

**ENVIRONMENTALLY SIGNIFICANT MORPHOLOGICAL AND
HYDRAULIC CHARACTERISTICS OF COBBLE AND
BOULDER BED RIVERS IN THE WESTERN CAPE**

VERNO JONKER

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Promoter: Professor A Rooseboom

Co-promoter: Professor AHM Görgens

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DECLARATION

I, the undersigned, hereby declare that the work contained in this dissertation is my own original work and that I have not previously in its entirety or in part submitted it at any university for a degree.

ABSTRACT

The interaction between moving water and the physical attributes of a river, as displayed by the channel morphology, determines the availability of physical habitat for aquatic species and thus also the condition of the ecosystem. As such, the environmental flow assessment process requires knowledge on how changes in the flow regime will affect both the morphological and hydraulic conditions within a river channel. With the increasing development of water-related infrastructure in mountain regions, knowledge of the morphological and hydraulic characteristics of rivers in the upper catchment areas is very important. Cobble and boulder bed rivers in the Western Cape are typical examples. They are characterized by steep gradients, great variability in sediment size and relatively low flow depths. The bed configuration contains a series of pools, steps, rapids, riffles and plane beds, while energy losses are high as a result of turbulence and hydraulic jumps. Due to their wide-ranging morphological and associated hydraulic attributes, the physical habitats within these rivers are extremely diverse, both on a spatial and temporal scale. This study addresses the interaction between moving water and the physical attributes of cobble and boulder bed rivers. Empirical, semi-empirical and theoretically based models are developed which define the hydraulic and morphological related characteristics of environmental flow components in cobble and boulder bed rivers. They cover macro scale channel deformation, the scouring of sand from the interstitial spaces between the cobbles as well as velocity-depth relationships, which prove to be key components in the assessment of environmental flow requirements in cobble and boulder bed rivers.

SAMEVATTING

Die wisselwerking tussen die vloeiende water en die fisiese eienskappe van 'n rivier, soos beskryf deur die morfologie, bepaal die beskikbaarheid van habitat vir akwatiese spesies en gevolglik ook die welstand van die ekosisteem. Kennis omtrent die impak van 'n veranderde vloeiregime op beide die morfologiese en die hidrouliese toestande in 'n rivierloop is dus nodig vir die bepaling van omgewingsvloeibehoeftes. Met die toenemende ontwikkeling van water-gerelateerde infrastruktuur in bergagtige gebiede is 'n grondige kennis van die morfologiese en hidrouliese eienskappe van riviere in hierdie bo-opvanggebiede gebiedend noodsaaklik. Spoelklipriviere in die Wes-Kaap is tipiese voorbeelde van sulke riviere. Hierdie riviere word gekenmerk deur steil hellings, 'n wye verskeidenheid sedimentgroottes, relatiewe lae vloeiëptes en hoë energieverliese as gevolg van turbulensie. Verder bevat die rivierbed afwisselend poele en stroomversnellings en gevolglik word 'n wye verskeidenheid habitat tipes in hierdie riviere aangetref. Hierdie studie fokus op die interaksie tussen bewegende water en die fisiese eienskappe van spoelklipriviere. Met behulp van empiriese, semi-empiriese en teoretiese modelle word die morfologiese en hidrouliese eienskappe van omgewingsvloeibehoeftes in spoelklipriviere aangespreek. Dit sluit makroskaal kanaalvervorming, die uitskuur van sand tussen die spoelklippe asook die verwantskap tussen vloeiëpte en vloeiëpte in. Hierdie aspekte kan beskou word as van die sleutel elemente vir die bepaling van omgewingsvloeibehoeftes in spoelklipriviere.

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Rivers have what man most respects and longs for in his own life and thought – a capacity for renewal and replenishment, continual energy, creativity, cleansing.

John M Kauffman, Flow East

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LIST OF SYMBOLS

a,b	: constants
A	: cross-sectional flow area
A_p	: wetted frontal area of a bed particle
A_w	: wetted roughness cross-sectional area
b_e	: function of effective roughness concentration
c	: resistance coefficient
C	: Chézy resistance coefficient
C_d	: drag coefficient
C_k	: kinetic energy loss coefficient
C_l	: transitional energy loss coefficient
C_m	: mobility coefficient
C_x	: large scale roughness resistance coefficient
d	: particle diameter
d_x	: particle size for which x % is smaller
D	: flow depth
\bar{D}	: mean flow depth over cross section
D_{cb}	: channel depth at bankfull level
f	: Darcy-Weisbach resistance coefficient
Fr	: Froude number
g	: gravitational acceleration
h_f	: friction loss
h_l	: transitional loss
j	: shape parameter
k	: absolute bed roughness
l	: reach length for ideal case being considered
L	: length of macro scale bedform
m	: mass of unit volume of water
n	: total number of bed particles in ideal reach with length l
n	: Manning resistance coefficient
\bar{n}	: number of bed particles along a streamline in ideal reach with length l
n_w	: number of vertical elements in cross section
P	: wetted perimeter
Q	: discharge
Q_{bf}	: discharge at bankfull level
Q_c	: characteristic discharge
Q_m	: measured discharge
Q_s	: sediment discharge

Q_x	: discharge with return period equal to x years
R	: hydraulic radius
R_d	: radius of turbulent eddy
R_0	: radius of a turbulent eddy next to the bed
s	: energy gradient \approx channel gradient (uniform flow conditions)
S_f	: energy gradient
S_0	: average channel gradient
t	: time for water to move distance equal to one cobble diameter
v	: local flow velocity
V	: average velocity over cross section
V^*	: shear velocity
V_{cr}^*	: critical shear velocity
V_{ss}	: settling velocity
v_{max}	: maximum flow velocity within a cross section
W_{ch}	: channel width at bankfull level
W_T	: flow width
x_a, x_b	: constants
y	: flow depth
y_c	: critical flow depth
y_i	: initial flow depth
y_n	: uniform flow depth
y_s	: sequent flow depth
y_0	: ordinate where the velocity is mathematically equal to zero
$\overline{y_0}$: mean ordinate value over the cross section where the velocity is mathematically equal to zero
z	: bed elevation above datum
α_r	: local riffle/rapid gradient
α_{WB}	: scale parameter (Weibull distribution)
α_{EXT}	: scale parameter (Extreme Type I distribution)
β_{WB}	: shape parameter (Weibull distribution)
β_{EXT}	: location parameter (Extreme Type I distribution)
ϕ	: angle of repose
μ	: statistical mean
ρ	: density of water
ρ_s	: particle density
σ	: standard deviation
τ	: shear stress
ν	: kinematic viscosity of water
Δz	: pool depth
Δ	: non-dimensional parameter reflecting increase in bed roughness at equilibrium

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1. INTRODUCTION

1.1 Background

The development and management of water resources have led to the alteration of the natural flow regimes of many rivers around the world and this has led to growing concern regarding the deterioration of river environments. A new scientific discipline, known as Environmental (or Instream) Flow Assessment, has been developed to predict the environmental impacts associated with water resource developments and to provide information on the amount and frequency of managed flows which are required to maintain a river in a pre-determined, environmentally acceptable condition. This then serves as a guideline for managing the river in order to provide for the ecological requirements of the riverine ecosystem.

A variety of factors control the abundance, distribution and productivity of aquatic organisms in rivers. These include competition for space, predation, chemical water quality, nutrient supplies, flow patterns, and flow variability and together they describe the biological, chemical and physical habitat (Gordon *et al.*, 1992). Chemical water quality has traditionally been viewed as the most important factor affecting the degradation of aquatic ecosystems (Hugues *et al.*, 1990). However, the physical habitat and its modifications have recently been identified as key elements in stream ecosystem functioning (Lamouroux *et al.*, 1995). The physical habitat refers to those factors which form the “structure” within which an organism makes its home. Physical factors are generally more predictable, less variable and more easily measured than biological or chemical ones, and are thus preferable for general, consistent descriptions of streams (Gordon *et al.*, 1992; Richards, 1976). Physical factors which are ecologically important include temperature, channel shape, morphological structure, substrate characteristics, flow velocity, flow depth, lift and drag forces, shear velocity and wake zones behind objects.

The essence of an environmental flow assessment is “to identify those fundamental components of a river’s flow regime that are considered essential for perpetuation of its valued ecological or water resource features, and to negotiate for these to be built into a modified flow regime” (King and Tharme, 1994). Various environmental flow assessment techniques have been developed and are continuously being refined. Generally, these techniques are based on the interpretation of hydrological, hydraulic and biological data. Since the physical habitat is determined by the interaction between moving water and the physical attributes of a river, as described by channel morphology, hydrological data in the form of simulated, daily discharges must be translated into local hydraulic conditions in order

to determine how the physical habitat changes with discharge and how this affects the habitat preferences of different species (Broadhurst *et al.*, 1997). The environmental flow assessment process therefore requires an understanding of the interrelationships between the morphological and hydraulic conditions within a river channel.

In South Africa, various research projects, which address these morphological-hydraulic interrelationships to some extent, have been initiated. The need for an understanding of the links between local hydraulic conditions, channel morphology and riverine ecosystem structure, was first recognized by Rogers *et al.* (1992) as part of an integrated, holistic research programme in the Kruger National Park. The need to establish these links was further highlighted in the development of the “ecohydraulics” concept, which deals with the links between physical (mainly hydraulic) conditions in a river and biotic distributions, and which places great emphasis on the diversity or the mosaic of hydraulic conditions within a reach. Relevant research projects include: an assessment of channel flow resistance on the Sabie river in order to link hydrological data to hydraulic variables (Broadhurst *et al.*, 1997); the development of a hierarchical geomorphological model for the classification of selected South African rivers (Wadeson and Rowntree, 1999), which presents a framework within which the impacts of water management on channel form and associated ecological processes can be assessed; an empirical linkage of abiotic and biotic patterns on a regional basis to determine whether the geomorphological character of a river is a useful guide to its ecological character (King and Schael, 2001).

It is within this context that the current research was initiated. With the increasing development of water-related infrastructure in mountain regions, knowledge of the hydraulic characteristics of rivers in the upper catchment areas has become very important, especially for determining environmental flow requirements. Cobble and boulder bed rivers in the Western Cape are important examples of such rivers. Due to their characteristic morphological and associated hydraulic attributes, the physical habitats within these rivers are extremely diverse, both on a spatial and a temporal scale. They are characterized by high gradients, great variability in sediment size and relatively low flow depths. The bed configuration contains a series of pools, steps, rapids, riffles and plane beds, while energy losses are high as a result of turbulence and local hydraulic jumps. It can thus be appreciated that in order to determine the impacts of a modified flow regime on the physical habitat in cobble and boulder bed rivers, it is essential to understand the mechanisms controlling the morphological and hydraulic conditions within these rivers.

1.2 Objectives and Methodology

The specification of environmental flow requirements is based on the understanding that different stages of the flow regime elicit different responses from a river (King and Louw, 1998). Recommendations for environmental flow requirements therefore specify flows of different magnitude and timing in an attempt to simulate the fundamental character encompassed in the natural flow regime.

The main flow components comprising an environmental flow requirement are low flows, freshes and channel forming flows. The low flow component maintains the basic ephemeral or perennial nature of the river and through its different magnitudes in the wet and dry seasons, creates fundamentally different seasonal conditions. Freshes, which are short pulses of higher flows, trigger spawning in some fish species, dilute poor quality waters and provide essential flow variability. Channel forming flows are necessary for maintaining the active channel form and removing encroaching riparian vegetation (King and Tharme, 1994). Another important flow component in cobble and boulder bed rivers, often referred to as a flushing flow or sediment maintenance flow, aims to flush fine sediments from the interstitial spaces between the cobbles.

This research programme aims to determine the discharge-, substrate-, morphological- and hydraulic interrelationships characteristic of different environmental flow components in cobble and boulder bed rivers. These relationships are addressed at both the macro (reach) and local scales. The following objectives, each related to one or more environmental flow component, were formulated:

i. To determine the regime behaviour of cobble and boulder bed rivers

Relevant environmental flow component: Channel forming flow

Maintaining the morphological character of cobble and boulder bed rivers, is an important prerequisite for sustainable ecosystem functioning and requires an understanding of the mechanisms controlling channel deformation in these rivers. The regime behaviour of cobble and boulder bed rivers will therefore be addressed by developing a model which defines the relationships between a characteristic discharge, channel form, substrate characteristics and macro scale bed deformation.

ii. To develop a sand scour model for cobble bed rivers

Relevant environmental flow component: Flushing / Sediment maintenance flow

The accumulation of fine sands in cobble bed rivers, which fill the interstitial spaces between the cobbles, can have a detrimental effect on the whole aquatic ecosystem. In order to accurately specify flushing flows for removing the sand from the interstitial spaces between the cobbles downstream of reservoirs, a sand scour model which describes the scouring of fine sands from between cobbles in terms of time and discharge dependent relationships, will be derived.

iii. To determine velocity-depth relationships in cobble and boulder bed rivers

Relevant environmental flow components: Freshes, Low flows

Cobble and boulder bed rivers are characterized by very complex velocity-depth relationships, especially during low flows. Due to the very different hydraulic mechanisms which generate flow resistance when the flow depth is not much greater than the bed particle size, the standard friction based equations representing the relationship between mean velocity and flow depth become increasingly unrepresentative. Furthermore, as a result of large scale roughness, reaches are characterized by complex hydraulic conditions and the average, cross-sectional hydraulic parameters as such do not represent the diversity of local hydraulic conditions occurring within a reach. The relationship between mean velocity and flow depth, as well as the variability of local flow velocities during low flows in cobble and boulder bed rivers will therefore be described.

1.3 Layout

Chapters 2 and 3 respectively, provide introductory overviews of the environmental flow assessment process and of channel morphology. The development of a regime model for cobble and boulder bed rivers is described in Chapter 4, while Chapter 5 is dedicated to the development of a sand scour model. Chapter 6 addresses velocity-depth relationships in cobble and boulder bed rivers. Final conclusions and recommendations are made in Chapter 7 and references are listed in Chapter 8.

2. ENVIRONMENTAL FLOW ASSESSMENT

2.1 Introduction

Instream flows are those flows that are retained along their natural flowpaths as opposed to water which is diverted for “offstream” uses such as irrigation, industry etc. Instream flows are important for maintaining the habitat of aquatic and riparian ecosystems, but can also support economically important activities such as transportation, production of hydroelectricity and waste disposal. These ecosystems are best preserved under natural, pristine conditions. However, the development and management of water resources for human utilisation have altered the natural flow regimes of most rivers around the world. A new scientific discipline has therefore been developed to predict the biological impacts associated with water resources developments and to provide information on the flows that are required in a particular river in order to maintain the river in a pre-determined, environmentally acceptable condition. This then serves as a guideline for managing the river in terms of operating rules for dams, water diversions etc., while still providing for the ecological requirements of the riverine ecosystem.

The process of determination of the amount of water which is required for environmental needs, is known as an “Environmental Flow Assessment” (EFA). Following the EFA, a modified flow regime is prescribed for the river. The amount of water required in the modified flow regime is that which is deemed to be necessary for maintaining the river in a pre-determined condition and is known as the “Environmental Flow Requirement” (EFR). EFR’s are based on an understanding (Brown and King, 2000) of how flow changes relate to changes in river condition, in order to describe flows that will:

- minimize or mitigate the impacts of a new water-resource development
- restore systems impacted by past developments
- allow calculation of the costs of compensating people affected by such impacts.

In general, the closer to natural the desired condition of the aquatic system, the greater the fraction of the original flow regime that will be required as an EFR (Figure 2.1).

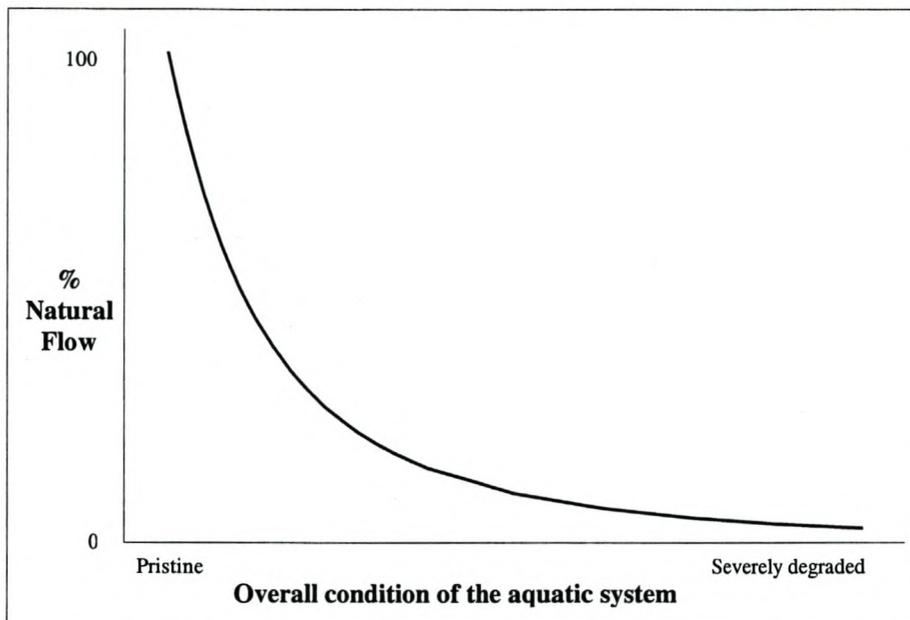


Figure 2.1 : Schematic illustrating the relationship between percentage of natural flow and river condition (Brown and King, 2000)

2.2 Environmental flow components

As indicated in Table 2.1, which describes the importance of different flow components from an ecological perspective, different parts of the flow regime elicit different habitat-related responses from a river. King and Louw (1998) list the most important characteristics of the natural flow regime of a river as:

- degree of perenniality
- magnitude of the low flows in the dry and wet season
- magnitude, timing and duration of floods in the wet season
- small floods that occur in the drier months

<p>The normal low flows in the river outside of floods</p>	<p>Low flows define the basic seasonality in a river – its dry and wet season, whether it flows all year or dries out for part of it. The different magnitudes of low flows in the dry and wet seasons create more or less wetted habitat and different hydraulic and chemical conditions, which directly influence what the balance of species will be in any season.</p>
<p>Freshes: small floods that occur several times within a year</p>	<p>Defined here as small pulses of higher flow, freshes are usually of most ecological importance in the dry season. These smaller floods stimulate spawning in fish, flush out poor quality water, mobilise sandy sediments, and contribute to flow variability. They re-set a wide spectrum of conditions in the river, triggering and synchronising activities as varied as upstream migration of fish and germination of riparian seedlings.</p>
<p>Large floods that occur less than once a year</p>	<p>Large, scouring floods dictate the form of the channel. They mobilise sediments and deposit silt, nutrients and seeds on floodplains. They inundate backwater areas, and trigger the emergence of flying adults of aquatic insects, which provide food for fish, frogs and birds. They maintain moisture levels in the banks, which support trees and shrubs, inundate floodplains, and scour estuaries thereby maintaining the link with the sea.</p>
<p>Flow variability</p>	<p>Variability of flow is essential for a healthy ecosystem. Different conditions are created through each day and season, controlling the balance of species and preventing dominance by pest species.</p>

Table 2.1: Effects of different components of a flow regime on aquatic ecosystems (Brown and King, 2000)

Recommendations for environmental flow requirements therefore usually specify flows of different magnitude and timing in an attempt to simulate the fundamental character encompassed in the natural flow regime. Figure 2.2 depicts the important features of a winter rainfall river's natural flow regime. Features 1 and 6 may recognize the perenniality of the river ; 2, 4 and 5 may recognize the importance of the difference between wet and dry season low flows; and 3 may recognize the timing of the first major flood of the wet season (King and Louw, 1998).

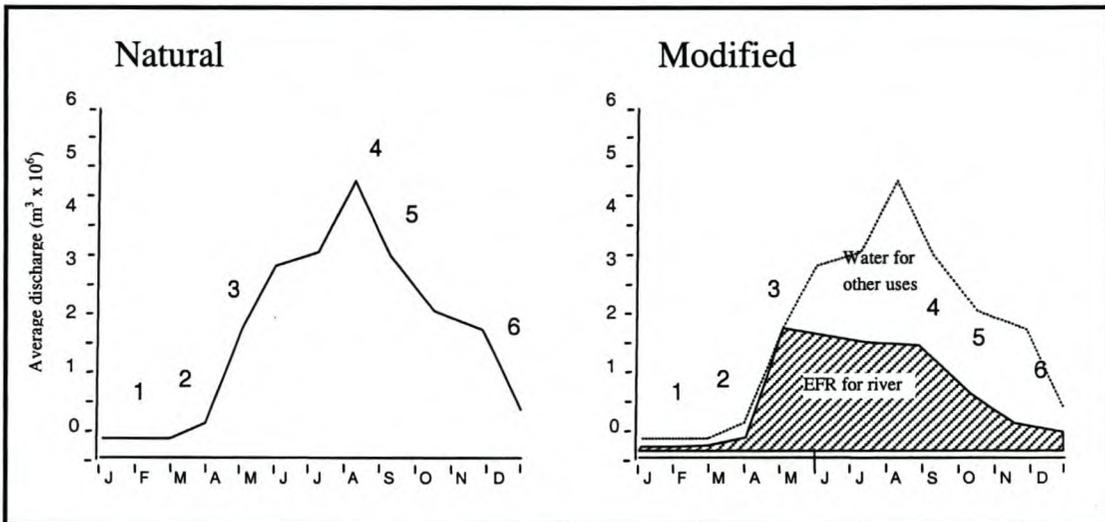


Figure 2.2 : Perceived important features of a winter rainfall river's natural flow regime (expressed as monthly averages) incorporated into the modified flow regime (King and Louw, 1998)

The main flow components comprising an EFR are:

- Low flows
- Freshes
- Channel forming flows

The low flow component maintains the basic ephemeral or perennial nature of the river and through its different magnitudes in the wet and dry seasons, creates fundamentally different seasonal conditions (King and Louw, 1998). Low flows can be further categorized into optimum, minimum and survival flows depending on the degree to which it meets the survival requirements of aquatic species. Some species may rely on low flows for a part of their life history, for others it is a time of stress (Gordon *et al.*, 1992). Short pulses of higher flows, known as freshes, trigger spawning in some fish species, mobilize sandy sediments during the wet and dry seasons, dilute poor quality waters and provide essential flow variability. Channel

forming flows are necessary for re-setting a wide spectrum of conditions in the river and riparian ecosystems. It maintains the active channel form and diversity of physical biotopes and removes encroaching riparian vegetation. Depending on the magnitude of the channel forming flow it is sometimes referred to as either a channel maintenance flow or a flood, with channel maintenance flows typically of a smaller magnitude than floods. Another important flow component relevant to cobble and boulder bed rivers is the so-called flushing flow or sediment maintenance flow. This flow aims to flush fine sands and sediments from the interstitial spaces between the cobbles and gravels downstream of reservoirs, introduced into this part of the river system from the incremental catchments downstream of a reservoir.

2.3 Environmental flow assessment methods

2.3.1 Overview

Research on environmental flow requirements, suggests that the flows which represent the normal characteristics of a specific river are the ones that the riverine biota are adapted to. In other words, the full range of natural intra- and interannual variation of flow regimes, and associated characteristics of timing, duration, frequency as well as rate of change, are critical in sustaining the integrity of aquatic ecosystems (Richter *et al.*, 1997). Therefore, flows that are not characteristic to a specific river, will constitute an atypical disturbance to the riverine ecosystem and could fundamentally change its character (King and Louw, 1998).

In establishing environmental flow requirements, the difficulty lies in deciding how much modification of the natural flow regime is acceptable. The lack of quantitative data on the effects of regulated flows on organisms is a limitation, which becomes especially critical when the preservation of aquatic habitat conflicts with other uses of water. Furthermore, financial and timing constraints do not allow extensive and detailed studies on species requirements in specific reaches and rivers. Cost effective, objective and consistent techniques, which provide reliable estimates of habitat requirements, have therefore been developed and are continuously being refined. These techniques are grouped into two broad approaches, viz. prescriptive and interactive approaches.

2.3.2 Prescriptive approaches

Methods based on the prescriptive approach usually address a narrow and specific objective and result in a recommendation for a single flow value or flow regime (Brown and King,

2000). They can be divided into three categories (Tharme, 1996), viz. hydrological index methods, hydraulic rating methods and holistic approaches.

i. Hydrological index methods

Examples of these methods are the Tennant or Montana method (Tennant, 1976) and the “Range of Variability Approach” (Richter *et al.*, 1997). These techniques recognize that hydrological variation plays a major part in structuring the biotic diversity within river ecosystems as it controls key habitat conditions within the river channel and floodplain (Richter *et al.*, 1997). They aim to maintain native aquatic biodiversity and ecosystem integrity by maintaining some semblance of natural flow variability, based on natural streamflow records. Flow requirements are either specified as a single minimum flow value or as different proportions of flow (% MAR) retained at different times of the year. The main attraction of these methods is the fact that an answer can be obtained rapidly if flow records are available, eliminating the time and cost of field data collection. However, little if any attention is given to the specific nature of the considered river or its biota (Brown and King, 2000). Furthermore, although these methods are specific about the magnitude of flow, they are vague about the timing and duration of flows and comprise no understanding or “feel” for the ecosystem (Gordon *et al.*, 1992).

ii. Hydraulic rating methods

These methods employ the relationship between the flow of the river (discharge) and simple hydraulic characteristics calculated from field data such as water depth, velocity or wetted perimeter, to advise on acceptable flows. However, they focus more on relationships between physical features of the river than on flow-related needs of the biota (Brown and King, 2000). Cross sections are usually chosen at ecologically critical sites, such as riffles, as it is reasoned that riffles will be affected more severely by flow alterations than pools, for example.

One example of this type of method is the Idaho method (Cochnauer, 1976 ; White, 1976). It employs a backwater calculation programme for calculating hydraulic parameters (flow depth, average velocity and wetted perimeter) at each transect for different discharges, which can be compared with known biological criteria.

Another example, the Wetted-Perimeter Method (Collings, 1972), is a low-resolution, river-specific method that is useful for determining seasonal flows required to maintain fish populations. It is relatively quick and cost-effective and is useful as a planning method at a catchment or greater level (Tharme, 1996). The method is based on the assumption that fish

rearing is related to food production, which in turn is related to how much of the river bed is inundated. It uses relationships between wetted perimeter and discharge, depth and velocity to set minimum discharges for fish food production and rearing (including spawning). The relationships are constructed from measuring the length of the wetted-perimeter at different discharges in the river of interest. The resulting recommended discharges are based on inflection points on the wetted-perimeter/discharge curve, each of which is assumed to represent the maximum habitat for minimum flow before the next inflection point.

iii. Early holistic approaches

These methods require collection of considerable river-specific data, and make structured links between flow characteristics of the river and the flow needs of the main biotic groups (fish, vegetation, invertebrates) (Brown and King, 2000). Examples include the Holistic Method and the Building Block Methodology.

The Holistic Method (Arthington *et al.* 1992) and the Building Block Methodology (King and Louw, 1998) were developed in parallel. The methodologies share the same basic tenets and assumptions, and both require early identification of the future desired condition of the river (Brown and King, 2000). An EFR is then described that should achieve and maintain this condition. Both involve the construction of a modified flow regime on a month-by-month basis, through separate consideration of different components of the flow regime. Each flow component represents “a well-defined feature of the flow regime intended to achieve particular ecological, geomorphological or water-quality objectives in the modified river ecosystem” (Arthington *et al.* 1992).

The Holistic Method is essentially an amalgamation of various methods and computer software, which relies heavily on expert opinion. It is used to ascertain the effects of reduced or increased flows on a river and to examine the impacts of periods of low, relatively constant flows, and of high, wet-season flows and floods, on the migration, spawning and dispersal requirements of fish (Brown and King, 2000). The Building Block Methodology (BBM) is based on the assumption that there are some flows within the total flow regime of a river that are more important than others for the maintenance of that river ecosystem. Such flows can therefore be identified in terms of their timing, duration, frequency and magnitude and combined into a recommended modified flow regime that is specific for that river. During identification of these flows, the focus is on the characteristic features of the natural flow regime of the river. As indicated in Figure 2.3, each of the identified flows is considered a building block that creates the modified flow regime or EFR and is deemed to perform a

required ecological or geomorphological function (King and Louw, 1998). The BBM was designed to cope with the southern African realities of limited data, money and time. It depends on available knowledge, expert opinion and some new data, which are used in a structured workshop session to describe an EFR (King and Louw, 1998). It considers the major components of the river ecosystem, both physical (hydrology, physical habitat, and chemical water quality) and biological (vegetation, fish and macroinvertebrates), as well as subsistence use of the river by riparian people. For each of these disciplines, all available data are synthesized and new data collected where necessary. Field measurements always include the surveying of cross-sections at representative sites along the river and development of the relationship between flow and water depth, velocity and area of inundation. The biological specialists also conduct field studies from which they develop an understanding of the links between aquatic species and the flow in the river at different times. The strength of the BBM therefore lies in its ability to incorporate any relevant knowledge, and to be used in both data-rich and data-poor situations (Brown and King, 2000).

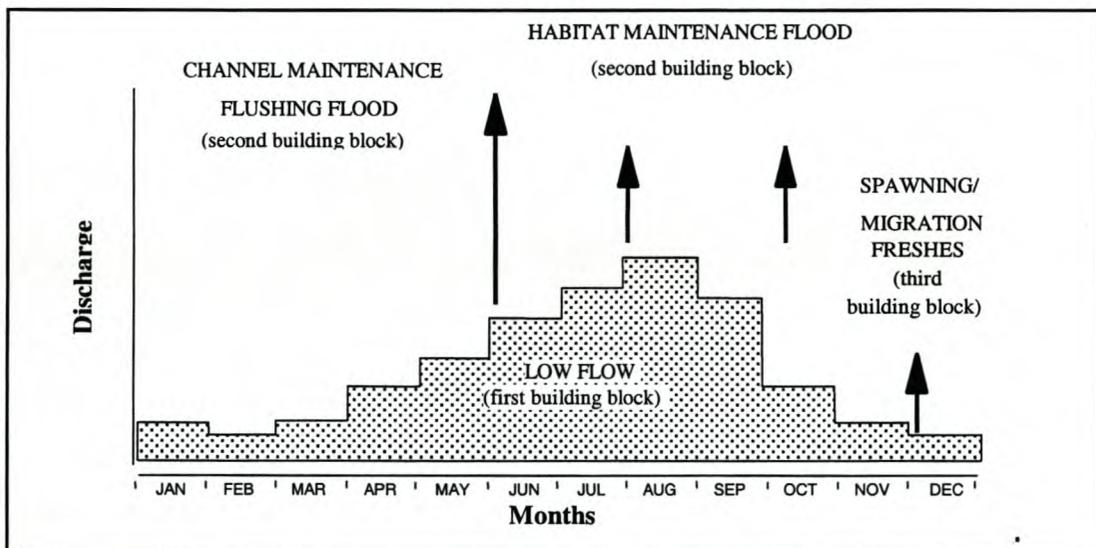


Figure 2.3 : The “building blocks” of the modified flow regime created using the BBM (King and Louw, 1998).

2.3.3 Interactive approaches

The EFA methods that use an interactive approach tend to be more complex than prescriptive methods and are predominantly limited to two types, viz. habitat simulation and holistic

methodologies. Both are essentially problem-solving tools, the output of which is a set of options (Brown and King, 2000). Each option quantitatively describes :

- a modified flow regime
- the resulting condition of the river, or species (whichever is being addressed)
- the effect on yield for offstream users
- the direct economic costs and benefits

i. Habitat simulation methods

Instream habitat simulation methods can be considered to be sophisticated hydraulic rating methods. Whereas hydraulic rating methods are based on the optimum ranges of velocity and depth determined for a species, habitat simulation methods use a continuous function to describe preferences and allow more flexibility in evaluating the effects of changing discharges on habitat availability (Brown and King, 2000). Furthermore, they consider not only how physical habitat changes with discharge, but combine such information with the habitat preferences of different species. (Table 2.2, for example, lists the typical habitat requirements for various fish species.) In this way the available habitats for specific species over a range of discharges are determined.

Species	Depth (m)	Velocity (m/s)	Substrate type
Blackfish	> 0,20	0 – 0,30	All
Brown trout	> 0,20	0 – 0,50	All
Redfin/Common carp	>1,0	0 – 0,20	Mud/Sand
Short-finned eel	> 0,20	0 – 0,30	All

Table 2.2 : Hydraulic habitat requirements for rearing habitat of fishes of Southwestern Victoria (Tunbridge, 1988)

The Instream Flow Incremental Methodology (IFIM) is considered to be the most sophisticated instream habitat simulation method. IFIM was devised by the United States Fish and Wildlife Service to assist in the assessment of environmental flow requirements of rivers (Bovee, 1982). Basically, IFIM is a problem-solving tool, comprising a collection of analytical procedures and computer programs, including the physical habitat simulation model, PHABSIM II. This model simulates hydraulic conditions over a range of discharges and then links the hydraulic information with habitat information on key riverine species to

produce habitat discharge relationships (Bovee and Milhous, 1978; Milhous *et al.*, 1989). It considers the effect of incremental changes in discharge on both the macrohabitat (channel characteristics, temperature and water quality) and microhabitat (distribution of hydraulic and structural features making up the living space for an organism). The two basic components of the model are hydraulic simulation and habitat simulation. Hydraulic simulation is based on measured and/or calculated data at selected cross sections and predicts conditions of velocity, water depth, substrate and hydraulic and vegetal cover over a range of discharges. Simulations are done at a level of resolution deemed to be ecologically relevant, by compartmentalizing the cross section into a grid of lateral cells extending halfway to adjacent cross sections. The hydraulic conditions in each cell are then simulated. Data are also collected on the habitat preferences of the selected riverine species. Based on the assumption that the habitat of aquatic species is determined by the hydraulic environment, habitat curves are constructed, showing species' preferences on a scale of 0 to 1 in terms of water depth, velocity, substrate and cover conditions. Typical examples of these curves, also known as "suitability index curves" or SI curves, are shown in Figure 2.4.

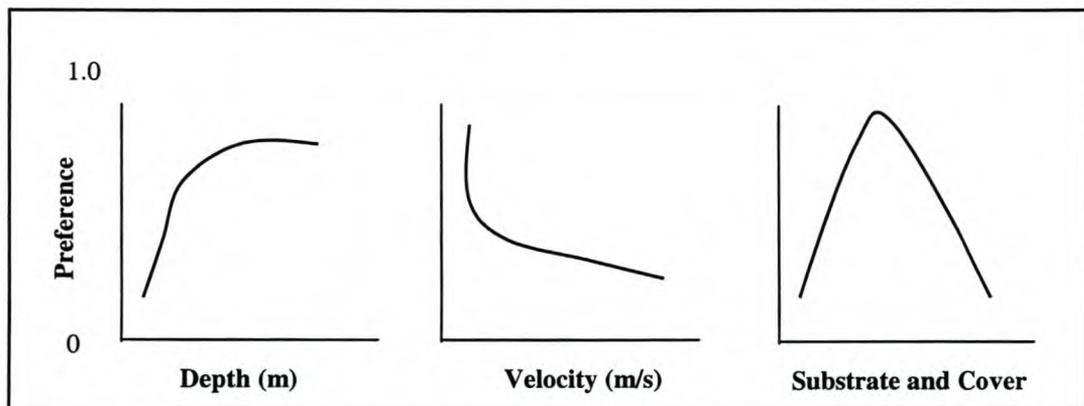


Figure 2.4 : Suitability index curves for a specific species' life stage

PHABSIM II then links the information on the hydraulic conditions within each cell with that on the preferred habitat of species, by using the SI curves to assess the suitability of hydraulic conditions at different discharges. The suitability for each cell at each discharge is then expressed as a combination of velocity, depth, substrate and cover conditions by a composite factor, known as "available flow-related microhabitat" or the "Weighted Usable Area" (WUA). WUA is therefore an indicator of the net suitability of use of a given reach by a certain life stage of a certain species and is expressed in units of area per unit length of river (Figure 2.5).

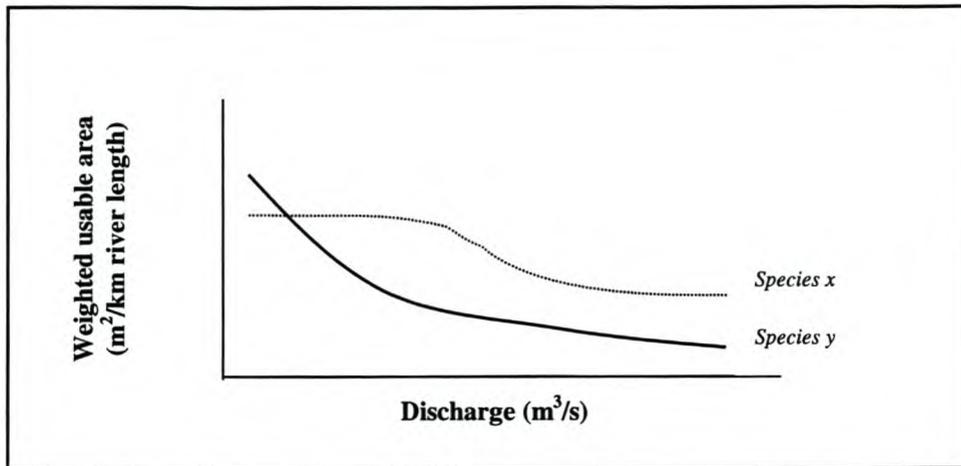


Figure 2.5 : Available habitat as a function of discharge

To extend the relevance of the PHABSIM output, the habitat-discharge functions may be combined with flow data to obtain monthly or daily habitat time series and habitat duration curves. Such curves are useful for comparing pre- and post project habitat availability.

Habitat simulation models are useful tools for determining instream flow requirements. However, some limitations have been identified. A critical limitation is the high cost of developing well-defined habitat-suitability curves. Furthermore, the curves may not be transferable from one stream to another. Another potential limitation is the inaccurate application of hydraulic calculation procedures such as backwater calculations or stage discharge relationships, due to inaccurate or insufficient cross sectional or roughness data.

ii. Holistic methodologies

Another type of interactive approach involves the so-called holistic methods, of which the DRIFT model is a typical example. DRIFT (Downstream Response to Imposed Flow Transformations) was developed for the assessment of environmental flows for the Lesotho Highlands Water Project. The methodology arose from, and its data-collection steps closely approximate those of, the BBM. Like the BBM, DRIFT culminates in one or more multidisciplinary workshops, but unlike the BBM, these are designed to produce an agreed number of biophysical and socio-economic scenarios (Brown and King, 2000). The central rationale of DRIFT is that different parts of the flow regime, e.g., low flows, freshes and floods of a river elicit different responses from the components of the riverine ecosystem. Thus, removal of one or more kinds of flow will affect the riverine ecosystem differently than removal of some other combination (Brown and King, 2000). Within DRIFT, the specialists

use component-specific methods to collect data and predict the consequences of flow changes in the way required by DRIFT.

Essentially DRIFT is a system for managing a great amount of data and knowledge in a structured way, following five main steps (Brown and King, 2000):

- Identification and isolation of wet-season and dry-season low flows, freshes, and floods from the long-term hydrological record.
- Description of the consequences for the river of partial or whole removal of each of these flow components.
- Creation of a biophysical database detailing the consequences.
- Use of the database to describe how river conditions will change with any future combination of high and low flows removed.
- Description of the socio-economic implications of changes in river condition.
(This and the previous step constitute the creation of EFR scenarios.)

Figure 2.6 is a diagrammatic representation of the DRIFT methodology.

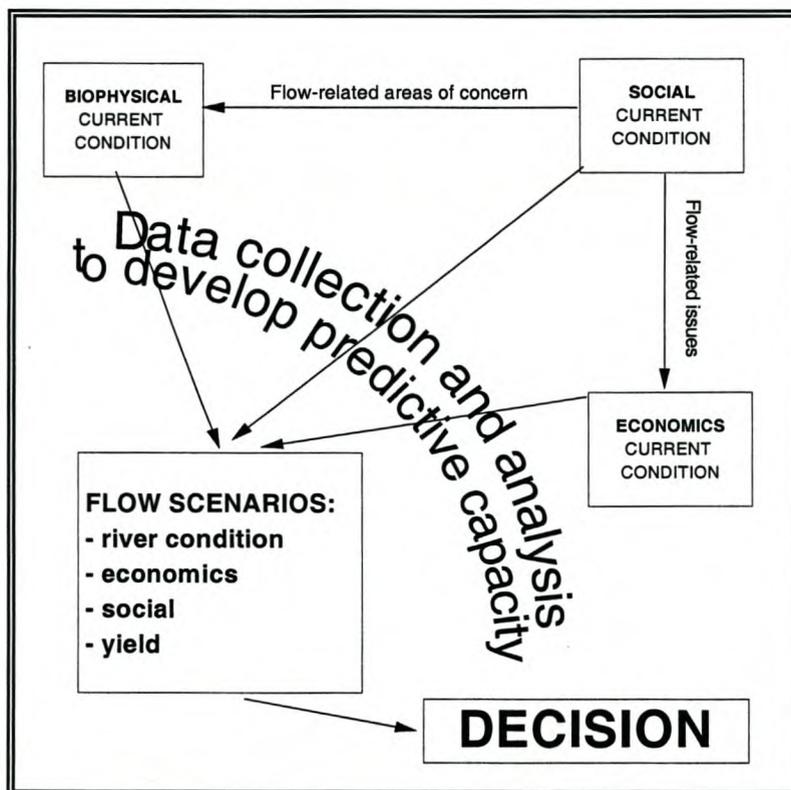


Figure 2.6 : Basic components of a DRIFT assessment (Brown and King, 2000)

The DRIFT methodology can also be used for advising on flow restoration for degraded rivers, by describing the consequences of adding rather than removing flow components (King, pers. comm., 2001)

2.4 Conclusion

As is evident from the above, various methodologies and approaches, representing different levels of detail across different spatial scales, have been developed for determining the quantity of water which is required for the environmental needs of a river. The essence of these techniques is to identify those flows that are required in a particular river in order to maintain the river in a pre-determined, environmentally acceptable condition. In order to achieve this, certain “tools” are required which serve to anticipate the effect of a modified flow regime on the morphological and associated hydraulic characteristics of a river, at both the reach and the local scales. This study focuses on the development of empirical, semi-empirical and theoretically based models aimed at addressing key morphological and hydraulic interrelationships for the assessment of environmental flow requirements in cobble and boulder bed rivers.

3. CHANNEL MORPHOLOGY: AN OVERVIEW

3.1 Introduction

Channel morphology (also referred to as fluvial geomorphology) is the branch of science that attempts to find systematic order in the wide array of landforms shaped by rivers and tries to understand the processes responsible for their development (Kellerhals and Church, 1989). The interaction between the moving water and the physical attributes of a river, as described by channel morphology, determines ecosystem functioning through the availability of physical (hydraulic) habitat for aquatic species. An understanding of the basic morphological concepts is therefore imperative when assessing environmental flow requirements, especially in the case of cobble and boulder bed rivers, which are characterized by macro bed forms and which, due to their characteristic morphological and associated hydrological attributes, display extreme habitat diversity.

This chapter describes the basic concepts related to fluvial geomorphology in terms of channel type, location within the longitudinal profile, channel pattern and channel structure, to provide a framework for the morphological classification of cobble and boulder bed rivers in the Western Cape.

3.2 Channel type

River channels can be classified into two broad types, viz. bedrock and alluvial channels. Richards (1987) defined bedrock channels as “channel segments which lack a coherent bed of active alluvium”. In bedrock channels the geology of the channel bed and its resistance to erosion are the main determinants of channel form (Rowntree and Wadeson, 1999). The gradient of bedrock channels therefore exhibits no necessary correlation with discharge and sediment characteristics. Bedrock sections are common along many rivers in mountainous regions, often occurring as short riffles or waterfalls, but most commonly form the headwater tributaries in otherwise alluvial channels (Richards, 1987).

As opposed to bedrock channels, alluvial channels are defined as channels that flow through sediments which they have previously deposited, with the form of the channel being a result of the balance between the available sediment and the transport capacity of the river. Alluvial channels are free to adjust their dimensions and as a result consistent relationships exist between discharge and channel width, depth and gradient. In alluvial rivers substrate characteristics are

most commonly described in terms of substrate size. Table 3.1 provides a classification of the different particle sizes as defined by the Wentworth scale, which is based on the length dimension of the median axis. (The phi (ϕ) scale is also often used to define particle size and is equal to the negative logarithm (base 2) of the particle size in millimetres.)

Class (Wentworth)	Diameter (mm)	Phi
Boulder	> 256	-12 to -8
Cobble	64 to 256	-8 to -6
Gravel	2 to 64	-6 to -1
Sand	0,0625 to 2	-1 to 4
Silt	0,0039 to 0,0625	4 to 8
Clay	< 0,0039	8 to 12

Table 3.1: Grade scales for substrate particle size (Adapted from Brakensiek *et al.*, 1979)

Other characteristics of the substrate include the particle shape, which is typically described in terms of roundness and sphericity, while the arrangement and associated bulk properties of the substrate are described in terms of orientation, stability, porosity, density and degree of embeddedness.

Channels which display a mixture of bedrock and alluvial sediments are known as mixed bed channels.

3.3 Longitudinal zonation

Longitudinal river zones provide a basis for within-river classification that can not only be used to identify geomorphologically similar streams, but also to retain the concept of longitudinal downstream changes (King and Schael, 2001). Various geomorphological classification systems, which are based on the concept of different zones along the length of a river, have been developed. They categorize river systems in terms of gradient (Davis, 1890), sediment production and mobility (Schumm, 1977) and substrate characteristics (Pickup, 1984). Rowntree and Wadson (1999) described the geomorphological zonation of South African rivers in terms of average channel gradient and characteristic morphological features. Table 3.2, which was originally produced by Rowntree (1996) but has been modified by King and Schael (2001) to comply with the stream types suggested by Rosgen (1994), shows the geomorphological zonation of South African rivers associated with “normal” profiles with a characteristic concave

shape. (Additional zones are associated with a “rejuvenated profile”, which exhibits steepening in its downstream segments.)

3.4 Channel patterns

Channel pattern classification refers to the planimetric form of the river. Generally, channel patterns can be classified into two broad categories, viz. single-thread and multi-thread channels.

Single-thread channels may further be classified into straight or meandering channels. Straight and meandering channels are discerned based on the degree of sinuosity, defined as the length of the active (thalweg) channel divided by the valley distance (Richards, 1982). Straight channels display a sinuosity of 1, while meandering channels are defined by a sinuosity of 1.5 or more. Channels with a sinuosity index of between 1 and 1.5 are referred to as sinuous. Multi-thread channels are classified as either braided or anastomosing. In the case of braided channels, two or more channels are divided by alluvial bars, while anastomosing channels are characterized by multi-thread channels separated by stable islands.

3.5 Channel structure : the morphological unit

Fluvial geomorphologists have developed the concept of the morphological unit to describe elements of channel morphology. The morphological unit is the basic structure or building block comprising the channel morphology and may be either an erosional or depositional feature. Although, in the long term, its characteristics are dependent on the imposed flow regime, which determines the erosion and sediment transport processes, in the short term it is considered a constant feature (Rowntree and Wadson, 1996). In alluvial channels, morphological units can be divided into two groups, viz. pools and bars. Pools are scour features which form upstream of hydraulic controls and which contain relatively slow flow and deep water at low flows. Bars, on the other hand, are depositional features, which can be classified according to the nature of the material composing them and by their location within the channel. Bars serve as energy dissipators that permit stable channel configurations to be maintained in the presence of sediment transport (Hey *et al.*, 1982). Table 3.3 provides a comprehensive list of morphological units associated with alluvial rivers.

Zone	Channel gradient (%)	Characteristic channel features
Source	Not specified	Low gradient, upland plateau or upland basin able to store water. Spongy or peat hydromorphic soils.
Mountain headwater	> 10	Very steep gradient stream dominated by vertical flow over bedrock with waterfalls and plunge pools. Normally first or second order streams. Reach types include bedrock fall and cascades
Mountain	4 – 9	Steep gradient stream dominated by bedrock and boulders, locally cobble or coarse gravels in pools. Reach types include cascade, bedrock fall, step-pool.
Mountain (Transitional)	2 – 4	Moderately steep stream dominated by bedrock or boulder. Reach types include plane bed, pool-rapid or pool-riffle. Confined or semi-confined valley floor with limited flood plain development.
Upper Foothills	0.5 - 2	Moderately steep, cobble bed or mixed bedrock-cobble bed channel, with plane bed, pool-riffle, or pool-rapid reach types. Length of pools and riffles/ rapids similar. Narrow flood plain of sand, gravel or cobble often present
Lower Foothills	0.1 – 0.5	Lower gradient mixed bed alluvial channel with sand and gravel dominating the bed, locally may be bedrock controlled. Reach types typically include pool-riffle or pool-rapid, sand bars common in pools. Pools of significantly greater extent than rapids or riffles. Flood plain often present.
Lowland river	0.01 – 0.1	Low gradient alluvial fine bed channel, typically regime reach type. May be confined, but fully developed meandering pattern within a distinct flood plain develops in unconfined reaches where there is an increased silt content in bed or banks

Table 3.2: Geomorphological zonation of South African river channels (King and Schael, 2001)

Morphological unit	Description
Pool	Topographical low point in an alluvial channel caused by scour, characterized by relatively finer bed material
Backwater	Detached side channel ; connected at lower end to main flow channel
Rip channel	High flow distributary channel on the inside of point bars or lateral bars; may form backwater at low flows
Plane bed	Topographically uniform bed formed in coarse alluvium, lacking well defined scour or depositional features
Lateral bar / side bar	Accumulation of sediment attached to channel margins, often alternating from one side to the other inducing a sinuous thalweg channel
Point bar	Formed on the inside of meander bends in association with pools. Lateral growth into the channel is associated with erosion on the opposite bank and migration of meander loops across the flood plain
Transverse bar	Bar forms across entire channel at an angle to the main flow direction
Riffle	Transverse bar formed of gravel or cobble, commonly separating pools
Rapid	Steep transverse bar formed from boulders
Step	Step-like features formed by large clasts (cobble and boulder) organized into discrete channel spanning accumulations ; steep gradient
Channel junction bar	Forms immediately downstream of a tributary junction due to the input of coarse material into a lower gradient channel
Lee bar	Accumulation of sediment in the lee of a flow obstruction
Mid-channel bar	Single bar formed within the middle of the channel, with strong flow on either side
Braid bar	Multiple mid-channel bars forming a complex system of diverging and converging thalweg channels
Sand waves / Lingoid bar	Large mobile feature formed in sand bed rivers which has a steep front edge spanning the channel and which extends for some distance upstream. Surface composed of smaller mobile dunes
Bench	Narrow terrace-like feature formed at edge of active channel abutting on to macro-channel bank
Islands	Mid-channel bars which have become stabilized due to vegetation growth and which are submerged at high flows due to flooding

Table 3.3 : Classification of alluvial morphological units (Rowntree and Wadeson, 1999)

Because cobble and boulder bed rivers are dominated by large clasts (detrital material consisting of fragments of broken rocks which have been eroded, transported and redeposited at a different site), which require high thresholds of stream power before movement takes place, the larger cobbles and boulders provide relatively immobile channel structures through which finer material is transported. Cobble and boulder bed rivers therefore frequently have a wide particle size range and are poorly sorted (Rowntree and Wadeson, 1999), but may be locally well sorted (King, pers. comm., 2001). The dominant bar types which occur in these reaches are transverse bars, characterized by coarse sediments, and point or alternate bars, especially along the lower reaches during low flows or when a high concentration of fine sediments is present.

3.6 Reach classification

Within a geomorphological context, Rowntree and Wadeson (1999) defined a reach as “a length of channel within which the local constraints on channel form are uniform, resulting in a characteristic channel pattern, degree of incision and cross section form within which a characteristic assemblage of channel morphologies occur.” Reaches in cobble and boulder bed rivers may be classified based on their assemblage of morphological units (Table 3.4).

Reach type	Description
Step-pool	Characterized by large clasts which are organized into discrete channel spanning accumulations that form a series of steps separating pools containing finer material
Plane bed	Characterized by plane bed morphologies in cobble bed or small boulder channels lacking well defined scour or depositional morphological units
Pool-riffle	Characterized by an undulating bed that defines a sequence of bars (riffles) and pools
Pool-rapid	Channels are characterized by long pools backed up behind fixed boulder deposits forming rapids

Table 3.4: Reach types in cobble and boulder bed rivers (Adapted from Grant *et al.*, 1990; Montgomery & Buffington, 1993 and Van Niekerk *et al.*, 1995)

The most common reach types associated with cobble and boulder bed rivers are the step-pool, plane bed, pool-rapid and pool-riffle.

