s_1 d_z p}} \quad 2.30$$

Jonker et al. (2002) proposed a theoretically-based equation for mean flow velocity in rivers under conditions of large-scale roughness:

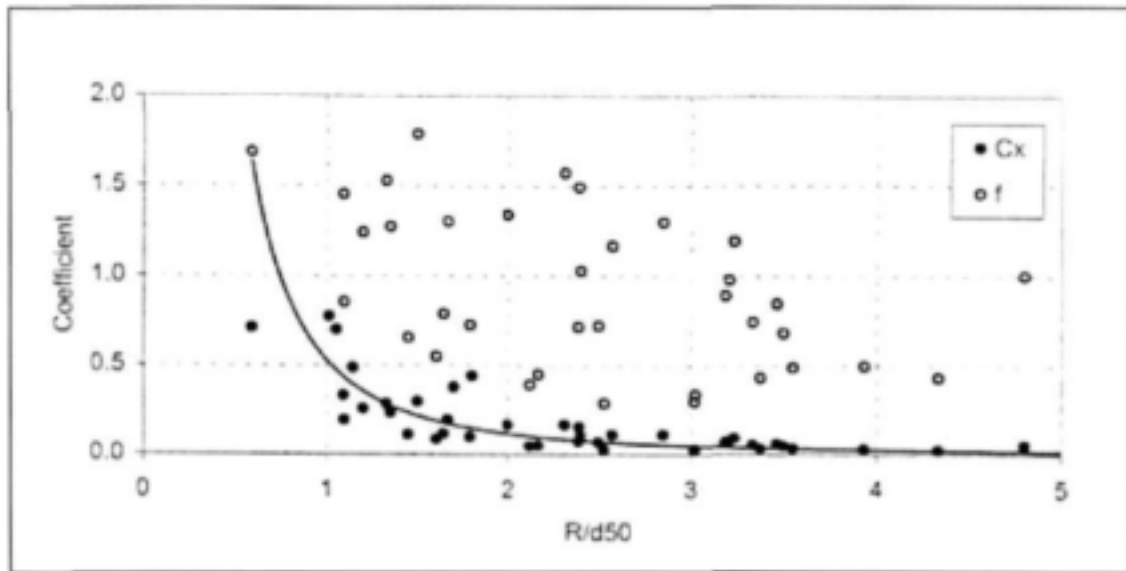
$$V = \sqrt{2g} \sqrt{\frac{d S_o}{C_x}} \quad 2.31$$

where d : cobble diameter,
 S_o : channel gradient, and
 C_x : resistance coefficient.

Extensive published data were used for evaluation of the applicability of equation 2.31. From these data, corresponding values of resistance coefficients (C_x) were calculated and plotted (Fig. 2.2). Correlation between resistance coefficient (C_x) and relative submergence (R/d_{50}) was derived as:

$$C_x = 0.5285 \left(\frac{R}{d_{50}} \right)^{-2.166} \quad 2.32$$

where R : hydraulic radius.

Figure 2-2 Relationship between C_x and R/d_{50} (Jonker et al., 2002)

It can be seen (Fig. 2-2) that good correlation exists between C_x and R/d_{50} , but it is evident that only one data point for relative submergence less than one was used. It is apparent that good correlation exists between theoretical and experimental values under conditions of intermediate scale roughness, but further investigation for the condition under large-scale roughness is still required.

Wohl and Ikeda (1998) used seven different configurations of bed roughness as well as a plane bed to study the effect of bed roughness configuration on velocity distribution. Sixteen conditions of varying discharge and slope were performed for each configuration. The flow resistance for each configuration was calculated in terms of the resistance coefficient:

$$\lambda_d = 22gdS_w / V^2 \quad 2.33$$

where λ_d : resistance coefficient,
 d : flow depth,
 S : water surface slope, and
 V : mean velocity.

Vertical velocity profiles were measured at 1 cm increments over the crest of the roughness element. The laboratory results indicate the following:

- The velocity profile shape remains fairly constant for a given roughness bed configuration and slope as discharge increases.
- Velocity profiles become less linear at the measurement point immediately downstream from the roughness element as slope increases.

- The maximum value of flow resistance in term of the resistance coefficient (equation (2.33)) was observed for all values of discharge and slope for roughness length/height ratio of about 9.
- Longitudinal patterns of roughness elements create much less flow resistance than transverse patterns.

2.1.2 Bed Roughness Characterization

Riverbed substrates influence properties of average flow, flow resistance, turbulence, and sediment motion. Characterization of a riverbed substrate for many different purposes such as channel roughness, bed load transport, and habitat description is therefore required as an essential part of river hydraulics and fluvial geomorphology (Wohl et al., 1996; Nikora et al., 1998). Flow resistance under large-scale roughness is controlled by substrate composition, and flow resistance prediction depends on quantitative description of the bed roughness. Methods for quantifying riverbed roughness are discussed below.

2.1.2.1 Surface Sampling Methods

Nikora et al. (1998) reviewed current methods for quantifying river bed roughness and combined them into two groups:

- Particle size approach, and
- Random field approach.

The first approach characterizes the surface of a riverbed by the characteristic diameter (D_n) that represents bed roughness, such as D_{50} , D_{84} or D_{90} . The second one considers the bed surface as a random field of bed elevation.

2.1.2.1.1 Particle Size Approach

The purpose of the particle size approach is to evaluate the size distribution of bed particles and to produce one measure of sediment size, such as D_{50} , D_{84} or D_{90} . This approach is widely used in river engineering, fluvial geomorphology, and stream ecology. The approach originated from Wolman (1954). Wolman (1954) proposed measuring the intermediate axes of 100 clasts sampled from a grid system, with the grid size determined by the size of the sampling area. This sample would represent the areal distribution of material of the bed. According to Wolman (1954), 100 clasts are sufficient to ensure that there are no significant differences between operators or between samples for a given operator (Wohl et al., 1996).

The techniques most often used for sampling bed surfaces can be divided into the following categories: areal, grid and transect. Grid and transect techniques are widely used for sampling coarse bed surfaces ($D > 8$ mm) and are not appropriate for small particles (Diplas and Sutherland, 1988).

Areal sampling:

An areal sample consists of the grains of the bed that are exposed to the flow within a predetermined area of the channel bed. The areal sample collected by using adhesives, clay or wax, is most often analysed as a frequency distribution by weight. The primary advantage of areal sampling is that adhesives remove the smaller particle size range while the disadvantage is that it requires the sample area to be dry (Diplas and Sutherland, 1988; Fripp and Diplas, 1993).

Grid sampling:

This technique is widely used for sampling coarse bed surfaces in the field. Grid sampling involves the removal by hand of stones found at specific points. These points can be established on the bed surface by using predetermined distances on a survey tape. The simplest variant of grid sampling is known as the Wolman walk method where an operator would stop at each pace and remove the stone found under his toe, the eyes should be averted or closed (Wolman, 1954). The grid sample is usually analysed as a frequency distribution of number. The use of the intermediate diameter of each sampled stone is recommended (Hey and Thorne, 1983).

Transect sampling:

This approach is similar to the previous one, involving the collection of the grains that are located along a predetermined line. The volume of the sample depends on the axes of the removed particles that are normal to the line (Diplas and Sutherland, 1988). The transect sample are interpreted in terms of a frequency by weight or by number for a line sample.

Accuracy of surface sampling for coarse sediments is associated with an operator and sampling errors, as well as with the choice of sampling procedure (Hey and Thorne, 1983; Wohl et al., 1996).

2.1.2.1.2 Random Field Approach

The other way to describe the bed roughness is a random field of bed elevations $Z(x, y, t)$, where x and y are the longitudinal and transverse coordinates, and t is time). In this approach, a quantitative description of riverbed roughness is reproduced by means of dimensional probability functions (m). From limited published information an important advantage of the random field approach that makes it preferable to the approach based on the use of the single particle size was recognised. Further investigation in using the random field approach for gravel-bed roughness characterization resulted in the development of a model based on the structure function parameterization (Nikora et al., 1998).

2.1.2.2 Standard Deviation Approach

Determination of flow resistance requires the definition of bed roughness geometry. As mentioned above, a single characteristic of grain size (D_s) derived from the grain size distribution of the surface material is incorporated into large number of flow resistance equations (e.g., Bray, 1979; Bathurst, 1985, 2002; Hey, 1979; Jonker, 2002). Disadvantages of this approach were reported and alternative parameters to be used as roughness definitions were proposed (Nikora et al., 1998; Smart et al., 2002; Aberle and Smart, 2003). The standard deviation of bed elevation s is one of the parameters that have introduced a new characterization of the riverbed geometry (Aberle and Smart, 2003). Through analysis of published experimental data sets of longitudinal bed profiles (Koll, 2002) it was shown that the use of the standard deviation s as a roughness parameter instead of the single grain size (D_s) leads to an improvement in prediction of flow resistance.

2.1.2.3 Volumetric Hydraulic Radius Approach

As already mentioned, the hydraulic radius is a parameter that becomes ambiguous when the bed roughness is large relative to flow depth. The conventional hydraulic radius is the ratio of cross-sectional flow area to wetted perimeter, but estimation of the wetted perimeter for conditions where the roughness elements intersect the water surface is practically impossible. The volumetric hydraulic radius (R_v) and standard deviation of bed surface elevation have been therefore proposed to be used to describe bed roughness for large-scale conditions (Smart et al., 2002). The volumetric hydraulic radius is defined as the volume of overlying water per unit plan area of bed, a definition used before by other researchers (Kellerhals, 1967; Bathurst et al., 1981). A practical procedure for determination of the volumetric hydraulic radius R_v and the standard deviation of bed surface elevation in the field is given in Smart (2001). A graphical representation of the volumetric hydraulic radius is shown in Fig. 2-3.

It was suggested that the standard deviation of bed surface elevations is a rational measure suitable for large relative roughness conditions. The results of the investigation showed that head-losses for large-scale relative roughness could be related to R_v/d_z . The R_v/d_z exponent of power law resistance equations increases from 1/6 to more than 1/2 as relative roughness increases.

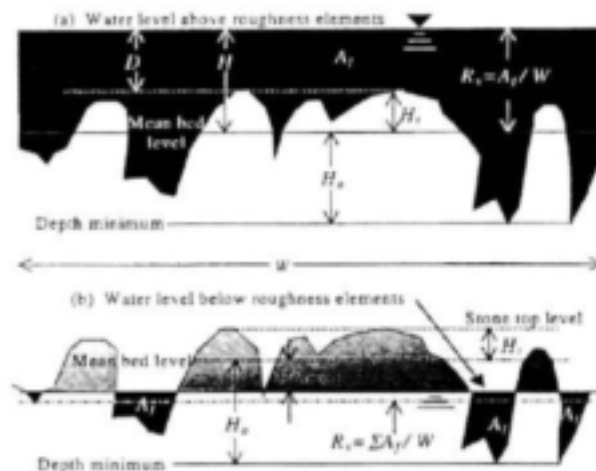


Figure 2-3 Graphical representation of the volumetric hydraulic radius
(Smart et al., 2002)

From the above it is clear that the resistance of flow over rough beds with large and intermediate scales is a complex subject. Prediction of flow resistance requires understanding of the resistance phenomenon. When roughness elements are situated far apart, each acts as an individual body, creating different hydraulic habitats for aquatic animals. Investigations related to flow around a single bed roughness element is therefore discussed in the next section.

2.1.3 Flow around a Single Bed Roughness Element

Flow around a single roughness element presents different patterns. The patterns can be recognized as vortex systems, wakes, and separation points on the bed in front of the roughness element, and on the roughness element itself. Shamloo et al. (2001) used a single hemisphere as a roughness element, and dye plumes which were introduced to upstream of the hemisphere to study formation of these flow patterns. Four flow regimes around the hemisphere based on the relative depth were classified:

Regime 1: When the relative depth is greater than 4, a bed element does not affect the water surface, and the top layer of flow does not mix with the wake. The vortex system around the bed element consists of a horseshoe and arch-vortices.

Regime 2: If the relative depth is in the range of 1.3 to 4, the regime is similar to regime 1, except that surface wakes are apparent.

Regime 3: For relative depths in the range of 1 to 1.3, the free shear layer from the roughness of the body causes mixing through the whole depth of flow as well as some backward flow at the water surface.

Regime 4: When the relative depth is less than 1, the top of the roughness element is above the water surface and a Karman vortex street is present with a strong backward flow behind the element.

The wake geometry, the velocity field and bed shear are different for the different regimes. Downstream of the hemisphere a closed wake recirculation region followed by an open turbulent wake was observed.

Flow around a single cylinder positioned in an open channel has three-dimensional flow patterns (Graf and Yulistiyanto, 1998). Due to the presence of the cylinder, the flow separation is composed of a complex flow system, known as the horseshoe-vortex system (Fig. 2-4).

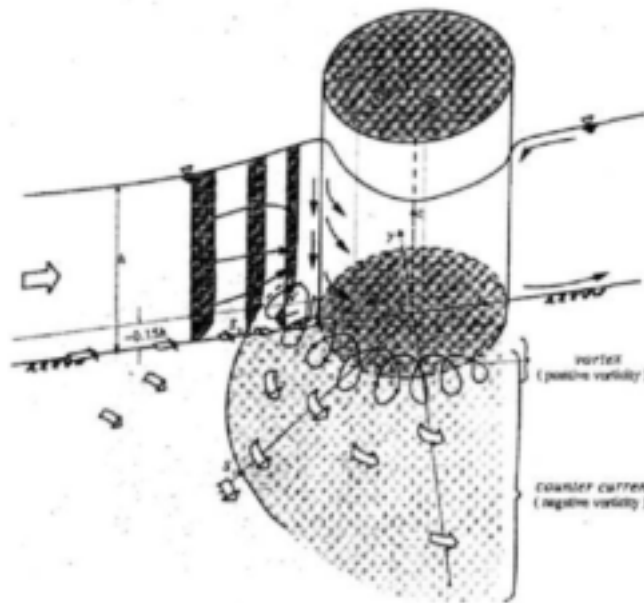


Figure 2-4 Scheme of the horseshoe-vortex system (Graf and Yulistiyanto, 1998)

An Acoustic Doppler Velocity Profiler was used to measure the complex vertical velocity distributions around the cylinder. The flow patterns around the cylinder can be described as follows:

- In the plane of symmetry, the horseshoe-vortex system consists of a vortex, driving a counter-current of negative vorticity.
- The system becomes stronger and closer to the base of the cylinder with increasing flow velocity.
- The horseshoe-vortex system stretches while moving around the cylinder in the streamwise direction. Downstream of the cylinder separation with flow reversal is evident.
- The horseshoe-vortex system produces a high bed-shear stress beneath it.

Lloyd and Stansby (1997) investigated recirculating flow around a conical obstacle with a gently sloping side. Four shapes of conical obstacles called islands were tested

for relative depths less than 1.0. A "wake stability parameter" (S) proposed by Ingram and Chu (1987) was used to classify the island wakes into "vortex shedding" or "unsteady bubble" types. The stability parameter (S) is a measure of the stabilizing effect of bed friction relative to the destabilizing influence of transverse shear given by

$$S = \frac{c_f D}{h} \quad 2.34$$

where c_f : bottom friction coefficient,
 D : cross-stream diameter for the body, and
 H : water depth.

The experiments were designed in such way that different stability parameters were examined. As the depth of the flow decreases, both the magnitude of the bed-friction coefficient (c_f) and the effective diameter of the island (D) increase. Experiments were performed for a range of the stability parameter (S) from 0.06 to 0.40, and blue dye was released upstream of the island to produce images of the recirculating wake zone. The results of the experiments showed that for small values of the wake stability parameter ($S < 0.20$) the island wakes were characterized by a well-organized vortex shedding system. When the stability parameter (S) increased, regular vortex shedding is still occurring, but has moved further downstream. With $S=0.4$, any form of well-organized vortex shedding has ceased to exist, and the wake as an "unsteady bubble" flow was appeared, confirming that bed friction can act to suppress the development of vortex shedding in the wake of conical islands. During the experiments, the time taken for the dye-saturated wake to clear was recorded as well. The results showed that with $S = 0.40$ the time was approximately nine times longer than for $S = 0.06$.

Laboratory investigation of the transition from localized supercritical to sub-critical flow around a single roughness element were carried out by Zgheib (1994). In order to simulate the supercritical-to-subcritical flow transition around a single roughness element, smooth and angular rocks of different size and shapes were placed in a laboratory flume. During the investigation the following hydraulic conditions were identified:

- A deflect jet occurs at the upstream face of the roughness element, resulting from the impact of the flow on the roughness element.
- A spillway effect results from flow of water around the roughness element.
- Intermixing of flow happens along the sides of the roughness element where largest mean velocities appear.
- Hydraulic jumps transform flow from supercritical to sub-critical.

- Tumbling flow is a condition of flow dominated by scattered regions of alternate acceleration and deceleration through critical flow over large bed elements.

Transitional tumbling flow of water around the single roughness element follows one of three possible surface profiles as shown in Figs 2-5 to 2-7.



Figure 2-5 Backward breaking jet (Zgheib, 1994)

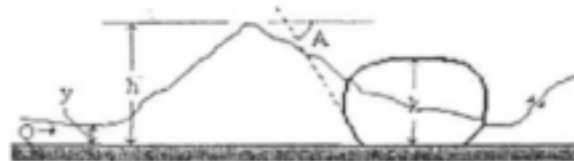


Figure 2-6 Stationary non-breaking jet (Zgheib, 1994)

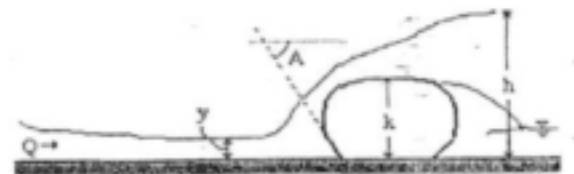


Figure 2-7 Forward shooting jet (Zgheib, 1994)

The conditions of occurrence of each profile depend on the roughness element size and shape, and flow condition. A dimensionless parameter in the form of the Froude number (Fr) is used to distinguish between the three surface profiles, i.e.

$$Fr = Q / (B - w) y^{3/2} g^{1/2} \quad 2.35$$

where B : flume width,
 w : width of the roughness element,
 Q : discharge,

- y : upstream flow depth, and
 g : acceleration of gravity.

For $Fr > 18$, the forward shooting jet occurs, for $12 < Fr < 18$, all three surface profiles are possible, and for $Fr < 12$ the backward breaking and the stationary non-breaking jets occur.

From the above it is clear that flow around the single roughness element is complex and not an easy task to deal with. Numerical simulation is therefore the other way to reproduce the complex flow around a single roughness element positioned in an open channel.

2.1.4 Vegetation and Flow Resistance

Prediction of the effect of vegetation on flow resistance is essential in environmental management of natural systems. A simulation model (James et al., 2001) has been developed to predict basic vegetation resistance by accounting for the fundamental processes involved. The model is based on force balance principles, and accounts for both bed roughness and vegetation resistance. The force applied to the vegetation is described using the well known drag force function with an effective drag coefficient.

The influence of vegetation on the overall resistance of a river channel is considerable, even if it occupies only a relatively small part of the channel, as has been demonstrated by James et al. (2001). It has been shown that the resistance depends not only on the areal extent of the vegetation, but also significantly on its distribution pattern. Partially reeded rivers are common and there is therefore a need for conveyance prediction methods that account for these effects.

Two basic types of reedbed distribution patterns were considered by James et al. (2001): longitudinal strips and discrete patches. Both of these occur naturally in rivers and need to be accounted for in conveyance prediction. Their resistance processes differ, however, and therefore require different treatment. In the case of marginal and mid-channel strips the additional resistance afforded by the reedbeds is through momentum transfer across the interface between the reeded zones, where basic resistance is high and the velocity low, and the clear channel zone where the basic resistance is relatively low and the velocity relatively high. For discrete reedbed patches there is also a significant form resistance contribution to overall resistance which is probably best addressed using a distributed drag force approach, similar to the treatment of individual stems in the basic reed resistance models, but with the applied force balanced by the bed resisting force and vegetation drag components at a larger scale.

Seven different longitudinal strip patterns were studied (James et al., 2001). All of these patterns contained the same total number of stems, with the same local density and the same overall areal coverage of 50% of the channel area - it is only the arrangement pattern was different. The relationships between Manning's n and flow depth for all the strip patterns showed that the distribution of the strips has a significant effect on overall channel resistance. It was found that for basic vegetation resistance, Manning's n varies strongly with flow depth, suggesting that it is inadvisable to use a single value of n for channels with bank vegetation or instream strips of vegetation. A method for predicting the conveyance of channels with strips of reeds, including the common case of strips along the banks was proposed, and is developed further in Chapter 7.

Experimental work on overall flow resistance of discrete patches was performed as well. Fifteen different patterns were tested. For these patterns the areal coverage wasn't constant as for the strip experiments, and ranged from 12.5% to 50% of the channel area. Flow resistance in term of the Manning's n showed a general increase of n with areal coverage, but the wide spread suggested that other influences are important. It was concluded that resistance is strongly influenced by the distribution pattern as well as the overall areal coverage. A method for predicting conveyance for channels with this kind of reedbed distribution has not yet been developed, and the issue is addressed in Chapter 7 of this report.

Naot et al. (1996) carried out an investigation on the hydrodynamics of turbulent flow in partly vegetated open channels. Three channel configurations were studied, consisting of a rectangular open channel with a vegetated bank, a vegetated corner and a vegetated floodplain. A phenomenological model was proposed to predict complex hydrodynamic behaviour. Two sets of experiments of three-dimensional turbulent flow in partly vegetated rectangular channels were conducted for comparison. A nondimensional vegetation density (N) was used to specify the cases, i.e.

$$N = 100nHD \quad 2.36$$

where n : vegetation density (rods per unit area),
 H : flow depth, and
 D : averaged rod diameter.

It was concluded that increase in vegetation density affects the streamwise velocity and the energy of turbulence until the nondimensional vegetation density (N) equals 32, after which the vegetated domain becomes practically impenetrable.

2.2 Multiple Local Controlled Conditions

2.2.1 Introduction

During low flow in a riffle area a wide range of flow depths and velocities occur that are controlled by rocks and boulders, creating local hydraulic features such as hydraulic jumps, local backups, contractions and critical controls. Furthermore, these features occur over the whole riffle area and change with discharge. Multiple local controlled conditions are very complex, and are expressions of rapidly varied flow, as discussed below.

2.2.2 Rapidly Varied Flow

Rapidly varied flow occurs as a sudden or local change of hydraulic conditions (particularly flow depth and velocity) due to variations in channel geometry (particularly bed elevation, channel width, and obstruction to flow) or a change in the regime of the flow. Sudden changes in the channel topography cause flow separations, create eddies and swirls of many forms, and the water profile changes over a short distance (Chadwick et al., 2004). Flow over a region of rapidly varied topography therefore includes local backups, transitions, critical controls and hydraulic jumps (Fig. 2-8).



Figure 2-8 Hydraulic controls resulting from multiple local controls of varying scale

Furthermore, the flow surface may become discontinuous if the flow depth changes rapidly, creating hydraulic jumps. The following characteristics related to rapidly varied flow should be noted (Chow, 1959):

- The pressure distribution cannot be assumed to be hydrostatic.
- The variation in flow regime takes place over a relatively short distance.

- When rapidly varied flow occurs in a sudden-transition structure, the physical characteristics of the flow are fixed by boundary geometry of the structure as well as by the state of the flow.
- The energy (or Coriolis) velocity distribution coefficient α and the momentum (or Boussinesq) velocity distribution coefficient β are usually far greater than unity and cannot be accurately determined.
- The separation zones, eddies, and rollers tend to complicate the flow pattern and to distort the velocity distribution in the stream.

Theory that assumes a parallel flow with hydrostatic pressure distribution, as used for uniform flow and gradually varied flows, does not apply for rapidly varied flow, even with a continuous flow profile.

Some distinct cases of rapidly varied flow phenomena are

- Channel Transitions
- Critical Controls
- Hydraulic Jumps

Under multiple local controlled conditions a combination of several isolated cases of rapidly varied flow exists, which makes estimation of the velocity and depth distributions more difficult. Lately, statistical numerical approaches have therefore become an alternative solution to generalize the velocity and depth distributions of a stream area under low flow conditions. Discussions related to the statistical and numerical approaches are presented in the following sections.

2.3 Statistical Approach

Statistical descriptions of the velocity and depth frequency distributions as functions of hydraulic parameters such as discharge, mean roughness, mean width, and mean depth, are a potentially useful approach for providing the necessary information for environmental flow assessment.

Based on a theoretical and statistical analysis, Dingman (1989) proposed a power law for the cumulative distribution of point velocity in a regular and highly irregular natural stream cross-section, given by:

$$F(v) = (v/V)^c \quad 2.37$$

- where v : local flow velocity,
 V : maximum cross-sectional velocity, and
 c : shape parameter equal to $1/b$, and
 b : slope of a plot of v versus $V(v)$ on log-log scale.

Values of the shape parameter, c , were calculated from field data, and they range from 0.356 to 0.942 for an extensive sample, and from 0.164 to 0.6118 and from 0.497 to 0.912 for intensive samples for pool and riffle cross sections respectively. There is no clear procedure for estimation of the maximum cross-sectional velocity (V) and the shape parameter (c).

Lamouroux et al. (1995) developed a velocity prediction model with distribution parameters that are related to descriptors of hydraulic variables in reaches. They analyzed velocity data of 37 French stream reaches including pools and riffle sites. Relative depths averaged velocities were estimated from three vertical measured points at 0.2, 0.4 and 0.8 above the bed. The measured frequency-velocity distributions varied from centred to decentred distributions.

A probability density function was proposed as a combination of a Gaussian distribution (centred), and Gaussian and exponential distributions, given by:

$$f(x = \frac{v}{V}, s) = s \left\{ 3.33e^{-\frac{x}{0.193}} + 0.117e^{-\left(\frac{x-2.44}{1.73}\right)^2} \right\} + (1-s) \left\{ 0.653e^{-\left(\frac{x-1}{0.864}\right)^2} \right\} \quad 2.38$$

where s : shape parameter of the point velocity distribution,
 v : point velocity,
 V : averaged reach velocity, and
 x : relative point velocity, equal to v/V .

The shape parameter of the velocity distribution, s , was suggested to be a function of Froude number (Fr) and relative roughness (D/H), as:

$$s = -0.275 - 0.237 \ln(Fr) + 0.27(D/H) \quad 2.39$$

where D : averaged dominant roughness, and
 H : averaged reach depth,

A similar approach was applied by Jonker et al. (2001) to predict the distributions of local flow velocities in cobble and boulder rivers of the Western Cape under low flow conditions. The rivers are characterized by different types of morphological units, and three dominant morphological units were recognized. These units are pool, plane bed, and rapid/riffle. Based on observed frequency distributions of relative velocity it was concluded that:

- The Weibull distribution provides the best estimate of the frequency distribution of the relative point velocities within pool morphological units.
- The Weibull and Extreme distributions both provide the best fit to the observed data within the plane bed morphological units.

- Within rapid and riffle morphological units the Extreme distribution displays the best fit to the observed data, followed by the Weibull distribution.

For predicting the frequency distribution of relative velocity in cobble and boulder reaches, the following hydraulic parameters were selected as possible explanatory variables:

- Froude number
- Velocity/depth ratio
- Reynolds number
- Relative submergence (y/d_{50} or y/d_{84})
- Width/depth ratio

Through analyses of the velocity data it was found that within pool and rapid/riffle morphological units the distribution of local sets of point velocities could be related to the average velocity, flow depth, flow width and relative roughness within a cross-section, and used to predict the frequency distribution of point velocity in these morphological units. For plane bed morphological units, no statistically significant relationships were found. The accuracy of the proposed predictions has not been checked, since the models have not been verified against independent field data.

Lamouroux (1998) proposed depth probability models developed based on analyses of data from different stream reaches in France and Germany. The reaches contained several pool-riffle sequences. It has been found that depth probability distributions tend from exponential to normal with increasing stage. The depth probability distribution was expressed as:

$$f\left(x = \frac{h}{H}, t\right) = t e^{-x} + 0.951(1-t) e^{-\left(\frac{x-t}{0.593}\right)^2} \quad 2.40$$

where t : shape parameter varying from 0 (normal distribution)
to 1 (exponential distribution),

x : relative depth, equal to h/H (h is flow depth, and H is mean reach depth).

Prediction of depth distribution by this model as a function of discharge requires an estimation of the shape parameter at a given stage and the depth-discharge relationship for the reach.

Stewardson and McMahon (2002) proposed a stochastic model of the joint depth and velocity probability distribution in streams. The model is based on theoretical considerations and samples of velocity and depth from a wide range of stream types, and was proposed to quantify the joint probability distribution of depth and velocity

for general application in river studies. The model is based on the assumption that if velocity increases with depth across a channel, and velocity decreases with depth along a channel, then velocity and depth are not independent variables. Then, depth and velocity are transformed to provide two independent variables, one (ψ_a) that varies across the channel but is invariant along the channel, the other (ψ_b) that is invariant across the channel, but varies longitudinally. It has been found that the probability distributions of ψ_a and ψ_b can be represented by a normal and a truncated normal density functions, respectively. Four parameters related to channel geometry were proposed for these density functions.

From the above it can be seen that:

- The model of Lamouroux et al. (1995) has been developed for pool-riffle sequences and not homogeneous geomorphologic features, but appears to be the most tested one.
- In general, all statistical models of the available velocity distribution in the literature do not distinguish between the two possible flow conditions: multiple local controlled and resistance controlled conditions.

2.4 Numerical Modelling

The flow structure over a rough bed is very complex, and two or three-dimensional numerical modelling is an option for simulation of such complex phenomenon.

Nikora et al. (2001) suggested that double averaged momentum equations could be used as a natural basis for the hydraulics of rough-bed open channel flows. Relationships for the vertical distribution of the total stress for 2D, steady, uniform, spatially averaged flow over a rough bed with flat free surface were derived. The relationships include the form-induced stresses and form drag as components of the total stress. Three types of rough bed flows were define as follows:

- Flow with high relative submergence.
- Flow with small relative submergence.
- Flow over a partially inundated bed.

The following relationships for the condition of small relative submergence (Eqs 2.41 and 2.42) as well as of partially inundated bed (Eqs 2.43 and 2.44) were suggested

$$\sqrt{\frac{8}{f}} = Cm \frac{H}{\delta} \quad 2.41$$

or

$$f = \frac{8}{C^2 m^2} \left[\frac{\delta}{H} \right]^2 \quad 2.42$$

and

$$\sqrt{\frac{8}{f}} = \frac{\alpha m}{\gamma} \frac{\delta}{H} \quad 2.43$$

or

$$f = \frac{8\gamma^2}{\alpha^2 m^2} \left[\frac{H}{\delta} \right]^2 \quad 2.44$$

where f : Darcy-Weisbach friction factor,
 α : slope,
 m : parameter depending on roughness geometry,
 C : parameter of velocity distribution,
 δ : boundary between logarithmic and linear flow regions, and
 H : maximum flow depth.

The developed relationships were compared with some measurements, and should be considered as preliminary approximations that need additional testing.

Recent advances in numerical modelling mean that now it is possible to use two and three dimensional models to simulate flow patterns under low flow conditions. Nicholas (2003) used Hydro2de to simulate flow within a braided reach of the Avoca River, New Zealand. Field measurements obtained at low flow conditions were used to validate performance of the model. Comparison of modelled and measured variables showed that field data exhibit greater spatial variability than model led results, but generally the model led results reproduce the systematic trends in measured data.

Two- and three-dimensional models are powerful tools that are now in common use for generating hydraulic information. These types of models have been used to simulate flow patterns within channels, overbank flows, and flows in estuaries (Hervouet and Van Haren, 1996; Connell et al., 1998; Stewart et al., 1999) showing good correlation with field data.

2.5 Conclusions

From the literature review it can be seen that hydraulics under low flow conditions has attracted much research attention. Different approaches and models for different scales (from a reach to a single obstacle) and conditions (large and intermediate relative roughness) have been proposed. Many different equations (logarithmic, power and semi-logarithmic) for prediction of flow resistance have been developed. However, some of them are modifications of Manning's, Darcy-Weisbach or Chézy

equations, while others are not clearly restricted to either the large or intermediate relative roughness condition.

Prediction of velocity and depth distributions is an essential part of an environmental study. A number of statistical models, based on field data have been developed. Application of these models must be carried out with care and understanding of their limitations.

Two- and three-dimensional models are powerful tools that are now in common use for generating hydraulic information.

3 EXPERIMENTAL INVESTIGATION OF RESISTANCE CONTROLLED CONDITIONS

3.1 Introduction

Large substrate material is a common feature of South African rivers. Under low flow conditions rocks are relatively large and exercise control on flow depth and velocity. Individual roughness elements within a natural channel vary in number, size, shape and distribution, and therefore create a wide range of physical habitats for aquatic animals. Prediction of flow depth and velocity for a given discharge is therefore an important part of environmental studies.

To gain better understanding of the influence of roughness elements on depth and velocity, laboratory experiments were conducted in the hydraulics laboratory of the School of Civil and Environmental Engineering, University of the Witwatersrand.

An experimental programme was carried out in laboratory flumes under controlled and idealized situations in order to establish the effects of roughness elements on flow resistance under different hydraulic conditions determined by bed slope and discharge, and to develop resistance prediction methods. Experiments were carried out using different sizes and areal densities of roughness elements. Roughness elements were simulated by hemispherical shells constructed of concrete. Laboratory experiments were conducted in Flume B (Series 1) and Flume C (Series 2). The results of the experimental work are presented in this chapter.

3.2 Experimental Investigation

3.2.1 Flume B Experiments

Under low flow conditions, resistance is determined by the largest or most exposed rocks on the bed. Their effect is thought to be related to their degree of submergence and preponderance in the substrate. The first set of experiments was carried out to assess these effects, using single-sized roughness elements at different areal densities and under different flow depths. The experiments were conducted in a 0.38m wide, 15.0m long, glass-sided tilting laboratory flume under controlled and idealized conditions. A tailgate fixed downstream of the flume was used to control the flow depth in the channel to ensure uniform flow. Water was supplied to the flume through a closed circulation system, and two valves situated in the supply pipe at the head of the experimental flume were used to control the discharge. The discharge was varied by opening/closing these control valves and measured using a V-notch, which was installed at the downstream end of the flume, as well as an electronic flow meter with sensors that are situated in the water pipe that discharges into the flume. All

experiments were carried out under uniform flow conditions. Experiments were carried out for two size of roughness elements, with diameters $D = 47\text{mm}$ (Series 1.1) and $D = 72\text{mm}$ (Series 1.2); the corresponding roughness heights, h , were therefore 23.5mm and 36mm. The roughness elements were arranged in a staggered pattern with equal longitudinal and transverse spacings (Fig. 3-1).



Figure 3-1 Arrangement of roughness elements in Flume B (Test 1.1.5 shown)

Series 1.1 experiments were performed with one size of roughness elements ($D = 47\text{mm}$), two bed slopes and four densities, with a range of discharges (Q) with corresponding flow depths (y) for each set-up (Table 3-1). (The density, Λ , is defined as the ratio of the plan area of the elements to the channel area.) The Darcy-Weisbach f values were calculated for each experiment using the side-wall correction procedure of Vanoni and Brooks (1957), and results are plotted in Fig. 3-2. The experimental conditions of Series 1.1 experiments are summarized in Table 3-1.

Table 3-1 Experimental conditions of Series 1.1 experiments

Test	Density, Λ (%)	Slope	y/h	Discharge, Q (l/s)
1.1.1	82	0.0011	1.45 – 6.14	0.6 – 15.2
1.1.2	82	0.0021	1.29 – 4.72	0.4 – 15.2
1.1.3	47	0.0011	2.28 – 6.43	1.4 – 14.8
1.1.4	30	0.0011	1.85 – 6.85	0.9 – 15.9
1.1.5	22	0.0011	1.26 – 6.19	0.4 – 13.9

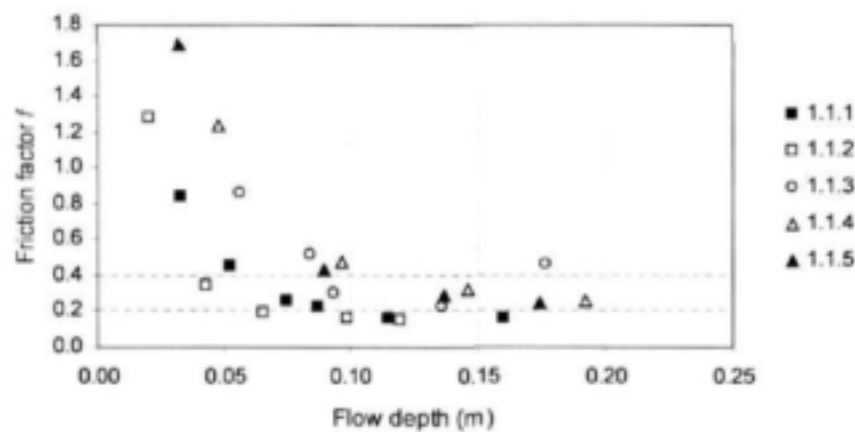


Figure 3-2 Variation of friction factor f with flow depth for Series 1.1 experiments

Tests 1.1.1 and 1.1.2 were carried out with the same density but different bed slopes, showing that the slope has a small influence, with resistance being slightly lower for the steeper slope (Fig. 3-2).

Tests 1.1.1, 1.1.3, 1.4 and 1.1.5 were conducted with the same bed slope but different densities. The influence of density on flow resistance for five different relative submergences is shown in Fig. 3-3.

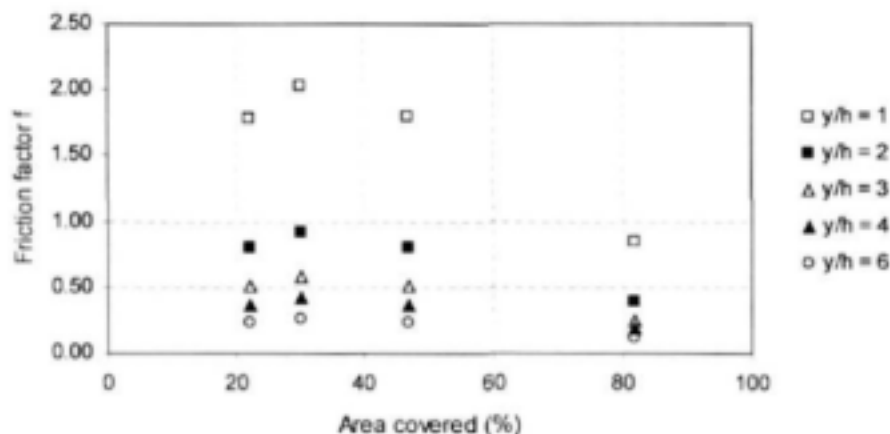


Figure 3-3 Influence of areal coverage on flow resistance in term of Darcy-Weisbach f for different relative submergences

From the graph (Fig. 3-3) it is clear that density of the roughness elements had significant effect on overall flow resistance. It also can be seen by comparing curve " $y/h = 1$ " (for the relative submergence equal to 1) with curve " $y/h = 6$ " (for the

relative submergence equal to 6) that the effect of density on overall resistance decreases with increasing relative submergence. Furthermore, the flow resistance increases with density, reaching its highest value at an areal coverage of 30%, and then decreasing for higher densities. This effect is consistent for all flow conditions, from large to small relative submergences.

The field data of Bathurst (2002) suggest that the effective friction factor under low-flow conditions depends on channel slope. Series 1.2 experiments were performed to investigate the influence of bed slope on flow resistance under controlled conditions. Experiments were carried out with one size of roughness element ($D = 72\text{mm}$), 5 bed slopes and one density (27%). Two (low and high) discharges for each slope were tested. The experimental conditions are listed in Table 3-2. As before, the flow resistance in terms of Darcy-Weisbach f was calculated using the side-wall correction procedure of Vanoni and Brooks (1957), and the results are plotted in Fig. 3-4.

Table 3-2 Experimental conditions for Series 1.2 experiments

Test	Slope	y/h	Discharge, Q (l/s)
1.2.1	0.00443	1.01 and 3.95	0.88 and 22.00
1.2.2	0.00329	0.95 and 3.96	0.63 and 18.00
1.2.3	0.00215	1.03 and 4.53	0.57 and 17.00
1.2.4	0.00101	1.01 and 5.42	0.29 and 13.00
1.2.5	0.00089	1.07 and 7.93	0.30 and 22.00

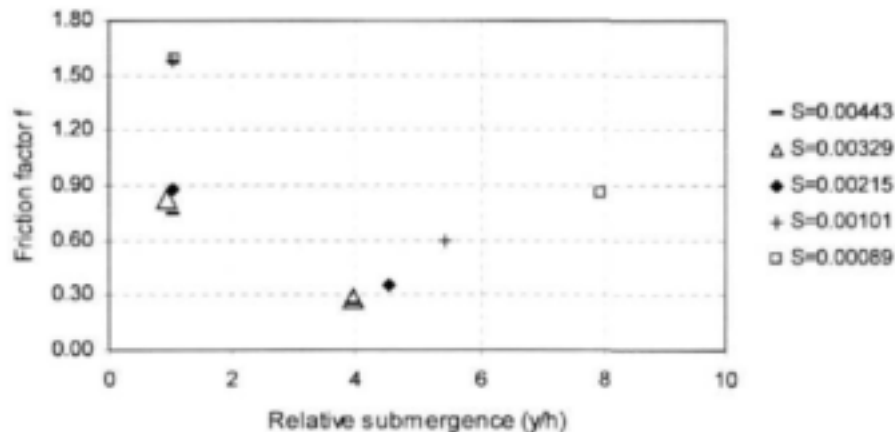


Figure 3-4 Influence of bed slopes on overall flow resistance

The results suggest that channel slope does have an influence on resistance; the f values are higher for milder slopes than for steeper one.

The Series 1 experimental data are listed in Appendix A, Table A-1.

3.2.2 Flume C Experiments

The largest rocks on a river bed are distributed randomly in space. This series of experiments was designed to examine some effects on overall resistance of the size of rocks and their distribution pattern, in particular the effect of longitudinal disruption of the flow and the effect of smaller rocks interspersed amongst the largest ones. Experiments were carried out in a 2.00m wide, 12.0m long laboratory flume. Two Series (2.1 and 2.2) of experiments were conducted to investigate these effects on flow resistance under controlled and idealized large and intermediate scale roughness conditions. Roughness elements were again simulated by hemispheres with constant shape.



Figure 3-5 Flume C with a staggered pattern of hemispheres

3.2.2.1 Series 2.1 experiments

The first series of experiments was carried out to investigate the influence of roughness element size and pattern on overall flow resistance. Eight different arrangements of roughness elements were tested, shown diagrammatically in Fig. 3-6 and described below; photographs of all patterns are also included in Appendix B.

Pattern 1: (Large hemispheres, LH, only) The hemispheres, $D = 116\text{mm}$, were arranged in a staggered grid pattern with equal longitudinal and transverse spacing of 285mm (Appendix B, Fig. B-1).

Pattern 2: (Large, LH, and small, SH, hemispheres) The large hemispheres had the same arrangement as in Pattern 1, and small hemispheres ($D = 54\text{mm}$) were placed between and in line with large hemispheres (Appendix B, Fig. B-2). The small hemispheres were arranged in a staggered pattern as well with the same spacing of 285mm as the large one.

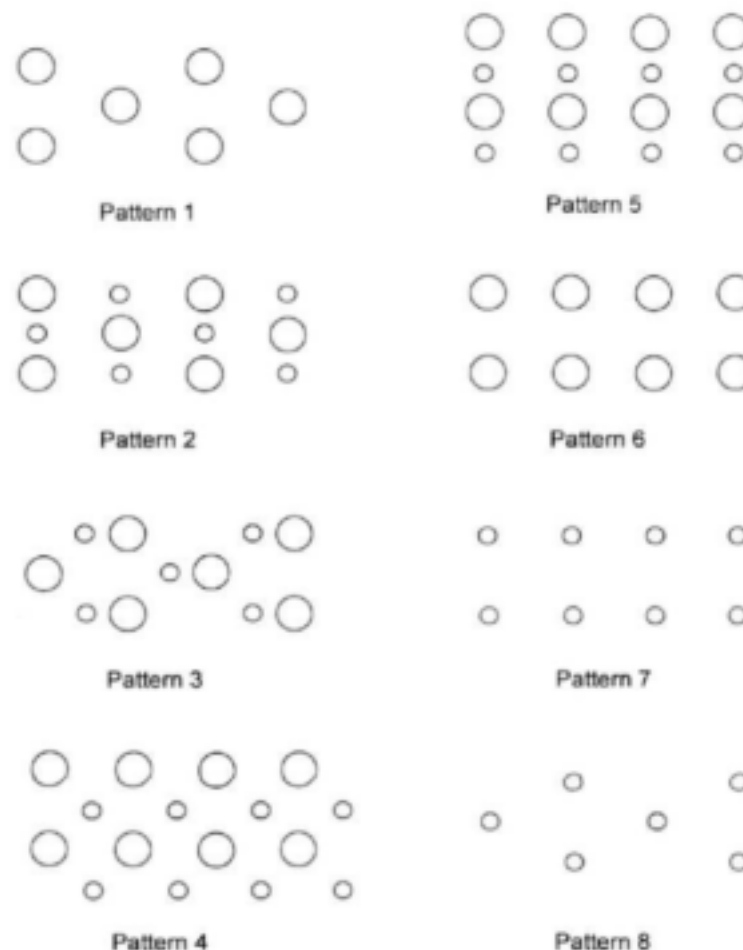


Figure 3-6 Roughness element arrangements for Series 2.1 experiments (flow is left to right)

Pattern 3: (Large, LH, and small, SH, hemispheres) The large hemispheres were placed as in Pattern 1 while the small ones were positioned longitudinally in line with, but transversely staggered with respect to the large hemispheres (Appendix B, Fig. B-3). Each size individually was arranged in a staggered pattern with spacing of 285mm .

Pattern 4: (Large, LH, and small, SH, hemispheres) The large hemispheres were arranged in a parallel grid pattern with equal longitudinal and transverse spacing of 285mm. The small hemispheres were arranged in the same pattern as the large ones, but longitudinally and transversely between them (Appendix B, Fig. B-4).

Pattern 5: (Large, LH, and small, SH, hemispheres) The hemispheres were arranged in a parallel grid pattern. The large hemispheres were arranged in a parallel pattern of 285mm spacing. A longitudinal row of small hemispheres was placed between each longitudinal row of large hemispheres (Appendix B, Fig. B-5).

Pattern 6: (Large hemispheres, LH, only) The hemispheres were arranged in a parallel grid pattern with equal longitudinal and transverse spacing of 285mm (Appendix B, Fig. B-6).

Pattern 7: (Small hemispheres, SH, only) The hemispheres were arranged in a parallel grid pattern with equal longitudinal and transverse spacing of 285mm (Appendix B, Fig. B-7).

Pattern 8: (Small hemispheres, SH, only) The hemispheres were arranged in staggered grid pattern with equal longitudinal and transverse spacing of 285mm (Appendix B, Fig. B-8).

Experiments were performed for both emergent and submerged conditions. Only one slope of 0.001 and two sizes of hemispheres, $D = 114\text{mm}$ (LH) and $D = 54\text{mm}$ (SH) were tested. A summary of experiments is listed in Table 3-3, and measured data are presented in Appendix B, Table B-1.

Table 3-3 Experimental conditions of Series 2.1 experiments

Pattern	Roughness shape	Discharge, Q (l/s)	Measured flow depth, y (mm)	Covered area (%)
1	LH	2.7 - 17.4	25 - 74	15
2	LH and SH	1.2 - 12.5	20 - 69	18
3	LH and SH	2.9 - 21.4	28 - 80	17
4	LH and SH	2.0 - 25.7	20 - 85	17
5	LH and SH	1.8 - 24.7	20 - 82	18
6	LH	3.2 - 27.3	25 - 85	15
7	SH	7.0 - 28.3	29 - 55	3
8	SH	5.3 - 28.3	25 - 56	3

LH - Large Hemispheres

SH - Small Hemispheres

Stage-discharge relationships for all 8 patterns, together with that corresponding to the basic resistance of the empty flume, are plotted in Fig. 3-7. The effect of roughness element arrangements in term of the Darcy-Weisbach friction factor f with the relative submergence for all 8 patterns is shown in Fig. 3-8. It should be noted that the relative submergence for Patterns 2, 3, 4 and 5, where two sizes of hemisphere comprised the bed arrangements, was calculated using the height of the large hemisphere.

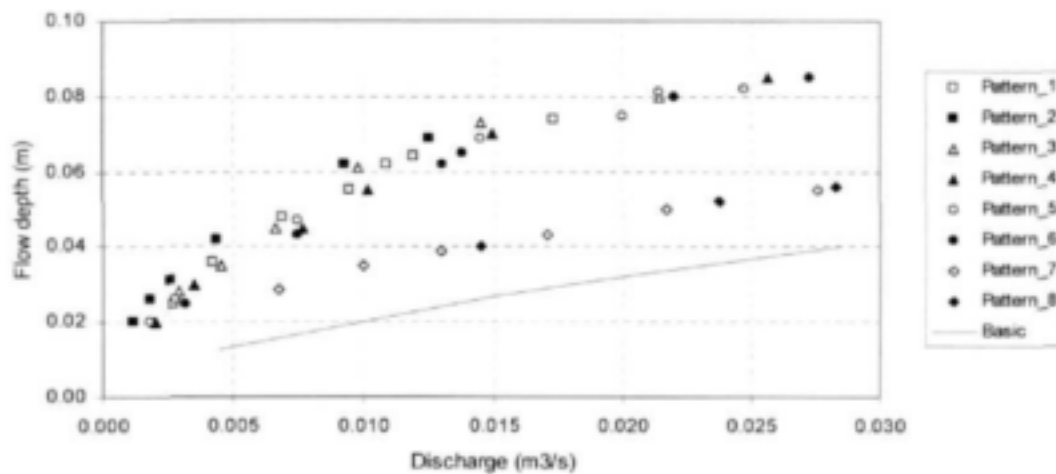


Figure 3-7 Stage-discharge relationship for Series 2.1 experiments

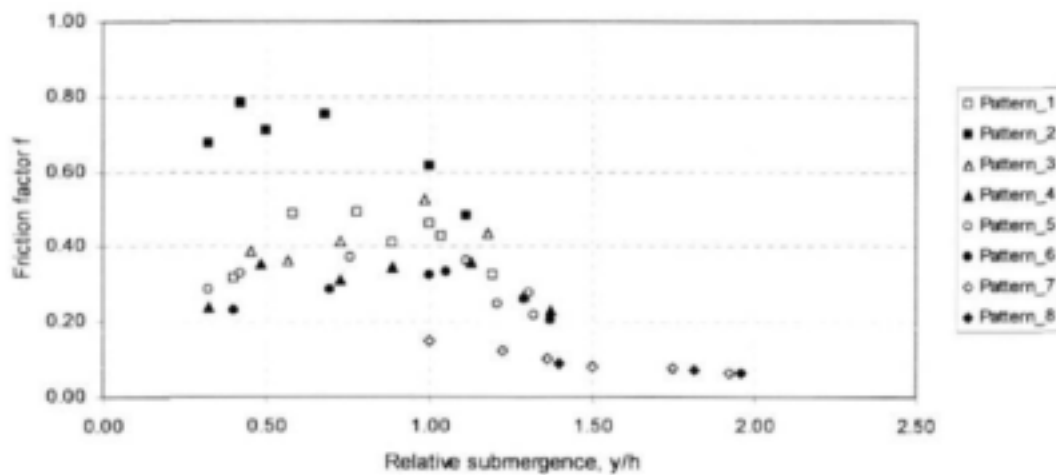


Figure 3-8 Flow resistance in term of Darcy-Weisbach f for Series 2.1 experiments

Pattern 1 and 8 were arranged in a staggered grid. The large hemispheres were used in Pattern 1, the small in Pattern 8. Patterns 6 and 7 were arranged in a parallel grid with the large and small hemispheres respectively. The effect of the roughness element pattern on overall flow resistance in term of Darcy-Weisbach f against the relative submergences is shown in Fig. 3-9. By comparing the graphs of Patterns 1 and 6 it can be seen that the arrangement of the roughness elements has a significant influence on overall resistance for relative submergences less than 1.0 (i.e. under large scale roughness condition) but with increasing the relative submergence (Pattern 7 and 8) the effect of pattern is insignificant.

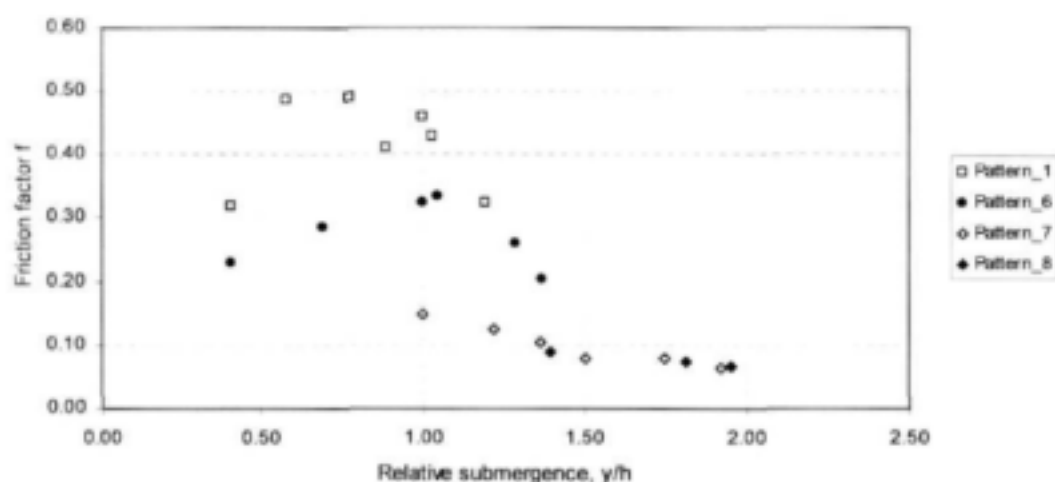


Figure 3-9 Effects of roughness element pattern and size on overall flow resistance, Series 2.1 experiments

In Patterns 2, 3, 4 and 5 the small hemispheres were arranged within staggered and parallel grids of the large hemispheres; their influence on overall resistance is shown in Fig. 3-10.

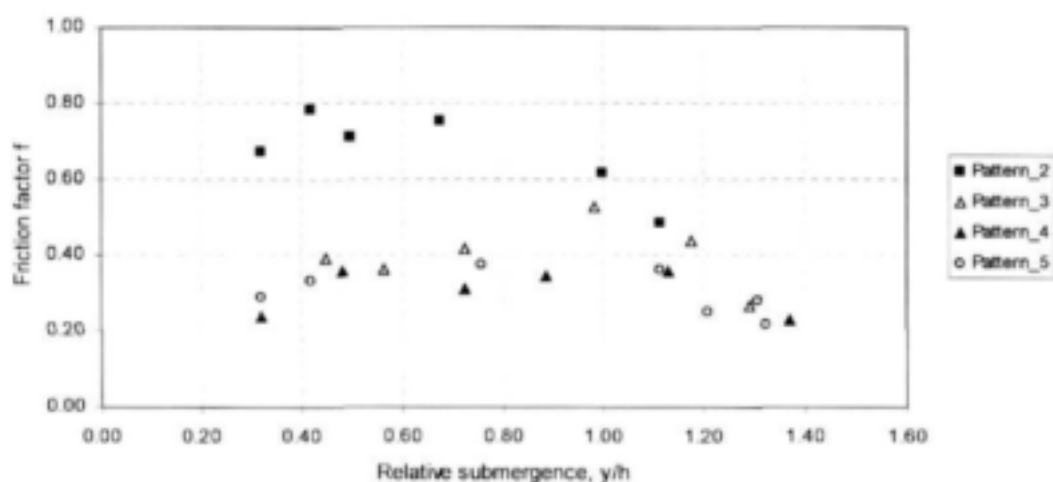


Figure 3-10 Influence of small elements interspersed between large ones on overall flow resistance, Series 2.1 experiments

From the graph it is clear that the roughness element arrangement in Pattern 2 has the highest influence on flow resistance under large-scale roughness condition, for relative submergence less than 1.0, while for the same flow condition, results of Patterns 3, 4 and 5 arrangements are similar. For the intermediate-scale roughness

flow condition with the relative submergence higher than 1.0, the effects of all the small hemisphere patterns within the large hemispheres are similar.

3.2.2.2 Series 2.2 experiments

Experiments were conducted with one slope of 0.0005 and three sizes of hemispheres named H1, H2 and H3 with diameters of $D1 = 108\text{mm}$, $D2 = 72\text{mm}$ and $D3 = 46\text{mm}$ respectively. A summary of the experimental conditions for the Series 2.2 experiments is listed in Table 3-3. Measured data from each experiment are included in Appendix D, Table D-1. Photographs of Patterns 1 to 17 are in Appendix D in Figs D-1 to D-17. The geometrical arrangements are similar to those used in Series 2.1 and shown in Fig. 3-6 : Patterns 1 to 7 are as for Series 2.1 Pattern 1, Patterns 8 to 14 as for Series 2.1 Pattern 3, and Patterns 15 to 17 as for series 2.1 Pattern 6.

Experiments with Patterns 1, 2, 3, 4, 15, 16 and 17 were carried out with hemispheres H1 only. The results of the experiments can be used to analyse the influence of roughness element density on overall flow resistance. Measured flow resistance, in term of Darcy-Weisbach f values, is plotted in Fig. 3-11. It is clear that roughness element density affects flow resistance significantly. The effect is much greater under large-scale roughness conditions (relative submergence less than 1.0) than under intermediate-scale roughness conditions (relative submergence greater than 1.0). It can also be seen that flow resistance increases with flow depth under large-scale roughness, reaching the highest resistance when the roughness elements are just submerged, before decreasing through the intermediate-scale roughness zone. The influence of the roughness element density decreases with increasing flow depth in the intermediate-scale roughness zone. The dependence of flow resistance on the areal coverage of roughness elements at the condition of maximum resistance is shown in Fig. 3-12, where f is plotted against areal coverage for relative submergences equal to 1.0. This shows that resistance increases with areal coverage, reaching a maximum value at an areal coverage of about 40%, and then reduces. The variation in Fig. 3-12 is consistent with the results of the Flume B experiments presented in Fig. 3-3.

Table 3-4 Experimental conditions for Series 2.2 experiments

Pattern No	Spacing H1 (mm)	Spacing H2 (mm)	Spacing H3 (mm)	Pattern grid	Discharge, Q (l/s)	Measured flow depth, y (mm)	Total areal coverage (%)
1	125/125			Staggered	0.6-55.7	32-189	54.64
2	200/200			Staggered	1.1-54.7	24-164	21.97
3	300/300			Staggered	2.0-55.2	23-129	12.02
4	400/400			Staggered	2.7-55.2	23-121	6.24
5		400/400		Staggered	3.2-55.2	23-107	2.89
6		200/200		Staggered	2.8-60.0	34-130	9.82
7		125/125		Staggered	4.1-55.2	50-149	23.98
8	200/200	200/200		Staggered	0.7-54.7	20-164	31.79
9	200/200		200/200	Staggered	0.8-43.4	20-145	26.13
10	300/300	300/300		Staggered	1.4-55.2	24-137	16.06
11	300/300		300/300	Staggered	2.0-55.2	26-131	13.73
12	400/400	400/400		Staggered	3.7-55.2	31-129	8.10
13	400/400	400/400		Staggered	2.5-52.2	27-131	8.10
14			200/200	Staggered	3.6-55.7	36-136	4.16
15	110/110			Parallel	0.4-25.0	43-123	74.91
16	110/125			Parallel	0.6-21.4	40-117	62.89
17	125/167			Parallel	0.6-7.8	31-82	41.62

Spacing of hemispheres indicated in the table is given as longitudinal/lateral centre to centre distances

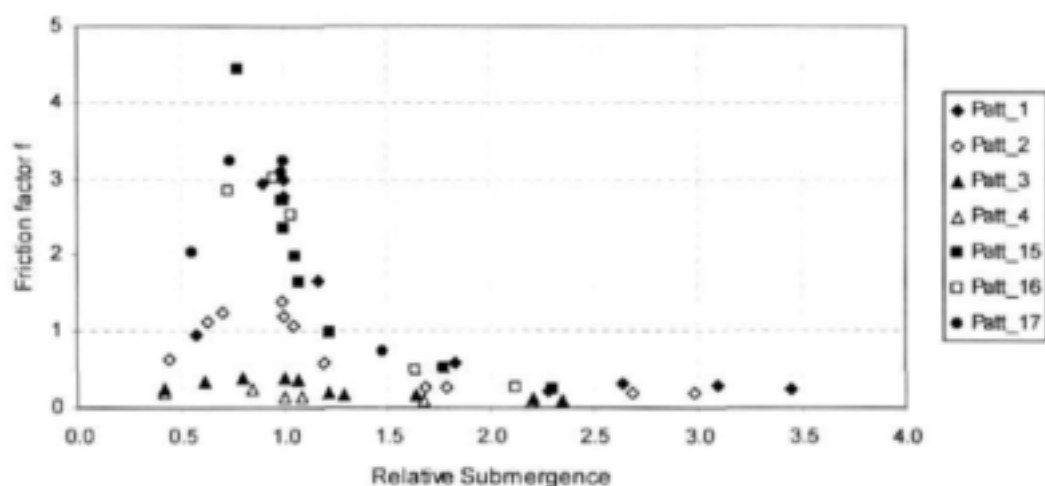


Figure 3-11 Flow resistance in term of Darcy-Weisbach f against relative submergence for patterns with hemispheres H1 only

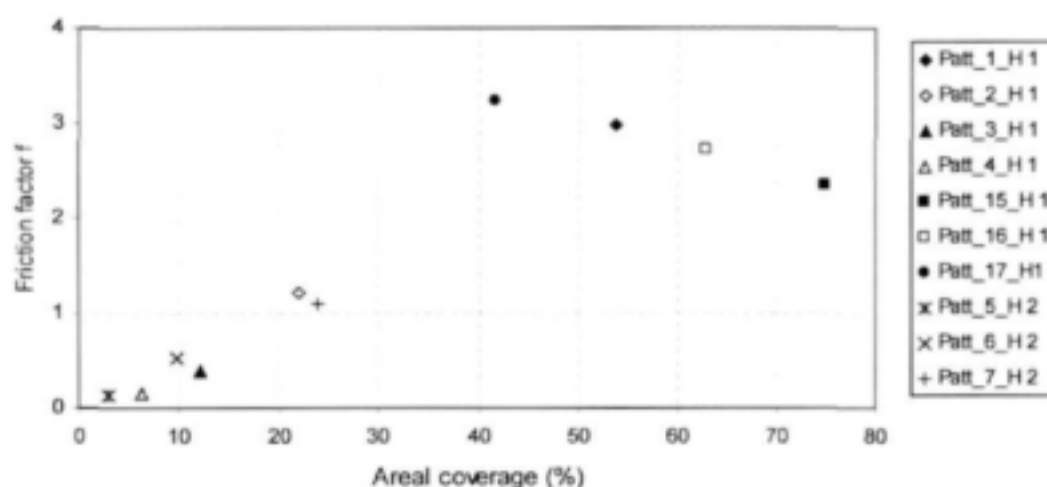


Figure 3-12 Influence of areal coverage on flow resistance in term of Darcy-Weisbach f

Experiments with Patterns 8, 9, 10, 11, 12 and 13 were performed to investigate the influence of smaller roughness elements (H2 and H3) within the patterns of the large (H1) hemispheres on overall resistance. Patterns 8, 10, 12 and 13 were carried out with hemispheres H1 and H2 placed in equal longitudinal and lateral spacings of 200mm, 300mm and 400mm (Table 3-4), while Patterns 9 and 11 consisted of hemispheres H1 and H3, arranged with spacings of 200 and 300mm respectively. Patterns 12 and 13 were performed with the same hemispheres and spacing but with a different pattern for the H2 hemispheres. The results of these experiments are presented in Fig. 3-13. It is evident that Patterns 8 and 9, with spacing of 200mm, produced the highest resistance, consistent with the previous observation of increasing resistance with density in the large-scale roughness zone. It can also be seen that the difference in resistance between Patterns 8 and 9 is not significant, suggesting that the size of interspersed smaller hemispheres has little influence on overall flow resistance.

Patterns 8 and 9 were arranged with hemispheres H1 and H2, and H1 and H3, respectively. The hemispheres H1 were positioned with spacing of 200mm; the smaller hemispheres were placed within the larger ones with the same spacing. Patterns 2, 6 and 14 were arranged with one size of hemispheres only with spacings of 200mm. Results of the experiments carried out for these patterns (Fig. 3-14) shows the influence of different hemisphere arrangements on overall flow resistance. The similarity of the results for Pattern 2 (hemispheres H1 only) with those for patterns 8 and 9 (with interspersed H2 and H3) suggests that resistance is caused primarily by the largest clasts in a cobble or boulder river bed. The relationship between the results for Patterns 2, 6 and 14 confirms the influence of areal coverage in the large-scale roughness zone.

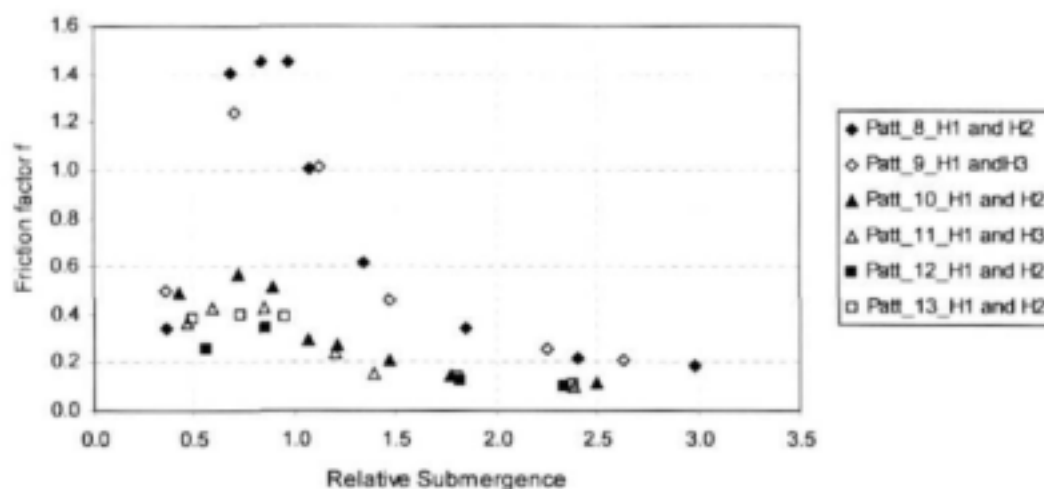


Figure 3-13 Variation of flow resistance in term of Darcy-Weisbach f for different combinations of hemisphere sizes

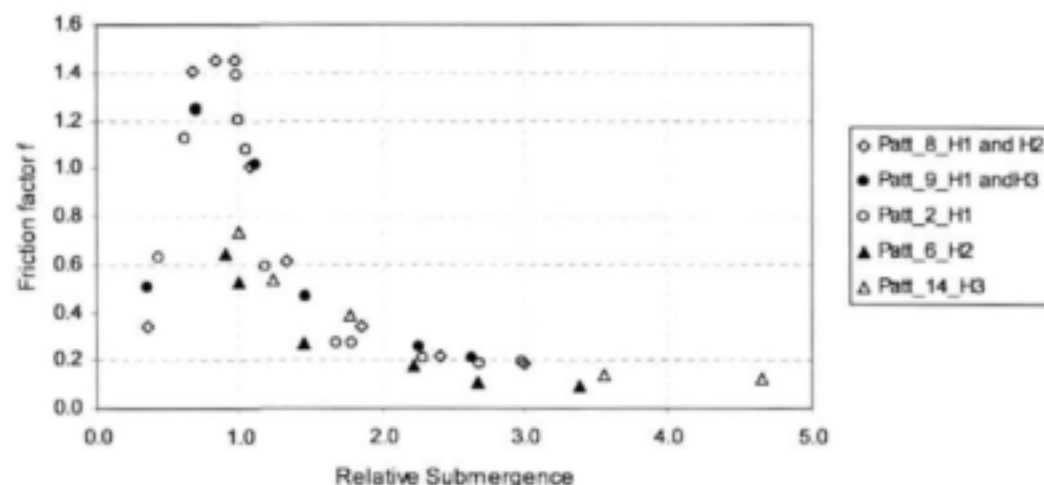


Figure 3-14 Flow resistance in term of Darcy-Weisbach for patterns with spacing of 200mm

3.2.3 Conclusions

This experimental investigation has shown that the density of large roughness elements on the bed of a channel has a significant influence on the overall flow resistance of the channel. The resistance due to the roughness elements increases with increasing density, reaching a maximum for an areal coverage in the range of 30% - 40%. Thereafter, overall resistance decreases with increasing density. This general variation occurs for all scales of relative roughness.

Experiments with different bed slopes indicate higher overall flow resistance for milder slopes than for steeper ones.

Experiments with different size of hemispheres suggest that resistance is caused primarily by the largest clasts in a cobble or boulder river bed.

Experiments with staggered and parallel arrangement patterns of large roughness elements indicate that the existence of continuous longitudinal flow paths decreases flow resistance significantly.

Experiments performed for large- and intermediate-scale roughness conditions confirmed that the resistance phenomenon under these conditions is different, and different methods are therefore required for its prediction. Flow resistance is a maximum for a relative roughness around 1.0; it decreases rapidly for deeper flows and gradually for shallower flows. The effects of roughness element size, density, pattern, and interspersed smaller elements are all much more pronounced for large-scale roughness conditions than for small-scale and intermediate scale conditions.

In channels with different sizes of large roughness elements, the overall resistance is caused primarily by the largest ones.

3.3 Velocity Distribution

Flow velocity is a physical parameter that is useful in the description of aquatic animals' habitats. The average velocity (ratio of discharge to cross-sectional flow area) is not a sufficient habitat descriptor for use in environmental studies, however, and prediction of local velocity distribution is usually required. The velocity distribution changes with flow conditions, making its prediction difficult. Laboratory tests were conducted to investigate how velocity distributions changes with discharge, and to provide a data base for testing the recommended prediction method (Chapter 6).

Point velocity measurements were taken around one hemisphere within Pattern 6 of the Series 2.2 described above (Appendix C, Fig. C-6) for two discharges of $Q = 21\text{ l/s}$ and $Q = 3\text{ l/s}$. A two-dimensional *Nortek Doppler Velocimeter* (NDV) was used to determine the velocity profile around a hemisphere. It was assumed that the velocity profile is symmetrical about the longitudinal axis of the hemisphere, and velocities were therefore measured only on one side. A grid of 20cm by 20cm around the hemisphere was defined (Fig. 3-15) and velocities were measured in each block, numbered as indicated.

3.3.1 Velocity Measurement for $Q = 3.0 \text{ l/s}$

A uniform depth of 36mm was measured for a discharge of 3.0l/s. A plan on the velocity measured grid is shown in Fig. 3-15. The hemispheres were just submerged, and only one point velocity was measured along the vertical at each grid block at 0.5 of the flow depth. Measured velocities are listed in Table D-1, Appendix D. An average velocity of 0.044m/s with discharge of 3l/s was calculated as the ratio of flow to cross-sectional flow area. The measured velocities were grouped into 2cm/s velocity classes. The velocity distribution in histogram format to display the frequency of occurrence of each value in the data set is plotted in Fig. 3-16.

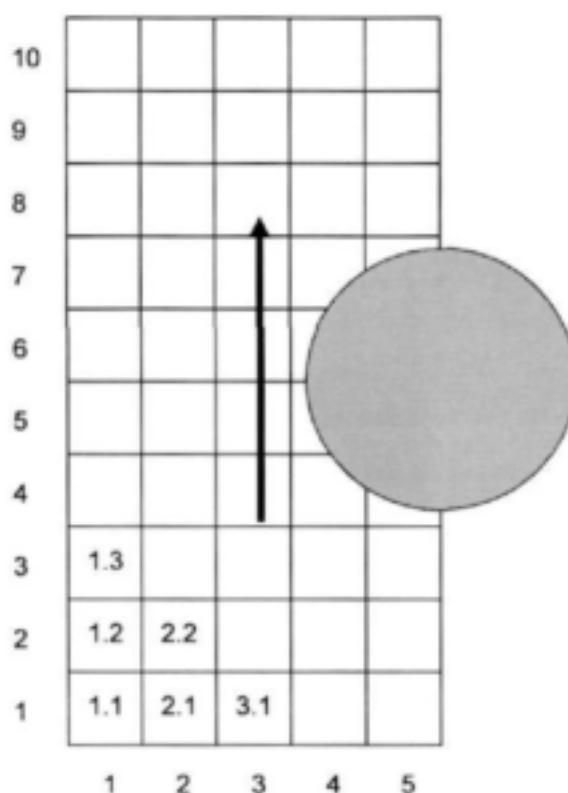


Figure 3-15 Velocity measurement grid

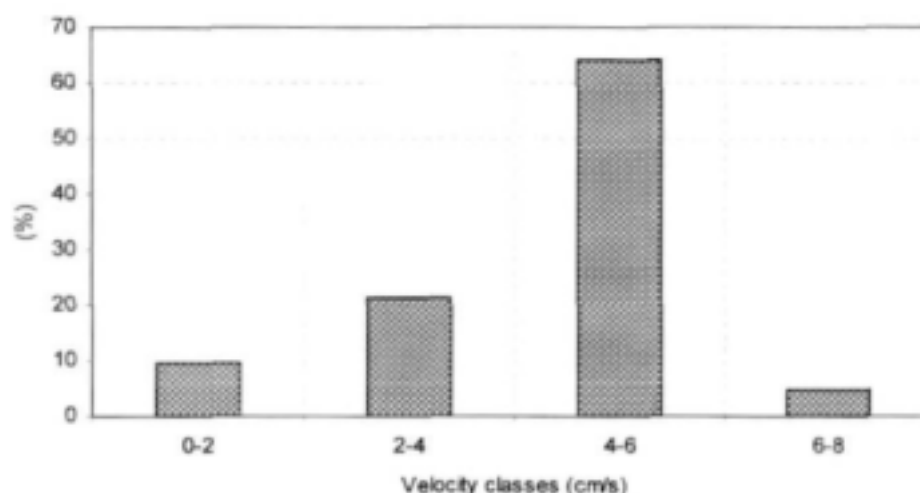


Figure 3-16 Velocity distribution histogram with 0.02 m/s velocity classes for $Q=3.0$ l/s

The histogram is unimodal (has one mode) and skewed to the left. The mode is in the class of 4 to 6 cm/s. The lowest and the highest measured velocities for the given discharge are in the 0 to 2 cm/s and 6 to 8 cm/s velocity ranges respectively.

The average velocity of 4.4 cm/s fell into the class of velocity that represents the highest (64.3) percent of measured values. The other measurements are distributed across three other classes, but with less frequent occurrences.

3.3.2 Velocity Measurement for $Q = 21$ l/s

The uniform flow depth for the test with a discharge of 21 l/s was 83 mm. Velocities were measured at the plan centre of each of the grid block. Seven vertical velocities were measured in each block at 1 cm intervals. The measured point and depth-averaged velocities at the 5 longitudinal sections (Fig. 3-15) are listed in Tables D-2 to D-6 (Appendix D). Vertical velocity distributions are plotted in Figs 3-17 to 3-21.

