SCALE MODELS OF GRAVEL BED RIVERS

By

ALAIN PAUL PARENT

B.Sc., Université de Montréal, 1985

A THESIS SUBMITTED IN PARTIAL FULFILLMENT

OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

Department of Geography

We accept this thesis as conforming to

the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

September 1988

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Department of Geography

The University of British Columbia
1956 Main Mall
Vancouver, Canada
V6T 1Y3

Date September 1, 1988

DE-6(3/81)

Abstract

The combined studies of flow processes in a full size gravel-bed river and its associated small-scale model have demonstrated the applicability of the similitude concepts as well as the potential of scale models for quantitative research in fluvial geomorphology. This scaling investigation considered the comparison of velocity profiles, shear stress measurements, flow structure and surface bed material samples. An extension of the strict comparison on a detailed level of field and laboratory processes in a specific case consisted in the proposition of a "generic model" framework according to which laboratory systems are viewed as part of the same family as field ones if some crucial conditions are satisfied. The pool-riffle sequence, used as a vehicle for the demonstration herein, is presented as a poorly known but ubiquitous river phenomenon which would benefit from such research methodology. The laboratory study performed herein revealed the need for an appraisal of the variability of shear stress estimates in intermediate relative roughness flows such as field size gravel-bed rivers. Knowledge of elements of the history of the prototype (field) river was also demonstrated to be important for the appraisal of model performance.

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List of symbols

b	Hydraulic geometry (depth/discharge) exponent
C_d	Drag coefficient
d	Mean channel depth
D_i	Grain size diameter (subscript refers to percentiles of the size distribution)
f	Hydraulic geometry (width/discharge) exponent
ff	Friction factor
F_s	Shields' parameter
F_d	Drag force
F_g	Gravitational force
F_i	Inertia force
F_v	Viscous force
g	Gravitational acceleration
k_s	Equivalent sand roughness
L	Length scale
Q	Channel discharge
R	Hydraulic radius
Re	Flow Reynolds number
Rp	Particle Reynolds number
s	Submerged specific weight of sediment

S	Energy gradient
V	Mean flow velocity
V'	Average velocity fluctuations
V_s	Particle settling velocity
V_*	Shear velocity
W	Mean channel width
y	Local flow depth
Φ	Function of
γ	Specific weight
κ	Von Karman constant
λ_x	Scale ratio (prototype/model) of characteristic parameter x
μ	Dynamic viscosity
ν	Kinematic viscosity
π_i	Dimensionless number
ρ	Density
τ_0	Boundary shear stress
τ_c	Shields' critical shear stress

Acknowledgements

It is a great pleasure to express my most sincere recognition to my supervisor, Dr Michael Church, for his thoughtful guidance and his intellectual contribution which greatly helped the proper completion of this research project. Dr Church provided the framework to narrow my vast initial intentions to a project of a size appropriate to the Master's level. I would also like to thank Dr Douw G. Steyn for his comments and his support with intrumentation. I am also grateful to Dr Olav Slaymaker, to Dr Edward Hickin from the Department of Geography at Simon Fraser University and to Dr Michael C. Quick from the Department of Civil engineering of the University of British Columbia who kindly loaned their current meters for the field measurements.

The help of Alastair Ross and Michel Lapointe during the field work (that means under very nasty conditions) was very much appreciated. Michel has also been a most interesting colleague to exchange with in Moliere's language. In the laboratory and in general, the help of and the interactions with John Wolcott were also a much appreciated stimulus. Discussions with my colleague David McLean were always lessons with intellectual fallouts. I am also grateful to the people at the U.B.C. Research Forest who provided equipment and support for the field work. Finally the help of Sylvie Lalonde in the field and with some of the figures is gratefully acknowledged.

Financial support to the author was provided via a grant and fellowship from the Natural Sciences and Engineering Research Council of Canada.

The Sands of Time were eroded by

The River of Constant Change

1.0 Introduction

Since the introduction of quantitative methods into geomorphology, the dominant, functional approach to landform study (Chorley, 1978) has yielded increasing insight into the behaviour of geomorphological systems. In fluvial geomorphology, the first milestone of those early quantitative developments is perhaps the classical textbook of Leopold and others (1964). Amongst the main contributions of functional studies to geomorphology, the recognition of some fairly consistent scale relationships involving morphological and/or hydrodynamic parameters of stream channels has been of utmost interest.

Empirical methods which first lead to those generalizations regarding alluvial channel behaviour also raised some crucial questions about the mechanisms which were involved. In consequence, the mainly empirical functional approach gradually gave way to process-oriented studies which were aimed at developing laws of stream behaviour. However, there are considerable difficulties as well as much work ahead before such laws can be developed. This situation results from the immense complexity and the size of the fluvial system, the episodic and irregular occurence of geomorphologically effective events, the temporal unsteadiness of their forcing functions, and from the difficulties to obtain representative quantitative measurements of the parameters which govern the response of the fluvial system.

Consequently, the change in the scope of research in fluvial geomorphology was necessarily accompanied by increased consideration to geomorphological subjects more restricted in space and time (the "realistic approach" of Chorley, 1978). Those smaller, more manageable systems appeared susceptible to bring insight to some particularly relevant problems concerning the behaviour of the fluvial system.

Concurrently, another fact rapidly emerged: the measurement and subsequent treatment of average hydrodynamic values could not fulfill the objective of a complete physical understanding of numerous features of the fluvial system. Average-value formulations in river studies have been helpful in many problems (mainly of practical nature) up to a point: for instance, Church (1985) has reviewed the usefulness and limitations of average-value formulations for the estimation of sediment transport in gravel-bed rivers. Also the mathematical formulations, which have been so popular since the emergence of digital computers (and which are, incidentally, mostly based on average hydrodynamic quantities), could not bring real progress to the subject of fluvial geomorphology. Acknowledging this situation and the fact that water flow can be described by its mean or fluctuating components, it has become increasingly evident that detailed observations, in space and time, of the flow dynamics and structure over particular configurations of interest, could improve our physical understanding of the interactions amongst river hydrodynamics, sediment transport phenomena and the geometrical configuration of stream boundaries.

However, detailed flow measurements are difficult and sometimes impractical to perform in full-scale situations: this is due to inherent characteristics of geomorphological systems mentioned above. Hence, an alternative to field studies has been to work under experimental conditions. Physical (or scale) models of rivers based on the theory of similarity have been extensively used in river engineering generally in order to solve some specific problems related to river management. In geomorphology, physical models generally have been used from a more descriptive perspective (for instance, consider the work reported by Schumm and others, 1987) although some authors have considered the principles of physical modelling, as prescribed by the theory of similarity, to study under controlled conditions some aspects of river behaviour (Southard and others, 1980,1984).

The following quotations summarize well the viewpoint of most geomorphologists regarding physical models:

> " Scale modelling is essentially an empirical exercise whose value is limited in a geomorphic context by the extended spatial scales common to geomorphological problems. Despite the use of various modelling strategies, geomorphology remains essentially a field science."
> (Knighton, 1984, p.6)

> " Although hardware models have been extensively used for descriptive and predictive purposes, they are inherently incapable of providing the last link in the chain of scientific investigation, the theory or the law."
> (Mosley and Zimpfer, 1978, p.450)

The bases for such statements, which may hold for many geomorphological situations, were however not formally exposed. In a recent book, Schumm and others (1987) remark on the various advantages of physical models but also consider scale modelling as too inexact a science at present to provide reliable quantitative results which can be applied to natural systems. The statements of Schumm and others are too general and susceptible to restrain quantitative fluvial geomorphology: scale modelling, which is supported by formal reasoning known as similarity theory, can be used in some circumstances as a tool for research. The range of conditions for which this is possible must be established.

Consequently, the hypothesis to be studied herein can be formulated as follows. Undistorted scale models of alluvial rivers constructed on similarity principles can be used for the collection and analysis of quantitative hydrodynamic measurements that would lead to a better understanding of mechanical processes in rivers and eventually to the development of laws or theories of alluvial river behaviour. Otherwise formulated, if the theoretical framework provided by the concept of similarity be respected, physical models can be used not solely as descriptive tools but also in order to systematically collect and analyze fundamental observations about river dynamics. Once a laboratory situation is shown to be similar to the field, generalization of the observations collected in the model is allowed.

A natural extension of the principal hypothesis of the current research, which focuses on a particular field-laboratory comparison, is the proposal of a "generic" modelling strategy for geomorphological research under controlled conditions. If the modelling exercise turns out to be appropriate for research and hence compatible with the processes observed in the field, physical modelling can be considered to be applicable to a range of rivers and consequently to a more general situation. The concept of the generic model is further discussed through this thesis.

In the perspective of the above arguments, this thesis is mainly concerned with the practical verification of similarity and hence with the possibility of working under controlled conditions for rivers. This exercise is performed for a particular geomorphological phenomenon which seems to deserve attention. The choice of the phenomenon of interest is also guided by the state of the system which prevails under formative conditions. Hence work in low transport intensity rivers represents a first practical aspect of selection. In the field, such a state is most often met in small to intermediate size gravel-bed rivers. Considering the physical limitations in the scaling of length scales of natural rivers (especially the bed material size distribution for sand and finer grained full-scale rivers), small to intermediate size, relatively coarse bedded rivers also represent a practical alternative for the current research strategy.

An immense advantage of working under experimental conditions is that the experimentalist has the freedom to manipulate the system and thereby to characterize its response with regard to the controlling parameters. Also the timescale of observation is considerably reduced. In addition, for the case of low transport intensity rivers, only the highest flows which transport a significant amount of sediment need to be modeled. This results in the simplification of the hydrograph of a particular situation and further contracts the timescale of operation. The principles of the theory of similarity, which bring the theoretical support to work under controlled, small-scale experimental conditions, are fully described in chapter 2.

The most fundamental characteristic feature of low transport intensity rivers consists of the pool-riffle sequence. Because of its ubiquity in such rivers, the pool-riffle sequence represents a natural vehicle for the current research. As the pool-riffle sequence is the river configuration in which measurements will be performed, a brief statement on the subject is in order. This material, contained in an appendix, demonstrates our limited understanding regarding the morphology and sedimentology of the pool-riffle sequence as well as the lack of knowledge of the flow processes which entail its formation and maintenance.

In this project, hydrodynamic measurements were performed in a field pool-riffle situation, Blaney creek (described in chapter 3), and in a corresponding physical model (described in chapter 4). The comparison of hydrodynamic measurements from the laboratory and field exercises, that is the test of the current research hypothesis, is also presented in chapter 4. Both chapters also feature some extended discussion about the observed processes and their implications regarding the creek condition.

2.0 Scale models of alluvial rivers

2.1 Generalities

This chapter presents theoretical and empirical support for hydrodynamic modelling and introduces the criteria which determine the range of situations for which undistorted physical models are a useful alternative for research practice.

Historically, scale modelling techniques were almost entirely developed in engineering. About a century ago, these techniques were considered as an intransmissible "art" and were largely based on the modeler's experience. However, scale model studies have gained theoretical bases and widened in scope of application since the introduction of dimensional analysis in the field of fluid mechanics at the beginning of the twentieth century (Rayleigh, 1915 ; Buckingham, 1915).

The complexity of the boundary conditions combined with the difficulties to formulate flow and sediment transport phenomena still constrain the use of mathematical models for the prediction or study of channel processes. Scale models in fluvial geomorphology as well as in engineering are useful for the study or prediction of channel flow, and of adjustments in the case of a movable bed model. However, for the physical model information to be valid, certain laws and limitations inherent to the physical characteristics of the open channel flow phenomenon must be respected.

2.2 Similitude

Similitude is an important concept in contemporary fluid mechanics. Similitude analysis and associated similitude theory concentrate primarily on the development of hydrodynamic model scaling laws. The concept of and rationale for scale models in engineering are made clear in the following quotation.

> " A model of some physical system (the prototype) may be thought of as another physical system, normally but not necessarily, of smaller size, which reproduces the physical phenomenon in such a way that measurements made in the model can be used , by the application of correct scaling factors, to predict accurately the phenomena to be expected in the prototype. "
>
> (Sharp, 1981, p.29)

Accordingly, two physical systems of different size which accurately reproduce the physical phenomena of each other are said to be "similar".

Physically similar systems can be described by a set of analytical equations which interrelate their characteristic parameters. The characteristic parameters of a physical system consist of the independent quantities which are required for a complete definition of the physical phenomenon (Yalin, 1971a). A distinction can also be made between the parameters and the variables of a system. The actual variables of a physical phenomenon are dimensionless and are composed of characteristic parameters.

Since three dimensions are generally used to describe open channel physics (i.e. length, time and mass), there are three conditions of similarity. Geometric similarity requires that the linear proportions are kept between the model and the prototype and hence that they have the same shape. Kinematic similarity requires that the shape of streamlines at any particular time is preserved. Finally dynamic similarity necessitates that the ratios of corresponding forces in the model and the full scale system are preserved.

However, one can call for a more rigorous definition of similarity. Novak and Cabelka (1981) introduce the more restrictive concept of mechanical similarity. Two systems are recognized as mechanically similar if they are geometrically similar and if, for proportional masses at homologous points, the paths described in proportional times are also geometrically similar. According to such a definition, mechanical similarity always includes geometric, kinematic and dynamic similarity. Meanwhile, dynamic similarity includes kinematic but not necessarily geometric similarity. The latter circumstance represents the case of a distorted model.

Mechanical similarity requires that forces are being proportionally scaled from one system to another. Hence, ratios of acting forces may represent adequate dimensionless variables to develop basic model laws. Forces in a fluid system can be characterized as: forces external to the fluid (gravitational and pressure difference forces), forces related to the physical properties of the fluid (viscous forces) and forces resulting from the fluid's motion (drag forces).

Another, fictitious, force might also be considered; the hypothetical inertia force. Inertia force is described as being equal to the resultant of all the forces acting in a particular system (i.e. of the same magnitude as the resultant) but acting in the opposite direction. This force thus describes the state of the physical system. It is hypothetical since it is not directly measurable. Because any fluid system experiences inertial forces, the ratio of the inertial force to acting forces constitutes the basis of similitude analysis. According to Newton's second law of mechanics, inertia force, F_i, can be expressed as (Kobus, 1980) :

$$F_i = \frac{V^2}{l}\rho l^3 = \rho l^2 V^2 = \rho Q^2/l^2 \qquad (2.1)$$

where ρl^3 is proportional to the mass, ρ is the mass density of the fluid and l is a characteristic length of the system, V is the velocity of the fluid and Q is the fluid discharge.

For the sake of this partial analysis, we will consider the three principal forces in open channel systems. The gravitational force, F_g, of a fluid particle is given by its weight, i.e.:

$$F_g = \rho l^3 g \qquad (2.2)$$

where g is the gravitational acceleration. The viscous force F_v may be derived from the equation of shear stress for a viscous fluid $\tau = \mu\ dv/dy$ i.e.:

$$F_v/l^2 = \mu v/l$$

$$\text{and} \quad F_v = \mu V l = \mu Q/l \qquad (2.3)$$

where μ is the coefficient of dynamic viscosity. Finally the resultant drag forces F_d can be expressed as follows :

$$F_d = \rho l^2 V^2 C_d \qquad (2.4)$$

where C_d is the drag coefficient.

Similitude analysis applies the principles of similarity theory by creating ratios of forces and, thus, a dimensionally homogeneous equation. If the dependent variable is represented by the drag forces, one can write :

$$F_d = \Phi (F_i , F_g , F_v) \qquad (2.5)$$

or in a nondimensional form:

$$\frac{F_d}{F_i} = \Phi \left(\frac{F_i}{F_g}, \frac{F_i}{F_v} \right) \qquad (2.6)$$

which also is equivalent to:

$$\frac{F_d}{\rho l^2 V^2} = \Phi \left(\frac{Q^2}{l^5 g} , \frac{\rho Q}{\mu l} \right) \text{ or } \left(\frac{V}{(gl)^{0.5}}, \frac{Vl}{\nu} \right) \qquad (2.7)$$

where $\nu = \mu/\rho$ is known as the coefficient of kinematic viscosity. The latter expression (2.7) contains two fundamental quantities on the right, namely the Froude and Reynolds numbers. Those two dimensionless parameters can be used to develop scaling relationships which define hydrodynamic modelling. However, the treatment of hydrodynamic scaling cannot be satisfactorily completed at this point, which represents an inherent limitation of the approach. Similitude analysis proceeds only via force ratios. As pointed out by Yalin (1971a), dimensional analysis consists of a more flexible and general method which also does not require any particular information about the formulation of the physical phenomenon.

This method will be considered in the following section.

2.3 Dimensional analysis and the development of scaling criteria

Dimensional analysis is a method of partial analysis, i.e. a method which is used in the characterization of a physical system when it has become too complex to be analyzed by well formulated methods (Sharp, 1981). The method, which is based on the principle of dimensional homogeneity, provides no analytical explanation of the physical system of interest but represents an excellent start for experimental investigations. Since open channel mechanics are rather complex, they are particularly susceptible to be studied via such a method.

Indeed, in contrast with similitude analysis, which requires that the researcher determines the relations amongst some parameters of a system, dimensional analysis necessitates only that one determines the characteristic parameters of the system (Yalin, 1971a). The application of simple and well-formulated rules will then determine possible dimensionless combinations from the dimensional quantities and, at the same time, the criteria for mechanical similarity. Two methods of dimensional analysis were introduced into the field of hydraulics early this century by Rayleigh and by Buckingham (op.cit.). The latter is perhaps the best known and will be reviewed briefly.

Buckingham (1915) presents the full deduction of his method as well as several of its applications to the field of fluid mechanics. Briefly, his method shows that the number of variables in a consistent, nondimensional and functional equation is related to the number n of characteristic parameters of the physical phenomenon of interest and to the number m of dimensions involved. Buckingham showed that the result of a consistent analysis would contain (n-m) dimensionless variables which he called π-terms (hence, his deductive method has become known as the π-theorem).

In the simplest form of the method, π-terms can be derived independently of each other. The method also necessitates the selection of some basic quantities (Yalin, 1971). Those parameters, which should be equal in number to the number of dimensions and which should contain all the fundamental dimensions of the phenomenon of interest, can be combined, one at a time, with each other parameter which describes the phenomenon in order to create dimensionless variables.

If we consider the case of an incompressible steady uniform flow over a granular bed under low sediment transport conditions, we can deduce the following list of characteristic parameters (we will ignore surface tension effects which should be irrelevant in the present analysis):

(1) Physical parameters of the fluid

ρ : water density

μ : water coefficient of dynamic viscosity

(2) Parameters of the flow

V : mean velocity of the flow

S : energy gradient

(3) Parameters which describe the geometry (i.e the channel) where the phenomenon occurs

w : channel width

d : channel mean depth

(4) Parameters which describe the characteristics of the boundary materials

D : a characteristic grain size diameter.

(5) The general constant g, the accelaration due to gravity.

We shall assume for now that the characteristic diameter accounts for the effects of the boundary material on the general flow situation. Generally, the median diameter of the grain size distribution, D_{50}, is employed. The selection of ρ, V and d as the basic quantities necessary for the application of dimensional analysis leads to the following dimensionless expression:

$$\pi i;\ i=1,5 = \{ \frac{V}{(gd)^{0.5}}, \frac{Vd}{\nu}, \frac{w}{d}, \frac{D}{d}, S \} \quad (2.8)$$

where ν is the coefficient of kinematic viscosity, as in equation (2.7). $\pi 1$ and $\pi 2$ represent the well-known Froude and Reynolds numbers which were also obtained from similitude analysis above. $\pi 3$ and $\pi 4$ consist of the width to depth ratio and of a relative roughness parameter respectively. $\pi 5$ is simply the energy gradient which already represents a dimensionless variable by itself.

If λ_x stands for the ratio prototype/model of any characteristic parameter x of the above list of dimensionless variables, then one can write, for similarity requirements:

$$\frac{\lambda_V{}^2}{\lambda_g \lambda_d} = 1.0$$

$$\lambda_S = 1.0$$

$$\frac{\lambda_V\ \lambda_d}{\lambda_\nu} = 1.0 \quad (2.9)$$

$$\frac{\lambda_w}{\lambda_d} = 1.0$$

$$\frac{\lambda_D}{\lambda_d} = 1.0$$

λ_d, λ_D and λ_w are all equivalent for the case of a geometrically similar, undistorted model and can be substituted by a unique length scale λ_l. λ_g is fixed on earth and λ_ν is also fixed if one works with the same fluid at about the same temperature. Consider $\pi 1$ and $\pi 2$ (all other π carrying redundant information) plus the following situation:

$$\lambda_g = 1 \quad \text{and} \quad \lambda_\nu = 1 \qquad (2.10)$$

According to the latter conditions, the realization of a smaller scale, mechanically similar model for any given prototype would appear impossible since Froude scaling law results for the velocity scale ($\lambda_V{}^2 = \lambda_l$) are incompatible with those from Reynolds' scaling law ($\lambda_V = 1/\lambda_l$). Therefore, a strictly mechanically similar small scale model would not be possible to realize under such conditions. However, as we shall discuss later, the relaxation of one of these two scaling criteria is admissible if some limiting conditions are respected.

The Froude law is considered predominant since, in both the model and the prototype, water flow is primarily governed by gravitational forces. If we accept that the action of gravity is overwhelming over that of other forces and if we also include geometric similarity as a basic requirement for mechanical similarity, i.e:

$$\lambda_W = \lambda_d = \lambda_D = \lambda_S = 1 \qquad (2.11)$$

we can further derive from $\pi 1$ the Froude scaling velocity scale

$$\lambda_V = (\lambda_l)^{0.5} \qquad (2.12)$$

By substitution into or manipulation of the latter scaling law, we can also obtain a series of scale ratios for other kinematic and dynamic variables of the physical system , for instance:

$$\lambda_Q = (\lambda_l)^{2.5}$$
$$\lambda_t = (\lambda_l)^{0.5}$$
$$\lambda_F = \lambda_\rho \lambda_l{}^3$$
$$\lambda_{TO} = \lambda_l \qquad (2.13)$$

where λ_t stands for the time scale, λ_F for the force scale and λ_{TO} for the shear stress scale. This set of scaling laws defines what is refered to as a Froudian model. Note that the same dimensionless variables or scaling laws could have been obtained from the similitude analysis presented in section 2.2: this confirms their physical significance.

2.4 Froudian scaling laws and natural river channels

2.4.1 Natural stream behaviour over a range of scales (hydraulic geometry results)

It is legitimate to ask what is the significance of Froudian scaling laws with respect to the behaviour of natural channels. Otherwise stated, the question " are natural streams Froude models of each other? " may be asked. The issue is of major importance regarding the appropriateness of using theoretically derived scaling relationships to model natural river behaviour at a smaller scale.

Downstream hydraulic geometry studies, which are numerous in fluvial geomorphology, as well as conceptually equivalent regime canals studies in engineering, provide the basic source of information for the present considerations. Those empirically obtained relations are customarily presented in the form of power functions, such as:

$$\psi_i = a\ Q_f^{\ x} \qquad (2.14)$$

where ψ_i represent any parameter part of the flow continuity equation ($Q = Vwd$), a is a constant of proportion and x an exponent. Q_f is referred to as formative discharge. The necessity for the latter discharge parameter is a result of the unsteady character of natural river flows. Most often, the bankfull discharge has been used as a surrogate to the "formative" discharge although its significance in terms of river morphology has not yet been demonstrated on a universal basis (Knighton, 1984). This reference discharge is nevertheless considered sufficient for the present analysis. Moreover, for the actual considerations, the width/discharge (exponent b) as well as the depth/discharge (exponent f) relations will be of most interest, since the third relation is then fixed.

Downstream hydraulic geometry results are summarized in various papers. In an unpublished manuscript, Church (1980) reports b exponents between 0.5 and 0.57 and f exponents between 0.32 and 0.47 (except one of 0.59). These hydraulic geometry relations commonly show high proportions of explained variance. The results about the channel width and depth relationship extend with discharge over nine orders of magnitude (Ferguson, 1986). The foregoing ranges of exponents considered some experimental channels as well as some irrigation canal and gravel bed river data, all for subcritical flow (i.e. Froude number < 1.0), and mostly North American. Classical regime studies (e.g. Inglis, 1949 ; Lacey, 1958 ; Simons and Albertson, 1960 ; Blench, 1969 ; etc.) typically suggest b and f values of 0.50 and 0.33 respectively for mostly Indian canals. Hey and Thorne (1986) also report b exponents of 0.50 and f exponents of 0.35 from British gravel bed rivers. Finally, Ming (1983) reports b exponents between 0.39 and 0.55 (mostly near 0.48-0.50) and f exponents between 0.30 and 0.43 (average of 0.36) from Chinese and Russian river studies.

Hence, according to worldwide studies from a variety of environments, the relative consistency of the b and f exponents is remarkable and it may be suggested that these bear average values of about 0.50 and 0.35 respectively. However, for a very wide range of scales, including the world's largest river systems, it appears that a b exponent of about 0.55 may be more appropriate (Kellerhals and Church, in press).

Let us now reconsider one result, which was presented in our similitude analysis above, in light of the likely predominance of the Froude number criterion for mechanical similarity of open channels. In equation (2.7), the Froude number was also given in terms of discharge. This form of the Froude number can be reformulated to set the required proportionality for Froude similarity between the discharge and the geometrical parameters of the channel, i.e:

$$L = \frac{Q^{0.40}}{g^{0.20}} \qquad (2.15)$$

If we consider L as the length scale, equation (2.15) shows that, for Froude similar channels, b and f exponents should both be equal to 0.40. Clearly this does not seem to be the case for alluvial channels according to the aforementioned empirical results of hydraulic geometry studies. The reasons for such a situation warrant further consideration.

2.4.2 Deconstrained hydraulic geometry plot

The approach to clarify the above concerns regarding the behaviour of natural channels has been to produce the equivalent of a downstream hydraulic geometry plot in which the constraints related to the roughness length scale (or grain size scale as a surrogate) which appear in natural drainage basins would be controlled. It is well documented in many geomorphological studies that the river bed median grain size diameter diminishes from the headwaters of a drainage basin to its outlet. Variation in the roughness length is less well documented but even if steeper bedforms tend to develop in ordinary finer grained, most downstream reaches, Froudian proportions do not seem to be kept, as demonstrated by the empirical results presented above.

Therefore, natural gravel bed river channel data (extracted from Church and Rood, 1983) were plotted on one (bankfull) discharge to width and depth graph together with some experimental data from Wolman and Brush (1961). Data were screened to ensure that flow conditions from those experimental channels were representative of field situations, i.e. subcritical and fully hydraulically rough (ref. section 2.5.1 below). The data were selected in such a way that a plot of the grain size (a controlling length scale) and discharge would approximately follow the Froudian similarity criterion as contained in equation (2.15): figure 1 presents the data and the relationship fitted by eye which has an exponent of 0.34.

Figure 2 shows the data points and the relationships fitted by eye between channel length scales (width and depth) and discharge. The results of this exercise are most interesting. The b and f exponents of the relationship between width/discharge and depth/discharge are respectively 0.425 and 0.417. It therefore appears that when the constraints of the natural environments are withdrawn, alluvial channels nearly conform to the requirements of Froudian similarity prescribed by equation (2.15). This occurred even though the control on the grain size scale in terms of the Froude requirements for similarity was not exact (see figure 1). It is thus proposed that figure 2 presents clear evidence that natural channels do follow the proportions prescribed by Froudian similarity laws under favorable conditions. The latter are not met in natural systems due to the constraint imposed upon by the boundary material as an underlying factor affecting resistance to flow, whence channel dimension.

