APPLICATION OF A RATIONAL MODEL OF STREAM EQUILIBRIUM FOR PREDICTING CHANNEL ADJUSTMENTS

by

BRUCE JONATHAN MACVICAR

B.Sc.(Eng.), The University of Guelph, 1996

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF APPLIED SCIENCE

in

THE FACULTY OF GRADUATE STUDIES

Department of Civil Engineering

We accept this thesis as conforming

to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

March 1999

Abstract

In this thesis, channel geometry adjustments and possible restoration efforts were modeled and interpreted for three streams in British Columbia using the physical model of Millar and Quick (1993). The major challenge of the project was to overcome limitations in our ability to quantify important physical processes such as flow resistance, sediment transport, and bank stability in order to access the inherent predictive and modeling capabilities of a rational approach. In practical terms this meant the model had to be calibrated to past adjustments of channel geometry before it could be used to predict future changes. The success of calibration was found to be dependent on the type of disturbance a stream was adjusting to.

Disturbances of bank stability were readily modeled. Calibration was facilitated by the sensitivity of modeled widths to changes in bank stability and the availability of air photographs to measure historical changes in channel width. Slesse Creek was found to have been disturbed by a reduction in bank stability due to forest harvesting in the riparian area of the creek. The creek adjusted by widening its channel and switching from a single to a multiple thread. Modeling results indicated that a moderate increase in bank stability could be used to reinstate a single-thread channel, reduce bank and floodplain erosion and allow vegetation to recover.

Disturbances to water discharges were also readily modeled provided that flow records existed or that past flows could be estimated from a clearly defined bankfull flow condition. Shovelnose Creek was found to have been disturbed by an increase in the discharge conveyed by the channel. The creek adjusted to the increase by widening and deepening its channel, and breaks in the slope were introduced. These breaks were now control points as channel discharge has been reduced to

pre-disturbance levels. Modeling indicated that the use of point deflectors to narrow and deepen the channel may be particularly suited to Shovelnose Creek due to the lateral and vertical stability of the channel.

Disturbances to the sediment transport regime were not readily modeled. The example of Harris Creek highlighted the difficulty of assessing simultaneous changes of particle sizes, channel roughness, and sediment transport. The longer time scale of sediment supply waves and effect of bedrock control on slope were additional difficulties.

The approach used to apply the rational model of Millar and Quick (1993) was advantageous because it focused on stream processes, produced exact numerical results and resulted in a stream response framework that was specific to each stream. The approach thus represents a step forward from other available approaches. Further research that could increase the applicability of the rational model is discussed.

Table of Contents

List of Appendices

List of Tables

List of Figures

List of Symbols

a = empirical coefficient

A = cross-section area of channel (m^2)

A_D = drainage area (km^2)

b = empirical exponent of width relation

c = soil cohesion

D_x = bed sediment diameter where *x*% is finer

d_i = cumulative mean at the *ith* year

F_1 = unnamed variable from Einstein-Brown relation

f = Darcy Weisbach friction factor

f' = empirical exponent of depth relation

G_b = Dry bedload transport rate (kg/s)

g_b = dry bedload transport rate per metre width (kg/s/m)

g^*_b = dimensionless unit sediment transport rate

g = gravitational acceleration (assumed = 9.81 m/s^2)

k_s = equivalent roughness (m)

n = Manning's roughness coefficient

P = channel wetted perimeter (m)

Q_{bf} = bankfull discharge (m^3/s)

Q_d = dominant discharge (m^3/s)

Q_i = discharge of the current year (m^3/s)

Q_2 = discharge with a 2-year return period (m^3/s)

$\overline{Q}$ = mean annual peak instantaneous discharge (m^3/s)

R_h = hydraulic radius (m)

r_c = meander arc length (m)

S = channel slope

S_v = valley slope

S* = transitional slope

SF = shear force

s = specific gravity of sediment (assumed = 2.65)

V = mean velocity (m/s)

W = bankfull width (m)

Y* = mean hydraulic depth (m)

Y_o = channel depth (m)

γ = unit weight of water (assumed = 9810 N/m^3)

λ = meander wavelength (m)

ρ_s = density of sediment (assumed = 2650 kg/m^3)

τ = mean boundary shear stress (N/m^2)

τ^*_{D50} = dimensionless shear stress for median grain diameter

τ_{crit} = critical shear stress (N/m^2)

θ = bank angle (°)

ϕ = friction angle of bank sediment (°)

ϕ' = modified friction angle of bank sediment (°)

ξ = sinuosity

Subscripts

bf = bankfull

*19*** = year

DI = During Inundation. Used to represent the period in which the lower reaches of Shovelnose Creek were regularly inundated by flows from the Squamish River.

Acknowledgment

I would like to thank my research supervisor, Dr. Robert Millar, for the time and effort he has put into this research project. At the University of British Columbia I have also been fortunate enough to have had some excellent teachers. In particular Drs. Dennis Russell, Michael Quick, Michael Church, and Robert Newbury have helped develop ideas presented here.

For practical support, the Steelhead Society of British Columbia and the Nanaimo Office of MoELP have assisted me and provided a lot of the background material for this report. Stephane D'Aoust, Dave Strajt and Chris Norris helped me with the field work, and Sue Greco helped me with editing.

This research has been funded by a scholarship from the National Sciences and Engineering Research Council of Canada.

Finally, I would like to thank my friends, family and especially Sue.

CHAPTER 1

INTRODUCTION

1.1 General Introduction

Within British Columbia and throughout the world, there is abundant evidence of damage caused to stream ecosystems as a result of human activities. Well known examples include agricultural and urban development, forest harvesting and dam construction. These activities can directly affect streams by straightening them, replacing them with buried pipes, and removing important elements from them such as fallen trees and streamside vegetation. They can also indirectly affect streams by altering hydrologic and sediment regimes. Urban drainage systems, for example, move water quickly off the land surface and into streams, often increasing the size of floods streams must regularly carry (Leopold, 1968). Alternatively, common logging practices such as clear-cutting can reduce the stability of hillslopes and lead to increased landsliding and sediment supply (Sauder *et.al.*, 1987). Ensuing damages can include loss of floodplain land through erosion, increased sedimentation, decreased water quality, and loss of fish and wildlife habitat (e.g. Emerson, 1971). There is a need for solutions to reduce the environmental degradation and economic losses that result.

Before focusing in on the specific objectives of this thesis, a number of terms need introduction. Firstly, *impacts* are the adverse effects visible in a stream ecosystem. They are what call attention to the problems in specific streams. These impacts are caused by *disturbances*, defined as events that interfere with the order of a system. In addition to human activities, disturbances can include many natural events such as landslides, extreme floods, and climatic shifts. *Recovery* is the tendency of a system to return to an ordered condition. In stream ecosystems processes of recovery include erosion, deposition, vegetation growth, and selective removal of small sediment sizes to armour a channel's bed.

Stream restoration is the group of activities whose objective it is to accelerate the natural processes of recovery (Bradshaw, 1994). If effective, stream restoration enables streams to stabilize at a faster rate than would occur without assistance (Milner, 1994). In the typical B.C. forested ecosystem, for example, full natural recovery "may be impossible to achieve in less than 500 years" (Bradshaw, 1994). It is because of the time required for natural recovery that people attempt to assist the process in order to reduce the adverse impacts that human induced disturbances have on stream ecosystems. This is not a simple task.

Bradshaw (1987) calls restoration projects the "acid test of our ecological understanding". They test not just how well we understand processes in isolation, but how well we understand the interrelation between all processes. The twin aims of this thesis were to interpret stream adjustments based on an understanding of what variables in the stream ecosystem have changed and to guide restoration efforts based on an understanding of how stream variables can be manipulated to accelerate recovery.

1.2 Background Theory

Stream ecosystems encompass physical, biological and chemical components. Where water quality is adequate, biological system function has been found to be largely a function of physical habitat (Frissell *et.al.*, 1986). As a result, a restoration strategy attempted, and the one investigated here, is to concentrate on accelerating the recovery of a stream's physical structure. This can be done on a range of scales from micro-habitats of specific organisms to an entire watershed. This study will focus on the *reach* scale (Schumm and Lichty, 1965). It is on this scale that fundamental theories of channel adjustment such as channel equilibrium have been developed and stream form can be defined by an average slope, width and depth.

Channel equilibrium has been defined by Blench (1957) as follows:

> "channels tend to adjust themselves to average breadths, depths, slopes and meander sizes that depend on (i) the sequence of water discharges imposed on them, (ii) the sequence of sediment discharges acquired by them from the catchment erosion, erosion of their own boundaries, or other sources, and (iii) the liability of their cohesive banks to erosion or deposition."

This definition is in line with previous work where streams in equilibrium were considered "graded" (Mackin, 1948) or "in regime" (Lindley, 1919).

Many studies in the past 50 years have used the concept of equilibrium to understand and predict the form of channels. Empirical approaches have analyzed statistical relations between important variables. They typically relate the size and slope of a channel to the amount of flow. Well-known examples are the early work of Leopold and Maddock (1953) and the regional study of Bray (1982b). In contrast, rational approaches have developed equations and models based on formulations of the physical processes taking place in a stream channel.

Ferguson (1986) has discussed the advantage of a rational approach. He stated that, "there are obvious difficulties in this approach but ultimately it promises greater geomorphologic understanding and predictive capability." This understanding and predictive ability are what is required for a solution to the problem of stream restoration. These abilities also separate it from empirical approaches. Statistical empiricism has led to identification of many patterns in stream systems but it is limited to restoration applications where conditions in a stream's watershed have not changed (Bray, 1982b).

A rational approach also has disadvantages. Ferguson (1986) alluded to "obvious difficulties" of accurately quantifying channel processes. Bray (1982b) stated that the main limitation to the development of a useful physically based model is the variation in sediment transport formulae. This thesis attempts to apply a rational model by working from the premise that it is possible to calibrate a model to fit the behaviour of individual streams.

This premise is dependent on two arguments. Firstly, there are many techniques available with which it is possible to extract useful information about stream systems. The air photo analysis of Mollard (1973) and the classification of valley and channel features by Kellerhals *et.al.* (1976) are two techniques which can quantitatively measure a stream's development with time. These techniques can provide definite measurements of equilibrium stream form and the values of independent variables which led to its development.

The second argument is if a rational model can be calibrated to the fixed baseline points, then the requirement of the process equations to predict precise values such as the specific amounts of sediment being transported, for example, is eliminated. What becomes more important are not the absolute values, but rather the relative values, i.e. how well the equations scale with changes to measurable stream parameters. With regards to sediment transport, Bagnold (1966) and Yang (1984) have argued that transport will scale with reach-averaged hydraulic parameters such as stream power. The study of Gomez and Church (1989) appears to confirm this and allows the possibility of applying a calibrated rational model.

Based on the arguments that past equilibrium can be measured and that available equations will scale accurately, the rational model of Millar and Quick (1993) was applied to streams in this thesis. The model combines process equations of flow resistance, sediment transport and bank stability to obtain equilibrium values for stream form.

The validity of applying an equilibrium model to stream restoration was also investigated in this thesis. Stevens *et.al.* (1975) and Roberts and Church (1986) have documented examples of "non-equilibrium behaviour" in alluvial streams. For these example streams, stream behaviour appeared to be chaotic because recurring disturbances prevented the streams from maintaining an equilibrium form. During time periods without disturbances, the streams still tended to self-adjust to a preferred form. The results indicate that disturbance regimes should be assessed prior to the application of results from an equilibrium analysis.

1.3 Study Sites

Three streams in British Columbia were used as case studies (Figure 1.1). Slesse, Shovelnose and Harris Creeks were chosen to represent types of common disturbances to British Columbia streams. Forestry harvesting in the riparian zone has been prevalent within the Slesse Creek watershed and the creek is typical of those disturbed by a reduction in bank stability. The lateral activity of the Squamish River has impacted Shovelnose Creek and the creek is an example of those disturbed by an extreme flood. Forest harvesting has been widespread on hillslopes within the Harris Creek watershed and the creek serves as an example of those disturbed by increased rates of landsliding and sediment supply to the channel.

1.4 Objectives

The objective of the thesis was to determine the applicability of the Millar and Quick (1993) model to both understanding responses of streams to disturbances and to guiding restoration efforts. In order to test model applicability, the following five objectives were addressed:

1. Calibrate the model of Millar and Quick (1993) to each of the 3 study streams;
2. Interpret past behaviour of each stream using the results of calibration;
3. Model possible restoration efforts for each study stream;
4. Assess limitations of the analyses; and
5. Develop recommendations for each study stream.

1.5 Thesis Outline

Chapter 1 is an introduction of the problem. Key concepts of disturbance, recovery, restoration, and equilibrium are introduced. An introduction to the key advantages and disadvantages of a rational approach is presented and the possibility of calibrating the model of Millar and Quick (1993) outlined. The three study sites are introduced and objectives are presented.

In Chapter 2, relevant literature is reviewed. Equilibrium is presented and discussed by looking at how the concept developed, some of its criticisms and limitations, and available methods for predicting its form. Various applications of prediction methods to stream restoration are presented and the abilities and limits of each approach discussed. The overall method as applied to each study site is broken down into a 9 step process.

In Chapter 3 the method used to calibrate and apply the rational model is outlined. Relevant formulae are presented and the model of Millar and Quick (1993) outlined.

Chapters 4, 5, and 6 are the case studies of Slesse, Shovelnose, and Harris Creeks respectively. Introductions to the problems of each stream are first presented along with descriptions of the watershed, the available history of fish populations, and restoration efforts to date. Histories of each watershed are presented, calibration of the model detailed and limitations investigated. Past stream behaviour is interpreted in each case and possible restoration efforts modeled. The potential for restoration is discussed and recommendations made. These chapters were intended to be reasonably complete studies of the respective streams.

Final conclusions and recommendations are presented in Chapter 7. These conclusions review specific results of this thesis, future work which may further this type of analysis, and the usefulness of a rational approach.

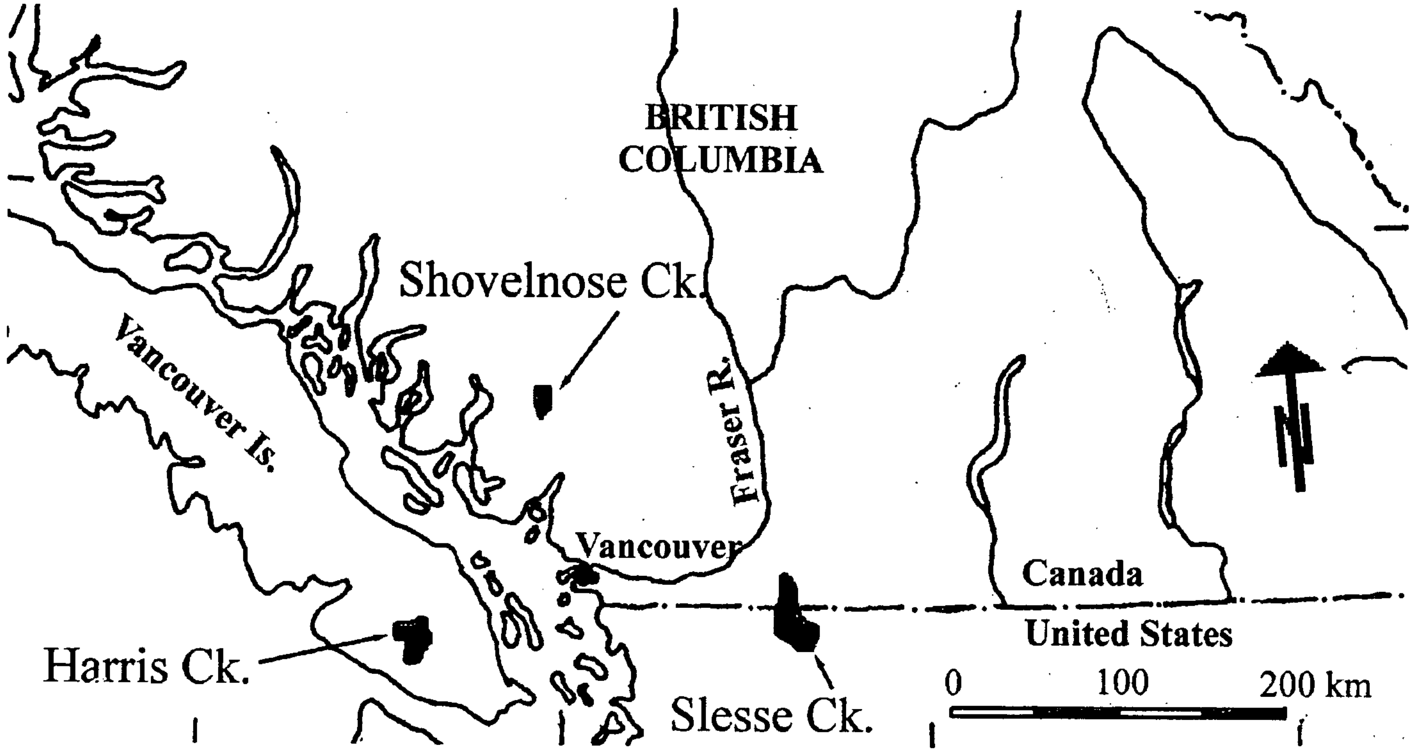


Figure 1.1 - Location of Study Sites

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Contents of this chapter have been grouped into three sections. Section 2.2 looks at the development of the equilibrium concept, the ranges over which it is valid, and criticisms of its applicability to natural streams. Section 2.3 looks at available methods for predicting a stream's equilibrium dimensions, how they have been applied to the problem of stream restoration, and the arguments for advancing a rational approach. In the third section, section 2.4, the model of Millar and Quick (1993) is briefly outlined.

2.2 Equilibrium

2.2.1 Development of the Concept

For the sake of brevity, this section is not a comprehensive review of the equilibrium concept. The intention was to present the ideas of equilibrium as clearly as possible so that the application of the concept in this thesis can be understood. For more detailed reviews, the reader is directed to the development papers of Mackin (1948) and Blench (1957) and more recent discussion papers such as Hickin (1983) and Lane and Richards (1997).

The concept of stream equilibrium arose from flume studies and observations of canals and natural streams in which stream dimensions that were free to change adjusted to preferred values, after which they remained near constant. Freidkin (1945) studied stream processes in flumes with bed and banks made of uniform sand. It was found that channel dimensions adjusted but eventually reached a condition of steady state in which variables no longer changed with time. This result has since been duplicated in flume experiments, including those of Wolman and Brush (1961) and Kellerhals (1967).

A similar result had been observed previously in India where canals "in regime" (Lindley, 1919) were found to transport supplied water and sediment loads without appreciable deposition or scour. Mackin (1948) and Blench (1957) extended the concept to natural rivers and called them "graded" or "in equilibrium". These streams exhibited consistency in slope and cross-section and were theorized to have adapted to supplied water and sediment loads, subject to local constraints.

Other work by Lane (1955b) and Schumm (1969) looked at stream adjustments and verified the chief diagnostic of the equilibrium concept. They found that alluvial streams (those flowing through transported sediment) responded to changes in their independent variables by adjusting their form, the dependent variables, to new values. If stream form had been changed artificially without changing the independent variables, streams were found to re-adjust dependent variables back to preferred, pre-disturbance values. Both Lane and Schumm developed qualitative relations shown below which can be used to anticipate the direction of changes.

Lane (1955b) suggested the following relation:

$$G_b D \propto Q_d S \tag{2.1}$$

where G_b is the bed material load, D is the sediment grain diameter, Q_d is the (dominant) water discharge, and S is the channel slope.

Schumm (1969) developed two relations as follows:

$$Q_d \propto \frac{W, Y^*, \lambda}{S} \tag{2.2}$$

$$G_b \propto \frac{W, \lambda, S}{Y^*, \xi} \tag{2.3}$$

where W is the channel width, Y^* is the mean channel depth, ξ is the sinuosity of the planform, and λ is the meander wavelength.

These relations do not provide quantitative predictions and commas are used to indicate the lack of a precise ratio, but the relations can be used to indicate direction of changes. For example, equation 2.3 indicates that width, meander wavelength and slope are likely to have a positive relation to changes in sediment transport while mean depth and sinuosity are likely to have a negative relation to changes in sediment transport.

2.2.2 Temporal and Spatial Ranges

Due to fluctuations in shorter scales and trends in the longer scales, equilibrium in streams can only be valid on intermediate scales of space and time (Schumm and Lichty, 1965). Long, intermediate and short scales are illustrated in Figure 2.1. The figure applies equally to temporal and spatial scales. Over long spatial scales, slope will vary from typically steep values in a stream's headwaters to typically milder values in the lower reaches. Over short scales, slope will vary depending on the location of the measurement. Riffles (faster flow sections) will have steeper than average slopes, while pools (slower flow sections) will have milder or even negative slopes. On the intermediate scale, an average slope can typically be found by measuring elevations of a repeating channel form over a number of repetitions. The definition of an appropriate length is called a *reach*, over whose length the downstream variation in independent variables will be small. Various guidelines for the division of streams into reaches are available. Those in the Channel Assessment Procedure for B.C. (Anon., 1996) were used in this thesis.

Streams in the long temporal scale, called cyclic or geologic time by Schumm and Lichty (1965), are subject to climatic, geologic, and valley slope changes. In the short or steady time scale, stream velocities and sediment transport will determined by the given channel form. In the intermediate or graded (Mackin, 1948) time scale, independent variables of flow, sediment transport, and bank stability will determine the channel form. Various definitions of appropriate temporal scales have been made. Mackin uses the phrase "over a period of years" in his definition of a graded stream, while Blench (1957) suggests 20-40 years as being a reasonable range. It is likely to be different for each stream depending on the rate of change of the valley slope, climate and geology. In this thesis it will be assumed that the graded time scale can be assumed for the design life of restoration measures, likely on the order of decades.

2.2.3 Independent and Dependent Variables

The following section describes the independent and dependent variables pertinent to an equilibrium analysis. From the aspect of a river reach, the independent variables are those that are imposed on the reach. The dependent variables, namely the hydraulic geometry, are those that vary in response to changes in imposed independent variables.

Independent Variables

Over graded or engineering time scales, independent variables are considered to be the climate, geology, vegetation type and density, relief, runoff, and sediment yield (Schumm and Lichty, 1965). Climate and geology are generally considered to determine the runoff, sediment size and sediment yield of the catchment. Vegetation is considered critical in determining the stability of the banks (Thorne, 1990). Relief can be represented by the valley slope (S_v). The independent variables are thus water discharge, sediment discharge, sediment size, bank stability, and valley slope. Representations of these parameters are discussed below.

Streams are subject to a range of discharges they must carry. The one most critical in determining the bankfull dimensions of the stream (Q_{bf}) has been characterized by different values in different studies. Lacey (1930) used the value most responsible for sediment transport and the formation of regime canals. This was termed the dominant discharge (Q_d) but is difficult to determine in practice because it requires information about sediment transport rates. Leopold and Maddock (1953) used the mean annual flow (Q_{ma}) in their regime analysis. Bray (1982a) found the highest statistical correlation for his data set to be between Q_{bf} and 2-year return period flow (Q_2). Church (1992) stated that there appears to be no universally consistent correlation between a particular flow frequency and the dominant flow, and indicated that smaller headwater streams may only be morphologically active in extreme events.

The characteristic sediment discharge (G_b) is assumed to be the bed material load corresponding to the characteristic flow (Hey and Thorne, 1986). A characteristic size is usually considered the median bed particle size, though some studies such as Bray (1982b) and Kellerhals (1967) have selected different characteristic sizes, and Parker (1978) has attempted to use the full range of observed particle sizes. The wash material load travels as suspended material through the reach and is not considered to exert an important role in stream morphology, though its deposition in slack water areas may be important for the development of the channel planform (Brice, 1968).

The identification of vegetation type and density as a critical variable is an indication of the importance of bank stability. Bank stability cannot, however, be measured directly. It has alternatively been ignored (e.g. Chang, 1982), related to a qualitative assessment of bank vegetation (Hey and Thorne, 1986), or related to bank soil properties (Millar and Quick, 1993, 1998; Darby and Thorne, 1996).

Dependent Variables

A variety of attempts have been made to identify the important dependent parameters of stream form. In perhaps the most comprehensive list, Hey (1988) identifies nine degrees of freedom as the width (W), mean channel depth (Y^*), slope (S), velocity (V), maximum depth (d_m), height of bedforms (Δ), sinuosity (ξ), wavelength of bedforms (λ), and meander arc length (r_c). This list can be simplified down to three primary variables of W, Y^*, and S. The other variables can generally be considered to be of secondary importance as they can be determined from the three primary variables as discussed below.

Sinuosity $\xi = S_v / S$ by definition and V can be determined from continuity, eliminating them from separate consideration. Variables d_m and Δ are related to resistance to flow. Resistance to flow has been alternately described using Manning's roughness (n) or equivalent roughness (k_s). Due to a poor understanding of how a channel will adjust its resistance to flow, this parameter is often assumed to be constant and imposed, although Davies and Sutherland (1983) and Yang (1987) indicate that it may vary or even optimize in some situations. One approach to determining λ and r_c is that of Leopold and Wolman (1957, 1960), and followed by Williams (1986), who developed empirical relations scaling with channel size. The work of Furbish (1988, 1991) and Quick (1974), however, indirectly critiques these relations by finding evidence of "chaotic behaviour" in channel planforms. Their work studied the movement of meanders and found them to be completely transitory elements that did not adjust to a preferred equilibrium value. Field work of Brice (1968) and Nanson and Hickin (1983) supported this claim. Average λ and r_c values are maintained by progression and cutoffs but they are not equilibrium features because they do not adjust to preferred values if disturbed.

Summary

Independent variables have been identified as bankfull discharge (Q_{bf}), bed material load (G_b), bed material size (D_x), bank stability (various formulations available), and valley slope (S_v). Because of the complexity and uncertainty associated with channel roughness, it will be assumed that the roughness coefficient (n or k_s) can also be treated as an independent variable. Primary dependent variables are the width (W), the mean depth (Y^*), the slope (S).

2.2.4 Disturbance and Stability

Stevens *et.al.* (1975) studied examples of streams exhibiting 'non-equilibrium' behaviour. The Gila River in Arizona, for example, was found to have exhibited large fluctuations in channel width (Figure 2.2). The river form was determined to be the function of the sequence of flood events. The primary indicator of this type of behaviour was concluded to be the ratio of the magnitude of extreme flood events to the average annual flood. In the Gila River the flood of record was 10 times the mean annual event. Rivers in regime typically had much smaller ratios. The flow of record in the middle Mississippi River at St. Louis, for example, was only 2.4 times the mean annual flood. In the Padma River in Bangladesh the largest recorded flow was only 25% above the mean annual peak flood. The form of these streams was more consistent with time (Stevens *et.al.*, 1975).

Roberts and Church (1986) detailed the adjustments of streams in the Queen Charlotte Islands to a change in sediment regimes. Due to logging within the watershed, sediment wedges had accumulated in the streams. The downstream movement of these wedges increased sediment transport up to 10 times, increased sediment residence time up to 100 times, and induced aggradation, braiding and lateral bank erosion. The approach and passing of a wedge crest often resulted in widened unvegetated channels that remained poor fish habitat for 20-30 years. Once the crest passed, deposits were stabilized by armouring, vegetation growth, the adoption of a single thread meandering channel, and the progressive decrease in the active channel width.

Both the above examples can be interpreted using an equilibrium approach. Disturbance has been defined earlier as events that interfere with a previous order. The Gila River has had multiple disturbances, exhibited by sudden increases in width. Once disturbances passed, however, width tended to self-adjust toward a preferred value. This value was not maintained because of repeated disturbances but the width did display the chief diagnostic of an equilibrium based system by tending to self-adjust to a preferred value (Mackin, 1948). The streams studied by Roberts and Church (1986) exhibit the impacts of a long term disturbance. Sediment waves move at a slower rate than flood waves and the disturbance events are stretched over 20-30 years. Once the crest passed, however, the processes of recovery began.

The impact of a disturbance will vary depending on the channel it is flowing through. The differences can be attributed to differences in *stability*, defined as the resistance to sudden change. Warner (1994) detailed the adjustments of a stream whose climate cycled between drought and flood dominated regimes. Sand-bed channels with thin vegetation and narrow floodplains were found to adjust quickly to regime changes, sometimes with each peak flow event. The result was a stream whose form reflected the flood history. In contrast, mixed load or gravel bed channels with good vegetation and wide floodplains adjusted slowly or not at all to regime changes. The result was a stream form that reflected a long term, regime condition.

Additional factors that have been found to stabilize channels include large woody debris (LWD) (Keller and Tally, 1979), boulder interlocking and stream bed armouring (Church, 1992), and cohesion of bank materials from silt and clay particles (Thorne, 1990). Very stable streams such as those controlled by step-pools will have their geometries imposed on them for much of the time, only being able to do work on the boundary in exceptional floods (Whittaker and Jaeggi, 1982). In extreme cases, current streams may be stabilized by past events such as glaciation that prevent them from adjusting their current form to match independent variables (Church and Slaymaker, 1989). Church (1992) and Yang (1987) suggest that these channels may respond by adjusting other parameters such as flow resistance.

2.2.5 Summary

Lane (1955b) stated that "it is believed that most alluvial streams may be said to be in this equilibrium or graded condition." While considerable evidence for equilibrium has been found, this statement has been challenged by a number of sources. Studies have shown that large fluctuations in water and sediment discharges can disturb streams and result in a variety of stream forms. Streams with low stability from factors such as vegetation and bed armouring have been found to reflect the recent flood history and not a long term average. Criticisms highlight the need to assess disturbance regimes and channel stability within each of the study sites. As stated by Lane and Richards (1997), "it may not be possible to conveniently separate time scales as a result of interacting water and sediment waves traveling at quite different rates", which indicates that restoring to an equilibrium condition will not be suitable for some streams.

Nevertheless, equilibrium remains fundamental to the understanding of channel adjustments, even in streams that have been disturbed. If we wish to accelerate the recovery of the stream ecosystem, the equilibrium condition represents the one condition that should be maintained by channel processes, initiating self-adjustment if it is disturbed. It thus represents a useful condition to predict for the purposes of guiding restoration efforts.

2.3 Prediction of Equilibrium Dimensions for Restoration

Strahler (1952) identified two types of stream behaviour studies. Historical studies trace the development of a particular region or stream with time. In contrast, analytical studies remove local and time-dependant contexts from a range of streams in order to identify patterns and predictability.

Shields (1996) identified three ways of applying available knowledge to restoration. Template (intuitive) methods directly apply historical studies to restoration. Empirical and rational approaches are based on analytical studies. Empirical approaches apply statistically derived relations of observed patterns, while rational approaches apply relations developed from an understanding of the important physical processes. These three approaches are discussed below.

2.3.1 Template Approaches

Templates do not explicitly attempt to determine equilibrium dimensions of a stream. Rather, they attempt to copy and apply the dimensions of desirable stream reaches as measured in historical studies. A variety of techniques, including analyses of limiting factors, preferred habitat reaches, and historical records such as air photos, maps, and floodplain excavations, have been used to identify and measure desirable reaches.

Limiting Factors

A limiting factor analysis investigates the habitat requirements of desired species and highlights which factors in the current stream appear to be limiting productivity. Applying these studies to restoration assumes that channel dimensions and properties can be altered based on the needs of biological organisms. Comprehensive lists of the physical factors that are important for fish productivity have been developed such as those of Ward and Slaney (1993) for West Coast salmonids. Reeves *et.al.* (1991) provides a synopsis of some of the attempts of altering streams for the purposes of creating fish habitat. High failure rates were a noted problem.

Preferred Habitat Reaches

Newbury and Gaboury (1993) develop fish habitat templates by searching for preferred habitats in the target stream and elsewhere within the nearby region. A regional search identifies templates that are both productive fish habitat and in equilibrium. In addition to characterizing the reach averaged parameters, attention is paid to characterizing micro-habitats important for the biology of stream ecosystems. An example of a preferred reach survey is shown in Figure 2.3.

Historical Records

Air photo interpretation techniques, described in Kellerhals *et.al.* (1976) and Mollard (1973), help to characterize morphological parameters such as lateral and vertical stability, sediment transport, the condition of the watershed, and measure dependent parameters such as the width, sinuosity and meander wavelength. Berger (1992) describes a stream restoration project in the Blanco River in which air photos are used to measure, copy, and impose the streams historical condition.

Brookes (1986) used maps, other reaches in the stream, and excavations of the historic floodplain to return a stream in the Netherlands to its historic condition. The channel had been straightened due to agriculture on the floodplain, but historical maps provided sufficient detail to determine the original location and planform geometry. Excavations mapped sediment patterns in the cross sections of relic channels (channels left from past flows) to determine channel and sediment size.

Advantages of Templates

There are two advantages of templates. Firstly, templates can model habitat complexity in small scales. Studies have shown that the smaller scales can be critical to the productivity of biological species (Newbury and Gaboury, 1993). Secondly, with adequate information and if properly used, historical techniques can describe and measure the stream at points in time where the stream was in equilibrium. Direct measurements of widths and sinuosities can be determined from air photos and field surveys of relic channels can obtain historical values of widths, depths and sediment sizes. These measurements can provide valuable points of calibration.

Limitations of Templates

The main limitation of templates is they do not look explicitly for an equilibrium solution. As a result, they make a restrictive implicit assumption and cannot be applied to determining reach-averaged solutions except in very restricted circumstances. Failure of template designs has been a problem (Reeves *et.al.*, 1991; Newbury and Gaboury, 1993; Rosgen, 1994). The implicit assumption in the use of templates is independent variables in the "poor" stream will be the same as in the "good" stream. Available techniques try to reduce risk by using similar streams, but this may not be valid even if the good stream is simply the historical condition of the poor stream. Natural fluctuations may have occurred, disturbances may still be ongoing, and human activities may have changed the independent variables. The only circumstances where reach scale templates can be reasonably applied are those where it can be shown that no changes within the stream's watershed have occurred.

2.3.2 Empirical Approaches

The second approach used for restoration is the empirical approach. Empirical relations are statistical relations of observed patterns from a range of streams. Indirect relations are statements of the patterns observed between form elements (e.g. $W \propto Drainage\ area$), while direct relations are those observed between dependent and independent variables (e.g. $W \propto Q_{bf}$).

Direct Approaches

Leopold and Maddock (1953) developed hydraulic geometry relations based on the downstream increase in discharge in a given catchment. These relations between flow and width, and flow and depth have the following power form:

$$W = aQ^b \tag{2.4}$$

$$Y^* = cQ^f \tag{2.5}$$

It was found that constants a and c varied between streams but exponents b and f were consistent across a range of streams. When plotted, equations 2.4 and 2.5 resulted in a series of mostly parallel lines (Figure 2.4). These results have since been duplicated in a number of studies. Sheilds (1996) shows a number of equations developed for sand and gravel bed channels. It has been widely reported that the exponent $b \approx 0.5$ and $f = 0.33$ or 0.40. Church (1992) indicates that most large channels ($D/Y^* < 0.1$) will take the first value for f, while smaller channels seem to take the second value. The reason for this behaviour was not clearly understood.

Kellerhals and Church (1989) plotted data from streams as small as laboratory flumes to streams as large as the Gulf Current and obtained a general correlation between width and discharge as shown in Figure 2.5. Over many log scales it indicates a predictable relation. On smaller scales, however, there is significant scatter. For example, at $Q = 100$ m^3/s, the grouping around the relation represents a variation in width between 25 and 100 m even if braided rives are ignored.

In order to reduce scatter, authors have tried to restrict their data sets to streams with similar physiographic settings. Kellerhals (1967) looked only at gravel-bed streams with negligible bed loads and included a measure of particle sizes to improve depth estimates. Bray (1982a) used only stable gravel-bed rivers in Alberta with greater than 5 years of hydrologic data and obtained good correlations for width and depth estimates. The second approach to reducing scatter has been to stratify streams according to various geomorphologic criteria. Hey and Thorne (1986) stratified streams according to the amount and quality of stream bank vegetation in order to improve width estimates.

Slope was examined empirically by Leopold and Wolman (1957). Scatter around any single relation with discharge was tremendous, and they chose to account for the scatter by classifying streams into different channel patterns as shown in Figure 2.6. A relation of the type shown below was then used as a dividing line between the stream patterns:

$$S = gQ^h \tag{2.6}$$

Other authors have attempted to reduce or at least understand the scatter by incorporating other variables. Charlton *et.al.*(1978) and Kellerhals (1967) found that particle size exerted a strong influence on channel slope. Bray (1982a) also analyzed slope, however, and was unable to improve the relation with bed particle sizes. Instead, he classified streams according to an island code defined in Kellerhals *et.al.* (1976), and found a progression of stream types generally divided by slope-discharge relations. Hey and Thorne (1986) were unable to stratify the slope vs. discharge relation based on their bank vegetation classification system. Because of the poor correlations, they concluded that sediment load could not be ignored when predicting slope.

Indirect Approaches

Brush (1961) looked at both direct and indirect relations in his study of Pennsylvanian streams. The indirect relations found between drainage area and channel dimensions were similar to direct relations found with dominant discharge. The substitution of drainage area for dominant discharge is often made for practical reasons of measurement, especially in small catchments where streams are often ungauged. In doing so, however, they assume hydrologic homogeneity between the catchments, restricting the range of their applicability. Newbury and Gaboury (1993) compiled regional relations of this type and used them for restoration projects in Manitoba.

A second type of indirect empirical study in use is the classification system of Rosgen (1994), now often recommended for stream restoration applications (e.g. Anon, 1994). This system uses a variety of geomorphologic parameters to separate streams into 7 major types and a host of subtypes (Figure 2.7). Partitions were based on empirical analyses of the various parameters which reportedly found stream types to group together. Individual data points, however, have not been published, and evaluation of scatter was not possible. It was observed that suggested ranges of the parameters are too broad to be applied for restoration. The slope of type C streams, for instance, have a suggested range of 0.1 to 2.0 %. Miller and Ritter (1996) protested the use of the Rosgen classification system for predictive purposes, citing a lack of consideration of equilibrium or hydrology. In response, Rosgen (1996) suggested that any references to the word "predict" in the original paper should be substituted with "imply".

Advantages of Empirical Approaches

Empiricism is based on the idea that streams will adjust to a preferred, ordered condition. The primary determinant of the condition has been observed to be the dominant discharge (Q_{bf}), and a coefficient has been used to account for other unmeasured processes. Empirical relations compiled on a regional basis can characterize the general pattern between discharge and channel size for local conditions. The advantage of this approach is that it is relatively easy to undertake, given the time and money for the necessary field work, because it does not require quantification of the difficult parameters of sediment transport and bank stability. Direct relations between Q_{bf} and W or Y^* can be used where streams are gauged or a hydraulic analysis has been done, and the drainage area (A_D) can be substituted for discharge in small ungauged catchments.