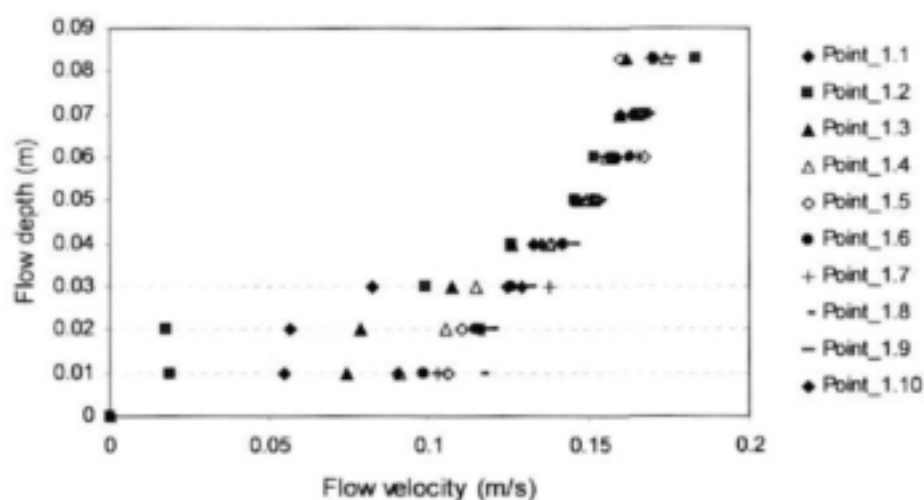


Figure 3-17 Vertical velocity distributions at longitudinal section 1

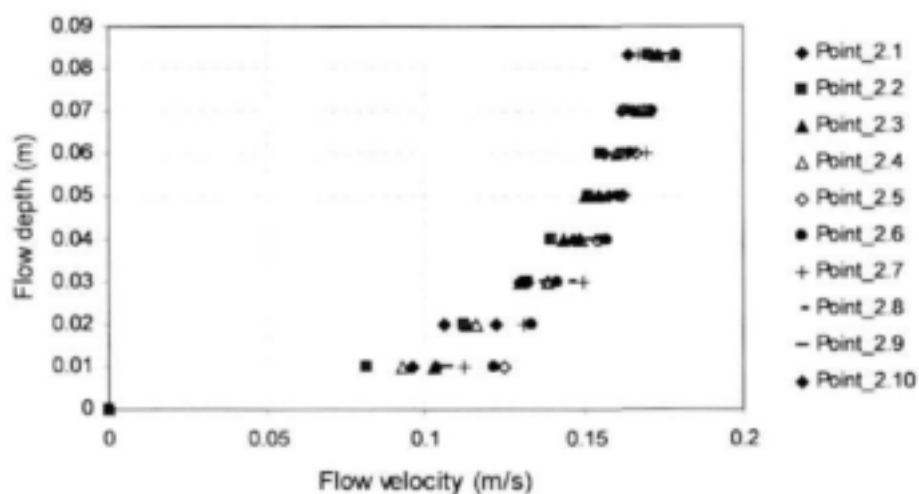


Figure 3-18 Vertical velocity distributions at longitudinal section 2

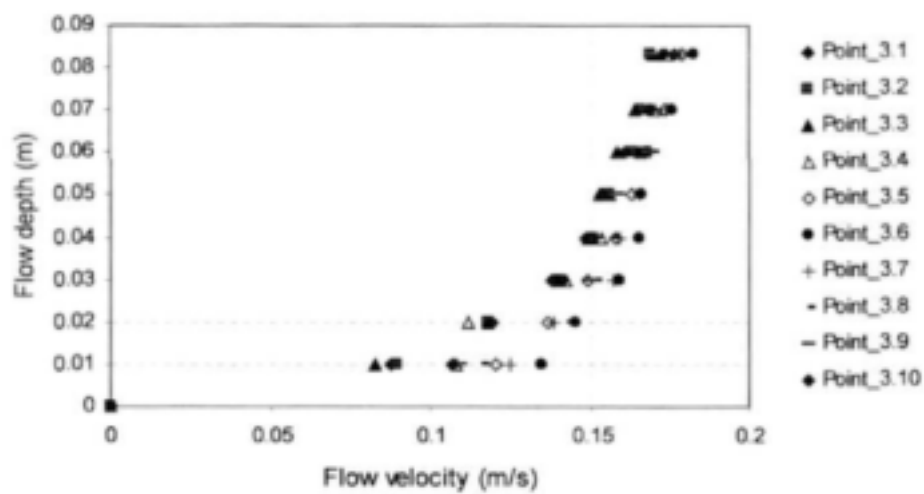


Figure 3-19 Vertical velocity distributions at longitudinal section 3

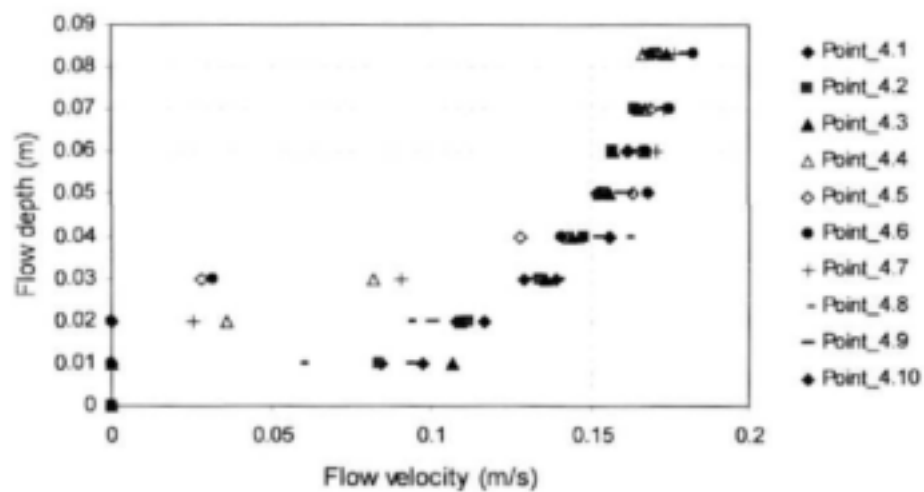


Figure 3-20 Vertical velocity distributions at longitudinal section 4

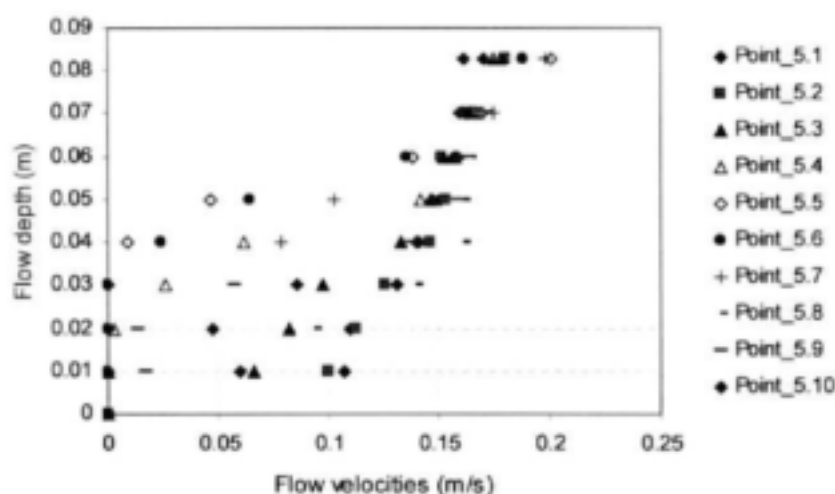


Figure 3-21 Vertical velocity distributions at longitudinal section 5

A depth-averaged velocity for each grid position considered was calculated, and the velocity distribution histogram with the same (as for discharge of 3l/s) velocity classes of 2cm/s is plotted in Fig. 3-22. The histogram is unimodal and skewed to the left. The mode is in the 12 to 14cm/s class. The lowest and the highest measured velocities are in the 4 to 6cm/s and 14 to 16cm/s velocity class respectively. Depth-averaged velocities lower than 4 cm/s were not calculated for this discharge.

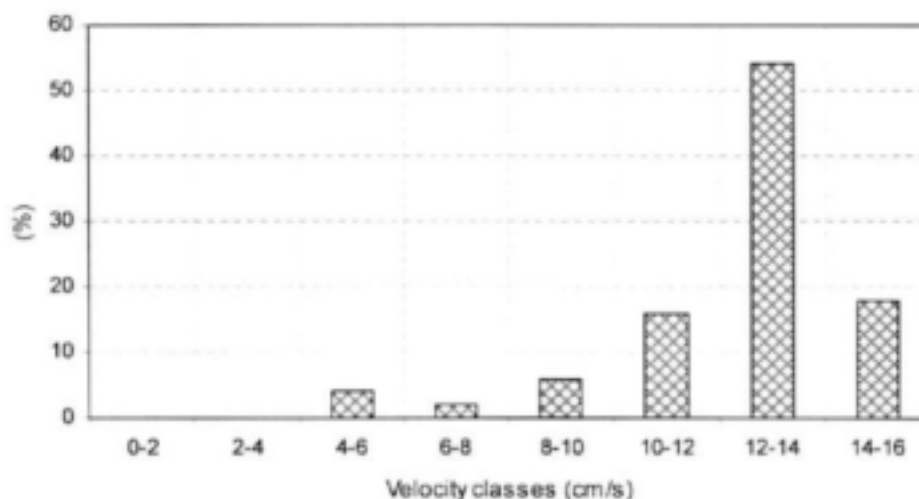


Figure 3-22 Depth-averaged velocity distribution histogram with velocity classes of 2cm/s for $Q=21l/s$

An average velocity of 0.13m/s for Pattern 6, Series 2.2 experiments was calculated as the ratio of flow to cross-sectional area. From the velocity distribution histogram

(Fig. 3-22) it can be seen that the average velocity falls within the same velocity class (12-14cm/s) as the mode.

The effect of resolution of hydraulic measurements was assessed by comparing velocity frequency distributions of measured point velocities with frequency distributions of depth-averaged velocity (that were calculated from the same measurements). Graphs of histograms for depth-averaged and point velocity are presented in Fig. 3-23. The histograms are different in terms of shape and the velocity range. As expected, the average velocity (0.13m/s) falls within the same velocity class as the mode for the depth-averaged velocity. However, most point velocities (58% measurements) are greater than 14cm/s. Thus different estimates of the predominant velocity occurring are produced. Point velocity is also spread over a wider range of velocity classes than depth-averaged velocity, including low velocities that were measured on the top of the hemisphere, and also including higher velocities (16-18cm/s) that the depth-averaged distribution. Use of too coarse a resolution may tend to under-estimate habitat diversity. Each resolution of measurement produces a distribution of velocity applicable only at this resolution.

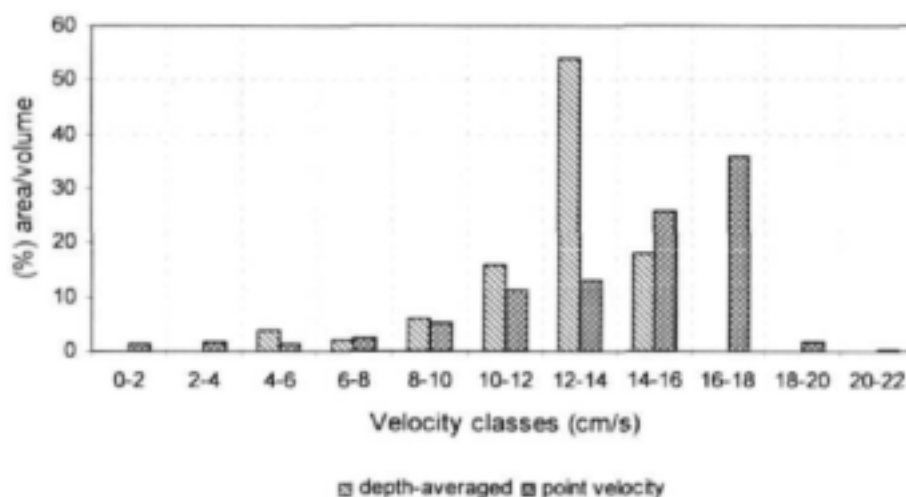


Figure 3-23 Depth-averaged and point velocity frequency distributions

3.3.3 Velocity Distributions with Discharges of 3.0 and 21.0l/s

The effect of discharge was assessed by comparing velocity frequency distributions of measured velocities for $Q=3.0$ l/s with the depth-averaged velocities for 21.0l/s. Graphs of histograms for $Q=3.0$ l/s and 21.0l/s are plotted together in Fig. 3-24. Both tests were performed under resistance controlled conditions. The histograms are unimodal and skewed to the left. As expected, the average velocity is lower for the smaller discharge, and the range of local velocities is wider for the higher discharge.

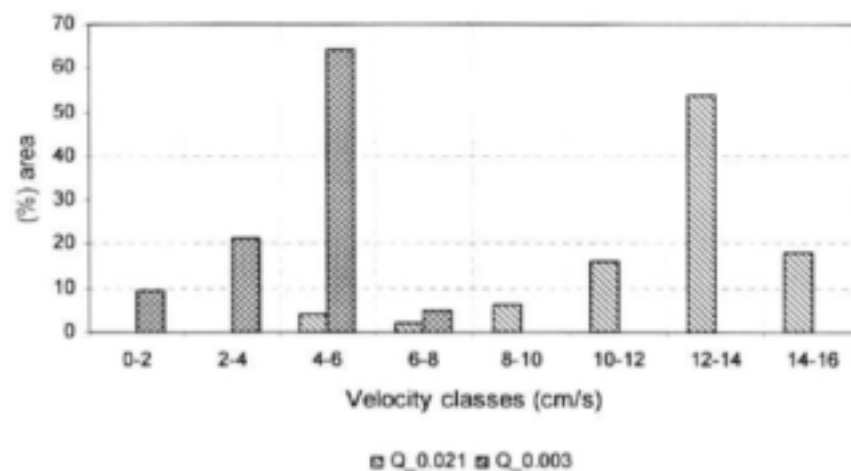


Figure 3-24 Velocity distribution histograms with 0.02 m/s velocity classes for $Q=21.0\text{l/s}$ and 3.0l/s

3.3.4 Conclusions

Velocities measured around one hemisphere under resistance controlled condition for two different discharges show:

- The shapes of the histograms are similar for both discharges.
- The average velocity is lower for the smaller discharge.
- The higher discharge provides the wider range of local velocity.
- The coarser resolution tends to underestimate the velocity frequency distribution.

4 PREDICTION METHODS FOR RESISTANCE CONTROLLED CONDITIONS

4.1 Introduction

The resistance to flow in open channels has attracted the attention of researchers for many years. Prediction of open channel flow resistance requires understanding of the influence of the underlying physical effects. Morris (1954) proposed a concept based on an assumption that the loss of energy over rough surfaces is related to the formation of wakes behind each roughness element. The main source of energy loss over a rough surface is the generation, spreading and dissipation of vortices from the wake and separation zones around every roughness element that influences the turbulence structure and energy dissipation phenomena. He presented a concept of flow over rough channel surface that is based on the effect of the longitudinal spacing of surface roughness elements. Three types of flow have been recognised:

- If the roughness elements are far apart, each acts as an isolated body, and the wake zone and vortex-generating zone are completely developed and dissipated before the next element is reached. The resulting resistance arises from the form drag of the individual roughness elements, together with the frictional resistance of the surface between the elements. This type of flow is designated as isolated-roughness flow. Under such condition the friction factor is resulted from the form drag of the roughness elements that is depend on the height of the projection of each element, and from the friction drag of the wall surface between elements. The ratio of the longitudinal spacing of the roughness elements to their height is therefore recognised as a significant correlating parameter for the isolated roughness flow condition.
- When the roughness elements are in close proximity, the zone of separation and vortex generation of each element is not completely developed before the next element is encountered. This type of flow is called wake-interference flow. For this condition the frictional resistance of the surface between the roughness elements is negligible, and flow structure is exemplified by complex vorticity and turbulent mixing. For this flow type the roughness height is less important in determining resistance, while the longitudinal spacing is the parameter that has the major influence.
- If the spacing between roughness elements is very small, there will be regions of dead water containing stable vortices between the elements. This type of flow is referred to as quasi-smooth or skimming flow.

Regarding open channel flow resistance, Rouse (1965) wrote: "Yet a glance at publications on the subject during the past decade or so will reveal many a significant anomaly. Momentum and energy analyses are at the same time over-simplified and confused with one other. Representation of parameters by symbols is mistaken for

empirical formulation of functions. Flow formulas are sometimes said to involve the Froude number when they do not, and yet the Froude number is as frequently ignored when it is actually essential. Boundary texture and cross-sectional nonuniformity are often discussed without distinction". Forty years later, have we improved our basic understanding of flow resistance? Can we predict flow resistance correctly for different roughness scales?

Three roughness scale types (small, large and intermediate) have been recognised, based on the degree of submergence of the roughness elements. The review in Chapter 2 presented a number of resistance prediction equations for large scale roughness conditions that have been developed. Most of these, however, are based on modifications of well known resistance equations that were developed for deep flow and their applicability to shallow flows is uncertain.

This chapter includes a short discussion related to small-scale roughness resistance prediction, and presents different types of equations for predicting flow resistance under large and intermediate scales roughness conditions.

4.2 Small-Scale Roughness

Under small-scale roughness conditions, the roughness elements are very small compared to the flow depth, and do not significantly alter the one dimensional character of the flow field. The Darcy-Weisbach (equation (2.1)), Chézy (equation (2.2)) and Manning's (equation (2.3)) equations are mostly used for this type of flow.

Various refinements have been made to the friction factor estimation. The Task Force on Friction Factors in Open channels (1963) reviewed the information available at the time, and recommended using the Darcy-Weisbach friction factor, f , for estimation of flow resistance. The following equations for estimating f for hydraulically rough (equation (4.1)), smooth (equation (4.2)) and transitional (equation (4.3)) flows were recommended:

$$\frac{1}{\sqrt{f}} = c \log \left(\frac{aR}{k_s} \right) \quad 4.1$$

$$\frac{1}{\sqrt{f}} = c \log \left(\text{Re} \frac{\sqrt{f}}{b} \right) \quad 4.2$$

$$\frac{1}{\sqrt{f}} = -\log \left(\frac{k_s}{aR} + \frac{b}{\text{Re} \sqrt{f}} \right) \quad 4.3$$

in which R : hydraulic radius

- k_s : roughness size
 Re : Reynolds number
 a, b, c : empirical coefficients

According to the Task Force, the recommended values of the coefficients a , b and c derived from various data sets are 12, 2.51 and 2 respectively.

In line with the Task Force recommendations, the Darcy-Weisbach friction factor, f , is used for all analyses in this study. Values of the Chézy C and Manning n can easily be obtained from f through the equivalence implied by equations (2.1), (2.2) and (2.3).

4.3 Large-Scale Roughness

4.3.1 Background

Flow resistance under large-scale roughness conditions has been investigated previously, and a number of important findings have been reported. It has been shown that a flow resistance equation for large scale roughness must include a parameter representing the proportion of the coarser elements in the bed surface layer (Bathurst, 1978; Hey, 1979; O'Laughlin and MacDonald, 1964). For describing a gravel-bed surface, it has been shown that the bed arrangement should be characterized by the size ratio between the bed layer and the coarse particles, and the concentration of boulders (Baiaomonte and Ferro, 1997; Ferro, 1999).

Flow resistance due to large-scale roughness is related to the form drag of the roughness elements and their disposition in a riverbed. In developing an equation for estimation of the resistance coefficient, it is therefore necessary to account for the processes that determine the drag of individual elements, and the roughness geometry. The form drag of an object varies according to whether the boundary layer on the object is laminar, turbulent, or transitional between these states, which are represented by the Reynolds number. The form drag is also influenced by deformation of the water surface by the object, which depends on the Froude number. The Froude number and Reynolds number are therefore variables that can be related to the drag of the roughness elements (Bathurst et al, 1981).

Flammer et al. (1970) performed laboratory experiments to determine the variables that affect the drag on a hemisphere for various flow conditions, from the relatively simple case of a semi-infinite flow to the more general case of a finite-flow with free surface effects. By dimensional analysis the following dimensionless parameters were obtained:

$$C_d = f(Re; Fr; P; \frac{y}{k}; \frac{b}{2k}) \quad 4.4$$

where Re : Reynolds number in terms of hemisphere diameter,
 Fr : Froude number,
 P : velocity profile parameter,
 k : hemisphere radius,
 y : flow depth,
 b : channel width,
 y/k : relative submergence, and
 $b/2k$: relative width.

From experiments related to the large-scale roughness within a flow regime characterized by 'pronounced free surface effects', the following findings were reported:

- Viscous forces are insignificant compared with gravity forces.
- The wave drag caused by the free surface increases the drag coefficient up to 10.
- The maximum wave drag occurs at a Froude number of about 0.5.

The geometry and disposition of the roughness elements have a significant influence on flow resistance, and the determination and incorporation of these influences into flow resistance prediction is essential. Different approaches to account for roughness geometry were discussed in Chapter 2.

Flow resistance of large-scale roughness has been investigated previously, and a general form of velocity equation under such conditions was proposed (James et al, 2001; Jordanova et al, 2004):

$$V = \frac{1}{F} \sqrt{S} \quad 4.5$$

where F : resistance coefficient.

The following equation for flow resistance coefficient F was proposed in terms of dimensionless parameters (Jordanova et al, 2004):

$$F = \left(\frac{W_{re}}{W_t} \right)^{0.225} \left(\frac{A_p}{A_b} \right)^{-0.465} \left(\frac{WS_{re}}{A_t} \right)^{0.374} \quad 4.6$$

where:

- The first parameter represents the roughness element concentration, and is expressed as the ratio of the volume of roughness elements within the flow element considered, W_{re} , to the volume of the considered element, W_t .
- The second parameter represents the roughness element shape, and is given as the ratio of the projected cross-sectional area of the individual roughness element, A_p , to its base bed area, A_b .
- The third parameter represents the roughness element spacing, as the ratio of the wetted surface area of the roughness elements, WS_{re} within the flow element considered, to the considered plan area of the bed, A_t .

Estimation of the flow resistance coefficient F in terms of dimensionless parameters (equation (4.6)) requires a lot of input data. It was therefore suggested that further investigation was required to express the flow resistance coefficient in a simpler form (Jordanova et al., 2004).

Laboratory experiments (Chapter 3) related to the large-scale roughness condition only were further analysed, and a prediction method for flow resistance is proposed.

4.3.2 Prediction Approach

Flow resistance under large-scale roughness condition is related to the drag force on the roughness elements, given by

$$F = \frac{1}{2} C_d \rho \bar{V}^2 A_p \quad 4.7$$

where F : drag force,
 C_d : drag coefficient,
 ρ : water density,
 $\bar{V}$: average velocity, and
 A_p : projected area of elements.

Under steady uniform flow conditions, this resisting force is balanced by the weight component of the water. For unit plan area, this component of the weight of water in the downstream direction is given by:

$$W = \gamma S(1 \times 1 \times y - N V_{rel}) \quad 4.8$$

in which γ : unit weight of water,
 y : flow depth,
 S : energy gradient,
 N : number of roughness elements per unit area, and
 V_{rel} : submerged volume of an individual roughness element.

The component in brackets is the volume of overlying water per unit plan area of bed and is known as the volumetric hydraulic radius, R_v . Equating equations (4.7) for unit plan area and (4.8), with $(1 \times 1 \times y - N V_{rel}) = R_v$ gives

$$V = \sqrt{\frac{1}{C_D N A_p}} \sqrt{R_v} \sqrt{2gS} \quad 4.9$$

It has been recognized that the drag coefficient, C_D , depends on a number of variables such as the Reynolds number and the Froude number (Flammer et al., 1970), and its estimation is therefore not easy or even possible (Lawrence, 2000; Smart et al., 2002) using a drag type prediction model. Furthermore, the projected area of the roughness elements projected area (NA_p) changes with flow depth and its estimation is complicated. A new, more general expression for resistance under large-scale flow is therefore adopted, as given by

$$V = \frac{1}{F} \sqrt{R_v} \sqrt{2gS} \quad 4.10$$

where F is the resistance coefficient for large scale roughness.

Equation (4.10) is superficially similar to the Darcy-Weisbach equation. However, in this case the inclusion of the volumetric hydraulic radius R_v arises from the driving force that is opposed by form drag (equation (4.8)) while the hydraulic radius in the Darcy-Weisbach equation arises from the resisting shear force at the boundary.

Experimental data were used to develop an approach for estimating the resistance coefficient F directly. As the flow resistance of large-scale roughness is mainly due to the form drag, it can be accounted for in terms of the Reynolds number and the Froude number (section 4.3.1, equation (4.4)). Furthermore, from the laboratory investigation the influence of the roughness elements' density on overall flow resistance is apparent in all the different tests. The following important variables were therefore selected as determinants of F :

- Froude number
- Roughness elements Reynolds number, and
- Areal coverage of resisting elements.

The resistance coefficient F is proposed to be expressed in terms of dimensionless parameters as

$$F = a Fr^b Re^c \Lambda^d \quad 4.11$$

where the symbol Λ represents the areal coverage calculated as proportion of the bed area covered by the roughness elements.

The experimental data presented in Chapter 3 that satisfy the large scale roughness criterion ($y/h < 1$) were divided into two sets; one set (listed in Table 4.1) is used in a multiple regression analysis to fit coefficients a , b , c and d for the resistance coefficient, F , and the other set (listed in Table 4.2) reserved for its verification.

The roughness element Reynolds number was calculated for each run as

$$Re = \frac{V_{eff} D}{\nu} \quad 4.12$$

where D : roughness element diameter
 ν : kinematic viscosity of water

The effective flow velocity, V_{eff} , is obtained as the ratio of the volumetric flow rate to the area available for flow,

$$V_{eff} = \frac{Q}{A} = \frac{Q}{Wy_{eff}} \quad 4.13$$

where W is the width of the channel, and y_{eff} is an effective depth calculated as

$$y_{eff} = y_m - N V_{rel} \quad 4.14$$

where y_m is the measured flow depth above the flume bottom. The submerged volume of a hemisphere can be obtained as

$$V_{rel} = \pi h^2 y_m - \frac{1}{3} \pi y_m^3 \quad 4.15$$

where h is the roughness elements height.

Table 4-1 Experimental data (Appendices B and C) used in multiple regression analysis

Series	Pattern	Discharge (l/s)	Measured depth (m)	Areal coverage	Slope
Series 2.1	1	2.7 – 9.5	0.025 – 0.055	0.149	0.001
	3	2.9 – 10.5	0.028 – 0.062	0.173	0.001
	5	1.8 – 12.0	0.020 – 0.062	0.177	0.001
Series 2.2	1	0.6 – 1.6	0.032 – 0.055	0.546	0.0005
	3	0.2 – 7.2	0.023 – 0.055	0.120	0.0005
	5	3.2 – 6.6	0.036 – 0.046	0.029	0.0005
	6	2.8 – 3.5	0.034 – 0.040	0.098	0.0005
	8	0.7 – 4.0	0.020 – 0.060	0.318	0.0005
	9	0.8 – 1.9	0.020 – 0.040	0.261	0.0005
	10	1.4 – 8.9	0.024 – 0.060	0.161	0.0005
	11	2.0 – 5.2	0.026 – 0.047	0.137	0.0005
	13	2.5 – 7.0	0.027 – 0.053	0.081	0.0005
	15	0.4 – 1.2	0.043 – 0.055	0.749	0.0005

Table 4-2 Experimental data (Appendices B and C) used for validation of proposed resistance coefficient F

Series	Pattern	Discharge (l/s)	Measured depth (m)	Areal coverage	Slope
Series 2.1	2	1.2 – 9.3	0.020 – 0.062	0.179	0.001
	4	2.0 – 12.5	0.020 – 0.062	0.170	0.001
	6	3.2 – 12.9	0.025 – 0.062	0.149	0.001
Series 2.2	2	1.1 – 3.7	0.024 – 0.055	0.220	0.0005
	4	2.7 – 12.1	0.023 – 0.055	0.062	0.0005
	12	3.7 – 6.3	0.031 – 0.047	0.081	0.0005
	16	0.6 – 1.6	0.040 – 0.057	0.629	0.0005
	17	0.6 – 1.7	0.031 – 0.055	0.416	0.0005

A multiple regression analysis was performed and the following equation for F was obtained with $R^2=0.965$:

$$F = 0.16 Fr^{-0.51} Re^{0.004} \Lambda^{0.222} \quad 4.16$$

4.3.3 Verification of Proposed Equations (4.10) and (4.16) for the Large-Scale Roughness Condition

The performance of the proposed equations (4.10) and (4.16) can be assessed by comparison of measured and predicted values of flow velocity with resistance coefficient F . There are two different questions that may needed to be answered: (1) What is the flow resistance in terms of the resistance coefficient F for a known hydraulic condition?, and (2) What is the flow velocity for a given flow depth? Two

procedures for verification of the proposed equations were therefore applied as follows:

- Procedure 1 - Estimation of Resistance Coefficient (F)

The predictive ability of equation (4.16) has been tested by using measured velocities to calculate values of Froude number and the roughness element Reynolds number, and hence resistance coefficient values from equation (4.16), which were then compared with the measured values.

- Procedure 2 – Prediction of Flow Velocity of a Given Depth

This estimation requires an iterative approach that was applied through four steps:

- a) For a specified flow depth a flow velocity, V is assumed, and the Froude number, Fr and the Reynolds number, Re in terms of the roughness element diameter are calculated.
- b) The resistance coefficient, F is estimated from equation (4.16).
- c) The flow velocity, V is calculated from equation (4.10).
- d) The calculated velocity (step (c)) is compared with the initially assumed value (step (a)), and if there is a difference, steps (a) to (c) are repeated until the assumed and calculated velocities are equal.

4.3.3.1 Verification of Proposed Equation (4.16) using Procedure 1

Values of resistance coefficient (F) for all experimental runs listed in Table 4-2 were predicted by equation (4.16). These values, designated as F_{pr} were then compared with the measured values, F_m ; prediction errors were calculated as absolute values of $(F_{pr} - F_m)/F_m * 100\%$.

Values of predicted and measured resistance coefficient, F for the verification data (Table 4-2) are plotted together with the line of perfect fit in Fig. 4-1. The average absolute prediction error was calculated as 8.10%, and the maximum and the minimum as 21.60% and 0.10% respectively, and the standard deviation of the prediction error was calculated as 6.80%.

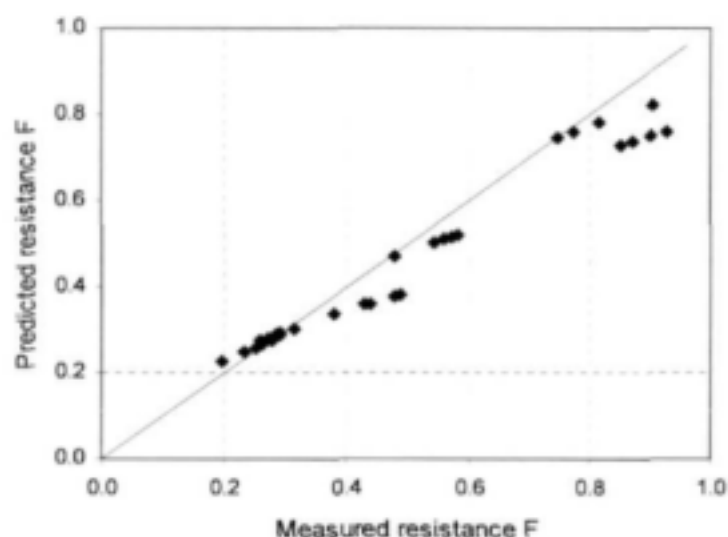


Figure 4-1 Measured and predicted (equation 4.16) values of resistance coefficient F together with the perfect fit line for experimental conditions listed in Table 4-2

Fig. 4-1 and the corresponding prediction errors show good correlation between predicted and measured values of the resistance coefficients.

4.3.3.2 Verification of Proposed Equations (4.10) using Procedure 2

Most flow resistance predictions would be required under conditions where the flow velocity is unknown. True verification of the velocity prediction (equation (4.10)) with the resistance coefficient (equation (4.16)) therefore requires the assumption that flow velocity is unknown. For each experimental run the iterative approach explained above was applied until the value of flow velocity assumed was equal to that calculated (equation (4.10)) with the resistance coefficient estimated by equation (4.16). These velocities and resistance coefficients were then compared with the measured values.

Measured and predicted values of resistance coefficient and flow velocity together with the best fit line and 30% accuracy limits for the verification data (Table 4-2) are plotted in Figs 4-2 and 4-3 respectively.

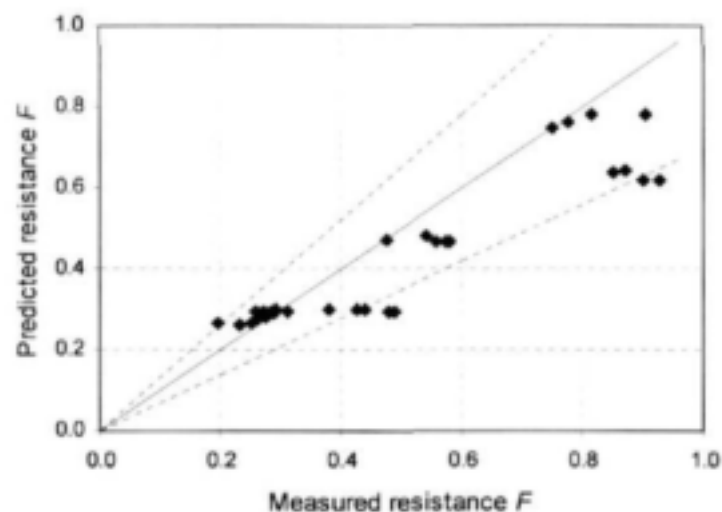


Figure 4-2 Measured and predicted values of resistance coefficient F together with the perfect fit line and 30% accuracy limits for experimental conditions listed in Table 4-2

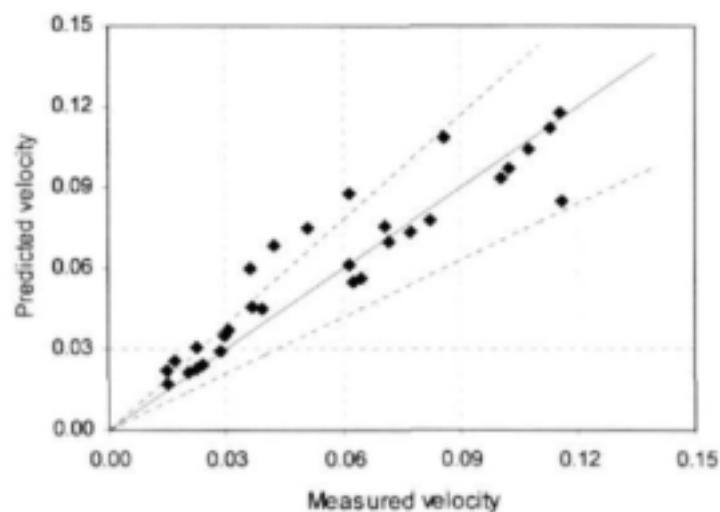


Figure 4-3 Measured and predicted values of flow velocity together with the perfect fit line and 30% accuracy limits for experimental conditions listed in Table 4-2

Predicted average, maximum and minimum absolute errors, and the standard deviations of the predicted errors for experiments listed in Tables 4-2 were calculated and presented in Tables 4-3 and 4-4 respectively.

Table 4-3 Resistance coefficient prediction errors in application of Procedure 2

Experimental Data	Average Error (%)	Minimum Error (%)	Maximum Error (%)	Standard deviation (%)
Table 4-2	15.10	0.10	39.30	12.60

Table 4-4 Flow velocity prediction errors in application of Procedure 2

Experimental Data	Average Error (%)	Minimum Error (%)	Maximum Error (%)	Standard deviation (%)
Table 4-2	19.20	0.10	64.60	18.90

Verification of the proposed equations (4.10) and (4.16) as assessed by using the iterative procedure (Procedure 2) shows their true predictive ability. It can be seen that prediction errors (Tables 4-3 and 4-4) are higher than were verified by Procedure 1 (section 4.3.3.1) but are still within acceptable accuracy limits. Verification of the proposed equations by the two different procedures shows clearly that Procedure 1 indicated better performance because the Froude number and the Reynolds number and hence the resistance coefficient were estimated with the observed flow velocity values. Verification of the proposed equations by Procedure 1 therefore has limited value for assessing velocity prediction. Further verification of the proposed equations (4.10) and (4.16) against field data will therefore be through the iterative approach only.

4.3.4 Equation Verification with Field Data

Depth and velocity distribution data collected in the Cotter River, Australia, are presented in Chapter 8. Two sites, named Vanities Crossing and Spur Hole, were selected for verification of the large-scale resistance prediction method. Photographs of the two sites are presented in Figs 4-4 and 4-5 respectively.

Measurements were carried out at three discharges. The data collected at the lowest discharge satisfy the large-scale roughness criterion, and were therefore used for verification of the proposed equations (4.10) and (4.16). As equation (4.10) was developed for estimation of the average cross-sectional velocity, the measured flow depths were used to calculate the flow area and the average velocity was then calculated as a ratio of the discharge to the flow area.

It should be noted that hydraulic data at these sites were collected for purposes other than estimating flow resistance, and substrate characteristics are given in terms of descriptive substrate classes only (Appendix G). To translate the descriptive substrate classes into particle sizes, the grade scale (adopted from Brakensiek et al (1979) in Gordon et al (1992)) was used for each cross-section considered. The substrate

diameters and the areal coverages required for application of the proposed equations ((4.10) and (4.16)) were estimated from the cross-sectional data (Appendix G), and these are listed in Table 4-5. This was done by the following steps:

- Estimation of the biggest substrate clasts present within a cross-section under consideration, and
- Estimation of the area which these clasts covered.

As an example, estimations for Spur Hole Site, cross-section 3 were carried out as follows. The data (Appendix G, cross-section 3) indicate that only two (gravel-cobble and cobble-boulder) substrate classes occur in the cross-section. The areal coverage of the bigger, cobble-boulder substrate size was calculated as the ratio of the number of points where this class was surveyed to the total number of points surveyed within the cross-sectional flow width.



Figure 4-4 The Cotter River, Vanities Crossing Site



Figure 4-5 The Cotter River, Spur Hole Site

Procedure 2 was applied to predict the flow velocity for the given discharges and measured and predicted velocities are plotted in Fig. 4-6. The average absolute error was calculated as 12.70%.

Table 4-5 The Cotter River field data

Site	Discharge (m ³ /s)	Water slope	Roughness diameter (m)	Areal coverage	V measured (m/s)
Vanities Crossing Cross-section 1	0.3305	0.00250	0.30	0.538	0.273
Vanities Crossing Cross-section 3	0.3305	0.00401	0.40	0.889	0.260
Spur Hole Cross-section 3	0.3427	0.00197	0.30	0.316	0.230
Spur Hole Cross-section 5	0.3427	0.00138	0.08	0.579	0.173

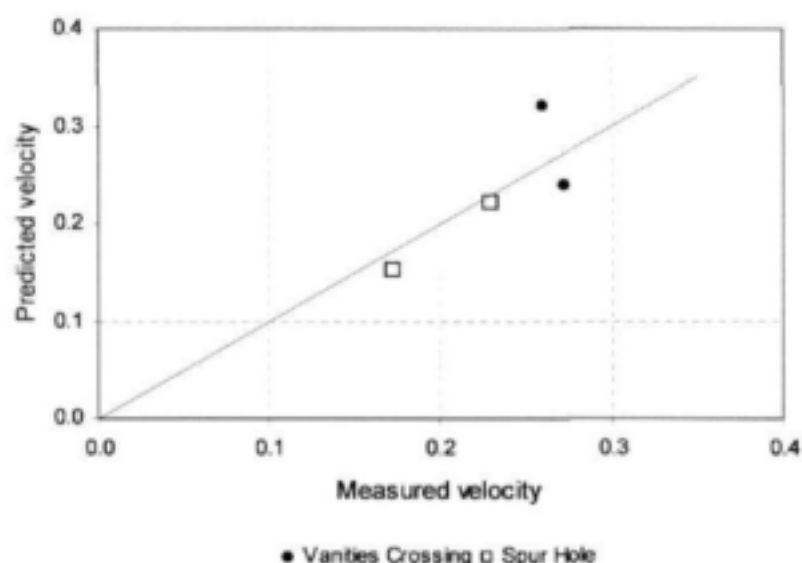


Figure 4-6 Measured and predicted flow velocities for Vanities Crossing and Spur Hole sites of the Cotter River.

From field data that satisfied the large-scale roughness condition it can be seen that the proposed equation (4.10) for prediction of the average velocity with the resistance coefficient defined by equation (4.16) reproduced measured flow velocities well. It can also be noted that prediction is better for both cross-sections of the Spur Hole site than for the Vanities Crossing site. The predicted results are quite reasonable considering the limitations of the data - they were collected by a different team, and the only information available includes cross-sections, depth-velocity measurements (Appendix G) and some photographs.

4.3.5 Modification of the Prediction Approach for Field Application

Application of the proposed equation (4.16) requires estimation of the areal coverage parameter, representing the plan area covered by larger elements over a total bed area under consideration. This parameter is not practical for use in field applications, as it is not directly measurable. It is, however, related to the bed sediment size distribution, which is more commonly measured.

Statistical parameters such as the geometric standard deviation are therefore more appropriate for representing the resistance-related characteristics of a river bed's substrate. The geometric standard deviation, as defined by Vanoni (1975), is

$$\sigma = \sqrt{\frac{D_{84}}{D_{16}}} \quad 4.17$$

For lognormally distributed bed material, equation (4.17) can also be expressed in terms of D_{84} and D_{50} as

$$\sigma = \frac{D_{84}}{D_{50}} \quad 4.18$$

Replacing Λ in equation (4.11) by σ (from equation (4.18)) leads to

$$F = a Fr^b Re^c \sigma^d \quad 4.19$$

Application of equation (4.19) requires re-estimation of coefficients a , b , c and d . Estimation of these empirical coefficients was carried out using published field data (Bathurst, 1978 and Bathurst, 1985) that satisfied the large-scale roughness criterion ($y/h < 1$). These data were collected from different rivers in Britain. At each site a survey of three cross-sections in sufficient detail was carried out. Mean site values of flow area, width and depth were obtained by averaging the three respective sectional values. The data used in the multiple regression analysis are listed in Table 4-6.

Table 4-6 Published field large-scale roughness data used in multiple regression analysis

Data source	River (site name)	Bed material D_{50} (mm)	Bed material D_{84} (mm)	Number of measurements
Bathurst (1978)	Upper River Tees (Whiddybank)	278	453	3
	Upper River Tees (Cronkley A)	207	380	3
	Upper River Tees (Cronkley B)	185	305	1
Bathurst (1985)	South Tyne	146	240	1
	Alwin	64	143	1
	Glen	60	113	1
	Ettrick	86	193	1
	Tweed	90	183	1
	Almond	118	307	1
	Braan	343	740	2
	Tromie-2	125	387	1
	Dulnain	251	500	2

For each field measurement, the resistance coefficient, F was calculated using equation (4.10). Multiple regression analysis to quantify equation (4.19) resulted in the empirical relationship (equation (4.20)) with $R^2=0.747$

$$F = 0.05 Fr^{-0.868} Re^{0.12} \sigma^{-0.228} \quad 4.20$$

4.3.5.1 Verification of Proposed Equation (4.20)

The publication of Hicks and Mason (1998) provided a reference data set for use in visually estimating resistance coefficients for New Zealand rivers. These data were collected from different rivers in New Zealand. The number of cross-sections per reach was generally from three to five, and the hydraulic parameters provided were calculated by averaging the surveyed cross-section values.

Verification of the proposed equation (4.20) for estimation of flow resistance was carried out by applying the iterative approach to Hicks and Mason (1998) field data that satisfied large-scale roughness condition (Table 4-7).

Table 4-7 Published Hicks and Mason (1998) field data

Data source	River (site name)	Bed material D_{50} (mm)	Bed material D_{84} (mm)	Number of measurements
Hicks and Mason (1998)	Ruakokapatuna	45	119	5
	Kapoiaiaia	78	212	1
	Waiau Water Race	46	80	1
	Stanley Brook	32	106	2

Predicted (equations (4.10 and 4.20)) and measured (Hicks and Mason (1998)) velocities together with the perfect fit line are plotted in Fig. 4-7.

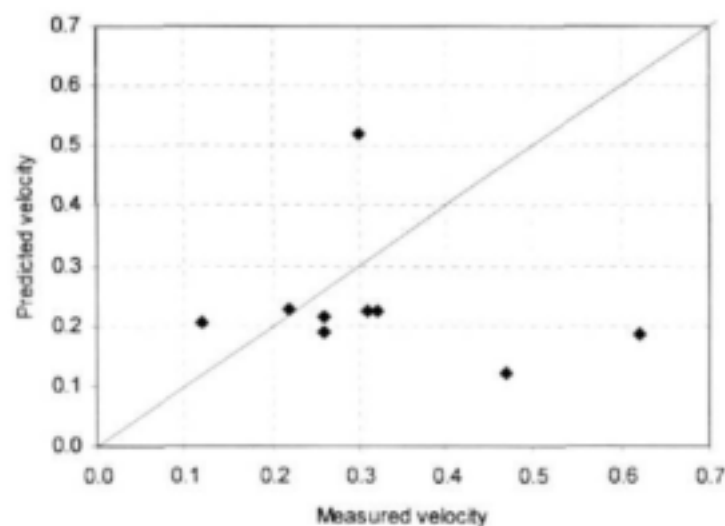


Figure 4-7 Measured and predicted (equations (4.10 and 4.20)) velocities for published Hicks and Mason (1998) field data listed in Table 4-6

Prediction average, minimum and maximum absolute errors and the standard deviation are given in Table 4-8.

Table 4-8 Flow velocity prediction (equations (4.10 and 4.20)) errors in application to field data (Table 4-7)

Field Data	Average Error (%)	Minimum Error (%)	Maximum Error (%)	Standard deviation (%)
Hicks and Mason (1998)	43.47	2.94	73.89	26.42

Taking into account the limitations and uncertainty associated with the field data that have been used in this development, the predictions of equations (4.10 and 4.20) are not unrealistic over a wide range of natural large-scale roughness conditions. While errors are appreciable, performance compares favourably with that of existing methods, as shown in the following section.

4.3.5.2 Bathurst (2002) equations

Based on twenty-seven published field datasets with $0.37 < y/D_{84} < 11$ and slopes in the range 0.2 – 4%, Bathurst (2002) proposed two equations ((2.18) and (2.19)), for different ranges of channel slope, for predicting flow resistance of rough beds.

Channel slope, $S < 0.8\%$:

$$(8/f)^{1/2} = 3.84 (y/D_{84})^{0.547} \quad 2.18$$

Channel slope, $S > 0.8\%$:

$$(8/f)^{1/2} = 3.10 (y/D_{84})^{0.93} \quad 2.19$$

These equations are based on data that include both large and intermediate scale roughness conditions. The data were considered together, with no distinction being made between these conditions. Furthermore, the equations have not been verified against independent field data.

For comparison of equations ((2.18) and (2.19)) with the proposed equations ((4.10) and (4.20)), equations ((2.18) and (2.19)) were first applied to the set of data that was used to develop equation (4.20). (This is the part of the same data set that was used for the development of equations ((2.18) and (2.19)) that satisfies the large scale roughness condition.)

Measured flow velocities are compared with those predicted by equations (2.18) and (2.19) (referred to as "Bathurst (2002)") and by equation (4.10) with resistance coefficient defined by equation (4.20) (referred to as "equation (4.20)") in Fig. 4-8.

The average, minimum and maximum absolute errors for flow velocity prediction by equations (2.18) and (2.19) are 15.80%, 1.60% and 44.35% respectively, with a standard deviation of 10.14%. It is clear that equations (2.18) and (2.19) predict these data better than the proposed equations (4.10 and 4.20), which produces corresponding errors of 38.65%, 1.94% and 98.9% with a standard deviation of 29.5%.

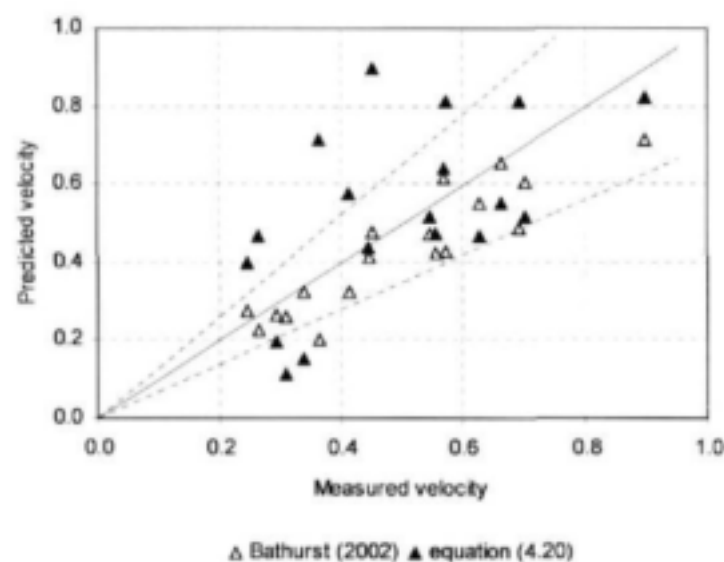


Figure 4-8 Measured and predicted (equations (2.18) and (2.19), and (4.10) and (4.20)) flow velocities for Bathurst (1978) and (1985) large scale roughness field data

Equations (2.18) and (2.19) and equations (4.10 and 4.20) were then applied to the Hicks and Mason (1998) field data for large scale roughness (Table 4-7). The measured and predicted flow velocities are plotted in Fig. 4-9.

The prediction errors presented in Table 4-9 show that the proposed equation (4.20) performs better than equations (2.18) and (2.19) for these data. It was shown (Fig. 4-8) that equations (2.18) and (2.19) predict flow velocity well for data (Table 4-6, Bathurst (1978) and (1985)) used for their development. Their performance is considerably poorer, however, for the independent field data of Hicks and Mason (1998).

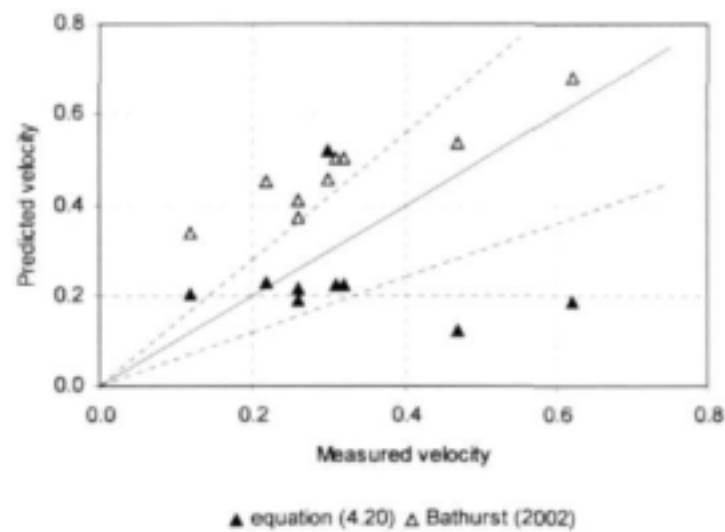


Figure 4-9 Measured and predicted (equations (2.18) and (2.19), and (4.10) and (4.20)) flow velocities for Hicks and Mason (1998) large scale roughness field data

Table 4-9 Prediction errors in application of equations ((2.18) and (2.19)) and equations ((4.10) and (4.20)) to Hicks and Mason (1998) field data

Prediction	Average Error (%)	Minimum Error (%)	Maximum Error (%)	Standard deviation (%)
Equations (2.18) and (2.19)	65.29	9.39	183.57	49.66
Equation (4.20)	43.47	2.94	73.89	26.42

4.4 Intermediate-Scale Roughness

When the relative submergence lies between 1.0 and about 4.0, the roughness scale is classified as intermediate. This regime represents a state of flow in which the influence of the roughness elements on flow resistance is manifest as a combination of both element drag and effective boundary shear, or friction. Under such conditions the total discharge can be considered to be the sum of the discharges below and above the tops of the large roughness elements. Alternatively, the discharge can be calculated using a velocity obtained as a weighted product of velocities reflecting the influences of roughness element drag and boundary friction, with the weighting factor depending on the relative submergence. Both hypotheses were investigated, and two approaches, Approach 1 and Approach 2 are presented below.

4.4.1 Resistance Prediction Approach 1

The intermediate-scale roughness condition is shown in Fig. 4-10 where $y_{measured}$ is the measured flow depth; y_{eff2} is an effective flow depth (equation (4.14)), and y_{eff1} is an effective flow depth for the discharge below the tops of the large roughness elements.

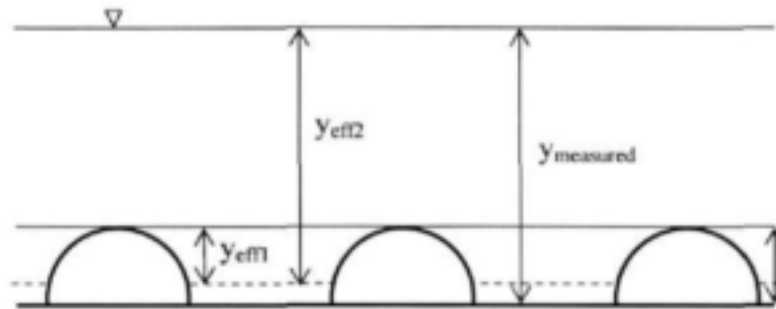


Figure 4-10 Intermediate-scale roughness condition

An equation to estimate the flow resistance under this condition (Fig.4-10) was deduced as follows.

Initially assuming flow to be effectively controlled by shear resistance, the unit width discharge can be estimated through the Darcy-Weisbach equation as

$$q = y_{eff2} \sqrt{\frac{8g}{f}} \sqrt{y_{eff2}} \sqrt{S} \quad 4.21$$

Under intermediate-scale roughness the total discharge (q) can be considered to be the sum of the discharges below (q_1) and above (q_2) the tops of the large roughness elements, and can be therefore be expressed as

$$q = q_1 + q_2 \quad 4.22$$

The lower zone discharge (q_1) is controlled by large-scale type resistance, and therefore, according to equation (4.10),

$$q_1 = y_{eff1} V_1 = y_{eff1} \frac{1}{F} \sqrt{2gy_{eff1}} \sqrt{S} \quad 4.23$$

with the value of F corresponding to large-scale roughness conditions.

The flow above the tops of the elements (q_2) is described as shear resisted flow and therefore, with $y_{eff2} - y_{eff1} = y_2$, can be calculated as

$$q_2 = y_2 \sqrt{\frac{8g}{f_2}} \sqrt{y_2} \sqrt{S} \quad 4.24$$

where f_2 is the friction factor for the flow above the tops of the roughness elements and would have the value corresponding to small-scale roughness conditions.

The total discharge is then

$$q = y_{eff1} \frac{1}{F} \sqrt{2gy_{eff1}} \sqrt{S} + y_2 \sqrt{\frac{8g}{f_2}} \sqrt{y_2} \sqrt{S} \quad 4.25$$

or

$$q = y_2 \left(\frac{y_{eff1}^{3/2}}{y_2} \frac{1}{F} + \sqrt{\frac{4}{f_2}} \sqrt{y_2} \right) \sqrt{2g} \sqrt{S} = y_2 \left(\frac{y_{eff1}^{3/2}}{y_2^{3/2}} \frac{1}{F} + \sqrt{\frac{4}{f_2}} \right) \sqrt{2g} \sqrt{y_2} \sqrt{S} \quad 4.26$$

Equating the total discharge from equations (4.21) and (4.26) enables a total effective friction factor to be determined, i.e.

$$\sqrt{\frac{4}{f}} y_{eff2}^{3/2} = y_2^{3/2} \left(\left(\frac{y_{eff1}}{y_2} \right)^{3/2} \frac{1}{F} + \sqrt{\frac{4}{f_2}} \right) \quad 4.27$$

from which

$$\sqrt{\frac{4}{f}} = \left(\frac{y_{eff1}}{y_{eff2}} \right)^{3/2} \frac{1}{F} + \left(1 - \frac{y_{eff1}}{y_{eff2}} \right)^{3/2} \sqrt{\frac{4}{f_2}} \quad 4.28$$

Now an average velocity can be calculated by equation (4.29) with $\sqrt{\frac{4}{f}}$ given by equation (4.28),

$$V = \sqrt{\frac{4}{f}} \sqrt{2 g y_{eff2}} \sqrt{S} \quad 4.29$$

The ratio y_{eff1}/y_{eff2} constitutes a weighting factor representing the relative importance of the form and effective shear resistance contributions to total resistance. For deep flows its value will be small, reflecting the minimal contribution of the drag resistance from individual roughness elements; the effective friction factor is then dominated by the second term of equation (4.28), which then closely approximates that for the bed under small-scale roughness conditions. In the intermediate-scale range of flow depths the ratio increases with decreasing flow depth, reflecting the increasing influence of form resistance. When the flow depth is equal to the height of the roughness elements, the ratio becomes equal to 1.0 and the second component of equation (4.28) will equal zero; equation (4.29) then reduces to the form of the equation proposed for large-scale roughness, i.e. equation (4.10).

4.4.1.1 Prediction Approach 1 Verification

The proposed equation (4.29) for estimation of an average velocity, with the effective friction factor given by equation (4.28), was evaluated by comparison of measured and predicted velocities for all laboratory tests performed under the intermediate-scale condition. The values of resistance coefficient F and Darcy-Weisbach friction factor f were determined from experimental data.

Measured and predicted velocities together with the perfect fit line and 25% accuracy limits for Series 2.1, 2.2 and 1.1 experiments are plotted in Figs 4-16, 4-17 and 4-18 respectively.

Average, minimum and maximum absolute errors for predicted velocity values for Series 1.1, 2.1 and 2.2 are listed in Table 4-9

Table 4-10 Predicted errors for application equations (4.28) and (4.29)

Experiments	Average absolute error (%)	Maximum absolute error (%)	Minimum absolute error (%)
Series 1.2	12.73	32.12	0.32
Series 2.1	8.08	21.35	0.68
Series 2.2	10.14	31.36	1.60

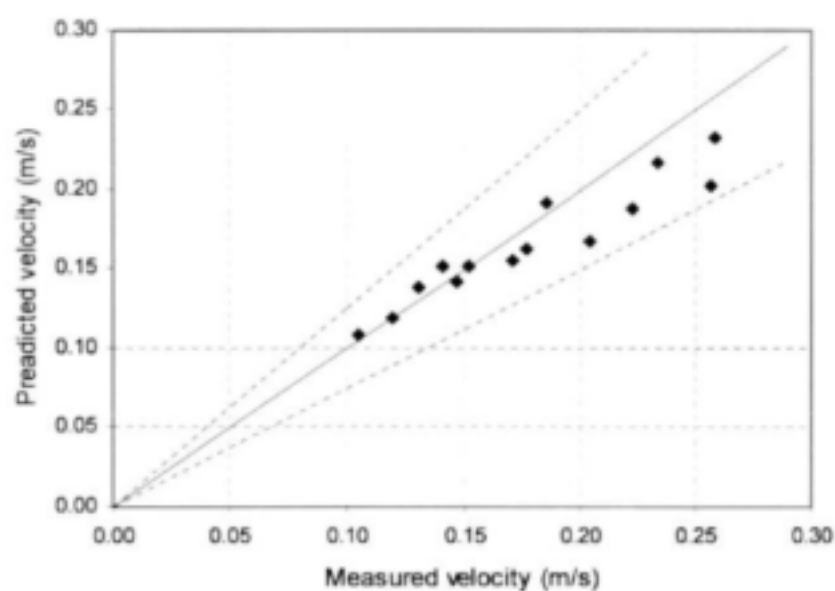


Figure 4-11 Measured and predicted (equations 4.28 and 4.29) velocities with 25% accuracy limits for Series 2.1 experiments.

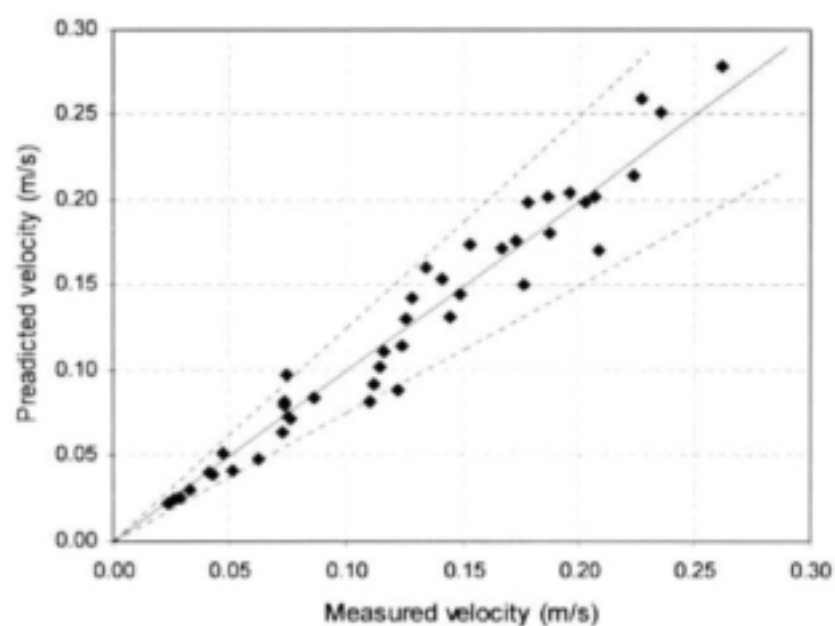


Figure 4-12 Measured and predicted (equations 4.28 and 4.29) velocity with 25% accuracy limits for Series 2.2 experiments

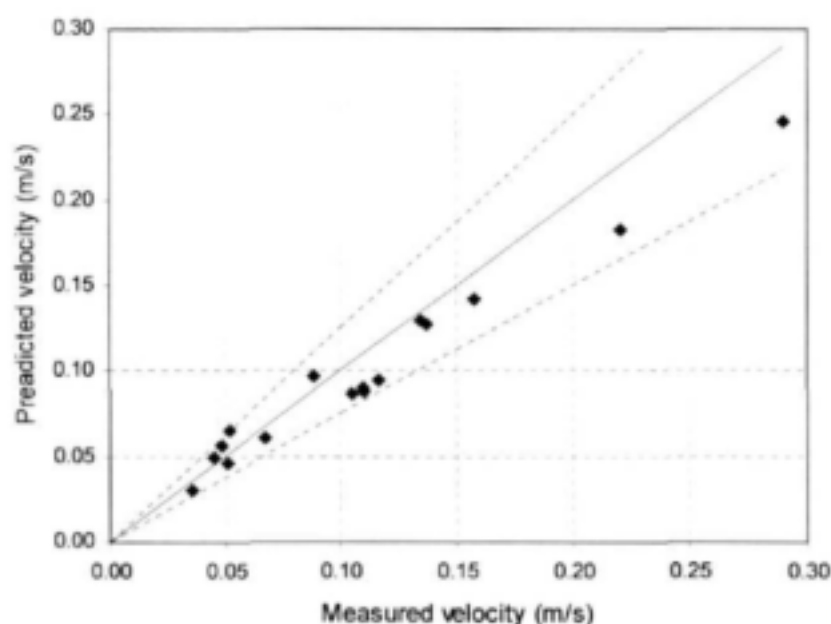


Figure 4-13 Measured and predicted (equations 4.28 and 4.30) velocity with 25% accuracy limits for Series 1.1 experiments

From the application of the proposed equations (4.28) and (4.29) it can be seen that predicted errors are acceptable. This approach has not been tested against field data.

4.4.2 Resistance Prediction Approach 2

Flow resistance under intermediate-scale roughness is imposed by a combination of roughness element drag and boundary friction. A proposed resistance prediction method is based on the following hypothesis:

- If the flow is deep and the relative submergence is greater than four, the boundary friction will dominate, and the velocity can be then calculated by equation (2.1).
- If the relative submergence is less than or equal to one, flow resistance will be dominated by the drag of roughness elements, and the proposed equation (4.10) should then be used.
- With increasing relative submergence from one to four, the dominant resisting effect changes from element drag to friction, and both drag and friction effects therefore contribute to flow resistance. Under such flow conditions, the velocity can be estimated by

$$V = \left(\frac{1}{F}\right)^a \left(\sqrt{\frac{4}{f}}\right)^{(1-a)} \sqrt{2gR_s\sqrt{S}} \quad 4.30$$

where a is a function of the relative submergence and varies from 1 to 0

When a is equal to 1, equation (4.30) reduces to the proposed equation (4.10) related to the large-scale roughness condition. With a equal to 0, equation (4.30) will take the form of equation (2.1) for small-scale roughness.

The experimental data related to the intermediate-scale roughness condition were divided into two sets. One set of data (Table 4-11) was used for development of a suitable functional relationship of the coefficient a as a function of the relative submergence.

Table 4-11 Experimental data used for functional development

Flume	Series	Pattern	Relative submergence	Number of runs
B	1.1	1.1.1	1.45 – 3.62	5
		1.1.4	1.02 – 3.55	3
		1.1.5	1.00 – 3.24	3
C	2.1	6	1.00 – 1.37	4
		8	1.00 – 1.96	5
C	2.2	1	1.00 – 3.44	6
		3	1.00 – 2.35	7
		5	1.00 – 2.86	3
		7	1.00 – 3.99	5
		14	1.00 – 4.65	5
		16	1.00 – 2.13	4

Equation (4.30) was applied to each experimental run. Application of equation (4.30) required input of the resistance coefficient F and friction factor f . These values were calculated from the experimental data for the satisfied flow conditions. Average velocities for each experiment were calculated (equation (4.30)) to be equal to the measured velocity by altering input values of the coefficient a only. Values of the coefficient a together with the related relative submergence are plotted in Fig. 4-14. A suitable relationship form of the coefficient a as a function of the relative submergence was fitted as

$$a = -0.67 \ln\left(\frac{y}{h}\right) + 0.992 \quad 4-31$$

The other experimental data (Table 4-12) were used for verification of the proposed equations (4.30) and (4.31). Measured and predicted (equations (4.30) and (4.31)) velocities together with the perfect fit line and 15 % accuracy limits for data measured in flumes B and C experiments are plotted in Fig. 4-15.

The average, maximum and minimum absolute prediction errors for Series 1.1, 2.1 and 2.2 experiments were calculated and are listed in Table 4-13.

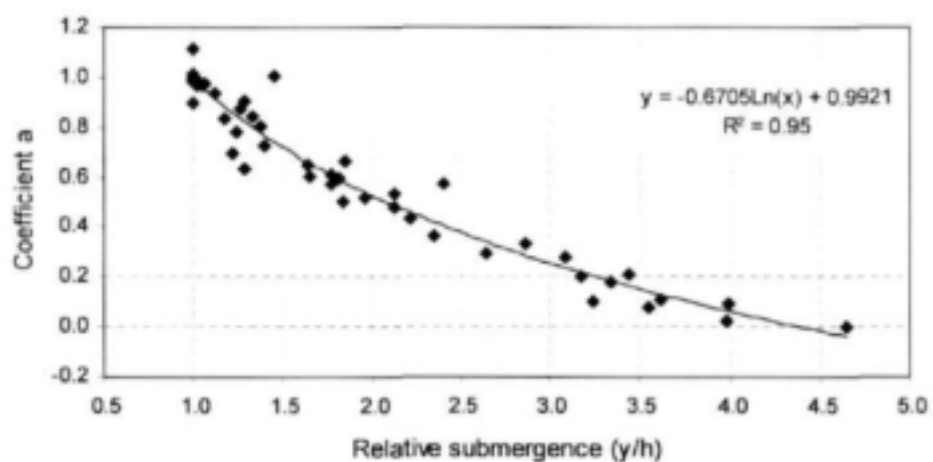


Figure 4-14 Functional relationship of relative submergence and coefficient a

Table 4-12 Experimental data used for verification of equation

Flume	Series	Pattern	Relative submergence	Number of runs
B	1.1	1.1.2	1.02 – 4.02	5
		1.1.3	1.02 – 3.57	4
C	2.1	1	1.00 – 1.19	3
		7	1.00 – 1.92	6
C	2.2	2	1.00 – 2.98	8
		4	1.00 – 2.20	4
		6	1.00 – 3.38	5
		15	1.00 – 2.30	6

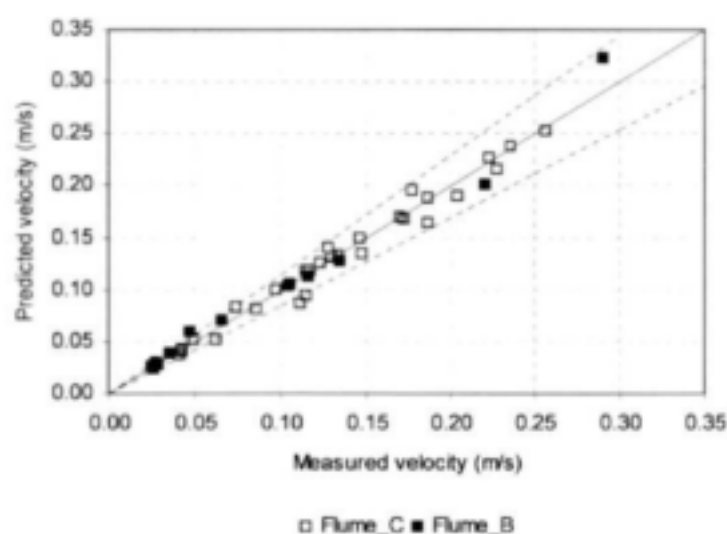


Figure 4-15 Measured and predicted (equations (4.30) and (4.31)) velocities with 15 % accuracy limits for experiments listed in Table 4-11

Table 4-13 Average, maximum and minimum absolute prediction errors for application of equations (4.30) and (4.31)

Flume	Series	Average (%)	Maximum (%)	Minimum (%)	St. Deviation of prediction error (%)
B	1.1	7.55	23.45	1.54	6.30
C	2.1	1.79	7.07	0.10	1.98
C	2.2	7.49	22.94	0.07	6.13

4.4.2.1 Verification of Proposed Equations (4.30) and (4.31) with Bathurst et al. (1981) published experimental data

Published experimental data of Bathurst et al (1981) were used for further verification of equations (4.30) and (4.31). Bathurst's experiments were carried out at Colorado State University in a flume with a length of 9.54m and a width of 1.168m width. The resistance of five bed materials classified 12.7, 19.05, 38.1, 50.8 and 63.5mm were tested. Experiments were performed with 3 flume slopes of 0.02, 0.05 and 0.08. Experimental data used for verification of proposed equations (4.30) and (4.31) are summarised in Table 4-14.

Table 4-14 Summary of Bathurst et al., (1981) experimental data

Bed material (mm)	D_{84} long axis (mm)	D_{84} median axis (mm)	D_{84} short axis (mm)	Discharge (m^3/s)	$y_{measured}$ (m)
12.7	17.0	11.5	7.8	0.0019-0.0490	0.012-0.046
19.5	28.0	19.3	12.3	0.0021-0.0546	0.016-0.054
38.1	59.0	43.0	27.0	0.0018-0.0802	0.023-0.101
50.8	73.0	47.0	34.0	0.0025-0.0495	0.041-0.095
63.5	90.0	58.0	44.0	0.0037-0.0497	0.049-0.108

Application of the proposed equations (4.30) and (4.31) required estimation of the resistance coefficient F and the friction factor f , and the relative submergence. It has been assumed that the short axis of D_{84} represents the height of the bed substrate. The relative submergence for each experiment was therefore calculated as the ratio of the measured depth to the short axis of D_{84} . Application of equation (4.30) required estimation of F and f . For each experimental run values of F (equation (4.10)) and f (equation (2.1)) were calculated. For each test, graphs of F and f as functions of the relative submergence were plotted and were extended, if necessary, to relative submergences equal to one for graphs of F and to four for graphs of f . These graphs were used to estimate the values of F and f . These values are listed in Table 4-15.

Table 4-15 Values of resistance coefficient F and friction factor f estimated from Bathurst's experimental data for use in equation (4.30)

Bed material (mm)	Slope	y/D_{84}	F	f
12.7	0.02	1.65 - 2.46	1.534	0.236
	0.05	2.53 - 5.91	0.734	0.208
	0.08	1.59 - 4.59	1.010	0.231
19.5	0.02	2.29 - 5.22	1.443	0.232
	0.05	1.66 - 4.36	0.890	0.172
	0.08	1.29 - 3.66	1.04	0.137
38.1	0.02	1.10 - 3.74	0.730	0.181
	0.05	0.85 - 2.93	0.673	0.169
	0.08	0.92 - 2.43	0.494	0.216
50.8	0.02	1.48 - 2.79	2.939	0.040
	0.05	1.30 - 2.32	1.978	0.146
	0.08	1.21 - 2.20	1.200	0.162
63.5	0.02	1.29 - 2.46	1.035	0.158
	0.05	1.11 - 2.02	0.937	0.089
	0.08	1.05 - 1.84	0.798	0.071

Measured and predicted (equations (4.30) and (4.31)) velocities together with the perfect fit line and 25 % accuracy limits for experiments with five flume beds are plotted in Fig. 4-16.

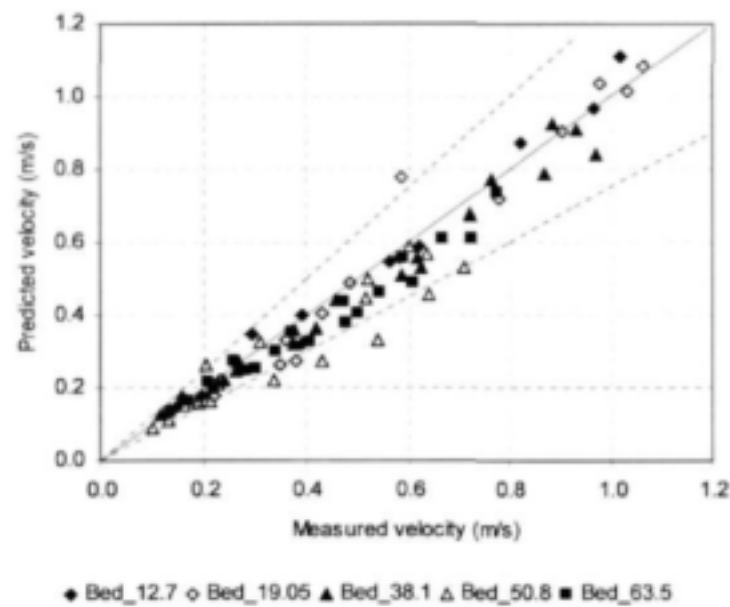


Figure 4-16 Measured and predicted (equations (4.30) and (4.31)) velocities with 25% accuracy limits for Bathurst's experiments listed in Table 4-14

Average, maximum and minimum absolute errors in prediction of flow velocity were calculated for each different bed material size and slope, and these are listed together with the standard deviation in Table 4-16.

Table 4-16 Average, maximum and minimum prediction (equations (4.30) and (4.31)) errors

Bed material (mm)	Slope	Average error (%)	Maximum error (%)	Minimum error (%)	St. deviation of prediction error (%)
12.7	0.02	2.50	2.73	2.27	0.23
	0.05	9.25	16.85	4.46	4.23
	0.08	7.76	18.06	0.18	6.98
19.5	0.02	19.70	32.57	1.30	11.52
	0.05	5.26	8.37	0.17	2.60
	0.08	10.26	28.62	1.27	10.28
38.1	0.02	8.48	13.83	1.16	4.40
	0.05	8.85	15.39	2.21	4.51
	0.08	6.76	13.40	1.66	3.85
50.8	0.02	7.06	15.33	1.63	4.66
	0.05	29.49	38.73	15.08	8.23
	0.08	14.60	27.40	3.67	8.85
63.5	0.02	14.75	20.13	6.19	4.71
	0.05	14.24	20.52	4.26	5.51
	0.08	5.84	8.50	2.70	1.68

The measured and predicted velocities plotted in Fig.4-16, and predicted errors listed in Table 4-16 show that the proposed approach can be recommended for estimation of flow velocity under intermediate-scale roughness conditions.

4.4.2.2 Verification of Proposed Equations (4.30) and (4.31) with Bathurst (1985) and Hicks and Mason (1998) published field data

Further verification of the performance of the proposed equations ((4.30) and (4.31)) was carried out by comparison of measured and predicted flow velocities of Bathurst (1985) and Hicks and Mason (1998) published field data that satisfied the intermediate-scale criteria. Data used for this verification are listed in Table 4-17.

The prediction approach was first applied to the field data of Bathurst (1985). Measured and predicted (equations (4.30) and (4.31)) flow velocities are plotted in Fig. 4-17. Predictions by Bathurst's (2002) equations ((2.18) and (2.19)) are included for comparison in Fig. 4-17, as these were derived for flow conditions with $y/D_{84} < 11$ representing the intermediate-scale roughness. Predicted (equations (2.18) and (2.19)) values are denoted "Bathurst (2002)" while equations ((4.30) and (4.31)) predictions are denoted "Equation (4.31)".

It can be seen (Fig. 4-17) that flow velocity prediction for both approaches is very similar. Average, maximum and minimum absolute prediction errors together with the standard deviation are presented in Table 4-18.