In the context of the present research project, the most significant impact of the results shown on figure 2 consists in the validation of Froude scaling laws for the study of some natural rivers and of their processes in small scale experiments. In fact, figure 2 easily spans a range between typical gravel-bed river prototypes and feasible controlled models. Appropriate model studies can only be realized if certain conditions, which will be the subject of the next

Figure 1. Grain size versus discharge plot

for the selected set of data

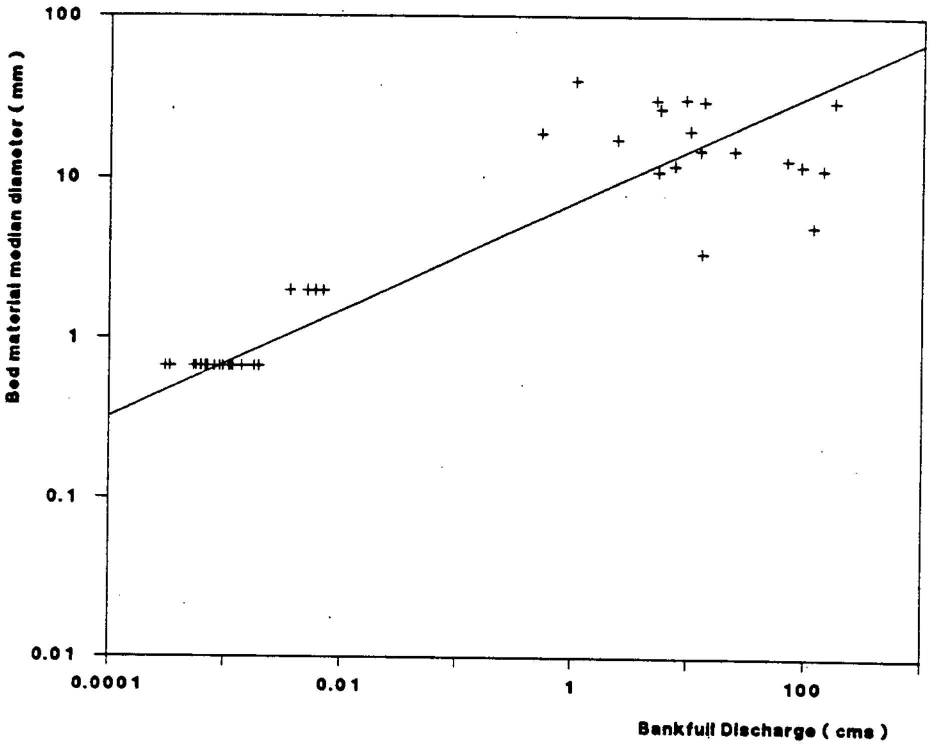
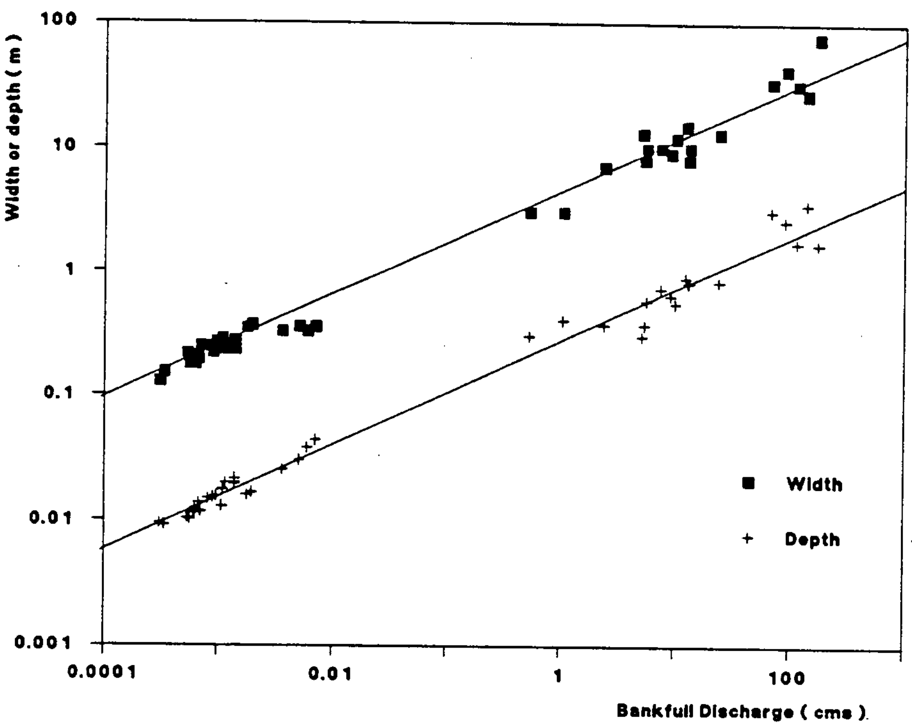


Figure 2. Deconstrained hydraulic geometry plot



section, are respected.

2.5 Criteria for a closer test of the representativeness of scale models

2.5.1 The reproduction of frictional characteristics or of roughness length

The balance between the propelling and resisting forces along the boundary of a channel reflects its frictional characteristics. The preservation of these forces is essential in the context of mechanical similarity. For rivers which have a relatively low bed sediment transport intensity, a rigid bed assumption is admissible for the purpose of the analysis of the flow resistance problem.

Channel flow resistance represents a very complex, composite phenomenon (see a review paper by Bathurst, 1982). Common simplifications of the problem include the assumption of steady uniform flow, the use of average geometric and hydraulic parameters, the latter of which are idealized as uniformly distributed, and the use of a representative diameter of the grain size distribution. In general terms, if we consider the propulsive gravitational force for any given cross-section to be resisted over the channel perimeter by a shear force (represented per unit area by the shear stress τ_0), and if we rearrange, we get the well-known uniform flow formulation (Henderson, 1966) :

$$\tau_0 = \gamma R S \tag{2.16}$$

where R is the hydraulic radius and S the energy slope of the flow (in most practical cases, identical with the water surface slope).

Let us consider the results of a dimensional analysis of the resistance problem for two-dimensional flow. Consider the following set of characteristic parameters for flow in a rough channel, i.e. μ, ρ, D, R and V. Any mechanical quantity related to the flow process can be expressed as a function of these five characteristic parameters (Yalin, 1971a). If we consider the average shear stress τ_0 acting on the boundary, which incorporates the driving gradient S of the system, we can get, using the π theorem, the following dimensionless grouping :

$$\frac{\tau_0}{\rho V^2} = f \left\{ \frac{VR}{\nu} , \frac{D}{R} \right\} \qquad (2.17)$$

Let us consider for the moment the significance of the term on the left side of equation (2.17).

Substitution of this term, in the form of the left member of equation (2.17), i.e.

$$\tau_0 = ff \ \rho \ V^2 \qquad (2.18)$$

(where ff is a coefficient termed the friction factor), into equation (2.16) gives

$$ff = gRS / V^2 \qquad (2.19)$$

Equation (2.19) is equivalent in form to the Darcy-Weisbach formula

$$ff = \frac{8gRS}{V^2} = \frac{8 \ V_*{}^2}{V^2} \qquad (2.20)$$

where $V_* = (gRS)^{0.5} = (\tau_0/\rho)^{0.5}$ is known as the shear velocity. The Darcy-Weisbach formula includes a dimensionless variable, a resistance coefficient, which accounts for the balance between retarding and propelling forces.

Shear distribution over an alluvial boundary is never uniform. Therefore, for detailed hydrodynamic measurements as intended in this thesis, a local expression for shear stress estimation is needed. The shear velocity introduced above also appears in the development of the classical boundary layer theory. Over rough boundaries, the velocity profile can be expressed as:

$$\frac{\tau_0}{\rho} = \frac{(V_2 - V_1)}{(2.3\kappa)\log_{10}(y_2/y_1)} \qquad (2.21)$$

where κ is known as the Von Karman constant. Equation (2.21) provides an estimate via measurement of the local velocity profile of the internal fluid shear at some distance above the bed. Equation (2.21) allows a closer test of the spatial distributions of the shear force in the model and the prototype: velocity profiles represent the principal measurements to do so.

From its dimensionless nature, the Darcy-Weisbach friction factor of equation (2.20) can be used to formulate the requirements for the preservation of mechanical similarity with regard to general frictional considerations. It can be reexpressed as follows:

$$ff = \text{constant}\,\frac{gR}{V^2}\,S \qquad (2.22)$$

i.e it can decomposed into the Froude number and the energy gradient. Equation (2.22) informs that the requirements for the preservation of resisting forces would be ensured by a Froude model. However, the preservation of the flow resistance condition must also consider some scale effects which will be the object of the discussion below.

In the above dimensional analysis of the resistance problem which led to equation (2.17), we have shown that the friction factor ff must be a function of the Reynolds number and the relative roughness of the boundary. Numerous experimental studies in fluid mechanics which considered pipe flow situations have revealed the nature of the relationships between the friction factor, the Reynolds number and the relative boundary roughness. These relationships can be represented by a plot of ff versus the Reynolds number for various relative roughness values, such as figure 3 (modified from Rouse, 1959 by Kobus, 1980).

The above analysis and the resulting so-called resistance diagram as figure 3 were first developed from pipe experiments but it is possible to use them for open channels if the hydraulic radius is used and if k_s, the equivalent roughness of the boundary materials, is properly defined (Middleton and Southard, 1984). The four regions which can be recognized on the resistance diagram (figure 3) correspond to distinct flow conditions i.e. laminar, turbulent hydraulically smooth, transitional and turbulent hydraulically rough. In the present research context, prototype conditions, like most river situations, belong to the hydraulically rough domain. It is

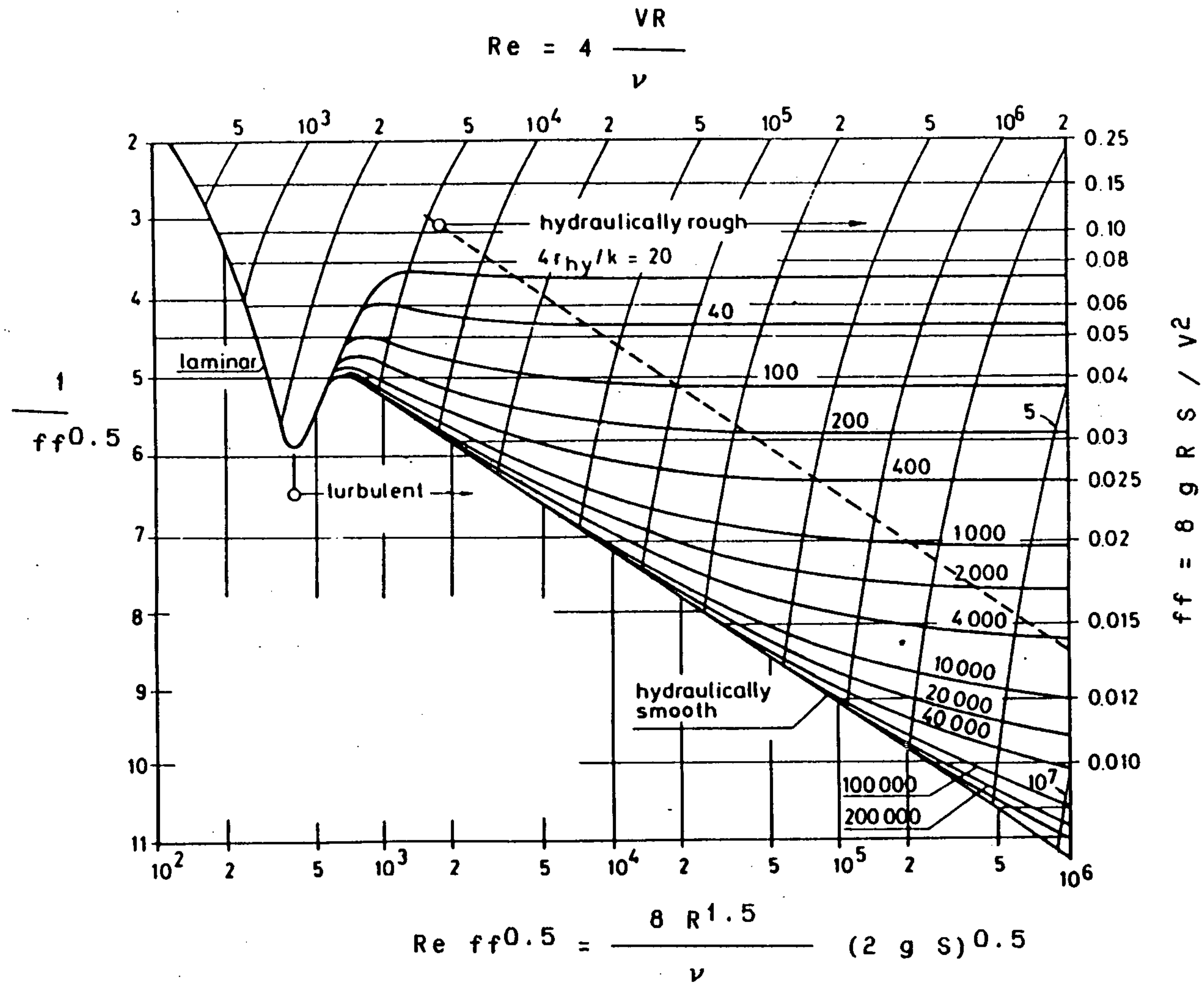


Figure 3. Resistance diagram

Modified from Rouse(1959) and Kobus(1980)' essential to preserve these conditions in the scale model in order to ensure mechanical similarity.

As we noted earlier, strict mechanical similarity can be satisfied only if the model and the prototype are the same. This is the result of the contradiction between the Reynolds and Froude scaling laws. We also noted that gravitational forces were predominant. Consequently, we must seek for the possibility of some compromise regarding the reproduction of viscous forces in order to realize a scale model. Figure 3 reveals that, under the hydraulically rough flow regime, for any relative roughness value, the friction factor is independent of the flow Reynolds number. This signifies that, under this flow regime, viscous forces are not relevant. It is therefore possible to obtain approximate mechanical similarity provided that the roughness length is preserved (that should be provided by the geometrical scaling of the bed material size distribution) and that the limiting condition represented by the dashed line on figure 3 is respected.

The limit between respectively hydraulically rough and transitional regions on figure 3 can be approximated by the following formula (Rouse, 1959) :

$$4 \; Re \; (ff)^{0.5} \; k_s/4R > 200 \qquad (2.23)$$

If the latter limiting condition or criterion is respected, the friction factor is preserved. Nevertheless, the similarity is said to be approximate since the Reynolds number is somehow sacrified. This however does not represent a major drawback if the phenomena to be looked at in the model do not strongly depend on small scale turbulence, such as is the case for the velocity profile (Middleton and Southard, 1984) or for large scale eddies and secondary flows (Yalin, 1982). When the Reynolds number is increased from a model to a prototype, the turbulent energy cascade (or the transfer of turbulence from the larger to the smaller scales) is lengthened and proportionally smaller scale eddies (Yalin, 1982) develop while the structure of the turbulence at larger scales is not much changed. This convenient phenomenon from the perspective of physical modelling is referred to as the Reynolds number similarity (Middleton and Southard, 1984).

2.5.2 Reproduction of sediment entrainment conditions and excess stress

The matter of reproducing the sediment entrainment conditions is strongly related to that of frictional characteristics as presented above. It is however approached from a different perspective which further qualifies the limiting conditions which frame physical modelling.

In the case of the reproduction of entrainment conditions one is concerned about the flow conditions around each particle on the stream bed. The reproduction of entrainment conditions must involve a correct albeit not necessarily exact scaling of the pressure field around any particle on the stream bed, a phenomenon which is determined by the combination of drag and lift forces.

In his classic study, Shields (1936) presented the results of a physical analysis of the sediment transport problem. Two dimensionless variables were proposed in order to describe the sediment entrainment conditions i.e. a particle Reynolds number

$$R_p = \frac{V_* \, k_s}{\nu} \qquad (2.24)$$

plus a variable which expressed the balance between the propelling force and the weight of a particle, i.e.

$$F_s = \frac{R \; S}{(s-1) \; D} \qquad (2.25)$$

where s represents the submerged specific weight of sediment. From low to high particle Reynolds number, as in the case of average flow analysis (section 2.5.1), the flow conditions around a grain on the stream bed are successively laminar, smooth turbulent, transitional or fully rough. These four flow states can be recognized in the Shields diagram, of which the latest version presented by Yalin and Karahan (1979a) will be considered herein.

Hydraulically rough conditions are found in the field situation and hence are desired in the model. Recent Shields diagrams reveal that the parameter of equation (2.25) uniquely determines the entrainment conditions for particle Reynolds number above 70 (Yalin, 1971a). Starting with the latter criterion and after some manipulation, for an undistorted, geometrically similar model, the following expression can define the limit (Yalin, 1971a) for the scaling factor of the model:

$$\lambda_L{}^{-1} = \left[\frac{70}{(\, V_*' k_s' / \, \nu \,)} \right]^{0.67} \quad (2.26)$$

where ' denotes prototype values. λ_L is the length ratio of the prototype to the model. The preservation of the latter limiting condition or criterion together with the use of Froudian scaling laws ensures that the mobility number i.e. τ_0/τ_c (where τ_c is the critical shear stress for sediment entrainment as found from the Shields diagram) is preserved in the model. The preservation of the mobility number appears to be of importance to the appropriate reproduction of bedform geometry and consequently of form roughness (Yalin and Karahan, 1979b). The latter issue is mostly important for sand bed rivers, as discussed by Yalin (1982).

2.5.3 Reproduction of flow patterns and bed shear stress distribution

A corollary of mechanical similarity in open channels is that at homologous points and homologous times between the model and the prototype the flowlines are geometrically similar (kinematic similarity) and the forces acting on the bed are appropriately scaled (dynamical similarity). However, it is very rarely verified if the model conforms to the prototype in those respects. In the eventuality of model studies of river processes, the issue is of utmost importance if generalization of the results is foreseen.

The prototype hydrodynamic measurements consist of the basic information for the comparison with the model situation: these principal measurements are the velocity measurements and the associated shear estimates obtained via equation (2.21) and its distribution. To effect a closer test on possible effects of the approximate mechanical similarity provided by Froudian methods of scaling, the flow structure must also be considered. These measurements were performed for the field and the model situation.

2.6 Generic model concept and geomorphological research

Both sections 2.5.1 and 2.5.2 presented explicit limiting conditions or criteria which respectively constrained the absolute size of the prototype and the absolute size of the sediments present on its bed. In fact, it is effectively impossible to perform laboratory investigation which could lead to scientific generalization for most prototype situations of fine bed, large scale rivers. In the latter cases, it is impossible to satisfy in ordinary circumstances equation (2.23) and (2.26) because of the large scale difference between the customary models and prototypes.

However, in the eventuality that the scaling ratio is rather small, it may be possible to model sand bed river processes by using warmer water in the model and even to meet the strict mechanical similarity requirements. This strategy will produce a decrease in water viscosity and provide additional albeit limited freedom to the the exercise. Southard, Boguchwal and Romea (1980) demonstrated the success of the latter method by a comparison of ripple geometrical parameters from field and laboratory studies. Complete mechanical similarity can eventually be obtained if the scaling ratio is smaller than or equal to about 2.5 by taking advantage of the change in water viscosity from 10 to 90 degrees Celcius.

In the case of small and medium scale gravel bed rivers, it is possible to consider reducing the size of the bed sediment while preserving the fully rough flow conditions at the threshold of transport. For a series of situations, the forces which act in natural streams can apparently be appropriately reproduced when some limiting conditions are respected (section 2.5.1). In such circumstances, scientific investigations in the laboratory would be legitimate.

An extension of this current hypothesis leads us to the introduction of the concept of a generic model. In an etymological sense, "generic" stands for an element which is part of a whole, a family. By extension, a generic model is one which is representative of a family in which individuals share something in common. If a laboratory channel respects the limiting conditions which were exposed in this chapter, it can thus be part of a family of situations and can be used to investigate physical processes in a quantitative manner. A generic modelling framework would not necessitate the reproduction of some exact boundary conditions, in contrast with the normal engineering approach to models.

The concept of generic modelling was approached by Hooke (1968) in a discussion of Bruun's (1966) paper. Hooke argued that geomorphologists are seldom interested in the study of a particular system but in general principles which apply to a population of systems. He therefore proposed a general laboratory procedure refered to as "similarity of process". The basic requirements and properties of "similarity of process" are that (1) the gross scaling relationships be met; (2) the model reproduces some morphologic characteristics of the prototype; and (3) the processes which produced the model's characteristics can logically be assumed to have the same effect in the prototype. Hooke further stated :

> " In this type of study the laboratory systems are treated as small systems in their own right, not as scale models of prototypes."
>
> (Hooke, 1968, p.392)

Hooke's statement goes beyond Schumm et al (1987) views on the use of scale models described in the introduction. However the requirements of Hooke's "similarity of process" introduced above do not encompass the criteria for mechanical similarity which were analytically developed in this chapter and which provide explicitly limiting conditions for scaling operations. The generic model concept includes in addition the recognition of some process-magnitude relation not formally recognized in Hooke's similarity of process.

The validity of the generic model framework will be confirmed only when it has been demonstrated that the processes can be the same in the model and the prototype. Such a test appears to be necessary in order to ensure that laboratory results will not be received sceptically in the geomorphological community. Therefore, a logical first step before fundamental research is being undertaken under the tenet of generic modelling is to verify if whether or not the theoretical background applies in a particular situation.

3.0 Field study

3.1 Site selection, location and description

The nature of this research project predetermined a number of criteria for field site selection. The size of the stream and of its bed material were important for an eventual scaling exercise in a 0.5 metre wide flume. Therefore, the stream needed to be small (about 5 to 8 metres in width) with well-developed alluvial pool and riffle forms and also needed to bear relatively coarse bed material, mostly in the gravel to cobble range. The former restriction regarding the size of the stream follows from operational and instrumental limitations. The latter restriction was important to ensure that the bed material could be scaled in the laboratory to manageable sizes (i.e. fine sands and coarser). Because of the restriction set by laboratory modelling, the selected stream reach would also need to be nearly straight. Final criteria were the accessibility of the site from the campus of the University of British Columbia in Vancouver, as well as its relative seclusion, for safety reasons.

A small stream in the lower Coast Moutains in the University of British Columbia Reseach Forest, about 50 km east of Vancouver, was found to best meet the above requirements. A 25 metres long reach of Blaney creek, upstream of Blaney lake (see figure 4) was selected. The reach, located downstream from a bedrock outcrop has an average width of 7 m, is nearly straight and bears well-developed alluvial pool-riffle features. The preparation of the field site involved pruning of overhanging vegetation, mapping, bed material sampling and installation of bridges and reference cables across the creek for the purpose of facilitating velocity measurements.

Conventional survey methods were used to produce a topographical map (figure 5) from about 200 points. The map shows the principal configurational elements of the reach which will be described below. These can be further appreciated from figure 6. A transverse log controls the downstream bed level of the study reach. The right bank of the creek is complex in form, generally vertical and undercut as well as partly made of and reinforced by organic debris. Immediately downstream of the outcrop the main flow impinges on the left reinforced bank. There, a large tree stump (the area was logged in 1959) holds coarse material between its roots. About 3-4 metres below this feature the left bank turns alluvial and constitutes the extension of the riverbed locally topped with some overbank sand deposits. The bankfull level is not particularly well defined along the creek, due to the complexity of the banks, but a few indicators (cutbanks, overbank fine deposits, limit of vegetation) still allow its determination.

Six important features (numbered from 1 to 6 on figure 5) can be recognized on the river bed (refer to figures 5 and 6).

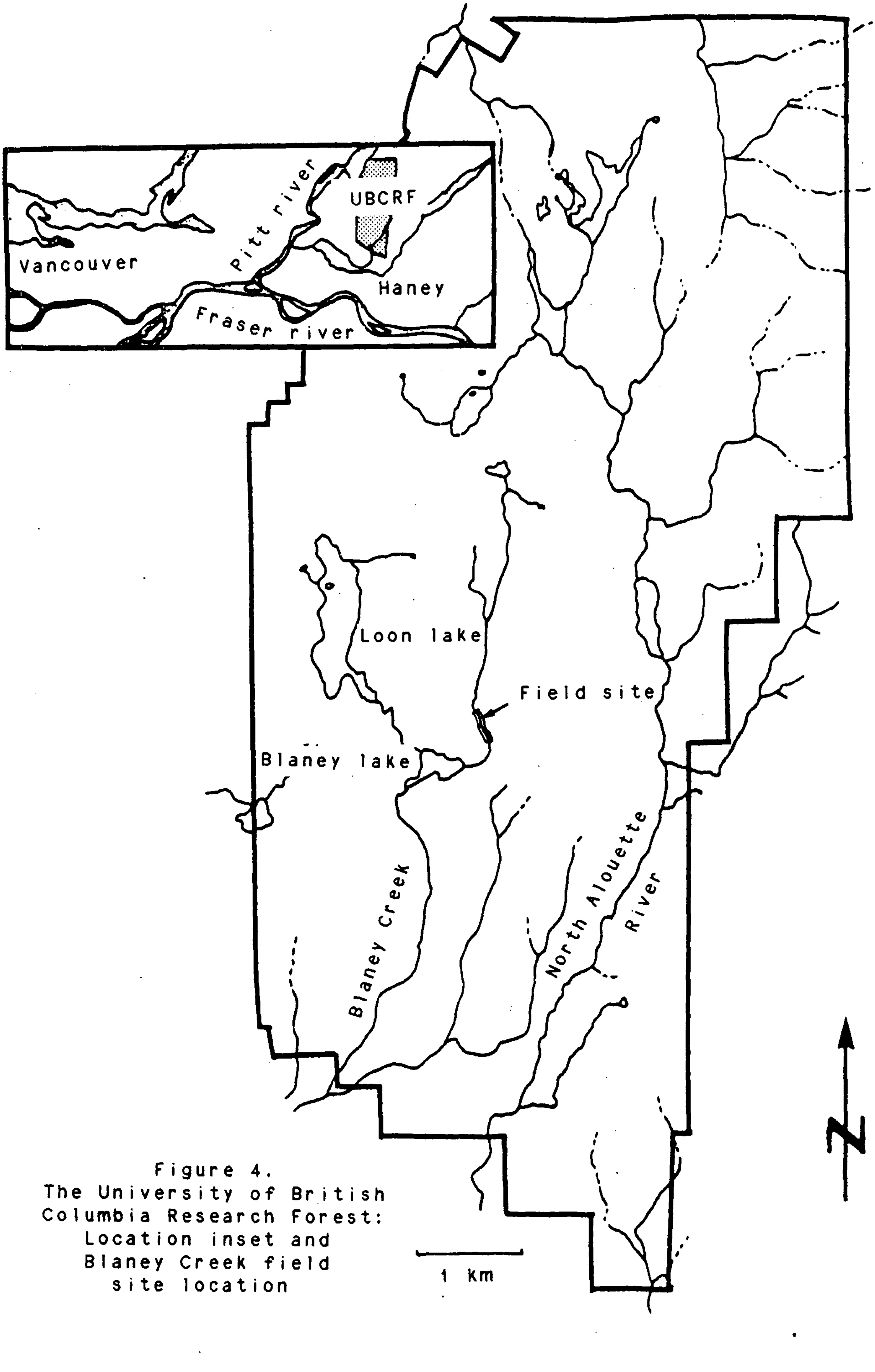


Figure 4.
The University of British
Columbia Research Forest:
Location inset and
Blaney Creek field
site location

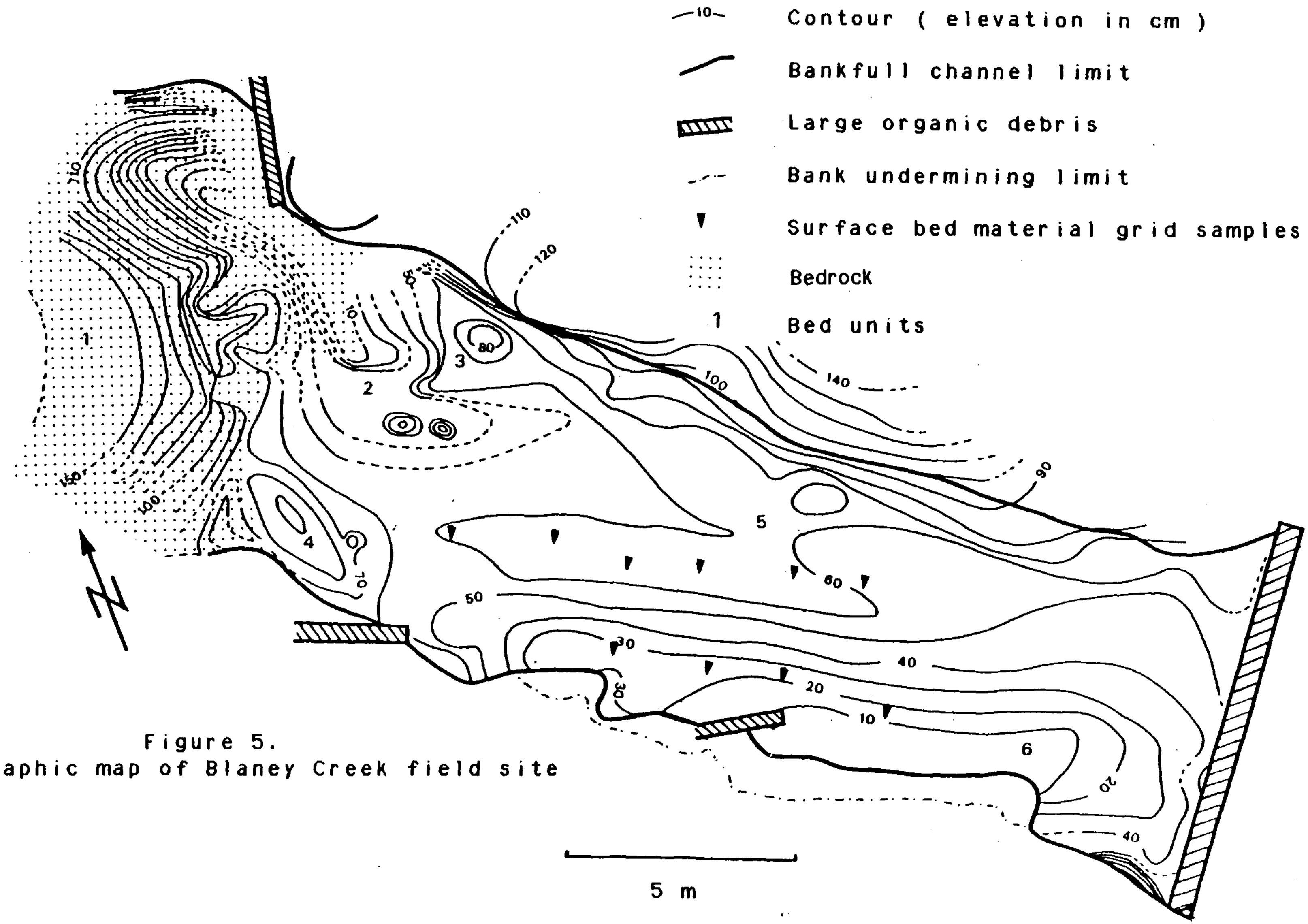


Figure 5.
Topographic map of Blaney Creek field site

Figure 6. Views of Blaney Creek field site

(a) Looking upstream, from the transverse log. In the foreground, riffle and distal pool. Note the character of the overhanging right bank . Tripod on bedrock outcrop in the background indicates from where the next photograph was shot.