Limitations of Empirical Approaches

Empirical approaches are limited due to the unmeasured variables in some situations. Relations between discharge and slope are not accurate because of the importance of sediment transport. Regional relations of channel size cannot be applied where conditions in the watershed have changed as the unmeasured variables have also likely changed. Common examples of these situations are where dams have been constructed and sediment transport has changed or where the riparian zone has been logged and bank stability changed. These activities will change the coefficients to new and unmeasurable values (Bray, 1982a) and prevent application of the empirical approach to channel restoration.

2.3.3 Rational Approaches

The third available approach is the rational approach. Rational approaches are developed from an understanding of the important physical processes in stream geomorphology. The important processes are considered to be flow resistance, sediment transport, and bank erosion or deposition. It is generally agreed that the problem is indeterminate (Hey, 1988), as there are more unknown variables than equations available to compute them. Extremal hypotheses formulate the nature of stream equilibrium and have been used to close the solution (e.g. Chang, 1982). The following sections will briefly review the physical processes and highlight limitations within current formulations of key processes. These limitations will reduce the accuracy of predictions from a rational model. References for full reviews of available formulations are provided. Formulations used in Millar and Quick (1993) are presented in section 2.4.

Flow Resistance

Flow resistance in natural rivers is a topic discussed in detail in Hey (1979), Bray (1982b), and Bathurst (1982). Available formulae are based on boundary layer theory in which the skin frictional resistance can be used to determine the velocity of the flow. The Keulegan (1933) form of the equation has been found to reasonably predict resistance by the above studies. In streams with relatively low gradients ($S < 0.01$), the equivalent roughness value (k_S) used in this equation has been correlated with characteristic grain diameters (Bray, 1982b). For high gradient streams ($S > 0.015$) flow resistance has been observed to better correlate with hydraulic radius and channel slope (Jarrett, 1984). These formulae assume the channel can be approximated as a straight uniform gravel-bed river (Hey, 1979).

Other forms of roughness in addition to skin friction include:

- internal distortion resistance generated by bends, discrete boulders or residual bed forms;
- spill resistance from acceleration and deceleration of the flow; and
- resistance due to movement of bed particles (Hey, 1979).

Applicable theory of these forms of resistance is not well developed. The presence of bedforms, large boulders, highly sinuous planforms, multiple channels, and large woody debris will thus decrease our ability to predict channel roughness. The relations are generally more applicable at higher flows such as bankfull conditions where the energy slope can be considered near constant.

Sediment Transport

Sediment transport formulae have been extensively reviewed by Henderson (1966), White *et.al.* (1975), and Gomez and Church (1989). Conclusions are almost universally negative. Highlighted problems include:

- the use of empirical coefficients generated with laboratory or limited field data sets;
- supply limiting in many Canadian rivers (Hickin, 1983);
- nonlinearity and the presence of thresholds,

- the importance of local values of hydraulic parameters over reach average values; and
- the inability to accurately measure sediment transport in field situations (Gomez and Church, 1989).

Simons and Senturk (1977) conclude that "the mechanics of sediment transport are so complex that it is extremely unlikely that a full understanding will ever be obtained. A universal sediment transport equation is not and may never be available." (p. 644).

The study of Bagnold (1966) was found to merit further study by Gomez and Church (1989) because it showed sediment transport to scale with stream power ($\Omega = \tau V$). Stream power and sediment transport were related as they represent the time rate of energy supply and dissipation respectively. The main problem is the unsolved question of the process efficiency. Transport formulae may be useful, however, if they can be calibrated to eliminate mean bias (Yang, 1984).

Bank Stability

Attempting to model width adjustments in rivers requires the inclusion of bank processes. For non-cohesive sediments, Millar and Quick (1993) developed a criterion based on the empirical study of the United States Bureau of Reclamation (USBR) in which the stability of non-cohesive banks was quantified with ϕ, the friction angle. The maximum value for coarse loose gravel sediments was found to be about 40° (Lane, 1955a). To account for the stabilizing effects of vegetation and particle cementation, ϕ was replaced with ϕ', the modified friction angle, and allowed to vary up to 90°.

Cohesive riverbank sediments have been modeled by Darby and Thorne (1996) and Millar and Quick (1998). They indicate cohesive bank stability to be dependent on the soil properties such as the cohesion (c), the friction angle (ϕ), the specific weight (γ), bank height (Y_o), and critical shear stress (τ_{crit}).

The inability to directly measure parameters such as ϕ' and τ_{crit} has limited the inclusion of bank stability in rational models. Millar and Quick (1993) developed a method for calculating ϕ' if a stable stream geometry is known but this technique is inapplicable to many restoration situations. A value of $\phi' = 40°$ can be assumed where banks are unvegetated. However, where the effects of vegetation, imbrication, or particle cementing are significant, values for ϕ' must be calibrated. Millar and Quick (1993) attempted this using the data of Hey and Thorne (1986). Hey and Thorne had measured streams in the United Kingdom and classified them according to the size and density of vegetation. By using their method of calculating ϕ' from stable stream geometries, Millar and Quick calculated average values of ϕ' for the various bank vegetation classes. Summarized results are listed in Table 2.1 and shows that the value of ϕ' increases with the density of bank vegetation.

Table 2.1 - ϕ' Determined Analytically by Millar and Quick (1993) using Data Sets of Hey and Thorne (1986)

Vegetation Type	Average ϕ'
I - Grassy Banks	44
II - 1- 5 % tree/shrub cover	52
III - 5 - 50 % tree/shrub cover	60
IV - > 50 % tree/shrub cover	66

Extremal Hypotheses

The indeterminate problem of a unique equilibrium geometry can be closed using an extremal hypotheses. Extremal hypotheses consider stream adjustments to be an optimization of a particular hydraulic variable. A variety of formulations exist. Their use is variously justified according to mathematical derivations (Parker, 1978), analogies to thermodynamics and general principles of least work (Yang, 1987), and agreement between resulting predictions and actual streams (Chang and Hill, 1977). Thus far, they have yielded the most success in closing the system (Bettress and White, 1987) but as noted by Ferguson (1986), their use has not been physically justified. Griffiths (1984) attacked the whole of extremal hypotheses as being an "illusion of progress", but his conclusions have been disputed because of an incorrect mathematical formulation (Song and Yang, 1986)

The minimization of energy dissipation rate theory of Yang and Song (1979) offers theoretical compatibility with Bagnold's (1966) study of sediment transport, as it was derived with an assumption of a closed (equilibrium) and dissipative mechanical system. Subject to constraints, the theory is purported to variously simplify to some of the other developed extremal hypotheses (Yang, 1987), including the maximization of sediment transport (Kirkby, 1977; White *et.al.*, 1982), minimization of stream power and minimization of slope (Chang and Hill, 1977), his own theory of minimum unit stream power (Yang, 1976), minimization of the Froude number (Yalin, 1992), and maximization of friction factor (Davies and Sutherland, 1983).

The use of extremal hypotheses can be justified based on their success when applied to natural streams. Simon and Thorne (1996) documented the adjustments of the Toutle River after the Mt. St. Helens explosion. Their results (Figure 2.8) directly support the minimization of unit stream power and slope, but results of friction factor did not indicate a maximization trend. Yang (1987) anticipated this by proposing that friction factor might only be maximized where constraints prevent other parameters such as slope from being minimized.

The success of extremal hypotheses is perhaps not surprising given their similarities to the concept of equilibrium. Gilbert (1914) first alluded to an extremal hypothesis when he proposed that rivers with a large supply of bedload adjust to transport it as efficiently as possible. In his definition of a graded stream, Mackin (1948) stated that "slope is delicately adjusted to provide .. just the velocity required for the transportation of the load supplied from the drainage basin." His use of velocity might currently be replaced with stream power, but the idea is there that the stream will form with just the slope required for the load, i.e. the slope will be minimized.

Advantages of a Rational Approach

A rational approach has long been recognized as having the most potential for understanding geomorphologic phenomenon such as the adjustments of a stream channel (Strahler, 1952; Mackin, 1963; Ferguson, 1987). The advantages are clear. Because the models are based on formulations of the physical processes, a causative relation can be determined. This allows them to be applied specifically to problems of modeling and prediction. For instance, in a channel where the dominant discharge has increased due to urbanization or climatic fluctuations it is known that the channel will enlarge. For restoration we need to know how it will enlarge. A rational approach allows the impacts of the flow increase to be predicted based on the relative strengths of the bed and banks. It also allows restoration efforts such as the addition of bed substrate or the reinforcement of banks to be modeled to determine the best option.

Limitations of a Rational Approach

The difficulty of applying a rational approach is the processes in natural streams are difficult to accurately quantify. Bray (1982a) and others have stated that the main limitation is the variation in sediment transport formulae. Other major problems identified in this section include the simplification of a number of complicated flow resistance phenomenon into a coefficient, and an inability to directly measure bank stability parameters such as ϕ' for non-cohesive soil and τ_{crit} for cohesive soil. These limitations had to be accounted for if a rational approach was to be applied. The conceptual approach is outlined as part of the summary below and detailed in Chapter 3.

2.3.4 Summary and Conceptual Method

Studies of stream behaviour were classified into historical types that look at particular streams and analytical types that study a range of streams. Three approaches to restoration have been developed from these studies. Templates apply historical studies to restoration by copying desirable stream reaches. Empirical approaches apply analytical studies by using statistical relations of the patterns found between relevant stream variables. Rational approaches also apply analytical studies but use formulations of the important physical processes in a stream system. All available approaches have limitations which prevent their application to channel restoration where independent variables have changed. Potential was perceived, however, in the complementary advantages of the three approaches. This led to the conceptual approach outlined below.

The difficulty of obtaining precise values of independent variables will limit the ability of a rational approach from predicting precise values of dependent variables. A rational approach has a distinct advantage, however, in that it is ideally suited to prediction because its process-based formulations entail a causative relationship. Conversely, historical and empirical techniques are limited in their ability to predict because important variables are lumped into coefficients during their development. The historical and empirical techniques have a useful advantage, however, because they can give a reasonable representation of historical or regime conditions without requiring the quantification of variables that are difficult to measure. This advantage complements that of a rational approach and allows a rational model to be calibrated. By first forcing a model to agree with observed local dependent variables, reasonable values of independent variables can be obtained. This then improves the confidence in the predictive ability of a rational model because it leaves it responsible only for the deviations from a known condition. This conceptual method is seen to minimize limitations while maximizing the understanding and predictive ability that can be gathered for restoration purposes.

2.4 Millar and Quick (1993)

This section will briefly review the model of Millar and Quick (1993) which was applied in this thesis. The model is a rational model in that it is based on formulations of the important physical processes in a stream. Important physical processes are considered to be flow resistance, sediment transport and bank stability. An extremal hypothesis is also used in the model. The model assumes a uniform trapezoidal channel as defined in Figure 2.9.

Input variables required by the model of Millar and Quick (1993) are the dominant or bankfull discharge (Q_{bf}), the median bed and bank particle sizes (D_{50}, $D_{50\,Bank}$), equivalent roughness (k_s), bedload transport (G_b) or channel slope (S), and bank stability (ϕ').

2.4.1 Flow Resistance

Flow resistance in Millar and Quick (1993) is calculated using the logarithmic Keulegan equation. This equation has the following form:

$$\frac{1}{\sqrt{f}} = 2.03\log\left(\frac{12.2R_h}{k_s}\right) \tag{2.7}$$

where f = friction factor, R_h = hydraulic radius (m), and k_s = equivalent roughness (m).

Velocity is calculated in the Darcy-Weisbach equation is as follows:

$$V^2 = \frac{8gR_hS}{f} \tag{2.8}$$

where V = velocity (m/s), g = gravitational acceleration (assumed = 9.81 m/s^2), and S = slope.

2.4.2 Bank and Bed Shear Stress

Millar and Quick (1993) distributed boundary shear stress into bed and bank components using the method of Knight (1981) and Knight *et.al.* (1984). In this method, the shear force *SF* is distributed between the bed and banks as follows:

$$SF_{total} = SF_{bed} + SF_{bank} \tag{2.9}$$

In the above papers shear forces were then distributed between bed and banks, but only for a rectangular channel. Flintham and Carling (1988) extended the analysis to trapezoidal channels. The percentage of the shear force being carried by the banks was given by:

$$\log \%SF_{bank} = -1.4026 \log\left(\frac{P_{bed}}{P_{bank}} + 1.5\right) + 2.247 \tag{2.10}$$

where P_{bed}, P_{bank} = the wetted perimeter of the bed and banks respectively (m). The mean bank and bed shear stresses are given by:

$$\tau_{bank} = \gamma Y_o S(0.01\%SF_{bank})\left[\frac{(W + P_{bed})\sin\theta}{4Y_o}\right] \tag{2.11}$$

$$\tau_{bed} = \gamma Y_o S(1 - 0.01\%SF_{bank})\left(\frac{W}{2P_{bed}} + 0.5\right) \tag{2.12}$$

where τ_{bed}, τ_{bank} = mean shear stress acting on the bed and banks respectively (N/m^2), θ = bank angle (°), Y_o = channel depth (m), and γ = unit weight of water (assumed = 9810 N/m^3),.

2.4.3 Bedload Transport

Bedload transport (G_b) was calculated using the Einstein-Brown equation:

$$g_b = \frac{g_b^*}{F_1 \rho_s \sqrt{(s-1) g d_{50}^3}} \tag{2.13a}$$

where $g_b = G_b/P_{bed}$ = dry bedload transport rate per unit width (kg/s/m), ρ_s = sediment density (kg/m^3), d_{50} = mean grain diameter of the bedload sediment, s = specific gravity of sediment (assumed = 2.65), g_b^* = the dimensionless bedload transport rate per unit width given by:

$$g_b^* = \begin{cases} 2.15e^{\left(-0.391/\tau_{d50}^*\right)} & \tau_{d_{50}}^* < 0.093 \\ 40\left(\tau_{d_{50}}^*\right)^3 & \tau_{d_{50}}^* \geq 0.093 \end{cases} \tag{2.13b}$$

τ_{d50}^* = dimensionless shear stress for the median bedload grain diameter given by:

$$\tau_{d_{50}}^* = \frac{\tau_{bed}}{\gamma(s-1)d_{50}} \tag{2.14c}$$

τ_{bed} = proportion of the shear stress acting on the bed (N/m^2), F_1 = unnamed variable given by:

$$F_1 = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd_{50}^3(s-1)}} - \sqrt{\frac{36\nu^2}{gd_{50}^3(s-1)}} \tag{2.14d}$$

and ν = kinematic viscosity of water (assumed = 0.000001 m^2/s). For gravel sediment $F_1 \cong 0.82$.

2.4.4 Bank Stability Parameter

The value of the bank stability parameter (ϕ') was calculated using the following equation, based on Lane (1955a) but modified by Millar and Quick (1993):

$$\frac{\tau_{bank}}{\gamma(s-1)D_{50Bank}} = 0.067\tan\phi'\sqrt{1-\frac{\sin^2\theta}{\sin^2\phi'}} \tag{2.15}$$

2.4.5 Extremal Hypothesis

The concept of maximum sediment transport capacity of White *et.al.*(1982) was used in the model of Millar and Quick (1993). This hypothesis is equivalent to the minimum power concept of Chang and Hill (1977) and computes the minimum slope.

2.4.6 Summary and Formulation

The model of Millar and Quick (1993) was used in this thesis. It calculates flow resistance with the Keulegan equation, bank and bed shear stresses with the analyses of Knight (1981), Knight *et.al.* (1984), and Flintham and Carling (1988), sediment transport with the Einstein-Brown equation, and bank stability using a stability criterion developed in Millar and Quick (1993). An extremal hypothesis of maximum sediment transporting capacity was used. A flow chart of the program is shown in Figure 2.10. Figure 2.10 a) shows a formulation of the variable-slope optimization and b) shows the formulation of the fixed-slope optimization.

2.5 Summary of Literature Review

In this chapter, equilibrium was investigated and defined. It was found to be applicable on intermediate spatial and temporal scales. The length of these scales will vary depending on the climate and geology affecting individual streams. Limits to the concept were found where streams are subject to frequent or long term disturbances. Various forms of stability were found which will help channels to maintain a consistent form, in the extreme case preventing the channel from adjusting at all to an equilibrium condition. Three approaches to applying available knowledge to restoration were identified. Of the three, a rational approach was found to be the only one applicable to problems involving prediction. Limitations in the ability of current formulations to quantify the important processes can be minimized by calibration of the model with the results of historical and empirical analyses. The model of Millar and Quick (1993) was reviewed and selected for application to channel restoration sites in British Columbia.

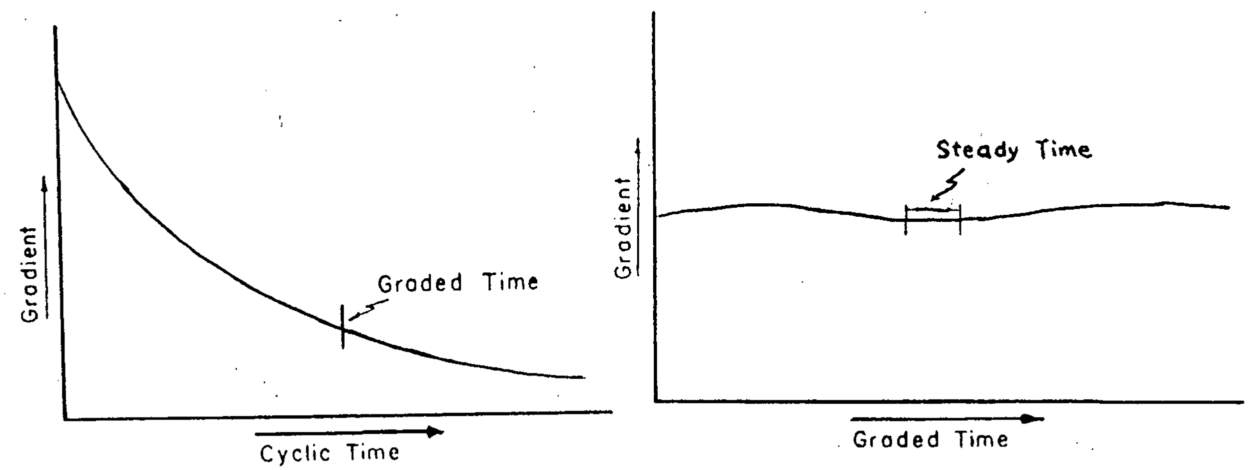


Figure 2.1 - Illustration of the concept of a graded stream on an intermediate time scale (Schumm and Lichty, 1965)

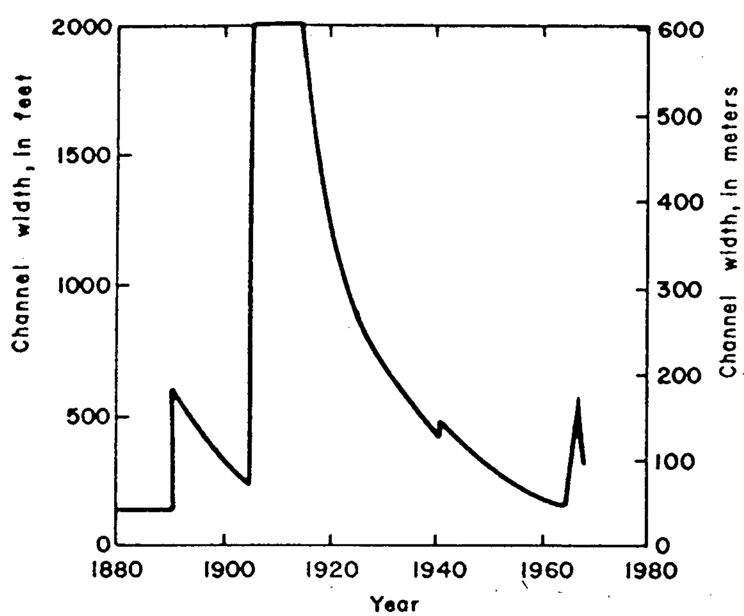


Figure 2.2 - History of the Gila River channel widths (Stevens et al., 1975)

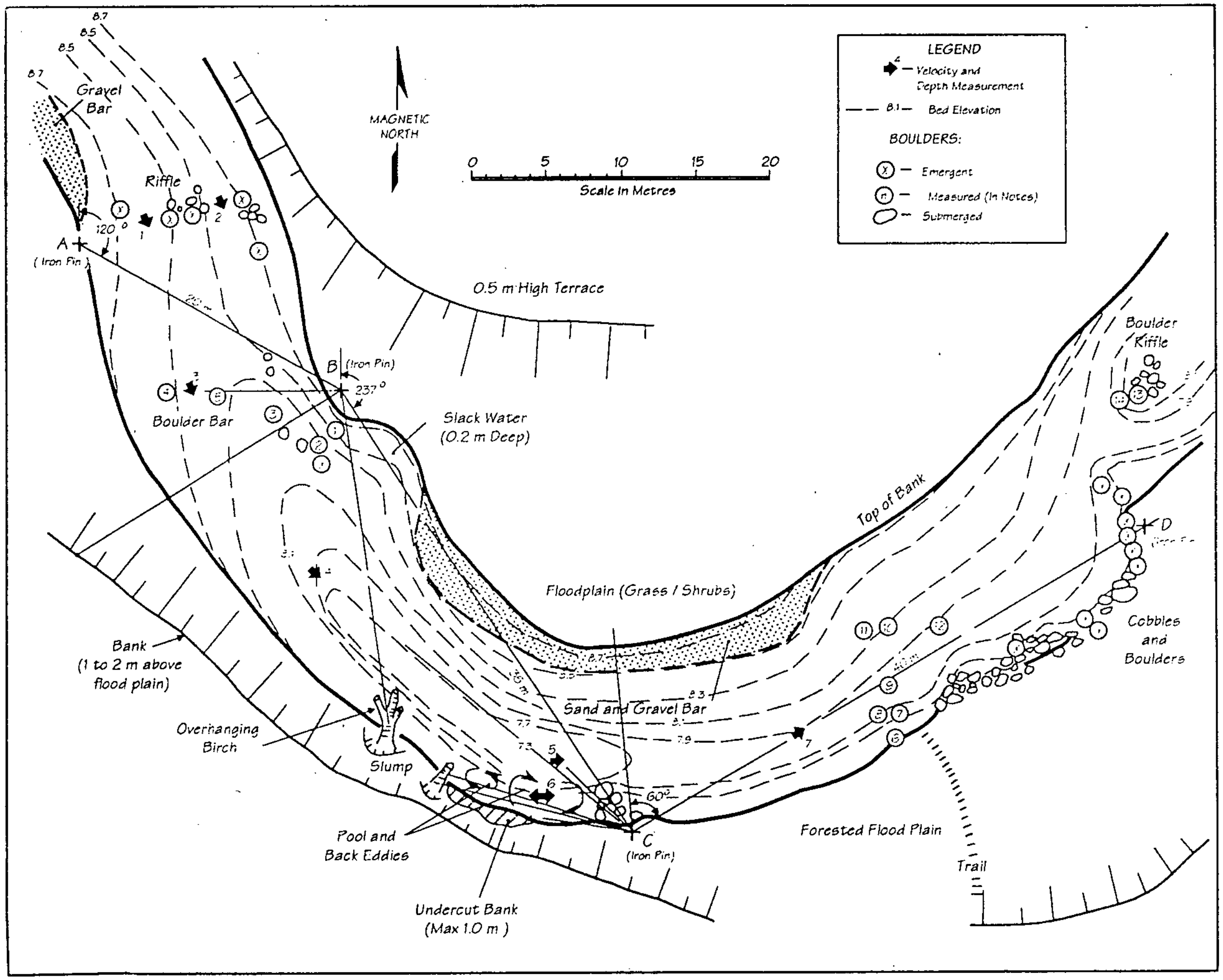


Figure 2.3 - A surveyed reach of preferred trout habitat on the lower Pine River (Newbury and Gaboury, 1993)

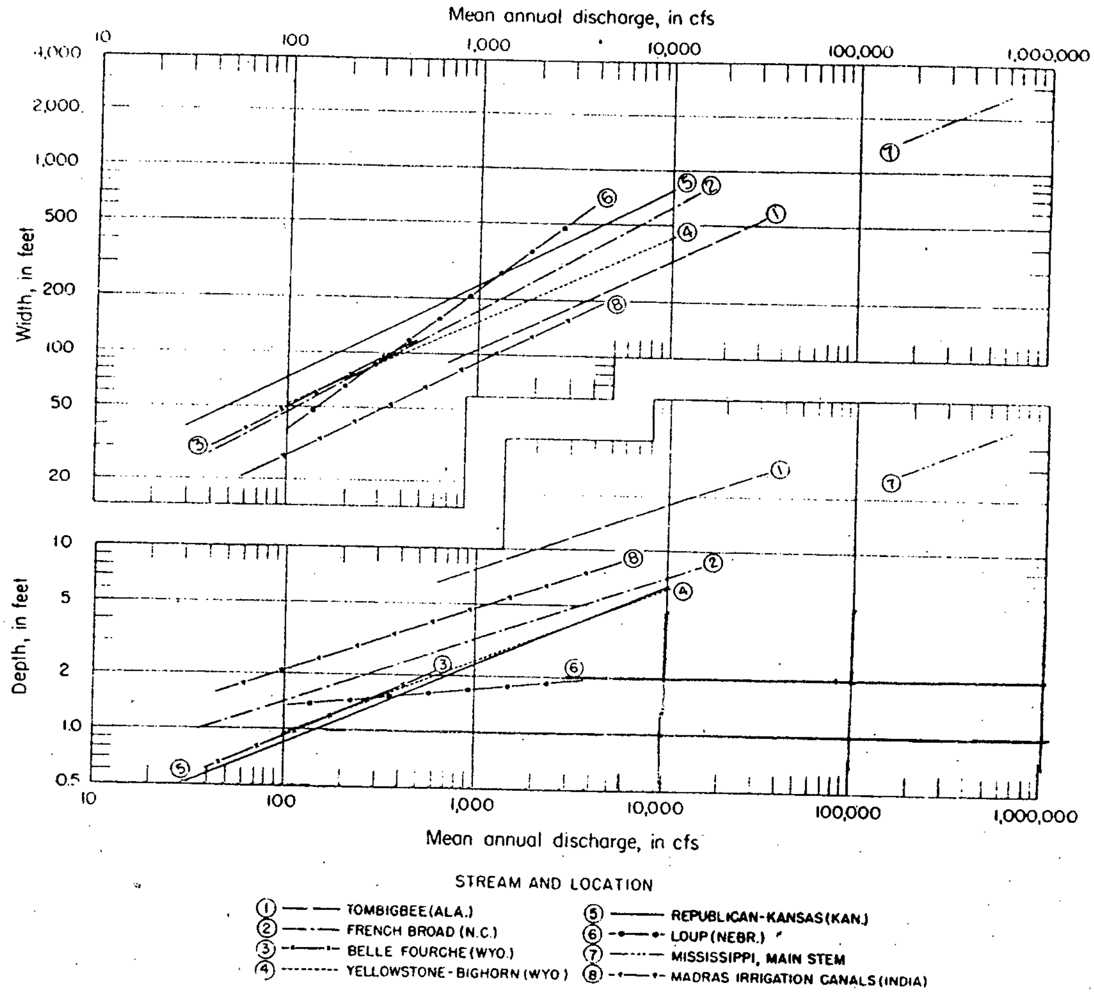


Figure 2.4 - Width and depth in relation to mean annual discharge as discharge increases downstream in various river systems (Leopold and Maddock, 1953)

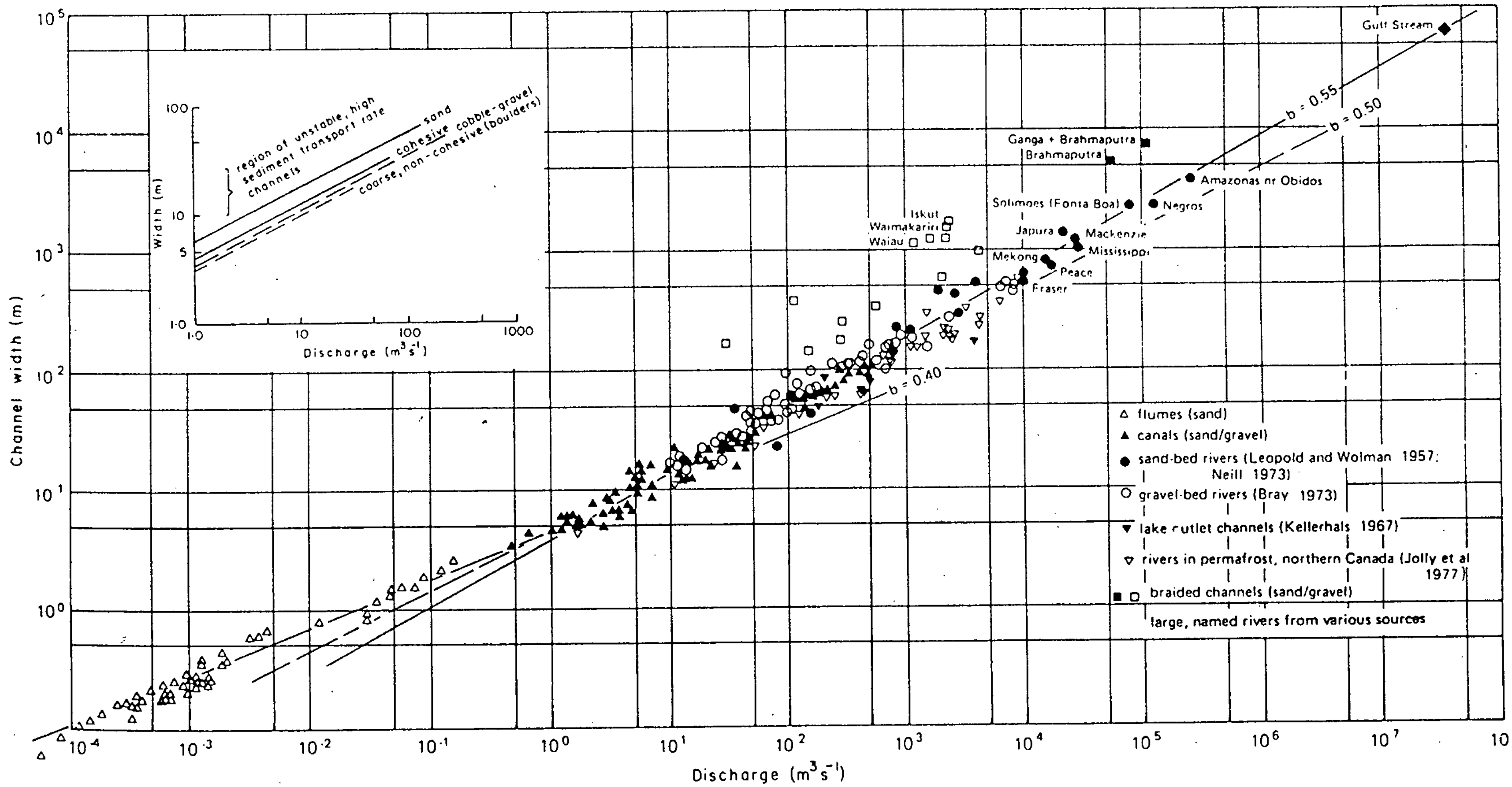


Figure 2.5 - Channel width vs. flow for a variety of data sets. Inset: Variation in channel width at a given discharge due to material (Kellerhals and Church, 1989).

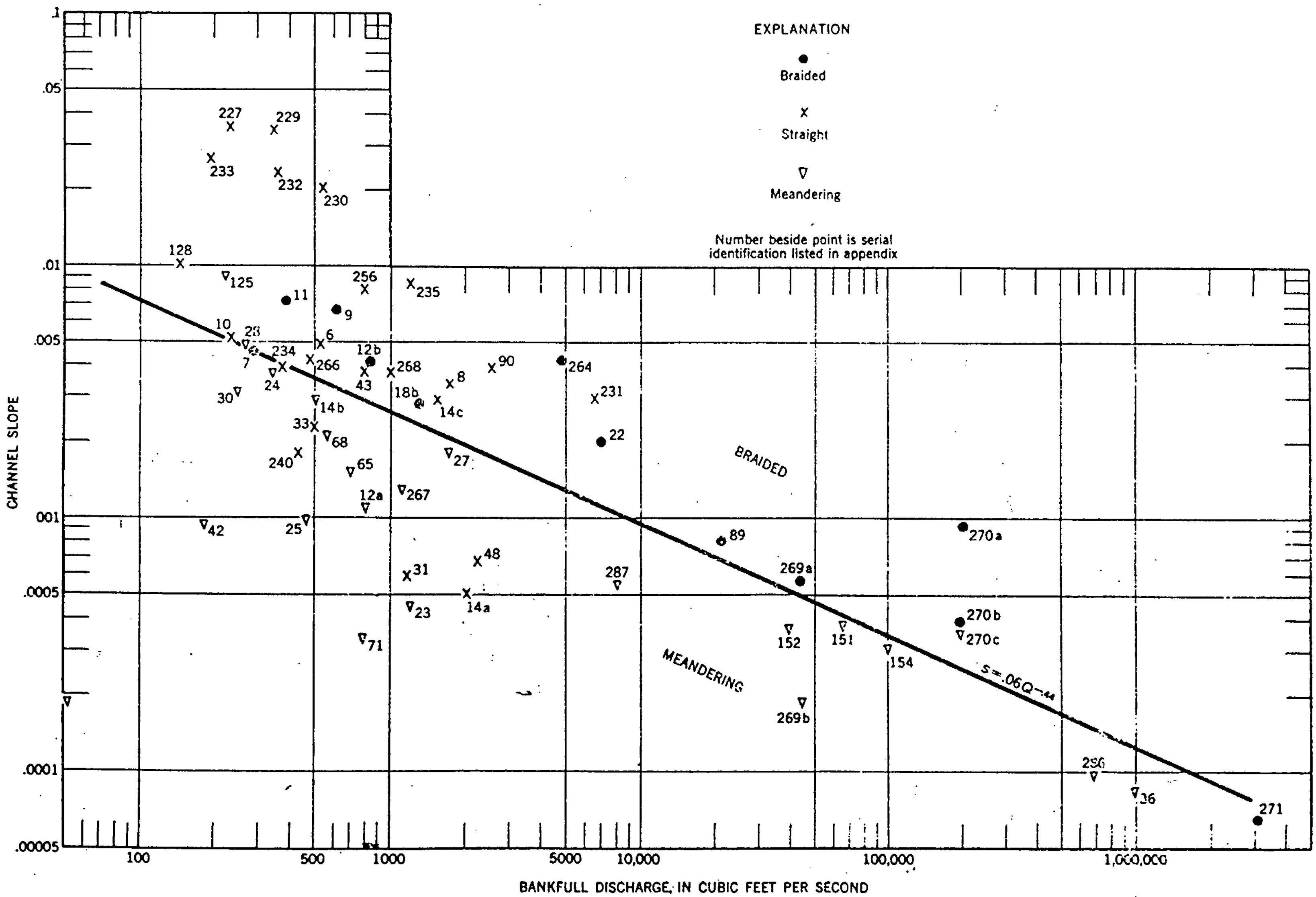
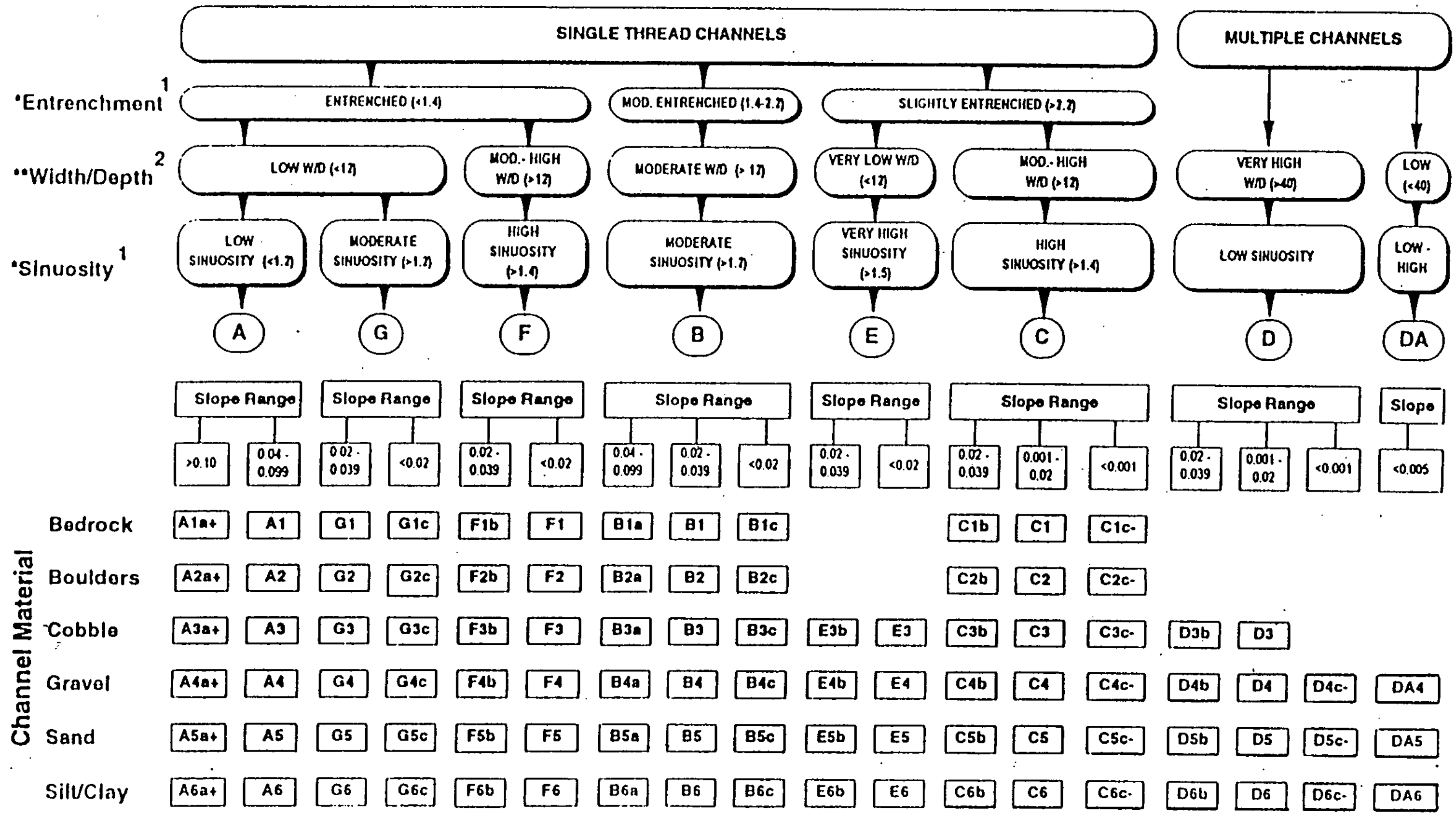


Figure 2.6 - Slope versus bankfull discharge for various natural channels. The drawn line represents the critical slope which separates braided from meandering channels (Leopold and Wolman, 1957)



1 Values can vary by ± 0.2 units as a function of the continuum of physical variables within stream reaches.
2 Values can vary by ± 2.0 units as a function of the continuum of physical variables within stream reaches.