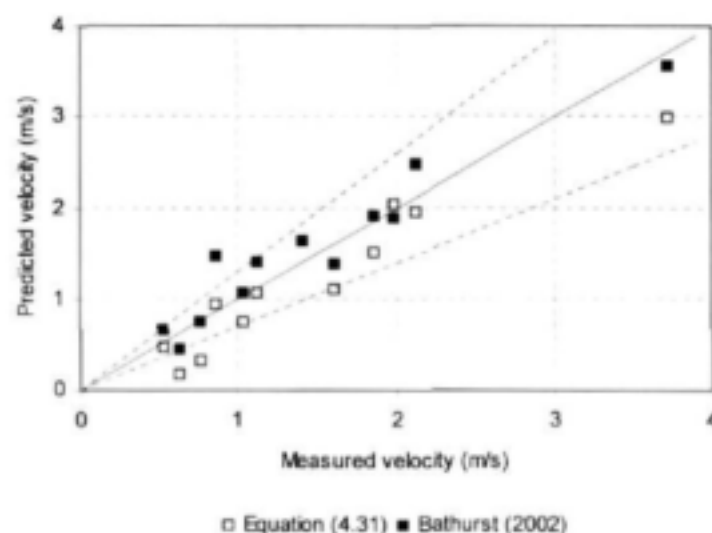


Figure 4-17 Measured and predicted (equations (4.30) and (4.31), and (2.18) and (2.19)) flow velocities for published field data of Bathurst (1985)

Table 4-17 Published field data used for verification of proposed equations (4.30) and (4.31)

Data source	River (site name)	Mean flow depth (m)	Bed material D_{50} (mm)	Number of measurements
Bathurst (1985)	South Tyne	0.50	240	1
	Ettrick	0.21 - 0.47	193	3
	Tweed	0.72	183	1
	Tromie-2	0.40 - 0.89	387	5
	Findhorn	0.30 and 0.45	140	2
Hicks and Mason (1998)	Waiau Water Race	0.22-0.30	80	3
	Cardrona	0.28 and 0.30	78	3
	Hutt	0.42 - 0.67	212	3
	Clarence	0.38 - 0.77	200	6
	Forks	0.28 and 0.39	104	2
	Waipapa	0.39 and 0.41	91	2
	Flaser	0.31 - 0.42	208	3
	Rowallanburn	0.62 and 0.86	250	2
	Northbrook	0.16 - 0.26	50	4
	Ruakokapatuna	0.24 and 0.42	119	2
	Kapoaiaia	0.26 - 0.54	212	5
	Butchers Creek	0.31 - 0.67	168	5
	Stanley Brook	0.32	106	1

Table 4-18 Average, maximum and minimum absolute predicted errors in application of equations (4.30) and (4.31), and (2.18) and (2.19) to Bathurst (1985) field data

Approach	Average error (%)	Maximum error (%)	Minimum error (%)	St. deviation of prediction error (%)
Equations (2.18) and (2.19)	17.66	69.18	1.60	18.08
Equations (4.30) and (4.31)	23.87	72.41	1.67	21.02

Prediction errors in application of the proposed equations ((4.30) and (4.31)) are slightly higher. Nevertheless, this approach was developed based on the laboratory data only while the development of equations ((2.18) and (2.19)) was based on this field data.

Equations (4.30) and (4.31), and (2.18) and (2.19) were also applied to the Hicks and Mason (1998) field data. Predicted and measured flow velocities together with the perfect fit line and 30% accuracy limits are plotted in Fig. 4-18. Average, maximum and minimum prediction errors and the standard deviations were calculated and are presented in Table 4-19.

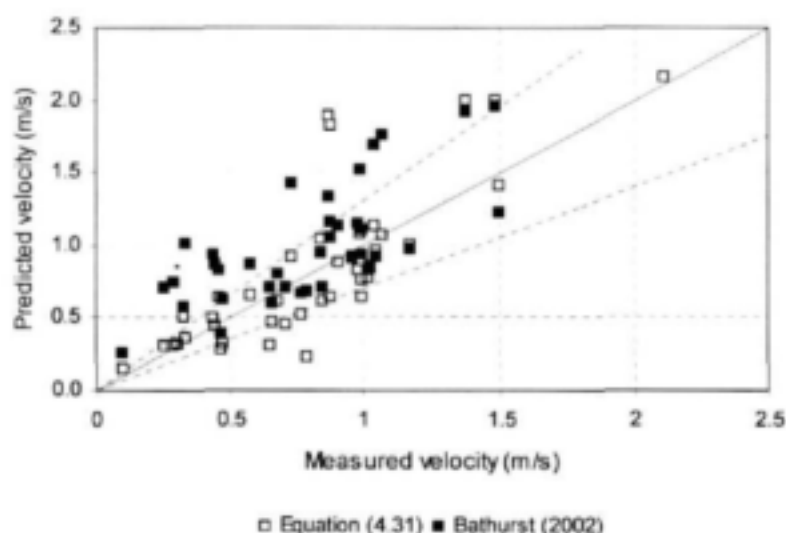


Figure 4-18 Measured and predicted (equations (4.30) and (4.31), and (2.18) and (2.19)) flow velocities with 30% accuracy limits for published Hicks and Mason (1998) field data

Table 4-19 Average, maximum and minimum absolute predicted errors in application of equations (4.30) and (4.31), and (2.18) and (2.19) to Hicks and Mason (1998) field data

Approach	Average error (%)	Maximum error (%)	Minimum error (%)	St. deviation of prediction error (%)
Equations (2.18) and (2.19)	51.48	197.29	0.87	52.88
Equations (4.30) and (4.31)	27.09	116.88	0.20	25.47

As before, when equations (2.18) and (2.19) are applied to the independent field data the resulting prediction is much poorer compared to predictions using the proposed method.

4.5 Conclusions

The different resistance effects in river channels with coarse substrates under small-, intermediate- and large-scale roughness conditions have been described by appropriate equations. Conventional shear resistance type equations are appropriate for small-scale roughness conditions. Equation (4.10) is proposed for estimating velocity under large-scale roughness condition with the resistance coefficient F given as a function of roughness element Reynolds number, Froude number and areal coverage by equation (4.16). These equations were tested against independent laboratory data and Cotter River field data, and proved to give satisfactory

performance. For field applications, equation (4.20) is proposed for estimating F . This is similar to equation (4.16) but accounts for bed roughness in terms of the geometric standard deviation of bed material particle sizes rather than the areal coverage of largest clasts. Application to field data showed performance to be at least as good as the best known, but less rationally justifiable alternative. Both equations (4.16 and 4.20) can be used for field applications depending on which parameter Λ or σ is estimated for a site under consideration.

Two approaches (Approach 1 and Approach 2) are proposed for estimating velocity under intermediate-scale roughness conditions. Approach 1 is based on an assumption that the total discharge is the sum of the discharges below and above the top of the large roughness elements. Based on this assumption, equation (4.29) for estimating the average velocity was theoretically developed and then applied to experimental data. Application of equation (4.29) to experimental results suggests that the approach performs satisfactorily. Its performance has yet to be tested against field data. An alternative new way (Approach 2) (equation (4.30)) for estimating velocities under intermediate-scale roughness conditions incorporating the influence of both large-and small-scale roughness is proposed. Coefficient a (equation (4.31)) effects partitioning of the influences of the two roughness scales. The proposed equations ((4.30) and (4.31)) were verified against experimental (Bathurst et al, 1981) and field (Bathurst, 1985; Hicks and Mason, 1998) data with promising results.

5 EXPERIMENTAL INVESTIGATION OF HYDRAULIC CONDITIONS WITH LARGE ROUGHNESS ELEMENTS

5.1 Introduction

Flow depth and velocity are hydraulic parameters used to describe aquatic animals' habitats and to predict biotic responses to discharge in a river. Under low flow conditions in a riffle area, a wide range of local flow depths and velocities occur, and the cross-section average depth and average velocity are insufficient to define the aquatic habitats at different flows. Prediction of flow depth and velocity distributions for a given discharge is therefore an important part of environmental studies.

At low flows, rocks and boulders control the local velocity and depth distributions. Flow is rapidly varied, and the occurrence of particular local velocities and depths is caused by the boundary geometry rather than flow resistance phenomena. Under such conditions, hydraulic features such as hydraulic jumps, local backup, contractions and critical controls occur over the whole area (Fig.5-1). Furthermore, all these features change with discharge. Under such conditions an average flow depth and velocity have limited value for ecological interpretation. As the discharge increases, the multiple local controls become submerged and the flow tends towards a resistance controlled condition, with consequent changes in the distributions of local flow depth and velocity.

Prediction of velocity distributions under multiple local control conditions is very difficult. Although the phenomena are easily understandable through elementary rapidly varied flow theory using concepts of Specific Energy, Momentum Function and the occurrence of critical flow, the complexity of the situation (such as shown in Fig.5-1, for example) makes direct application of this theory practically impossible.

An alternative to deterministic prediction is statistical description, and statistical models (Chapter 2) have been proposed for quantifying velocity and depth distributions. These are generally based on field data and do not account explicitly for multiple local control conditions.

Fig. 5-1 shows a wide range of flow depths and velocities, typical of multiple local controlled conditions. An understanding of the effect of various controlling factors is required for prediction of velocity and depth distributions under multiple local controlled conditions, whether a deterministic or a statistical approach is followed. For this reason, laboratory experiments were conducted in the hydraulics laboratory of the School of Civil and Environmental Engineering, University of the Witwatersrand. The results of this experimental work are presented in this chapter.

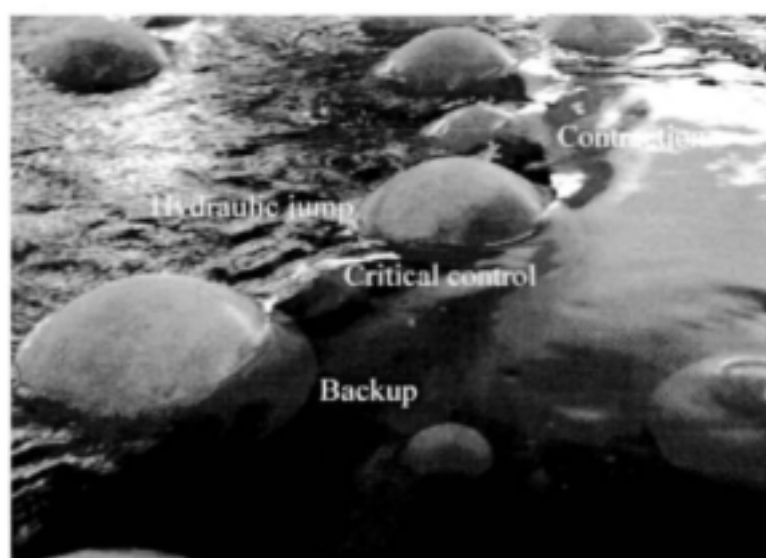


Figure 5-1 Example of multiple local controlled conditions created in the laboratory flume

5.2 Laboratory Investigations of Local Velocity Distributions: Flume C Experiments

5.2.1 Experimental Conditions

A series of experiments were carried out to test the influence of the transverse spacing between roughness elements and the control condition (multiple local control or resistance control) on the statistical distribution of local velocities. The experiments were conducted in a 0.5m wide channel formed within a 2.0m wide, 15.0m long, tilting laboratory flume under controlled and idealized conditions. The flume slope was set to 0.0005 for all experiments. Water was supplied to the flume through a closed circulation system, and a control valve situated in the supply pipe at the head of the experimental flume was used to control the discharge. The discharge was measured with a V-notch, installed downstream of the flume, and by an electronic flow meter with sensors in the water supply pipe. The discharge was $0.0161\text{m}^3/\text{s}$ for all experiments. The downstream flow depth was controlled by an adjustable tailgate at the end of the flume.

Experiments were carried out to investigate velocity distributions under local controlled conditions created by two hemispheres ($D = 0.112\text{m}$) representing natural roughness elements. The hemispheres were placed in line along the cross section at Chainage 5.5m from the flume entrance (Fig.5-2), with three different spacings. Three different tailgate settings were used to induce multiple local controlled or

resistance controlled conditions for the different arrangements. The experimental conditions are listed in Table 5-1.

A two-dimensional *Nortek Doppler Velocimeter* (NDV) was used to measure velocity. Velocities were measured in the longitudinal (x) and transverse (y) directions. A total of 1000 samples of a 9mm sampling volume at mid-flow depth were recorded at each measuring point for 40 seconds at a frequency of 25 Hz. Time-averaged velocities obtained by the NDV were determined using the CollectV Data Acquisition Program.

For all the experiments, mid-flow depth velocities were measured on a grid extending from Chainage 5.00m (0.50 m upstream of the hemispheres) to Chainage 7.00m (1.50 m downstream of the hemispheres). Measurements were taken over 20 cross sections at 0.05 m transverse intervals. The cross sections were located with varying spacing, to ensure adequate description of longitudinal variations. From Chainage 5.00m the cross sections were spaced at 0.10m up to Chainage 5.50m, then at 0.05m up to Chainage 5.80m, at 0.10m up to Chainage 6.20m, and at 0.20m up to Chainage 7.00m. It must be recognized that a statistical representation of a velocity distribution in situations such as this will depend significantly on the sample area. The experiments were conducted only to enable comparison of the effects of the different arrangements and are not attempts to define usable distributions.

Table 5-1 Experimental conditions

Test	Discharge (m ³ /s)	Slope	Spacing (m)	Tailgate position (m)	Control Condition
1	0.0161	0.0005	0.14	0.1005	MLC
2	0.0161	0.0005	0.24	0.1005	MLC
3	0.0161	0.0005	0.38	0.1005	MLC
4	0.0161	0.0005	0.24	0.1012	RC
5	0.0161	0.0005	0.38	0.1012	RC
6	0.0161	0.0005	0.24	0.1030	RC
7	0.0161	0.0005	0.38	0.1030	RC

MLC: Multiple Local Control

RC: Resistance Control

Flow depths were also measured at the velocity measurement locations but are not presented here because the water surface disturbances in the supercritical flow regions were of the same order of magnitude as the flow depth. The error margins on the measurements are therefore too great for statistical interpretation in the same way as for the velocity measurements.

5.2.2 Results

For better understanding of how local velocities change with spacing between obstacles and with hydraulic conditions (multiple local control or resistance control), a statistical method was used to analyze the measured data. A frequency distribution approach was applied to all the experimental tests to count numbers of occurrences of each measured value in each data set over the same area. A bar chart graphic format was used for showing the results. The measured velocities are presented in Appendix E, in Tables E-1 to E-7 for each of the 7 conditions tested.

To be able to compare velocity frequency distributions for different experimental runs, the graphs should be plotted in the same velocity scale. The measured velocities (Appendix E) were therefore analyzed to establish the maximum that was measured. Then a decision of how many classes to have and how wide the classes should be was made. As the maximum velocity was measured in Test 2 (0.655 m/s), all results presented here are therefore plotted with 7 classes within the class width of 0.1m/s.

For Test 1, a locally controlled condition was created by setting the tailgate position appropriately (Fig. 5-2). The distribution of local velocities is shown by the histogram in Fig. 5-3.

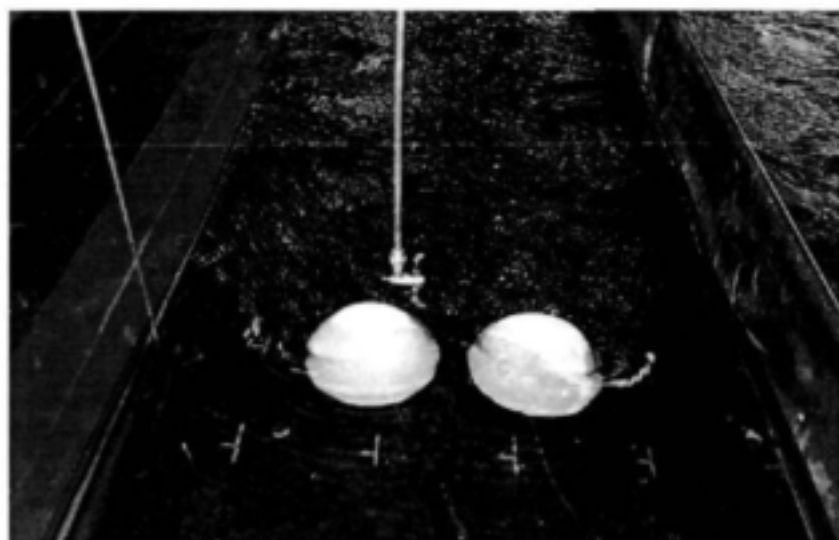


Figure 5-2 Tests 1, two hemispheres with spacing of 0.14 m

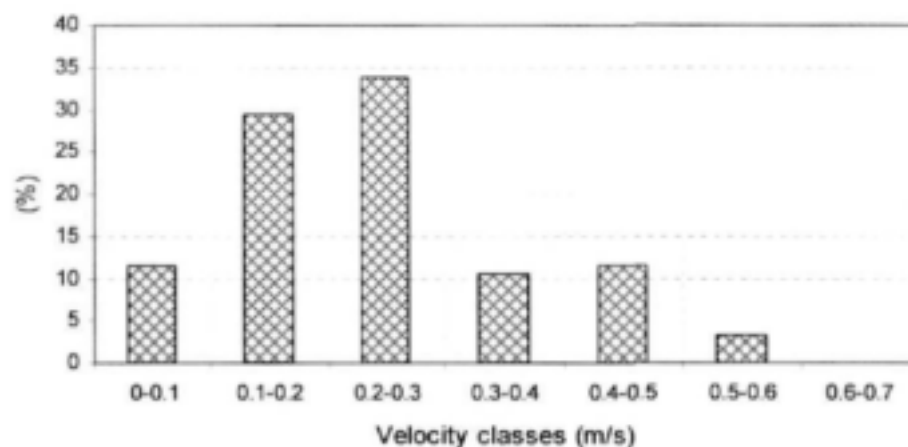


Figure 5-3 Test 1, velocity distribution histogram

The histogram is unimodal and skewed to the right. The mode is within the 0.2 to 0.3m/s velocity class. It can be seen that the highest velocity class is void, presumably because the narrow gap restricted the development of high velocities.

For Test 2, the spacing between the hemispheres was increased to 0.24m (Fig. 5-4) and local controlled conditions were maintained. The velocity distribution histogram is shown in Fig. 5-5.

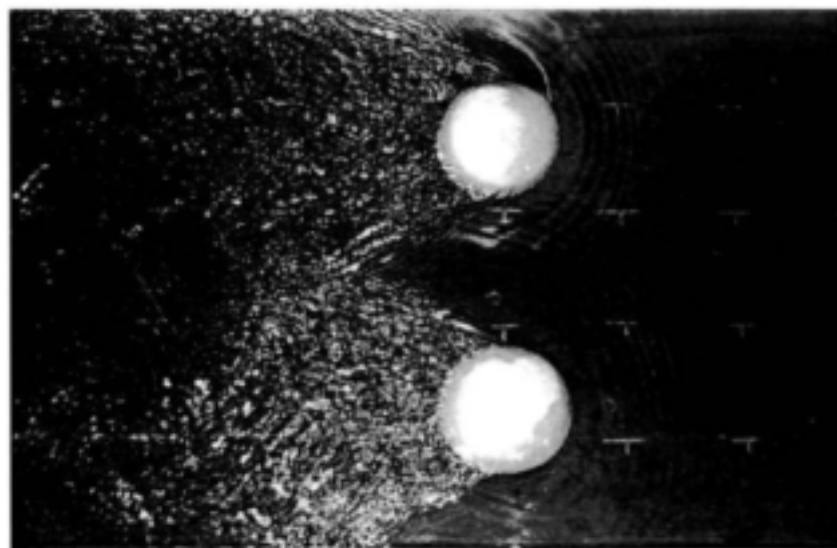


Figure 5-4 Test 2, hemispheres with spacing of 0.24 m

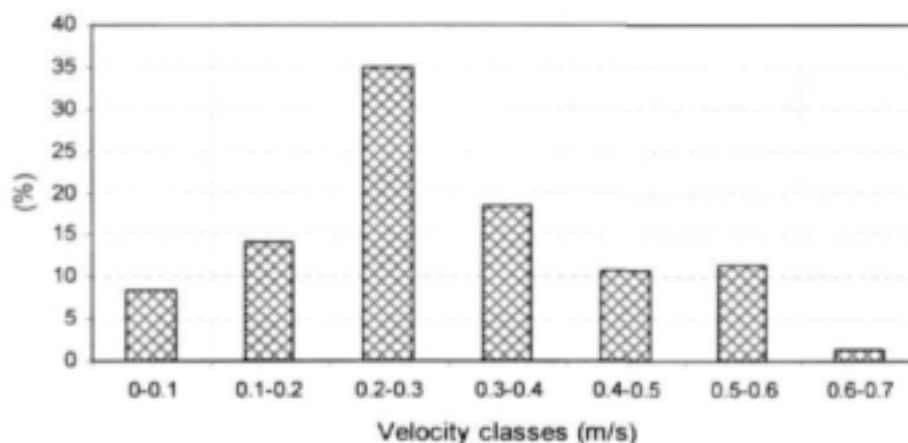


Figure 5-5 Test 2, velocity distribution histogram

From the histogram can be seen that with an increased spacing of 0.24m, a wider range of velocities occurred.

For Test 3 the two hemispheres were positioned with spacing of 0.38m (Fig. 5-6), again with local control. A histogram of the measured velocities is in Fig.5-7. The histogram is unimodal, but skewed to the left.

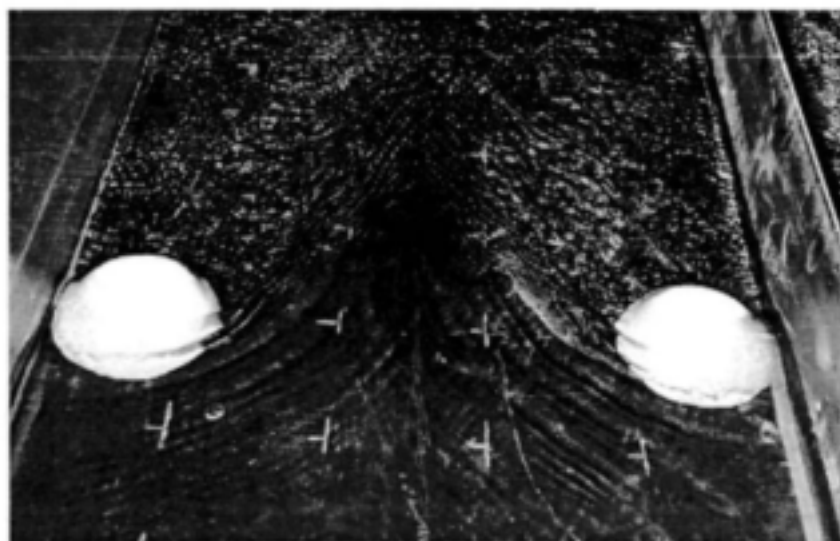


Figure 5-6 Test 3, hemispheres with spacing of 0.38 m

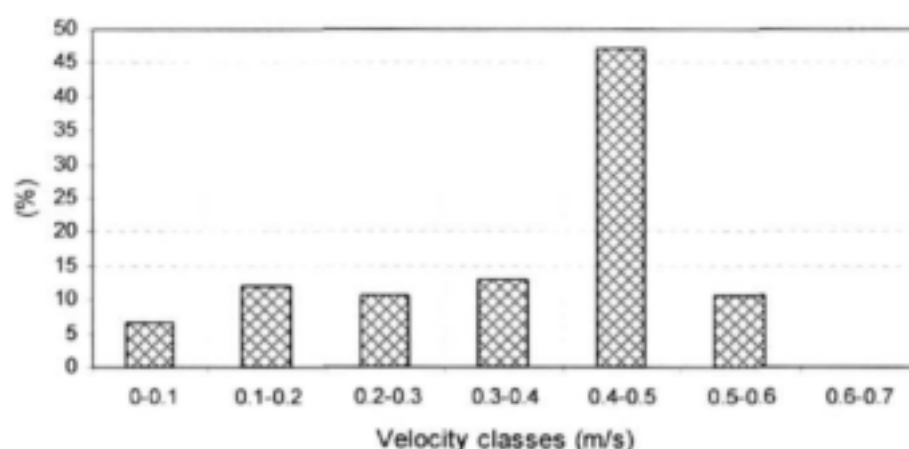


Figure 5-7 Histogram of velocity distribution for test 3

From Fig. 5-7 it can be seen that the highest velocity class is void, as for the narrower gap (Fig. 5-5), and the strong mode characterizes the uniformity.

For Test 4, the hemispheres were positioned as for Test 2, but the tailgate was raised by 0.007m in order to submerge the local control influences. The resulting velocity histogram is shown in Fig. 5-8. The mode is within the same 0.2 to 0.3 velocity class as for Test 2. With this condition another higher velocity class (0.5-0.6m/s) is void, and the mode is within the 0.1-0.2m/s velocity class, indicating greater uniformity than under locally controlled conditions.

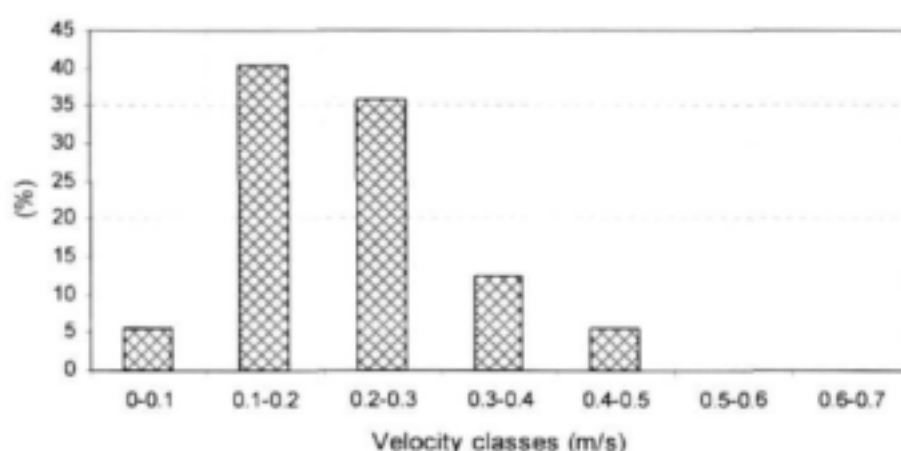


Figure 5-8 Test 4, velocity distribution histogram

For Test 5, the hemispheres were positioned as in Test 3, and the position of the tailgate was as in Test 4, again to submerge the local control effects. The histogram of velocity distribution is presented in Fig. 5-9. This histogram (Fig. 5-9) is very different from that for Test 3 (Fig. 5-3). No velocities occur in the 0.5-0.6 class, while the other five classes have similar occurrences, showing a more consistent frequency distribution.

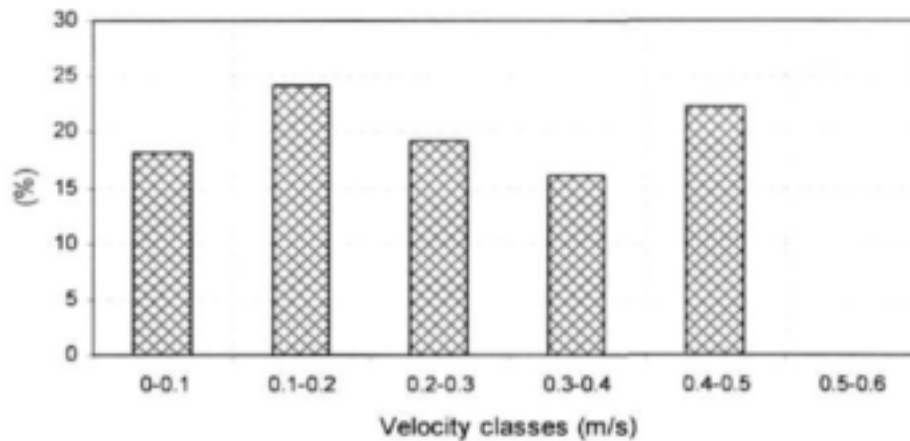


Figure 5-9 Test 5, velocity distribution histogram

Test 6 was run to investigate increased submergence by raising the height of the tailgate to 0.018m. The velocity distribution histogram is shown in Fig. 5-10. About 80% of the measured velocities are within 0.1-0.2m/s class. There are no measured velocities higher than 0.4m/s. The strong mode characterizes the uniformity.

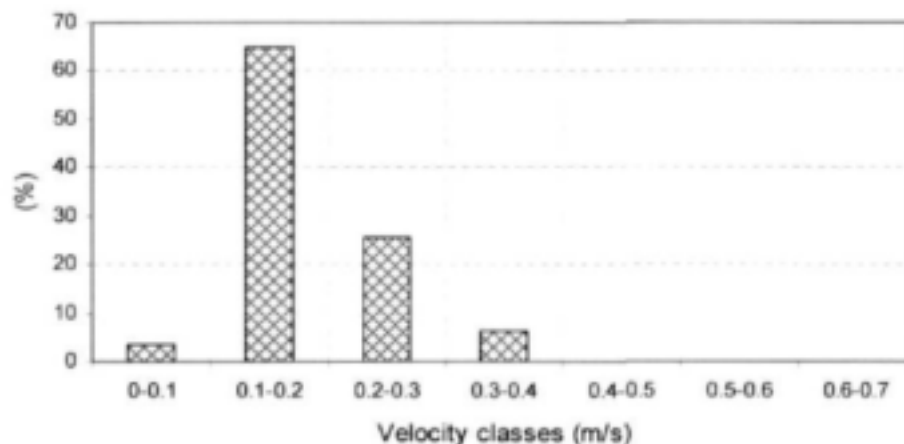


Figure 5-10 Test 6, velocity distribution histogram

Test 7 was conducted for spacing between hemispheres of 0.38m, and the tailgate height as in Test 6. The velocity distribution histogram is plotted in Fig. 5- 11. All the measured velocities are within the range of 0 to 0.4m/s, with only four classes.

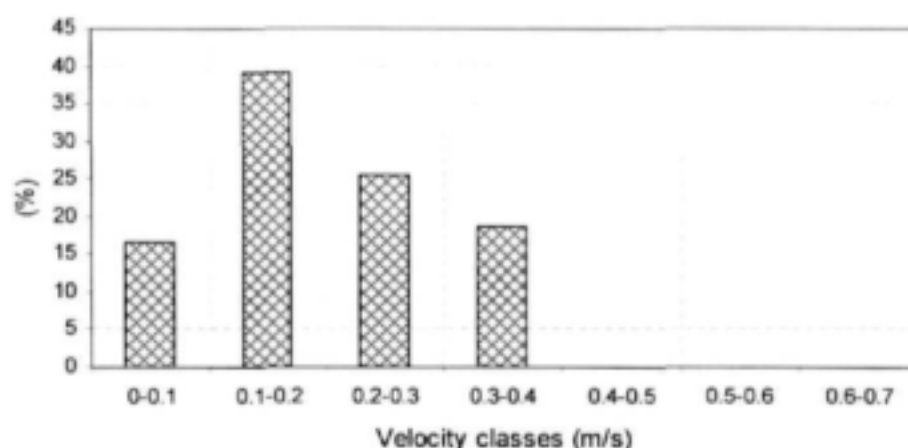


Figure 5-11 Test 7, velocity distribution histogram

5.2.3 Discussion

The influence of the different spacings of 0.14, 0.24 and 0.38m in Tests 1, 2 and 3 respectively on the velocity distribution under local controlled conditions is shown in the comparison of velocity distribution histograms in Fig. 5-12. This shows that with the same tailgate setting, the velocity range is similar for the different spacings, but the distributions are rather different. The highest range of velocities occurred with the intermediate spacing of 0.24m (Test 2); the highest velocity class was void for a spacing of 0.14m (Test1) and also for the widest spacing (Test 3). It is therefore concluded that spacing between the large roughness elements is one of the parameters that influences the velocity distribution, with large spacing resulting in high local velocities under multiple local control conditions.

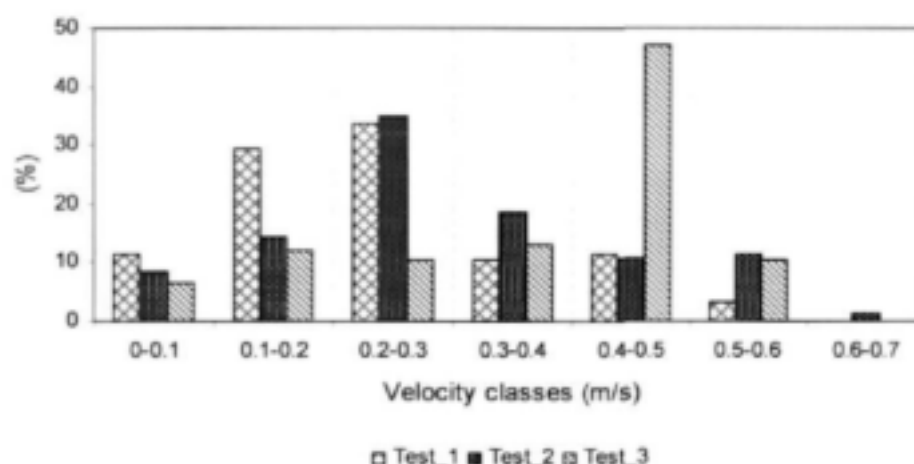


Figure 5-12 Velocity distribution histogram for tests 1, 2 and 3

The influence of downstream conditions on the velocity distributions for tests 2, 4 and 6 with spacing of 0.24m and tests 3, 5 and 7 with spacing of 0.38m are shown in Figs 5-13 and 5-14 respectively. Spatially explicit velocity distributions for spacings 0.24m (Tests 2 and 6) and 0.38m (Tests 3 and 7) are presented in Figs 5-15 and 5-16 respectively. It is apparent that locally controlled conditions (Tests 2 and 3) produce a wider range of local velocities than occur under resistance controlled conditions (Tests 6 and 7). Submergence of local controls (Tests 2, 4 and 6, and 3, 5 and 7) therefore decreases the hydraulic diversity and multiple local controlled conditions can be expected to provide a wider variety of physical habitats than resistance controlled conditions. This difference emphasizes the importance of being able to predict velocity distributions in environmental studies in which aquatic animals' habitats need to be specified.

These results suggest that the lack of generality of statistical descriptions of velocity distributions severely limits their useful application in practice.

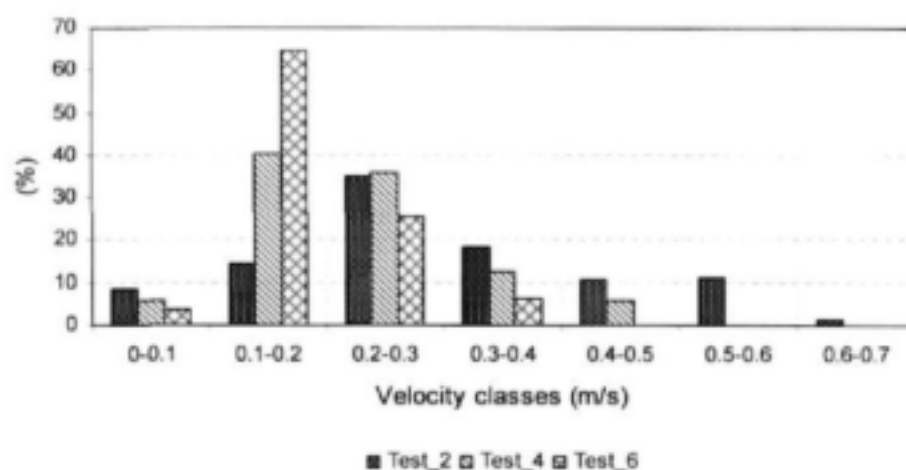


Figure 5-13 Velocity distribution histogram for tests 2, 4 and 6

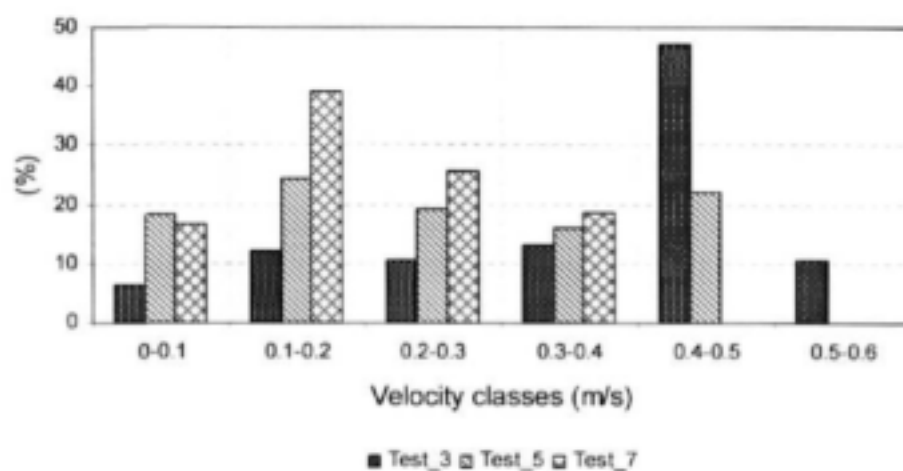


Figure 5-14 Velocity distribution histogram for tests 3, 5 and 7

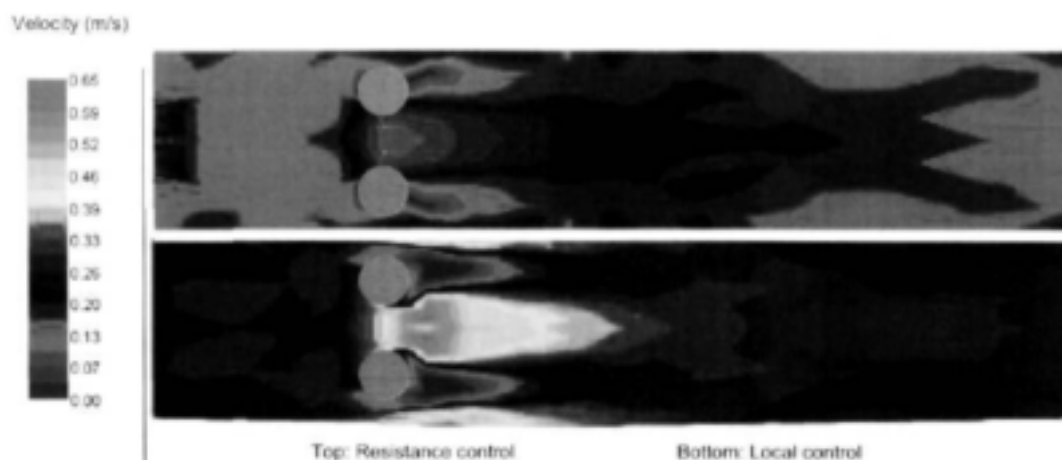


Figure 5-15 Explicit velocity distributions for spacing of 0.24m for Test 2 (bottom graph) and Test 6 (top graph)

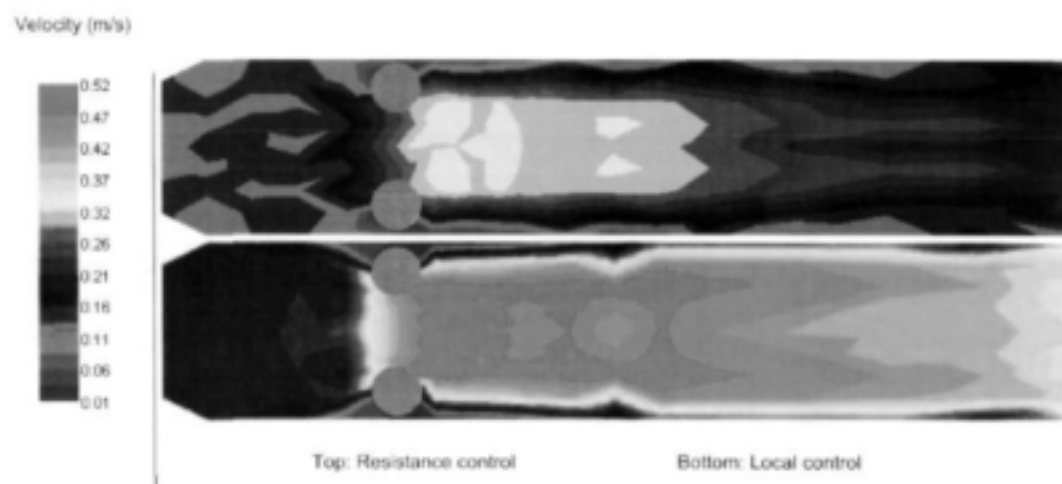


Figure 5-16 Explicit velocity distributions for spacing of 0.38m for Test 3 (bottom graph) and Test 7 (top graph)

5.3 Laboratory Investigations of Local Velocity Distributions: Flume B Experiments

The Flume C experiment results have been presented in the form of local velocity histograms. These give representations of the relative occurrences of different velocities within specified classes in a reach of channel, without conveying any idea of where the velocities occur in relation to the roughness elements. Aquatic animals often use relatively localized areas of a stream for particular functions, and experience

a wide range of hydraulic conditions even around single rocks. It is useful to be able locate suitable habitat areas in a spatially explicit way, and to understand the influence of channel characteristics on hydraulic conditions at particular specified locations. This set of experiments was conducted to gain understanding of the influence on the velocity distributions around a single roughness feature of other features in the channel.

5.3.1 Experimental Conditions

These experiments were conducted in a 0.38m wide, 15.0m long, glass-sided tilting laboratory flume set at a slope of 0.001136. A tailgate fixed downstream of the flume was used to establish the uniform flow for a given discharge. Water was supplied to the flume through a closed circulation system, and two valves situated in the supply pipe at the head of the flume were used to control the discharge. The discharge was measured with a V-notch installed downstream of the flume, and by an electronic flow meter with sensors in the supply pipe. The discharge was 0.005m³/s for all experiments, resulting in resistance controlled conditions.

Two shapes of roughness element, cylindrical and hemispherical, were tested. Test 1 was first carried out in the empty flume to determine the uniform flow depth and associated cross-sectional velocity distribution to establish a basis for assessing the effects of the large roughness elements. Tests 2 to 6 were performed using concrete hemispheres with $D = 0.114\text{m}$ placed on the flume bed at different chainages (measured from the beginning of the sloping part of the flume), while in Test 7 the roughness element was simulated by a cylinder with $D = 50\text{mm}$. Experimental conditions of all tests are listed in Table 5-2.

Velocities were measured in the longitudinal (x direction) and transverse (y direction) channel directions, using the same NDV apparatus as used in the Flume C experiments. A total of 500 samples in a 9mm sampling volume were recorded at each measuring point for 20 seconds at a frequency of 25 Hz. Time-averaged velocities obtained by the NDV were assessed using CollectV Data Acquisition Program. All measured data are included in Appendix F.

Velocities were measured on a grid extending from Chainage 3.5m in each case to an appropriate downstream location. Measurements were taken over a number of cross sections at 0.02m transverse intervals. The cross sections were located at different positions for each experiment to capture the distinct characteristics of each roughness pattern. Velocities were measured over half the channel width in each case.

Table 5-2 Experimental conditions

Test	Element Type	D (mm)	Number of Elements	Chainage (m)	Slope	Discharge (m ³ /s)
1	Empty	N/A	N/A	N/A	0.001136	0.005
2	Hemisphere	114	1	4.5	0.001136	0.005
3	Hemisphere	114	2	4.5 and 5.5	0.001136	0.005
4	Hemisphere	114	3	4.5, 5.5 and 6.5	0.001136	0.005
5	Hemisphere	114	4	4.5, 5.5, 6.5 and 7.5	0.001136	0.005
6	Hemisphere	114	2	4.5 and 8.5	0.001136	0.005
7	Cylinder	50	1	4.5	0.001136	0.005

5.3.2 Results

The measured velocities are presented in Appendix F, in Tables F-1 to F- 7 for each of the 7 conditions tested. Note that the velocity profiles presented here are plotted over the full channel width, assuming symmetrical repetition of the measurements over half the width.

The first experiment (Test 1) was conducted to establish uniform conditions and therefore was run without any roughness elements. The measured uniform depth was 0.035m. The transverse velocity distribution is shown in Fig. 5-17.

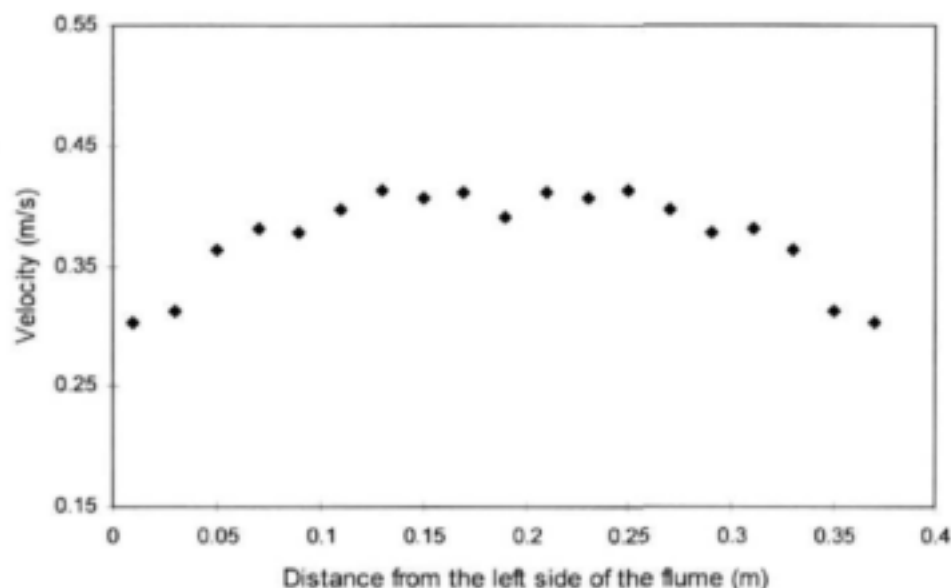


Figure 5-17 Transverse velocity distribution under uniform conditions (Test 1)

For Test 2, a single hemisphere was placed at chainage 4.50m. Velocities were measured at 19 cross-sections along the flume. Transverse velocity distributions at

Chainages 3.50, 4.00, 4.40, 5.50 and 6.60m together with that for the empty flume are shown in Fig. 5-18. A photograph of the experimental set up of Test 2 is included as Fig. F-1 in Appendix F. A plan-view of the spatial distribution of longitudinal velocity is illustrated in Fig. 5-19. Variations of longitudinal velocity together with the uniform velocity are shown on Figs F- 7 to F- 10 in Appendix F.

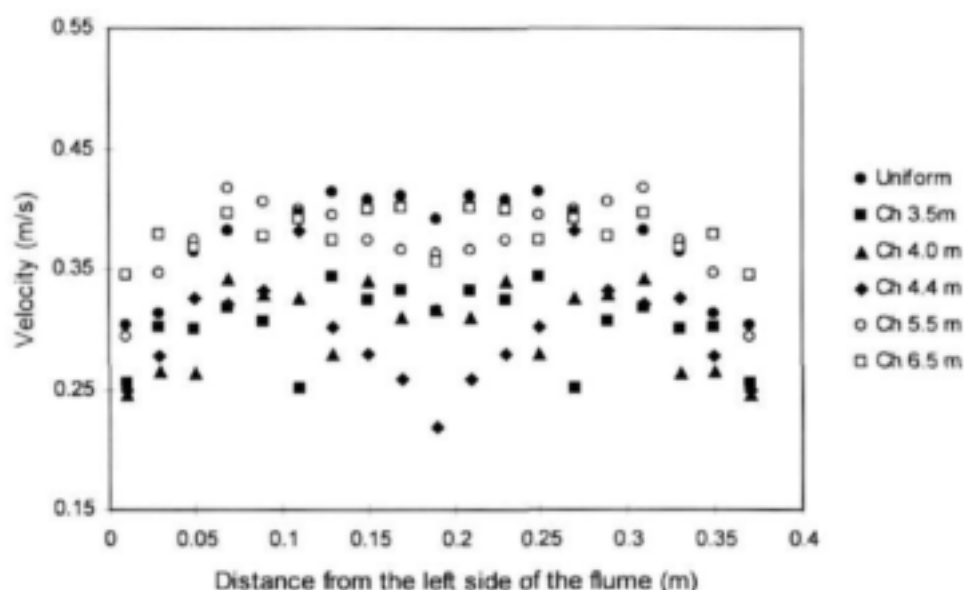


Figure 5-18 Measured transverse velocity distributions for Test 2

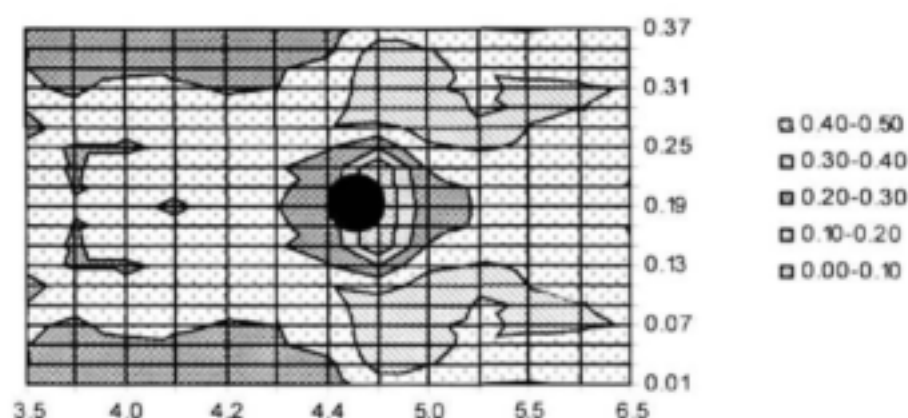


Figure 5-19 Plan-view of spatial distribution of longitudinal velocity for Test 2

For Test 3, two hemispheres were positioned on the bottom of the flume at Chainages 4.50 and 5.50m. Velocity measurements were taken at 24 cross-sections. Transverse

velocity distributions at Chainages 3.50, 4.40, 4.70, 5.00, 5.60, 6.10 and 7.00m are shown in Fig. 5-20. A photograph of the Test 3 experiment is given as Fig. 2 in Appendix F. The spatial distribution of longitudinal velocity is presented as Fig. 5-21. Graphs of the longitudinal distributions of velocity, together with the uniform velocity are included in Appendix F as Figs F-17 to F-26.

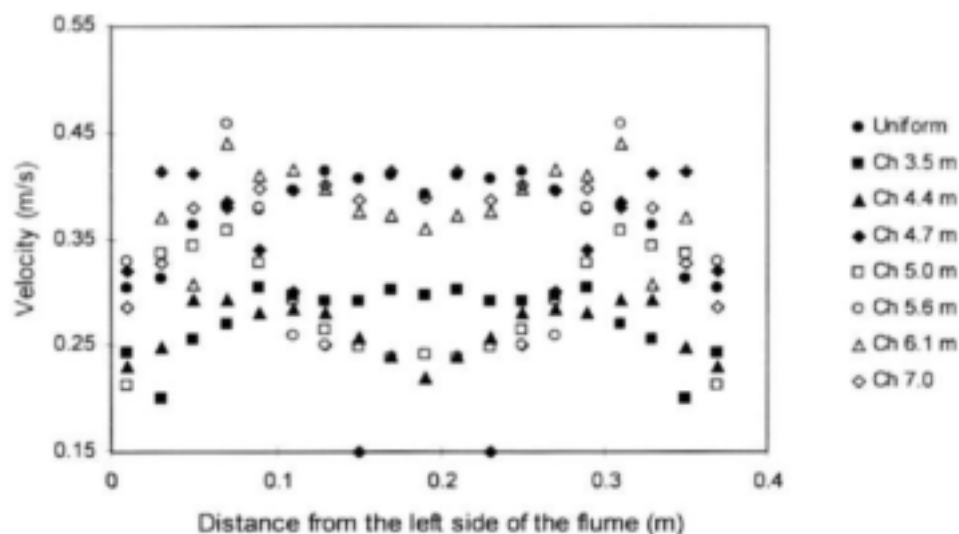


Figure 5-20 Measured transverse velocity distributions for Test 3

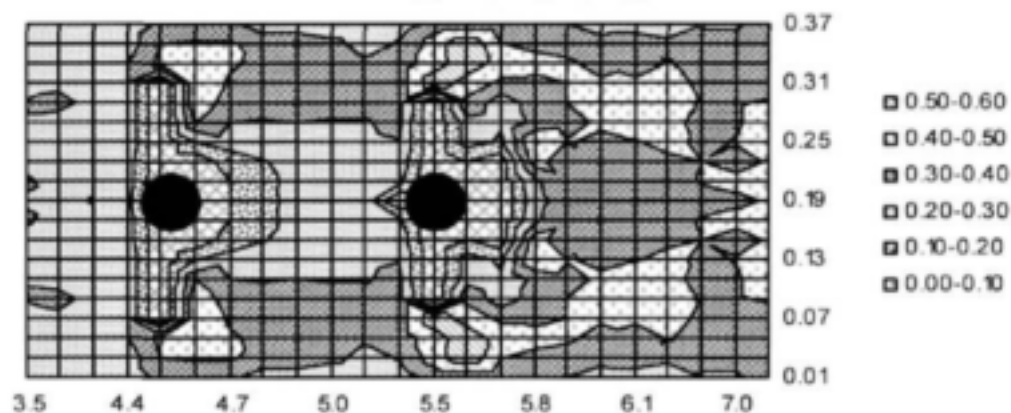


Figure 5-21 Plan-view of spatial distribution of longitudinal velocity for Test 3

One more hemisphere was placed at Chainage 6.50m for Test 4, and velocities were measured at 24 cross-sections. Cross-sectional measured velocities are shown in Fig.

5-22. A view of the spatial distribution of longitudinal velocity is presented in Fig. 5-23. A photograph of the Test 4 experiment is present in Fig. F-3 (Appendix F).

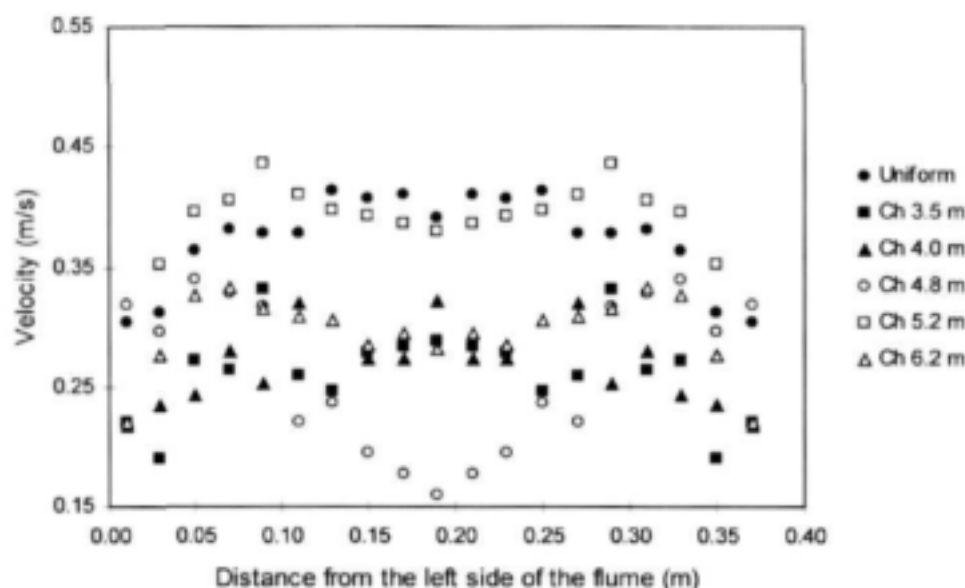


Figure 5-22 Measured transverse velocity distributions for Test 4

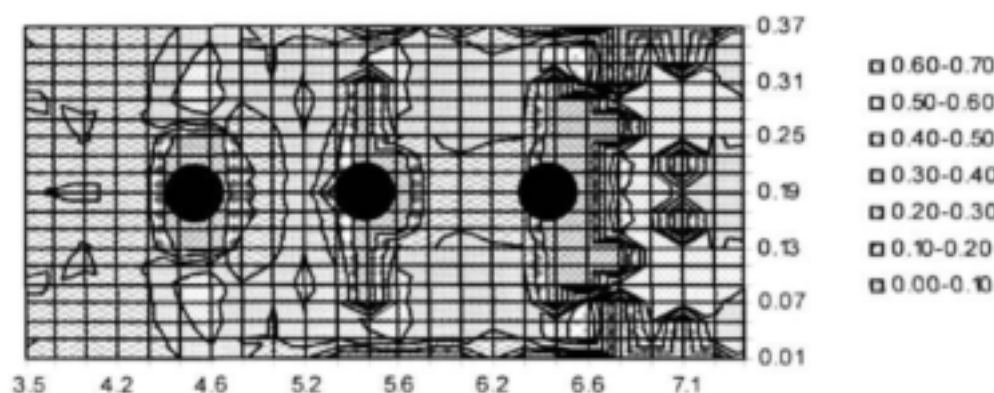


Figure 5-23 Plan-view of spatial distribution of longitudinal velocity for Test 4

Test 5 was conducted with four hemispheres positioned at Chainages 4.50, 5.50, 6.50 and 7.50m. Velocities were measured at 28 cross-sections; the results are shown in Fig. 5-24. A photograph of the experimental setup is showed in Fig. 4 in Appendix F. A view of the longitudinal velocity distribution is presented in Fig. 5-25.

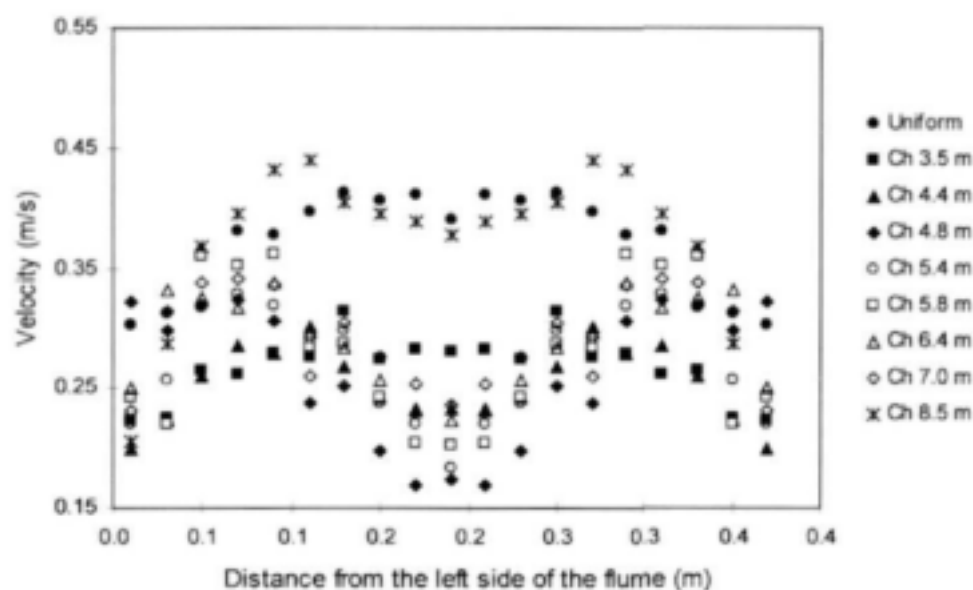


Figure 5-24 Measured transverse velocity distribution for Test 5

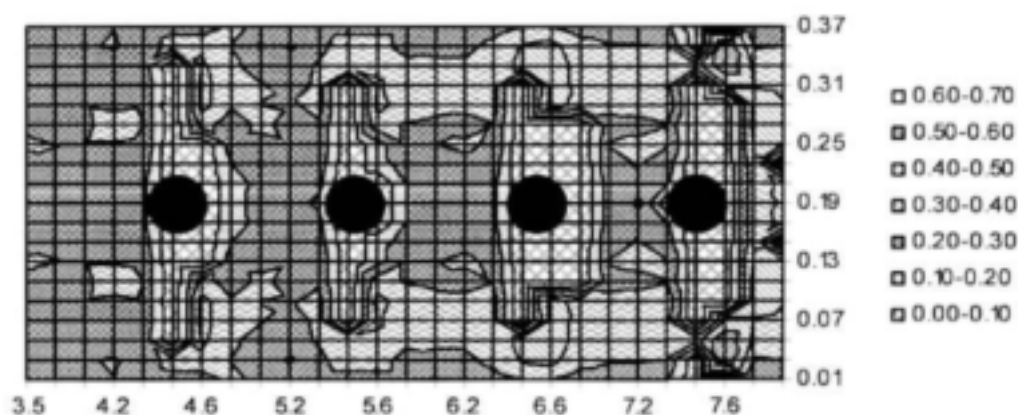


Figure 5-25 Plan-view of spatial distribution of longitudinal velocity for Test 5

The influence of two hemispheres on velocity and depth distributions was investigated in Test 6. Hemispheres were installed at Chainages 4.50 and 8.50m, and velocities measured at 14 cross-sections. The results are shown on Fig. 5-26. Graphs of longitudinal velocities are shown in Figs F-27 to F-36, in Appendix F. The experimental setup is shown in Fig. F-5 of Appendix F. A plot of the velocity distribution is given in Fig. 5-27.

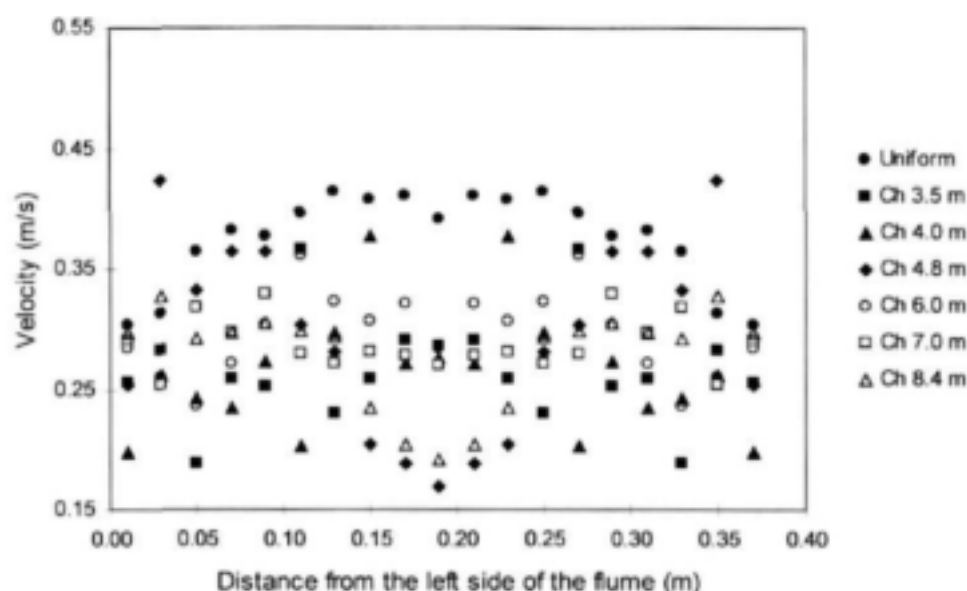


Figure 5-26 Measured transverse velocity distributions for Test 6

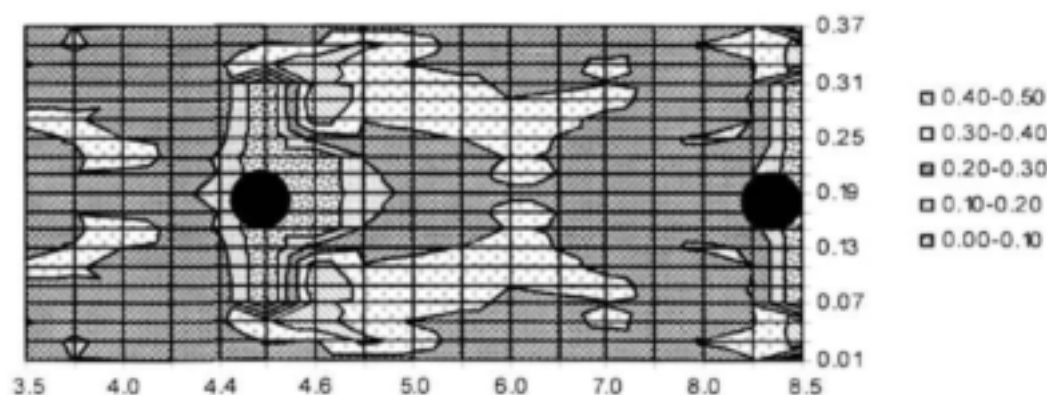


Figure 5-27 Plan-view of spatial distribution of longitudinal velocity for Test 6

The last experiment (Test 7) was carried out to examine the effect of roughness element shape on velocity distributions. In rivers many large rocks emerge through the water surface, and are better represented by cylinders than hemispherical shapes. The influence of a single cylinder on the velocity distributions was tested here. The cylinder was installed at Chainage 4.50m. The test is similar to Test 2 where the hemisphere was placed at the same chainage. Velocities were recorded at 16 cross-sections, and the results are plotted in Fig. 5-28. A photograph of the experiment is

shown in Fig. F-6 (Appendix F). A view of the velocity distribution is presented in Fig. 5-29.

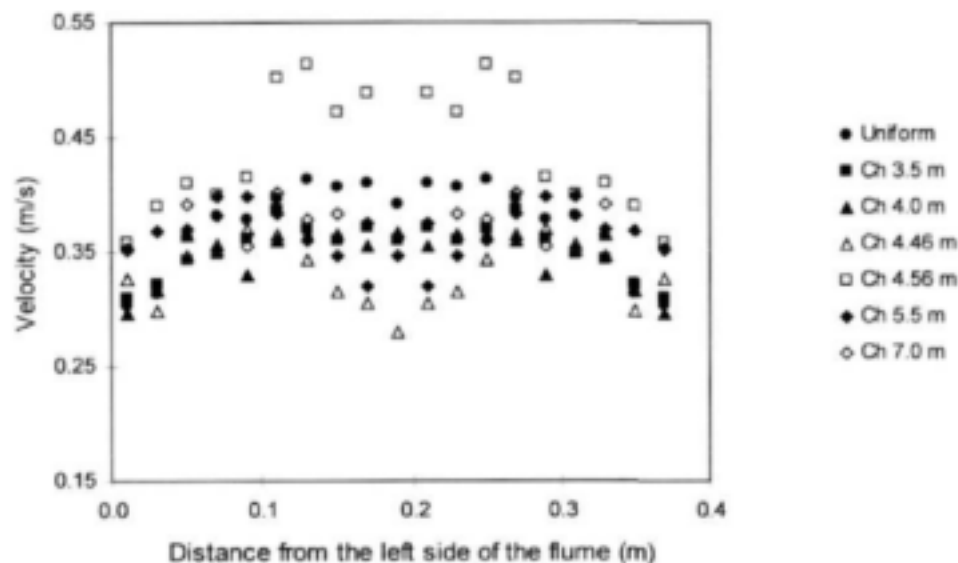


Figure 5-28 Measured transverse velocity distributions for Test 7

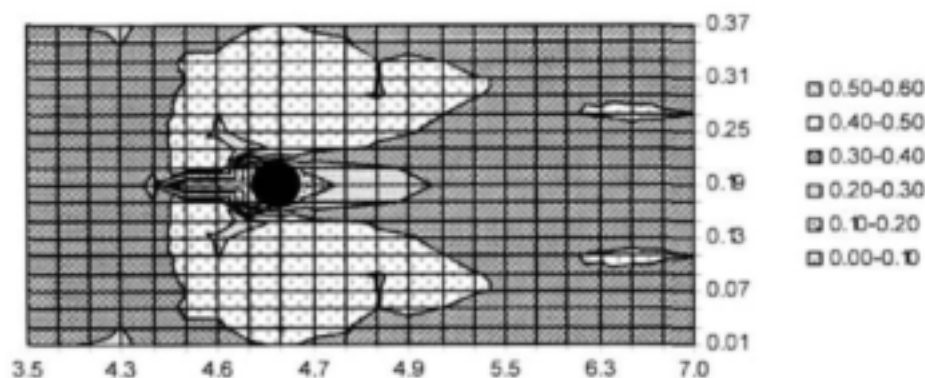


Figure 5-29 Plan-view of spatial distribution of longitudinal velocity for Test 7

5.3.3 Discussion

The experimental results for Test 2 (Fig. 5-18) show that a single obstacle affects the velocities for a considerable distance upstream and downstream. The velocities

upstream are all reduced by the presence of the obstacle, and even at the first measurement position (about 9 times the obstacle diameter (D) upstream) the maximum velocity is only about 81% of that for the unobstructed flow. At the most downstream section (about $18D$ downstream), the distribution has recovered almost completely, although the near wall velocities are actually slightly elevated.

The addition of a second hemisphere $1.0m$ ($9D$) downstream of the first (Test 3, Fig. 5-20) has a further significant effect on the velocity distribution. The maximum velocity $9D$ upstream is now only about 70% of that for the unobstructed flow. The profile has recovered completely by $9D$ downstream of the second hemisphere.

For a third hemispheres $1.0m$ ($9D$) further downstream (Test 4, Fig. 5-22), the upstream velocity distribution is even further affected. The velocity in the centre of the channel is reduced to 64% of the unobstructed value, but the profile has become distorted with the maximum value off centre and about 87% of the unobstructed value at its location.

A fourth hemisphere $1.0m$ ($9D$) downstream of the third (Test 5, Fig. 5-24) has no further effect on the velocity distribution upstream of the first.

Test 6 (Fig. 5-26) had only two hemispheres in line, but spaced very far apart. The influence of the second one on velocities upstream of the first one appears to be much the same as when there are more hemispheres in line over a similar distance.

These results suggest that obstacles in the lee zones of upstream ones have a significant effect on flow velocities upstream of the first one in line, but that recovery of the transverse profile below the last one in line is not greatly affected by obstacles upstream of the last one.

Comparison of Test 2 (for a single hemispherical obstacle) and Test 7 (for a single cylindrical obstacle) (Figs 5-20 and 5-28), shows the effect of the cylinder on the velocity distributions to be slightly less than that of the hemisphere, suggesting that obstacle shape could be an important consideration.

5.4 Conclusion

The Flume C experimental results show that local controlled conditions present greater hydraulic diversity in a channel than resistance controlled conditions. The distributions are also dependent on the spacings between roughness elements. The change in velocity distributions with flow condition makes their prediction very difficult.

The Flume B experimental results show that the distribution of velocity around an obstacle is influenced strongly by other obstacles in its proximity, even if these are within its lee zone. The shape of an obstacle also has an effect on the surrounding velocity distribution.

Use of statistical descriptions of velocity distributions must take cognizance of their sensitivity to channel bed geometry and flow conditions. Computational modelling provides a potentially more reliable approach, and is pursued in the following chapter.

6 PREDICTION METHODS FOR VELOCITY DISTRIBUTIONS WITH LARGE SCALE ROUGHNESS

6.1 Multiple Local and Resistance Controlled Conditions

The laboratory investigations of velocity distributions under local and resistance controlled conditions presented in Chapter 5 showed that the velocity frequency distributions induced by the different control conditions are different in nature. The differences are illustrated by the results for the case of two hemispheres spaced 0.24m apart in the Flume C experimental series (Tests 2 and 6) and 0.38m apart (Tests 3 and 7), which are reproduced in Figs 6-1 and 6-2 respectively. The range of velocity classes is wider under locally controlled conditions (Tests 2 and 3) than under resistance controlled conditions (Tests 6 and 7); the diversity of hydraulic conditions is therefore decreased by submergence of local control features. Both conditions commonly occur at the same location in coarse substrate rivers, with local control occurring at very low discharges and submergence taking place as discharge increases. This means that the frequency distributions of velocity at a particular site can be expected to change as discharge varies. Prediction methods of the velocity frequency distributions should clearly be able to distinguish between these control conditions, associate their occurrences with discharge values, and provide the appropriate histograms. Standard regional distribution functions developed from measured data at particular study sites are unable to do this – they do not generally distinguish between the conditions or allow for realistic ranges of discharge.

Computational modelling provides an alternative to data-based statistical distributions for predicting velocity frequency characteristics. There are now a number of commercial and public domain two-dimensional (2D) and three-dimensional (3D) hydraulic models available that may be used for these purposes. They have the advantage of generality in not being based on site-specific data, but rather on rigorous theory; they should therefore be expected to be able to account for changes in control phenomena with discharge. They are also able to predict both velocity and depth simultaneously. They have the disadvantages of cost (at least for the commercial packages), requiring a high level of user expertise, and requiring detailed site-specific topographical information, which is expensive to obtain.

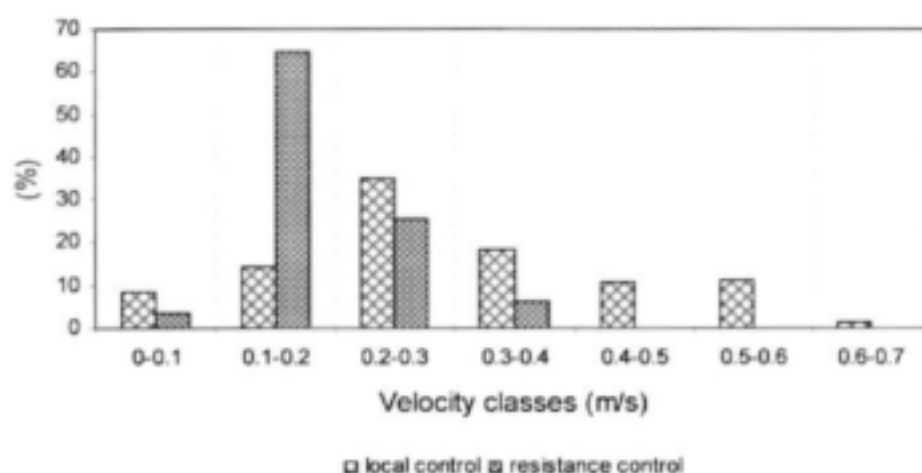


Figure 6-1 Velocity distribution histograms for local control (Test 2) and resistance control (Test 6) (Flume C experiments)

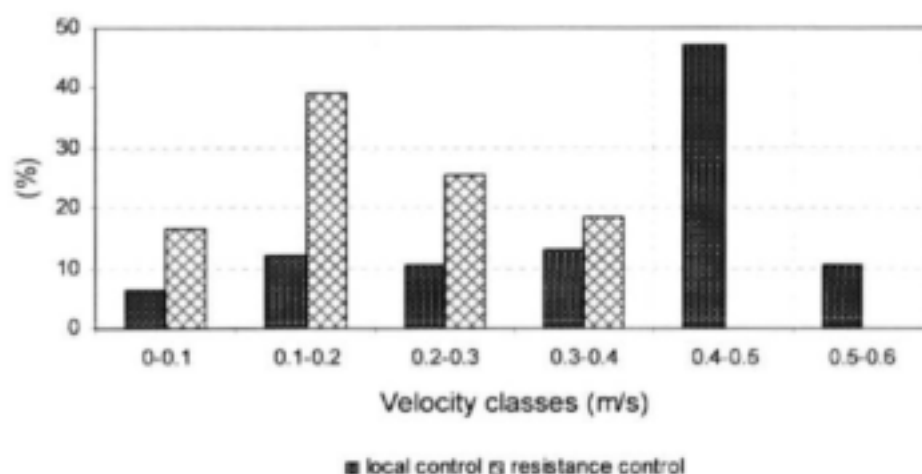


Figure 6-2 Velocity distribution histograms for local control (Tests 3) and resistance control (Test 7) (Flume C experiments)

A public domain 2D model, River2D has been tested for suitability in predicting velocity distributions for environmental flow analysis purposes. In the following sections the model is described briefly, tested for its ability to simulate locally controlled flow conditions, and tested for accuracy against some of the laboratory results presented in Chapter 3.

6.2 River2D Modelling

Implementation of River2D for eco-hydraulic modelling in South Africa is being investigated as part of Water Research Commission project K5/1508 (Hirschowitz et al., in preparation). That project will report on the model's applicability for different levels of Ecological Reserve determination, and present a set of guidelines for its use in South Africa.

River2D is a two-dimensional, depth average hydrodynamic and fish habitat model developed specifically for use in natural streams and rivers. It is a Finite Element model, based on the basic physical principles of conservation of mass and momentum, and on a set of constitutive laws which relate the driving and resisting forces to fluid properties and motions. It features subcritical-supercritical and wet-dry area solution capabilities.

River2D is one of a suite of four programmes which also includes R2D_Bed, R2D_Mesh and R2D_Ice. The general modelling procedure applies the following steps.

- Develop a bed topography using R2D_Bed (and R2D_Ice where applicable).
- Develop a mesh (a computational discretization) using R2D_Mesh
- Apply River2D to solve for water depths and velocities.

6.2.1 Trans-Critical Flow Simulation

The River2D suite has been used to simulate some hypothetical rapidly varied flow conditions to test its ability to describe the trans-critical flow conditions associated with multiple local controlled conditions. The simulations have been performed for different sizes of roughness elements (D from 0.10m to 0.40m). The number of obstacles was varied from 2 to 10. The predicted distributions of Froude number are shown in Figs 6-3 to 6-8.

The simulations presented in Figs 6-3 and 6-4 show that the spacing between two obstacles influences the flow pattern. Here, two hemispherical roughness elements are located on the same cross section in a 1.8m wide channel with their centres 0.36m and 1.6m apart, similar to the situations presented in the adjacent photographs. In both cases the situation is clearly locally controlled, and the simulations show realistic transitions between subcritical and supercritical flow. They also show expected differences in flow pattern associated with the different spacings.