(b) Looking downstream, from the bedrock outcrop. In the foreground,proximal pool (note coarse bed material), cross-over area and finer sediment accumulation bar. In the background, man stands on transverse log which is the downstream limit of the mapped reach

At the upstream limit of the study reach, (1) a 1 m relief bedrock outcrop is found transverse to the creek. All of the water at low flow stage is discharged on its lower left side. At high stages the outcrop is entirely covered by water although most of the water still flows on its left. The upper right side of the outcrop is actually topped by vegetated gravelly deposits (not shown on the map) which further encourages most of the water to flow via the left side. Immediately below the outcrop, on the left side of the creek, a pool (the proximal pool - 2) has developed : its origin is to be related to the stump towards which the main thread of upstream flow is oriented. The proximal pool bed is either bedrock or covered by large boulders (some of which show up on the map) especially at its downstream end . On the left side and downstream end of the proximal pool an accumulation of coarse material reinforced with some organic debris produces a mound which is easily discernable on the topographic map (3). Opposite the proximal pool near the right bank a small accumulation bar (4) has developed on the edge of the bedrock outcrop, just above the area where the flow crosses over to the second (or distal) pool at low flow stages. The material of this bar and adjacent cross-over area is clearly finer than that of the (5) riffle and (6) distal pool.

Finally, the riffle and the distal pool represent the two features of most interest herein. The riffle which constitutes the extension of the proximal pool shoaling is of diagonal nature and has well-defined crest and slipface which are about 15 metres long. The distal pool located along and subparallel to the right bank gradually widens towards its downstream end which is fixed by the transverse log. The flow measurements were taken over these features.

3.2 Bed material sampling

To avoid disrupting the stability of the bed features within the selected study reach, it was necessary to sample the bed material on the next downstream riffle. A complete bed material sampling (i.e. volumetric, surface layer, areal and surface grid) was performed at that location, following the procedures described in Church et al (1987). Moreover surface grid samples were taken on the riffle and in the distal pool within the study reach. The latter samples would ascertain by comparing grid surface samples if the complete sampling programme at the downstream location could be considered representative of the site's conditions.

A 1 X 1 m grid sample was used to sample 25 stones on the surface of the riffle site downstream of the study reach. After this sample was completed and the stones saved to be included in the next areal sample, the material within the 1 X 1 m area was painted and later collected. Afterwards the depth of the largest stone which determines the surface layer depth was noted and this sample accordingly excavated. Finally, a volumetric (bulk) sample was taken after the surface layer had been cleared. No bedding change was encountered during excavation. The subsurface material, mostly granite, was noted to be oxidized albeit not decayed. In the laboratory, samples were oven-dried and sieved using conventional procedures.

To enable comparison with other samples which are dimensionally equivalent, the areal sample size distribution was adjusted following the method proposed by Kellerhals and Bray (1971). Figure 7 presents the results of the sieve analysis of the sediment samples from the downstream riffle location. The deficiency in coarse material at the surface with respect to the subsurface materal is especially evident if one considers the grid sample. The surface layer and volumetric samples show similar distributions.

In the study reach, five (on the riffle) and four (in the distal pool) 1 X 1 m grid samples of 25 clasts each were taken. The sampling stations are indicated on figure 5. No spatial trend in size distribution characteristics could be detected on either feature. However, one additional sample taken near the cross-over area showed a clearly finer size distribution than any other sample. Figure 8 compares the size distribution of within reach pool and riffle bed surface material samples as well as that of the riffle site downstream of the study reach. The comparison of the size distributions within the study reach reveal that riffle material is slightly coarser in general than the pool but that median diameter of both size distribution are almost identical. The size distribution of the grid sample from the riffle downstream of the study reach is not much different than both previous samples, especially in the cobble range. This validates consideration of the volumetric sample from the downstream area as representative of the study reach subsurface material. Therefore, it was decided to use the

Figure 7.

Size distributions of bed material samples

at downstream riffle site, Blaney Creek

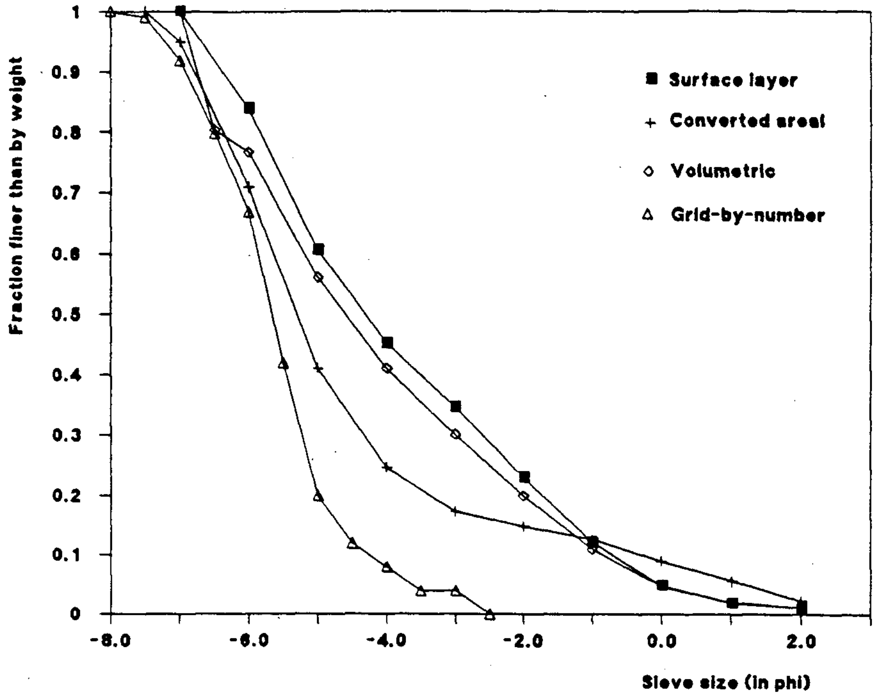
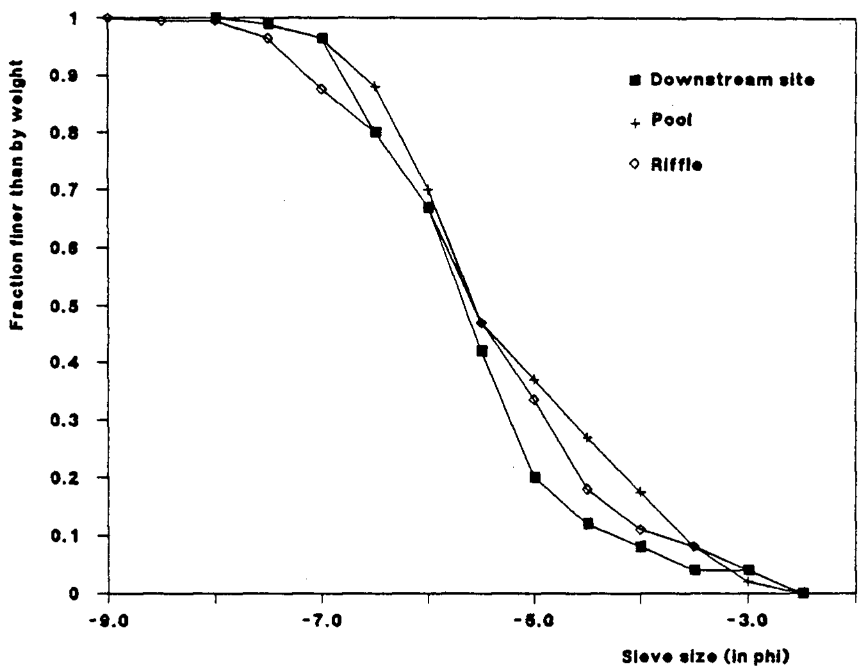


Figure 8. Comparison of pooled size distributions

for grid samples from downstream riffle

and within reach pool and riffle sites, Blaney Creek



downstream bulk sample and the within reach pool and riffle grid samples for scale modelling considerations. Some grain size distribution parameters for these samples are given in table 1.

The nomographs of Church et al (1987) suggest that all samples were actually too small for confident characterization (i.e. the largest stone should represent 0.1 % of the sample weight) of the bed material. The representativeness of the volumetric sample is an important issue in the context of scale modelling of bed material. The largest clast of the size distribution of the bulk sample (about 128 mm) represented about 4 percent of the total sample weight (according to Church et al, 1987), a situation which was judged to be reasonable (it is in fact common in many field measurement programmes) albeit not ideal.

3.3 Measurement arrangements, instruments and estimation of shear stress

In order to perform measurements safely at high flows, three transverse 60 cm wide, 7 m long aluminum bridges were laid across the creek at about six metre intervals. They were landed on the top of the right bank and mounted on pillars along the left bank. A fourth bridge 40 cm wide, 7 m long was moved around on the three fixed bridges and onto one landing on a piece of wood debris on the right bank to allow measurements at intermediate locations and in the cross-over area.

A set of nine reference levelled cables was installed at

Table 1

Grain size distribution percentile diameters (in mm) for selected bed material samples, Blaney creek

Sample	Grid (downstream site)	Volumetric (Bulk)	Grid (study reach) Pool	Grid (study reach) Riffle
Percentiles				
D16	26.9	3.1	15.4	21.0
D35	41.6	11.3	30.7	34.8
D50	51.3	25.1	47.8	49.9
D65	61.8	43.4	59.7	65.8
D84	99.0	91.8	83.9	119.4
D90	112.2	97.7	97.7	148.1

about 2.5 m above the lowest point on the bed. From the cable network, water surface and bed elevations could be determined for each velocity sampling station. The latter elevations could be compared with topographical map elevations while the former were useful to estimate the water surface slope during the measurement programme. A staff gauge was intalled in the reach to monitor the changes in water level during the measurement periods.

The velocity profiles were measured with six propellor current-meters (C1 and C2 Ott laboratory current meters, five with 30 mm diameter/10 cm pitch - maximum velocity of 2 m/s - and one with 50 mm diameter/25 cm pitch - maximum velocity of 4 m/s) clamped to a 3 m long, 20 mm diameter rod (see figure 9). Analog signals were transmitted via 15 m cables to six channels on a CR5 pulse counter data-logger (Campbell Scientific Instruments) which was installed on the right bank. The supporting rod had a small 10.5 X 6 cm plate at its base that ensured it would rest nearly on the top of bed particles. A minimum distance of two times the propellor diameter from the boundary and between meters appeared sufficient according to laboratory tests on wall proximity effects and mutual interactions between propeller meters performed by Benini (1962). Therefore, the first meter's axis was set at six centimeters from the plate and thus about six centimeters on average above the top of bed particles, which corresponds roughly to surface $D_{90}/2$ (see D_{90} values on table 2). This was assumed to be sufficient to avoid the meter being in the wake of protruding bed particles. The next two meters'

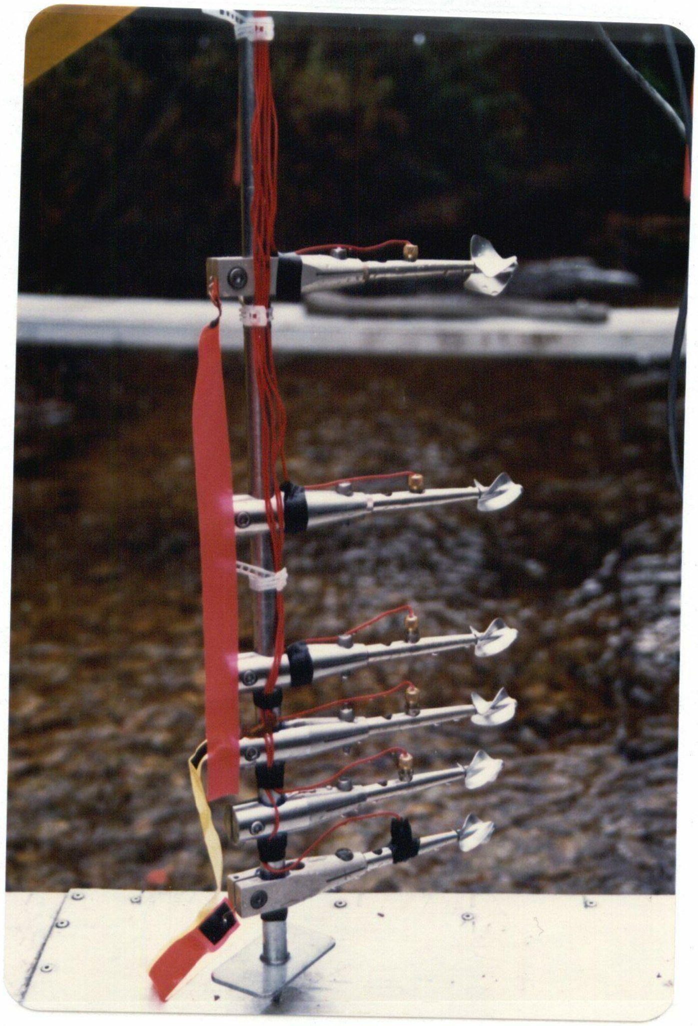


Figure 9.
Ott laboratory current meter rod-mounted array

axes were set at 12 and 18 cm respectively from the plate. The fixed distance between near-bed current meters would also ensure accurate determination of local internal shear. Finally, the three other meters were approximately logarithmically spaced through the remaining water depth. The upper meter was usually set about two diameters below the water surface. In shallowest flows, some of the three upper meters could either be out of the water or separated by 6 cm.

Velocity sampling stations were determined partly on morphological grounds but also to obtain a complete coverage of the riffle and distal pool. In order to observe the structure of the flow, 50 cm flags were attached to the bottom and near surface meter clamps. Their orientation was measured from the bridges with a compass. It was believed that the foregoing measurements were accurate within 2-5 degrees. The flowline information through the depth was used to align the six Ott meters with the local flow direction. In actuality propellor current meters will give accurate readings when they are oriented within 15 degrees from the mean current axial direction (A.S.M.E., 1971). However in the deepest, fastest November 23 flows, the near-bed flag was not everywhere visible and near-bed alignment had to be judged from that of neighbouring stations.

Propellor current meters generally overestimate axial velocity pulsations and underestimate transverse components (Jepson, 1967). However, if the average amplitude of velocity fluctuation or turbulence intensity is less than 0.2 times the mean velocity, Ott meter error is less than 1 % (Kulin and

Compton, 1975). Experimental measurements for high relative roughness flows such as Blaney Creek (Nowell and Church, 1979) suggest values of turbulence intensity V'/V_* (V' being the average velocity fluctuations) in the order of 2.0 for the lowest third of the water depth. Using the latter figure, equation (2.21) and mean friction factor values of measured events (see table 3 and description below), the turbulence intensity near the bed of the creek would approximately average 0.3 to 0.35 V. According to Kulin and Compton's (1975) rule of thumb, Ott meter errors due to velocity fluctuations under Blaney Creek conditions could exceed 1 % near the bed, but not in a drastical fashion. The latter conditions were considered relatively conservative and it was assumed that axial or lateral effects were relatively insignificant.

The time interval of velocity profile measurements was selected to be one minute. It was assumed that a one minute time interval would be sufficient to obtain representative velocity profiles, a situation which appeared not unreasonable for a 7 m wide creek. In any case, it was difficult to consider longer time intervals due to the manipulations and procedures associated with each measurement. These could consume at least 5 minutes and since Blaney creek was hypothesized to bear a flashy flood regime, it was reasoned that longer time intervals for velocity measurement would have resulted in the end in a more restricted spatial coverage of the creek. This concern was borne out in the subsequent high flow measurements.

Due to some technical difficulties, most often 4 current meters were actually in working condition at various times during field velocity measurements. Fortunately the two most important near-bed meters were operational at all times. The flow speed and orientation information gained can be used to explore the hydrodynamics of the riffle and distal pool. Velocity profile data were collected in order to allow bed shear stress estimations for several locations across the study site. For these purposes the logarithmic law of the wall for hydraulically rough flow (equation 2.21) was assumed to be appropriate. In actuality, the law of the wall provides an estimation of the internal fluid shear at some distance above the bed.

There is relatively abundant experimental evidence that the law applies under controlled field and laboratory conditions, albeit often only approximately through the whole water depth. Some authors have observed velocity profiles to be kinked and have proposed that the law of the wall is applicable in the lowest 15-20 % of the water depth only. However, the latter issue depends on the relative roughness as well as on the roughness element spacing (Nowell and Church, 1979) at the solid boundary of the flow. Most of our profiles suggested that the velocity distribution was logarithmic near the bed. The use of equation (2.21) in this area appeared justified to estimate bed shear. However, because of the limited number of points, it could not be satisfactorily determined at which average relative height above the bed the the law of the wall fails.

Unfortunately, due to some uncertainties about the exact location of the actual or idealized (in the case of a granular surface) bed level, our velocity profile data could not allow near bed shear stress estimation using the concept of the displacement height (Jackson, 1981). In practice, this parameter is found by linearizing non-linear velocity profiles but our profiles had too few points to do so. The physical significance and hence formulation of the displacement height are not well-known (Middleton and Southard, 1984). Although some authors report on the relative consistency of the displacement height with respect to the roughness height under controlled conditions (Jackson, 1981; Bayazit, 1982), the marked nonuniformity of the flow conditions at the study site would appear to preclude such generalizations. Due to the above concerns about the exact location of the current meters with respect to an idealized bed level, and to the form of equation (2.21), we will consider shear estimates herein as actually describing the local momentum transfer through the fluid. Those internal shear estimates taken relatively close to the bed will be assumed to be comparable to the the actual force acting per unit area of bed.

3.4 Results from flow measurements

Two hydrological events will be considered. The first one which occurred on November 20, 1986 corresponds to approximately half-bankfull conditions. A second event on November 23, 1986 during which the bankfull level (about 63 cm on the staff gauge) was reached will be of most interest.

Twenty-one stations were sampled during both events. Figure 10 shows the variation in water stage during the period of measurement, with labels for velocity sampling stations. November 20 measurements were taken a few hours after the water stage approached bankfull stage. The stage gradually diminished by about two centimeters during the measurement period and all the measured profiles may thus be compared. In contrast, November 23 measurements were taken at the peak and during the recession of a flood. Consequently, the data from stations A to K and L to U will be considered in two separate groups. As a consequence of the unsteadiness of the flow, near bankfull measurements were limited in number but provided a reasonable spatial coverage of the reach.

The average hydraulic parameters of both events are summarized in table 2. These values (except the slope) were obtained from one cross-sectional estimate (stations D to H on November 23 and e to k on November 20), discharge being estimated by the velocity-area method. November 23 data correspond to near bankfull stage and will be particularly useful for modelling considerations developed in the next chapter. The water surface slope could be estimated from 5 reliable water elevation pairs along the creek on November 23 (at near bankfull stage) and 18 pairs on November 20.

Detailed hydrodynamic measurements will now be examined by considering the shape and local and temporal variability of the velocity profiles as well as the shear and flow patterns through the studied reach.

Let us first consider what indicates the shape of the

Figure 10. Temporal changes in water level

November 20 and 23, 1986 events

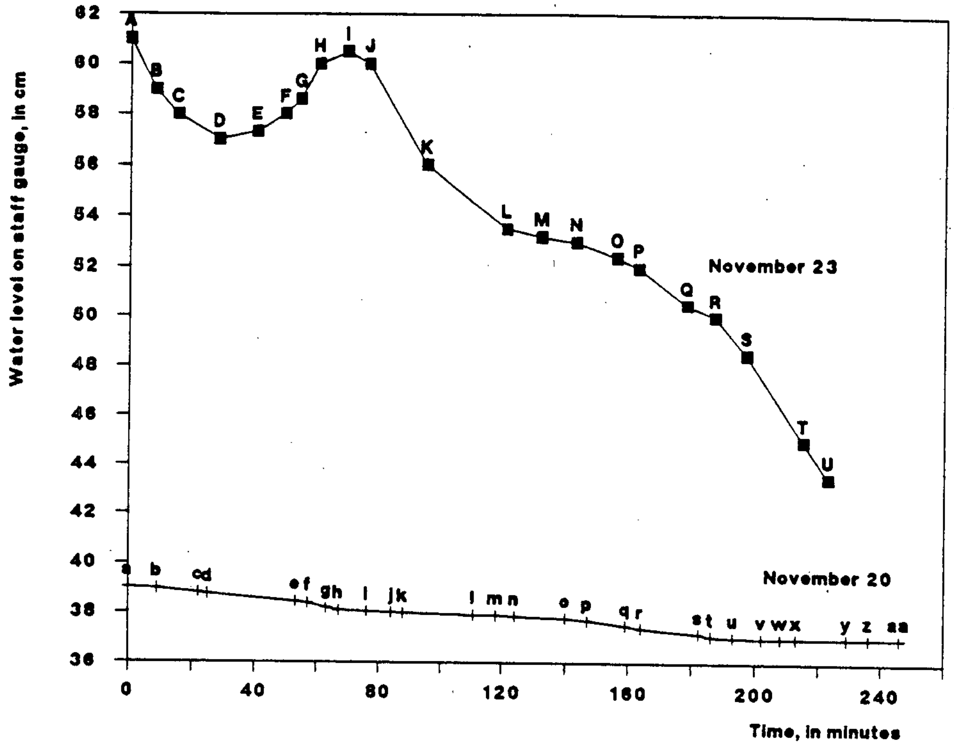


Table 2

Mean hydraulic parameters of November 20 and 23, 1986 hydrological events, Blaney creek

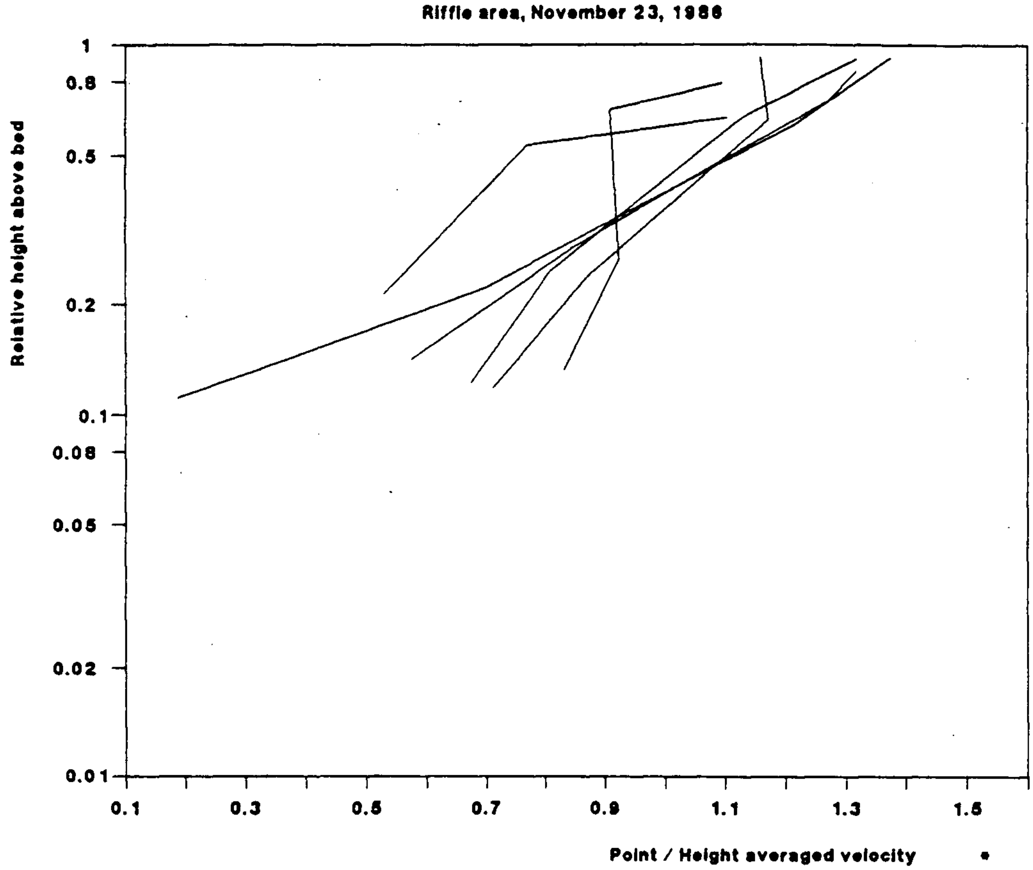
	November 20	November 23
Discharge ($m^3\ s^{-1}$)	2.62	5.00
Mean velocity ($m\ s^{-1}$)	1.11	1.30
Mean depth (m)	0.34	0.55
Mean water surface slope	0.0095	0.0102
Hydraulic radius (m)	0.31	0.475
Froude number	0.61	0.56
Reynolds number	240 000	470 000
Friction factor	0.19	0.225

velocity profiles through the study area in terms of physical processes. Apart from a few stations located close to the right bank which bear more complex forms, most of the November 23 profiles were kinked. Close to the bed, a wake layer was found, the characteristics of which are mostly determined by the boundary materials and their arrangement. Above it, an outer layer could be recognized, the characteristics of which are rather determined by the channel configuration. Being dependent on local factors wake layer characteristics were more variable than those of the outer layer, as found by Nowell and Church (1979).

It can be shown from equation (2.21) that the slope of the velocity profile is proportional to the internal shear intensity. Figures 11, 12 and 13 respectively show dimensionless velocity profiles for the riffle, the pool-group 1 (at high stage) and the pool-group 2 (during recession). In general, these figures indicate that the highest velocity gradients in the wake layer were measured over the riffle area. One can also note that in a majority of situations internal shear through the outer layer was greater than that of the wake layer for pool profiles: in contrast, the internal shear is more similar throughout the whole depth over the riffle even though the profiles at the latter location still appear kinked.

November 20 velocity profiles (not shown herein) were more regular through the pool and often appeared to comprise only a wake layer, especially over the riffle, as the flow depth was reduced. In actuality some stations over the riffle were too shallow to allow satisfactory velocity profiles measurements.

Figure 11. Dimensionless velocity profiles



*
Height averaged velocity was approximated by the velocity at 0.2 and 0.8 the water depth

Figure 12. Dimensionless velocity profiles

Pool area, November 23 1986

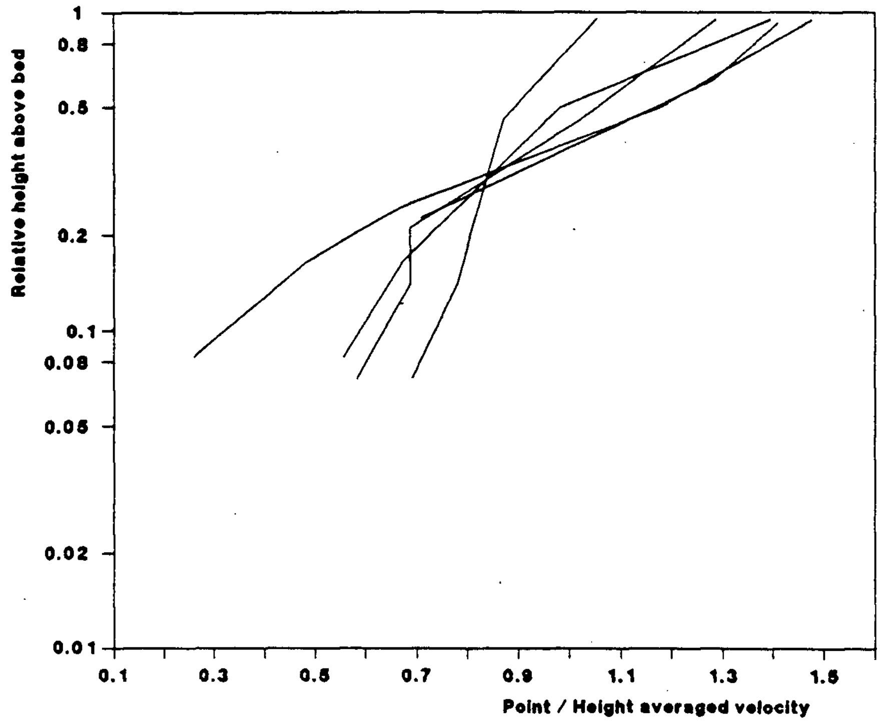
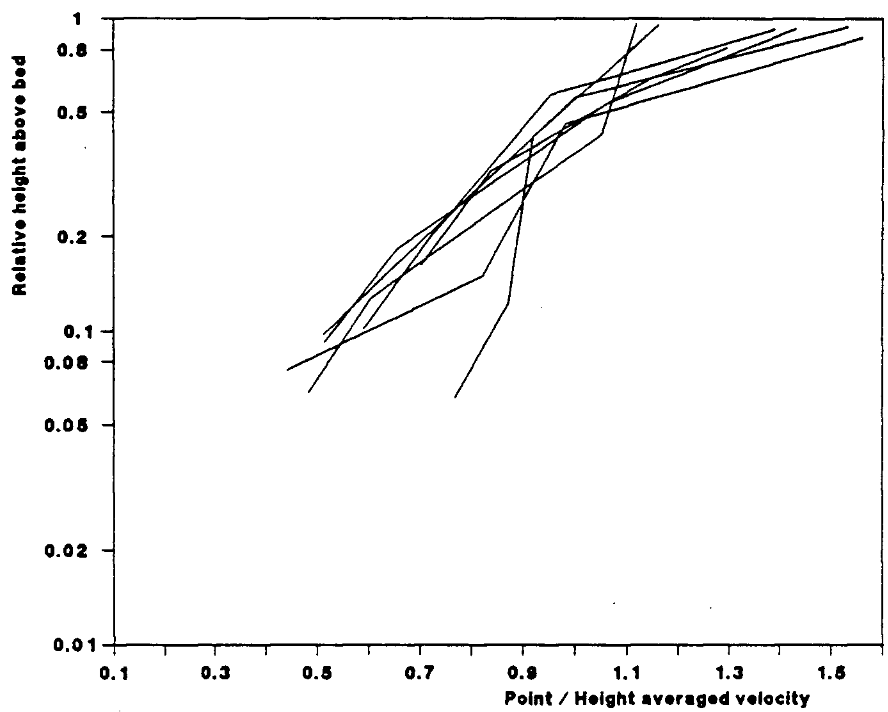


Figure 13. Dimensionless velocity profiles

Pool area(lower stage),November 23 1986



During the November 23 event, no evidence was noted of jetting (Rood, 1980) of the flow from the riffle into the distal pool. In general, the velocity profiles in the pool were complex but (except close to the right bank) bear the wake shedding shape characteristic of high relative roughness streams, as can be seen from figures 12 and 13.