3.6.1 Step-Pool

Step-pool reaches are characterized by large clasts organized into discrete channel spanning accumulations that form a series of steps separating scour pools containing finer material (Grant *et al.*, 1990). The channel width is of the same order of magnitude as that of the clasts themselves. There is a strong vertical component to the flow in step-pool reaches, contrasted to the more lateral flow in lower gradient pool-riffle reaches (Rowntree and Wadeson, 1999). The largest volume of sediment transported through this reach type is in the sand size (Leopold, 1992), with boulder and cobble fractions making up the major features of channel morphology, which remains stable except in rare flood events (Rowntree and Wadeson, 1999).

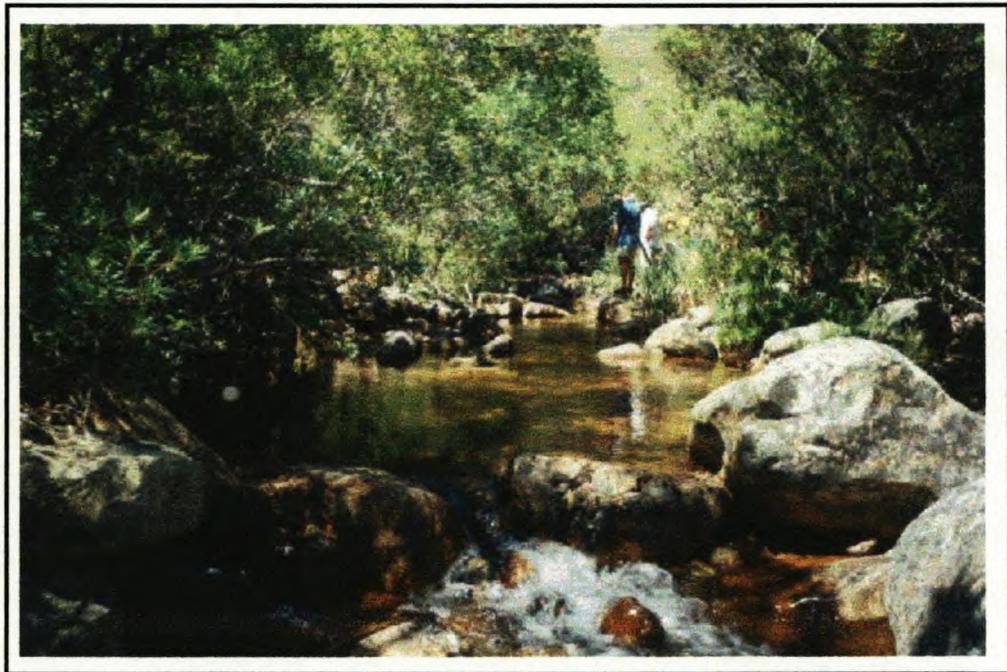


Figure 3.1: A typical step-pool reach type

3.6.2 Plane bed

Another reach type commonly associated with cobble and boulder bed rivers is a plane bed unit, which describes channels developed in coarse bed material which lack well-defined bedform (Montgomery and Buffington, 1993). These features are quite distinct from both step-pool and pool-riffle channels in that they lack rhythmic bedforms. They appear to occur naturally at gradients and relative roughnesses intermediate between step-pool and pool-riffle reaches. The larger clasts are generally scattered over the channel bed and at low to moderate flows project out of the water (Wadeson and Rowntree, 1999).



Figure 3.2: Plane bed morphological unit

3.6.3 Pool-Riffle

The alternating pool-riffle bedform is most common on streams with bed materials ranging from “pea” to “watermelon” size (Knighton, 1984). In South Africa they are dominated by substrate in the size class of cobbles and boulders (Rowntree and Wadeson, 1999). Riffles are topographic highs and are formed by the accumulation of coarse material to form a transverse bar with a steeper gradient (Selby, 1985). Pools are topographic lows, which are scour features located between riffles. Their positions are often coincident with point bars situated on meander bends.

The pool-riffle bedform is considered a means of self-adjustment by streams which acts to regulate energy expenditure. They are usually formed during larger floods (Gordon *et al.*, 1992).

Pools and riffles are easily distinguished at low flows. However, various techniques have been developed for their objective classification. These include bed material size (Leopold *et al.*, 1964; Mosley, 1982), water surface slope (Yang, 1971; Jowett, 1993), the ratio of average velocity to average depth (Wolman, 1955; Jowett, 1993), Froude number (Jowett, 1993) and bed topography (O'Neill and Abrahams, 1984). At low discharges flow through pools is deep relative to that over riffles and the surface water gradient and flow velocity are low. Pools are therefore areas of deposition of fine material during low flow periods. Riffles, on the other hand, are characterized by coarser material and fast, shallow flow with high velocity and a high surface water gradient relative to the pool.



Figure 3.3: An example of a pool-riffle reach type

3.6.4 Pool-Rapid

This reach type is very similar to pool-riffle reaches, except that it occurs at relatively higher channel gradients and is associated mainly with boulders instead of cobbles. The rapids are characterized by highly turbulent flows and high flow velocities.

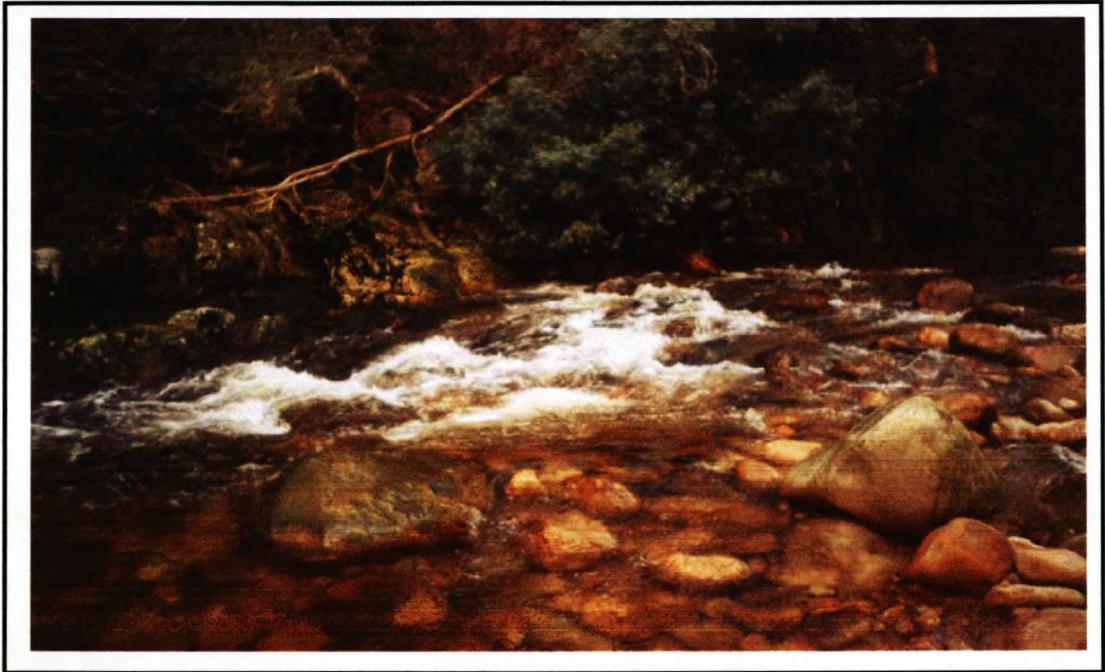


Figure 3.4: Rapid morphological unit

4. REGIME BEHAVIOUR OF COBBLE AND BOULDER BED RIVERS

4.1 Introduction

A characteristic morphological feature of cobble and boulder bed rivers is the presence of macro scale bed forms (pool-riffle or pool-rapid structures) and the general absence of smaller scale bed forms which may include ripples, dunes, etc. Due to the presence of these bed forms and their associated morphological and hydraulic attributes, the physical habitat in cobble and boulder bed rivers is extremely diverse, both on a spatial and a temporal scale. The high gradients of rapids and riffles lead to turbulence and high oxygen levels, while deeper pools are associated with lower water temperatures and lower velocities. Gordon *et al.* (1992) state that “in terms of physical habitat, the pool-riffle structure provides a great diversity of bed forms, substrate materials and local velocities”. This is echoed by Scheuerlein (1999) who, referring to cobble and boulder bed rivers, states that “a high variation of abiotic patterns e.g. channel width, flow depth, flow velocity, turbulence, bed structure, etc., provides the basis for valuable ecosystems”. Maintaining the morphological character of cobble and boulder bed rivers is therefore an important prerequisite for sustainable ecosystem functioning.

Cobble and boulder bed rivers are classified as alluvial rivers and the concept of a regime channel as defined by Richards (1987) applies, i.e. a self-formed channel which, when subject to relatively uniform governing conditions, is expected to show a consistency of form or average geometry adjusted to transmit the imposed water and sediment regime. In order to predict the long term, morphological impacts associated with a modified flow regime on cobble and boulder bed rivers, it is therefore essential to understand the mechanisms controlling the regime behaviour of these rivers, specifically the process of macro scale bed deformation.

This part of the research aims to establish fundamental, theoretically based relationships between discharge, channel geometry, bed deformation and substrate characteristics in cobble and boulder bed rivers. Section 4.2 describes the typical morphological characteristics of cobble and boulder bed rivers in the Western Cape, while a model is proposed in section 4.3 which addresses the regime behaviour of cobble and boulder bed rivers.

4.2 Morphological characteristics of cobble and boulder bed rivers in the Western Cape

4.2.1 Overview

Based on field observations it was found that, in terms of general morphological characteristics, rivers in the Western Cape can be classified into two broad categories viz. steep, coastal rivers and the typical longitudinal concave-shaped, alluvial or mixed bed rivers. Steep coastal rivers are characterized by bedrock and boulders and contain series of waterfalls and scour pools. The channel gradient is steep and fairly constant from estuary to source. Examples of these rivers are the Rooiels and Steenbras Rivers. Alluvial or mixed bed, concave-shaped rivers, typically rise in the mountainous areas from where they flow through foothills and low-lying areas until eventually reaching either the Atlantic or Indian Ocean. The upper part of the mountain zone is dominated by bedrock and boulders and is characterized by waterfalls and deep pools. Further downstream, in the mountain transitional and upper foothill zones, reaches display a step-pool, plane bed or pool-rapid morphology, dominated by boulders. Progressing down into the foothill zones, grain sizes decrease to within the range of cobbles and pool-riffle reach types start to dominate. Still further downstream, the substrate changes to gravel, sand and eventually to silt, while the channel follows a meandering pattern (refer to Figure 4.1). Examples of these rivers within the study area are the Berg-, Breede- and Jonkershoek Rivers.

4.2.2 Data collection

In order to determine the typical morphological character of cobble and boulder bed rivers in the Western Cape, extensive field data were collected in the cobble and boulder bed reaches which dominate in the mountain transitional and foothill zones of the concave shaped, alluvial rivers within the study area. Thirteen study reaches were selected, representing a range of reach types in terms of substrate size, morphological structure and position within the longitudinal profile. Reach lengths typically varied from about 60 m along the upper reaches to more than 250 m along the lower reaches with each reach including at least one sequence of pool-rapid or pool-riffle structures. Table 4.1 lists the study reaches and contains general, descriptive information. The definitions as provided in the previous chapter were used to assign each study reach to a reach type and longitudinal zone. Similarly, the identification of the dominant substrate class within each study reach was based on the definitions as per Chapter 3. (Appendix A contains a photograph of each study reach.)

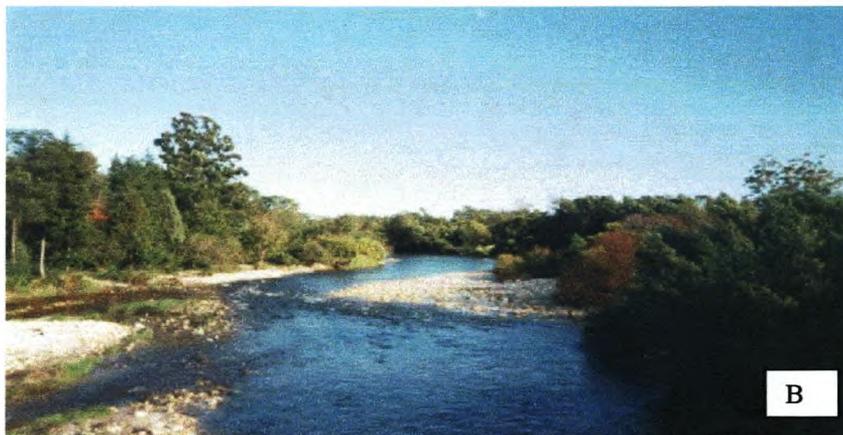
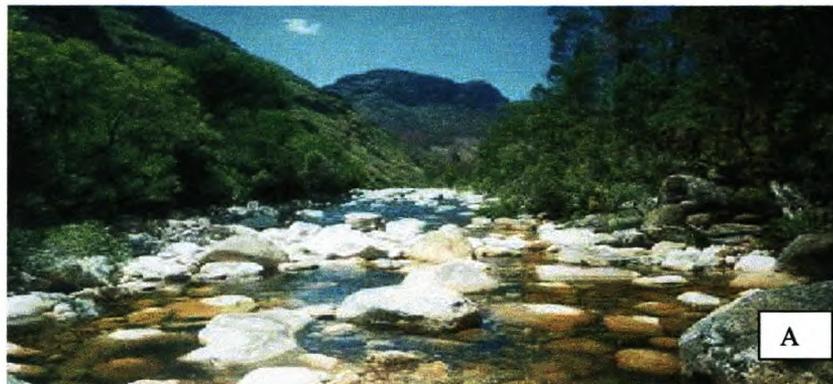
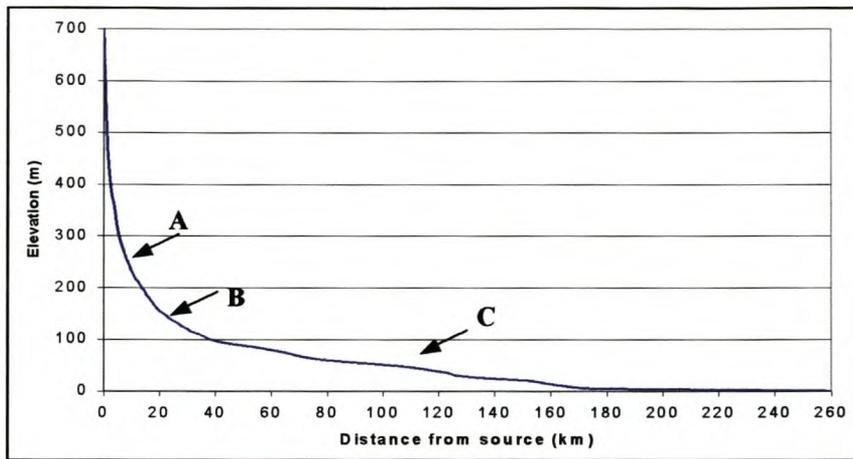


Figure 4.1 : Longitudinal profile and associated morphological characteristics in the Berg River

Study reach	Catchment name and area (km ²)	Lat.	Long.	Longitudinal zone	Dominant substrate	Reach type	***Gradient (S ₀) (%)
**Whitebridge	Jonkershoek (10)	33 59 37	18 58 40	Mountain	Boulder ; Bedrock	Step-Pool	5
**Jonkershoek	Jonkershoek (15)	33 59 20	18 58 01	Mountain trans.	Boulder	Pool-Rapid	2.7
*/**Vergenoeg	Jonkershoek (53)	33 56 10	18 53 35	Upper foothills	Cobble	Pool-Riffle	1.4
*/**Vlottenburg	Jonkershoek (192)	33 58 21	18 48 03	Lower foothills	Cobble	Pool-Riffle	0.7
Smalblaar	Breede (23)	33 43 50	19 06 50	Mountain trans.	Boulder ; Bedrock	Step-Pool	2.5
*/**Elands	Breede (61)	33 44 03	19 07 04	Upper foothills	Boulder	Pool-Rapid	1.3
*/**Molenaars I	Breede (92)	33 43 10	19 08 10	Upper foothills	Cobble	Pool-Riffle	0.8
*Molenaars II	Breede (111)	33 43 02	19 10 20	Upper foothills	Boulder	Pool-Rapid	1.2
**Berg I	Berg (20)	33 59 10	19 03 40	Mountain trans.	Boulder	Pool-Rapid	3.3
*/**Berg II	Berg (38)	33 57 30	19 04 10	Upper foothills	Boulder	Pool-Rapid	1.4
Berg III	Berg (172)	33 52 20	19 02 01	Upper foothills	Cobble	Pool-Riffle	0.8
*Berg IV	Berg (526)	33 45 04	18 56 21	Lower foothills	Cobble	Pool-Riffle	0.3
**Du Toits	Riviersonderend (21)	33 56 13	19 10 10	Mountain trans.	Boulder	Pool-Rapid	2.2

* Reaches in which the regime model was verified (refer to paragraph 4.3.3)

** Reaches in which cross section and velocity data were collected (refer to Chapter 6)

*** Determined from 1:10 000 orthophotos

Table 4.1 : List of study reaches

No chemical or biological data were collected. Only data on the physical (morphological) attributes of reaches were collected. This included :

i. Bed form geometry

With a survey instrument and staff, data on the length of the bed form (L), the local riffle or rapid gradient (α_r) and the pool depth (Δz) for at least one pool-riffle or pool-rapid structure in each study reach were collected (these parameters are defined in Figure 4.2). In some of the study reaches the entire longitudinal profile was surveyed. (Appendix B contains these longitudinal profiles as well as the locations of measured L , α_r and Δz values.)

ii. Channel width and depth

Within each study reach, the bankfull channel width and depth were measured at three transects within a typical pool-riffle or pool-rapid sequence. The transects represented, respectively, a pool, a riffle or rapid, and the length of channel in between. Average values of bankfull width (W_{ch}) and depth (D_{ch}) were then calculated for each study reach.

iii. Substrate size distribution

The determination of the substrate size distribution within each study reach was based on the Wolman sampling method (Wolman, 1954), which included the following steps:

- In the reach under consideration, a grid system with 100 nodes was established over a length of channel which always included at least one pool-riffle or pool-rapid sequence. (The Wolman sampling areas are indicated in Appendix B.)
- The median diameters of the 100 substrate particles located underneath the nodal points on the grid, were measured.
- Based on the measured diameters, a cumulative frequency curve for each sample was plotted and values of the median diameter (d_{50}) and other percentile values determined. (Cumulative frequency curves of the substrate size distributions within the various study reaches are included as Appendix C.)

Table 4.2 summarizes the data that were collected. Note that, where applicable, average values were calculated for each study reach. (R denotes a rapid/riffle and P a pool, while σ represents the standard deviation of the bed particle size distribution, assumed equal to $0.5 \log (d_{84}/d_{16})$.)

Site	d_{50} (mm)	σ	W_{ch} (m)	D_{ch} (m)	L (m)	Δz (m)	α_r (%)
Whitebridge	200	0.431	12 (R)	1.2 (R)	15	0.45	10.1
			10	1	13	0.34	9.8
			8 (P)	1 (P)			
		Avg.	10	1.1	14	0.40	10
Jonkershoek	215	0.398	8.5 (R)	1.5 (R)	23	0.60	6.4
			9	1.5	27	0.40	5.7
			6.5 (P)	1.6 (P)			
		Avg.	8	1.5	25	0.50	6.0
Vergenoeg	170	0.389	13 (R)	1.3 (R)	50	0.60	3.1
			16	0.9			
			16 (P)	1.8 (P)			
		Avg.	15	1.3	50	0.60	3.1
Vlottenburg	140	0.181	24 (R)	1.7 (R)	95	0.80	2.4
			20	1.6			
			21 (P)	2.5 (P)			
		Avg.	22	1.9	95	0.80	2.4
Smalblaar	240	0.361	12 (R)	1.0 (R)	30	0.48	4.8
			15	1.0	31	0.33	6.9
			15 (P)	0.9 (P)			
		Avg.	14	1.0	30	0.40	5.7
Elands	250	0.301	22 (R)	1.3 (R)	75	0.55	4.1
			25	0.5			
			19 (P)	1.9 (P)			
		Avg.	22	1.2	75	0.55	4.1
Molen. I	130	0.332	29 (R)	1.4 (R)	120	0.40	2.8
			31	0.9			
			30 (P)	2.3 (P)			
		Avg.	30	1.5	120	0.40	2.8
Molen. II	220	0.338	28 (R)	1.2 (R)	75	0.60	4.2
			29	1.0	65	0.70	3.8
			32 (P)	2.3 (P)			
		Avg.	29	1.5	70	0.65	4.1
Berg I	275	0.331	15 (R)	1.1 (R)	55	0.45	7.0
			15	1.2			
			16 (P)	1.5 (P)			
		Avg.	15	1.3	55	0.45	7.0
Berg II	160	0.415	18 (R)	1.3 (R)	50	0.55	3.8
			15	1.2			
			16 (P)	1.8 (P)			
		Avg.	17	1.4	50	0.55	3.8
Berg III	170	0.220	30 (R)	1.3 (R)	110	0.55	2.5
			32	1.3			
			28 (P)	1.5 (P)			
		Avg.	30	1.4	110	0.55	2.5
Berg IV	90	0.239	37 (R)	1.9 (R)	145	0.65	1.6
			33	1.5			
			40 (P)	2.0 (P)			
		Avg.	38	1.8	145	0.65	1.6
Du Toits	150	0.417	14 (R)	0.9 (R)	30	0.40	4.0
			14	0.8			
			15 (P)	0.9 (P)			
		Avg.	14	0.9	30	0.40	4.0

Table 4.2 : Morphological data

4.2.3 Bed form geometry

Step-pool, pool-rapid and pool-riffle structures may be defined by the parameters L , Δz and α_r , as shown in Figure 4.2. In order to investigate the changes in bed form geometry along the length of the channel, the measured values of L , Δz and α_r within each study reach were plotted against the average reach gradient (determined from 1:10 000 orthophotos) as shown in Figure 4.3.

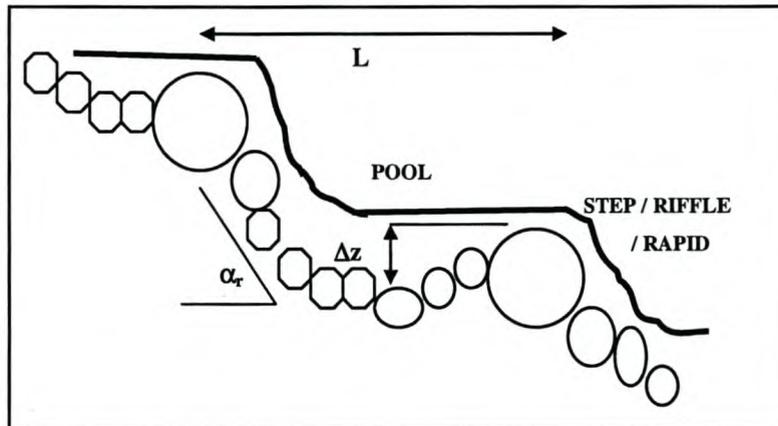


Figure 4.2 : Definition of reach parameters

Figure 4.3 shows that there is a definite correlation between the geometry of macro bed forms and their location within the longitudinal profile. It is found that:

- The local gradients of step, rapid and riffle morphological units increase in direct relation to the average reach gradient.
- The depth of pools tends to be slightly larger at the lower gradient reaches.
- The length of macro scale bed forms decreases with increasing channel gradient.

4.2.4 Channel geometry

A common feature of alluvial rivers with erodible boundaries is their consistency of form in relation to discharge. Channel width and depth, where both are measured at the bankfull level, are useful indices for describing channel form.

With regard to the variation of bankfull width and depth within a pool-riffle or pool-rapid structure itself, Table 4.2 indicates that the bankfull depth within pool sections is almost consistently larger than the bankfull depth within the corresponding riffle or rapid sections. However there does not seem to be any correlation between bankfull width and pool or riffle sections.

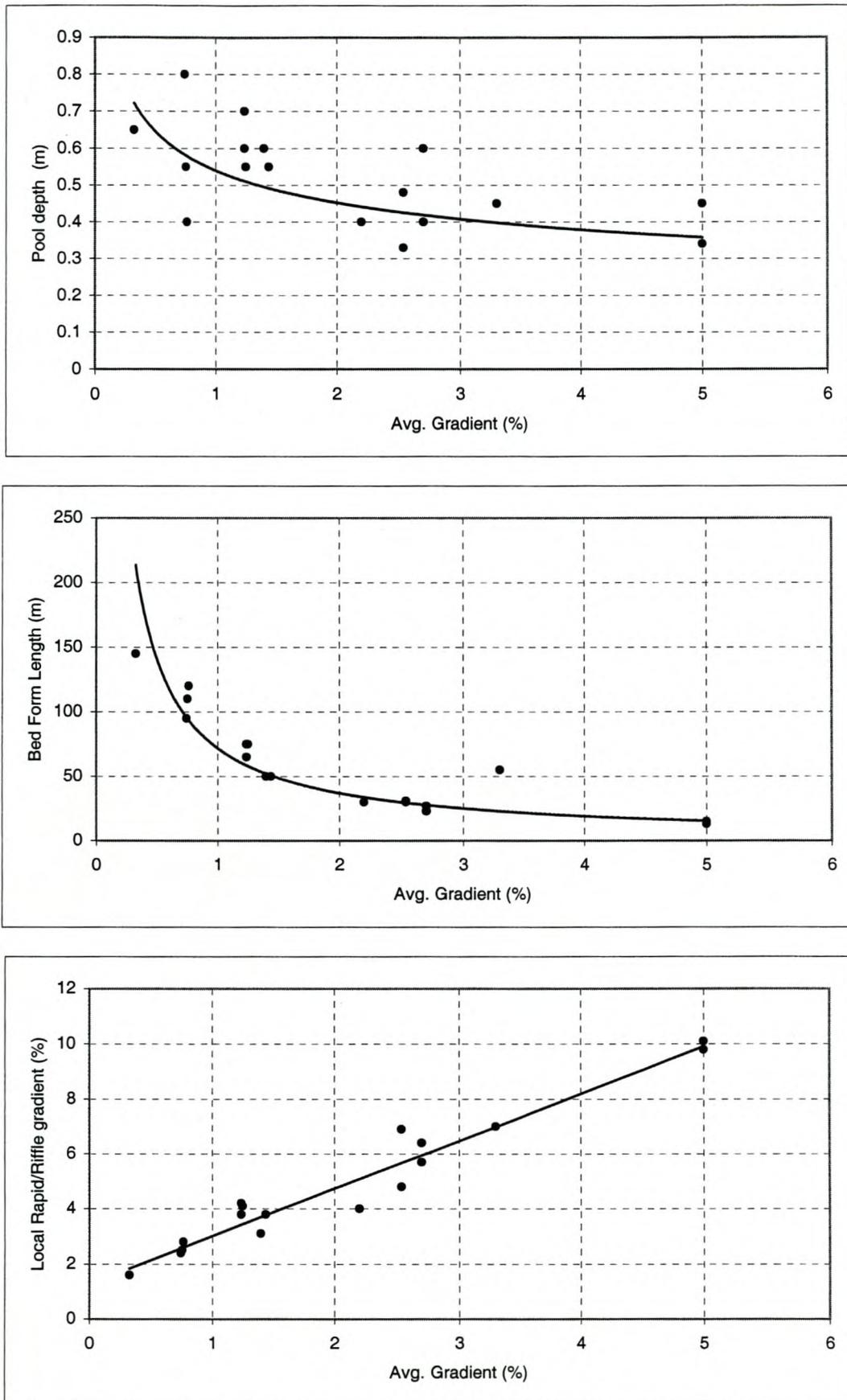


Figure 4.3: Changes in macro scale bed form geometry with average reach gradient

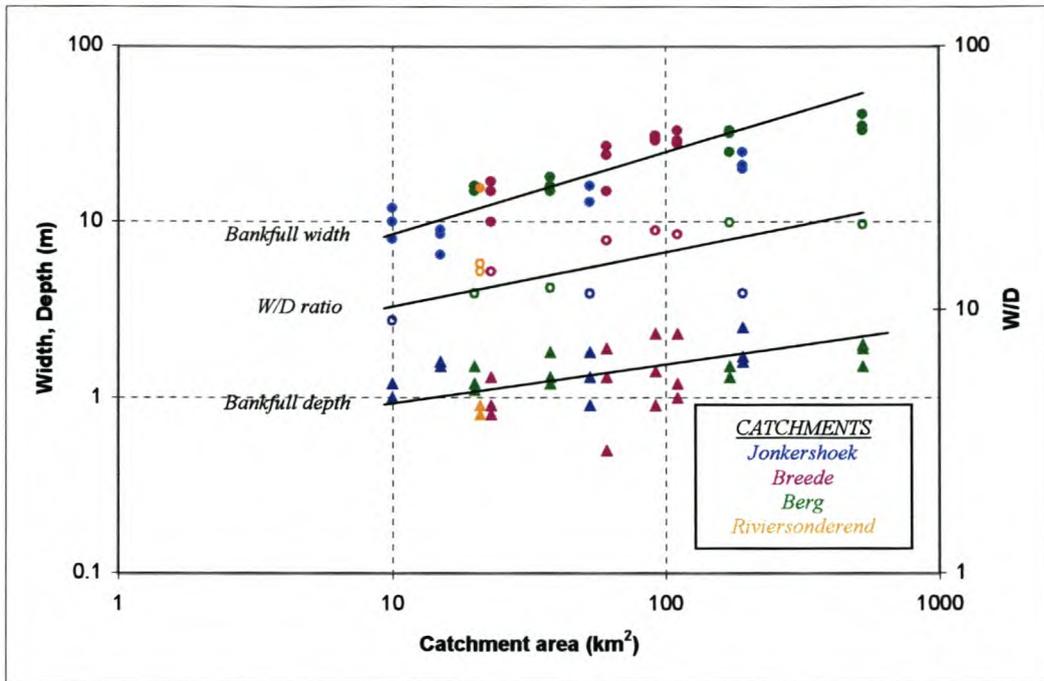


Figure 4.4: Changes in channel geometry with catchment area

Since, in the Western Cape, flood discharge typically increases with distance downstream because of the increasing area of drainage, the catchment area may be used to represent discharge. For the various study reaches, downstream changes in the measured values of bankfull width and depth as well as the reach-averaged ratio of bankfull width to depth in relation to catchment area, are shown in Figure 4.4. (Different catchments are indicated in different colours.) It shows that the magnitude of all three parameters increase in the downstream direction in direct relation to the catchment area. Furthermore it indicates that even though the various study reaches are situated in four different catchments, very similar relationships exist between catchment area and channel geometry and that a case can be made for regional consistency.

4.2.5 Substrate characteristics

The common feature of all the rivers under consideration in this study is the characteristic concave shape of the longitudinal profile in the downstream direction, with the slope decreasing from the upper “eroding” reaches to the lower “depositional” ones. This is typical of many alluvial streams and is associated with both an increase in discharge and a decrease in sediment particle diameter in the downstream direction. Figure 4.5, which indicates the relationship between particle size and average reach gradient within the study reaches, clearly indicates this increase in particle diameter size with channel gradient.

Another common feature of cobble and boulder bed rivers is the wide range of particle sizes within any particular reach. This variability in bed particle size may be represented by the standard deviation. (Bathurst (1978) found that the size distribution of larger sized bed particles is approximately log-normal, in which case the standard deviation may be expressed as $0.5 \log (d_{84}/d_{16})$. This was therefore also assumed to apply to the cobble and boulder bed rivers of the Western Cape.) Figure 4.6, which shows the relationship between the standard deviation in bed particle size and the average reach gradient within the study reaches, suggests that the surface armouring particles become more uniform as one progresses down the river, which may be attributed to the decrease in sediment transport capacity in the downstream direction.

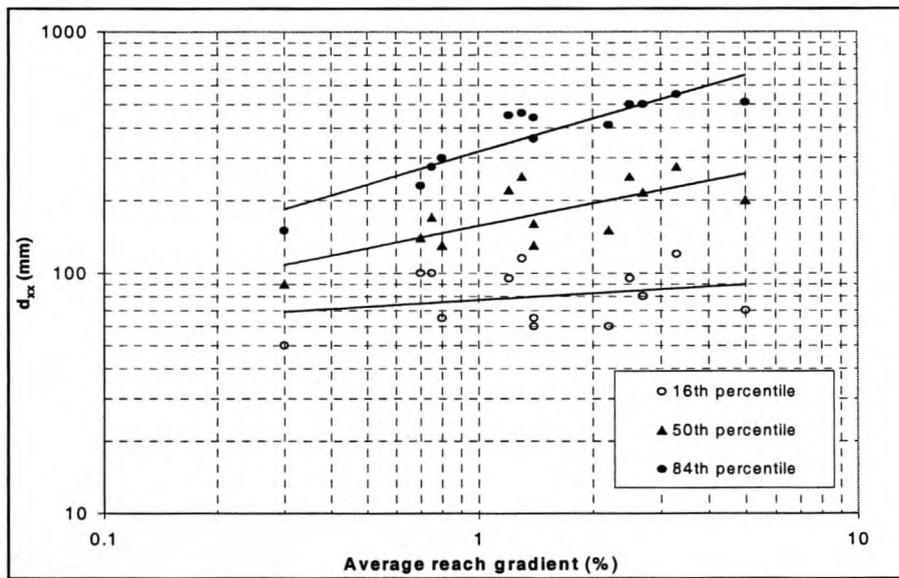


Figure 4.5 : The variation of substrate size with average reach gradient

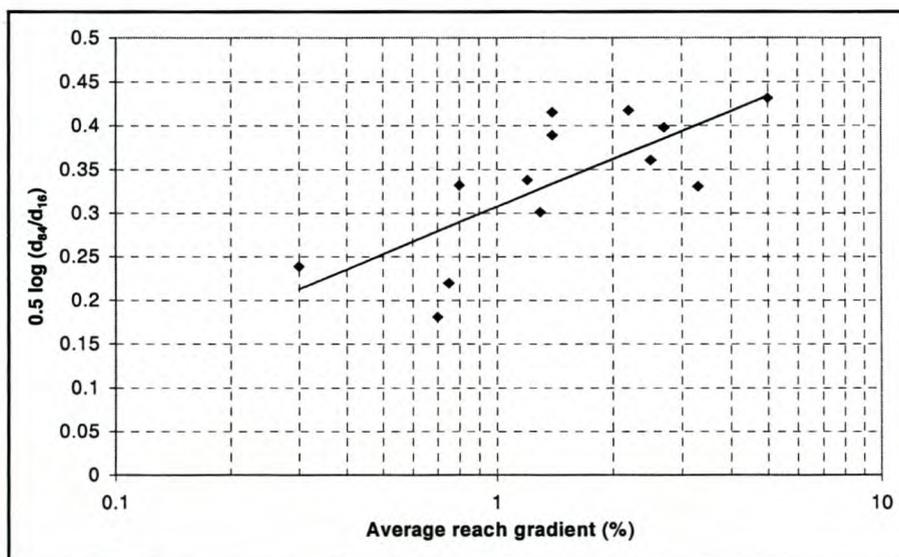


Figure 4.6 : The standard deviation in bed particle size vs. average reach gradient

4.3 The development of a regime model for cobble and boulder bed rivers

4.3.1 Background

Regime theory attempts to establish the relationships that exist within a river channel between a characteristic discharge, sediment characteristics, channel form and channel gradient. Whereas the discharge and sediment characteristics are usually known variables, channel width, depth and gradient need to be determined analytically. Three equations are therefore needed, of which two are generally available, i.e. a flow resistance equation and an equation defining sediment transport characteristics. However, a third equation is usually not readily available. Various approaches to overcome this problem have subsequently been developed. These approaches may be classified into two broad categories, viz. empirical methods and rational (analytical) methods.

Empirical methods rely upon experimental or field data for determining empirical relationships between a characteristic discharge and the variables defining channel dimensions and gradient. Although empirical regime methods are not based on theoretical considerations, the nature of their derivation and their history have resulted in general acceptance of their approximate representation of dominant channel deformation mechanisms (Le Grange, 1994). However, the inadequacy of the empirical methods in explaining the cause of the dynamic adjustment of rivers has prompted the development of the so-called rational regime methods. Most rational methods can be classified as extremal methods, which are motivated by the conviction that a regime channel is formed because a certain physical quantity tends towards a minimum or maximum value (Yalin, 1992). Once the value is reached, the channel is “in regime”. Various physical quantities have been considered which include the minimization of stream power hypothesis (Chang and Hill, 1977; Song and Yang, 1982), the maximum sediment transport rate hypothesis (White *et al.*, 1982), the maximum flow resistance hypothesis (Davies and Sutherland, 1983; Abrahams *et al.*, 1995) and the hypothesis of minimum unit stream power (Yang, 1984).

Since not all discharges have the same capacity to perform work, a critical issue related to regime theory is the identification of a characteristic discharge which “dominates” channel formation and which can be defined either in terms of its hydrological significance (linked to a certain return period; the long term average flow; the mean annual flood; or rare and large floods), its hydraulic significance (bankfull flow or a flow which exceeds the critical shear velocity of the bed sediments) or both. Although the concept that a range of discharges

govern the shape and size of alluvial channels has been established by some researchers (e.g. Harvey (1969) and Knighton (1984)), it is often assumed when describing the regime behaviour of alluvial rivers that the bankfull discharge - the discharge that fills the river channel to its banks and above which spilling into the floodplain occurs - dominates the formation of the channel. This assumption is based on the results of numerous studies on the regime behaviour of alluvial rivers: Dury (1954) and Leopold and Wolman (1957) for example, demonstrated empirically that consistent relationships exist between bankfull discharge and the channel pattern, particularly the spacing of pools and riffles and the wavelength of meander bends. Similarly, Ackers and Charlton (1970), in an attempt to determine the discharge responsible for the meander geometry of sand bed streams, conducted experimental research and confirmed bankfull flow as the “condition which generates the characteristic meander length”. Andrews (1980) found that the bankfull discharge is nearly equal to the effective discharge, defined as “the increment of discharge that transports the largest fraction of the annual sediment load over a period of years”, and which can be assumed to play a dominant role in channel formation. Petts and Foster (1985) state that flow resistance reaches a minimum value at bankfull stage and thus the channel operates most efficiently for the transport of water at this level. The significance of the bankfull discharge is further emphasized when its consistency in terms of frequency of occurrence is considered. Dury (1959, 1961) found that on certain English and American rivers bankfull discharge has a recurrence interval of between one and two years. In 1964, Leopold *et al.* found that recurrence intervals for natural bankfull discharge on nineteen American rivers range from 1 to 4 years and suggested a norm of 2 years. In recently incised channels in Australia, Woodyer (1968) used bench levels corresponding to the present floodplain level to derive average recurrence intervals between 1.2 and 2.7 years, while Andrews (1980) found that the recurrence interval of the effective (bankfull) discharge ranged from 1.2 to 3.3 years.

4.3.2 A review of existing regime models for rivers with large sized bed particles

Extensive research has been conducted to describe the regime behaviour of sand bed rivers and numerous sand bed regime models have been developed, including models that describe the development of small scale bed forms under lower and upper regime conditions. However, it is only relatively recently that attention has been directed towards the regime characteristics of rivers with larger sized bed particles, i.e. gravel, cobble and boulder bed rivers.

Generally, models which describe the regime behaviour of these rivers can be categorized into those that predict channel geometry, i.e. channel width, depth and gradient and those that address the formation of macro scale bed forms.

i. Channel geometry

Since 1960, various equations have been derived for predicting the regime dimensions of stable gravel and cobble bed rivers. Most of these are of the same form and relate channel width (W_{ch}), depth (D_{ch}) and gradient (S_0) to one or more independent variables viz. a characteristic discharge (Q_x), sediment discharge (Q_s) and a characteristic bed particle diameter (d_{xx}). Some equations also accommodate the effect of vegetation type found on the river banks.

Table 4.3 lists some of the existing empirical regime equations for gravel and cobble bed rivers.

Reference	Channel Width (m)	Channel Depth (m)	Channel Gradient (S_0)	d_{50} (mm)
Kellerhals (1967)	$3.26Q_{bf}^{0.50}$	$0.182d_{90}^{-0.12}Q_{bf}^{0.40}$	$0.086d_{90}^{0.92}Q_{bf}^{-0.40}$	-
Hey (1982)	$2.2Q_s^{-0.05}Q_{bf}^{0.54}$	$0.16d_{50}^{-0.15}Q_{bf}^{0.41}$	$0.68Q_s^{0.13}d_{50}^{0.97}Q_{bf}^{-0.53}$	21 – 90
Bray (1982)	$2.08d_{50}^{-0.07}Q_2^{0.53}$	$0.26d_{50}^{-0.025}Q_2^{0.33}$	$0.097d_{50}^{0.586}Q_2^{-0.334}$	19 – 145
<u>Hey & Thorne(1983)</u>				
(Veg. I)	$3.98Q_{bf}^{0.54}Q_s^{-0.01}$	$0.16Q_s^{-0.02}d_{50}^{-0.15}Q_{bf}^{0.39}$	$0.087Q_s^{0.1}d_{50}^{-0.09}Q_{bf}^{-0.43}$	10 – 180
(Veg. II)	$3.08Q_{bf}^{0.54}Q_s^{-0.01}$	$0.19Q_s^{-0.02}d_{50}^{-0.15}Q_{bf}^{0.39}$	$0.087Q_s^{0.1}d_{50}^{-0.09}Q_{bf}^{-0.43}$	10 – 180
(Veg. III)	$2.52Q_{bf}^{0.54}Q_s^{-0.01}$	$0.19Q_s^{-0.02}d_{50}^{-0.15}Q_{bf}^{0.39}$	$0.087Q_s^{0.1}d_{50}^{-0.09}Q_{bf}^{-0.43}$	10 – 180
(Veg. IV)	$2.17Q_{bf}^{0.54}Q_s^{-0.01}$	$0.20Q_s^{-0.02}d_{50}^{-0.15}Q_{bf}^{0.39}$	$0.087Q_s^{0.1}d_{50}^{-0.09}Q_{bf}^{-0.43}$	10 – 180

Q_{bf} : Bankfull discharge; Q_2 : Two year flood discharge; Veg. I: Grassy banks with no trees or shrubs; Veg. II: 1-5% tree/shrub cover; Veg. III: 5-50% tree/shrub cover; Veg. IV: >50% shrub cover or incised into flood plain

Table 4.3 : Empirical regime equations for gravel and cobble bed rivers

To address the local variability in channel geometry due to the presence of pools and riffles, Hey and Thorne (1986) evaluated data from 62 stable gravel and cobble bed rivers (d_{50} between 14 mm and 176 mm). They found that on average at bankfull stage, the channel past riffles is 3.4 % wider and 5 % shallower than the average bankfull width and depth respectively, while the channel past pools is 3 % narrower and 5 % deeper than the average bankfull width and depth respectively.

The earliest rational analyses for describing the regime geometry of river channels with large sized bed particles (> 2 mm) were based on the threshold theory and related the flow conditions and channel form to incipient motion of the bed particles and bank stability (Glover and Florey, 1951; Lane *et al.*, 1959; Li *et al.*, 1976). In 1980, Chang developed a rational regime model for gravel and cobble bed streams based on the concept of minimum stream power. He reasoned that for an alluvial channel, the necessary condition of equilibrium occurs when the stream power per unit channel length is a minimum. He subsequently derived the following equations describing the regime channel width, depth and gradient of gravel and cobble bed rivers ($16\text{mm} < d_{50} < 265\text{mm}$): (units used are feet and cubic feet per second)

$$W_{\text{ch}} = \left[1.905 + 0.249 \left(\ln \frac{0.001065 d_{50}^{1.15}}{S_0 Q_{\text{bf}}^{0.42}} \right)^2 \right] Q_{\text{bf}}^{0.47} \quad (4.1)$$

$$D_{\text{ch}} = \left[0.2077 + 0.0418 \ln \left(\frac{0.000442 d_{50}^{1.15}}{S_0 Q_{\text{bf}}^{0.42}} \right) \right] Q_{\text{bf}}^{0.42} \quad (4.2)$$

$$S_0 = 0.000442 d_{50}^{1.15} Q_{\text{bf}}^{-0.42} \quad (4.3)$$

Another example of a rational regime model for describing the geometry of gravel and cobble bed rivers, is that of Yalin (1992), who used a dimensional formulation of the regime channel to show that the Froude number is the only parameter, apart from channel width, depth and gradient, which varies during channel deformation. He concluded that the Froude number tends towards an extremal (minimum) value during regime channel formation and in conjunction with the Chézy equation and by setting the critical shear velocity equal to $0.90 d_{50}^{0.5}$, established the following dimensional, non-homogeneous relationships which were calibrated with experimental data from rivers ($14\text{mm} < d_{50} < 190\text{mm}$):

$$W_{\text{ch}} = 1.50 d_{50}^{-0.25} Q_{\text{bf}}^{0.50} \quad (4.4)$$

$$D_{\text{ch}} = 0.15 d_{50}^{-0.07} Q_{\text{bf}}^{0.43} \quad (4.5)$$

$$S_0 = 0.55 d_{50}^{1.07} Q_{\text{bf}}^{-0.43} \quad (4.6)$$

The derivations of all of the above equations were either restricted to straight channels with plane beds or defined reach averaged values of channel width, depth and gradient, with no account being taken of the pool-riffle variability characteristic of these rivers.

ii. Macro scale bed deformation

Macro scale bed deformation in gravel, cobble and boulder bed rivers typically refers to the formation of pool-riffle or pool-rapid structures. These bed forms have been described empirically by Leopold *et al.* (1964) and Hey and Thorne (1986), who found that riffle spacing in gravel bed rivers is usually between 5 and 7 times the bankfull width. However, their findings were mostly based on observations in lower river reaches, which displayed a high degree of sinuosity, and were not representative of pool-riffle or pool-rapid sequences characteristic of the middle and upper reaches.

Three theories which have been proposed towards a fundamental understanding of the physical processes controlling macro scale bed deformation in gravel and cobble bed rivers are the antidune theory, dispersion and sorting theory and velocity reversal theory.

The antidune theory assumes that the formation of macro scale bed forms in gravel, cobble and boulder bed rivers corresponds to the formation of antidunes in sand bed rivers, where the wave form of the bed is in phase with the form of a standing wave on the water surface. Shaw and Kellerhals (1977) used equations developed by Kennedy (1961), Reynolds (1965) and Parker (1975), which link the dimensions of antidunes in sand with the corresponding flow parameters, and found a similarity between bed forms observed in a gravel bed river and gravel antidunes developed in the laboratory. However, in both cases (the river and laboratory) the bed material was fairly uniform and not representative of typical heterogeneous bed material found in cobble and boulder bed rivers. Furthermore, the bedforms, which they described as “transverse ridges more or less evenly spaced in the direction of flow with crests 2 m long and a crest spacing of 2 to 3 m”, do not resemble typical pool-riffle or pool-rapid structures. Whittaker and Jaeggi (1982) and Chin (1999) applied the antidune theory to step-pool systems. They found that although the deformation process leading an initial plane bed to step-pool formation is similar to that which produces antidunes, effects due to the heterogeneity of the bed sediments disturb the regularity of the process. They concluded that true antidune wave trains may apply only in some cases under natural conditions, such as along the more gentle reaches of the channel where bed sediments are more uniform and bedforms are more adjustable and easily submerged, but are difficult to obtain in steeper channels dominated by larger roughness elements.

The second theory, the so-called dispersion and sorting theory of Yang (1971), applies the entropy concept introduced by Leopold and Langbein (1962) to a stream system which leads to the law of least time rate of energy expenditure. This law states that “during the evolution

towards its equilibrium condition, a natural stream chooses its course of flow so that the time rate of potential energy expenditure per unit mass of water along its course, is a minimum". In combination with the continuity principle and a resistance equation, Yang (1971) developed a conceptual model which proved that when the discharge, lateral geometry and other constraints are the same for all possible courses between two points in space, the course of uniform depth and slope is the course of maximum time rate of potential energy expenditure per unit mass of water. Subsequently, this course will be avoided by natural streams and as a result, pools and riffles will be formed. The actual formation of riffles and pools, Yang ascribed to pressure differences between the riffle and pool areas, which will increase bed elevation at the riffles as coarse materials migrate to the surface due to a grain dispersion process as reported by Bagnold (1954). With reference to this proposed formation process, Whittaker and Jaeggi (1982) state that "for these effects to occur, the whole bed must be in a state of general shearing motion to a considerable depth, which in a gravel river, would only occur under debris flow conditions". Furthermore, Yalin's proposed mechanism for the actual formation of pool-riffle structures has never been verified with field or laboratory experiments and no evidence for the proposed pressure differences between pools and riffles exist.

The third theory, which is known as the velocity reversal theory, was developed by Keller (1971). He measured velocities in pools and riffles and found that, with increasing discharge, there is a greater increase in velocity in the pools than along the riffles. From extrapolation of his data, he concluded that there would be a velocity reversal at high flows, i.e. the velocity in pools will exceed the velocity in riffles, which would move coarse grains from a riffle through a downstream pool to be deposited in the next riffle, thus maintaining the feature. Although the theory appears in standard texts (Richards, 1982; Gordon *et al.*, 1992) and underpins various conceptual models related to the pool-riffle structure (e.g. Keller and Melhorn, 1974; Andrews, 1982; Lisle, 1979; Sidle, 1988), convincing evidence for the ubiquitous occurrence of such a reversal in a range of channel geometries is currently unavailable (Carling, 1991). Carling (1991), who reviewed the theory as well as attempts to validate it, concluded that, based on continuity considerations, riffles need to be considerably wider than pools for a reversal in the velocity to occur under conditions of high stage and stable morphology. Referring to studies which demonstrate a velocity and shear stress reversal in the East Fork River in England (Andrews, 1979; Lisle, 1979), Carling states that the observed velocity reversal can be ascribed to the unique fact that the riffle where the reversal occurred was 74 % wider than the pool at bankfull stage! Furthermore, based on observations of the hydraulic geometry of stable pool-riffle sequences in the river Severn in England, Carling found that neither the sectionally-averaged velocity, nor the near-bed shear

velocity was sensibly greater in the pools than over the riffles during bankfull or near bankfull flow. Another objection to the velocity reversal theory is made by Bhowmik and Demissie (1982), who argue that, because of the supposedly higher shear stresses in pools during higher flows, the lag sediments in the bottom of pools should be coarser than the riffle sediments – a statement which is in conflict with numerous observations that material in riffles is generally coarser than that in pools (Hirsch and Abrahams, 1981; Milne, 1982). Further criticism of the velocity reversal theory is made by Whittaker and Jaeggi (1982), who state that the proposed mechanism is essentially one of maintenance of pool-riffle bed forms and fail to explain the development thereof.

4.3.3 An analytical approach towards the development of a model for predicting the regime characteristics of cobble and boulder bed rivers

The above review of existing regime models for rivers with large sized bed particles revealed that none of these models provide a complete, theoretically based description of the regime behaviour of cobble and boulder bed rivers in terms of those attributes which are important for ecosystem functioning, viz. substrate characteristics, pool depths, the local gradients of riffles and rapids and the spacing or length of bed forms. Whereas the equations which relate a characteristic discharge to channel geometry only provide information on channel width, depth and average gradient, the proposed theories on macro scale bed deformation are essentially conceptual models describing the process of bed deformation without providing quantifiable relationships between discharge and channel and bed form geometry. In order to predict the morphological impacts associated with a modified flow regime, a complete morphological description of the mechanisms controlling channel deformation in these rivers is required. Consequently, a theoretically based regime model which addresses the regime behaviour of cobble and boulder bed rivers is developed in this section.

i. Hypothesis

Cobble and boulder bed reaches are typically characterized by relatively steep gradients. During higher flows, this results in extremely high shear velocities along the river bed which may lead to large scale erosion. Consistent with theories by Keller and Melhorn (1978), Leopold *et al.* (1964) and Yang (1971), it is therefore hypothesized that the formation of pools and riffles is essentially a mechanism of self-adjustment by the stream system towards obtaining dynamic equilibrium. This is achieved by reducing the erosive capacity of the river through the effective dissipation of excess energy. By transforming the bed profile into a series of macro scale bed

forms, each of which acts as a natural control structure, the water is forced through critical depth with a subsequent hydraulic jump downstream. Furthermore, it is hypothesized that during this bed deformation process the river is removing smaller bed particles from the river bed and in so doing increases the effective absolute bed roughness, which leads to a reduction in the amount of unit turbulent stream power applied along the river bed as well as to a reduction in the sediment transport capacity of the river (Rooseboom, 1974). Channel and bed deformation therefore continues until, on average along the river bed, the rate of deposition of particles equals the rate of resuspension. At this point, in similar fashion to a sand bed river under conditions of dynamic equilibrium (Rooseboom and Le Grange, 2000), it follows that individual bed particles are continually crossing the movement threshold, i.e. critical conditions in terms of sediment movement prevail.