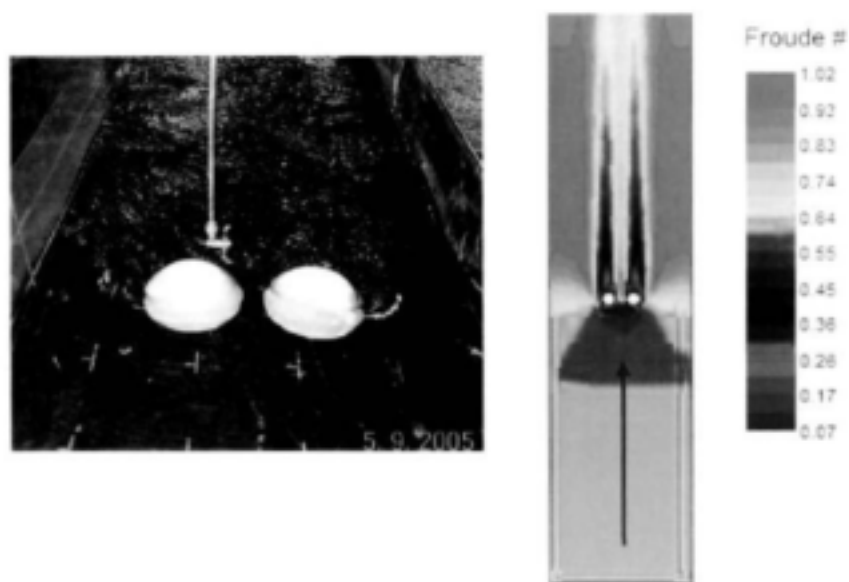


Figure 6-3 River2D modelling, $D=0.10$ m, centre to centre spacing = 0.36m

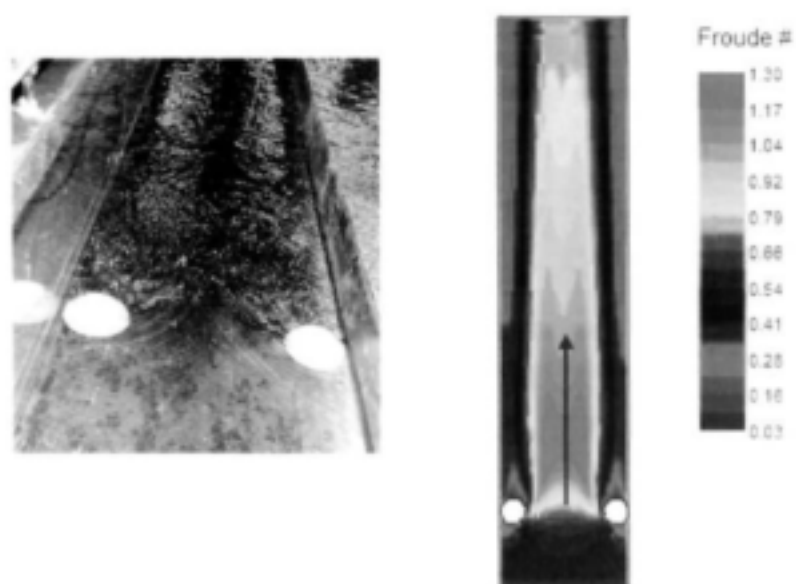


Figure 6-4 River2D modelling, $D=0.20$ m, centre to centre spacing = 1.6m

The effect of differences in size of roughness elements is shown in Fig 6-5, where simulations with different sizes as well as spacing and channel width are presented. The Froude number varies over a wider range in the right hand graph where the roughness elements size is bigger ($D = 0.35\text{m}$) compared with the left hand one ($D = 0.25\text{m}$). In Fig. 6-6 the combined influence of longitudinal spacing and size of obstacles is shown ($D = 0.35\text{m}$ in the left hand graph and $D = 0.40\text{m}$ in the right hand one). The influence of the number of obstacles and their arrangement on the flow pattern is shown in Figs 6-7 and 6-8.

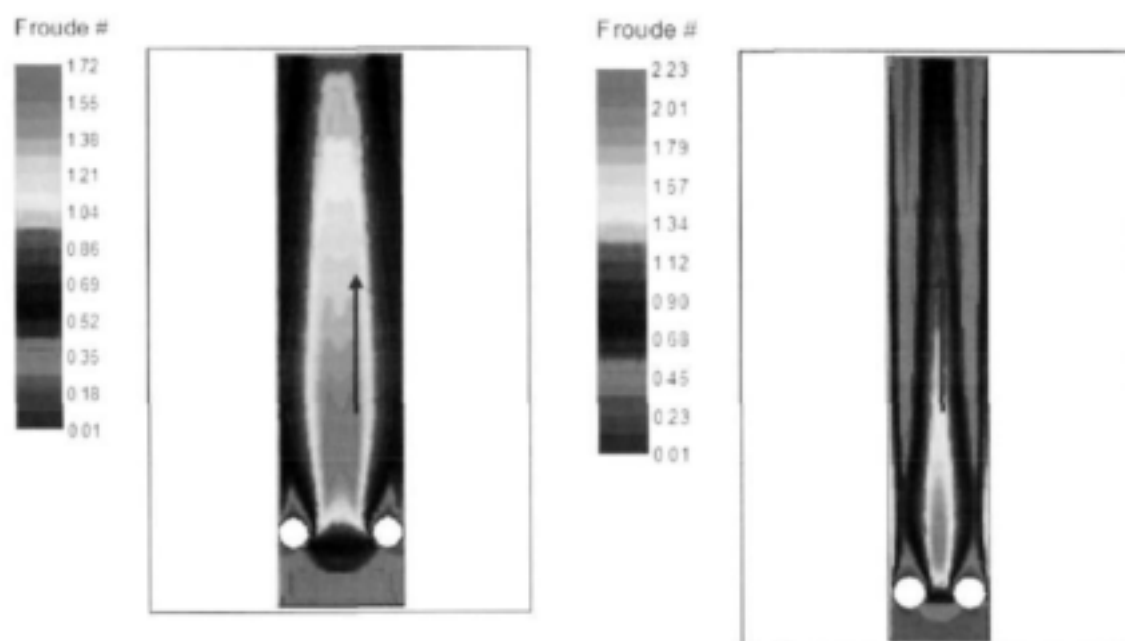


Figure 6-5 River2D modelling, (left) $D=0.25\text{m}$, centre to centre spacing = 1.5m ,
(right) $D=0.35\text{m}$, centre to centre spacing = 1.2m

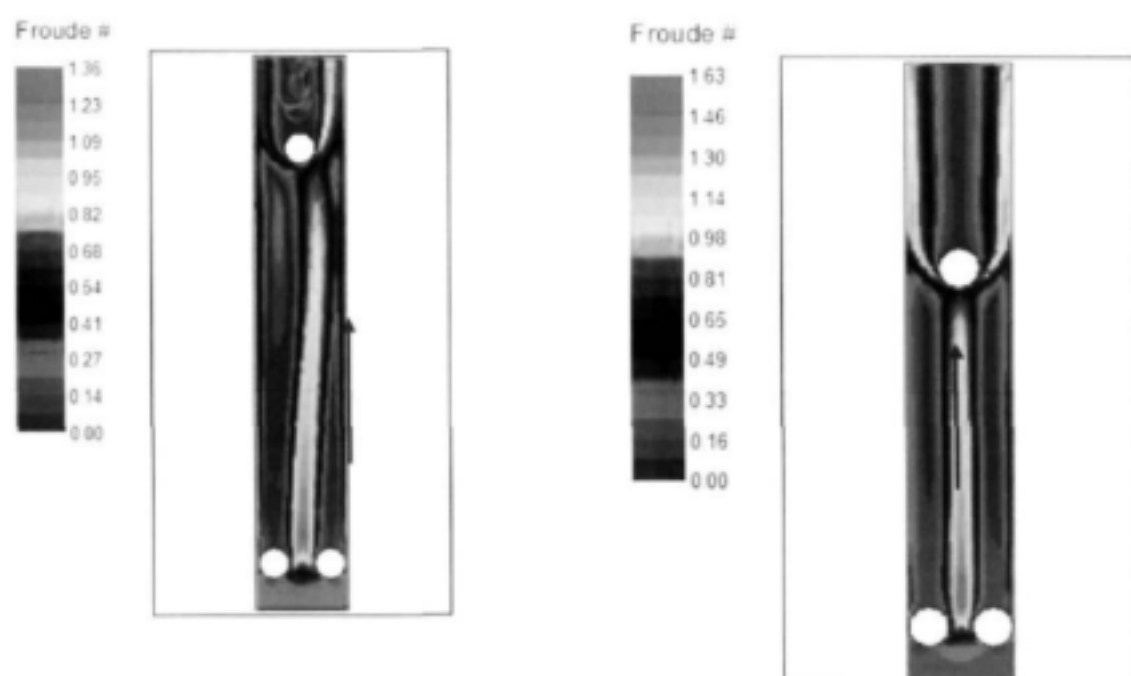


Figure 6-6 River2D, (left) $D=0.35\text{m}$, c/c transverse=1.20m, longitudinal=9m
(right) $D=0.40\text{m}$, c/c transverse=1.20m, longitudinal=7m

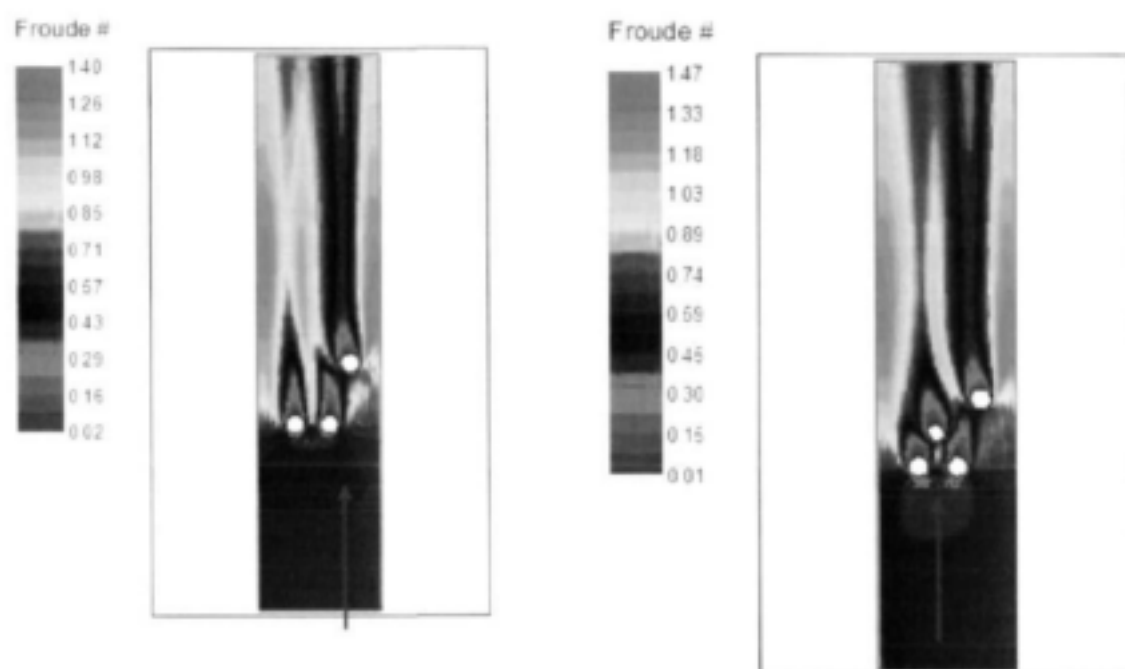


Figure 6-7 River2D modelling, $D=0.15\text{m}$ at the left and the right plots

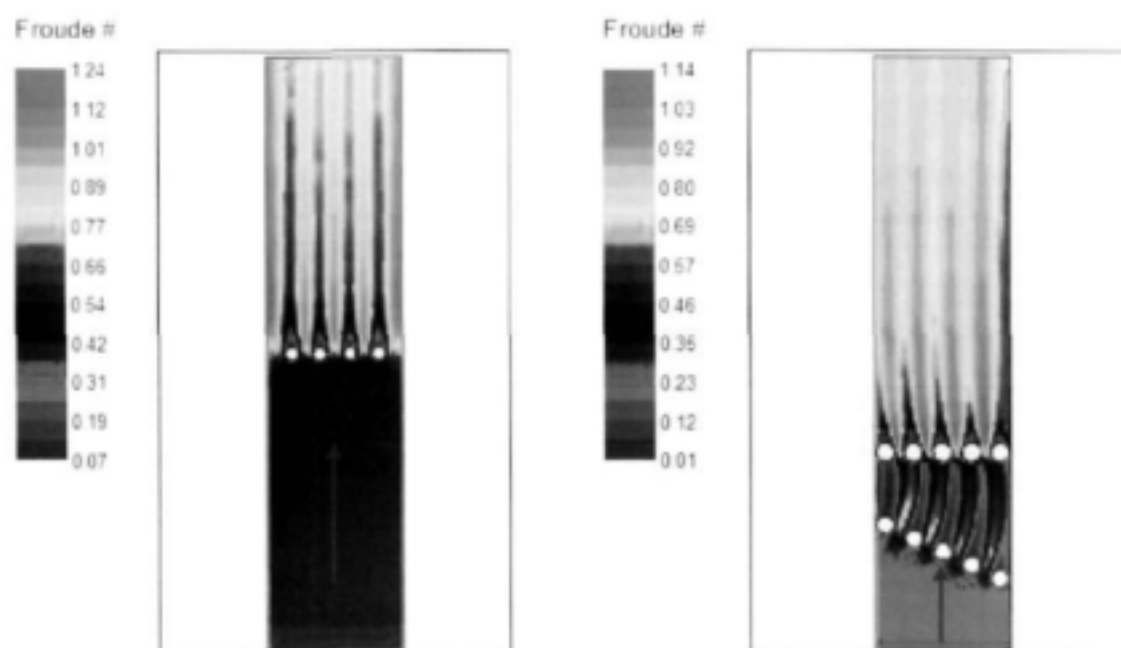


Figure 6-8 River2D modelling, $D=0.15\text{m}$

These hypothetical applications show that it is possible to describe a complex bed geometry comprising large roughness elements using River2D_Bed. The model is able to simulate realistically the complex flow patterns associated with locally controlled conditions, and particularly transitions between subcritical and supercritical flow.

6.2.2 Prediction of Velocity Frequency Distributions

The previous section established that River2D is able to simulate complex flow patterns associated with locally controlled flow. In this section its accuracy in predicting velocity distributions around large roughness elements under resistance controlled conditions is tested. The laboratory measurements described in Section 3.3 provide the test data. Here, the local velocities around one of the hemispheres in the experiment for Series 2.2, Pattern 6 (Fig. C-6, Appendix C) were measured. The channel was 12.00m long and 2.00m wide, set on a slope of 0.0005. Hemispheres with 0.072 m diameter were arranged in staggered pattern with 0.20m centre to centre spacings.

To model this situation, a mesh of 0.05m size was set up over the whole bed, and refined to 0.02m over each hemisphere for more accurate representation of the bed topography.

Modelling was performed for the two experimental discharges, $Q = 3.0\text{ l/s}$ and $Q = 21.0\text{ l/s}$, for which velocity distributions around the one hemisphere were measured. Details of the experiments related to velocity measurements are given in Chapter 3, sections 3.3.1 and 3.3.2 for $Q = 21.0\text{ l/s}$ and $Q = 3.0\text{ l/s}$ respectively.

The local velocities predicted by River2D were extracted in Excel format, and were analysed using the same velocity classes as the measured data.

Modelled (River2D) and measured velocity distributions in histogram format for $Q = 3.0\text{ l/s}$ with velocity classes of 0.01 m/s and 0.02 m/s , and for $Q = 21.0\text{ l/s}$ with velocity classes of 0.01 and 0.02 m/s are plotted in Figs 6-9, 6-10, 6-11 and 6-12 respectively.

The predicted velocity frequency distributions agree well with the measured ones with both specified velocity classes for $Q = 3.0\text{ l/s}$. The dominant velocity is slightly underestimated and the highest velocity classes are slightly over-represented, but the shape of the distribution is well reproduced. In contrast, the predicted frequency distributions for $Q = 21.0\text{ l/s}$ are rather poor. For the finer resolution histogram, the dominant velocity is in the wrong class, and in the coarser resolution histogram the dominant class is over-represented. The distributions are also skewed the wrong way, with the higher classes being emphasized rather than the lower ones.

The difference in performance for the two cases is a result of the different flow conditions and the appropriateness of the type of model for each of these. In 2D modelling a vertical distribution of velocity at each calculation point is assumed, which is realistic in some situations, but not others. The flow condition for $Q = 3.0\text{ l/s}$ is a large-scale roughness one. Under such conditions the vertical velocity profiles are not unduly distorted by the roughness elements, and the assumption made in 2D modelling is reasonable. (In fact, for this experiment only a single velocity – at mid flow depth – was measured, and was regarded as representative of the depth-averaged value). The flow condition for 21 l/s is one of intermediate-scale roughness. The vertical velocity profiles are highly variable from position to position, and a single assumed profile must be a very poor representation at many locations. This situation requires 3D modelling for accurate representation of local velocities. However, it must be acknowledged that for environmental flow analysis purposes, velocities and their distributions are usually required at coarser scales under these conditions. In other words, while the flow is highly nonuniform at the bed roughness scale, it is relatively uniform at the flow depth scale. Velocity variations of concern at this scale would be determined by channel characteristics at the cross-section scale, rather than the roughness element scale. The choice of 1D, 2D or 3D model is ultimately dictated by the question being asked and the scale of features inducing nonuniformity as well as the resources available to carry out the study.

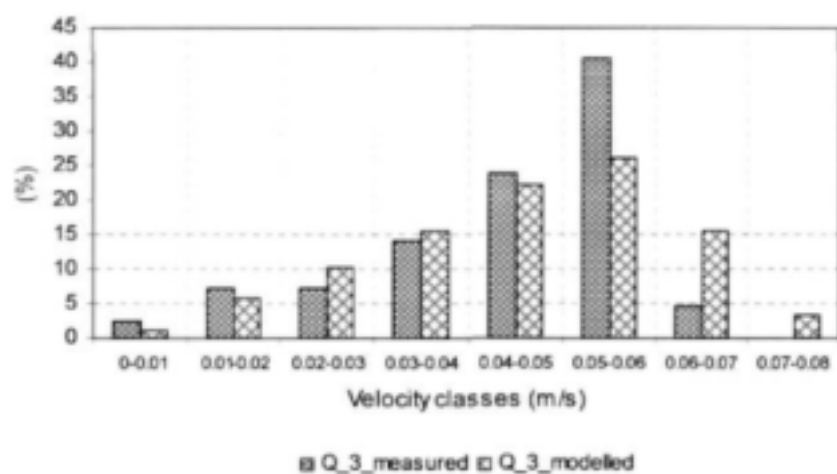


Figure 6-9 Measured and modelled velocity distributions with 0.01 m/s classes for $Q=3l/s$

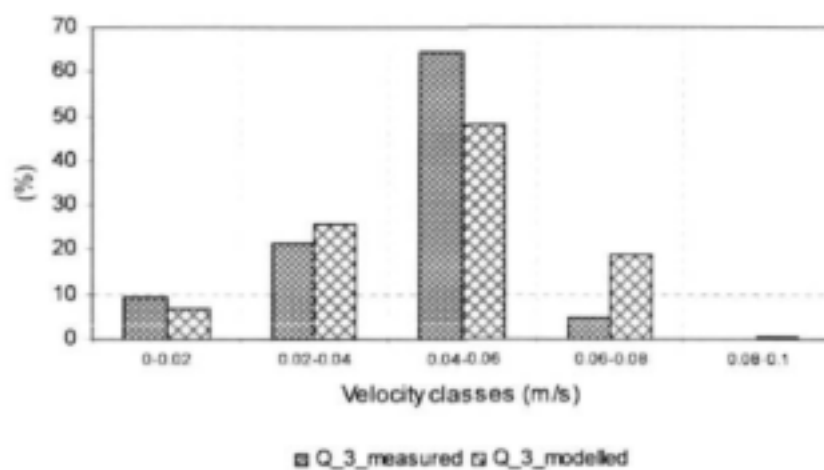


Figure 6-10 Measured and modelled velocity distributions with 0.02 m/s classes for $Q=3l/s$

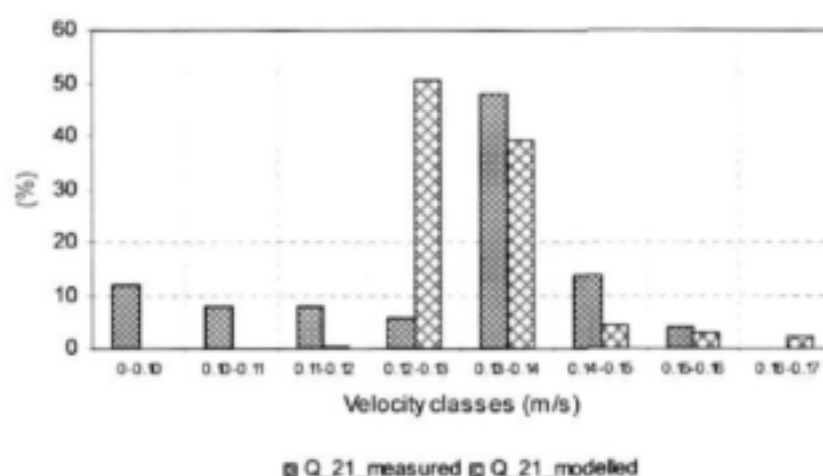


Figure 6-11 Measured and modelled velocity distributions with 0.01 m/s classes for Q=21l/s

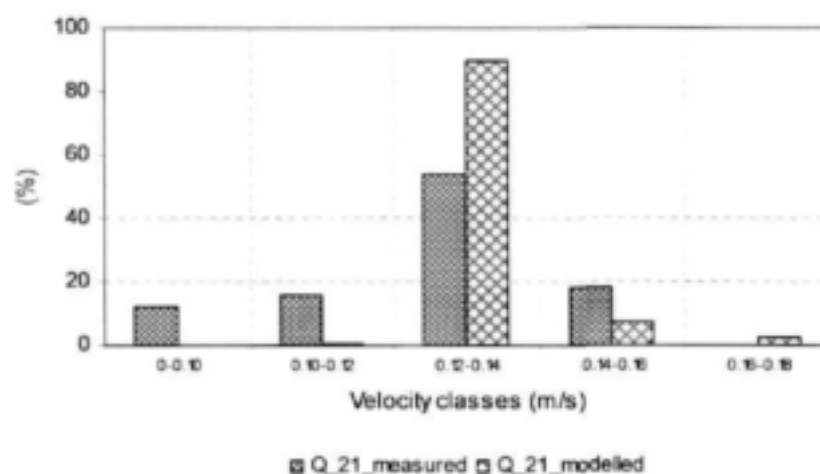


Figure 6-12 Measured and modelled velocity distributions with 0.02 m/s classes for Q=21l/s

6.3 Conclusions

Statistical models for describing velocity frequency distributions in rivers that are based on measured field data generally do not distinguish between multiple local control and resistance control flow conditions. They are therefore not generally applicable, and cannot account for velocity distribution changes associated with varying discharge.

Complex bed geometry can be modelled by R2D_Bed.

The River2D model can describe multiple local control conditions realistically, and trans-critical flows in particular.

River2D can predict velocity frequency distributions reliably under large-scale roughness conditions.

7 VEGETATION FLOW RESISTANCE

7.1 Estimation of Flow Resistance through Emergent Vegetation

7.1.1 Introduction

Reeds are a vegetation type widely present in South African rivers and wetlands, and the ability to quantify reed resistance is essential for engineering applications and environmental management of these systems. The effect of reeds on flow resistance is a complex phenomenon, and is determined by the morphology of the individual reed stems and characteristics of the whole reedbed. Plant characteristics such as stem diameter, stem height and the distribution of leaves and branches influence the drag imposed on the flow, and all vary seasonally. As part of a study of reedbed dynamics, associated hydraulics and sedimentation and their mutual interaction, James et al. (2001) developed a rigorous simulation model (REEDFLO) for flow through reeds that accounts for the fundamental processes influencing flow resistance.

REEDFLO provides a realistic description of the resistance phenomenon and can account for the influential parameters with a high level of detail. It is, however, too complex and cumbersome for practical application, and provides a level of resolution beyond what is usually necessary. The computational model of vegetation-influenced flow has been applied hypothetically to identify the variables most significant in determining resistance to flow through emergent reed-type vegetation, and to develop a simple procedure for estimating conveyance. A rational form of stage-discharge relationship is adopted and a formulation for the resistance coefficient in terms of the significant variables (stem spacing, stem diameter and stem drag coefficient) is derived through application of the computational model. Experimental results (James et al., 2001) are used to relate the stem drag coefficient to stem Reynolds number and foliage state. The stage-discharge and resistance coefficient equations and the drag coefficient relationship are applied through a simple 6-step iterative procedure for routine conveyance estimation (Jordanova et al., 2006). The procedure is verified by comparison of predicted and measured discharges for flow through natural vegetation.

7.1.2 The REEDFLO Model (James et al., 2001)

REEDFLO is based on force balance principles. The classic drag force equation is used to describe stem drag, with local approach velocity determined through the defect model of Petryk (1969). Both submerged and emergent conditions can be accounted for, using eddy-viscosity and mixing length functions to describe flow through and above the stem zone. The model includes some empirical content, requiring calibration with experimental data. For a specified discharge, REEDFLO predicts the flow depth, the vertical distribution of velocity, the bed shear stress, the total stem drag, the effective drag coefficient based on average velocity, and the

effective channel resistance in term of Manning's n or Darcy-Weisbach friction factor f . The development for emergent conditions is described below.

The model uses numerical techniques to obtain solutions to the finite-difference equations describing the balance of applied and resisting forces acting on flow elements (Li and Shen, 1973; Petryk and Bosmajian, 1975; Christensen, 1976; Lindner, 1982; Kosorin, 1983)

$$F_A = F_B + F_V \quad 7.1$$

where F_A : the applied force per unit plan area,
 F_B : the resistance force per unit plan area contributed by the bed, and
 F_V : the resistance force per unit plan area contributed by the vegetation.

The applied force per unit plan area is given by

$$F_A = \gamma y S_f (1 - n_x n_y \frac{\pi D^2}{4}) \quad 7.2$$

where γ : the unit weight of water,
 y : the flow depth,
 S_f : the energy slope,
 n_x : the numbers of stems in the longitudinal direction
 n_y : the numbers of stems in the lateral direction, and
 D : the stem diameter.

The resistance force contributed by the bed per unit plan area is related to bed shear stress by

$$F_B = \tau_b (1 - n_x n_y \frac{\pi D^2}{4}) \quad 7.3$$

in which τ_b is the bed shear stress.

The resistance force contributed by the vegetation per unit plan area is determined by the empirical drag force relationship for vertical cylinders

$$F_V = \frac{l}{2} C_{De} \rho y D n_x n_y u_a^2 \quad 7.4$$

- where u_a : the asymptotic approach velocity (the velocity attained within a large stand of stems),
 ρ : the water density, and
 C_{De} : the effective drag coefficient that accounts for the influence of adjacent obstructions and surface wave effects in a multi-stem arrangement, as proposed by Richter (1973), i.e.

$$C_{De} = \left(1 + 1.9C_D \left(\frac{D}{a_y} \right) \right) C_D \quad 7.5$$

- where a_y : the distance between reed stems in the lateral direction, and
 C_D : the drag coefficient for isolated cylindrical elements, which is a function of the cylinder Reynolds number.

7.1.3 Development of a Practical Resistance Equation (James et al., 2001)

REEDFLO is too computationally intensive for practical direct application and predicts flow characteristics through emergent reeds at a higher level of resolution than is generally necessary or justified, considering the uncertainties associated with vegetation characteristics and drag coefficient values. The model has therefore been applied hypothetically to develop a simple resistance relationship for flow through emergent vegetation stems, using the following steps (described in the sections that follow) (James et al., 2001):

1. Identification of factors that determine flow resistance;
2. Determination of the form of the flow resistance equation;
3. Estimation of drag coefficient values;
4. Development of a resistance coefficient relationship.

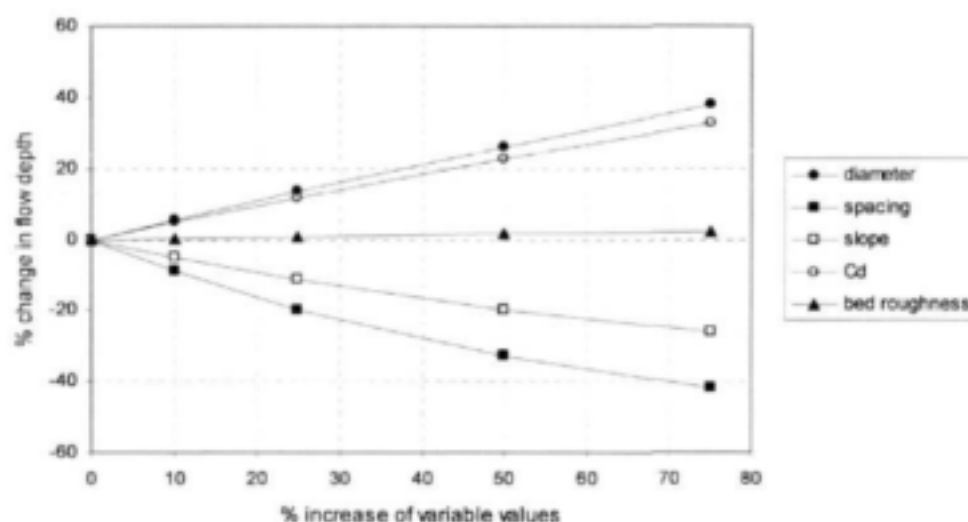
7.1.3.1 Identification of factors that determine flow resistance

The development of a general resistance equation for different conditions requires identification of the factors that significantly influence flow resistance. The impact of vegetation depends on many complex interacting factors, including parameters related to vegetation - stems diameter, spacing and drag coefficient, and channel properties represented by slope and substrate. Flow conditions were therefore simulated for a wide range of input variable values, which were varied systematically to explore their effects on flow resistance, and to enable identification of the most important ones. The variables investigated and the ranges of their variations are listed in Table 7-1. Simulations were performed for discharges of 0.01 and 0.05 m³/s/m.

Table 7-1 Variables and their range used in REEDFLO simulations

Variable	Range	Variables value				
		Base	+10 % -10 %	+25 % -25 %	+50 % -50 %	+75 % -75 %
Stem diameter, D (mm)	5.0-20.0	5.0	5.5 4.5	6.3 3.8	7.5 2.5	8.8 1.3
Stem spacing, s (m)	0.05-0.45	0.05	0.0550 0.0450	0.0625 0.0375	0.0750 0.0250	0.0875 0.0125
Bed slope, S	0.0005-0.005	0.001	0.0011 0.0009	0.0013 0.0008	0.0015 0.0005	0.0018 0.0003
Bed roughness, k_s (m)	0.0125-0.003	0.0125	0.0138 0.1130	0.0156 0.0094	0.0188 0.0063	0.0219 0.0031
Drag coefficient, C_D	1.0-2.2	1.15	1.2650 1.0350	1.4375 0.8625	1.7250 0.5750	2.0125 0.2875

In the first series of simulations, the base values of the variables were increased by up to 75 %. The responses of flow depth for a discharge of $q=0.05 \text{ m}^3/\text{s/m}$ (Fig. 7-1) show that changes in different variables affect flow resistance (and hence flow depth) differently. Change in stem diameter (D) and spacing (s) has significant influence on flow resistance, change in slope (S) and drag coefficient (C_D) has moderate influence, while the influence of bed roughness (k_s) change is small.

Figure 7-1 Effect of increasing variable values on change of flow depth for $q=0.05 \text{ m}^3/\text{s/m}$

In the second series of investigations, the values of the variables were decreased by up to 75 % (Table 7-1). The responses of flow depth for a discharge of $q=0.05 \text{ m}^3/\text{s/m}$ are shown in Fig. 7-2. The curves for stem spacing (Fig. 7-2) and slope show significant influence: flow depth increases by 285 % for a decrease in stem spacing of 75 % and by 100% for a decrease in bed slope of 75 %. As in Fig. 1 the effect of bed

roughness change on flow depth is very small. Stem spacing and diameter are therefore important in determining flow resistance, and further investigation of their influence was carried out. Figure 3 shows the influence on flow depth of three realistic diameter values ($D=5.0$ mm, $D=10.0$ mm and $D=20.0$ mm) at a slope of 0.0005, a discharge of $0.05 \text{ m}^3/\text{s/m}$, and stem spacings in the range 0.05 to 0.18 m. The influence of diameter on flow depth can be seen to decrease slightly with increased stem spacing, suggesting that the influence can not be accounted for in terms of relative spacing (s/D) only, but also depends on absolute values of stem spacing and stem diameter. For example (Fig. 7-3), for a spacing of 0.1 m and diameter of 20.0 mm (relative spacing of 5) the flow depth is about 1.3 m, while for the same relative spacing with a stem spacing and stem diameter of 0.05m and 10.0 mm, respectively, the depth is 1.75 m.

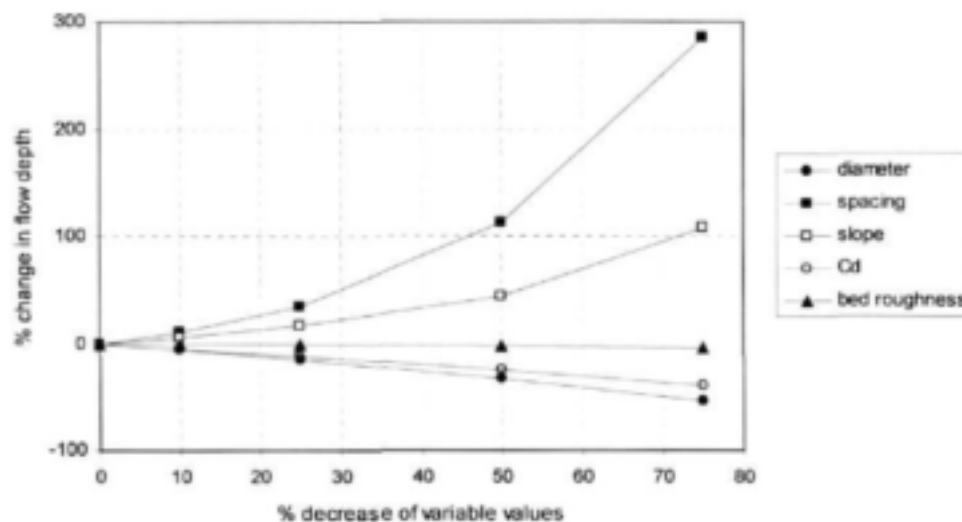


Figure 7-2 Effect of decreasing values on change in flow depth for $q=0.05 \text{ m}^3/\text{s/m}$

The results of simulations performed enable us to investigate how different parameters affect the flow resistance. It has been found that bed roughness, as was expected, has a small or even negligible effect on flow resistance. Vegetation parameters - stem diameter, stem spacing and drag coefficient have a significant effect on flow resistance. This is understandable, because resistance in reedbeds is dominated by stem drag, which depends strongly on the morphology and density of stems (James et al., 2001). Bed slope also strongly influences flow resistance indicating that it would be a parameter in defining the retardance to flow (Kouwen and Unny, 1973; Wu et al., 1999).

On the basis of this analysis, the vegetation parameters of stem diameter, stem spacing and drag coefficient, and the bed slope have been identified as the most important parameters determining flow resistance.

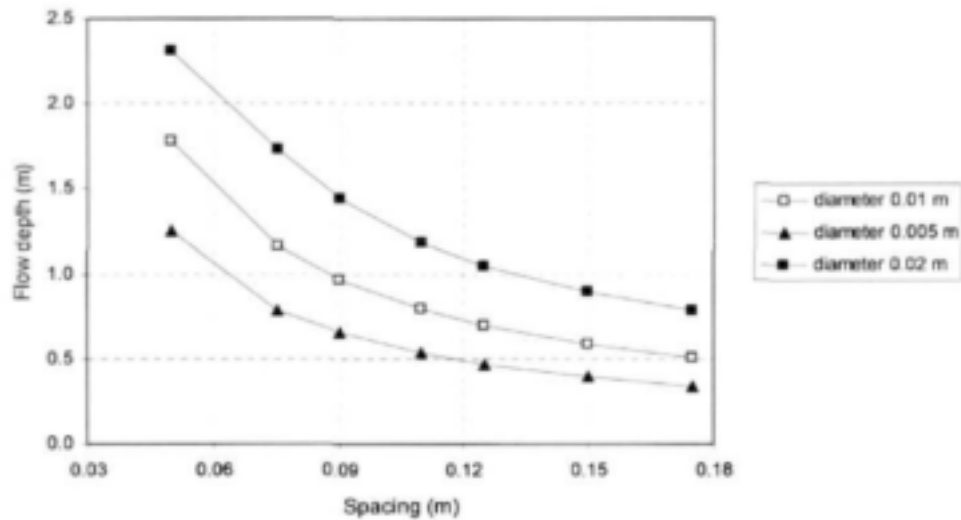


Figure 7-3 Effect of stem spacing on flow depth for stem diameters of 0.005, 0.01 and 0.02 m for slope of 0.0005 and $q=0.05 \text{ m}^3/\text{s/m}$

7.1.3.2 Determination of form of the flow resistance equation

Although widely used, Manning's equation is poorly suited to vegetated channels (James et al., 2001; Turner et al., 1978; Turner and Chanmeesri, 1984; James et al., 2004) and a more general form of resistance equation, such as proposed by Turner and Chanmeesri (1984) and Smith et al. (1990) is considered here. The discharge-depth equation offered by Turner and Chanmeesri (1984) is given by

$$q = G^{-1} y^m S^{0.5} \quad 7.6$$

where q : the discharge per unit width,
 G : a coefficient of roughness which is independent of slope, and
 S : the channel slope.

The roughness coefficient (G) and exponent (m) need to be determined experimentally.

Smith et al. (1990) recommended a similar equation,

$$q = a S_f^b y^c \quad 7.7$$

in which a , b and c are experimentally fitted parameters. However, the suggested parameters were determined from experiments for a particular crop type, geometry and flow conditions, and this limits their application.

James et al. (2001, 2004) showed that from the balance of forces driving and resisting the water movement, where the resistance to flow is caused exclusively by stem drag rather than boundary shear, the velocity is independent of flow depth. The proposed equation is

$$V = \frac{1}{F} \sqrt{S} \quad 7.8$$

in which F is a resistance coefficient dependent on stem diameter (D), stem density (N) and drag coefficient (C_D) given by

$$\frac{1}{F} = \sqrt{\frac{2g}{C_D N D} \left(1 - \frac{N \pi D^2}{4} \right)} \quad 7.9$$

Equation (7.8) suggests that the exponent of S in equation (7.7) should be 0.5, and flow depth is for continuity reasons only, and its exponent should be 1.0. The discharge-depth equation is accordingly given by

$$q = yV \quad 7.10$$

Equation (7.9) was developed theoretically for cylindrical rods and simple hydraulic conditions without accounting for the influence of adjacent obstructions and surface wave effects in a multi-stem arrangement.

The computationally intensive REEDFLO model determines the velocity at a longitudinal position within a multi-stem arrangement by deducting from the velocity in the absence of vegetation, V_0 , the sum of the velocity defects arising from all upstream stems and adjacent obstructions. The velocity per unit flow width in a multi-stem arrangement is given by (James et al., 2001; Li and Shen, 1973)

$$V = V_0 - \frac{1}{W} \int_{-s_w/2}^{s_w/2} u_d dz \quad 7.11$$

where V : the average velocity,
 W : the flow width,
 s_w : the spread of the wake,

- u_d : the flow velocity defect, and
 z : the lateral distance relative to the obstruction.

Application of REEDFLO allows development of an empirical resistance coefficient (F) that accounts for surface wave and blockage effects within a large reed stand. The most suitable way of expressing the functional relationship in terms of dimensionless parameters was found to be

$$F = f\left(\frac{s}{D}, \frac{D}{y}, C_D\right) \quad 7.12$$

7.1.3.3 Estimation of drag coefficient values

The range of drag coefficient (C_D) values for reeds has been determined from results of laboratory experiments with real reed and morphologically similar bulrush stems (James et al., 2001). Two reed stems (reed 1 and 2 in Table 7-2) and a bulrush stem were harvested from stands of *Phragmites australis* and *Typha capensis* in the Braamfontein Spruit, in Johannesburg. The characteristics of the freshly cut stems are listed in Table 7-2.

Table 7-2 Drag coefficient values for reed and bulrush stems

Stem Type	Foliage	Foliage Area, (m ²)	Stem diameter, (mm)	Re	C_D
Reed 1	Full foliage	0.0292	10.8	638-4838	1.35-3.46
Reed 2	Full foliage	0.0340	8.40	457-4347	1.75-6.79
Reed 2	6 leaves	0.0318	8.40	246-4347	1.55-16.2
Reed 2	3 leaves	0.0158	8.40	246-4731	1.27-16.2
Reed 2	Stem only	0	8.40	255-4686	1.25-4.34
Bulrush	Full foliage	0.0339	11.57	501-5981	2.22-5.34

Drag force experiments were performed in a 24 m long, 0.915 m wide, horizontal flume. Vegetation stems were held normal to the flow in a rectangular aluminium frame mounted on a pivot that allowed the applied force to be measured by maintaining moment equilibrium. Flow velocities were measured at the stem level with the stem removed.

The drag force (F_D) on a stem is related to flow and stem characteristics by

$$F_D = C_D A_{pr} \frac{1}{2} \rho V^2 \quad 7.13$$

- where A_{pr} : the stem area projected in the flow direction, and
 V : the local flow velocity.

The drag coefficient (C_D) depends on the stem size and shape and the Reynolds number (Re) expressed in terms of stem diameter (D)

$$Re = \frac{VD}{\nu} \quad 7.14$$

Experimentally determined values of C_D with corresponding Reynolds numbers for all three stems are plotted together with the standard curve presented by Albertson et al. (1960) in Fig. 7-4. The stem of reed 2 was progressively stripped of leaves and branches to determine the relative contributions of the main stem and the foliage to drag. It was tested first with all its leaves and branches, then with 6 leaves, 3 leaves, and finally with only the bare stem. The foliage areas of the stems were measured by tracing the outlines on to squared paper (Table 7-2). Values of C_D are plotted against Reynolds number (Re) in Fig. 7-5 for the reed stem with no leaves, and in Fig. 7-6 for the reed with 6 and 3 leaves. All the experimentally determined C_D values are plotted together in Fig. 7-7, which includes curves for the upper and lower limits and a best-fit curve through all the values.

The relationship between drag coefficient, C_D , and stem Reynolds number (Re) can be represented by

$$C_D = \alpha Re^\beta \quad 7.15$$

Best-fit values of coefficients α and β for all the experimental conditions are listed in Table 7-3.

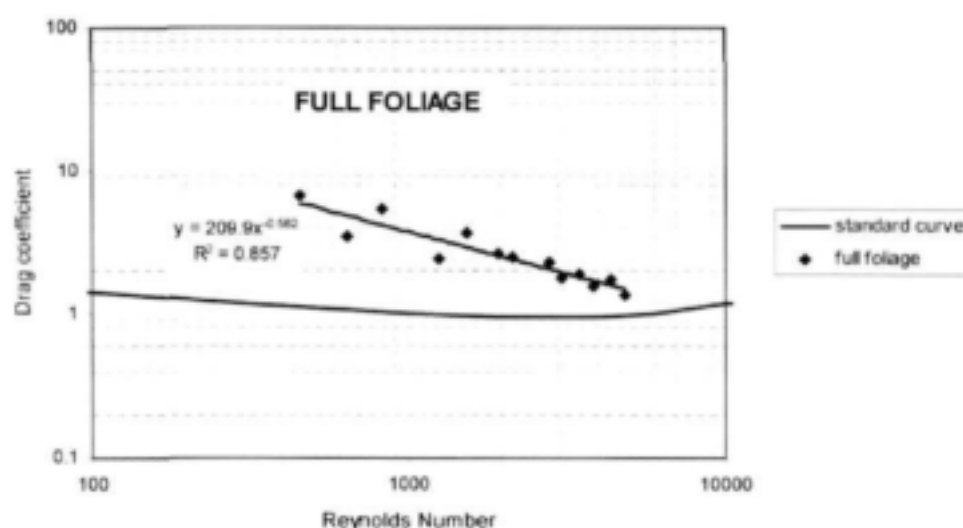


Figure 7-4 Drag coefficient for natural reed and bulrush stems

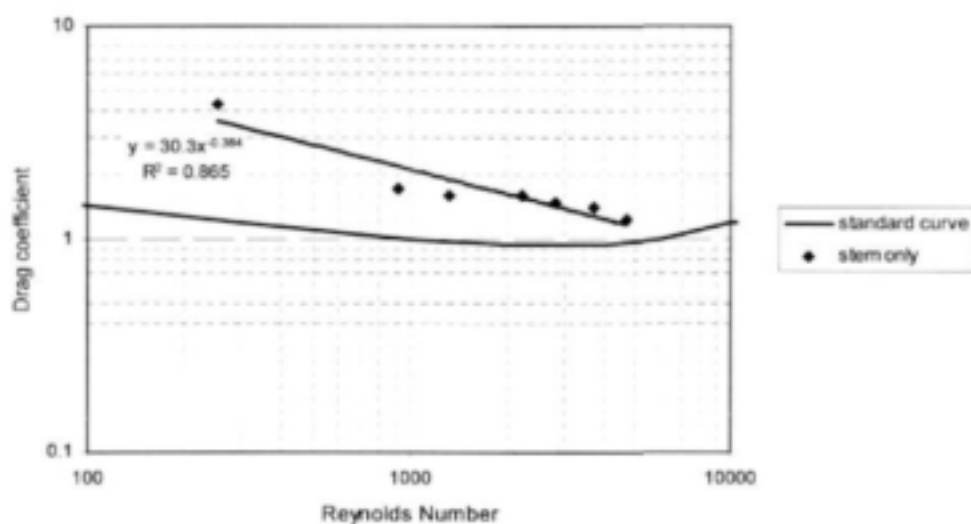


Figure 7-5 Drag coefficient for reed stem with no leaves

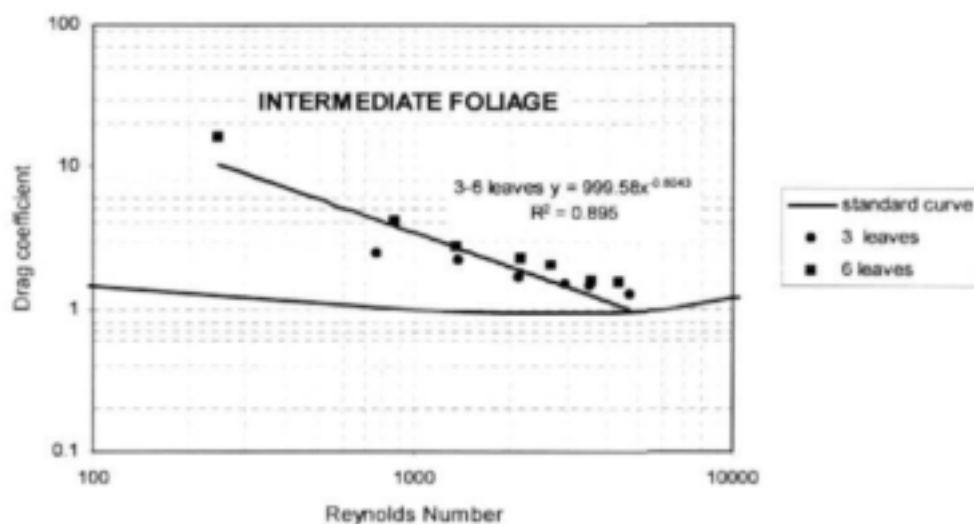


Figure 7-6 Variation of drag coefficient with varying degree of leaf foliage

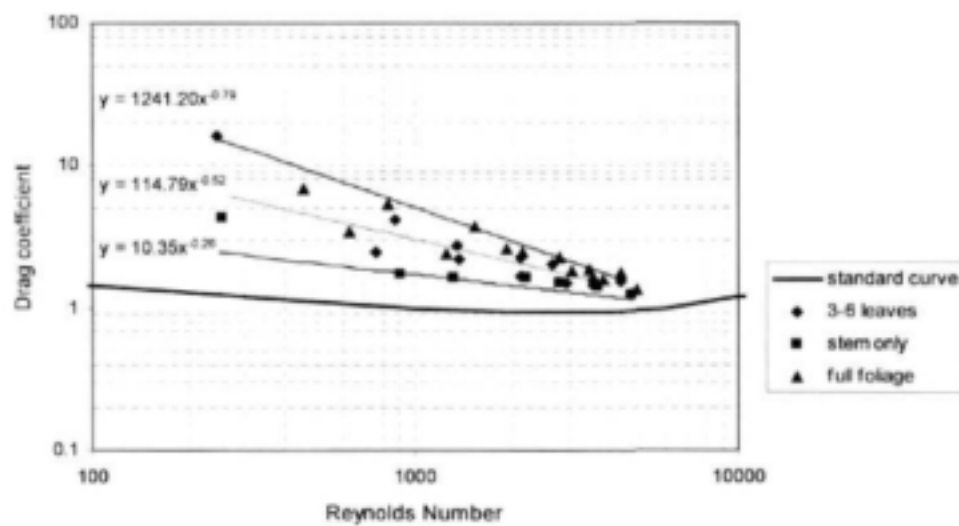


Figure 7-7 Variation of drag coefficient with degree of foliage

Table 7-3 Values of α and β coefficients for estimation of the drag coefficient as a function of the stem Reynolds number

Description	α	β
Stem only	30.3	-0.38
3 – 6 leaves	999.58	-0.80
Full Foliage	209.9	-0.58
Upper limit	1241.2	-0.79
Lower limit	10.35	-0.28
Average	114.79	-0.62

7.1.3.4 Development of resistance relationship

The functional relationship for F (Eq. 7.12) has been quantified by multiple regression analysis of the results of 270 REEDFLO simulations for a wide range of all the input variables. Realistic ranges of values of stem spacing, stem diameter and bed slope were selected according to field data from the Sabie, Letaba and Sand Rivers within the Kruger National Park, South Africa (Kotschy, 2003; Carter, 1995; van Coller et al., 1997). This established appropriate ranges of relevant parameter values for practical application of the proposed equation. The ranges of input variables used in the simulation runs are given in Table 7-4.

Table 7-4 Range of variables for which the resistance equation (7.16) is applicable

Variable	Range
Discharge, q ($\text{m}^3\text{s}^{-1}\text{m}^{-1}$)	0.005-0.5
Bed slope, S	0.0005-0.002
Stem diameter, D (mm)	5.0-20.0
Stem spacing, s (m)	0.05-0.1

The following relationship has been derived for the conditions listed in Table 7-4.

$$F = 1.885 \left(\frac{s}{D} \right)^{-0.65} \left(\frac{D}{y} \right)^{0.07} C_D^{0.48} \quad 7.16$$

with r^2 equal to 0.95. The predictions of equation (7.16) are compared with values derived from the REEDFLO simulations in Fig. 7-8. Equation (7.6) reproduces the REEDFLO values with an absolute error of 11.5 %, a standard deviation of 8.4 % and a maximum error of 33.3 %.

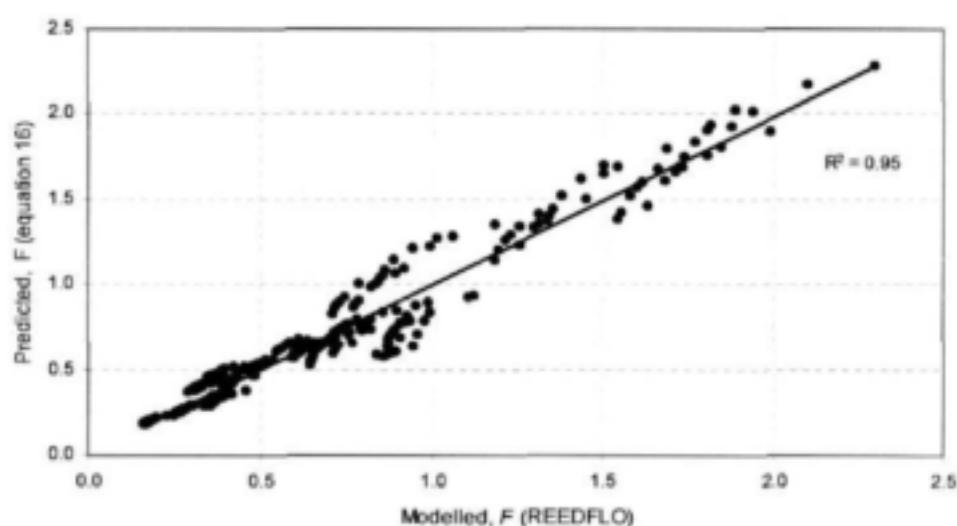


Figure 7-8 Predicted (equation (7.16)) and modelled (REEDFLO) resistance coefficient F (James et al., 2001)

7.1.4 Proposed Procedure for Practical Application (Jordanova et al., 2006)

A simple procedure using equations (7.8), (7.10), (7.14), (7.15) and (7.16) is proposed for practical stage-discharge determination for waterways with emergent vegetation.

Application of the procedure consists of 6 steps:

1. From field data, obtain representative values for stem diameter (D), stem spacing (s), bed slope (S) and the average height of stems from the bed to the first leaf.
2. For a specified flow depth assume a flow velocity (V) and calculate the Reynolds number in terms of the stem diameter (D) (equation (7.14)).
3. Depending on the flow depth of interest in relation to the height of the first leaf, estimate the drag coefficient (C_D) from Fig. 7-7 or by using Eq. 7.15 and Table 7-3, according to the degree of foliage.
4. From equation (7.16) calculate the resistance coefficient, F , and from equation (7.8) calculate the velocity, V .
5. Compare the calculated velocity with the initially assumed one, and if there is a difference, repeat step 2, 3 and 4 until the assumed and calculated velocities are the same.
6. From equation (7.10) calculate the discharge (q) for the nominated flow depth.

7.1.5 Procedure Verification

Assessment of the proposed procedure (Section 4) for practical application requires field or experimental data on flow resistance with real reeds or similar plants. Published data on flow resistance with real vegetation is very limited, and often does not include information about hydraulic conditions. Suitable experimental data for flow through crops of wheat, reeds and bulrushes are, however, available from the work of Turner and Chanmeesri (1984), Hall and Freeman (1994), Meijer and van Velzen (1999) and James et al. (2004). For each data set the discharge was calculated for each experimental flow depth and compared with the measured value.

Turner and Chanmeesri (1984)

Laboratory experiments with crops of wheat were carried out in a long concrete channel on a fixed slope, and in a smaller flume with a variable slope. The tests were conducted for different sowing patterns, stem densities, stages of growth, and slopes. The experimental data used to verify the proposed procedure (section 4) were extracted from their observations, and are presented in Table 7-5.

Table 7-5 Data from experiments of Turner and Chanmeesri (1984)

Test	Discharge, Q (m^3/s)	Flow depth, y (m)	Stem diameter, D (mm)	Spacing, s (m)	Slope, S
B	0.0007	0.0161	2.72	0.03	0.002
	0.0011	0.0247	2.72	0.03	0.002
	0.0015	0.0328	2.72	0.03	0.002
	0.0020	0.0445	2.72	0.03	0.002
	0.0025	0.0555	2.72	0.03	0.002
	0.0031	0.0665	2.72	0.03	0.002
	0.0036	0.0750	2.72	0.03	0.002
	0.0040	0.0860	2.72	0.03	0.002
H	0.0010	0.0140	2.89	0.05	0.0028
	0.0015	0.0220	2.89	0.05	0.0028
	0.0018	0.0260	2.89	0.05	0.0028
	0.0026	0.0360	2.89	0.05	0.0028
	0.0032	0.0415	2.89	0.05	0.0028
	0.0036	0.0495	2.89	0.05	0.0028
	0.0040	0.0535	2.89	0.05	0.0028
	0.0046	0.0585	2.89	0.05	0.0028
	0.0049	0.0670	2.89	0.05	0.0028

Hall and Freeman (1994)

Hall and Freeman (1994) carried out laboratory experiments on flow through bulrushes (*Scirpus validus*) at different growth stages. A weir installed downstream of the channel was used to control backwater. The first sets of experiments were conducted for "low" and "high" tailwater conditions, the stem diameter was 7.0 mm and the density of the stems was 403 per square metre. The second set of experiments was run for "high" tailwater conditions only, when the stem diameter was 7.6 mm and the stem density was 807 per square metre. The experimental stage-discharge data used for verification are listed in Table 7-6.

Table 7-6 Data from experiments of Hall and Freeman (1994)

Test	Discharge, Q (m^3/s)	Flow depth, y (m)	Stem diameter, D (mm)	Spacing, s (m)	Slope, S
June 1992 Tests (low tailwater)	0.009	0.103	7.0	0.05	0.0088
	0.026	0.215	7.0	0.05	0.0105
	0.044	0.268	7.0	0.05	0.0145
	0.057	0.306	7.0	0.05	0.0145
June 1992 Tests (high tailwater)	0.009	0.313	7.0	0.05	0.0010
	0.026	0.339	7.0	0.05	0.0035
	0.044	0.403	7.0	0.05	0.0040
	0.057	0.432	7.0	0.05	0.0050
November 1992 Tests	0.010	0.347	7.6	0.035	0.0028
	0.026	0.374	7.6	0.035	0.0085
	0.044	0.417	7.6	0.035	0.0120
	0.064	0.448	7.6	0.035	0.0198

Meijer and van Velzen (1999)

In addition to their studies with artificial vegetation represented by steel rods, Meijer and van Velzen (1999) also carried out some tests with natural reeds in a 3.0 m wide, 100 m long laboratory flume. The data for these were used for verification and are given in Table 7-7.

Table 7-7 Data from experiments of Meijer and van Velzen (1999)

Test	Discharge, Q (m^3/s)	Flow depth, y (m)	Stem diameter, D (mm)	Spacing, s (m)	Slope, S
Emergent reeds	0.36	1.22	5.7	0.063	0.0013
	0.53	1.27	5.7	0.063	0.0026

James et al. (2004)

The hydraulic and resistance characteristics of flow through reeded channels were investigated using harvested reed (*Phragmites australis*) in a 0.38 m wide, 0.66 m high and 10.4 m long flume. Two test series were undertaken to differentiate between the leaf and stem resistance of reeds, the first with foliated and the second with defoliated reeds set vertically in a regular rhomboidal pattern. Experimental data are listed in Table 7-8.

Table 7-8 Data from experiments of James et al. (2004)

Test	Discharge, Q (m ³ /s)	Flow depth, y (m)	Stem diameter, D (mm)	Spacing, s (m)	Slope, S
Full foliage	0.0014	0.047	9.5	0.071	0.0015
	0.0029	0.134	9.5	0.071	0.0015
	0.0049	0.331	9.5	0.071	0.0015
	0.0064	0.482	9.5	0.071	0.0015
Stem only	0.0030	0.078	9.5	0.071	0.0015
	0.0067	0.165	9.5	0.071	0.0015
	0.0096	0.239	9.5	0.071	0.0015
	0.0152	0.353	9.5	0.071	0.0015

The proposed procedure (Section 4) was applied to the four sets of experimental data. A summary of comparisons between predicted and measured discharges in terms of maximum, minimum and average absolute errors, and their standard deviations is given in Table 7-9. The average absolute error in the predicted discharge for all 39 tests is 12.9%. An application of the proposed procedure was performed on the same sets of experimental data using the resistance coefficient (equation (7.9)), and the average absolute error in the predicted discharge is 18.1%. The difference in performance of the proposed procedure with the resistance coefficients (equations (7.9) and (7.16)) in terms of the average absolute error shows equation (7.16) to be superior. The performance of equation (7.9) deteriorates significantly as the amount of foliage increases.

Available data required for verification of prediction of the proposed procedure is limited, and for the data used, not all information on hydraulic and vegetation conditions was available. At this stage it is therefore difficult to evaluate why the method performed better or worse for different conditions. However, considering the limited data on which the method is based, particularly for C_D , the proposed procedure performs reasonably well. It provides a workable method that can be improved as new data become available.

Table 7-9 Summary of comparison between predicted and measured discharges

Source	Test	No. of tests	Discharge computation			
			Min absolute error (%)	Max absolute error (%)	Average absolute error (%)	Standard deviation (%)
Turner and Chanmeesri (1984)	B	8	3.7	12.6	7.4	2.5
	H	9	0.3	9.5	3.7	8.8
Hall and Freeman (1994)	Low tailwater	4	3.7	23.3	13.5	6.9
	High tailwater	4	0.9	30.5	17.2	12.6
	November 1992	4	9.2	20.3	13.2	4.3
Meijer and van Velzen (1999)	Emergent reeds	2	3.1	9.2	6.1	3.1
James et al. (2004)	Full foliage	4	1.5	17.0	11.6	6.1
	Stem only	4	28.2	31.7	30.5	1.4

7.1.6 Conclusions

A computational model of vegetation-influenced flow has been applied to identify the characteristics that most significantly influence resistance to flow through emergent reed-type vegetation, and to develop a simple, direct approach for resistance calculations.

The most important variables influencing emergent vegetation resistance are the vegetation stem diameter, the stem spacing, the stem drag coefficient and the channel slope. Bed roughness has little influence in drag dominated flows.

The proposed stage-discharge relationship (equation (7.10)) is rational in origin, but includes an empirical resistance coefficient. A formulation for this coefficient is presented (equation (7.16)) that accounts rationally for the underlying variables and processes (through the REEDFLO model), and reduces the empirical content to the stem drag coefficient only.

The stem drag coefficient depends on the stem Reynolds number and the stem foliage, as presented in Fig. 7-7, based on available real vegetation stem drag data.

Resistance to flow through emergent vegetation can be estimated through a simple iterative procedure (Section 4) employing equations (7.10) and (7.16) and the drag coefficient diagram (Fig. 7-7). Predictions show satisfactory agreement with measured stage-discharge data for emergent vegetation.

Field data, required for Step 1, should be collected with an understanding of the influence of each parameter on overall resistance. Reed stem attribute data have to be subjectively sampled to cover the range of densities, stem diameters and leaf distributions. Seasonal changes in stem characteristics need to be understood and taken into account where necessary for effective prediction of the interaction between reeds and flow.

The proposed procedure has been tested only against the limited available laboratory data, and has not yet been verified against field data. Nevertheless, it provides a simple conveyance estimation method for flow through emergent vegetation with a reliability consistent with current knowledge, and which can be refined as suitable field data become available.

7.2 Conveyance Estimation for Channels with Emergent Vegetation Boundaries

7.2.1 Introduction

Emergent vegetation is a common feature along river banks. It can influence flow resistance significantly even if it occupies only a relatively small part of the channel (James et al., 2001), and should be accounted for in estimation of the channel conveyance. Vegetation increases overall resistance by imposing greater local resistance where it occurs, and by retarding flow in adjacent clear water zones.

The well-structured distribution of roughness presented by vegetation occurring in strips along the channel banks suggests channel subdivision as a suitable strategy for practical conveyance calculation. Discharges can be calculated separately for the vegetated and clear zones of a cross section, and added together to obtain the total discharge. The effects of flow interaction between the zones can be quantified for this purpose through recognition of an apparent shear stress at the zonal interface, tending to resist flow in the clear channel and propel it within the vegetation.

The flow contribution of the vegetated zones for channels with bank vegetation is usually small, and may often be ignored. Where it is significant, the discharge may be estimated using the procedure presented in section 7.1, although this method will overpredict the discharge slightly because of the propelling influence of the adjacent clear channel flow.

The conveyance estimation method presented in this section for the clear channel zone between the bank vegetation zones has been proposed by James and Makoa (2006).

7.2.2 Clear channel zone discharge

A clear channel between vegetation strips can be considered as one of composite roughness, with different roughnesses on the sides and the bed. If resistance coefficients for the different surfaces are known, then an equivalent coefficient for the whole clear channel can be determined by well-known composite roughness formulae (Chow, 1959; French, 1985). Different assumptions regarding the flow characteristics in the subsections have led to different formulae for an equivalent value of Manning's n (n_e). For the following formulations the channel cross section is divided into N subsections, each with a wetted perimeter (P_i) (which does not include the interfaces with adjacent subsections), and with a known subsection value of the resistance coefficient, n_i .

Horton (1933) assumed that the velocities in the subsections are all equal to the average velocity for the whole cross section. This assumption leads to the following expression for the equivalent, overall, value of Manning's n :

$$n_e = \left(\frac{\sum_{i=1}^N (P_i n_i^{3/2})}{P} \right)^{2/3} \quad 7.17$$

where n_e : the equivalent value and
 P : the total wetted perimeter.

Pavlovski (1931) assumed that the total boundary shear force for the whole cross section is the sum of the subsection shear forces calculated separately. The equivalent value of Manning's n is then given by

$$n_e = \left(\frac{\sum_{i=1}^N (P_i n_i^2)}{P} \right)^{1/2} \quad 7.18$$

By assuming the total discharge for the whole cross section to be the sum of the subsection discharges, calculated separately, Lotter (1933) formulated the following expression for the equivalent resistance coefficient:

$$n_e = \frac{P R^{5/3}}{\sum_{i=1}^N \frac{P_i R_i^{5/3}}{n_i}} \quad 7.19$$

where R_i : the hydraulic radius of the i th subsection, and
 R : the total hydraulic radius.

Colebatch (1941) assumed local flow conditions to be associated with flow area rather than wetted perimeter, and proposed that the equivalent resistance coefficient is given by

$$n_e = \left(\frac{\sum_{i=1}^N (A_i n_i^{3/2})}{A} \right)^{2/3} \quad 7.20$$

where A_i : the area of the i th subsection, and
 A : the total area.

These equations have been tested for the situation of vegetation-type side boundaries using laboratory data obtained by James et al. (2001) and field data from the Okavango Delta obtained by McCarthy et al. (1991).

7.2.2.1 Laboratory experiments (James et al., 2001)

The influence of vegetation strips (including those wholly within the channel as well as along the banks) on channel conveyance was investigated experimentally by James et al. (2001). Experiments were carried out in a 12.26 m long, 1.0 m wide, rectangular channel lined with cement plaster and set on a slope of 0.00107. Water was supplied from an elevated constant head tank, and controlled by a valve upstream of the channel. Discharge was measured with two triangular weirs installed in parallel in a forebay between the control valve and the head of the channel. Uniform flow was ensured by adjustment of a hinged overflow gate downstream of the channel. Flow depths were measured along the channel centre line using a pointer gauge mounted on a travelling instrument bridge.

Vegetation stems were represented by 5 mm diameter steel rods set in a regular, staggered grid pattern with centre spacings of 25 mm in both the longitudinal and transverse directions. The rods were secured above the water surface in wooden frames, each 1.0 m long and 0.125 m wide and holding 200 rods, enabling their arrangement in the 7 distribution patterns shown in Fig. 7-9, all extending over a distance of 11.0 m. All the patterns contain the same total number of stems with the same local density and the same overall coverage (50% of the channel area). Pattern 1 represents the condition being investigated, but the others were tested to assist interpretation of the resistance phenomenon, and to provide data for checking the robustness of the conveyance prediction method proposed.

Stage-discharge measurements were taken for all the distribution patterns, and also for the basic channel with no stems. For the basic channel, flow depths were measured for 9 discharges between 5 l/s and 35 l/s. For the vegetated channels, flow depths were measured at discharges of 5 l/s, 10 l/s, 12.5 l/s and 15 l/s for all the patterns, and up to 22.5 l/s for Pattern 1 and 20 l/s for Pattern 5. Longitudinal flow velocities were also measured with a miniature propeller over one cross section in the clear channels to enable the clear channel discharges to be determined by integration. Velocities were measured at a number of verticals, the spacing depending on the clear channel width. At each vertical the velocity was measured at 0.2, 0.4 and 0.8 of the flow depth from the bed to enable a depth-averaged velocity (V) to be determined. No velocities were measured amongst the stems. Water temperatures were also measured to enable accurate specification of viscosity values.

Bed shear stresses across the clear channel were also measured over the full or half flow width for three flow depths with Pattern 1. This was done using a pitot-static tube as a Preston tube, with the calibration relationships of Patel (1965) used to relate shear stress to dynamic pressure head.

Further details of experimental procedure and additional measurements are presented by James et al. (2001).

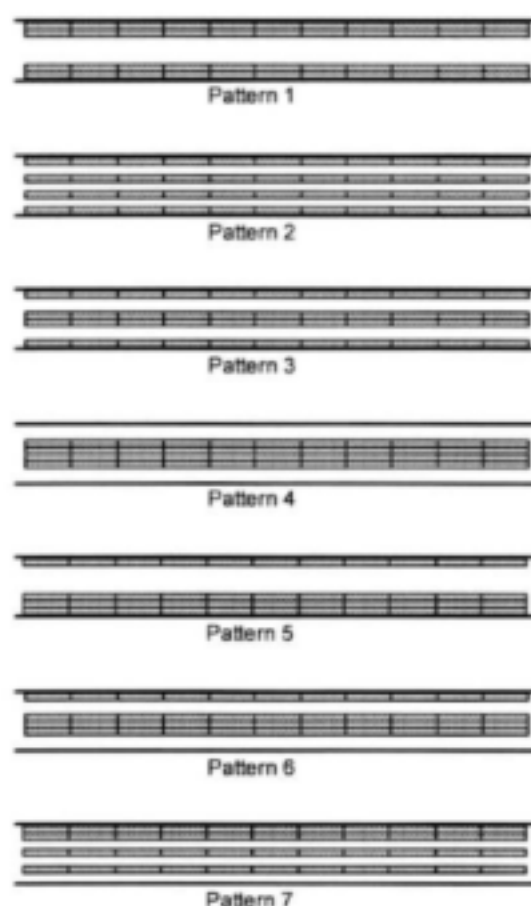


Figure 7-9 Experimental vegetation strip patterns (James et al, 2001)

7.2.2.2 Analysis of laboratory data

Application of the equivalent resistance coefficient formulae presented above (equations (7.17) to (7.20)) requires resistance coefficient values for the plaster bed and side boundaries of the basic channel and for the stem-water interfaces. Manning's n for the solid boundaries (n_b) was calculated from the stage-discharge measurements in the basic channel (without stems), giving a range from 0.0099 to 0.0106, with an average of 0.0102 and standard deviation of 0.00024; there was no discernible trend with flow condition. Values of the Darcy-Weisbach friction factor for both solid boundaries (f_b) and stem-water interfaces (f_s) were also determined from the integrated clear channel discharges and associated flow depths for Patterns 1, 3 and 5 by an inverse application of the side-wall correction procedure proposed by Brownlie (1981). The corresponding values of Manning's n were then calculated from the f values using the hydraulic radius values for the bed and interface determined by the side-wall correction calculations. The calculated values of n_b ranged from 0.0096 to 0.0102, with an average of 0.0098 and a standard deviation of 0.00017, while the values of n_s ranged from 0.0390 to 0.0474, with an average of 0.0432 and standard deviation of 0.0024. Neither n_s nor n_b showed any discernible trend with flow condition.

Values of f_s were also estimated from the measured bed shear stress profiles. For each case the interface shear force was calculated from a balance of forces, as the difference between the downslope weight component of the flow and the bed shear force, as determined by integration of the shear stress profiles (assuming symmetry where these were measured over only half the channel width). The shear force was used to calculate the interfacial shear stress by dividing by the interface area as defined by the flow depth, and thence f_s through the Darcy-Weisbach equation. It is not possible to determine corresponding Manning n values, because the hydraulic radius associated with the interface is not known. However, the average value of f_s determined by the force balance (0.233) agrees remarkably closely with that determined by the side-wall correction procedure (0.243), which adds credence to the side-wall correction results.

In the following analyses the value of n_b determined from the basic channel results, i.e. 0.0102, is used for the solid boundaries and the average value of n_s determined from the side-wall correction analysis, i.e. 0.0432, is used for the stem-water interfaces.

To test the composite roughness concept, the actual values of n_e were determined directly from Manning's equation for the clear channels of Patterns 1, 2, 3 and 5 (where all sides are stem boundaries) using discharges determined by integration of the measured velocity distributions. These equivalent values depend on the relative contributions of the different surfaces, and vary consistently with the width to depth ratios of the clear channels (Fig. 7-10). The n_e values were also predicted using the composite roughness formulas, equations (7.17) to (7.20). For calculation of subsection areas, the clear channel subdivision was made by bisecting the wall-bed angle (90° in these cases), and projecting the intersection of the subdivision planes from either side vertically in cases where the intersection point was below the water surface (as done by Flinham and Carling (1991) in a study of composite roughness with simple, solid boundary channels). The composite roughness methods assume no interaction between the subareas, implying no shear stress on the subdivision surfaces. Successful application therefore requires, ideally, specification of shear-free surfaces for the cross section subdivision. Such surfaces may exist, but their forms and locations can only be determined by more detailed velocity distribution description than obtained from these experiments. The angle-bisector planes are therefore approximations to shear-free surfaces, and their correctness is unknown. However, Flinham and Carling (1991) investigated the effect of replacing the angle-bisector planes with planes defined by assumed velocity contours, ensuring no shear stress, and found that discharge predictions were not sensitive to the location of the planes. The n_e values predicted using equations (7.17) to (7.20) show the same trend with width to depth ratio as the measured data (Fig. 7-10), suggesting that the variation is explained by the composite roughness effect. The method of Lotter (1933) clearly

exaggerates the relative influence of the bed roughness over the entire range of conditions. The methods of Horton (1933) and Pavlovski (1931) perform the best, with Horton's being superior in the more realistic higher range of width to depth ratios.

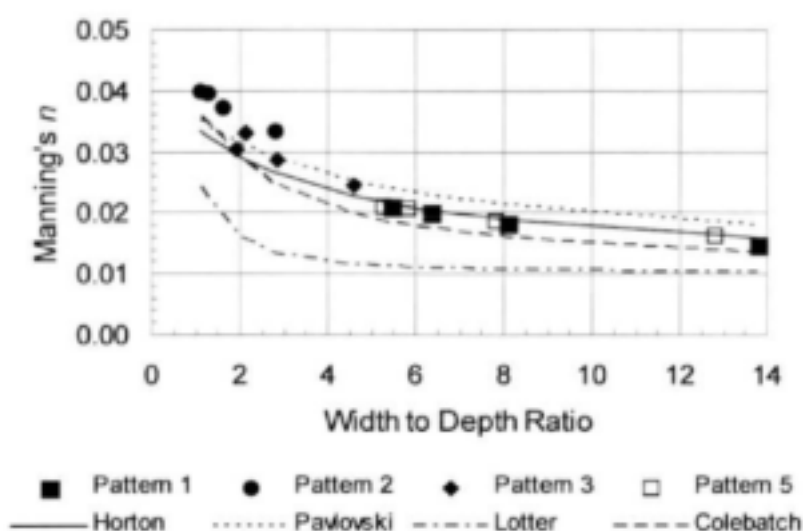


Figure 7-10 Clear channel equivalent Manning's n (n_e) values for Patterns 1, 2 and 5

(James and Makoa, 2006)

Total clear channel discharges have been predicted for all the experimental conditions using n_e from equations (7.17) to (7.20). The average absolute prediction errors are listed in Table 7-10. The method of Lotter (1933) is clearly unsatisfactory (as was also found by Flinham and Carling (1991)). Colebatch's (1941) formula consistently overpredicts discharge, with higher average error and standard deviation of absolute errors than the remaining two methods. The method of Pavlovski (1931) performs best overall and for most patterns, being inferior to Horton's (1933) method only for Patterns 1 and 5; discharge predictions are both over and under the measured values. Horton's (1933) method is also satisfactory, again with discharge being both under- and over-predicted. Its better performance for Patterns 1 and 5 suggests that it may be superior for wide channels with bank vegetation only, although these are two of the three patterns used for estimating the stem-water interface resistance in the first place. It is concluded that both the methods of Pavlovski (1931) and Horton (1933) give satisfactory results, and there is little to choose between them. These two methods have the added advantage that only the channel perimeter (and not the flow area) needs to be subdivided, obviating the necessity of attempting to define shear-free surfaces. The methods of Pavlovski (1931) and Horton (1933) were therefore selected for further application.

Vegetated zone and total discharge calculations for the laboratory experiments are presented by James et al. (2001) and James and Makoa (2006).