Let us now address the issue of the local variability of the velocity profiles. During the November 23 event, some simple tests were done in order to determine the representativeness of one local velocity profile measurement. At stations L (pool near right bank) and P (near riffle crest), two additional one-minute profiles at a distance of approximately 10 cm from the station were collected. These two stations can be located on figure 18. These considerations about the local divergence of the velocity profiles are relevant to the spatial variation of the internal shear and further qualify how these hydrodynamic estimates might be interpreted. For station L, one profile shows a clear divergence in the outer layer (figure 14). This might be attributable to the proximity of the banks and in particular of a hanging log (see figure 5 or 6) which may distort the near-surface local flow field. Wake layer gradients were substantially similar but their intensity was somewhat different. Internal shear estimates from the bottom of the profiles differed at the most by 20 %. Station P showed some consistency in the outer layer gradient between profiles although the wake layers were markedly different (figure 15). Internal shear estimates in this case differed by up to 3.6

Figure 14. Local variability in velocity profiles

Station L, November 23, 1986

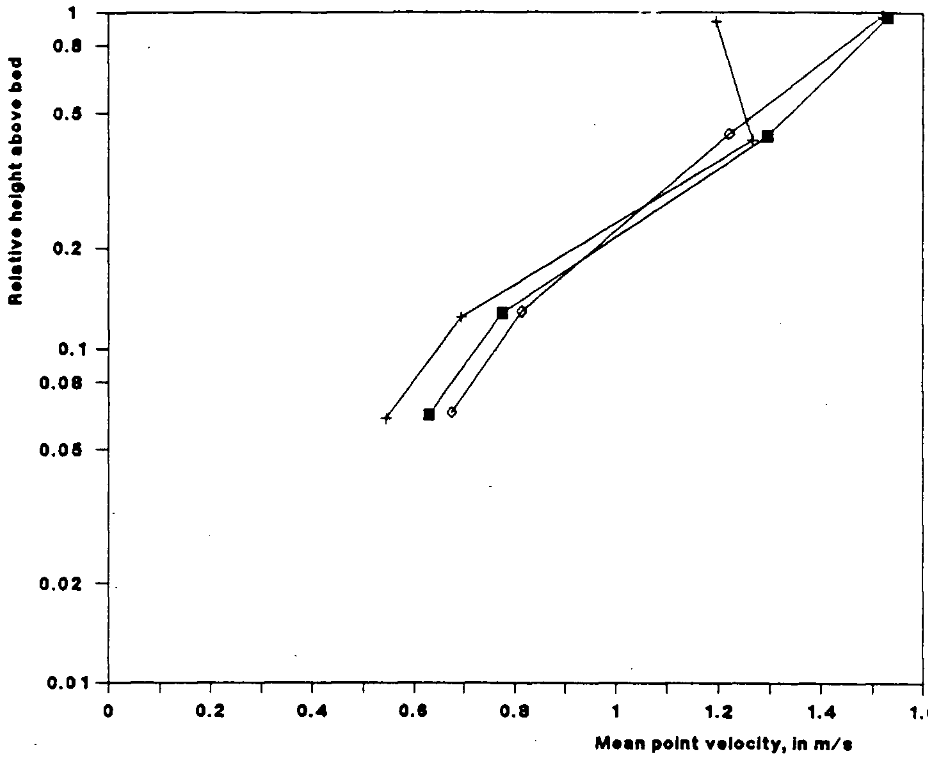
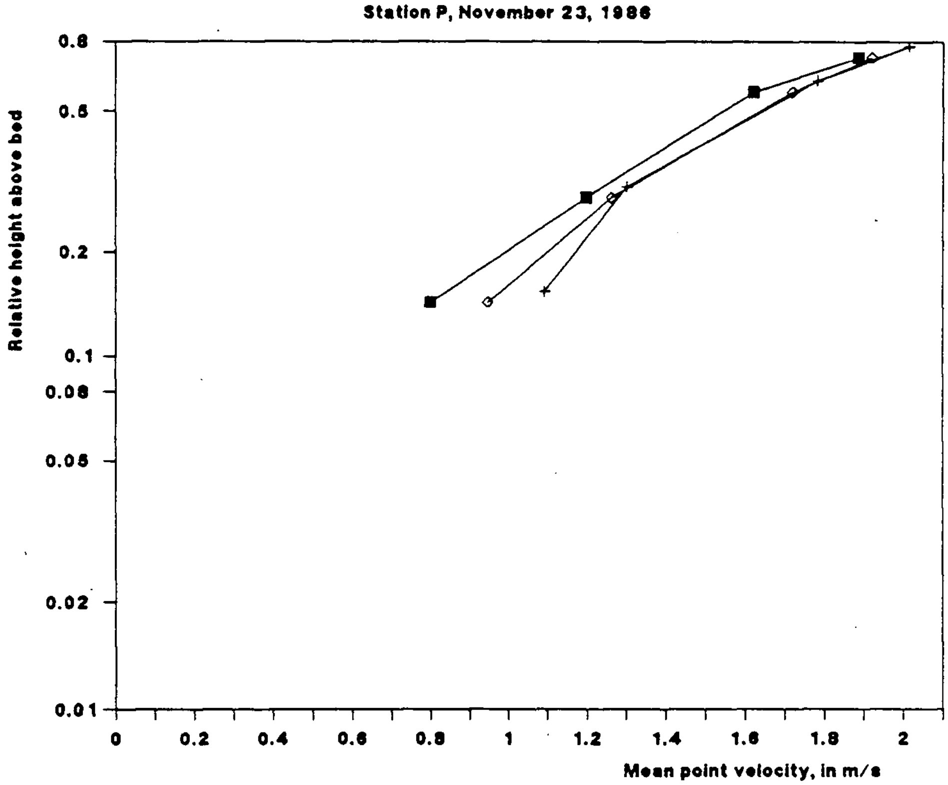


Figure 15. Local variability in velocity profiles

Station P, November 23, 1986



times, which is considerable. Station P is also located near the riffle crest, which may correspond to a most spatially variable area in terms of velocity distribution. It was unfortunately impossible to study this issue further in the field.

Some simple tests were made at the end of the November 23 period on the replicability of a velocity profile in order to investigate potential sources of variability from a temporal viewpoint. Ten or nine sucessive measurements were taken while the water level was nearly constant at Station V and W respectively (stations shown on figure 18), the former located at the downstream end of the distal pool and the latter over the distal part of the riffle. Station V profile was located in the intense shearing area along the right bank (see further description below). The velocity profiles were indeed rather complex (figure 16) and this may be attributable to the local protusion of the bank as well as to the intense mixing and perhaps to jetting effects. In any case, the variability in the one-minute average profiles for this pool location is interesting, especially near the boundary. Outer layer behaviour is seen to be more consistent. Station W (figure 17) on the riffle shows kinked profiles from which wake and outer layers can be recognized. The average profiles for the latter station are more consistent than for the pool location even near the bed. Therefore, at least for some pool locations, the temporal variation in the form of the velocity profiles may further discourage strong generalization from a unique profile for one particular station. This consideration, combined with

Figure 16. Scans at downstream end of pool

Station V, November 23, 1986

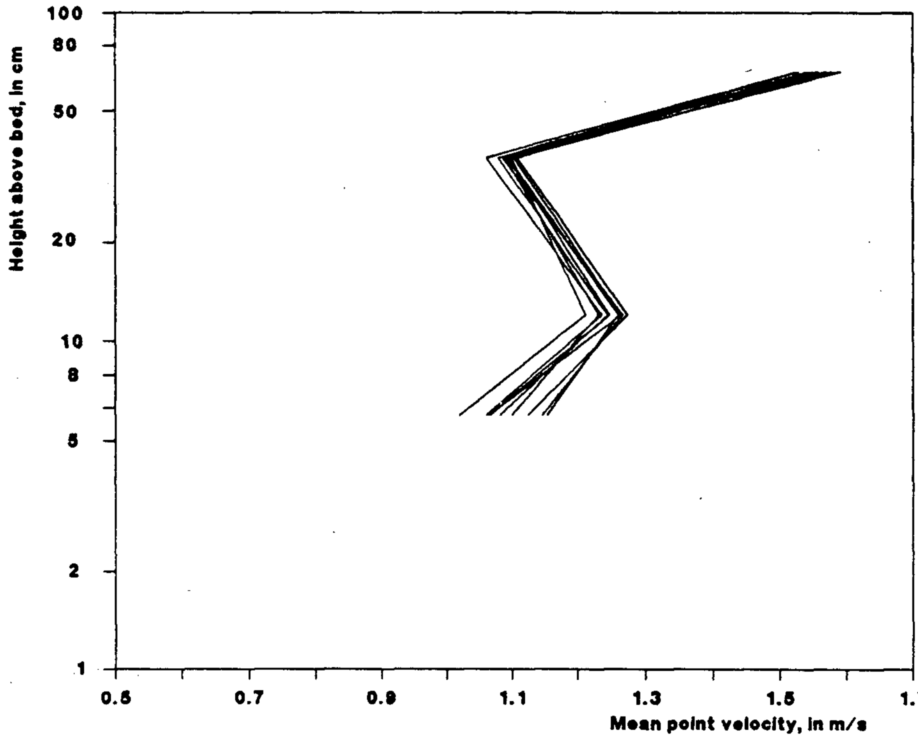
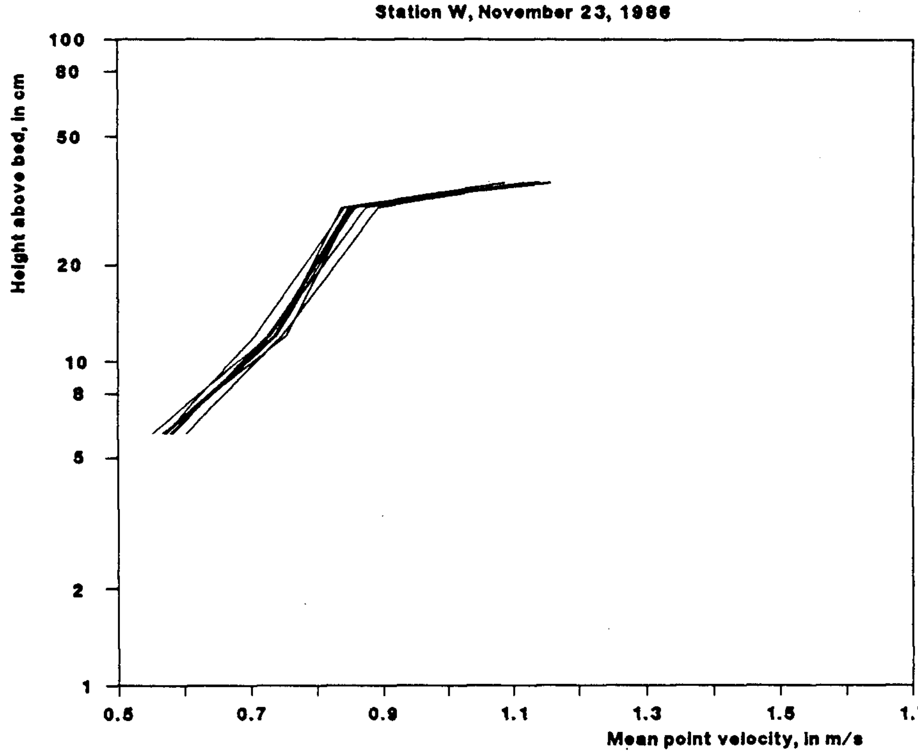


Figure 17. Scans at downstream end of riffle

Station W, November 23, 1986



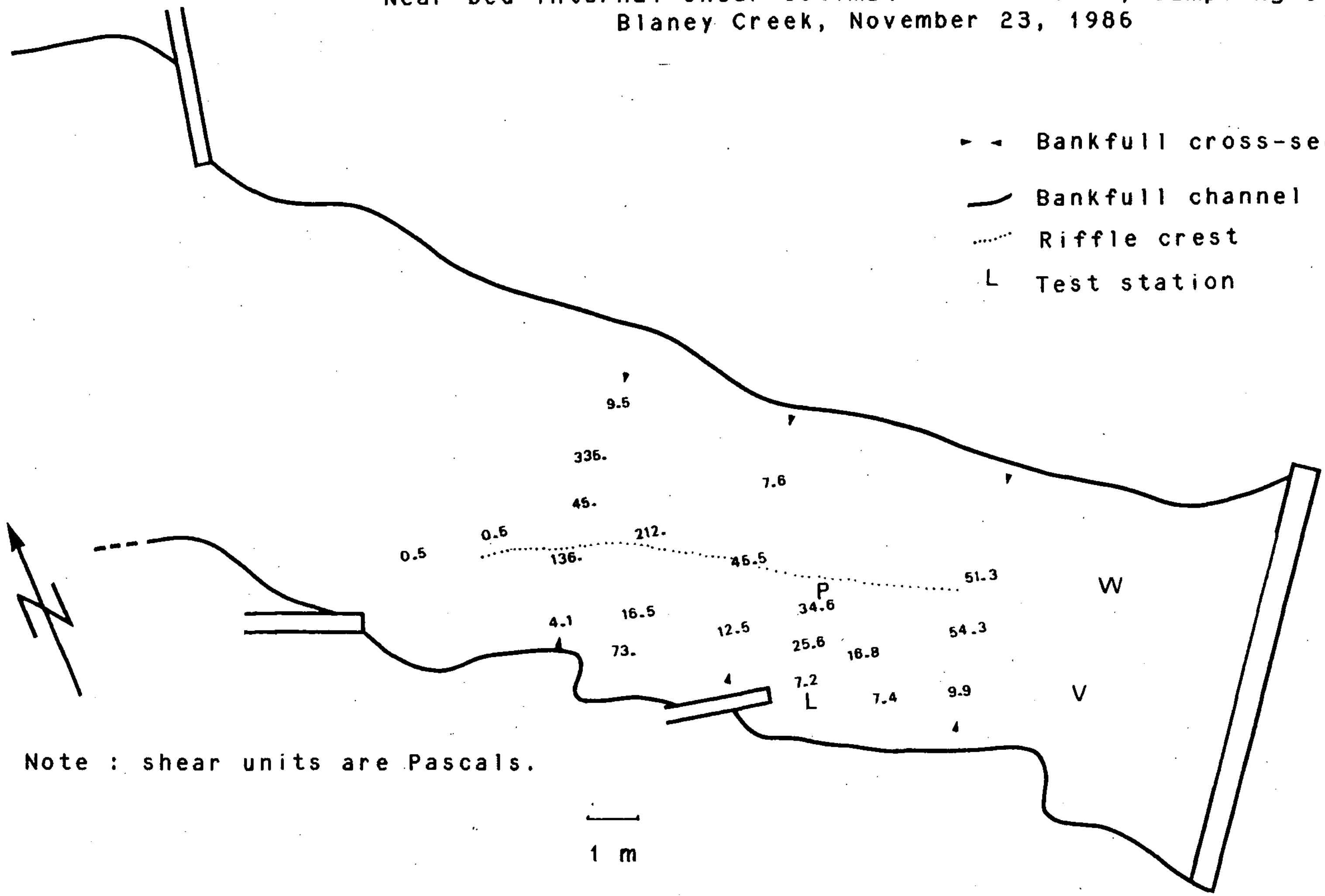
the local variability of some profiles, was important to consider before any interpretation of the shear estimates can be produced.

Figure 18 shows the distribution of the values of internal shear estimated via equation (2.21) for November 23. For those stations which were replicated, average values are considered. According to 11 comparable estimates, average internal shear at high flow stage was larger on the riffle (82 Pa) than in the pool (44 Pa). Using the uniform flow formulation (equation 2.16), the shear stress is estimated to be 47.5 Pa, i.e. somewhat lower than the average of the 11 near bankfull stations: however, these stations did not cover sufficient bed area to argue on a strict comparison. At lower stage on November 23, only one value was measured over the riffle (212 Pa) while the average of seven shear estimates in the pool was 25.8 Pa. Despite local variability, November 23 measurements tend to suggest that the highest shear values (335, 212 and 136 Pa) were recorded over the riffle or near the riffle crest as well as in the most upstream part of the study reach.

According to the measured stations average shear at the lower discharge of November 20 was 22.2 Pa on the riffle and 21.8 pa in the pool, i.e. the shear appeared to be similar on the average, although quite variable within each bed unit (figure 19). These shear estimates are somewhat smaller than the uniform flow approximation which amounts to 28.9 Pa. Once again, highest shears were measured in the upper part of the study reach.

Figures 20 and 21 show the flowlines for the November 23

Figure 18.
Near-bed internal shear estimates at velocity sampling stations
Blaney Creek, November 23, 1986



Note : shear units are Pascals.

Figure 19.
Near-bed internal shear estimates at velocity sampling stations
Blaney Creek, November 20, 1986

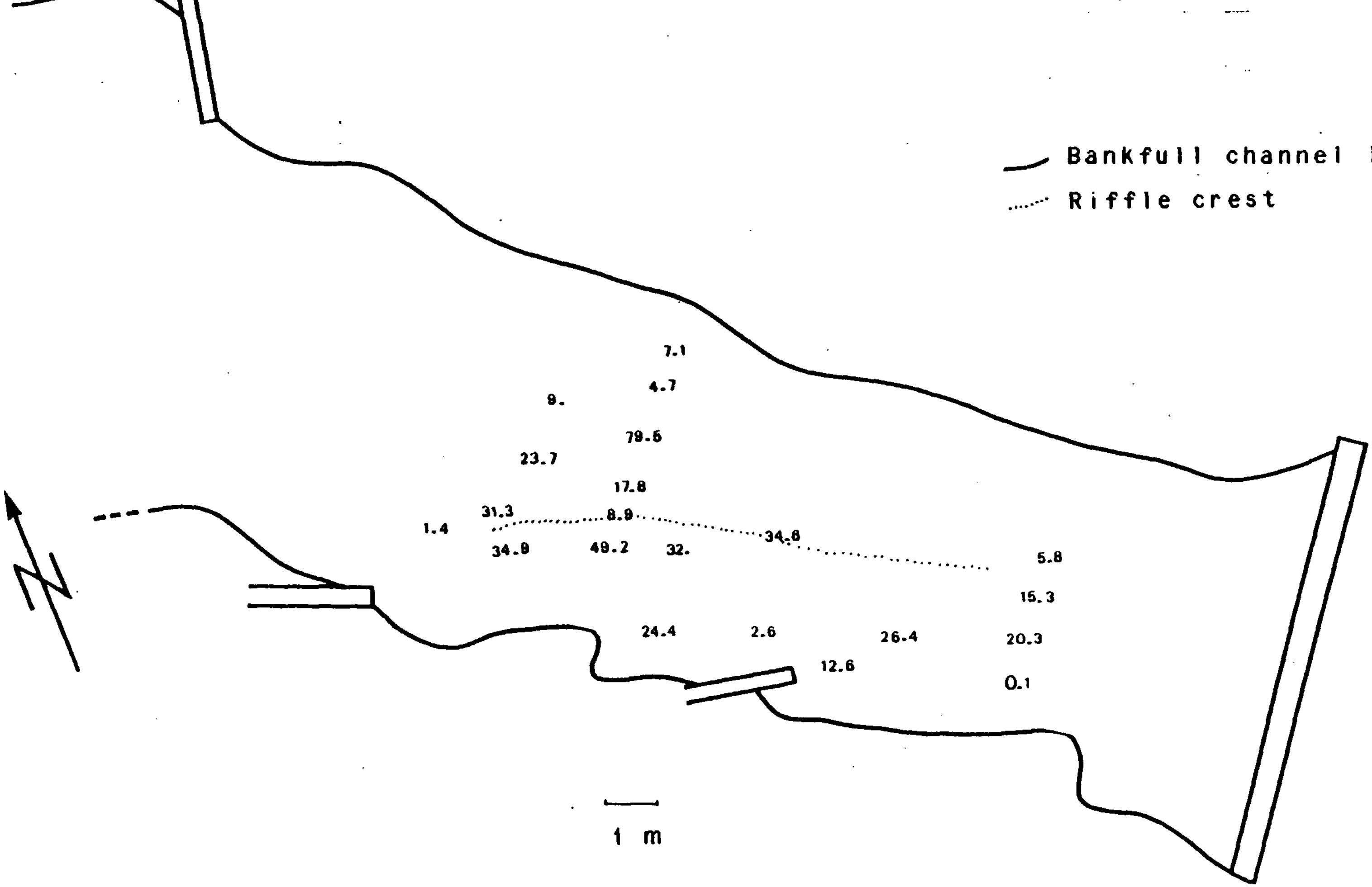


Figure 20.
Near-bottom and near-surface flowlines, Blaney Creek, at near bankfull stage, November 23, 1986.

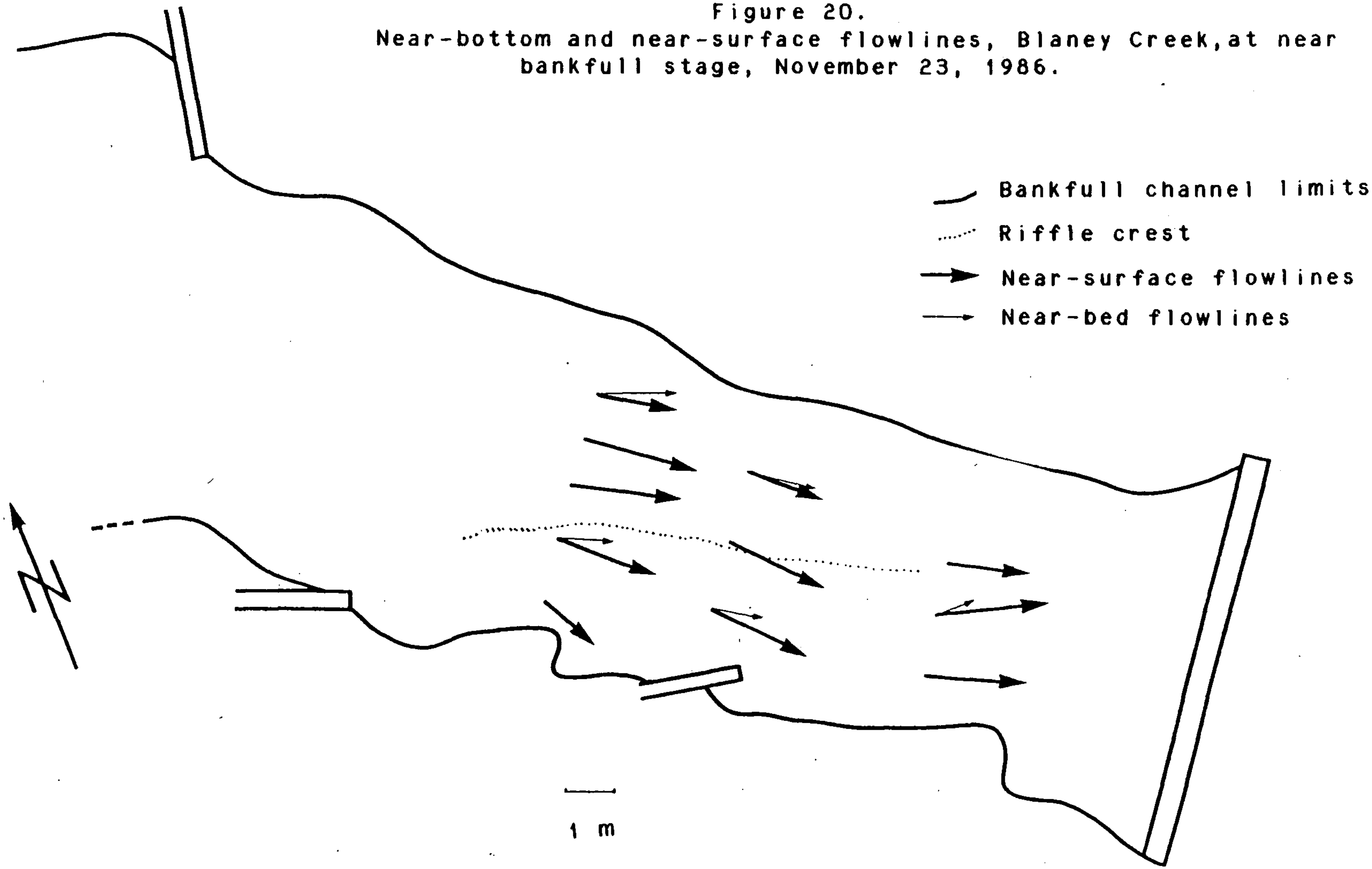
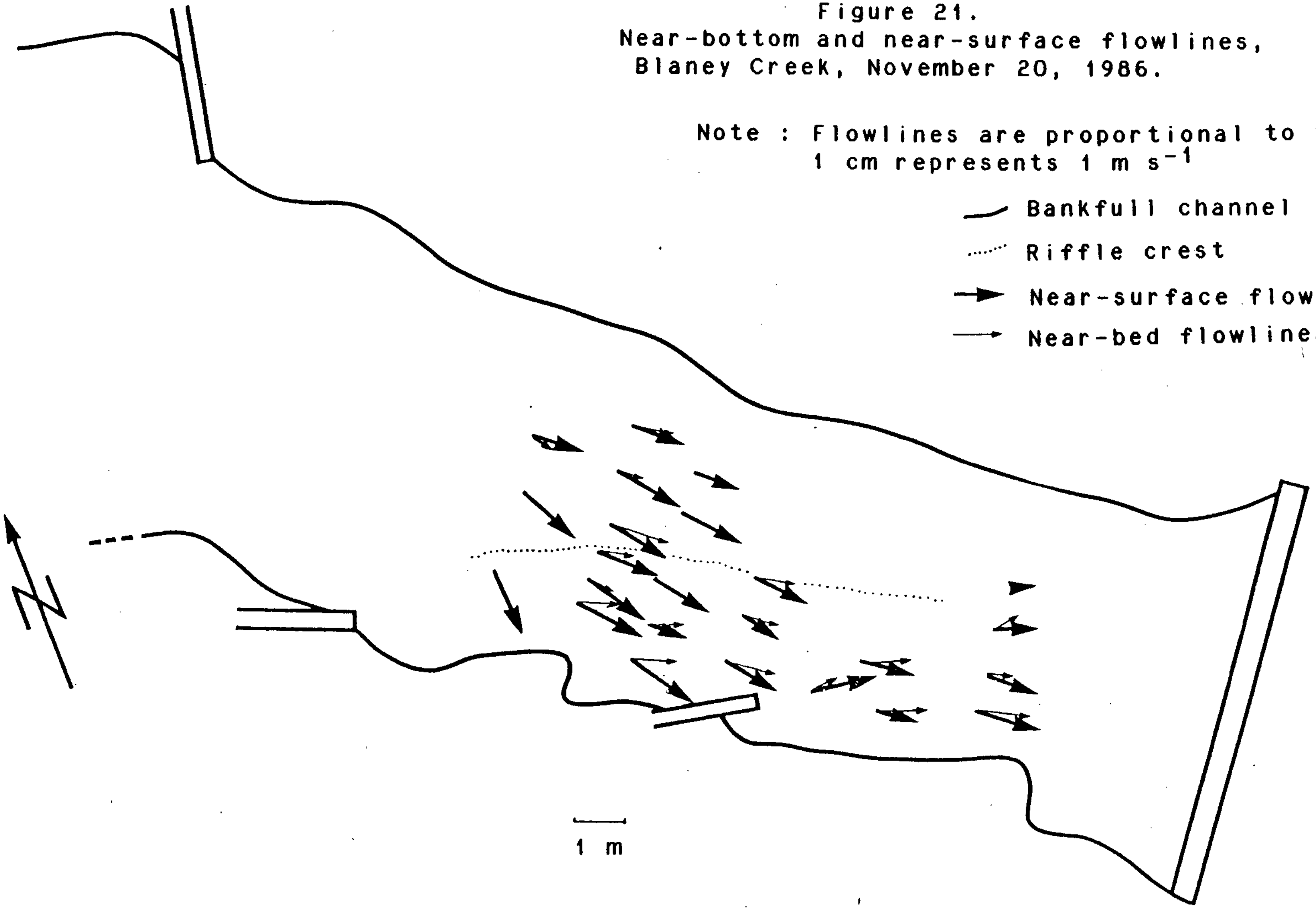


Figure 21.
Near-bottom and near-surface flowlines,
Blaney Creek, November 20, 1986.