With reference to Figure 4.7, which displays a schematic representation of the hydraulic conditions assumed to prevail at the channel forming discharge, the hypothesis therefore states that at each start of a riffle or rapid a control section exists where the energy level is minimized and the flow depth is critical. After passing the control section, the water accelerates down the slope to form a so-called riffle or rapid. The momentum of the fast flowing water is checked by the slower moving water in the pool, with the water level in the pool being determined by the energy level at the start of the next (downstream) riffle or rapid and so on. The flow depth in the pool is therefore linked directly to the flow velocity of the water that enters the pool, which in turn, is determined by the channel width, local gradient and bed roughness of the riffle or rapid at a specific discharge. When equilibrium conditions prevail, the flow depth of the water in the pool is just sufficient to dissipate the excess energy without net removal of sediment from the bed, while the flow depth along the riffle or rapid is just sufficient to equal the momentum of the water in the pool, without net removal of sediment from the riffle area. At a characteristic discharge and bed particle size distribution, the river thus adjusts its width, average gradient, local gradient of the riffle or rapid and absolute bed roughness until equilibrium exists in terms of the power applied along the bed and the power required to suspend bed particles as well as in terms of momentum exchange between the accelerating water along the riffle and the slower moving water in the pool.

The hypothesis therefore implies that consistent relationships should exist between channel geometry, the configuration of the macro scale bed forms, the critical shear velocity of bed particles (related to bed particle size) and a characteristic discharge, i.e. the channel forming discharge.

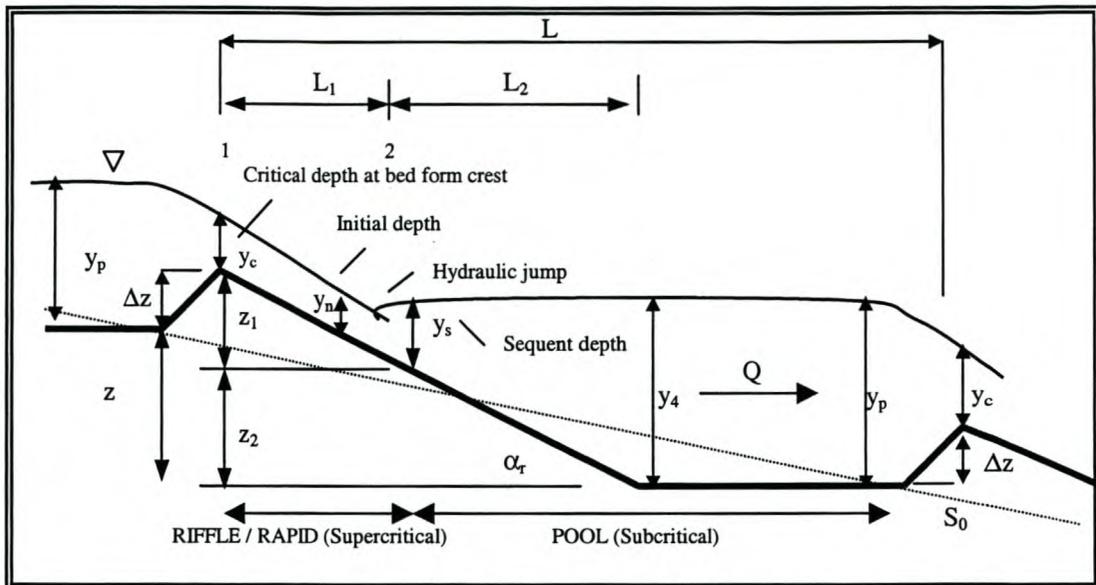


Figure 4.7 : Schematic representation of the hydraulic conditions assumed to prevail during dynamic equilibrium in a deformed cobble and boulder bed river (not to scale)

ii. The relationship between bed configuration and discharge

As a first step towards the development of the regime model, equations defining the relationship between discharge and the bed form geometry as depicted in Figure 4.7 were derived. Through application of the energy and momentum principles and based on certain assumptions, equations 4.7 to 4.16 were derived:

Applying the energy principle between sections 1 and 2 and assuming that a uniform flow depth (y_n) prevails at the start of the hydraulic jump, it follows that

$$z_1 + y_c + V_c^2/2g = y_n + V_n^2/2g + h_f \tag{4.7}$$

- with z_1 : bed elevation above datum (m)
- y_c : critical flow depth at the bed form crest (m)
- y_n : uniform flow depth along the riffle (m)
- V_c : average flow velocity at bed form crest ($= Q / (W_{ch}y_c)$) (m/s)
- V_n : average flow velocity along riffle/rapid at depth y_n ($= Q / (W_{ch}y_n)$) (m/s)
- h_f : friction losses along the riffle/rapid (m)

However,

$$\begin{aligned} h_f &= S_f L_1 \\ &= S_f z_1 / \alpha_r \\ &\approx \frac{z_1}{2\alpha_r} \left[\frac{V_c^2 f}{8gy_c} + \frac{V_n^2 f}{8gy_n} \right] \end{aligned} \quad (4.8)$$

with S_f : energy gradient along riffle / rapid
 g : gravitational acceleration (m/s^2)
 f : Darcy-Weisbach resistance coefficient

Therefore, from equations 4.7 and 4.8:

$$z_1 = \frac{\left(y_n + \frac{V_n^2}{2g} - y_c - \frac{V_c^2}{2g} \right)}{1 - \frac{0.5}{\alpha_r} \left(\frac{V_c^2 f}{8gy_c} + \frac{V_n^2 f}{8gy_n} \right)} \quad (4.9)$$

Furthermore, from the momentum principle, it can be shown that

$$y_s = \frac{y_n}{2} \left[\sqrt{1 + \frac{8V_n^2}{gy_n}} - 1 \right] \quad (4.10)$$

Based on the findings of Bradley and Peterka (1957), it is assumed that in the case of a hydraulic jump on a sloped surface, the equivalent sequent flow depth on a horizontal surface may be defined by (refer to Figure 4.7)

$$y_4 = 1.3 y_s \quad (4.11)$$

and furthermore that

$$L_2 = 0.82 y_s (\alpha_r)^{-0.78} \quad (4.12)$$

Next, assuming that energy losses within the pool are negligible, $y_p \approx y_4$. Therefore, if it is assumed that at the channel forming discharge the height (Δz) of the control section is just sufficient to force the water through critical depth at the bed form crest, it follows that

$$\Delta z = (y_4 + V_4^2/2g - y_c - V_c^2/2g) \quad (4.13)$$

Furthermore, from Figure 4.7

$$z = z_1 + z_2 - \Delta z \quad (4.14)$$

and

$$z_2 = L_2 \alpha_r \quad (4.15)$$

Assuming that a consistent relationship will exist between the height of a bed form (z), the length of the bed form (L) and the average gradient of the reach (S_0), i.e. that the spacing of bed forms fits in with the channel topography, the length of a bed form (L) may be calculated from the channel topography as follows:

$$L = z / S_0 \quad (4.16)$$

Equations 4.7 to 4.16 can therefore be used to define the relationship between channel geometry, bed form geometry and the discharge assumed to prevail during dynamic equilibrium conditions in a deformed cobble and boulder bed river and enables the hypothesis on the formation of macro scale bed forms in cobble and boulder bed rivers to be verified.

iii. Verification

The morphological data that were collected in the study reaches (listed in Tables 4.1 and 4.2) were used to verify the above hypothesis on channel deformation. (Some of the study reaches in the mountain and mountain transitional zones - Whitebridge, Jonkershoek, Smalblaar and Berg I - which are characterized by large clasts of rocky material which have found their way into the river by falling down from steep banks and cliffs alongside the river, were not included in this analysis. The Du Toits and Berg III reaches were also excluded due to the fact that they were situated close to a weir and bridge crossing respectively.) It was assumed that the observed channel and bed form geometry in each reach correspond to the channel and bed form geometry which prevail during channel forming (i.e. dynamic equilibrium) conditions. Through application of equations 4.7 to 4.16, a characteristic discharge (Q_c), which by implication equals the channel forming discharge, was then calculated based on the observed channel and bed form geometry and substrate size distribution within each reach. This involved a repetitive process whereby the measured values of channel width, the local gradient of the riffle or rapid and the average reach gradient were substituted into the equations and a discharge selected and refined until the calculated value of the bed form length equalled the measured value thereof. (A sample calculation is included as Appendix D

- In order to make provision for the effects of large scale roughness, the value of the Darcy Weisbach resistance coefficient in equations 4.8 and 4.9 was calculated from the Griffiths (1981) equation, which, as shown in Chapter 6, has been found to provide a good estimate of flow resistance in the cobble and boulder bed rivers of the Western Cape.) The calculated discharges were then evaluated in terms of their hydrological significance, their relationship to critical conditions for sediment entrainment and their relationship to bankfull discharges.

Hydrological significance

For those study reaches where observed peak flow records at nearby gauging weirs were available, the calculated values of Q_c were compared with the results of an annual flood frequency analysis (Appendix E). The results (Table 4.4) confirm that within all these reaches, Q_c corresponds to floods with a recurrence interval of between 1 and 4 years. The significance of this is further emphasized when it is considered that floods with a recurrence interval of between 1 and 4 years have been shown to be equivalent to the bankfull or “dominant” discharge (refer to section 4.3.1).

Reach (gauging station)	Q_c (m ³ /s)	Recurrence interval (Years)
Elands (H1H017/33)	203	3.84
Molenaars I (H1H018)	281	1.43
Molenaars II (H1H018)	222	1.20
Berg IV (G1H020)	243	1.53

Table 4.4: The recurrence interval of the characteristic discharge (Q_c)

Critical conditions for sediment movement

In section 4.3.3 (i) it was hypothesized that the regime or equilibrium condition in a deformed cobble and boulder bed river is characterized by critical conditions in terms of sediment movement, which implies that at the characteristic discharge (Q_c) calculated above, critical conditions for sediment movement prevails. With the aid of field data collected in the study reaches, this was verified.

Because of the variation in hydraulic conditions during macro scale bed deformation (as depicted in Figure 4.7) different areas along the river bed will be characterized by different shear velocities during the channel deformation process. Three distinct areas within the

deformed bed profile were therefore identified where the relationship between actual shear velocity and critical shear velocity was investigated, viz. at the bed form crest, along the riffle or rapid and within the pool.

Within each of these areas, the shear velocity (V^*) was calculated from equation 4.17:

$$V^* = \sqrt{gyS_f} \quad (4.17)$$

with y : flow depth
 S_f : energy gradient = $(V^2 f)/(8gR)$
 V : average flow velocity = $Q_c / (W_{ch}y)$
 R : hydraulic radius (assumed equal to y)
 f : Darcy Weisbach coefficient

Assuming a rectangular channel, the critical flow depth at the bed form crest was calculated from equation 4.18:

$$y_c = \sqrt[3]{\frac{Q_c^2}{gW_{ch}^2}} \quad (4.18)$$

Along the riffle (or rapid), the flow depth was assumed equal to the uniform flow depth (y_n), which was calculated by solving equation 4.19:

$$Q_c = y_n W_{ch} \sqrt{\frac{8gy_n S_f}{f}} \quad (4.19)$$

Within the pool, the flow depth (y_p) was calculated by solving equation 4.20

$$y_p + \frac{\left(\frac{Q_c}{W_{ch}y_p}\right)^2}{2g} = \Delta z + 1.5y_c \quad (4.20)$$

Critical shear velocities (V^*_{cr}) were calculated from equation 4.21 (Rooseboom, 1974), which represents the initiation of movement of a cohesionless sediment particle under uniform, turbulent boundary conditions.

$$V^*_{cr} = 0.12V_{ss} \quad (4.21)$$

with V_{ss} representing the settling velocity of a sediment particle under turbulent flow conditions, i.e.

$$V_{ss} = \sqrt{\frac{4(\rho_s - \rho)gd_x}{3\rho C_d}} \quad (4.22)$$

with ρ_s : substrate particle density (kg/m^3) - set equal to 2670 kg/m^3 as determined from actual river cobbles
 ρ : water density (kg/m^3)
 d_x : characteristic particle diameter (m)
 C_d : drag coefficient

If the value of C_d in equation 4.22 is assumed constant ($= 0.4$), which is true for larger diameters ($d > 6\text{mm}$), it follows from equations 4.21 and 4.22 that the critical shear velocity of a bed particle (V^*_{cr}) is directly related to the square root of its diameter (\sqrt{d}). In terms of Froude scale laws, it therefore follows that for larger particles the critical particle diameter varies directly according to the length scale (which is not the case for small particles).

Furthermore, the differing contributions of the gravitational force component in the entrainment of bed particles had to be taken into account. Within pools, the gravitational force works directly against the lifting forces which act upon the bed particles. At the downstream side of the bed form crest, gravity begins to assist in the entrainment of bed sediments, whilst further downstream, on the riffle/rapid slope, the full gravitational force component comes into play. In order to make provision for the gravity effect along the riffle/rapid, which may be significant due to the steep gradient and which may contribute to particle mobility in this area, the critical shear velocity along the riffle/rapid was reduced with a coefficient calculated from equation 4.23 (Armitage, 2002):

$$C_m = \sqrt{\cos \alpha_r \left(1 - \frac{\tan \alpha_r}{\tan \phi} \right)} \quad (4.23)$$

with C_m : mobility coefficient
 ϕ : angle of repose ($\approx 40^\circ$ for large bed particles (Van Rijn, 1993))

Figure 4.8, which displays the degree of correlation between the shear velocities and critical shear velocities related to different percentile values of the overall substrate size distribution within a reach, shows that at different locations within the various study reaches, the shear

velocity at a discharge equal to Q_c consistently approximates the critical shear velocity for a certain relative particle size ($\pm d_{95}$ in rapids or riffles, $\pm d_{84}$ at the crest and $\pm d_{35}$ in pools). This confirms the hypothesis that during macro scale bed deformation, critical conditions in terms of sediment movement may be assumed to prevail along the bed. The different values of critical shear velocity at the different locations, which reflect the different particle sizes that are in equilibrium in terms of the effective shear velocity (related to the power applied along the river bed) and the critical shear velocity (according to the power required to suspend the bed particles), may be attributed to the differences in armouring effects due to the variation in shear velocities. Figure 4.8 confirms that consistent relationships exist between bed form geometry and the bed particle size distribution at the characteristic discharge and implies that at a discharge of Q_c , the river is able to remove larger particles from the riffle areas (up to the 95th percentile value) than from the pool areas (up to the 35th percentile value), i.e. the bed particles along riffles should be coarser than in pools. This is consistent with a statement by Leopold *et al.* (1964) that “higher shear velocities at the riffles cause finer material to be washed away and deposited in the pools downstream, which leads to only the coarser particles which can withstand the higher bed shear stress remaining at the riffle”. This is supported by observations within the study reaches as well as by numerous other observations that rapids and riffles exhibit relatively coarser bed particles than pools (Leopold *et al.*, 1964; Bhowmik and Demissie, 1982; Keller, 1971; Church, 1972; Lisle, 1979; Richards, 1976; Hirsch and Abrahams, 1981; Milne, 1982). Hirsch and Abrahams (1981) also found that the bed sediments in riffle sections are not only coarser than in pools, but also better sorted – a finding which is expected when one considers that smaller sized particles are washed away from the riffles, leaving larger and more uniform particles behind. Finally, the above results, which show that at the channel forming discharge the river is able to move up to the d_{95} percentile value of the bed particle size, are in agreement with the results of Andrews (1983), who found that particles as large as the d_{90} of the substrate size distribution were entrained by the bankfull discharge in nine Colorado rivers. The fact that data from different sites conform to the same patterns in Figure 4.8 in terms of shear velocities and critical particle diameters also confirms that Froude similarity laws hold and that critical particle sizes vary according to the linear (undistorted) scale.

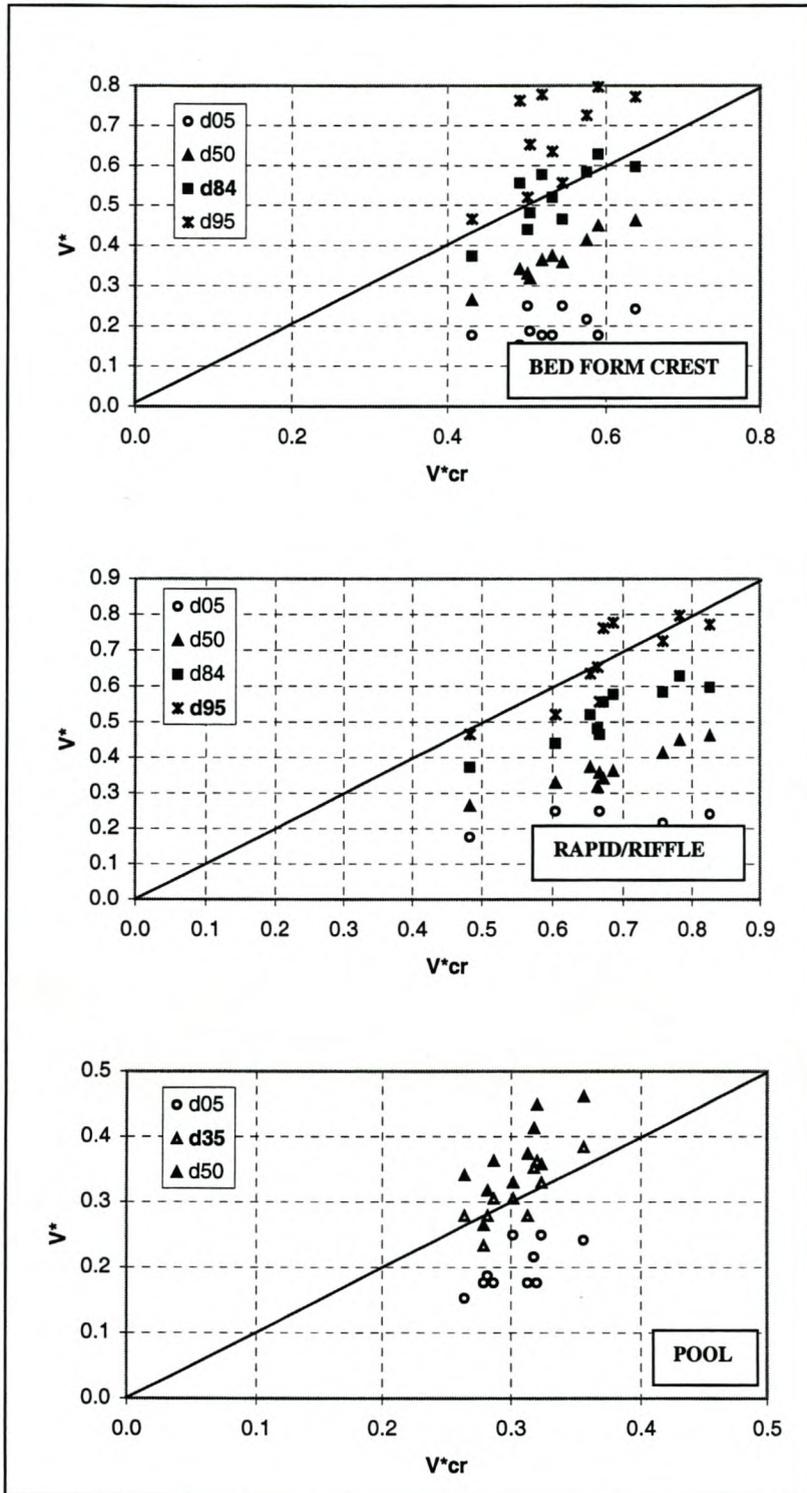


Figure 4.8 : The relationship between shear velocity (V^*) and critical shear velocity (V^*_{cr}) at a discharge equal to Q_c

Channel geometry

As discussed in section 4.3.1, bankfull discharge (Q_{bf}) is often considered to be responsible for determining the form of alluvial channels. Consequently it may be assumed that the characteristic discharge (Q_c) which was calculated above and which, in accordance with the adopted hypothesis is responsible for channel deformation, should approximate the bankfull discharge. As no measured data on observed bankfull discharges in the study reaches were available, an indirect approach was followed to investigate the relationship between Q_c and Q_{bf} :

Q_c was calculated based on measured values of bankfull width, the average reach gradient, the local riffle gradient, the bed form length and the substrate size distribution within each study reach (refer to Appendix D). Therefore, since the measured channel depth at bankfull level was not used during the calculation of Q_c , a comparison of the observed channel depth at bankfull level and the calculated flow depth associated with Q_c provides an independent verification of the relationship between Q_c and Q_{bf} (Figure 4.9). It shows that the observed bankfull depths at the riffle or rapid sections are comparable to the calculated flow depths along the riffles (y_n) at Q_c . However, the calculated pool depths (y_p) at a discharge of Q_c are substantially larger than the observed values. A possible explanation for this might be that the large flow depths calculated in the pool areas only occur under dynamic equilibrium conditions. Then, as the flow rate and sediment transport capacity of the river decreases, smaller bed particles are deposited in the pool areas, which leads to shallower pools being observed during low flow conditions.

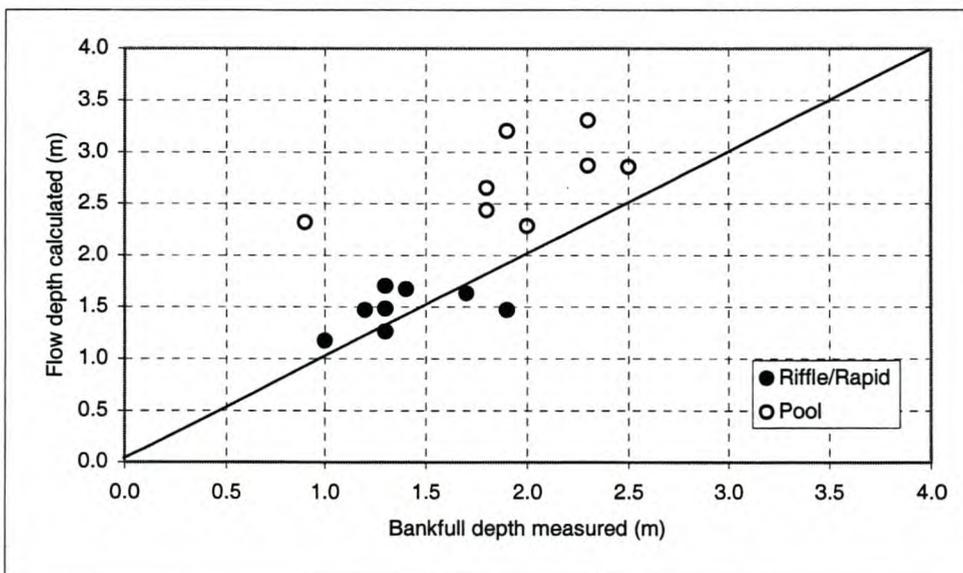


Figure 4.9 : The relationship between observed bankfull depth and flow depths associated with the characteristic discharge

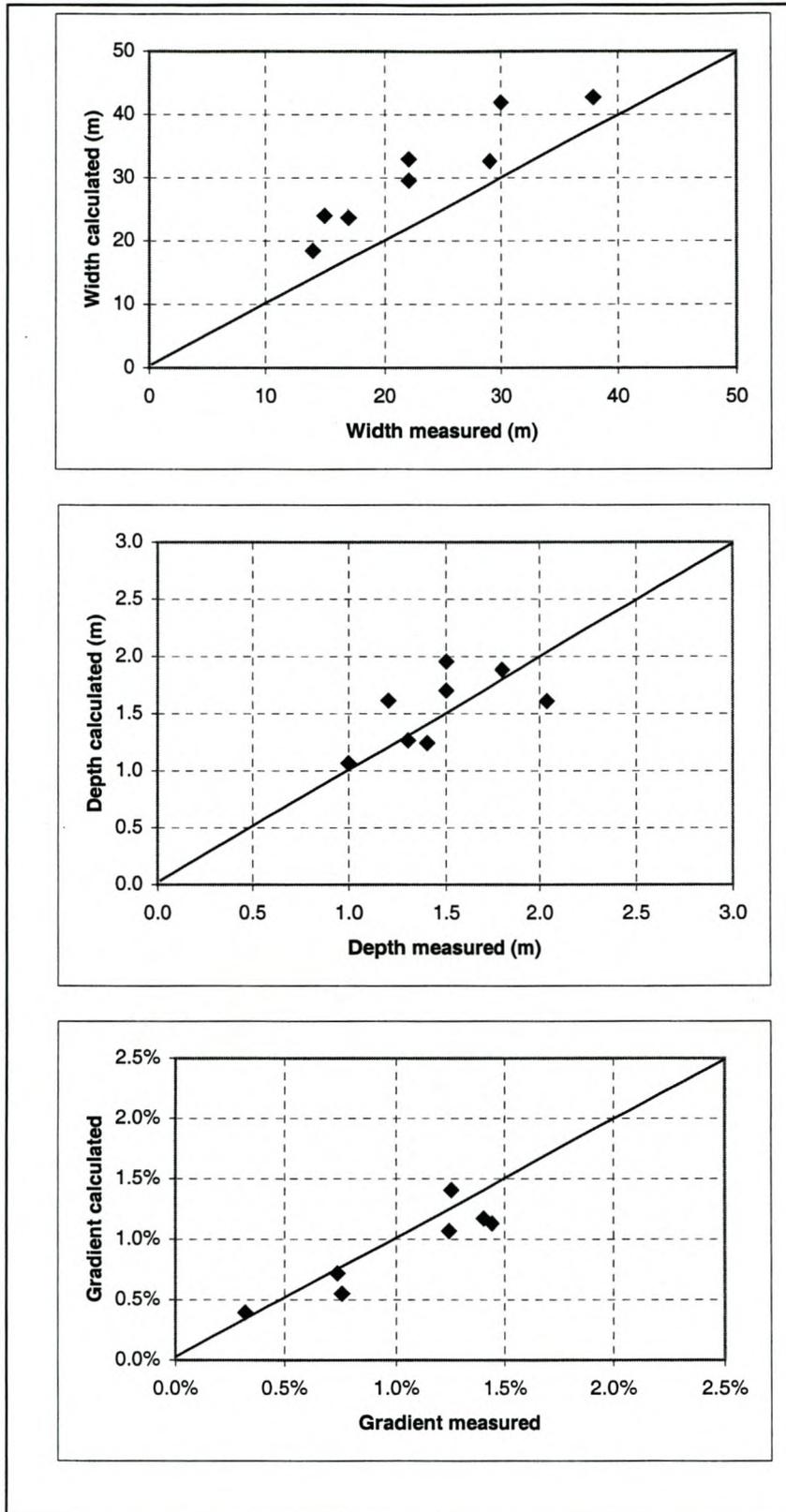


Figure 4.10: A comparison of the observed channel geometry with the channel geometry as predicted by Yalin's (1992) regime equations at a discharge equal to Q_c

Another methodology which was employed for investigating the relationship between Q_c and Q_{bf} , involved an estimation of the channel geometry from existing regime equations at a discharge equal to Q_c , and a comparison of the calculated values of channel width, depth and gradient with observed values. The equations (4.4 to 4.6) developed by Yalin (1992), which can be regarded as “averages” of various regime equations, were used for this purpose. Figure 4.10 displays the degree of correspondence between the calculated and observed values of channel width, depth and gradient and shows that relatively good correlation exists, which, indirectly, therefore implies that there is good correlation between Q_c and Q_{bf} .

4.4 Conclusion

In the first part of this Chapter, the morphological characteristics of cobble and boulder bed rivers in the Western Cape were discussed. It was found that in the concave-shaped, alluvial rivers of the Western Cape, cobble and boulder bed reaches typically occur in the mountain transitional zone, as well as in the upper and lower foothill zone. These reaches are characterized by four dominant types of morphological units, viz. pools, plane beds, riffles and rapids. Furthermore, it was shown that consistent relationships exist between channel geometry, catchment area (assumed to be related to discharge) and bed form geometry within these rivers.

A review of existing regime models for rivers with large sized bed particles revealed that none of these models provides a complete morphological description in terms of those attributes which are important for ecosystem functioning, viz. substrate characteristics, pool depth, the local gradient of riffles or rapids and the spacing or frequency of bed forms. In order to predict the morphological impacts associated with a modified flow regime, a theoretically based regime model was developed, which addresses the regime behaviour of cobble and boulder bed rivers. The model, which can be classified as a rational regime model, is based on the hypothesis that the formation of pools and riffles is essentially a mechanism of self-adjustment by the stream system towards obtaining dynamic equilibrium, i.e. at a characteristic discharge and bed particle size distribution, the river adjusts its width, average gradient, local gradient of the riffle or rapid and absolute bed roughness until equilibrium exists in terms of the power applied along the bed and the power required to suspend bed particles as well as in terms of momentum exchange between the accelerating water along the riffle and the slower moving water in the pool. This behaviour is comparable to the formation of bed forms in sand bed rivers, where flow resistance is increased in order

to dissipate excess energy and to limit the scouring of sand from the river bed (Rooseboom and Le Grange, 2000).

With the aid of field data the model was verified in typical cobble and boulder bed rivers in the Western Cape by calculating a characteristic discharge, which, in accordance with the hypothesis, is equivalent to the channel forming discharge. This discharge was then evaluated in terms of its hydrological significance, its ability to entrain bed particles and its relationship to bankfull discharge. It was found that:

- the discharge consistently corresponds to a flood with a return period of between 1 and 4 years
- at different locations along the bed profile, the shear velocity at this discharge consistently approximates the critical shear velocity for a certain particle size as a percentile value of the overall substrate size distribution within the reach ($\pm d_{95}$ in rapids or riffles, $\pm d_{84}$ at the crest and $\pm d_{35}$ in pools)
- based on indirect methods, the discharge shows good correlation with bankfull discharge

The regime model which has been developed therefore defines the relationship between a characteristic (channel forming) discharge and channel morphology (channel geometry, bed form geometry and substrate size) in cobble and boulder bed rivers. Based on the calibration of the model in cobble and boulder bed rivers of the Western Cape it was found that the characteristic discharge has a return period of between 1 and 4 years and approximates the bankfull discharge. Furthermore, at different locations within the deformed bed profile, consistent relationships were observed between the shear velocity at this characteristic discharge and the critical shear velocity of certain percentile values of the overall substrate size distribution within the reach. Using these relationships, the model can therefore be used to anticipate the long term morphological impacts of changes in a river's flow regime. In addition, the model can be used to determine the discharge that is required to maintain or restore pool-riffle or pool-rapid structures. However, cognisance should be taken of the fact that the relationship between discharge and channel form is complex and that the above relationships still need to be verified under actual channel forming conditions and further refined by addressing aspects such as the duration of discharge and the effects of sinuosity and sediment transport. Preferably this should include both field and laboratory experiments.

5. A SAND SCOUR MODEL FOR COBBLE BED RIVERS

5.1 Introduction

Whereas Chapter 4 addressed the morphological characteristics of cobble and boulder bed rivers in terms of their regime behaviour on a macro scale, this chapter is concerned with equilibrium conditions in these rivers at a much smaller scale. More specifically, the scouring of sand from the interstitial spaces between the cobbles is addressed.

In cobble bed rivers many aquatic species are dependent on the interstitial spaces between the cobbles for their survival. Salmonids, for example, use these spaces for laying their eggs while the spaces also provide habitat and sheltering for various benthic insects (Gordon *et al.*, 1992). The accumulation of fine sediments in cobble bed rivers, which fill these interstitial spaces, can therefore have a detrimental effect on the whole aquatic ecosystem. Although natural phenomena such as catchment erosion after fires may lead to excessive sediment loads being introduced and deposited on cobble river beds, natural floods ensure the periodic removal thereof. The construction of a reservoir, however, leads to a change in flood peaks and flooding frequency as well as changes in the sediment transporting capacity in the river channel downstream. Fine sediments introduced into this part of the river system from the incremental catchment downstream of the reservoir, may therefore accumulate on parts of the river bed. In order to flush these unwanted fine sands from the interstitial spaces between the cobbles and gravels, special reservoir releases known as flushing flows or sediment maintenance flows, may be specified (Wilcock *et al.*, 1996).

The range of effective flushing flows is relatively narrow. Whereas the rate and efficiency of fine sediment removal increase with discharge, so does the potential cost in the form of lost economic opportunity as the released water is lost for storage or power generation. The transport rate of larger sized sediments also increases with discharge and may need to be kept within limits. The size of a flushing flow may be further constrained by the release capacity of the dam, financial and legal liabilities associated with the creation of an artificial flood as well as the availability of stored water at the appropriate time (Wilcock *et al.*, 1996). Other points to be considered in the determination of flushing flow requirements, include the location of sediment sources, the effect of land use changes on sediment influxes and the sensitivity of biota to sediment deposition.

Although there is a clear need to specify flushing flows as accurately as possible, relatively crude methods are often used due to a lack of appropriate models. Flushing flow methods can generally be classified into three categories viz. hydrological, morphological and sedimentological methods (Gordon *et al.*, 1992). Hydrological methods are based on an index obtained from flow records e.g. a discharge with a certain return period or probability of exceedance, while morphological methods typically specify flushing flows as a percentage of bankfull flow. Usually these methods are based on observations at the site of interest or in other similar channels and as such are empirical in nature. The sedimentological methods on the other hand are physically based and require knowledge of channel form, gradient and substrate composition as well as sediment entrainment and transport theory.

The determination of flushing flows based on sedimentological methods is subject to uncertainty due to the complexity of flow and sediment transport patterns in cobble bed rivers. Most sedimentological methods assume that fine sediment will be flushed out when the threshold of motion of some percentage of the fine particles is reached. However, due to the bimodal character of the substrate (sand size sediments and cobble sizes), the conventional sediment entrainment equations, which were derived for uniform sediment sizes (e.g. the Shield's equation), are inaccurate as they do not allow for the effects of shielding, packing or armouring. Extensive research has been conducted in order to estimate the critical shear stress in mixed bed sediments. Based on measurements of the critical shear stress of natural sediments, Wilcock (1993) for example proposed a parameter which relates the bimodality of a sediment to the critical shear stress of different size fractions. Wiberg and Smith (1987) assumed that the velocity profile of the water could be extrapolated down to the level of the grains on the bed allowing the forces on the grains to be calculated and the critical shear stress for each grain to be predicted. By allowing for two length scales (the particle diameter and the bed roughness) they found that, due to the relative protrusion of particles into the flow as well as differences in the particle angle of repose, the particles at the surface of a poorly sorted bed can have critical shear stresses that differ significantly from the critical shear stresses associated with the same particles when placed on a well-sorted bed of the same size. Kuhnle (1993) investigated the initiation of sediment beds with bimodal size distributions (one mode in the sand and one in the gravel size range). He calculated the critical bed shear stress by interpolating or extrapolating the bed shear stress for a very small transport rate, with the aid of techniques developed by Parker *et al* (1982) and Wilcock and Southard (1988), and found that values of the critical shear stress for the sand and gravel fractions of the bed increased as the amount of gravel in the bed sediment and thus also the absolute bed roughness, increased.

Effective flushing flow strategies need to consider the magnitude, timing and duration of the flushing flow in order to allow for the entrainment and removal of fine sediments. An attempt to develop a comprehensive flushing flow strategy for gravel bed rivers was made by Wilcock *et al.* (1996). They developed a basis for evaluating the trade-offs among discharge, flow duration and pool dredging which determine rates of bed mobilization, sand removal and gravel loss. This involved the development of a set of simple functions representing sand and gravel transport, gravel entrainment, subsurface sand supply and pool sediment trapping and the combination of these functions into a sediment routing algorithm to evaluate flushing alternatives for the Trinity River in California. They recommended a flow of moderate size, which limits gravel loss and maximizes sand trapping by pools, from where the sand can then be removed by dredging. In another flushing flow study, O'Brien (1987) determined the flow needed to mobilize sand trapped within a cobble bed, based on flume studies and field data collected in the Yampa River in Colorado. From actual bed load measurements, sediment load-discharge relationships were developed and used to calculate the "effective" discharge, i.e. the flow that transported the most sediment over a long period of time. The peak of the flushing flow hydrograph was then set at this effective discharge.

The literature review on flushing flows revealed that, while the empirical flushing flow methods are based on experimental observations, the theoretically based models are mainly concerned with defining incipient motion conditions in mixed bed sediments and provide no relationship between discharge and scour depth. Similarly the work of Wilcock *et al.* (1996) and O'Brien (1987), although addressing the relationship between flow and sediment load, did not provide quantifiable relationships between discharge and absolute scour depth. The existing flushing flow methodologies therefore lack a fundamental theoretical basis which is needed for flushing flows to be specified in terms of time and discharge dependent relationships describing the depth of scour of fine sands from between cobbles.

Ligon *et al.* (1995) and Kondolf and Wilcock (1996) noted that the goals of flushing flows need to be stated in terms of measurable changes to the physical habitat rather than the abundance of organisms. In line with this philosophy, this Chapter focuses on the development of theoretically based relationships which define the depth and rate of scour of fine sands from between the interstitial spaces in a cobble bed. The aim of the model is not to address all aspects related to flushing flows, but to rather focus on the process of fine sand scouring from between cobbles, and as such provide a theoretical basis for sand scour which can then be combined with the relevant hydraulic and sediment transport models to provide a complete flushing flow strategy.

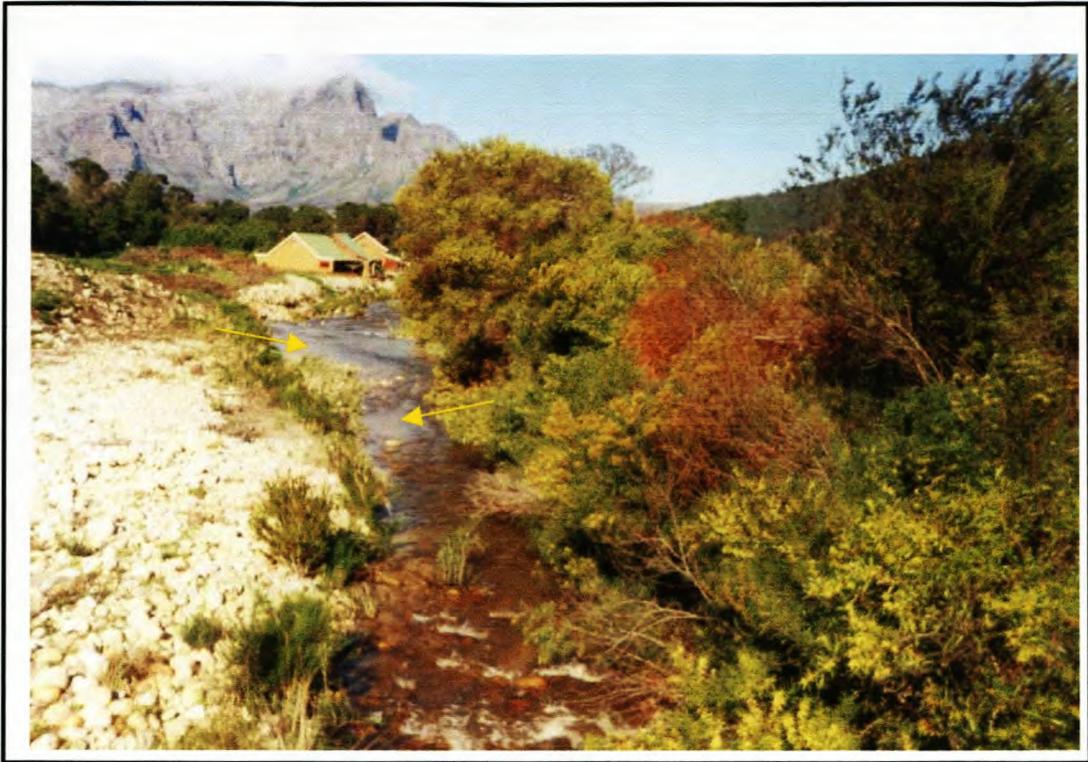


Figure 5.1 : Typical cobble bed river sedimentation (Wemmershoek River below Wemmershoek Dam)

5.2 Theoretical background

5.2.1 Sediment transport and the principle of least applied power

The law of conservation of power has been found to provide good insight into the sediment transport characteristics of streams. This law, which is defined in scalar terms and directly related to time, is a third derivative of Newton's second law together with the laws of conservation of energy and momentum (Rooseboom, 1974 ; 1998). Rooseboom (1974) defined the law of conservation of power under conditions of steady, uniform flow as

$$\int_{y_0}^D \rho g s v dy = \int_{y_0}^D \tau \frac{dv}{dy} dy \quad (5.1)$$

- with
- ρ : fluid density (kg/m^3)
 - g : gravitational acceleration (m/s^2)
 - s : energy gradient \approx channel gradient
 - v : velocity at distance y above the bed (m/s) ($\approx \frac{1}{\kappa} \sqrt{gDs} \ln \frac{y}{y_0}$)

- D : flow depth (m)
 y_0 : $\approx k/30$; ordinate where velocity is mathematically equal to zero (m)
 τ : shear stress at distance y above the bed (N/m^2)
 k : absolute bed roughness (m)
 κ : von Karman coefficient ($\approx \frac{1}{2\pi}$)

(The parameter ρgsv in equation 5.1 represents the amount of unit power made available by the flowing stream, whereas the parameter $\tau \frac{dv}{dy}$ represents the power applied per unit volume to maintain motion.)

Where alternative modes of flow exist, that mode of flow which requires the least amount of unit power will be followed and it therefore follows that fluid flowing over movable material would only transport the material, if it will result in a decrease in the amount of unit power being applied (Rooseboom, 1974; 1998). As the power applied along the bed of a river varies depending on whether laminar or turbulent flow conditions prevail at the bed, the critical condition for sediment movement also depends on whether the flow at the bed is laminar or turbulent.

Under conditions of laminar or smooth turbulent flow, Rooseboom (1974) showed that the unit stream power applied along the bed equals

$$\frac{(\rho g s D)^2}{\rho \nu} \quad (5.2)$$

with ν : kinematic viscosity (m^2/s).

The applied power required per unit volume to entrain a particle with density ρ_s and settling velocity V_{ss} in a fluid with density ρ , equals

$$(\rho_s - \rho)g V_{ss} \quad (5.3)$$

Stokes's law (Graf, 1971), defines the settling velocity of a particle with diameter d under viscous conditions as

$$(V_{ss})_{LAM} \propto d^2 g \frac{\rho_s - \rho}{\rho \nu} \quad (5.4)$$

The critical condition for the movement of sediment particles is reached when the power applied along the bed exceeds the power required to suspend the sediment particles. In laminar or smooth turbulent flow therefore, a relationship defining the threshold for sediment transport under viscous conditions can be defined from equations 5.2, 5.3 and 5.4. This relationship, calibrated with data by Grass (1970) and Yang (1973), was found to be:

$$\frac{\sqrt{gDs}}{V_{ss}} = \frac{1.6}{\frac{\sqrt{gDs}}{\nu} \cdot d} \quad (5.5)$$

for values of $\frac{\sqrt{gDs} \cdot d}{\nu} < 13$, i.e. with smooth turbulent or completely laminar flow over a smooth bed (Rooseboom, 1974 ; 1998).

Under conditions of rough, turbulent flow, Rooseboom (1974; 1998) showed that the unit applied power near the bed (at y_0), where $D-y_0 \approx D$, is

$$\left(\tau \frac{dv}{dy} \right)_0 \approx \frac{30\rho g s D \sqrt{2\pi g s D}}{d} \quad (5.6)$$

Furthermore, under conditions of turbulent flow, the settling velocity as expressed by Graf (1971) equals

$$(V_{ss})_{TURB} = \sqrt{\frac{4(\rho_s - \rho)gd}{3\rho C_d}} \quad (5.7)$$

with C_d : drag coefficient (assumed constant for larger diameters).

From equations 5.3, 5.6 and 5.7 the critical condition for the movement of sediment along an even bed in rough turbulent flow can thus be defined by

$$\frac{\sqrt{gDs}}{V_{ss}} = \text{Constant} \quad (5.8)$$

This relationship was calibrated with measured data from Yang (1973) and the value of the constant was found to be 0.12 for values of $\frac{\sqrt{gDs} \cdot d}{v} > 13$ (Rooseboom, 1974 ; 1998).

5.2.2 Dynamic equilibrium on a sand streambed

The condition of dynamic equilibrium in a sand bed river is reached when the average rate of deposition of particles equals the average rate of re-suspension. The stream thus maintains a constant transporting capacity, while the average bed level remains constant. By applying the unit stream power approach, Rooseboom and Le Grange (2000) derived and calibrated a universally applicable relationship representing the full spectrum of bed conditions and associated roughness values under conditions of dynamic equilibrium on a sand streambed. The strength of their model lies in the fact that the unit stream power approach allows the direct comparison of a stream's sediment transporting capacity with the effort required to entrain sediment as both are defined in scalar terms. A brief description of their dynamic equilibrium theory is provided below:

Equation 5.6, which represents the amount of stream power applied along a bed under conditions of rough turbulent flow, can be written as

$$\left(\tau \frac{dv}{dy} \right)_0 = \frac{14.8 \rho g s D \sqrt{2 \pi g s D}}{R_0} \quad (5.9)$$

with $\left(\tau \frac{dv}{dy} \right)_0$: unit applied power near the bed

R_0 : $\approx d/2$; the radius of a turbulent eddy next to the bed (m).

Based on the concept of minimum applied power, equation 5.9 therefore suggests that once sediment is being transported, a further reduction in the amount of unit turbulent stream power applied along an alluvial bed is possible by means of (Rooseboom, 1974) :

- The formation of a pseudo-viscous zone of high concentration suspension along the bed (increasing R_0)
- The formation of bedforms i.e. ripples, dunes etc. (increasing R_0)
- Creating a meandering course (decreasing slope)

Furthermore, it can be shown (Rooseboom, 1992) that the sediment concentration at any level within a stream is directly proportional to (applied power)² and subsequently that the sediment carrying capacity of a stream is proportional to (applied power along the bed)² with z defined as

$$z = \frac{5\sqrt{2\pi} \cdot V_{ss}}{6\sqrt{gDs}} \quad (5.10)$$

It therefore follows that as an alluvial stream reduces the amount of unit stream power applied along the bed by deformation of the bed, the stream is also in effect decreasing its sediment carrying capacity. This process continues until a condition is reached where the size of the boundary eddies that fit in with the bedforms become so large that the average rate of deposition of the particles is equal to the average rate of re-suspension of the particles. When this point is reached, the stream is in a condition of steady state sediment transport or dynamic equilibrium.

Rooseboom and Le Grange (2000) hypothesized that under conditions of equilibrium sediment transport in a sand bed river, with particles on the bed being entrained and deposited at the same rate, on average along the deformed sand bed river critical conditions must prevail at the bed where individual particles are continually passing across the movement threshold. This was confirmed experimentally by Wang and White (1993), who, in a paper on alluvial resistance in transition regime, state that: "It was observed during the experiments that the flow velocity near the bottom of the eddy in the trough was close to that for incipient motion of bed material". Furthermore, since it is not possible to have sand particles less than about 2 mm in diameter at rest under turbulent boundary conditions on an even bed as the absolute roughness is too small to induce a turbulent boundary layer, Rooseboom and Le Grange (2000) hypothesized that during dynamic equilibrium along a sand bed, laminar boundary conditions have to prevail at the bed. (As shown in section 5.2.1, a stream is able to entrain bed particles under laminar boundary conditions in accordance with the principle of least applied power (Rooseboom, 1974).)

From equations 5.3 and 5.4, the unit stream power required to suspend a particle of diameter d for laminar conditions along a smooth bed is directly proportional to

$$\frac{(\rho_s - \rho)^2 g^2 d^2}{\rho \nu} \quad (5.11)$$

In similar fashion, when turbulent flow conditions prevail along an even bed, the unit stream power applied in maintaining motion along the bed approximates (from equation 5.6)

$$\frac{30\rho g s D \sqrt{2\pi g s D}}{k} \quad (5.12)$$

with k : absolute bed roughness (\approx particle diameter (d) with an even bed) (m).

As an even sand bed becomes deformed (i.e. the value of k increases), the applied turbulent power at the bed is therefore reduced until equilibrium is reached. At the point of dynamic equilibrium, critical conditions exist at and below the interface between the thin laminar boundary layer along the bed and the turbulent eddies above. Rooseboom and Le Grange therefore reasoned that the applied turbulent unit stream power at the bed, expressed in terms of the absolute bed roughness, must be proportional to the unit power which is required to bring particles into suspension under laminar conditions. Therefore, from equations 5.11 and 5.12

$$\frac{30\rho g s D \sqrt{2\pi g s D}}{k} \propto \frac{(\rho_s - \rho)^2 g^2 d^2}{\rho v}$$

which can be written as

$$\frac{(30\sqrt{2\pi} v)(\rho g s D)^2}{(\sqrt{g D s} k)(\rho v)} = \text{constant} \frac{(\rho_s - \rho)^2 g^2 d^2}{\rho v}$$

and which leads to

$$\frac{\sqrt{g D s}}{V_{ss}} = \left[\frac{\sqrt{g D s} \cdot k}{\sqrt{2\pi} \cdot v} \right]^{\frac{1}{2}} \text{constant} \left[\frac{v}{\sqrt{g D s} \cdot d} \right] \quad (5.13)$$

Equation 5.13 represents the condition of dynamic equilibrium in a sand bed river and defines the relationship between absolute roughness and bed deformation under steady state sediment transport conditions. Comparison of equation 5.13, describing dynamic equilibrium in an alluvial river with bedforms, with equation 5.5, representing critical conditions in laminar

flow, reveals an additional parameter $\left[\frac{\sqrt{g D s} \cdot k}{\sqrt{2\pi} \cdot v} \right]^{\frac{1}{2}}$ in equation 5.13. Based on an analysis of

recorded flood levels and scour depths in Southern-African sand bed rivers, Rooseboom and

Le Grange (2000) calibrated equation 5.13 and showed that the parameter $\left[\frac{\sqrt{g D s} \cdot k}{\sqrt{2\pi} \cdot v} \right]^{\frac{1}{2}}$,

referred to as delta (Δ), reflects the decrease in transporting capacity due to deformation of

the bed with k , the absolute roughness, reflecting the size of the boundary eddies that fit in with the changing bed forms. Their findings confirmed the existence of a laminar boundary layer at equilibrium conditions in a deformed sand bed river and also proved that the threshold between the movement and non-movement of bed particles is the key in describing all equilibrium bed conditions.

Since scouring of fine sediments along a cobble river bed is associated with changes in absolute bed roughness as the cobbles become exposed during scouring, it is hypothesized that *the process of fine sediment scouring in a cobble bed river and the associated change in absolute bed roughness is similar to the process of bed deformation in a sand bed river, except for the fact that the cobbles will limit the extent of bed deformation.* For a cobble bed, the relationship that exists between the sand particle characteristics and the maximum scour depth, should therefore be similar to the relationship that exists between absolute roughness (as represented by Δ) and particle characteristics under conditions of dynamic equilibrium on a deformed sand bed river, provided that allowance is made for the presence of the cobbles. In order to evaluate this hypothesis, laboratory studies were undertaken.

5.3 Development of a sand scour model for cobble bed rivers

5.3.1 Experimental procedure

During the laboratory experiments, various combinations of sand and cobble sizes were used and relationships were developed between applied power, particle characteristics, the rate of scour and the maximum scour depth. Tests were performed in a horizontal flume of 1.0 m width and 40 m length in the hydraulics laboratory of the Department of Civil Engineering of the University of Stellenbosch. Cobbles were collected from rivers and sorted with a gravelometer. Two to three layers of uniform, spherical cobbles were arranged in a closely packed pattern in order to create an artificial cobble bed. The length of the cobble bed was approximately 2.4 m and provided a sufficient area in which to investigate the scouring effect. The cobbles were then completely covered with sand of a fairly uniform grading. At the upstream side of the cobble bed, a transition of gravel and smaller cobbles was constructed to allow full development of the turbulent flow profile. Flow depths were controlled with an adjustable weir at the downstream end of the flume. Average velocities above the cobble bed were measured with an Ott-meter. The laboratory setup is shown diagrammatically in Figure 5.2.