Figure 2.7 - Key to Rosgen Classification System (Rosgen, 1994)

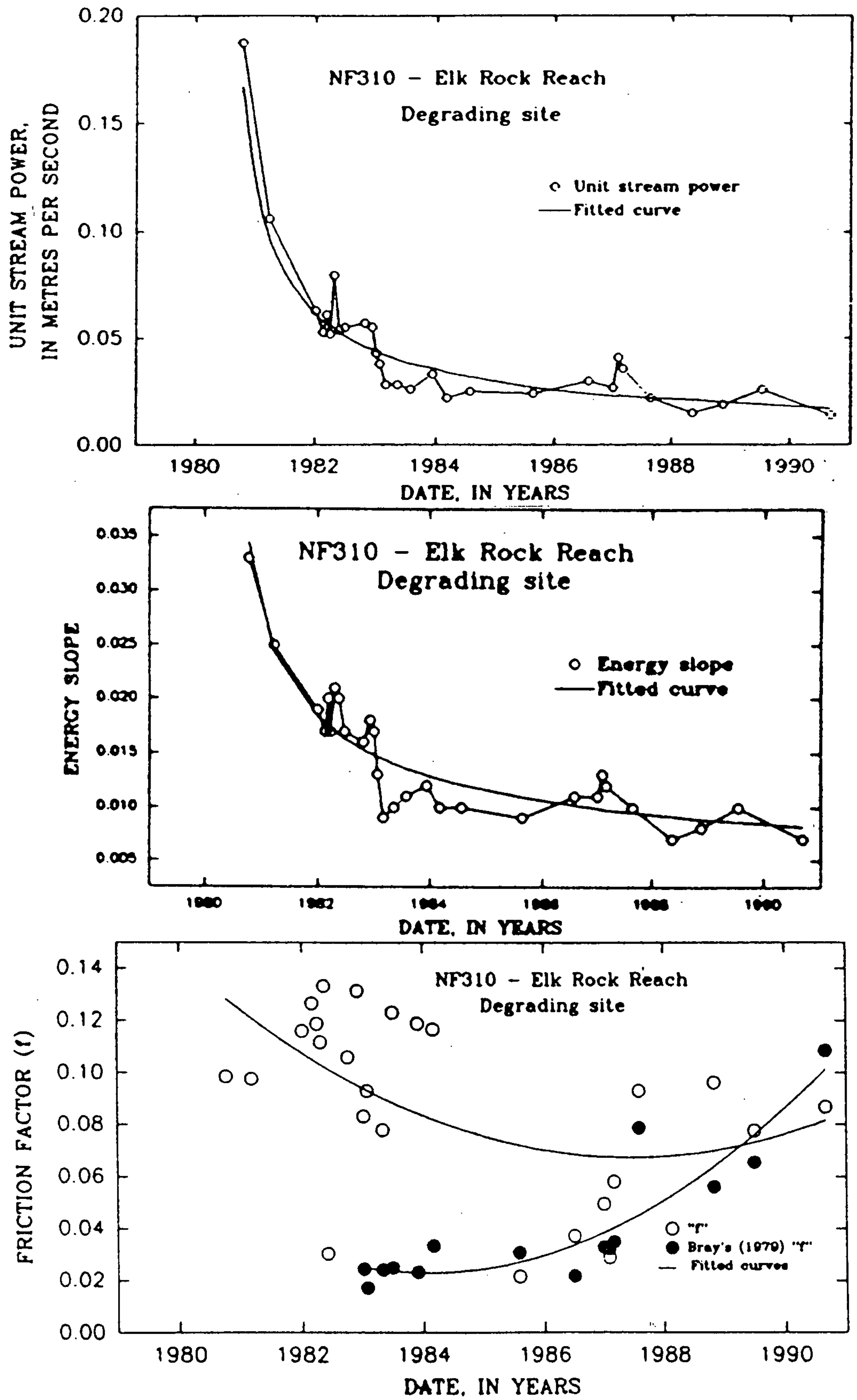
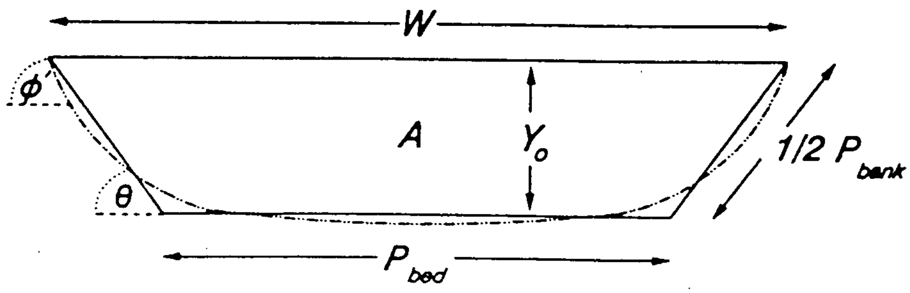


Figure 2.8 - Variation of a) unit stream power b) energy slope, and c) friction factor with time, Elk Rock Reach, Toutle River (Simon and Thorne, 1996)



$$R_h = \frac{A}{P_{bed} + P_{bank}} \qquad Y^* = \frac{A}{W}$$

-·-·-·-·-·- Actual Channel Cross Section

———— Simplified Trapezoidal Cross Section

Figure 2.9 Definition sketch of simplified trapezoidal channel for Millar and Quick (1993) model

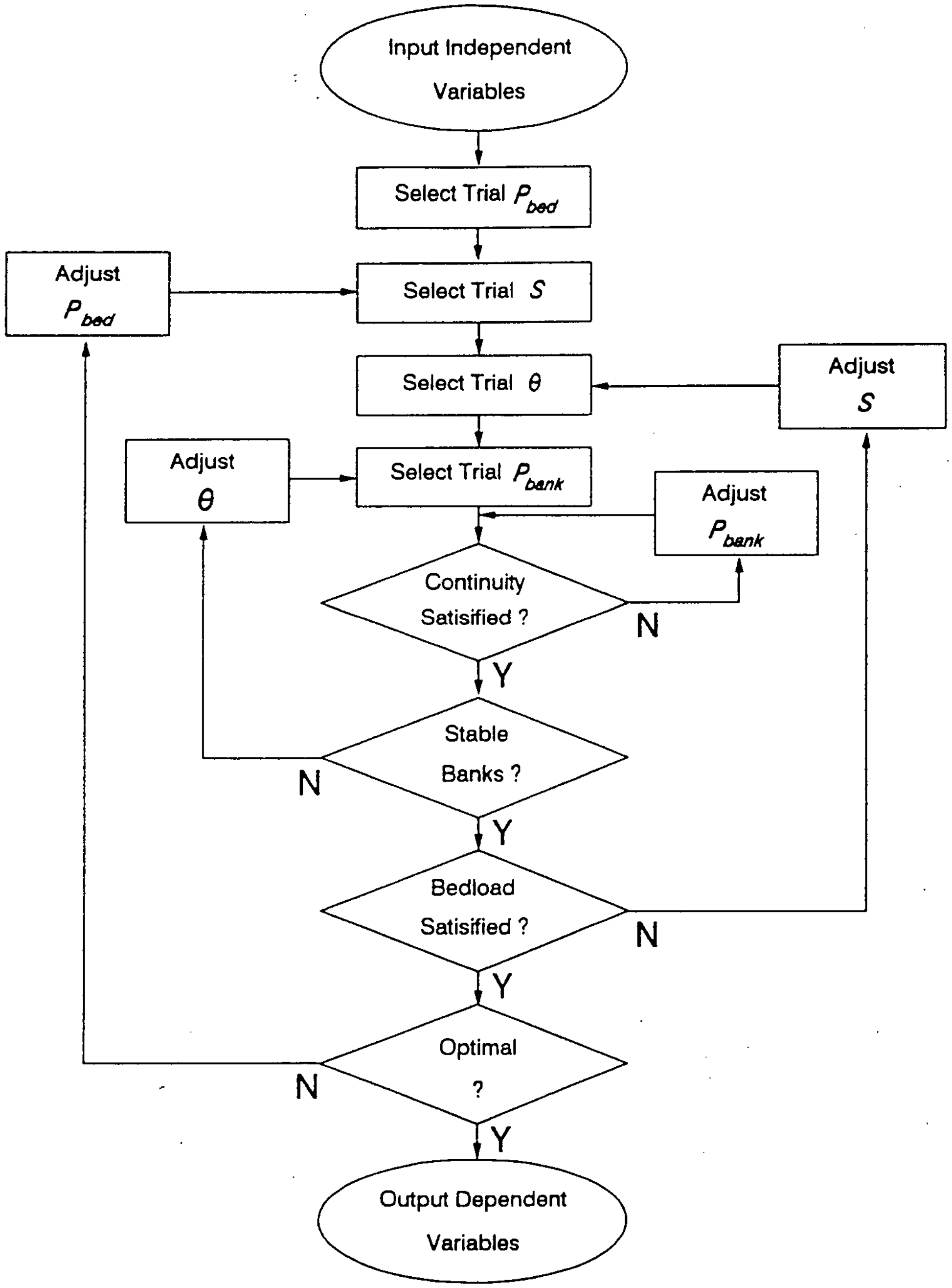


Figure 2.10 a) Flow chart for Millar and Quick (1993) - Variable-Slope version

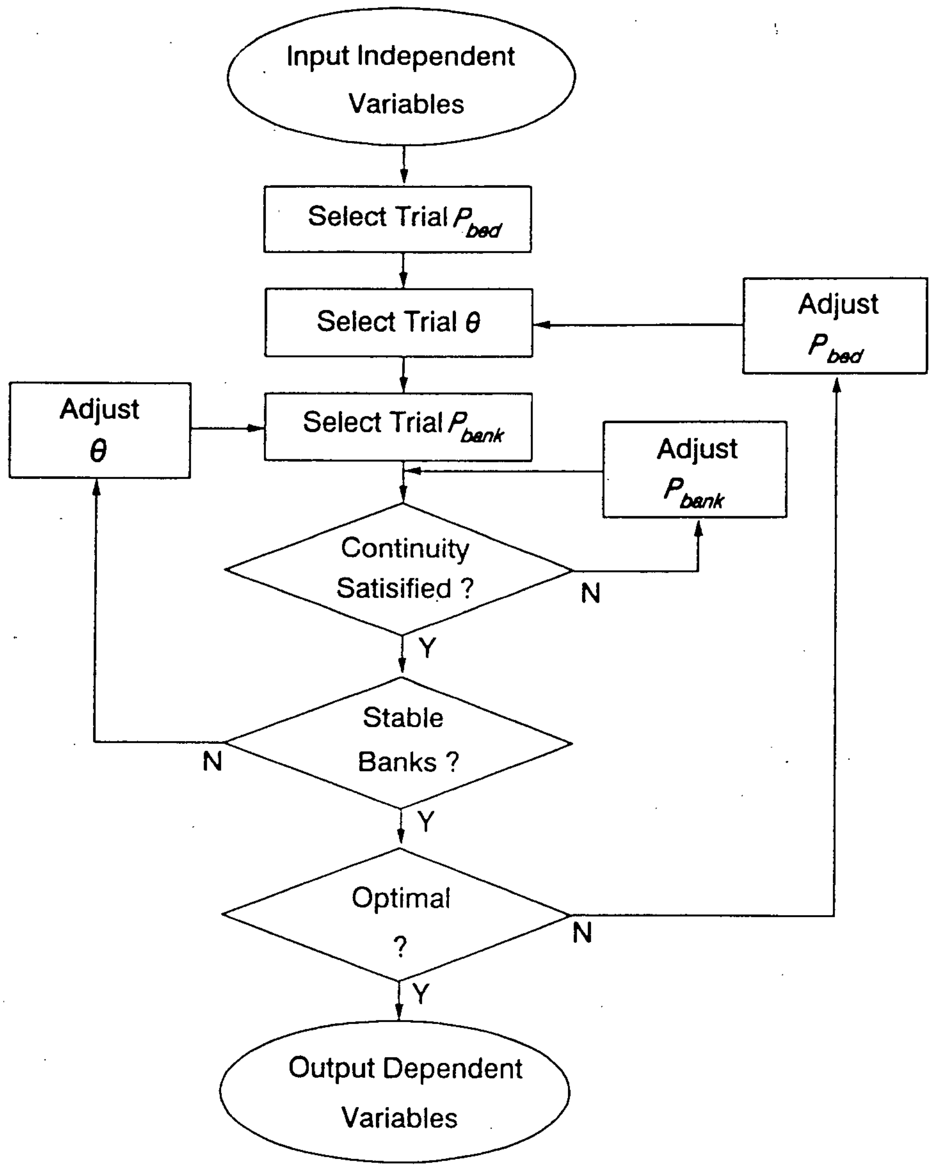


Figure 2.10 b) Flow chart for Millar and Quick (1993) - Fixed-Slope version

CHAPTER 3

METHODS

3.1 Introduction

This chapter will detail the methods used in this thesis. For each of the three study sites, the following steps were completed:

1. Review past reports;
2. Analyze flow records of the stream and other streams in region;
3. Classify and measure stream parameters using an air photo analysis;
4. Survey the current stream channel;
5. Establish values for independent variables used in the model;
6. Check the stream relative to a meandering-braiding transition;
7. Interpret stream behaviour;
8. Model restoration options; and
9. Analyze limitations.

From these steps, recommendations specific to the field site and more general recommendations regarding the general applicability of the method were made.

3.2 Step 1 - Review Existing Reports

Each stream used in this thesis as a test case was already being studied as part of ongoing restoration efforts. The first step reviewed what had been learned to date. Specific information for each site varied, but it generally included: an assessment of the problem; a review of past human activities including logging within the watershed; a review of changes to channel morphology; some information of channel flows; and an assessment of fish populations. These reports also usually divided the stream into reaches and directed attention to those of interest.

3.3 Step 2 - Analyze Flow Records

Flow records were analyzed to determine the mean annual peak instantaneous discharge, the year and relative magnitude of extreme flow events, and trends in the flow record over time. Where the record lengths were at least 20 years, mean annual peak instantaneous discharges ($\overline{Q}$) were calculated from the record. The year and relative magnitude of peak events were established from flow records. Nearby gauged streams were used to identify years of high floods for ungauged streams. To identify trends, cumulative departures from the mean were calculated using the following equation:

$$d_i = \sum\left(Q_i - \overline{Q}\right) \tag{3.1}$$

where d_i = cumulative mean at the i^{th} year and Q_i = current year flow. The cumulative mean (d_i) was then plotted versus the year and changes of slope used to identify trends. Results were interpreted by considering: periods with slopes parallel to the overall mean to have a mean equal to the overall mean; periods with negative slopes to have mean bankfull flows less than the overall mean; and periods with positive slopes to have a mean greater than the overall mean.

3.4 Step 3 - Air Photo Analysis

Historic air photos were analyzed and the channel measured using techniques described in Mollard (1973). Measured parameters were the width (W) and sinuosity (ξ). The reach-averaged value of W was established using the average of a minimum of five measurements, roughly equally spaced along the reach to ensure a representative distribution. The classification system of Kellerhals *et.al.* (1976) was used to describe observed changes. In addition to changes in the stream morphology, the floodplain and watershed condition were also noted, in particular the amounts and locations of forest harvesting activities.

3.5 Step 4 - Field Surveys

Current hydraulic geometry was surveyed using techniques as described in *Stream Channel Reference Sites: an illustrated guide to field technique* (Harrelson et. al., 1994), and *Stream Analysis and Fish Habitat Design: a field manual* (Newbury and Gaboury, 1993). A minimum of 5 cross-sections were used to establish average wet and bankfull widths and hydraulic mean depths. Pool, riffle, and glide sections were measured in equal ratios to that observed in the field. A longitudinal profile of thalweg (area of cross section with greatest amount of flow) elevations was completed, and repeating stream forms such as riffle crests were used to establish the average energy gradient. Bed particle size distributions and representative sediment sizes (e.g. D_{50}) were measured using the Wolman pebble count technique (Wolman, 1954). Where a significant amount of bed material appeared immobile, attempts were made to distinguish material being transported from material remnant from a past flow regime. Bulk samples were taken and sieve analyses done where the pebble count contained a significant amount of material less than 5 mm. Visible relic channels were also surveyed and roughly dated with vegetation growth.

3.6 Step 5 - Establish Model Inputs

Input variables required by the model of Millar and Quick (1993) are the dominant or bankfull discharge (Q_{bf}), the median bed and bank particle sizes (D_{50}, $D_{50\,Bank}$), equivalent roughness (k_s), bedload transport (G_b) or channel slope (S), and bank stability (ϕ'). Values of these variables were established using the methods described below.

3.6.1 Bankfull Discharge (Q_{bf})

Bankfull discharge (Q_{bf}) for the current and historic channels was established from the analysis of flow records. It was assumed that $Q_{bf} = \overline{Q}$. If the cumulative discharge plot was of sufficient length (> 25 years) and indicated that there was a trend or a shift that had taken place, then the means from each period were used as Q_{bf}. Where streams were not gauged, means were established from nearby gauged streams using the following relation (Harris, 1986):

$$\overline{Q}_U = \overline{Q}_G \left(\frac{A_U}{A_G} \right)^n \tag{3.2}$$

where $\overline{Q}_U$, A_U and $\overline{Q}_G$, A_G are the mean annual peak instantaneous discharges and drainage areas of the ungauged and gauged streams respectively and n is an adjustment factor with a typical value of $n = 0.75$.

3.6.2 Median Bed and Bank Particle Sizes (D_{50}, $D_{50\,Bank}$)

Current values for D_{50} and $D_{50\,Bank}$ were established from particle size distributions obtained from field surveys. Historic particle sizes were established from particle size distributions from relic channels where available. The techniques of Mollard (1973) were used to broadly categorize particle sizes from air photos where positive identifications could be made. Where historical particle sizes could not be measured, current values were used.

3.6.3 Equivalent roughness (k_s)

Equivalent roughness (k_s) was established using two opposing approaches to assess the applicability of various formulae. For low gradient streams ($S < 0.015$), the friction factor (f) was first back-calculated using established values of Q_{bf} together with measured channel dimensions:

$$f = \frac{8gA^2Y^*S}{Q_{bf}^{\,2}} \tag{3.3}$$

where A = cross-sectional area (m^2). By inverting the Keulegan equation, k_s was obtained:

$$k_s = \left(\frac{Y^*}{12.2}\right)^{\left(\frac{f^{\frac{1}{2}}}{2.03}\right)} \tag{3.4}$$

The value of k_s from Eq. 3.4 was then compared to the empirical estimate of Bray (1982b) where:

$$k_s = 6.8D_{50} \tag{3.5}$$

For high gradient streams ($S > 0.015$) roughness has been observed to correlate better with channel dimensions (Jarrett, 1984). For these streams Manning's roughness (n) was first back-calculated using established values of Q_{bf} together with measured channel dimensions:

$$n = \frac{A}{Q_{bf}} Y^{*\frac{2}{3}} S^{\frac{1}{2}} \tag{3.6}$$

The value of n from Eq 3.6 was then compared to the empirical estimate of Jarrett (1984) where:

$$n = 0.39 S^{0.38} R_h^{-0.16} \tag{3.7}$$

Calculation of historic values of roughness required measurements of S, Y^* and/or particle sizes. If there had been no significant lateral movement, S was assumed to be unchanged. If there had been lateral movement, pre-disturbance slope was established from current and historic values for ξ and the current S. Historical Y^* and particle sizes were established using relic channels where possible. Without relic channels, the current value of roughness was assumed applicable.

3.6.4 Bedload transport (G_b)

Sediment transport was calculated using the method outlined in Millar and Quick (1993). This involved the distribution of bank and bed shear stresses and the calculation of sediment transport using the Einstein-Brown equation. Relevant equations were presented in section 2.4. In order to eliminate a circular argument, the sediment transporting capacity of the upstream reach was calculated and used as the input to the model. Where the channel was laterally and vertically constraine, the fixed-slope version of the Millar and Quick (1993) model was used. An input value of sediment transport was not required for this version of the model.

3.6.5 Bank Stability (ϕ') and Model Calibration

Where the impact of vegetation was anticipated to be negligible, it was assumed that $\phi' = 40°$ (Millar and Quick, 1993). Where stable stream geometries could be measured, ϕ' was back calculated using equation 2.15. This required an estimate of τ_{bank} which was obtained from equations 2.10 and 2.11.

No procedure has yet been developed for independantly determining the value of the bank sediment friction angle (ϕ') where banks are stabilized by vegetation. Instead, ϕ' was obtained by calibrating the model of Millar and Quick (1993). Known input values were used in the model and ϕ' varied within its known range of 40° to 90°. ϕ' was then obtained based on an agreement between modeled and measured widths

3.7 Step 6 - Meandering-Braiding Transition

A meandering-braiding transition criterion has been recently developed by Millar (1998) based on the transition slope criterion of Parker (1976). Millar improved on the earlier formulation by including the effects of bank stability, a variable whose importance has been emphasized by Ferguson (1987). The Millar (1998) criterion finds a transitional slope (S*) as follows:

$$S^* = 0.0002 D_{50}^{0.61} \phi'^{1.75} Q_{bf}^{-0.25} \tag{3.8}$$

Channels with gradients steeper than S* tend to braid, while meandering rivers have gentler gradients. Meandering is used here in a loose sense and is more correctly defined as single-thread. The criterion thus separates channels that tend to have a single thread from those that tend to have multiple threads.

3.8 Step 7 - Stream Behaviour Interpretation

Following calibration, stream behaviour was numerically interpreted based on the results of calibration. The specific goals of this step were to a) clearly identify which independent variables had changed and to what degree and to b) separate the changes primarily responsible for the observed impacts from those whose impact had been minor. If the calculated changes to independent variables corresponded to what had been observed in a qualitative manner, this gave confidence to using the model to predict the impact of restoration alternatives.

3.9 Step 8 - Restoration Modeling

Restoration alternatives were modeled using the calibrated model based on possible changes to the local conditions in the reach. Sediment and water discharges were assumed to be a function of the watershed and independent of restoration work. Local variables that could be altered were channel roughness, bank sediment size, bank strength, and bed sediment size.

3.10 Step 9 - Analysis of Limitations

In order to assess the limitations of the analysis, three steps were taken. First, sensitivity was calculated. Following that, errors and assumptions in the measurement and calculation procedures were assessed and compared with parameter sensitivity. Finally, disturbance and stability were assessed.

3.10.1 Sensitivity

The sensitivity of the calibration was analyzed by modeling 10 and 25% over and under-estimates in each of the independent variables. Results were summarized in tables that assessed width, depth, and slope estimates to be insensitive, moderately sensitive and very sensitive to errors in each independent variable. Results were considered: insensitive if a 25% error produced less than a 10% change in the estimate; moderately sensitive if the same error produced between a 10 and 25% change; and very sensitive if the same error produced a greater than 25% change.

3.10.2 Sources of Error

Sources of error were examined including inaccurate, incomplete, or non-representative measurements, and assumptions or limitations in calculation methods. The significance of each error was assessed using the sensitivity analysis.

3.10.3 Disturbance and Stability

The role of disturbance was analyzed by investigating short term and long term fluctuations in water and sediment discharges. Trends in flow records and the occurrence of flood peaks many times greater than the mean annual flood were considered as risk factors of disturbance. Air photo analyses were used to analyze the role of short term fluctuations in sediment supply as well as the possibility of long term waves due to logging or other human activities.

Stability was defined as the resistance to sudden change. Stability can arise from any of a number of factors such as boulders and LWD as discussed in Chapter 2. The channel was assessed for stability by looking at the role that these factors were playing in each stream. Stability or instability in channel morphology was assessed from the air photo analysis to understand the influence stabilization factors had exerted on channel form.

3.11 Summary

The steps used to analyze each stream case study have been presented in this section. Within each study site the first objective was to calibrate calculations of stream processes to measured dimensions of the channel. Broadly, flow resistance was calibrated to the bankfull dimensions, sediment transport was calibrated to slope, and bank stability was calibrated to width. A successful calibration allowed restoration efforts to be modeled by varying local independent variables. To check limitations of the analysis, errors were discussed and compared to a sensitivity of model predictions. Disturbance and stability were also assessed to determine the limitations of an equilibrium analysis. Recommendations for restoration were developed for each stream from the outlined analysis.

CHAPTER 4

SLESSE CREEK

4.1 Introduction

Slesse Creek is a mountain stream straddling the Canada - U.S. border that supported large populations of salmonid fish species in the past. Increased lateral instability within the past twenty years has severely degraded the quality of fish habitat (Babikaiff and Associates, 1997). The stream's geometry has changed with active widths increasing by over three times as it shifted from a single to a multiple-thread, braided channel. The changes have occurred in spite of the protection of 60% of the catchment south of the border in a wilderness area. Forest harvesting in the riparian zone on the Canadian side of the border has been the chief cause of the problems. Specifically, the removal of mature forest from the banks and floodplain has destabilized the banks and left them subject to erosion at high flows. Restoration of the hillslopes and the stream has begun under the Watershed Restoration Program of B.C. through the Steelhead Society of British Columbia. The focus of this study is to develop recommendations for restoration of the stream channel.

4.1.1 Watershed Description

Slesse Creek drains 166 km^2 of the Chilliwack River catchment. Approximately 100 km^2 of the upper creek and headwaters are located south of the border in the Mount Baker Wilderness Area. Slesse Creek flows north from there into B.C. and joins the Chilliwack River approximately 19 km upstream of Vedder Crossing (Figure 4.1). It is a fourth-order stream (based on 1:20,000 scale maps), and the largest tributary in the Chilliwack system. Its valley is glacially carved and hillslopes are steep with frequent gullies. Upper reaches are often confined by bedrock and directly coupled with hillslopes. Lower reaches have typically wandered across a wide floodplain. The channel is vertically controlled due to the presence of large boulders remnant from glacial activity and a few points of bedrock control.

Reach Division

This report will use reaches lettered A-J as defined by Babikaiff and Associates Geoscience (Babikaiff and Associates, 1997). Reaches A-F are shown on Figure 4.2. Brief descriptions of the reaches are included to provide an overview and introduce specific reaches of interest.

Reaches A and B are located immediately upstream of the confluence with the Chilliwack River. Combined they are 1.6 km long and have an average slope of 2 - 3 %. Reach A is alluvial and has been the subject of some past restoration efforts. Reach B is confined within a bedrock canyon. Reaches C to G are a combined 5.8 km long and lies at slopes between 2 - 5 %. They are predominantly alluvial with a continuous channel flat and currently very wide and braided. Reaches H to J are 4.6 km long and have slopes between 5 - 7 %. The floodplain in these reaches is discontinuous and the channel is frequently confined. Beyond Reach J the channel lies within the United States and was not investigated.

This report will concentrate on Reach D for the following reasons:

- Reach D is an alluvial reach with a wide floodplain and little influence from bedrock controls;
- the morphology of the reach is relatively homogeneous with no major tributary inflows;
- the Reaches C, D, and E have experienced the greatest decrease in fish habitat and are currently limited by a lack of pools and overhead cover (Whelan and Associates, 1996); and
- current restoration efforts are being focused within Reach D (Babikaiff and Associates, 1997).

4.1.2 Fish Populations

Limited data of fish populations are available. No historic surveys or assessments of fish populations were found, although Babikaiff and Associates (1997) indicates that Slesse Creek has historically supported chinook, coho and sockeye salmon, steelhead, cutthroat, and resident rainbow trout, and Dolly Varden Char. Current fish populations were assessed by Whelan and Associates (1996). Electrofishing and dive surveys found coho salmon and rainbow trout present but not abundant in all reaches, bull trout upstream of Reach G, and adult chinook salmon in Reach A.

4.1.3 Restoration

There is active interest in restoring Slesse Creek. The project is being guided by the Steelhead Society of British Columbia under the Watershed Restoration Program (WRP). Road, landslide, and gully assessments have been done by Terrasol (1996), and some rehabilitative prescriptions have been implemented for road deactivation and off-channel fish habitat (Whelan and Associates, 1996). Further recommendations for fish habitat restoration have been made by Babikaiff and Associates (1997) and two bar stabilization projects were implemented in the summer of 1998 (Merideth Brown, SSBC, personal communication).

4.2 Watershed History

In order to apply the model of Millar and Quick (1993) to Slesse Creek, it was necessary to understand both the current condition and the changes that have occurred. In this section the history of stream morphology is reviewed along with the hydrology and forest harvesting activities. The analysis extends from current conditions back to the 1930's which is the limit of the air photo record.

4.2.1 Stream Morphology

Stream morphology was described from air photos of Slesse Creek shown in Figure 4.3 using the terminology of Kellerhals (1976). In 1936, Reach D was a wandering gravel/cobble bed stream with irregular meanders. Islands, side bars, and mid-channel bars were frequent. Boulders were frequent minor obstructions. The channel appears to have been laterally moderately unstable, with bank erosion focused at bend apexes and clearly visible avulsion tracks. By 1973, average channel width had decreased, although the meandering remained, and the colonization of bars by vegetation had formed islands that split the flow in some locations. Avulsions changed the location of the channel and slightly decreased sinuosity, but it remained predominantly a single thread channel. By 1993, channel width had increased by 4 or 5 times and the planform had changed to braided or multi-thread and appeared to be highly unstable. Sinuosity of the overall channel had decreased although the sinuosity of observable channels remained close to historic values. Most of the islands visible in 1973 had disappeared though new ones had been carved out of the old floodplain due to lateral activity.

Reach-averaged channel geometries of Reach D are shown in Table 4.1. Listed values were obtained from air photos in 1936, 1973 and 1993, and a field survey in April 1998. In the field visit, braided and single thread sections of the channel were observed. The width of the braided section was measured using air photos and the slope from the field survey. Rapids were observed to be controlled by accumulations of large boulders. Survey results and calculations are included in Appendix A.

Upstream, reaches are often controlled by bedrock and have tended to be more stable in widths and planforms. They are often directly coupled to hillslopes, and erosion and gully failures are sources of sediment input. Aggradation in upper reaches has resulted in periodic movements of sediment waves or slugs into Reach D, most likely coincident with periods of high flow

(Babikaiff and Associates, 1997). The catchment beyond the U.S./Canada border is pristine, and it was assumed that no major changes in stream morphology had occurred.

Table 4.2.1 - Channel Geometry of Slesse Creek - Reach D

Year	1936	1973	1993	1997 (dyked)
Source	Air Photos	Air Photos	Air Photos	Surveyed
W (m)	28	21	145	41
ξ	1.15	1.12	1.06	1.06
S	0.019	0.020	0.021	0.021
Y^* (m)	-	-	-	2.0
Planform	Single thread, wandering	Single thread, wandering	Braided	Single thread

4.2.2 Hydrology

Slesse Creek is gauged (Station # 08MH056) and records are almost complete since 1960. The gauge is situated just upstream of the confluence with the Chilliwack River and measures flow from 96% of the catchment. Figure 4.4 shows a record of maximum annual instantaneous discharges in Slesse Creek and an overall mean of 92 m^3/s. Using a plot of the cumulative departures from the mean (Figure 4.5), a clear shift was observed. There was a low flow period prior to 1977, during which the mean annual instantaneous peak discharge was 67 m^3/s. The largest flow of 110 m^3/s occurred in 1963. Since 1978 a high flow period was observed during which the mean has increased to 117 m^3/s. The six largest flows on record have occurred in this second period with the largest flow of 212 m^3/s occurring in 1978.

The observed peak flow fluctuations can be attributed to climatic fluctuations. Moore (1991) analyzed long-term climatic records from Agassiz and concluded that precipitation has remained constant over the last six decades. A climatic temperature increase in the mid 1970's, however, has led to less of the precipitation falling as snow. Church and Miles (1987) looked at the same records and found above average precipitation in the periods of 1917-24, 1948-56, and 1971-84.

Jordan (1990) looked at the records of Hope, BC and found above average rainfall intensities in the periods of 1906-21 and 1980-95. These studies indicate that climatic fluctuations similar to the one that has recently occurred are part of the regional hydrologic regime.

The change to peak flows is also coincident with but appears to be unconnected to forest activities in the watershed. Babikaiff and Associates (1997) notes that only a moderate amount of harvest has occurred in or above the transitional snow elevations, and none in the headwaters, suggesting the effect of forest harvest on the peakedness of rain-on-snow events has been slight.

4.2.3 Forest Harvesting

Information of forestry activities is summarized here from Babikaiff and Associates (1997). Forest harvest predates the earliest air photos of the watershed, taken between 1936-1940. Activities to that time consisted of roads and forest harvest near to the confluence with the Chilliwack River. Between 1936 and 1956, an extensive road network was built on the hillslopes up to Reach F. Most of the low elevation timber up to the E/F Reach boundary, including the riparian zone, was cut in this period. A riparian buffer strip was generally left, but it was narrow, and landings for cross-stream yarding were common. Between 1956 and 1973, logging progressed in an upstream and upslope direction, cutting most of the lower elevation timber up to the border, and the mid to high elevation up to Reach F. Since 1973, the smaller parcels of remaining forest have been cut.

Two forestry-related impacts were identified. Firstly there has been an increase in sediment supply from landsliding and torrenting. Failures within harvested gullies and debris slides from logging roads were most apparent in 1973 air photos. The relative magnitude of the increase over the level of natural sediment supply, however, appears to be small for two reasons. Firstly, the natural level of supply is high. A large number of avalanche chutes were visible on even the earliest photos. These chutes are much larger than the landslides and torrents attributed to logging activities, contribute material directly to the creek, and are expected to be active every melt season. Secondly, most of the watershed upstream of Reach D has not been changed. Hay and Company (1992) determined that 18% of the watershed had been harvested by 1992, and with a hydrologic recovery factor, the equivalent clear-cut area (ECA) was only 5% of the total watershed. The total area cut above Reach D was not calculated but has primarily consisted of low-elevation timber, and 60% of the total watershed is protected in the Mt. Baker Wilderness Area. These factors indicate that the impact of forest harvesting on water and sediment supply has been small.

The second impact has been a decrease in bank stability. The loss of the binding effect of tree roots and mature forest vegetation will lead to decreased resistance of the banks to erosion (Thorne, 1990). Impacts of root strength loss are likely to be lagged from the date of harvest due to the time it takes for roots to deteriorate. Babikaiff and Associates (1997) have identified eroded banks as the major source for material currently within the channel boundary.

4.2.4 Summary

The history and stream morphology of Slesse Creek have been examined. It was found that prior to 1973, the stream was moderately unstable and that channel avulsions and erosion at the outside of bends were common areas of activity. Since 1973 the creek has dramatically increased its width and changed to a highly unstable braided planform. Slope changes within Slesse Creek were minor and largely due to avulsions. Accumulations of coarse lag boulders are a clear indication of external vertical control (Kellerhals and Church, 1989).

Possible reasons for the changes to stream morphology have been investigated. Upstream sediment supply and hydrology have been only slightly affected by forest harvesting. A decrease in bank stability has resulted from widespread riparian logging in the watershed prior to 1956. Impacts were also coincident with an increase in peak floods due to climatic fluctuations.

4.3 Analysis

The application of the model of Millar and Quick (1993) was undertaken in four steps. First, model input values were quantified and the model was calibrated to the existing and past geometries. Second, the position of Slesse Creek relative to a meandering-braiding transition criterion developed by Millar (1998) was assessed for existing and past geometries. Third, past stream behaviour was interpreted based on model findings. Fourth, restoration options were modeled by varying input parameters over feasible ranges.

4.3.1 Model Inputs and Calibration

The fixed-slope version of the Millar and Quick (1993) model was used due to the vertical control exerted on the channel via coarse lag deposits. Figure 2.10*b* shows a flow chart of the model formulation. Required input variables were the dominant or bankfull discharge (Q_{bf}), the median bed and bank particle sizes (D_{50}, $D_{50\,Bank}$), equivalent roughness (k_s), channel slope (S), and bank stability (ϕ'). This section describes how these values were obtained for the current and historic conditions of Slesse Creek.

Bankfull Discharge (Q_{bf})

The bankfull or dominant discharge was assumed to be equal to the mean annual peak instantaneous discharge. As discussed in section 4.2.2, the available record indicates that Q_{bf} has not been constant. The recent channel has formed during the period of above average flooding, and $Q_{bf\,1993}$ = 117 m^3/s based on the short-term mean of flows between 1978 to 1995. The 1973 channel formed during a period of low flows, and $Q_{bf\,1973}$ = 67 m^3/s based on the short term mean of recorded flows between 1960 to 1977. No progressive trend was found in climatic records dating back to the beginning of the century, and the creek in 1936 was assumed to have formed with the long term mean, giving $Q_{bf\,1936}$ = 92 m^3/s.

Sediment Sizes (D_x)

The Wolman (1954) technique for pebble counts was used and samples taken in the single thread dyked section of the channel and in the wide braided section during field surveys. The pebble count taken in the braided section was not used because it appeared later to have been made in a deposition area. Only the pebble count from the single thread section were used, giving D_{50} = 0.133 m. Measurements of bed and bank particle sizes found $D_{50} = D_{50\,Bank}$. The particle size distributions are included in Appendix A. Due to a lack of historical information, particle size distributions were assumed to be representative of historical channels.

Flow Resistance (k_s)

Slesse Creek has a high slope (≈ 2 %), high bedload transport, and large bed material. The empirical formula of Jarrett (1984) has been developed for natural channels with high slopes and relates roughness to the hydraulic radius and slope. The formula obtained an estimate for Manning's n = 0.086, which corresponds to a value of k_s = 3.6 m. This value corresponds well with the value back-calculated from $Q_{bf\,1993}$ using equation 3.6 (n = 0.085). Historic values of k_s were assumed to be equal to the current value.

Channel Slope (S)

The current value of channel slope was determined from field surveys. Historic values were established from current values together with measuring sinuosities from air photos.