Note : Flowlines are proportional to velocity.
1 cm represents 1 m s^{-1}



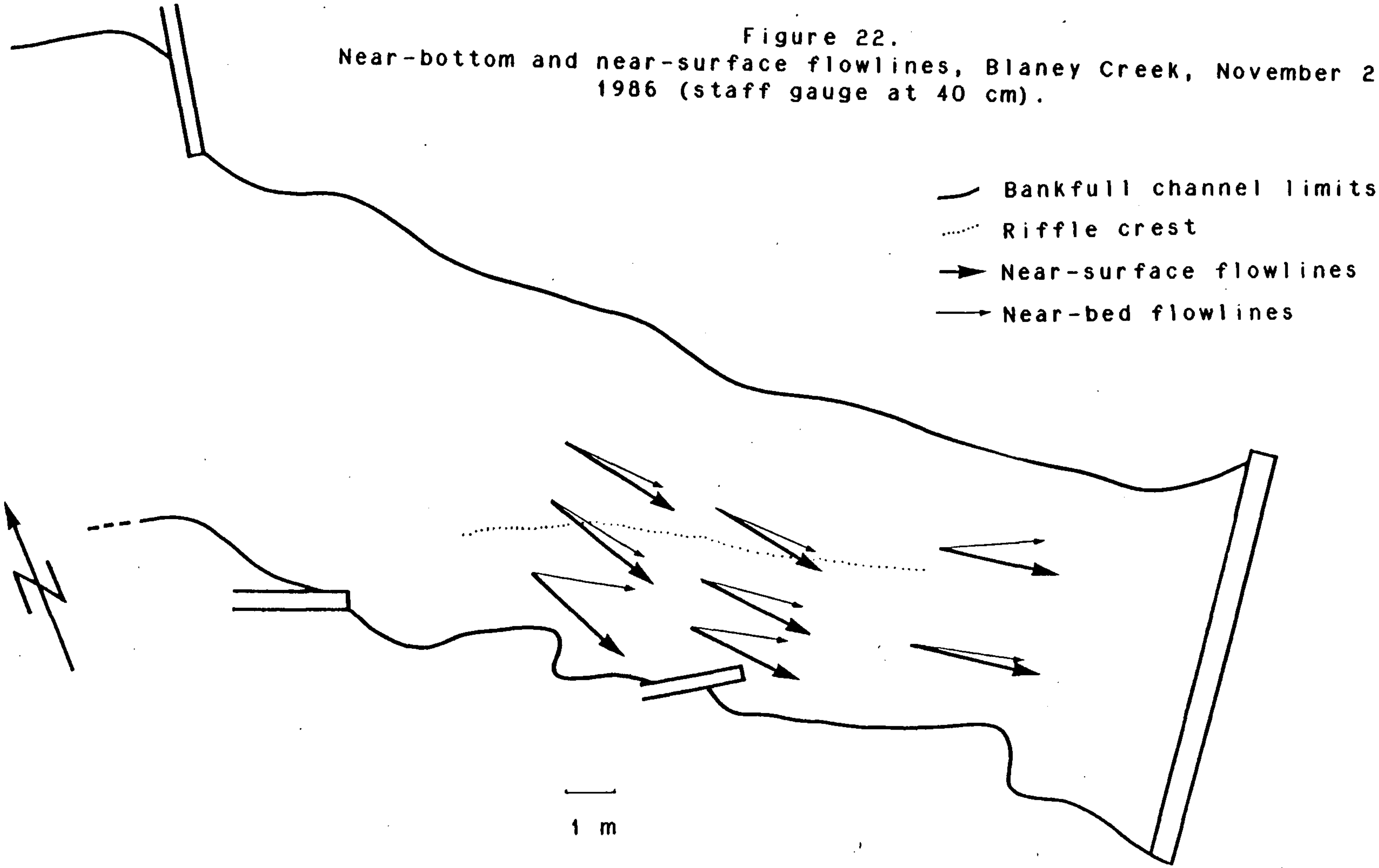
(at high stage only) and November 20 events respectively. The structure of the secondary flows in the distal pool could be related to the pressure distribution arising from surface flow impingement even if at a low angle (Gallinatti, 1984). The flow structure is particularly well developed and warrants some qualitative comments.

The comparison of figures 20 and 21 reveals that particularly the surface flow tends to be more aligned with the banks at higher discharge. Figure 22 presents the results of flow structure observations made at the end of November 23 event when the flow stage was almost that of the event of November 20 (water level on staff gauge was 40 cm). These observations were collected from the bridges with 3 m long plastic flags and a compass. The wide divergence between the surface and bottom flowlines is in every case evident. Divergence is greater at low flows than at higher stages. In the distal pool the bottom flowlines tend to be parallel with the riffle face. Surface flow is oblique to the riffle axis in the upstream part of the distal pool but subparallel to it in its downstream end. This well developed circulation may have some consequences for sediment evacuation from the pool. It also will be useful to investigate the field-flume comformation in the following chapter.

3.5 Problems raised by the field measurements

The topics in this section cover the reversal hypothesis (described in the appendix) plus observations on bedload

Figure 22.
Near-bottom and near-surface flowlines, Blaney Creek, November 23, 1986 (staff gauge at 40 cm).



transport and transport conditions as well as flow structure during November 23 event.

If one believes the above internal shear values and if one makes abstraction of their limitations due to restricted spatial coverage (particularly over the riffle), it would appear that the internal shear increases more over the riffle than in the pool as discharge increases. In the same perspective, it is perhaps worthwhile to mention that the analysis of the velocity patterns for the lowest current meters also suggested that near-bed velocities were higher over the riffle at near bankfull conditions. These measurements appear contrary to the velocity or shear stress reversal hypotheses described in the appendix. Although the bed topography through the reach is also controlled by the obstructions to the flow, the apparently lower shear in the distal pool raises unavoidably the issue of its maintenance. If in Blaney creek sediment transport of some calibre and intensity occurs at near bankfull, how can the pool then not fill? The spatial coverage of the field measurements is not sufficiently dense to settle the latter issue.

However, some other, more general considerations about the conditions in Blaney Creek on November 23 are worth discussing.

On November 23, only three bedload transport sediment samples from the lower bridge were taken at bankfull conditions, using a 76 mm opening Helley-Smith sampler. Those very restricted, one minute samples indicated that particles at least of medium gravel size (10 mm) were transported and that most transport seemed to occur along the base of the riffle face. It could not be directly determined if larger material was actually mobile. Although there is no definitive indication that the discharge we measured, albeit near bankfull, was of formative character, the apparent trend in the shear stress is unlikely to change much until the creek goes overbank. Lower stresses in the distal pool, in the face of the existence of the pool, would either indicate low transport intensity or preferential routes for sediment of various calibre. The latter possibility would be related to the lateral sorting of sediment downstream from the flow obstruction in the proximal pool. Gallinatti's (1984) qualitative model of the flow field around streambank obstructions, described in the appendix, may provide the basic mechanism for lateral sorting to occur. This process could not be verified directly during the hydrological events but was to be examined in the flume experiments. Nevertheless, the facts that finer sediments are found near the right bank below the outcrop and that very coarse materials accumulated immediately downstream of the obtruction suggest that such a process may be active. Lateral sorting in the proximal pool can further be appreciated from figure 23.

The transport conditions on November 23 can be further characterized by considering the ratio of the average dimensionless shear stress to the dimensionless critical shear stress necessary to entrain some characteristic bed particles. Using Andrews' (1983) results, which were developed from a series of observations on various size particles transported in single thread, gravel bed rivers, the critical dimensionless shear stress necessary to entrain the D_{50} of surface materials for Blaney creek would be 0.046. Using the D_{50} of the surface and the hydraulic radius, the dimensionless shear stress for the bankfull flow is calculated to be 0.057, a typical value for small gravel bed streams with reinforced banks (Andrews, 1984). Therefore the critical conditions for surface material entrainment would have been exceeded by 27% (mobility number of 1.27) for November 23 bankfull flow. This exceedance is low compared to most active gravel bed rivers with alluvial pool-riffle bedforms which on the average bear an exceedance of about 100 % (mobility number of 2.0) at bankfull discharge (this was estimated from the data of Andrews, 1984).

Simple calculations, using the D/S90/s as a criterion, reveal that an increase of more than 50 % in discharge or shear could mobilize the bed in general. A lesser increase would probably initiate local movement on the bed since the shear is not uniformly distributed over the bed. Nevertheless, such a rise in flow would cause overbank flow which would increase flow resistance and consequently lower τ_0 (Hirsch and Abrahams, 1981).

It might therefore be proposed that Blaney creek experiences low mobility conditions at near bankfull stage, although the creek is not paved in the sense of Bray and Church (1980). The observation that the subsurface material is oxidized but not decayed tends to support such assertions. Perhaps low transport intensity can be reconciled with the continued existence of the distal pool in the eventuality that the latter represents a relict feature. Whether this is

Figure 23.
Lateral bed material sorting in proximal pool, Blaney Creek. On the left edge of the photograph begins the small accumulation bar.

Figure 24.
November 23, 1986 hydrological event, Blaney Creek. Staff gauge is approximately at 45 cm (i.e. 18 cm below bankfull). Yet one can clearly notice by the water surface appearance the existence of a shear line along the right bank.

reasonable or not will be discussed in chapter 4.

We may also explore some characteristics of the flow on a general level. It was noted during the field measurements that an 'eddy' or shear line such as described by Lisle (1986) developed in the distal pool at some distance from the right bank. Figure 24 shows the November 23 event during receding flow. A shear line is clearly visible along the right bank. Outside the range of the photograph the shear line extended to near the cross-over area. The same feature of the flow structure is present at higher flows but begins only at some distance into the pool. Its upstream extent appears to be related to the changing orientation of the streamlines with flow stage, as we described above. This flow feature suggested that the approaching flow was obliquely toward the relatively reinforced and cohesive right bank and thus that the origin of the distal pool could be similar to that of the proximal, i.e. obstruction controlled.

4.0 Laboratory study and conformation

4.1 Flume description, instrumentation and hydrodynamic measurement procedures

The flume used in this study is located in the Geography Department at the University of British Columbia. The tilting recirculating flume with transparent acrylic walls (see figure 25) is supported on a steel truss. A levelling jack near its downstream end allows bed slope to be adjusted up to about 0.02. The apparatus is about six metres long and has a bed width of 47 cm. A large reservoir at its downstream end prevents drawdown or backing up during pumping. From the bottom of the reservoir the water flows through a tilting 15.25 cm diameter plastic return pipe to a variable speed impeller pump driven by a 1.0 horsepower motor situated at its upstream end. Water is pumped up to the flume inlet where it flows through a honeycomb mesh before entering the flume bed. Sediment in transport falls into a downstream reservoir where only grains smaller than 2 mm can enter the return pipe and be recirculated. Particles coarser than 2 mm could not safely be recirculated through the pump.

The flume had no installed device for discharge

Figure 25.
General view of the water recirculating flume
at the Department of Geography,
The University of British Columbia

regulation. Discharge was thus estimated via the velocity-area method from one documented cross section (7 profiles, each with 5 to 7 one-minute interval measurements) at some distance from the flume entry. Because of this drawback, the discharge could be set only approximately for each run. A water level recorder was set to ensure that discharge stayed constant during runs.

Velocities in the flume were measured using a TSI 1239W ruggedized hot film probe with 1.5 mm hemispherical tip fixed on a 3.2 mm elbow rod. The very soft water from the municipal water supply substantially reduced the usual problem of probe contamination during measurements. During runs, water was constantly renewed from the tap.

The probe had been calibrated in a rotating drum during autumn 1986 by John F. Wolcott. It was operated at 19 degrees celcius overheat ratio. During measurements, the analog output signal from the probe was sampled at 10 Hz, processed by a TSI operating module and sent to a CR21X (Campbell Scientific Instrument) data logger. The data logger allowed the calculation of the mean voltage as well as the extraction of the minimum and maximum values during the one minute interval of measurement. One minute is a very sufficient interval to characterize the velocity field. Froudian similarity regarding field measurement time interval would require 15 seconds in the flume (equation 2.13). Accordingly, flume velocity profiles were likely to bear lesser variability. One-minute average as well as voltage values were converted to mean velocity values assuming that they followed a log-normal distribution.

At each measurement station, the probe's distance from the water surface and the water depth were measured manually using a millimetre ruler. Because of its size and shape, the probe could hardly be set closer than 2 mm from the bed. For proper operation, the probe must not be set too close to the water-air interface, i.e. about 5-6 mm below the water surface. Hence although we could easily get more detailed profiles than under the field conditions we could not get measurements as close to the water surface, all proportions kept.

Finally, water surface elevations were measured using a millimetre ruler set carefully at the water surface and read from a level. The same procedure was used in order to construct topographical maps of the final bed configuration of flume runs. Near-surface and near-bed flowlines were determined with respect to the flume walls using a flag and a protractor mounted on the same rod and read from above the flume. Errors in the latter readings were considered to be of the same magnitude as in the field, i.e. 2-5 degrees.

4.2 Scale modelling of Blaney Creek : strategy

4.2.1 General considerations

As was argued in chapter 2, in order to strictly verify field to laboratory conformation on a hydrodynamical basis, the present modelling exercise needed to follow undistorted Froude model scaling laws. The width of the flume determined the scale ratio from prototype to model for near bankfull conditions, i.e. the flume width was 16 times smaller than that of the creek. Accordingly the length of the study reach came up to be about 1.6 m in the flume. Such a scale ratio satisfies the requirements of equation (2.26), using k_s = 3.5 D_{90} for gravel-bed rivers (Bray, 1982). The other important length scale is that of the bed material, which will be discussed in some detail.

4.2.2 Bed material scaling

The matter of bed material specification for a modelling exercise is not trivial (recall sections 2.5.1 and 2.5.2). In the case of most sand bed rivers, it is this practical consideration which prevents in many situations the utilisation of an undistorted model. The paragraphs which follow describe the procedures which were taken to scale the bed material size distribution of Blaney Creek in order to best meet mechanical similarity requirements.

A first step in the scaling of the volumetric bed material sample consisted of truncating it at the suspended-bed material load limit using the flow conditions of November 23. The following suspension criterion (Parker et al, 1982) was used :

$$\frac{V*}{V_s} > 1.0 \qquad (4.1)$$

where V_s is a particle settling velocity as defined by the Rubey-Watson law (Dingman, 1984). Accordingly, the upper limit for suspension for November 23 conditions was found to be about 1.25 mm. This resulted in the truncation of about 7.6 % of the size distribution.

The truncated size distribution was then Froude-scaled to obtain a target, model size distribution. Truncation eliminated some particle sizes for which scaled particle Reynolds number would have likely been relatively low (all finer than very fine sand in the model) and which would not have allowed proper entrainment conditions to be preserved, as discussed in section 2.5.2. The truncated field size distribution has D_{16}, D_{50} and D_{84} respectively of 5, 29 and 93 mm (see figure 26).

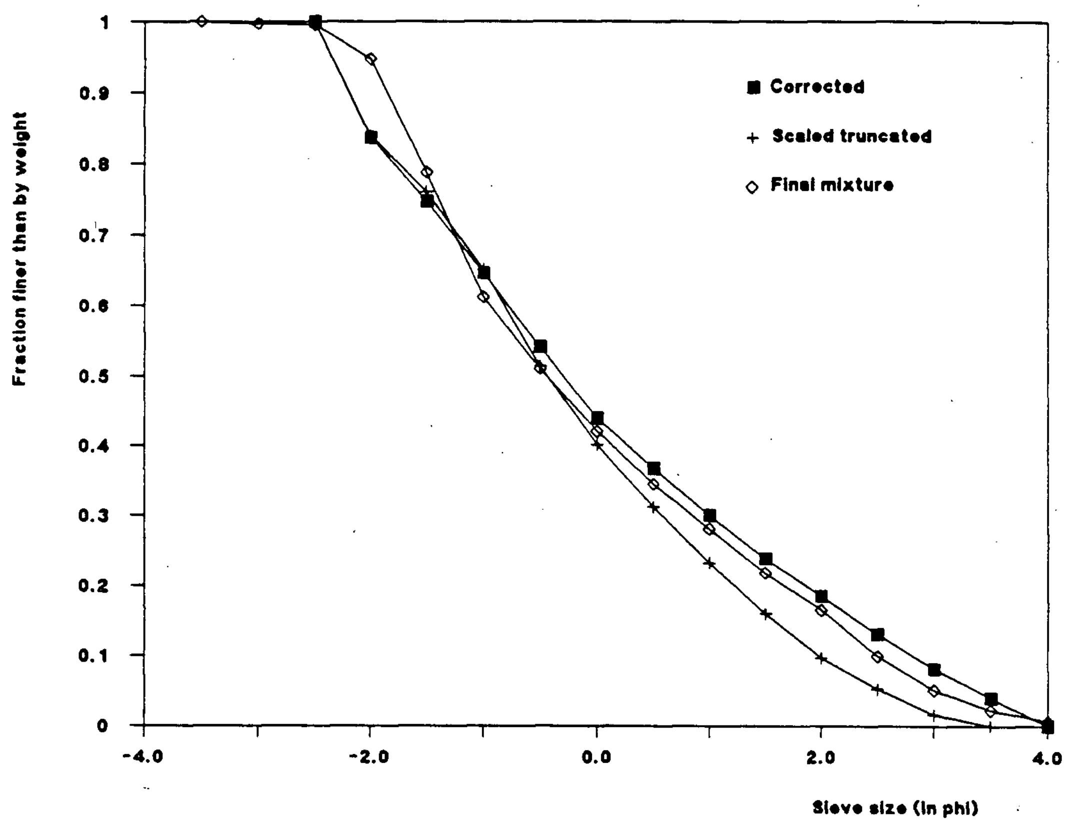
Ripples can occur in the model if a certain percentage of the size distribution has a particle Reynolds number smaller than 5, i.e. the situation for which laminar flow conditions occur in the wake of the particle. Jaeggi (1986) commented that these conditions can interfere with bar formation in models of gravel-bed rivers. He also recommended that a particle Reynolds number of 5 should represent a minimum condition for gravel-bed river model sediments. According to the above considerations, a correction procedure as proposed in Jaeggi (1986) can be applied in order to improve the situation. This procedure is concerned with the model size distribution particles in the transitional flow regime (i.e. particle Reynolds number from 5 to 70). In flow mechanics terms, and according to the shape of bed material entrainment curves in several sediment transport studies, the critical shear velocity (or resistance) in the model would be too small for those field particles with fully turbulent wake which fall in the transitional particle Reynolds range when scaled. The

Figure 26.

Comparison of laboratory sediment mixture with

the corrected and non-corrrected scaled size distributions

of the truncated volumetric sample of Blaney Creek

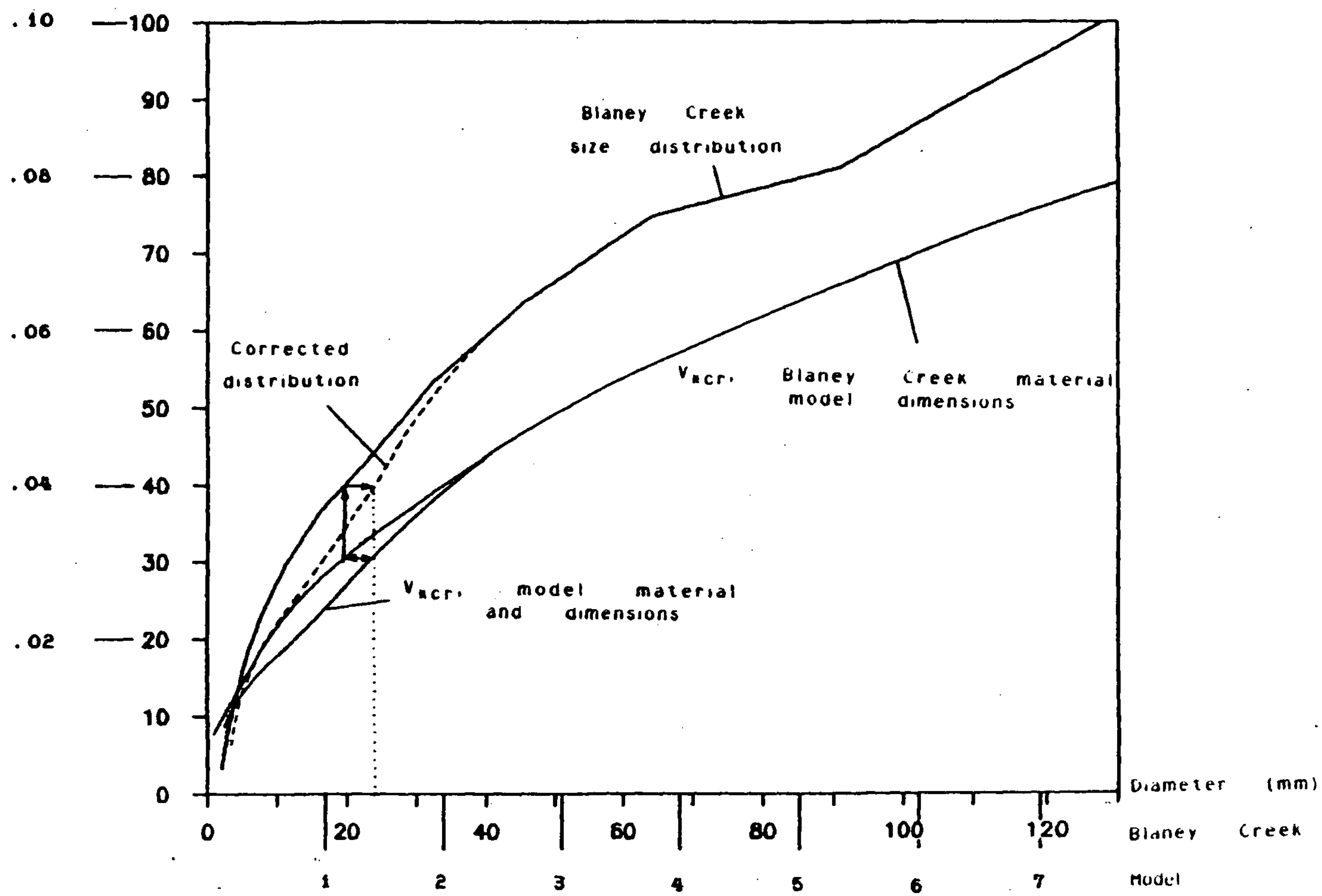


correction procedure is graphical and it is shown in figure 27. It will now be described in some detail.

On figure 27, two scales are present on the abcissa, i.e. that of the prototype and that of the Froude-scaled model grain size distribution. Also, the curves of critical shear velocities at model dimensions for both size distributions are shown. Following Jaeggi (1986), we assumed in order to calculate the critical shear velocities that the Shields factor (equation 2.25) was constant at 0.03 for $5<Rp<10$, changing linearly for $10<Rp<70$ and constant at 0.05 for $Rp>70$. At model flow conditions, for grains larger than about 2.3 mm in the model, the grain resistance or entrainment conditions with respect to the field are properly reproduced and the two V_{*cr} curves overlap. The divergence of those curves for grains smaller than 2.3 mm indicates the difference in resistance conditions around bed particles in the model if only Froude scaling is applied. Shifting of the grain size distribution curve by the amount corresponding to the discrepancy between the two V_{*cr} curves, as shown in figure 27, results in a correction of the size distribution. The finally adopted curve is somewhat coarser than the simple geometrically scaled truncated size distribution and is aimed to give a better reproduction in the model of entrainment conditions present in the prototype.

Figure 26 also presents the final mixture used in the modelling exercise. Also shown on figure 26 are the Froude scaled (scale ratio of 17 had been used at that time) truncated and the corrected size distributions for comparison. It is

Figure 27.
Critical shear velocity-based graphical correction procedure to the truncated volumetric sample of Blaney Creek



seen that the mixture adopted reproduces sufficiently well the scaled and corrected size distribution. The latter actually falls between the the two target size distributions below the D_{50}. The match is pretty good in the vicinity of the D_{50}. Above it, the selected mixture departs from the desired curve, actually being finer than desired above the D_{80}. In any case, field samples usually only represent approximations (Jaeggi, 1986) so this mixture is deemed acceptable, despite some differences. Finally, it should be mentioned that the field and laboratory sediment had the same specific gravity, i.e. just about 2700 kg per cubic metre.

4.2.3 Working assumptions regarding field conditions

Since it is associated with a discontinuity in morphologic (Williams, 1978) and hydrodynamic properties, the bankfull stage is often assumed to bear some morphogenetic significance (Knighton, 1984) in alluvial rivers. Although in a variety of rivers the long-term morphology is likely adjusted to a range of formative discharges, it is perhaps in those streams for which excess dimensionless shear stress at bankfull conditions is small that the equation of the dominant and bankfull discharge is most reasonable. A working assumption in this research project was to accept the latter statement in order to attempt to develop bed morphology from an initially flat bed. Perhaps the greatest risk associated with this assumption should be related to the memory of a particularly high flood or to the inherent historical component of any environmental system, as stated by G.G.Simpson (1963). Therefore, in order to perform laboratory experiments focusing on the selected pool-riffle sequence hydrodynamics, we had to collect a set of velocity profiles at near bankfull conditions.

The latter decisions were also apparently supported by some hydrodynamic measurements and sediment transport considerations introduced in chapter 3. The fact that transport of bed material sediment appeared relatively low at a supposedly near formative discharge suggested that it was possible to run the flume without feeding sediment at the inlet as must usually be done in the case of a more mobile river. Therefore, the approach was to run the flume until the bed stabilized and transport rate approached zero, a condition which usually occurred after a few hours of running. When the such conditions were attained velocity measurements were performed.

The next considerations regard the details of the configuration of Blaney creek field site. Early in the laboratory work it was hypothesized that the details in the creek configuration were not all important in determining the creek's bed topography and flow processes. Hence, the presence of the bedrock outcrop and bedrock control for the proximal pool bed level as well as of an obstruction to the flow and of downstream control provided by the transverse log were judged to be most relevant to the reproduction of the field situation. Correspondingly one apparent advantage of Blaney creek site was that those well-defined boundary conditions would allow the comparison of ongoing fluvial processes at a local scale.

The characteristics of the creek also entailed a shift from a purely generic to a more specific modelling approach. The geometrical scaling of the upstream and downstream controls immediately determined the local, imposed bed slope: whence, the flume slope was accordingly set to zero. However, the final value of the hydraulic gradient will also be affected by the resistance of the boundary materials which, in turn, depends on appropriate reproduction of the flow conditions, as reported in chapter 2.

Another initial assumption was that bank resistance did not represent an important element in the reproduction of Blaney Creek hydrodynamics. On the one hand, this was motivated by the fact that bank resistance is rather small compared to bed resistance for channels with width to depth ratio above 10 to 15 (Knight, 1981). On the other hand, it also represented a working assumption as it appeared exceedingly difficult to model the roughness of Blaney creek banks, especially considering the undercut and sinuous character of the right bank (refer to figures 5 and 6). Therefore, at the beginning of laboratory work, the banks were smooth and straight.

The first runs were performed with the most simple configuration described above. The bedrock outcrop was scaled geometrically although the straight flume wall constraint introduced small lateral distortion. Figure 28 shows the model outcrop, built using lego blocks, a material which also provided some notional roughness. Comparison with figure 5 (the field site map) reveals the simplifications of its shape.

Figure 28.
Lego-built model of Blaney Creek bedrock outcrop

Furthermore, the obstruction to the flow and the distance between the upstream and downstream controls were also modeled geometrically. The scale ratio was 16 in all cases. Another important feature of the initial configuration is the upstream deflection of most of the upstream incoming flow towards the left bank as occurs in the field. This was also done by constraining the upstream channel to about 27 cm, considering its field width and the scaling ratio. This way, most of the incoming water flowed via the left side of the outcrop and impinged at a similar angle on the obstruction.