Table 7-10 Average absolute errors (%) for prediction of experimental clear channel discharge

Pattern	Clear Channel Discharges			
	Pavlovski (1931)	Horton (1933)	Colebatch (1941)	Lotter (1933)
1	14.28 (2.07)	4.00 (2.11)	13.12 (2.89)	70.71 (17.49)
2	7.47 (1.21)	14.12 (2.50)	11.45 (8.75)	80.70 (31.06)
3	3.71 (2.22)	8.33 (4.40)	15.00 (8.23)	104.3 (14.81)
4	6.51 (3.73)	19.60 (4.02)	27.52 (3.29)	215.1 (54.82)
5	12.51 (0.71)	1.96 (0.97)	15.56 (1.33)	76.20 (13.03)
6	4.64 (1.82)	5.30 (2.22)	12.87 (6.22)	147.4 (22.04)
7	2.09 (1.13)	7.48 (2.62)	10.33 (7.46)	161.1 (23.00)
All	7.32 (4.68)	8.68 (6.41)	15.12 (8.10)	122.2 (57.7)

7.2.2.3 Field application

Practical application of the above techniques requires resistance coefficient values for the channel bed and the interface between the vegetated and clear channel zones. Field data collected from the main waterway of the Okavango Delta in Botswana (Fig. 7-11) by McCarthy et al. (1991) have been used to determine a typical value of the interface Manning resistance coefficient, n_s , and also to test the applicability of the composite roughness formulae of Pavlovski (1931) and Horton (1933) for estimating clear channel conveyance. The clear water channels in the delta are close to rectangular in cross section (Fig. 7-12) and are flanked on both sides by effectively infinite emergent vegetation (dominated by *Cyperus papyrus*, with subordinate *Phragmites* species and *Miscanthus junceus*), making them well suited for this purpose. The stems do not extend to the channel bed, but are rooted in a permeable substrate of partly decomposed plant material that contains more than 50% free water and constitutes a boundary condition hydraulically similar to the stems. Measurements were taken at 10 sites over a distance of approximately 300 km, nine of which provide useful data for this analysis. A single flow condition was recorded at each of 7 of the sites, and two flow conditions at the other two. Stage-discharge relationships were therefore not obtained, but collectively the cross sections represent

a wide range of width to depth ratios (Table 7-11). At each site the measurements included cross-sectional depth profiles, flow velocities at 1.0 m vertical intervals at varying distances across the channels, water surface slope, bed load sediment transport rate and turbidity. The flow velocity and depth measurements were contoured and spatially integrated to determine average velocities and cross-sectional areas. The data relevant to this investigation are listed in Table 7-11.

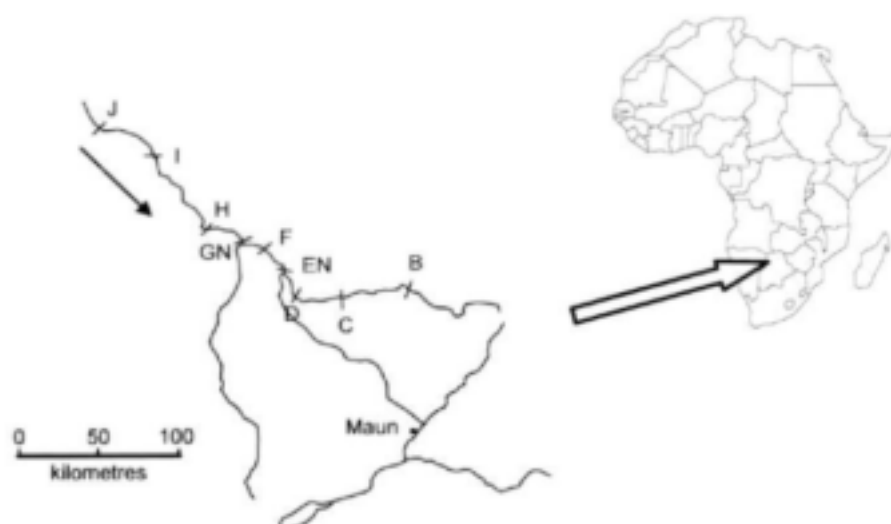


Figure 7-11 Field site locations in the Okavango Delta (after McCarthy *et al.*, 1991)

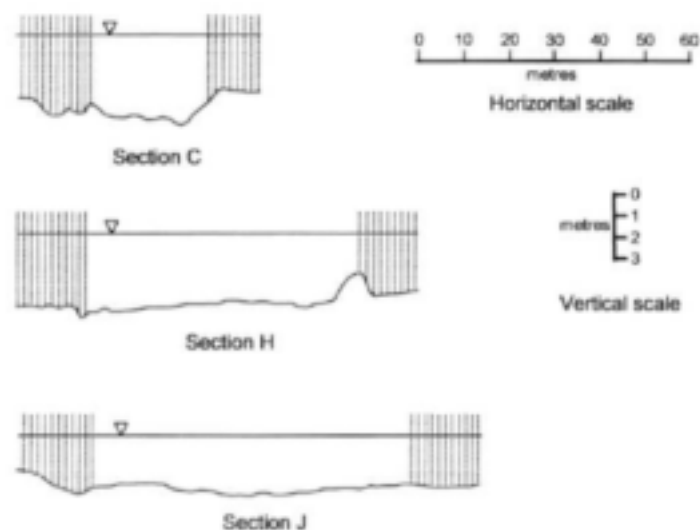
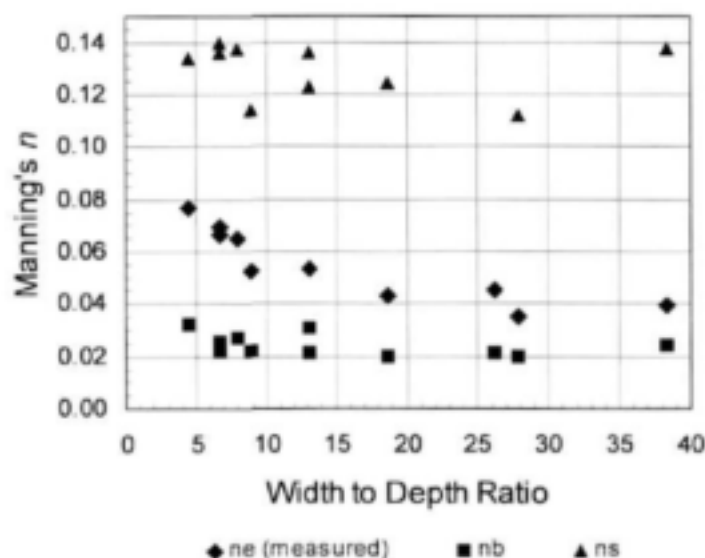


Figure 7-12 Typical cross-sections along the main channel in the Okavango Delta
(after McCarthy *et al.*, 1991)

Table 7-11 Okavango Delta channel data (after McCarthy et al., 1991)

Site	Width (m)	Mean Depth (m)	Width/Depth	Discharge (m ³ /s)	Bedload Discharge (kg/m/s)	Water Surface Slope	D ₅₀ (mm)	D ₉₀ (mm)
J	70	2.51	27.9	129.9	0.057	0.00022	0.35	0.50
I	92	2.40	38.3	143.6	0.045	0.00022	0.28	0.39
H	60	3.22	18.6	117.9	0.013	0.00017	0.37	0.62
GN	55	4.19	13.1	117.6	0.0047	0.00013	0.23	0.33
F	38	4.26	8.9	100.3	0.021	0.00020	0.35	0.47
EN	32	4.00	8.0	81.9	0.037	0.00037	0.41	0.90
D	26	3.85	6.8	66.0	0.038	0.00045	0.35	0.50
C	26	3.92	6.6	54.0	0.015	0.00031	0.28	0.37
B	17	3.76	4.5	30.7	0.015	0.00038	0.34	0.62
H _{repeat}	60	4.60	13.0	168.2	0.030	-	0.37	0.62
I _{repeat}	88	3.36	26.2	204.1	0.040	-	0.28	0.39

Values of n_e calculated from the measured data for all the sites show a consistent declining trend with width to depth ratio (Fig. 7-13), similar to the laboratory data (Fig. 7-10). The data do not provide direct estimates of the Manning resistance coefficient for the bed (n_b) or sides (n_s), and these were therefore determined indirectly.

Figure 7-13 Measured n_e and calculated n_b and n_s values for the Okavango Delta main channel

The resistance coefficient for the bed (n_b) is directly related to bed shear stress which, in turn, is related to the bed load discharge. It is therefore possible to infer values of n_b from the measured bed load. Because of the high degree of uncertainty inherent in sediment discharge equations, the analysis has been carried out using three well-

known methods, viz. those of Meyer-Peter and Müller (1948), van Rijn (1984a), and Ackers and White, as updated by Ackers (1993).

Meyer-Peter and Müller (1948) proposed that the bed load discharge in terms of submerged weight, g_s' , is related to flow and sediment characteristics by

$$\frac{\gamma R \left(\left(\frac{k}{k'} \right)^{3/2} \right) S}{d} - 0.047(\gamma_s - \gamma) = 0.25 \rho^{1/3} \frac{(g_s')^{2/3}}{d} \quad 7.21$$

where γ and γ_s : the specific weights of water and sediment respectively,
 d : the mean sediment particle size,
 $k (= 1/n_b)$: the total resistance coefficient (including contributions of both grain roughness and the form roughness associated with bed forms), and
 k' : the resistance coefficient related to grain roughness only.

Müller (1943) proposed that k' be calculated from the sediment size by

$$k' = \frac{26}{d_{90}^{1/6}} \quad 7.22$$

with d_{90} in metres. The hydraulic radius, R , in a composite channel must be that associated with the portion on which the bed load transport is taking place, which requires assignment of a corresponding portion of the flow area. Flinham and Carling (1991) recommended subdividing the flow area with planes from the bed/side intersections inclined at 45° to the bed with their intersection projected vertically if this is below the water surface. In view of the findings of James et al., (2001) that the influence of bank vegetation on clear channel flow might be more extensive than previously supposed, the analysis was repeated with inclination angles of 30° and 15° . The average values of $n_b (= 1/k)$ obtained by solving equations (7.21) and (7.22) for all the field sites are listed in Table 7.12 for the different subdivision angles used.

According to van Rijn (1984), the bed load discharge in volumetric units, q_b , is given by

$$\frac{q_b}{\sqrt{(S_s - 1)g} d_{50}^{1.5}} = 0.053 \frac{T^{2.1}}{D_s^{0.3}} \quad 7.23$$

where S_s : the specific gravity of the sediment,
 d_{50} : the median particle size,

- D_* : a dimensionless representation of d_{50} , and
 T : a transport stage parameter representing the excess of sediment-moving shear stress over the critical value at incipient motion.

The transport stage parameter, T , was calculated for each site using equation (7.23) with the measured values of bed load discharge and d_{50} . The resulting value of T and the measured flow depth were then used to determine the bed form roughness according to the bed form geometry and roughness relationships proposed by van Rijn (1984b). The total bed roughness (k_s) was then determined by adding this to the bed roughness associated with the grains (given by van Rijn (1984b) as $3d_{90}$). The Chézy resistance coefficient, C , was then calculated as

$$C = 18 \log \left(\frac{12 R}{k_s} \right) \quad 7.24$$

where the hydraulic radius (R) is that associated with the bed, calculated as for the Meyer-Peter and Müller (1948) analysis. The value of n_b was then determined from C by equating the Chézy and Manning formulae. The average values for all the field sites are listed in Table 7-13 for the different assumed subdivision angles.

The Ackers and White sediment transport equation was expressed by Ackers (1993) as a relationship between the flow velocity (V) and the volumetric sediment concentration (C_v), i.e.

$$\frac{V}{\sqrt{g(S_s - 1)d}} = K \left(1 + J C_v^{1/n} \right) \quad 7.25$$

with

$$K = \frac{A_{gr} \left(11.3 \left(\frac{y}{d} \right)^{0.1} \right)^{1-n}}{\left(\frac{f}{8} \right)^{5/2}} \quad 7.26$$

and

$$J = \left[\left(\frac{y}{d} \right) \left(\frac{f}{8} \right)^{5/2} \left(\frac{1}{C} \right) \right]^{1/m} \quad 7.27$$

- where y : the flow depth, and
 m, A_{gr}, n and C : empirical values depending on the sediment grain characteristics, as given by Ackers (1993).

Table 7-12 Bed and side Manning's n values and errors in predicted equivalent values for Okavango channels (values in parentheses are standard deviations)

Bed Load Equation	Channel Subdivision Angle	Average n_b	Average n_s	Absolute Error in n_s Prediction (%)	
				Pavlovski (1931)	Horton (1933)
Meyer-Peter & Müller (1948)	45°	0.0328 (0.0104)	0.1125 (0.0145)	5.45 (1.54)	14.14 (6.45)
	30°	0.0306 (0.0088)	0.1254 (0.0154)	0.94 (0.69)	11.07 (6.09)
	15°	0.0240 (0.0056)	0.1487 (0.0165)	10.38 (2.28)	6.81 (5.34)
Van Rijn (1984a)	45°	0.0281 (0.0094)	0.1136 (0.0144)	5.55 (0.82)	14.65 (2.73)
	30°	0.0281 (0.0009)	0.1239 (0.0158)	0.97 (0.88)	10.28 (2.18)
	15°	0.0285 (0.0012)	0.1385 (0.0178)	9.28 (2.61)	3.95 (2.29)
Ackers & White (Ackers, 1993)	45°	0.0267 (0.0052)	0.1188 (0.0103)	6.23 (0.94)	17.44 (3.42)
	30°	0.0252 (0.0043)	0.1315 (0.0113)	1.06 (0.83)	13.62 (3.40)
	15°	0.0209 (0.0048)	0.1530 (0.0127)	11.89 (2.64)	8.07 (3.58)

Replacing the flow depth (y) by the hydraulic radius associated with the bed, as before, the value of f ($= f_b$) was calculated for each site using equations (7.26) to (7.27) and converted to n_b . The resulting average values for all sites and each of the channel subdivision angles are presented in Table 7-12.

Once n_b had been established, the shear stress on the bed was calculated from the relationship between boundary shear stress and Manning's n ,

$$\tau_o = \frac{V^2 \gamma n}{R^{1/6}} \quad 7.28$$

using the cross-sectional average velocity for V and R associated with the bed. The shear stress on the sides could then be calculated by force balance, and n_s calculated by applying equation (7.28) to the sides, using the cross-sectional average velocity for

V and R associated with each side. The average values of n_s calculated for all sites using the different sediment transport equations and subdivision angles are presented in Table 7-12.

The calculated values of n_b clearly depend both on the sediment transport equation used and on the assumed positions of the subdivision planes. Considering the uncertainty normally associated with sediment transport predictions, the differences are relatively modest, however; the ranges of values obtained by the different equations are between 20% and 30% of the mean values for the different subdivision angles. The effect of the choice of subdivision angle is just as important, producing similar difference in n_b with the Meyer-Peter and Müller (1948) and Ackers and White (Ackers, 1993) equations, but little difference with the van Rijn (1984a) equation. The calculated values of n_s are much less sensitive than n_b to the sediment transport equation used, ranging only between 5% and 10% of the mean values for the different assumed subdivision angles. They are just as sensitive as n_b to choice of subdivision angle, however. The van Rijn (1984a) analysis produced the least variance of n_b with subdivision angle and greatest constancy with W/D , while the Ackers and White (Ackers, 1993) analysis performed best in these respects for n_s . The measured values of n_e are plotted in Fig. 7-13 together with the values of n_b and n_s as calculated using the Ackers and White equation with a subdivision angle of 30° , as an example of the analysis results.

The calculated values of n_b and n_s were used to test the prediction of effective Manning resistance coefficient, n_e , by the formulae of Horton (1933) (equation (7.17)) and Pavlovski (1931) (equation (7.18)). The average absolute prediction errors for all Okavango sites are listed in Table 7-12. The best predictions (about 1.0% absolute error) are by the formula of Pavlovski (1931), with n_b predicted by any of the sediment transport equations, using flow area subdivision by planes at 30° to the bed. The predicted values of n_e (using n_b and n_s from the Ackers and White analysis) are shown together with the measured values on Fig. 7-14. Also shown on Fig. 7-14 are values calculated for each site using the average values of n_b and n_s for all sites, rather than the local values; agreement is still good, with an average absolute error of 5.7%. The close agreement of the predicted with the measured values confirms that the composite resistance approach accounts for the variation of overall resistance with width to depth ratio. The average value of n_s is about 0.13. In general, this will depend on vegetation type and state as well as the flow condition, and further field investigation is required to establish representative values for a range of conditions.

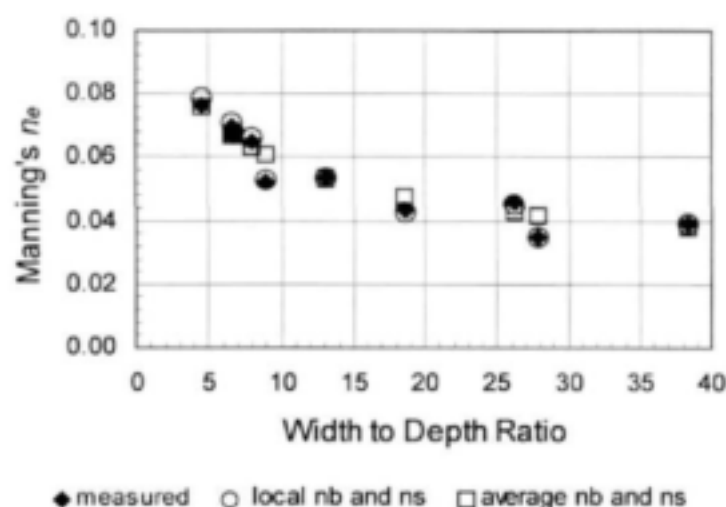


Figure 7-14 Measured and predicted n_e values for the Okavango Delta main channel

7.2.3 Conclusions

Conveyance in channels with emergent bank vegetation can be estimated by a zonal approach, whereby the total discharge is calculated as the sum of discharges in the vegetated and clear channel zones, calculated separately.

Both laboratory and field applications show that clear channel discharges can be reliably estimated using conventional resistance equations, such as Manning's. The variation of Manning's n with width to depth ratio can be attributed to the varying proportional contributions of composite roughness components, and can be reliably accounted for by the Horton (1933) (equation (7.17)) and Pavlovski (1931) (equation (7.18)) formulae.

Resistance coefficient values for a channel bed and vegetated side boundaries can be estimated from field stage-discharge and sediment bed load measurements. The bed value can be obtained by back-calculation from sediment transport equations, and the side values thence by force balance. The portions of flow area associated with the sides and bed were found to be best defined by subdivision planes at 30° to the bed. A value of Manning's n for the surface presented by reed-type emergent bank vegetation was found for a field situation to be about 0.13.

7.3 Modelling of Flow Resistance for Discrete Reed Patches

7.3.1 Introduction

Vegetation is one of the river features that have significant contribution to overall flow resistance, and its prediction is therefore essential in hydraulic modelling. A

prediction method for estimating the average velocity and hydraulic resistance within vegetation is presented in section 7.1. Conveyance estimation for channels with continuous emergent vegetation strips is discussed in section 7.2. Besides the longitudinal strips, vegetation within a channel often forms discrete, longitudinally discontinuous patches. The influence of discrete reed patches on overall flow resistance has been investigated (James et al., 2001) and it was found that the resistance is strongly influenced by the overall areal coverage, and varies significantly with the overall distribution pattern of the patches, the size and shape of the patches, and the degree of longitudinal and transverse fragmentation. The resistance phenomenon under such conditions is very complex, and conveyance cannot be described by a single equation or simple procedure. Computational modelling in two or three dimensions provides a realistic alternative approach.

In this section an attempt to model the flow resistance with discrete patches using River2D software is presented.

7.3.2 River2D Software

River2D is a two-dimensional hydraulic and habitat simulation model, capable of predicting values of hydraulic parameters such as depth and velocity in a spatially explicit way. The programme is designed for both steady and unsteady flow analyses, and the numerical solutions are obtained using finite element methods.

River2D consists of four modules that run separately:

- R2D_bed - bed topography module,
- R2D_ice - ice cover module,
- R2D_mesh - finite element mesh module, and
- River2D - flow and habitat analysis module.

River2D software and documentation are available free of charge for download from website <http://www.river2d.ualberta.ca/>.

Implementation of River2D for modelling in South Africa is a part of a present Water Research Commission project K5/1508. The project will present research related to the best use of River2D as a hydraulic analysis tool. Discussions of modelling considerations, inputs and parameters, the sensitivity of two-dimensional hydraulic model to these inputs and parameters, and a set of guidelines for the use of River2D for eco-hydraulic modelling in South Africa will be included in the final report.

7.3.3 Laboratory Experiments (James et al., 2001)

Discrete vegetation patch experiments were performed in the same facility used for the longitudinal strip experiments. The discrete patches were simulated by the same frames (section 7.2.2.1). The frames were arranged in 15 different discrete patch

distribution patterns, and numbering of the distribution patterns used here is the same as in James et al. (2001) in their Fig. 8-24.

Experiments were conducted for different areal coverages. The maximum total channel area covered by the patches was 50%, while the minimum was 12.5%. Percentages of total area covered by patches for each pattern are as follows:

Pattern 8:	25.00
Pattern 9:	37.50
Pattern 10:	37.50
Pattern 11:	50.00
Pattern 12:	28.125
Pattern 13:	28.125
Pattern 14:	28.125
Pattern 15:	50.00
Pattern 16:	50.00
Pattern 17:	12.50
Pattern 18:	12.50
Pattern 19:	25.00
Pattern 20:	37.50
Pattern 21:	25.00
Pattern 22:	12.50

Flow depths were measured at different discharges for different patterns, and the flow resistance in terms of Manning's n for each measurement was calculated. Manning's n values calculated for measured depths at a discharge of 10 l/s are plotted against proportion of total area covered (AC) by patches for all patterns in Fig. 7-15. It is evident that resistance increases with increasing total area covered, but it is also clear that for the same area covered it varies over a very wide range for different patch arrangement patterns.

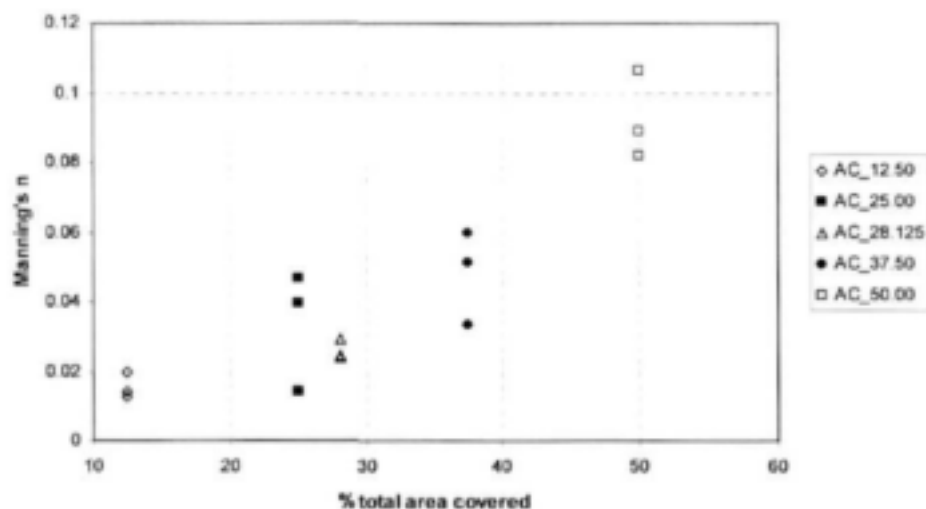


Figure 7-15 Manning's n variation with proportion of total area covered by patches, $Q=10$ l/s

The experiments to investigate resistance due to longitudinal vegetation strips (section 7.2.) were all carried out with a total areal coverage of 50 %. The overall resistance (Manning's n) values for a discharge of 10 l/s are included in Fig. 7-16 for comparison with other patch distribution patterns with the same areal coverage.

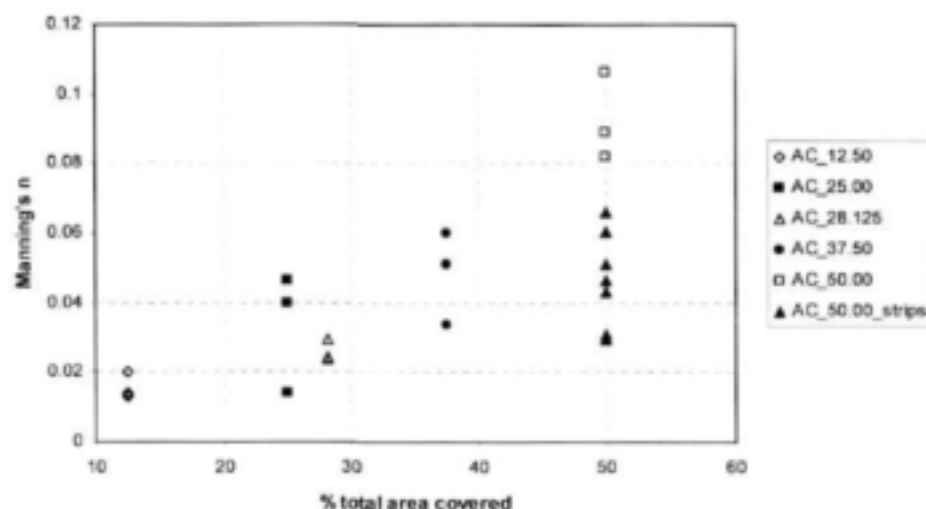


Figure 7-16 Flow resistance in term of Manning's n against area covered by vegetation including longitudinal strips

From Fig. 7-16 it can be seen that Manning's n for an areal coverage of 50 % varies from 0.03 to 0.106, showing that overall resistance depends significantly on the degree of fragmentation of patches and the length of stem-clear water interface. As vegetation patches are often irregular in shape and distribution, conveyance prediction

for such channels is not simple and requires computational modelling for adequate description.

Application of River2D model to reproduce experimental results of some of the 15 discrete patch patterns is discussed in the next section.

7.3.4 Modelling channels with discrete patches using River2D

7.3.4.1 Introduction

As mentioned above, a current Water Research Commission project (K5/1508) related to eco-hydraulic modelling includes investigation of the applicability of River2D model to provide prediction of effects of vegetation on overall resistance. The two ways of modelling flow through vegetation by the model – by treating it either as large scale roughness or as a porous material – have been investigated. It has been concluded that River2D can be used to predict the resistance of vegetated channel with reasonable accuracy.

The following sections describe the application of River2D to discrete patch patterns 11, 15, 16, 20 and 22.

7.3.4.2 Modelling Setup

The layouts of modelled Patterns 11, 15, 16, 20 and 22 are shown in Figs 7-17, 7-18, 7-19, 7-20 and 7-21 respectively. In each case the vegetated channel has a length of 12.00 m, a width of 1.00 m, and a slope of 0.00107.

A mesh of 0.25 m size was set up over the whole bed, and was refined to 0.05 m over the clear channel, spaces between the discrete patches. Modelling was carried out for a discharge of 10 l/s only, and this value was set as a boundary condition of inflow.

Vegetation patches were modelled as:

- large scale roughness with a high value of the bed roughness k_s in the vegetation zones referred to as bed_1, and
- zones of porous material, treated as groundwater flow by River2D, and referred to as bed_2.



Figure 7-17 Layout of Pattern 11. Vegetation in Red



Figure 7-18 Layout of Pattern 15. Vegetation in Red



Figure 7-19 Layout of Pattern 16. Vegetation in Red



Figure 7-20 Layout of Pattern 20. Vegetation in Red



Figure 7-21 Layout of Pattern 22. Vegetation in Red

7.3.4.3 Large Scale Roughness Approach

Vegetation patches were modelled as areas with large scale roughness, requiring specification of a corresponding value of roughness size k_s . Laboratory experiments on flow resistance within vegetation (James et al., 2001) indicate values of the Chézy resistance coefficient (used in River2D) as low as $2.7 \text{ m}^{1/2}/\text{s}$. The vegetation used in the laboratory experiments was very dense (with a stem spacing of 25 mm), resulting in this very high resistance. The calculated value of flow resistance exceeds the limit that can be specified in the River2D. The highest permissible value of the bed roughness k_s was therefore used, and it was therefore expected that the modelled flow resistance might be lower than measured. Measured and modelled flow depths and corresponding resistance in term of Manning's n for considered patterns are presented in Figs 7-22 and 7-23 respectively. Modelled data for this condition are labelled "bed_1".

7.3.4.4 Groundwater Water Approach

River2D includes groundwater flow equations that can be used to model vegetation patches as a porous material. The programme automatically switches from the surface flow equations to groundwater flow equations when no water occurs over a part of the modelled area. The vegetation patch levels were therefore increased to above the water level so that the groundwater flow equations would be implemented in the discrete patch regions. Modelled flow depths and flow resistances in terms of Manning's n for each pattern, labelled "bed_2", are presented in Figs 7-22 and 7-23 respectively.

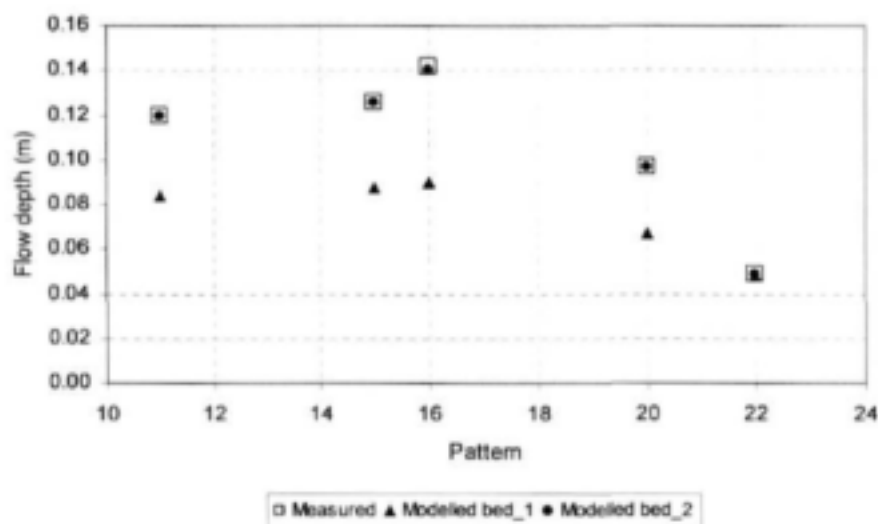


Figure 7-22 Measured and modelled River2D flow depths

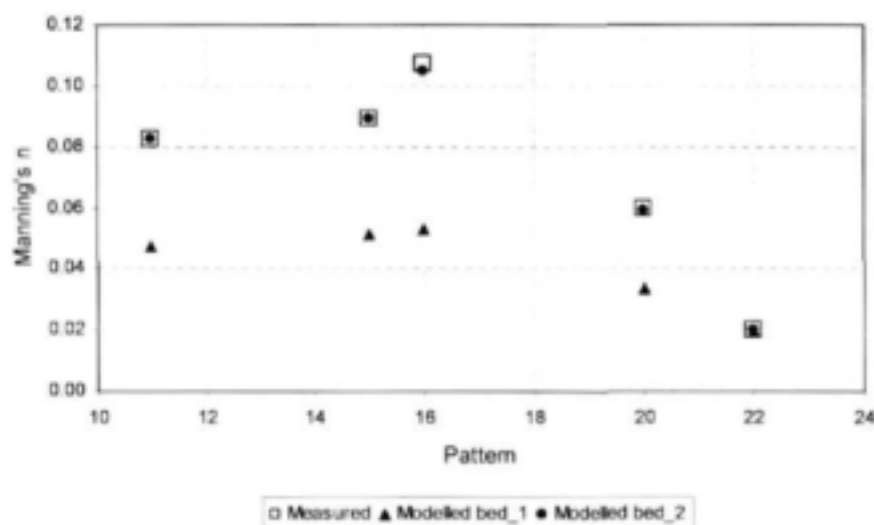


Figure 7-23 Flow resistance in term of Manning's n for measured and modelled River2D data

7.3.4.5 Results and Discussion

Results of River2D modelling for discrete vegetation patch distribution patterns in terms of flow depth and Manning's n resistance coefficient are shown in Figs 7-22 and 7-23 respectively. Modelling vegetation as a porous material reproduced the measured flow depth and the resistance well for all the patterns considered. Modelling vegetation as a surface with large scale roughness produced flow depths and resistance values lower than were measured; this was expected as the true roughness exceeded the input limit of River2D. The difference between measured and modelled values also varies between the different patch patterns. The highest modelled error is for pattern 16, while the modelled flow depth for pattern 22 with areal coverage of 12.50 % is about the same as was measured. It also can be seen that the modelled error for pattern 16 is higher than for patterns 11 and 15, even though they all have the same areal coverage of 50 %. This difference can be explained by lateral momentum transfers flow caused by the vegetation patches. Pattern 16 has the largest patches, and hence the largest in-vegetation flow areas that would not be affected by momentum transfer and where the resistance is closest to the uninfluenced vegetation value; it is therefore the pattern most affected by the input limitation on resistance coefficient.

7.3.5 Conclusions

The River2D model has been tested for prediction of flow resistance in channels with discrete patches. Experimental data of James et al. (2001) were reproduced, and modelled and measured resistance values were compared. Discrete vegetation patches were represented as regions of large scale roughness with high values of bed roughness size k_s ("bed_1" model) and as regions of porous material ("bed_2" model). Modelled results showed that the "bed_2" representation reproduced the experimental data very well, while the "bed_1" model underestimated the measured flow resistance. The difference in performance of two models can be explained by the very dense vegetation used in the laboratory experiments, which had a very high flow resistance that exceeded the limitation of the programme. On the other hand, the "bed_2" model has an un-realistic flow through the vegetation patches, while the "bed_1" model provides a better cross-sectional velocity distribution.

In general, River2D can be used to predict the flow resistance of vegetation patches with reasonable accuracy. More information related to use of River2D for modelling of channels with vegetation for South African application will be provided in the report for WRC project K5/1508, currently in preparation.

8 RIVER2D APPLICATION TO FIELD DATA

8.1 Introduction

The model River2D is recommended in this report for predicting velocity and flow depth distributions under low flow conditions. Its performance has been demonstrated in Chapters 6 and 7 by comparison of its predictions with laboratory data. In this chapter, its application to two field situations where appropriate data are available is presented. Two sites, Spur Hole and Vanities Crossing, on the Cotter River in Australia and one site, Site A, of the Driehoeks River in South Africa were used for this application. The sites are very different in terms of dominant river-bed substratum (Figs 8-2, 8-3 and 8-26). The Spur Hole site is dominated by cobble and gravel substrate. Bedrock and cobble represent the Vanities Crossing site, while Site A is characterized by a gravel bed with vegetation patches.

8.2 Cotter River Modelling

8.2.1 Introduction

The Cotter River in Australia has been studied intensively by the Riverine Landscapes Research Laboratory at the University of Canberra. This river presents features and hydraulic characteristics similar to those of many South African rivers, and is suitable for assessing the capabilities of River2D. The data used for this application have been made available by Professor Martin Thoms (pers. comm.).

The Cotter River catchment is situated in the Brindabella Range within the Australian Capital Territory (ACT), Australia, and occupies an area of 48,300 hectares (Fig. 8-1). The river flows in a northerly direction before joining the Murrumbidgee River approximately 72 km downstream at Cotter Reserve. The area consists of steep slopes and rock outcrops, with granite, limestone, siltstone and shales present. Most of the catchment (88%) lies within the Namadgi National Park with land-use dominated by native forest. Plantation forestry of predominantly pine trees occupies the lower catchment. Because the majority of the catchment is managed for nature conservation and protection of water supply, the Cotter River flows through a largely unmodified area. It is an upland boulder, cobble and gravel-bed river with a highly heterogeneous bed topography including bedrock outcrops.



Figure 8-1 Location of the Cotter River Catchment and study area (Dyer and Thoms, 2006)

8.2.2 Selection of Sites

Two sites were selected, i.e. one at Spur Hole and one a short distance downstream from Vanities Crossing. The selected sites are shown in Figs 8-2 and 8-3 respectively.

Standard techniques (Elliott et al., 1996) were used to collect data on site topography, substrate and hydraulics. Survey markers were used to identify the locations of the cross-sections, transect-profiles were surveyed and substrate sizes present were visually assessed (Appendix G). The two sites were visited under three different flows between April and June 2002 in order to collect field data across the range of discharges of prime interest to this study.



Figure 8-2 Cotter River, Spur Hole Site



Figure 8-3 Cotter River, Vanities Crossing Site

8.2.3 Cross-section Data

At Spur Hole, cross-sections 2, 3, 4 and 5 were surveyed relative to each other and therefore bed heights are also relative to each other. Cross-section 1 is a 'stand alone' cross-section some distance downstream from the other four, and bed heights are not relative to other cross-sections. An arbitrary datum of 100 m was given to the right

bank marker at cross-sections 1 and 5. At Vanities Crossing, all cross-sections are in close proximity and were surveyed relative to each other. Therefore bed heights are also relative to each other. An arbitrary datum of 100 m was given to the left bank marker at cross-section 5.

The bed profiles for each cross-section with the water surface levels during each of the three surveys are illustrated in Appendix G, Figs G-1 and G-2 for Spur Hole site and in Figs G-3 and G-4 for Vanities Crossing. Cross-sectional surveyed and substrate data are also presented in Appendix G.

8.2.4 Flow Data

The two sites were visited under three different flow conditions between April and June 2002 in order to collect field data across the range of discharges of prime interest to this study. During each of the three site visits, water surface elevations were recorded. Cross-sectional water velocities and water depths were measured at 0.5m intervals across all transects during the low and medium flow calibration data collection, and at a limited number of cross-sections (i.e. those that could be accessed safely) during the high flow data collection.

Table 8-1 summarises the dates when the field data were collected and the flows (in m^3/s and Ml/d) that were present during each of the surveys.

Table 8-1 Flow data.

Field data collection	Spur Hole		Vanities Crossing	
	m^3/s	Ml/d	m^3/s	Ml/d
Low flow (24/4/02)	0.3427	29.6	0.3305	28.6
Medium flow (20/6/02)	1.2686	109.6	1.308	113.1
High flow (30/5/02)	2.2498	194.4	2.1118	182.5

8.2.5 River2D Modelling

River2D modelling of two sites, (Spur Hole and Vanities Crossing, both on the Cotter River) with two discharges (low and medium) (Table 8-1) were performed by the following steps:

- Processing of survey data
- Specification of bed roughness (k_s)
- Mesh generation
- Boundary condition specification
- Analysis
- Evaluation of results

8.2.5.1 Modelling Approach

All survey data, at five cross-sections for each site, were linked in a "xyz" co-ordinate system. During the field work substrate sizes were assigned at each surveyed point. The value of the bed roughness (k_s) for the all surveyed points was estimated using the Conveyance Estimation System of Wallingford Software (2004). Resistance coefficients in term of Manning's n that they suggest (for a flow depth of 1m) have been converted to equivalent values of roughness k_s (Hirschowitz et al., in preparation) and these were used in the model. Values of roughness k_s for different bed materials are showed in Table 8-2. Topography and bed roughness data of the considered sites were saved in a text file using the format required by River2D. A mesh of 0.5m size was set up over the whole bed. The upstream boundary condition was specified as a discharge, while a water surface elevation was set up as the downstream boundary condition.

Table 8-2 Natural bed material resistance according to Wallingford Software (2004)

Bed Material	Manning's n ($s^{-1/3}m^{-1}$)			Equivalent k_s (m)		
	Typical	Minimum	Maximum	Typical	Minimum	Maximum
Bedrock	0.025	0.023	0.028	0.072	0.046	0.125
Cobbles	0.035	0.031	0.039	0.311	0.194	0.452
Coarse gravel	0.027	0.025	0.03	0.105	0.072	0.169
Gravel	0.025	0.020	0.028	0.072	0.020	0.125
Fine gravel	0.022	0.017	0.025	0.036	0.007	0.072
Sand	0.020	0.017	0.025	0.020	0.007	0.072
Silt	0.018	0.015	0.022	0.010	0.002	0.036
Clay	0.020	0.018	0.023	0.020	0.010	0.046
Peat	0.020	0.018	0.023	0.020	0.010	0.046
Earth	0.020	0.018	0.023	0.020	0.010	0.046

8.2.5.2 Spur Hole Site Modelling and Results

River2D steady flow modelling was performed with two discharges ($0.343m^3/s$ and $1.27m^3/s$). Before running the model, it is necessary to specify boundary locations and conditions. Locations for the upstream and downstream boundaries of the model should be cross-sections. For steady models, the upstream boundary condition is usually specified as a flow rate, and the downstream boundary condition is specified as a set water surface elevation.

The following locations and conditions for the upstream and downstream boundaries were specified:

- Location of the upstream boundary - 1st upstream cross-section
- Location of the downstream boundary - the last downstream cross-section

- Condition of the upstream boundary – $Q=0.343\text{m}^3/\text{s}$ and $Q=1.27\text{m}^3/\text{s}$ for runs 1 and 2 respectively
- Condition of the downstream boundary - the last downstream cross-section water surface elevations related to $Q=0.343\text{m}^3/\text{s}$ and $Q=1.27\text{m}^3/\text{s}$ for runs 1 and 2 respectively.

A plan view of the velocity distribution for $Q=0.343\text{m}^3/\text{s}$ is shown in Fig. 8-4 as an example of River2D output results.

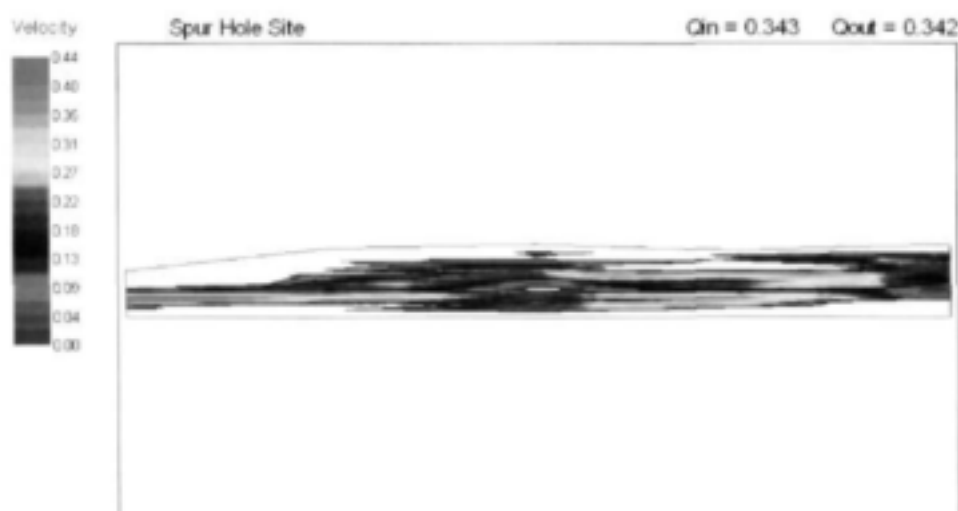


Figure 8-4 Spur Hole Site, overview of velocity distribution for $Q=0.343\text{ m}^3/\text{s}$

More meaningful interpretations of the predictions are possible from comparison of modelled and measured frequency distributions of velocity and depth in the form of histogram. Figs 8-5 and 8-6 present the histograms for sections 2, 3 and 4 for the discharge $Q = 0.343\text{m}^3/\text{s}$. Figs 8-7 and 8-8 are for the discharge $Q = 1.27\text{m}^3/\text{s}$.

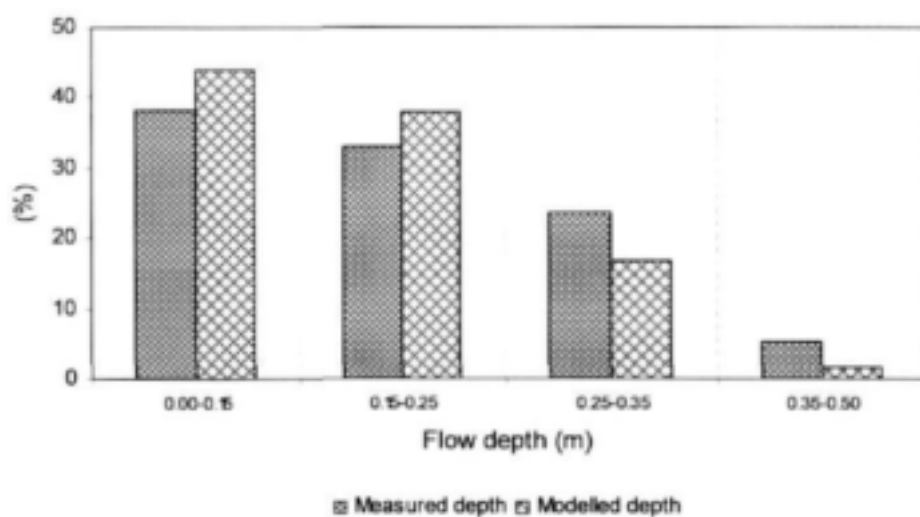


Figure 8-5 Spur Hole Site, modelled and measured frequency-depth distributions through cross-sections 2, 3 and 4, for $Q = 0.343\text{m}^3/\text{s}$

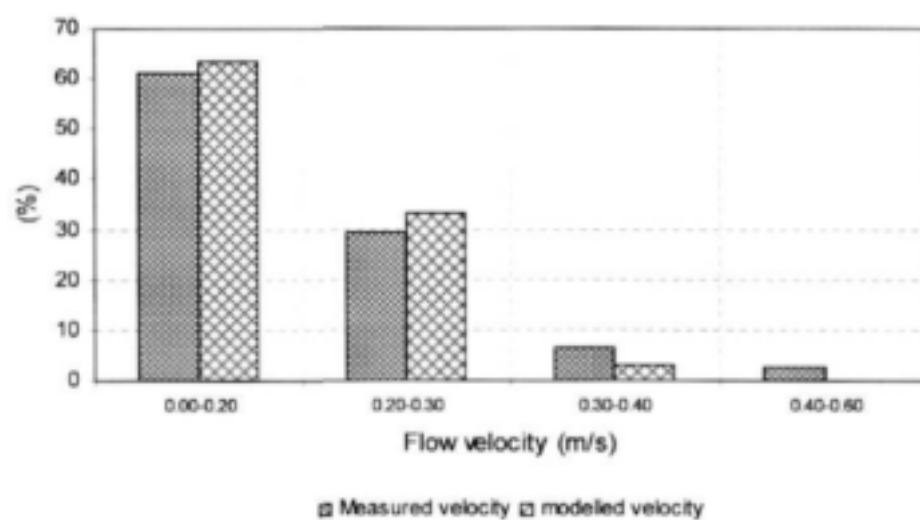


Figure 8-6 Spur Hole Site, modelled and measured frequency-velocity distributions through cross-sections 2, 3 and 4 for, $Q = 0.343\text{m}^3/\text{s}$

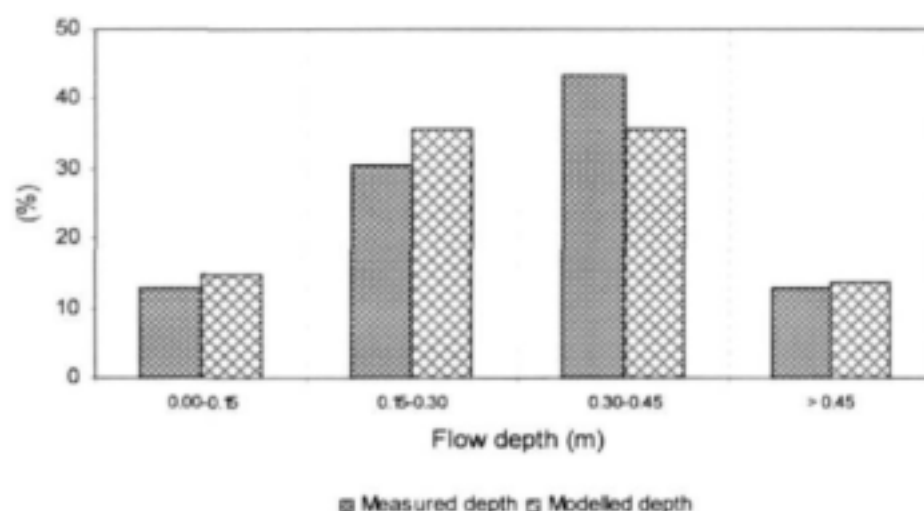


Figure 8-7 Spur Hole Site, modelled and measured frequency-depth distributions through cross-sections 2, 3 and 4, for $Q = 1.27\text{m}^3/\text{s}$

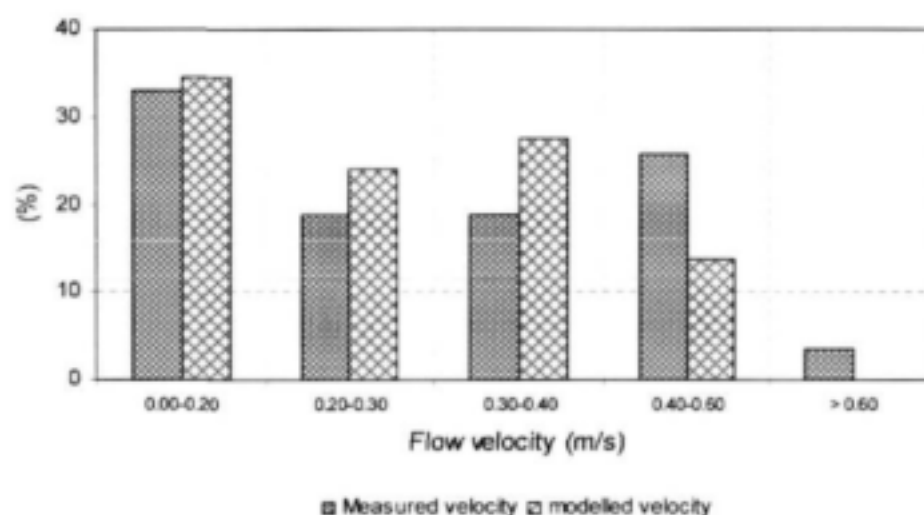


Figure 8-8 Spur Hole Site, modelled and measured frequency-velocity distributions through cross-sections 2, 3 and 4, for $Q = 1.27\text{m}^3/\text{s}$

Generally, the modelled frequencies of velocity and depth reproduce the measured values well. It also can be seen that River2D performs better in terms of depth distribution than velocity distribution. The model did not predict occurrence of velocities higher than 0.4m/s and 0.6m/s for $Q=0.343$ and $1.27\text{m}^3/\text{s}$ respectively (Figs 8-6 and 8-8).

For South African rivers, Kleynhans (1999) has recommended characterization of hydraulic habitats for fish according to the following four velocity-depth classes:

- Slow-Deep (SD): depth > 0.5m, and velocity < 0.3m/s (including deep pools and backwaters)
- Slow-Shallow (SS): depth < 0.5m, and velocity < 0.3m/s (including shallow pools and backwaters)
- Fast-Deep (FD): depth > 0.3m, and velocity > 0.3m/s (including deep runs, rapids and riffles)
- Fast-Shallow (FS): depth < 0.3m, and velocity > 0.3m/s (including shallow runs, rapids and riffles).

These depth-velocity classes, together with features that provide cover for fish, are usually used in Reserve determination studies. For further evaluation of River2D performance, modelled and measured velocity and depth were therefore classified into these classes. A comparison was performed for $Q = 0.343\text{m}^3/\text{s}$ and $1.27\text{m}^3/\text{s}$ for cross-sections 2, 3 and 4 combined. These results are plotted as depth/velocity class frequency distributions in Figs 8-9 and 8-10 respectively. The combined results for cross sections 2, 3 and 4 will be referred to as “cross sectional scale” results.

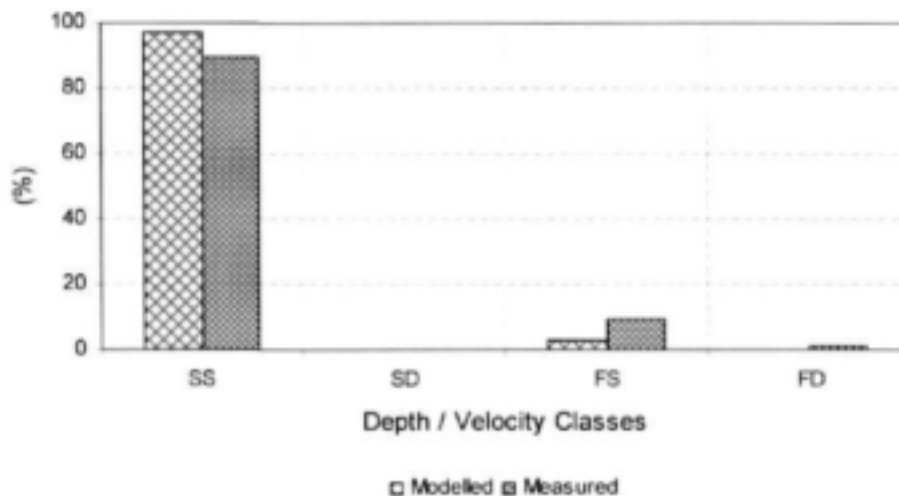


Figure 8-9 Spur Hole Site, modelled and measured frequency-depth/velocity class distributions, for $Q = 0.343\text{m}^3/\text{s}$

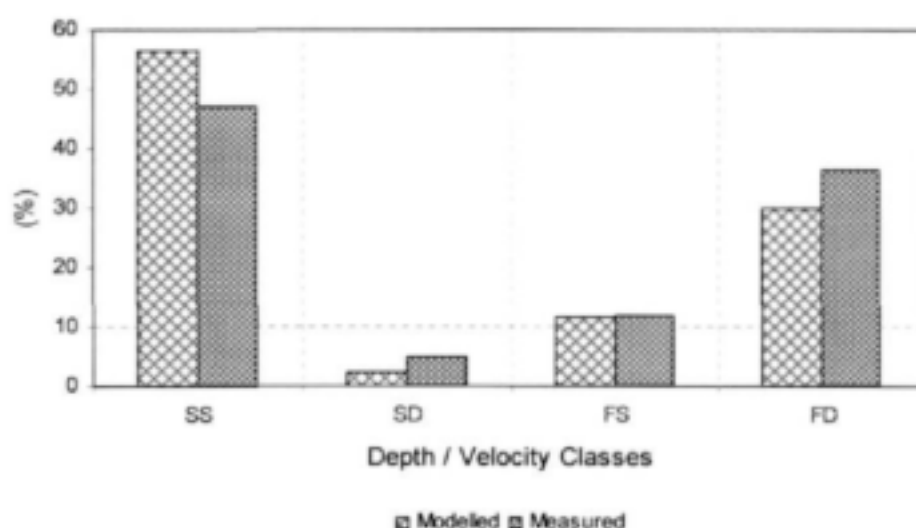


Figure 8-10 Spur Hole Site, modelled and measured frequency-depth/velocity class distributions for $Q = 1.27\text{m}^3/\text{s}$

From Figs 8-9 and 8-10 it can be concluded that River2D reproduces the measured class distributions reasonably well, with better performance for a discharge of $1.27\text{m}^3/\text{s}$.

As aquatic scientists interest is usually related to the whole area of a site rather than just at the surveyed cross-sections, modelled velocities and depths were extracted in a longitudinal and transverse grid of 0.5m (the same as used in the field for depth and velocity measurements) to show depth-velocity class distributions over the whole modelled river bed (from cross-section 1 to cross-section 5) of the site. Modelled depth/velocity class frequency distributions for the two discharges, ($Q = 0.343\text{m}^3/\text{s}$ and $1.27\text{m}^3/\text{s}$) are shown in Fig. 8-11. In further discussions these results will be referred to as “areal scale” results.

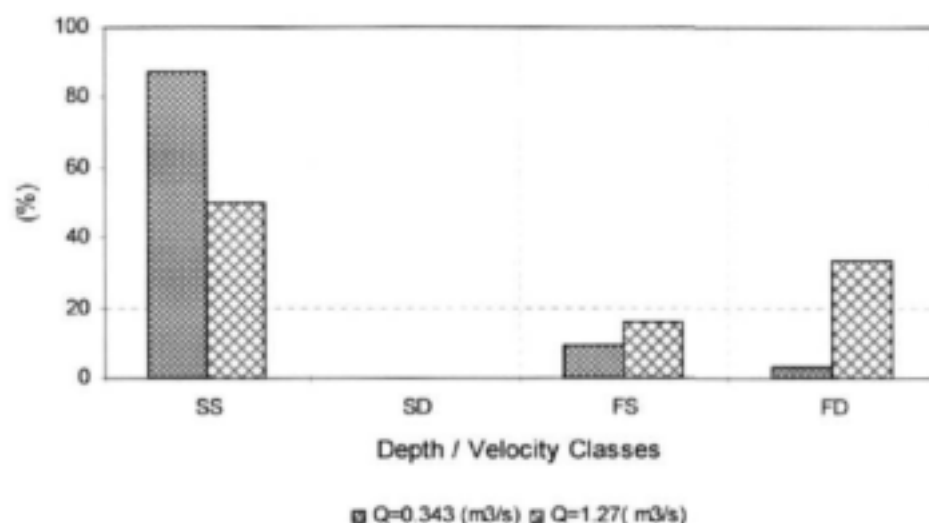


Figure 8-11 Spur Hole Site, modelled frequency-depth/velocity class distributions over the whole river bed for $Q = 0.343 \text{ m}^3/\text{s}$ and $1.27 \text{ m}^3/\text{s}$

From Fig. 8-11 it can be seen that the modelled frequency depth/velocity class distributions obtained at the areal scale are different from those related to the cross sectional scale (Figs 8-9 and 8-10). For a discharge of $0.343 \text{ m}^3/\text{s}$ two classes, slow-shallow (SS) and fast-shallow (FS) are represented in the cross sectional scale results (Fig.8-9) while with the areal scale results includes the fast-deep (FD) class as well, with 3.3% frequency. Comparison of the results for a discharge of $1.27 \text{ m}^3/\text{s}$, Figs 8-10 and 8-11, shows that the frequencies of slow-shallow (SS), fast-shallow (FS) and fast-deep (FD) classes are similar for both the cross-sectional and the areal scales, while frequency of slow-deep (SD) class being 2.3% in the cross-sectional scale (Fig. 8-10) decreased to 0.2% (too small to be seen in Fig. 8-11) in the areal scale. The differences between the two analyses are small, and it can therefore be noted that modelled results can be extracted at any resolution and scale depending on questions we have to answer.

8.2.5.3 Vanities Crossing Site Modelling and Results

The same approach was applied to modelled and measured data for the Vanities Crossing Site. All results at a cross sectional scale with discharges of $0.331 \text{ m}^3/\text{s}$ and $1.308 \text{ m}^3/\text{s}$ are presented in Figs 8-12, 8-13 and 8-16, and Figs 8-14, 8-15 and 8-17 respectively.

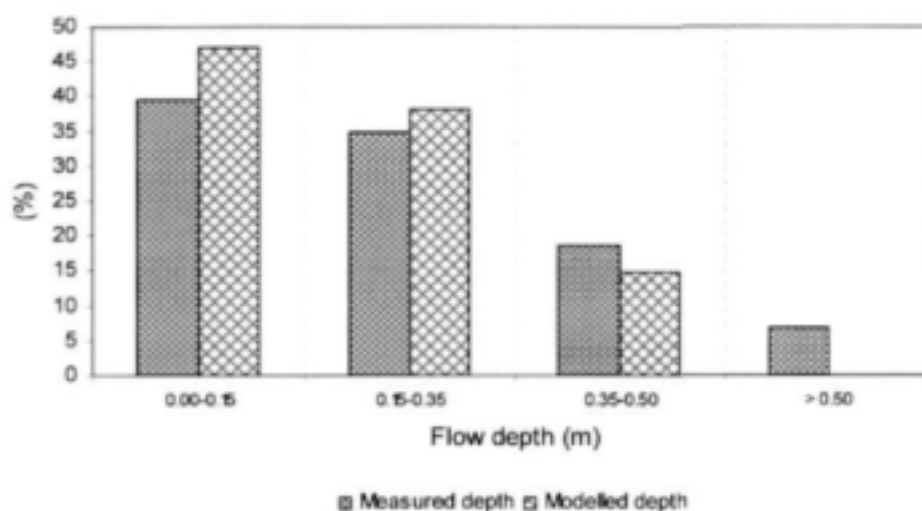


Figure 8-12 Vanities Crossing Site, modelled and measured frequency-depth distributions for $Q = 0.331 \text{ m}^3/\text{s}$

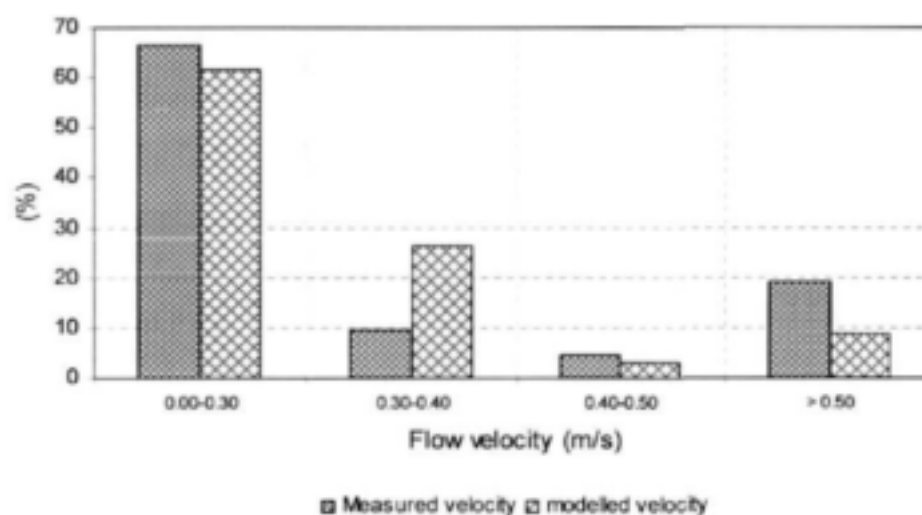


Figure 8-13 Vanities Crossing Site, modelled and measured frequency-velocity distributions for $Q = 0.331 \text{ m}^3/\text{s}$

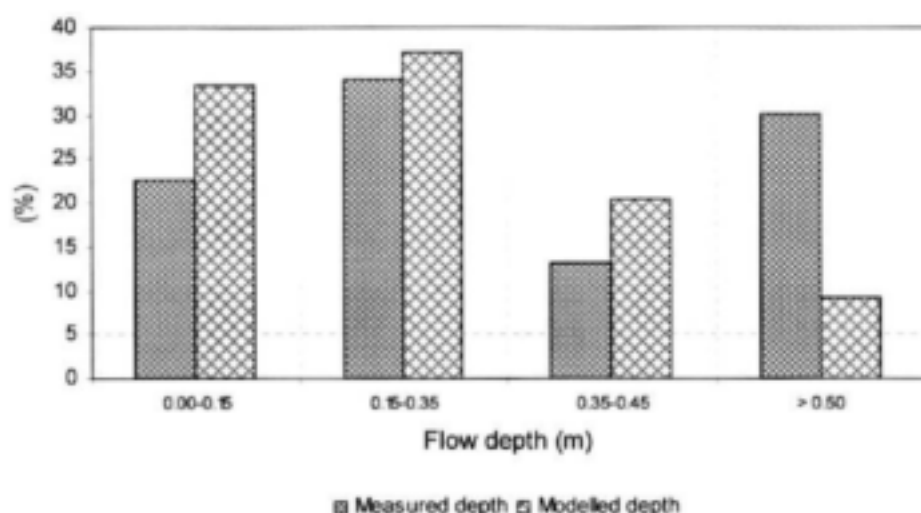


Figure 8-14 Vanities Crossing Site, modelled and measured frequency-depth distributions for $Q = 1.308\text{m}^3/\text{s}$

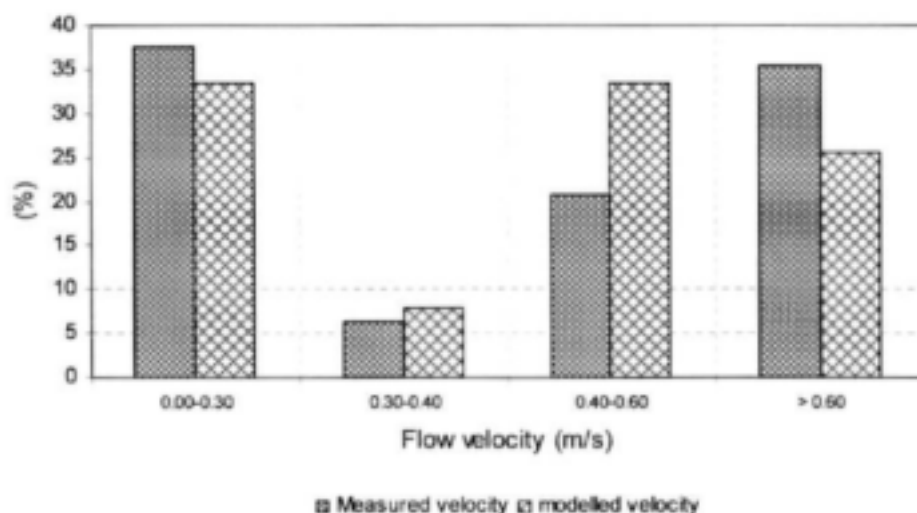


Figure 8-15 Vanities Crossing Site, modelled and measured frequency-velocity distributions for $Q = 1.308\text{m}^3/\text{s}$

From the modelled depth and velocity frequency distributions it can be seen that: Modelled results of the depth frequency distribution with discharges of $0.331\text{m}^3/\text{s}$ and $1.308\text{m}^3/\text{s}$ (Figs 8-12 and 8-14) reproduced the measured data well. For a discharge of $0.331\text{m}^3/\text{s}$ the modelled result is better for a depth less than 0.5m, while the class of flow depth ">0.5" is not reproduced. For a discharge of $1.308\text{m}^3/\text{s}$ all depth classes

appeared, showing that model performance is better for the higher discharge than for the lower one.

Modelled results of the velocity frequency distribution for the two discharges show good agreement with the measured data (Figs 8-13 and 8-15). All four classes appear in the modelled distributions. The velocity class scale is very coarse, (smaller or bigger than 0.3 m/s) therefore it can be seen (Figs 8-13 and 8-15) that modelled and measured frequency velocity distributions at this scale show reasonable correlation.

Measured and modelled depths and velocities were combined into depth/velocity classes for discharges of $0.331\text{m}^3/\text{s}$ and $1.308\text{m}^3/\text{s}$, and are plotted in frequency distribution form in Figs.8-16 and 8-17 respectively.

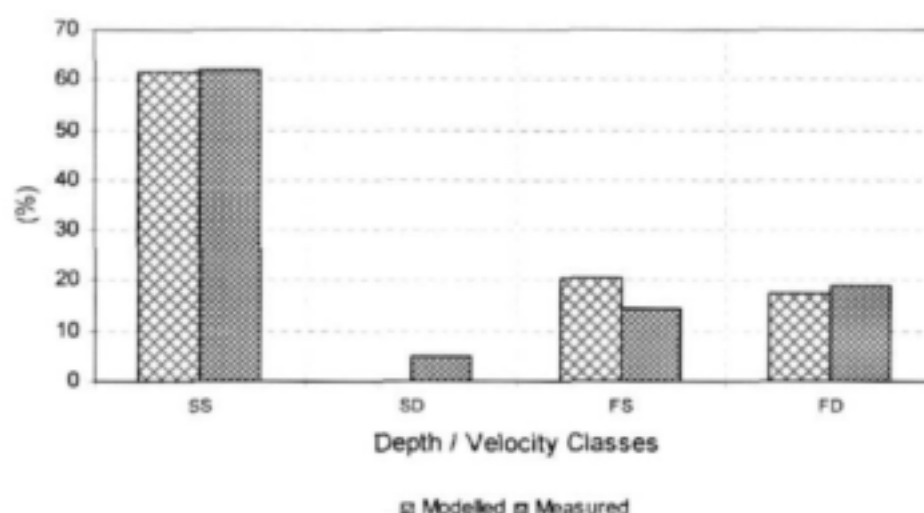


Figure 8-16 Vanities Crossing Site, modelled and measured frequency-depth/velocity class distributions for $Q = 0.331\text{m}^3/\text{s}$

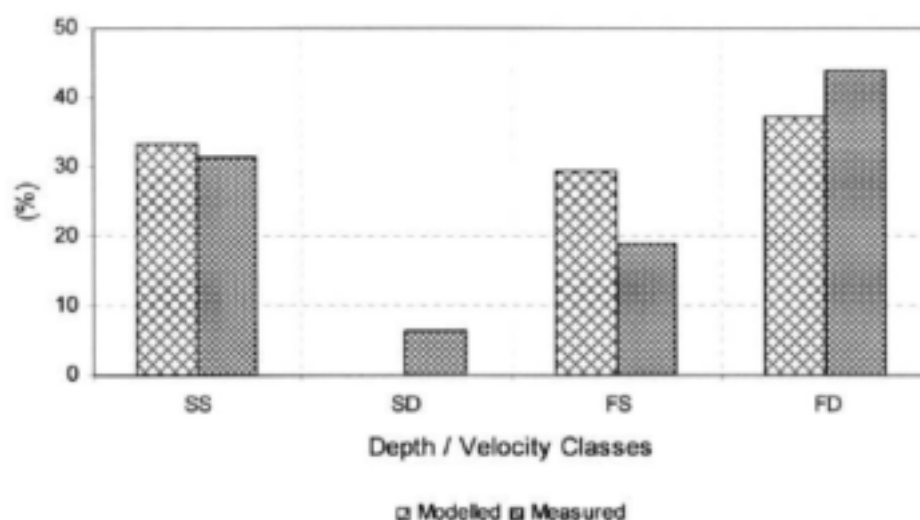


Figure 8-17 Vanities Crossing Site, modelled and measured frequency-depth/velocity class distributions for $Q = 1.308 \text{ m}^3/\text{s}$

The modelled depth/velocity class frequency distributions show good agreement with the measured ones for slow-shallow (SS), fast-shallow (FS) and fast-deep (FD) classes (Figs 8-16 and 8-17). However, the slow-deep class is not represented in the modelling results.

Modelled result of depth/velocity class frequency distribution at the areal scale with discharges of $0.331 \text{ m}^3/\text{s}$ and $1.308 \text{ m}^3/\text{s}$ are shown in Fig. 8-18.

Comparing the cross sectional scale modelling results (Figs 8-16 and 8-17) with the areal scale one (Fig. 8-18) shows that modelled depth/velocity class frequency distributions are similar for both scales. It also can be noted that a small frequency of 0.5% for the slow-deep (SD) class is represented for a discharge of $0.331 \text{ m}^3/\text{s}$.

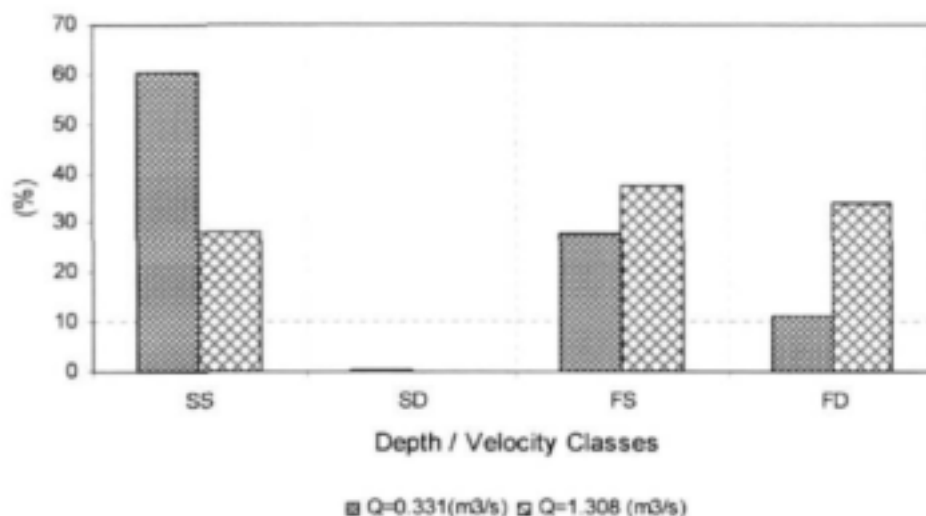


Figure 8-18 Vanities Crossing Site, modelled frequency depth/velocity class distribution in areal scale for $Q=0.331$ and $1.308 \text{ m}^3/\text{s}$

8.2.5.4 One Cross-Section Approach

In the previous sections, the River2D modelling was performed using 5 surveyed cross-sections for each site. In South Africa, time allocated for field work depends on the level at which a Reserve determination is to be done. For rapid and intermediate levels, the time for a river survey is restricted, and insufficient to survey as many cross-sections as are needed for confident hydraulic modelling. Often, one cross-section with a longitudinal slope is used in hydraulic modelling. In this section, an experiment to evaluate the use of only one cross-section in River2D modelling is discussed. This modelling will be referred to as the “one cross-section” approach.

Cross-section 3 of the Spur Hole site was chosen to be tested for the one cross-section approach. Survey data of cross-section 3, together with two additional cross-sections, one 25 m down stream and the other 35 m up stream, were linked in a “xyz” coordinate system. The values of the bed roughness (k_s) for all surveyed points were kept the same as in the cross-section scale modelling. The modelled bed topography and bed roughness data were saved in a text file. A mesh of 0.3m size was used. The upstream boundary condition was specified as a discharge. The water surface elevation at cross-section 3 was extended 25m downstream using the water slope for a given discharge, and this water surface elevation was set as the downstream boundary condition.

Modelling was performed with two discharges, $Q = 0.343 \text{ m}^3/\text{s}$ and $1.27 \text{ m}^3/\text{s}$. Modelled flow depths and velocities were extracted for cross-section 3 and compared

with the measured values. Modelled and measured cross-sectional flow depths with a discharge of $0.343\text{m}^3/\text{s}$ are shown in Fig. 8-19. It can be seen that model reproduced the measured depths reasonably well.

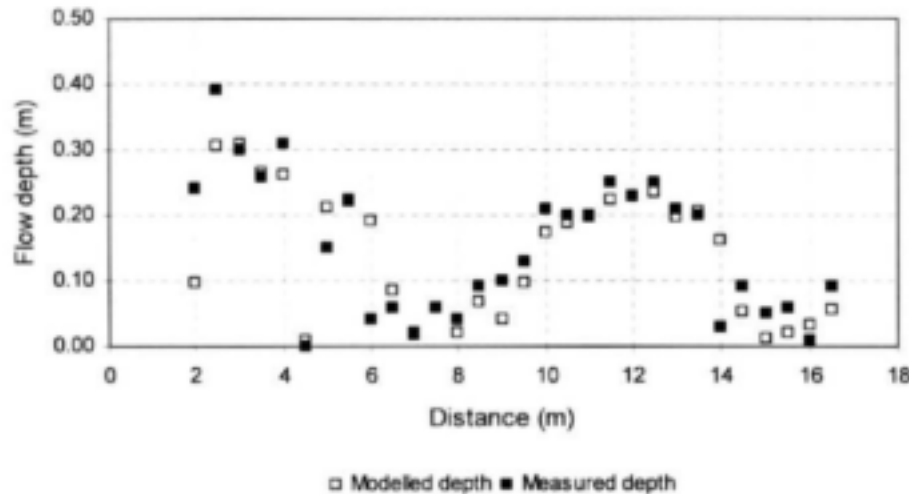


Figure 8-19 Spur Hole Site, modelled and measured cross-sectional flow depth distribution for cross section 3 with $Q = 0.343\text{m}^3/\text{s}$

Modelled and measured flow velocities with a discharge of $0.343\text{m}^3/\text{s}$ are shown in Fig. 8-20. Modelled velocities at distance 2 m to 5 m are very different to the measured velocities. This difference can be explained by limited river bed topography information, resulting from the use of data for one cross-section only. The photograph of the Spur Hole Site (Fig. 8-1) shows clearly a lot of substrate features upstream and downstream of the cross-section that would influence flow velocity.

Modelled and measured results of depth/velocity class frequency distributions with a discharge of $0.34\text{m}^3/\text{s}$ are shown in Fig. 8-21. Only the slow-shallow (SS) depth/velocity class was predicted to occur.

Modelling was also performed for a discharge of $1.27\text{m}^3/\text{s}$, and the results are shown in Figs 8-22, 8-23 and 8-24. The results showed better agreement between modelled and measured flow depths than for flow velocities.