After preparation of the sand-covered cobble bed, the following basic procedure was followed:

- With the adjustable downstream weir controlling the water level in the flume, water was allowed to fill the flume very slowly in such a way that no sand movement took place along the cobble bed.
- The water supply to the flume was slowly increased until the desired discharge was obtained. The discharge was calculated from velocity and depth measurements above the cobble bed as well as from the level at the downstream weir. A large enough depth was maintained above the cobble bed to ensure that no sand movement took place.
- The adjustable weir was then rapidly lowered until the desired water depth above the experimental cobble bed area was reached. This represented the start of the experiment.
- At pre-determined positions along the bed, the level of the sand (depth of scour) was measured as the experiment progressed until no further change in the scour level could be observed.

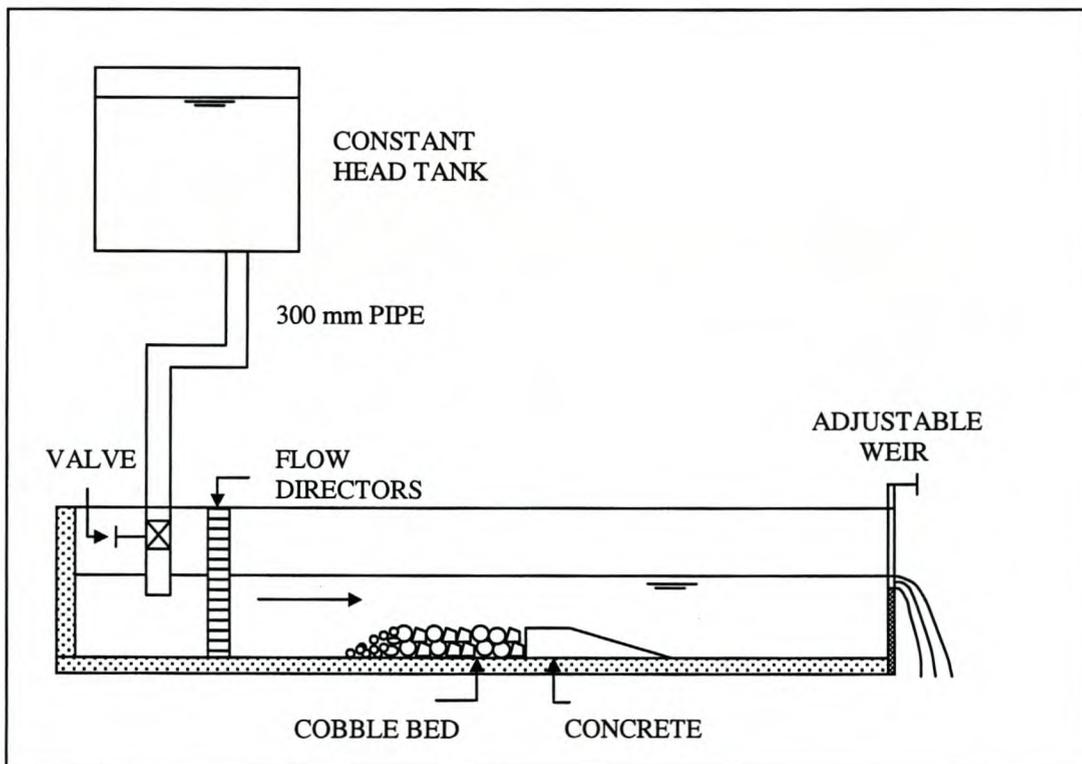


Figure 5.2 : Laboratory setup

In total, 64 experiments were performed. Different sand-cobble combinations, which represented a broad spectrum of cobble and sand particle sizes, were tested (see Table 5.1.)

Experimental setup	Median sand diameter (mm)	Cobble diameter (mm)
I	0.22	60
II	0.83	60
III	0.22	71
IV	0.54	80
V	0.54	130
VI	0.22	130

Table 5.1 : Sand-cobble combinations

The difference in energy and associated water levels between the upstream and downstream sections of the cobble bed reach was too small to measure accurately in order to determine the energy gradient. As the flow depth above the cobble bed was controlled by the downstream weir, the flow in the flume could be classified as gradually varied. Based on the procedure for calculating the energy slope under conditions of gradually varied flow (direct- and standard step methods), the Chézy formula was used to calculate the energy slope above the cobble bed area from the measured average velocity. The scour depth beneath the top of the cobbles was used to represent the absolute roughness (k) in the Chézy equation. Due to the difference in resistance between the cobble bed of the flume and the glass sidewalls, it was assumed that the total flow resistance would be affected only slightly by the sidewalls. The flow was assumed to be two-dimensional and the hydraulic radius in the Chézy equation was substituted with the average flow depth above the cobble bed. In order to limit the uncertainty associated with large-scale roughness, the ratio of flow depth to absolute roughness was maintained at values above three (Chow, 1959). (Appendix F lists the experimental results.) The outcome of a typical scour experiment is shown in Figure 5.3.

5.3.2 Laminar boundary conditions

In order to gain insight into the hydraulic conditions that exist at the bed when the maximum depth of sand scour is reached in a cobble bed, the laboratory data were analyzed in terms of the parameters $\frac{\sqrt{gD_s}}{V_{ss}}$ and $\frac{\sqrt{gD_s} \cdot d}{v}$. (As already shown, the threshold condition for both laminar and turbulent boundary conditions may be defined in terms of these parameters.) The maximum scour depth reached below the cobble crests during each experiment, was used to represent the absolute roughness value (k) for calculation of the energy slope in the above

parameters, while V_{ss} represents the settling velocity of the sand particles (diameter d) under laminar conditions. The flow depth (D) was set equal to the average flow depth along the cobble bed, i.e. the reference bed level was located somewhere between the top of the cobbles and the level of scour between the cobbles. Figure 5.4 shows that scour data with similar values of Δ display the same pattern as that displayed by the incipient motion curve (Liu, 1957; Rooseboom, 1992) for particles smaller than 2 mm on an even bed with laminar boundary layer conditions. In line with Rooseboom and Le Grange's findings, Figure 5.4 therefore indicates that, in correspondence with dynamic equilibrium in a sand bed river, the condition of maximum scour depth in a cobble bed goes hand in hand with the formation of a laminar sublayer below the turbulent eddies that fit in with the shapes of the exposed cobbles.

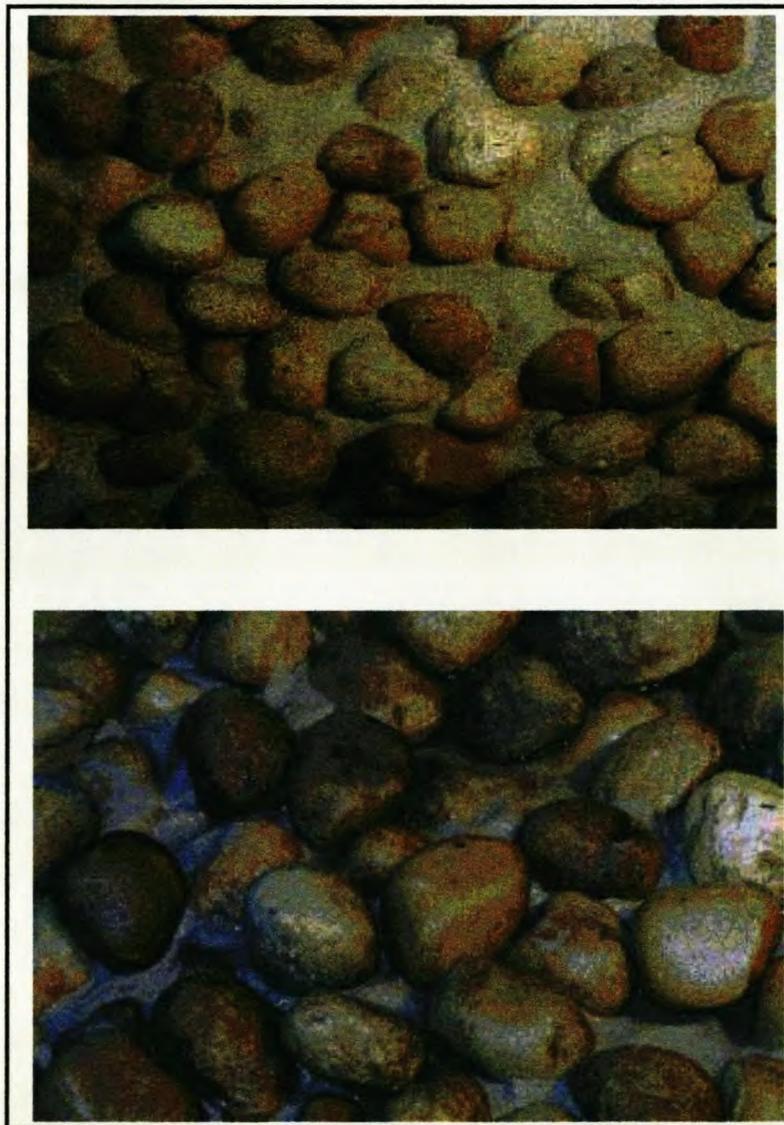


Figure 5.3: The outcome of typical scour experiments, showing exposed cobbles after sand scour (In the top photograph, the maximum scour depth is about half the cobble diameter, while in the bottom photograph it approximates one cobble diameter)

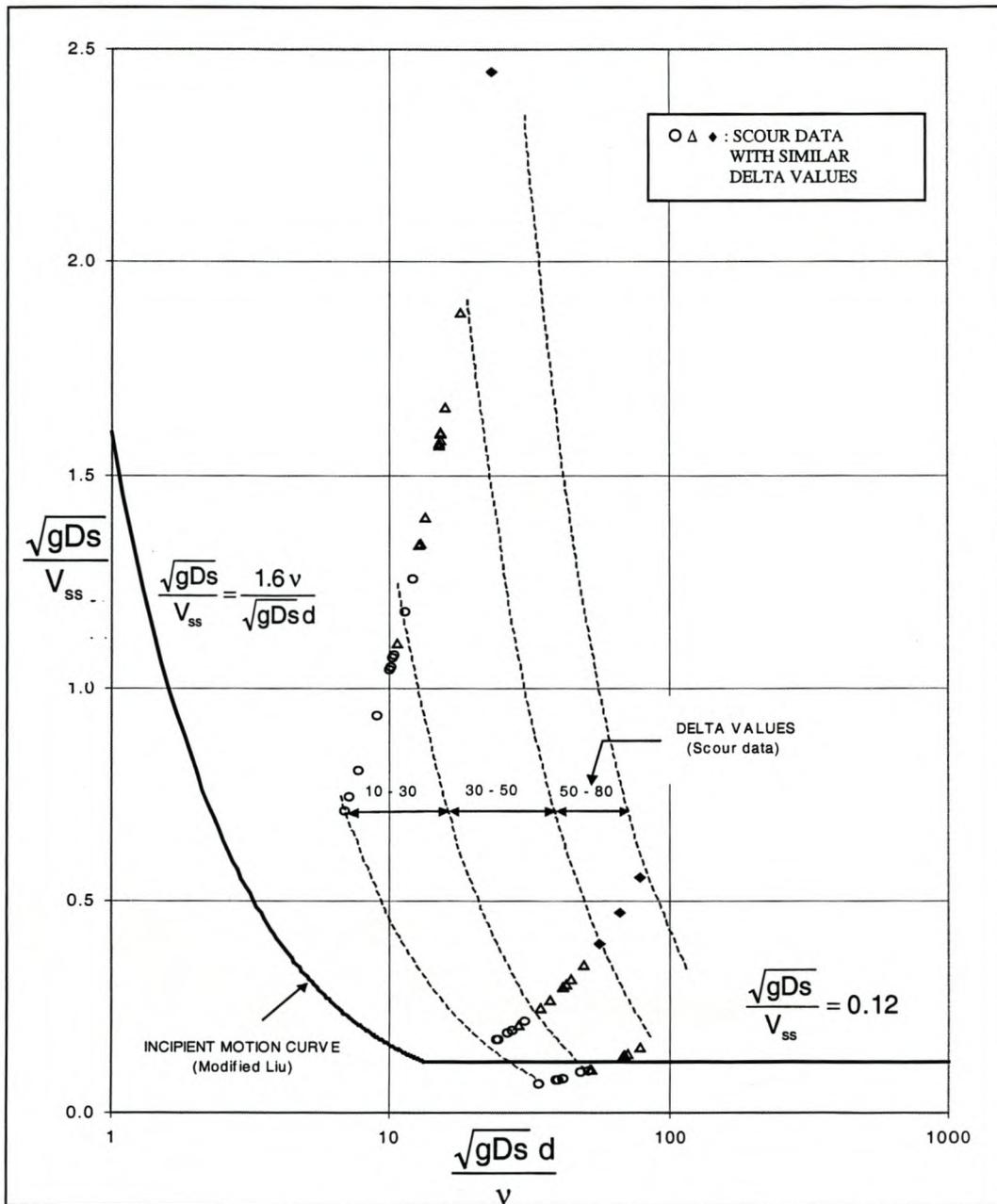


Figure 5.4: Scour data in relation to critical conditions for cohesionless sediment particles

5.3.3 The maximum depth of scour

The theory of dynamic equilibrium enabled Rooseboom and Le Grange (1994) to represent the full spectrum of bed conditions in a sand bed river, from lower regime to upper regime, within a single system. Figure 5.5 (Rooseboom and Le Grange, 2000) displays the typical progression of equilibrium conditions in a sand bed river for three different particle diameters. It indicates that for any particular sand diameter, as $\frac{\sqrt{gDs}}{V_{ss}}$ increases, equilibrium

with lower regime bedforms goes hand in hand with increasing k or $\left[\frac{\sqrt{gDs} \cdot k}{\sqrt{2\pi} \cdot v} \right]^{1/2}$ values and laminar boundary conditions. This continues until a sharp turning point is reached when the boundary conditions below the turbulent eddies switch to being turbulent. The bed becomes unstable and while $\frac{\sqrt{gDs}}{V_{ss}}$ values remain constant, k -values decrease dramatically until a new turning point is reached, from where upper regime bedforms develop and laminar boundary conditions again dominate.

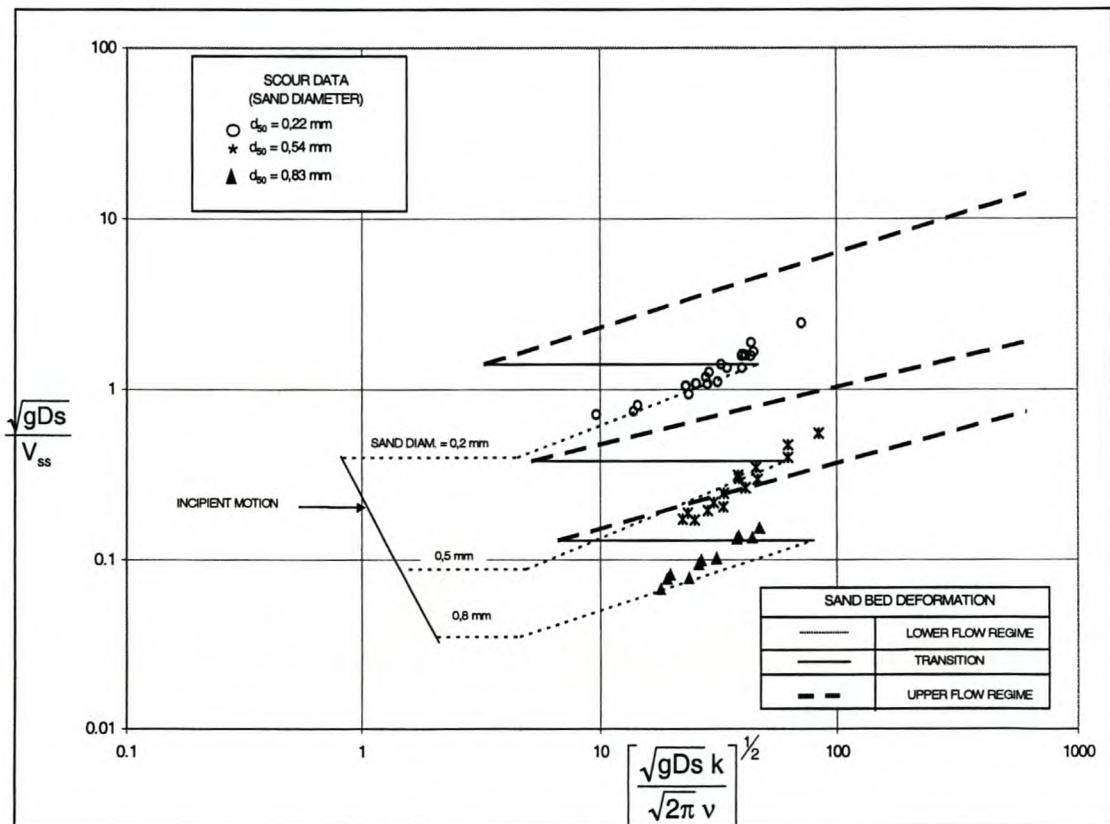


Figure 5.5 : Delta values during dynamic equilibrium in a sand bed river with the new cobble bed scour data superimposed

If the results of the scour experiments are superimposed in Figure 5.5, it is clear that for a particular sand diameter, as the value of $\frac{\sqrt{gDs}}{V_{ss}}$ increases, the maximum scour depth that is reached (represented by Δ) increases in similar fashion to the progression of equilibrium conditions and associated bedforms in a sand bed river. It is important to take cognisance of the fact that in the case of the scour experiments, the absolute roughness (k) was assumed to be equal to the scour depth between the cobbles, as opposed to equilibrium conditions on a deformed sand bed, where the absolute roughness is determined by the changing bed forms.

This may explain the slight difference between the slopes of the cobble bed scour data and the slopes of the lines representing lower regime conditions on a sand bed river, as evident from Figure 5.5. In addition, Figure 5.5 shows that unlike sand bed rivers, where transition from lower to upper regime bedforms occur as $\frac{\sqrt{gD_s}}{V_{ss}}$ increases (as indicated by the horizontal, solid lines), no transition occurs in the case of the 0.22 mm and 0.54 mm sand. This is expected as the “bedforms” in a cobble bed are fixed by the exposed cobbles, which cannot be washed away. Figure 5.5 therefore confirms the hypothesis that the hydraulic conditions during sand scouring along a cobble bed river are comparable to those that prevail under conditions of dynamic equilibrium in a sand bed river.

The next diagram (Figure 5.6), which was developed by Rooseboom and LeGrange (2000), provides a complete averaged picture of the relationship between absolute roughness and particle characteristics under different flow conditions in an alluvial sand bed river. Again, if the results of the scour experiments are superimposed in Figure 5.6, it proves that not only do similar relationships exist between the maximum scour depth in a cobble bed and dynamic equilibrium conditions in a sand bed river as represented by Δ , but that the Δ -values themselves are of the *same order of magnitude*. The observed Δ values can thus be extrapolated according to the pattern for sand bed rivers, for use in calculating the discharges which are required to scour sand from cobble beds down to certain depths.

5.3.4 The rate of scour

As a flushing flow management programme should specify the magnitude and duration of flows, it is imperative that apart from predicting the maximum scour depth, a scour model should also be time-related. In order to define the rate of scour on a cobble bed, a relationship needs to be established between the stream’s ability to entrain sand particles from the interstitial spaces between the cobbles and the effort needed to entrain the particles. In terms of stream power principles, this can be interpreted as the ratio $\left[\frac{\text{Unit power applied along bed}}{\text{Unit power required to suspend particles}} \right]$, which reflects the ratio of the stream’s capacity to entrain bed particles relative to the minimum power required to keep the particles in suspension (Rooseboom and Le Grange, 2000) and should serve as an indicator of the rate of scour. On the assumption that laminar boundary conditions only come fully into play at the point when the maximum scour depth is reached and not during the scour process, turbulent boundary conditions can be assumed to exist during the progression of scour. From equations 5.3 and 5.9 therefore, the above ratio is directly proportional to

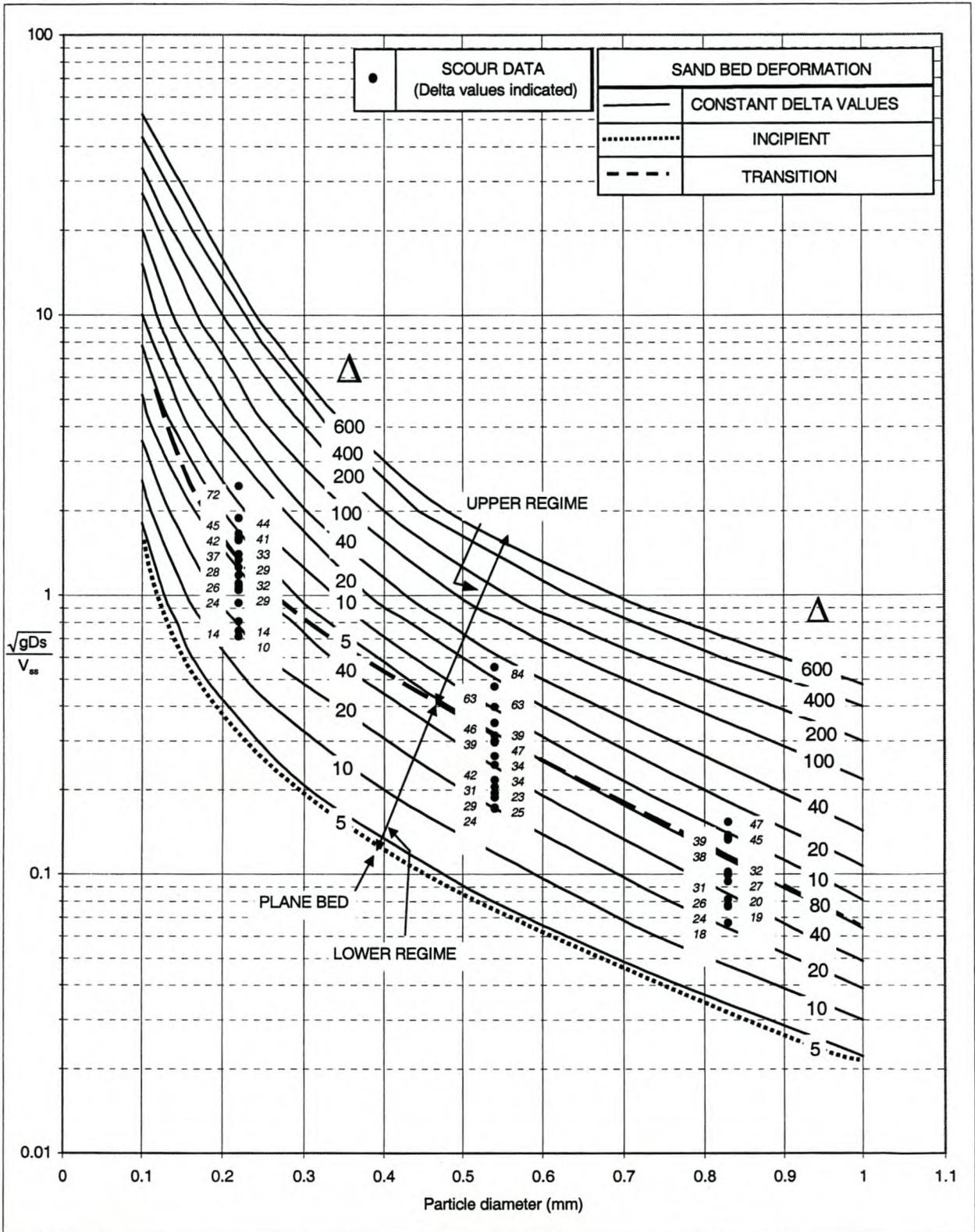


Figure 5.6 : Δ-values and bedforms in a sand bed river with the new cobble bed scour data superimposed

$$\frac{\left(\tau \frac{dv}{dy}\right)_0}{V_{ss}} \tag{5.14}$$

with $\left(\tau \frac{dv}{dy}\right)_0$ and V_{ss} respectively representing the applied power at the bed and the settling velocity under turbulent conditions. If this parameter is compared with the scour rate, as measured during the laboratory experiments, Figure 5.7 shows that a definite trend exists, albeit with a large degree of scatter.

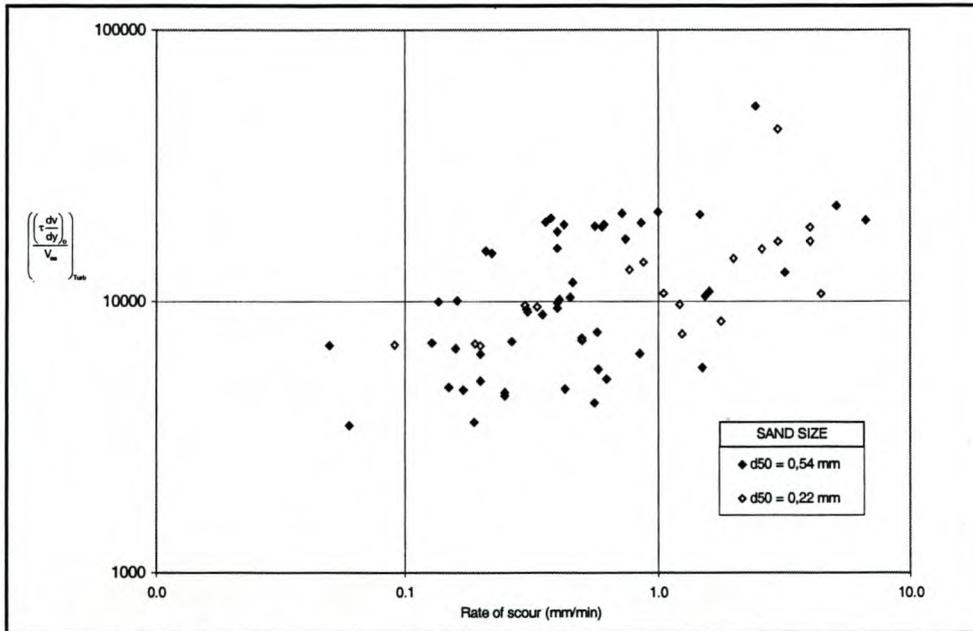


Figure 5.7 : The relationship between rate of scour, turbulent applied power and turbulent settling velocity

(In conjunction with the relationships of Rooseboom (1992), who has shown that the sediment concentration at any level in a river is proportional to the power applied along the bed, the above relationship can be refined in order to account for the effects of sediment concentration. Although this phenomenon was not investigated as part of this study, it is important that cognisance be taken thereof.)

5.4 Conclusions

A scour model has been developed, which predicts the maximum depth of scour as well as the rate of scour of fine sands in a cobble bed. The maximum scour depth represents the level below the top of the cobbles at which no further scour is observed, while the rate of scour refers to the progression of the scour level below the top of the cobbles. The model has been developed from a relationship defining the condition of dynamic equilibrium for steady state

sediment transport in a sand bed river. With the aid of physical model studies, distinct relationships were derived between the absolute roughness or maximum depth of scour, represented by Δ , sand particle characteristics and the relative applied power ($\frac{\sqrt{gDs}}{V_{ss}}$). The findings are based on average conditions within the cobble bed area. Because of the uncertainties involved when scour depths become large, the experiments only focused on scour depths of up to one cobble diameter. Cognisance should be taken of the fact that the experiments were conducted under clear water conditions. In practice therefore, the results are only applicable to the upstream section of a sand-covered cobbled bed where the flushing water is still relatively clear and void of suspended sediment. Although the experiments were conducted under clear water conditions, the results indicate similar patterns for the clear water experiments and for equilibrium sediment transport in sand bed rivers. This suggests that in both cases the equilibrium condition is characterized by particles at the bed continuously crossing the threshold between deposition and entrainment.

Table 5.1 indicates that different cobble sizes were used in combination with the 0.22mm and 0.54mm sand. It was expected that the different cobble diameters would affect the sizes of the boundary eddies, which in turn determine the power being applied at the bed and eventually the maximum scour depth. However the results indicate that the effect of cobble size is small and that cobble diameter as such does not seem to play a significant role in the size of the boundary eddies, which were assumed to be determined by the depth of scour. It rather seems that the size of these eddies is primarily a function of the scour depth or absolute roughness, which suggests that the laboratory results can be extrapolated to represent larger scour depths associated with larger cobbles. This is further confirmed by Figure 5.5, which indicates an almost linear increase in the value of Δ with increasing values of $\frac{\sqrt{gDs}}{V_{ss}}$ on double log scales. Finally, based on the experimental results, a diagram was calibrated which represents the relationship between the absolute bed roughness or maximum scour depth as represented by Δ and particle characteristics under different flow regimes in a cobble bed (Figure 5.8). It shows that the Δ -values typically range from about 10 to 80 for the range of cobble and sand diameters that were tested. Larger cobbles would lead to larger scour depths and higher Δ -values at higher values of $\frac{\sqrt{gDs}}{V_{ss}}$.

It is recommended that further research should concentrate on the effects of:

- absolute cobble sizes
- non-uniform cobble-sized beds

- steady state sediment transport conditions
- sediment concentration

Field experiments should also be conducted.

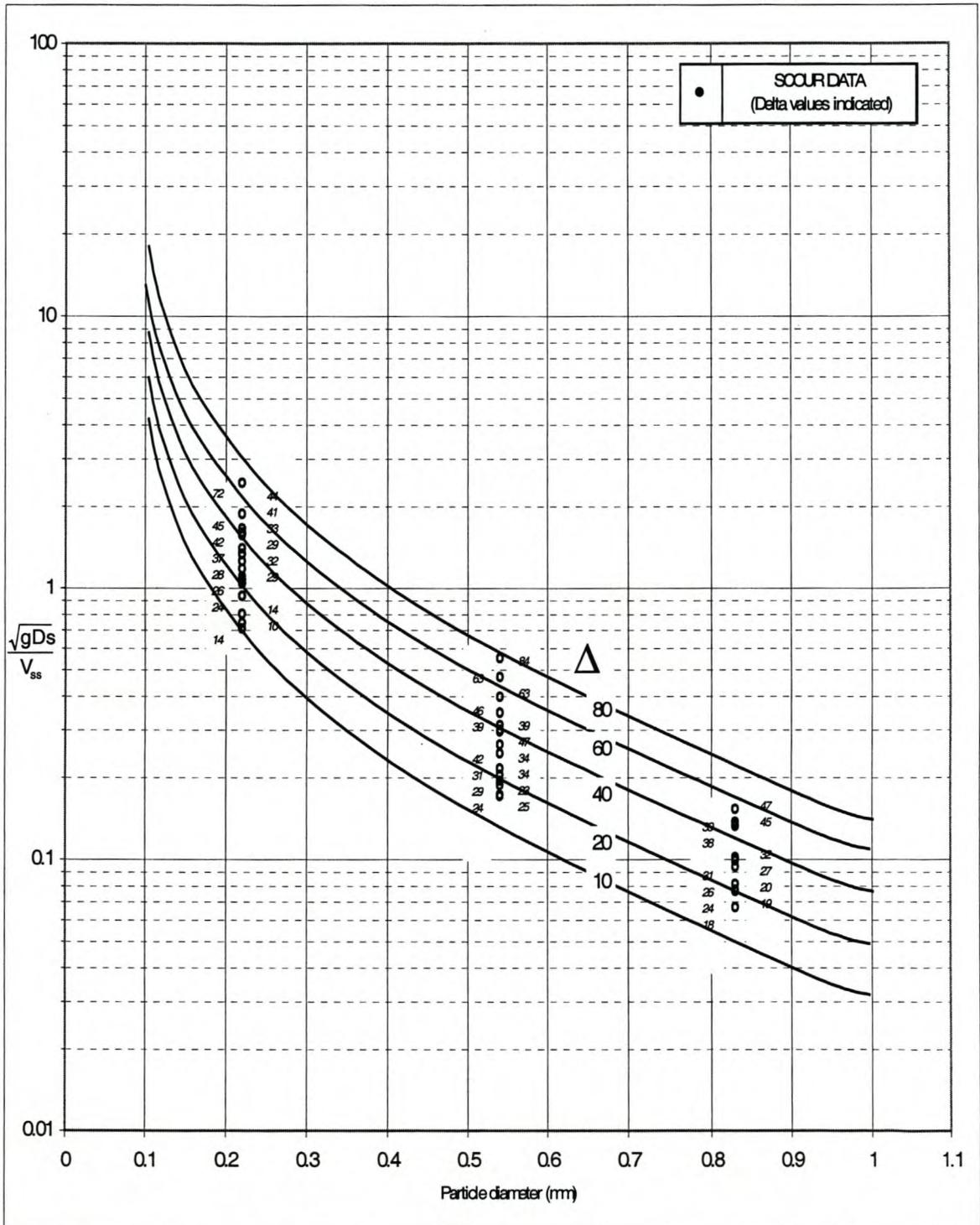


Figure 5.8 : Maximum scour depth in a cobble bed (clear water conditions)

6. VELOCITY-DEPTH RELATIONSHIPS IN COBBLE AND BOULDER BED RIVERS

6.1 Introduction

During the environmental flow assessment process, hydrological data in the form of a simulated, daily discharge regime is translated into point discharge values and further into local hydraulic conditions in order to assess the ecological impact of an altered discharge regime. The most common hydraulic parameters used by ecologists to describe the flow in rivers are flow depth and flow velocity (Bovee, 1982). Water depth affects water temperature, light penetration and hydrostatic pressure, while stream velocity determines the rate at which nutrients and oxygen are supplied and waste products are removed.

Various quantitative models for predicting habitat quality in terms of flow velocity and flow depth have been developed. A central issue in the application of these models is to define the relationship between flow velocity and flow depth, which is required to calculate discharge, to determine sediment transport characteristics and to predict how the physical (hydraulic) habitat changes in relation to discharge. Traditionally, the relationship between flow velocity and flow depth is described in terms of the average values within a cross section. However, recently there has been ever-increasing recognition of the importance of habitat patchiness or physical heterogeneity in freshwater ecosystems and dissatisfaction with the inability to adequately describe this with cross-sectional data (King and Schael, 2001). Subsequently, in order to describe hydraulic habitat diversity, much more emphasis is being placed on the spatial variability or heterogeneity of hydraulic conditions within a reach, with recent studies emphasizing the importance of the variability of local sets of point velocities, flow depths and roughness, or combinations thereof, on hydrodynamic habitats (Lamouroux *et al.*, 1995; Waters, 1976; Bovee and Cochnauer, 1977; Orth and Maughan, 1982).

Cobble and boulder bed rivers, which are characterized by steep gradients and relatively low flow depths in relation to bed particle size, display complex velocity-depth relationships. Due to the very different hydraulic processes of flow resistance, with the flow depth and the bed particles being of the same order of magnitude, the standard friction based equations defining the relationship between mean velocity and flow depth become increasingly inaccurate as the flow depth decreases. Furthermore, as a result of large scale roughness, the average, cross-sectional hydraulic parameters as such do not represent the diversity of local hydraulic conditions occurring within the reach, especially during low flows. In order to provide more

insight into velocity-depth relationships in cobble and boulder bed reaches, this part of the research therefore addresses both the relationship between mean velocity and average flow depth (section 6.2) as well as the spatial variability of local flow velocities within these rivers (section 6.3).

6.2 The estimation of mean velocity under conditions of large scale roughness

The estimation of mean velocity within a river channel is commonly defined by means of friction-based formulae, which, under uniform flow conditions, relate the mean velocity of flow to the flow depth or hydraulic radius, the energy gradient and a resistance coefficient. These equations take the following general form:

$$V = c R^{a_x} s^{b_x} \quad (6.1)$$

with V : average velocity (m/s)
 c : resistance coefficient
 R : hydraulic radius (m)
 s : energy gradient \approx channel gradient
 a_x : constant
 b_x : constant

The resistance coefficient in equation 6.1 depends upon various factors, viz. surface roughness, vegetation, channel irregularities and alignment, silting and scouring, channel size and shape, obstruction to flow and the relative roughness (Chow, 1959). The significance of these factors varies widely depending upon the character of the stream. For conditions of small scale roughness, where the physical roughness elements are small relative to the flow depth, flow resistance may be attributed mainly to surface drag on the channel boundary (boundary or skin resistance), for which case various forms of equation 6.1 have been developed based on a variety of empirical, semi-empirical and theoretical considerations. Examples are the Manning and Chézy equations, which are originally empirical in nature, and the Darcy Weisbach equation, which is partially based on theoretical boundary layer considerations and which defines the resistance coefficient by a semi-logarithmic function of the ratio of flow depth to a characteristic particle size on the boundary (Keulegan, 1938; Schlichting, 1979) expressed as

$$\sqrt{\frac{8}{f}} = a_2 + b_2 \log \frac{D}{d_x} \tag{6.2}$$

with f : Darcy Weisbach resistance coefficient
 d_x : characteristic particle size (m)
 D : flow depth (m)

As the relative roughness (d_x/D) increases and the bed particles and flow depth become of the same order of magnitude, boundary resistance becomes less dominant, while form drag around the individual particles composing the channel bed and disturbance of the free water surface become more significant (Bathurst, 1985). Consequently, due to the different hydraulic mechanisms of flow resistance under conditions of large scale roughness, the conventional friction based equations become increasingly inaccurate as they underestimate channel resistance significantly – by up to 100% according to Bathurst (1985). For this reason, Chow (1959) recommended that the standard resistance coefficients only be used when the relative roughness is less than 0.3. Similarly, Bathurst *et al.* (1981) distinguishes between small-, intermediate- and large scale roughness (Table 6.1), with the understanding that the conventional resistance coefficients are only valid for the condition of small scale roughness.

Roughness scale	Relative submergence	
Small scale	$\frac{D}{d_{50}} > 7.5$	$\frac{D}{d_{84}} > 4$
Intermediate scale	$2 < \frac{D}{d_{50}} < 7.5$	$1.2 < \frac{D}{d_{84}} < 4$
Large scale	$\frac{D}{d_{50}} < 2$	$\frac{D}{d_{84}} < 1.2$

Table 6.1: Roughness scales

Generally, two approaches may be followed to estimate flow resistance under conditions of large scale roughness. The first approach involves empirical and semi-empirical resistance equations, which relate the resistance coefficient to the relative submergence (D/d_x), channel shape or other physical channel parameters. Alternatively, theoretically, process based resistance equations may be derived, which attempt to quantify flow resistance in terms of the resistance processes which dominate under conditions of large scale roughness.

6.2.1 Empirical and semi-empirical large scale roughness resistance equations

Numerous empirical and semi-empirical equations, which define flow resistance under conditions of large and intermediate scale roughness, have been developed. Most of these define resistance in terms of the dimensionless Darcy Weisbach friction factor, which is then used to calculate the mean velocity under the assumption of uniform flow. In general, these equations are of the same form and can be classified into two broad categories, viz.

- non-dimensional power equations:

$$\frac{1}{\sqrt{f}} = a_3 \left(\frac{R}{d_x} \right)^{b_3} \quad (6.3)$$

- non-dimensional semi-logarithmic equations:

$$\frac{1}{\sqrt{f}} = a_4 \log \left(\frac{R}{d_x} \right) + b_4 \quad (6.4)$$

(Equation 6.4 is essentially a modification of equation 6.2, the semi-logarithmic resistance equation derived from boundary layer theory.)

Table 6.2 lists typical large and intermediate scale roughness equations from literature. These equations, which were calibrated with data from gravel or cobble bed rivers, clearly display the inverse relationship between flow resistance and relative submergence, i.e. the value of the resistance coefficient increases as the relative submergence decreases.

Various other types of empirical resistance equations have been developed for conditions of large and intermediate scale roughness (e.g. Thompson and Campbell, 1979; Ferro and Giordano, 1991; Pyle and Novak, 1981), which relate the resistance coefficient to additional physical parameters such as flow width, the standard deviation of bed particle size, effective roughness concentration, wetted frontal area of bed elements, etc. However, these equations become increasingly impractical for application in the field, due to the detailed input data which are required.

Reference	Equation	Particle d_{50} range	Relative submergence
Non-dimensional power equations			
6.5 Charlton et al.(1978)	$\sqrt{1/f} = 1.27 (D/d_{50})^{0.23}$	40 – 220 mm	-
6.6 Bray (1979)	$\sqrt{1/f} = 1.36 (D/d_{50})^{0.281}$	-	3 – 150
6.7 Griffiths (1981)	$\sqrt{1/f} = 1.33 (R/d_{50})^{0.287}$	15 – 301 mm	2 – 200
Non-dimensional semi-logarithmic equations			
6.8 Limerinos (1970)	$\sqrt{1/f} = 2.03 \log (R/d_{50}) + 0.35$	-	-
6.9 Charlton et al (1978)	$\sqrt{1/f} = 1.94 \log((D + d_{84})/d_{84}) + 0.78$	40 – 220 mm	-
6.10 Bray (1979)	$\sqrt{1/f} = 2.36 \log (D/d_{50}) + 0.248$	-	3 – 150
6.11 Griffiths (1981)	$\sqrt{1/f} = 1.98 \log (R/d_{50}) + 0.76$	15 – 301 mm	2 – 200
6.12 Hey (1979)	$\sqrt{1/f} = 2.03 \log ((aR)/3.5 d_{84})$	50 – 250 mm	0.6 – 20
6.13 Bathurst (1985)	$\sqrt{8/f} = 5.62 \log (D/d_{84}) + 4$	273 mm	0.5 – 10

Table 6.2 : Empirical and semi-empirical resistance equations

In general, the disadvantage of empirical and semi-empirical resistance equations is the fact that they tend to be site specific and consequently, great care has to be taken when applying these equations to conditions that are different to those under which they were calibrated (Bathurst, 1978; 1985). In order to develop a more generally applicable, theoretically based equation for the calculation of mean velocity under conditions of large scale roughness, a fundamental approach was therefore considered.

6.2.2 A fundamental approach towards the estimation of mean velocity under conditions of large scale roughness

A detailed literature survey confirmed that no process-based, flow resistance equations for conditions of large scale roughness exist. The only equation that attempts to describe the resistance processes which dominate under conditions of large and intermediate scale roughness, was developed by Bathurst *et al.* (1981). Assuming that the resistance to flow under conditions of large scale roughness is related mainly to the form drag of the roughness elements and their disposition in the channel, Bathurst accounted separately for the resistance effects related to Reynolds and Froude numbers, which determine the drag of individual elements, and the processes of roughness and channel geometry, and as such determined the

combined effect of the roughness elements on the flow. Through the use of semi-empirical analyses, supported by extensive flume data, the following equation was derived:

$$\sqrt{\frac{8}{f}} = \left(\frac{0.28}{b_e} Fr \right)^{\log(0.755/b_e)} 13.434 \left(\frac{W_T}{d_{50}} \right)^{0.492} b_e^{1.025(W_T/d_{50})^{0.118}} \left(\frac{A_w}{W_T D} \right) \quad (6.14)$$

- with Fr : Froude number
 b_e : function of effective roughness concentration
 W_T : flow width (m)
 A_w : wetted roughness cross-sectional area (m²)

Apart from the fact that this equation requires very detailed input data, which complicates its application, Thorne and Zevenbergen (1985) applied this equation to field conditions and found that it does not produce accurate estimates of flow resistance.

As part of this research, a simpler, generally applicable methodology for defining mean velocity-depth relationships under conditions of large scale roughness was sought, using two approaches: Firstly, the principles of conservation of energy, momentum and power were applied to an ideal cobble bed river and, based on specific assumptions regarding the resistance processes that dominate under conditions of large scale roughness, equations for mean velocity were derived. Secondly, since velocity-depth relationships may be expressed in terms of the size of the boundary eddies (Rooseboom, 1974), the relationship between relative roughness, cobble diameter and the diameter of eddies which form next to the bed under conditions of large scale roughness, was investigated.

i. The principles of conservation of energy, momentum and power

Consider a river reach of length *l* and gradient S₀ as depicted in Figure 6.1, with the bed consisting of a single uniform layer of spherical cobbles of diameter *d*. Assume that the flow is uniform and let the number of cobbles within the reach along a streamline be equal to \bar{n} , while the total number of cobbles in the reach equals *n*.

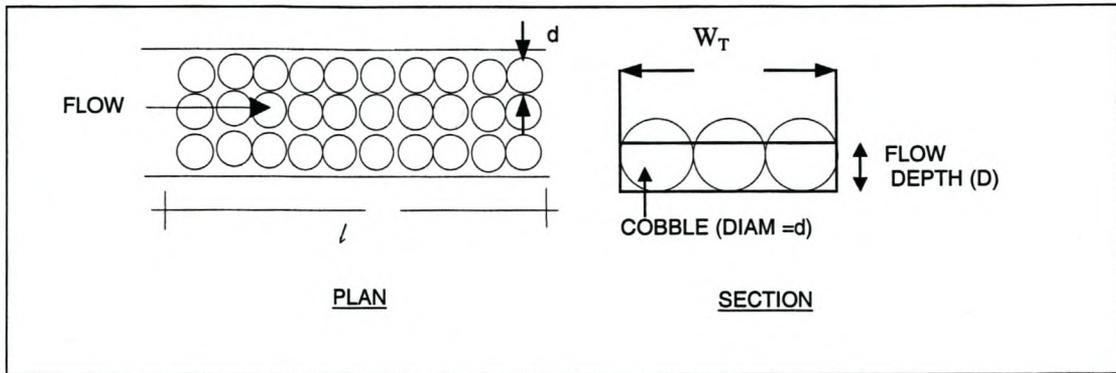


Figure 6.1: Schematic representation of ideal cobble bed river

The energy principle

Assume that during low flows in the river, energy losses can be ascribed mainly to transition losses which, in the Bernoulli equation, can be defined as:

$$h_t = C_t \frac{V^2}{2g} \tag{6.15}$$

- with h_t : transition loss (m)
 C_t : transition loss coefficient
 V : average flow velocity (m/s)

If equation 6.15 represents the transition loss due to one element, the transition losses along a streamline in the direction of flow, due to \bar{n} elements, add up to

$$(h_t)_l = \bar{n} C_t \frac{V^2}{2g} \tag{6.16}$$

However, for the ideal case being considered, $\bar{n} \approx l/d$ and therefore

$$(h_t)_l = \frac{l}{d} C_t \frac{V^2}{2g} \tag{6.17}$$

The energy gradient over the reach (S_f), which is equal to the channel gradient (S_0) assuming uniform flow, equals $(h_1)/l$ and therefore, from equations 6.16 and 6.17, the average velocity (V) may be expressed as

$$V = \sqrt{\frac{2gdS_0}{C_f}} \quad (6.18)$$

The momentum principle

The momentum principle states that “the sum of all the external force components acting upon a body of fluid equals the difference between the sum of the momentum flux components of the outflows and the inflows” (Rooseboom, 1974). Under conditions of uniform flow, where the water depth and velocity remain constant, the momentum principle simplifies to a balance between the external forces in terms of the weight component of the enclosed fluid and the force exerted on the enclosed fluid by the roughness elements.

Consider the section of river in Figure 6.1. Assume that the roughness elements act individually, generating a total resistance force based on the sum of their profile drags. (Profile drag is composed of form drag and skin friction, the latter being small compared with the former under conditions of large scale roughness.) From the momentum principle, the weight component of the body of water (with length l , width W_T and depth D) in the direction of flow, must equal the force exerted on the enclosed fluid by the roughness elements.

The weight component of the water body in the direction of flow may be expressed as

$$\rho g W_T l D S_0 \quad (6.19)$$

while the drag force exerted by all the elements equals

$$(n)(0.5 \rho C_d V^2 A_p) \quad (6.20)$$

with A_p : wetted frontal area of a roughness element (m^2)
 C_d : drag coefficient

Equating equations 6.19 and 6.20 leads to the following equation for the average velocity:

$$V = \sqrt{\frac{2g}{C_d}} \sqrt{DS_0} \sqrt{\frac{lW_T}{nA_p}} \quad (6.21)$$

The ratio $\left(\frac{nA_p}{lW_T}\right)$ in equation 6.21 is termed the “roughness concentration” (Bathurst, 1978)

and refers to the ratio of the wetted frontal area of all the bed elements to the plan area of the bed. For the ideal case being considered, and when the flow depth is less than the particle diameter, this ratio can be simplified as follows:

A_p may be approximated by the product of the ratio of flow depth to cobble diameter (D/d) and the frontal area of a cobble $\left(\frac{\pi d^2}{4}\right)$. Furthermore, from the plan view on Figure 6.1,

(lW_T) may be approximated by $n\left(\frac{\pi d^2}{4}\right)$. The roughness concentration within the channel of

length l and width W_T , may thus be approximated by

$$\frac{n\left[\left(\frac{\pi d^2}{4}\right)\left(\frac{D}{d}\right)\right]}{n\left(\frac{\pi d^2}{4}\right)} \approx D/d \quad (6.22)$$

Equation 6.21 then becomes

$$V = \sqrt{\frac{2gdS_0}{C_d}} \quad (6.23)$$

The principle of conservation of stream power

Again referring to Figure 6.1, consider a unit volume of water with mass m and average velocity V . The kinetic energy of this water element equals $(1/2)mV^2$. Assume that the kinetic energy loss per channel length equal to one cobble diameter (d) equals

$$C_k (1/2) m V^2 \quad (6.24)$$

with C_k : kinetic energy loss coefficient

Let t represent the time it takes for the water element to move a distance equivalent to one cobble diameter. The rate of kinetic energy loss per length of channel equal to one cobble diameter then equals

$$[C_k (\frac{1}{2}) m V^2] / t \tag{6.25}$$

However, $t = V/d$ and for a unit volume of water with density ρ , $m = \rho$. Equation 6.25 therefore becomes

$$[C_k (\frac{1}{2}) \rho V^2] V / d \tag{6.26}$$

which represents the power applied to maintain motion and, in terms of the law of conservation of stream power, should equal the power made available by the decrease in potential energy of the element (Rooseboom, 1974). Therefore

$$[C_k \rho V^3] / 2d = \rho g S_0 V \tag{6.27}$$

with $\rho g S_0 V$: power made available per unit volume of water

From equation 6.27, the average velocity of the water element may then be expressed as

$$V = \sqrt{\frac{2gdS_0}{C_k}} \tag{6.28}$$

From the above analyses it is evident that the principles of conservation of energy, momentum and power yield equations for mean velocity (equations 6.18, 6.23 and 6.28) which display the same form, viz.

$$V = \sqrt{2g} \sqrt{\frac{dS_0}{C_x}} \tag{6.29}$$

with C_x : energy loss/resistance coefficient

This equation differs from the conventional form of equation defining mean flow velocity in rivers (equation 6.1) and implies that under conditions of large scale roughness, the flow velocity is independent of flow depth. However, this may not be unrealistic considering that under conditions of large scale roughness energy losses are dominated by local effects e.g. drag and wake effects, local accelerations and decelerations, local hydraulic jumps, etc, which are related to individual bed elements.

ii. Eddy theory

The second fundamental approach investigated the relationship between cobble diameter, the diameter of eddies that form next to the bed and the relative submergence under conditions of large scale roughness. Rooseboom (1974; 1998) has shown that under conditions of small scale roughness and uniform flow, the diameter of eddies next to the bed fit in with the size of the roughness elements. He subsequently derived an equation defining the mean velocity under turbulent boundary conditions in terms of the size of these boundary eddies. It was therefore hypothesized that, since turbulent flow prevails under conditions of large scale roughness in cobble bed rivers and shear stresses are generated by eddying motion on a large scale, the mean velocity under these conditions can also be expressed in terms of the size of the boundary eddies. Furthermore, if it can be shown that consistent relationships exist between the cobble diameter, the diameter of eddies which form next to the bed and the ratio of flow depth to cobble diameter, these relationships can be used to determine the mean velocity of flow under conditions of large scale roughness.