Finally, a thickness of 10.3 cm of bed material filled the modeled study reach flush to the downstream control level. The level of the non-alluvial proximal pool bed was however controlled by a plate glued with gravels and set at geometrically similar height. Before starting the pump the material was compacted and levelled. Then the pump was started and the discharge was gradually raised in order to first saturate the bed and then to a flow rate near or slightly below the desired one. Cross-sectional velocity measurements were then performed to determine if the flow rate needed to be readjusted.

4.2.4 Initial experience with and adjustments to the model

Bed topography developed very rapidly in the first few hours of each run and it could be compared with the target field site morphology. In each successive run, necessary readjustments were made one at a time in order to discriminate the effect of each configurational element on bed morphology. It was found that some additional factors needed to be considered, amongst them the necessity of notional roughness on banks (which prevented local scour along flume sides) and the modelling of the mound found downstream of the obstruction as well as of some reinforcement on the upper right side of the proximal pool in order to reproduce its non-erodible character (refer to chapter 3, field site description, and to figure 5). The mound adjusted the local amount of water discharge across the stream. Bank roughness was simulated using lego boards and the mound area and proximal pool reinforcement by small cobbles. Even with those adjustments it was impossible to grow the most important distal pool even if runs all lasted over 24 hours and even when the discharge was increased by a factor of 2 above the scaled near bankfull value.

Accordingly, the sinuous bankline of the right creek bank was geometrically scaled using aluminum sheet on which some sandpaper was glued to produce some notional roughness. Its introduction did not result in distal pool growth either. At this point in the laboratory work the hypothesis emerged that current creek bed morphology could be inherited from a different stream configuration. Some field evidences for a different stream configuration in the past can be observed about 10 m upstream from the bedrock outcrop. There are some signs there of an abandoned channel which would have flowed on the right side of the outcrop in the past. This channel is now blocked by an accumulation of large organic debris and sediments. The perturbed state of the low areas around the creek (attributed to the logging operations nearly three decades ago) unfortunately precludes any detailed field study of past creek configuration.

The emergence of the above proposal concerning the distal pool status was further stimulated by the fact, apparent from field measurements, that the internal or bed shear in the distal pool appeared to be lower than over the riffle at assumed formative discharge, a situation which appeared contradictory with respect to pool existence (as discussed in section 3.5). One possible past flow configuration could be tested within the straight wall flume by removing the deflection of the incoming flow made above the outcrop and thereby forcing more water to flow along the right bank. After several hours of running under the latter configuration, the current creek configuration was reinstalled and the run further continued. This did not result in appropriate distal pool growth either.

Another past event which affected Blaney Creek dramatically was the introduction of the transverse log which acted as a new downstream control. It is possible that the actual lowest bed level in the distal pool is related to the absence of the downstream log at some point in the past. However, altering the configuration of the creek in the laboratory in order to reconstitute the actual bed morphology and history was difficult due to straight flume walls.

It must be understood however that any scenario which either proposes that the distal pool is relict or that it has never been a pool in a morphogenetic sense implies low transport rate (at least over the pool) or the maintenance of the competence in the actual pool area, even if not for the largest clasts carried by the stream (refer to the lateral sorting hypothesis in section 3.5). For all the foregoing flume runs, it was observed that transport in what corresponds to the distal pool area only occurred during the process of surface coarsening early in the run. In contrast, active transport occurred for a period of several hours over the building riffle whose crest, less prominent due to the absence of the distal pool, was nearly similar to the field. In contrast to the distal pool, riffle growth is associated to the actual bed configuration since it consists of the extension of the proximal pool.

In light of those latter considerations, the final approach to the modelling of the Blaney creek site was to dig at scale the distal pool below the initial flat bed level and see if it would be preserved in the process of a run. The arrangement of the eventual riffle with a manually excavated pool was then going to be compared with the creek morphology. For these excavated pool runs (runs 13 and 14 below), all configurational elements described above were preserved and the upstream deflection of the flow reinstalled.

Before any comment is made on the results of these runs some general remarks about the relict pool hypothesis, the field site characteristics and the aim of this research project appear to be warranted. As pointed out by Simpson (1963), physical models "abstract what are believed to be the essential configurational similarities of historical events". Hence, despite the manipulation done with the creek configuration in the flume, the status of the actual distal pool cannot be fully ascertained. Finally, the situation with regard to the distal pool is an interesting lesson learned from the laboratory exercise rather than a drawback on the goals of this research project. In actuality, the comparison of field and laboratory flow characteristics will clarify the distal pool situation and will still allow the verification of similarity principles.

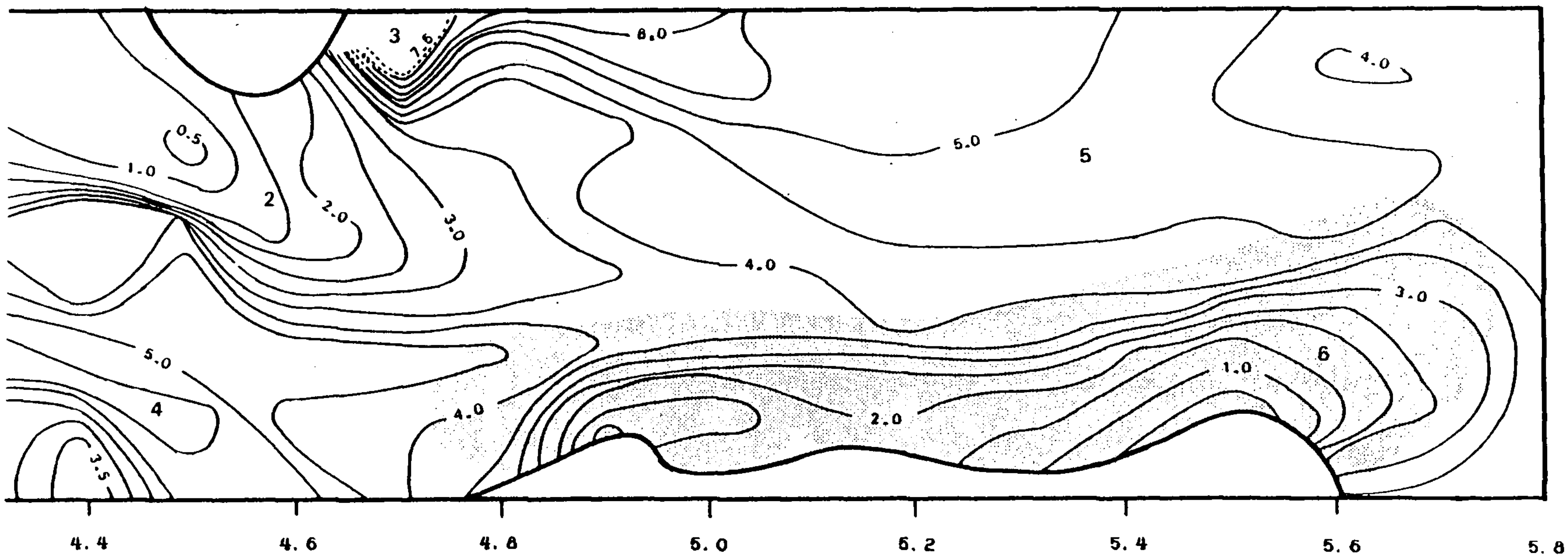
4.3 Laboratory results and their conformation to field

4.3.1 Bed morphology conformation

The results from two runs (i.e. runs 13 and 14) with excavated distal pool will be considered in this paragraph. Runs 13 and 14 lasted about 100 and 200 hours respectively. The flow dynamics and bed material characteristics of the latter run will be further analyzed in the paragraphs below. The two runs had slightly different initial configurations and thus slightly distinct developments. The topographical map of run 13 (figure 29) is introduced in order to show that a final bed configuration such as the one of run 14 (figure 30) can be approximately replicated following the current laboratory procedures. We note from figures 29 and 30 the existence of the same general features on the bed, which are in turn similar to the features found in the field (figure 5). This situation appears to support the assumption regarding the significance of November 23 discharge, the no-feed procedure and the pool

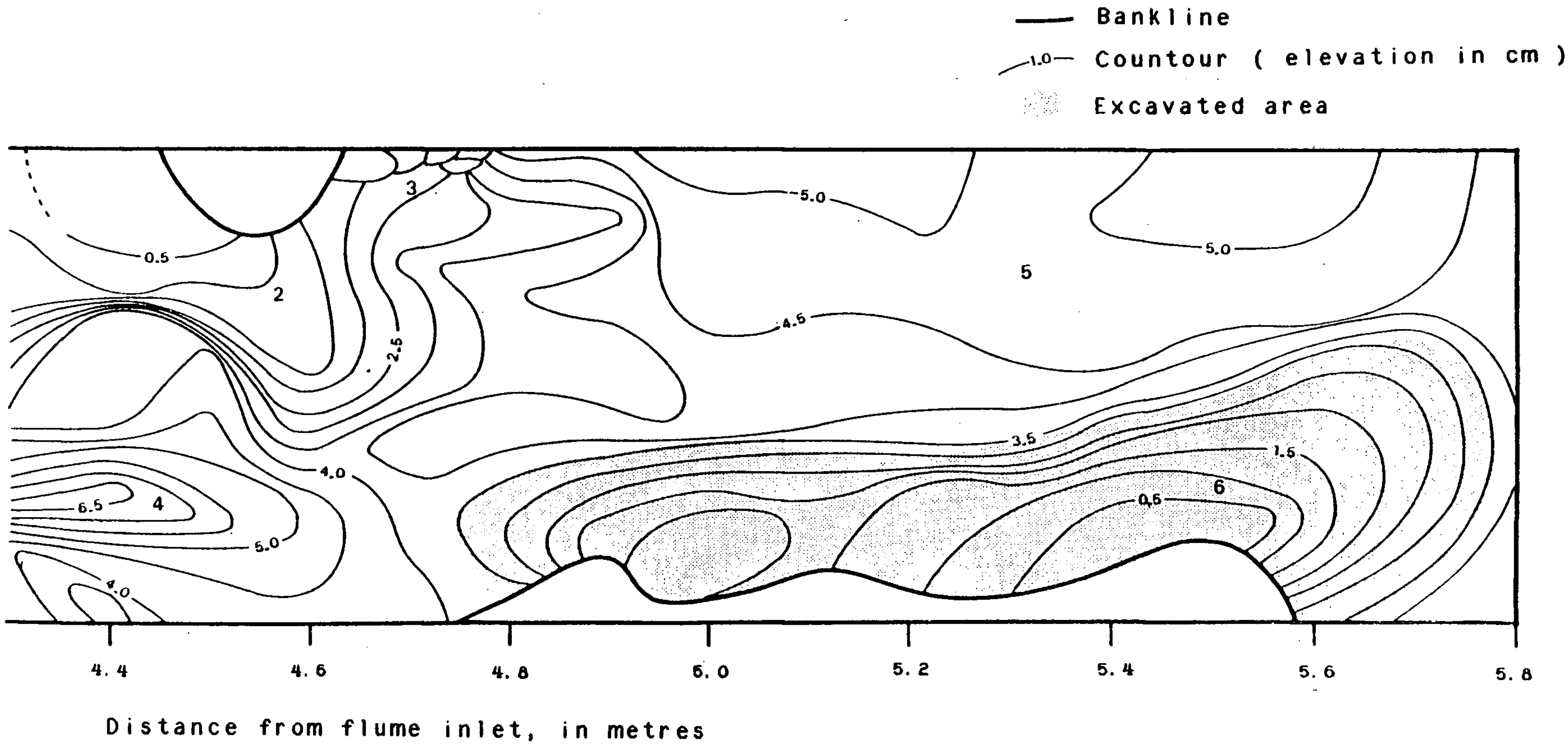
Figure 29.
Topographical map of run 13 final bed configuration

Bankline
1.0 Countour (elevation in cm)
Excavated area



Distance from flume inlet, in metres

Figure 30.
Topographical map of run 14 final bed configuration



excavation.

Boundary conditions were the same for each run, with the exception of the mound (3) area, downstream of the simulated obstruction to the flow. The latter was more prominent in run 13 and this caused the the proximal pool (2) to be more linear and the cross-over to be displaced further downstream. It also caused the excavated distal pool (6) to shrink. Figure 31 shows the initial and final configurations for run 14. Whereas the proximal pool in run 14 is perhaps wider than it should be, the distal pool had been preserved and the cross-over area is at an homologous location with that in the field. The closeness in topography between field and run 14 topography was judged to be sufficient considering the unavoidable degree of simplification introduced by the scaling of the site's various configurational elements. Note however that the topography of the small accumulation bar (4) is not to be considered for strict comparison with the field since it appears to depend on upstream supply of sediment and was simply grown herein by dumping sediment in the area. Also the distal area (i.e. below 5.55 m) of the riffle (5) was created by introducing material by hand. This suggests that the latter feature's growth may depend on upstream sediment supply, even if of low intensity. In summary, below the cross-over area the main difference between the field and laboratory bed topography of run 14 consists of the shape of the proximal pool (2) extension and shoaling area. The discussion below will further consider the similarity of the stream hydrodynamics over the riffle and distal pool which represent the main test of the hydrodynamic

Figure 31.
Initial and final conditions, run 14

scaling done herein.

4.3.2 Flow measurement conformation

Only the near bankfull conditions can be considered because there was insufficient flow depth in the flume to allow velocity profile measurements for a scaled November 20 event. In the first instance, it is essential to verify that the general flow and entrainment conditions for the scaled bankfull discharge reproduced those of the field. The criterion for those conditions to be met was presented in section 2.5.1. The geometric and hydraulic parameters of run 14 and November 23 data are provided for an homologuous cross-section in Table 3. According to these values and those from bed material sampling (see below), the conditions set by equation (2.23) (accepting k_s = 3.5 D_{90}, using the surface values) is met. Therefore flow in the model was in the hydraulically rough regime and resisting forces were accordingly appropriately reproduced. In fact, the latter situation is further supported by the close correspondance of the friction factor (table 3).

Let us concentrate on table 3. The last column presents the ratios with target values, which are always within 7 % of unity. The water surface slope estimate which was obtained from seven longitudinal elevation pairs appears to be exactly the same (in actuality the risk of error in the measurement of this quantity renders this exact correspondance rather coincidental). The model may then be said to be very mildly dysfunctional but some uncertainties mainly related to slope

Table 3

Mean hydraulic and geometric parameters from run 14 and comparison with target values as found from Froude similarity principles

	Run 14 value A	Target value from Froude law (1/16) B	Ratio A/B
Discharge (l s^{-1})	5.21	4.88	1.07
Mean velocity (cm s^{-1})	33.1	32.5	1.02
Mean width (cm)	43.0	43.8	0.98
Mean depth (cm)	3.66	3.43	1.07
Hydraulic radius (cm)	3.13	2.97	1.05
Water surface slope	0.0102	0.0102	1.00
Froude number	0.55	0.56	0.98
Reynolds number	8000	n/a	n/a
Friction factor	0.229	0.225	1.02

measurements do not allow any further development of these considerations. The latter statement is prefered to that of mildly distorted model from Dhamotharan and others (1980) which has a geometric connotation. Possible sources which could account for this mild dysfunction are perhaps the facts that sediments (scaled using a scale ratio of 17) and channel width (the latter due to the introduction of the sinuous bank later in the progress of the study) are slightly smaller than the scale ratio would require. In actuality, the differences must mostly arise from the simple fact that discharge was slightly higher than the target value during the run. Other items like ignorance of strict scaling for bank roughness did not seem to make a significant difference.

We can now consider the issue of the shape of the velocity profiles and their conformation with the field ones. It was possible to measure a much larger number of profiles under the steady flow conditions of the laboratory. Figures 32 and 33 show all the dimensionless velocity profiles in the distal pool and over the riffle respectively. These figures clearly suggest that there is more variability in the pool flow structure than over the riffle. This is somewhat similar to the situation observed in the field.

The large number of profiles sampled in the laboratory allows further subdivisions within the site into upper and lower areas of the riffle and of the pool (figures 34, 35 , 36 and 37). Two general comments emerge from the latter series of figures. First, the greater variability in the shape of the profiles in the pool, as mentioned above, can even be better

Figure 32. Dimensionless velocity profiles

in pool, run 14

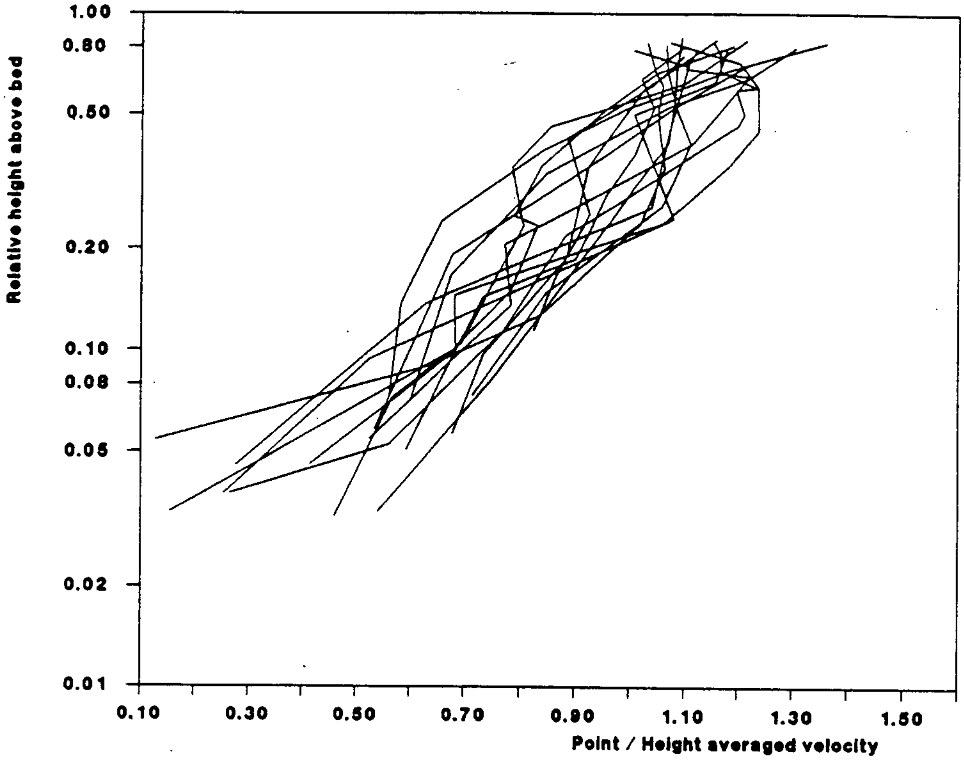


Figure 33. Dimensionless velocity profiles

over riffle, run 14

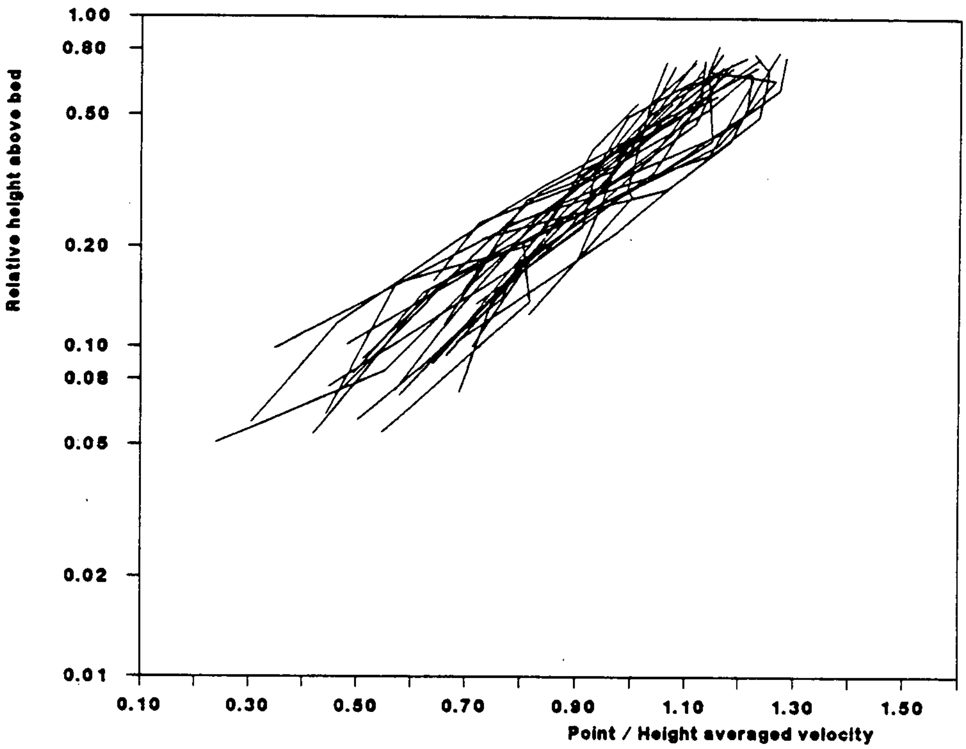


Figure 34. Dimensionless velocity profiles

Downstream part of riffle, run 14

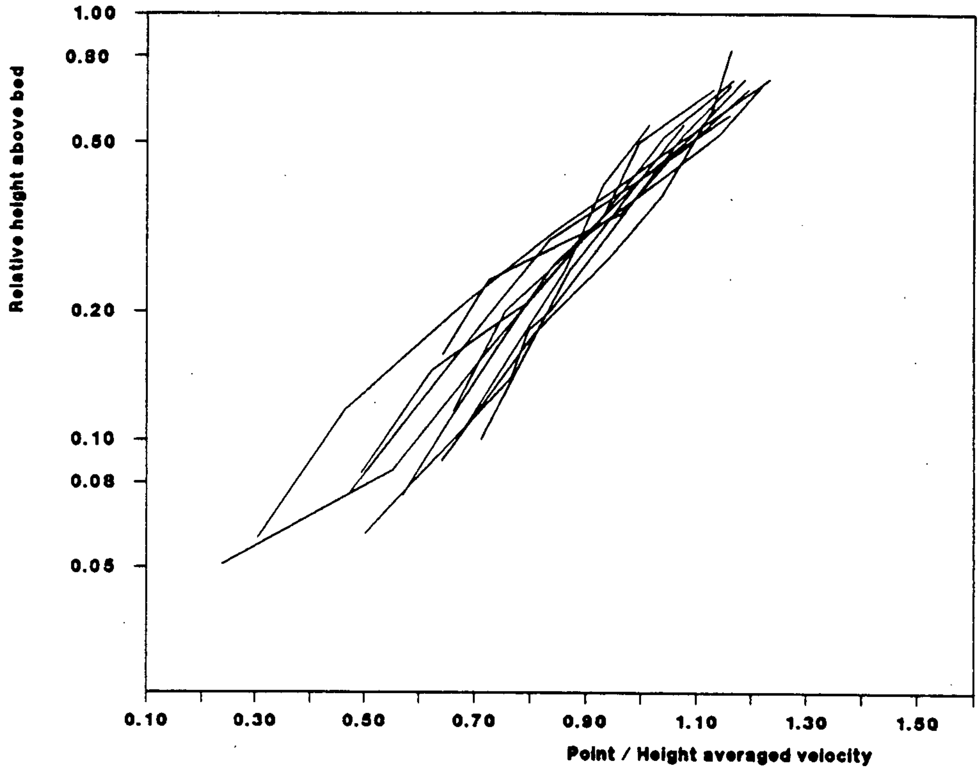


Figure 35. Dimensionless velocity profiles

Upstream part of riffle, run 14

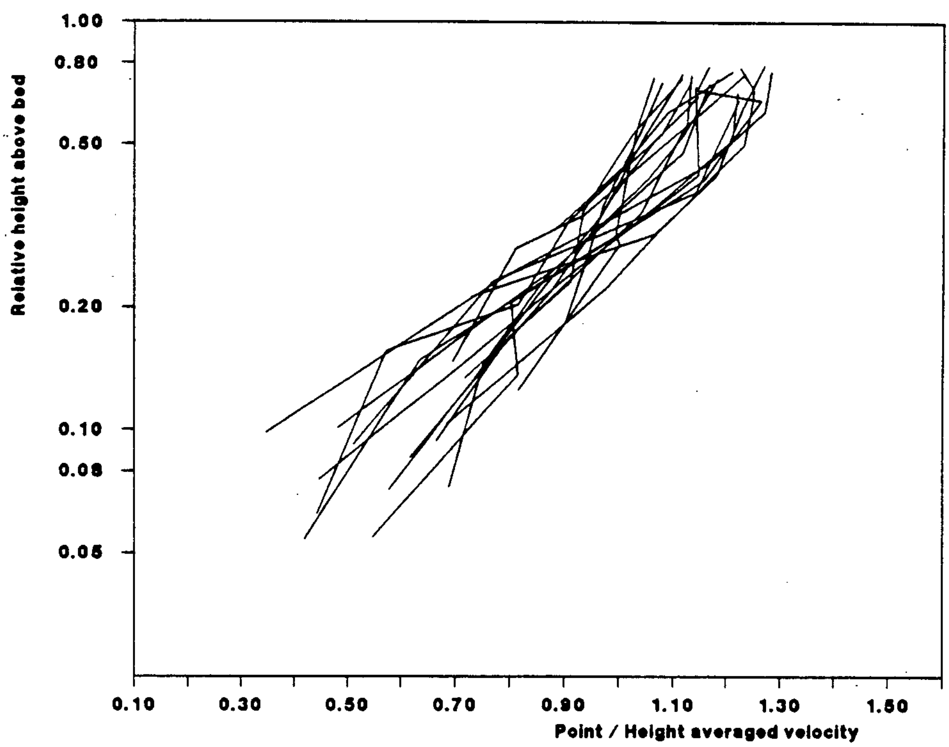


Figure 36. Dimensionless velocity profiles

Downstream part of pool, run 14

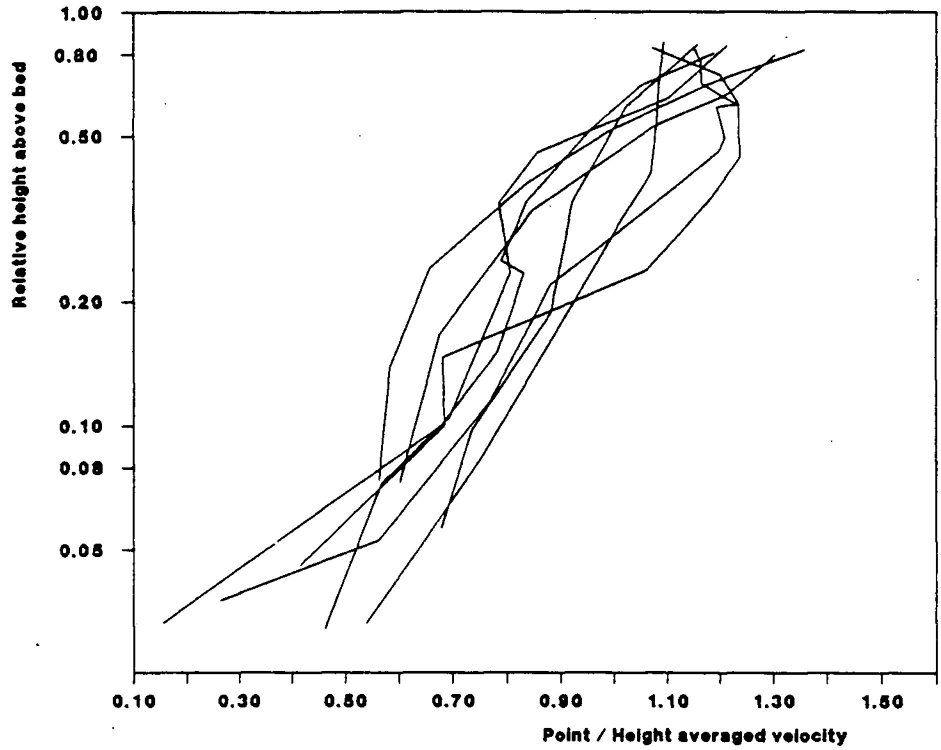
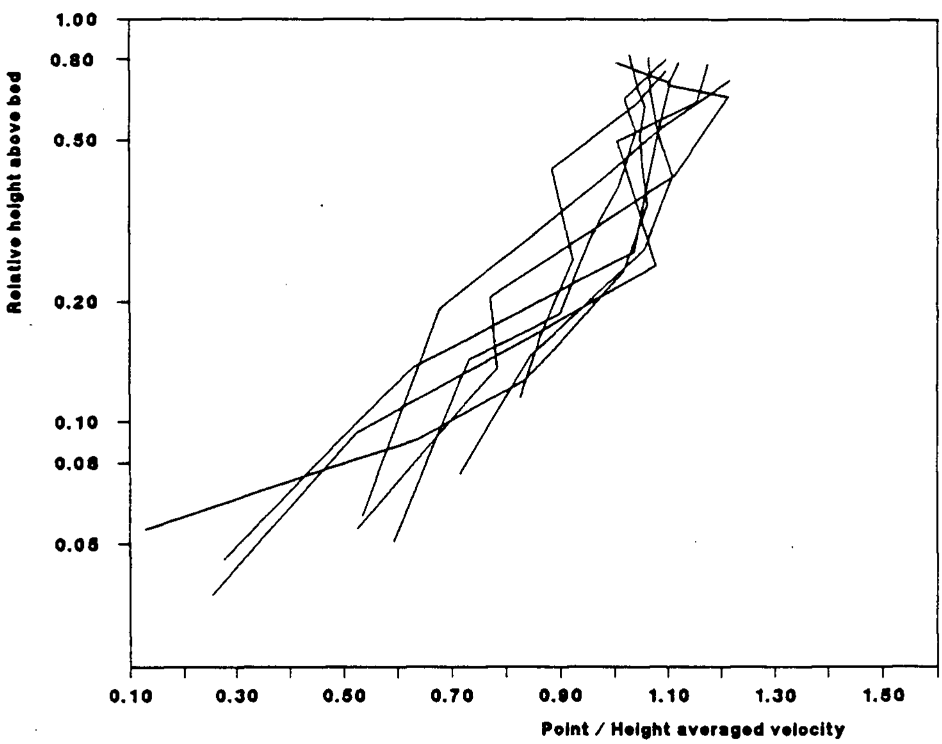


Figure 37. Dimensionless velocity profiles

Upstream part of pool, run 14

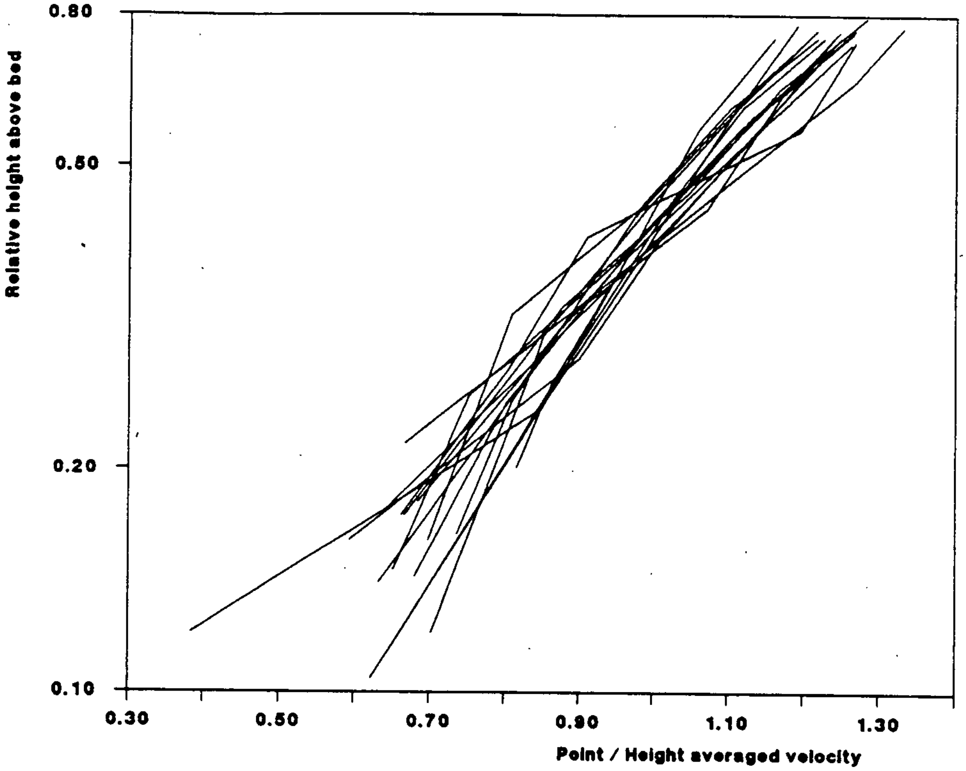


appreciated. Second, as in the field situation the profiles are less kinked over the riffle whereas the shape of the pool profiles is much more capricious. These latter observations may represent a first indication from comparative river hydrodynamics that the distal pool may actually effectively be inherited. In addition, another source of variability valid for the distal pool is the complex flow structure engendered by the banks. Specific observations from figures 34 to 37 are (1) that velocity profiles in the upper part of the riffle are more variable in shape than in the lower part; (2) that the situation is contrary in the pool. Observation (1) should be expected since the flow in the riffle's upper part actually constitutes the extension of the proximal pool which is not uniform in character. Observation (2) is possibly related to the extensive shearing which occurs in the downstream, distal pool area (further discussed below).