Bank Stability (ϕ') and Model Calibration

No procedure has yet been developed for determining the value of the bank sediment friction angle (ϕ') where banks are stabilized by vegetation unless depths and bank angles are known. For unvegetated banks, the minimum value of 40° can be assumed (Millar and Quick, 1993), giving ϕ'_{1993} = 40°. Using this value together with $Q_{bf\,1993}$ = 117 m^3/s, the model predicts a value of W = 140 m. This is close to the observed width of 145 m determined from the 1993 air photos. For past channel geometries where bank stability had been influenced by mature riparian vegetation, ϕ' was obtained by calibrating the model of Millar and Quick (1993). Q_{bf}, k_s, S, D_{50}, and $D_{50\,Bank}$ were input into the model, and ϕ' varied within its known range of 40° to 90°. Three plots of W versus ϕ' are shown in Figure 4.6. Each of the three curves corresponds to a different value of Q_{bf} as indicated in the legend. Estimated values of ϕ'_{1936} = 73° and ϕ'_{1973} = 75° have been interpolated from Figure 4.6.

Summary of Input Parameters

Values of input variables used in the rational model are shown in Table 4.2.

Table 4.2 - Input Variables for Slesse Creek

Year	1936	1973	1998
Q_{bf} (m^3/s)	92	67	117
D_{50} (m)	0.133	0.133	0.133
$D_{50\,Bank}$ (m)	0.133	0.133	0.133
k_s (m)	3.6	3.6	3.6
S	0.019	0.020	0.021
ϕ' (°) estimated	-	-	40
ϕ' (°) calibrated	73	75	-

4.3.2 Meandering-Braiding Transition

Millar (1998) recently developed a meandering-braiding transition criterion using the criterion of Parker (1978) and the Millar and Quick (1993) rational model. Equation 3.8 was used to calculate values for S^* shown in Table 4.3.

Table 4.3 - Meandering-Braiding Criterion for Slesse Creek

Year	1936	1973	1998
S	0.019	0.020	0.021
S^*	0.034	0.039	0.011
Predicted Planform	Meandering (single-thread)	meandering (single-thread)	braiding (multi-thread)
Observed Planform	Wandering (single-thread)	wandering (single-thread)	braiding (multi-thread)

4.3.3 Interpretation of Stream Behaviour

Interpretations of stream behaviour were made based on model results. Between 1936 and 1973, Slesse Creek decreased in width through vegetation growth on channel margins and mid-channel bars. This appears to have been caused by a short term reduction in Q_{bf} below the long term mean. Although channel banks had been logged, ϕ' remained high, suggesting that root networks were still intact and protecting the soil. Between 1973 and 1993 Q_{bf} increased above the long term mean and banks failed. Calibration of the model indicated logging had reduced ϕ' from 75° to 40°.

The relative significance of the high flows and reduced bank stability can be seen on Figure 4.6. If an increase of Q_{bf} from 67 m^3/s to 117 m^3/s had occurred without a decrease in ϕ' the width would only have increased between 5 and 10 m and the channel would have remained single thread. This modeled increase in width is within the range of widths observed in the 1936 channel, indicated that flow fluctuations are part of the natural regime. The critical change that led to over 90% of the increase in width has been the decrease in ϕ'.

4.3.4 Restoration Modeling

The rational model was used to assess the impact that altering bank stability (ϕ') and bank sediment size ($D_{50\,Bank}$) will have on the equilibrium channel geometry. Other parameters were held constant while ϕ' and $D_{50\,Bank}$ were varied one at a time through feasible ranges. Modeling results for W and Y^* are shown in Figure 4.7. Although calculated by the model, G_b is not shown because the slope of Slesse Creek is vertically controlled.

The meandering-braiding transition criterion was used to calculate threshold values of ϕ' and $D_{50\,Bank}$ that would induce a single-thread planform at the current slope. It was found from equation 3.13 that an increase of bank particle size to $D_{50\,Bank}$ = 0.37 m or an increase of bank stability to ϕ' = 57° would cause a reduction in channel width to $W \approx$ 60 m and induce a planform change back to a single thread geometry. Meandering-braiding thresholds are shown on Figure 4.7 as dashed vertical lines.

4.3.5 Summary

Model inputs were quantified and calibrated to understand the behaviour of Slesse Creek. Dominant discharge was below average in the 1960's and 1970's with Q_{bf} = 67 m^3/s and above average since with Q_{bf} = 117 m^3/s. The bank stability parameter (ϕ') has decreased from approximately 75° to 40° coincident with period of above average flooding as a result of logging in the riparian area. The decrease in ϕ' was found to be responsible for more than 90% of the observed channel widening. Increases in ϕ' and $D_{50\,Bank}$ were modeled to lead to the desired restoration goals of channel narrowing and increased stability. A single thread channel can be induced either by increasing $D_{50\,Bank}$ to 0.4 m (W = 45 m), or increasing ϕ' to 60° (W = 60 m).

4.4 Limitations of Analysis

In order to assess the impact of assumptions and errors in the analysis, three steps were taken. First, sensitivity was calculated. Following that, errors and assumptions in the measurement and calculation procedures were assessed and compared with parameter sensitivity. Finally, disturbance and stability were assessed.

4.4.1 Sensitivity

The sensitivity of the 1936 calibration to 10 and 25% errors in input variables was calculated. Graphs of the sensitivity of the estimate of the 1936 channel geometry are included in Appendix A. Results are summarized in Table 4.4.

Table 4.4 - Sensitivity of Modeling for Slesse Creek

Dimension	Insensitive to	Moderately Sensitive to	Highly Sensitive to
W	S, D_{50}	Q_{bf}, k_s, $D_{50\,Bank}$	ϕ'
Y^*	Q_{bf}, k_s, D_{50}	S, $D_{50\,Bank}$	ϕ'

4.4.2 Sources of Error

Sources of error were identified as the roughness relation of Jarrett (1984), which produced an unrealistically high estimate of k_s, several difficulties with the measurement of sediment particle sizes, the difficulty of extending a hydraulic analysis to a multi-thread channel, and judgment and measurement error during field surveys and air photo analyses.

Roughness Calculation

The roughness relation of Jarrett (1984) abandons the use of relative roughness and instead relates flow resistance to slope and hydraulic radius. The value of equivalent roughness is found to be very high at $k_s = 3.6$ m. This is bigger than the stream is deep and larger than any material in the creek. It indicates that other forms of friction besides skin friction are important if not dominant. From Hey (1979), spill resistance, internal distortion resistance and the additional resistance due to a mobile bed may all be significant in Slesse Creek. The calibration of W is moderately sensitive to errors in k_s values, though Y^* is not.

Particle sizes

Three errors related to particle size distributions were identified as follows:

1. Sediment measurements were restricted by dangerous velocities in some riffle sections. This resulted in an incomplete picture of the sediment distributions and may have underestimated sediment sizes. The particle sizes not measured were mostly boulders remnant from past glacial activity.
2. It was not possible to measure changes in the size distributions. From Schumm (1969) it is known that D_{50} will decrease in low flow periods and vice-versa, but air photos cannot provide this information, and no other historical records were available. The magnitude of the changes is unknown.
3. A pebble count for the braided section of the current channel could not used as it had been made in a deposition area.

Sediment measurement errors mean that the listed value for D_{50} may be incorrect for the current channel and almost certainly incorrect for the historic channel. From Table 4.4, width and depth are insensitive to errors in D_{50} though both are moderately sensitive to errors in $D_{50\,Bank}$. Additional field work to measure bed and bank material sizes in the braided section could improve the accuracy of input values as listed in Table 4.2.

Modeling a Braided Channel

The model cannot account for multiple channels and predictions for the current channel are likely to contain error. The major difficulty of accounting for multiple channels, however, lies in quantifying sediment transport. Slesse Creek was found to be slope controlled and sediment supply limited which reduced the impact and this error did not prevent application of the model.

Judgment Error

The final error relates to the collection of data from field work and air photo analyses. Field work and data collection will always be subject to judgment and measurement error. In particular, bankfull dimensions in the field are difficult to define and the photos used to define historic channel widths often have small scales. The impact of human errors was felt to be small in this case as a hydraulic analysis resulted in an estimate for Q_{bf} very close to the mean annual peak instantaneous discharge from flow records.

4.4.3 Disturbance and Stability

Disturbances will limit the applicability of obtained results to stream restoration because they produce a channel that will often be in a transitory state. Restoration efforts based on a single equilibrium condition for such a stream are likely to fail. Two potential sources of disturbances were identified. Firstly, Q_{bf} fluctuates due to climate changes. Flow records do not extend to 1936 and fluctuations at the time are unknown, but variation in width was noted by Babikaiff and Associates (1997), and it did appear that the channel had been wider some time prior to 1936. These findings indicate that the form of Slesse Creek is not independent of time.

The second potential source of disturbance is variability of sediment supply. Tributaries are subject to debris torrents, and sediment tends to be supplied in waves from the upper catchment. Sediment waves have been found by Roberts and Church (1986) to result in extended periods of instability and recovery in other channels. The frequency of avulsions and the wandering nature of Slesse Creek in air photos may be an indication of sediment waves.

Within Slesse Creek, a number of factors contribute to stability. Firstly, Reach D has a wide floodplain that will help to disperse the energy of peak floods. Secondly, high flows are typically diverted into ephemeral armoured channels. Thirdly, the large boulders and bedrock outcrops that control slope will restrict vertical degradation. The fourth and final factor in the past was large vegetation. Its role was to keep banks and soil in place during large flood events, but the majority of current vegetation is immature and banks are highly unstable. Some sections of the creek were observed to have medium-size second growth vegetation.

4.4.4 Summary

The main sources of error for the analysis were the lack of historical sediment size and roughness data. These parameters were expected to have varied with observed variations in channel dimensions and water discharge, but changes were unmeasurable. This limitation introduces uncertainty with respect to values of D_{50}, $D_{50\,Bank}$ and k_s. Width estimates are moderately sensitive to $D_{50\,Bank}$ and k_s. Peak flow fluctuations and variability in sediment supply were identified as disturbances that may continue to influence the form of Slesse Creek.

4.5 Potential for Restoration

The goal of restoration is recovery acceleration. Long term recovery of Slesse Creek will depend on the establishment and growth of riparian vegetation. Current instability, however, will make it difficult for vegetation to establish itself. The twin aims of short term restoration should thus be to reduce instability in the channel while accelerating vegetation growth. Due to natural variation, it will not be possible to fix the location or size of the stream in the long term and any design is likely to fail. A successful restoration plan will first have delayed this eventuality as long as possible by not trying to be over ambitious and only decreasing width a moderate amount. Secondly, the plan will have prepared for eventual failure by using the delay as a chance to establish vegetation on banks and in the floodplain so that natural features will be able to resume their role as a primary stabilization factor for Slesse Creek in the future.

For these reasons, a moderate goal is recommended. Figure 4.7 a) and b) indicate changes to $D_{50\,Bank}$ and ϕ' that will lead to reduced widths. On each figure, the change necessary to bring the channel down to attain a single thread is indicated. This is suggested as a reasonable goal that can be attained by increasing $D_{50\,Bank}$ to 0.4 m or increasing ϕ' to 60°. It is recommended that a combination of boulder placements and bank stabilization techniques be used due to a lack of a direct connection between bank stabilization techniques and changes to ϕ'. This armouring of the banks will permit a reduction of width from 145 m to 60 m.

Instability in channel form means that bank stabilization will be subject to high risks as the channel may move, and the location of projects should be selected carefully. Ideal locations are downstream of sections where stream location is fixed. A potential location is downstream of the E/F Reach boundary. Lateral movement in this section has been limited and second growth forest has established itself to some degree. Restoration proceeding downstream from that point would decrease the risk of the flow moving behind the bank stabilization works.

Avulsions in Slesse Creek are common and are anticipated to occur in the future. To minimize the damage that will result, restoration efforts should mimic the historical stream by maintaining a secondary channel. It is envisioned that this channel would be used in periods of high flow or in the event a sediment plug blocked the channel mainstem. Its main goal would be to deflect stress away from unprotected banks and floodplain while vegetation is still young and unestablished. To prevent erosion, the overflow channel should be of similar dimensions and restored in the same manner as the main channel.

4.6 Conclusions and Recommendations

Conclusions are:

- W has increased dramatically and the planform has switched from a single to a multi-thread channel. The primary factor was identified as a decrease in ϕ' due to forest harvesting in the floodplain using the rational model of Millar and Quick (1993);

- Q_{bf} has changed due to climatic fluctuations but modeling indicates this factor was of secondary importance;
- additional field work in the reach immediately upstream, consisting of measuring sediment particle sizes and estimating sediment transport capacity, would improve confidence in results;
- flows and sediment supply are subject to waves, indicating that Slesse Creek may be in a transient rather then steady state much of the time; and
- increases in bank stability and bank material sizes were modeled and anticipated to reduce widths and increase depths.

Restoration recommendations are:

- restore Slesse Creek in a downstream direction beginning at the Reach E/F boundary to reduce risk of outflanking;
- encourage a single thread channel to form at W = 60 m with available bank stabilization techniques to increase ϕ' to 60° and boulder placements to increase $D_{50\,Bank}$ to 0.4 m.
- maintain an overflow/avulsion channel, matching restoration efforts in the channel with those in the main part of the stream; and
- accelerate vegetation growth on floodplain and channel banks.

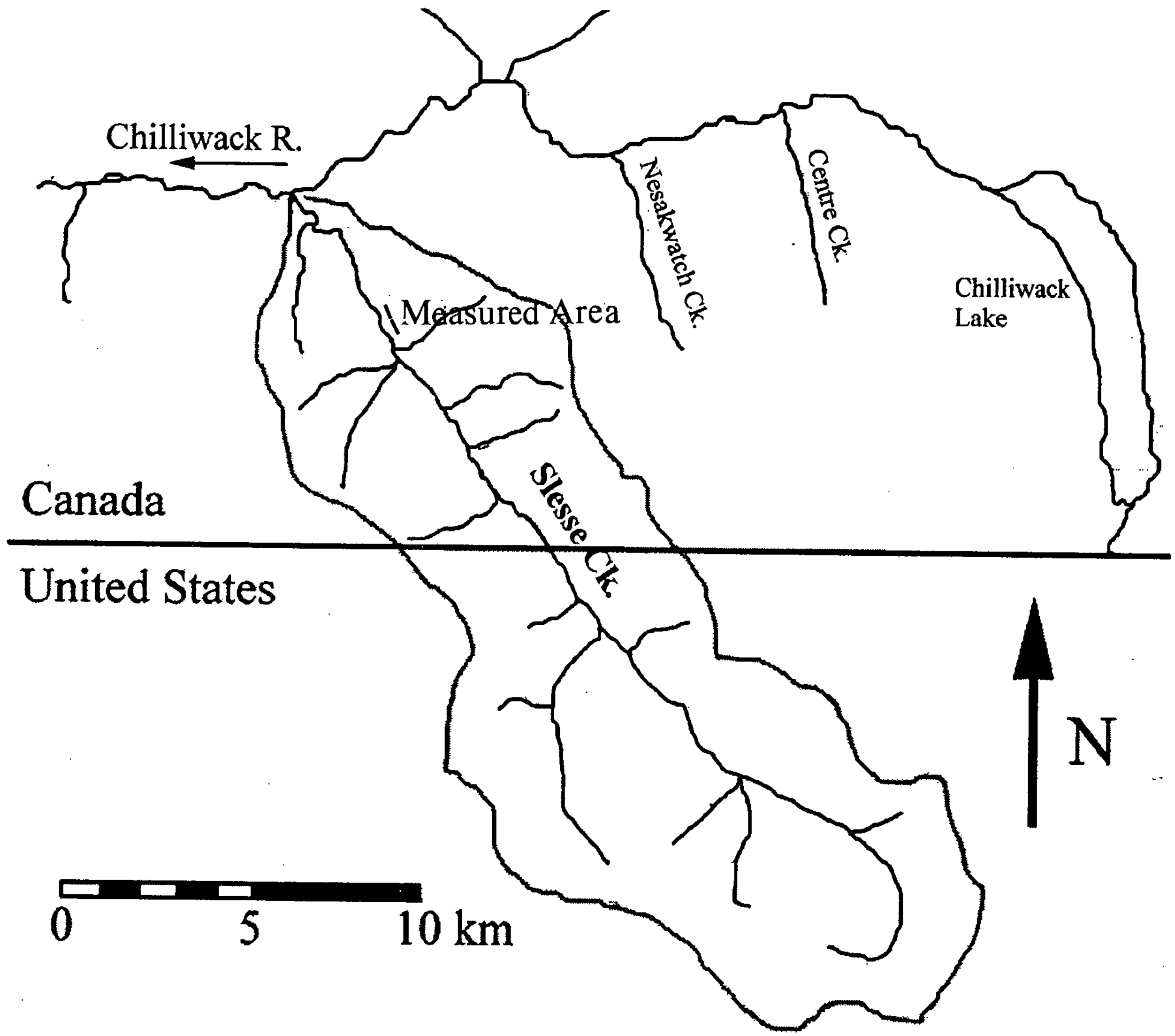
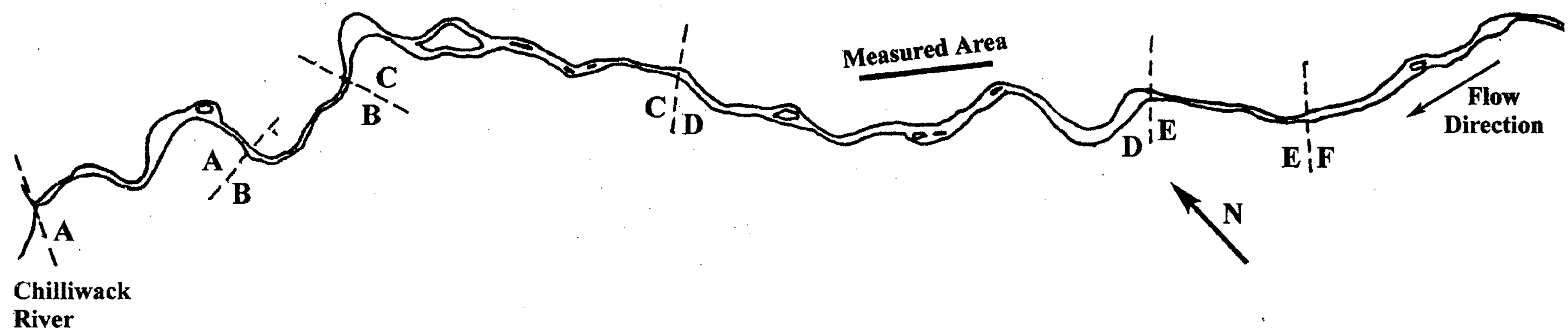


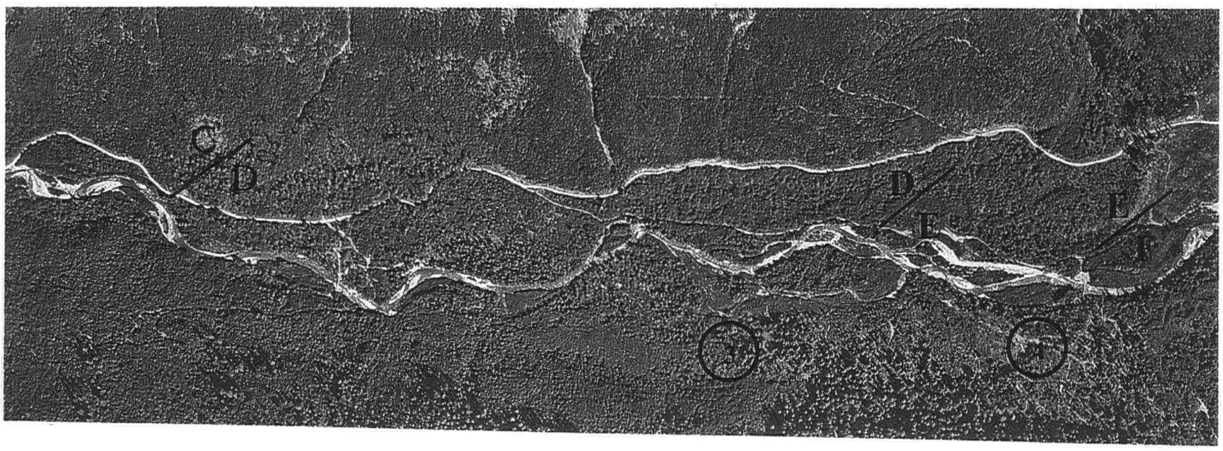
Figure 4.1 - Slesse Creek Watershed



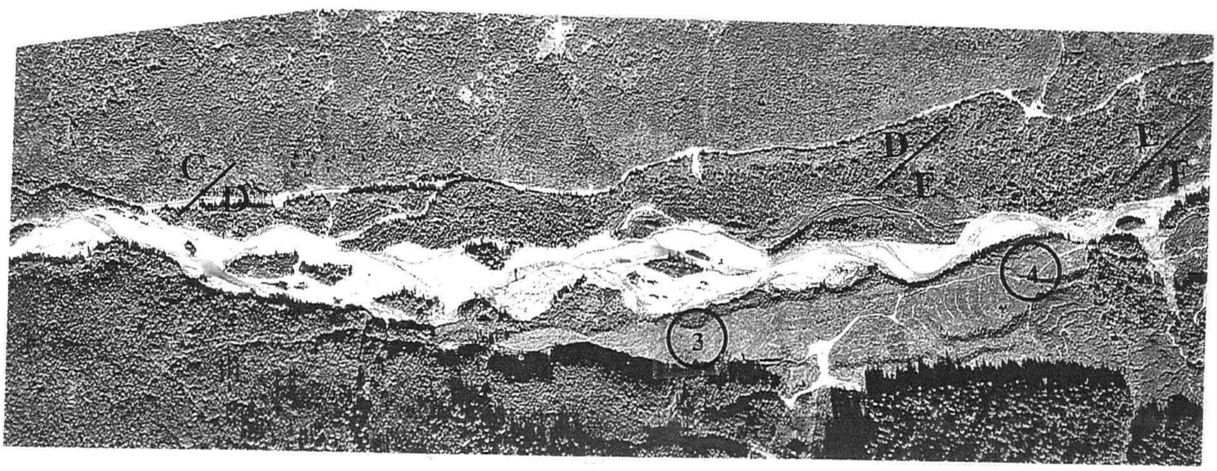
Scale 1:22,200

Figure 4.2 - Slesse Creek, Map of Lower Watershed in 1936

a) 1936 Scale 1:22,200



b) 1973 Scale 1:19,050



c) 1993 Scale 1:17,650

Figure 4.3 Slesse Creek Airphotos

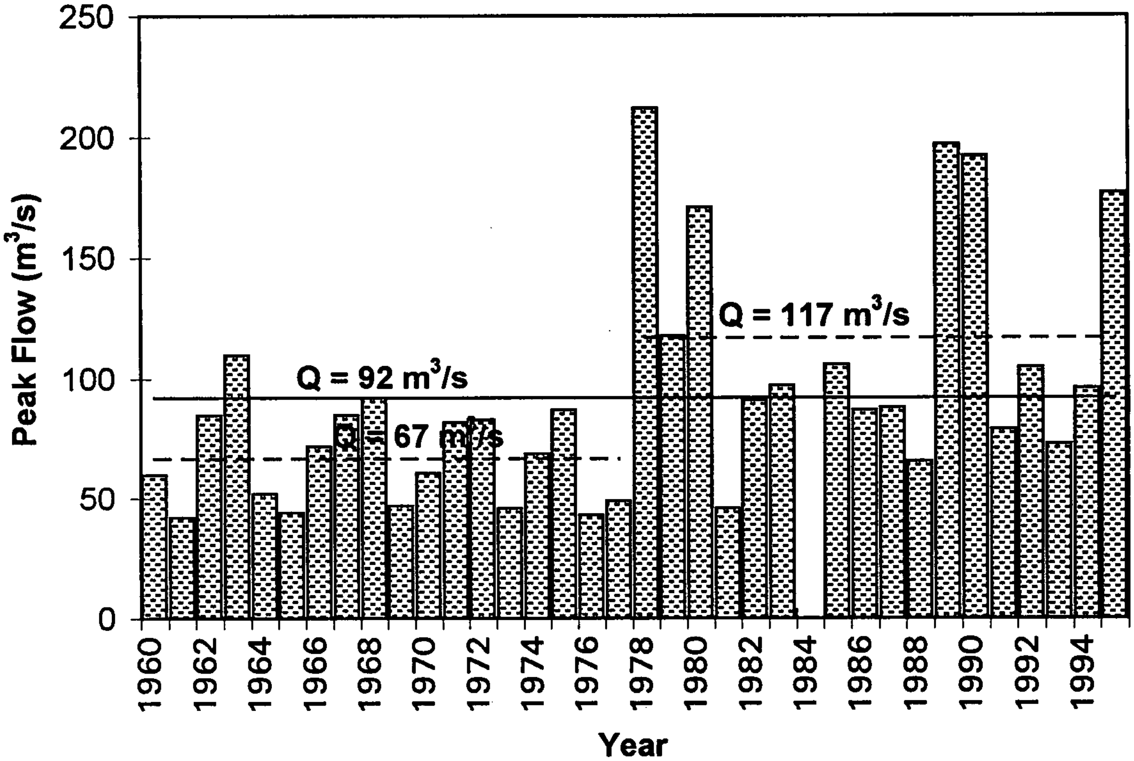


Figure 4.4 - Instantaneous Peak Flow Record, Slesse Creek, Station # 08MH056

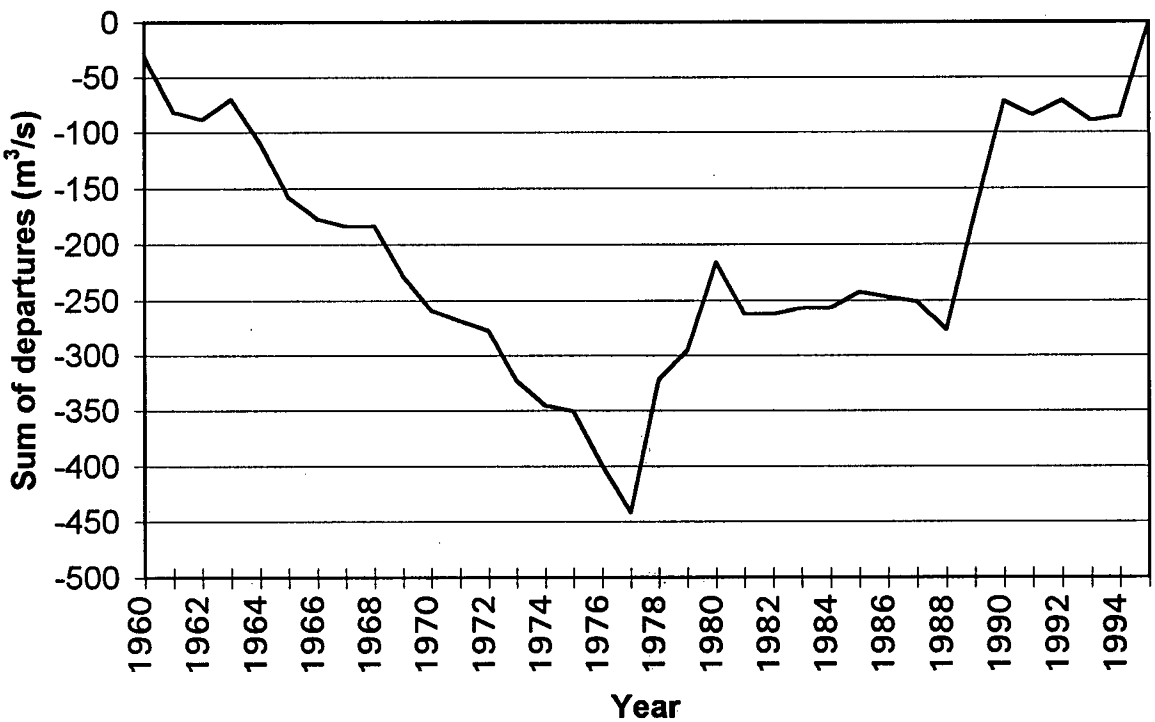


Figure 4.5 - Cumulative Departures from the Mean, Slesse Creek, Station # 08MH056

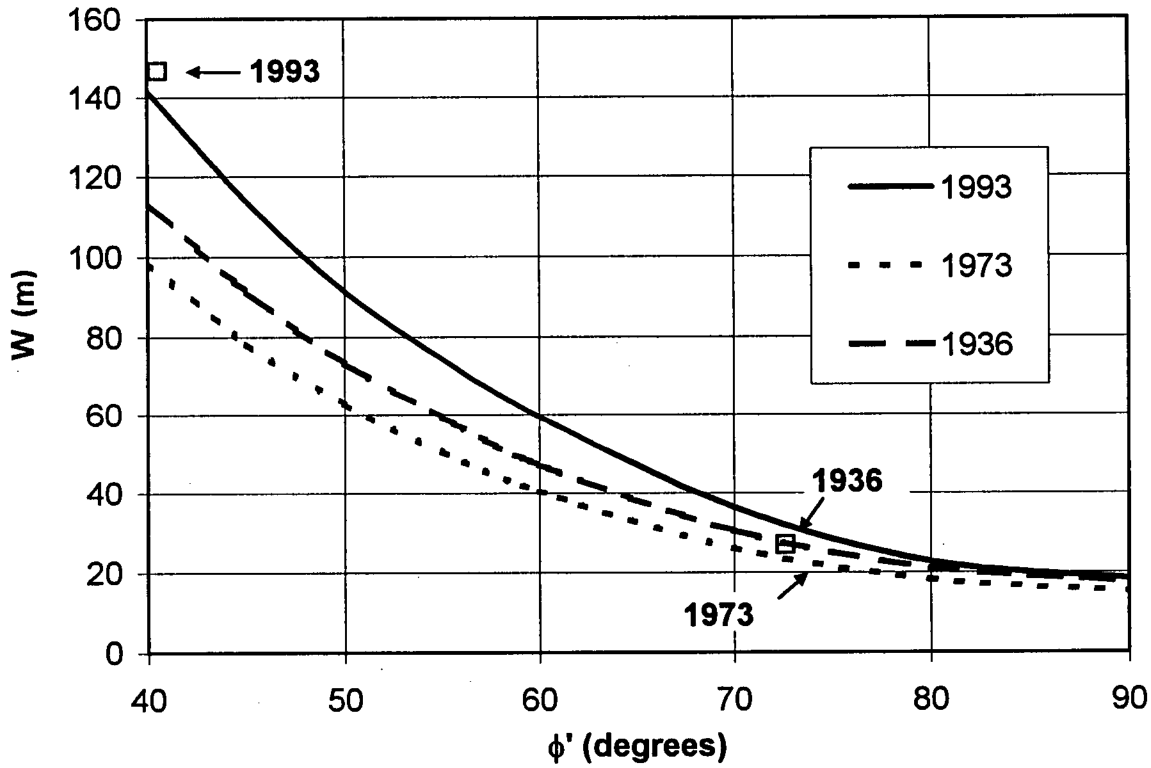


Figure 4.6 - Slesse Creek Calibration, variation of W with φ'

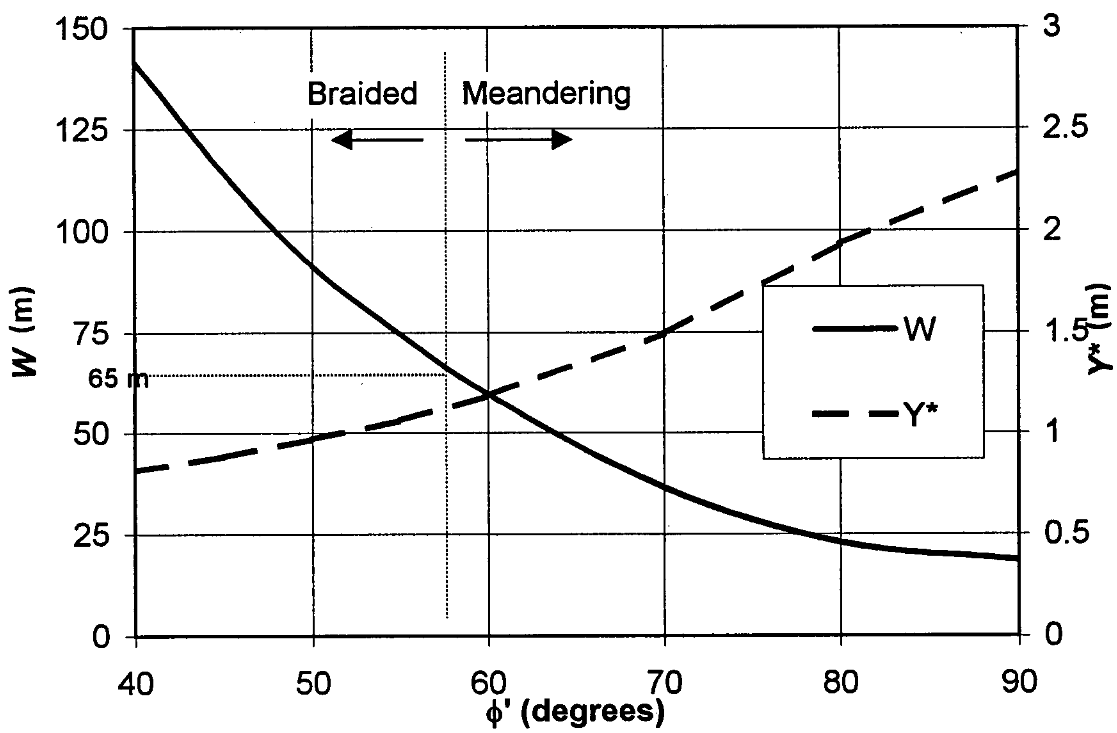


Figure 4.7 a) - Restoration of Slesse Creek, Changing Bank Stability, $D_{50\,Bank}$ = 0.13 m

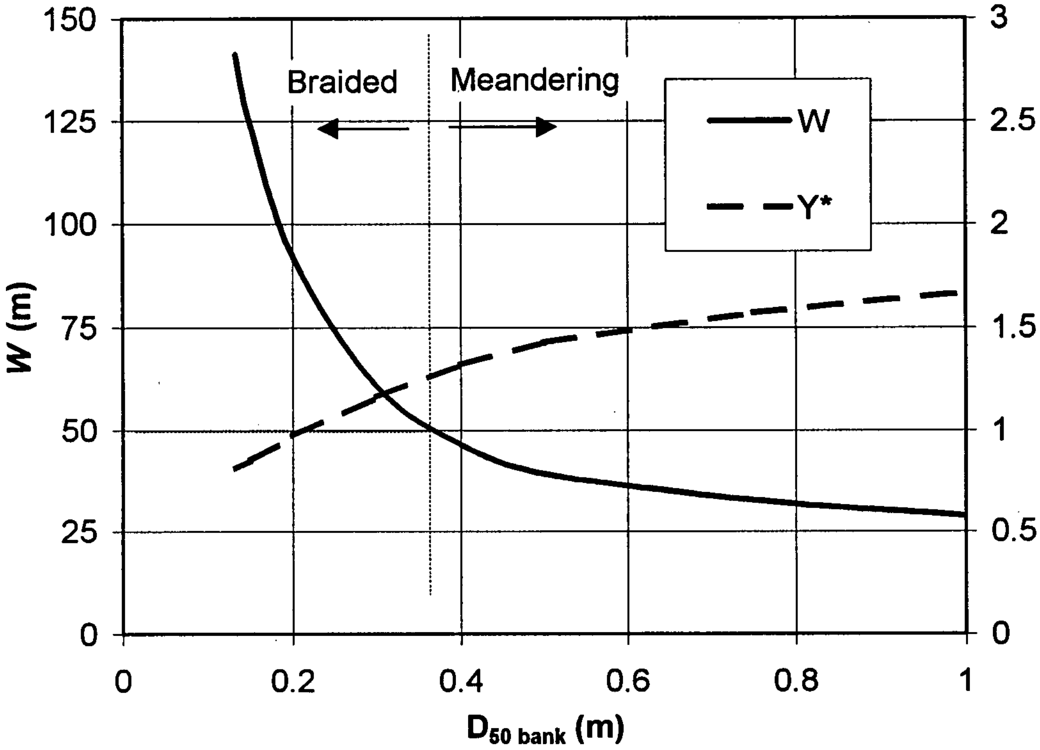


Figure 4.7 b) - Restoration of Slesse Creek, Changing Bank Particle Sizes, $\phi' = 40$

CHAPTER 5

SHOVELNOSE CREEK

5.1 Introduction

Shovelnose Creek is a small stream with a big role as habitat for spawning and rearing steelhead trout in the Squamish River watershed. A combination of good water quality and favourable slopes has made the stream extremely productive until recent years (SSBC, 1996). Disturbance has come both from within the catchment in the form of debris flows and from outside of the catchment as a result of a Squamish River avulsion into the lower reaches of Shovelnose Creek, and the most productive reaches prior to disturbance were left wide, uniform and unproductive. Restoration under the Steelhead Society of British Columbia (SSBC) has begun with funding from the Watershed Restoration Program (WRP). The route of avulsion from the Squamish has been blocked, and in-stream structures have been built in some reaches to create deep pools and overhead cover for fish.

5.1.1 Watershed Description

Shovelnose Creek is a mountain stream draining 25.5 km^2 of the Squamish River catchment. It is located approximately 45 km north of the town of Squamish, British Columbia (see Figure 1.1). It is a third order stream (based on 1:20,000 scale maps), and the southernmost of three creeks draining the Mt. Cayley region, an inactive Quaternary volcano (Figure 5.1). Shovelnose Creek is distinct from Mud and Turbid creeks because it drains granitic instead of volcanic soil. The catchment is very steep, with stream slopes in excess of 50%, and is glaciated in the upper sections. Hillslopes in the area are unstable and the three creeks are major sources of bedload to the Squamish River. Immediately downstream of the creeks, the Squamish switches from a confined single thread to a braided stream morphology.

Reach Division

The stream was divided into three channel links based on differences in channel slope (Figure 5.1). Link 3 encompasses all of the creek upstream of the fan. Slopes are greater than 30% and frequently confined in a bedrock canyon. Link 2 is the section of the creek on the alluvial fan. It has a slope between 5 and 10 % and is approximately 1 km long. It was productive fish habitat but was disturbed by a debris torrent in the 1980's. Link 1 is the section of the creek disturbed by the avulsion from the Squamish River. It lies downstream of the fan on the floodplain of the Squamish River with gradients of less than 1 %. It is usually about 2.5 km long, but the point of confluence with the Squamish changes depending on lateral movements of the creek and the river.