In order to investigate the above relationships, an understanding of eddy theory is required. The following paragraphs therefore provide a brief overview of eddy theory and define the relationships between mean velocity, flow depth and the size of boundary eddies:

Under conditions of steady uniform flow, from turbulent flow theory and the principle of momentum exchange across an eddy of radius R_d , it follows that (Rooseboom, 1974):

$$\tau(y) = \rho g s (D - y) = \frac{\rho}{2\pi} R_d^2 \left(\frac{dv}{dy} \right)^2 \quad (6.30)$$

$$\Rightarrow \frac{dv}{dy} = \frac{\sqrt{2\pi g s (D - y)}}{R_d} \quad (6.31)$$

with $\frac{dv}{dy}$: angular velocity of the eddy (m/s)

$\tau(y)$: shear stress at distance y above the bed (N/m^2)

In turbulent flow, as layers of fluid cannot slip relative to each other due to the eddying motion, a thin vertical element therefore has to move as a unit (Rooseboom, 1974). As the velocity next to the boundary (at point 0) has to be equal to zero, the only way in which such

an element can momentarily move as a unit, is by relative rotation around 0. However, such rotation is not possible, unless it is accompanied by translation of the centre of rotation. If V_0 equals the translatory velocity of the centre of rotation, it therefore follows that

$$V_0 = y \frac{dv}{dy} = \frac{y\sqrt{2\pi g s(D-y)}}{R_d} = \text{Constant} \quad (6.32)$$

since the centre of rotation is common to all elements in the vertical.

Close to the bed, R_d mathematically equals y and therefore, if $y = y_0$, the ordinate of the level at which the velocity is mathematically equal to 0, y can be equated to R_d . Therefore, equation 6.32 becomes

$$V_0 = \sqrt{2\pi g s(D-y_0)} \quad (6.33)$$

Furthermore, from the principle of conservation of power, it follows that

$$\int_{y_0}^D \rho g s v dy = \int_{y_0}^D \tau \frac{dv}{dy} dy \quad (6.34)$$

After integration, and by substituting equation 6.31 into equation 6.34, an equation for average velocity can be derived, viz.

$$V = \frac{V_0}{D-y_0} D \left(\ln \frac{D}{ey_0} + \frac{y_0}{D} \right) \quad (6.35)$$

Under conditions of small scale roughness, Rooseboom (1974) has proven that equation 6.35 simplifies to an equation with the form of equation 6.1, by showing that the ratio of y_0 to R_d is constant and by assuming that $D-y_0 \approx D$. However, under conditions of large scale roughness, due to the relative large boundary eddies in relation to flow depth and the nature of a heterogeneous cobble bed, the assumptions of a constant ratio between y_0 and R_d or that $D-y_0 \approx D$ are not realistic. Furthermore, under conditions of large scale roughness in a cobble and boulder bed river, the flow depth varies considerably within a cross section, which makes it difficult to define the location of the origin of the y -axis. To overcome this problem, average values of the parameters in equation 6.35 were defined. This was done by considering

a cross section consisting of n_w vertical elements with height (y) and width (dW_T) as shown in Figure 6.2. The cross sectional flow area (A) therefore equals

$$A = \int y dW_T = dW_T \sum y = n_w dW_T \frac{\sum y}{n_w} = W_T \bar{D} \quad (6.36)$$

which means that \bar{D} (the mean flow depth over the cross section) can be expressed as A/W_T .

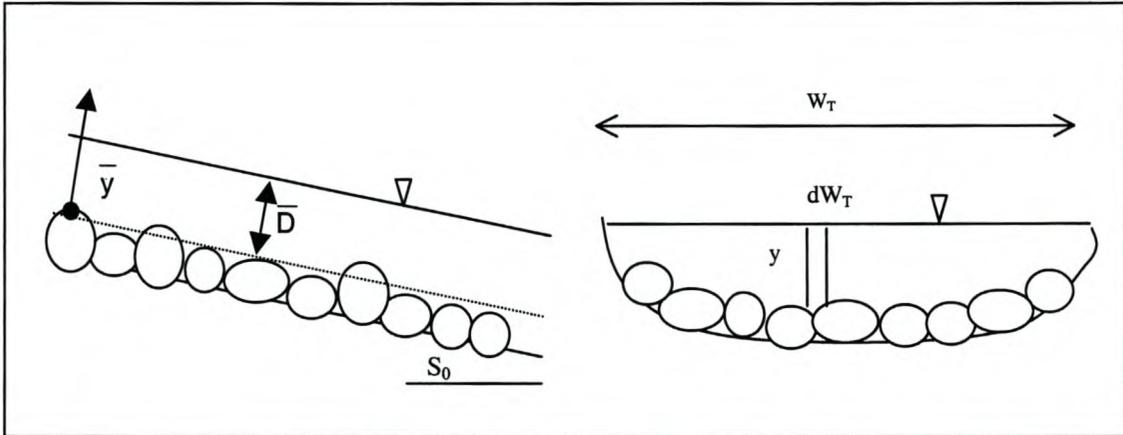


Figure 6.2 : Definition diagram (not to scale)

The parameter \bar{D} may therefore be used to define a reference bed level in a heterogeneous cobble bed where the vertical axis (y axis) originates, as indicated in Figure 6.2. At low flows in a heterogeneous cobble bed river, y_0 can therefore be replaced by \bar{y}_0 , representing the average ordinate level over the cross section at which the velocity is mathematically equal to zero. Equations 6.33 and 6.35 then become

$$\bar{V}_0 = \sqrt{2\pi g s (\bar{D} - \bar{y}_0)} \quad (6.37)$$

and

$$\bar{V} = \frac{\bar{V}_0}{\bar{D} - \bar{y}_0} \bar{D} \left(\ln \frac{\bar{D}}{e \bar{y}_0} + \frac{\bar{y}_0}{\bar{D}} \right) \quad (6.38)$$

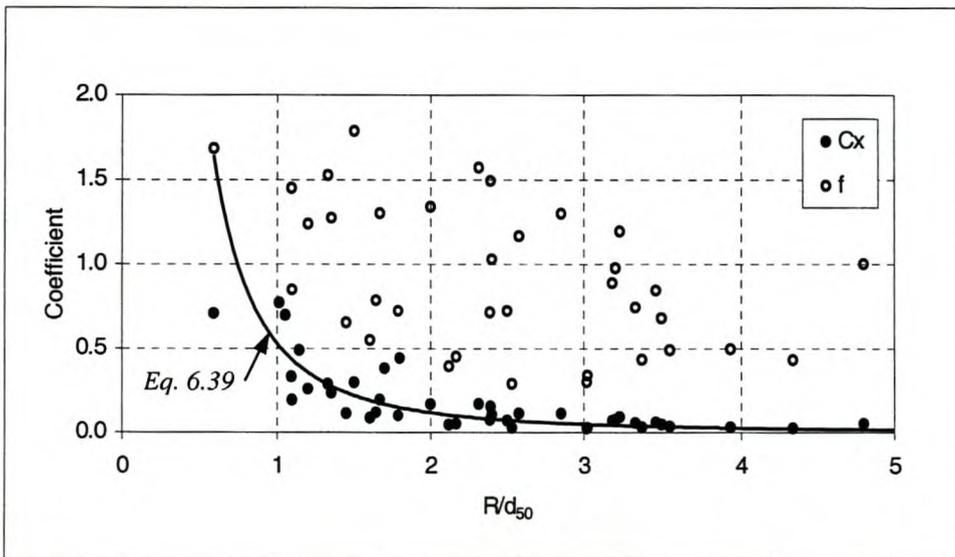
respectively. Equation 6.38 therefore represents the relationship between average flow velocity and mean flow depth under conditions of large scale roughness and uniform flow.

iii. Verification

In order to evaluate the applicability of the two theoretically based equations which were derived (equations 6.29 and 6.38), an extensive dataset was compiled from literature (Bathurst, 1978; Jarret, 1984; Bathurst, 1985), representing fifty measured values of discharge, bed particle size distribution, energy gradient, flow area and hydraulic radius over a range of discharges in cobble and boulder bed rivers, under conditions of large and intermediate scale roughness (refer to Appendix G). From this data, corresponding values of the parameter C_x (equation 6.29) and \bar{y}_0 (equation 6.38) were calculated, and their relationship with various physical parameters (e.g. absolute cobble size (d_{50}), relative submergence (R/d_{50}) and the width to depth ratio (W/D)) was investigated. In the case of both C_x and \bar{y}_0 , it was found that the only significant correlation that exists, is with the relative submergence (R/d_{50}).

Figure 6.3 shows the good correlation that exists between C_x and R/d_{50} . This is confirmed by the high r^2 value (0.87) of a power form regression line (defined by equation 6.39) fitted to the data:

$$C_x = 0.5285 \left(\frac{R}{d_{50}} \right)^{-2.166} \quad (6.39)$$



**Figure 6.3: The relationship between C_x and R/d_{50}
 (Darcy Weisbach f-values also indicated)**

Also indicated on Figure 6.3 are corresponding values of the Darcy Weisbach resistance coefficient (f) as calculated from the Darcy Weisbach equation (equation 6.40). It shows that, for the same data, the correlation between C_x and R/d_{50} is much higher than the correlation between f and R/d_{50} . The implication of this is that C_x provides a more consistent and accurate quantification of flow resistance under conditions of large scale roughness and does not seem to be as site specific as the Darcy Weisbach resistance coefficient.

In similar fashion as Figure 6.3, Figure 6.4 displays the relationship between the relative submergence and the ratio \bar{y}_0/d_{50} and suggests that, as the relative submergence (R/d_{50}) decreases, there is a corresponding decrease in the value of \bar{y}_0 . If it is assumed that \bar{y}_0 is directly related to the size of the boundary eddies that fit in with the bed particles (which is true for conditions of small scale roughness), Figure 6.4 therefore implies that under conditions of large scale roughness, the development of boundary eddies is suppressed by the lower ratio of flow depth to bed particle size. A regression curve fitted to the data indicates a very poor correlation ($r^2 = 0.36$) which suggests that, under conditions of large scale roughness, \bar{y}_0 (and therefore also the size of the boundary eddies) might also be related to other variables, e.g. particle shape, roughness concentration, standard deviation of particle size, etc., which would be difficult to define under field conditions.

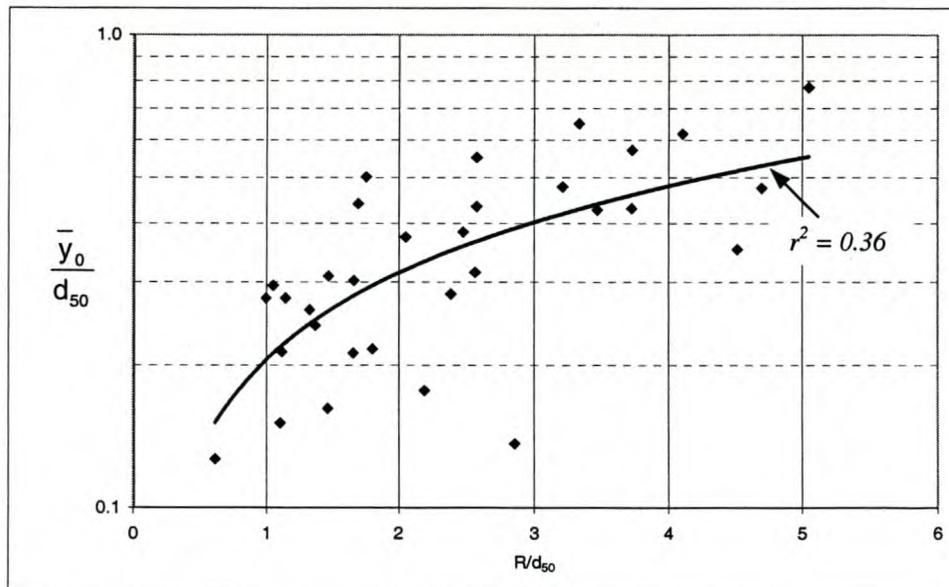


Figure 6.4: The relationship between the ratio \bar{y}_0/d_{50} and relative submergence

From the above analysis it was concluded that equation 6.38 is not suited for the calculation of mean velocity under conditions of large scale roughness. However, from Figure 6.3 it is evident that equation 6.29 can be used for the calculation of mean velocity under large scale roughness conditions. This equation is verified in section 6.2.3.

6.2.3 The application of large scale roughness resistance equations to cobble and boulder bed rivers in the Western Cape

In this section the existing empirical and semi-empirical large scale roughness resistance equations listed in Table 6.2 (equations 6.5 to 6.13), as well as the newly derived, theoretically based equation (equation 6.29), are applied to cobble and boulder bed rivers in the Western Cape. Three study reaches were selected for this exercise. They are located in the Elands, Molenaars and Jonkershoek Rivers respectively. The reaches were between 30 and 70 m in length, straight and relatively uniform, with no disturbances due to vegetation or macro scale bed form irregularities. This allowed average flow conditions within the reaches to be considered uniform, even though locally the flow was non-uniform with zones of separation, acceleration and deceleration. Within each reach, the Wolman sampling method was used to determine the overall particle size distribution within the reach, while bed and water levels at 0.5 m intervals along three cross sections within each reach were surveyed. As far as possible, cross sections were spaced evenly over the entire reach length. At different discharges, values of the wetted perimeter, flow area and hydraulic radius within each cross section were calculated from which a reach-averaged value for the flow area and hydraulic radius was then calculated. (This approach is consistent with numerous other studies on large scale roughness where, due to the irregular nature of the bed, reach-averaged values represented by more than one cross section are used (Jarret, 1984; Bathurst, 1985; Bathurst 1978; Thorne and Zevenbergen, 1985)). At each discharge the average energy gradient (assumed equal to the channel gradient S_0) over the length of the reach was also determined, based on the difference in water surface elevation between upstream and downstream measurements. The actual discharge (Q_m) within each reach was determined from rating curves at nearby gauging weirs (Molenaars: H1H018; Elands: H1H033 and Jonkershoek: G2H037). Table 6.3 lists the relevant cross sectional data. (From Table 6.3 it is evident that in the case of Jonkershoek River, at each of the different discharges, the third cross section is characterized by a much smaller flow area and wetted perimeter than the other two cross sections. This might be attributed to the presence of large boulders which project through the water surface during low flows. However, since this is typical of cobble and boulder bed rivers during low flows, this cross sections was included in the calculations.)

At each of the different discharges and based on the reach-averaged values of the hydraulic radius, the flow area and the energy gradient, values for the Darcy-Weisbach resistance coefficient (f) and the large scale roughness resistance coefficient (C_x) within each reach were calculated from equations 6.5 to 6.13 and 6.39 respectively. These values were subsequently substituted into the Darcy Weisbach equation (equation 6.40) and equation 6.41 (derived from equations 6.29 and 6.39) respectively, to calculate corresponding discharge values, which were then compared with the measured discharge values.

River	Dataset	Cross Section No.	Qm (m ³ /s)	A (m ²)	P (m)	R(m) (m)	Sf (%)	R/d50
Jonkershoek d ₅₀ =0.144m	A	1	0.691	1.758	7.478	0.235	0.30	1.908
		2		1.941	7.569	0.256		
		3		1.258	3.78	0.333		
		Average		1.652	0.275			
	B	1	0.495	1.54	6.997	0.220	0.31	1.770
		2		1.503	6.754	0.223		
		3		1.148	3.565	0.322		
		Average		1.397	0.255			
	C	1	0.175	0.969	5.957	0.163	0.22	1.301
		2		1.047	6.664	0.157		
		3		0.763	3.148	0.242		
		Average		0.926	0.187			
	D	1	0.104	0.88	5.75	0.153	0.28	1.046
		2		0.732	6.137	0.119		
		3		0.431	2.403	0.179		
		Average		0.681	0.151			
Elands d ₅₀ =0.18m	A	1	0.19	1.292	8.48	0.152	0.40	0.782
		2		1.327	9.314	0.142		
		3		1.494	11.715	0.128		
		Average		1.371	0.141			
	B	1	1.115	4.037	12.26	0.329	0.21	1.609
		2		3.767	13.005	0.290		
		3		4.24	16.96	0.250		
		Average		4.015	0.290			
	C	1	0.365	2.759	10.258	0.269	0.28	1.190
		2		2.303	10.465	0.220		
		3		2.435	15.842	0.154		
		Average		2.500	0.214			
	D	1	0.19	1.331	7.9	0.168	0.5	0.788
		2		1.466	9.869	0.149		
		3		1.371	12.65	0.108		
		Average		1.389	0.142			
Molenaars d ₅₀ =0.16m	A	1	0.405	1.617	6.278	0.258	0.39	1.309
		2		1.222	9.148	0.134		
		3		2.041	8.608	0.237		
		Average		1.627	0.209			
	B	1	0.383	1.369	5.668	0.242	0.46	1.307
		2		1.558	8.908	0.175		
		3		2.118	10.038	0.211		
		Average		1.682	0.209			
	C	1	0.36	1.53	6.091	0.251	0.38	1.368
		2		1.583	7.946	0.199		
		3		1.717	8.32	0.206		
		Average		1.610	0.219			

Table 6.3: Cross section data and measured discharges

$$Q = A \sqrt{\frac{8g}{f}} \sqrt{RS_o} \tag{6.40}$$

$$Q = A \sqrt{\frac{2gd_{50} S_0}{0.5285 (R/d_{50})^{-2.166}}} \tag{6.41}$$

Table 6.4 displays the percentage error between the calculated and measured discharge values. Firstly, it indicates that compared with the other equations, equation 6.41 consistently estimates the actual discharge fairly accurately (for eight of the eleven cases within 25%). Secondly, it shows that discharge values calculated with the Darcy-Weisbach equation vary significantly depending on the resistance equation that was used. Only for the case where the semi-logarithmic Griffiths (1981) equation (eq. 6.11) was used to calculate the resistance coefficient, is there relatively good correspondence between calculated and measured discharge values. Thirdly, it indicates that there is no apparent correlation between relative submergence and percentage error. This is surprising as one would expect the estimation of mean velocity (and discharge) to become more accurate as relative submergence increases and the effects of large scale roughness thus become less dominant. However, cognisance should be taken of the fact that limited data were used and that all the R/d_{50} ratios were well below 3, which implies that large scale roughness effect dominated throughout.

Study reach	R/d ₅₀	Percentage error between calculated and measured discharge									
		Eq. 6.41	Darcy Weisbach (eq. 6.40)								
			Resistance coefficient (Eq. No.)								
			6.5	6.6	6.7	6.8	6.9	6.10	6.11	6.12	6.13
JKH A	1.91	-39	-10	-1	-3	-44	-23	-45	-20	-42	103
JKH B	1.77	-33	2	12	10	-40	-13	-41	-12	-38	122
JKH C	1.30	-24	28	39	37	-45	8	-51	-6	-42	129
JKH D	1.05	-16	53	64	60	-54	28	-65	-5	-50	123
ELNDS A	0.78	-7	87	97	93	-79	57	-99	-15	-75	81
ELNDS B	1.61	-29	11	22	20	-39	-6	-42	-8	-37	130
ELNDS C	1.19	13	96	112	108	-25	65	-37	35	-21	225
ELNDS D	0.79	1	101	113	108	-77	69	-99	-7	-72	98
MOL A	1.31	-18	37	49	46	-40	13	-47	1	-44	128
MOLB	1.31	-3	63	77	73	-29	34	-37	19	-33	120
MOLC	1.37	-6	56	70	66	-28	29	-35	18	-32	169

Note: Shading indicates % error within 25 %

Table 6.4 : Percentage error between calculated and measured discharges for Western Cape data

The fact that only one of the nine empirical resistance equations (eq. 6.5 to 6.13) consistently provides a good estimate of flow resistance (and discharge), confirms the site specific nature of these equations and suggests that equation 6.11 was probably calibrated under conditions which are very similar to those prevailing in the study reaches. This was confirmed when a semi-logarithmic form equation (equation 6.42) was calibrated with the data in Table 6.3 and displayed very similar coefficients to those of equation 6.11 (indicated in brackets):

$$\sqrt{\frac{1}{f}} = 0.745 (0.76) + 2.4(1.98) \log \frac{D}{d_{50}} \quad (6.42)$$

The above results therefore confirm that the theoretically based equation (equation 6.41) can be considered more generally applicable than the site specific empirical and semi-empirical equations listed in Table 6.2. To further illustrate this point, equations 6.41 and 6.40 (with the flow resistance coefficient calculated from equation 6.11) were applied to an additional set of large scale roughness data from literature (Thorne and Zevenbergen, 1985). Figure 6.5, which displays the degree of correspondence between the calculated and measured discharge values, shows that whereas equation 6.41 again provides relatively accurate estimates of the actual discharge, equation 6.40 is less accurate.

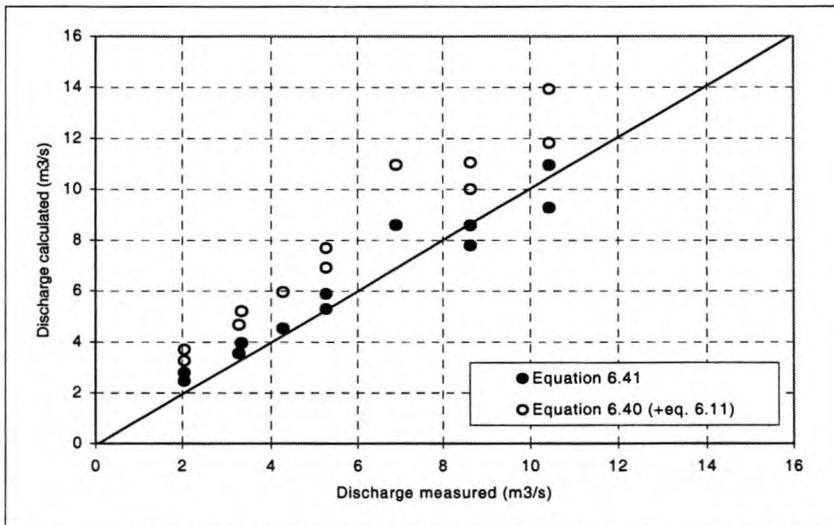


Figure 6.5: A comparison of calculated and measured discharge values for data from Thorne and Zevenbergen (1985)

6.2.4 Conclusion

Based on the principles of conservation of energy, momentum and power, a fundamental approach towards the estimation of mean velocity under conditions of large scale roughness led to the development of a relatively simple and generally applicable equation. With the aid of an extensive dataset from literature, the resistance coefficient in this equation (C_x) was calibrated. It was found that C_x displays good correlation with relative submergence (R/d_{50}) and that the relationship between C_x and R/d_{50} is more consistent across different rivers or reaches than the Darcy Weisbach resistance coefficient (f), which is usually used to quantify flow resistance under conditions of large scale roughness. The application of the newly derived theoretically based equation on cobble and boulder bed rivers in the Western Cape,

proved that it is able to consistently provide a fairly accurate estimate ($\pm 25\%$) of mean flow velocity under conditions of large scale roughness. The application of the Darcy Weisbach equation to cobble and boulder bed rivers in the Western Cape, with the resistance coefficient calculated from existing empirical and semi-empirical large scale roughness resistance equations, confirmed the site specific nature of these equations. Whereas some equations provided fairly accurate estimates of discharge, others led to significant over- or under estimations.

It is therefore recommended that the new equation (equation 6.41) be used as a generally applicable equation for obtaining first order estimates of mean velocity-depth or stage discharge relationships under conditions of large scale roughness. However, whenever more accurate estimates of mean velocity (or discharge) are required, it is recommended that a resistance coefficient be calibrated over a range of discharges under conditions similar to those under which they will be applied. Furthermore it is recommended that the resistance coefficient in equation 6.41 be further refined by investigating its sensitivity to additional variables, e.g. roughness concentration, particle shape, etc.

6.3 The probability distribution of local velocities in cobble and boulder bed rivers

Due to the complex nature of flow patterns under conditions of large scale roughness in cobble and boulder bed rivers, the estimation of the flow resistance, which is subsequently used to calculate the mean velocity under the assumption of uniform flow, provides no information on the spatial distribution of local velocities within the reach under consideration. The statistical description of the velocity distribution under these conditions is therefore an attractive alternative. Dingman (1989) investigated the probability distribution of flow velocities in channel cross sections. From existing hydraulic and statistical relationships for idealized channel cross sections, he derived the following power-law to define the probability distribution of velocities in natural channel cross sections:

$$F(v) = \left(\frac{v}{v_{\max}} \right)^j \quad (6.43)$$

with v : local flow velocity
 v_{\max} : maximum velocity in cross section
 j : shape parameter

With the aid of field measurements, Dingman confirmed that the velocity distribution in natural channel cross sections is well characterized by the power-law distribution over a wide range of stream types and conditions, including large and intermediate scale roughness. However, he failed to relate the distribution parameters v_{\max} and j to any basic variables. A more practical model was developed by Lamouroux *et al.* (1995), who analyzed velocity data on several stream segments with intermediate to large scale roughness statistically. The frequency distribution of point velocities relative to the mean velocity, was expressed as a combination of a centered (Gaussian) model, grouped around the mean, and a decentered (Gaussian and exponential) model. A shape parameter, describing the point velocity frequency distribution, was defined and dimensional analysis was used to model this parameter, and subsequently the velocity distribution, as a function of average parameters within the reach, e.g. roughness, flow depth and flow width.

Given the relative simplicity of the Lamouroux model in terms of input data requirements, as well as the important link between the reach scale (average cross-sectional hydraulic parameters) and the local scale (point velocities), a similar approach was adopted in this research programme to determine the velocity distribution of local flow velocities in cobble and boulder bed rivers of the Western Cape during low flows.

6.3.1 Data collection

The dominant morphological units as identified in the cobble and boulder bed rivers of the Western Cape (i.e. pool, plane bed, riffle and rapid morphological units), served as the basic elements in providing the spatial framework for the hydraulic classification of zones or areas within which similar velocity patterns occur.

The analysis was based on data collected during the months of November to May, representing the period of summer low flows in Western Cape rivers. The motivation for this was: Firstly, summer low flows represent a critical period for the aquatic habitat and the hydraulic conditions during this period are of great ecological significance. Secondly, especially in cobble and boulder bed rivers, low flow hydraulics are extremely diverse and complex and very difficult to describe with conventional methods. Thirdly, by collecting all the data during the same stage of the annual flow regime viz. summer low flows, a consistent database was created, making comparison between the various study reaches possible. Finally, the low flows facilitated data collection in terms of accessibility and subsequently led to improved accuracy.

Velocity data were collected along nine of the thirteen study reaches in which morphological data were previously collected (refer to Table 4.1). Due to time constraints, data could not be collected for all thirteen reaches. However, the nine reaches are representative of all the catchments and reach types under consideration and provided an extensive dataset for further analyses.

Within each study reach, cross sections were surveyed and velocity measurements were taken. A maximum of six cross sections were surveyed per reach, with at least one cross section per type of morphological unit. The surveys were performed with a theodolite and survey staff and provided information on flow area, flow width, wetted perimeter and the hydraulic depth. (The cross sections are attached as Appendix H - their locations within the longitudinal profile are shown in Appendix B). Velocity measurements were taken at regular intervals (0.5 m or 1.0 m depending on channel width) within each cross section. Based on these measurements, the velocity area method was used to calculate the discharge. Random velocity measurements, on a grid basis, were also completed within each morphological unit represented by a cross section (See Figure 6.6). Between 20 and 60 measurements were made per morphological unit. Velocities were measured with an electromagnetic flow meter at 0.4 times the flow depth from the channel bottom. It is realized that due to large-scale roughness and its effects on the logarithmic velocity profile, these measurements do not necessarily reflect the average flow velocity. However, by taking all velocity measurements at the same relative level, consistency was ensured. The measured flow velocities represent flow in the dominant flow direction. This was almost always the downstream direction, although it occasionally represented flow in other directions, e.g. transverse or even reverse flow. (Point velocity data are attached as Appendix I).

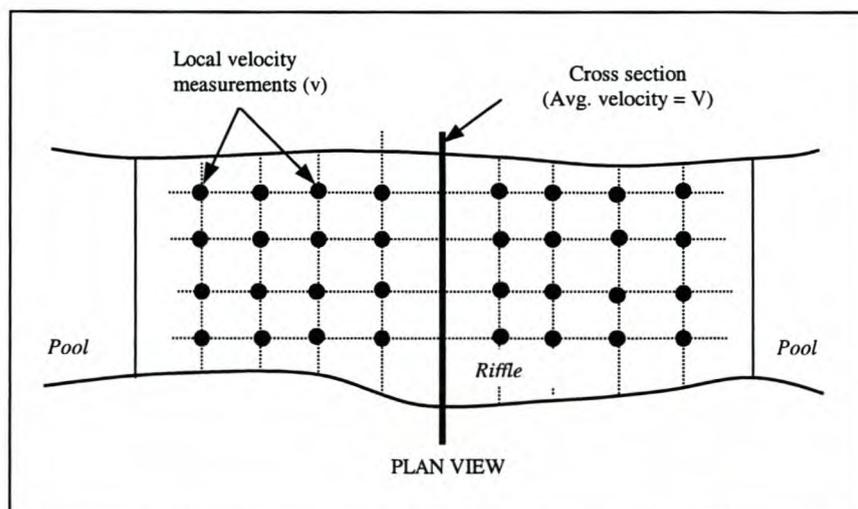


Figure 6.6: Velocity measurements within a typical morphological unit

6.3.2 Statistical analysis

The randomly measured point velocities (v) within each morphological unit were expressed as “relative velocities” defined as the ratio $\frac{v}{V}$, with V the average, cross-sectional velocity within the cross section representing the morphological unit. Probability distributions were then fitted to the observed data and relationships identified between the statistical parameters and the hydraulic parameters describing the average conditions within each type of morphological unit. (In some instances, relative velocities from similar morphological unit types within the same reach were combined, e.g. from two pools or from two riffles. This was done to provide a larger data set for the subsequent statistical analyses. However, the criterion was that the cross-sections had to display similar average hydraulic parameters.)

Table 6.5 lists the data sets that were used in the statistical analysis.

POOLS		PLANE BEDS		RIFFLES/RAPIDS	
REACH	XS No.	REACH	XS No.	REACH	XS No.
Whitebridge	2, 5	Whitebridge	3	Whitebridge	4
Berg I	5	Berg I	1, 2	Whitebridge	1
Jonkershoek	1, 6	Jonkershoek	2	Berg I	3, 4
Du Toits	1	Jonkershoek	3	Jonkershoek	5
Du Toits	3	Du Toits	2	Berg II	1, 5
Berg II	2, 4	Elands	1, 2	Elands	3, 4
Elands	5, 6	Vergenoeg	5, 6	Vergenoeg	3
Vergenoeg	1, 2	Berg II	3	Vergenoeg	4
Molenaars	3	Molenaars I	2	Molenaars I	1
Vlottenburg	1			Vlottenburg	2, 3
				Du Toits	4
				Du Toits	5

Table 6.5 : Data sets for statistical analyses

Appendix J shows the observed frequency distributions of relative velocity. Generally, it is a skewed distribution, which in some cases displays extreme values.

To identify probability distribution(s) that could be used to predict the frequency distribution of relative velocities in cobble and boulder bed rivers, a broad spectrum of probability distribution types were fitted to the observed data. These include the Normal, Log-normal, Extreme Type I, Gamma, Rayleigh and Weibull distributions and represent centered, decentered and extreme type distributions (refer to Table 6.6).

As a preliminary process of elimination, probability plots were utilized to visually identify those distributions which display a poor fit of the observed data. These were then eliminated from further analyses for that particular data set.

The next step involved goodness-of-fit tests to test the validity of the assumed distributions. The Chi-square and Kolmogorov-Smirnov tests were used to determine the absolute statistical significance as well as the relative goodness-of-fit for each of the theoretical probability distributions. (The results of these tests are found in Table 6.7)

Generally, the results confirm that in almost all cases, the probability distributions which were evaluated are statistically significant at a significance level of $\alpha = 5\%$. Based on the relative goodness-of-fit of the various distributions, it was consequently concluded that:

- within most of the pool morphological units, the Weibull distribution consistently provides the best estimate of the frequency distribution of relative point velocities.
- within plane bed morphological units, the Extreme and Weibull distributions both display the best fit to the observed data, relative to the other distributions.
- within rapid- and riffle morphological units in general, the Extreme distribution displays the best fit to the observed data, followed by the Weibull distribution.

(In Appendix J these distributions are fitted to the observed data)

<i>Normal distribution</i>	
$f(x) = \frac{1}{\sqrt{2\pi}\sigma} e^{-(x-\mu)^2 / 2\sigma^2} \quad -\infty < x < \infty$	
μ	: mean
σ	: standard deviation
<i>Log-normal distribution</i>	
$f(x) = \frac{1}{\sqrt{2\pi}\beta} x^{-1} e^{-(\ln x - \alpha)^2 / 2\beta^2} \quad x > 0; \beta > 0$	
α	: scale parameter
β	: shape parameter
<i>Extreme Type I distribution</i>	
$f(x) = \alpha \exp[-\alpha(x - \beta) - e^{-\alpha(x - \beta)}] \quad -\infty < x < \infty$	
α	: scale parameter
β	: location parameter
<i>Gamma distribution</i>	
$f(x) = \frac{1}{\beta^\alpha \Gamma(\alpha)} x^{\alpha-1} e^{-x/\beta} \quad x > 0; \alpha > 0; \beta > 0$	
α	: shape parameter
β	: scale parameter
<i>Rayleigh distribution</i>	
$f(x) = \frac{x}{\beta^2} e^{-x^2 / 2\beta^2} \quad 0 \leq x < \infty; \beta > 0$	
β	: scale parameter
<i>Weibull distribution</i>	
$f(x) = \alpha\beta x^{\beta-1} e^{-\alpha x^\beta} \quad x > 0; \alpha > 0; \beta > 0$	
α	: scale parameter
β	: shape parameter

Table 6.6 : Probability distribution types which were considered

STYDY REACH Cross section(s)	n	NORMAL		LOGN		GAMMA		EXTREME		RAYLEIGH		WEIBULL	
		$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)	$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)	$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)	$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)	$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)	$\frac{\sum_{i=1}^k (\eta - e_i)^2}{e_i}$	D _n (max)
POOLS													
Whitebridge (XS 5 & 2)	48	3.40	0.11	-	0.15	1.90	0.13	2.20	0.14	1.00	0.12	1.00	0.12
Berg I (XS 5)	40	32.20	0.24	22.10	0.27	20.30	0.26	25.30	0.27	32.50	0.30	14.60	0.23
Jonkershoek (XS 1 & 6)	48	4.60	0.14	5.10	0.14	2.00	0.12	2.00	0.15	1.20	0.19	1.30	0.13
Du Toits (XS 1)*	20	-	0.15	-	0.16	-	0.14	-	0.13	-	0.18	-	0.13
Du Toits (XS 3)*	24	-	0.17	-	0.19	-	0.19	-	0.18	-	0.31	-	0.19
Berg II (XS 2 & 4)	46	19.40	0.16	10.60	0.11	7.40	0.10	12.00	0.11	21.10	0.21	6.70	0.09
Elands (XS 5 & 6)	56	12.90	0.15	23.10	0.14	13.00	0.12	13.60	0.12	-	0.20	10.00	0.11
Vergenoeg (XS 1 & 2)	51	3.90	0.11	4.20	0.11	2.20	0.09	2.70	0.09	0.70	0.07	1.20	0.09
Molenaars (XS 3)	49	3.50	0.08	6.30	0.18	3.20	0.15	3.10	0.13	1.90	0.12	2.20	0.11
Vlottenburg (XS 1)	33	9.80	0.12	11.20	0.22	7.60	0.18	8.00	0.15	8.70	0.16	7.00	0.15
PLANE BEDS													
Whitebridge (XS 3)*	22	-	0.15	-	0.10	-	0.11	-	0.12	-	0.12	-	0.12
Berg I (XS 1 & 2)	51	12.00	0.09	29.60	0.23	15.50	0.19	13.10	0.15	11.90	0.15	11.20	0.17
Jonkershoek (XS 2)*	21	-	0.20	-	0.13	-	0.16	-	0.19	-	0.39	-	0.16
Jonkershoek (XS 3)*	24	-	0.10	-	0.14	-	0.09	-	0.09	-	0.13	-	0.08
Du Toits (XS 2)*	23	-	0.17	-	0.12	-	0.09	-	0.10	-	0.22	-	0.11
Elands (XS 1 & 2)	48	4.00	0.12	-	0.14	1.90	0.10	2.10	0.10	1.80	0.10	0.90	0.09
Vergenoeg (XS 5 & 6)	44	2.70	0.08	-	0.16	-	0.12	4.40	0.10	2.40	0.11	2.60	0.09
Berg II (XS 3)	39	2.80	0.11	9.40	0.18	2.80	0.13	1.00	0.07	0.10	0.09	1.10	0.09
Molenaars (XS 2)	44	7.10	0.09	-	0.15	4.40	0.11	5.00	0.11	4.80	0.10	4.10	0.10
RAPIDS and RIFFLES													
Whitebridge (XS 4)*	19	-	0.17	-	0.23	-	0.20	-	0.18	-	0.28	-	0.18
Whitebridge (XS 1)*	20	-	0.12	-	0.20	-	0.14	-	0.09	-	0.15	-	0.11
Berg I (XS 3 & 4)	54	10.20	0.09	3.00	0.12	1.20	0.09	3.10	0.07	4.90	0.12	1.80	0.06
Jonkershoek (XS 5)*	22	-	0.15	-	0.19	-	0.17	-	0.16	-	0.29	-	0.17
Du Toits (XS 4)*	23	-	0.15	-	0.13	-	0.12	-	0.14	-	0.20	-	0.12
Du Toits (XS 5)	39	22.50	0.13	4.50	0.15	3.90	0.12	13.00	0.12	33.50	0.26	3.90	0.11
Berg II (XS 1 & 5)	49	6.20	0.09	13.70	0.23	4.00	0.17	2.90	0.12	4.80	0.17	1.80	0.14
Elands (XS 3 & 4)	55	7.60	0.11	3.20	0.09	2.50	0.07	3.70	0.08	3.10	0.11	2.60	0.08
Vergenoeg (XS 3)*	23	-	0.08	-	0.20	-	0.16	-	0.14	-	0.15	-	0.13
Vergenoeg (XS 4)*	21	-	0.15	-	0.18	-	0.13	-	0.11	-	0.12	-	0.12
Molenaars (XS 1)	48	6.10	0.14	-	0.15	2.40	0.10	2.40	0.07	2.80	0.13	1.60	0.07
Vlottenburg (XS 2 & 3)	49	6.30	0.09	-	0.16	-	0.11	9.10	0.10	-	0.10	6.00	0.10

* Chi-square test inaccurate due to (k<5) and/or (e_i<5)

Note: Bold values indicate statistical significance at the 5% level.

Table 6.7 : Goodness-of-fit-tests results

6.3.3 Parameter estimates

Having identified the most suitable probability distributions for predicting the frequency distribution of relative velocity in cobble and boulder bed reaches, the statistical parameters describing these distributions needed to be related to one or more explanatory variables. The following hydraulic parameters were selected as possible explanatory variables (V represents the average velocity in the cross section and D is the mean flow depth):

- Froude number (V / \sqrt{gD})
- velocity/depth ratio (V/D)
- Reynolds number (VD / ν)
- relative submergence (D/d_{50} or D/d_{84})
- width/depth ratio (W_T/D)

The first two parameters were selected because of the significant variation in flow velocity and flow depth between pool, plane bed and riffle or rapid morphological units. The kinematic viscosity of water changes only gradually with temperature and therefore it was assumed that the Reynolds number is directly related to the discharge per unit width (the product of flow velocity and depth), thus providing an indication of the magnitude of flow. At a given slope, the unit discharge as well as the power being released by the stream are thus represented by the Reynolds number. The relative submergence was included because of its relationship to flow resistance in cobble and boulder bed rivers under conditions of large-scale roughness. Finally, the effect of channel shape, may be described by the ratio of flow width to flow depth.

Simple and multiple linear regression techniques were used to investigate the relationships between the statistical parameters of the probability distributions and the relevant explanatory variables. (Since the parameters, which define the Weibull and Extreme probability distributions, may be estimated from the sample mean and variance, the sample mean (μ) and standard deviation (σ) were also included as dependent variables.)

Table 6.8 lists the average hydraulic parameters for each data set, along with the relevant statistical parameters.

POOLS	STATISTICAL PARAMETERS						EXPLANATORY VARIABLES					
	Mean	St. Dev.	Weibull		Extreme		Fr	V/D	W/D	D/d ₅₀	D/d ₈₄	Re
			Scale	Shape	Location	Scale						
Whitebridge (XS 5 & 2)	1.247	0.678	1.412	1.977	-	-	0.045	0.289	21.458	1.200	0.470	1.650
Jonkershoek (XS 1 & 6)	1.044	0.638	1.169	1.684	-	-	0.050	0.283	16.113	1.450	0.620	2.700
Du Toits (XS 1)	1.517	0.964	1.698	1.646	-	-	0.019	0.122	23.971	1.500	0.550	0.630
Du Toits (XS 3)	1.552	1.285	1.636	1.162	-	-	0.033	0.265	33.613	1.040	0.380	0.650
Berg II (XS 2 & 4)	1.115	0.822	1.225	1.394	-	-	0.035	0.186	19.574	2.050	0.700	2.350
Elands (XS 5 & 6)	0.772	0.506	0.856	1.530	-	-	0.042	0.215	36.127	1.370	0.750	3.000
Vergenoeg (XS 1 & 2)	1.112	0.572	1.260	2.090	-	-	0.040	0.231	20.560	2.500	0.850	2.150
Molenaars (XS 3)	1.063	0.538	1.196	2.066	-	-	0.063	0.412	95.868	1.750	0.760	2.140
Vlottenburg (XS 1)	0.949	0.529	1.061	1.800	-	-	0.017	0.053	14.685	7.286	4.435	5.860
PLANE BEDS												
Whitebridge (XS 3)	1.194	0.684	1.354	1.910	0.890	0.499	0.160	1.385	27.574	0.650	0.260	2.360
Berg I (XS 1 & 2)	1.200	0.745	1.318	1.511	0.838	0.659	0.059	0.423	28.502	0.800	0.390	1.670
Jonkershoek (XS 2)	1.084	1.091	1.045	0.926	0.605	0.736	0.119	0.800	16.118	1.000	0.430	3.770
Jonkershoek (XS 3)	1.303	0.851	1.443	1.544	0.911	0.679	0.067	0.405	16.280	1.260	0.540	2.990
Du Toits (XS 2)	1.300	1.019	1.434	1.396	0.880	0.662	0.088	0.740	17.021	0.930	0.340	1.430
Elands (XS 1 & 2)	1.139	0.647	1.276	1.796	0.828	0.555	0.100	0.616	38.494	0.970	0.530	4.150
Vergenoeg (XS 5 & 6)	1.174	0.557	1.322	2.251	0.899	0.515	0.113	0.860	27.500	1.500	0.550	2.700
Berg II (XS 3)	1.078	0.651	1.198	1.651	0.777	0.534	0.188	1.389	26.833	1.060	0.370	4.500
Molenaars (XS 2)	1.209	0.647	1.361	1.948	0.895	0.569	0.110	0.777	79.677	1.500	0.650	3.000
RAPIDS and RIFFLES												
Whitebridge (XS 4)	1.141	0.845	1.215	1.232	0.740	0.696	0.350	2.833	16.000	0.750	0.290	6.400
Whitebridge (XS 1)	1.334	0.917	1.450	1.389	0.913	0.729	0.284	2.507	17.120	0.630	0.250	3.950
Berg I (XS 3 & 4)	1.458	0.915	1.620	1.605	1.036	0.723	0.261	2.186	35.714	0.510	0.250	4.300
Jonkershoek (XS 5)	1.469	1.238	1.491	1.041	0.903	0.948	0.301	2.560	20.632	0.630	0.270	4.700
Berg II (XS 1 & 5)	1.220	0.816	1.313	1.334	0.835	0.690	0.289	2.497	27.465	0.780	0.270	4.350
Elands (XS 3 & 4)	1.441	0.825	1.629	1.860	1.060	0.646	0.214	1.598	53.056	0.650	0.360	4.900
Vergenoeg (XS 3)	1.179	0.552	1.316	2.203	0.909	0.517	0.396	4.258	51.142	0.650	0.230	3.100
Vergenoeg (XS 4)	1.104	0.650	1.244	1.819	0.813	0.503	0.272	2.279	21.429	1.080	0.370	4.470
Molenaars (XS 1)	1.336	0.807	1.490	1.679	0.963	0.646	0.270	2.589	150.400	0.820	0.350	2.930
Vlottenburg (XS 2 & 3)	0.908	0.426	1.021	2.261	0.694	0.406	0.314	2.247	63.700	1.360	0.830	8.300
Du Toits (XS 4)	1.515	1.029	1.673	1.494	1.048	0.774	0.214	2.375	46.875	0.530	0.200	1.500
Du Toits (XS 5)	1.318	1.060	1.399	1.199	0.839	0.787	0.181	1.789	48.402	0.670	0.250	1.800

Table 6.8 : Statistical parameters and relevant explanatory variables

The approach that was followed during the regression analysis is as follows:

- Scatterplots were used to identify relationships between all pairs of variables. If outliers were identified, the calculations and data were checked and corrected if possible. If the values could not be corrected, the data set was rejected.
- A correlation matrix was used to determine the correlation coefficients. (It is realized that a low correlation coefficient does not imply that the variable will not be a useful predictor in the case of multiple regression.)
- Regression equations were fitted, which define each of the statistical parameters as a function of one or more explanatory variables.

In the case of both simple and multiple linear regression, the *t*-distribution was used to test the statistical significance of the respective regression coefficients, while the ANOVA *F*-test was used as an additional significance test in the case of multiple linear regression.

i. Pool morphological units

Table 6.9 shows the correlation between the different variables. (The shaded values indicate the dependent variables.) It shows that both the mean and standard deviation display good correlation with the relative submergence (D/d_{84}) and Reynolds number, while only the scale parameter (α_{WB}) of the Weibull distribution shows any significant relationship to any explanatory variable – in this case the Reynolds number.

	Fr	V/D	W/D	D/d ₅₀	D/d ₈₄	Re	μ	σ	α_{WB}	β_{WB}
Fr	1.00									
V/D	0.92	1.00								
W/D	0.62	0.72	1.00							
D/d ₅₀	0.07	-0.11	-0.01	1.00						
D/d ₈₄	0.37	0.08	0.25	0.81	1.00					
Re	0.60	0.24	0.06	0.35	0.71	1.00				
μ	-0.59	-0.25	-0.17	-0.31	-0.74	-0.97	1.00			
σ	-0.64	-0.33	-0.24	-0.43	-0.77	-0.82	0.87	1.00		
α_{WB}	-0.58	-0.26	-0.18	-0.26	-0.70	-0.97	0.99	0.80	1.00	
β_{WB}	0.52	0.43	0.31	0.45	0.50	0.25	-0.30	-0.72	-0.20	1.00

Table 6.9 : Correlation coefficients : Pool morphological units (Bold values are statistically significant at $p < 0.05$)

The following regression equations were subsequently derived: (The 95 % confidence intervals of the regression coefficients are provided in brackets, while the r^2 value and the statistical significance are also indicated.)

Mean and standard deviation

Mean (μ)

$$\text{Equation} \quad : \mu = 1.722 (\pm 0.139) - 0.285 (\pm 0.069) \text{ Re} \quad (6.44a)$$

$$r^2 \quad : 0.95$$

Stat. Significance : $\alpha = 5\%$

Standard deviation (σ)

$$\text{Equation} \quad : \sigma = 1.224 (\pm 0.355) - 0.248 (\pm 0.171) \text{ Re} \quad (6.44bi)$$

$$r^2 \quad : 0.68$$

Stat. Significance : $\alpha = 5\%$

$$\text{Equation} \quad : \sigma = 1.272 (\pm 0.411) - 43.62 (\pm 24.54) \text{ Fr} + 5.04 (\pm 3.72) \text{ V/D} \quad (6.44bii)$$

$$r^2 \quad : 0.81$$

Stat. Significance : $\alpha = 5\%$

Weibull parameters

Scale (α) Parameter

$$\text{Equation} \quad : \alpha = 1.877 (\pm 0.162) - 0.299 (\pm 0.078) \text{ Re} \quad (6.45a)$$

$$r^2 \quad : 0.936$$

Stat. Significance : $\alpha = 5\%$

Shape (β) Parameter

$$\begin{aligned} \text{Equation} \quad : \beta = 1.556 (\pm 0.732) + 241.262 (\pm 191.18) \text{ Fr} - 28.54 (\pm 23.95) \text{ V/D} \\ \quad \quad \quad - 1.351 (\pm 1.143) \text{ Re} \quad (6.45b) \end{aligned}$$

$$r^2 \quad : 0.767$$

Stat. Significance : F -test p -value $> 5\%$

These regression equations show that in the case of pool morphological units, the mean and standard deviation of the relative velocity distribution can be related to either the Reynolds number, or the Froude number and velocity-depth ratio at a significance level of $\alpha = 5\%$ and with relatively high r^2 values. However, although the scale parameter of the Weibull distribution shows good correlation to the Reynolds number, the Weibull shape parameter can not be significantly expressed as a function of any basic variable.

ii. Plane bed morphological units

Table 6.10 shows that there is no significant correlation between any of the dependent and explanatory variables in the case of plane bed morphological units. This was confirmed during the regression analyses, which resulted in no statistically significant regression equations for estimation of the statistical parameters. This might be attributed to various factors. Firstly, whereas pools, riffles and rapids are fairly easy to identify visually, plane beds are not, and this might have led to the inconsistent classification of plane bed morphological units. Furthermore, the frequency distribution of point velocities within plane beds might be related to other explanatory variables which were not considered, e.g. the local energy gradient, standard deviation of bed particle size, etc.

	Fr	V/D	W/D	D/d ₅₀	D/d ₈₄	Re	μ	σ	α_{WB}	β_{WB}	β_{EXT}	α_{EXT}
Fr	1.00											
V/D	0.96	1.00										
W/D	0.15	0.07	1.00									
D/d ₅₀	-0.15	-0.26	0.45	1.00								
D/d ₈₄	-0.27	-0.46	0.60	0.89	1.00							
Re	0.24	-0.05	0.19	0.27	0.52	1.00						
μ	-0.45	-0.25	-0.10	0.09	-0.06	-0.63	1.00					
σ	-0.18	-0.17	-0.55	-0.32	-0.38	-0.09	0.08	1.00				
α_{WB}	-0.29	-0.10	0.12	0.13	0.02	-0.57	0.91	-0.32	1.00			
β_{WB}	0.27	0.31	0.48	0.41	0.33	-0.05	0.11	-0.93	0.49	1.00		
β_{EXT}	-0.13	0.02	0.28	0.24	0.14	-0.45	0.73	-0.61	0.94	0.75	1.00	
α_{EXT}	-0.56	-0.59	-0.43	-0.12	-0.08	-0.03	0.08	0.87	-0.31	-0.92	-0.59	1.00

Table 6.10 : Correlation coefficients : Plane bed morphological units (Bold values are statistically significant at $p < 0.05$)

iii. Rapid- and riffle morphological units

From Table 6.11 it is observed that all the statistical parameters display a statistically significant correlation with the relative submergence (either D/d₅₀ or D/d₈₄). In some cases there is also a good correlation with the Reynolds number.