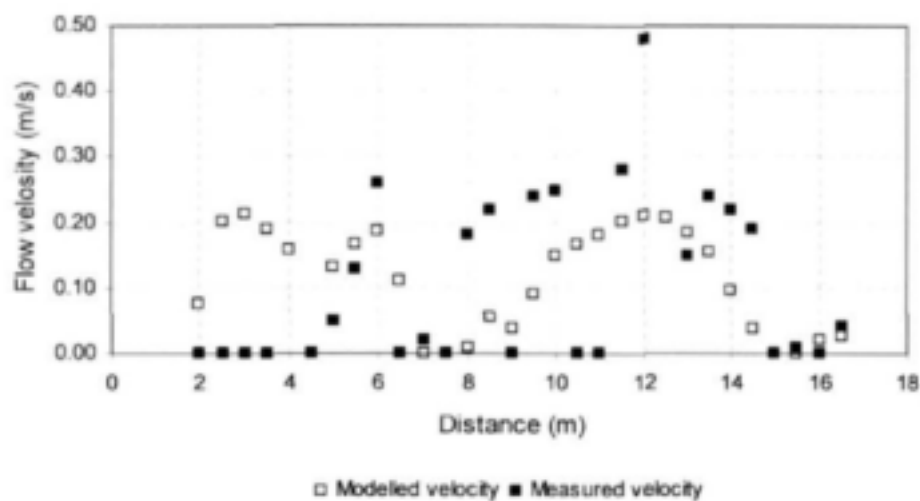


Figure 8-20 Spur Hole Site, modelled and measured cross-sectional velocity distribution for cross-section 3 with $Q = 0.343 \text{ m}^3/\text{s}$

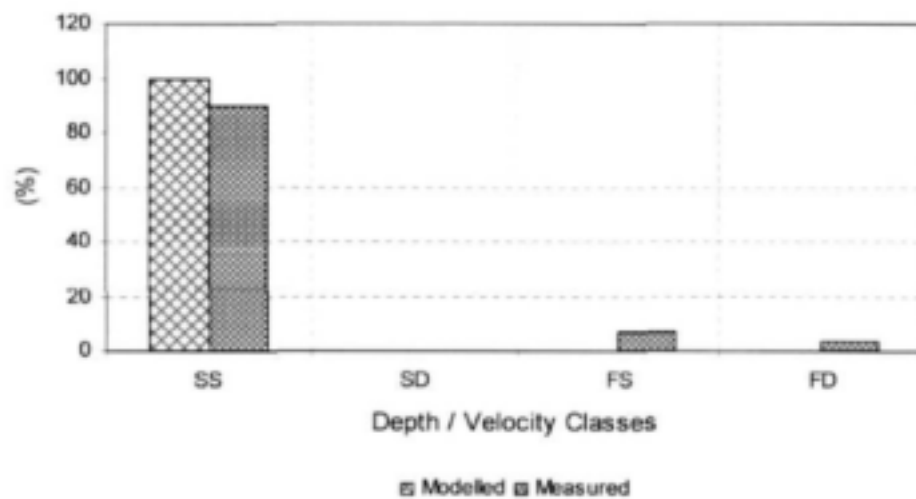


Figure 8-21 Spur Hole Site, modelled and measured depth/velocity classes for cross-section 3 with $Q = 0.343 \text{ m}^3/\text{s}$

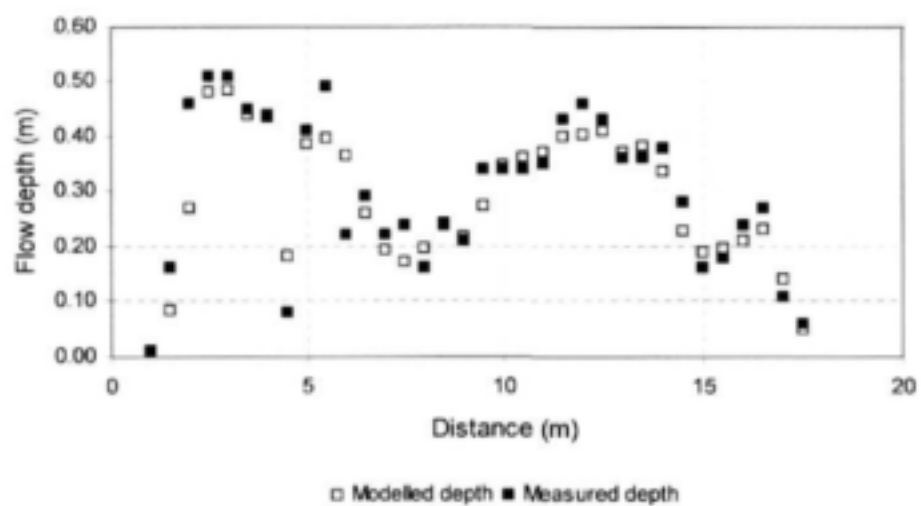


Figure 8-22 Spur Hope Site, modelled and measured cross sectional depth distribution for cross-section 3 with $Q = 1.27\text{m}^3/\text{s}$

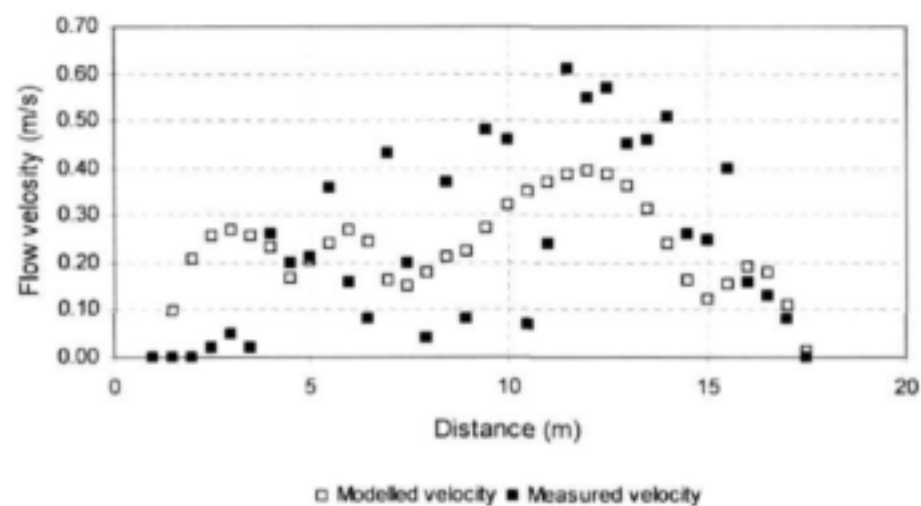


Figure 8-23 Spur Hole Site, modelled and measured cross sectional velocity distribution for cross-section 3 with $Q = 1.27\text{m}^3/\text{s}$

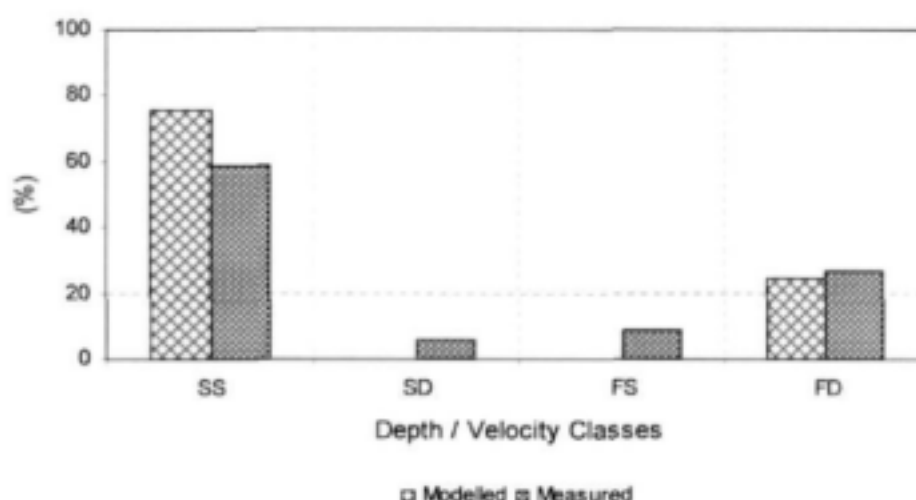


Figure 8-24 Spur Hole Site, modelled and measured depth/velocity class frequency distributions for cross-section 3 with $Q = 1.27\text{m}^3/\text{s}$

From the above it can be seen that the one cross-section approach can be used if appropriate field data are collected. It is important to note that the survey of a river site for River2D modelling must be performed by somebody who has hydraulic and River2D knowledge and expertise, somebody who understands low flow hydraulics phenomena, and who can identify river bed features (such as vegetation patches, big isolated boulders or rocks) that will influence depth and velocity distributions. Such features must be incorporated in the riverbed topography description, and upstream and downstream controls must be correctly identified and included in River2D modelling.

8.3 Driehoeks River Modelling

8.3.1 Introduction

The Driehoeks River is being studied in Water Research Commission Project K5/1403: Multi-scale Habitat Use and Movement of Freshwater Fish Species in a Large River System: Implications for Dam Placement, Operation and Design. The river has also been selected as a case study in WRC project K5/1508: Eco-Hydraulic Modelling in River Systems. A map of the catchment of the Olifants and Doring Rivers, showing the location of the Driehoeks River is presented in Fig. 8-25.



Figure 8-25 The Olifants and Doring Rivers catchment and location of the Driehoeks River

8.3.2 Field Data

A site selection and data collection field trip was undertaken together with a member of the research team of Project K5/1405 from 30 June to 11 July 2005. Five sites were selected, named Sites 1, 2, A, B and D, surveys of bed topography and sediment type were carried out, and comprehensive hydraulic data (845 measurements of flow depth and flow velocity) were collected. These data form part of the set compiled for Project K5/1508 and will be included in that project's report. Two further field trips were carried out and comprehensive velocity and depth data were obtained. In this report, only Site A has been used to verify River2D performance. The complexity of this site can be seen in Fig. 8-26.



Figure 8-26 Driehoeks River Site A

8.3.3 River2D Modelling

River2D modelling was performed with $Q = 0.13 \text{ m}^3/\text{s}$. The topography of the river bed was modelled from the survey data. A mesh of 0.5m size was set up over the whole bed. The bed roughness was represented by $k_s = 0.1 \text{ m}$, while all vegetation patches were modelled with a higher k_s value of 0.6 m. Modelled flow depths and velocities are presented in Figs 8-27 and 8-28 respectively. Measured and modelled velocities in histogram form are presented in Fig. 8-29. It can be seen (Fig. 8-29) that modelled and measured frequency-velocity distributions are very similar, confirming that River2D can be used for rivers under low flow conditions.

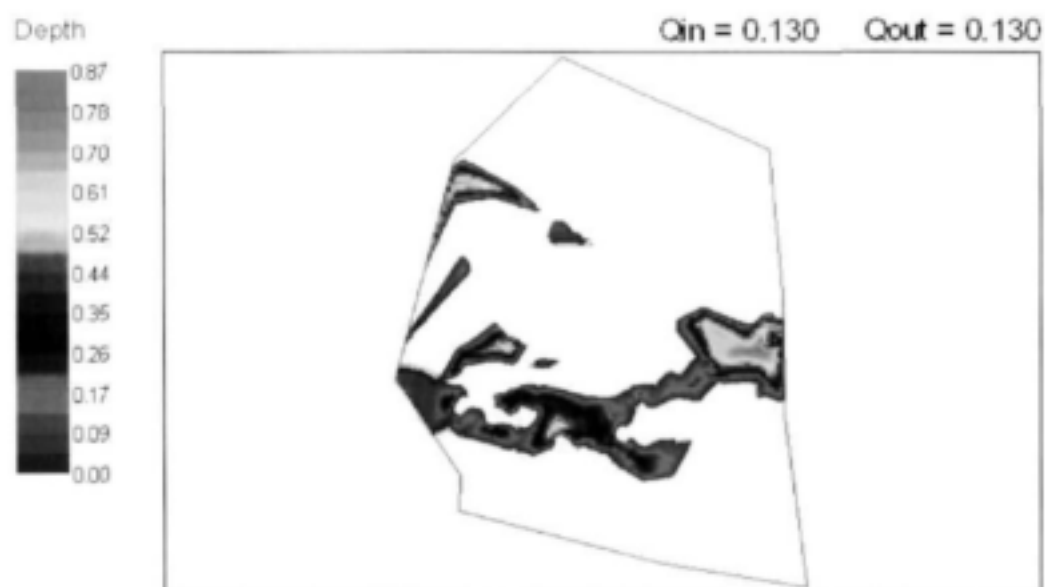


Figure 8-27 River2D predicted flow depths at Driehoeks River Site A with $Q = 0.13\text{m}^3/\text{s}$

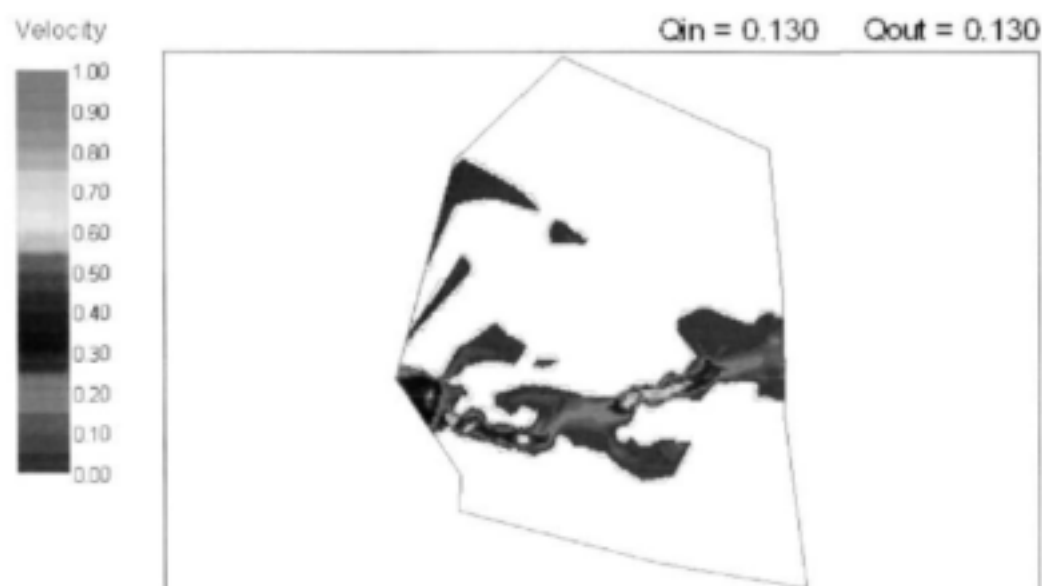


Figure 8-28 River2D predicted flow velocities at Driehoeks River Site A with $Q = 0.13\text{m}^3/\text{s}$

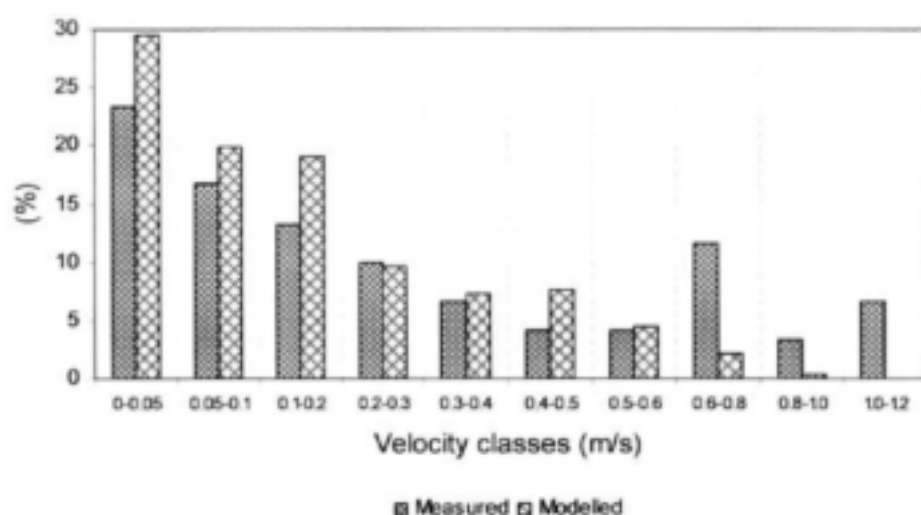


Figure 8-29 Measured and modelled velocity frequency distribution for Driehoeks River Site A for $Q = 0.13 \text{ m}^3/\text{s}$

More detailed modelling of the Driehoeks River sites is part of WRC Project K5/1508 and will be included in that project report (Hirschowitz et al., in preparation).

8.4 Conclusions and Recommendations

River2D modelling was performed for two sites, Spur Hole and Vanities Crossing, of the Cotter River in Australia and one site, Site A, of the Driehoeks River in South Africa. The sites are very different in term of dominant river-bed substratum containing wide range of river-bed features such as big rocks and vegetated patches. Modelled results of all sites showed good agreement with measured data. River2D output results can be seen in different type of plots as well as can be extracted at different resolutions in Excel format. Modelling success depends on careful, interpretive surveys of river sites, which must be done by a hydraulics expert with knowledge and experience in River2D modelling. Correct identification of the river bed features (such as vegetation patches, big isolated boulders or rocks) that influence depth and velocity distributions, and the relevant upstream and downstream controls, requires intuitive understanding of low flow hydraulics phenomena.

9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

This project aimed to develop appropriate methods for describing the hydraulic characteristics of South African rivers under conditions of low discharge, and the influence of vegetation and large bed roughness.

Conventionally, flow resistance in rivers is described using equations (such as those of Chézy, Darcy-Weisbach and Manning) that implicitly assume the dominant resistance phenomenon to be boundary shear stress. Such equations are inherently unsatisfactory for low flow conditions, where the size of roughness elements is comparable to the flow depth and resistance is dominated by form drag. An experimental investigation was carried out (Chapter 3), and a new method for predicting flow resistance under conditions of large-scale roughness when the flow depth is less than the height of the roughness elements is proposed (Chapter 4). Two methods for predicting flow resistance under conditions of intermediate-scale roughness when the flow depth is between one and four times the height of the roughness elements, combining both roughness element drag and boundary friction effects, are proposed. The new methods present an advance on current practice by explicitly accounting for the distinctly different resistance phenomena encountered under small- and large-scale roughness conditions. Their rational basis gives them greater generality and reduces reliance on case-specific calibration of resistance coefficients using inappropriate equation forms.

Conventional resistance equations are also not applicable for vegetated channels. Alternative practical conveyance prediction methods are proposed for three situations pertaining to the occurrence of vegetation in rivers and wetlands, representing three different levels of complexity, viz. flow through extensive emergent vegetation, flow in channels with emergent vegetation boundaries, and flow in channels with discrete vegetation patches (Chapter 7).

Flow through extensive and effectively uniform emergent vegetation is the simplest situation, and requires the simplest solution – application of a single resistance equation. A practical equation is proposed for emergent, reed-type vegetation, based on research carried out in the earlier WRC project (K5/856) (James et al., 2001). This equation has a different form from the conventional resistance equations, reflecting the dominant role of stem drag rather than boundary shear stress. The appropriate resistance coefficient is consequently less variable with situation and flow condition than the force-fitted values required when using the conventional equations.

Flow in channels with emergent vegetated banks represents the next level of complexity. The effect on flow resistance of transverse fragmentation of vegetation into longitudinal strips was demonstrated and elucidated in Project K5/856 (James et al., 2001). This situation cannot be described by a single equation, and a zonal approach is proposed where conveyances in the vegetated and unvegetated zones are calculated separately using appropriate equations. The effect of the interaction between the zones on the clear channel flow is accounted for by well-established composite roughness equations, requiring specification of a resistance coefficient for the interface. This approach has been confirmed by application to field data from the main channel of the Okavango delta and shown to be practical, reliable and simple.

The third level of complexity occurs when vegetation is fragmented longitudinally as well as transversely. It was shown in Project K5/856 (James et al., 2001) that the distribution pattern of vegetation patches, as well as the overall areal coverage, has a significant influence on flow resistance. This situation cannot be described by hand-calculation based methods, and recourse must be made to computational modelling. The two-dimensional model River2D was tested against the extensive laboratory data on flow in channels with fragmented vegetation patches that were obtained in Project K5/856. It is concluded that a river bed with vegetation patches can be modelled by River2D, and vegetation patches can be represented either as areas of extreme roughness or zones of flow through a porous medium. Owing to limitations of the software in specification of roughness values, the porous medium representation was more successful in these applications although it is less effective in describing velocity distributions.

The spatial distributions of local flow depth and velocity in a river reach are important determinants of habitat suitability. It is shown (Chapter 5) that the distributions are highly dependent on discharge and whether the flow is controlled by general distributed resistance or multiple local occurrences of critical flow, with greater diversity produced by the multiple local control condition. The use of statistical descriptions based on field data must therefore be used with caution and cognizance of their sensitivity to channel bed geometry and flow conditions. The statistical representation of distributions is also sensitive to the scale and resolution of both the collection and presentation of data. Computational modelling provides a potentially more reliable approach.

Application of River2D to laboratory and field data (Chapters 6 and 8) shows that complex bed geometry can be modelled by R2D_Bed. The River2D model can describe multiple local control conditions realistically, and trans-critical flows in particular. River2D can predict velocity frequency distributions reliably under large-scale roughness conditions.

9.2 Recommendations for further research

The project has produced methods for predicting low flow conditions in rivers that are complete and usable. As with all existing methods that rely wholly or partly on empiricism, their generality and reliability could be enhanced by extending the data base underlying the empirical content. Most of the relationships presented in this report are based on laboratory data, and further field confirmation would be valuable.

Flow in rivers with large roughness elements has been inadequately studied in the field, except for steep mountain streams which present rather different hydraulic phenomena to those in the riffle situations that are important in South African applications. Field-based investigations would be valuable to test and refine the approaches presented for predicting resistance under intermediate- and large-scale conditions. In particular, it would be useful to be able to recommend a percentile bed material size to define the roughness size, and to assess the contribution of the relatively small material between the largest elements on overall resistance. Surprisingly, even relationships for resistance in terms of bed characteristics under small-scale roughness conditions are limited and unreliable; a reassessment of currently used equations and a detailed laboratory and field study of appropriate coefficients would be valuable.

The method for predicting vegetation resistance depends on knowledge of the stem drag coefficient. It would be useful to carry out extensive measurements to relate the coefficient directly to vegetation morphological characteristics. Further field data for quantifying the resistance coefficient of vegetated bank boundaries directly, or developing a method for its estimation would be valuable.

Most guidelines for estimating resistance coefficients do so in terms of equations (such as those of Chézy, Darcy-Weisbach and Manning) that implicitly assume boundary shear to be the dominant resisting influence. In many situations, particularly with low flows, the resistance is dominated by form drag, and the equation form presented for vegetation and the large-scale roughness condition is more appropriate. Reinterpretation of existing guidelines in terms of this form would reduce the uncertainty of resistance coefficient estimation significantly.

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APPENDIX A

Table A-1 Flume B experimental data

Flume B

Series 1 Experiments

	radius (m)	height (m)
Hemisphere	0.047	0.029

Test	Q (cumecs)	y_measured (m)	Covered area	y/h	Slope
1_1.1	0.0152	0.178	0.82	6.1	0.0011
1_1.2	0.0094	0.133	0.82	4.6	0.0011
1_1.3	0.0053	0.105	0.82	3.6	0.0011
1_1.4	0.0039	0.092	0.82	3.2	0.0011
1_1.5	0.0018	0.070	0.82	2.4	0.0011
1_1.6	0.0006	0.050	0.82	1.7	0.0011
1_1.7	0.0004	0.042	0.82	1.4	0.0011
1_2.1	0.0055	0.083	0.82	2.9	0.0021
1_2.2	0.0022	0.061	0.82	2.1	0.0021
1_2.3	0.0004	0.038	0.82	1.3	0.0021
1_2.4	0.0109	0.117	0.82	4.0	0.0021
1_2.5	0.0152	0.137	0.82	4.7	0.0021
1_3.1	0.0014	0.066	0.47	2.3	0.0011
1_3.2	0.0033	0.094	0.47	3.2	0.0011
1_3.3	0.0089	0.147	0.47	5.1	0.0011
1_3.4	0.0148	0.187	0.47	6.4	0.0011
1_3.5	0.0042	0.104	0.47	3.6	0.0011
1_4.1	0.0009	0.054	0.30	1.8	0.0011
1_4.2	0.0043	0.103	0.30	3.6	0.0011
1_4.3	0.0096	0.153	0.30	5.3	0.0011
1_4.4	0.0159	0.199	0.30	6.8	0.0011
1_5.1	0.0139	0.180	0.22	6.2	0.0011
1_5.2	0.0091	0.142	0.22	4.9	0.0011
1_5.3	0.0040	0.094	0.22	3.2	0.0011
1_5.4	0.0004	0.037	0.22	1.3	0.0011

Appendix A

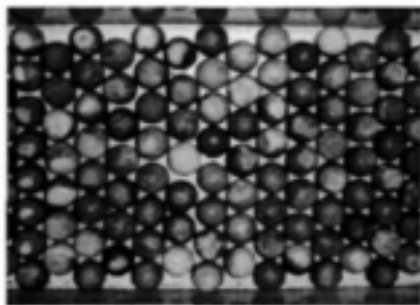


Figure A-1 Series 1, Tests 1 and 2

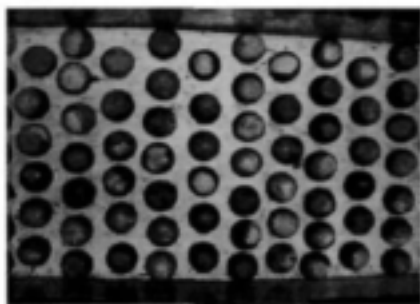


Figure A-2 Series 1, Test 3

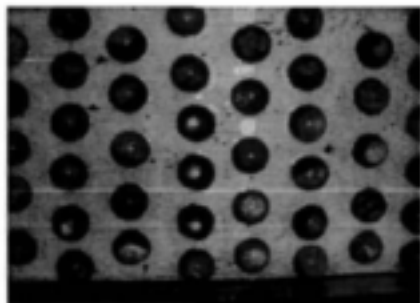


Figure A-3 Series 1, Test 4

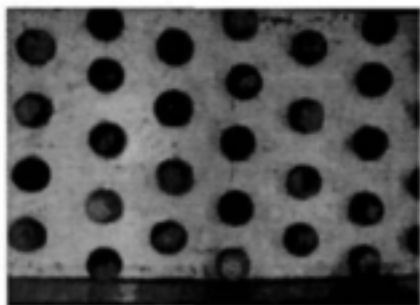


Figure A-4 Series 1, Test 5

APPENDIX B

Table B-1 Flume C, Series 2.1 experiments

Flume C		
Series 2.1 Experiments		
Bed Slope	0.001	
	radius	height
Big Hemisphere		
(BH)	0.058	0.062
Small Hemisphere		
(SH)	0.027	0.0286

Patterns	Q (cumecs)	y_measured (m)	No BH/m	No SH/m	y/h (BH)	y/h (SH)
Pattern 1 (BH)	0.0027	0.025	28	0	0.40	0
	0.0042	0.036	28	0	0.58	0
	0.0069	0.048	28	0	0.77	0
	0.0095	0.055	28	0	0.89	0
	0.0119	0.064	28	0	1.03	0
	0.0174	0.074	28	0	1.19	0
Pattern 2 (BH+SH)	0.0012	0.020	28	26	0.32	0.70
	0.0018	0.026	28	26	0.42	0.91
	0.0026	0.031	28	26	0.50	1.08
	0.0043	0.042	28	26	0.68	1.47
	0.0093	0.062	28	26	1.00	2.17
	0.0125	0.069	28	26	1.11	2.41
Pattern 3 (BH+SH)	0.0029	0.028	28	20	0.45	0.98
	0.0045	0.035	28	20	0.56	1.22
	0.0066	0.045	28	20	0.73	1.57
	0.0098	0.061	28	20	0.98	2.13
	0.0145	0.073	28	20	1.18	2.55
	0.0214	0.080	28	20	1.29	2.80
Pattern 4 (BH+SH)	0.0020	0.020	28	18	0.32	0.70
	0.0035	0.030	28	18	0.48	1.05
	0.0076	0.045	28	18	0.73	1.57
	0.0102	0.055	28	18	0.89	1.92
	0.0149	0.070	28	18	1.13	2.45
	0.0257	0.085	28	18	1.37	2.97
Pattern 5 (BH+SH)	0.0018	0.020	28	24	0.32	0.70
	0.0028	0.026	28	24	0.42	0.91

Appendix B

Patterns	Q (cumecs)	y_measured (m)	No BH/m	No SH/m	y/h (BH)	y/h (SH)
	0.0075	0.047	28	24	0.76	1.64
	0.0145	0.069	28	24	1.11	2.41
	0.0200	0.075	28	24	1.21	2.62
	0.0214	0.081	28	24	1.31	2.83
	0.0247	0.082	28	24	1.32	2.87
Pattern 6 (BH)	0.0032	0.025	28	0	0.40	0
	0.0075	0.043	28	0	0.69	0
	0.0138	0.065	28	0	1.05	0
	0.0220	0.080	28	0	1.29	0
	0.0273	0.085	28	0	1.37	0
Pattern 7 (SH)	0.0070	0.029	0	28	0	1.01
	0.0100	0.035	0	28	0	1.22
	0.0129	0.039	0	28	0	1.36
	0.0171	0.043	0	28	0	1.50
	0.0217	0.050	0	28	0	1.75
	0.0276	0.055	0	28	0	1.92
Pattern 8 (SH)	0.0053	0.025	0	26	0	0.87
	0.0088	0.032	0	26	0	1.12
	0.0145	0.040	0	26	0	1.40
	0.0238	0.052	0	26	0	1.82
	0.0283	0.056	0	26	0	1.96



Figure B-1 Pattern 1

Appendix B



Figure B-2 Pattern 2



Figure B-3 Pattern 3

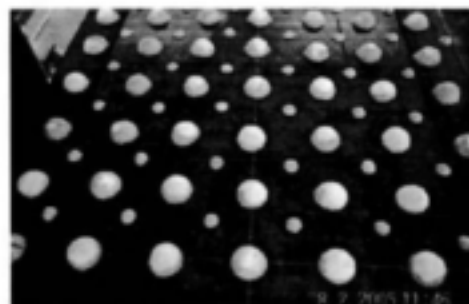


Figure B-4 Pattern 4

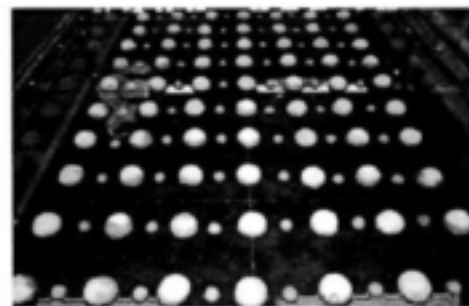


Figure B-5 Pattern 5

Appendix B

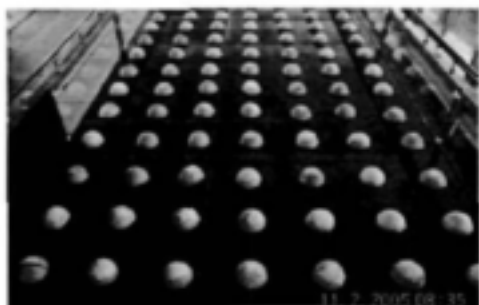


Figure B-6 Pattern 6



Figure B-7 Pattern 7



Figure B-8 Pattern 8

APPENDIX C

Table C-1 Flume C Series 2.2 experiments

Flume C Experiments

Series 2.2 Experiments

Bed Slope	H1_radius	H1_height	H2_radius	H2_height	H3_radius	H3_height
0.0005	0.054	0.054	0.036	0.036	0.023	0.023

Spacing (m)	Q (cumecs)	y_measured (m)	No H1/m	No H2/m	No H3/m	y/h (H1)	y/h (H2)	y/h (H3)
Pattern 1	0.0006	0.0319	116	0	0	0.59	0	0
H1	0.0012	0.0495	116	0	0	0.92	0	0
spacing 0.125m	0.0016	0.0552	116	0	0	1.02	0	0
	0.0029	0.0643	116	0	0	1.19	0	0
	0.0119	0.1012	116	0	0	1.87	0	0
	0.0311	0.1452	116	0	0	2.69	0	0
	0.0417	0.1698	116	0	0	3.14	0	0
	0.0557	0.1894	116	0	0	3.51	0	0
Pattern 2	0.0011	0.024	47.5	0	0	0.45	0	0
H1	0.0016	0.035	47.5	0	0	0.64	0	0
spacing 0.200m	0.0019	0.039	47.5	0	0	0.72	0	0
	0.0033	0.055	47.5	0	0	1.01	0	0
	0.0042	0.058	47.5	0	0	1.07	0	0
	0.0071	0.066	47.5	0	0	1.22	0	0
	0.0187	0.093	47.5	0	0	1.72	0	0
	0.0206	0.099	47.5	0	0	1.83	0	0
	0.0344	0.126	47.5	0	0	2.33	0	0
	0.0475	0.148	47.5	0	0	2.73	0	0
	0.0547	0.164	47.5	0	0	3.04	0	0
Pattern 3	0.0552	0.129	26	0	0	2.39	0	0
H1	0.0468	0.122	26	0	0	2.25	0	0
spacing 0.300m	0.0246	0.091	26	0	0	1.68	0	0
	0.0160	0.071	26	0	0	1.31	0	0
	0.0137	0.067	26	0	0	1.24	0	0
	0.0082	0.059	26	0	0	1.08	0	0
	0.0050	0.044	26	0	0	0.81	0	0
	0.0034	0.034	26	0	0	0.62	0	0
	0.0020	0.023	26	0	0	0.42	0	0
Pattern 4	0.0552	0.121	14	0	0	2.24	0	0
H1	0.0333	0.092	14	0	0	1.71	0	0

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spacing 0.400m	0.0140	0.060	14	0	0	1.10	0	0
	0.0072	0.046	14	0	0	0.86	0	0
	0.0027	0.023	14	0	0	0.43	0	0
Pattern 5	0.0552	0.107	0	14	0	0	2.98	0
H2	0.0322	0.080	0	14	0	0	2.21	0
spacing 0.400m	0.0066	0.036	0	14	0	0	1.00	0
	0.0032	0.023	0	14	0	0	0.65	0
Pattern 6	0.06	0.13	0	47.5	0	0	3.52	0
H2	0.0360	0.100	0	47.5	0	0	2.78	0
spacing 0.200m	0.01	0.05	0	47.5	0	0	1.52	0
	0.0028	0.034	0	47.5	0	0	0.95	0
Pattern 7	0.0552	0.149	0	116	0	0	4.14	0
H2	0.0360	0.125	0	116	0	0	3.48	0
spacing 0.125m	0.0088	0.066	0	116	0	0	1.84	0
	0.0041	0.050	0	116	0	0	1.38	0
Pattern 8	0.0547	0.164	47.5	47.5	0	3.04	4.57	0
H1 and H2	0.0358	0.132	47.5	47.5	0	2.45	3.68	0
spacing 0.200m	0.0187	0.102	47.5	47.5	0	1.89	2.83	0
	0.0080	0.074	47.5	47.5	0	1.37	2.05	0
	0.0043	0.060	47.5	47.5	0	1.10	1.65	0
	0.0029	0.054	47.5	47.5	0	0.99	1.49	0
	0.0022	0.046	47.5	47.5	0	0.85	1.28	0
	0.0015	0.038	47.5	47.5	0	0.70	1.05	0
	0.0007	0.020	47.5	47.5	0	0.37	0.56	0
Pattern 9	0.0008	0.020	47.5	0	47.5	0.37	0	0.88
H1 and H3	0.0019	0.039	47.5	0	47.5	0.72	0	1.69
spacing 0.200m	0.0048	0.062	47.5	0	47.5	1.14	0	2.69
	0.0112	0.081	47.5	0	47.5	1.50	0	3.52
	0.0302	0.124	47.5	0	47.5	2.30	0	5.39
	0.0434	0.145	47.5	0	47.5	2.69	0	6.30
Pattern 10	0.0552	0.137	26	19.5	0	2.54	3.81	0
H1 and H2	0.0283	0.098	26	19.5	0	1.81	2.72	0
spacing 0.300m	0.0179	0.081	26	19.5	0	1.50	2.25	0
	0.0114	0.066	26	19.5	0	1.23	1.84	0
	0.0089	0.059	26	19.5	0	1.09	1.63	0
	0.0051	0.049	26	19.5	0	0.91	1.37	0
	0.0033	0.040	26	19.5	0	0.74	1.11	0
	0.0014	0.024	26	19.5	0	0.44	0.66	0
Pattern 11	0.0552	0.131	26	0	19.5	2.43	0	5.70
H1 and H3	0.0193	0.076	26	0	19.5	1.41	0	3.32

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spacing 0.300m	0.0121	0.066	26	0	19.5	1.22	0	2.87
	0.0052	0.047	26	0	19.5	0.87	0	2.03
	0.0028	0.033	26	0	19.5	0.61	0	1.42
	0.0020	0.026	26	0	19.5	0.47	0	1.11
Pattern 12 H1 and H2 spacing 0.400 (1)	0.0037	0.031	13.5	9	0	0.57	0.86	0
	0.0063	0.047	13.5	9	0	0.88	1.31	0
	0.0333	0.100	13.5	9	0	1.85	2.78	0
	0.0552	0.129	13.5	9	0	2.38	3.57	0
Pattern 13 H1 and H2 spacing 0.400 (2)	0.0552	0.131	13.5	9	0	2.43	3.64	0
	0.0316	0.100	13.5	9	0	1.84	2.76	0
	0.0070	0.053	13.5	9	0	0.98	1.46	0
	0.0045	0.040	13.5	9	0	0.75	1.12	0
	0.0025	0.027	13.5	9	0	0.51	0.76	0
Pattern 14 H3 spacing 0.200m	0.0557	0.136	0	47.5	47.5	0	3.78	5.92
	0.0348	0.104	0	47.5	47.5	0	2.90	4.53
	0.0073	0.052	0	47.5	47.5	0	1.44	2.26
	0.0036	0.036	0	47.5	47.5	0	1.01	1.58
Pattern 15 H1 spacing 0.110m	0.0004	0.043	162	0	0	0.78	0	0
	0.0018	0.059	162	0	0	1.07	0	0
	0.0015	0.058	162	0	0	1.06	0	0
	0.0011	0.054	162	0	0	0.99	0	0
	0.0033	0.068	162	0	0	1.23	0	0
	0.0103	0.098	162	0	0	1.78	0	0
	0.0250	0.127	162	0	0	2.30	0	0
	0.0012	0.055	162	0	0	1.00	0	0
Pattern 16 H1 spacing 0.110/0.125m	0.0016	0.057	136	0	0	1.04	0	0
	0.0012	0.052	136	0	0	0.95	0	0
	0.0006	0.04	136	0	0	0.74	0	0
	0.0099	0.09	136	0	0	1.64	0	0
	0.0214	0.117	136	0	0	2.13	0	0
	0.0014	0.055	136	0	0	1.00	0	0
Pattern 17 H1 spacing 0.110/0.167m	0.0009	0.041	90	0	0	0.74	0	0
	0.0006	0.031	90	0	0	0.56	0	0
	0.0018	0.055	90	0	0	0.99	0	0
	0.0078	0.082	90	0	0	1.48	0	0

Appendix C



Figure C-1 Pattern H 1 spacing 0.125 m



Figure C-2 Pattern H 1 spacing 0.200 m



Figure C-3 Pattern H 1 spacing 0.300 m

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Figure C-4 Pattern H 1 spacing 0.400 m

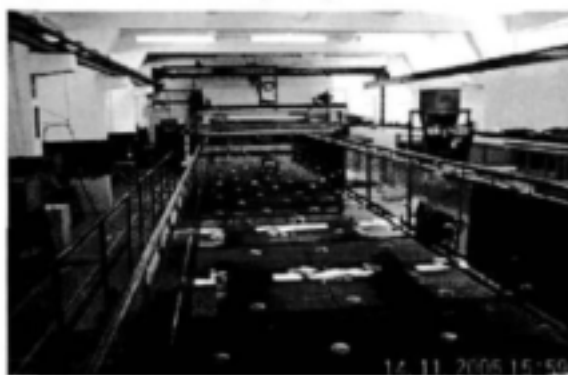


Figure C-5 Pattern H 2 spacing 0.400 m



Figure C-6 Pattern H 2 spacing 0.200 m

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Figure C-7 Pattern H 2 spacing 0.125



Figure C-8 Pattern H 1 and H 2 spacing 0.200 m

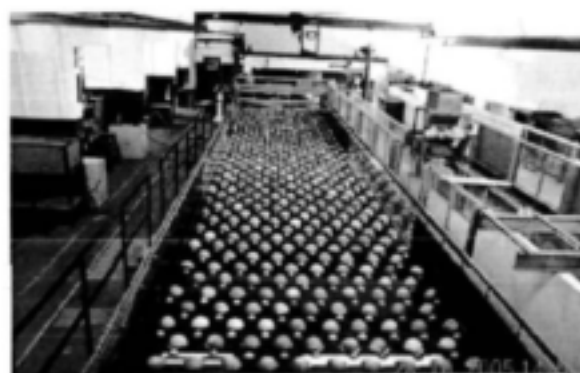


Figure C-9 Pattern H 1 and H 3 spacing 0.200 m

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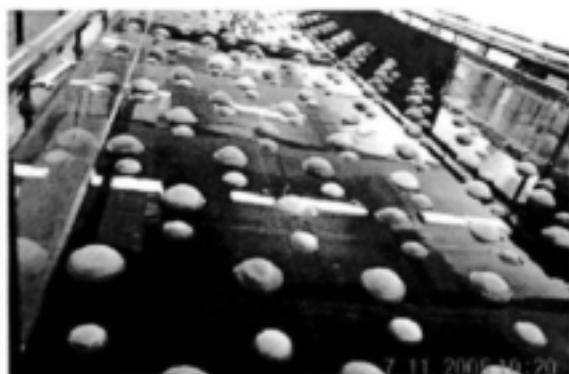


Figure C-10 Pattern H 1 and H 2 spacing 0.300 m

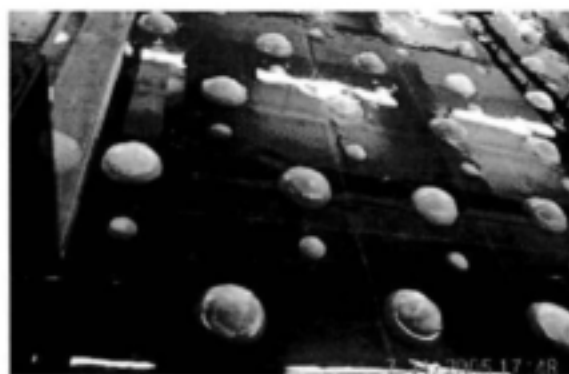


Figure C-11 Pattern H1 and H 3 spacing 0.300 m



Figure C-12 Pattern H1 and H 2 spacing 0.400 m

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Figure C-13 Pattern H 2 and H3 spacing 0.400 m

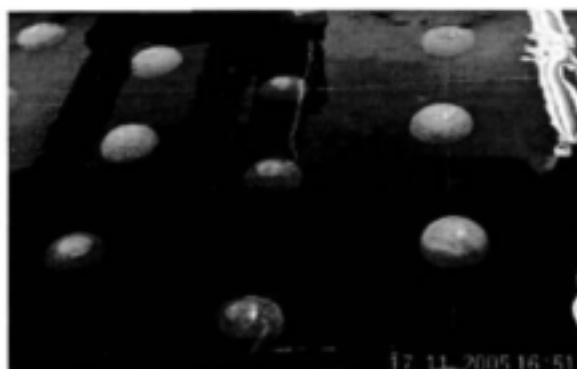


Figure C-14 Pattern H 3 spacing 0.200m



Figure C-15 Pattern H1 spacing 0.110m

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Figure C-16 Pattern H1 spacing cross-sectional 0.110m, longitudinal 0.125m



Figure C-17 Pattern H1 spacing cross-sectional 0.110m, longitudinal 0.167m

APPENDIX D

Velocity measurements around one hemisphere

Hemisphere	Spacing (m)	Uniform depth (m)	Slope	Discharge (m ³ /s)
H2	0.200	0.083	0.0005	0.0214

Table D-1 Measured velocities around a hemisphere (Q=3.0 l/s)

Measured point	x (m)	y (m)	vel (cm/s)	vel (m/s)
1.1	0.01	0.01	2.81	0.028
2.1	0.01	0.03	3.28	0.033
3.1	0.01	0.05	4.73	0.047
4.1	0.01	0.07	3.59	0.036
5.1	0.01	0.09	4.98	0.050
1.2	0.03	0.01	2.96	0.030
2.2	0.03	0.03	3.17	0.032
3.2	0.03	0.05	3.82	0.038
4.2	0.03	0.07	5.09	0.051
5.2	0.03	0.09	4.04	0.040
1.3	0.05	0.01	0.71	0.007
2.3	0.05	0.03	4.33	0.043
3.3	0.05	0.05	4.47	0.045
4.3	0.05	0.07	5.48	0.055
5.3	0.05	0.09	2.64	0.026
1.4	0.07	0.01	4.72	0.047
2.4	0.07	0.03	4.49	0.045
3.4	0.07	0.05	3.76	0.038
4.4	0.07	0.07	6.24	0.062
2.5	0.09	0.03	5.17	0.052
3.5	0.09	0.05	5.48	0.055
2.6	0.11	0.03	5.52	0.055
3.6	0.11	0.05	5.25	0.053
1.7	0.13	0.01	5.68	0.057
2.7	0.13	0.03	5.25	0.053
3.7	0.13	0.05	5.37	0.054
4.7	0.13	0.07	6.39	0.064
1.8	0.15	0.01	1.59	0.016
2.8	0.15	0.03	5.44	0.054
3.8	0.15	0.05	5.92	0.059
4.8	0.15	0.07	5.68	0.057
5.8	0.15	0.09	1.22	0.012
1.9	0.17	0.01	5.66	0.057
2.9	0.17	0.03	5.29	0.053
3.9	0.17	0.05	5.65	0.056
4.9	0.17	0.07	4.76	0.048
5.9	0.17	0.09	1.95	0.020
1.10	0.19	0.01	4.97	0.050
2.10	0.19	0.03	5.19	0.052
3.10	0.19	0.05	4.58	0.046
4.10	0.19	0.07	5.32	0.053
5.10	0.19	0.09	3.44	0.034

Appendix D

Table D-2 Measured velocities at longitudinal section 1

Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
Measured point 1.1					
0	0	0	0		
0.01	0.0552	-0.0083	0.0558		
0.02	0.0567	-0.0064	0.0571		
0.03	0.0826	-0.0020	0.0826		
0.04	0.1326	0.0031	0.1326		
0.05	0.1533	0.0046	0.1534		
0.06	0.1589	0.0053	0.1590		
0.07	0.1645	0.0077	0.1647		
0.083	0.1701		0.1704		
				0.1120	0.1122
Measured point 1.2					
0	0	0	0		
0.01	0.0194	-0.0031	0.0196		
0.02	0.0177	-0.0022	0.0178		
0.03	0.0993	-0.0021	0.0993		
0.04	0.1261	0.0021	0.1261		
0.05	0.1458	0.0037	0.1458		
0.06	0.1521	0.0068	0.1523		
0.07	0.1676	0.0082	0.1678		
0.083	0.1831		0.1833		
				0.1043	0.1044
Measured point 1.3					
0	0	0	0		
0.01	0.0744	0.0058	0.0746		
0.02	0.0790	-0.0014	0.0790		
0.03	0.1075	0.0022	0.1075		
0.04	0.1258	0.0008	0.1258		
0.05	0.1465	0.0032	0.1465		
0.06	0.1576	0.0072	0.1578		
0.07	0.1596	0.0052	0.1597		
0.083	0.1616		0.1616		
				0.1183	0.1183
Measured point 1.4					
0	0	0	0		
0.01	0.0912	0.0019	0.0912		
0.02	0.1054	-0.0035	0.1055		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.03	0.1150	-0.0013	0.1150		
0.04	0.1375	0.0019	0.1375		
0.05	0.1510	0.0046	0.1511		
0.06	0.1557	0.0054	0.1558		
0.07	0.1650	0.0072	0.1652		
0.083	0.1743		0.1745		
				0.1275	0.1276
Measured point 1.5					
0	0	0	0		
0.01	0.1059	0.0079	0.1062		
0.02	0.1106	0.0021	0.1106		
0.03	0.1244	0.0026	0.1244		
0.04	0.1380	0.0048	0.1381		
0.05	0.1492	0.0075	0.1494		
0.06	0.1674	0.0045	0.1675		
0.07	0.1600	0.0051	0.1601		
0.083	0.1600		0.1527		
				0.1313	0.1311
Measured point 1.6					
0	0	0	0		
0.01	0.0990	0.0900	0.1338		
0.02	0.1149	0.0034	0.1150		
0.03	0.1257	0.0050	0.1258		
0.04	0.1418	0.0047	0.1419		
0.05	0.1519	0.0083	0.1521		
0.06	0.1631	0.0061	0.1632		
0.07	0.1666	0.0085	0.1668		
0.083	0.1701		0.1704		
				0.1326	0.1385
Measured point 1.7					
0	0	0	0		
0.01	0.1031	0.0094	0.1035		
0.02	0.1175	0.0021	0.1175		
0.03	0.1377	0.0022	0.1377		
0.04	0.1415	0.0069	0.1417		
0.05	0.1509	0.0066	0.1510		
0.06	0.1652	0.0046	0.1653		
0.07	0.1673	0.0073	0.1675		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.083	0.1694		0.1697		
				0.1356	0.1357
Measured point 1.8					
0	0	0	0		
0.01	0.1168	0.0060	0.1170		
0.02	0.1198	0.0088	0.1201		
0.03	0.1316	0.0068	0.1318		
0.04	0.1452	0.0047	0.1453		
0.05	0.1475	0.0075	0.1477		
0.06	0.1624	0.0090	0.1626		
0.07	0.1686	0.0083	0.1688		
0.083	0.1748		0.1750		
				0.1370	0.1372
Measured point 1.9					
0	0	0	0		
0.01	0.10375	0.0003	0.1038		
0.02	0.118	0.0013	0.1180		
0.03	0.131	0.0069	0.1312		
0.04	0.1447	0.0058	0.1448		
0.05	0.1539	0.0053	0.1540		
0.06	0.1552	0.0081	0.1554		
0.07	0.1651	0.0096	0.1654		
0.083	0.1750		0.1753		
				0.1344	0.1345
Measured point 1.10					
0	0	0	0		
0.01	0.0907	0.0056	0.0909		
0.02	0.1162	0.0053	0.1163		
0.03	0.1288	0.0048	0.1289		
0.04	0.135	0.0073	0.1352		
0.05	0.1466	0.0089	0.1469		
0.06	0.1565	0.0091	0.1568		
0.07	0.1635	0.0095	0.1638		
0.083	0.1705		0.1708		
				0.1293	0.1295

Appendix D

Table D-3 Measured velocities at longitudinal section 2

Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
Measured point 2.1					
0	0	0	0		
0.01	0.0962	-0.0109	0.0968		
0.02	0.1063	-0.0125	0.1070		
0.03	0.1297	-0.0065	0.1299		
0.04	0.1484	-0.0042	0.1485		
0.05	0.1576	-0.0012	0.1576		
0.06	0.1607	0.0019	0.1607		
0.07	0.1623	0.0044	0.1624		
0.083	0.1639		0.1640		
				0.1324	0.1326
Measured point 2.2					
0	0	0	0		
0.01	0.0821	-0.0022	0.0821		
0.02	0.1126	-0.0075	0.1128		
0.03	0.1323	-0.0056	0.1324		
0.04	0.1397	-0.0041	0.1398		
0.05	0.1518	0.0011	0.1518		
0.06	0.1555	0.0034	0.1555		
0.07	0.1629	0.0025	0.1629		
0.083	0.1703		0.1703		
				0.1293	0.1293
Measured point 2.3					
0	0	0	0		
0.01	0.1034	-0.0107	0.1040		
0.02	0.1128	-0.0079	0.1131		
0.03	0.1299	-0.005	0.1300		
0.04	0.1437	-0.0027	0.1437		
0.05	0.1502	0.0015	0.1502		
0.06	0.1601	0.0034	0.1601		
0.07	0.1668	0.0039	0.1668		
0.083	0.1735		0.1736		
				0.1337	0.1338
Measured point 2.4					
0	0	0	0		
0.01	0.0932	-0.004	0.0933		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.02	0.1162	-0.0098	0.1166		
0.03	0.1387	-0.0047	0.1388		
0.04	0.1489	-0.0042	0.1490		
0.05	0.1549	-0.0001	0.1549		
0.06	0.1624	0.0026	0.1624		
0.07	0.1703	0.0077	0.1705		
0.083	0.1782		0.1785		
				0.1359	0.1360
Measured point 2.5					
0	0	0	0		
0.01	0.1249	-0.0036	0.1250		
0.02	0.1223	-0.0058	0.1224		
0.03	0.1384	-0.0014	0.1384		
0.04	0.1543	-0.0023	0.1543		
0.05	0.162	0.0001	0.1620		
0.06	0.1665	0.0051	0.1666		
0.07	0.1687	0.0033	0.1687		
0.083	0.1709		0.1709		
				0.1430	0.1431
Measured point 2.6					
0	0	0	0		
0.01	0.1217	-0.0006	0.1217		
0.02	0.1335	0.0005	0.1335		
0.03	0.1417	-0.0023	0.1417		
0.04	0.1573	-0.0003	0.1573		
0.05	0.161	0.0054	0.1611		
0.06	0.1644	0.0048	0.1645		
0.07	0.1715	0.0061	0.1716		
0.083	0.1786		0.1787		
				0.1447	0.1447
Measured point 2.7					
0	0	0	0		
0.01	0.1125	0.0076	0.1128		
0.02	0.1311	0.0043	0.1312		
0.03	0.1498	0.0035	0.1498		
0.04	0.1552	0.0023	0.1552		
0.05	0.1626	0.0065	0.1627		
0.06	0.1698	0.0057	0.1699		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.07	0.1683	0.0087	0.1685		
0.083	0.1668		0.1672		
				0.1438	0.1440
Measured point 2.8					
0	0	0	0		
0.01	0.1049	0.0069	0.1051		
0.02	0.1309	0.008	0.1311		
0.03	0.1454	0.0076	0.1456		
0.04	0.1525	0.0051	0.1526		
0.05	0.1619	0.0071	0.1621		
0.06	0.1643	0.0104	0.1646		
0.07	0.1689	0.01	0.1692		
0.083	0.1735		0.1738		
				0.1414	0.1416
Measured point 2.9					
0	0	0	0		
0.01	0.1068	0.0113	0.1074		
0.02	0.1131	0.011	0.1136		
0.03	0.1378	0.0095	0.1381		
0.04	0.1524	0.0068	0.1526		
0.05	0.1555	0.0102	0.1558		
0.06	0.1609	0.0101	0.1612		
0.07	0.1687	0.0085	0.1689		
0.083	0.1765		0.1766		
				0.1377	0.1380
Measured point 2.10					
0	0	0	0		
0.01	0.1035	0.0083	0.1038		
0.02	0.1226	0.0125	0.1232		
0.03	0.1323	0.0098	0.1327		
0.04	0.1467	0.0075	0.1469		
0.05	0.1545	0.0101	0.1548		
0.06	0.1567	0.0105	0.1571		
0.07	0.1651	0.0116	0.1655		
0.083	0.1735		0.1740		
				0.1353	0.1357

Appendix D

Table D-4 Measured velocities at longitudinal section 3

Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
Measured point 3.1					
0	0	0	0		
0.01	0.0873	-0.0057	0.0875		
0.02	0.1187	-0.0062	0.1189		
0.03	0.1411	-0.0058	0.1412		
0.04	0.1487	-0.0032	0.1487		
0.05	0.155	-0.0005	0.1550		
0.06	0.1643	0.0011	0.1643		
0.07	0.1684	0.004	0.1684		
0.083	0.1725		0.1726		
				0.1351	0.1352
Measured point 3.2					
0	0	0	0		
0.01	0.0896	-0.0044	0.0897		
0.02	0.118	-0.0075	0.1182		
0.03	0.1404	-0.0043	0.1405		
0.04	0.1511	-0.0033	0.1511		
0.05	0.1561	0.0001	0.1561		
0.06	0.1627	0.003	0.1627		
0.07	0.1658	0.0021	0.1658		
0.083	0.1689		0.1689		
				0.1350	0.1351
Measured point 3.3					
0	0	0	0		
0.01	0.0824	-0.0037	0.0825		
0.02	0.1181	-0.009	0.1184		
0.03	0.1396	-0.0076	0.1398		
0.04	0.1488	-0.0058	0.1489		
0.05	0.1529	-0.0009	0.1529		
0.06	0.1583	0.0011	0.1583		
0.07	0.164	0.0027	0.1640		
0.083	0.1697		0.1697		
				0.1323	0.1324
Measured point 3.4					
0	0	0	0		
0.01	0.1079	-0.0153	0.1090		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.02	0.1119	-0.0113	0.1125		
0.03	0.1424	-0.0076	0.1426		
0.04	0.1535	-0.0066	0.1536		
0.05	0.1558	-0.0003	0.1558		
0.06	0.1659	0.0022	0.1659		
0.07	0.17	0.0011	0.1700		
0.083	0.1741		0.1741		
				0.1392	0.1395
Measured point 3.5					
0	0	0	0		
0.01	0.1207	-0.013	0.1214		
0.02	0.1368	-0.0122	0.1373		
0.03	0.1488	-0.0094	0.1491		
0.04	0.1579	-0.0047	0.1580		
0.05	0.1629	-0.0016	0.1629		
0.06	0.1671	0.0005	0.1671		
0.07	0.1731	0.0069	0.1732		
0.083	0.1791		0.1794		
				0.1469	0.1471
Measured point 3.6					
0	0	0	0		
0.01	0.1346	-0.0006	0.1346		
0.02	0.1455	-0.0023	0.1455		
0.03	0.1593	-0.0046	0.1594		
0.04	0.165	-0.0009	0.1650		
0.05	0.1661	0.0014	0.1661		
0.06	0.1684	0.0049	0.1685		
0.07	0.1755	0.0061	0.1756		
0.083	0.1826		0.1827		
				0.1534	0.1535
Measured point 3.7					
0	0	0	0		
0.01	0.125	0.0153	0.1259		
0.02	0.1387	0.0073	0.1389		
0.03	0.1573	0.0041	0.1574		
0.04	0.1588	0.0023	0.1588		
0.05	0.1651	0.0078	0.1653		
0.06	0.169	0.0078	0.1692		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.07	0.1737	0.01	0.1740		
0.083	0.1784		0.1788		
				0.1498	0.1501
Measured point 3.8					
0	0	0	0		
0.01	0.116	0.0231	0.1183		
0.02	0.1426	0.015	0.1434		
0.03	0.1513	0.0131	0.1519		
0.04	0.1574	0.0091	0.1577		
0.05	0.1641	0.0096	0.1644		
0.06	0.1695	0.0074	0.1697		
0.07	0.1751	0.0109	0.1754		
0.083	0.1807		0.1812		
				0.1477	0.1484
Measured point 3.9					
0	0	0	0		
0.01	0.1093	0.0196	0.1110		
0.02	0.1184	0.0172	0.1196		
0.03	0.1493	0.0144	0.1500		
0.04	0.1522	0.0098	0.1525		
0.05	0.1581	0.0115	0.1585		
0.06	0.1681	0.0114	0.1685		
0.07	0.1687	0.0095	0.1690		
0.083	0.1693		0.1694		
				0.1412	0.1419
Measured point 3.10					
0	0	0	0		
0.01	0.1067	0.018	0.1082		
0.02	0.1193	0.0154	0.1203		
0.03	0.1378	0.0156	0.1387		
0.04	0.15	0.0116	0.1504		
0.05	0.1537	0.011	0.1541		
0.06	0.1611	0.0121	0.1616		
0.07	0.1689	0.0105	0.1692		
0.083	0.1767		0.1769		
				0.1377	0.1384

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Table D-5 Measured velocities at longitudinal section 4

Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
Measured point 4.1					
0	0	0	0		
0.01	0.0843	0.0048	0.0844		
0.02	0.1169	-0.0001	0.1169		
0.03	0.139	-0.0013	0.1390		
0.04	0.1558	0.0008	0.1558		
0.05	0.1533	0.0042	0.1534		
0.06	0.1613	0.0017	0.1613		
0.07	0.1655	0.0042	0.1656		
0.083	0.1697		0.1698		
				0.1336	0.1336
Measured point 4.2					
0	0	0	0		
0.01	0.0836	0.0031	0.0837		
0.02	0.1121	-0.0059	0.1123		
0.03	0.134	-0.0002	0.1340		
0.04	0.148	-0.0024	0.1480		
0.05	0.1543	0.0012	0.1543		
0.06	0.1571	0.002	0.1571		
0.07	0.1638	0.0023	0.1638		
0.083	0.1705		0.1705		
				0.1312	0.1312
Measured point 4.3					
0	0	0	0		
0.01	0.1069	-0.012	0.1076		
0.02	0.1096	-0.0079	0.1099		
0.03	0.1359	-0.0074	0.1361		
0.04	0.1443	-0.0031	0.1443		
0.05	0.156	0.0004	0.1560		
0.06	0.1571	0.0024	0.1571		
0.07	0.1654	0.0024	0.1654		
0.083	0.1737		0.1737		
				0.1355	0.1357
Measured point 4.4					
0	0	0	0		
0.01	0	-0.0015	0.0015		
0.02	0.0358	-0.0044	0.0361		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.03	0.0818	-0.0021	0.0818		
0.04	0.1429	-0.006	0.1430		
0.05	0.1525	-0.0025	0.1525		
0.06	0.1671	-0.0009	0.1671		
0.07	0.1669	0.003	0.1669		
0.083	0.1667		0.1668		
				0.1026	0.1029
Measured point 4.5					
0	0	0	0		
0.01	0	0	0.0000		
0.02	0	-0.0009	0.0009		
0.03	0.028	0.0026	0.0281		
0.04	0.1282	-0.0015	0.1282		
0.05	0.1631	-0.0024	0.1631		
0.06	0.166	0.0017	0.1660		
0.07	0.1688	0.0035	0.1688		
0.083	0.1716		0.1717		
				0.0917	0.0918
Measured point 4.6					
0	0	0	0		
0.01	0	0	0.0000		
0.02	0	0.0001	0.0001		
0.03	0.0319	-0.0076	0.0328		
0.04	0.1411	-0.0032	0.1411		
0.05	0.1684	0.0004	0.1684		
0.06	0.1676	0.0038	0.1676		
0.07	0.1751	0.0057	0.1752		
0.083	0.1826		0.1827		
				0.0959	0.0961
Measured point 4.7					
0	0	0	0		
0.01	0	0.0003	0.0003		
0.02	0.0257	-0.0073	0.0267		
0.03	0.0909	-0.0142	0.0920		
0.04	0.1554	0.0014	0.1554		
0.05	0.1685	0.0062	0.1686		
0.06	0.1707	0.0074	0.1709		
0.07	0.1735	0.0093	0.1737		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.083	0.1763		0.1766	0.1088	0.1092
Measured point 4.8					
0	0	0	0		
0.01	0.0596	0.024	0.0643		
0.02	0.0934	0.0187	0.0953		
0.03	0.14	0.0131	0.1406		
0.04	0.1613	0.0096	0.1616		
0.05	0.1615	0.0095	0.1618		
0.06	0.1644	0.0088	0.1646		
0.07	0.1712	0.0087	0.1714		
0.083	0.1780		0.1782	0.1311	0.1322
Measured point 4.9					
0	0	0	0		
0.01	0.0943	0.0332	0.1000		
0.02	0.1008	0.0246	0.1038		
0.03	0.1314	0.0192	0.1328		
0.04	0.153	0.0117	0.1534		
0.05	0.1597	0.0126	0.1602		
0.06	0.1634	0.0129	0.1639		
0.07	0.1672	0.0135	0.1677		
0.083	0.1710		0.1716	0.1340	0.1357
Measured point 4.10					
0	0	0	0		
0.01	0.0973	0.0282	0.1013		
0.02	0.1082	0.0228	0.1106		
0.03	0.1292	0.0175	0.1304		
0.04	0.147	0.0133	0.1476		
0.05	0.1521	0.0123	0.1526		
0.06	0.1623	0.0131	0.1628		
0.07	0.1652	0.0125	0.1657		
0.083	0.1681		0.1685	0.1324	0.1338

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Table D-6 Measured velocities at longitudinal section 5

Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
Measured point 5.1					
0	0	0	0		
0.01	0.1074	0.0072	0.1076		
0.02	0.1098	0.0076	0.1101		
0.03	0.1312	0.0036	0.1312		
0.04	0.1399	0.0034	0.1399		
0.05	0.151	0.0065	0.1511		
0.06	0.1568	0.0054	0.1569		
0.07	0.1636	0.0059	0.1637		
0.083	0.1704		0.1705		
				0.1332	0.1333
Measured point 5.2					
0	0	0	0		
0.01	0.1002	0.0011	0.1002		
0.02	0.1121	0.0044	0.1122		
0.03	0.1255	0.0041	0.1256		
0.04	0.146	0.002	0.1460		
0.05	0.1535	0.0051	0.1536		
0.06	0.1516	0.0015	0.1516		
0.07	0.166	0.0093	0.1663		
0.083	0.1804		0.1809		
				0.1325	0.1326
Measured point 5.3					
0	0	0	0		
0.01	0.0658	-0.0045	0.0660		
0.02	0.082	-0.0043	0.0821		
0.03	0.0975	0.0074	0.0978		
0.04	0.1326	0.0021	0.1326		
0.05	0.1465	0.005	0.1466		
0.06	0.1529	0.0036	0.1529		
0.07	0.164	0.0059	0.1641		
0.083	0.1751		0.1753		
				0.1169	0.1170
Measured point 5.4					
0	0	0	0		
0.01	0	0.0001	0.0001		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.02	0.0027	-0.0007	0.0028		
0.03	0.0259	0.0014	0.0259		
0.04	0.0615	0.0017	0.0615		
0.05	0.1416	0.0053	0.1417		
0.06	0.1576	0.0046	0.1577		
0.07	0.1683	0.0051	0.1684		
0.083	0.1790		0.1791		
				0.0819	0.0820
Measured point 5.5					
0	0	0	0		
0.01	0	0	0.0000		
0.02	0	0	0.0000		
0.03	0	0	0.0000		
0.04	0.0089	0.0012	0.0090		
0.05	0.0461	0.0001	0.0461		
0.06	0.1382	0.0079	0.1384		
0.07	0.1695	0.0055	0.1696		
0.083	0.2008		0.2008		
				0.0566	0.0566
Measured point 5.6					
0	0	0	0		
0.01	0	0	0.0000		
0.02	0	0	0.0000		
0.03	0	0	0.0000		
0.04	0.0237	0.0027	0.0239		
0.05	0.0646	0.0098	0.0653		
0.06	0.1358	0.0005	0.1358		
0.07	0.162	0.0056	0.1621		
0.083	0.1882		0.1884		
				0.0589	0.0591
Measured point 5.7					
0	0	0	0		
0.01	0	0	0.0000		
0.02	0	0	0.0000		
0.03	0.0003	0.0003	0.0004		
0.04	0.0783	0.0113	0.0791		
0.05	0.103	-0.0012	0.1030		
0.06	0.1515	0.0034	0.1515		

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Vertical position (m)	Velocity (x) (m/s)	Velocity (y) (m/s)	Total velocity (m/s)	Average (x) (m/s)	Average total (m/s)
0.07	0.1749	0.0105	0.1752		
0.083	0.1983		0.1989		
				0.0738	0.0739
Measured point 5.8					
0	0	0	0		
0.01	0.0596	0.024	0.0643		
0.02	0.0934	0.0187	0.0953		
0.03	0.14	0.0131	0.1406		
0.04	0.1613	0.0096	0.1616		
0.05	0.1615	0.0095	0.1618		
0.06	0.1644	0.0088	0.1646		
0.07	0.1712	0.0087	0.1714		
0.083	0.1780		0.1782		
				0.1311	0.1322
Measured point 5.9					
0	0	0	0		
0.01	0.0167	0.0258	0.0307		
0.02	0.0135	0.007	0.0152		
0.03	0.0573	0.0051	0.0575		
0.04	0.1378	0.0087	0.1381		
0.05	0.158	0.0103	0.1583		
0.06	0.1628	0.0096	0.1631		
0.07	0.169	0.0095	0.1693		
0.083	0.1752		0.1755		
				0.1003	0.1030
Measured point 5.10					
0	0	0	0		
0.01	0.0598	0.0303	0.0670		
0.02	0.0477	0.0053	0.0480		
0.03	0.0861	0.0025	0.0861		
0.04	0.1408	0.005	0.1409		
0.05	0.1474	0.008	0.1476		
0.06	0.1588	0.0085	0.1590		
0.07	0.1602	0.0081	0.1604		
0.083	0.1616		0.1618		
				0.1113	0.1126

APPENDIX E

Test 1

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.14 m

Tailgate position 0.1005

Table E-1 Velocity measurements

Longitudinal distance (m)	Distance from right side (m)	Total velocity (cm/s)	Total velocity (m/s)
5.00	0.25	21.08	0.211
5.00	0.30	20.40	0.204
5.00	0.35	21.02	0.210
5.00	0.40	21.15	0.211
5.00	0.45	19.96	0.200
5.10	0.25	21.11	0.211
5.10	0.30	20.42	0.204
5.10	0.35	21.27	0.213
5.10	0.40	21.55	0.216
5.10	0.45	20.23	0.202
5.20	0.25	20.34	0.203
5.20	0.30	20.21	0.202
5.20	0.35	21.52	0.215
5.20	0.40	21.39	0.214
5.20	0.45	20.29	0.203
5.30	0.25	19.53	0.195
5.30	0.30	20.12	0.201
5.30	0.35	20.46	0.205
5.30	0.40	20.29	0.203
5.30	0.45	21.94	0.219
5.40	0.25	11.75	0.117
5.40	0.30	11.17	0.112
5.40	0.35	19.42	0.194
5.40	0.40	24.66	0.247
5.40	0.45	21.53	0.215
5.55	0.25	50.79	0.508
5.55	0.30	1.59	0.016
5.55	0.45	46.53	0.465
5.60	0.25	43.83	0.438
5.60	0.30	2.08	0.021
5.60	0.35	0.97	0.010
5.60	0.40	52.87	0.529
5.60	0.45	48.98	0.490
5.65	0.25	30.35	0.304
5.65	0.30	11.46	0.115
5.65	0.35	1.05	0.010
5.65	0.40	52.31	0.523
5.65	0.45	42.90	0.429
5.70	0.25	23.04	0.230
5.70	0.30	14.47	0.145
5.70	0.35	1.44	0.014
5.70	0.40	49.97	0.500
5.70	0.45	48.25	0.483

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Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.75	0.25	19.67	0.197
5.75	0.30	12.49	0.125
5.75	0.35	5.45	0.054
5.75	0.40	44.85	0.449
5.75	0.45	46.59	0.466
5.80	0.25	16.04	0.160
5.80	0.30	13.18	0.132
5.80	0.35	9.96	0.100
5.80	0.40	43.80	0.438
5.80	0.45	42.09	0.421
5.90	0.25	11.39	0.114
5.90	0.30	10.27	0.103
5.90	0.35	16.36	0.164
5.90	0.40	40.26	0.403
5.90	0.45	33.53	0.335
6.00	0.25	8.29	0.083
6.00	0.30	8.12	0.081
6.00	0.40	38.85	0.389
6.00	0.45	36.78	0.368
6.10	0.25	8.96	0.090
6.10	0.35	21.52	0.215
6.10	0.30	9.03	0.090
6.10	0.40	35.49	0.355
6.10	0.45	22.66	0.227
6.20	0.25	10.30	0.103
6.20	0.30	11.06	0.111
6.20	0.35	23.01	0.230
6.20	0.40	37.51	0.375
6.20	0.45	37.70	0.377
6.40	0.25	10.77	0.108
6.40	0.30	10.40	0.104
6.40	0.35	22.47	0.225
6.40	0.40	31.95	0.320
6.40	0.45	36.93	0.369
6.60	0.25	15.08	0.151
6.60	0.30	10.23	0.102
6.60	0.35	21.71	0.217
6.60	0.40	26.74	0.267
6.60	0.45	25.28	0.253
6.80	0.25	13.70	0.137
6.80	0.30	15.24	0.152
6.80	0.35	19.03	0.190
6.80	0.40	23.28	0.233
6.80	0.45	21.66	0.217
7.00	0.25	15.81	0.158
7.00	0.30	14.01	0.140
7.00	0.35	19.83	0.198
7.00	0.40	20.16	0.202
7.00	0.45	21.81	0.218

Appendix E

Test 2

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.24 m

Tailgate position 0.1005

Table E-2 Velocity measurements

Longitudinal distance (m)	Distance from right side (m)	Total velocity (cm/s)	Total velocity (m/s)
5.00	0.25	20.79	0.208
5.00	0.30	20.38	0.204
5.00	0.35	20.13	0.201
5.00	0.40	20.51	0.205
5.00	0.45	20.35	0.204
5.10	0.25	20.52	0.205
5.10	0.30	18.95	0.189
5.10	0.35	19.08	0.191
5.10	0.40	21.17	0.212
5.10	0.45	20.94	0.209
5.20	0.25	19.92	0.199
5.20	0.30	18.24	0.182
5.20	0.35	19.59	0.196
5.20	0.40	21.24	0.212
5.20	0.45	19.51	0.195
5.30	0.25	20.27	0.203
5.30	0.30	20.59	0.206
5.30	0.35	19.77	0.198
5.30	0.40	19.74	0.197
5.30	0.45	21.10	0.211
5.40	0.25	22.40	0.224
5.40	0.30	19.00	0.190
5.40	0.35	17.03	0.170
5.40	0.40	18.02	0.180
5.40	0.45	23.81	0.238
5.50	0.30	40.74	0.407
5.55	0.25	52.43	0.524
5.55	0.30	18.16	0.182
5.60	0.25	45.13	0.451
5.60	0.26	52.66	0.527
5.60	0.28	58.70	0.587
5.60	0.30	57.32	0.573
5.60	0.32	45.82	0.458
5.60	0.34	11.45	0.115
5.60	0.35	4.93	0.049
5.60	0.40	1.16	0.012
5.60	0.45	40.31	0.403
5.65	0.25	54.62	0.546
5.65	0.31	51.72	0.517
5.65	0.35	10.96	0.110
5.65	0.41	11.38	0.114
5.65	0.45	43.44	0.434
5.70	0.25	46.97	0.470
5.70	0.31	42.99	0.430

Appendix E

Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.70	0.35	15.44	0.154
5.70	0.41	13.25	0.133
5.70	0.45	50.18	0.502
5.80	0.25	44.86	0.449
5.80	0.31	35.50	0.355
5.80	0.35	22.11	0.221
5.80	0.41	20.20	0.202
5.80	0.45	39.55	0.395
5.90	0.25	37.17	0.372
5.90	0.31	33.99	0.340
5.90	0.35	21.88	0.219
5.90	0.41	24.83	0.248
5.90	0.45	34.54	0.345
6.00	0.25	37.03	0.370
6.00	0.30	33.61	0.336
6.00	0.35	26.87	0.269
6.00	0.40	25.44	0.254
6.00	0.45	34.00	0.340
6.10	0.25	34.36	0.344
6.10	0.30	30.75	0.308
6.10	0.35	27.49	0.275
6.10	0.40	27.71	0.277
6.10	0.45	31.00	0.310
6.20	0.25	31.12	0.311
6.20	0.30	30.64	0.306
6.20	0.35	30.04	0.300
6.20	0.40	28.78	0.288
6.20	0.45	29.52	0.295
6.40	0.25	28.13	0.281
6.40	0.30	27.14	0.271
6.40	0.35	29.21	0.292
6.40	0.40	25.86	0.259
6.40	0.45	22.15	0.221
6.60	0.25	27.04	0.270
6.60	0.30	27.64	0.276
6.60	0.35	25.24	0.252
6.60	0.40	23.87	0.239
6.60	0.45	24.99	0.250
6.80	0.25	27.38	0.274
6.80	0.30	27.05	0.270
6.80	0.35	25.51	0.255
6.80	0.40	23.33	0.233
6.80	0.45	19.13	0.191
6.80	0.05	19.13	0.191
7.00	0.25	25.52	0.255
7.00	0.30	23.01	0.230
7.00	0.35	25.28	0.253
7.00	0.40	21.98	0.220
7.00	0.45	25.28	0.253

Appendix E

Test 3

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.038 m

Tailgate position 0.1005

Table E-3 Velocity measurements

Longitudinal distance	Distance from right side	Total velocity	Total velocity
(m)	(m)	(cm/s)	(m/s)
5.00	0.25	19.85	0.199
5.00	0.30	19.13	0.191
5.00	0.35	20.37	0.204
5.00	0.40	18.83	0.188
5.00	0.45	17.37	0.174
5.10	0.25	18.83	0.188
5.10	0.30	18.98	0.190
5.10	0.35	19.45	0.194
5.10	0.40	20.19	0.202
5.10	0.45	19.53	0.195
5.20	0.25	19.90	0.199
5.20	0.30	19.94	0.199
5.20	0.35	19.02	0.190
5.20	0.40	20.24	0.202
5.20	0.45	18.19	0.182
5.30	0.25	22.31	0.223
5.30	0.30	21.77	0.218
5.30	0.35	21.25	0.212
5.30	0.40	20.35	0.203
5.30	0.45	16.12	0.161
5.40	0.25	24.79	0.248
5.40	0.30	20.03	0.200
5.40	0.35	18.12	0.181
5.40	0.40	22.47	0.225
5.40	0.45	5.91	0.059
5.50	0.25	39.47	0.395
5.50	0.30	39.66	0.397
5.50	0.35	44.08	0.441
5.55	0.25	44.68	0.447
5.55	0.30	45.07	0.451
5.55	0.35	49.83	0.498
5.60	0.25	49.03	0.490
5.60	0.30	49.45	0.495
5.60	0.35	50.68	0.507
5.60	0.40	33.70	0.337
5.60	0.45	0.85	0.008
5.65	0.25	49.20	0.492
5.65	0.30	51.75	0.518
5.65	0.35	50.52	0.505
5.65	0.40	36.42	0.364
5.65	0.45	0.99	0.010
5.70	0.25	49.04	0.490
5.70	0.30	47.36	0.474