Link 1 was subdivided into A, B, and C reaches progressing in an upstream direction (Figure 5.2). Reach A is the reach just upstream of the confluence in which opposing point deflectors have been constructed. Reach B is the location of restoration efforts in the immediate future. It has a cobble bed with overlying sand. Reach C is closest to the fan and has a gravel bed.

5.1.2 Fish Populations

Shovelnose Creek has historically supported significant fish populations. Prior to the 1980's, escapments of coho salmon averaged 250 adults with peaks of 750, chum salmon averaged 500 with peaks of 3000, and chinook salmon averaged 100 with peaks of 200 (SSBC, 1996). Steelhead populations in a 1979 survey were 400 smolts and 50 adults (Clark, 1988). No recent surveys have been undertaken, but a "dramatic reduction" is fish production has been observed since the disturbances and only small spawning runs of chum and chinook salmon have returned to the creek in recent years (SSBC, 1996).

5.1.3 Restoration

There is active interest in restoring Shovelnose Creek. The SSBC has already completed a number of restoration projects under the WRP. In 1994 a training berm was constructed at the site of the 1984 Squamish avulsion (Figure 5.2). The purpose of this berm was to prevent disturbance of Shovelnose Creek by inundation from the Squamish River at flows lower than a 20 year return period. In 1995 an open surface water intake and a small berm were constructed to provide an extension channel with year round flow (Figure 5.2). In 1996, opposing wing deflectors were constructed in Reach A to create deep pools in the channel. LWD structures were placed in the reach in 1997 both to increase the potential for fish habitat and to encourage deposition and channel narrowing. Meanders and LWD structures were constructed in Reach B in 1998 to narrow the channel, improve fish habitat in the reach (SSBC, 1999). Due to steep slopes, Hay and Company (1995) specifically warned against attempting to restore link 2. This analysis was undertaken prior to the restoration efforts of the Steelhead Society in 1998 and focused on the restoration of Reach B (Figure 5.2).

5.2 Watershed History

In order to model Reach B of Shovelnose Creek, it was necessary to understand both the current condition and the changes that have occurred. In this section the changes to stream morphology since 1964 are reviewed from available air photos. 1964, 1974, and 1994 air photos are included in Figure 5.3. Scales of the photos have not been altered. The division between links 1 and 2 is shown on each of the photos for orientation. The stability of hillslopes in the region, the hydrology, and forest harvesting will also be reviewed to assess possible causes of the disturbances.

5.2.1 Stream Morphology

In 1964 and up to 1974, link 1 of Shovelnose Creek was a sinuous channel with occasional islands, a pool and riffle flow sequence, and point bars. It appeared to be laterally stable. Upstream in Link 2 on the alluvial fan, flow was split into two channels. One channel was wide and braided while the other was narrower and more stable up to 1974 air photos. By 1994 the morphology of the two channels had reversed. The former braided channel was narrower with very low flows while the former narrow channel was braided and considerably wider. In 1994 Link 1 was considerably wider, straighter and more uniform than in earlier photos. The low flow channel is visible as a dark area between extensive bars that line both sides of the channel. Occasional riffles are visible as areas of faster flow.

Reach-averaged channel dimensions of Shovelnose Creek are shown in Table 5.1. Listed values were obtained from 1974 and 1994 air photos and a field survey in October 1997. Measurements were not taken from the 1964 photos as the scale was small and there were no major changes in channel morphology between 1964 and 1974. Channel banks and bed material from the period of inundation by the Squamish River were visible and surveyed. The slope measurement during inundation is the overall slope of link 1, as surveyed for SSBC (Dave Duff, personal communication). A conceptual diagram of the slopes is shown in Figure 5.4. The 1974 slope was calculated from the slope of link 1 using the relative sinuosities measured from air photos. Survey data and calculations are included in Appendix B.

From the air photo analysis, the influence of the Squamish River on the geomorphology of Slesse Creek was noted. In 1964 photos, a road ran along the north side of the Squamish River. The river had multiple channels downstream of its confluence with Shovelnose Creek, but islands were predominantly vegetated and there was little exposed bar surface. In 1974 the Squamish River upstream of the confluence had moved and extensive bars were now visible. With the larger scale of the 1974 photos, past avulsion tracks were visible and indicated the recurrence of flows from the Squamish River to the lower reaches of Shovelnose Creek. By 1994 the Squamish River upstream of the confluence had eroded further towards Shovelnose Creek, removing the old logging road in the process, and a channel between the Squamish River and Shovelnose Creek was visible. The activity of the Squamish River also changed the location of its confluence with Shovelnose Creek, moving it further downstream. Few traces of vegetated islands within the Squamish River remained.

Table 5.1 - Channel Geometry of Shovelnose Creek

Year	1974	During Inundation (DI)	1997	
Reach	A-C	B-C	B	C
Source	Air Photos	Survey	Survey	Survey
W (m)	15	53	36	30
ξ	1.14	1.06	1.06	1.06
S	0.0074	0.008	0.0031	0.0043
Y^* (m)	-	2.5	0.87	0.87
Bed Material	-	Cobbles	Sand	Gravel

5.2.2 Hillslope Stability

The Mt. Cayley region has a history of natural instability. The two creeks north of Shovelnose, aptly named Mud and Turbid Creeks (Figure 5.1), have very poor water quality due to high amounts of suspended material in addition to very high bed material loads. These creeks drain Mt. Cayley, an extinct volcano, whereas Shovelnose Creek drains granitic bedrock and water quality has been good. Brooks and Hickin (1991) studied the stability of the region by looking at terraces. They concluded the Squamish River has been dammed at least 3 times in the last 5000 years due to major debris landslides from Mt. Cayley. The most recent dam left a prominent terrace at the 200 m elevation level. Additionally, an unknown number of smaller events have led to partial blockages of the Squamish River, the most recent of which occurred in 1984. This event temporarily dammed the Squamish River upstream of Shovelnose Creek and the subsequent flood, created when the dam was overtopped and gave way, caused the avulsion from the Squamish River into Shovelnose Creek.

Within the Shovelnose watershed, naturally occurring landslides and debris torrents have been documented. Baumann (1994) completed a preliminary terrain and hydrologic assessment of the Shovelnose Creek watershed and concluded that slopes and tributaries on the south side of the creek are typical of systems subject to periodic debris flows. Hay and Company (1995) looked at landsliding in the creek and found many natural landslides and a prevalence of steep slopes with loose material above the treeline. J.M. Ryder and Associates (1994) mapped terrain and slope stability and found that 68% of the terrain below the tree line is either potentially unstable (class IV) or actively unstable (class V). SSBC (1996) concluded that the avulsion and erosion on the alluvial fan, possibly taking place in 1991, was the result of a naturally-sourced debris torrent.

5.2.3 Hydrology

Shovelnose Creek is not gauged. Nearby flow records from the Squamish and the Elaho Rivers (Station No. 08GA022 and 08GA071) were used by Hay and Company (1995) to establish flood magnitudes. The mean annual peak instantaneous discharge in Shovelnose Creek was estimated at 45 m^3/s, and the 200 year flood was estimated at 130 m^3/s. Baumann (1994) found that flows of 150 m^3/s may occur in the watershed during major storms. The two highest flows on record in the Squamish River occurred in 1984 and 1991 (shown in Figure 5.5). These years are the same as those earlier identified as the likely years for the two main disturbances to Shovelnose Creek.

Hydrologic trends were also assessed for the Squamish River. The overall mean peak annual instantaneous discharge in the Squamish River was found to be 1410 m^3/s. Using a plot of cumulative departures from the mean (Figure 5.6), two different means were identified representing a shift of 22% in peak discharges. The five highest flows on record have occurred between 1980 and 1991. The reason for the increase in peak events is not known. Due to similarities with trends in other streams such as Slesse Creek, the most likely cause was climatic variability. The increase in peak events does indicate that increased instability in Shovelnose Creek and the Squamish River may be associated with hydrologic trends.

5.2.4 Forest Harvesting

Forest harvesting in Shovelnose Creek watershed has consisted of road building and clear-cut logging. Trees on the floodplain of the Squamish River in 1964 air photos appear to be small and uniform, suggesting post-logging second growth (Figure 5.3). The alluvial fan of Shovelnose Creek was nearly bare at this point and indicated recent logging. No buffer strips are visible in the clear-cut areas. There was no additional logging visible in 1974 photos. A fresh cut area above the alluvial fan on the north side of the creek is visible in 1994 photos.

According to two reports, logging has not significantly altered the sediment or hydrologic regimes of the watershed. Hay and Company (1995) measured the total logged area to be 1.67 km^2. This represents 7.4% of the basin and 14.9% of the area below the treeline. Only one logging related landslide was found, although unstable areas were identified in the most recent clear-cut area. Slides are expected in the future as root strengths deteriorate. Baumann (1994) found logging had been done in a "proficient manner", although it likely aggravated bank erosion in Links 1 & 2.

5.2.5 Summary

The history and stream morphology of Shovelnose Creek was investigated. It was found that observed changes in the stream morphology were the result of two major disturbances. Firstly, an avulsion from the Squamish River took place in 1984 that inundated the lower reaches of

Shovelnose Creek. This event has been tied to high flows and hillslope instability in the Mt. Cayley region that led to a temporary partial damming of the Squamish River. Downstream flooding resulted when the dam overtopped and gave way. Forest harvesting on the Squamish River floodplain may have aggravated the impacts of this flood event.

The second disturbance was an avulsion in the upper catchment of Shovelnose Creek, likely in 1991, which destabilized a productive secondary channel on the alluvial fan. This event has been tied to high flows and a debris flow within the Shovelnose Creek watershed. Impacts were likely aggravated by logging to the stream banks on the alluvial fan of Shovelnose Creek.

There have been three major impacts to the lower reaches of Shovelnose Creek. Firstly, the channel is now much larger than required to transport current flows. Secondly, slopes of the lower reaches have been changed and are controlled by accumulations of large boulders. Thirdly, bed and bank sediments have been coarsened by the selective removal of fine sediments during the period of high flows.

5.3 Analysis

The application of the model of Millar and Quick (1993) was done in three steps. Firstly, model input values were quantified and the model was calibrated to the existing and past geometries. Secondly, the past stream behaviour was interpreted based on the calibration. Thirdly, restoration options were modeled by varying input parameters over feasible ranges.

5.3.1 Model Inputs

The fixed-slope version of the Millar and Quick (1993) model was used due the vertical and lateral control currently exerted on the channel via the large bed and bank material remnant from the period of inundation by the Squamish River. Required input variables were the bankfull discharge (Q_{bf}), median bed and bank particle sizes (D_{50}, $D_{50\,Bank}$), equivalent roughness (k_s), channel slope (S), and bank stability (ϕ').

Bankfull Discharge (Q_{bf})

The bankfull or dominant discharge was assumed equal to the mean annual peak instantaneous discharge. Based on Hay & Co.'s (1995) analysis for Shovelnose Creek, $Q_{bf\,1997} = 45$ m^3/s. It was not possible to assume that trends observed in the Squamish River flow record would be reflected in Shovelnose Creek and it was assumed $Q_{bf\,1974} = Q_{bf\,1997}$.

Bankfull flow values during the period of inundation by the Squamish River were not known. To obtain an estimate, the median bed particle size was used in equation 3.5 to estimate roughness. Roughness and measured relic bankfull channel dimensions were then used as inputs to the Keulegan (equation 3.4) and Darcy Weisbach equations (equation 3.3). This procedure found $k_{s\,DI} = 0.78$ m and $Q_{bf\,DI} = 540$ m^3/s, or approximately 12 times the mean annual flood.

Sediment Sizes (D_x)

Reach B currently has a sand bed overlying cobbles. The sand had recently been transported, but cobbles had extensive algae growth and were not mobile. Upstream in Reach C, gravel bed material was recently transported. A gradient of particle sizes was observed, gradually coarsening in an upstream direction. From this, a single value was required as a sediment size estimate. Measured from repeating bar forms, bed slope was near constant in Reach C except in the region nearest to the fan. Pebble counts were done in the middle of this constant slope section and found $D_{50\,1997} = 0.044$ m. Bank material was visually similar and assumed to be forming from the same material. Without historical information, this material size was also assumed applicable to the 1974 channel. For the channel during inundation, pebble counts were done in Reach B of the immobile material remnant from the period of Squamish flooding, giving $D_{50\,DI} = 0.115$ m. Pebble counts of the channel banks found $D_{50\,Bank\,DI} = 0.265$.

Flow Resistance (k_s)

Shovelnose Creek has a mild slope ($S = 0.3$ - 0.8 %) and equation 3.5 calculated $k_{s\,1997} = 0.31$. This was close to estimates of k_s from equations 3.3 and 3.4 for Reaches A and C and was used as the input value. Estimates of k_s from Reach B were not used due to poor bankfull markers.

Slope (S)

Channel slopes were established from field surveys as described earlier and listed in Table 5.1.

Bank Stability (ϕ') and Calibration

ϕ' was obtained by calibrating the model of Millar and Quick (1993). A plot of ϕ' versus W is shown in Figure 5.7. Three curves are shown, each modeled with the independent variables of the years indicated in the legend. $\phi'_{1974} = 73°$, $\phi'_{DI} = 46°$, and $\phi'_{1997} = 44°$ based on an agreement between modeled and measured widths.

Summary of Input Values

Values of input variables used in the rational model are shown in Table 5.2. As they are of interest for restoration, calculated sediment transport capacities are also shown.

5.3.2 Interpretation of Stream Behaviour

The following interpretations of stream behaviour were made based on model calibration. Prior to 1984 Shovelnose Creek had high bank stability ($\phi' = 73°$) and was able to maintain a narrow channel suitable for fish spawning and rearing. In 1984, dominant discharges in the channel increased from about 45 m^3/s to about 540 m^3/s due to an avulsion from the Squamish River.

This flow enlarged the channel, increased sediment transport capacity and coarsened the bed and bank material. Changes appear to have been aggravated by the removal of riparian vegetation as reflected in a decrease of ϕ' from 73° to 45°.

Since 1994, flows from the Squamish River has been blocked. Flows are now close to pre-disturbance values but historical bed and bank material has been removed. Sediment from the Shovelnose Creek catchment appears to be returning and depositing within the channel boundary. The historical sediment transport rate is much higher than the current capacity of Reach C which is turn higher than the current capacity of Reach B. The channel is vertically controlled by coarse lag material at the riffles and laterally controlled by terraces. Both riffles and terraces are remnant from the Squamish floods.

Table 5.2 - Input Variables for Shovelnose Creek

Year	1974	During Inundation (DI)	1997
Q_{bf} (m^3/s)	45	540	45
D_{50} (m)	0.044	0.115	0.044
$D_{50\ Bank}$ (m)	0.044	0.265	0.044
k_s (m)	0.31	0.78	0.31
S	0.0074	0.008	0.0043
ϕ' (°) calibrated	73	46	44
G_b (kg/s) calculated	63	760	3

5.3.3 Restoration Modeling

The objectives of restoration in Reach B were to narrow and deepen the channel while reducing sand deposition in the thalweg. The fixed-slope version of Millar and Quick (1993) was used to model the impact that altering ϕ' and $D_{50\,Bank}$ has on the equilibrium channel geometry and G_b. Other parameters were held constant while ϕ' and $D_{50\,Bank}$ were varied through feasible ranges.

An input value for the channel slope was required, but the controlled slope varied between the reaches (Figure 5.4). It was assumed that the slope of Reach B would be adjusted to $S = 0.0043$, the current slope of Reach C, in order to prevent aggradation. Modeling results for W, Y^*, and G_b are shown in Figure 5.8. Both an increase in ϕ' and $D_{50\,Bank}$ were found to deepen and narrow the channel while increasing the sediment transport capacity of the channel. Width approached a minimum value of 13 m where $D_{50\,Bank} \geq 0.20$ m. The corresponding $Y^* = 1.5$ m and $G_b = 13$ kg/s. An increase in ϕ' from 45° to 65 ° produced similar results.

5.3.5 Summary

Model inputs were quantified and calibrated. Dominant discharge during the period of inundation by the Squamish River increased flows more than ten times over the current mean annual flood in Shovelnose Creek. Changes to the sediment transporting capacity have resulted from changes to the dominant discharge, but the current capacity has been left much less than historical values due to the vertical and lateral control of the current slope. Bank stability has been decreased concurrent with the increase in flows, most likely due to the prior harvest of riparian vegetation.

Increases in ϕ' and $D_{50\,Bank}$ were modeled to lead to the desired restoration goals of channel narrowing and deepening and increased sediment transport capacity. It was assumed for the modeling that the slope of Reach B would be adjusted to match that of Reach C to prevent continued aggradation. An increase of $D_{50\,Bank}$ to 0.20 m or ϕ' to 65° was predicted to narrow the channel from $W = 30$ m to 13 m, deepen the channel from $Y^* = 0.9$ m to 1.5 m and increase sediment transporting capacity from $G_b = 3$ kg/s to 13 kg/s.

5.4 Limitations of Analysis

In order to assess the impact of assumptions and errors in the analysis, three steps were taken. First, a sensitivity analysis was done. Following that, errors and assumptions in the measurement and calculation procedures were assessed and compared with parameter sensitivity. Finally, disturbance and channel stability were assessed.

5.4.1 Sensitivity

The sensitivity of the 1974 calibration to 10 and 25% errors in input variables was calculated. Graphs of the sensitivity of the estimate of the 1974 channel geometry are included in Appendix B. Results are summarized in Table 5.3.

Table 5.3 - Sensitivity of Modeling for Shovelnose Creek

Dimension	Insensitive to	Moderately Sensitive to	Highly Sensitive to
W	S, k_s, D_{50}	Q_{bf}, $D_{50\,Bank}$	ϕ'
Y^*	Q_{bf}, $D_{50\,Bank}$, k_s, D_{50}	S	ϕ'
G_b	k_s, $D_{50\,Bank}$	-	Q_{bf}, S, ϕ', D_{50}

5.4.2 Sources of Error

Potential sources of error were the assumption of a single particle size and the application of this particle size to model the 1974 channel, the calculation of Q_{bf} during inundation by the Squamish River, and incomplete field work.

Particle Sizes

At the junction between Shovelnose Creek and the Squamish River floodplain there is an alluvial fan. The influence of this alluvial fan is to create a gradient of particle sizes in Reach C. It was assumed that this gradient of particle sizes can be considered constant in time. Between Reach B and Reach C there is also a difference in particle sizes. Aggradation and a continuous change in particle sizes will occur in Reach B as sediment moves downstream. It was assumed that this process cannot be considered constant in time and restoration in Reach B was modeled based on the sediment in Reach C. The accuracy of modeling was dependant on these two assumptions. Changes of bed particle sizes with time should be monitored to verify the assumptions.

A third assumption relating to particle sizes was that current particle sizes will be representative of particle sizes in 1974. The slope of the 1974 channel, however, was much greater than the current slope and this third assumption not likely to be correct. No other value, however, was available. For calibration, however, the lack of sensitivity of width and depth to D_{50}. means that this limitation was not critical.

Calculation of Q_{bf} During Inundation

The calculation of bankfull discharge during inundation (Q_{bfDI}) by the Squamish river was not expected to be accurate as it was calculated only from a hydraulic analysis and not confirmed from gauge records. Good measures of width, depth and sediment size were available and ϕ' could be reasonably calculated. The value of Q_{bfDI} was not critical for calibration or modeling.

Other Sources of Error

Some field work is seen to be incomplete. Firstly, only two cross-sections were surveyed in Reach B. Cross-sections in this reach were very uniform, however, and this reach was not used for a hydraulic analysis or to calibrate the model. Secondly, particle size distributions in Reach C were done with an inadequate sample size. Confidence would be increased by ensuring statistical validity and measurements could be completed as part of monitoring sediment changes.

5.4.4 Disturbance and Stability

Three sources of disturbances were identified. They are rapid aggradation rates, natural instability within the catchment, and reccurrence of disturbance from the Squamish River. Rapid aggradation rates between the alluvial fan and Reach C will limit the applicability of an equilibrium analysis because it will result in changing values of D_{50}, k_s, and G_b in the short term. This problem is part of the error in determining a value for D_{50} as discussed earlier.

The second potential sources of disturbances are debris flows and landslides. These events are part of the natural regime and the alluvial fan of Shovelnose Creek was unstable even prior to logging. Downstream of the fan the channel appeared stable in 1964 and 1974 air photos, indicating that the effect of the extreme events may have been moderated or eliminated by the fan.

The third source of disturbance is the Squamish River. It is not known whether the event of 1984 is expected to occur again in the near future. Meander extension, however, has moved the Squamish closer to Shovelnose Creek, and the constructed berm has only been designed for flows up to a 20 year return period.

The lower reaches of Shovelnose Creek are stabilized by a number of factors. Firstly, steep headwater streams and neighbouring cliffs contribute large boulders that make up an important part of the creek's bed and bank material. Secondly, previous avulsions from the Squamish River may have stabilized the form of the stream by creating resistant terraces. Vegetation has also been a stabilizing factor, though its influence has decreased due to past logging. Finally, the Squamish River floodplain may have dissipated the energy of peak flows from the upper catchment of Shovelnose Creek.

5.4.5 Summary of Limitations

The main source of error in Shovelnose Creek was aggradation resulting in changing values of D_{50}, k_s, and G_b with time. Further monitoring is necessary to verify assumption that aggradation of the fan can be ignored in the short term and that restoration efforts in Reach B can be designed based on characteristics of Reach C. Calibration was not limited by the problems, however, and restoration was modeled with confidence due to a lack of sensitivity of W to k_s, D_{50}, and S. Restoration will be limited by disturbance. Future avulsions from the Squamish River are likely. Large floods from within the Shovelnose Creek watershed are also expected.

5.5 Potential for Restoration

The rate of natural disturbance in Shovelnose Creek is high. Natural landslides and debris torrents are common and are likely to reoccur in the future. The most productive reaches of the creek are located on the Squamish River floodplain and subject to avulsions from the larger river during high flows. Further lateral movement may eliminate Reaches A - C of Shovelnose Creek. These factors should limit restoration in Shovelnose Creek to short-term efforts.

If restoration proceeds, modeling results indicate that Reach B is regrading in response to oversupply from Reach C. Any restoration attempts are likely to be buried unless the slope in Reach B is adjusted to match the upstream reach. Currently, large boulder riffles are controlling slopes. By decreasing the height of the riffle downstream of Reach B, a slope equal to that of Reach C can be imposed.

The rational model was used to show the effects of changes to ϕ' and $D_{50\,Bank}$. Increases in ϕ' and $D_{50\,Bank}$ were found to reduce W, increase Y^*, and increase G_b. Of the two options, changes to $D_{50\,Bank}$ have the advantages of readily available material and ease of implementation. The main disadvantage is that armouring the bank will not provide the variety of habitat, food sources, and local hydraulics important for the biological part of the stream ecosystems. Changes to ϕ' through the re-establishment of riparian vegetation are envisioned to result in good biological habitat, but they have the disadvantage that they are hard to implement and are at risk while vegetation is small.

Point deflector design as described in Allan and Lowe (1997) seems particularly suited to implementation in Shovelnose Creek, and this technique has already been applied in Reach A. Point deflectors are wedges made out of boulders extending from channel banks into the stream. They serve to concentrate flow and create deep pool areas. They can be built low profile so that large flows pass over them. Also, by concentrating the flow, they maintain a variety of hydraulic habitats and encourage deposition and bank development behind the wedges where flow is slower. The presence of the terraces limit lateral movement and the risk of outflanking.

Margins of safety are recommended to reduce forces on banks. From Figure 5.7 a) and b), a reduction of W from 30 to 15 m using point deflectors with a $D_{50\,Bank} \geq 0.20$ m is recommended. Acceleration of vegetation growth in deposition areas is also recommended to encourage deposition and bank development.

5.6 Conclusions and Recommendations

Conclusions are:

- Lower reaches of Shovelnose Creek have been disturbed by a period of extreme floods. The frequency of these large floods has been decreased by the construction of a training berm which divert flows up to a 20 year return period;
- A decrease in bank stability (ϕ'), most likely due to forest harvesting in the floodplain has also disturbed the channel;

- Impacts are breaks in slope have been introduced due to accumulations of large boulders, the current channel is much larger than required for current flows, and bed and bank materials have been coarsened;
- The model of Millar and Quick (1993) was calibrated to the observed changes;
- Aggradation is resulting in changing values of D_{50}, k_s, and G_b with time, decreasing confidence in results;
- Flows and sediment supply are subject to extreme peaks, indicating that Slesse Creek may be in a transitory state much of the time; and
- Increases in bank stability and bank material sizes were modeled to meet restoration goals of reduced width, increased depth, and increased sediment transporting capacity.

Restoration recommendations are:

- Monitor particle sizes and sediment transporting capacity in Reaches B and C;
- Concentrate on short term restoration efforts due to the likely recurrence of disturbance from the Squamish River;
- Regrade Reach B by altering the elevation of the downstream remnant riffle;
- Restore Reach B using a point deflector technique as applied in Reach A with $W \geq 15$ m and $D_{50\,Bank} \geq 0.20$ m; and
- Accelerate growth of vegetation in deposition areas.

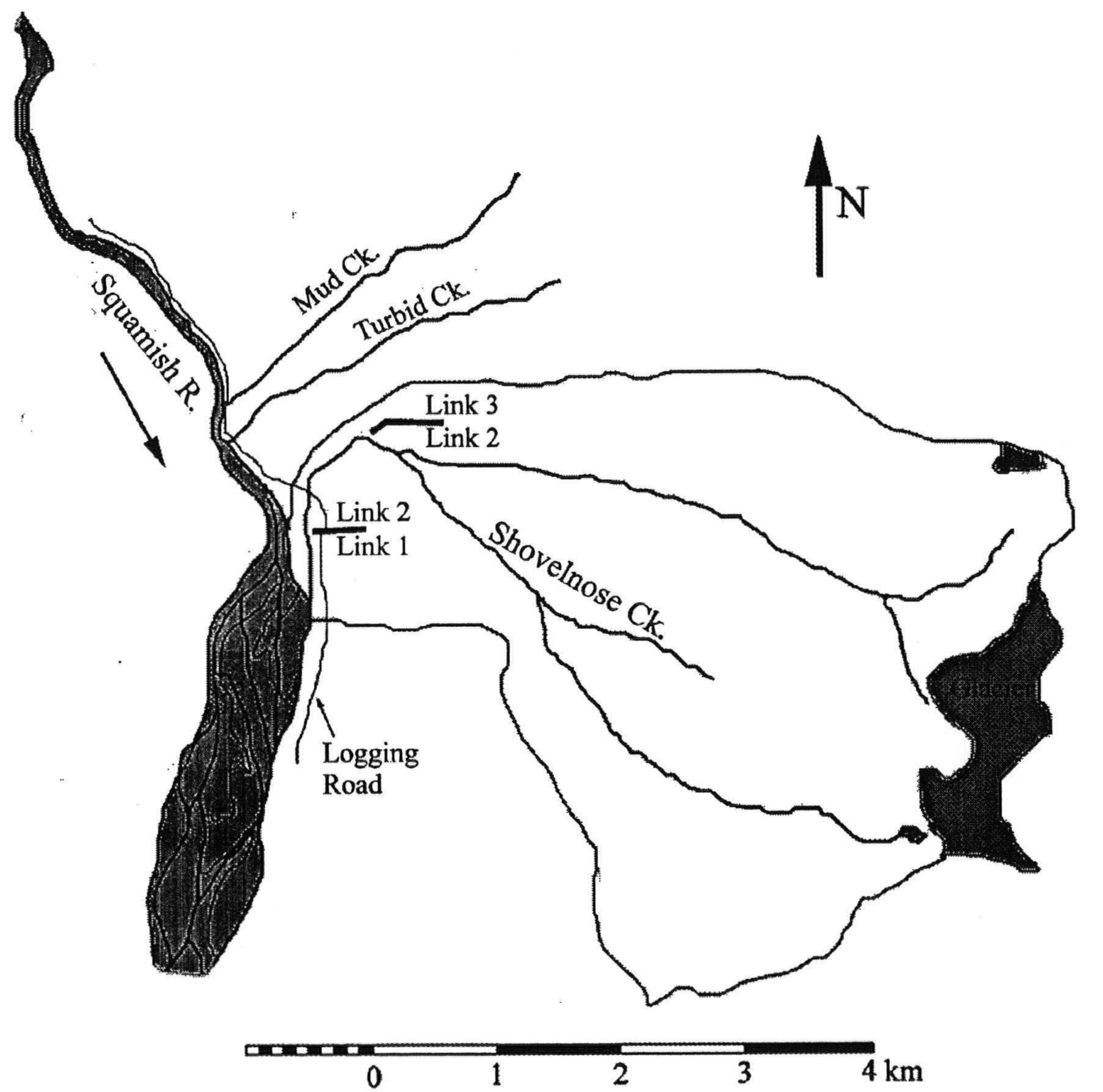


Figure 5.1 - Shovelnose Creek Watershed

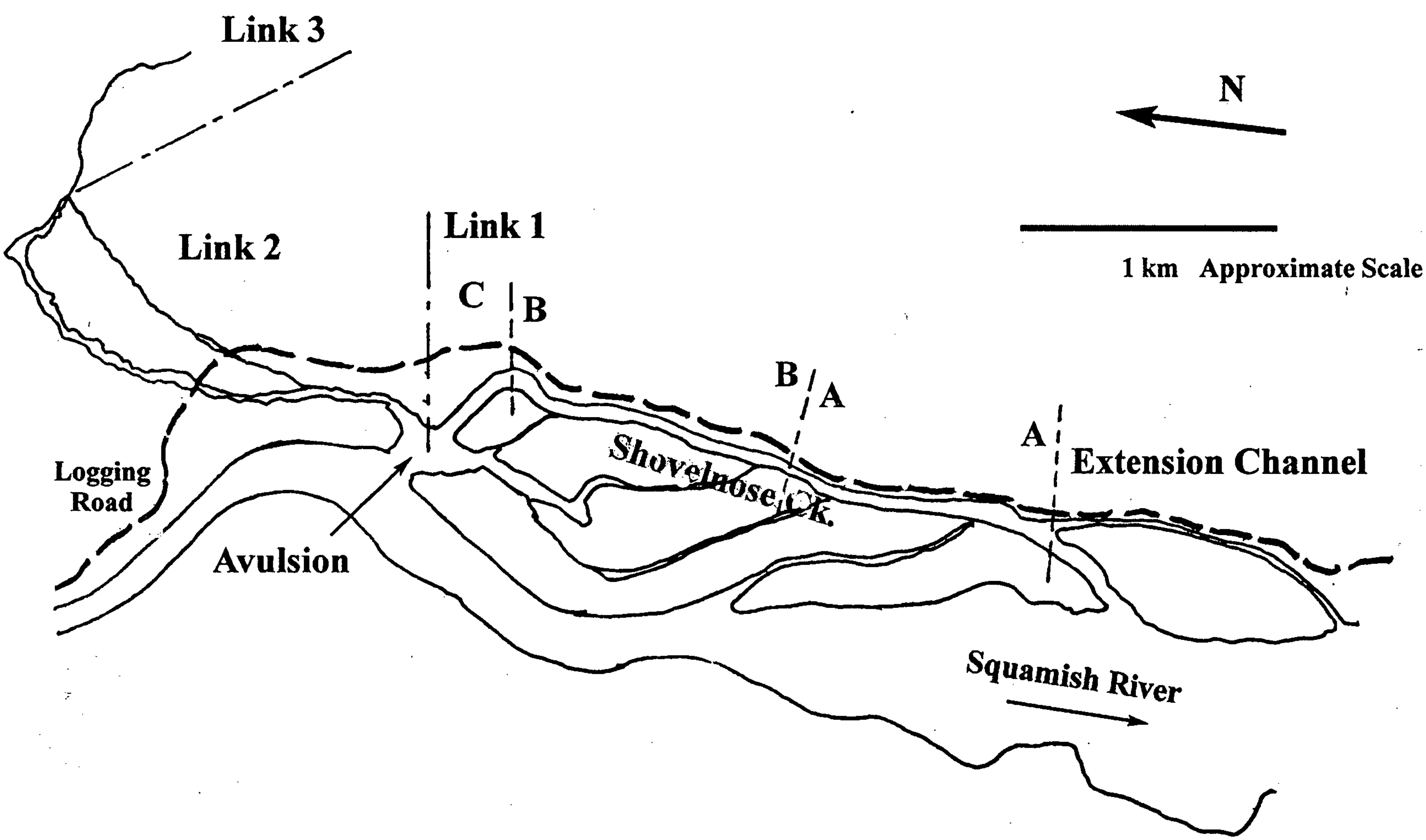
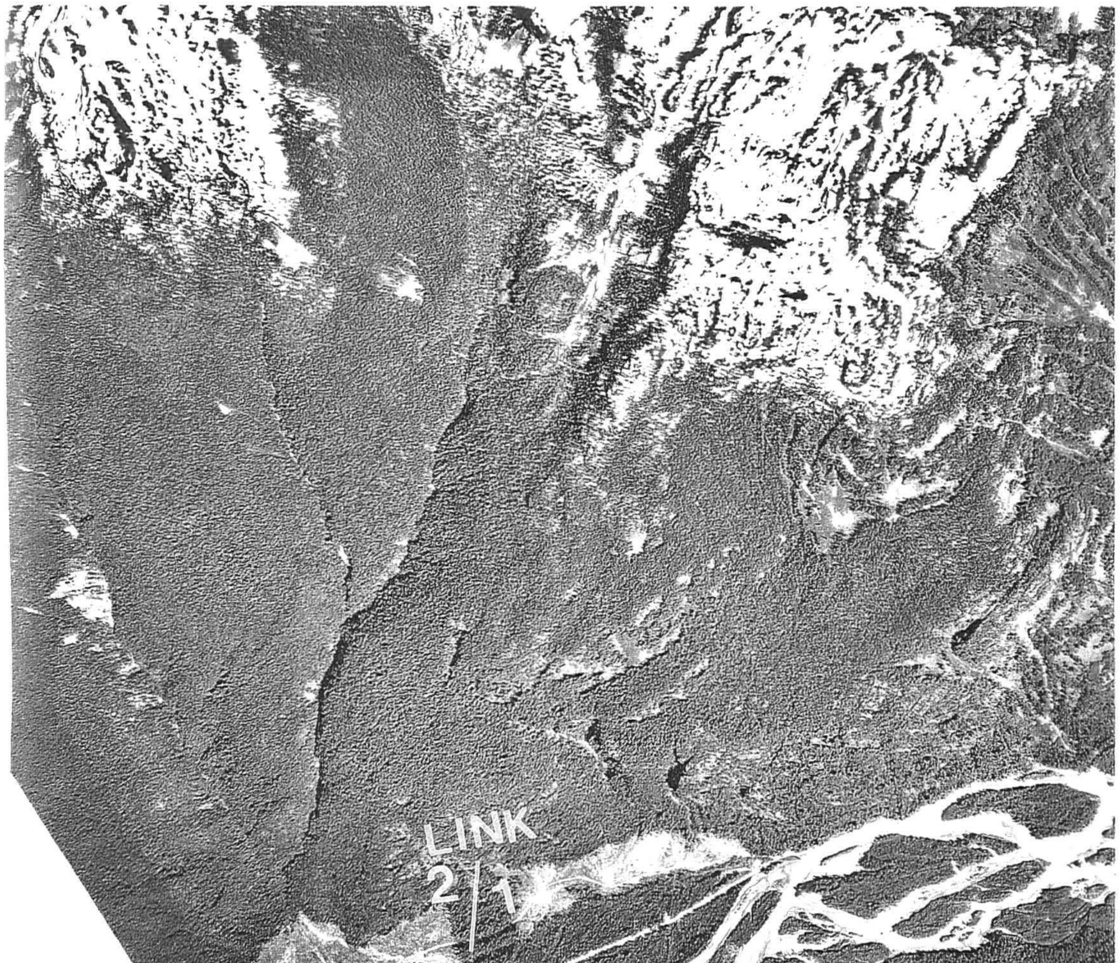