	Fr	V/D	W/D	D/d ₅₀	D/d ₈₄	Re	μ	σ	β _{EXT}	α _{EXT}	α _{WB}	β _{WB}
Fr	1.00											
V/D	0.87	1.00										
W/D	-0.06	0.07	1.00									
D/d ₅₀	0.06	-0.22	0.11	1.00								
D/d ₈₄	0.10	-0.22	0.23	0.86	1.00							
Re	0.40	-0.08	-0.20	0.63	0.76	1.00						
μ	-0.33	-0.08	0.07	-0.88	-0.68	-0.62	1.00					
σ	-0.34	-0.18	-0.18	-0.73	-0.64	-0.49	0.82	1.00				
β _{EXT}	-0.28	0.00	0.25	-0.76	-0.54	-0.58	0.88	0.46	1.00			
α _{EXT}	-0.22	-0.08	-0.19	-0.76	-0.65	-0.44	0.80	0.99	0.44	1.00		
α _{WB}	-0.37	-0.10	0.15	-0.84	-0.63	-0.63	0.97	0.68	0.96	0.65	1.00	
β _{WB}	0.26	0.22	0.34	0.50	0.58	0.30	-0.49	-0.86	-0.05	-0.87	-0.32	1.00

Table 6.11: Correlation coefficients: Rapid- and riffle morphological units (Bold values are statistically significant at $p < 0.05$)

The following regression equations were derived:

Weibull parameters

Scale (α) Parameter

$$\text{Equation} \quad : \alpha = 2.116 (\pm 0.279) - 0.092 (\pm 0.077) V/D + 0.002 (\pm 0.002) W/D - 0.738 (\pm 0.216) D/d_{50} \quad (6.46a)$$

$$r^2 \quad : 0.87$$

Stat. Significance : $\alpha = 5\%$

Shape (β) Parameter

$$\text{Equation} \quad : \beta = 1.153 (\pm 0.456) + 1.348 (\pm 1.25) D/d_{84} \quad (6.46b)$$

$$r^2 \quad : 0.34$$

Stat. Significance : $\alpha = 5\%$

Extreme Type I parameters

Scale (α)

$$\text{Equation} \quad : u = 1.0 (\pm 0.205) - 0.442 (\pm 0.253) D/d_{50} \quad (6.47a)$$

$$r^2 \quad : 0.57$$

Stat. Significance : $\alpha = 5\%$

Location (β)

Equation : $\alpha = 1.13 (\pm 0.16) + 0.001 (\pm 0.001) W/D - 0.38 (\pm 0.19) D/d_{50}$ (6.47b i)

r^2 : 0.68

Stat. Significance : Regression coefficient of W/D not significant at $\alpha = 5\%$

Equation : $\alpha = 1.167 (\pm 0.169) - 0.360 (\pm 0.207) D/d_{50}$ (6.47b ii)

r^2 : 0.57

Stat. Significance : $\alpha = 5\%$ **Mean and standard deviation**Mean (μ)

Equation : $\mu = 1.99 (\pm 0.264) - 0.076 (\pm 0.073) V/D - 0.693 (\pm 0.205) D/d_{50}$ (6.48a)

r^2 : 0.95

Stat. Significance : $\alpha = 5\%$ Standard deviation (σ)

Equation : $\sigma = 1.659 (\pm 0.480) - 0.719 (\pm 0.372) D/d_{50} - 0.114 (\pm 0.132) V/D$ (6.48b i)

r^2 : 0.66

Stat. Significance : Regression coefficient of V/D not significant at $\alpha = 5\%$

Equation : $\sigma = 1.329 (\pm 0.328) - 0.648 (\pm 0.403) D/d_{50}$ (6.48b ii)

r^2 : 0.53

Stat. Significance : $\alpha = 5\%$

Equations 6.47 (a and b ii) and 6.48 (a and b ii) suggest that both parameters of the Extreme Type I distribution, as well as the mean and standard deviation can be related to average hydraulic parameters at a significance level of 5 % and with relatively high r^2 values. However, although the scale parameter of the Weibull distribution shows a good correlation to several explanatory variables, the equation defining the shape parameter has a very low r^2 value.

6.3.4 Conclusion

Due to the spatial variability of local hydraulic conditions in cobble and boulder bed rivers and in light of the importance of this variability for the description of the aquatic habitat in freshwater ecosystems, a statistical model was developed for predicting the probability distribution of local flow velocities in cobble and boulder bed rivers during low flows.

It was demonstrated that the distribution of local sets of point velocities within pool, rapid and riffle morphological units respectively, may be related to simple, average cross-sectional hydraulic parameters. The regression models which were subsequently developed, suggest that knowledge of the average velocity, flow depth, flow width and relative roughness within a cross section, can be used to predict the frequency distribution of point velocity in these morphological units. However, before these equations are applied it is imperative that they be verified with independent data sets.

For plane bed morphological units, no statistically significant relationships were found to exist. This might be ascribed to inconsistent or inaccurate identification of plane bed morphological units, or suggests that the frequency distribution of point velocities within plane beds might be related to variables which were not considered, e.g. the local energy gradient, the standard deviation of bed particle size or the roughness concentration.

7. FINAL CONCLUSIONS AND RECOMMENDATIONS

7.1 Final conclusions

The objectives which were defined at the start of this research, have been met through the development of empirical, semi-empirical and theoretically based models, which address both at the macro scale and the local scale, ecologically significant morphological and hydraulic characteristics of cobble and boulder bed rivers. These models, which may serve as tools for determining the hydraulic and morphological interrelationships characteristic of different environmental flow components in cobble and boulder bed rivers, address:

- the regime characteristics of cobble and boulder bed rivers
- fine sand scouring in cobble beds
- velocity-depth relationships under conditions of large scale roughness

The main conclusions that have been reached are:

- The process of macro scale bed deformation in cobble and boulder bed rivers may be considered as a means of self adjustment towards obtaining dynamic equilibrium. The river bed transforms its profile into a series of consecutive macro scale bed forms in order to create natural control structures, each of which forces the water through critical depth with a subsequent hydraulic jump downstream. This deformation continues until equilibrium exists in terms of the power applied along the bed and the power required to suspend bed particles, as well as in terms of momentum exchange between the accelerating water along the riffle and the slower moving water in the pool. A theoretically based regime model has been developed which defines the relationship between a characteristic discharge (assumed to represent the channel forming discharge) and channel morphology (channel and bed form geometry, substrate size) in cobble and boulder bed rivers, and as such provides a first step towards predicting the long term morphological impacts of a modified flow regime.
- The process of scouring of fine sands in a cobble bed and the associated change in absolute bed roughness is similar to the process of bed deformation in a sand bed river, and may be defined by relationships similar to those that exist between absolute bed roughness and particle characteristics under conditions of dynamic equilibrium on a deformed sand bed river.
- A fundamental approach towards the estimation of mean velocity under conditions of large scale roughness led to the development of a relatively simple and generally applicable equation. The application of this new equation to cobble and boulder bed rivers

in the Western Cape, proved that it is able to provide a fairly accurate estimate of mean flow velocity under conditions of large scale roughness and can therefore be used to obtain first order estimates of mean velocity-depth relationships during the environmental flow assessment process.

- Probability distributions of point velocities within pool, rapid and riffle morphological units respectively, may be related to average cross-sectional hydraulic parameters.

7.2 Recommendations

- Although all the models that have been developed as part of this research programme have been calibrated with extensive data sets, they have not, due to time constraints and a lack of readily available data, been verified with independent data sets. Before application of these models, it is therefore imperative that:
 - the regime model is verified under actual channel forming discharge conditions.
 - the sand scour model is verified in the field, possibly by means of controlled reservoir releases.
 - the fundamental, theoretically based equation for calculating the mean velocity under large scale roughness conditions is verified with additional field data over a range of discharges and channel gradients.
 - the velocity distribution model be verified with independent field data.

As an additional verification exercise, it is recommended that the models are applied to rivers where an EFA has already been completed in order to determine the morphological and hydraulic characteristics of the specified environmental flow components.

- The sand scour model should be further refined by investigating the effects of absolute and non-uniform cobble sizes as well as steady state sediment transport conditions. It should be combined with hydraulic and sediment transport models to provide a comprehensive flushing flow strategy.
- In the case of the regime model, the effects of the duration of discharge and sediment transport need to be investigated and possibly incorporated into the model. The effects of discharges less than or exceeding the characteristic discharge need to be determined in order to provide a complete understanding of the behaviour of the channel in response to the whole range of discharges included in a typical flow regime.
- The benefits of introducing additional physical variables into the theoretically based equation for mean velocity under conditions of large scale roughness as well as the velocity distribution model (e.g. particle shape, roughness concentration etc.) should be investigated.

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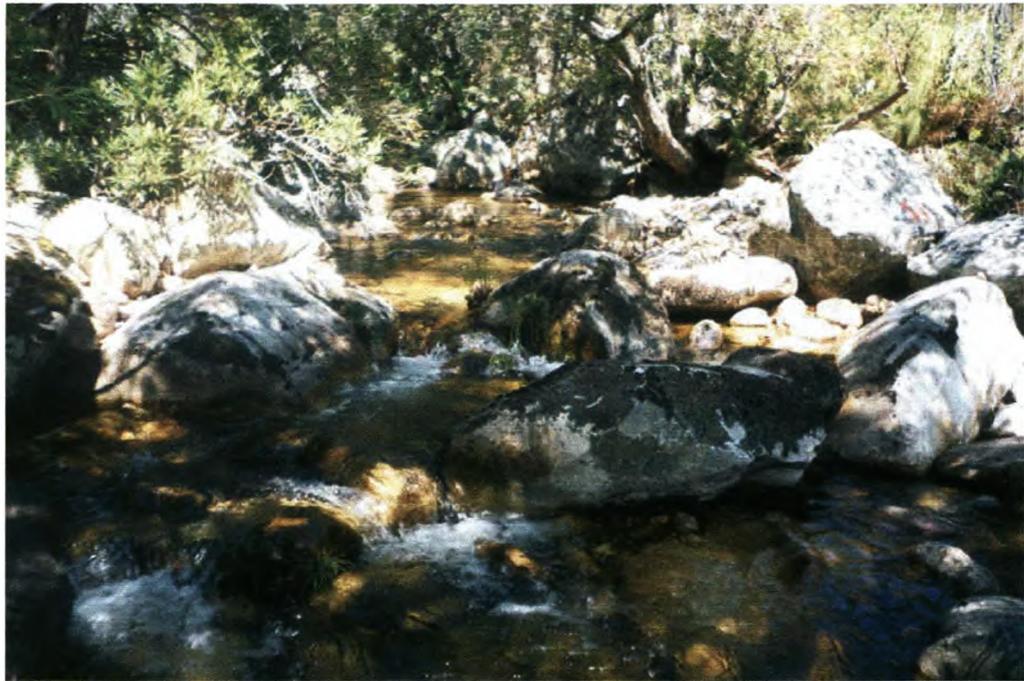
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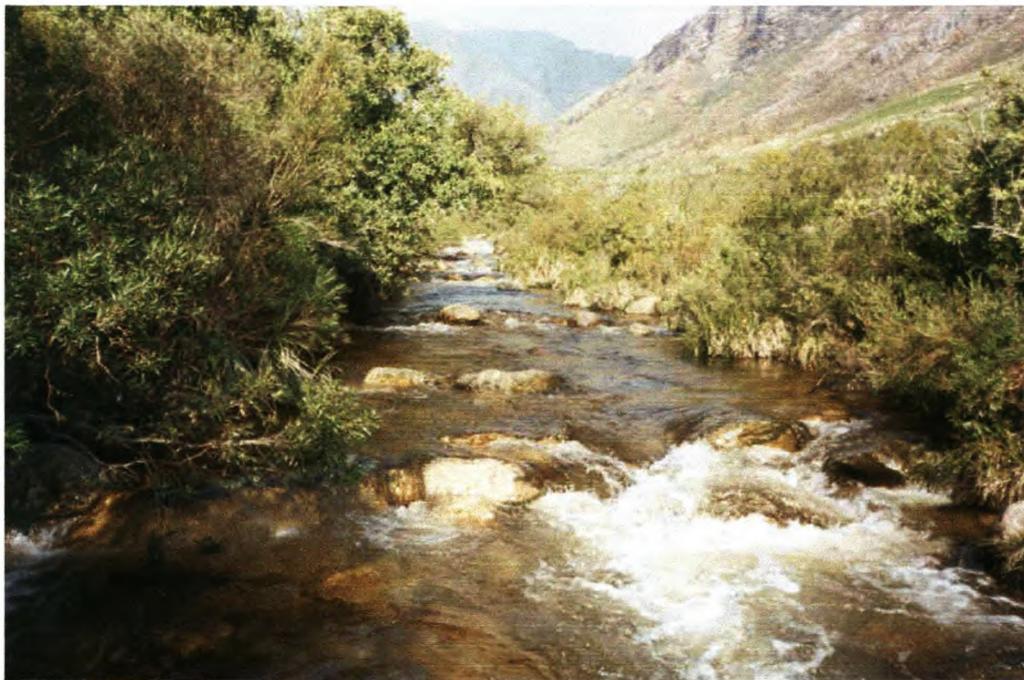
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APPENDIX A

Photographic record of study reaches



WHITEBRIDGE



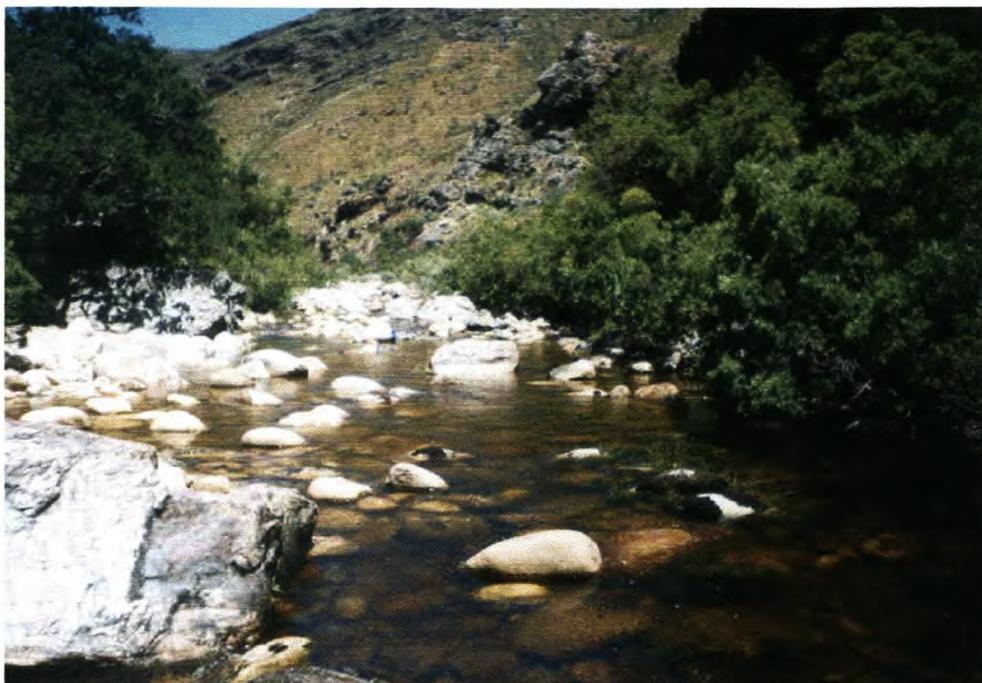
JONKERSHOEK



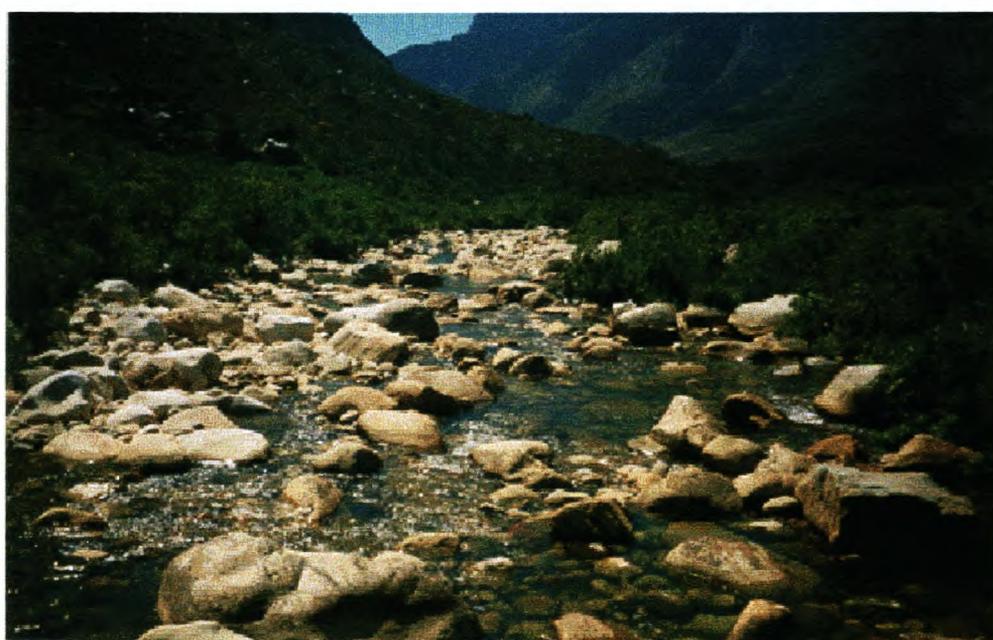
VERGENOEG



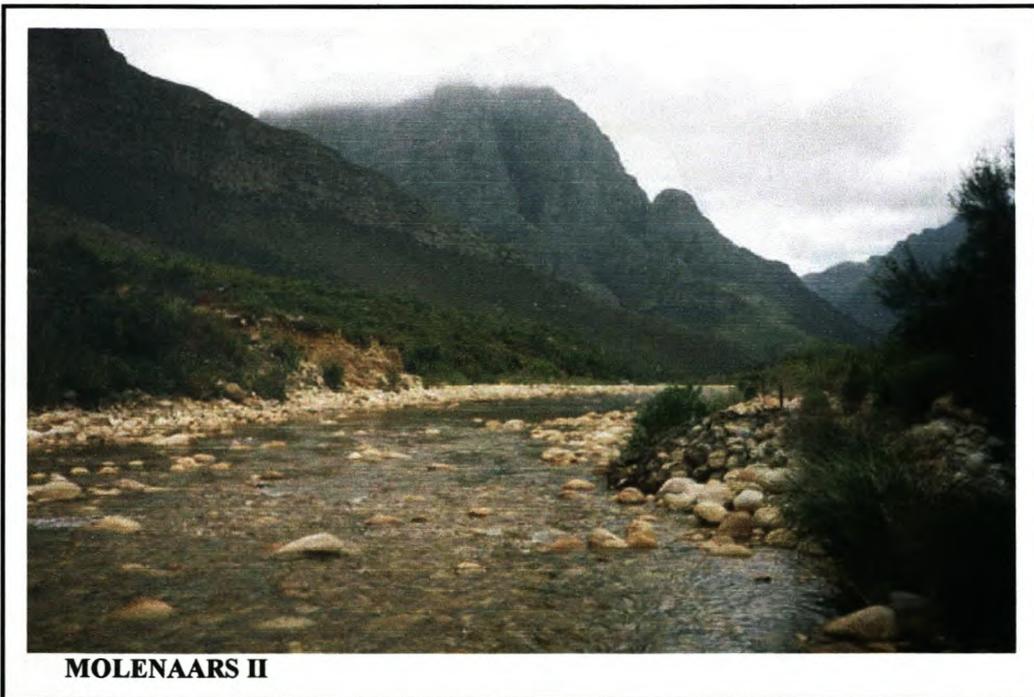
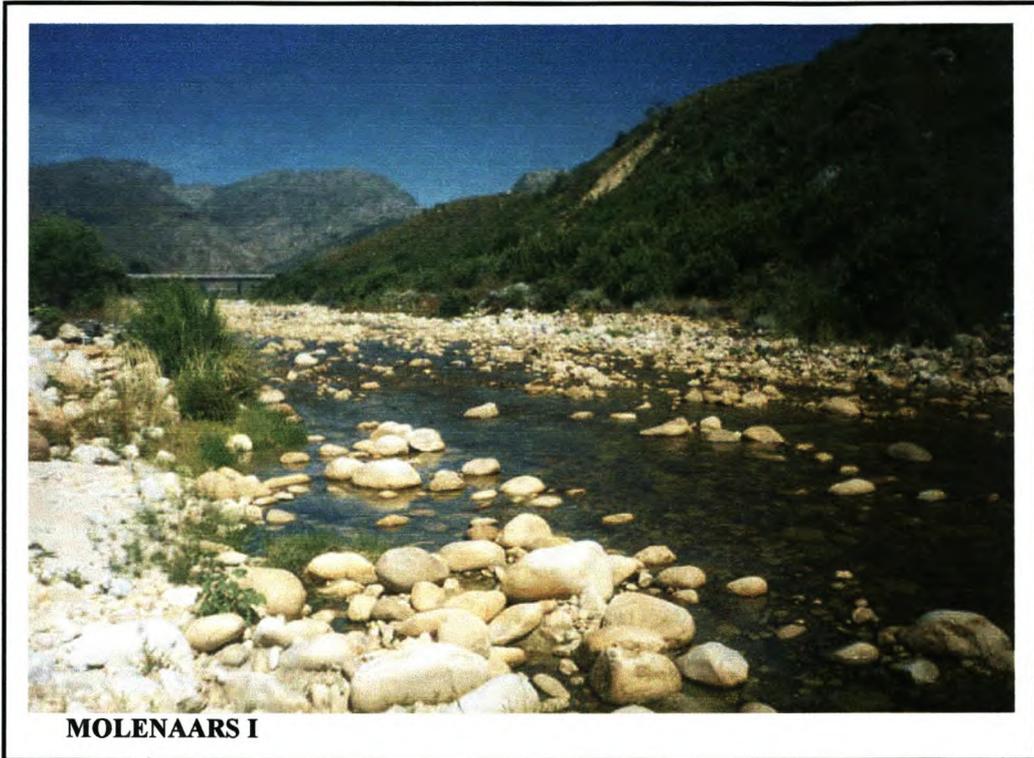
VLOTTENBURG

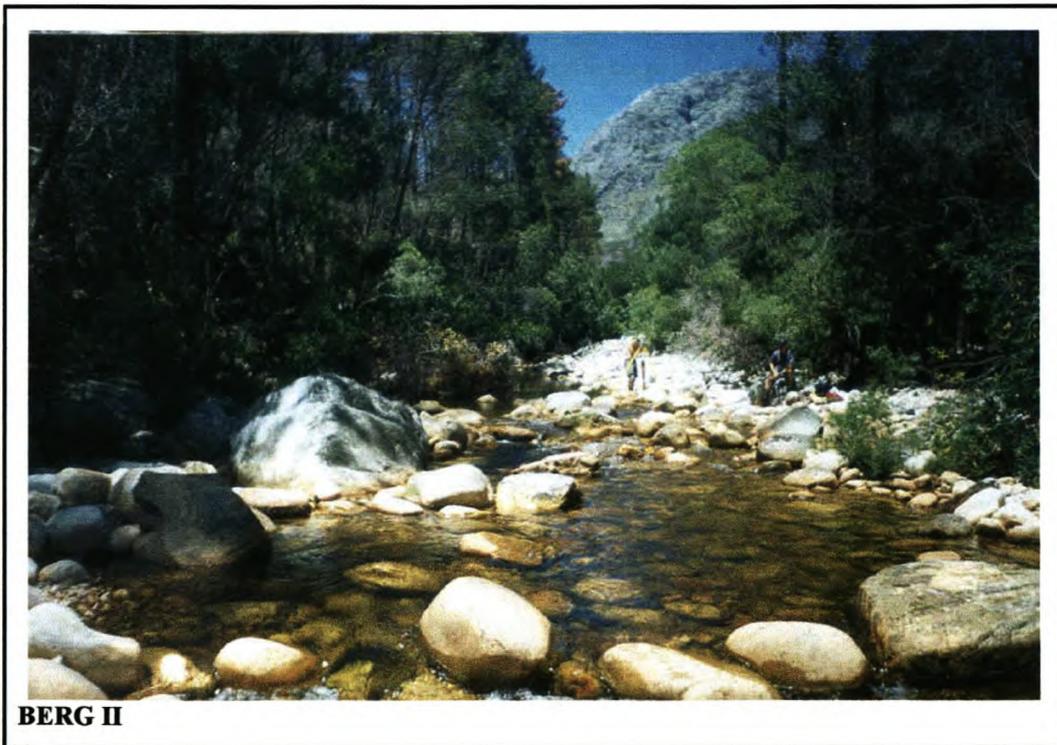
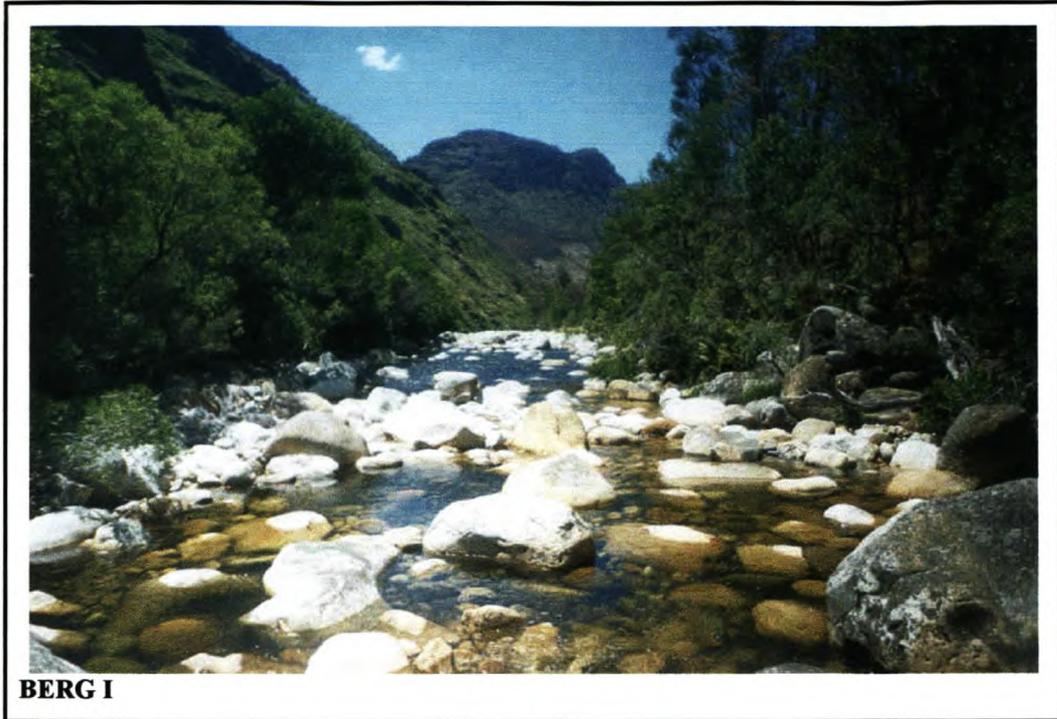


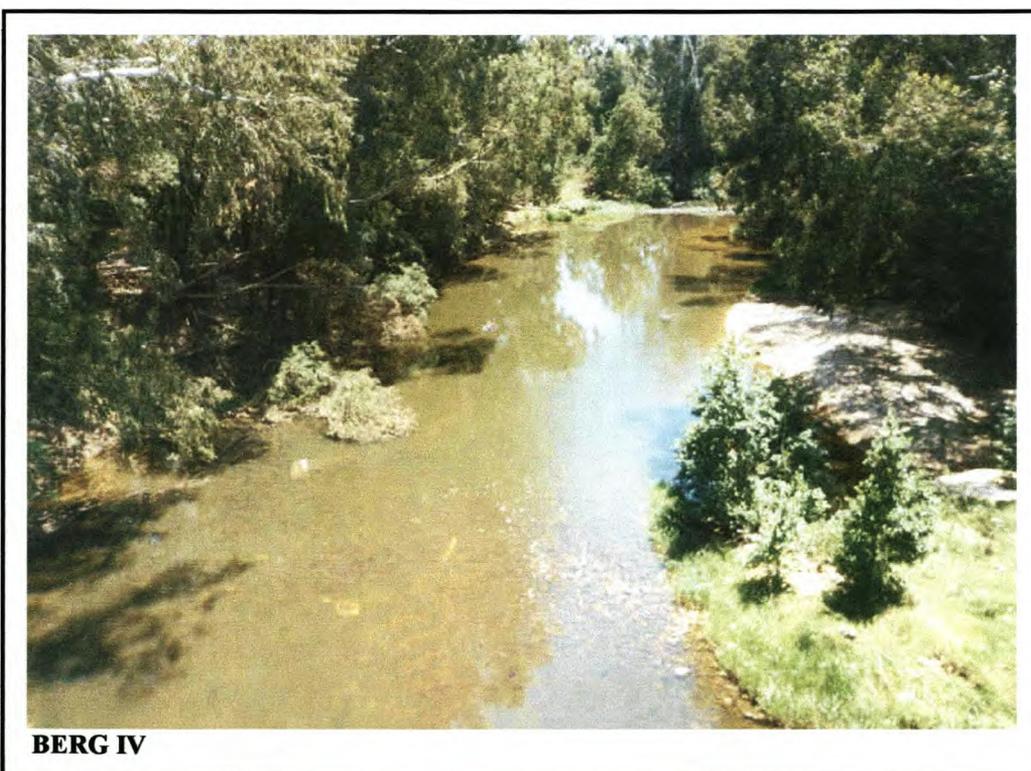
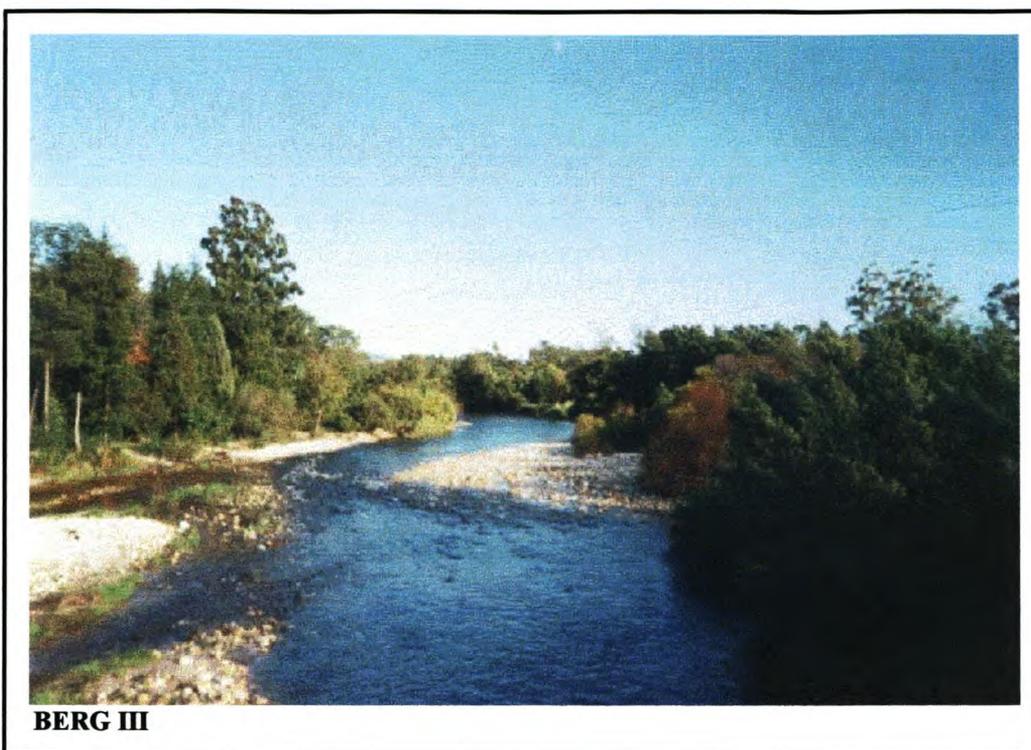
ELANDS



SMALBLAAR









DU TOITS

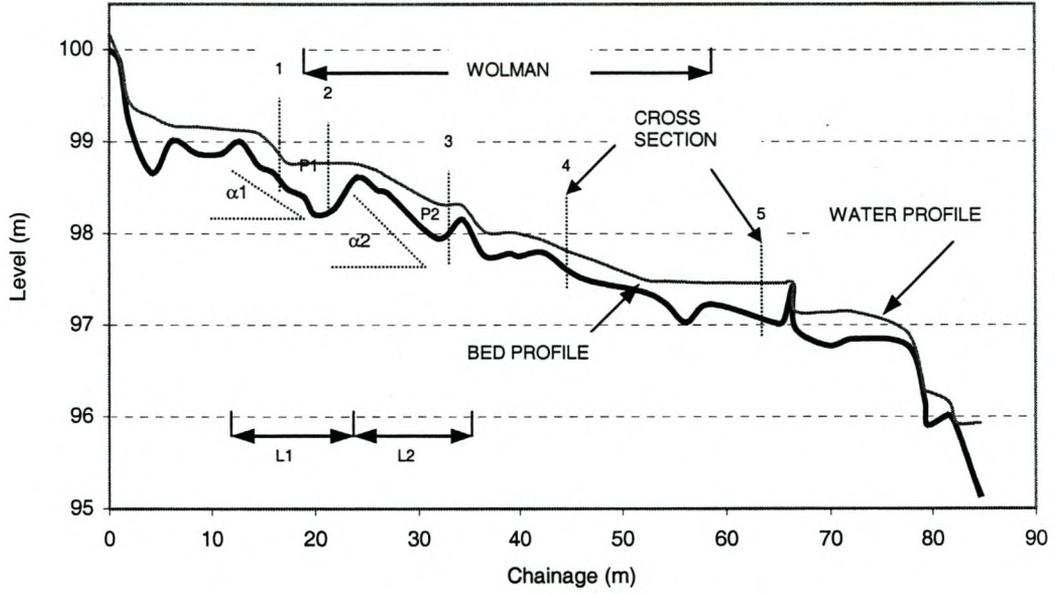
APPENDIX B

Longitudinal thalweg profiles Location of cross sections and Wolman sampling areas

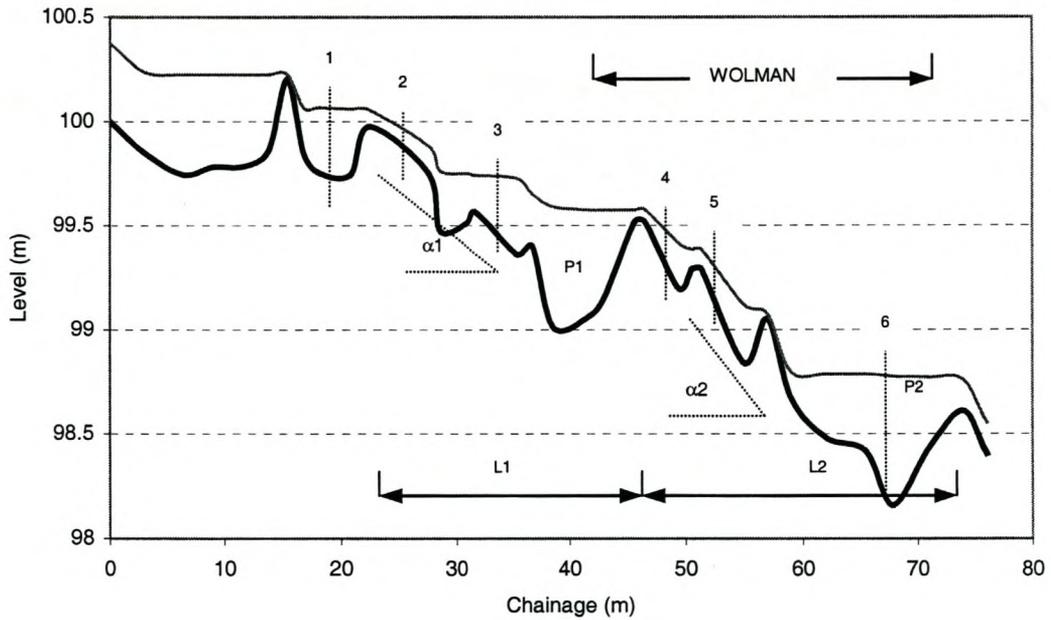
Notation

- α : rapid/riffle gradient
P : pool
L : bedform length

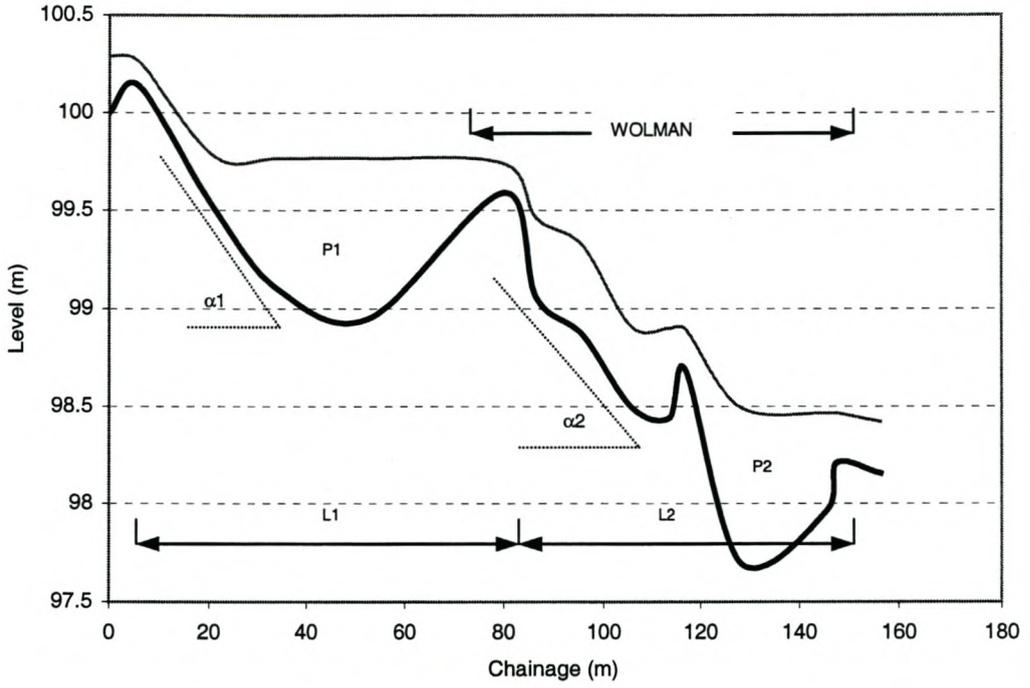
WHITEBRIDGE



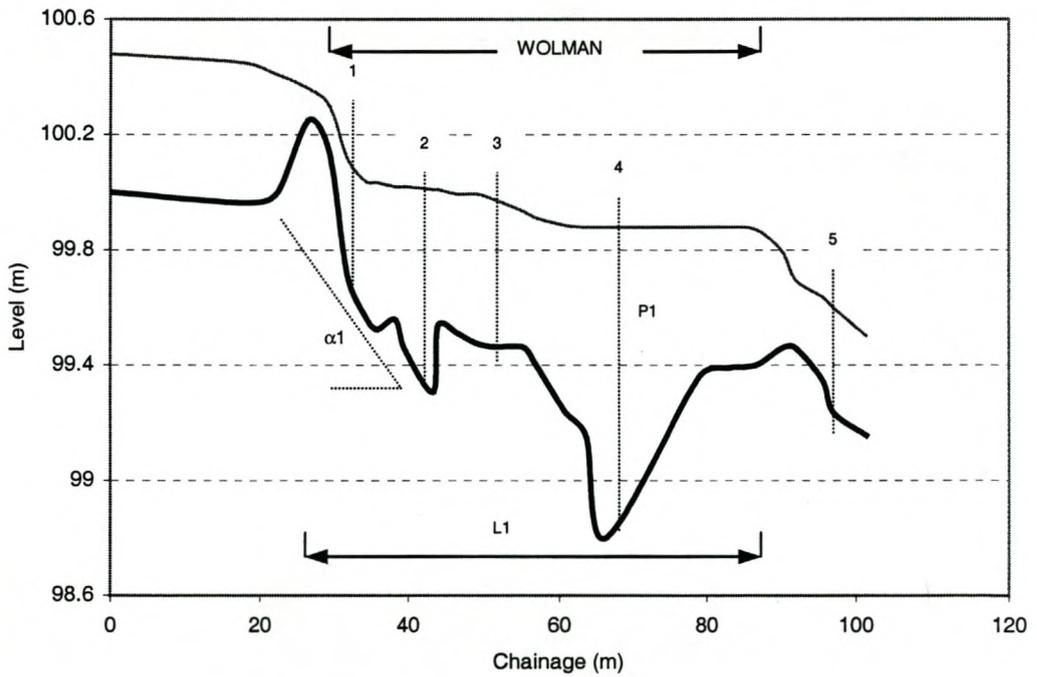
JONKERSHOEK



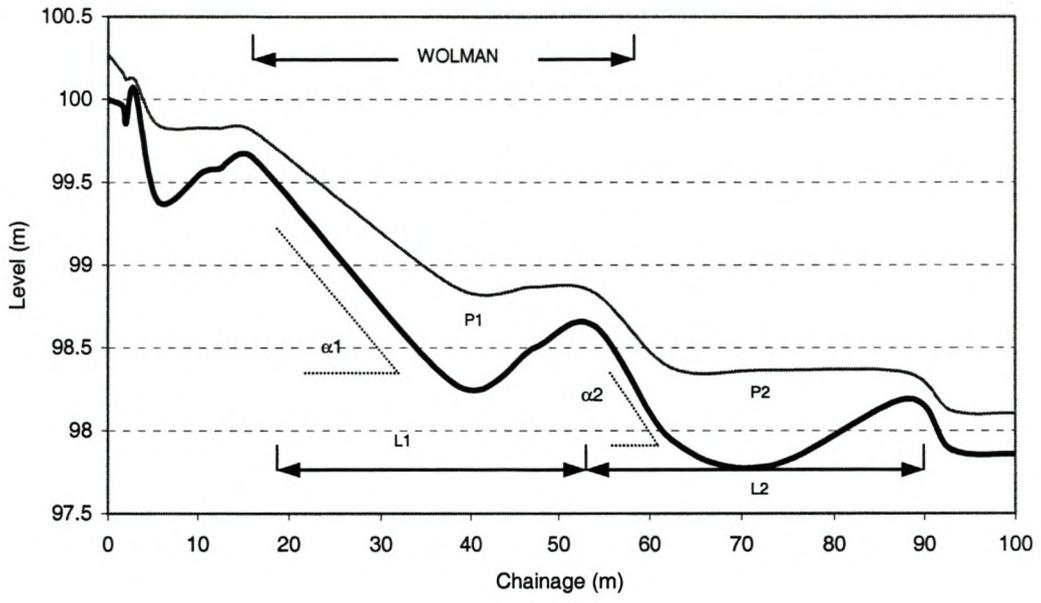
MOLENAARS II



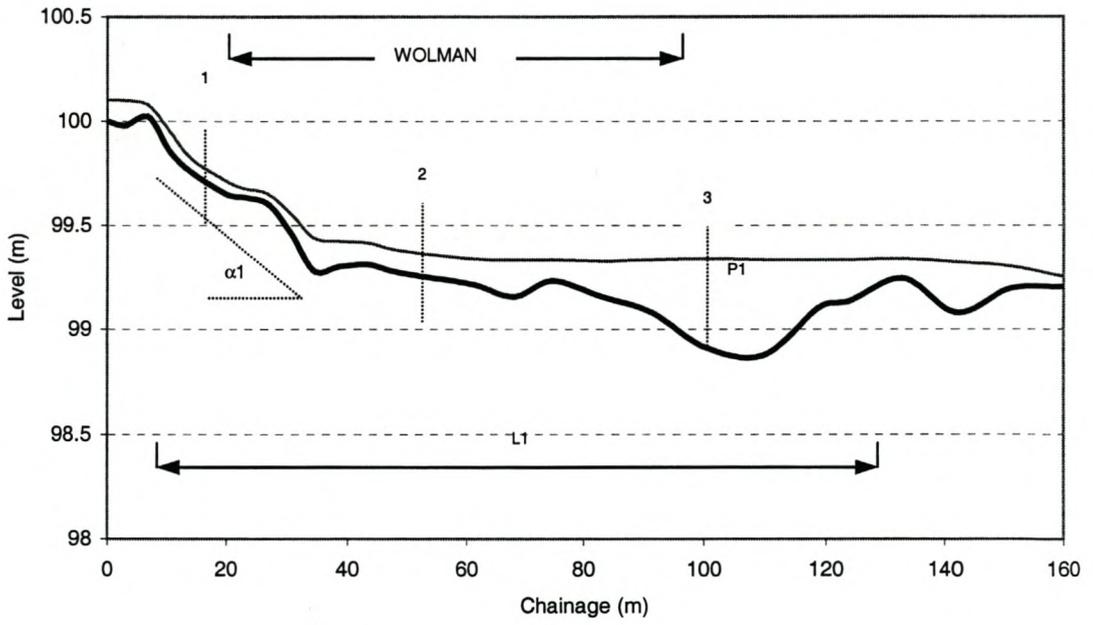
BERG II



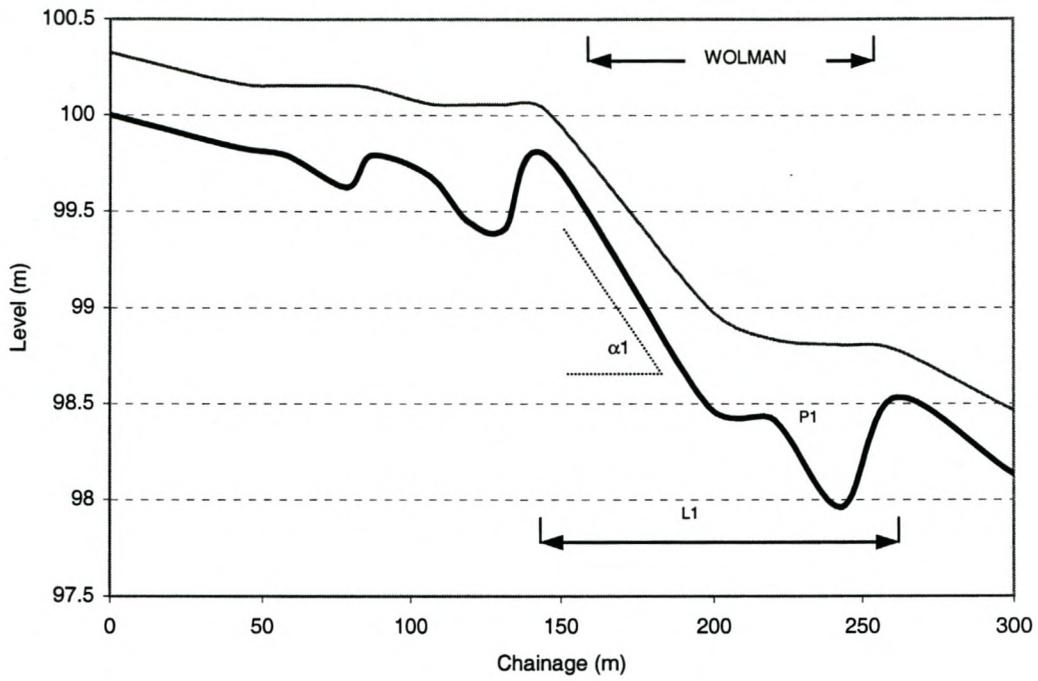
SMALBLAAR



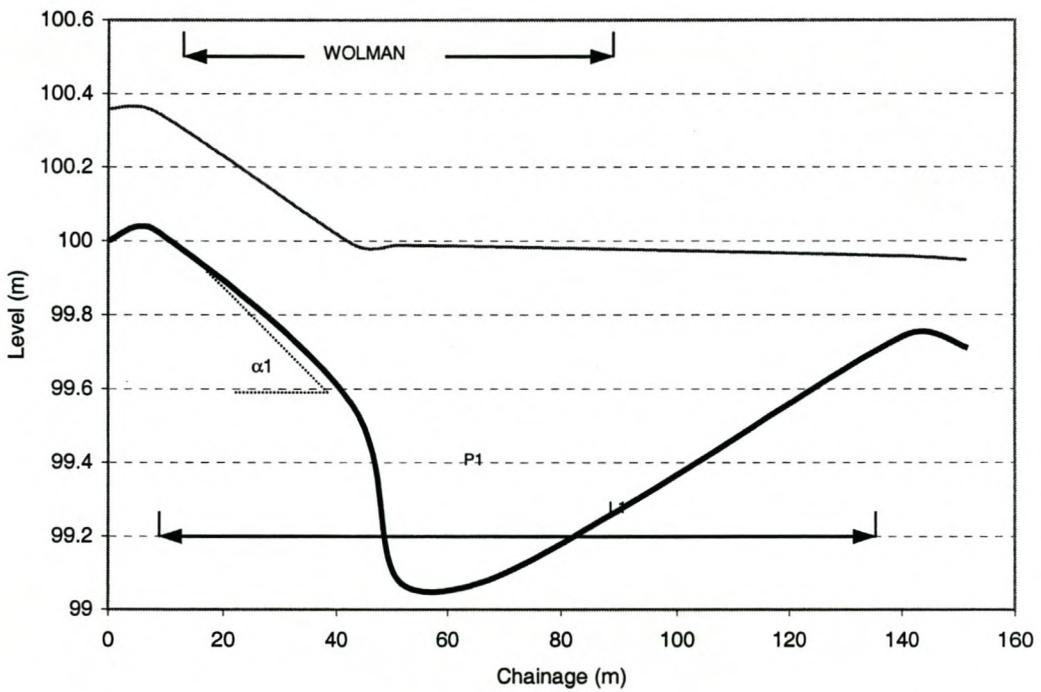
MOLENAARS I



BERG III

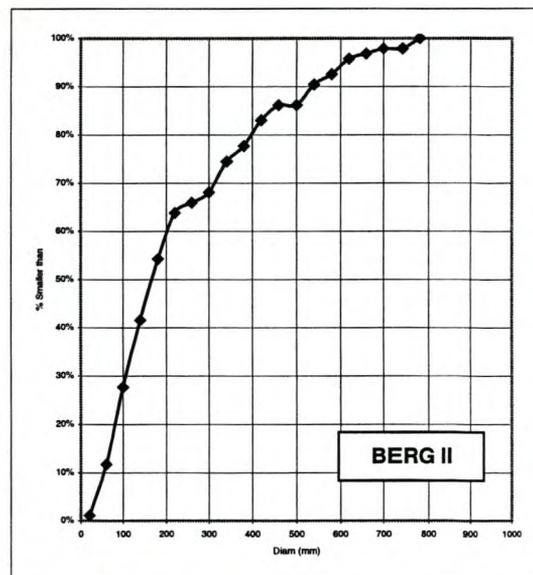
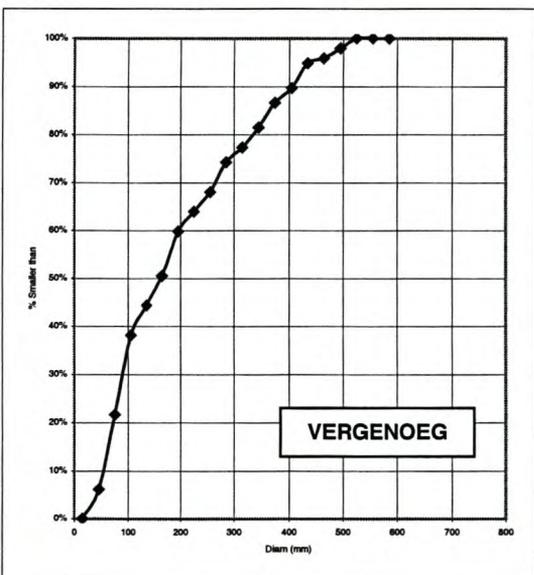
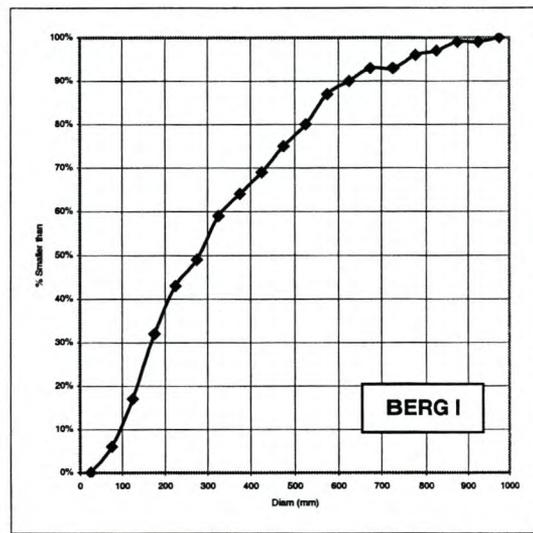
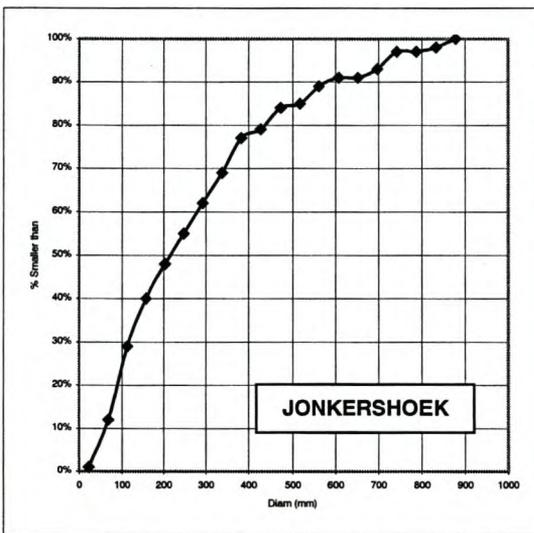
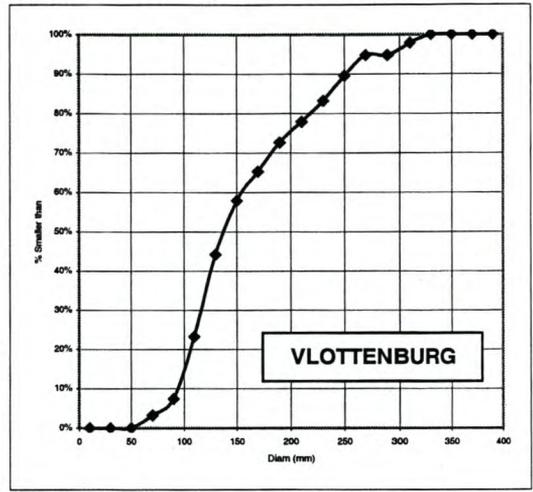
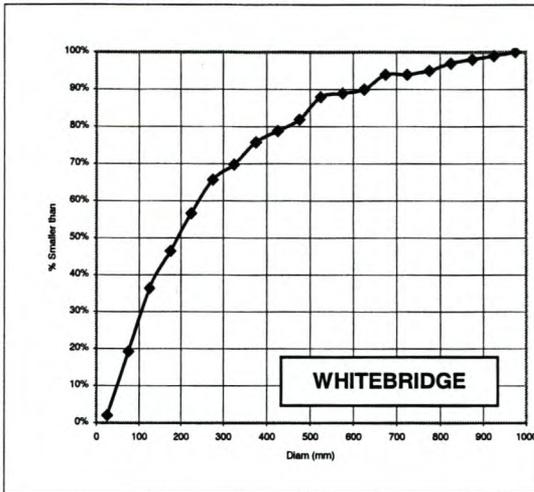