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5.70	0.35	48.10	0.481
5.70	0.40	36.91	0.369
5.70	0.45	2.37	0.024
5.75	0.25	47.03	0.470
5.75	0.30	47.10	0.471
5.75	0.35	48.46	0.485
5.75	0.40	37.38	0.374
5.75	0.45	3.11	0.031
5.80	0.25	45.40	0.454
5.80	0.30	45.92	0.459
5.80	0.35	48.51	0.485
5.80	0.40	39.20	0.392
5.80	0.45	4.78	0.048
5.90	0.25	47.01	0.470
5.90	0.30	48.02	0.480
5.90	0.35	49.34	0.493
5.90	0.40	42.34	0.423
5.90	0.45	11.29	0.113
6.00	0.25	42.21	0.422
6.00	0.30	44.89	0.449
6.00	0.35	48.81	0.488
6.00	0.40	41.64	0.416
6.00	0.45	13.62	0.136
6.10	0.25	47.37	0.474
6.10	0.30	47.95	0.479
6.10	0.35	50.28	0.503
6.10	0.40	46.16	0.462
6.10	0.45	25.27	0.253
6.20	0.25	44.58	0.446
6.20	0.30	45.08	0.451
6.20	0.35	48.32	0.483
6.20	0.40	44.93	0.449
6.20	0.45	21.88	0.219
6.40	0.25	41.26	0.413
6.40	0.30	44.54	0.445
6.40	0.35	45.70	0.457
6.40	0.40	44.65	0.447
6.40	0.45	25.69	0.257
6.60	0.25	40.60	0.406
6.60	0.30	39.25	0.393
6.60	0.35	44.32	0.443
6.60	0.40	42.12	0.421
6.60	0.45	26.72	0.267
6.80	0.25	41.02	0.410
6.80	0.30	40.02	0.400
6.80	0.35	41.94	0.419
6.80	0.40	40.72	0.407
6.80	0.45	17.69	0.177
7.00	0.25	36.97	0.370
7.00	0.30	35.31	0.353
7.00	0.35	34.72	0.347
7.00	0.40	38.49	0.385
7.00	0.45	30.19	0.302

Appendix E

Test 4

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.24 m

Tailgate position 0.1012

Table E-4 Velocity measurements

Longitudinal distance (m)	Distance from right side (m)	Total velocity (cm/s)	Total velocity (m/s)
5.00	0.25	17.74	0.177
5.00	0.30	17.96	0.180
5.00	0.35	18.61	0.186
5.00	0.40	19.61	0.196
5.00	0.45	19.62	0.196
5.10	0.25	19.33	0.193
5.10	0.30	18.02	0.180
5.10	0.35	18.35	0.183
5.10	0.40	19.42	0.194
5.10	0.45	17.19	0.172
5.20	0.25	19.68	0.197
5.20	0.30	19.57	0.196
5.20	0.35	17.73	0.177
5.20	0.40	19.20	0.192
5.20	0.45	23.32	0.233
5.30	0.25	17.25	0.172
5.30	0.30	17.11	0.171
5.30	0.35	19.34	0.193
5.30	0.40	19.45	0.195
5.30	0.45	20.21	0.202
5.40	0.25	21.07	0.211
5.40	0.30	18.81	0.188
5.40	0.35	18.68	0.187
5.40	0.40	17.69	0.177
5.40	0.45	18.46	0.185
5.50	0.30	36.60	0.366
5.55	0.26	47.52	0.475
5.55	0.30	45.56	0.456
5.55	0.36	1.57	0.016
5.60	0.25	44.75	0.447
5.60	0.30	44.65	0.446
5.60	0.35	9.03	0.090
5.60	0.40	4.31	0.043
5.60	0.45	33.30	0.333
5.70	0.25	39.23	0.392
5.70	0.30	34.75	0.348
5.70	0.35	19.75	0.197
5.70	0.40	11.31	0.113
5.70	0.45	27.87	0.279
5.80	0.25	32.53	0.325
5.80	0.30	30.69	0.307
5.80	0.35	20.26	0.203
5.80	0.40	18.87	0.189

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Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.80	0.45	28.31	0.283
5.90	0.25	30.34	0.303
5.90	0.30	28.56	0.286
5.90	0.35	20.46	0.205
5.90	0.40	20.19	0.202
5.90	0.45	25.39	0.254
6.00	0.25	30.23	0.302
6.00	0.30	28.61	0.286
6.00	0.35	23.12	0.231
6.00	0.40	21.77	0.218
6.00	0.45	26.19	0.262
6.10	0.25	30.32	0.303
6.10	0.30	26.32	0.263
6.10	0.35	23.78	0.238
6.10	0.40	22.75	0.227
6.10	0.45	24.87	0.249
6.20	0.25	30.71	0.307
6.20	0.30	28.37	0.284
6.20	0.35	24.22	0.242
6.20	0.40	22.16	0.222
6.20	0.45	21.28	0.213
6.40	0.25	28.42	0.284
6.40	0.30	29.68	0.297
6.40	0.35	23.88	0.239
6.40	0.40	18.28	0.183
6.40	0.45	19.29	0.193
6.60	0.25	19.94	0.199
6.60	0.30	24.45	0.245
6.60	0.35	18.65	0.186
6.60	0.40	15.61	0.156
6.60	0.45	20.62	0.206
6.80	0.25	24.21	0.242
6.80	0.30	22.79	0.228
6.80	0.35	19.81	0.198
6.80	0.40	16.18	0.162
6.80	0.45	12.66	0.127
6.80	0.05	12.66	0.127
7.00	0.25	25.60	0.256
7.00	0.35	20.43	0.204
7.00	0.30	21.66	0.217
7.00	0.40	18.09	0.181
7.00	0.45	15.94	0.159

Appendix E

Test 5

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.38 m

Tailgate position 0.1012

Table E-5 Velocity measurements

Longitudinal distance	Distance from right side	Total velocity	Total velocity
(m)	(m)	(cm/s)	(m/s)
5.00	0.25	14.14	0.141
5.00	0.30	13.86	0.139
5.00	0.35	16.28	0.163
5.00	0.40	12.43	0.124
5.00	0.45	13.99	0.140
5.10	0.25	16.93	0.169
5.10	0.30	14.13	0.141
5.10	0.35	15.80	0.158
5.10	0.40	10.63	0.106
5.10	0.45	15.60	0.156
5.20	0.25	16.51	0.165
5.20	0.30	15.83	0.158
5.20	0.35	14.46	0.145
5.20	0.40	27.40	0.274
5.20	0.45	5.12	0.051
5.30	0.25	16.87	0.169
5.30	0.30	17.23	0.172
5.30	0.35	15.71	0.157
5.30	0.40	14.68	0.147
5.30	0.45	9.86	0.099
5.40	0.25	22.52	0.225
5.40	0.30	16.31	0.163
5.40	0.35	23.78	0.238
5.40	0.40	19.17	0.192
5.40	0.45	5.25	0.052
5.50	0.25	35.25	0.352
5.50	0.30	35.22	0.352
5.50	0.35	35.85	0.358
5.50	0.40	2.67	0.027
5.55	0.25	39.53	0.395
5.55	0.30	37.29	0.373
5.55	0.35	42.07	0.421
5.55	0.40	0.54	0.005
5.60	0.25	41.64	0.416
5.60	0.30	43.22	0.432
5.60	0.35	41.40	0.414
5.60	0.40	21.16	0.212
5.60	0.45	1.71	0.017
5.65	0.25	42.73	0.427
5.65	0.30	43.62	0.436
5.65	0.35	43.06	0.431

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Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.65	0.40	21.30	0.213
5.65	0.45	1.08	0.011
5.70	0.25	42.34	0.423
5.70	0.30	42.19	0.422
5.70	0.35	39.84	0.398
5.70	0.40	21.44	0.214
5.70	0.45	3.00	0.030
5.75	0.25	42.32	0.423
5.75	0.30	42.44	0.424
5.75	0.35	42.13	0.421
5.75	0.40	24.26	0.243
5.75	0.45	4.62	0.046
5.80	0.25	41.23	0.412
5.80	0.30	41.66	0.417
5.80	0.35	41.93	0.419
5.80	0.40	24.38	0.244
5.80	0.45	4.16	0.042
5.90	0.25	40.56	0.406
5.90	0.30	39.93	0.399
5.90	0.35	40.87	0.409
5.90	0.40	25.39	0.254
5.90	0.45	5.03	0.050
6.00	0.25	41.80	0.418
6.00	0.30	41.85	0.418
6.00	0.35	39.58	0.396
6.00	0.40	20.46	0.205
6.00	0.45	4.05	0.040
6.10	0.25	40.34	0.403
6.10	0.30	41.32	0.413
6.10	0.35	39.37	0.394
6.10	0.40	24.55	0.246
6.10	0.45	7.59	0.076
6.20	0.25	40.11	0.401
6.20	0.30	39.31	0.393
6.20	0.35	37.25	0.372
6.20	0.40	25.16	0.252
6.20	0.45	1.18	0.012
6.40	0.25	35.42	0.354
6.40	0.30	36.98	0.370
6.40	0.35	32.50	0.325
6.40	0.40	14.94	0.149
6.40	0.45	2.58	0.026
6.60	0.25	29.77	0.298
6.60	0.30	31.60	0.316
6.60	0.35	25.06	0.251
6.60	0.40	15.41	0.154
6.60	0.45	4.80	0.048
6.80	0.25	29.76	0.298
6.80	0.30	30.44	0.304
6.80	0.35	25.10	0.251
6.80	0.40	15.36	0.154

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Longitudinal distance	Distance from right side	Total velocity	Total velocity
6.80	0.45	15.31	0.153
7.00	0.25	29.06	0.291
7.00	0.30	29.70	0.297
7.00	0.35	20.57	0.206
7.00	0.40	14.38	0.144
7.00	0.45	5.97	0.060

Test 6

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.24 m

Tailgate position 0.103

Table E-6 Velocity measurements

Longitudinal distance	Distance from right side	Total velocity	Total velocity
(m)	(m)	(cm/s)	(m/s)
5.00	0.25	15.59	0.156
5.00	0.30	27.21	0.272
5.00	0.35	16.06	0.161
5.00	0.40	13.95	0.140
5.00	0.45	13.72	0.137
5.10	0.25	15.86	0.159
5.10	0.30	15.00	0.150
5.10	0.35	15.67	0.157
5.10	0.40	15.99	0.160
5.10	0.45	17.73	0.177
5.20	0.25	15.41	0.154
5.20	0.30	15.75	0.158
5.20	0.35	15.75	0.158
5.20	0.40	15.86	0.159
5.20	0.45	15.99	0.160
5.30	0.25	15.19	0.152
5.30	0.30	15.62	0.156
5.30	0.35	15.31	0.153
5.30	0.40	15.77	0.158
5.30	0.45	13.18	0.132
5.40	0.25	18.71	0.187
5.40	0.30	14.55	0.146
5.40	0.35	14.34	0.143
5.40	0.40	14.60	0.146
5.40	0.45	17.05	0.171
5.50	0.26	34.19	0.342
5.50	0.30	32.30	0.323
5.50	0.36	1.04	0.010
5.60	0.25	32.40	0.324
5.60	0.30	30.00	0.300
5.60	0.35	15.86	0.159

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Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.60	0.40	1.19	0.012
5.60	0.45	26.07	0.261
5.70	0.25	27.59	0.276
5.70	0.30	26.54	0.265
5.70	0.35	15.94	0.159
5.70	0.40	11.13	0.111
5.70	0.45	23.38	0.234
5.80	0.25	25.28	0.253
5.80	0.30	23.80	0.238
5.80	0.35	18.17	0.182
5.80	0.40	15.89	0.159
5.80	0.45	23.70	0.237
5.90	0.25	23.52	0.235
5.90	0.30	21.99	0.220
5.90	0.35	19.47	0.195
5.90	0.45	15.46	0.155
6.00	0.25	22.86	0.229
6.00	0.30	21.17	0.212
6.00	0.35	18.50	0.185
6.00	0.40	19.10	0.191
6.00	0.45	21.28	0.213
6.10	0.25	21.38	0.214
6.10	0.30	20.25	0.202
6.10	0.35	19.38	0.194
6.10	0.40	18.27	0.183
6.10	0.45	18.55	0.185
6.20	0.25	22.76	0.228
6.20	0.30	18.04	0.180
6.20	0.35	20.93	0.209
6.20	0.40	18.94	0.189
6.20	0.45	20.55	0.205
6.40	0.25	20.47	0.205
6.40	0.30	19.22	0.192
6.40	0.35	18.02	0.180
6.40	0.40	16.37	0.164
6.40	0.45	12.87	0.129
6.60	0.25	18.67	0.187
6.60	0.30	17.79	0.178
6.60	0.35	17.49	0.175
6.60	0.40	11.20	0.112
6.60	0.45	10.51	0.105
6.80	0.25	16.47	0.165
6.80	0.30	12.57	0.126
6.80	0.35	16.79	0.168
6.80	0.40	17.85	0.179
6.80	0.45	12.41	0.124
7.00	0.25	14.21	0.142
7.00	0.30	12.84	0.128
7.00	0.35	15.12	0.151
7.00	0.40	15.37	0.154
7.00	0.45	22.75	0.227

Appendix E

Test 7

Slope = 0.0005

Discharge = 0.016 m³/s

Spacing 0.38 m

Tailgate position 0.103

Table E-7 Velocity measurements

Longitudinal distance (m)	Distance from right side (m)	Total velocity (cm/s)	Total velocity (m/s)
5.00	0.25	11.21	0.112
5.00	0.30	13.63	0.136
5.00	0.35	9.01	0.090
5.00	0.40	14.68	0.147
5.00	0.45	8.72	0.087
5.10	0.25	14.26	0.143
5.10	0.30	12.43	0.124
5.10	0.35	14.14	0.141
5.10	0.40	9.08	0.091
5.10	0.45	8.19	0.082
5.20	0.25	13.65	0.137
5.20	0.30	13.79	0.138
5.20	0.35	12.25	0.122
5.20	0.40	14.28	0.143
5.20	0.45	13.08	0.131
5.30	0.25	11.55	0.116
5.30	0.30	15.28	0.153
5.30	0.35	12.24	0.122
5.30	0.40	14.98	0.150
5.30	0.45	13.74	0.137
5.30	0.45	12.62	0.126
5.40	0.25	17.96	0.180
5.40	0.30	13.57	0.136
5.40	0.35	18.70	0.187
5.40	0.40	11.86	0.119
5.40	0.45	10.70	0.107
5.50	0.25	26.55	0.265
5.50	0.30	23.69	0.237
5.50	0.35	23.68	0.237
5.55	0.25	31.69	0.317
5.55	0.30	27.48	0.275
5.55	0.35	29.37	0.294
5.60	0.25	31.04	0.310
5.60	0.30	32.17	0.322
5.60	0.35	31.05	0.310
5.60	0.40	23.10	0.231
5.60	0.45	2.74	0.027
5.65	0.25	31.10	0.311
5.65	0.30	32.73	0.327
5.65	0.35	32.59	0.326
5.65	0.40	20.16	0.202

Appendix E

Longitudinal distance	Distance from right side	Total velocity	Total velocity
5.65	0.45	1.31	0.013
5.70	0.25	32.09	0.321
5.70	0.30	30.52	0.305
5.70	0.35	30.55	0.306
5.70	0.40	19.87	0.199
5.70	0.45	1.17	0.012
5.75	0.25	32.29	0.323
5.75	0.30	32.62	0.326
5.75	0.35	31.51	0.315
5.75	0.40	20.20	0.202
5.75	0.45	3.97	0.040
5.80	0.25	31.21	0.312
5.80	0.30	30.92	0.309
5.80	0.35	30.17	0.302
5.80	0.40	21.05	0.211
5.80	0.45	4.75	0.048
5.90	0.25	29.72	0.297
5.90	0.30	29.45	0.294
5.90	0.35	29.47	0.295
5.90	0.40	22.25	0.222
5.90	0.45	11.23	0.112
6.00	0.25	30.76	0.308
6.00	0.30	32.08	0.321
6.00	0.35	29.25	0.293
6.00	0.40	19.86	0.199
6.00	0.45	4.74	0.047
6.10	0.25	29.20	0.292
6.10	0.30	30.84	0.308
6.10	0.35	29.30	0.293
6.10	0.40	23.50	0.235
6.10	0.45	5.22	0.052
6.20	0.25	27.92	0.279
6.20	0.30	28.80	0.288
6.20	0.35	27.48	0.275
6.20	0.40	20.93	0.209
6.20	0.45	14.97	0.150
6.40	0.25	22.66	0.227
6.40	0.30	25.52	0.255
6.40	0.35	23.81	0.238
6.40	0.40	16.47	0.165
6.40	0.45	6.85	0.068
6.60	0.25	19.29	0.193
6.60	0.30	24.43	0.244
6.60	0.35	19.50	0.195
6.60	0.40	13.32	0.133
6.60	0.45	9.10	0.091
6.80	0.25	19.97	0.200
6.80	0.30	25.13	0.251
6.80	0.35	19.50	0.195
6.80	0.40	13.97	0.140
6.80	0.45	16.49	0.165

Appendix E

Longitudinal distance	Distance from right side	Total velocity	Total velocity
7.00	0.25	18.02	0.180
7.00	0.30	18.14	0.181
7.00	0.35	18.64	0.186
7.00	0.40	19.92	0.199
7.00	0.45	11.91	0.119

APPENDIX F

Test 1

Slope = 0.001136

Discharge = 0.005 m³/s

Uniform depth = 0.035 m

Condition: Empty flume

Table F-1 Test 1 velocity measurements

Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
0.19	39.13	-0.11	39.13	0.391
0.17	41.06	-0.49	41.06	0.411
0.15	40.69	-0.16	40.69	0.407
0.13	41.32	-0.05	41.32	0.413
0.11	39.64	-0.21	39.64	0.396
0.09	37.76	0.07	37.76	0.378
0.07	38.16	-0.77	38.17	0.382
0.05	36.37	-0.52	36.37	0.364
0.03	31.25	-0.14	31.25	0.313
0.01	30.36	-0.14	30.36	0.304

Test 2

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: One hemisphere at chainage 4.5 m



Figure F-1 Experimental set up of Test 2

Table F-2 Test 2 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	31.54	0.58	31.55	0.315
	0.17	33.20	0.04	33.20	0.332
	0.15	32.50	-0.37	32.50	0.325
	0.13	34.38	-1.08	34.40	0.344
	0.11	25.05	1.51	25.10	0.251
	0.09	30.71	-0.10	30.71	0.307
	0.07	31.78	-0.80	31.79	0.318
	0.05	30.08	-0.88	30.09	0.301
	0.03	30.12	-1.56	30.16	0.302
	0.01	25.59	-0.34	25.59	0.256

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
3.75	0.19	30.40	0.77	30.41	0.304
	0.17	29.64	0.46	29.64	0.296
	0.15	29.18	0.54	29.18	0.292
	0.13	28.55	0.89	28.56	0.286
	0.11	37.76	-2.50	37.84	0.378
	0.09	32.71	-0.52	32.71	0.327
	0.07	27.85	-0.37	27.85	0.279
	0.05	24.68	-1.22	24.71	0.247
	0.03	22.64	-0.99	22.66	0.227
	0.01	24.35	-0.01	24.35	0.244
4.0	0.19	31.70	0.44	31.70	0.317
	0.17	31.06	0.42	31.06	0.311
	0.15	33.97	-0.79	33.98	0.340
	0.13	27.99	0.87	28.00	0.280
	0.11	32.67	-0.28	32.67	0.327
	0.09	32.84	-1.08	32.86	0.329
	0.07	34.12	-1.64	34.16	0.342
	0.05	26.36	-0.46	26.36	0.264
	0.03	26.53	-0.14	26.53	0.265
	0.01	24.61	-0.19	24.61	0.246
4.1	0.19	28.64	1.03	28.66	0.287
	0.17	32.14	0.23	32.14	0.321
	0.15	31.56	-0.08	31.56	0.316
	0.13	32.89	-1.00	32.91	0.329
	0.11	31.12	-0.26	31.12	0.311
	0.09	30.67	-0.52	30.67	0.307
	0.07	31.19	-0.29	31.19	0.312
	0.05	28.52	-0.27	28.52	0.285
	0.03	27.51	-1.29	27.54	0.275
	0.01	24.45	-0.22	24.45	0.245
4.2	0.19	33.44	-0.33	33.44	0.334
	0.17	31.69	0.17	31.69	0.317
	0.15	30.91	-0.12	30.91	0.309
	0.13	30.63	-0.66	30.64	0.306
	0.11	33.77	-1.20	33.79	0.338
	0.09	32.16	-0.81	32.17	0.322
	0.07	29.32	-0.85	29.33	0.293

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
	0.05	27.09	-1.13	27.11	0.271
	0.03	25.71	-1.21	25.74	0.257
	0.01	24.46	-0.27	24.46	0.245
4.3	0.19	30.22	0.20	30.22	0.302
	0.17	31.42	-0.14	31.42	0.314
	0.15	30.31	-0.71	30.32	0.303
	0.13	32.97	-1.79	33.02	0.330
	0.11	31.23	-1.23	31.25	0.313
	0.09	30.11	-1.13	30.13	0.301
	0.07	29.90	-1.50	29.94	0.299
	0.05	29.40	-0.99	29.42	0.294
	0.03	22.97	-0.67	22.98	0.230
	0.01	25.72	-0.47	25.72	0.257
4.4	0.19	21.86	1.23	21.89	0.219
	0.17	25.80	-1.70	25.86	0.259
	0.15	27.86	-3.14	28.04	0.280
	0.13	29.90	-4.74	30.27	0.303
	0.11	37.58	-7.02	38.23	0.382
	0.09	32.94	-4.76	33.28	0.333
	0.07	31.75	-4.46	32.06	0.321
	0.05	32.41	-3.01	32.55	0.325
	0.03	27.67	-2.40	27.77	0.278
	0.01	12.12	3.67	12.66	0.127
4.5	0.05	45.84	-5.70	46.19	0.462
	0.03	42.99	-3.04	43.10	0.431
5.0	0.15	30.19	2.34	30.28	0.303
	0.13	38.59	2.25	38.66	0.387
	0.11	44.14	2.75	44.23	0.442
	0.09	47.23	1.02	47.24	0.472
	0.07	41.41	-0.16	41.41	0.414
	0.05	40.93	-0.02	40.93	0.409
	0.03	40.20	-1.60	40.23	0.402
	0.01	36.22	-3.22	36.36	0.364
5.2	0.19	30.34	-0.09	30.34	0.303
	0.17	30.62	-0.07	30.62	0.306
	0.15	34.94	0.07	34.94	0.349
	0.13	40.62	-0.64	40.63	0.406

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
	0.11	41.76	0.07	41.76	0.418
	0.09	38.97	-1.09	38.99	0.390
	0.07	37.77	-0.62	37.78	0.378
	0.05	39.05	-0.04	39.05	0.391
	0.03	33.12	-1.94	33.18	0.332
5.5	0.19	36.27	0.13	36.27	0.363
	0.17	36.62	0.19	36.62	0.366
	0.15	37.46	0.25	37.46	0.375
	0.13	39.40	-0.48	39.40	0.394
	0.11	39.95	0.16	39.95	0.400
	0.09	40.64	0.10	40.64	0.406
	0.07	41.70	0.26	41.70	0.417
	0.05	37.40	-1.69	37.44	0.374
	0.03	34.60	-1.23	34.62	0.346
	0.01	29.34	-0.32	29.34	0.293
6.0	0.19	36.96	0.23	36.96	0.370
	0.17	38.62	0.61	38.62	0.386
	0.15	38.81	-0.15	38.81	0.388
	0.13	38.04	-0.05	38.04	0.380
	0.11	36.87	-0.25	36.87	0.369
	0.09	40.14	-0.89	40.15	0.401
	0.07	40.87	-0.08	40.87	0.409
	0.05	37.32	-0.94	37.33	0.373
	0.03	34.46	-1.70	34.50	0.345
	0.01	30.86	-0.14	30.86	0.309
6.5	0.19	35.68	1.51	35.71	0.357
	0.17	40.06	0.46	40.06	0.401
	0.15	39.90	0.25	39.90	0.399
	0.13	37.44	0.42	37.44	0.374
	0.11	39.13	0.03	39.13	0.391
	0.09	37.71	0.43	37.71	0.377
	0.07	39.61	0.03	39.61	0.396
	0.05	36.71	-0.88	36.72	0.367
	0.03	37.80	-0.09	37.80	0.378
	0.01	34.59	-0.21	34.59	0.346

Test 3

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: One hemisphere at chainage 4.5 m and other at 5.5 m



Figure F-2 Experimental set up of Test 3

Table F-1 Test 3 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	29.69	-0.97	29.71	0.297
	0.17	30.31	-0.70	30.32	0.303
	0.15	29.23	-0.67	29.24	0.292
	0.13	29.19	-1.70	29.24	0.292
	0.11	29.67	-0.81	29.68	0.297
	0.09	30.43	-1.39	30.46	0.305
	0.07	27.06	-0.94	27.08	0.271
	0.05	25.58	-1.07	25.60	0.256
	0.03	20.03	0.69	20.04	0.200
	0.01	24.27	-0.73	24.28	0.243
4.0	0.19	29.36	0.26	29.36	0.294
	0.17	28.45	-0.52	28.45	0.285

Longitudinal chainage (m)	Transverse distance (m)	V_x (cm/s)	V_y (cm/s)	V (cm/s)	V (m/s)
	0.15	28.49	-0.63	28.50	0.285
	0.13	27.62	-0.99	27.64	0.276
	0.11	29.13	-0.41	29.13	0.291
	0.09	30.50	-1.77	30.55	0.306
	0.07	29.49	0.94	29.50	0.295
	0.05	26.31	-0.62	26.32	0.263
	0.03	22.79	-0.22	22.79	0.228
	0.01	23.65	-0.58	23.66	0.237
4.2	0.19	30.14	-0.80	30.15	0.302
	0.17	28.56	-0.32	28.56	0.286
	0.15	27.60	-0.77	27.61	0.276
	0.13	26.68	-1.22	26.71	0.267
	0.11	26.63	-1.39	26.67	0.267
	0.09	29.33	-2.09	29.40	0.294
	0.07	28.33	-1.45	28.37	0.284
	0.05	27.73	-1.83	27.79	0.278
	0.03	23.32	-0.81	23.33	0.233
	0.01	21.86	-0.28	21.86	0.219
4.4	0.19	21.89	1.00	21.91	0.219
	0.17	23.95	-2.00	24.03	0.240
	0.15	25.46	-3.43	25.69	0.257
	0.13	27.70	-5.18	28.18	0.282
	0.11	27.98	-5.11	28.44	0.284
	0.09	27.66	-4.99	28.11	0.281
	0.07	29.11	-3.80	29.36	0.294
	0.05	29.15	-3.62	29.37	0.294
	0.03	24.70	-2.33	24.81	0.248
4.5	0.05	42.06	-6.27	42.52	0.425
	0.03	39.81	-4.61	40.08	0.401
4.6	0.09	48.04	3.49	48.17	0.482
	0.07	46.88	1.92	46.92	0.469
	0.05	49.16	-0.30	49.16	0.492
	0.03	41.16	-0.70	41.17	0.412
4.7	0.09	33.78	4.59	34.09	0.341
	0.07	38.31	3.94	38.51	0.385
	0.05	41.12	2.36	41.19	0.412
	0.03	41.43	0.93	41.44	0.414
4.8	0.19	17.45	-1.89	17.55	0.176

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.17	18.04	0.41	18.04	0.180
	0.15	19.28	1.45	19.33	0.193
	0.13	23.34	1.41	23.38	0.234
	0.11	28.84	1.06	28.86	0.289
	0.09	35.26	1.57	35.29	0.353
	0.07	34.70	1.00	34.71	0.347
	0.05	38.04	1.48	38.07	0.381
	0.03	36.76	1.10	36.78	0.368
	0.01	24.19	-1.87	24.26	0.243
4.9	0.19	22.54	-0.93	22.56	0.226
	0.17	21.90	-0.90	21.92	0.219
	0.15	23.25	0.03	23.25	0.233
	0.13	26.07	-0.34	26.07	0.261
	0.11	30.07	0.52	30.07	0.301
	0.09	34.67	0.88	34.68	0.347
	0.07	35.39	0.61	35.40	0.354
	0.05	36.57	0.56	36.57	0.366
	0.03	36.39	0.97	36.40	0.364
	0.01	23.60	-1.38	23.64	0.236
5.0	0.19	24.22	-0.42	24.22	0.242
	0.17	23.78	0.22	23.78	0.238
	0.15	24.84	-0.12	24.84	0.248
	0.13	26.39	-1.55	26.44	0.264
	0.11	29.26	0.37	29.26	0.293
	0.09	32.79	0.07	32.79	0.328
	0.07	35.76	0.70	35.77	0.358
	0.05	34.35	0.41	34.35	0.344
	0.03	33.74	0.50	33.74	0.337
	0.01	21.18	-1.44	21.23	0.212
5.2	0.19	25.83	-1.78	25.89	0.259
	0.17	26.09	-0.65	26.10	0.261
	0.15	27.2	-0.38	27.20	0.272
	0.13	29.06	-0.17	29.06	0.291
	0.11	29.55	-0.64	29.56	0.296
	0.09	31.22	-0.34	31.22	0.312
	0.07	33.51	0.90	33.52	0.335
	0.05	31.13	0.34	31.13	0.311
	0.03	25.90	-0.64	25.91	0.259

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.01	20.74	0.03	20.74	0.207
5.4	0.17	22.37	-2.72	22.53	0.225
	0.15	25.11	-4.61	25.53	0.255
	0.13	28.78	-6.95	29.61	0.296
	0.11	30.64	-7.56	31.56	0.316
	0.09	33.12	-7.17	33.89	0.339
	0.07	34.56	-6.80	35.22	0.352
	0.05	35.16	-5.18	35.54	0.355
	0.03	30.82	-3.71	31.04	0.310
5.5	0.05	44.40	-4.12	44.59	0.446
	0.03	44.41	-3.04	44.51	0.445
5.6	0.07	45.45	6.25	45.88	0.459
	0.05	56.42	5.01	56.64	0.566
	0.03	59.31	3.63	59.42	0.594
5.65	0.07	37.47	4.10	37.69	0.377
	0.05	45.02	4.17	45.21	0.452
	0.03	40.08	2.13	40.14	0.401
5.75	0.19	26.24	-2.78	26.39	0.264
	0.17	27.65	1.90	27.72	0.277
	0.15	44.01	4.25	44.21	0.442
	0.13	47.72	6.89	48.21	0.482
	0.09	32.13	-3.04	32.27	0.323
	0.07	35.40	-3.21	35.55	0.355
5.85	0.17	37.50	0.00	37.50	0.375
	0.15	32.52	1.03	32.54	0.325
	0.13	40.47	0.52	40.47	0.405
	0.05	41.81	-1.40	41.83	0.418
5.95	0.19	35.41	-1.29	35.43	0.354
	0.17	38.01	-1.60	38.04	0.380
	0.15	38.49	-0.47	38.49	0.385
	0.11	38.12	-0.66	38.13	0.381
	0.09	41.59	-1.50	41.62	0.416
	0.07	44.49	0.30	44.49	0.445
	0.05	41.32	-1.25	41.34	0.413
6.1	0.19	35.93	-1.08	35.95	0.359
	0.17	37.19	-0.84	37.20	0.372
	0.15	37.54	-0.61	37.54	0.375
	0.13	39.75	-0.71	39.76	0.398

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V_x (cm/s)	V_y (cm/s)	V (cm/s)	V (m/s)
	0.11	41.48	-0.37	41.48	0.415
	0.09	40.92	-0.97	40.93	0.409
	0.07	44.01	-0.32	44.01	0.440
	0.05	30.74	-0.78	30.75	0.307
	0.03	36.93	-3.00	37.05	0.371
6.25	0.19	37.52	-0.75	37.53	0.375
	0.17	38.49	-0.65	38.50	0.385
	0.15	39.07	-0.23	39.07	0.391
	0.13	41.21	-1.07	41.22	0.412
	0.11	41.91	-1.62	41.94	0.419
	0.09	41.03	-1.66	41.06	0.411
	0.07	43.15	-0.38	43.15	0.432
	0.05	40.88	-1.89	40.92	0.409
	0.03	33.69	-1.83	33.74	0.337
	0.01	19.95	0.82	19.97	0.200
6.5	0.19	38.42	-0.51	38.42	0.384
	0.17	40.21	-0.72	40.22	0.402
	0.15	39.94	-0.79	39.95	0.399
	0.13	39.10	-1.04	39.11	0.391
	0.11	37.65	-0.46	37.65	0.377
	0.09	39.64	-1.75	39.68	0.397
	0.07	38.33	-1.52	38.36	0.384
	0.05	35.25	-2.19	35.32	0.353
	0.03	32.44	-1.44	32.47	0.325
	0.01	29.06	-0.32	29.06	0.291
7	0.19	38.79	0.02	38.79	0.388
	0.17	41.37	-0.01	41.37	0.414
	0.15	38.60	-0.17	38.60	0.386
	0.13	40.17	-0.30	40.17	0.402
	0.11	39.54	-0.38	39.54	0.395
	0.09	39.67	-1.09	39.68	0.397
	0.07	37.98	-1.22	38.00	0.380
	0.05	37.88	-0.56	37.88	0.379
	0.03	32.67	-1.04	32.69	0.327
	0.01	28.57	0.03	28.57	0.286
8.0	0.19	40.58	0.17	40.58	0.406
	0.17	40.58	-0.16	40.58	0.406
	0.15	40.33	-0.17	40.33	0.403

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V_x (cm/s)	V_y (cm/s)	V (cm/s)	V (m/s)
	0.13	42.06	-1.27	42.08	0.421
	0.11	41.67	-1.10	41.68	0.417
	0.09	40.39	-0.83	40.40	0.404
	0.07	36.27	-0.79	36.28	0.363
	0.05	36.65	-1.23	36.67	0.367
	0.03	34.39	-0.77	34.40	0.344
	0.01	34.67	-0.17	34.67	0.347

Appendix F

Test 4

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: Three hemispheres at chainages 4.5, 5.5 and at 6.5 m



Figure F-3 Experimental set up of Test 4

Table F-2 Test 4 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	28.76	0.06	28.76	0.288
	0.17	28.33	0.06	28.33	0.283
	0.15	27.81	-0.10	27.81	0.278
	0.13	24.60	-0.25	24.60	0.246
	0.11	25.89	-0.01	25.89	0.259
	0.09	33.02	-3.07	33.16	0.332
	0.07	26.33	-0.33	26.33	0.263
	0.05	27.16	-0.15	27.16	0.272
	0.03	18.97	-0.61	18.98	0.190
	0.01	22.03	-0.24	22.03	0.220
4.0	0.19	32.14	0.86	32.15	0.322
	0.17	27.42	-0.91	27.44	0.274
	0.15	27.39	-1.18	27.42	0.274

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.13	30.46	-2.55	30.57	0.306
	0.11	32.00	-2.46	32.09	0.321
	0.09	25.17	-2.84	25.33	0.253
	0.07	28.00	-0.78	28.01	0.280
	0.05	24.32	-0.85	24.33	0.243
	0.03	23.56	-0.32	23.56	0.236
	0.01	21.72	-0.43	21.72	0.217
4.2	0.19	27.17	-0.31	27.17	0.272
	0.17	27.07	-0.35	27.07	0.271
	0.15	25.50	-0.51	25.51	0.255
	0.13	22.17	-0.14	22.17	0.222
	0.11	22.59	-1.31	22.63	0.226
	0.09	26.39	-1.75	26.45	0.264
	0.07	28.23	-1.17	28.25	0.283
	0.05	23.32	-1.14	23.35	0.233
	0.03	23.02	-0.03	23.02	0.230
	0.01	20.19	-0.11	20.19	0.202
4.4	0.19	21.55	-0.48	21.56	0.216
	0.17	21.32	-2.31	21.44	0.214
	0.15	23.47	-3.48	23.73	0.237
	0.13	24.87	-5.12	25.39	0.254
	0.11	30.28	-6.90	31.06	0.311
	0.09	28.76	-4.86	29.17	0.292
	0.07	28.04	-4.41	28.38	0.284
	0.05	28.09	-3.08	28.26	0.283
	0.03	23.18	-2.34	23.30	0.233
4.5	0.05	37.74	-4.36	37.99	0.380
	0.03	35.65	3.74	35.85	0.358
4.6	0.11	38.20	4.21	38.43	0.384
	0.09	45.84	2.89	45.93	0.459
	0.07	42.77	1.75	42.81	0.428
	0.05	43.34	0.07	43.34	0.433
	0.03	40.72	-0.74	40.73	0.407
4.8	0.19	16.02	0.39	16.02	0.160
	0.17	17.68	0.08	17.68	0.177
	0.15	19.39	1.82	19.48	0.195
	0.13	23.68	1.33	23.72	0.237
	0.11	22.04	1.09	22.07	0.221
	0.09	31.66	2.42	31.75	0.318

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.07	32.85	1.97	32.91	0.329
	0.05	33.96	1.27	33.98	0.340
	0.03	29.54	-0.17	29.54	0.295
	0.01	31.86	0.00	31.86	0.319
5.0	0.19	24.85	-0.38	24.85	0.249
	0.17	22.46	0.43	22.46	0.225
	0.15	24.17	0.29	24.17	0.242
	0.13	26.68	-0.06	26.68	0.267
	0.11	30.44	0.87	30.45	0.305
	0.09	30.98	0.34	30.98	0.310
	0.07	30.03	1.43	30.06	0.301
	0.05	28.99	0.81	29.00	0.290
	0.03	29.17	0.44	29.17	0.292
	0.01	20.60	-0.77	20.61	0.206
5.2	0.19	37.99	-0.50	37.99	0.380
	0.17	38.63	-0.80	38.64	0.386
	0.15	39.32	-0.94	39.33	0.393
	0.13	39.67	-1.16	39.69	0.397
	0.11	40.99	-1.44	41.02	0.410
	0.09	43.56	-2.36	43.62	0.436
	0.07	40.54	-2.20	40.60	0.406
	0.05	39.49	-2.66	39.58	0.396
	0.03	35.08	-2.38	35.16	0.352
5.4	0.17	19.45	-2.28	19.58	0.196
	0.15	23.19	-4.39	23.60	0.236
	0.13	25.99	-5.57	26.58	0.266
	0.11	29.03	-7.10	29.89	0.299
	0.09	31.49	-6.53	32.16	0.322
	0.07	31.47	-4.52	31.79	0.318
	0.05	31.15	-4.75	31.51	0.315
	0.03	30.34	-3.02	30.49	0.305
5.5	0.05	41.68	-4.21	41.89	0.419
	0.03	37.70	-3.51	37.86	0.379
5.6	0.13	28.65	9.05	30.05	0.300
	0.11	38.70	8.04	39.53	0.395
	0.09	43.38	4.10	43.57	0.436
	0.07	41.67	2.05	41.72	0.417
	0.05	42.94	0.83	42.95	0.429
	0.03	40.92	-0.91	40.93	0.409

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
5.8	0.19	26.93	-0.14	26.93	0.269
	0.17	25.98	-1.33	26.01	0.260
	0.15	28.72	0.48	28.72	0.287
	0.13	29.73	0.53	29.73	0.297
	0.11	34.70	0.94	34.71	0.347
	0.09	31.76	0.60	31.77	0.318
	0.07	33.12	1.17	33.14	0.331
	0.05	35.90	0.49	35.90	0.359
	0.03	33.07	0.05	33.07	0.331
6.0	0.19	28.17	-0.38	28.17	0.282
	0.17	27.93	-0.95	27.95	0.279
	0.15	29.70	-0.02	29.70	0.297
	0.13	31.56	-0.99	31.58	0.316
	0.11	31.83	-0.91	31.84	0.318
	0.09	31.49	-1.00	31.51	0.315
	0.07	32.26	0.09	32.26	0.323
	0.05	34.18	-0.33	34.18	0.342
	0.03	31.91	-0.50	31.91	0.319
6.2	0.19	28.10	-0.54	28.11	0.281
	0.17	29.48	-0.53	29.48	0.295
	0.15	28.46	-1.23	28.49	0.285
	0.13	30.51	-1.79	30.56	0.306
	0.11	30.94	-0.79	30.95	0.310
	0.09	31.51	-1.66	31.55	0.316
	0.07	33.3	-0.43	33.30	0.333
	0.05	32.65	-1.75	32.70	0.327
	0.03	27.72	-0.76	27.73	0.277
6.4	0.19	22.39	3.74	22.70	0.227
	0.17	23.24	0.94	23.26	0.233
	0.15	22.93	-2.19	23.03	0.230
	0.13	29.89	-4.96	30.30	0.303
	0.11	30.64	-5.24	31.08	0.311
	0.09	31.55	-5.84	32.09	0.321
	0.07	34.81	-3.19	34.96	0.350
	0.05	33.98	-3.82	34.19	0.342
	0.03	30.1	-2.98	30.25	0.302
6.5	0.05	44.76	-4.96	45.03	0.450
	0.03	44.50	-4.15	44.69	0.447
6.6	0.07	53.26	7.82	53.83	0.538

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.05	60.16	5.74	60.43	0.604
	0.03	58.33	2.91	58.40	0.584
6.85	0.19	50.59	-1.40	50.61	0.506
	0.17	54.30	0.20	54.30	0.543
	0.15	50.79	0.04	50.79	0.508
	0.09	48.38	-3.50	48.51	0.485
7.0	0.19	43.04	-1.61	43.07	0.431
	0.17	42.79	-0.15	42.79	0.428
	0.15	45.07	0.14	45.07	0.451
	0.13	45.66	0.11	45.66	0.457
	0.11	47.11	-0.02	47.11	0.471
	0.09	48.08	-1.21	48.10	0.481
	0.07	46.42	-2.12	46.47	0.465
	0.05	40.51	-2.36	40.58	0.406
	0.03	33.11	-2.32	33.19	0.332
7.1	0.19	38.48	-1.79	38.52	0.385
	0.13	43.12	-3.10	43.23	0.432
	0.11	46.45	-3.70	46.60	0.466
	0.09	45.09	-2.87	45.18	0.452
	0.07	41.36	-1.94	41.41	0.414
7.75	0.19	37.99	-0.50	37.99	0.380
	0.17	38.83	-0.80	38.84	0.388
	0.15	39.32	-0.94	39.33	0.393
	0.13	39.67	-1.16	39.69	0.397
	0.11	42.99	-1.44	43.01	0.430
	0.09	43.56	-2.36	43.62	0.436
	0.07	40.54	-2.20	40.60	0.406
	0.05	39.49	-2.66	39.58	0.396
	0.03	35.08	-2.38	35.16	0.352
8.5	0.19	36.79	-0.54	36.79	0.368
	0.17	38.51	-0.53	38.51	0.385
	0.15	38.70	-0.79	38.71	0.387
	0.13	40.75	-1.53	40.78	0.408
	0.11	42.05	-1.61	42.08	0.421
	0.09	40.65	-1.91	40.69	0.407
	0.07	37.82	-1.61	37.85	0.379
	0.05	37.56	-2.20	37.62	0.376
	0.03	34.13	-2.12	34.20	0.342
	0.01	27.63	0.27	27.63	0.276

Appendix F

Test 5

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: Four hemispheres at chainages 4.5, 5.5, 6.5 and at 7.5 m



Figure F-4 Experimental set up of Test 5

Table F-3 Test 5 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	28.07	0.59	28.08	0.281
	0.17	28.29	0.67	28.30	0.283
	0.15	27.49	0.69	27.50	0.275
	0.13	31.37	-0.38	31.37	0.314
	0.11	27.54	0.13	27.54	0.275
	0.09	27.87	-0.57	27.88	0.279
	0.07	26.12	-0.09	26.12	0.261
	0.05	26.55	-0.19	26.55	0.266
	0.03	22.44	0.28	22.44	0.224
	0.01	22.27	0.21	22.27	0.223
4.0	0.19	26.53	-0.88	26.54	0.265
	0.17	27.28	-0.96	27.30	0.273

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.15	28.44	-0.84	28.45	0.285
	0.13	28.95	-0.97	28.97	0.290
	0.11	25.59	-0.97	25.61	0.256
	0.09	27.75	-1.88	27.81	0.278
	0.07	23.87	0.98	23.89	0.239
	0.05	22.48	-0.99	22.50	0.225
	0.03	20.56	-0.86	20.58	0.206
	0.01	21.77	-0.50	21.78	0.218
4.2	0.19	24.11	-0.29	24.11	0.241
	0.17	27.25	-0.49	27.25	0.273
	0.15	26.54	-1.30	26.57	0.266
	0.13	27.99	-1.27	28.02	0.280
	0.11	38.36	-1.84	38.40	0.384
	0.09	27.27	-1.76	27.33	0.273
	0.07	27.84	-0.84	27.85	0.279
	0.05	25.54	-1.89	25.61	0.256
	0.03	19.25	-1.74	19.33	0.193
	0.01	20.10	-0.58	20.11	0.201
4.4	0.19	23.23	-0.12	23.23	0.232
	0.17	22.98	-2.44	23.11	0.231
	0.15	23.97	-3.38	24.21	0.242
	0.13	26.34	-4.89	26.79	0.268
	0.11	29.83	-4.45	30.16	0.302
	0.09	27.38	-5.44	27.92	0.279
	0.07	28.31	-4.06	28.60	0.286
	0.05	25.79	-3.45	26.02	0.260
	0.03	22.25	-0.51	22.26	0.223
4.5	0.05	35.73	-5.35	36.13	0.361
4.6	0.11	33.42	0.95	33.43	0.334
	0.09	43.43	1.46	43.45	0.435
	0.07	43.52	0.72	43.53	0.435
	0.05	41.56	-0.95	41.57	0.416
	0.03	38.57	-1.75	38.61	0.386
4.8	0.19	17.31	1.20	17.35	0.174
	0.17	16.92	-0.24	16.92	0.169
	0.15	19.68	1.28	19.72	0.197
	0.13	25.15	1.50	25.19	0.252

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
	0.11	23.79	0.61	23.80	0.238
	0.09	30.66	0.71	30.67	0.307
	0.07	32.43	1.01	32.45	0.324
	0.05	31.81	0.47	31.81	0.318
	0.03	29.77	-0.27	29.77	0.298
	0.01	32.17	-0.55	32.17	0.322
5.0	0.19	23.16	0.51	23.17	0.232
	0.17	22.31	0.25	22.31	0.223
	0.15	23.83	-0.56	23.84	0.238
	0.13	26.58	0.54	26.59	0.266
	0.11	32.76	0.88	32.77	0.328
	0.09	29.87	-0.79	29.88	0.299
	0.07	30.73	0.10	30.73	0.307
	0.05	25.66	1.19	25.69	0.257
	0.03	25.19	-0.28	25.19	0.252
	0.01	28.17	-0.19	28.17	0.282
5.2	0.19	24.49	-0.25	24.49	0.245
	0.17	24.33	-1.14	24.36	0.244
	0.15	24.87	-1.36	24.91	0.249
	0.13	28.52	-1.21	28.55	0.285
	0.11	26.37	-1.95	26.44	0.264
	0.09	29.04	-2.35	29.13	0.291
	0.07	24.00	1.22	24.03	0.240
	0.05	24.44	0.97	24.46	0.245
	0.03	18.87	-0.28	18.87	0.189
	0.01	24.73	-0.33	24.73	0.247
5.4	0.19	18.33	-1.63	18.40	0.184
	0.17	21.82	-2.90	22.01	0.220
	0.15	23.22	-5.17	23.79	0.238
	0.13	29.29	-5.34	29.77	0.298
	0.11	28.52	-6.05	29.15	0.292
	0.09	31.09	-7.37	31.95	0.320
	0.07	32.56	-4.20	32.83	0.328
	0.05	31.55	-3.28	31.72	0.317
	0.03	25.59	-1.70	25.65	0.256
5.5	0.05	39.01	-5.54	39.40	0.394
	0.03	38.52	-2.89	38.63	0.386

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
5.6	0.15	14.51	6.54	15.92	0.159
	0.13	16.05	4.63	16.70	0.167
	0.11	36.68	5.54	37.10	0.371
	0.09	42.12	1.93	42.16	0.422
	0.07	41.23	0.93	41.24	0.412
	0.05	38.35	0.06	38.35	0.384
	0.03	36.64	-0.69	36.65	0.366
5.8	0.19	20.33	0.29	20.33	0.203
	0.17	20.36	0.03	20.36	0.204
	0.15	24.24	0.58	24.25	0.242
	0.13	28.64	1.22	28.67	0.287
	0.11	28.33	0.26	28.33	0.283
	0.09	36.18	1.78	36.22	0.362
	0.07	35.17	1.05	35.19	0.352
	0.05	36.01	0.85	36.02	0.360
	0.03	21.81	2.75	21.98	0.220
	0.01	24.30	0.72	24.31	0.243
6.0	0.19	24.24	0.35	24.24	0.242
	0.17	23.85	-0.32	23.85	0.239
	0.15	26.33	-0.75	26.34	0.263
	0.13	29.79	-0.71	29.80	0.298
	0.11	26.84	-3.76	27.10	0.271
	0.09	34.05	-1.30	34.07	0.341
	0.07	36.52	-0.51	36.52	0.365
	0.05	33.02	0.33	33.02	0.330
	0.03	25.83	-0.90	25.85	0.258
	0.01	27.42	-0.03	27.42	0.274
6.2	0.19	25.75	-0.20	25.75	0.258
	0.17	26.13	-0.99	26.15	0.261
	0.15	27.03	-0.69	27.04	0.270
	0.13	31.85	-0.57	31.86	0.319
	0.11	25.72	-3.25	25.92	0.259
	0.09	32.53	-1.31	32.56	0.326
	0.07	31.84	-1.00	31.86	0.319
	0.05	32.22	-0.66	32.23	0.322
	0.03	26.73	-1.51	26.77	0.268
	0.01	22.28	0.12	22.28	0.223

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
6.4	0.19	22.27	0.44	22.27	0.223
	0.17	23.25	-1.11	23.28	0.233
	0.15	25.45	-3.43	25.68	0.257
	0.13	28.05	-4.59	28.42	0.284
	0.11	29.12	-5.58	29.65	0.296
	0.09	33.42	-5.37	33.85	0.338
	0.07	31.36	-4.91	31.74	0.317
	0.05	32.35	-3.54	32.54	0.325
	0.03	33.11	-2.75	33.22	0.332
6.5	0.05	42.95	-5.98	43.36	0.434
	0.03	40.79	-3.95	40.98	0.410
6.6	0.09	46.31	2.24	46.36	0.464
	0.07	44.85	-0.11	44.85	0.449
	0.05	44.46	-0.01	44.46	0.445
	0.03	42.87	-0.18	42.87	0.429
6.8	0.09	34.40	-0.76	34.41	0.344
	0.07	36.59	0.77	36.60	0.366
	0.05	37.14	0.26	37.14	0.371
	0.03	34.08	0.57	34.08	0.341
7.0	0.19	23.53	-1.22	23.56	0.236
	0.17	25.31	-1.01	25.33	0.253
	0.15	27.56	-1.00	27.58	0.276
	0.13	30.45	-0.89	30.46	0.305
	0.11	25.93	-1.17	25.96	0.260
	0.09	33.55	-1.20	33.57	0.336
	0.07	34.07	0.10	34.07	0.341
	0.05	33.82	-0.24	33.82	0.338
	0.03	31.38	-0.52	31.38	0.314
7.2	0.01	23.10	-0.24	23.10	0.231
	0.19	30.77	-1.52	30.81	0.308
	0.17	27.39	-1.56	27.43	0.274
	0.15	30.36	0.43	30.36	0.304
	0.13	32.44	-0.67	32.45	0.324
	0.11	26.81	-2.33	26.91	0.269
	0.09	35.13	-1.70	35.17	0.352
	0.07	35.56	-0.21	35.56	0.356
	0.05	34.09	-0.73	34.10	0.341

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	Vx (cm/s)	Vy (cm/s)	V (cm/s)	V (m/s)
	0.03	30.61	-1.06	30.63	0.306
	0.01	23.32	0.12	23.32	0.233
7.4	0.17	21.43	-1.57	21.49	0.215
	0.15	23.32	-4.20	23.70	0.237
	0.13	30.26	-6.17	30.88	0.309
	0.11	28.64	-7.45	29.59	0.296
	0.09	36.27	-6.17	36.79	0.368
	0.07	34.35	-5.93	34.86	0.349
	0.05	35.37	-5.51	35.80	0.358
	0.03	29.45	-2.89	29.59	0.296
7.5	0.05	33.11	-9.75	34.52	0.345
7.6	0.07	39.23	4.99	39.55	0.395
	0.05	59.59	7.00	60.00	0.600
	0.03	60.88	5.07	61.09	0.611
8.5	0.19	37.80	-0.18	37.80	0.378
	0.17	38.97	-0.29	38.97	0.390
	0.15	39.50	-0.58	39.50	0.395
	0.13	40.55	-1.17	40.57	0.406
	0.11	43.97	-2.59	44.05	0.440
	0.09	43.13	-1.63	43.16	0.432
	0.07	39.40	-2.31	39.47	0.395
	0.05	36.79	-2.53	36.88	0.369
	0.01	20.54	0.82	20.56	0.206
9.0	0.19	40.67	-0.71	40.68	0.407
	0.17	40.49	-0.41	40.49	0.405
	0.13	41.10	-2.06	41.15	0.412
	0.11	41.92	-1.26	41.94	0.419
	0.09	40.53	-1.35	40.55	0.406
	0.07	38.72	-1.05	38.73	0.387
	0.05	31.00	-0.85	31.01	0.310
	0.01	28.32	-0.06	28.32	0.283

Appendix F

Test 6

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: Two hemispheres at chainages 4.5 and at 8.5 m



Figure F-5 Experimental set up of Test 6

Table F-4 Test 6 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	28.52	0.64	28.53	0.285
	0.17	29.14	0.05	29.14	0.291
	0.15	25.95	0.67	25.96	0.260
	0.13	22.93	1.84	23.00	0.230
	0.11	36.47	-3.02	36.59	0.366
	0.09	25.27	-1.51	25.32	0.253
	0.07	25.90	1.03	25.92	0.259
	0.05	18.86	-0.78	18.88	0.189
	0.03	28.25	-0.78	28.26	0.283
	0.01	25.59	-0.34	25.59	0.256
3.75	0.19	25.70	0.36	25.70	0.257
	0.17	28.70	-0.37	28.70	0.287

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V_x (cm/s)	V_y (cm/s)	V (cm/s)	V (m/s)
	0.15	27.28	-0.11	27.28	0.273
	0.13	30.92	-4.15	31.20	0.312
	0.11	33.35	-0.45	33.35	0.334
	0.09	25.52	0.31	25.52	0.255
	0.07	29.77	1.95	29.83	0.298
	0.05	24.87	-0.82	24.88	0.249
	0.03	17.53	1.22	17.57	0.176
	0.01	19.37	1.28	19.41	0.194
4	0.19	27.60	0.59	27.61	0.276
	0.17	27.17	0.39	27.17	0.272
	0.15	37.57	-3.11	37.70	0.377
	0.13	29.66	-1.92	29.72	0.297
	0.11	20.33	-0.98	20.35	0.204
	0.09	27.39	-0.68	27.40	0.274
	0.07	23.39	-1.94	23.47	0.235
	0.05	24.30	1.49	24.35	0.243
	0.03	26.26	0.32	26.26	0.263
	0.01	19.80	-1.01	19.83	0.198
4.4	0.19	17.59	-0.98	17.62	0.176
	0.17	19.77	-2.87	19.98	0.200
	0.15	18.11	-3.88	18.52	0.185
	0.13	23.13	-6.32	23.98	0.240
	0.11	24.28	-8.47	25.71	0.257
	0.09	27.61	-6.02	28.26	0.283
	0.07	26.65	-3.87	26.93	0.269
	0.05	28.07	-4.33	28.40	0.284
	0.03	23.06	-1.98	23.14	0.231
4.5	0.05	38.93	-5.68	39.34	0.393
	0.03	30.69	-3.24	30.86	0.309
4.6	0.13	24.04	4.50	24.46	0.245
	0.11	46.68	3.96	46.85	0.468
	0.09	49.48	3.54	49.61	0.496
	0.07	42.46	1.55	42.49	0.425
	0.05	45.26	0.16	45.26	0.453
	0.03	29.65	1.44	29.68	0.297
4.8	0.19	16.98	-0.68	16.99	0.170
	0.17	18.85	1.12	18.88	0.189

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.15	20.34	1.39	20.39	0.204
	0.13	28.07	2.49	28.18	0.282
	0.11	29.99	4.21	30.28	0.303
	0.09	36.25	4.24	36.50	0.365
	0.07	36.32	3.41	36.48	0.365
	0.05	33.26	1.62	33.30	0.333
	0.03	42.32	-0.74	42.33	0.423
	0.01	25.42	-0.92	25.44	0.254
5	0.19	21.60	-0.26	21.60	0.216
	0.17	22.37	0.85	22.39	0.224
	0.15	23.55	0.91	23.57	0.236
	0.13	28.41	0.61	28.42	0.284
	0.11	31.81	0.17	31.81	0.318
	0.09	33.90	-0.31	33.90	0.339
	0.07	33.04	-0.25	33.04	0.330
	0.05	29.31	2.31	29.40	0.294
	0.03	36.20	2.20	36.27	0.363
	0.01	26.58	0.33	26.58	0.266
5.25	0.19	25.34	-2.58	25.47	0.255
	0.17	26.94	0.21	26.94	0.269
	0.15	25.56	2.14	25.65	0.256
	0.13	30.32	0.58	30.33	0.303
	0.11	30.94	0.57	30.95	0.309
	0.09	34.71	-0.47	34.71	0.347
	0.07	34.06	0.46	34.06	0.341
	0.05	22.40	2.06	22.49	0.225
	0.03	25.92	-0.30	25.92	0.259
	0.01	21.44	-0.44	21.44	0.214
6	0.19	28.48	0.23	28.48	0.285
	0.17	32.12	0.09	32.12	0.321
	0.15	30.61	0.39	30.61	0.306
	0.13	32.26	-0.45	32.26	0.323
	0.11	36.02	-1.75	36.06	0.361
	0.09	30.31	-0.14	30.31	0.303
	0.07	27.10	0.35	27.10	0.271
	0.05	23.69	0.70	23.70	0.237
	0.03	26.08	-0.44	26.08	0.261

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
7	0.01	28.48	0.49	28.48	0.285
	0.19	27.04	-0.05	27.04	0.270
	0.17	27.81	0.24	27.81	0.278
	0.15	28.15	0.48	28.15	0.282
	0.13	27.13	0.32	27.13	0.271
	0.11	28.02	0.45	28.02	0.280
	0.09	32.84	-0.34	32.84	0.328
	0.07	29.71	0.29	29.71	0.297
	0.05	31.80	-0.06	31.80	0.318
	0.03	25.44	1.08	25.46	0.255
8	0.01	29.04	0.94	29.06	0.291
	0.19	26.82	-0.08	26.82	0.268
	0.17	27.41	0.09	27.41	0.274
	0.15	28.61	0.47	28.61	0.286
	0.13	30.92	-0.70	30.93	0.309
	0.11	23.23	1.04	23.25	0.233
	0.09	22.65	0.69	22.66	0.227
	0.07	26.50	-0.28	26.50	0.265
	0.05	23.82	0.45	23.82	0.238
	0.03	30.34	0.40	30.34	0.303
8.4	0.01	26.90	1.39	26.94	0.269
	0.19	19.06	1.12	19.09	0.191
	0.17	20.44	-0.38	20.44	0.204
	0.15	23.03	-4.71	23.51	0.235
	0.13	28.96	-5.45	29.47	0.295
	0.11	29.30	-6.06	29.92	0.299
	0.09	30.05	-5.48	30.55	0.305
	0.07	29.47	-3.71	29.70	0.297
	0.05	29.07	-3.43	29.27	0.293
	0.03	32.70	-2.27	32.78	0.328
8.5	0.01	29.66	0.13	29.66	0.297
	0.05	44.18	-4.36	44.39	0.444
	0.03	43.12	-3.32	43.25	0.432

Appendix F

Test 7

Slope = 0.001136

Discharge = 0.005 m³/s

Condition: One cylinder at chainage 4.5 m

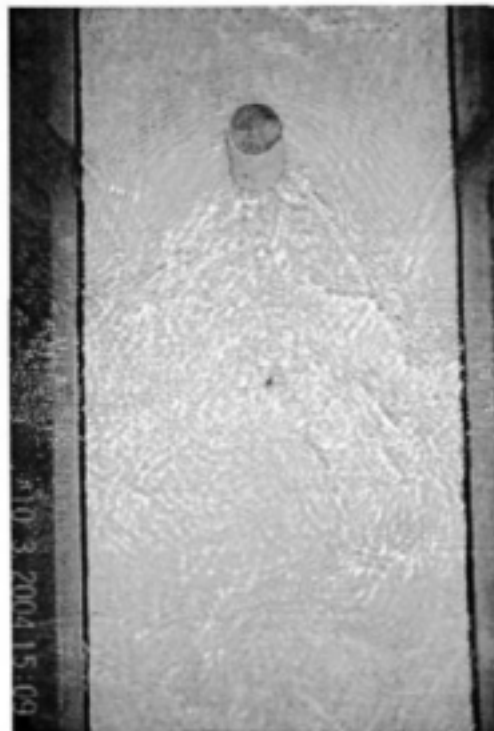


Figure F-6 Experimental set up of Test 7

Table F-5 Test 7 velocity measurements

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
3.5	0.19	35.92	-0.06	35.92	0.359
	0.17	37.14	-0.15	37.14	0.371
	0.15	36.06	-0.02	36.06	0.361
	0.13	36.92	-0.44	36.92	0.369
	0.11	38.72	-1.14	38.74	0.387
	0.09	36.22	-0.57	36.22	0.362
	0.07	34.82	-0.41	34.82	0.348
	0.05	34.38	-0.34	34.38	0.344
	0.03	32.18	-0.79	32.19	0.322
	0.01	30.93	-0.60	30.94	0.309
4.0	0.19	36.63	-0.15	36.63	0.366
	0.17	35.48	-0.22	35.48	0.355

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.15	36.48	-0.28	36.48	0.365
	0.13	36.57	-0.55	36.57	0.366
	0.11	36.06	-0.78	36.07	0.361
	0.09	32.98	-0.38	32.98	0.330
	0.07	35.20	-0.68	35.21	0.352
	0.05	34.66	-0.56	34.66	0.347
	0.03	31.61	-0.70	31.62	0.316
	0.01	29.70	-0.35	29.70	0.297
4.25	0.19	34.92	0.28	34.92	0.349
	0.17	35.99	-0.27	35.99	0.360
	0.15	35.78	-0.34	35.78	0.358
	0.13	37.05	-0.85	37.06	0.371
	0.11	35.81	-0.67	35.82	0.358
	0.09	35.13	-0.55	35.13	0.351
	0.07	34.72	-0.77	34.73	0.347
	0.05	33.47	-0.33	33.47	0.335
	0.03	29.85	-0.32	29.85	0.299
	0.01	28.53	-0.49	28.53	0.285
4.46	0.19	27.93	0.68	27.94	0.279
	0.17	30.46	-1.98	30.52	0.305
	0.15	31.44	-2.92	31.58	0.316
	0.13	34.13	-3.69	34.33	0.343
	0.11	36.34	-3.79	36.54	0.365
	0.09	36.85	-3.69	37.03	0.370
	0.07	35.60	-2.86	35.71	0.357
	0.05	36.41	-2.67	36.51	0.365
	0.03	35.04	-2.14	35.11	0.351
	0.01	32.63	-1.32	32.66	0.327
4.5	0.17	48.27	7.31	48.82	0.488
	0.15	46.67	6.54	47.13	0.471
	0.09	41.10	-5.73	41.50	0.415
	0.07	39.73	-4.55	39.99	0.400
	0.05	40.85	-3.48	41.00	0.410
	0.03	38.99	-2.65	39.08	0.391
	0.01	35.87	-1.06	35.89	0.359
4.56	0.19	0.00	0.00	0.00	0.000
	0.17	0.00	0.00	0.00	0.000

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.15	52.18	0.31	52.18	0.522
	0.13	51.26	0.72	51.27	0.513
	0.11	50.15	0.96	50.16	0.502
	0.09	45.88	0.03	45.88	0.459
	0.07	45.62	-0.92	45.63	0.456
	0.05	44.05	-0.92	44.06	0.441
	0.03	42.14	-0.98	42.15	0.422
	0.01	39.01	-0.60	39.01	0.390
4.6	0.19	0.00	0.00	0.00	0.000
	0.17	12.66	3.93	13.26	0.133
	0.15	48.56	4.37	48.76	0.488
	0.13	47.69	2.36	47.75	0.477
	0.11	46.70	1.74	46.73	0.467
	0.09	43.91	1.20	43.93	0.439
	0.07	43.09	0.62	43.09	0.431
	0.05	43.51	0.28	43.51	0.435
	0.03	39.81	0.37	39.81	0.398
	0.01	40.73	-0.31	40.73	0.407
4.65	0.19	9.78	1.25	9.86	0.099
	0.17	23.81	3.70	24.10	0.241
	0.15	41.68	5.40	42.03	0.420
	0.13	44.31	4.13	44.50	0.445
	0.11	45.01	2.60	45.09	0.451
	0.09	40.95	1.98	41.00	0.410
	0.07	41.48	0.78	41.49	0.415
	0.05	41.81	0.01	41.81	0.418
	0.03	41.28	-0.37	41.28	0.413
	0.01	39.12	-0.31	39.12	0.391
4.7	0.19	14.86	0.47	14.87	0.149
	0.17	27.40	3.45	27.62	0.276
	0.15	39.90	4.68	40.17	0.402
	0.13	42.74	3.04	42.85	0.428
	0.11	42.27	2.15	42.32	0.423
	0.09	40.34	1.13	40.36	0.404
	0.07	42.03	1.13	42.05	0.420
	0.05	41.43	0.41	41.43	0.414
	0.03	40.73	-0.33	40.73	0.407

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V_x (cm/s)	V_y (cm/s)	V (cm/s)	V (m/s)
	0.01	38.63	-0.53	38.63	0.386
4.75	0.19	22.39	-0.07	22.39	0.224
	0.17	28.61	2.48	28.72	0.287
	0.15	39.03	3.75	39.21	0.392
	0.13	42.71	2.94	42.81	0.428
	0.11	41.31	1.94	41.36	0.414
	0.09	39.71	1.08	39.72	0.397
	0.07	39.94	0.76	39.95	0.399
	0.05	39.90	-0.04	39.90	0.399
	0.03	34.97	0.42	34.97	0.350
	0.01	37.28	-0.35	37.28	0.373
4.8	0.19	25.14	0.85	25.15	0.252
	0.17	30.53	2.55	30.64	0.306
	0.15	38.03	2.49	38.11	0.381
	0.13	40.84	2.38	40.91	0.409
	0.11	41.72	1.54	41.75	0.417
	0.09	40.94	0.72	40.95	0.409
	0.07	40.79	-0.12	40.79	0.408
	0.05	41.21	0.22	41.21	0.412
	0.03	37.60	0.01	37.60	0.376
	0.01	36.79	-0.39	36.79	0.368
4.9	0.19	28.28	0.57	28.29	0.283
	0.17	32.12	1.17	32.14	0.321
	0.15	37.19	2.09	37.25	0.372
	0.13	40.29	1.95	40.34	0.403
	0.11	40.41	1.27	40.43	0.404
	0.09	40.30	0.07	40.30	0.403
	0.07	40.53	-0.17	40.53	0.405
	0.05	40.03	-0.18	40.03	0.400
	0.03	35.42	-0.77	35.43	0.354
	0.01	35.52	-0.33	35.52	0.355
5.0	0.19	30.71	0.31	30.71	0.307
	0.17	32.00	0.86	32.01	0.320
	0.15	34.60	1.16	34.62	0.346
	0.13	38.84	0.83	38.85	0.388
	0.11	40.06	0.74	40.07	0.401
	0.09	39.83	0.43	39.83	0.398

Appendix F

Longitudinal chainage (m)	Transverse distance (m)	V _x (cm/s)	V _y (cm/s)	V (cm/s)	V (m/s)
	0.07	39.83	-0.43	39.83	0.398
	0.05	37.03	-0.25	37.03	0.370
	0.03	36.88	-0.75	36.89	0.369
	0.01	35.12	-0.78	35.13	0.351
5.5	0.19	34.63	-0.48	34.63	0.346
	0.17	34.72	-0.09	34.72	0.347
	0.15	36.57	0.30	36.57	0.366
	0.13	36.02	0.64	36.03	0.360
	0.11	38.32	0.31	38.32	0.383
	0.09	39.09	0.23	39.09	0.391
	0.07	39.03	-0.25	39.03	0.390
	0.05	37.82	-0.83	37.83	0.378
	0.03	36.08	-0.35	36.08	0.361
	0.01	34.74	-0.68	34.75	0.347
6.5	0.19	36.39	-1.39	36.42	0.364
	0.17	37.49	-1.13	37.51	0.375
	0.15	38.34	-0.87	38.35	0.383
	0.13	38.13	-1.29	38.15	0.382
	0.11	41.15	-1.34	41.17	0.412
	0.09	35.55	-0.56	35.55	0.356
	0.07	38.18	-0.46	38.18	0.382
	0.05	39.21	-1.32	39.23	0.392
	0.03	36.85	-1.13	36.87	0.369
	0.01	35.34	-1.22	35.36	0.354
7.0	0.19	36.31	-0.99	36.32	0.363
	0.17	36.92	-0.99	36.93	0.369
	0.15	38.10	-1.17	38.12	0.381
	0.13	37.77	-1.19	37.79	0.378
	0.11	40.20	-1.87	40.24	0.402
	0.09	37.87	-1.46	37.90	0.379
	0.07	37.81	-1.45	37.84	0.378
	0.05	38.86	-1.19	38.88	0.389
	0.03	41.56	-1.85	41.60	0.416
	0.01	34.69	-1.16	34.71	0.347