Figure 5.2 - Shovelnose Creek, Map of Lower Watershed in 1994

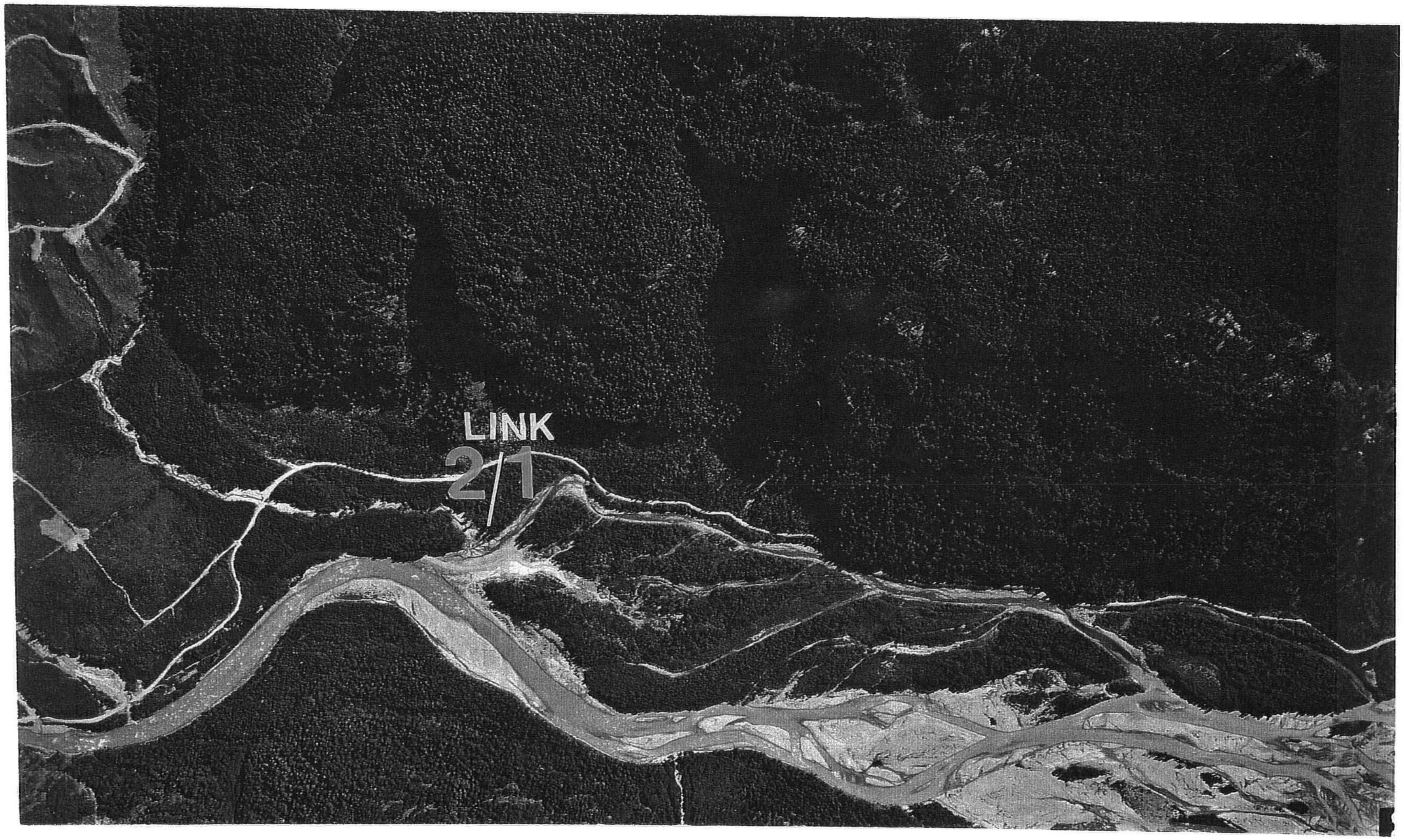


a) 1964, scale ~ 1:40,000

Figure 5.3 - Shovelnose Creek Air Photographs

b) 1974, scale 1:12,700

Figure 5.3 - Shovelnose Creek Air Photographs



c) 1994, scale 1:21,000

Figure 5.3 - Shovelnose Creek Air Photographs

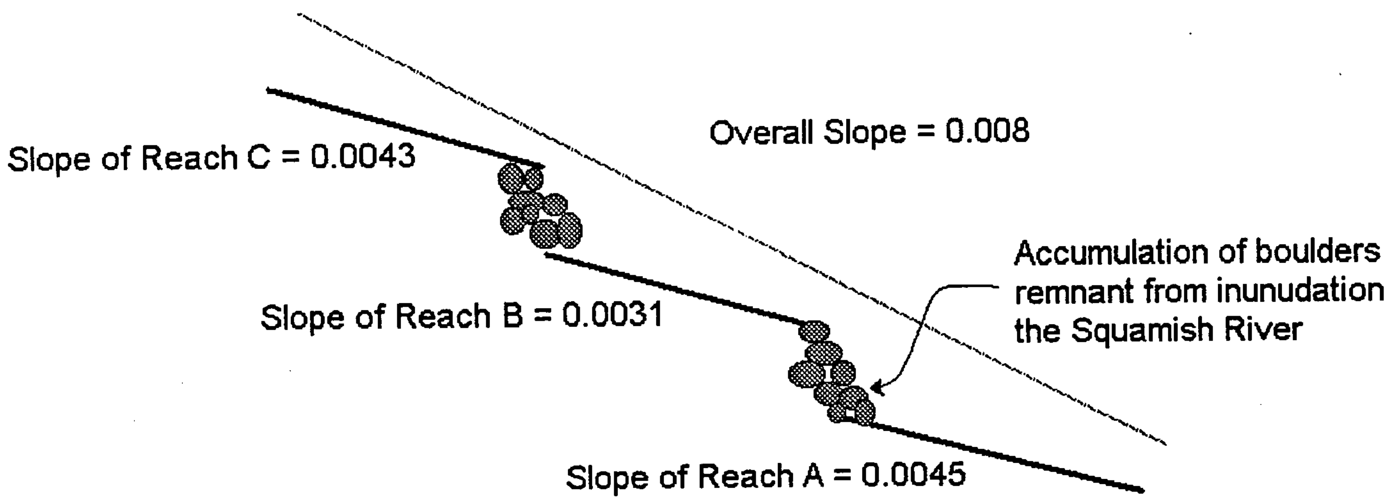


Figure 5.4 Conceptual Diagram of Slopes in Shovelnose Creek

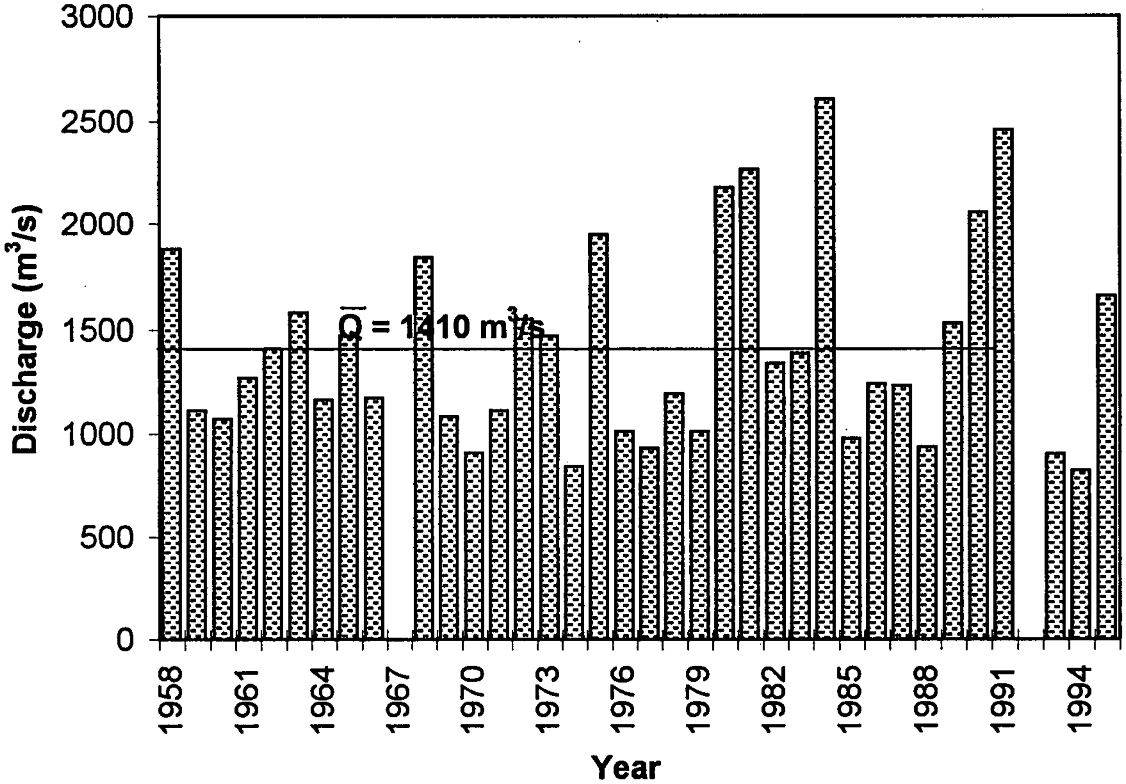


Figure 5.5 Peak Annual Instantaneous Discharges, Squamish River, Station #08GA022

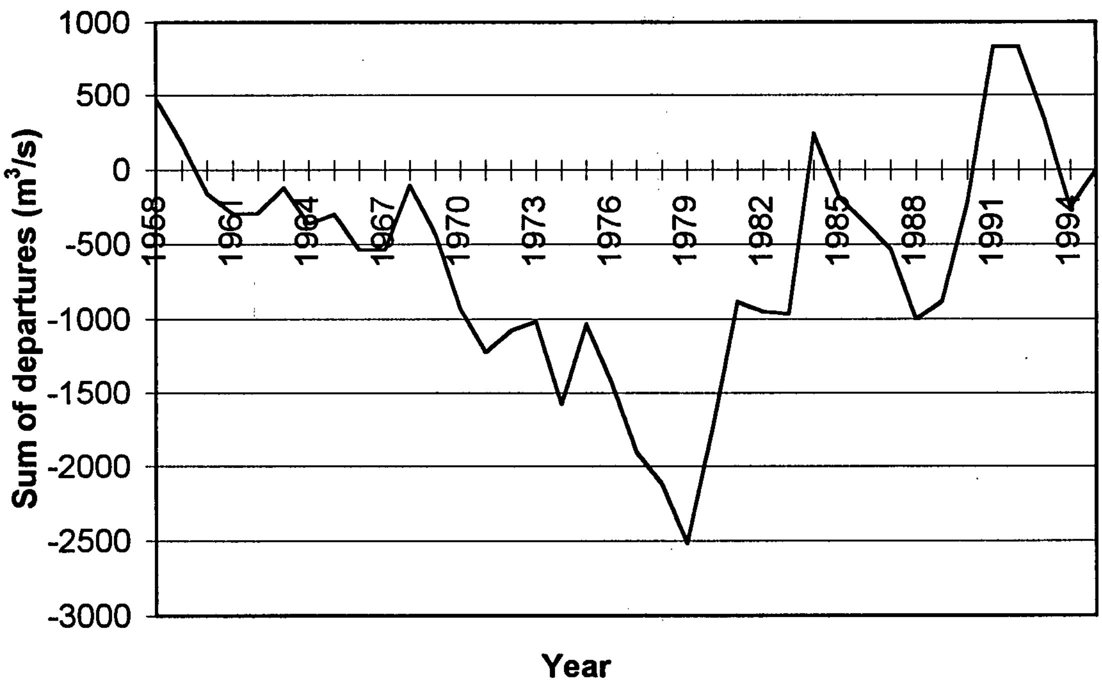


Figure 5.6 Cumulative Departures from the Mean, Squamish River, Station #08GA022

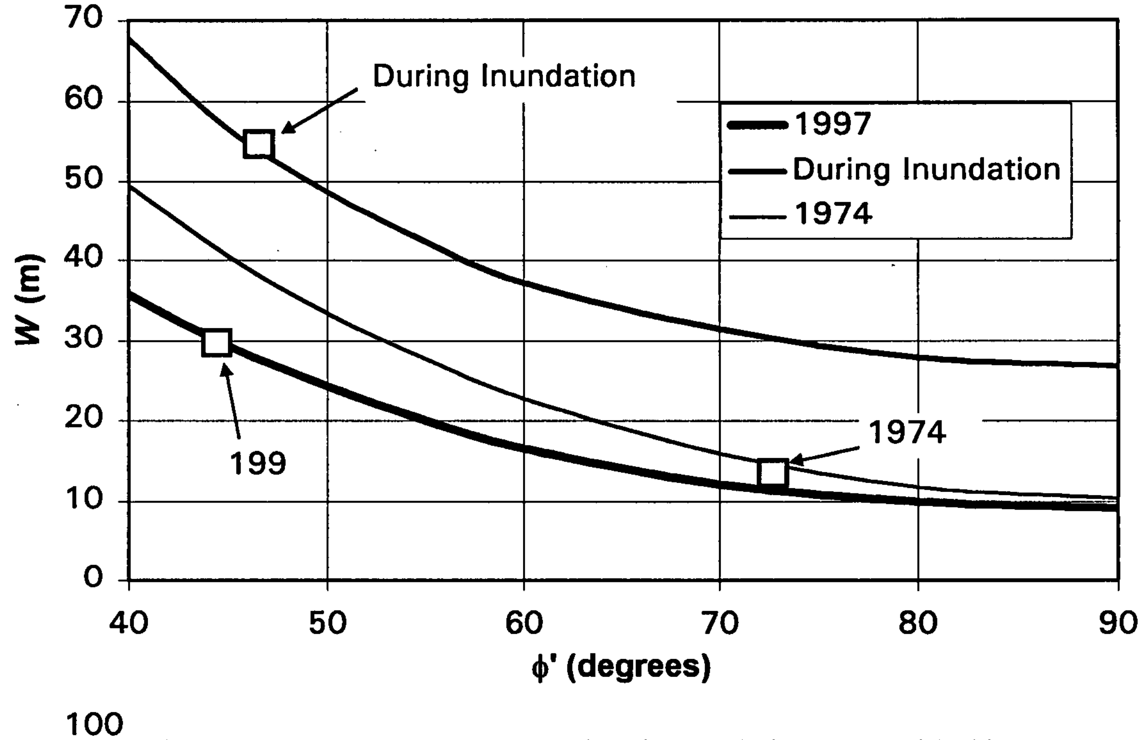
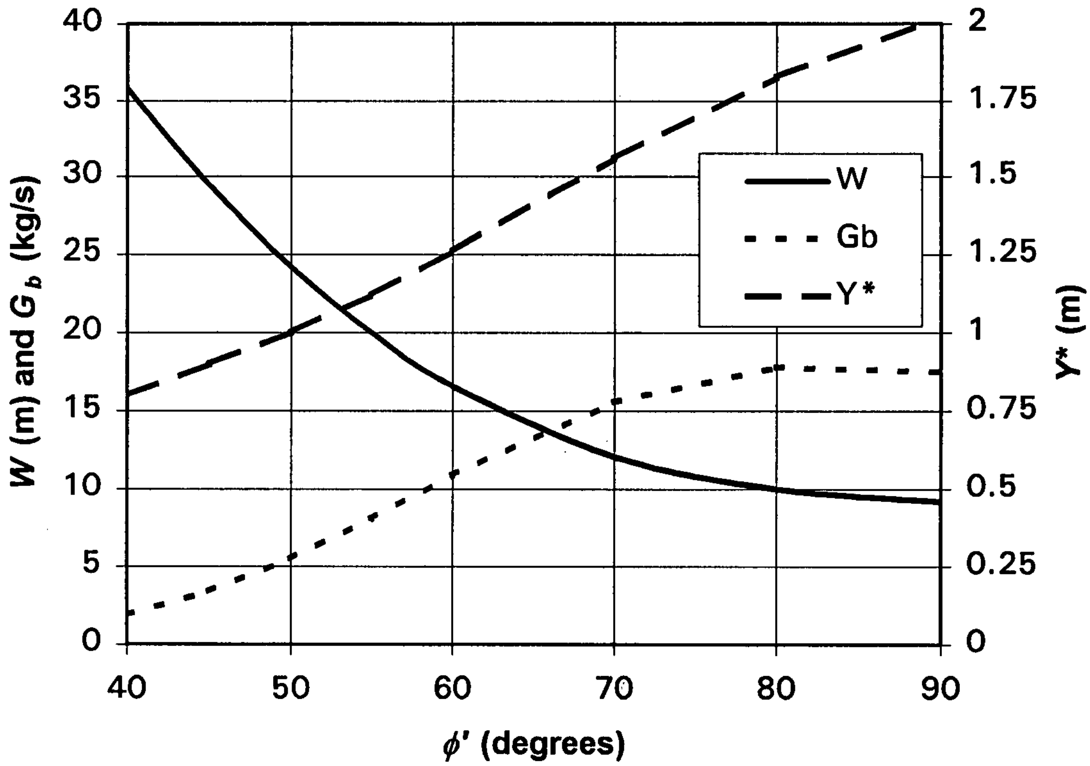
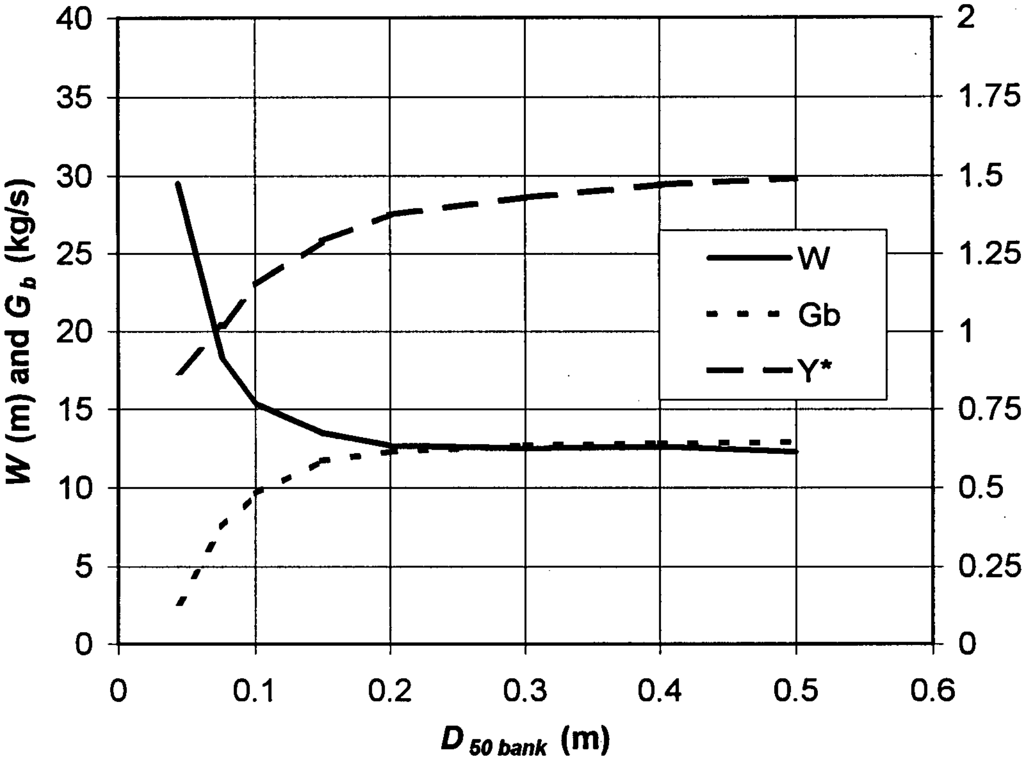


Figure 5.7 - Shovelnose Creek Calibration, variation of W with ϕ'



a) Changing Bank Stability

Figure 5.8 - Restoration of Shovelnose Creek



b) Changing Bank Material Size
Figure 5.8 - Restoration of Shovelnose Creek (cont'd)

CHAPTER 6

HARRIS CREEK

6.1 Introduction

Harris Creek is a stream on the west coast of Vancouver Island and one of the top destinations in British Columbia for steelhead trout fishing. Current fish populations are severely depressed from historical levels. Habitat problems in Harris Creek are related to sediment supply. Increased landsliding from logged terrain has led to the accumulation of sediment wedges in tributaries. Channel widening, large bar accumulations, infilling of spawning and rearing habitat and lateral erosion have resulted as wedges move downstream. There is active interest in improving fish habitat in Harris Creek.

6.1.1 Watershed Description

Harris Creek is located near the west coast of Vancouver Island (Figure 1.1) within the Vancouver Island Ranges physiographic region. It is the largest tributary to the San Juan River, draining 145 km^2 or 20 % of the larger watershed area. Mountain peaks in the watershed reach up to 1200 m, and the confluence with the San Juan River is at 10 m elevation. Harris Creek is frequently bedrock controlled in its upper reaches. In the lower reaches, the channel is unconfined, but thick glacial till remains in some locations.

Reach Division

This report will use channel reaches numbered H1-H4 as defined in Northwest Hydraulic Consultants (NHC, 1994) and shown in Figure 6.1. Reaches H1 and H2 are upstream of the confluence with Hemmingsen Creek. They are confined and narrow with an average channel slope of 1.5%. Reach H3 is located immediately downstream of the confluence with Hemmingsen Creek. It is confined in a canyon. H4 is the lowermost reach and flows on the alluvial fan of the creek. Sections from Reaches H2 and H4 were investigated in this report. Reach H2 is of active interest for restoration and Reach H4 is the most alluvial section of the river.

6.1.2 Fish Populations

Harris Creek is important for steelhead trout. It has supported two runs a year and the 15th busiest sport fishery since the early 1900's (NHC, 1994). Cutthroat trout and coho salmon also spawn in Harris Creek. Populations are considered severely depressed at the current time. Figures for coho show a decrease between 60 and 90% from 1960 levels (NHC, 1994).

6.1.3 Restoration

Due to the decline of fish populations and the importance of the fishery, there is active interest in restoring Harris Creek. Information for this study was provided by the Nanaimo Office of the Ministry of Environment Lands and Parks. Funding for the restoration of Harris Creek is being provided through the Watershed Restoration Program (WRP). NHC (1994) provided a report on the impact of forest harvesting in the San Juan Watershed. This report provided much of the background material for the current study.

6.2 Watershed History

In order to model Harris Creek, it was necessary to understand both the current condition and the changes that have occurred. In this section the history of stream morphology is reviewed along with the hydrology and forest harvesting activities. The analysis extends from current conditions back to the 1950's which is the limit of the air photo record. Air photos are shown in Figure 6.2.

For Reach H4 1952, 1970, 1984, and 1992 photos are shown and the location of the bridge crossing can be used for orientation. The 1984 photos were included instead of the 1980 photos used by NHC (1994) due to their larger scale. For Reach H2 1970, 1984, and 1992 photos are shown and a white X has been placed at the location of a channel bifurcation for orientation.

6.2.1 Stream Morphology

Reach H4 of Harris Creek is a sinuous gravel bed stream. In 1952, channel side bars were common, and the overall flow pattern was an irregular pool and riffle-sequence. Irregularities may have been due to the presence of glacial tills. Laterally it was slightly unstable, with one large scar from a past channel loop. Between 1952 and 1992 photos, the stream retained its overall appearance and location. The main change was the erosion of the outside of banks at bends. This activity has increased the width of the channel and constructed extensive point bars. The low flow channel is predominantly wide and uniform.

Upstream in Reach H2, changes are more difficult to assess. The stream is frequently confined in canyons, and width has remained nearly constant. Areas of deposition and scour are visible and there is a near complete lack of LWD in the channel. Occasional islands were observed, but similar to the lower reach, the channel retains its overall location and appearance.

The reach-averaged values of the channel geometry for Reaches H4 and H2 are shown in Table 6.1. Historic channel measurements were obtained from the report of NHC (1994) who used 1952, 1970, and 1980 air photos. A field survey in April 1998 was used to measure current channel dimensions. Historic banks were also visible and surveyed during the field visit. Only one cross-section of historic banks in Reach H4 was possible. In Reach H2, a second bank line was visible above the first one, marked by the roots of large stumps, and a reach-averaged survey of the historic hydraulic geometry was obtained. These lines were assumed to mark the pre-logging bankfull width and depth. Full survey data and calculations are included in Appendix C.

Table 6.1 - Hydraulic Geometry of Harris Creek

Reach	H4					H2	
Year	pre-logging	1952	1970	1980	1998	pre-logging	1998
Source	Surveyed	Air photo NHC, '94	Air photo NHC, '94	Air photo NHC, '94	Surveyed	Surveyed	Surveyed
W (m)	34[1]	37	40	42	51	32	29
ξ	-	1.1	1.1	1.1	1.1	-	-
S	-	0.0032	0.0032	0.0032	0.0032	0.0069	0.0069
Y^* (m)	2.6[1]	-	-	-	2.0	2.0	1.9

Note: [1] Single cross-section only

6.2.2 Hydrology

Assessing the hydrology of Harris Creek was difficult due to a lack of accurate data. The creek is gauged (Station # 08HA070), but records are preliminary, consisting of intermittent daily discharge records for the years 1996-98 (shown along with San Juan records for the same period in Figure 6.3). A gauge has been in operation on the San Juan River (Station # 08HA010) since 1960 and is currently located about 2.5 km downstream of the confluence of Harris Creek (Figure 6.4). The lower end of the discharge rating curve is considered good, but damage to the gauge has meant that few measurements of extreme discharges have been collected (NHC, 1994). It was difficult to assess trends without reliable information about extreme discharges, but a plot of the cumulative departures from the mean (Figure 6.5) does show a possible low flow period in the 1970's. The pattern was similar to that observed in the flow records of the Squamish River and Slesse Creek.

6.2.3 Forest Harvesting

Forest harvest information was derived from air photos and a Riparian Overview Assessment completed by Timberwest (1997). The earliest logging probably began before the turn of the century and concentrated on the San Juan delta. By 1952, logging via train had cut most of the old-growth forest in the Harris Creek watershed up to the confluence of the creek with its main tributary, Hemmingsen Creek. Some banks and patches of the floodplain around Reach H4 were unlogged. By 1968/70, logging had proceeded upstream into the tributaries, concentrating at first in the upper Harris and later moving into Hemmingsen Creek. Since that time, NHC (1994) estimates that about 20 to 40 km^2 of the upper Harris Creek watershed has been cut.

Impacts of forestry activities include increased landsliding, debris torrents and sediment transport. Table 6.2, adopted from NHC (1994) shows how rates have changed during the different periods. Annual landsliding was calculated for the watershed by assuming a constant depth and density for all disturbances visible on air photos. The minimum bedload transport was calculated in the report using the morphological technique of Neill (1971). This technique estimated the minimum amount of material transported by measuring eroded and deposited areas from air photos.

Table 6.2 - Sediment Supply and Transport in Harris Creek, Reach H4 from Northwest Hydraulic Consultants (1994)

Period	Annual Landsliding (tonnes/year)	% Logging Related	Minimum Bedload Transport (tonnes/year)
1952-1970	4,300	63	500 - 1,000
1970-1980	13,500	59	150 - 300
1980-1992	5,000	97	1,000 - 2,000

6.2.4 Summary

Reach H4 has increased its width and Reach H2 has slightly decreased its width. Significant increases in sediment supply to the creek have been attributed to landsliding on hillslopes post logging. The upper Harris has been less prone to landsliding in recent years. Much of the material deposited into this reach appears to have moved downstream into lower Harris Creek where it is resulting in large sediment accumulations. Additional material to the lower creek is being supplied from Hemmingsen Creek which has seen increased landsliding activity in recent years. Decreased bank stability may also be a problem in the lower reaches due to some logging to the banks, though the presence of glacial till and areas with large vegetation appear to have restricted lateral activity. Lateral activity in the upper reaches is controlled by boulders and bedrock outcrops. Trends in hydrologic data are weak but correspond with those observed for other catchments. They indicate that the period since the late 1970's has been one of increased peak floods, a possible connection to observed sediment accumulations.

6.3 Analysis

Application of the rational model required a number of steps. Firstly, input values were quantified and the model was calibrated to existing and past geometries. Secondly, stream behaviour was interpreted. The final step of modeling restoration was not possible due to limitations.

6.3.1 Model Inputs

The fixed-slope version of the Millar and Quick (1993) model was used due to vertical and lateral control of the channel. Required input variables were the bankfull discharge (Q_{bf}), median bed and bank particle sizes (D_{50}, $D_{50\,Bank}$), equivalent roughness (k_s), slope (S) and bank stability (ϕ').

Bankfull Discharge (Q_{bf})

Records for Harris Creek were not long enough to assess the mean annual peak instantaneous discharge. Equation 3.2 was used to calculate mean annual floods for the two reaches studied in

Harris Creek. A typical exponent of $n = 0.75$ was used (Harris, 1986). Bankfull flows were assumed to be the mean annual instantaneous peak discharges.

Table 6.3 - Harris Creek Bankfull Flows

	San Juan River	Harris Creek Reach H4	Harris Creek Reach H2
Area (km^2)	733	145	56
Q_{bf} (m^3/s)	840	250	120

Sediment Sizes (D_x)

Sediment sizes were measured using the Wolman (1954) pebble count technique. A sieve analysis of material transported and deposited in the margins of Reach H2 was also done. Bank sediment was difficult to distinguish from bed sediment due to the large size of material and it was assumed $D_{50} = D_{50\,Bank}$. Based on a visual inspection, the same assumption was made for Reach H4.

As sediment supply to the creek changes, the bed material can be expected to change as well. These changes cannot be measured from air photos and no records are available. Values are expected to vary significantly in Harris Creek, as channels are often confined and glacial till restricts lateral and vertical movement. Beds are expected to coarsen as a result of low sediment supply, and vice versa (Schumm, 1969). Sediment supply changes are considered to be the major change affecting Harris Creek. No assumptions could be made regarding historic particle sizes.

Flow Resistance (k_s)

Flow resistance was calculated using equation 3.5 from Bray (1982b). It predicted $k_s = 0.44$ m for Reach H4 and $k_s = 1.56$ m for Reach H2. These values could not be confirmed using hydraulic analyses. It was not possible to measure changes in roughness for historical channels.

Slope (S)

Slope in both reaches was controlled and the fixed slope version of the model was used. Changes in sinuosity were found negligible from the air photo analysis. Slope was measured during the field survey as described earlier.

Bank Stability and Calibration (ϕ')

ϕ' of the current channels was obtained by calibrating the model of Millar and Quick (1993). A plot of ϕ' versus W is shown in Figure 6.6. Two curves are shown, one for each reach. ϕ'_{H4} = 49° and ϕ'_{H2} = 44° based on an agreement between modeled and measured width.

Summary

Established values of independent variables are listed in Table 6.4. Collected data and calculations are included in Appendix C.

Table 6.4 - Input Variables of Harris Creek

Reach	H4				H2	
Year	undisturbed section	1952	1980	1998	pre-logging	1998
Q (m^3/s)	250	250	250	250	120	120
D_{50} (m)	-	-	-	0.065	-	0.23
$D_{50\,Bank}$ (m)	-	-	-	0.065	-	0.23
$D_{50\,Bulk}$ (m)	-	-	-	-	-	0.017
k_s (m)	-	-	-	0.44	-	1.56
S	-	0.0033	0.0033	0.0033	0.0069	0.0069
ϕ' (°)	-	-	-	49	-	44

6.3.2 Interpretation of Stream Behaviour

The lowest reach of Harris Creek has experienced an increase in sediment supply, and has responded by increasing width. Reach H2 of Harris Creek has experienced a decrease in channel width. Changes within Harris Creek are predominantly related to sediment supply and associated changes of particle sizes and roughness. These changes are not easily measured with available techniques and it was not possible to interpret the behaviour of Harris Creek.

6.3.3 Restoration Modeling

Because the model had not been calibrated to understand the behaviour of Harris Creek, it was not used to model the possible restoration efforts.

6.4 Limitations of Analysis

This step was followed to clarify the reasons the model was inapplicable to Harris Creek. Sources of error and disturbances were assessed.

6.4.1 Sources of Error

Sources of error were identified as the calculation of flows from the gauge on the San Juan River and the inability to model changes to sediment transport.

Calculation of Q_{bf}

A primary source of error was the calculation of Q_{bf}. No gauge records were available, and calculation of Q_{bf} using the San Juan River gauge data and equation 3.2 produced estimates that were too large for the measured channels. The errors were felt to be beyond what could be expected from errors of measuring the bankfull condition. Better information is needed to increase the confidence of observations and predictions.

Inability to Directly Measure Sediment Transport

The major disturbance identified was an increase in sediment supply. It was not possible to directly measure an increase in sediment transport. Without historic values of bed material sizes, depth or channel roughness, it was also not possible to calculate the changes to sediment transport capacity. NHC (1994) found the rate of sediment transport associated with lateral erosion to be varying, but the utilized technique does not necessarily correspond with the overall rate of sediment transport.

6.4.2 Disturbance

The impact of logging activities has been an increase in sediment supply. This increase has resulted in waves of sediment passing through Harris Creek. These waves take many years to pass and represent a long-term disturbance to the creek (Roberts and Church, 1986). Reach H4 is also located on an alluvial fan, indicating equilibrium may not be the normal state of the reach.

6.4.3 Summary

Limitations in the analysis were assessed in order to identify reasons the analysis was limited. While calculation of bankfull flows was a problem, the primary limitation was the inability to directly measure sediment transport. Also, sediment supply waves are an ongoing disturbance.

6.5 Conclusions

Conclusions are that:

- Width has increased in the lower reach and decreased in the upper reach, primarily as a result of changes to sediment supply;
- It was not possible to calculate historic values of depth, roughness or sediment transport
- Sediment supply waves are an ongoing long term disturbance; and
- It was not possible to model Harris Creek with the model of Millar and Quick (1993).

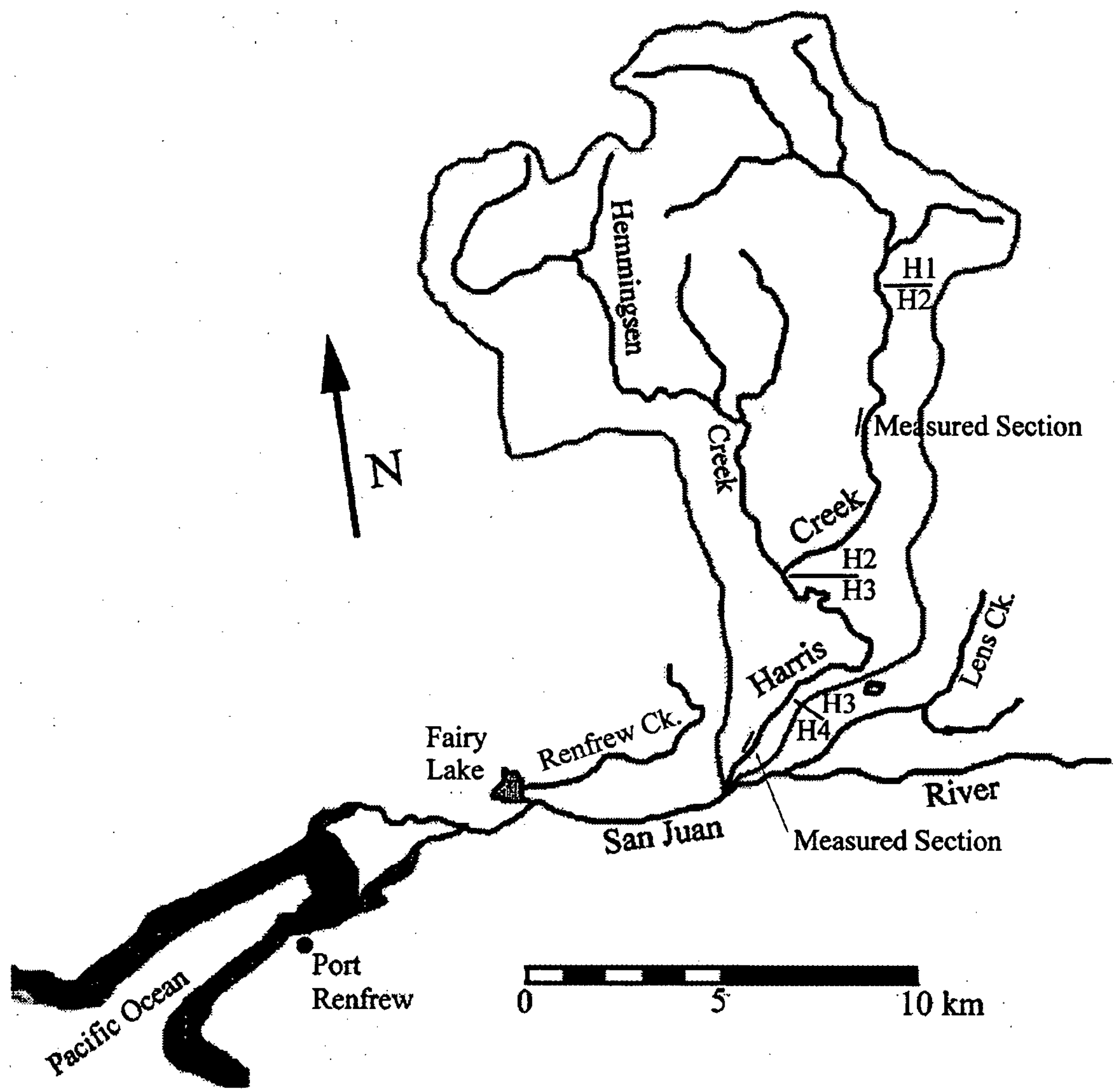
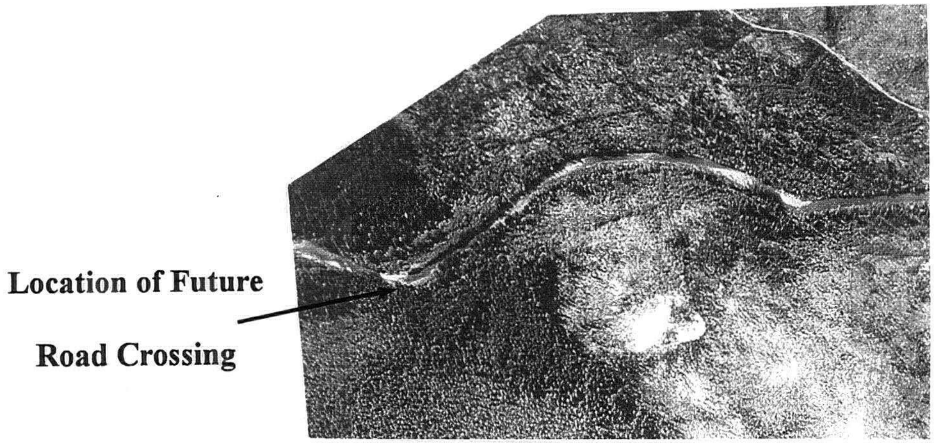
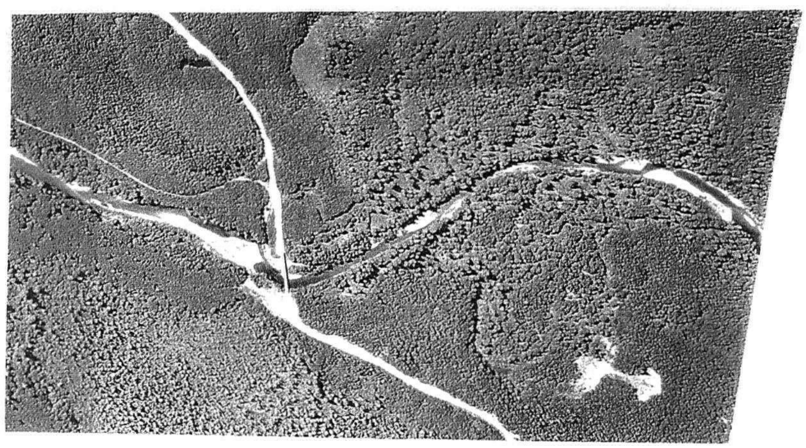