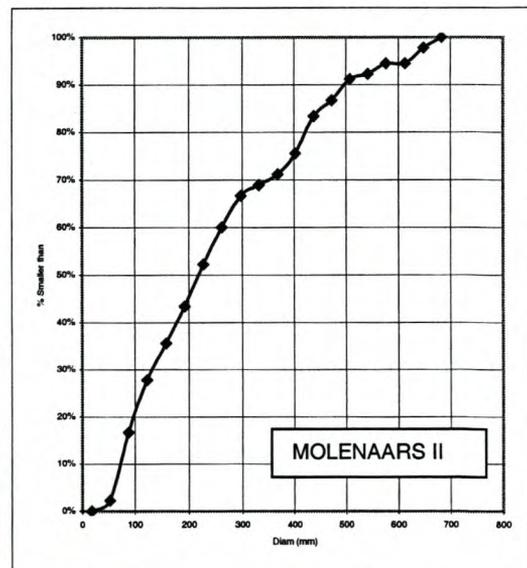
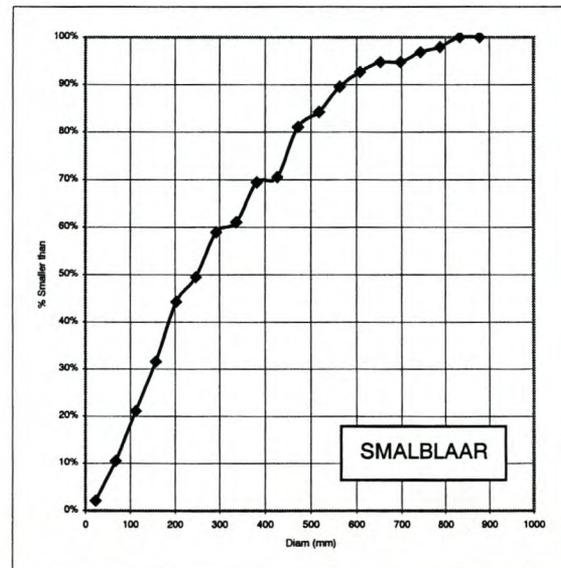
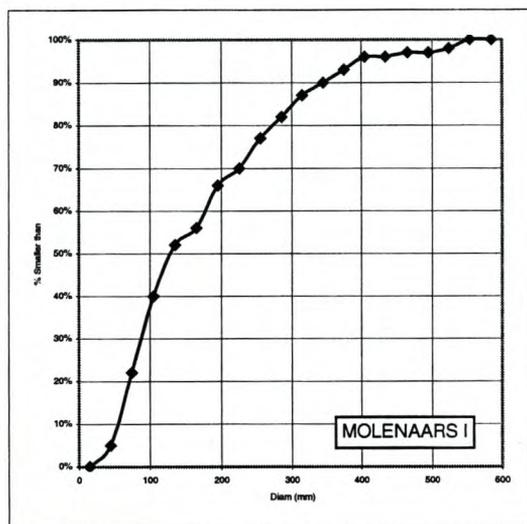
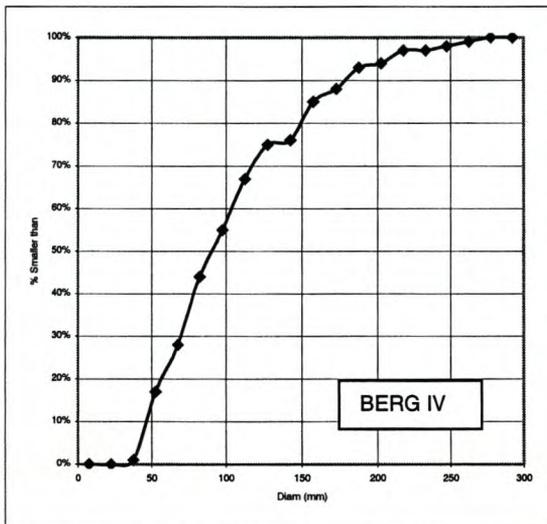
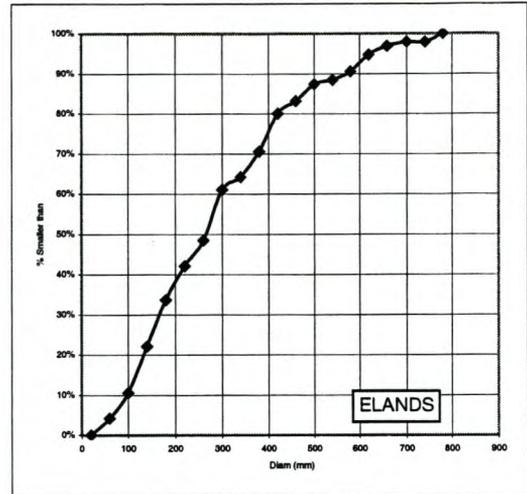
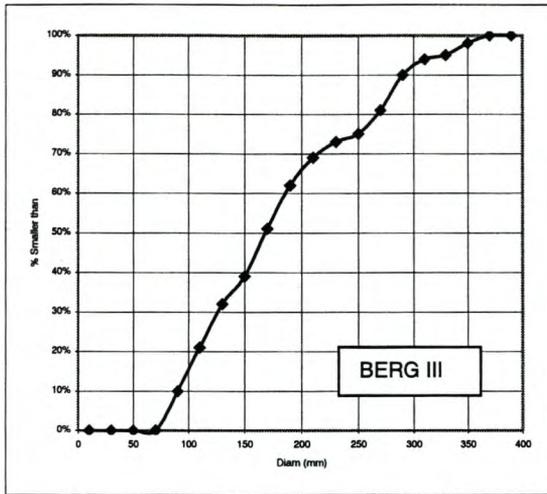
BERG IV

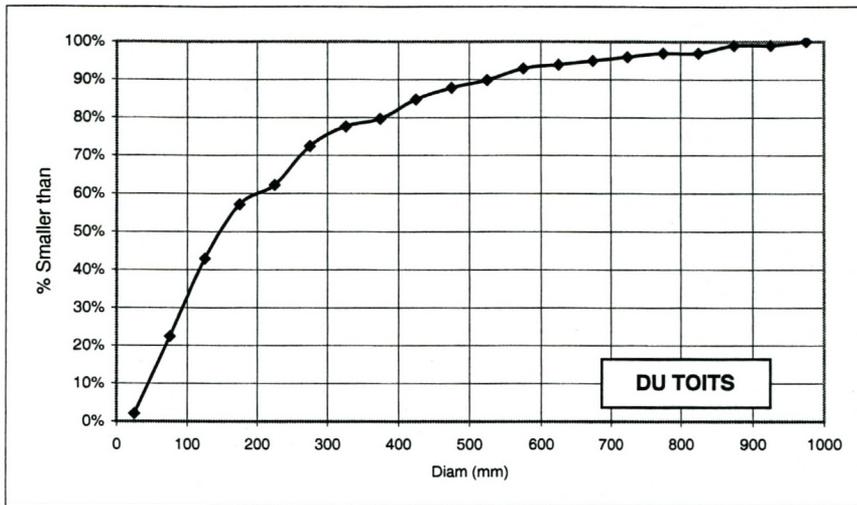


APPENDIX C

Substrate size distributions







APPENDIX D

Sample calculation of channel forming discharge

Calculation of channel forming discharge based on measured channel and bed form geometry

Study reach: Vlottenburg

Known variables

Channel width (W_{ch}): 22 m
 Avg. reach gradient (S_0): 0.0074
 Local riffle gradient (α_r): 0.024
 d_{50} : 0.14 m
 Bedform length (L): 95 m

1. Estimate a discharge, say

$$Q = 180 \text{ m}^3/\text{s}$$

2. Calculate the critical flow depth at the bed form crest (eq. 4.18)

$$y_c = \sqrt[3]{\frac{Q^2}{gW_{ch}^2}} = \sqrt[3]{\frac{180^2}{9.81 * 22^2}} = 1.90 \text{ m}$$

The flow velocity at the bed form crest (V_c) therefore equals

$$\frac{Q}{W_{ch} \cdot y_c} = \frac{180}{22 * 1.90} = 4.32 \text{ m/s}$$

and the specific energy at the bed form crest (E_c) equals

$$1.5 y_c = 1.5 (1.90) = 2.85 \text{ m.}$$

In order to calculate the energy gradient at the bed form crest, the value of the Darcy Weisbach resistance coefficient (f) is first calculated from the Griffiths (1981) equation (eq. 6.11):

$$\sqrt{1/f} = 1.98 \log (R/d_{50}) + 0.76$$

and assuming that $R \approx y_c$, an f -value of 0.111 is calculated.

The energy gradient at the bed form crest (S_c) can now be calculated, i.e.

$$S_c = \frac{V_c^2 f}{8gy_c} = \frac{4.32^2 * 0.111}{8 * 9.81 * 1.90} = 0.014.$$

3. Calculate the uniform flow depth along the riffle (y_n) by solving the following equation (eq. 4.19):

$$Q_c = y_n W_{ch} \sqrt{\frac{8gy_n S_f}{f}}$$

i.e.

$$180 = y_n * 22 \sqrt{\frac{8 * 9.81 * y_n * 0.024}{f}}$$

(The value of f in the above equation is again calculated from the Griffiths (1981) equation, assuming that $R \approx y_n$. This gives an f -value of 0.12.)

A value for y_n of 1.63 m is then calculated.

The flow velocity at a flow depth of y_n equals

$$\frac{Q}{W_{ch} \cdot y_n} = \frac{180}{22 \cdot 1.63} = 5.02 \text{ m/s}$$

and the specific energy (E_n)

$$y_n + \frac{V_n^2}{2g} = 1.63 + \frac{5.02^2}{2 \cdot 9.81} = 2.92 \text{ m}$$

4. Calculate z_1 (refer to Figure 4.7) from the following equation (eq. 4.9)

$$z_1 = \frac{\left(y_n + \frac{V_n^2}{2g} - y_c - \frac{V_c^2}{2g} \right)}{1 - \frac{0.5}{\alpha_r} \left(\frac{V_c^2 f}{8gy_c} + \frac{V_n^2 f}{8gy_n} \right)} = \frac{(2.92 - 2.85)}{1 - \frac{0.5}{0.024} (0.014 + 0.024)}$$

$$= 0.336 \text{ m}$$

5. Calculate y_s (eq. 4.10):

$$y_s = \frac{y_n}{2} \left[\sqrt{1 + \frac{8V_n^2}{gy_n}} - 1 \right] = \frac{1.63}{2} \left[\sqrt{1 + \frac{8 \cdot 5.02^2}{9.81 \cdot 1.63}} - 1 \right]$$

$$= 2.20 \text{ m}$$

6. Calculate:

$$y_4 = 1.3 \cdot y_s = 2.86 \text{ m (eq. 4.11)}$$

$$V_4 = \frac{Q}{W_{ch} \cdot y_4} = \frac{180}{22 \cdot 2.86} = 2.87 \text{ m/s}$$

$$L_2 = 0.82 y_s (\alpha_r)^{-0.78} = 33.0 \text{ m (eq. 4.12)}$$

$$z_2 = L_2 \alpha_r = 0.79 \text{ m (eq. 4.15)}$$

$$\Delta z = (y_4 + \frac{V_4^2}{2g} - y_c - \frac{V_c^2}{2g}) = 0.43 \text{ m (eq. 4.13)}$$

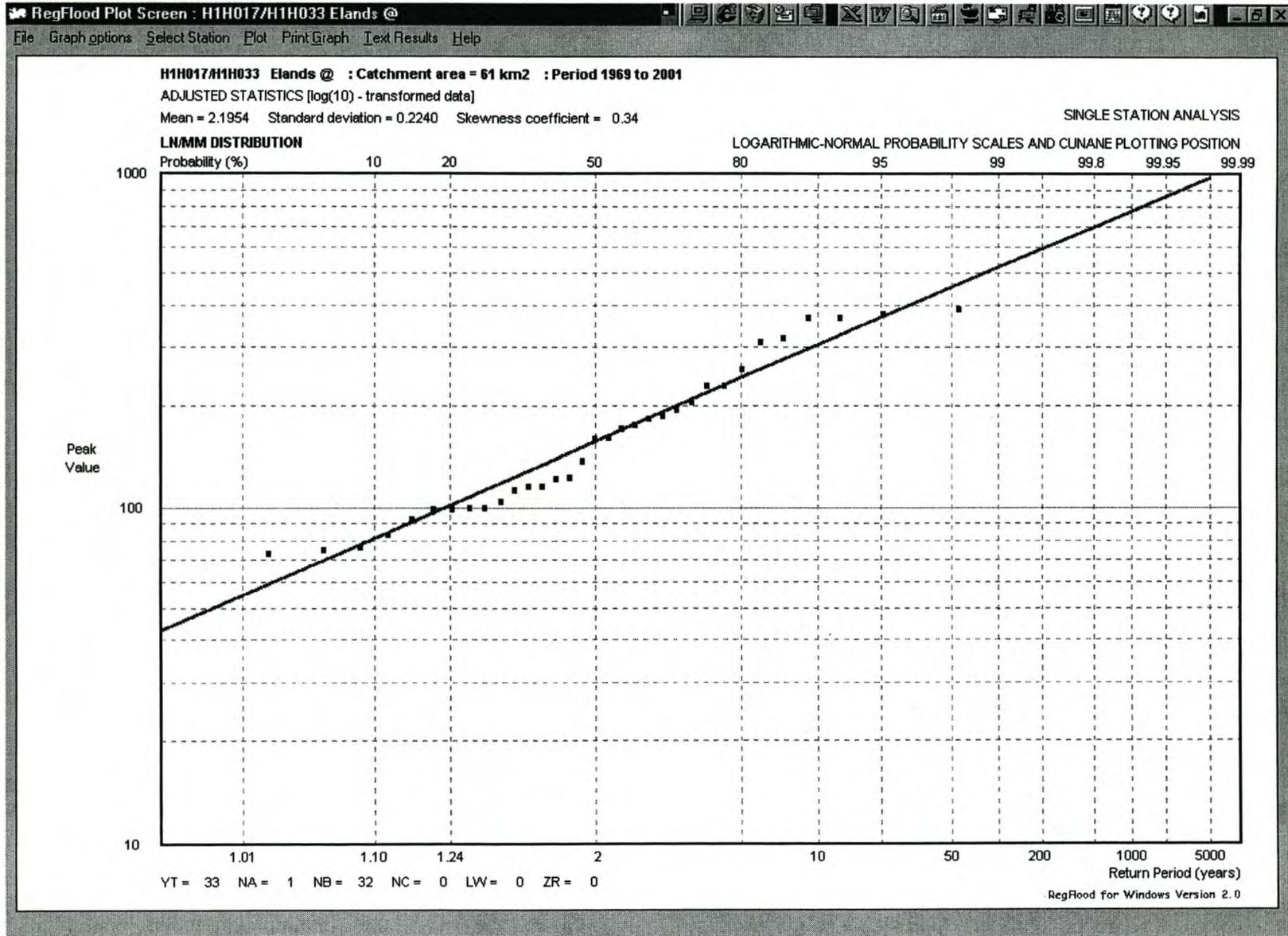
$$z = z_1 + z_2 - \Delta z = 0.70 \text{ m (eq. 4.14)}$$

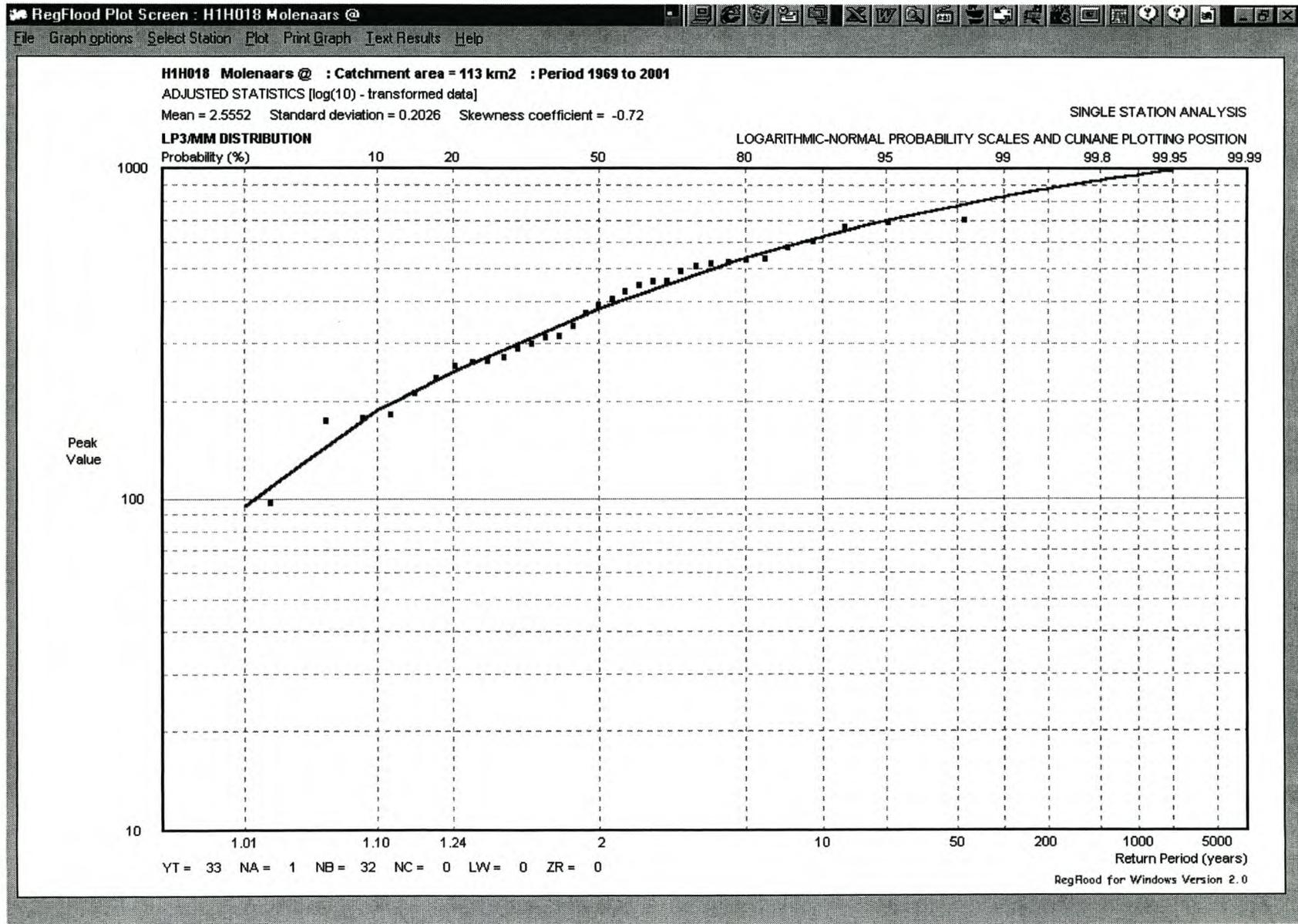
$$L = z / S_0 = 94.6 \text{ m (eq. 4.16)}$$

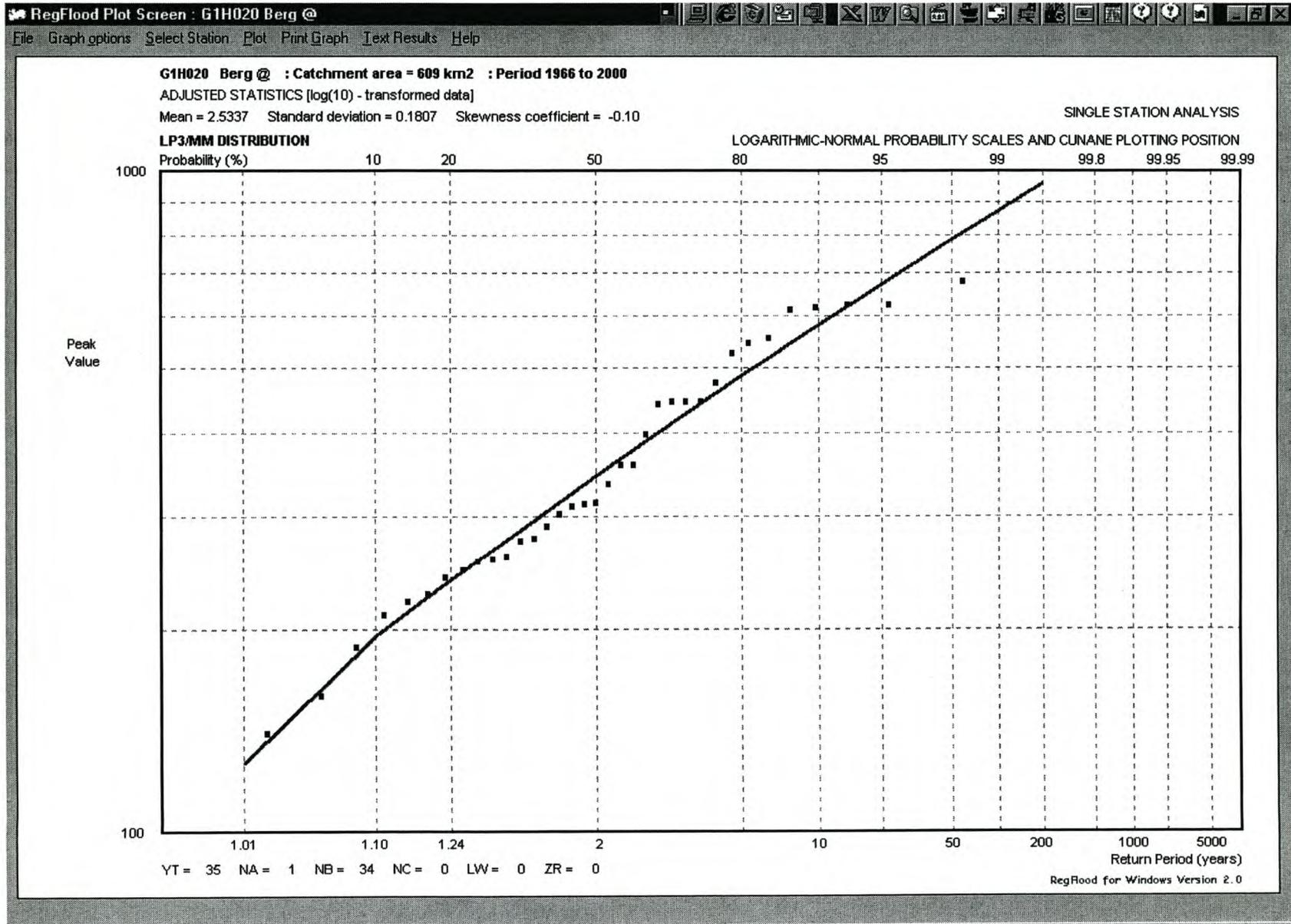
QED

APPENDIX E

Flood frequency analysis results







APPENDIX F

Sand scour experimental results

SCOUR RATE DATA								
Exp No.	Time (min)	Scour level	Diameter		Avg. flow depth (m)	Discharge (m ³ /s)	Avg. velocity (m/s)	Scour depth (mm)
			Sand (m)	Cobble (m)				
46	0	261						0
	25	251						10
	55	239	0.00054	0.08	0.200	0.109	0.545	22
	85	235						26
	105	231						30
47	0	268						0
	10	227						41
	20	219	0.00054	0.08	0.273	0.226	0.828	49
	40	215						53
	58	211						50
48	0	263						0
	20	240						23
	50	234	0.00054	0.08	0.406	0.237	0.584	26
	70	230						27
	90	228						29
49	0	269						0
	5	228						41
	10	218	0.00054	0.08	0.278	0.256	0.921	51
	30	210						59
	40	201						59
50	0	271						0
	10	255						16
	20	241	0.00054	0.08	0.567	0.333	0.587	30
	60	227						33
	110	224						36
51	0	268						0
	10	233						35
	30	224	0.00054	0.08	0.405	0.387	0.956	44
	50	216						46
52	0	267						0
	5	255						12
	35	241	0.00054	0.08	0.448	0.346	0.772	26
	55	232						35
	75	223						44
	95	216						45
53	0	260						0
	10	244						16
	20	228	0.00054	0.08	0.698	0.525	0.752	32
	50	221						39
	90	219						42

SCOUR RATE DATA (cont.)							
Exp No.	Time (min)	Diameter		Avg. flow depth (m)	Discharge (m ³ /s)	Avg. velocity (m/s)	Scour depth (mm)
		Sand (m)	Cobble (m)				
57	0						0
	5						73
	12	0.00054	0.130	0.339	0.332	0.979	80
	19						85
	30						91
	35						94
56	0						0
	7						35
	21	0.00054	0.130	0.308	0.236	0.766	67
	37						71
55	0						0
	6						2
	20						20
	45	0.00054	0.130	0.289	0.162	0.561	35
	60						45
	80						50
90						53	
61	0						0
	15						12
	20						21
	24	0.00022	0.130	0.235	0.120	0.511	28
	28						31
	34						33
38						36	
62	0						0
	5						11
	8						32
	12						42
	17	0.00022	0.130	0.268	0.160	0.597	43
	22						45
	32						48
	45						52
63	0						0
	3						4
	7						21
	9	0.00022	0.130	0.340	0.243	0.715	28
	11						33
	14						41
	17						45
	30						64

MAXIMUM SCOUR DEPTHS / EQUILIBRIUM SCOUR RESULTS							
No.	Diameter		*Flow			*Scour	
	Sand (m)	Cobble (m)	Depth (m)	Discharge (m ³ /s)	*Time (min)	*Velocity (m/s)	Depth (m)
1	0.00022	0.06	0.400	0.200	70	0.500	0.008
2	0.00022	0.06	0.071	0.025	45	0.354	0.015
3	0.00022	0.06	0.600	0.300	90	0.500	0.015
4	0.00022	0.06	0.200	0.100	105	0.500	0.030
5	0.00022	0.06	0.500	0.300	55	0.600	0.030
6	0.00022	0.06	0.600	0.400	50	0.667	0.040
7	0.00022	0.06	0.800	0.600	45	0.750	0.040
8	0.00022	0.06	0.107	0.050	75	0.467	0.045
9	0.00022	0.06	0.150	0.100	50	0.667	0.045
10	0.00022	0.06	0.300	0.200	50	0.667	0.045
11	0.00022	0.06	0.800	0.500	85	0.625	0.045
12	0.00022	0.06	0.250	0.200	35	0.800	0.060
13	0.00022	0.06	0.400	0.300	40	0.750	0.060
14	0.00022	0.06	0.500	0.400	45	0.800	0.060
15	0.00022	0.06	0.600	0.500	35	0.833	0.060
16	0.00022	0.06	0.300	0.300	45	1.000	0.060
17	0.00022	0.06	0.501	0.500	60	0.998	0.060
18	0.00022	0.06	0.600	0.600	40	1.000	0.060
19	0.00022	0.06	0.400	0.400	35	1.000	0.060
20	0.00022	0.06	0.500	0.600	40	1.200	0.060
21	0.00022	0.06	0.407	0.500	35	1.229	0.060
22	0.00022	0.071	0.123	0.050	40	0.408	0.036
23	0.00022	0.071	0.201	0.099	45	0.494	0.036
24	0.00022	0.071	0.400	0.200	70	0.500	0.036
25	0.00022	0.071	0.804	0.502	75	0.624	0.053
26	0.00022	0.071	0.299	0.200	55	0.669	0.071
27	0.00022	0.071	0.601	0.403	40	0.671	0.071
28	0.00022	0.071	0.600	0.499	60	0.832	0.071
29	0.00022	0.071	0.150	0.100	48	0.667	0.071
30	0.00022	0.071	0.251	0.200	38	0.797	0.071
31	0.00022	0.071	0.502	0.403	45	0.802	0.071
32	0.00022	0.071	0.402	0.403	45	1.003	0.071
33	0.00083	0.06	0.305	0.200	46	0.656	0.020
34	0.00083	0.06	0.499	0.295	51	0.592	0.020
35	0.00083	0.06	0.703	0.505	50	0.718	0.020
36	0.00083	0.06	0.400	0.295	41	0.738	0.030
37	0.00083	0.06	0.599	0.394	39	0.658	0.030
38	0.00083	0.06	0.601	0.505	48	0.841	0.030
39	0.00083	0.06	0.501	0.394	41	0.786	0.040
40	0.00083	0.06	0.709	0.600	50	0.846	0.040
41	0.00083	0.06	0.397	0.394	39	0.992	0.045
42	0.00083	0.06	0.502	0.505	42	1.006	0.045
43	0.00083	0.06	0.303	0.295	48	0.973	0.060
44	0.00083	0.06	0.603	0.600	37	0.995	0.060
45	0.00083	0.06	0.497	0.602	34	1.212	0.060
46	0.00054	0.08	0.200	0.109	105	0.545	0.029
47	0.00054	0.08	0.273	0.226	58	0.828	0.049
48	0.00054	0.08	0.406	0.237	90	0.584	0.028
49	0.00054	0.08	0.278	0.256	35	0.921	0.059
50	0.00054	0.08	0.567	0.333	110	0.587	0.036
51	0.00054	0.08	0.405	0.387	50	0.956	0.046
52	0.00054	0.08	0.448	0.346	95	0.772	0.045
53	0.00054	0.08	0.698	0.525	90	0.752	0.042
54	0.00054	0.08	0.374	0.465	-	1.243	0.080
55	0.00054	0.13	0.289	0.162	90	0.561	0.053
56	0.00054	0.13	0.308	0.236	40	0.766	0.071
57	0.00054	0.13	0.339	0.332	35	0.979	0.095
58	0.00054	0.13	0.321	0.186	-	0.579	0.041
59	0.00054	0.13	0.320	0.228	-	0.713	0.063
60	0.00054	0.13	0.399	0.530	-	1.328	0.122
61	0.00022	0.13	0.235	0.120	40	0.511	0.036
62	0.00022	0.13	0.268	0.160	45	0.597	0.053
63	0.00022	0.13	0.340	0.243	30	0.715	0.064
64	0.00022	0.13	0.399	0.390	-	0.977	0.121

*Equilibrium

APPENDIX G

Large scale roughness data from literature

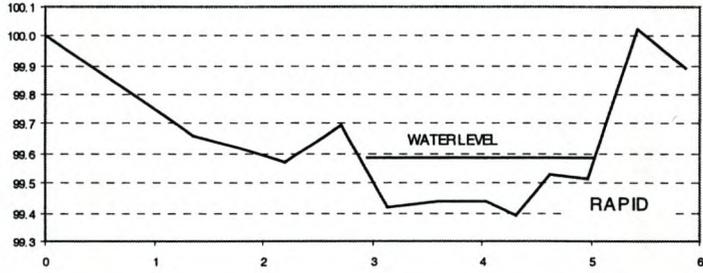
Reference	d50 (m)	Q(m ³ /s)	Area (m ²)	R(m)	Sf(%)	
Bathurst(1978)	0.28	0.90	2.46	0.17	1.74	
	0.28	3.90	5.62	0.31	1.71	
	0.28	7.20	8.04	0.40	1.66	
	0.21	1.37	5.59	0.21	1.14	
	0.21	4.00	8.98	0.28	1.15	
	0.21	7.10	11.32	0.34	1.16	
	0.19	1.10	3.72	0.20	0.80	
	0.19	4.00	7.42	0.33	0.81	
	0.19	7.10	9.45	0.40	0.81	
	Bathurst (1985)	0.34	6.74	12.30	0.46	1.28
0.34		10.30	15.50	0.57	1.28	
0.34		23.30	24.20	0.88	1.56	
0.26		2.00	3.81	0.28	3.73	
0.26		2.69	4.36	0.30	3.60	
0.26		15.90	10.80	0.63	3.14	
0.26		24.80	15.80	0.84	3.64	
0.13		2.07	4.58	0.30	1.30	
0.13		3.30	6.22	0.40	1.06	
0.13		12.20	10.70	0.66	1.19	
0.13		25.50	13.60	0.83	1.13	
0.13		31.60	14.80	0.89	1.54	
0.13		102.00	27.40	1.31	1.06	
0.09		0.62	1.97	0.14	0.50	
0.09		11.30	6.96	0.47	0.83	
0.09		76.90	21.50	0.96	1.04	
Thorne & Zev. (1985)		0.16	2.05	3.67	0.35	1.43
		0.16	3.28	4.37	0.40	1.51
	0.16	5.27	5.32	0.47	1.63	
	0.16	8.64	6.71	0.57	1.47	
	0.16	10.45	7.38	0.62	1.46	
	0.13	2.05	3.96	0.29	1.83	
	0.13	3.34	4.70	0.33	1.93	
	0.13	4.28	5.10	0.35	1.89	
	0.13	5.27	5.92	0.39	1.90	
	0.13	6.91	6.85	0.49	1.85	
	0.13	8.64	7.27	0.45	1.94	
	0.13	10.45	8.22	0.50	1.98	
	Jarret (1984)	0.18	1.50	4.00	0.31	1.50
0.18		6.06	6.60	0.46	1.70	
0.18		10.20	9.48	0.61	1.80	
0.18		21.67	13.11	0.79	1.90	
0.15		0.88	1.95	0.27	3.00	
0.15		3.26	3.35	0.37	3.40	
0.15		7.96	4.00	0.46	3.30	
0.30		4.19	6.32	0.37	1.90	
0.30		23.51	13.66	0.65	2.30	
0.30		38.53	17.20	0.77	2.40	
0.12		1.36	2.97	0.18	2.60	
0.12		2.61	4.28	0.24	2.60	
0.12		9.38	8.46	0.43	2.50	
0.12		11.59	11.81	0.59	2.10	
0.43		26.20	23.15	0.99	2.60	
0.43		41.08	31.60	1.22	2.30	
0.43		60.06	37.83	1.36	2.10	
0.43		78.19	42.20	1.48	2.50	
0.43		128.33	48.89	1.68	2.60	
0.12		5.78	11.43	0.37	0.30	
0.12		6.35	11.62	0.41	0.40	
0.12		6.60	12.55	0.43	0.40	
0.12	16.35	21.01	0.62	0.40		
0.12	65.16	41.18	1.07	0.40		
0.12	105.10	49.08	1.23	0.40		

APPENDIX H

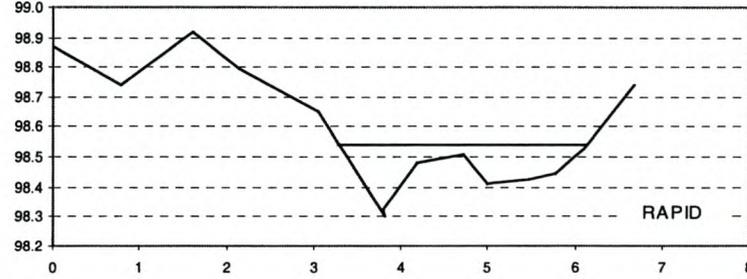
Cross sections

Whitebridge

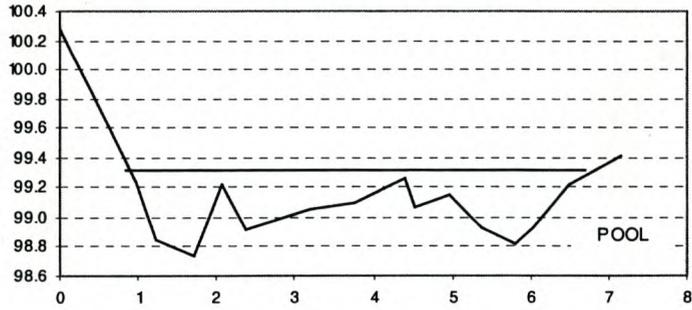
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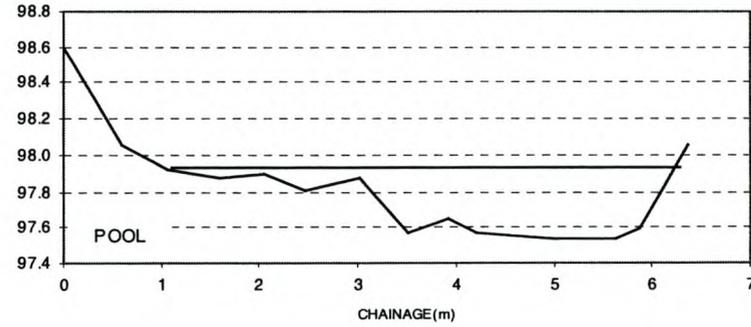
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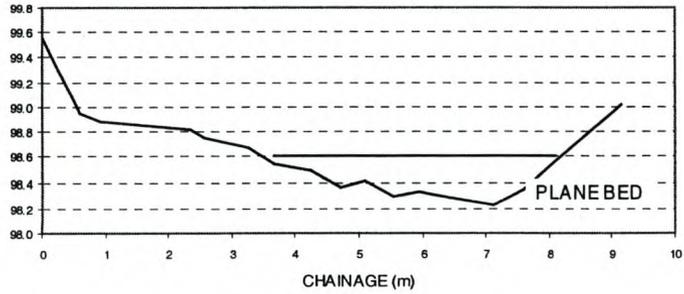
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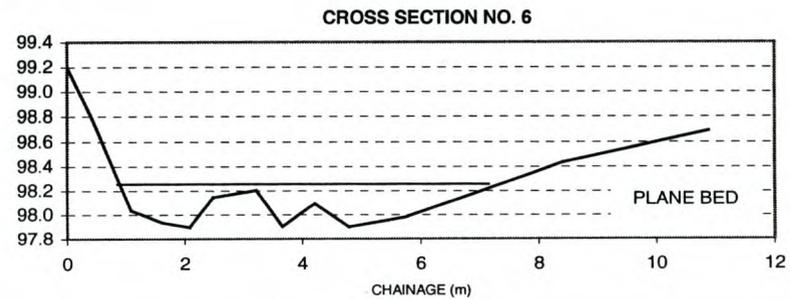
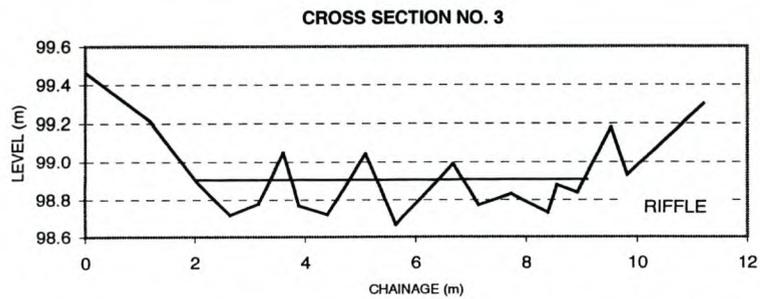
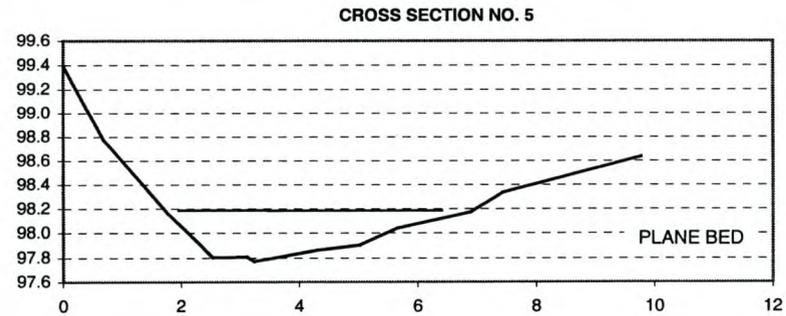
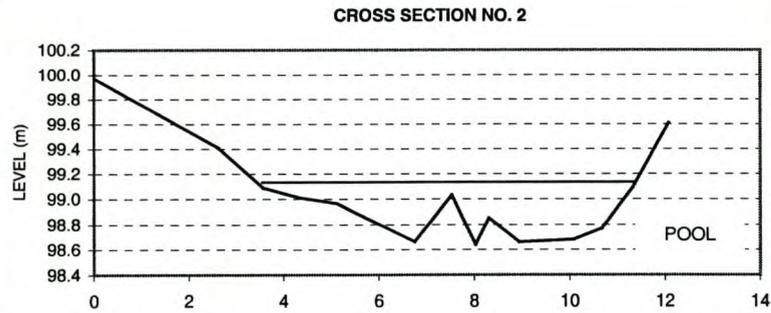
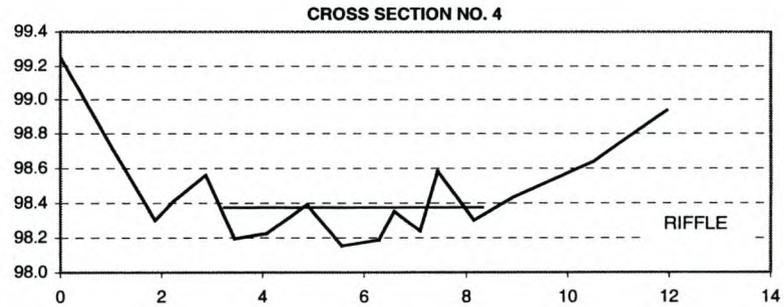
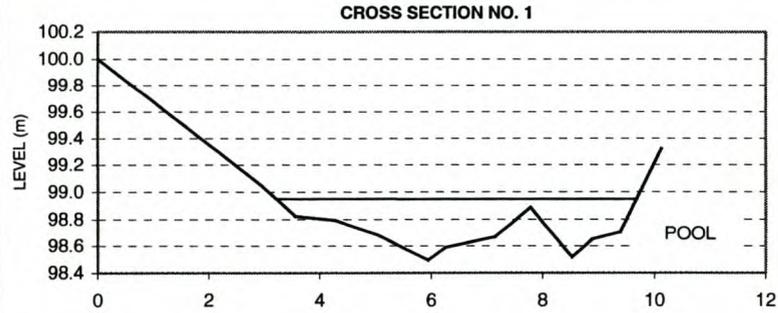
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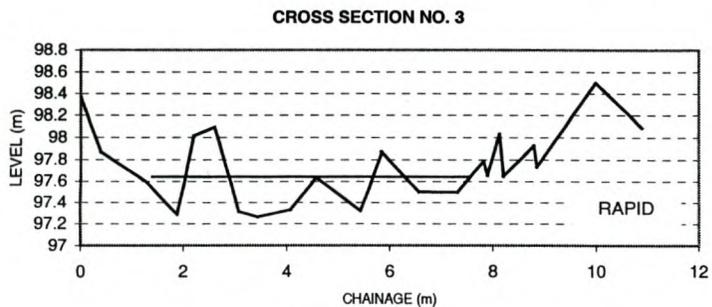
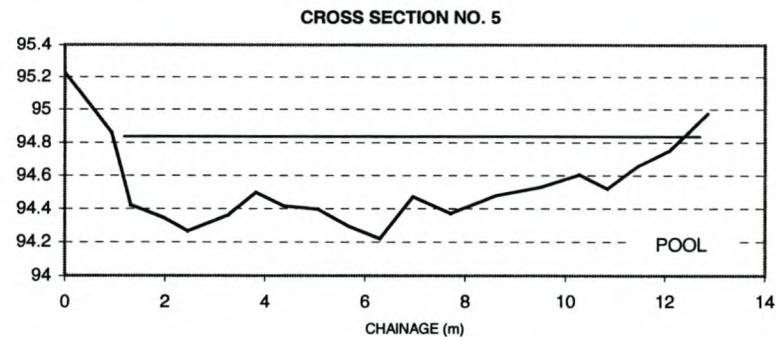
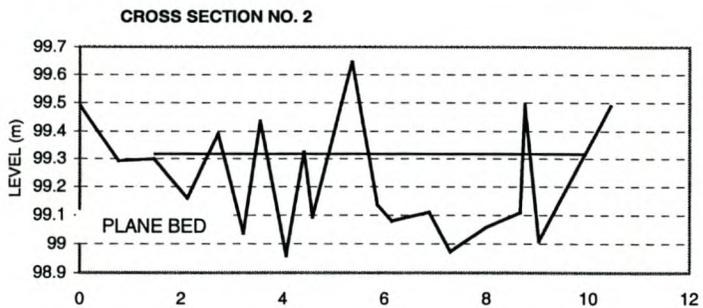
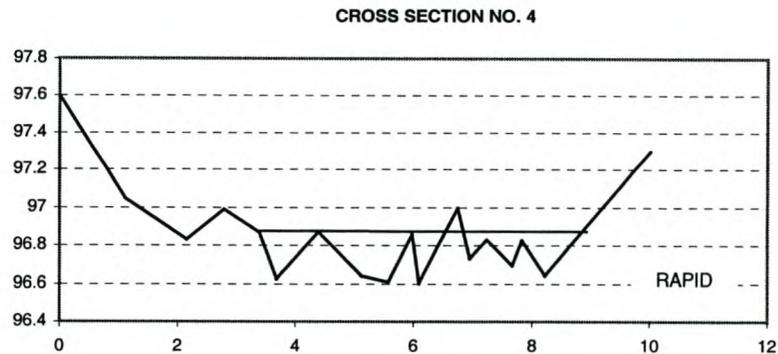
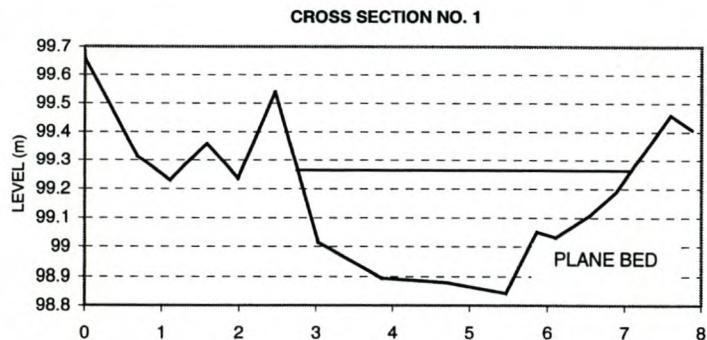
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Vergenoeg

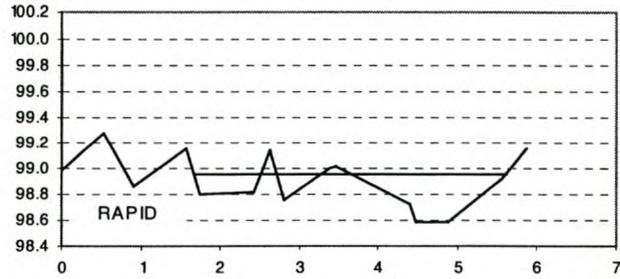


Berg 1

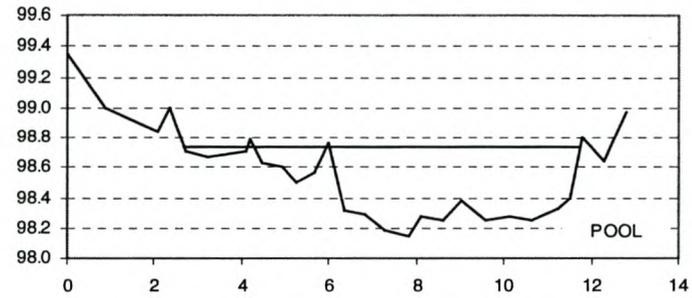


Berg 2

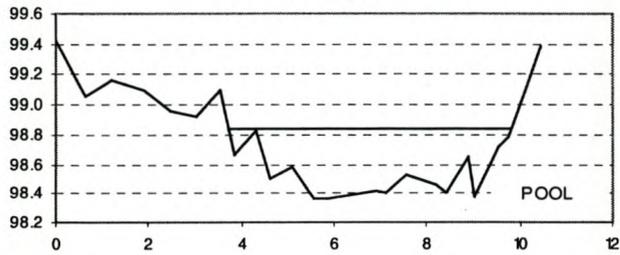
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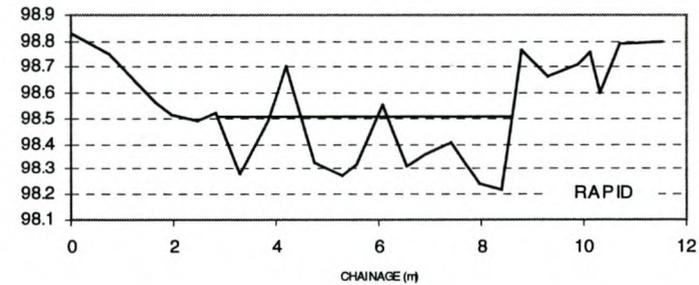
CROSS SECTION NO. 4