Just as in chapter 3, let us now consider the variability of the velocity profile measurements. A test of local variability in velocity profiles was made on a 4 X 4, five mm spaced grid near the riffle crest at 5.16 m along the flume (refer to figure 30). Figure 38 shows dimensionless velocity profiles for the sixteen stations. It shows that variability in the gradient of the velocity profiles occurs for the wake layer mostly. Internal near-bottom shear from those profiles varied from 0.55 to 9.71 Pa, with average of 2.76 and standard deviation of 2.54 Pa. Therefore, on a local scale, the internal shear appears to vary considerably, at least over the riffle. The same test was not performed in the distal pool.

Figure 38. Local shear stress variability test

Dimensionless velocity profiles, run 14



However, it appears reasonable to suggest that the variation in the pool is at least of the same magnitude since pool profiles appear to be more variable. With regard to the local variability of internal shear, the close spacing of the 16 profile measurements appears to suggest the possibility of individual particle effects on wake layer gradient, although this could not be verified. Finally one must consider the fact that the grid test also includes temporal variability effects that may be locally important, as suggested by the field observations. However, temporal velocity profile variability could not be investigated in the laboratory since we did not have a rod-mounted array of hot-films.

Next, consider the internal, near-bed shear distribution through the study area. Figure 39 presents a map of the distribution of near-bed shear estimates. The average shear is 3.121 Pa over the riffle (26 stations) and 2.636 Pa in the pool (18 stations). These values, when multiplied by the scaling ratio, give 49.94 and 42.18 Pa respectively, i.e. nearly the field uniform flow estimate of shear stress (section 3.4). Although we apparently get larger stress over the riffle in the laboratory, the actual data set which covers a much greater bed area than under the restrictive conditions of the field indicates that the absolute difference may be smaller than suggested by the field mesurements. Accordingly, the laboratory results would appear to suggest that field shear estimates at near bankfull conditions are overestimated for the riffle and more accurate for the pool.

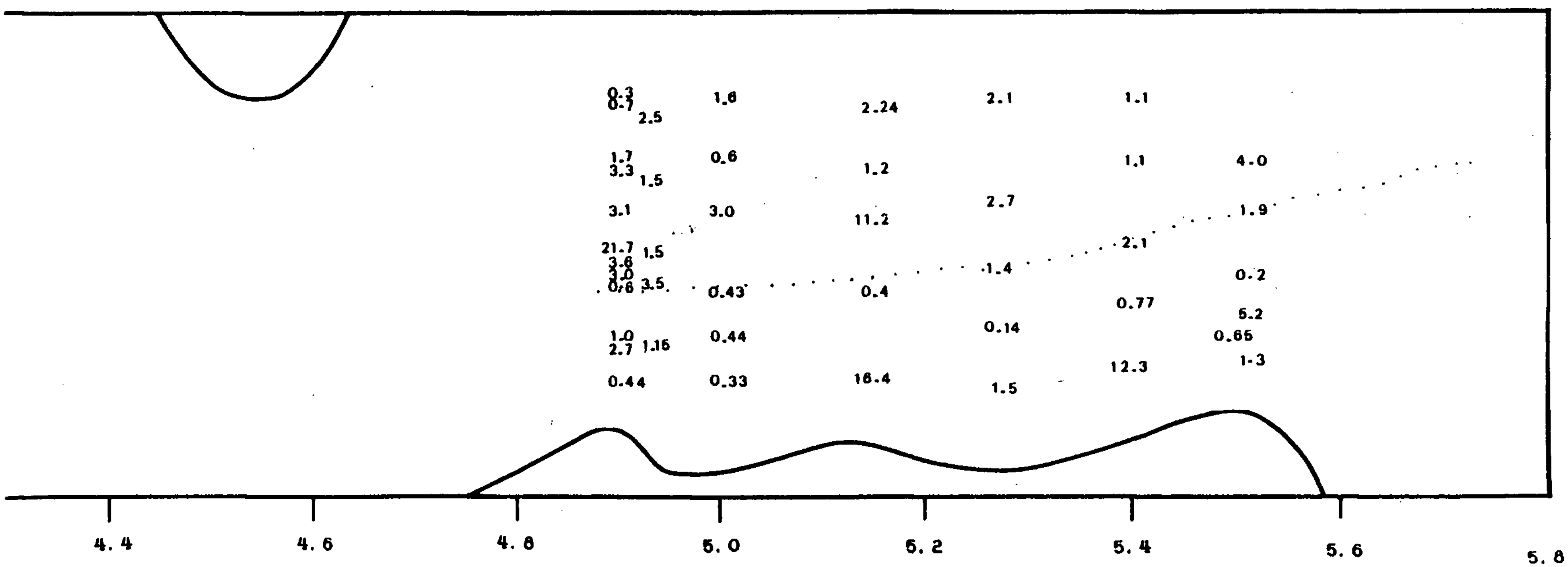
However, the most significant point in the context of this

Figure 39.
Near-bed internal shear estimates at velocity sampling stations, run 14

Banks

Riffle crest

Note : shear units are Pascals.



Distance from flume inlet, in metres

research project is that the magnitude of near-bed force per unit area appears to be reasonably reproduced, supporting the similarity theory as well as the various decisions made about the bed material scaling and the configuration in the current hydrodynamic modelling effort.

Local variability of the shear estimates deserves further attention before any conclusion can be addressed regarding the existence of patterns in shear distribution at the bed unit scale. Discussion is easier from laboratory measurements. Figure 39 offers sufficiently closely spaced shear estimates to confirm that the local variability can be considerable, as we also found from the grid test discussed two paragraphs above. Considering this reality, it may become extremely difficult to recognize any pattern of shear within the limit of bed units, such as advocated by the tenet of a reversal in bottom tractive force (described in the appendix). For instance, run 14 results do not support the field observation that higher shear through the sequence would be concentrated in the upper part of the reach. The only generalizations which may be attempted from figure 39 concern the upper part of the distal pool and the downstream area of the riffle. In the former region, the internal shear appears effectively to be very low and in the latter it seems to be rather uniform compared to other areas. It remains that the local variability overshadows these general statements.

This situation warrants some consideration of the rationale behind and the characteristics of such variability. Perhaps one source of variability comes from the intrinsic characteristics of high relative roughness flows. As suggested above, perhaps individual particles on the bed do have significant effect on the velocity profile on a local basis and thereby reduce the physical significance of a single local measurement. This could not be tested in this project. It is however expected that a spatially averaged shear estimate from an intensive coverage must bear some significance within given bed units. For instance, the bulk flow shear estimate from equation (2.16) for run 14 is 3.13 Pa, which is essentially the riffle area near-bed internal shear average, where the flow is more uniform than in the distal pool. Close correspondance between the local spatially averaged and uniform flow methods for shear estimation over relatively high relative roughness flows was also reported by Einstein and El Samni (1949).

A brief consideration of run 13 shear estimates further characterizes the variation in average or internal shear. Run 13, despite a slightly different configuration, was performed at the same discharge as run 14. Average internal shear calculated from velocity profiles was found to be 3.26 Pa and 1.36 Pa for the riffle and distal pool respectively. The difference comes from the fact that large local shears in the distal pool as found in run 14 were not measured for run 13 although the spatial coverage was comparable. Because of some morphological differences the patterns of shear within the units cannot be compared at a detailed level. Therefore, it is difficult to conclude whether or not the distal pool and the riffle spatially averaged shears are different. Nevertheless, two cases (i.e. the field and run 13 measurements) tend to show that distinctly greater average shear takes place over the riffle.

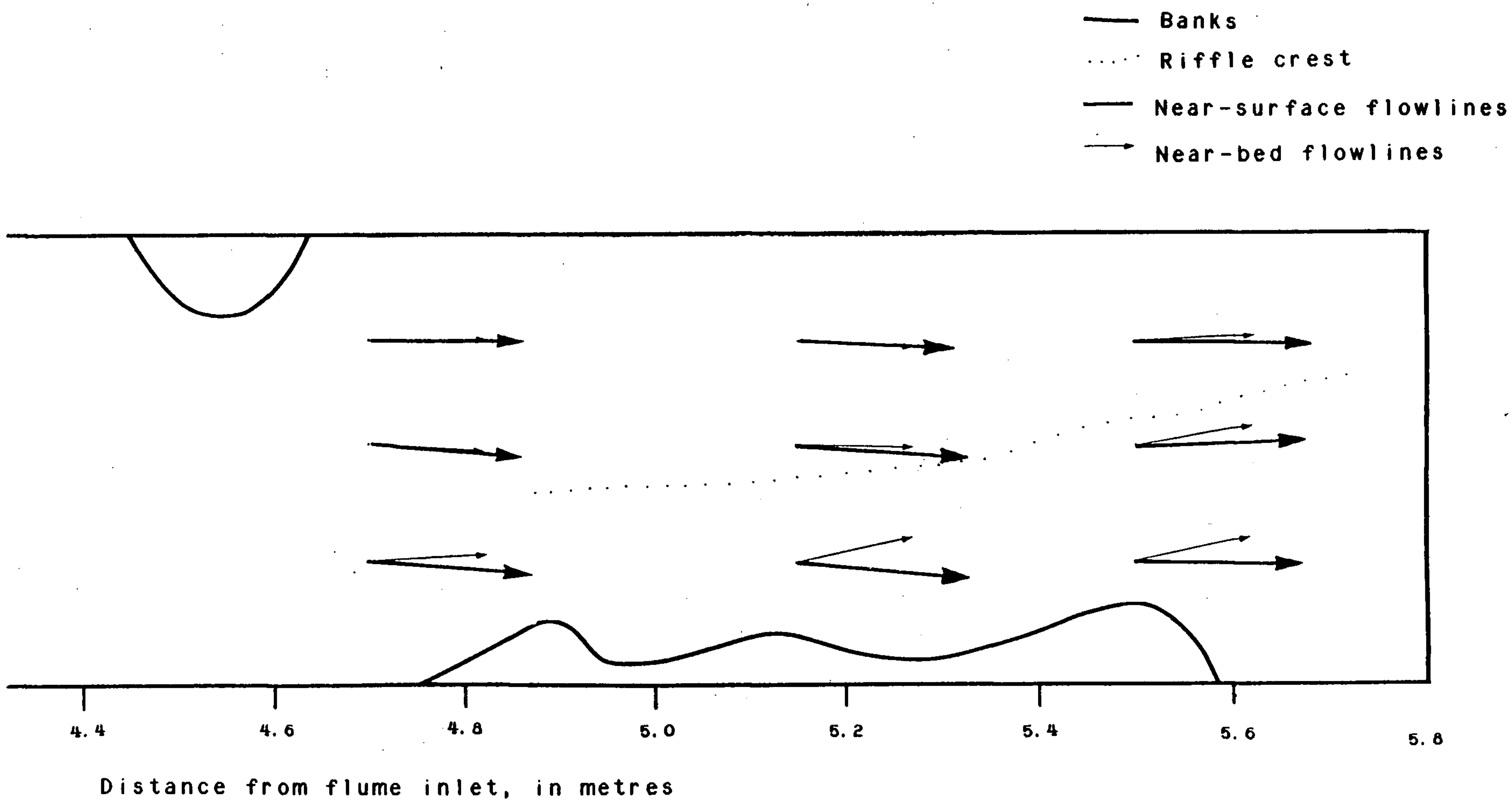
Finally, the flow structure as revealed by the flowlines can be compared. Figure 40 shows the flowlines at channel forming discharge. By comparing it with figure 21, one realizes that patterns are similar. This appears to validate the simplification of the configuration and the pool excavation approach in the sense that the macro flow structure is adequately reproduced. Its reproduction combined with appropriate scaling of the internal shear further confirm that the field and flume situations are hydrodynamically similar. However, a final test of the hydrodynamic similarity is introduced in the next section.

4.3.3 Surface bed material characteristics and conformation

The comparison of the characteristics of the surface materials between the field and flume situation represents a further test of the success of the modelling programme. It specifically addresses the reproduction of entrainment conditions in the model (section 2.5.2) for if there was excess shear force for a certain class (the finest particles) of bed sediment, the characteristics of the surface layer and likely the frequency of bed material transport would not be dynamically equivalent in the field and the model.

Surface coarsening of a widely distributed mixture has been observed in a variety of laboratory experiments by G. Parker and associates (e.g. Parker et al, 1982). Coarsening

Figure 40.
Near-bottom and near-surface flowlines, run 14



appears not to be related to preferential entrainment of some particle sizes but rather acts to regulate the various sediment sizes in transport in proportion to their presence in the subsurface materials, a process which has become known as equilibrium transport. Armour development under steady flow (Parker et al, 1982) was an indication that coarsening constitutes a mobile bed phenomenon. Reproduction of the armour layer characteristics in the scale model would thus indicate that interactions of bed particles during bed formation were properly accounted for. A visual appreciation of surface coarsening which occurred during run 14 can be gained from figure 31.

The surface materials were sampled over the riffle using a 10 X 10 cm, 1 cm space grid which was set at two side by side locations. Hence 200 stones were collected. In the pool, 50 stones were collected at the same spacing but using a ruler disposed randomly. In both cases the stones were collected with tweezers and their b-axes were measured with calipers. The results from the sampling exercise are shown on figure 41 where the 'combined' sample curve actually combines the pool and riffle stones. It is seen that the difference between the pool and riffle samples is small, the riffle being slightly coarser than the pool, but not quite in the same way as in the field.

In general, the D_{50} of the surface in the laboratory are from 3.41 to 4.06 mm. Scaled up, this is equivalent to 54.6 to 65 mm, i.e. slightly coarser than for the field situation (refer to table 2). Figure 42 shows the grain size

Figure 41. Size distributions of surface bed material samples

run 14

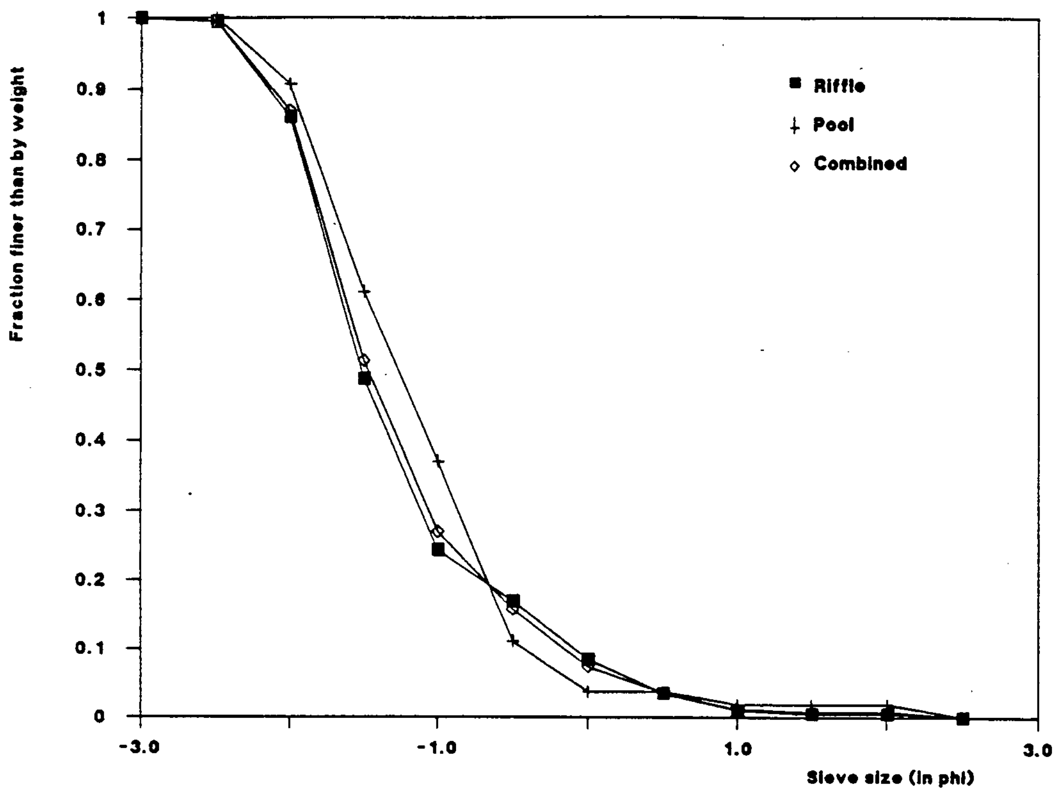
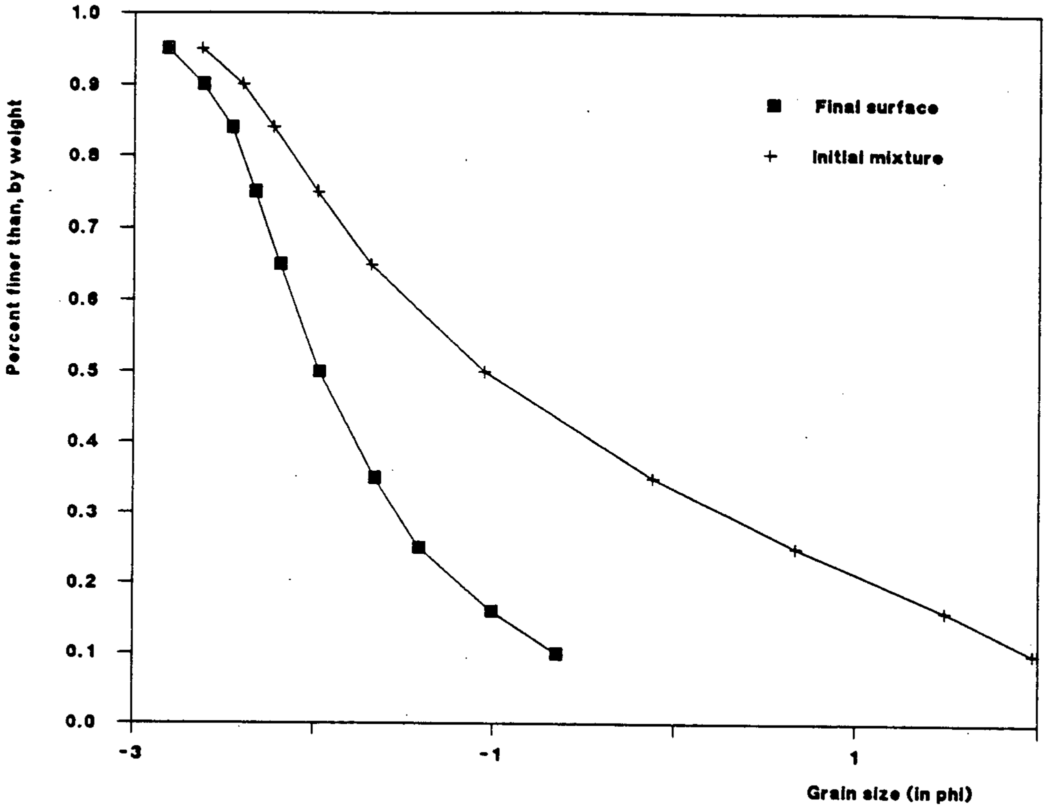


Figure 42. Grain size distribution, run 14

Initial and final surface materials



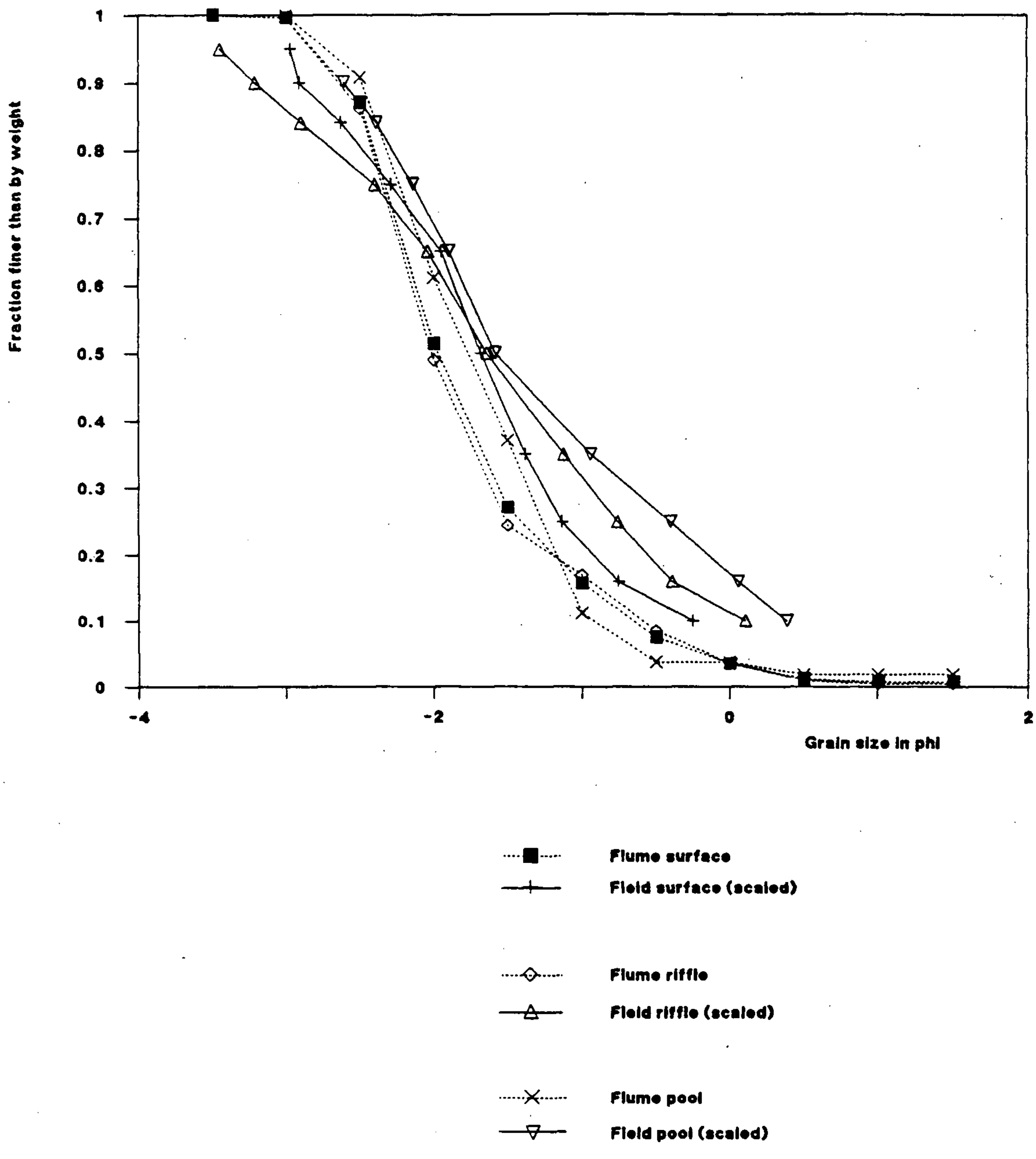
distribution of the initial mixture and that of the surface material at the end of the run. It can be easily seen that surface coarsening involved the selective entrainment of finer particles in this no-feed situation. For the final surface, the D_{10}, D_{50} and D_{90} are respectively 6.15, 1.89 and 1.16 times coarser than for the initial mixture. These ratios compare well with those from field data (table 1). This correspondance suggests that sediment transport is probably quite low in the field and supports the no-feed procedure in the laboratory work.

Figure 43 compares the various samples with their scaled field equivalents. Flume surface and scaled field surface (actually from the downstream sampling site) compare very well over all grain sizes. However, a comparison of pool as well as riffle samples further reveals that the surface materials were coarser in the flume especially below the D_{50}. Coarser size distribution characteristics for the finest part of flume size distribution is perhaps to be expected because of the truncation and correction applied to the flume mixture. The size distributions of the pool samples are seen to be relatively close above the D_{50} whereas riffle sediments seem finer than wanted above D_{75}. It is also worthwhile to note that the run 13 samples (not shown herein) also corroborate the latter observations.

The first thing to consider in order to assess the foregoing discrepancies is sampling limitations. The grids used to sample surface sediments in the field and laboratory were not geometrically similar. More fundamentally samples are

Figure 43. Grain size distribution comparison

Run 14 and field (scaled) surface samples



always approximations to a certain degree. Indeed it is well-known that there exists a practical lower limit to which the surface materials may be sampled (Church et al, 1987). This reality is more critical for the finer flume sediment and may also partly account for the fact that some laboratory samples appear coarser. Another element consists of the fact that the field volumetric sample was truncated and further adjusted for modelling purposes in order to get closer to field resistance or entrainment conditions in the laboratory. The possibility of some finer particles in the laboratory actually experiencing lower dimensionless critical shear stress than the corresponding field ones would encourage the lower part of the size distribution to become coarser. Finally, additional effects may arise from the fact that coarse material in the selected mixture was underrepresented from the start (see figure 26).

Perhaps some operating assumptions made for the the laboratory work have a significant influence as well. The assumption that the bed morphology is mainly determined via a single or restricted range of discharge may not have allowed development of exactly similar surface characteristics. Perhaps the decision to ignore sediment feeding to the system could have been detrimental. However, the arguments presented above regarding the similarity of the size distribution percentiles between the field and laboratory seem to constrain this latter eventuality.