Figure 6.1 - Harris Creek Watershed



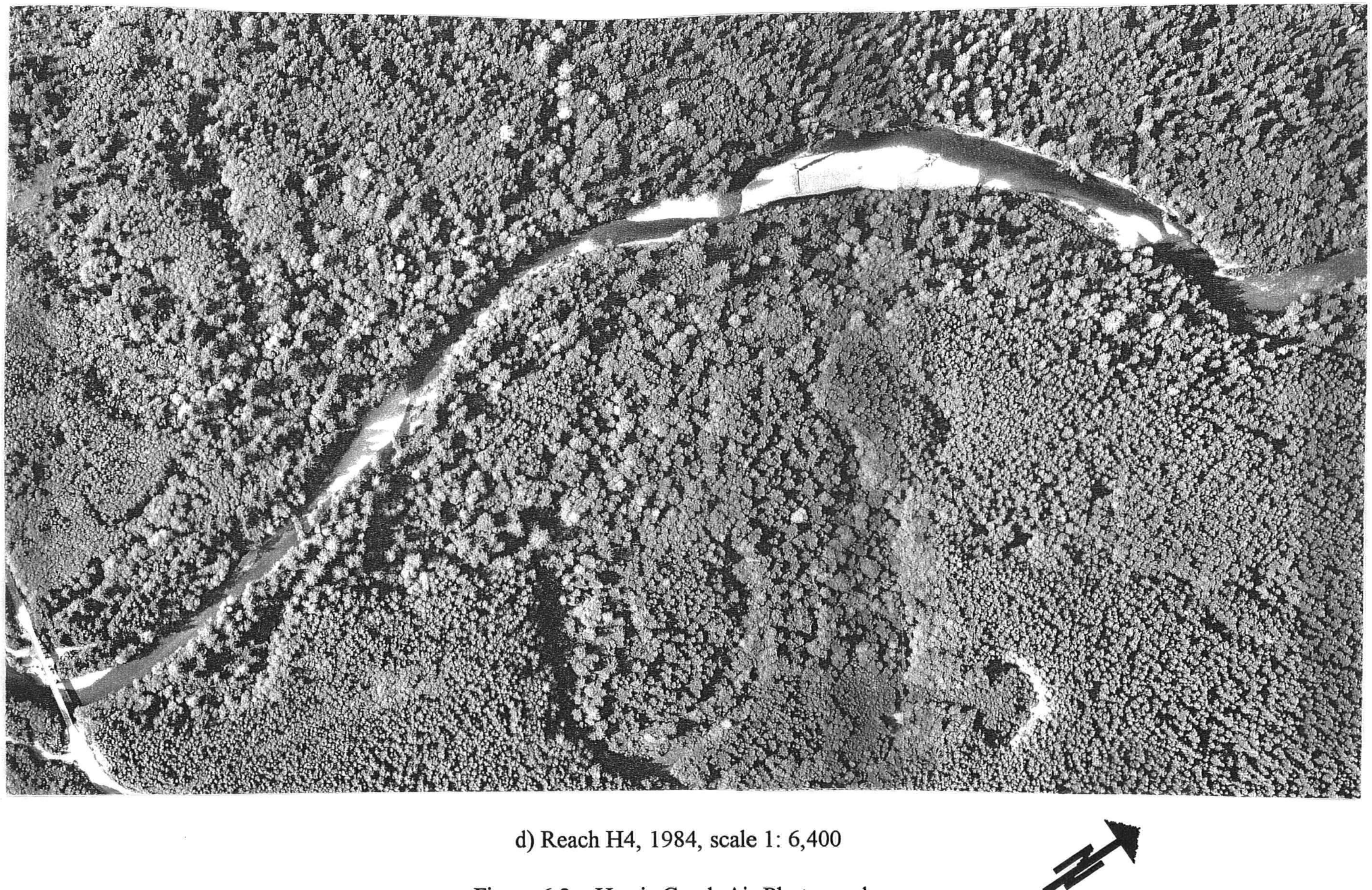
a) Reach H4, 1952, scale 1:21,300



b) Reach H4, 1970, scale 1:18,500

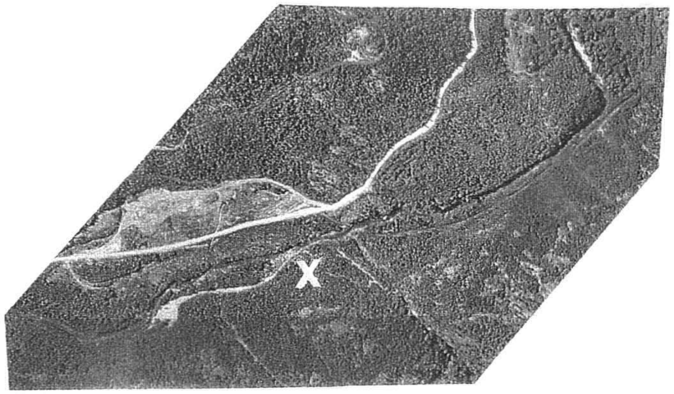
c) Reach H4, 1992, scale 1:23,000

Figure 6.2 – Harris Creek Air Photographs

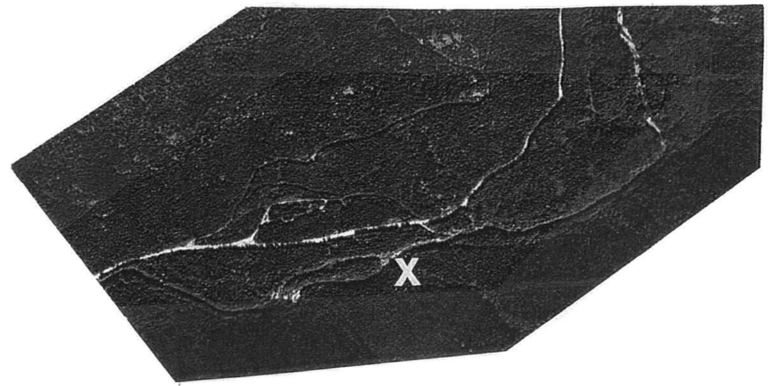


d) Reach H4, 1984, scale 1: 6,400

Figure 6.2 – Harris Creek Air Photographs



e) Reach H2, 1970, scale 1:18,500



f) Reach H2, 1992, scale 1:23,000

g) Reach H2, 1984, scale 1:6,400

Figure 6.2 – Harris Creek Air Photographs

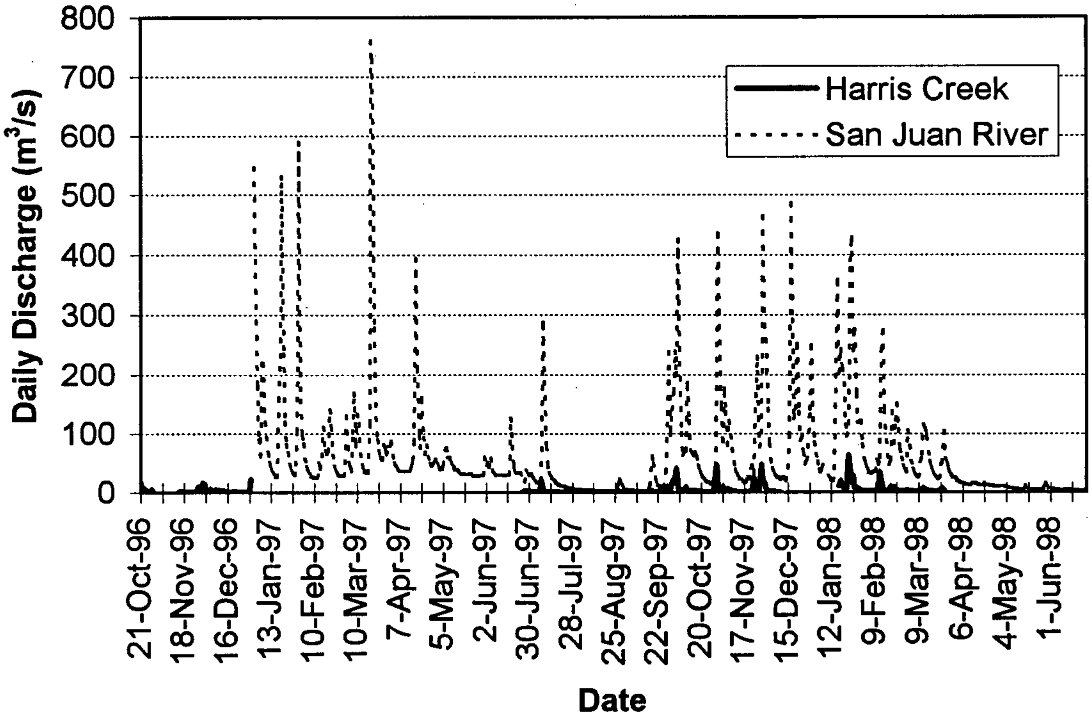


Figure 6.3 - Daily Discharge Record for Harris Creek and the San Juan River
Station # 08HA070 and # 08HA010

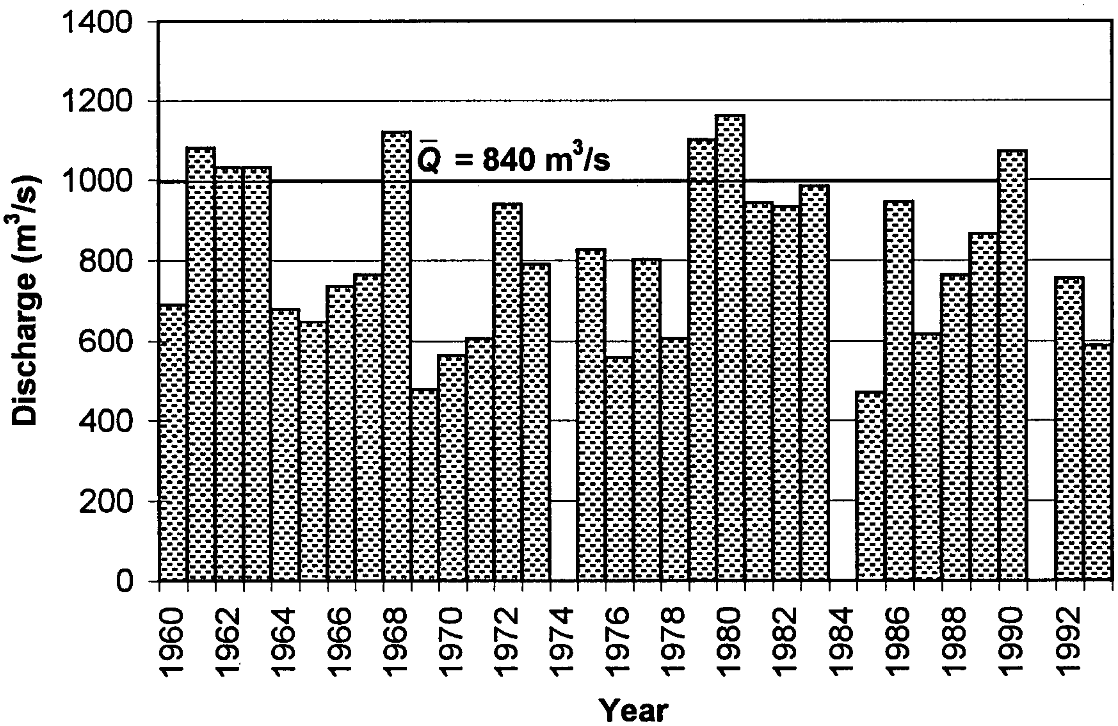


Figure 6.4 - Annual Peak Instantaneous Discharges, San Juan River Station # 08HA010

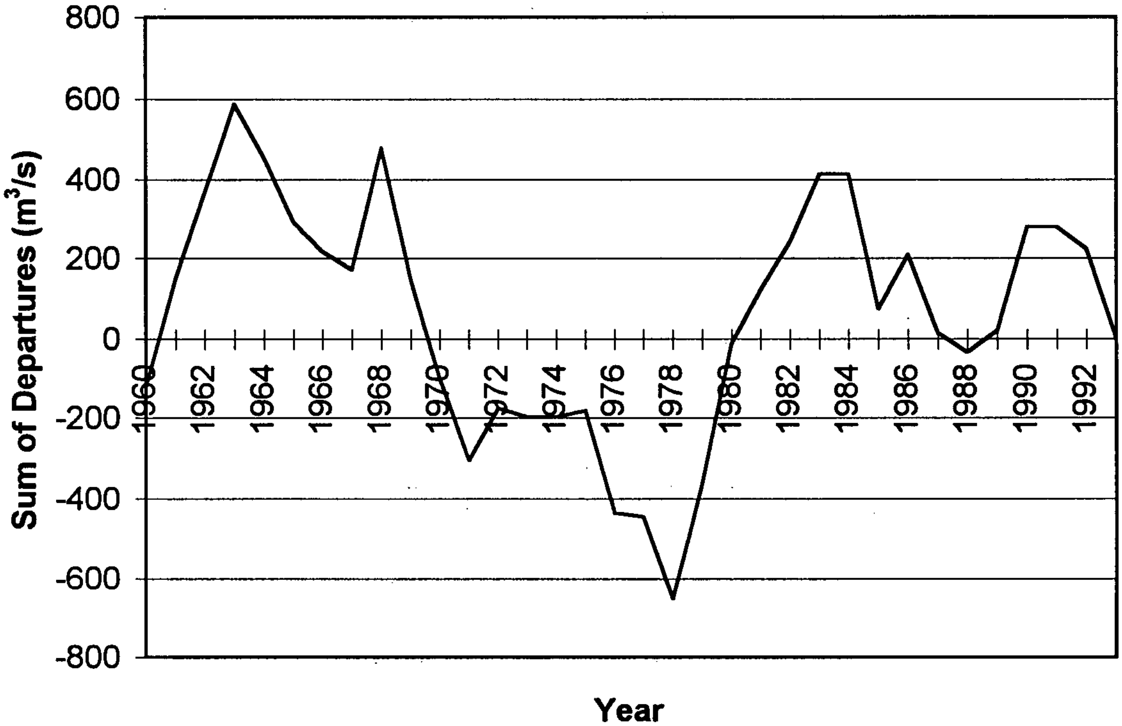


Figure 6.5 - Cumulative Departures from the Mean, San Juan River, Station # 08HA010

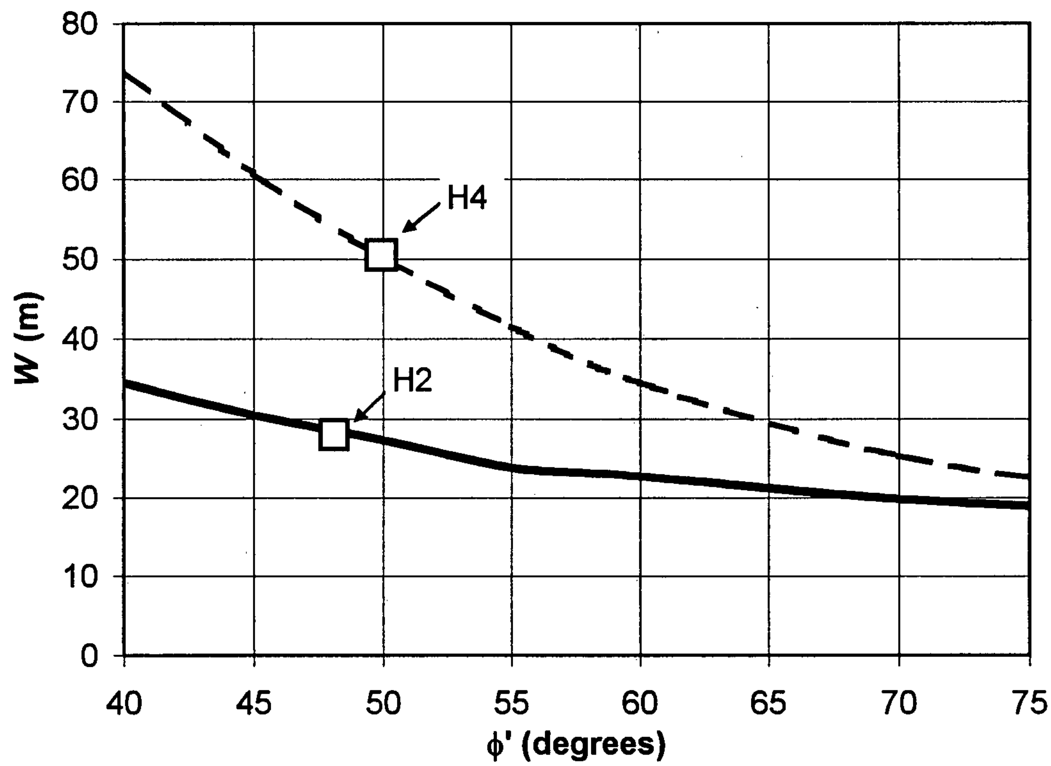


Figure 6.6 - Harris Creek Calibration, variation of W with ϕ'

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Introduction

Results are discussed in this chapter in five sections. Section 7.2 summarizes the case studies of Slesse, Shovelnose and Harris Creeks and details implications for the restoration of those streams. Section 7.3 discusses the calibration and measurement techniques used in this thesis. Section 7.4 highlights which types of disturbances are more accurately understood with the developed approach. Section 7.5 examines what future work could improve this type of analysis. Section 7.6, the final section, reviews the key advantages of a calibrated rational model that separate it from other approaches.

7.2 Case Studies

Three disturbed streams in British Columbia were investigated in order to calibrate the physically based model of Millar and Quick (1993) and predict stream adjustments. These streams were chosen to represent different types of common disturbances. Slesse Creek has been disturbed by a decrease in bank stability. Shovelnose Creek has suffered a period of extreme flooding due to an avulsion from a larger river and was used as an example of streams disturbed by high flows. Harris Creek has been subject to sediment waves and was chosen as an example of streams disturbed by increases in sediment supply.

7.2.1 Slesse Creek

Slesse Creek has experienced a dramatic change in its channel planform coincident with human activities. The width of the creek has increased from $W_{1936} = 30$ m to $W_{1993} = 145$ m and a single thread channel has been replaced by a multiple thread braided channel. Disturbances were identified as an increase in peak discharges from $Q_{bf1936} = 90$ m^3/s to $Q_{bf1993} = 120$ m^3/s as obtained from flow gauge records, and a decrease in bank stability from $\phi'_{1936} = 75°$ to $\phi'_{1993} = 40°$ as obtained from the calibration of the model of Millar and Quick (1993). The historical behaviour of the creek and modeling results indicate that the decrease in bank stability was the dominant disturbance. The source of the disturbance appears to have been widespread forest harvesting within the riparian zone of the creek.

A restoration plan was conceived and modeled based on twin objectives of reducing lateral instability and narrowing channel widths. It calls for a moderate decrease of width to $W = 60$ m using a combination of increased bank material size from $D_{50} = 0.13$ m to $D_{50} = 0.40$ m and/or an increase in the bank stability parameter to $\phi' = 60°$. Efforts should proceed downstream from a confined section of the channel in order to minimize the risk of outflanking. If successful, a single channel would be maintained and channel stability would be increased in the short term while the long term stability of the creek would be increased by the regrowth of riparian vegetation. Risk of failure is high due to long term trends in peak flows and sediment supply waves.

7.2.2 Shovelnose Creek

Shovelnose Creek has also experienced a significant change in its channel planform coincident with human activities. The width of the creek has changed, first increasing from $W_{1974} = 15$ m to $W_{DI} = 50$ m then decreasing to $W_{1997} = 30$ m. The current stream is oversized for current flows, slopes are controlled by accumulations of large sediment, and sand and small gravels are depositing over a cobble bed which is immobile at current flows. Disturbances were identified as an increase in peak flows from $Q_{bf\,1974} = 45$ m^3/s to $Q_{bf\,DI} = 540$ m^3/s as obtained from a regional hydraulic analysis and a measurement of a relic channel, and a decrease in bank stability from $\phi'_{1974} = 70°$ to $\phi'_{DI} = 45°$ as obtained from calibration with Millar and Quick (1993). Sources of the disturbances were a channel avulsion from the Squamish River connected to natural hillslope instabilities, and a decrease in bank root strength related to forest harvesting in riparian areas.

Restoration of Shovelnose Creek was modeled based on a need to narrow and deepen the channel while increasing transport of fine sediment sizes. Increases in ϕ' and $D_{50\,Bank}$ were found to achieve the desired goals and it was recommended that point deflectors be constructed to a minimum width of $W > 15$ m with $D_{50\,Bank} \geq 0.20$ m. Accuracy of results may be affected by slope discontinuities which are likely to result in changing values of sediment sizes, roughness and sediment transport over time. Long term restoration of Shovelnose Creek was not considered due to the likelihood of repeated avulsions from the Squamish River.

7.2.3 Harris Creek

The quality of fish habitat in Harris Creek has decreased coincident with human activities. Two reaches of Harris Creek were studied. From the air photo record the upstream reach has decreased its width while the downstream reach has increased its width. The disturbance was variations in sediment supply. The source of these sediment waves was found to be forest harvesting on steep hillslopes in the watershed that resulted in periods of increased landsliding and sediment delivery.

The model of Millar and Quick (1993) could not be calibrated to interpret the behaviour of Harris Creek. The major difficulty was the inability to directly measure changes to sediment transport. An equilibrium approach was limited in this case as sediment waves were a long term disturbance.

7.3 Usefulness of Measurement Techniques for Calibration

A variety of techniques developed by other authors were used in this thesis. This section summarizes the success of applying those techniques towards calibrating the model of Millar and Quick (1993) to particular streams.

7.3.1 Analysis of Hydrologic Records

Hydrologic records were a critical source of information for the analysis. Accurate, long term records were used to establish dominant discharges, extreme flood events and long term trends. Dominant discharges were identified in Chapter 2 as the most important variable in determining channel size. The analysis of Harris Creek was limited by a lack of reliable flow information and because calculated flows could not be calibrated to observed channel geometries.

Flood events and long term trends were used to isolate disturbance events. A lack of long term flow records in Shovelnose Creek prevented the separation of flow disturbances from disturbances to bank stability, leaving open the possibility that the model was inaccurately calibrated. By comparison, flow records for Slesse Creek allowed a plot of cumulative departures from the mean to be constructed and flow trends identified. Flow trends were instrumental in calibrating the model of Millar and Quick (1993) to Slesse Creek.

7.3.2 Analysis of Air Photographs

Air photos were also a critical source of information. For many areas of British Columbia, no other historical records of stream form are available. The classification system of Kellerhals *et.al.* (1976) and the techniques of Mollard (1973) provided systematic methods for making a variety of measurements and assessments of stream condition that would not otherwise be possible. In particular, width and sinuosity measurements were important for calibration, and the time series of photographs were used to qualitatively indicate changes to sediment supply and bank stability.

7.3.3 Field Measurements

The field guides of Harrelson *et.al.* (1994) and Newbury and Gaboury (1993) were very useful for making consistent field measurements. The two critical measurements were found to be slope and particle sizes as these variables could not be measured reliably from maps or air photographs. Widths could be obtained from air photos, although values were more reliable if checked with field results. A significant problem was that confidence in estimates of the bankfull depth was often low due to unreliable bankfull indicators, but field measurements were the only method to estimate this variable. In general, field measurements were critical for calculating flow resistance and sediment transport. To improve confidence in the results, more that one reach should be measured.

7.3.4 Measurement of Relic Channels

Relic channels, or those left from previous flow regimes were critical for measuring historic particle sizes. Mean channel depth was also useful where reliable bankfull markers were found. The analyses of Shovelnose and Harris Creeks were limited because relic channels were not found, and sizes of bed material were expected to have changed significantly.

7.4 Types of Disturbance

The applicability of the model depended on the type of disturbance observed. The reasons for the success of the model in different situations are discussed in this section by looking at changes to bank stability, flow, and sediment transport. Changes to sediment sizes and channel roughness are considered to be part of changes to sediment transport and were discussed together.

Changes to bank stability were the most easily understood and the most accurately modeled. The primary reason for this was the importance of ϕ' in determining the width (W) and the ease with which W could be measured. The model of Millar and Quick (1993) is thus suitable for calibrating and modeling the restoration of streams disturbed by changes to bank stability.

Where gauge records existed, changes to Q_{bf} were also successfully modeled. Flow changes were found to account for adjustments in Slesse Creek between 1936 and 1973. This was later important in isolating future flow changes between 1973 and 1993 from the changes to bank stability that occurred in the same period. The model was thus suitable for application to streams disturbed by changes to peak flows. For the analyses of Harris and Shovelnose Creeks, flow records were not available and trends could not be reliably established, limiting the analyses.

Changes to sediment sizes, roughness and sediment transport were the most difficult problems to assess. It was not possible to model Harris Creek because changes to sediment transport could not be calculated and because the anticipated sediment waves are a disturbance event that may be stretched over 20-30 years and the application of an equilibrium model will not be suitable during this interval. Modeling of both Slesse and Shovelnose Creeks was possible because it was possible to assume an unchanged rate of sediment supply. Sediment supply in both watersheds was dominated by natural sources that regularly contributed bed material directly to the stream. The model of Millar and Quick was thus not applied to streams disturbed by changes to sediment supply.

7.5 Future Work

This section lists and discusses possible areas of future research.

Increased Understanding of ϕ' - Changes to bank stability resulted in both Slesse and Shovelnose Creeks due to the effects of past logging. These changes were calibrated and it was found that the change could be simulated in both cases by a decrease in ϕ' from above 70° to somewhere in the low 40° range. This agreement in the findings indicates that there may be a predictable relation between ϕ' and the age of vegetation. Two approaches to further research can be recommended. The first is to develop a regional approach to predicting ϕ' based on vegetation. This approach was applied by Millar (1994) to streams in the U.K. (data from Hey and Thorne, 1986). This type of relation may find patterns that could be used to predict ϕ' for streams in a homogeneous region. The danger of this approach is that an empirical estimate of ϕ' could mask rather than reveal trends. A second approach is to develop a rational method to determine ϕ'. This approach could eventually allow ϕ' to be calculated directly from an analysis of the banks. Other than bank vegetation, factors such as the size of bank material, and the size of the stream should be investigated.

Significance of Cumulative Departures from the Mean - The cumulative departures from the mean method of isolating trends was found useful, though more research should be used to establish what signifies a trend. Significance could be based on both a trend length and/or a relative deviation from the long term mean. Significance could also be tied to the rate and thresholds of channel adjustments.

Increased Measurement of Relic Channels - Brookes (1986), shows the potential benefits of increasing the effort put in to measuring relic channels. He used floodplain excavations to determine historic sediment sizes. This method would be most applicable in streams subject to avulsions or those artificially moved.

Inclusion of Other Sediment Transport Relations - It would be useful to be able to use a variety of sediment transport formulae in the model of Millar and Quick (1993). Gomez and Church (1989) have identified that some formulae will scale better than others. The formula of Bagnold

(1966) was found to scale particularly well. The formula used will affect the ability of the model to accurately predict channel changes relative to a known starting form.

Modeling Adjustments of Channel Roughness - Roughness was assumed to be constant due to an incomplete knowledge of how it adjusts with changes to independant variables. Slesse Creek had a variety of resistance processes hidden in the empirical coefficient, reducing the robustness of the model when analysing the response of the stream to changes in its independent variables. More work is needed to separate the components of roughness.

Another approach would be to allow maximization of roughness as an extremal hypotheses. Griffiths (1984) have mathematically found and Simon and Thorne (1996) have practically demonstrated that no roughness maximization trend will occur when W, Y^*, and S are unconstrained. Yang (1987), however, has indicated that it may occur where the channel is constrained. Harris Creek was an example of a stream where slope was constrained while sediment supply varied. The formulation of Davies and Sutherland (1983) should be examined and possibly included within the fixed slope version of the Millar and Quick (1993) model.

Morphologic Techniques of Calculating Sediment Transport - A major limitation of analysis is the reliance on indirect calculations of sediment transport. A solution would find a direct method of establishing sediment transport and eliminate the weakness of calculating it from channel geometries. Morphological techniques such as those of Neill (1971) hold some promise, although a couple of problems are evident. Firstly, current techniques calculate only the minimum transport rate and cannot measure the amount of material that is transported though the reach without being deposited or eroded from an identifiable channel feature. Secondly, even if an accurate morphologic technique is found, it cannot easily be related to a characteristic discharge.

7.6 Usefulness of a Rational Approach

In this thesis a rational approach to the problem of stream restoration was identified as having potential above other approaches because it was specifically suited to problems of modeling and prediction. The rational model of Millar and Quick (1993) was applied to streams subject to flow and bank stability disturbances by calibration with various measurements of current and historical parameters. The model was then used to interpret past stream behaviour and develop restoration recommendations in these streams.

There are three main advantages of a rational approach over the application of template and empirical approaches to stream restoration. Firstly, in a rational approach, attention is focused on stream processes, providing a crucial link between observed disturbances and a stream's hydraulic geometry. This link may increase our understanding of our impacts on the stream ecosystem and reduce the misapplication of restoration practices in disturbed streams.

The second advantage is that exact numerical results are produced. These results can be used to develop restoration recommendations and calculate the costs of a restoration project. They can also be used to test the accuracy of the model of Millar and Quick (1993), limits to available measurement techniques, and limits and assumptions in available mathematical formulations.

The third and main advantage of a rational approach is that a stream response framework specific to each stream is produced. This framework can be used to understand stream adjustments, guide restoration efforts, test alternative approaches and make stream management decisions. Stream response frameworks can also be improved with time by monitoring future changes to streams and as mathematical formulations of stream processes are improved.

REFERENCES

Allan, J.H., and S. Lowe, 1997: "Rehabilitating mainstem holding and rearing habitat." In *Fish Habitat Rehabilitation Procedures*, P.A. Slaney and D. Zaldokas (editors), Watershed Restoration technical Circular No. 9. B.C. MoELP and Ministry of Forests

Anon, 1994: *Natural Channel Systems: An Approach to Managment and Design,* Ontario Ministry of Natural Resources, 103 p.

Anon, 1996: *Channel Assessment Procedure Guidebook.* Province of B.C. Forest Practices Code, 37 p.

Babikaiff and Associates Geoscience, 1997: *Slesse Creek: Channel assessment and prescription development (Draft).* Prepared for Steelhead Society Habitat Restoration Corp. 72 p.

Bagnold, R.A., 1966: "An approach to the sediment transport problem from general physics." *Professional Paper 422-I,* U.S. Geological Survey, 37 p.

Bathurst, J.C., 1982: "Theoretical aspects of flow resistance." In *Gravel Bed Rivers,* R.D. Hey, J.C. Bathurst, and C.R. Thorne (Eds.), John Wiley & Sons, Chicester, 83 -105.

Baumann, F.W., 1994: *Preliminary Terrain and Hydrologic Assessment, Shovelnose Creek area.* Prepared for the Steelhead Society of British Columbia.

Berger, J.J., 1992: "The Blanco River." In *Restoration of Aquatic Ecosystems,* S. Maurizi and F. Poillon (Eds.), National Research Council, Washington National Academy Press, 470-477.

Bettress, R., and W.R. White, 1987: "Extremal hypotheses applied to river regime." In *Sediment Transport in Gravel-bed Rivers,* C.R. Thorne, J.C. Bathurst, and R.D. Hey (Eds.), Wiley & Sons, Chichester, 767-778.

Blench, T., 1957: *Regime Behaviour of Rivers.* Butterworths, London, England, 138 p.

Bradshaw, A.D., 1987: "Restoration: an acid test for ecology." *In Restoration Ecology: a synthetic approach to ecological research,* I.H. Rorison, J.P. Fime, R. Hunt, G.A.F. Hendry, and D.H. Lewis (Eds.), Academic Press, London, 3-21.

Bradshaw, A.D., 1994: "Underlying principles of restoration." *Canadian Journal of Fish and Aquatic Science.* **53** (Suppl. 1): 3-9.

Bray, D.I., 1982a: "Regime equations for gravel-bed rivers." In *Gravel Bed Rivers,* R.D. Hey, J.C. Bathurst, and C.R. Thorne (Eds.), John Wiley & Sons, Chicester, 517-552.

Bray, D.I., 1982b: "Flow resistance in gravel-bed rivers." In *Gravel Bed Rivers,* R.D. Hey, J.C. Bathurst, and C.R. Thorne (Eds.), John Wiley & Sons, Chicester, 109-135.

Brice, J., 1968. "Meandering pattern in the White River in Indiana - an analysis." In *Restoration of Rivers and Streams*. J. Gore (Ed.), 178-200.

Brookes, A., 1986: "Restoring the sinuosity of artificially straightened stream channels." *Environmental Geology and Water Science*, **10** (1), 33-41.

Brooks, G.R., and E.J. Hickin, 1991: "Debris avalanche impoundments of the Squamish River, Mount Cayley area, southwestern British Columbia." *Canadian Journal of Earth Sciences*, **28** (9), 1375-1385.

Brush, L.M., 1961: "Physiographic and hydraulic studies of rivers." *Professional Paper 282-F*, U.S. Geological Survey, 175 pp.

Chang, H.H., and J.C. Hill, 1977: "Minimum stream power for rivers and deltas." *Journal of the Hydraulic Division*, ASCE, **103** (12), 1375-1389.

Chang, H.H., 1982: "Mathematical model for erodible beds." *Journal of the Hydraulics Division*, ASCE, **108** (5), 678-689.

Charlton, F.G., P.P. Brouwn, and R.W. Benson, 1978: "The hydroulic geometry of some gravel-bed rivers in Britain." *Report IT 180*, Hydraulic Research Station, Wallingford.

Church, M., 1992: "Channel morphology and typology." In *The Rivers Handbook, v. 1*, P. Calow and G.E. Petts (Eds.), Oxford, Blackwell Science, 122-143.

Church, M., and M.J. Miles, 1987: "Meteorological antecedents to debris flow in southwestern British Columbia: Some case studies." In *Debris Flows/Avalanches: Process, recognition and mitigation.* J.E. Costa and G.F. Wieczorek (editors). Reviews in Engineering Geology Volume VII, 63-79.

Church, M., and O. Slaymaker, 1988: "Disequilibrium of holocene sediment yield in glaciated British Columbia." *Nature*, **337**, 452-454.

Clark, B.J., 1988: *Squamish Steelhead Investigations 1977-1979.* Regional Fisheries Report No. LM 001. Ministry of the Environment.

Darby, S.E., and C.R. Thorne, 1996: "Numerical simulation of widening and bed deformation of straight sand-bed rivers. I: Model devleopment." *Journal of Hydrulic Engineering*, ASCE, **122** (4), 184-193.

Davies, T.R.H., and A.J. Sutherland, 1983: "Extremal hypotheses for river behaviour." *Water Resources Research*, **19** (1), 141-148.

Ferguson, R.I., 1986: "Hydraulics and hydraulic geometry." *Progress in Physical Geography*, **10**, 1-31.

Ferguson, R.I., 1987: "Hydraulic and sedimentary controls of channel pattern." In *River Channels: Environment and Process*, K.S. Richards (editor), Blackwell, Oxford, UK, 129-158.

Flintham, T.P., and Carling, P.A., 1988: "The prediction of mean bed and wall boundary shear in uniform and compositely roughened channels." In *International Conference on River Regime*, W.P. White (editor), Wiley, Chichester, 267-287.

Frissell, C.A., W.J. Liss, C.E. Warren, and M.D. Hurley, 1986: "A hierarchical framework for stream habitat classification: viewing streams in a watershed context." *Environmental Management*, **10** (2), 199-214.

Friedkin, J.F., 1945: *A Laboratory Study of the Meandering of Alluvial Rivers,* U.S. Waterways Experimental Station, Vicksburg, 40 pp.

Furbish, D.J., 1988: "River-bend curvature and migration: How are they related?" *Geology,* **16**, 752-755.

Furbish, D.J., 1991: "Spatial autoregressive structure in meander evolution." *Bulletin,* Geological Society of America, **103**, 1576-1589.

Fusillo, T.V., G.H. Nieswand, and T.H. Shelton, 1977: "Sediment yields in a small watershed under suburban

Gilbert, G.K., 1914: "The transportation of debris by running water." *Professional Paper 86,* U.S. Geological Survey, 263 p.

Gomez, B., and M. Church, 1989: "An assessment of bed load sediment transport formulae for gravel bed rivers." *Water Resources Research,* **25** (6), 1161-1186.

Griffiths, G.A., 1984: "Extremal hypotheses for river regime: an illusion of progress." *Water Resources Journal,* **20 (1)**, 113-118.

Harris, J.D., 1986: "Design flood estimation for medium and large watersheds." Chapter 8 of MTC Drainage Manual, Ontario Ministry of Transportation and Communications, p H4-7.

Harrelson, C.C., C.L. Rawlins, and J.P. Potyondy, 1994: *Stream Channel Reference Sites: an illustrated guide to field technique*. General Technical Report RM-245, USDA and Forest Service, Rocky Mountain Forest and Range Experiment Station, Fort Collins, CO, 61p.

Hay and Company Consultants Inc., 1992: *Chilliwack River Hazard Management Study.* Interime Report. Prepared for the Fraser Cheam Regional District, 116 p.

Hay and Company Consultants Inc., 1995: *Creek Inspections and Landslide Inventory: Coho and Shovelnose Creeks.* Prepared for the Steelhead Society of British Columbia.

Hey, R.D., 1979: "Flow resistance in gravel bed rivers." *Journal of the Hydraulics Division,* **105** (4), 365-379.

Hey, R.D., and C.L. Thorne, 1986: "Stable Channels with mobile gravel beds." *Journal of Hydraulic Engineering.* **112** (8), 671-689.

Hey, R.D., 1988: "Mathmatical models of channel morphology." In *Modelling Geomorphological Systems*, M.G. Anderson (Ed.), Wiley & Sons, Chichester, 99-125.

Henderson, F.M., 1966: *Open Channel Flow*. MacMillan Publishing Company, New York, 522 p.

Hickin, E.J., 1983: "River channel changes: retrospect and prospect." *Special Publications International Association of Sedimentologists,* **6**, 61-83.

Jarrett, R.D., 1984: "Hydraulics of high-gradient streams." *Journal of Hydraulic Engineering.* **110** (11), 1519-1539.

Jordan, P., 1990: *Hydrology of the November 1989 Chilliwack River flood, and some observations on the impacts of forest management.* Prepared for the B.C. Ministry of Forests, Chilliwack Forest District.

Keller, E.A., and T. Tally, 1979: "Effects of large organic debris on channel form and fluvial processes in the coastal redwood environment." In *Adjustments of the Fluvial System*, D.D. Rhodes and G.P. Williams (Eds.) Dubuque, Iowa, Kendall/Hunt: 169-197.

Kellerhals, R., 1967: "Stable channels with gravel-paved beds." *Proceedings, Journal of hte Waterways and Habours Division,* ASCE, **93** (1), 63-84.

Kellerhals, R., M. Church, and D. Bray, 1976: "Classification and analysis of river processes." *Journal of the Hydraulics Division,* **102** (7), 813-829.

Kellerhals, R., and M. Church, 1989: " The morphology of large rivers: characterization and managemnet." In *Proceedings of the International Large River Symposium*, D.P. Dodge (Ed.), Canadian Special Publications of Fish and Aquatic Sciencs, **106**, 31-48.

Keulegan, G.H., 1938: "Laws of turbulent flow in open channels." *Journal of Nat. Bur. Stand.*, 21 Res. Pap., 1151, 707-741.

Kirkby, M.J., 1977: "Maximum sediment efficiency as a criterion for alluvial channels." In *River Channel Changes* K.J. Gregory (editor), Wiley, Chichester, 429-442.

Knight, D.W., 1981: "Boundary shear in smooth and rough channels." *Journal of the Hydraulics Division*, ASCE, **107** (7), 839-851.

Knight, D.W., and Demetriou, J.D., and M.E. Hamed, 1984: "Boundary shear in smooth rectangular channels." *Journal of the Hydraulics Division,* ASCE, **110** (4), 405-422.