Appendix F

Test 2

Slope = 0.0011

Discharge = 0.005 m³/s

Condition: One hemisphere at chainage 4.5 m

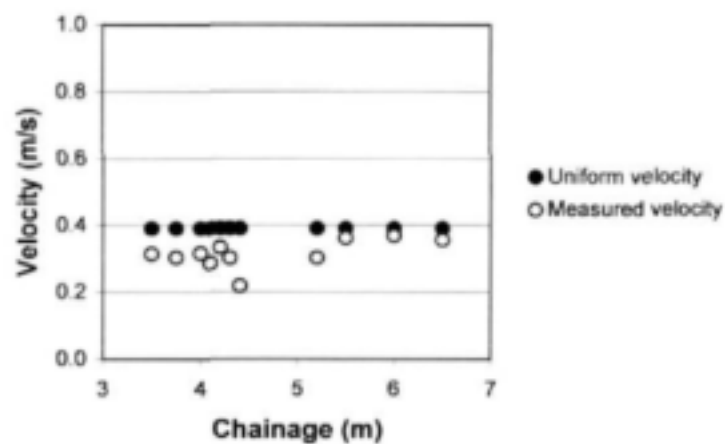


Figure F-7 Velocity measurements at 0.19 m from left side of the flume

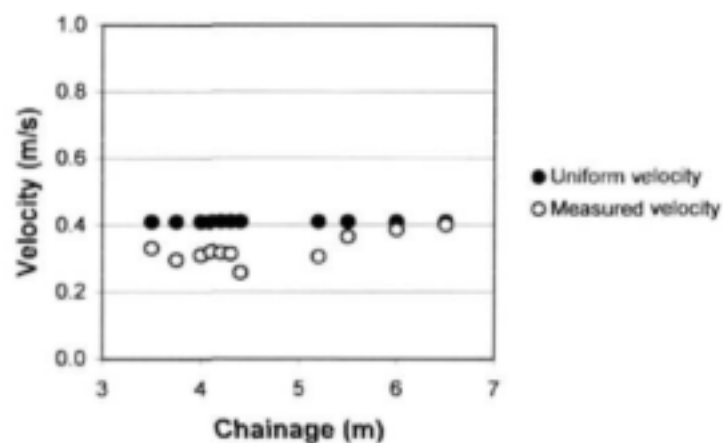


Figure F-8 Velocity measurements at 0.17 m from left side of the flume

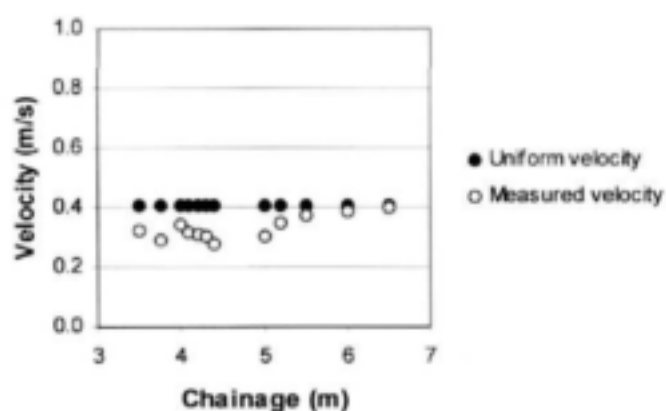


Figure F-9 Velocity measurements at 0.15 m from left side of the flume

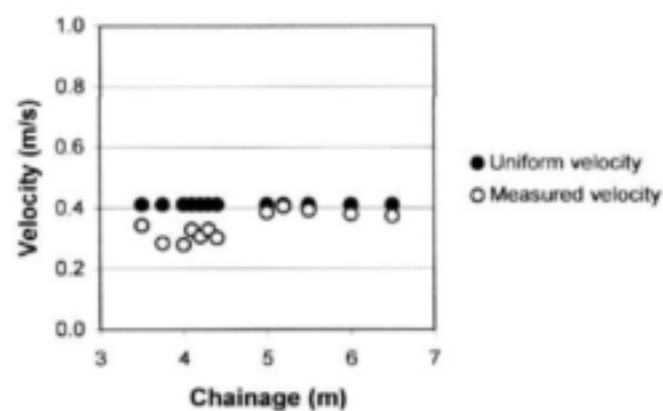


Figure F-10 Velocity measurements at 0.13 m from left side of the flume

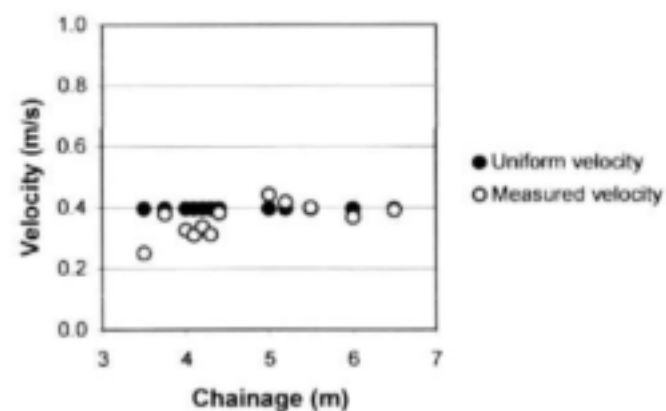


Figure F-11 Velocity measurements at 0.11 m from left side of the flume

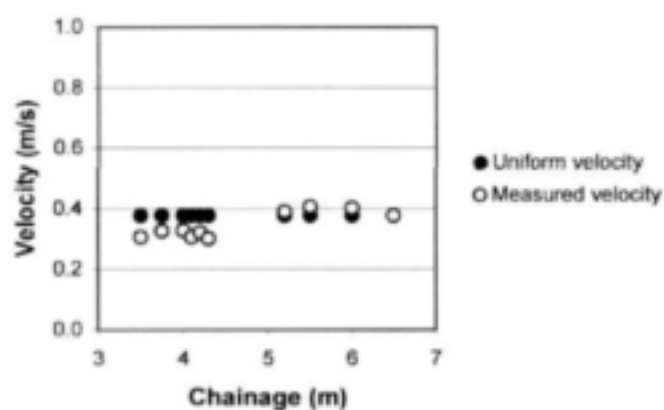


Figure F-12 Velocity measurements at 0.09 m from left side of the flume

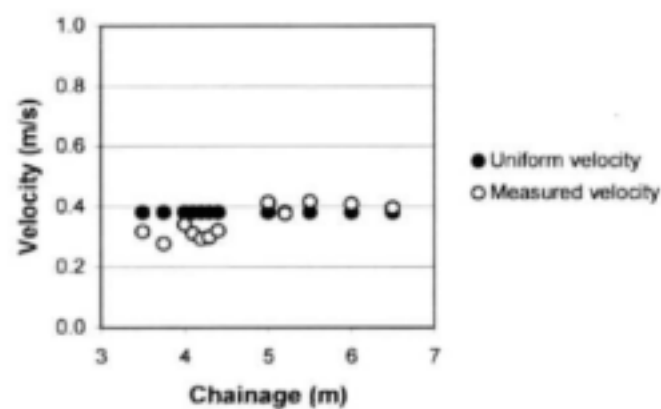


Figure F-13 Velocity measurements at 0.07 m from left side of the flume

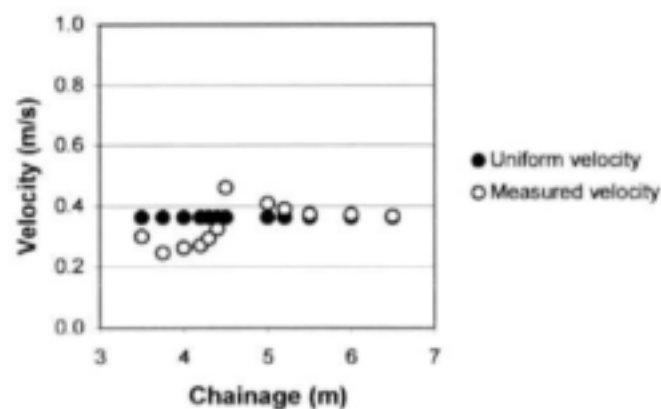


Figure F-14 Velocity measurements at 0.05 m from left side of the flume

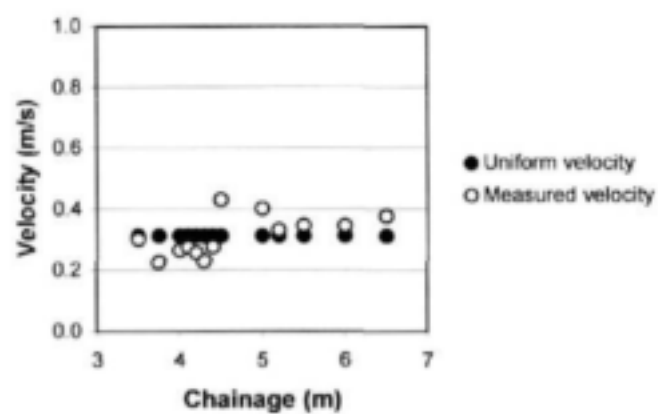


Figure F-15 Velocity measurements at 0.03 m from left side of the flume

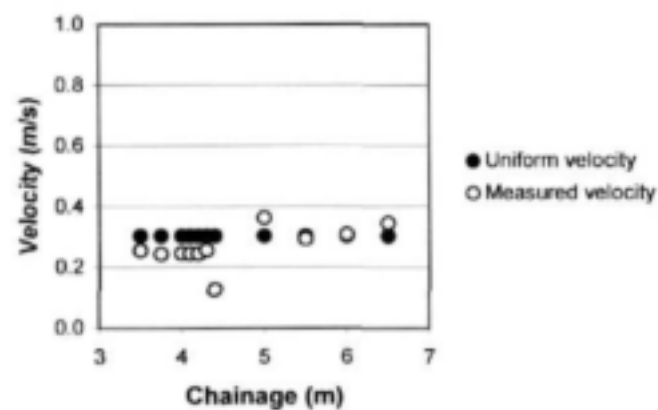


Figure F-16 Velocity measurements at 0.01 m from left side of the flume

Appendix F

Test 3

Slope = 0.0011

Discharge = $0.005 \text{ m}^3/\text{s}$

Condition: Two hemispheres at chainage 4.5 m and 5.5

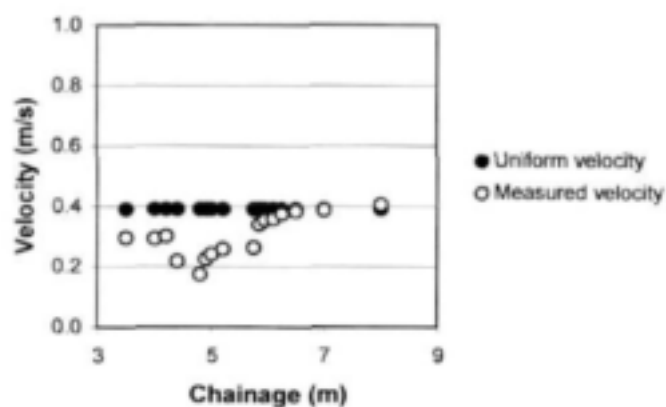


Figure F-17 Velocity measurements at 0.19 m from left side of the flume

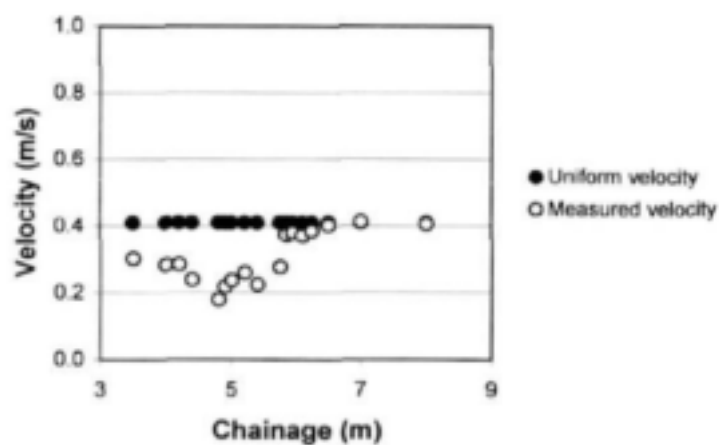


Figure F-18 Velocity measurements at 0.17 m from left side of the flume

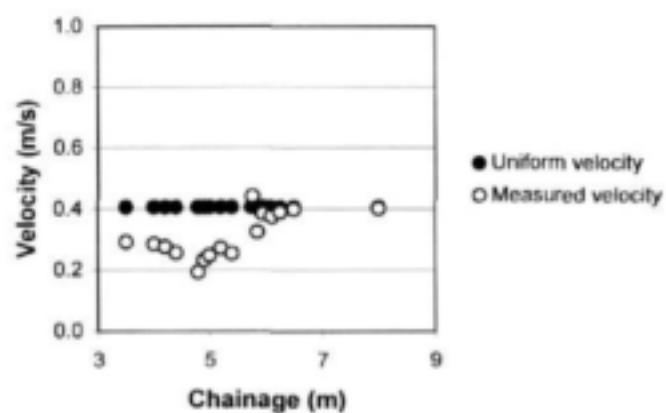


Figure F-19 Velocity measurements at 0.15 m from left side of the flume

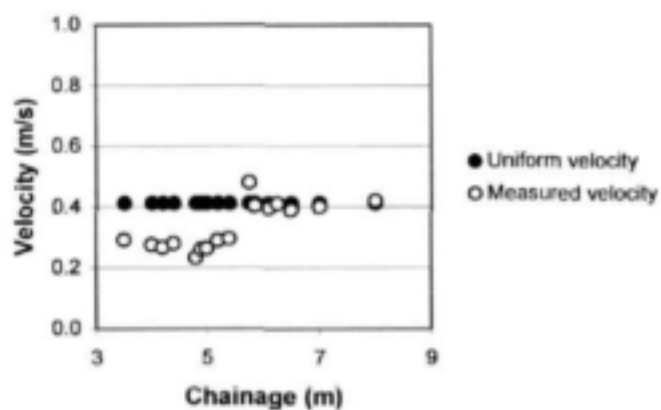


Figure F-20 Velocity measurements at 0.13 m from left side of the flume

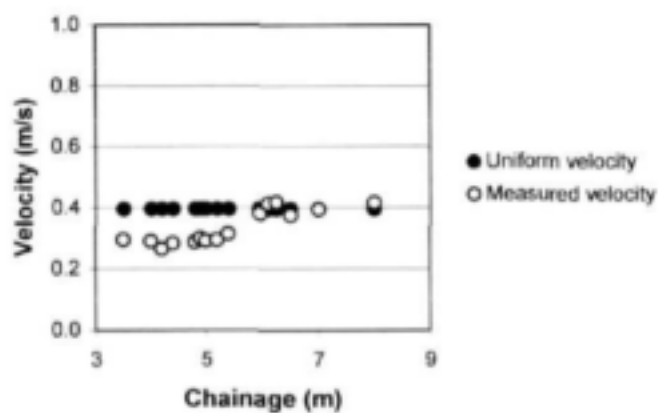


Figure F-21 Velocity measurements at 0.11 m from left side of the flume

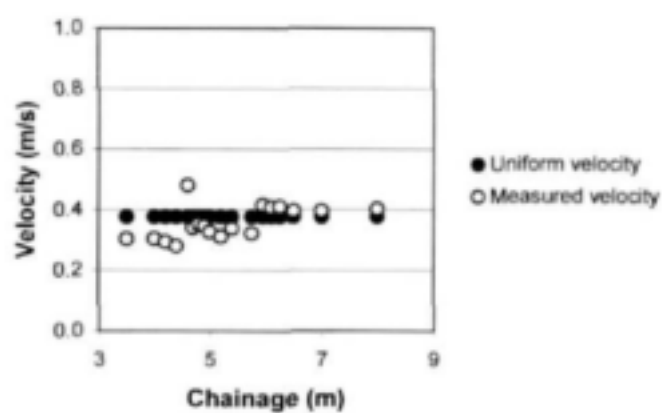


Figure F-22 Velocity measurements at 0.09 m from left side of the flume

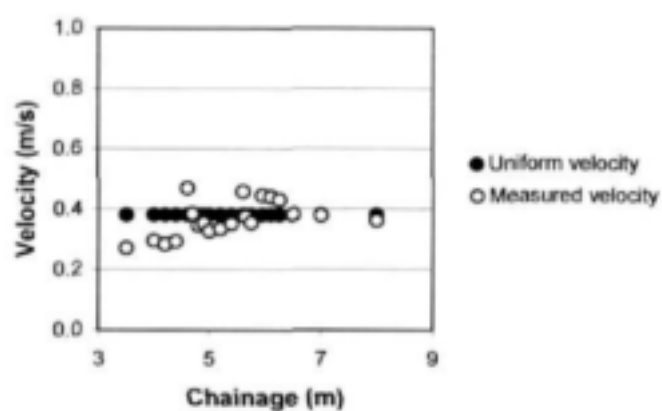


Figure F-23 Velocity measurements at 0.07 m from left side of the flume

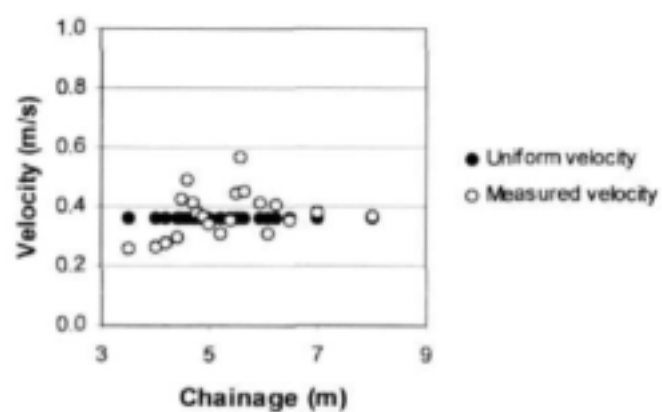


Figure F-24 Velocity measurements at 0.05 m from left side of the flume

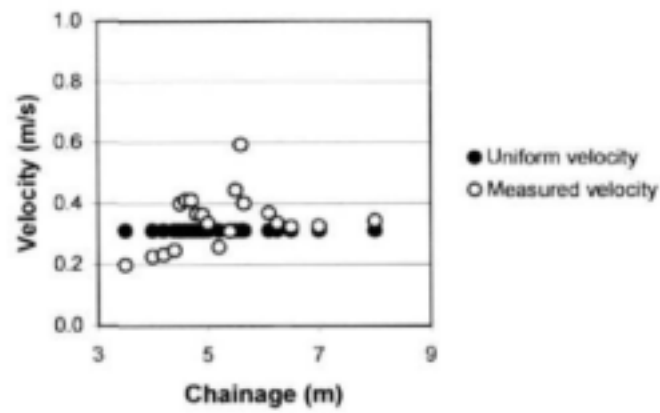


Figure F-25 Velocity measurements at 0.03 m from left side of the flume

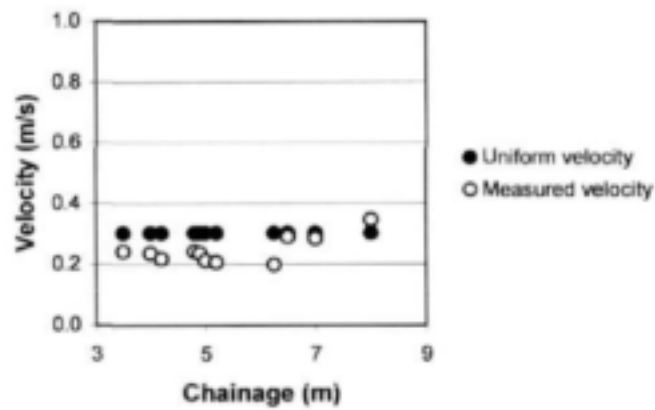


Figure F-26 Velocity measurements at 0.01 m from left side of the flume

Appendix F

Slope = 0.0011

Discharge = 0.005 m³/s

Condition: Two hemispheres at chainage 4.5 and 6.5 m

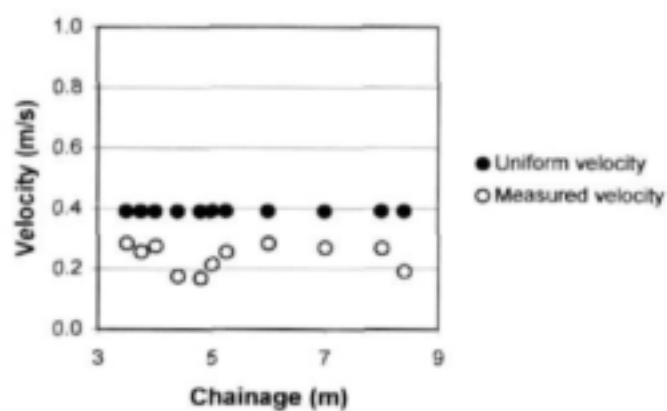


Figure F-27 Velocity measurements at 0.19 m from left side of the flume

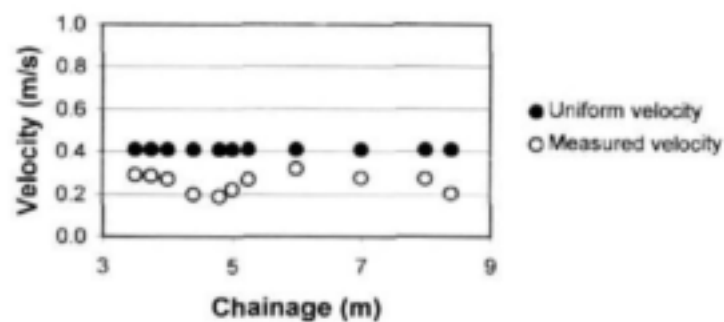


Figure F-28 Velocity measurements at 0.17 m from left side of the flume

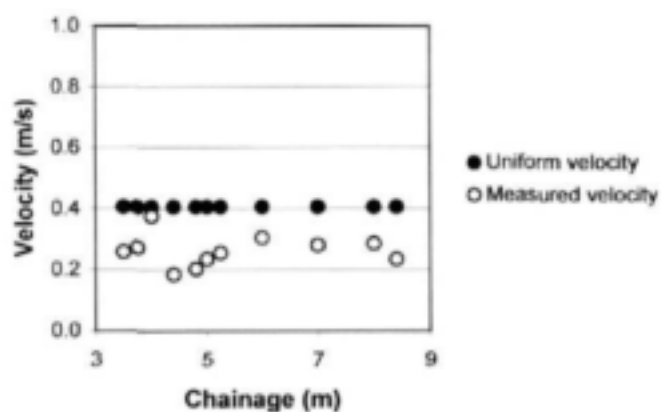


Figure F-29 Velocity measurements at 0.15 m from left side of the flume

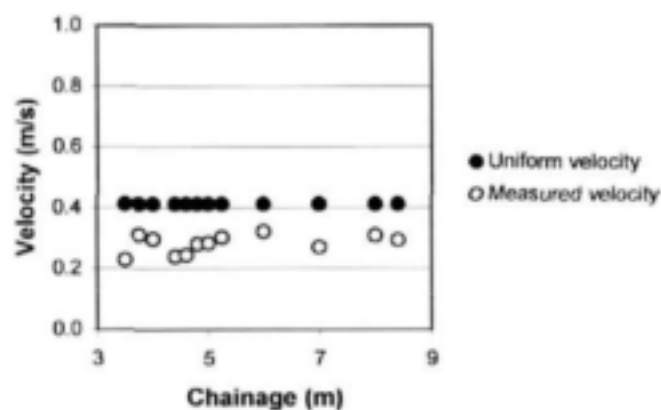


Figure F-30 Velocity measurements at 0.13 m from left side of the flume

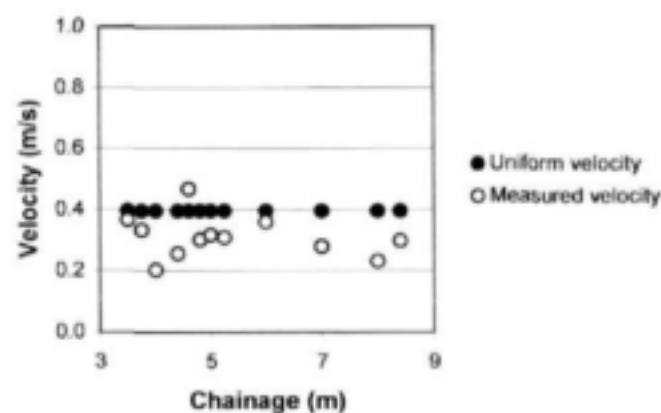


Figure F-31 Velocity measurements at 0.11 m from left side of the flume

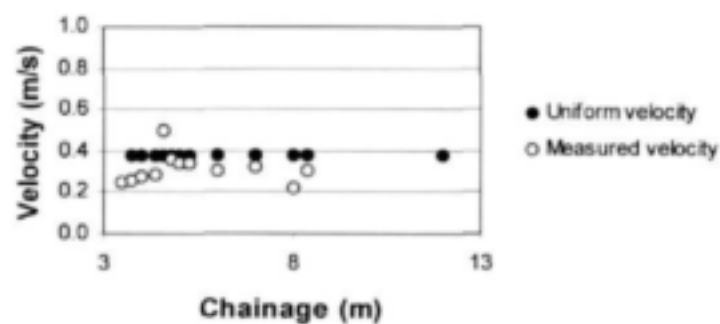


Figure F-32 Velocity measurements at 0.09 m from left side of the flume

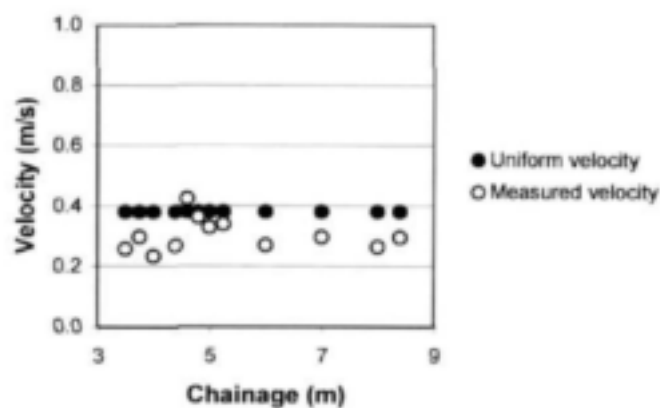


Figure F-33 Velocity measurements at 0.07 m from left side of the flume

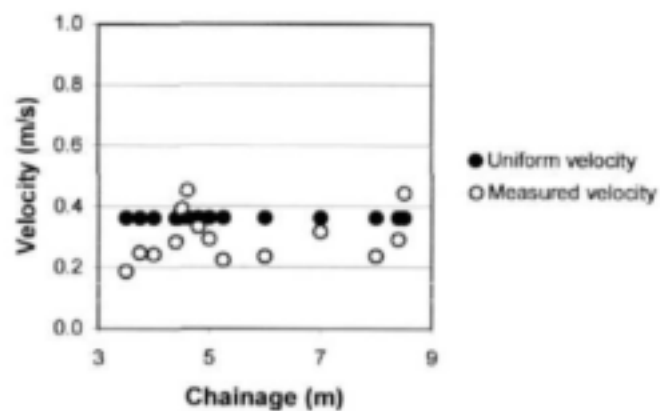


Figure F-34 Velocity measurements at 0.05 m from left side of the flume

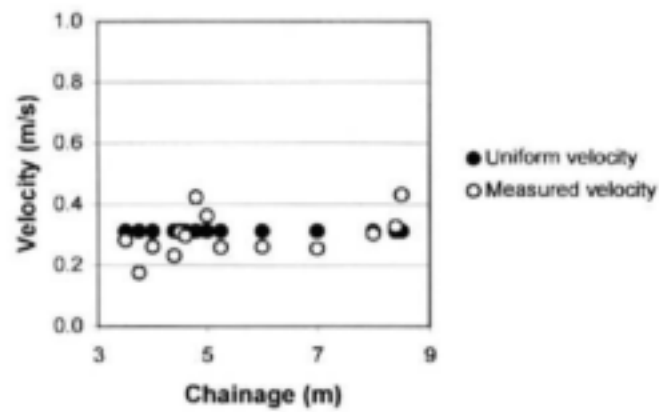


Figure F-35 Velocity measurements at 0.03 m from left side of the flume

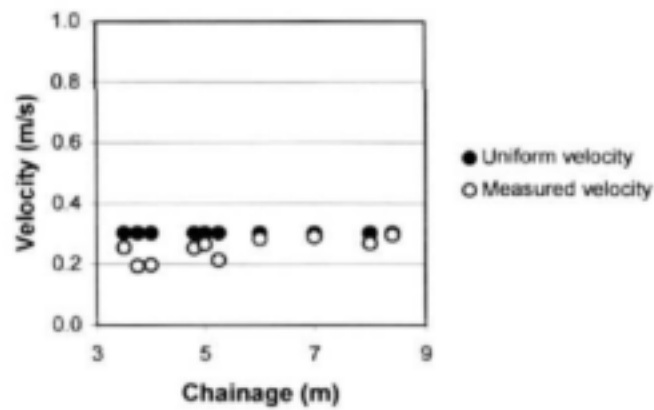


Figure F-36 Velocity measurements at 0.01 m from left side of the flume

APPENDIX G**COTTER RIVER CROSS-SECTIONAL DATA**

Table G-1 Spur Hole Site

Cross-section 1			Cross-section 2			Cross-section 3			Cross-section 4			Cross-section 5		
Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC
0.00	99.55	9.0	0.00	98.92	9.0	0.00	99.53	9.0	0.00	99.17	9.0	0.00	99.17	9.0
0.50	99.43	9.0	0.50	98.83	9.0	0.50	98.76	9.0	0.50	99.02	9.0	0.50	99.02	9.0
1.00	99.01	9.0	1.00	98.73	9.0	1.00	98.66	9.0	1.00	98.82	9.0	1.00	98.82	5.0
1.50	98.83	5.3	1.50	98.46	7.0	1.50	98.59	2.0	1.50	98.74	7.0	1.50	98.88	5.0
2.00	98.80	5.9	2.00	98.52	7.0	2.00	98.40	7.0	2.00	98.98	7.0	2.00	98.97	5.0
2.50	98.64	5.8	2.50	98.41	7.0	2.50	98.14	7.0	2.50	98.65	7.0	2.50	98.97	5.2
3.00	98.56	5.2	3.00	98.67	7.0	3.00	98.17	7.0	3.00	98.44	3.0	3.00	98.96	5.1
3.50	98.63	5.4	3.50	98.69	7.0	3.50	98.24	6.6	3.50	98.35	5.0	3.50	98.91	5.1
4.00	98.37	5.9	4.00	98.03	6.9	4.00	98.17	6.5	4.00	98.42	5.7	4.00	98.93	6.7
4.50	98.27	6.0	4.50	98.05	6.8	4.50	98.55	7.0	4.50	98.27	5.7	4.50	98.77	6.7
5.00	98.25	5.9	5.00	98.12	5.8	5.00	98.24	6.8	5.00	98.45	5.8	5.00	98.51	6.0
5.50	98.38	5.8	5.50	98.09	5.9	5.50	98.26	6.0	5.50	98.44	5.7	5.50	98.55	5.5
6.00	98.41	5.7	6.00	98.14	5.9	6.00	98.29	5.9	6.00	98.38	5.6	6.00	98.59	5.7
6.50	98.56	8.0	6.50	98.18	6.3	6.50	98.40	5.8	6.50	98.39	5.7	6.50	98.64	5.7
7.00	98.67	8.0	7.00	98.18	5.9	7.00	98.47	6.2	7.00	98.43	6.6	7.00	98.57	5.2
7.50	98.64	8.0	7.50	98.25	5.9	7.50	98.49	5.0	7.50	98.43	6.2	7.50	98.65	5.3
8.00	98.69	8.0	8.00	98.19	6.1	8.00	98.46	5.9	8.00	98.40	5.4	8.00	98.61	5.2
8.50	98.89	8.0	8.50	98.23	5.8	8.50	98.41	5.7	8.50	98.37	6.3	8.50	98.48	5.6
9.00	99.14	9.0	9.00	98.21	5.8	9.00	98.45	5.8	9.00	98.36	6.8	9.00	98.42	5.6
9.50	99.11	9.0	9.50	98.28	6.2	9.50	98.39	5.3	9.50	98.44	6.6	9.50	98.43	5.4
10.00	99.15	9.0	10.00	98.28	6.1	10.00	98.30	5.8	10.00	98.42	5.6	10.00	98.49	5.7
10.50	99.18	9.0	10.50	98.34	5.9	10.50	98.30	5.8	10.50	98.43	5.7	10.50	98.56	5.8
11.00	99.23	9.0	11.00	98.27	6.1	11.00	98.29	5.2	11.00	98.40	5.4	11.00	98.43	5.6
11.50	99.40	9.0	11.50	98.62	7.0	11.50	98.26	5.4	11.50	98.48	5.7	11.50	98.47	5.6
12.00	99.62	9.0	12.00	98.38	5.9	12.00	98.26	5.6	12.00	98.49	6.7	12.00	98.36	5.7
12.08	100.0	9.0	12.50	98.41	5.9	12.50	98.24	5.4	12.50	98.92	7.0	12.50	98.26	5.8
			13.00	98.56	6.8	13.00	98.30	5.5	13.00	99.02	7.0	13.00	98.36	5.7
			13.50	98.45	6.0	13.50	98.27	5.2	13.50	98.84	7.0	13.50	98.27	5.4
			14.00	98.45	8.0	14.00	98.31	6.6	14.00	98.63	6.7	14.00	98.29	5.6
			14.50	98.41	6.8	14.50	98.44	6.6	14.50	98.55	5.2	14.50	98.32	5.6
			15.00	98.76	7.0	15.00	98.48	6.4	15.00	98.60	5.6	15.00	98.41	5.4
			15.50	98.72	5.5	15.50	98.46	5.2	15.50	98.54	5.7	15.50	98.47	5.3
			16.00	98.90	6.1	16.00	98.45	5.8	16.00	98.60	5.5	16.00	98.56	5.0

Appendix G

Cross-section 1			Cross-section 2			Cross-section 3			Cross-section 4			Cross-section 5		
Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC
			16.50	98.90	6.0	16.50	98.41	5.4	16.50	98.62	6.2	16.50	98.62	5.0
			17.00	98.98	9.0	17.00	98.52	6.2	17.00	98.98	9.0	17.00	98.66	5.0
			17.50	99.00	9.0	17.50	98.61	2.0	17.50	99.17	9.0	17.50	98.71	9.0
			17.68	99.04	9.0	18.00	98.74	2.0	17.65	99.22	9.0	18.00	99.10	9.0
						18.50	98.93	9.0				18.50	99.53	9.0
						18.80	99.03	9.0				18.83	100.0	9.0

Ch – Distance across from Left Bank

H – Adjusted Bed Height

SC – Substrate Code

Sub Code = Substrate Code

1 = plant detritus

2 = mud / clay

3 = silt

4 = sand

5 = gravel

6 = cobble

7 = boulder

8 = bedrock

9 = terrestrial vegetation

FT = Flow Type

BSW = Broken Standing Wave

USW = Unbroken Standing Wave

RIPP = Rippled

SBT = Smooth Boundary Turbulent

NPF = No Perceptible Flow

BOI – Boil (Upwelling)

Q=Discharge (Ml/d)

WL = Water Level

LB = Left Bank

d/s = downstream

u/s = upstream

Substrate sizes – If x.0, then 100% of size 'x' present. Other decimals indicate a mix of neighbouring size classes. No. after decimal is the proportion of next size up that is present. E.g. 5.7 = 30% gravel and 70% cobbles.

Bed Heights and Water Levels are in metres above an arbitrary datum. Bed Height in **bold** is thalweg.

Appendix G

Spur Hole

Cross-section No. 1 (d/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 24/4/02			Med 20/6/02			High 30/5/02		
			Q=29.6	WL= 98.673		Q=109.6	WL= 98.845		Q=194.4	WL= 98.801	
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	99.546	9.0									
0.50	99.432	9.0									
1.00	99.005	9.0									
1.50	98.829	5.3									
2.00	98.801	5.9				0.05	0.03	SBT			
2.50	98.641	5.8	0.03	0.00	NPF	0.19	0.11	SBT			
3.00	98.565	5.2	0.11	0.18	SBT	0.32	0.76	SBT			
3.50	98.625	5.4	0.12	0.42	RIPP	0.33	0.67	RIPP			
4.00	98.374	5.9	0.31	0.59	RIPP	0.51	0.76	RIPP			
4.50	98.267	6.0	0.45	1.03	BSW	0.66	0.93	USW			
5.00	98.252	5.9	0.40	0.50	USW	0.68	1.08	BSW			
5.50	98.379	5.8	0.30	-0.12	RIPP	0.56	-0.13	BSW			
6.00	98.408	5.7	0.25	-0.01	SBT	0.5	0.1	BSW			
6.50	98.561	8.0	0.12	-0.05	NPF	0.32	0.92	BSW			
7.00	98.672	8.0	0.07	-0.03	NPF	0.21	0.42	RIPP			
7.50	98.635	8.0	0.06	-0.03	NPF	0.33	0.05	SBT			
8.00	98.685	8.0				0.23	-0.03	SBT			
8.50	98.894	8.0									
9.00	99.142	9.0									
9.50	99.111	9.0									
10.00	99.152	9.0									
10.50	99.182	9.0									
11.00	99.227	9.0									
11.50	99.396	9.0									
12.00	99.618	9.0									
12.08	100.000	9.0									

unsafe to measure depth & velocity

Appendix G

Spur Hole

Cross-section No. 2 (d/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 24/4/02			Med 20/5/02			High 30/5/02		
			Q=29.6 Depth (m)	WL= 98.460 Velocity (m/s)	FT	Q=109.6 Depth (m)	WL= 98.634 Velocity (m/s)	FT	Q=194.4 Depth (m)	WL= 98.723 Velocity (m/s)	FT
0.00	98.922	9.0									
0.50	98.833	9.0									
1.00	98.729	9.0									
1.50	98.455	7.0	0.02	0.00	NPF	0.17	0.22	SBT	0.28	0.17	SBT
2.00	98.515	7.0				0.10	0.21	SBT	0.36	0.22	SBT
2.50	98.406	7.0	0.08	0.17	SBT	0.33	0.24	RIPP	0.46	0.48	RIPP
3.00	98.671	7.0							0.06	0.41	RIPP
3.50	98.694	7.0							0.05	0.34	RIPP
4.00	98.026	6.9	0.36	0.00	NPF	0.55	0.00	RIPP	0.65	-0.01	RIPP
4.50	98.047	6.8	0.34	0.11	SBT	0.39	0.42	RIPP	0.69	0.16	SBT
5.00	98.116	5.8	0.37	0.21	SBT	0.54	0.37	SBT	0.66	0.42	SBT
5.50	98.093	5.9	0.34	0.18	SBT	0.48	0.27	SBT	0.60	0.39	SBT
6.00	98.140	5.9	0.30	0.06	SBT	0.46	0.24	SBT	0.56	0.25	SBT
6.50	98.183	6.3	0.30	0.23	SBT	0.38	0.26	SBT	0.55	0.37	SBT
7.00	98.179	5.9	0.25	0.22	SBT	0.41	0.39	SBT	0.56	0.31	SBT
7.50	98.248	5.9	0.22	0.27	SBT	0.41	0.43	SBT	0.55	0.44	SBT
8.00	98.192	6.1	0.25	0.26	SBT	0.43	0.43	SBT	0.53	0.48	SBT
8.50	98.233	5.8	0.24	0.33	SBT	0.41	0.36	SBT	0.52	0.53	SBT
9.00	98.212	5.8	0.23	0.21	SBT	0.42	0.34	RIPP	0.49	0.45	RIPP
9.50	98.277	6.2	0.12	0.24	RIPP	0.36	0.47	SBT	0.48	0.50	RIPP
10.00	98.283	6.1	0.17	0.18	SBT	0.40	0.45	SBT	0.44	0.61	RIPP
10.50	98.336	5.9	0.15	0.34	RIPP	0.34	0.60	RIPP	0.46	0.65	RIPP
11.00	98.273	6.1	0.19	0.24	SBT	0.39	0.46	RIPP	0.43	0.54	RIPP
11.50	98.619	7.0				0.01	-	SBT	0.12	0.61	RIPP
12.00	98.384	5.9	0.09	0.05	SBT	0.25	0.37	RIPP	0.39	0.47	SBT
12.50	98.408	5.9	0.04	0.05	SBT	0.24	0.22	SBT	0.37	0.31	SBT
13.00	98.556	6.8				0.12	0.32	SBT	0.16	0.29	SBT
13.50	98.452	6.0	0.02	0.00	NPF	0.20	0.22	SBT	0.25	0.27	SBT
14.00	98.453	8.0	0.01	0.00	NPF	0.20	0.17	SBT	0.27	0.31	SBT
14.50	98.410	6.8	0.03	0.00	NPF	0.21	0.14	SBT	0.39	0.18	SBT
15.00	98.762	7.0									
15.50	98.721	5.5							0.06	0.00	NPF
16.00	98.899	6.1									
16.50	98.904	6.0									
17.00	98.977	9.0									
17.50	99.001	9.0									
17.68	99.036	9.0									

Appendix G

Spur Hole

Cross-section No. 3 (mid)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02 Q=29.6 WL= 98.499			Med 20/6/02 Q=109.6 WL= 98.658			High 30/5/02 Q=194.4 WL= 98.755		
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	99.529	9.0									
0.50	98.757	9.0									
1.00	98.655	9.0				0.01	0.00	NPF	0.11	0.00	NPF
1.50	98.587	2.0				0.16	0.00	NPF	0.27	0.01	NPF
2.00	98.397	7.0	0.24	0.00	NPF	0.46	0.00	NPF	0.53	0.04	NPF
2.50	98.143	7.0	0.39	0.00	NPF	0.51	0.02	NPF	0.68	0.06	SBT
3.00	98.170	7.0	0.30	0.00	NPF	0.51	0.05	SBT	0.56	0.08	SBT
3.50	98.236	6.6	0.26	0.00	SBT	0.45	0.02	SBT	0.55	0.05	SBT
4.00	98.173	6.5	0.31	0.17	SBT	0.44	0.26	SBT	0.57	0.22	SBT
4.50	98.547	7.0				0.08	0.20	SBT	0.18	0.25	SBT
5.00	98.240	6.8	0.15	0.05	SBT	0.41	0.21	SBT	0.41	0.25	SBT
5.50	98.261	6.0	0.22	0.13	SBT	0.49	0.36	SBT	0.56	0.44	SBT
6.00	98.286	5.9	0.04	0.26	SBT	0.22	0.16	SBT	0.34	0.12	SBT
6.50	98.402	5.8	0.06	0.00	NPF	0.29	0.08	SBT	0.40	0.07	SBT
7.00	98.468	6.2	0.02	0.02	SBT	0.22	0.43	SBT	0.31	0.46	RIPP
7.50	98.489	5.0	0.06	0.00	NPF	0.24	0.20	SBT	0.36	0.17	SBT
8.00	98.462	5.9	0.04	0.16	SBT	0.16	0.04	SBT	0.30	0.01	SBT
8.50	98.413	5.7	0.09	0.22	RIPP	0.24	0.37	SBT	0.32	0.64	RIPP
9.00	98.445	5.8	0.10	-0.04	RIPP	0.21	0.08	SBT	0.36	0.24	BOI
9.50	98.387	5.3	0.13	0.24	RIPP	0.34	0.48	SBT	0.44	0.46	SBT
10.00	98.304	5.8	0.21	0.25	RIPP	0.34	0.46	SBT	0.44	0.52	SBT
10.50	98.295	5.8	0.20	-0.01	NPF	0.34	0.07	SBT	0.49	0.11	SBT
11.00	98.287	5.2	0.20	-0.02	NPF	0.35	0.24	SBT	0.49	0.45	SBT
11.50	98.255	5.4	0.25	0.28	SBT	0.43	0.61	RIPP	0.50	0.73	RIPP
12.00	98.255	5.6	0.23	0.48	RIPP	0.46	0.55	RIPP	0.56	0.58	RIPP
12.50	98.239	5.4	0.25	0.52	RIPP	0.43	0.57	RIPP	0.55	0.62	SBT
13.00	98.297	5.5	0.21	0.15	SBT	0.36	0.45	RIPP	0.53	0.67	SBT
13.50	98.268	5.2	0.20	0.24	RIPP	0.36	0.46	SBT	0.49	0.53	SBT
14.00	98.308	6.6	0.03	0.22	RIPP	0.38	0.51	SBT	0.44	0.59	SBT
14.50	98.442	6.6	0.09	0.19	RIPP	0.28	0.26	SBT	0.42	0.34	SBT
15.00	98.475	6.4	0.05	-0.07	SBT	0.16	0.25	SBT	0.36	0.30	RIPP
15.50	98.462	5.2	0.06	0.01	NPF	0.18	0.40	SBT	0.34	0.47	RIPP
16.00	98.452	5.8	0.01	0.00	NPF	0.24	0.16	SBT	0.33	0.25	SBT
16.50	98.413	5.4	0.09	0.04	NPF	0.27	0.13	SBT	0.34	0.35	SBT
17.00	98.519	6.2				0.11	0.08	SBT	0.10	0.20	SBT
17.50	98.605	2.0				0.06	0.00	NPF	0.16	0.01	SBT
18.00	98.741	2.0							0.06	0.00	NPF
18.50	98.929	9.0									
18.80	99.029	9.0									

Appendix G

Spur Hole

Cross-section No. 4 (u/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02			Med 20/5/02			High 30/5/02		
			Q=29.6	WL= 98.641	FT	Q=109.6	WL= 98.766	FT	Q=194.4	WL= 98.842	FT
			Depth (m)	Velocity (m/s)		Depth (m)	Velocity (m/s)		Depth (m)	Velocity (m/s)	
0.00	99.171	9.0									
0.50	99.016	9.0									
1.00	98.821	9.0							0.02	0.00	NPF
1.50	98.744	7.0				0.05	0.00	NPF	0.11	0.00	NPF
2.00	98.983	7.0									
2.50	98.648	7.0							0.05	0.00	NPF
3.00	98.437	3.0	0.28	-0.01	NPF	0.32	0.02	SBT	0.44	0.04	SBT
3.50	98.345	5.0	0.29	0.07	SBT	0.41	0.24	SBT	0.50	0.19	SBT
4.00	98.423	5.7	0.22	0.20	SBT	0.28	0.14	SBT	0.44	0.41	SBT
4.50	98.269	5.7	0.36	0.07	SBT	0.50	0.12	SBT	0.59	0.29	SBT
5.00	98.454	5.8	0.27	0.02	SBT	0.36	0.39	SBT	0.41	0.56	SBT
5.50	98.444	5.7	0.13	0.30	SBT	0.42	0.44	SBT	0.48	0.62	RIPP
6.00	98.383	5.6	0.26	0.26	SBT	0.41	0.42	SBT	0.51	0.44	RIPP
6.50	98.391	5.7	0.25	0.32	RIPP	0.41	0.33	SBT	0.50	0.46	RIPP
7.00	98.434	6.6	0.21	0.23	SBT	0.38	0.54	SBT	0.45	0.63	RIPP
7.50	98.429	6.2	0.24	0.11	RIPP	0.28	0.51	SBT	0.49	0.64	RIPP
8.00	98.399	5.4	0.24	0.20	SBT	0.36	0.31	SBT	0.51	0.49	RIPP
8.50	98.368	6.3	0.27	0.14	SBT	0.39	0.02	RIPP	0.51	0.46	RIPP
9.00	98.365	6.8	0.27	0.16	RIPP	0.41	0.38	RIPP	0.55	0.53	RIPP
9.50	98.439	6.6	0.18	0.13	SBT	0.33	0.38	RIPP	0.37	0.64	RIPP
10.00	98.420	5.6	0.23	0.23	RIPP	0.36	0.44	RIPP	0.49	0.59	RIPP
10.50	98.433	5.7	0.20	0.20	RIPP	0.36	0.40	SBT	0.45	0.56	RIPP
11.00	98.403	5.4	0.23	0.05	SBT	0.38	0.31	SBT	0.46	0.58	RIPP
11.50	98.475	5.7	0.24	0.01	NPF	0.29	0.02	SBT	0.36	0.10	RIPP
12.00	98.491	6.7	0.18	0.23	RIPP	0.06	0.32	RIPP	0.31	0.76	USW
12.50	98.922	7.0									
13.00	99.021	7.0									
13.50	98.844	7.0									
14.00	98.629	6.7	0.06	0.30	SBT	0.21	0.69	RIPP	0.32	0.82	RIPP
14.50	98.549	5.2	0.09	0.07	SBT	0.19	0.49	RIPP	0.34	0.41	RIPP
15.00	98.595	5.6	0.05	0.00	SBT	0.16	0.30	RIPP	0.29	0.50	SBT
15.50	98.535	5.7	0.12	0.17	SBT	0.25	0.14	SBT	0.34	0.21	SBT
16.00	98.604	5.5	0.07	0.00	SBT	0.08	0.00	NPF	0.02	0.00	NPF
16.50	98.621	6.2	0.02	0.00	NPF	0.04	0.00	NPF	0.12	0.00	NPF
17.00	98.984	9.0									
17.50	99.172	9.0									
17.65	99.218	9.0									

Appendix G

Spur Hole

Cross-section No. 5 (u/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02 Q=29.6 WL= 98.671			Med 20/6/02 Q=109.6 WL= 98.800			High 30/5/02 Q=194.4 WL= 98.893		
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	99.172	9.0									
0.50	99.016	9.0									
1.00	98.822	5.0							0.08	0.00	NPF
1.50	98.884	5.0							0.04	0.00	NPF
2.00	98.969	5.0									
2.50	98.967	5.2									
3.00	98.959	5.1									
3.50	98.912	5.1									
4.00	98.930	6.7									
4.50	98.773	6.7				0.02	0.00	NPF	0.09	0.00	NPF
5.00	98.506	6.0	0.07	0.00	NPF	0.28	0.02	NPF	0.36	-0.01	NPF
5.50	98.553	5.5	0.08	0.03	NPF	0.23	0.07	SBT	0.31	0.03	SBT
6.00	98.593	5.7	0.09	-0.03	SBT	0.24	0.11	SBT	0.35	0.12	SBT
6.50	98.639	5.7	0.04	0.07	SBT	0.27	0.26	RIPP	0.35	0.27	SBT
7.00	98.568	5.2	0.09	0.08	SBT	0.23	0.43	RIPP	0.34	0.68	RIPP
7.50	98.652	5.3	0.04	0.01	SBT	0.22	0.12	SBT	0.31	0.40	RIPP
8.00	98.607	5.2	0.06	0.10	SBT	0.31	0.49	RIPP	0.39	0.44	RIPP
8.50	98.476	5.6	0.16	-0.03	NPF	0.36	0.43	RIPP	0.45	0.55	RIPP
9.00	98.416	5.6	0.22	-0.01	SBT	0.39	0.09	SBT	0.46	0.63	RIPP
9.50	98.427	5.4	0.22	0.03	SBT	0.39	-0.07	SBT	0.49	0.09	SBT
10.00	98.488	5.7	0.20	0.17	SBT	0.36	0.13	SBT	0.50	0.32	RIPP
10.50	98.559	5.8	0.19	0.24	SBT	0.35	0.54	RIPP	0.46	0.87	RIPP
11.00	98.432	5.6	0.25	0.22	SBT	0.42	0.49	RIPP	0.51	0.51	SBT
11.50	98.474	5.6	0.26	0.18	SBT	0.43	0.22	SBT	0.55	0.56	SBT
12.00	98.363	5.7	0.27	0.19	SBT	0.48	0.34	SBT	0.60	0.47	SBT
12.50	98.263	5.8	0.37	0.22	SBT	0.54	0.28	SBT	0.61	0.38	SBT
13.00	98.357	5.7	0.38	0.14	SBT	0.56	0.25	SBT	0.64	0.37	SBT
13.50	98.267	5.4	0.37	0.10	SBT	0.55	0.23	SBT	0.64	0.31	SBT
14.00	98.291	5.6	0.37	0.19	SBT	0.53	0.23	SBT	0.56	0.39	SBT
14.50	98.317	5.6	0.35	0.30	RIPP	0.51	0.26	SBT	0.63	0.44	SBT
15.00	98.408	5.4	0.26	0.29	RIPP	0.40	0.43	SBT	0.52	0.65	RIPP
15.50	98.466	5.3	0.17	0.08	SBT	0.35	0.39	RIPP	0.46	0.60	SBT
16.00	98.562	5.0	0.11	-0.01	NPF	0.26	0.31	RIPP	0.39	0.24	SBT
16.50	98.625	5.0	0.05	0.00	NPF	0.21	0.11	SBT	0.31	0.34	SBT
17.00	98.663	5.0	0.01	0.00	NPF	0.16	0.00	NPF	0.26	0.01	SBT
17.50	98.708	9.0				0.11	0.00	NPF	0.20	0.00	NPF
18.00	99.098	9.0									
18.50	99.526	9.0									
18.83	100.000	9.0									

Table G-2 Vanity's Crossing Site

Cross-section 1			Cross-section 2			Cross-section 3			Cross-section 4			Cross-section 5		
Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC	Ch	H	SC
0.00	98.409	9.0	0.00	98.811	8.0	0.00	98.889	9.0	0.00	99.424	8.0	0.00	100.000	8.0
0.50	98.285	9.0	0.50	98.517	8.0	0.50	98.733	9.0	0.50	99.253	8.0	0.50	99.668	8.0
1.00	98.280	8.0	1.00	98.618	8.0	1.00	98.582	9.0	1.00	98.969	8.0	1.00	99.487	8.0
1.50	98.189	8.2	1.50	98.421	8.0	1.50	98.440	8.0	1.50	98.767	8.0	1.50	99.361	8.0
2.00	98.056	8.0	2.00	98.209	8.0	2.00	98.389	8.0	2.00	98.798	8.0	2.00	99.180	8.0
2.50	97.847	8.0	2.50	98.098	8.0	2.50	98.203	8.0	2.50	98.804	8.0	2.50	99.199	8.0
3.00	98.185	8.0	3.00	98.310	8.0	3.00	98.271	8.0	3.00	98.761	8.0	3.00	99.328	8.0
3.50	98.394	8.0	3.50	98.319	8.0	3.50	98.428	8.0	3.50	98.721	8.0	3.50	98.892	8.0
4.00	98.389	8.0	4.00	98.325	8.0	4.00	97.965	6.3	4.00	98.779	8.0	4.00	98.642	8.0
4.50	98.196	8.0	4.50	98.310	8.0	4.50	98.064	6.8	4.50	98.781	8.0	4.50	98.285	8.0
5.00	97.981	8.0	5.00	98.214	8.0	5.00	98.285	6.7	5.00	98.670	7.3	5.00	98.294	6.2
5.50	98.191	7.2	5.50	98.019	8.0	5.50	98.049	6.8	5.50	98.598	6.6	5.50	98.139	6.2
6.00	97.928	8.0	6.00	97.930	8.0	6.00	97.875	6.8	6.00	98.360	6.6	6.00	98.020	6.2
6.50	97.945	6.0	6.50	97.898	8.0	6.50	98.108	7.2	6.50	98.254	6.7	6.50	98.125	6.2
7.00	97.860	6.1	7.00	97.693	6.6	7.00	98.881	8.0	7.00	98.233	6.8	7.00	98.258	5.7
7.50	97.865	6.2	7.50	97.838	6.6	7.50	98.736	8.0	7.50	98.468	8.0	7.50	98.344	6.0
8.00	97.882	5.9	8.00	98.080	5.8	8.00	98.262	5.7	8.00	98.288	7.8	8.00	98.888	8.0
8.50	97.847	5.7	8.50	98.098	5.8	8.50	98.277	6.8	8.50	98.461	7.6	8.50	98.885	8.0
9.00	97.821	5.9	9.00	98.181	5.8	9.00	98.189	7.0	9.00	98.561	6.7	9.00	98.963	8.0
9.50	97.819	5.8	9.50	98.236	5.8	9.50	98.489	6.9	9.50	98.425	6.0	9.50	98.958	8.0
10.00	97.901	6.0	10.00	98.205	5.7	10.00	98.082	6.6	10.00	98.492	5.9	10.00	98.881	8.0
10.50	98.182	7.0	10.50	98.421	5.7	10.50	98.418	7.3	10.50	99.008	6.4	10.50	99.309	8.0
11.00	97.780	6.1	11.00	98.326	5.7	11.00	98.433	6.6	10.60	99.079	6.4	11.00	99.420	5.0
11.50	97.917	6.1	11.50	98.456	5.6	11.50	98.343	6.7				11.50	99.167	5.2
12.00	97.861	6.2	12.00	98.680	9.0	12.00	98.328	6.8				12.00	99.347	5.7
12.50	97.952	5.7	12.50	98.722	5.0	12.50	98.529	6.0				12.50	99.586	5.1
13.00	98.254	5.0	13.00	98.743	4.8	13.00	98.780	4.8				12.70	99.687	5.1
13.50	98.546	5.3	13.25	98.690	4.3	13.50	99.019	9.0						
14.00	98.646	5.3				13.60	99.225	9.0						
14.50	98.653	5.0												
14.85	99.114	5.2												

Ch – Distance across from Left Bank

H – Adjusted Bed Height

SC – Substrate Code

Appendix G

Vanity's Crossing

Cross-section No. 1 (d/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02 Q=28.6 WL= 98.073			Med 20/6/02 Q=113.1 WL= 98.195			High 30/5/02 Q=182.5 WL= 98.260		
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	98.409	9.0									
0.50	98.285	9.0									
1.00	98.280	8.0									
1.50	98.189	8.2				0.03	0.00	NPF	0.10	0.04	NPF
2.00	98.056	8.0	0.03	0.00	NPF	0.12	0.00	NPF	0.16	0.05	NPF
2.50	97.847	8.0	0.25	0.00	NPF	0.37	0.01	NPF	0.40	0.07	NPF
3.00	98.185	8.0				0.01	0.00	NPF	0.07	0.05	NPF
3.50	98.394	8.0									
4.00	98.389	8.0									
4.50	98.196	8.0							0.08	0.12	SBT
5.00	97.981	8.0	0.02	0.00	NPF	0.21	0.03	SBT	0.24	0.14	RIPP
5.50	98.191	7.2				0.19	0.09	RIPP	0.27	0.37	RIPP
6.00	97.928	8.0	0.19	-0.03	SBT	0.31	0.06	SBT	0.41	0.37	RIPP
6.50	97.945	6.0	0.12	0.35	RIPP	0.33	0.69	RIPP	0.36	0.82	RIPP
7.00	97.860	6.1	0.23	0.14	RIPP	0.37	0.41	RIPP	0.37	0.84	RIPP
7.50	97.865	6.2	0.21	0.10	SBT	0.39	0.46	RIPP	0.36	0.49	RIPP
8.00	97.882	5.9	0.20	0.33	SBT	0.38	0.46	RIPP	0.42	0.66	RIPP
8.50	97.847	5.7	0.22	0.39	SBT	0.38	0.43	RIPP	0.32	0.72	RIPP
9.00	97.821	5.9	0.15	0.20	SBT	0.31	0.73	RIPP	0.38	0.62	USW
9.50	97.819	5.8	0.15	0.12	SBT	0.37	0.78	RIPP	0.38	1.27	USW
10.00	97.901	6.0	0.22	0.62	RIPP	0.33	1.03	RIPP	0.39	1.23	USW
10.50	98.182	7.0				0.37	0.52	BSW	0.41	0.70	RIPP
11.00	97.780	6.1	0.18	0.58	USW	0.38	0.82	RIPP	0.44	0.81	RIPP
11.50	97.917	6.1	0.23	0.19	RIPP	0.35	0.59	RIPP	0.40	1.09	RIPP
12.00	97.861	6.2	0.23	0.48	RIPP	0.36	0.53	RIPP	0.36	0.57	RIPP
12.50	97.952	5.7	0.08	0.18	SBT	0.28	0.09	SBT	0.31	0.41	SBT
13.00	98.254	5.0							0.10	-0.06	SBT
13.50	98.546	5.3									
14.00	98.646	5.3									
14.50	98.653	5.0									
14.85	98.114	5.2									

Appendix G

Vanity's Crossing

Cross-section No. 2 (d/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02			Med 20/5/02			High 30/5/02		
			Q=28.6 Depth (m)	WL= 98.198 Velocity (m/s)	FT	Q=113.1 Depth (m)	WL= 98.319 Velocity (m/s)	FT	Q=182.5 Depth (m)	WL= 98.456 Velocity (m/s)	FT
0.00	98.811	8.0									
0.50	98.517	8.0									
1.00	98.618	8.0									
1.50	98.421	8.0									
2.00	98.209	8.0				0.03	0.00	NPF			
2.50	98.098	8.0	0.09	0.00	NPF	0.29	0.15	RIPP			
3.00	98.310	8.0				0.08	0.11	RIPP			
3.50	98.319	8.0									
4.00	98.325	8.0									
4.50	98.310	8.0				0.02	-	RIPP			
5.00	98.214	8.0	0.08	0.03	NPF	0.36	0.73	RIPP			
5.50	98.019	8.0	0.12	0.29	RIPP	0.34	1.21	USW			
6.00	97.930	8.0	0.05	0.55	RIPP	0.32	1.03	BSW			
6.50	97.898	8.0	0.20	0.27	RIPP	0.58	0.63	BSW			
7.00	97.693	6.6	0.50	1.01	BSW	0.56	0.92	BSW			
7.50	97.838	6.6	0.32	0.33	RIPP	0.39	0.73	RIPP			
8.00	98.080	5.8	0.20	0.01	RIPP	0.26	0.54	RIPP			
8.50	98.098	5.8	0.10	0.10	RIPP	0.22	0.21	SBT			
9.00	98.181	5.8	0.03	0.00	NPF	0.10	0.49	RIPP			
9.50	98.236	5.8				0.16	0.21	RIPP			
10.00	98.205	5.7				0.12	0.37	RIPP			
10.50	98.421	5.7									
11.00	98.326	5.7									
11.50	98.456	5.6									
12.00	98.680	9.0									
12.50	98.722	5.0									
13.00	98.743	4.8									
13.25	98.690	4.3									

unsafe to measure depth & velocity

Appendix G

Vanity's Crossing

Cross-section No. 3 (mid)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02 Q=28.6 WL= 98.472			Med 20/6/02 Q=113.1 WL= 98.494			High 30/5/02 Q=182.5 WL= 98.579		
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	98.889	9.0									
0.50	98.733	9.0									
1.00	98.582	9.0				0.09	0.00	NPF			
1.50	98.440	8.0	0.04	0.00	NPF	0.23	0.01	SBT			
2.00	98.389	8.0	0.04	0.00	NPF	0.34	0.03	SBT			
2.50	98.203	8.0	0.20	0.00	NPF	0.46	0.06	SBT			
3.00	98.271	8.0	0.15	-0.01	SBT	0.41	0.05	RIPP			
3.50	98.428	8.0	0.06	-0.21	SBT	0.25	-0.03	RIPP			
4.00	97.965	6.3	0.53	-0.09	RIPP	0.72	0.65	RIPP			
4.50	98.064	6.8	0.35	0.30	RIPP	0.66	0.72	RIPP			
5.00	98.285	6.7	0.28	0.45	RIPP	0.53	0.59	USW			
5.50	98.049	6.8	0.28	0.82	USW	0.56	0.75	USW			
6.00	97.875	6.8	0.45	0.70	USW	-	1.38	USW			
6.50	98.108	7.2	0.23	0.28	RIPP	0.51	0.47	USW			
7.00	98.881	8.0									
7.50	98.736	8.0									
8.00	98.262	5.7	0.08	0.00	NPF	0.10	0.00	NPF			
8.50	98.277	6.8	0.15	0.00	SBT	0.26	-0.06	RIPP			
9.00	98.189	7.0	0.12	0.27	SBT	0.31	0.47	RIPP			
9.50	98.489	6.9				0.06	0.39	RIPP			
10.00	98.082	6.6	0.30	0.01	SBT	0.42	0.39	RIPP			
10.50	98.418	7.3				0.33	0.57	RIPP			
11.00	98.433	6.6				0.28	0.82	RIPP			
11.50	98.343	6.7				0.13	0.09	RIPP			
12.00	98.328	6.8				0.13	0.72	RIPP			
12.50	98.529	6.0									
13.00	98.780	4.8									
13.50	99.019	9.0									
13.60	99.225	9.0									

unsafe to measure depth & velocity

Appendix G

Vanity's Crossing

Cross-section No. 4 (u/s)

Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02			Med 20/5/02			High 30/5/02		
			Q=28.6 Depth (m)	WL= 98.809 Velocity (m/s)	FT	Q=113.1 Depth (m)	WL= 98.940 Velocity (m/s)	FT	Q=182.5 Depth (m)	WL= 99.032 Velocity (m/s)	FT
0.00	99.424	8.0									
0.50	99.253	8.0									
1.00	98.969	8.0									
1.50	98.767	8.0	0.07	0.00	NPF	0.12	0.01	NPF			
2.00	98.798	8.0	0.23	0.00	NPF	0.46	0.05	SBT			
2.50	98.804	8.0	0.05	0.00	NPF	0.25	0.04	SBT			
3.00	98.761	8.0	0.07	0.00	NPF	0.23	0.09	SBT			
3.50	98.721	8.0	0.10	0.00	NPF	0.26	0.14	SBT			
4.00	98.779	8.0	0.10	0.00	NPF	0.11	0.55	RIPP			
4.50	98.781	8.0	0.03	0.00	NPF	0.21	1.17	RIPP			
5.00	98.670	7.3	0.15	0.00	NPF	0.22	0.89	RIPP			
5.50	98.598	6.6	0.32	-0.03	NPF	0.54	0.06	RIPP			
6.00	98.360	6.6	0.35	-0.01	SBT	0.42	0.13	RIPP			
6.50	98.254	6.7	0.49	0.08	SBT	0.69	0.25	RIPP			
7.00	98.233	6.8	0.43	0.03	SBT	0.72	0.51	RIPP			
7.50	98.468	8.0	0.41	0.27	SBT	0.56	1.05	RIPP			
8.00	98.288	7.8	0.64	0.25	SBT	0.74	0.66	RIPP			
8.50	98.461	7.6	0.46	0.21	SBT	0.65	0.92	RIPP			
9.00	98.561	6.7	0.40	-0.06	SBT	0.61	0.43	SBT			
9.50	98.425	6.0	0.28	0.20	SBT	0.56	0.52	SBT			
10.00	98.462	5.9	0.22	-0.04	SBT	0.55	0.23	SBT			
10.50	99.008	6.4									
10.60	99.079	6.4									

unsafe to measure depth & velocity

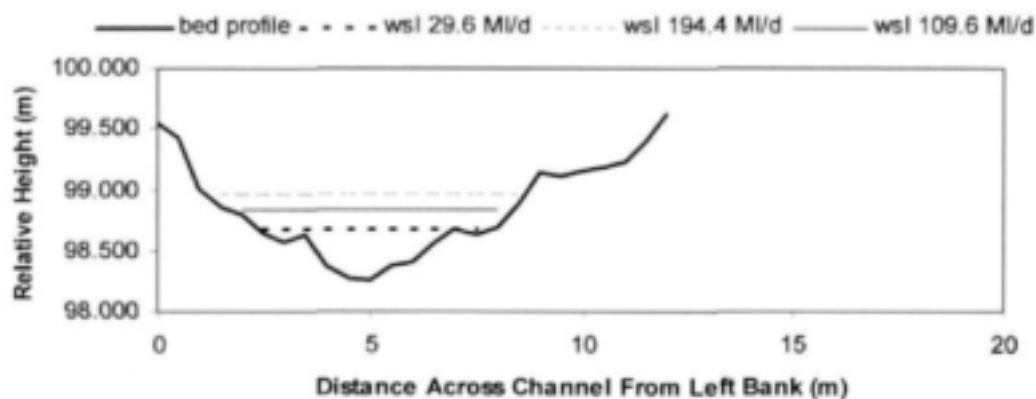
Vanity's Crossing

Cross-section No. 5 (u/s)

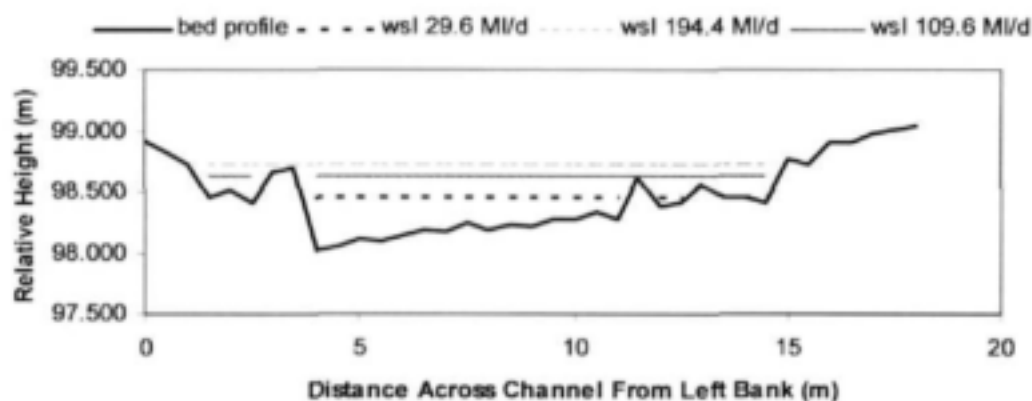
Distance Across from LB (m)	Adjusted Bed Height (m)	Sub Code	Low 23/4/02 Q=28.6 WL= 98.874			Med 20/6/02 Q=113.1 WL= 98.983			High 30/5/02 Q=182.5 WL= 99.101		
			Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT	Depth (m)	Velocity (m/s)	FT
0.00	100.000	8.0									
0.50	99.868	8.0									
1.00	99.487	8.0									
1.50	99.361	8.0									
2.00	99.180	8.0									
2.50	99.199	8.0									
3.00	99.328	8.0									
3.50	98.892	8.0				0.12	-0.22	RIPP			
4.00	98.642	8.0	0.36	0.09	RIPP	0.34	0.16	RIPP			
4.50	98.285	8.0	0.56	0.28	RIPP	0.32	0.79	RIPP			
5.00	98.294	6.2	0.61	0.24	RIPP	0.72	0.75	RIPP			
5.50	98.139	6.2	0.70	0.25	RIPP	0.93	0.73	RIPP			
6.00	98.020	6.2	0.65	0.05	RIPP	0.84	0.36	RIPP			
6.50	98.125	6.2	0.56	-0.01	RIPP	0.76	0.49	RIPP			
7.00	98.258	5.7	0.46	0.12	RIPP	0.66	0.45	RIPP			
7.50	98.344	6.0	0.44	-0.01	RIPP	0.62	0.09	RIPP			
8.00	98.888	8.0				0.10	-0.12	RIPP			
8.50	98.885	8.0				0.01	-	SBT			
9.00	98.963	8.0				0.04	-	USW			
9.50	98.958	8.0				0.26	0.25	USW			
10.00	98.881	8.0									
10.50	99.309	8.0									
11.00	99.420	5.0									
11.50	99.167	5.2									
12.00	99.347	5.7									
12.50	99.586	5.1									
12.70	99.687	5.1									

unsafe to measure depth & velocity

Spur Hole C/S 1



Spur Hole C/S 2



Spur Hole C/S 3

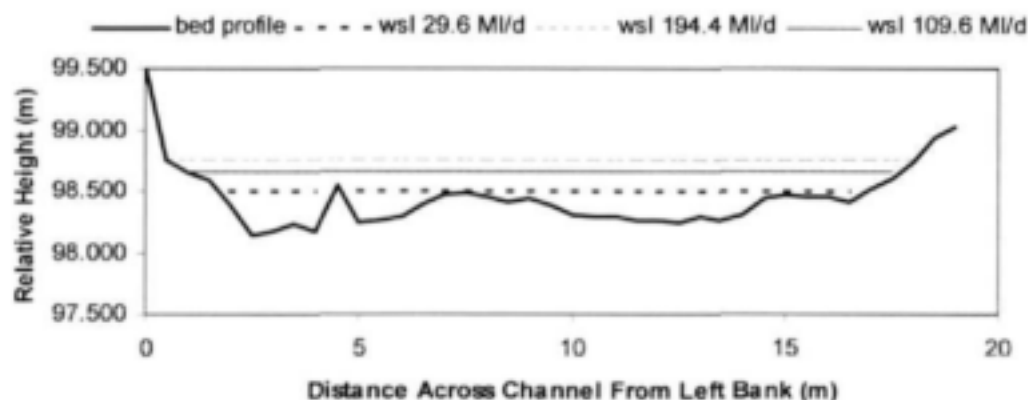
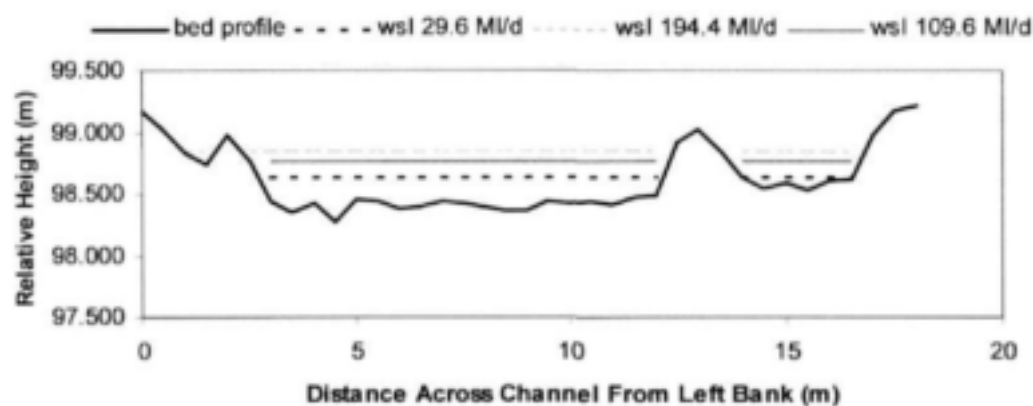


Figure G-1 Bed profiles for cross-sections 1, 2 and 3 at Spur Hole

Spur Hole C/S 4



Spur Hole C/S 5

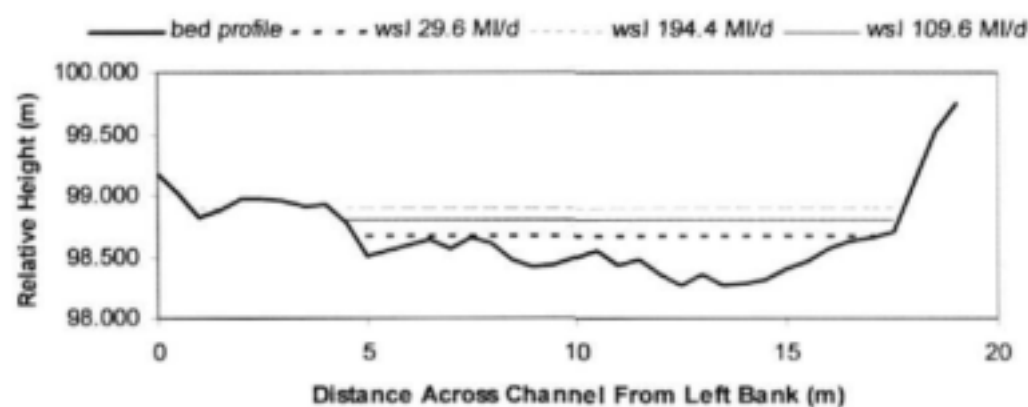


Figure G-2 Bed profiles for cross-sections 4 and 5 at Spur Hole

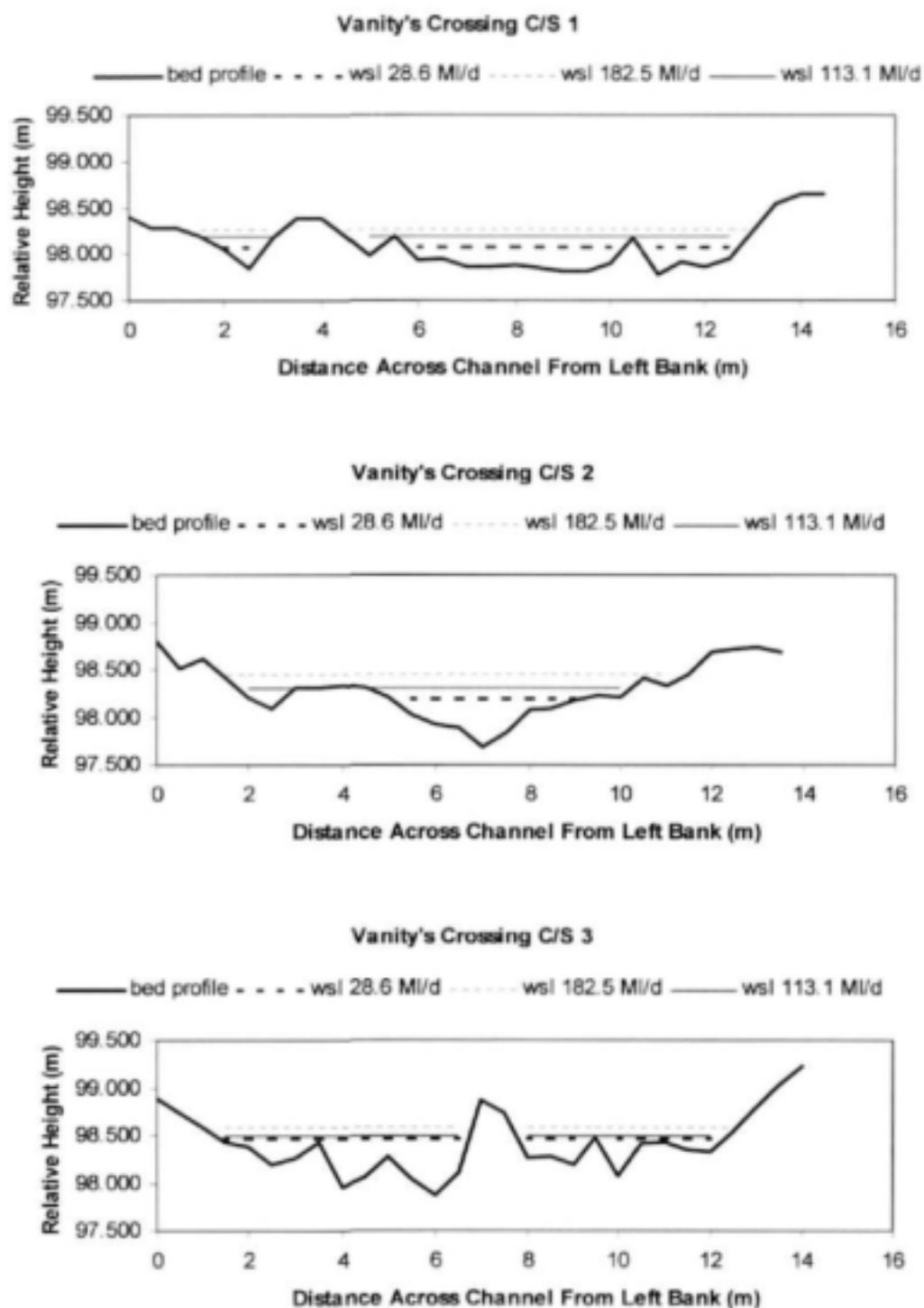


Figure G-3 Bed profiles for cross-sections 1, 2 and 3 at Vanity's Crossing

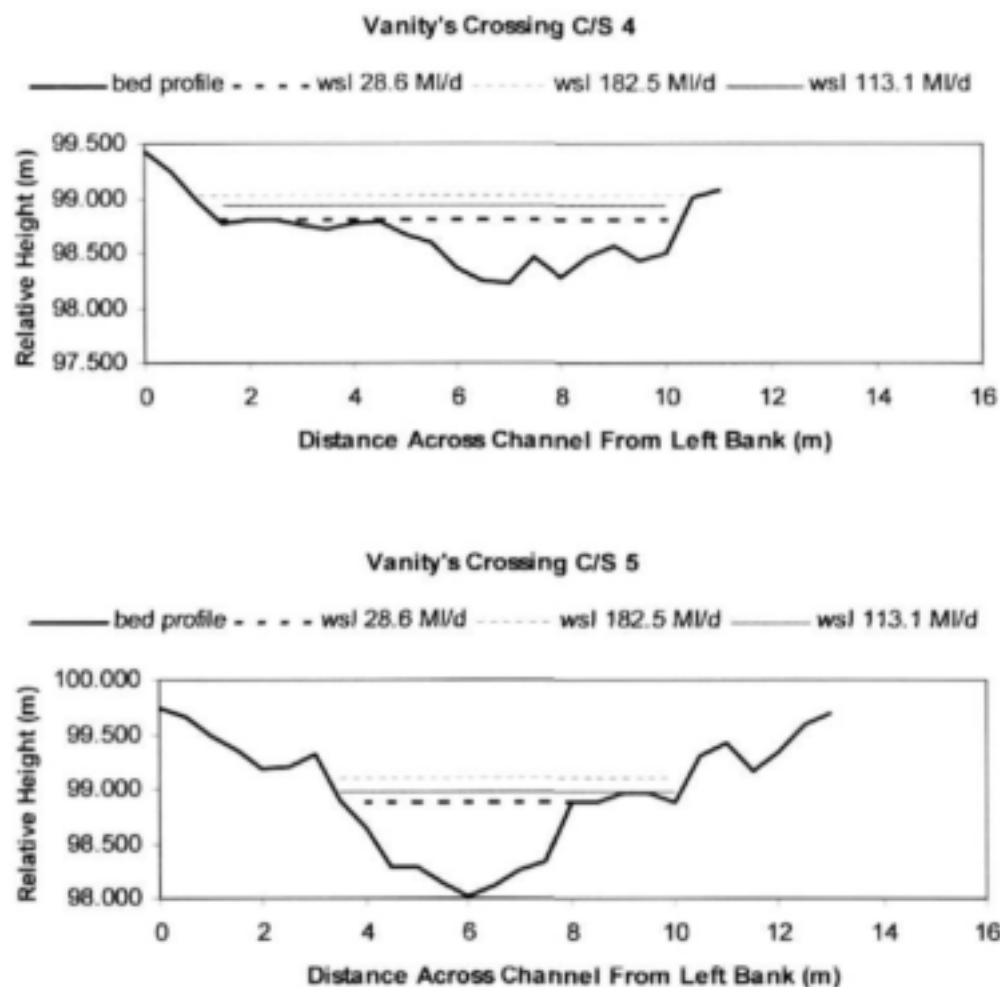


Figure G-4 Bed profiles for cross-sections 4 and 5 at Vanity's Crossing

Other related WRC reports available:

Hydraulics for determination of the ecological reserve for rivers.

Jordanova AA; Birkhead AL; James CS; Kleynhans CJ

Implementation of the ecological Reserve requires the integration of the hydrology and the biotic requirements. This project aims to provide this integration in a way that may be used by experts and managers for both the intermediate and comprehensive Reserve, and also to define the hydraulic characteristics of instream habitats.

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