In summary, despite some minor discrepancies, the actual size distribution characteristics of the field and flume can be considered close. It must also be considered that the frictional characteristics of the stream which mostly depend on a bed material length scale were appropriately reproduced in the laboratory. It is therefore suggested that the actual model run reproduced satisfactorily the characteristics of the surface bed material and that dysfunction, if any, was relatively small.

4.4 Extended observations on study site characteristics from field and laboratory

Turbulence characteristics were not specifically measured within the limits of this project. However the minimum and maximum velocity which were recorded at each point measurement in the flume can be looked upon as an index of turbulence characteristics. Figures 44 and 45 present a graph of the deviation from average velocity for the minimum and maximum values recorded within the one-minute period versus the relative height, for the pool and riffle unit respectively. These figures suggest three things: (1) the minimum velocity versus the relative height bear a linear relationship, (2) the maximum velocity versus the relative height bear a logarithmic relationship, and (3) for both the minimum and the maximum velocity, the deviations are greater at the riffle stations. The latter point suggest greater turbulence intensity in general over the riffle area. In this particular study, it appears that those macroflow structures which bring the greatest deviation from the average velocity are more active

Figure 44. Deviations from one-minute average velocity

Maximum and minimum values in pool

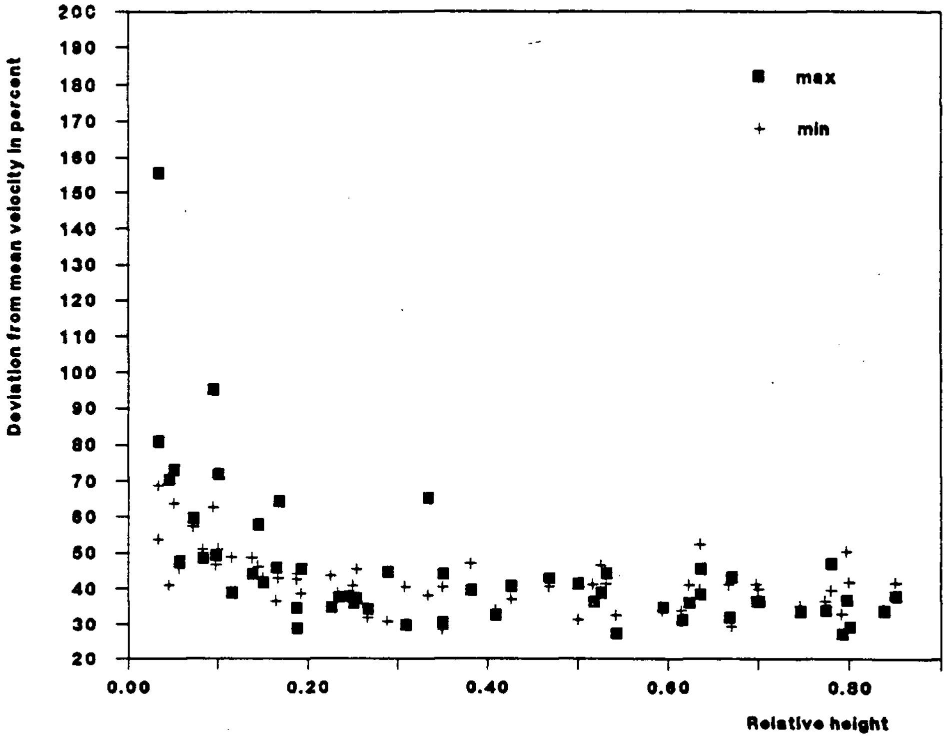
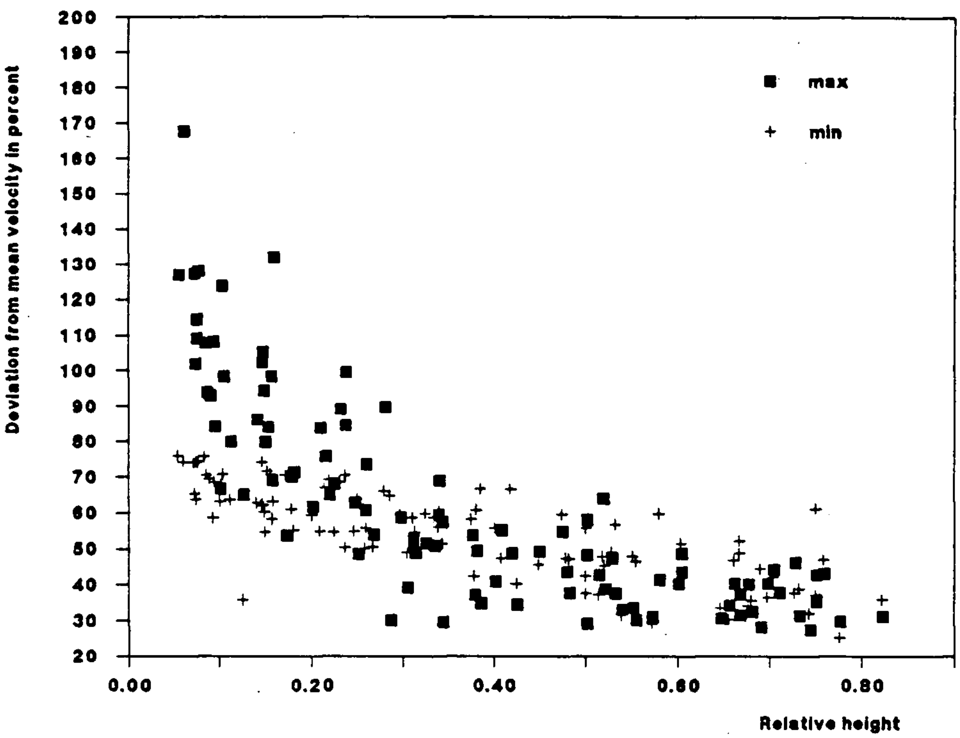


Figure 45. Deviations from one-minute average velocity

Maximum and minimum values over riffle



over the riffle area than over the distal pool. These are exported from the extremely turbulent proximal pool into the riffle area and dissipate before reaching the distal pool. Similar measurements for the case of a fully alluvial pool-riffle sequence could help clarify the question of macro-turbulence through the sequence, as raised by Rood (1980).

It is worthwhile to describe briefly the results from dye injections at the end of run 14 in the distal pool near the right bank. Flow patterns of figure 40 were confirmed. The existence of a riffle slip face vortex could be detected. More interestingly the presence of a shear line through the distal pool, which was noted during the November 23 high flow event (see figure 21), was also detected. Injection of dye near the right bank above the distal pool (near the cross-over) revealed that the local water flowed close to the bank (within a few centimeters) without any mixing with the main flow. Conversely, dye, when injected over the riffle, revealed that the main water body did not flow to the right of the shear line, i.e. along the right bank. These observations hold wherever the dye was injected and at any depth within the distal pool. Thus, apparent longitudinal shearing from the field site could be reproduced and further observed in the laboratory. This represents a further indication of the success of the scaling exercise and of the modelling strategy.

Injection of dye near the bed at the position of the shear line revealed divergence of the current at the bed. Although these phenomenon occurred, it would appear as though they had no morphogenetic significance with respect to the distal pool since the latter was dug and seems to represent a relict feature, as we discussed above. The macro flow structure would then merely represent an adjustment to an imposed morphology from the start. Nevertheless it may help pool preservation by routing some of the entering sediment along the riffle face and not along the right bank.

During upstream pool development, for each single run, the phenomenon of lateral sorting during the scouring of the proximal pool could be observed. The coarsest particles travelled on the side of the obstruction while finest material moved mostly along the face of the developing riffle. Accordingly, only the finest entrained materials entered the distal pool in which the shear was large enough to evacuate them. Hence the preservation of the distal pool would appear truly to be linked to the flow hydrodynamics imposed by the configuration of the creek and to the associated sediment transport routes.

Now, concerning the general sediment transport conditions, it may be asked if bedforms would develop in Blaney Creek study site if there were no obstructions to the flow (i.e. if the channel were straight). The question is especially valid since the excess shear at high flows is rather low, as discussed in chapter 3. To approach this issue, an additional flume run was performed with the same sediment mixture (run 15) after the main body of the research had been accomplished. The flume bed was set at 0.01 (the actual slope value in the study reach at near bankfull discharge). The flow rate was near that of run

14 (see table 4 for complete information about hydraulic and geometric parameters for run 15). The run lasted two days and no bedforms developed. The slope appeared to slightly increase and the average depth was lower than in run 14. The average shear from six velocity profiles and via uniform flow formulation was 2.66 and 2.56 Pa respectively. Although a bed armour developed (not analyzed), no planar segregation of sediment occurred. Therefore, it appears that, without the presence of obstructions to the flow, no undulation would develop on the bed at bankfull flow.

Table 4
Hydraulic and geometric parameters for run 15

Discharge (l s^{-1})	5.10
Mean velocity (cm s^{-1})	41.0
Width (cm)	47.0
Mean depth (cm)	2.64
Hydraulic radius (cm)	2.37
Water surface slope	0.011
Froude number	0.81
Reynolds number	7000
Friction factor	0.13

5.0 Conclusions

The present study has confirmed our hypothesis that undistorted physical modelling based on the principles of similitude represents a reasonable alternative to field research in geomorphology for the case of low transport intensity and small to intermediate size rivers. A specific verification of some detailed hydrodynamic quantities for a small pool-riffle, gravel bed stream (Blaney Creek) revealed that, despite some uncontrollable variability mostly related to the nature of the environment and the limitations of the measurements, field processes can be convincingly reproduced. Hence the claims of Knighton (1984) and Mosley and Zimpfer (1978) as well as the viewpoint taken by Schumm and others (1987), introduced in chapter 1, appear not to be valid for the geomorphological system which was studied herein.

Concurrently, the subject of local variability in the velocity profiles and shear estimates over a river bed of relatively high roughness was shown to deserve some attention in future research. This issue must probably be tackled before any process oriented studies designed to study salient characteristics (notably temporal variability or intermittency) and effects of macroflow structures in pool-riffle streams can be undertaken.

The present research also intended to demonstrate a scaling strategy designated as "generic" modelling for a pool-riffle sequence, guided by a particular field prototype. In a "generic model" the object is to reproduce certain key features of some full size phenomenon, in this case, the pool-riffle couplet. This particular model of Blaney Creek failed to do this spontaneously and this raised the question whether the pool-riffle sequence in the prototype stream is relict. In the present situation, part of the boundary conditions had to be manipulated: this represented a shift from a "generic" to a more "specific" model.

A very important lesson from the latter situation is that historical specifics (the configurational aspects in Simpson's (1963) sense) cannot be considered in a generic model: it appears that some pool-riffle features are of this type. These discoveries in fact represent a fundamental and comprehensive lesson on the nature of geomorphological investigations but do not invalidate the concept of "generic" modelling as a tool for research on the pool-riffle topic. From another perspective, these elements represent a contribution made by the laboratory experiment to the understanding of the historical development of the river reach of interest. It appears that obstruction to the main thread of flow in Blaney Creek controls riffle and pool development and location, as found by Gallinatti(1984) and Lisle(1986). The observations also provided additional elements for the understanding of the mechanisms and response of very low transport intensity streams. In the case of Blaney Creek, it appears that low transport intensity combined with preferential transport routes encourage the preservation of the distal pool.

Above all, the main contribution of this research project lies in the recognition that model studies are legitimate tools for research with respect to full-scale gravel-bed river situations. This emerging fact can be embodied into a concept of generic modelling according to which quantitative observations done in a model situation for which proper consideration of scale effects is made can be generalized and used to interpret river behaviour. Generic models in fluvial geomorphology are restricted to some family of field situations determined basically by economical and practical limitations of laboratory flumes themselves and/or by the nature of prototype bed material. In a "generic" model, a prototype reach may be used to guide the model scales but the object is not to reproduce exact boundary details of the reach. Such models can in fact be said to be part of the same family as the field situation. The importance of these conceptual arguments lies in the fact that measurement programme in many field situations are impractical, indeed impossible, to perform.

An immediate fallout of such an assertion is that rapid progress on gravel-bed river problems potentially could be realized in a relatively short period of time, laboratory conditions allowing the manipulation by the experimentalist of the factors controlling river processes. In particular, the pool-riffle sequence was merely the vehicle for a model-field conformation exercise herein. However, the numerous poorly known aspects of pool-riffle characteristics could be examined in a thorough experimental programme combined with field observations.

This would undoubtly improve our level of understanding and perhaps eventually lead to the establishment of some physically-based statements about the pool-riffle sequence and gravel-bed river behaviour in general.

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Appendix: Bed configurations in low mobility rivers

A.1 General considerations

A ubiquitous bed configuration in low-mobility, gravel-bed rivers consists of more or less regular undulations of the bed, in the order of several times the channel width. Topographically high areas of mostly aggradational nature are termed riffles whereas low portions of the bed which experience mostly scouring conditions at some formative discharge are known as pools. In some channels, storage bars are also present. In actuality, these three features, i.e the bar-pool-riffle, can be considered as a single fundamental unit of channel organization which bears morphological, hydrodynamical and sedimentological (Church and Jones, 1982 ; Ferguson and Werrity, 1983 ; Thompson, 1986) as well as biological (Beschta and Platts, 1986) significance.

Although sedimentological (Leopold et al, 1964) and hydrodynamic (Wolman, 1955) criteria have been invoked, bed elevation records provide the simplest and most natural information to distinguish between pools and riffles. Regression (Richards, 1976a , Milne, 1982a) or the more sophisticated bedform differencing (O'Neill and Abrahams, 1984) techniques have been advocated in order to discriminate between pools and riffles from a record of longitudinal bed elevations. However, these procedures inevitably introduce some simplification. In reality, pools and riffles display a variety of tridimensional morphological expressions related to the hydrodynamic and transport conditions of any given situation.

Bed elevation series are nevertheless useful to study the spacing of the pool-riffle sequence to see if any significant scale constrains their behaviour. Several authors have emphasized the relative consistency of mean pool-to-pool or riffle-to-riffle spacing (Leopold et al, 1964 ; Keller, 1972, ; Harvey, 1975 ; Richards, 1976a, 1976b ; Keller and Melhorn, 1978 ; Milne, 1982a ; Church and Jones, 1982), being on the average 5 to 7 times the channel width. These studies suggest an apparent consistency over a wide range of scales of this isometric scaling of pool-riffle sequences with channel width, apparently valid even for the case of bedrock channels (Keller and Melhorn, 1978).

However, consideration of the scatter of these proposed isometric relationship reveals that some other controls on spacing do exist. Some obvious but local controls are the inhomogeneity of floodplain materials and planform irregularities (Milne, 1982a). The latter have a determining effect on pool location (Gallinatti, 1984 ; Lisle, 1986). Possible effects of the characteristics of boundary materials and stream ability to arrange them via transport into pool-riffle units have not been addressed yet. The latter issue would appear to be particularly susceptible to be studied in controlled and properly scaled laboratory channels.

In some situations, perhaps spacing variability can be accounted for by the coexistence of different large-scale bedforms arising from watershed controls (Church and Jones, 1982) or by the lag before bedform addition as a stream increases its sinuosity in the process of meander development (Keller and Melhorn, 1973 ; Milne, 1982a). For such creation of additional bedforms to occur along a river length, Thompson (1986) proposed an inter-riffle threshold distance of 10 times the channel width.

A more basic issue consists of the origin of these bed undulations in gravel-bed rivers. This represents a rather fundamental problem of fluvial morphology which has often been linked with the meander initiation enigma. This rapprochement is based on the fact that in many albeit not all field situations, bar or pool locations alternate from one bank to the other, imitating the meandering pattern, even in straight channels. Laboratory studies in straight channels have also sometimes resulted in the development of alternating bars (Chang et al, 1971 ; Sukegawa, 1974 ; H.Ikeda, 1973, 1983 ; Jaeggi, 1980,1983,1984 ; S.Ikeda, 1984): these would appear to represent the first stage of meander development. Moreover, alternate bars do bear some analogies with field pool-riffle sequences on morphological and hydrodynamical grounds.

However, it is yet unclear what is the actual link between the pool-riffle sequence and the meandering phenomenon in alluvial river channels. Some authors who consider the pool-riffle sequence to be prerequisite for meander development have accordingly proposed conceptual models of alluvial river development (e.g. Keller, 1972 ; Lewin, 1976 ; Thompson, 1986). The rapprochement and causal link between these two phenomena are apparently further justified by the relative consistency in the proportion between half meander wavelength and interriffle distance over a wide range of scales. However, the fact that meandering channels with fine-grained bed material do not show a pool-riffle sequence proper indicates that both phenomena may coincide only under particular alluvial circumstances. In addition, the meandering phenomenon occurs under varied hydrodynamic conditions, i.e. microstreams over hydrophobic plates (Goricky, 1973), in glacier ice, in ocean currents and in the atmosphere (Leopold et al, 1964 ; Zeller, 1967). Meandering thus appears to represent a flow-induced phenomenon for which development may be triggered by a distinct perturbing factor for each of these situations (Parker, 1976).

A.2 Bedform classification and conditions of pool-riffle occurrence

Bedforms in general and the pool-riffle sequence in particular are generated by shearing flow over a granular bed and display a wide variety of shape and size related to the characteristics of the environment in which they grow. Attempts to unify in a unique conceptual model bed configurations of various scales are rare in the literature. Indeed the dominant flow-regime classifications of bedforms (e.g. Simons and Richardson, 1961 ; Southard, 1971) do not cover the wide range of conditions met in natural systems, being mostly based on sand flume experiments. Jackson (1975) proposed a unifying model of bedforms based on conceptual arguments pionered in Russian literature. This model has the merit of apparently being based on fluid-dynamic processes (not established or well-formulated in some cases) and spatio-temporal parameters. Jackson recognized three groups of bed configurations, namely microforms, mesoforms and macroforms, the latter which are the realm of pool-riffle sequences and channel bars. Leeder (1983) also offered an apparently genetic bedform classification based on bedform mechanics which integrated large-scale features.

Evidences of juxtaposed domains for meso and macroforms are provided by H.Ikeda's (1973,1975) experimental and field results. The limits of occurrence of each type of bed configuration can presumably be described via some hydrodynamic parameters. Some authors have tried to describe requirements for various bar types to develop in terms of the stream energy gradient (Kopaliani and Romashin, 1970 ; Church and Jones, 1982; Florsheim, 1985). However, those descriptions do not take into account the characteristics of the bed material to be entrained under a given energy gradient in a relative manner and are unlikely to be generalizable. Preliminary classification attempts by the writer have tend to suggest that in fully rough subcritical flows the succession of bedforms from bar forms to dunes is governed by the mobility number (described in section 2.5.2). Pool-riffle sequences as well as alternate bars (of sand-bed laboratory experiments) occur when the mobility number is low (i.e. generally below 3). Dune beds appear at higher relative stresses for a given grain size, and their shape also appears to be a function of the mobility parameter (Yalin and Karahan, 1979b).

The existence of a wide variety in the morphological expressions of bedforms in mountain rivers has actually been long recognized in the East European and Russian literature (Gergov, 1983). Increasing mobility number is expected to have an effect on pool-riffle morphology although this has not been documented. For instance, an increase in mobility number can be related to an increase in sediment transport or in the amount of material in transit, and hence to a greater importance of the bar component in the fundamental pool-riffle unit.

The mobility number magnitude has also some influence on the migration of the sequence. Alternate bars in laboratory experiments and pool-riffle sequences of low sinousity rivers are sometimes migrating (H.Ikeda, 1975 ; Ferguson and Werrity, 1983 ; Jaeggi, 1985). The speed of migration, which is very variable but relatively small to nil in most gravel-bed rivers, is undoubtly related to the low excess stress available to entrain the bed particles. Factors which promote riffle or bar stability either originate from planform control (Kinoshita and Miwa, 1974 ; Lisle, 1986) or from sedimentological and hydrodynamical factors (Church and Jones, 1982). As we shall further discover below, bar mobility is likely related to bar shape.

In addition, it is interesting to note that some criteria for alternate bar occurrence (Sukegawa, 1974 ; Jaeggi, 1984) also included the mobility number. However, some confusion between bar types (Shen, 1961 ; Jaeggi, 1983, 1984) and possible scaling or regime difficulties (the former related to the flow conditions in some laboratory channels and the latter to the fixed width of the channel, ref. Chang, 1985) in some experiments impede the use of these criteria for pool-riffle sequence occurrence. The same reasons cause the results on laboratory bar geometry (height and spacing) of S.Ikeda (1984) to be received sceptically.

The lower limit for pool-riffle occurrence would correspond to threshold conditions. However, in channels in which planform irregularities or streamside obstruction intersect the local flow, creating large scale vortices, pool-riffle sequences can possibly develop even if the flow average mobility number is smaller than 1.0 (i.e. the so-called forced bars of Kinoshita and Miwa, 1974). In the case of planform controlled pool and riffle sequences, whether or not the flow average mobility number is below or above 1.0, pool and riffle location and geometry are mostly determined by the intersection to the flow in plan (Gallinatti, 1984 ; Lisle, 1986).

Classifications of bar-pool-riffle sequences in fully alluvial milieus are primarily based on bar form. There is a confusing array of terms concerning bar classification (Smith, 1978). The latter situation is likely a consequence of their morphological diversity and the lack of a global approach to the large-scale bedform aspect in full-scale rivers. Riffle planform may vary from transverse, sometimes with medial bar (e.g. Richards, 1976b), to diagonal or alternate bar shape (e.g. Ferguson and Werrity, 1983). (Note that the transverse step-pool systems of mountain streams (Whittaker and Jaeggi, 1982) occur under different hydrodynamic conditions and represent another domain of bed formation.) Church and Jones (1982) described a gradation in riffle and bar morphology which is possibly related to the mobility number; at greater mobility, the bar component fills the channel and the riffle represents merely its leading edge. This proposal remains to be verified.

Church and Jones (1982) also attempted a classification for alluvial gravel-bed channels macroforms. In addition to morphological criteria, this classification distinguishes between the primarily resistance function of riffles and the sediment storage role of bars. In the discussion of the latter paper, P. Ashmore proposes a classification scheme similar to theirs which also includes bar complexes and remnants, a refinement particularly relevant for the case of multithread rivers channels. These attempts however remain descriptive and hence not based on hydrodynamic quantities.

In partly alluvial milieus, macroform classification can either be based on pool type for obstruction-controlled pool-riffle sequences (e.g. Beschta and Platts, 1986) or bar type in the case of bedrock canyons (e.g. Baker, 1984).

The emergence of a complete classification for large-scale bedforms is highly desirable in order to provide a universal frame of reference which will eliminate the confusions due to unregulated nomenclature in the literature.

A.3 Flow processes, origin and evolution of the pool-riffle phenomenon

Studies of flow processes in pool-riffle units at formative flows are rare. Velocity measurements performed through a selected sequence were reported by Keller (1971). Therein only near-bed velocities for low to medium discharges were measured along two cross-sections, respectively in a pool and over a riffle. For low discharges, Keller found the bottom velocities (which he assumed proportional to the tractive force acting on the bed) to be larger over the riffle than in the pool. However, Keller noted a tendency for the bottom velocities to equalize in the pool and the riffle at medium discharge and hypothesized that a reversal (i.e. the bottom velocity in the pool exceeding that over the riffle) may occur for the highest, formative discharges. A reversal in terms of tractive force was never systematically measured while his set of original bottom velocity data (Keller, 1969), taken at 1.5 cm from the bed (which has a D_{50} of about 32 mm over the riffle), showed rather large local variability and possibly individual particle effects.

Nevertheless some authors have argued that the apparent sorting of surface bed material, riffle material being in general coarser than that of adjacent pool, could result from a reversal in flow competence between pools and riffles (Lisle, 1979 ; Hirsch and Abrahams, 1981). Andrews (1979) reported a reversal of the mean velocities for the East Fork river, a pool-riffle stream. Lisle (1979) actually reported for the same river a reversal of average shear stress (equation 2.16) at high flows. A recent study by Petit (1987) in a small pebble-bed stream seemed not to support the hypothesis of bottom velocity reversal. Petit also estimated the tractive force on the bed and concluded that a reversal of the latter quantity occurred. His approach however suffers from weaknesses regarding the way the stress was actually estimated. Only local velocity profile measurements which would allow bed shear stresses to be properly estimated through the sequence could clarify the velocity or shear stress reversal issue.

The occurrence or sedimentological importance of the velocity or shear stress reversal has also been refuted by some authors (Teleki, 1972 ; Bhowmik and Demissie, 1982). Milne (1982b) pointed out that lateral sorting between the pool and the adjacent bar in a relatively sinuous channel was generally more significant than longitudinal sorting from pool to riffle. The most appealing aspect of the reversal hypothesis remains that it can account for the maintenance albeit not the initiation of the pool-riffle sequence (Lisle, 1979 ; Keller, 1971, 1983).

The initiation of the pool-riffle sequence is a difficult problem to study due to natural feedback between bed morphology and flow structure at formative discharge. Several attempts at explanation have nevertheless been offered but no systematic physical observations have been collected. Two alternate views are possible; pool-riffle sequences are viewed as either a flow or sediment transport phenomenon.

On the one hand, Langbein and Leopold (1968) proposed that pool-riffle sequence initiation was determined by some property of the moving bedload materials which would travel as kinematic waves. Thompson (1986) argued that pool-riffle spacing may effectively be under control of such a mechanism. Lekach and Schick (1983) observed that bedload transport over a non-undulating confined river bed had a pulse-like character. The fact that a pool-riffle sequence developed some distance downstream from the canyon reach was taken as an indication that kinematic wave transport (which could represent an inherent sediment transfer phenomenon) could be responsible for pool-riffle initiation. Wavelike transport which is characteristic of gravel-bed rivers has been noted in other pool-riffle sequences (e.g. Campbell and Sidle, 1985 ; Milligan et al, 1980). In this case, the creation of organized bedforms would be prerequisite to the organization of flow patterns.

On the other hand, several ideas about how patterns of velocity or stress could establish in the flow have also been proposed. Some authors also appealed to minimization principles (Yang, 1971 ; Cherkauer, 1973) but their arguments were not valid under channel-forming conditions. In contrast, other authors attempted to account for the origin of distinct but periodic flow structure in pools and riffles, as described by Keller and Melhorn (1973), Ferguson and Werrity (1983) and Thompson (1986). Richards (1976a) adapted Yalin's (1971b) probabilistic arguments to the apparent periodic occurrence of zones of relatively higher and lower stresses in the longitudinal direction. Nearly a decade earlier, Einstein and Shen (1964) proposed that secondary circulation of alternating nature could originate from the growth and decay of wall turbulence produced by a vorticity generating mechanism. A more sophisticated and distinct treatment by Parker (1976), using instability analysis, suggested that sediment transport and friction but not helicity were necessary for the development of alternating zones of scour and deposition. However, none of the foregoing ideas have been well-tested.

For the case of streamside obstruction-controlled pool-riffle sequence development, a qualitative albeit theoretical treatment is proposed in Gallinatti (1984). Near an obstruction the impinging flow generates high pressure whereas the pressure is lower near the opposite bank. This pressure gradient, combined with an adverse pressure gradient through the water depth at the face of the obstruction, results in the formation of an helical flow structure directed downward at the obstruction and upward near the inner bank. The shape and extent of this flow structure will depend on the particular configuration of the reach. Scour appears along the obstruction. The circulation persists for some distance after the obstruction during which the advection of the high velocity core and turbulent diffusion gradually promote the creation of a lobate deposition area which resembles an alternate bar.

A.4 Pool-riffle sequences : perspectives and prospects

The mechanisms proposed for alluvial pool-riffle or incipient meandering development or maintenance do not produce universal agreement. The physical processes involved in the development of the pool-riffle sequence are indeed extraordinarily difficult to measure, spatially distributed quantities being involved in this two-phase phenomenon. From this situation follow instrumental and data resolution constraints.

In this perspective, it is also interesting to note that many simpler aspects of pool-riffle sequence behaviour introduced in this chapter are not yet well understood, like the controls on their varying morphology, the criteria for their occurrence and their hydrodynamics at equilibrium. Some of the above remarks, notably those about the problems of classification, illustrate that it is necessary to begin to study the pool-riffle phenomenon by addressing some fundamental questions, hence perhaps to ignore most of the work done until now. A reflexion about the actual level of knowledge on the pool-riffle problem and the need for extensive and detailed information lead to consideration of the possibility of working under controlled laboratory conditions. Such working conditions would allow widely varying combinations of some influential hydrodynamic and sedimentological parameters as well as detailed hydrodynamic measurements in order to explore several crucial issues.

The writer believes that rapid scientific progress could be made under such working conditions. If it is demonstrated a priori that the physical processes acting in full-scale systems can be appropriately reproduced at a smaller scale, according to the theory introduced in chapter 2, laboratory results can be transferred to the field without scepticism. The latter statement describes the motivation and rationale behind the current research design. In that sense, the current project represents a first, necessary step on the way to truly fundamental research regarding the bed morphology in rivers.