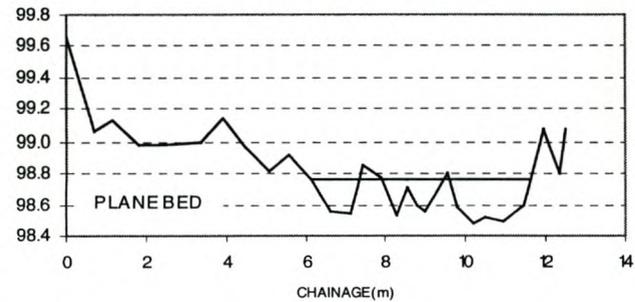
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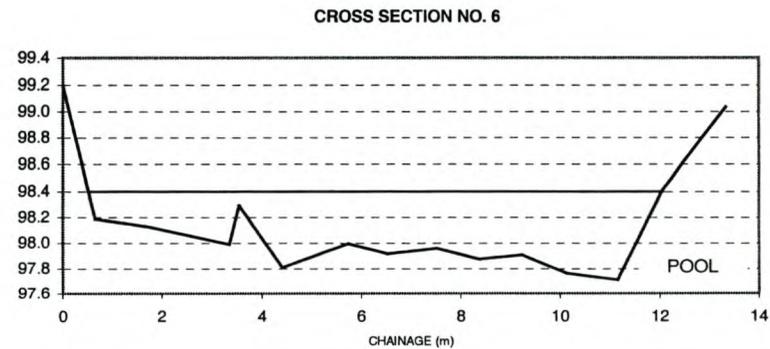
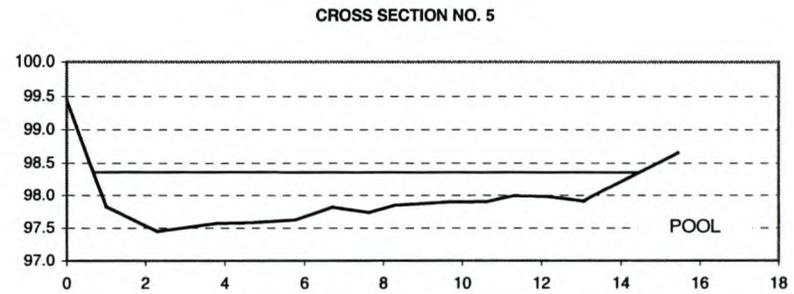
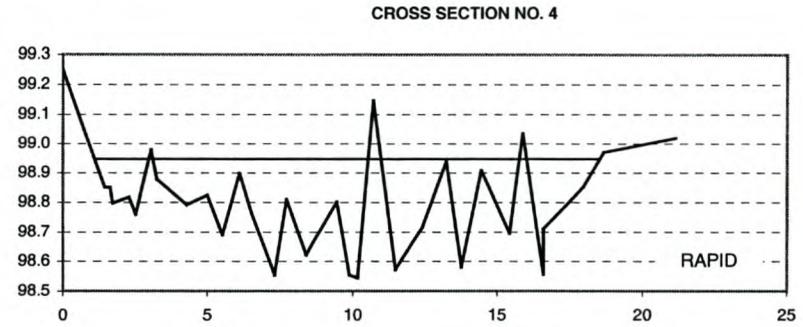
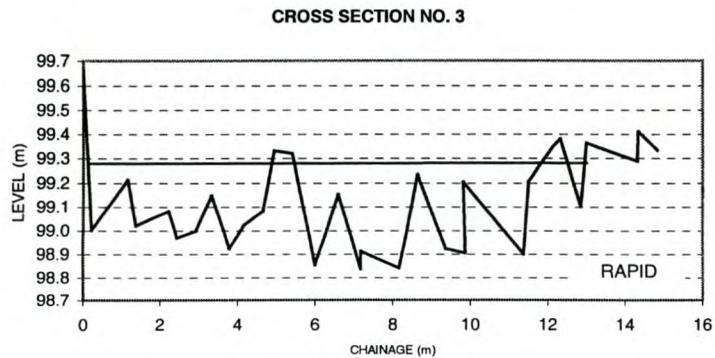
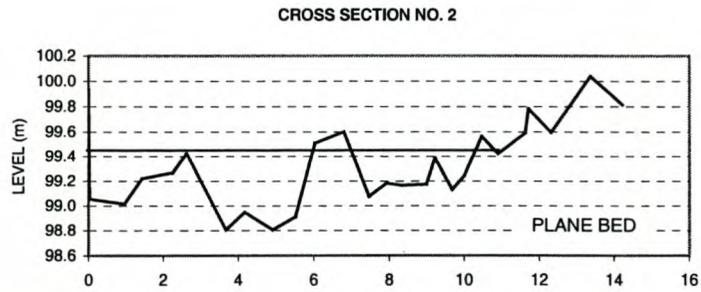
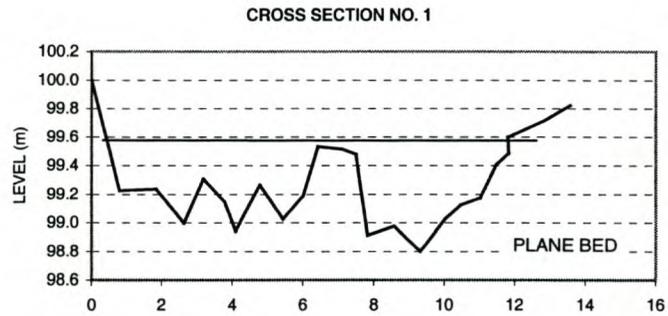
CROSS SECTION NO. 5



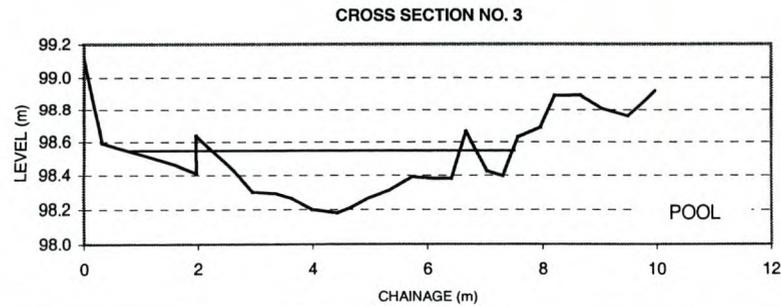
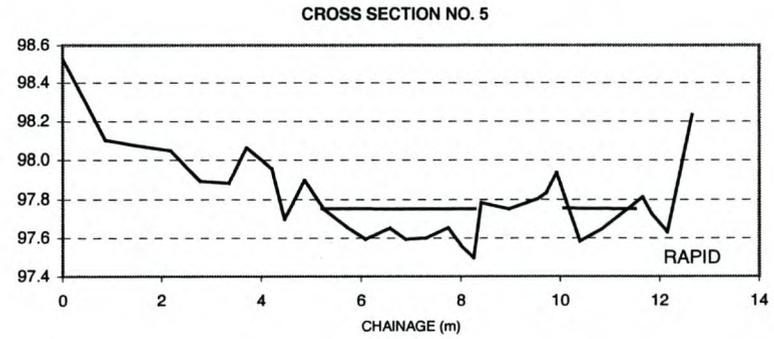
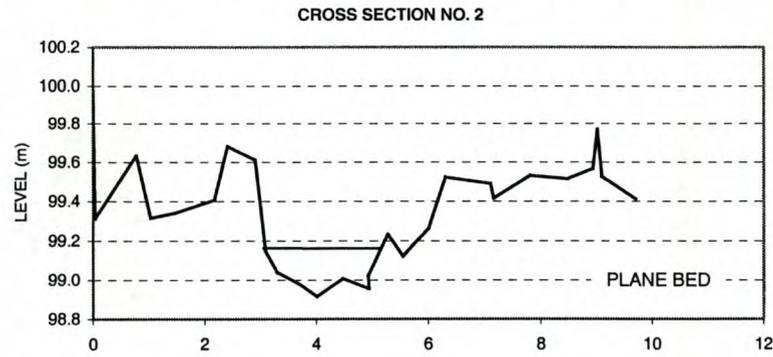
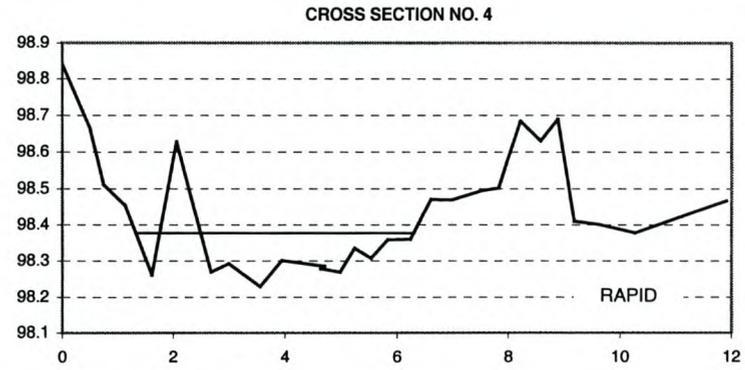
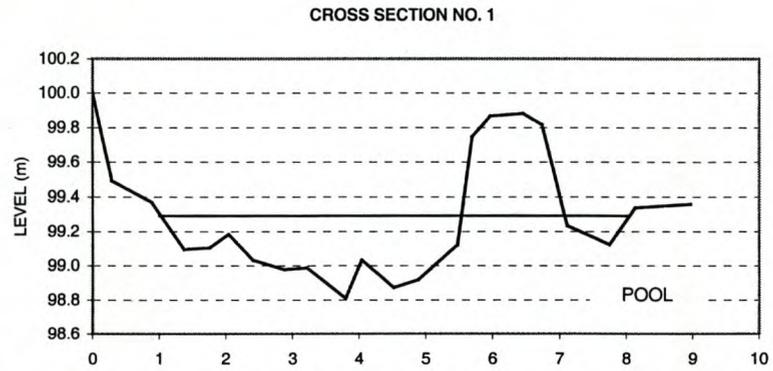
CROSS SECTION NO. 3



Elands

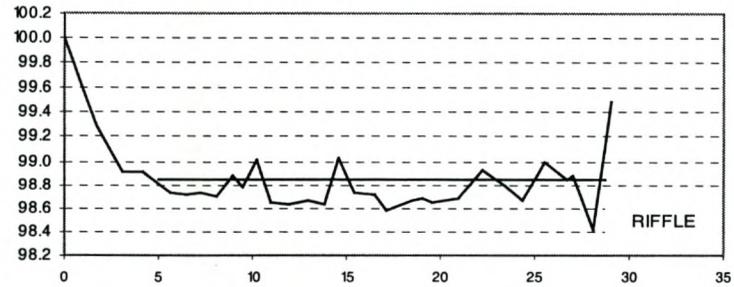


Du Toits

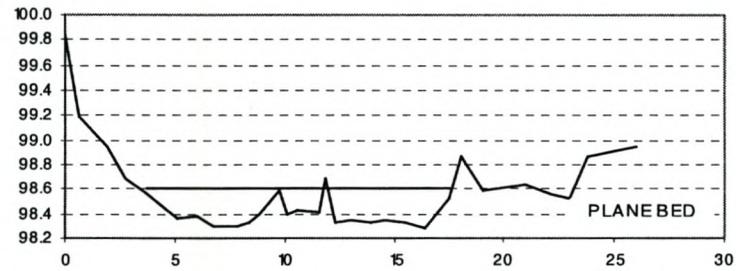


Molenaars 1

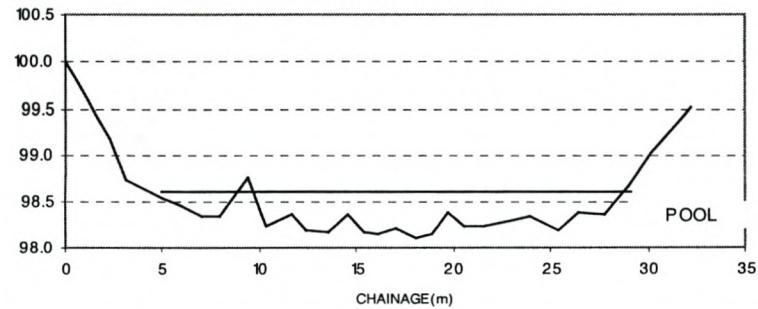
CROSS SECTION NO. 1



CROSS SECTION NO. 2

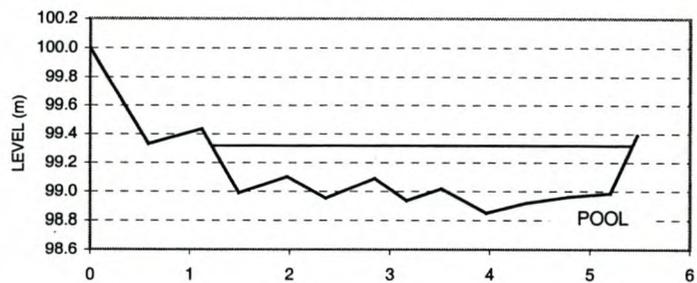


CROSS SECTION NO.3

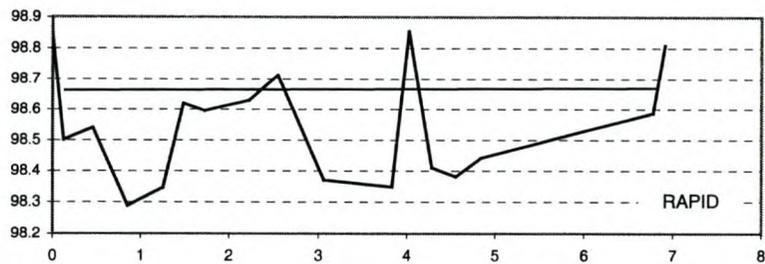


Jonkershoek

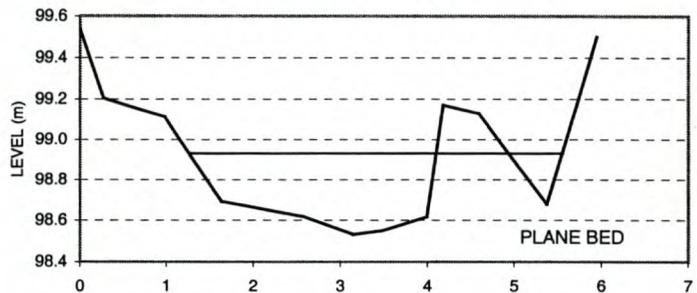
CROSS SECTION NO. 1



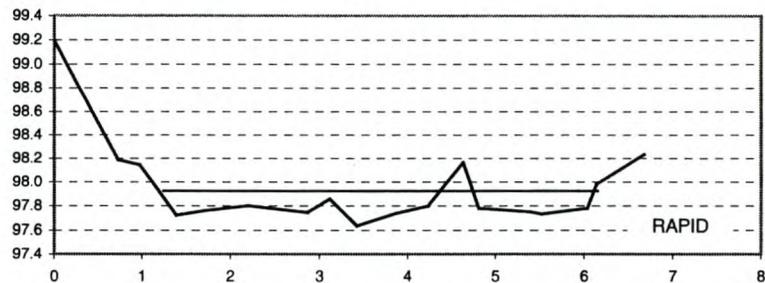
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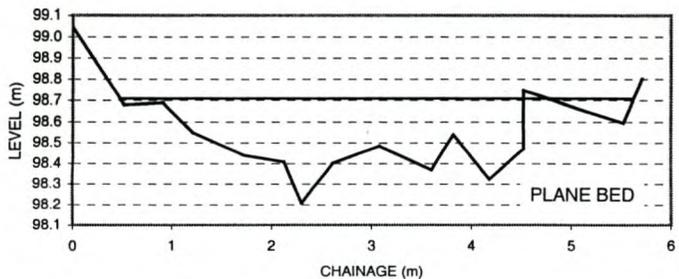
CROSS SECTION NO. 2



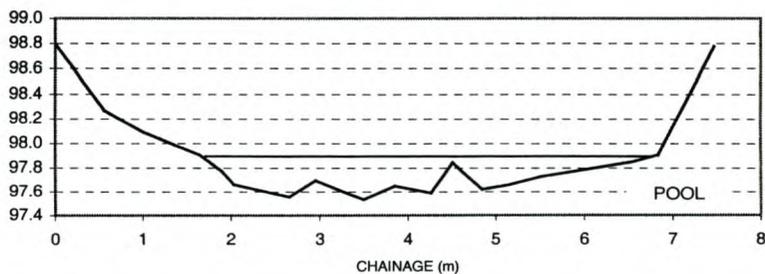
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CROSS SECTION NO. 3

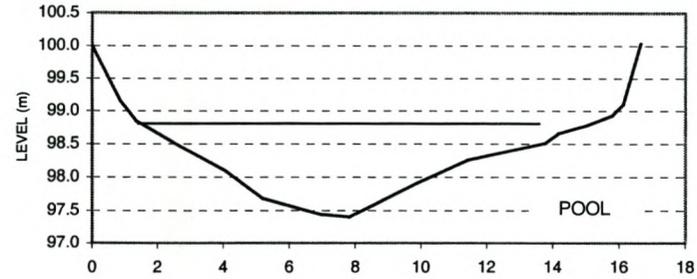


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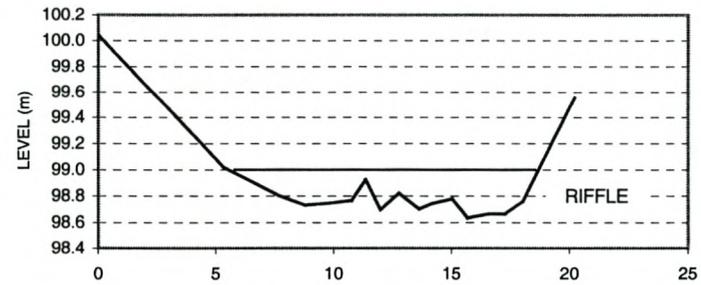


Vlottenburg

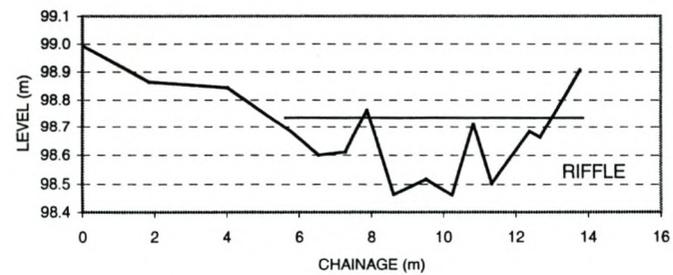
CROSS SECTION NO. 1



CROSS SECTION NO. 2



CROSS SECTION NO. 3



APPENDIX I

Point velocity data

Discharge during data collection

Site	Discharge (m ³ /s)
Whitebridge	0.085
Jonkershoek	0.132
Vergenoeg	0.134
Vlottenburg	0.903
Elands	0.577
Molenaars	0.468
Berg1	0.094
Berg2	0.140
DuToits	0.034

JONKERSHOEK

***A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS**

Cross section :			
	1	2	3
Morphological Unit :	Pool	Plane bed	Plane bed
No.	Velocity (m/s)		
1	0.09	0.01	0.08
2	0.11	0.06	0.26
3	0.13	0.45	0.18
4	0.15	0.52	0.16
5	0.16	0.17	0.1
6	0.2	0.02	0.23
7	0.09	0.16	0.21
8	0.04	0.4	0.11
9	0.04	0.52	0.23
10	0.05	0.19	0.26
11	0.04	0.08	0.09
12	0.13	0.03	0.38
13	0.1	0.04	0.01
14	0.1	0.04	0.16
15	0.18	0.16	0.11

Cross section :			
	4	5	6
Morphological Unit :	Rapid	Rapid	Pool
No.	Velocity (m/s)		
1	0.02	0.03	0.04
2	0.45	0.1	0.14
3	0.3	0.24	0.24
4	0.87	0.03	0.07
5	0.62	0.56	0.1
6	0.05	0.99	0.07
7	0.32	0.77	0.02
8	0.03	0.83	0.2
9	0.38	0.71	0.04
10	0.81	0.65	0.18
11	0.39	1.55	0.16
12	0.34	1.15	0.18
13	0.71	0.43	0.04
14		0.21	0.12
15		0.35	0.03

* Based on a random grid basis.

** From left bank, looking downstream ; At spacing of 0.5 or 1.0 m.

****B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS**

Cross section :			
	1	2	3
Morphological Unit :	Pool	Plane bed	Plane bed
No.	Velocity (m/s)		
1	0.08	0.04	0.02
2	0.07	0.03	0.05
3	0.04	0.48	0.03
4	0.07	0.35	0.15
5	0.13	0.11	0.26
6	0.12	0.01	0.13
7	0.09		0.06
8	0.11		0.04
9	0.06		0.13

Cross section :			
	4	5	6
Morphological Unit :	Rapid	Rapid	Pool
No.	Velocity (m/s)		
1	0.05	0.71	0.01
2	0.2	1.07	0.02
3	0.22	0.66	0.04
4	0.13	0.07	0.04
5	0.58	0.11	0.11
6	0.1	0.06	0.11
7	0.21	0.03	0.14
8	0.05		0.02
9	0.02		0.01

VERGENOEG

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :	5 & 6	3	1 & 2	4
Morphological Unit :	Plane bed	Riffle	Pool	Riffle
No.	Velocity (m/s)			
1	0.13	0.34	0.07	0.25
2	0.1	0.25	0.1	0.68
3	0.3	0.85	0.1	0.34
4	0.13	0.41	0.14	0.43
5	0.25	0.5	0.08	0.31
6	0.15	0.46	0.09	0.13
7	0.1	0.51	0.13	0.06
8	0.09	0.12	0.08	0.49
9	0.03	0.02	0.07	0.69
10	0.15	0.3	0.05	0.87
11	0.26	0.53	0.13	0.1
12	0.19	0.64	0.05	0.23
13	0.17	0.42	0.14	0.38
14	0.18	0.62	0.1	0.33
15	0.23	0.52	0.02	0.51
16	0.22		0.04	
17	0.04		0.03	
18	0.26		0.07	
19	0.16		0.05	
20	0.17		0.03	
21	0.16		0.12	
22	0.11		0.04	
23	0.21		0.04	
24	0.23		0.05	
25	0.23		0.03	
26	0.26		0.05	
27	0.24		0.09	
28	0.14		0.1	
29	0.21		0.15	
30	0.04		0.11	

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :	1	2	3
Morphological Unit :	Pool	Pool	Riffle
No.	Velocity (m/s)		
1	0.03	0.03	0.28
2	0.02	0.07	0.24
3	0.03	0.06	0.23
4	0.06	0.08	0.54
5	0.06	0.13	0.7
6	0.1	0.12	0.63
7	0.08	0.08	0.24
8	0.21	0.1	0.41
9	0.08	0.12	
10	0.05	0.07	
		0.04	

Cross section :	4	5	6
Morphological Unit :	Riffle	Plane bed	Plane bed
No.	Velocity (m/s)		
1	0.09	0.24	0.2
2	0.43	0.06	0.22
3	0.26	0.17	0.12
4	0.29	0.04	0.15
5	0.29	0.36	0.22
6	0.26	0.18	0.08
		200	110

VLOTTENBURG

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :		3 & 4	1
Morphological Unit :		Riffle	Pool
No.	Velocity (m/s)		
1	0.08	0.1	
2	0.45	0.12	
3	0.6	0.13	
4	0.06	0.11	
5	0.06	0.09	
6	0.27	0.07	
7	0.56	0.05	
8	0.55	0.1	
9	0.64	0.05	
10	0.62	0.06	
11	0.5	0.1	
12	0.41	0.09	
13	0.05	0.1	
14	0.34	0.08	
15	0.6	0.09	
16	0.36	0.06	
17	0.13	0.04	
18	0.11	0.05	
19	0.32	0.04	
20	0.57	0.02	
21	0.21	0.01	
22	0.43	0.01	
23	0.35	0.01	
24	0.6	0.01	
25	0.47	0.01	
26	0.34	0.02	
27	0.24	0.05	
28	0.18	0.07	
29	0.15	0.1	
30	0.33	0.08	

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :		1	3	4
Morphological Unit :		Pool	Riffle	Riffle
No.	Velocity (m/s)			
1	0.08	0.37	0.44	
2	0.07	0.23	0.52	
3	0.07	0.3	0.25	
4	0.01	0.7	0.7	
5	0.06	0.28	0.34	
6	0.03	0.51	0.6	
7	0.13	0.32	0.6	
8		0.64	0.56	
9		0.31	0.42	
			0.47	

ELANDS

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section : Morphological Unit :	5 & 6 Pool	3&4 Rapid	1 & 2 Plane bed
No.	Velocity (m/s)		
1	0.01	0.84	0.04
2	0.05	0.87	0.09
3	0.1	0.67	0.22
4	0.13	0.6	0.29
5	0.13	0.52	0.15
6	0.12	0.49	0.29
7	0.13	0.7	0.28
8	0.02	0.18	0.12
9	0.03	0.92	0.14
10	0.04	0.14	0.38
11	0.11	0.88	0.32
12	0.08	0.8	0.41
13	0.11	1.18	0.04
14	0.05	0.17	0.42
15	0.04	0.25	0.37
16	0.07	0.46	0.15
17	0.04	0.34	0.27
18	0.02	0.1	0.03
19	0.04	0.52	0.22
20	0.03	0.29	0.15
21	0.01	0.42	0.02
22	0.02	0.8	0.27
23	0.01	0.17	0.21
24	0.03	0.56	0.17
25	0.04	0.51	0.18
26	0.04	0.78	0.34
27	0.12	0.36	0.35
28	0.09	0.4	0.03
29	0.11	0.2	0.16
30	0.07	0.15	0.31

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section : Morphological Unit :	1 Plane bed	2 Plane bed	3 Rapid
No.	Velocity (m/s)		
1	0.12	0.29	0.37
2	0.17	0.08	0.5
3	0.13	0.15	0.22
4	0.25	0.4	0.68
5	0.24	0.13	0.15
6	0.1	0.44	0.08
7	0.12	0.23	0.48
8	0.28	0.09	0.25
9	0.17		0.25
10	0.03		0.2

	4 Rapid	5 Pool	6 Pool
No.	Velocity (m/s)		
1	0.12	0.01	0.08
2	0.3	0.04	0.09
3	0.35	0.01	0.07
4	0.69	0.01	0.1
5	0.3	0.04	0.13
6	0.24	0.01	0.15
7	0.3	0.02	0.11
8	0.43	0.06	0.06
9	1.04	0.07	0.18
10	0.82	0.06	0.18
11	0.52	0.09	0.13
12	0.54	0.09	0.03
13	0.3	0.09	
14	0.43	0.09	
15	0.54		

MOLENAARS I

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :	3	2	1
Morphological Unit :	Pool	Plane bed	Riffle
No.	Velocity (m/s)		
1	0.01	0.15	0.1
2	0.08	0.21	0.42
3	0.09	0.19	0.4
4	0.13	0.19	0.5
5	0.18	0.23	0.4
6	0.21	0.14	0.58
7	0.15	0.05	0.31
8	0.09	0.29	0.21
9	0.15	0.04	1.01
10	0.03	0.08	0.15
11	0.01	0.07	0.82
12	0.04	0.17	0.27
13	0.08	0.16	0.65
14	0.11	0.12	0.78
15	0.1	0.24	0.61
16	0.15	0.3	0.68
17	0.18	0.1	0.31
18	0.11	0.31	0.68
19	0.1	0.26	0.04
20	0.09	0.28	0.52
21	0.09	0.06	0.27
22	0.06	0.19	0.32
23	0.02	0.26	0.18
24	0.05	0.08	0.38
25	0.06	0.16	0.57
26	0.12	0.15	0.16
27	0.13	0.31	0.33
28	0.14	0.14	0.22
29	0.14	0.39	0.19
30	0.1	0.36	0.1

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :	1	2	3
Morphological Unit :	Riffle	Plane bed	Pool
No.	Velocity (m/s)		
1	0.12	0.04	0.03
2	0.3	0.12	0.03
3	0.07	0.06	0.03
4	0.28	0.23	0.12
5	0.24	0.15	0.16
6	0.19	0.28	0.08
7	0.54	0.28	0.07
8	0.31	0.23	0.1
9	0.33	0.25	0.11
10	0.37	0.32	0.16
11	0.56	0.16	0.15
12	0.29	0.13	0.14
13	0.33	0.03	0.11
14	0.44	0.02	0.1
15	0.53		0.09
16	0.76		0.05
17	0.12		0.04
18	0.01		0.06

BERG I

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :	1 & 2	3 & 4	5
Morphological Unit :	Plane bed	Rapid	Pool
No.	Velocity (m/s)		
1	0.16	0.29	0.02
2	0.19	0.27	0.01
3	0.18	0.3	0.03
4	0.11	1.1	0.03
5	0.19	0.17	0.01
6	0.15	0.17	0.02
7	0.12	0.98	0.01
8	0	0.57	0.01
9	0.06	0.63	0.02
10	0.31	0.57	0.04
11	0.23	1.01	0.01
12	0.15	0.41	0.01
13	0.23	0.36	0.01
14	0.08	0.64	0.01
15	0.15	0.35	0.02
16	0.13	0.01	0.04
17	0.17	0.16	0.02
18	0.18	0.47	0.03
19	0.02	0.52	0.05
20	0.016	0.45	0.05
21	0.01	0.83	0.07
22	0.08	0.64	0.03
23	0.13	0.81	0.03
24	0.13	0.88	0.01
25	0.08	1.2	0.01
26	0.13	0.47	0.01
27	0.14	0.58	0.01
28	0.22	0.37	0.05
29	0.18	0.17	0.02
30	0.16	0.84	0.02

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :	1	2	3
Morphological Unit :	Plane bed	Plane bed	Rapid
No.	Velocity (m/s)		
1	0	0.04	0.19
2	0.03	0.01	0.1
3	0.01	0.01	0.21
4	0.12	0.05	0.13
5	0.12	0.11	0.44
6	0.04	0.01	0.79
7	0.01	0.09	0.63
8	0.16	0.11	0.55
9	0.19	0.14	0.46
10	0.03	0.17	0.25
11	0.06	0.14	0.24
12		0.08	0.29

Cross section :	4	5
Morphological Unit :	Rapid	Pool
No.	Velocity (m/s)	
1	0.03	0
2	0.13	0.02
3	0.25	0.01
4	0.72	0.02
5	0.39	0.01
6	0.51	0.04
7	0.2	0.04
8	0.54	0.01
9	0.43	0.01
10	0.17	0.01
11	0.09	0.02
12	0.45	

BERG II

A : VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :			
Morphological Unit :	1	2	3
	Rapid	Pool	Plane bed
No.	Velocity (m/s)		
1	0.54	0.09	0.44
2	0.21	0.03	0.57
3	0.61	0.14	0.03
4	0.04	0.06	0.3
5	0.26	0.04	0.26
6	0.43	0.01	0.74
7	0.47	0.05	0.23
8	0.51	0.16	0.13
9	0.27	0.14	0.39
10	0.68	0.08	0.34
11	0.8	0.07	0.19
12	0.76	0.12	0.2
13	0.73	0.26	0.45
14	0.02	0.07	0.42
15	0.5	0.07	0.36

Cross section :			
Morphological Unit :	4	5	
	Pool	Rapid	Plane bed
No.	Velocity (m/s)		
1	0.02	0.88	0.01
2	0.09	0.38	0.03
3	0.08	0.72	0.21
4	0.13	0.28	0.31
5	0.02	0.47	0.2
6	0.01	0.48	0.04
7	0.01	0.45	0.27
8	0.02	0.62	0.45
9	0.08	0.22	0.18
10	0.13	0.05	0.27
11	0.04	0.61	
12	0.08	0.24	
13	0.13	0.32	
14	0.03	0.51	
15	0.01	0.35	

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :			
Morphological Unit :	1	2	3
	Rapid	Pool	Plane bed
No.	Velocity (m/s)		
1	0.38	0.01	0.14
2	0.25	0.07	0.24
3	0.1	0.04	0.17
4	0.3	0.06	0.27
5	0.47	0.13	0.09
6	0.36	0.07	0.08
7	0.02	0.04	0.19
8	0.64	0.11	0.5
9		0.09	0.26
10		0.09	
11		0.02	

Cross section :			
Morphological Unit :	4	5	
	Pool	Rapid	Plane bed
No.	Velocity (m/s)		
1	0	0.04	
2	0.01	0.05	
3	0.02	0.03	
4	0.08	0.4	
5	0.16	0.42	
6	0.09	0.48	
7	0.01	0.67	
8		1.33	
9		0.33	
10		0.01	
11		0.03	

DU TOITS

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :			
Morphological Unit :	1	2	5
	Pool	Plane bed	Rapid
No.	Velocity (m/s)		
1	0.01	0.06	0.02
2	0.03	0.17	0.29
3	0.03	0.14	0.43
4	0.02	0.12	0.28
5	0	0.19	0.32
6	0.04	0.06	0.39
7	0.09	0.16	0.29
8	0.1	0.12	0.03
9	0.03	0.07	0.32
10	0.01	0.29	0.15
11	0.07	0.31	0.53
12	0.03	0.06	0.8
13	0.05	0.08	0.39
14	0.08	0.06	0.05
15	0.06	0.03	0.05
CONT.			0.47
			0.21
			0.16
			0.26
			0.03
			0.08
			0.17
			0.49
			0.57
			0.09
			0.49
			0.44
			0.35
			0.03
			0.1
			0.27
			0.23
			0.02
			0.55
			0.2
			0.49

B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

Cross section :					
Morphological Unit :		1	2	3	
		Pool	Plane bed	Pool	
No.		Velocity (m/s)			
1		0.05	0.12	0.07	
2		0.01	0.04	0.11	
3		0.06	0.17	0.11	
4		0.09	0.09	0.06	
5		0.04	0.02	0.01	
6		0.01	0.02	0.01	
7		0	0.14	0.01	
8		0	0.44	0.01	

Cross section :			
Morphological Unit :		4	5
		Rapid	Rapid
No.		Velocity (m/s)	
1		0	0.02
2		0.13	0.06
3		0.16	0.11
4		0.14	0.09
5		0.33	0.19
6		0.18	0.26
7		0.13	0.02
8		0.14	0.09
9		0.09	0.13

WHITEBRIDGE

A : RANDOM VELOCITY DATA WITHIN MORPHOLOGICAL UNITS

Cross section :		4	6
Morphological Unit :		Rapid	Pool
No.		Velocity (m/s)	
1		0.1	0.13
2		0.7	0.16
3		1.06	0.05
4		0.99	0.18
5		0.38	0.04
6		0.05	0.11
7		0.7	0.23
8		0.21	0.09
9		0.51	0.1
10		0.05	0.08
11		0.73	0.09
12		0.63	0.13
13		0.05	0.05
14		0.56	0.11
15		0.77	0.15

Cross section :		3	1	2
Morphological Unit :		Plane bed	Rapid	Pool
No.		Velocity (m/s)		
1		0.28	0.52	0.03
2		0.2	0.4	0.04
3		0.21	0.51	0.04
4		0.19	0.3	0.03
5		0.12	0.05	0.1
6		0.15	0.19	0.13
7		0.1	0.25	0.09
8		0.41	0.71	0.02
9		0.12	0.25	0.09
10		0.08	0.48	0.08
11		0.16	0.03	0.12
12		0.45	0.82	0.12
13		0.22	1.54	0.1
14		0.24	0.39	0.03
15		0.41	0.95	0.03

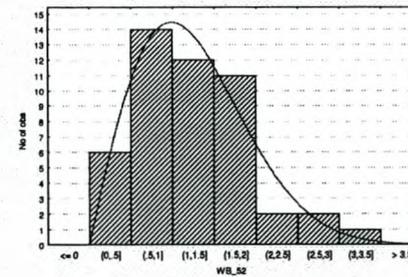
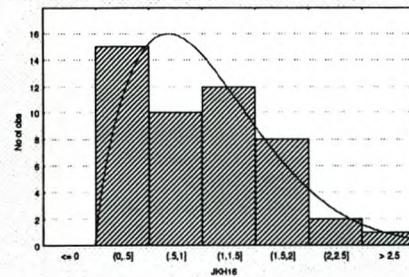
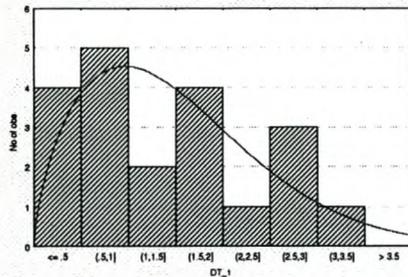
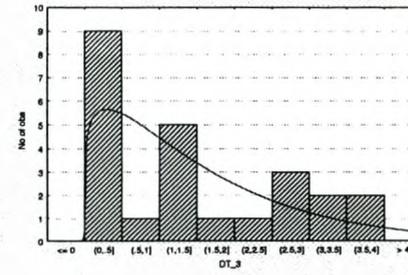
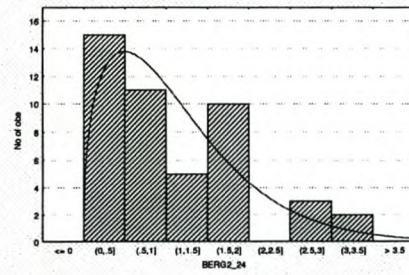
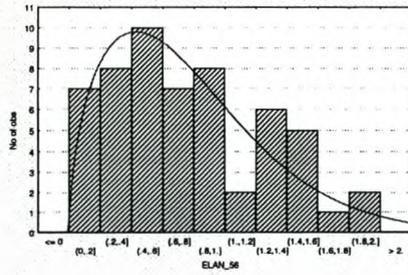
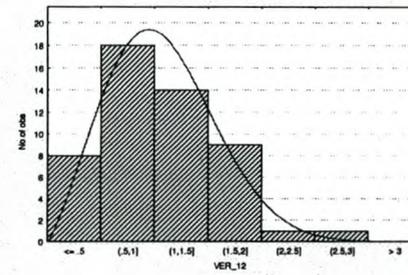
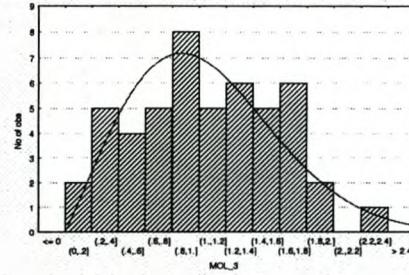
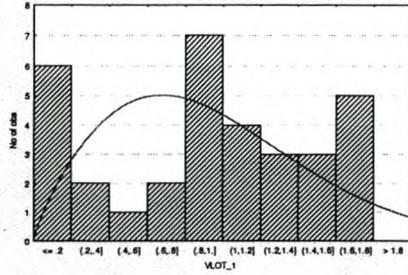
B : VELOCITY DATA IN SPECIFIC CROSS SECTIONS

		4	6
		Rapid	Pool
No.		Velocity (m/s)	
1		1.14	0.03
2		0.49	0.06
3		0.11	0.08
4		0.09	0.2
5			0.11
6			0.06
7			0.04
			380

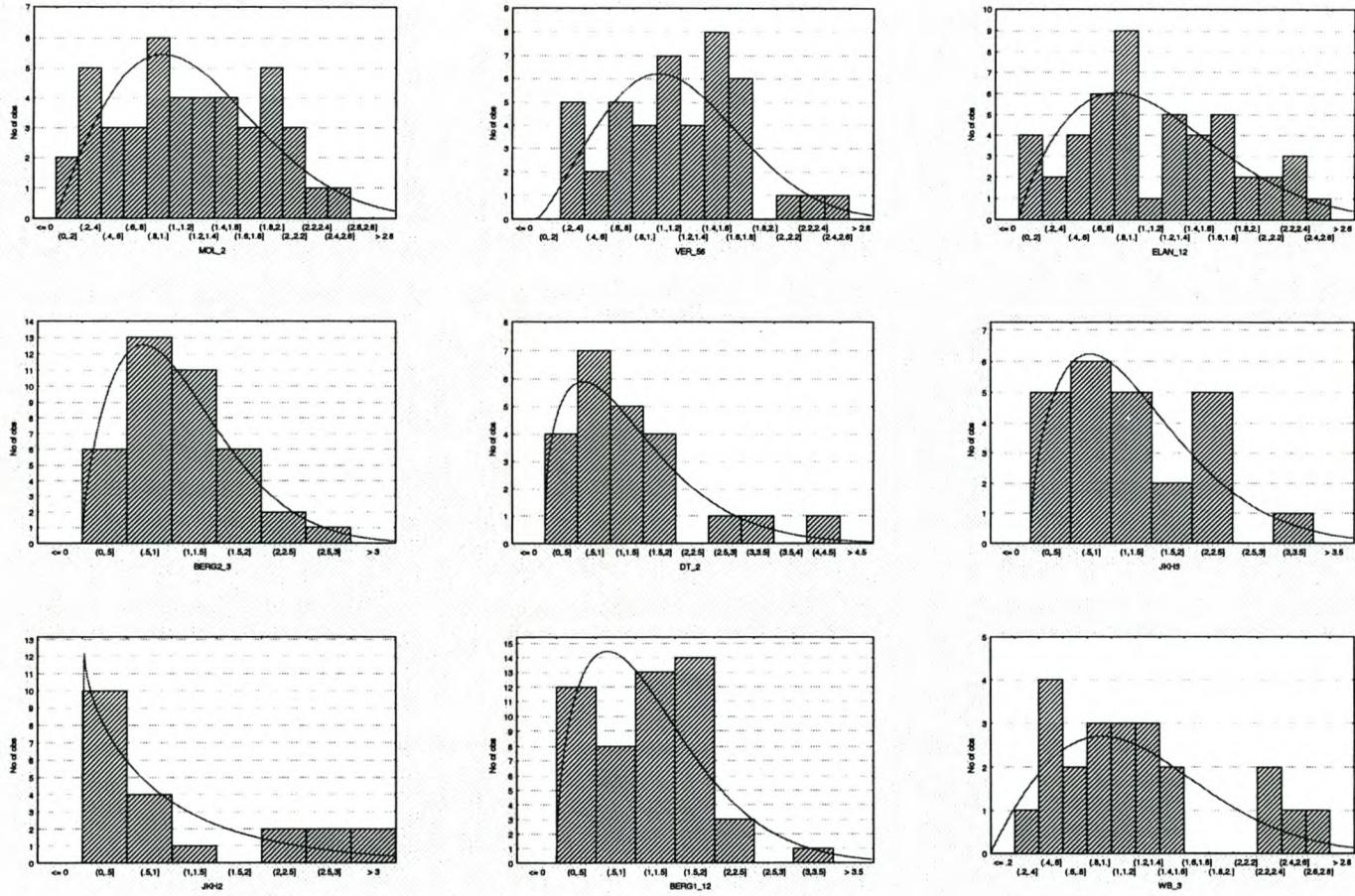
		3	1	2
		Plane bed	Rapid	Pool
No.		Velocity (m/s)		
1		0.23	0.02	0.07
2		0.47	0.19	0.1
3		0.18	0.37	0.09
4		0.1	0.28	0.04
5		0.06	0.58	0.04
6		0.08		0.07
7		0.27		0.12
8				0.11
9				0.12
10				0.04

APPENDIX J

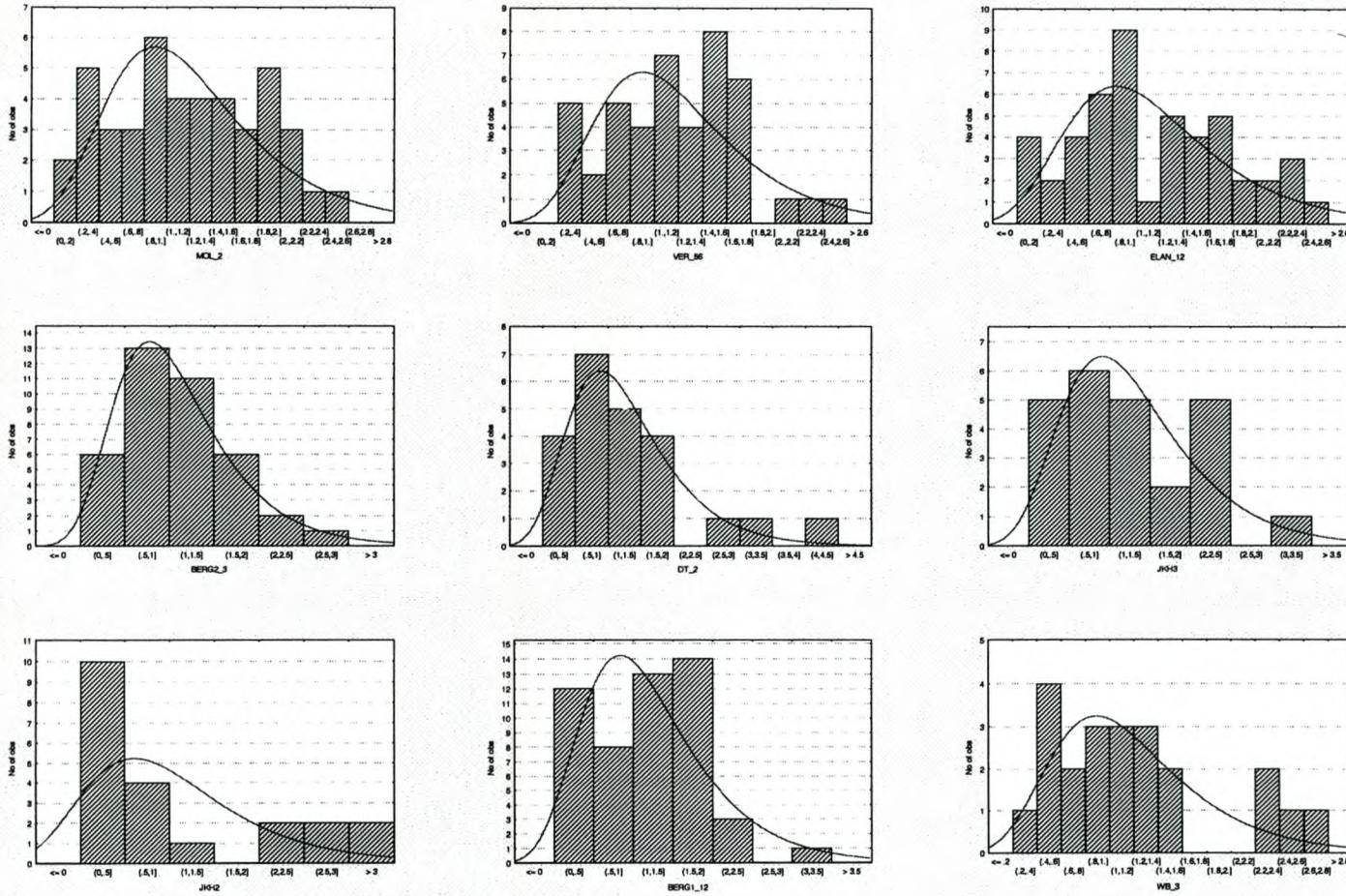
Histograms of relative point velocity



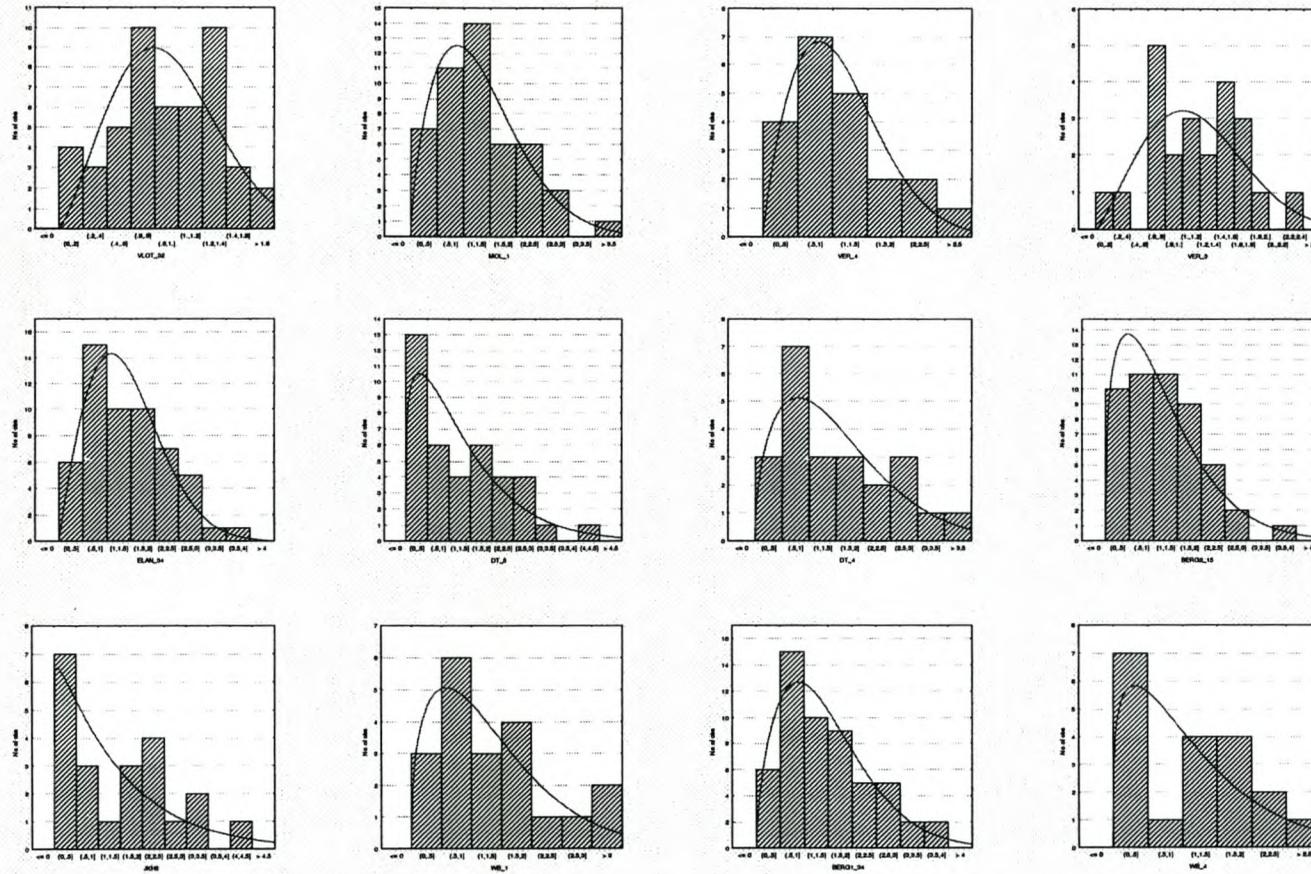
POOLS : Weibull distribution fitted



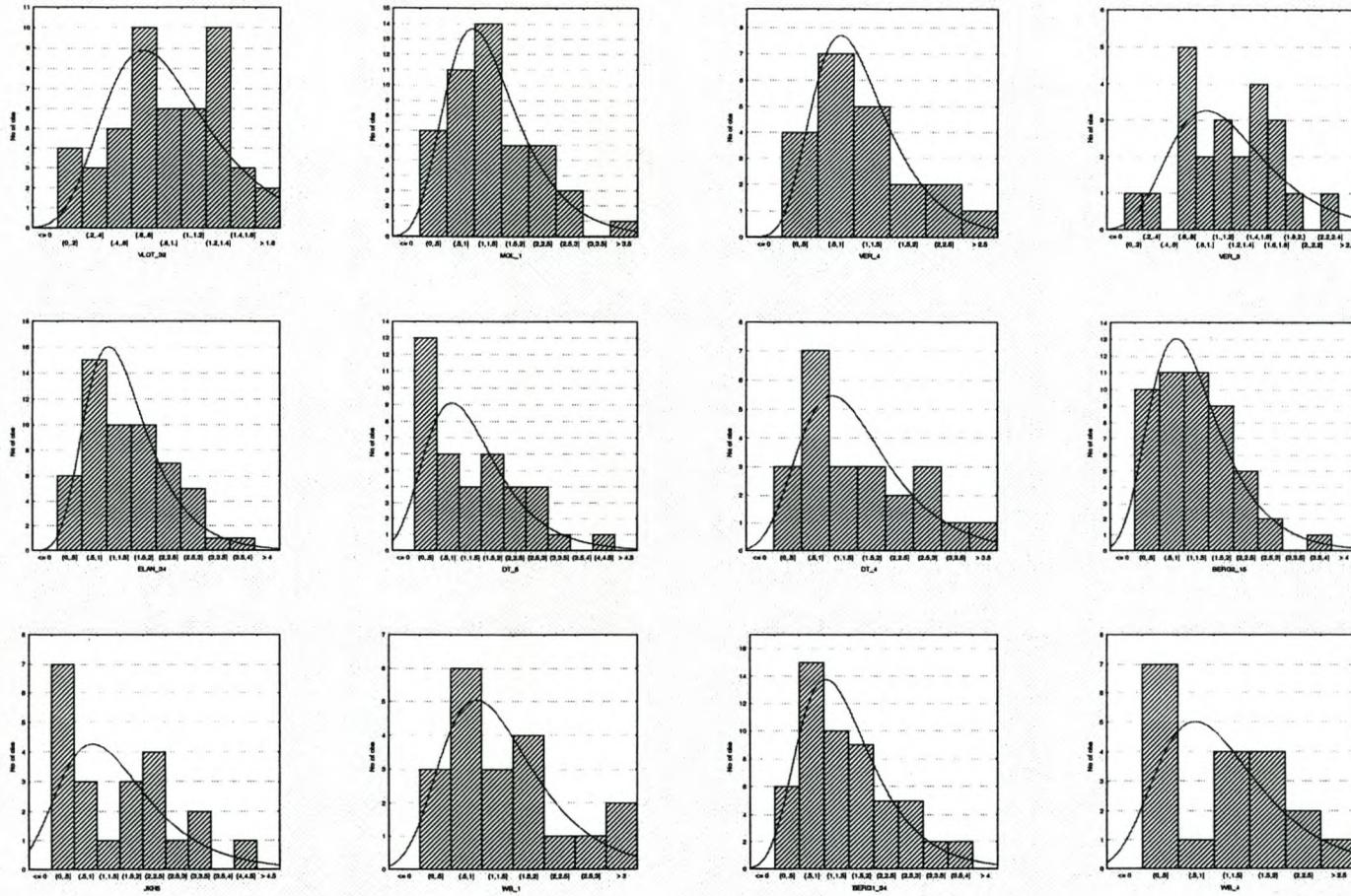
PLANE BEDS: Weibull distribution fitted



PLANE BEDS: Extreme Type I distribution fitted



RAPIDS and RIFFLES : Weibull distribution fitted



RAPIDS and RIFFLES: Extreme Type I distribution fitted