Lacey, G., 1930: "Stable channels in alluvium." *Proceedings of the Institute of Civil Engineers*, 29, 259-292.

Lane, E.W., 1955a: "The design of stable channels." *Trans.* ASCE, **120**, 1234-1279.

Lane, E.W., 1955b: "The importance of fluid morphology in hydraulic engineering" *Proceedings,* ASCE, **81** (746), 1-17.

Lane, S.N., and K.S. Richards, 1997: "Linking river channel form and process: time, space and causality revisited." *Earth Surface Processes and Landforms,* **22**, 249-260.

Lindley, E.S., 1919: "Regime channels" *Proceedings Punjab Engineering Conference.*

Leopold, L.B., and T.R. Maddock, 1953: "Hydraulic geometry of stream channels and some physiographic implications." *Professional Paper 252,* U.S. Geological Survey, p.

Leopold, L.B., and M.G. Wolman, 1957: "River channel patterns: braided, meandering and straight." *Professional Paper 282-B*, U.S. Geological Survey, 85 p.

Leopold, L.B., and M.G. Wolman, 1960: "River meanders." *Bulletin of the Geological Society of America,* **71**, 769-794.

Mackin, J.H., 1948: "Concept of the graded river." *Bulletin,* U.S. Geological Survey, 59, 463-512.

Mackin, J.H., 1963: "Rational and empirical methods of investigation in geology." In *The Fabric of Geology,* C.C. Albritton (editor), Freeman, Stanford, 135-163.

Miller, J.R., and J.B. Ritter, 1996: "An examination of the Rosgen classification of natural rivers." *Catena,* **27**, 295-299.

Millar, R.G., and M.C. Quick, 1993: "Effect of bank stability on geometry of gravel rivers." *Journal of Hydraulic Engineering,* ASCE, **119** (12), 1343-1363.

Millar, R.G., and M.C. Quick, 1998: "Stable geometry of gravel-bed rivers with cohesive banks." *Journal of Hydraulic Engineering.*, ASCE, *Submitted for review.*

Millar, R.G., 1998: "Meandering-Braiding Transition." *7th International Symposium on River Sedimentation,* Hong Kong.

Milner, A.M. 1994. "System Recovery." In *The Rivers Handbook: hydrological and ecological principles,* P. Calow, and G.E. Petts (Eds.), Blackwell Scientific Publications, London, **2**, 76-97.

Mollard, J.D., 1973: "Air photo interpretation of fluvial features." *Proceedings of the 7th Canadian Hydrology Symposium,* 341-380.

Moore, R.D., 1991: "Hydrology and water supply in the Fraser River basin." In *Water in Sustainable Development: Exploging our common future in the Fraser River Basin.* A. Dorsey, and J.R. Griggs (editors), Westwater Research Centre, University of British Columbia.

Nanson, G.C., and E.J. Hickin, 1983: "Channel migration and incision on the Beatton River." *Journal of Hydraulic Engineering,* ASCE, **109** (3), 327-337.

Neill, C.R., 1971: "River bed transport related to meander migration rates." *Journal of the Waterways and Harbours Division,* ASCE, **97** (4), 783-

Newbury, R., and M. Gaboury, 1993: "Exploration and rehabilitation of hydraulic habitats in stream using principles of fluvial behaviour." *Freshwater Biology*, **29**, 195-210.

Northwest Hydraulics Consultants Ltd., 1994: *Impact of Forest Harvesting on Terrain Stability, Stream Channel Morphology, and Fisheries Resources of the San Juan Watershed, Vancouver Island.* Prepared for Ministry of Environment, Lands and Parks, 66 p.

Parker, G., 1976: "On the cause and characteristic scales of meandering and braiding in rivers." *Journal of Fluid Mechanics,* ASCE, **76** (3), 457-478.

Parker, G., 1978: "Self-formed straight rivers with equilibrium banks and mobile bed, Part 2, The gravel river." *Journal of Fluid Mechanics,* ASCE, **89** (1), 127-146.

Quick, M.C., 1974: "Mechanism for streamflow meandering." *Journal of the Hydraulics Division,* ASCE, **100** (HY6), 741-753.

Reeves, G.H., J.D. Hall, T.D. Roelofs, T.L. Hickman, and C.O. Baker, 1991: "Rehabilitating and modifying stream habitats." In *Influences of Forest and Rangeland Management on Salmonid Fishes and Their Habitats,* Special Publication 19, W.R. Meehan (Ed.) American Fisheries Society, Bethesda, Maryland, 519-557.

Roberts, R.G., and M. Church. 1986. "Sediment budget in severely disturbed watersheds, Queen Charlotte Ranges, British Columbia." *Canadian Journal of Forestry Resources,* **16** (5), 1092-1106.

Rosgen, D.L., 1994: "A classification of natural rivers." *Catena*, **22**, 169-199.

Rosgen, D.L., 1996: "A classification of natural rivers: reply to the comments be J.R. Miller and J.B. Ritter." *Catena*, **27**, 301-307.

Ryder, J.M. and Associates, 1994: *Terrain and slope stability mapping of Shovelnose Creek watershed.* Prepared for Weldwood of Canada Ltd.

Schumm, S.A., and R.W. Lichty, 1965: "Time, space and causality in geomorphology." *American Journal of Science*, **263**, 110-119.

Schumm, S.A., 1969: "River metamorphosis." *Journal of the Hydraulic Division,* ASCE, **95** (1), 255-273.

Shields, F.D., 1996: "Hydraulic and hydrologic stability." In *River Channel Restoration: Guiding Principles for Sustainable Projects,* A Brookes, and F.D. Shields Jr. (Eds.), John Wiley & Sons, Chichester, England, 23-74.

Simons, D.B., and F. Senturk, 1977: *Sediment Transport Technology.* Water Resources Publications, Fort Collins, Colorado.

Steelhead Society of British Columbia, 1996: *Watershed Restoration Program Progress Report: Squamish River, Shovelnose Creek and Ashlu Creek Restoration.* Prepared for the Watershed Restoration Program.

Stevens, M.A., D.B. Simons, and E.V. Richardson, 1975: "Nonequilibirum river form." *Journal of the Hydraulics Division,* ASCE, **101** (5), 557-566.

Strahler, A.N., 1952: "Dynamic basis of geomorphology." *Bulletin,* U.S. Geological Survey, **63**, 923-938.

Song, C.C.S., and C.T. Yang., 1986: "Comment on "Extremal Hypotheses for River Regime: An Illusion of Progress" by George A. Griffiths." *Water Resources Research,* **22** (6) 993-994.

Terrasol Environmental Consulting, 1996: *Slesse Creek Watershed Restoration Level II Deactivation.* Prepared for the Ministry of Forests, Chilliwack River District, 9 p.

Thorne, C.R., 1990: "Effects of vegetation on riverbank stability and erosion," In Thornes, J.B., Ed., *Vegetation and Erosion.,* John Wiley & Sons, Chichester, England.

Timberwest, 1997: *Riparian Overview Assessment: San Juan River.* 9 p.

Ward, B.R., and P.A. Slaney, 1993: "Habitat manipulations for the rearing of fish in British Columbia." In *Le Development du saumon Altantique au Quebec,* G. Shooner and S Asselin (editors), Quebec, p. 142-148.

Warner, R.F., 1995: "Predicting and managing channel changin in Southeast Australia." *Catena,* **25**, 403-418.

Whelan, M.A. and Associates Ltd., 1996: *Chilliwack Watershed Stream Inventory and Level 1 Fish Habitat Assessment, Late Summer 1995.* Prepared for the Ministry of the Environment, Lands and Parks and the Steelhead Society Habitat Restoration Corporation, 54 p.

White, W.R., H. Milli, and A.D. Crabbe, 1975: "Sediment transport theories: a review." *Proceedings of the Institure of Civil Engineering,* **59**, 265-292.

White, W.R., R. Bettress, and E. Paris, 1982: "An analytical approach to river regime." *Journal of Hydraulic Engineering,* ASCE, **108** (10), 1179-1193.

Whittaker J.G., and M.N.R. Jaegi, 1982: "Origin of step-pool systems in mountain streams." *Journal of the Hydraulics Division,* ASCE, **108**, 758-773.

Williams, G.P., 1986: "River meanders and channel size." *Journal of Hydrology,* **88**, 147-164.

Wolman, M.G., 1954: "A method of sampling coarse river bed material." *Transactions of the American Geophysical Union,* **35** (6), 951-956.

Wolman, M.G., and L.M. Brush Jr., 1961: "Factors controlling the size and shape of stream channels in coarse noncohisive sands." *Professional Paper 282-G,* U.S. Geological Survey, 180-210.

Yalin, M.S., 1992: *River Mechanics.* Pergamon Press, Oxford, UK.

Yang, C.T., 1976: "Minimum unit stream power and fluvial hydraulics." *Journal of the Hydraulics Division*, ASCE, **102** (7), 919-934.

Yang, C.T., and C.C.S. Song, 1979: "Theory of minimum rate of energy dissipation." *Journal of the Hydraulics Division*, ASCE, **105** (7), 769-784.

Yang, C.T., 1984: "Unit stream power equation for gravel." *Journal of the Hydraulics Division*, ASCE, **110** (12), 1783-1797.

Yang, C.T., 1987: "Energy dissipation rate approach in river mechanics." In *Sediment Transport in Gravel-bed Rivers,* C.R. Thorne, J.C. Bathurst, and R.D. Hey (Eds.), Wiley & Sons, Chichester, 735-766.

APPENDIX A

SLESSE CREEK

APPENDIX A.1 SLESSE CREEK - AIRPHOTOS

Air photos used in this study are listed in the table below.

Slesse Creek Airphotos

Year	Roll Number	Picture Numbers	Scale
1936	BC207	54-56	1:22,200
1973	BC(C)87	262-264	1:19,050
1993	BCB93026	76-78	1:17,650

APPENDIX A.2

Input Data and Reach Analysis
Slesse Creek
Reach D

Data Collected
Mar 18 -19, 1998

By
Bruce MacVicar
and Dave Strajt

Summary Data

W =	41	m
Y =	1.4	m
S =	0.021	

Pbed =	35.0	m
Pbank =	6.6	m
Ybed =	1.51	m
Rh =	1.3	m

	overall	1960-77	1978-95	
Q =	92	67	117	m^3/s

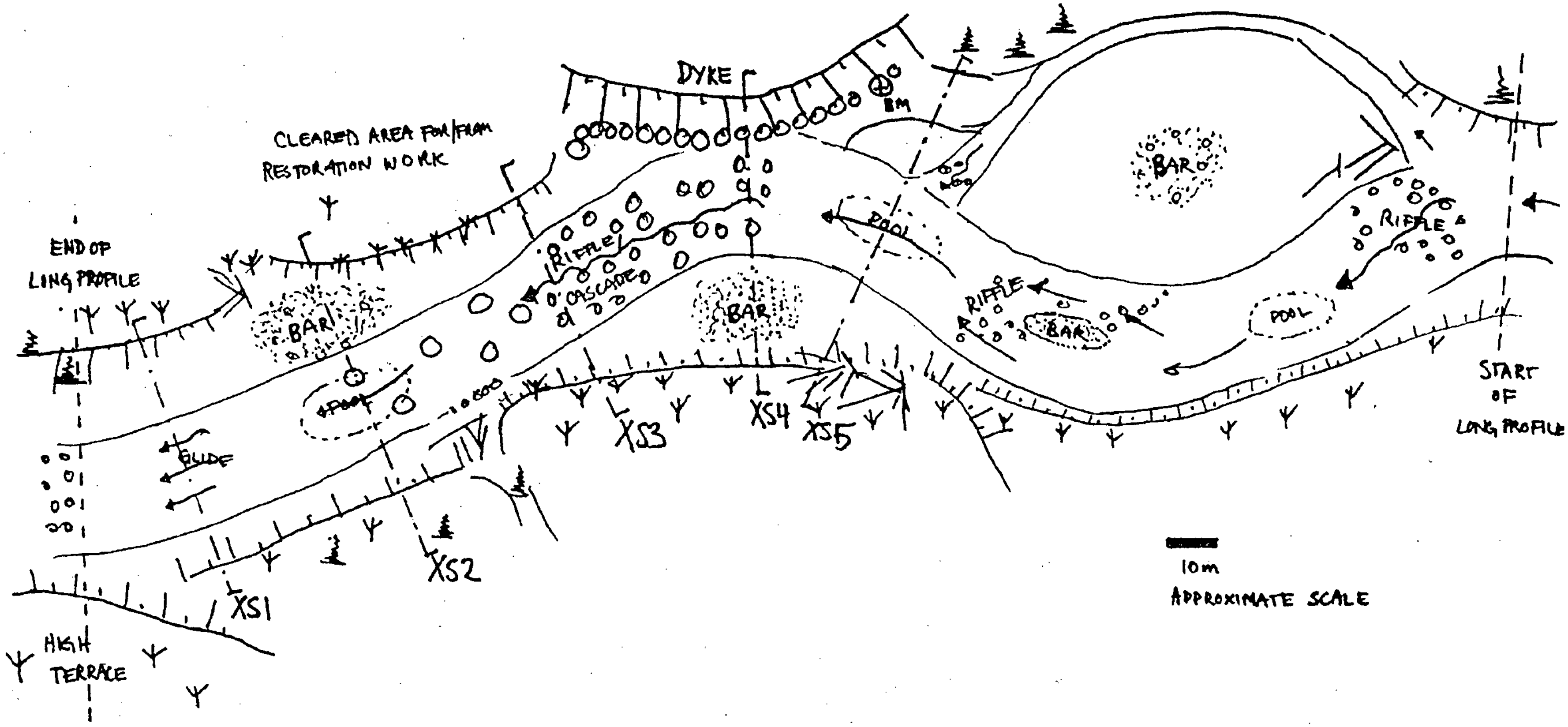
	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	0.11	0.13	0.19	0.31	0.34	m

D50bank =	0.13	m
ks =	3.6	m

	E-B	
Gb =	1600	kg/s

	mature alder	young alder	
ϕ' =	57	40	degrees

SLESSE CREEK - REACH D
MARCH 19/1998

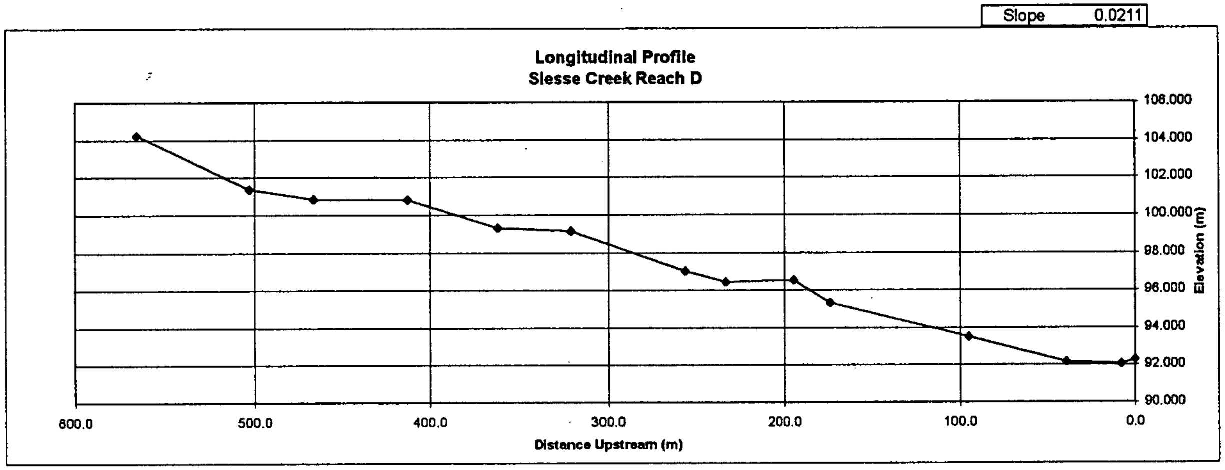


Longitudinal Profile - Slesse Creek, Reach #1

		BS	HI	FS	ELEV	
BM		0.585	100.585		100.000	
TP				2.400		
	2	3.149	101.334		98.185	
				0.088		
	3	2.885	104.131		101.246	
				0.000		
	4	2.483	106.614		104.131	
		0.604	98.789			
				3.790		
	1	-0.040	94.959		94.999	from TP2
		3.445	98.444			
				2.089		
	5	1.158	97.513		96.355	from TP1

	Elevation					Distance								
Note	TP elev	Level H	HI	Thalweg	Thalweg E	Level D	Low	Mid	High	Dist 1	Dist 2	from level	Thalweg D	
	94.999	-0.040	94.959	2.676	92.283	8						-8.0	0.0	
XS5, LP1	94.999	-0.040	94.959	2.911	92.048	8						0.0	8.0	
XS4	94.999	-0.040	94.959	2.820	92.139	8	0.795	0.949	1.105	15.4	15.6	31.0	39.0	
XS3	94.999	-0.040	94.959	1.469	93.490	8	1.035	1.469	1.905	43.4	43.6	87.0	95.0	
XS2	94.999	3.790	98.789	3.485	95.304	194.5							173.5	
LP2	94.999	3.790	98.789	2.277	96.512	194.5	1.585	1.689	1.795	10.4	10.6	21.0	194.5	0.0217
XS1	94.999	3.790	98.789	2.360	96.429	194.5	0.713	0.909	1.105	19.6	19.6	39.2	233.7	
	94.999	3.790	98.789	1.781	97.008	194.5	1.472	1.781	2.090	30.9	30.9	61.8	256.3	
LP3	98.185	3.149	101.334	2.209	99.125	320.8							320.8	0.0207
	98.185	3.149	101.334	2.017	99.317	320.8	1.812	2.017	2.219	20.5	20.2	40.7	361.5	
	98.185	3.149	101.334	0.538	100.796	320.8		0.538	0.998		46	92.0	412.8	0.0182
	101.246	2.885	104.131	3.294	100.837	449.5	1.744	1.829	1.915	8.5	8.6	17.1	466.6	
	101.246	2.885	104.131	2.792	101.339	449.5	1.078	1.344	1.609	26.6	26.5	53.1	502.6	
	104.131	2.483	106.614	2.384	104.230	538.3	1.896	2.030	2.168	13.4	13.8	27.2	565.5	

Slope	0.0211

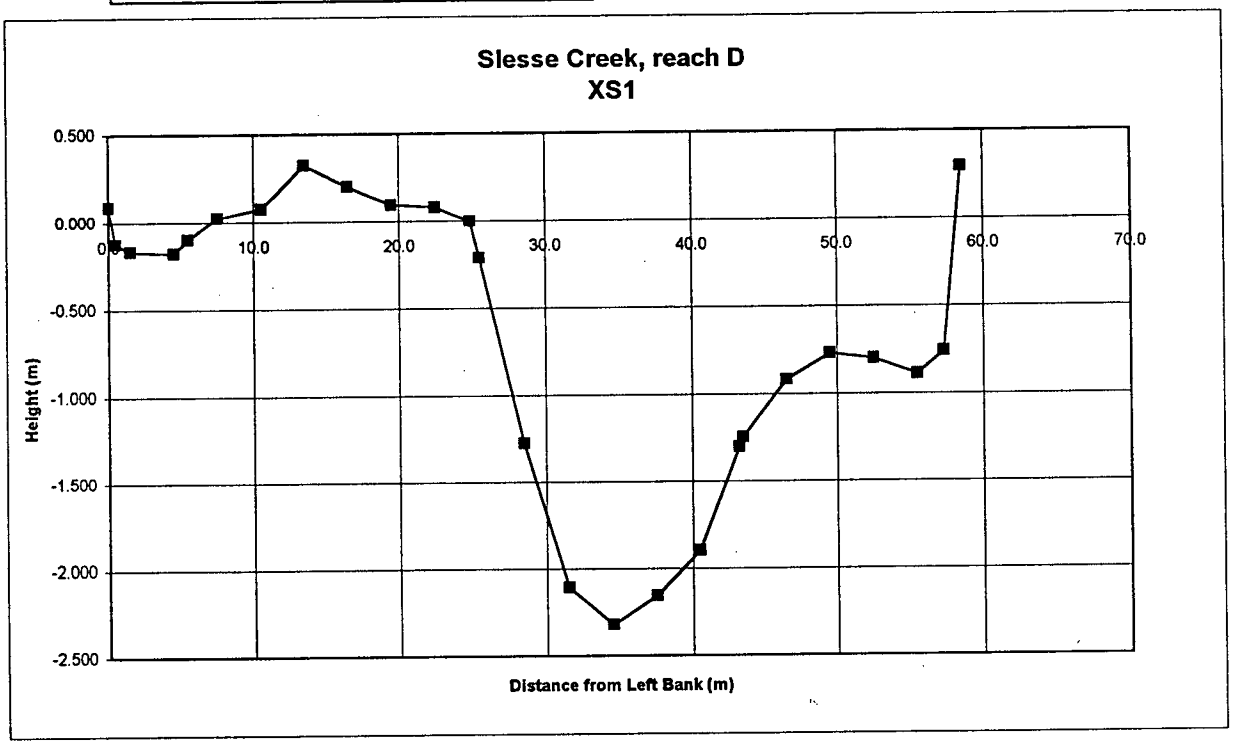


Cross Section #	1			
Level Elevation:	100.585	m	FS	from BF
Bankfull FS (BF):	1.719	m water elev (WE)	2.995	-1.276

distance (m)	FS (m)	D from LB	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
59.5	1.632	0.0	98.953	0.087			1.363			
59.0	1.845	0.5	98.740	-0.126	-0.1	0.5	1.150	0.0	0.0	0.0
58.0	1.888	1.5	98.697	-0.169	-0.2	1.0	1.107	0.0	0.0	0.0
55.0	1.899	4.5	98.686	-0.180	-0.5	3.0	1.096	0.0	0.0	0.0
54.0	1.819	5.5	98.766	-0.100	-0.1	1.0	1.176	0.0	0.0	0.0
52.0	1.694	7.5	98.891	0.025	0.0	0.0	1.301	0.0	0.0	0.0
49.0	1.643	10.5	98.942	0.076	0.0	0.0	1.352	0.0	0.0	0.0
46.0	1.394	13.5	99.191	0.325	0.0	0.0	1.601	0.0	0.0	0.0
43.0	1.517	16.5	99.068	0.202	0.0	0.0	1.478	0.0	0.0	0.0
40.0	1.624	19.5	98.961	0.095	0.0	0.0	1.371	0.0	0.0	0.0
37.0	1.638	22.5	98.947	0.081	0.0	0.0	1.357	0.0	0.0	0.0
34.6	1.719	24.9	98.866	0.000	0.0	0.0	1.276	0.0	0.0	0.0
34.0	1.933	25.5	98.652	-0.214	-0.1	0.6	1.062	0.0	0.0	0.0
31.0	2.995	28.5	97.590	-1.276	-3.8	3.0	0.000	0.0	0.0	0.0
28.0	3.825	31.5	96.760	-2.106	-6.3	3.0	-0.830	-2.5	3.0	-6.3
25.0	4.040	34.5	96.545	-2.321	-7.0	3.0	-1.045	-3.1	3.0	-7.0
22.0	3.872	37.5	96.713	-2.153	-6.5	3.0	-0.877	-2.6	3.0	-6.5
19.0	3.615	40.5	96.970	-1.896	-5.7	3.0	-0.620	-1.9	3.0	-5.7
16.3	3.018	43.2	97.567	-1.299	-3.5	2.7	-0.023	-0.1	2.7	-3.5
16.0	2.964	43.5	97.621	-1.245	-0.4	0.3	0.031	0.0	0.0	0.0
13.0	2.637	46.5	97.948	-0.918	-2.8	3.0	0.358	0.0	0.0	0.0
10.0	2.489	49.5	98.096	-0.770	-2.3	3.0	0.506	0.0	0.0	0.0
7.0	2.519	52.5	98.066	-0.800	-2.4	3.0	0.476	0.0	0.0	0.0
4.0	2.608	55.5	97.977	-0.889	-2.7	3.0	0.387	0.0	0.0	0.0
2.2	2.479	57.3	98.106	-0.760	-1.4	1.8	0.516	0.0	0.0	0.0
1.0	1.425	58.5	99.160	0.294	0.0	0.0	1.570	0.0	0.0	0.0
					45.6	37.9		10.2	14.7	28.9

Summary Data

Bankfull Width =	37.9	m
Hydraulic Mean Depth=	1.20	m
Wet Width =	14.7	m
Wet Mean Depth =	0.69	m
Ybed =	1.97	m

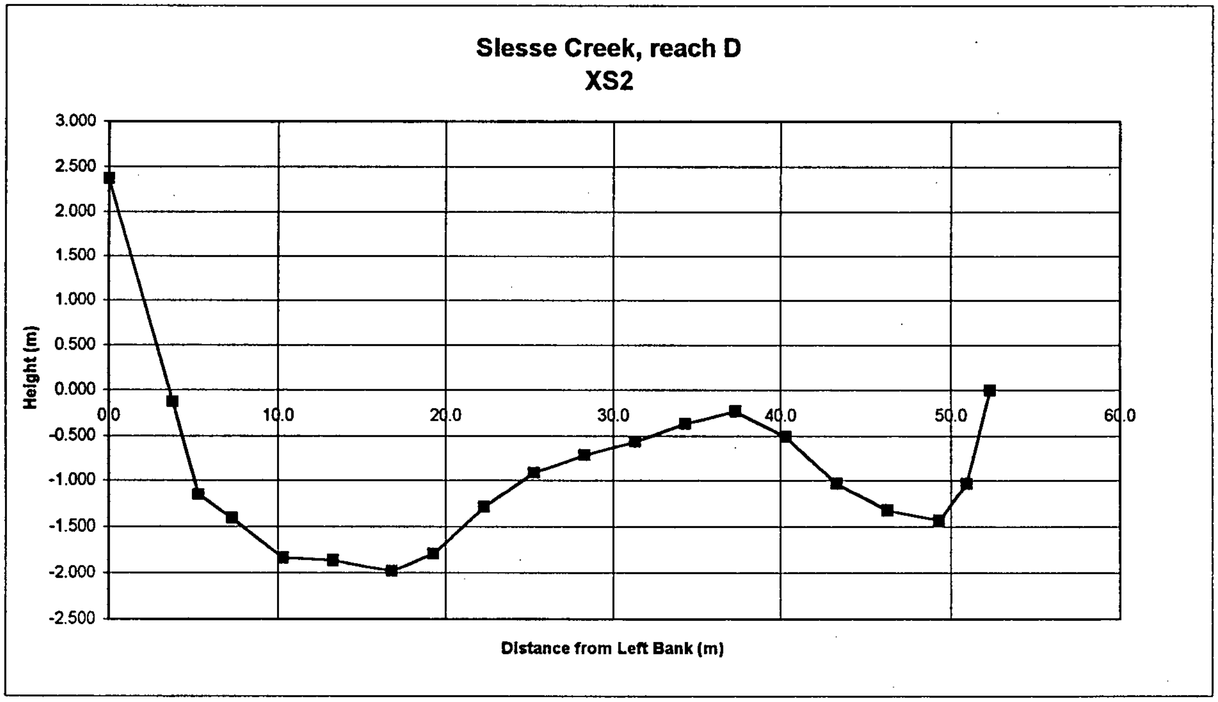


Cross Section #	2				
Level Elevation:	98.444	m		FS	from BF
Bankfull FS (BF):	1.353	m	water elev (WE)	2.508	-1.155

distance (m)	FS (m)	D from LB	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
53.3	-1.018	0.0	99.462	2.371			3.526			
49.5	1.482	3.8	96.962	-0.129	-0.5	3.8	1.026	0.0	0.0	0.0
48.0	2.508	5.3	95.936	-1.155	-1.7	1.5	0.000	0.0	0.0	0.0
46.0	2.758	7.3	95.686	-1.405	-2.8	2.0	-0.250	-0.5	2.0	-2.8
43.0	3.188	10.3	95.256	-1.835	-5.5	3.0	-0.680	-2.0	3.0	-5.5
40.0	3.215	13.3	95.229	-1.862	-5.6	3.0	-0.707	-2.1	3.0	-5.6
36.5	3.325	16.8	95.119	-1.972	-6.9	3.5	-0.817	-2.9	3.5	-6.9
34.0	3.141	19.3	95.303	-1.788	-4.5	2.5	-0.633	-1.6	2.5	-4.5
31.0	2.642	22.3	95.802	-1.289	-3.9	3.0	-0.134	-0.4	3.0	-3.9
28.0	2.262	25.3	96.182	-0.909	-2.7	3.0	0.246	0.0	0.0	0.0
25.0	2.065	28.3	96.379	-0.712	-2.1	3.0	0.443	0.0	0.0	0.0
22.0	1.918	31.3	96.526	-0.565	-1.7	3.0	0.590	0.0	0.0	0.0
19.0	1.721	34.3	96.723	-0.368	-1.1	3.0	0.787	0.0	0.0	0.0
16.0	1.585	37.3	96.859	-0.232	-0.7	3.0	0.923	0.0	0.0	0.0
13.0	1.861	40.3	96.583	-0.508	-1.5	3.0	0.647	0.0	0.0	0.0
10.0	2.388	43.3	96.056	-1.035	-3.1	3.0	0.120	0.0	0.0	0.0
7.0	2.679	46.3	95.765	-1.326	-4.0	3.0	-0.171	-0.5	3.0	-4.0
4.0	2.782	49.3	95.662	-1.429	-4.3	3.0	-0.274	-0.8	3.0	-4.3
2.4	2.387	50.9	96.057	-1.034	-1.7	1.6	0.121	0.0	0.0	0.0
1.0	1.353	52.3	97.091	0.000	0.0	0.0	1.155	0.0	0.0	0.0
					54.3	50.9		10.8	23.0	37.4

Summary

Bankfull Width =	50.9	m
Hydraulic Mean Depth=	1.07	m
Wet Width =	23.0	m
Wet Mean Depth =	0.47	m
Ybed =	1.63	m

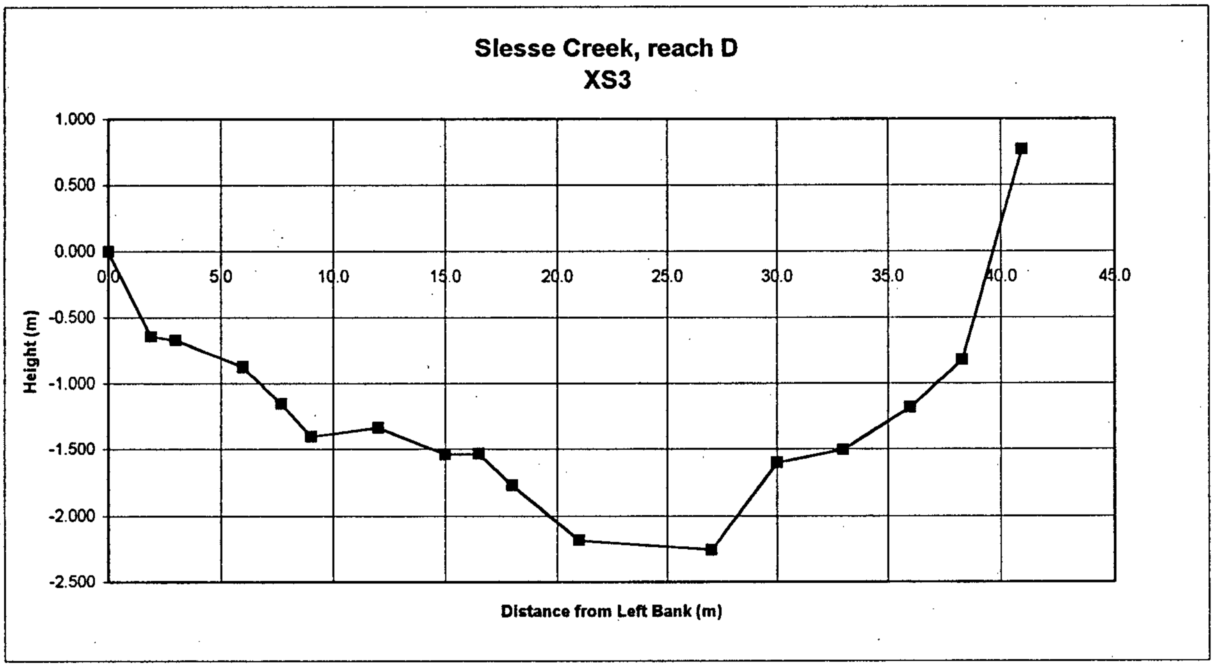


Cross Section #	3				
Level Elevation:	96.378	m		FS	from BF
Bankfull FS (BF):	0.569	m	water elev (WE)	2.101	-1.532

distance (m)	FS (m)	D from LB	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
43.0	0.569	0.0	95.809	0.000			1.532			
41.1	1.215	1.9	95.163	-0.646	-1.2	1.9	0.886	0.0	0.0	0.0
40.0	1.242	3.0	95.136	-0.673	-0.7	1.1	0.859	0.0	0.0	0.0
37.0	1.444	6.0	94.934	-0.875	-2.6	3.0	0.657	0.0	0.0	0.0
35.3	1.721	7.7	94.657	-1.152	-2.0	1.7	0.380	0.0	0.0	0.0
34.0	1.972	9.0	94.406	-1.403	-1.8	1.3	0.129	0.0	0.0	0.0
31.0	1.905	12.0	94.473	-1.336	-4.0	3.0	0.196	0.0	0.0	0.0
28.0	2.105	15.0	94.273	-1.536	-4.6	3.0	-0.004	0.0	3.0	-4.6
26.5	2.101	16.5	94.277	-1.532	-2.3	1.5	0.000	0.0	0.0	0.0
25.0	2.337	18.0	94.041	-1.768	-2.7	1.5	-0.236	-0.4	1.5	-2.7
22.0	2.754	21.0	93.624	-2.185	-6.6	3.0	-0.653	-2.0	3.0	-6.6
16.0	2.825	27.0	93.553	-2.256	-13.5	6.0	-0.724	-4.3	6.0	-13.5
13.0	2.169	30.0	94.209	-1.600	-4.8	3.0	-0.068	-0.2	3.0	-4.8
10.0	2.073	33.0	94.305	-1.504	-4.5	3.0	0.028	0.0	0.0	0.0
7.0	1.748	36.0	94.630	-1.179	-3.5	3.0	0.353	0.0	0.0	0.0
4.7	1.391	38.3	94.987	-0.822	-1.9	2.3	0.710	0.0	0.0	0.0
2.1	-0.2	40.9	96.578	0.769	0.0	0.0	2.301	0.0	0.0	0.0
					56.8	38.3		6.9	16.5	32.2

Summary Data

Bankfull Width =	38.3	m
Hydraulic Mean Depth=	1.48	m
Wet Width =	16.5	m
Wet Mean Depth =	0.42	m
Ybed =	1.95	m



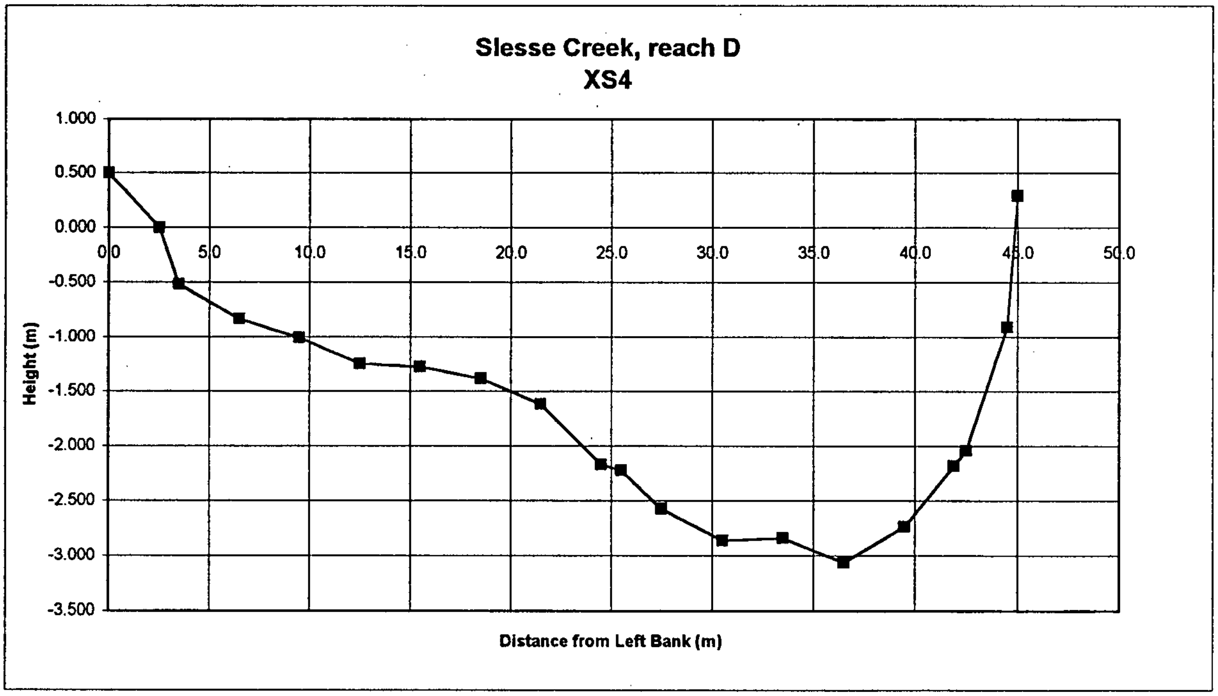
Cross Section # 4

Level Elevation:	97.513	m		FS	from BF
Bankfull FS (BF):	0.216	m	water elev (WE)	1.740	-1.524

distance (m)	FS (m)	D from LB	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
49.5	-0.284	0.0	97.797	0.500			2.024			
47.0	0.216	2.5	97.297	0.000	0.0	0.0	1.524	0.0	0.0	0.0
46.0	0.731	3.5	96.782	-0.515	-0.5	1.0	1.009	0.0	0.0	0.0
43.0	1.050	6.5	96.463	-0.834	-2.5	3.0	0.690	0.0	0.0	0.0
40.0	1.225	9.5	96.288	-1.009	-3.0	3.0	0.515	0.0	0.0	0.0
37.0	1.462	12.5	96.051	-1.246	-3.7	3.0	0.278	0.0	0.0	0.0
34.0	1.493	15.5	96.020	-1.277	-3.8	3.0	0.247	0.0	0.0	0.0
31.0	1.597	18.5	95.916	-1.381	-4.1	3.0	0.143	0.0	0.0	0.0
28.0	1.830	21.5	95.683	-1.614	-4.8	3.0	-0.090	-0.3	3.0	-4.8
25.0	2.388	24.5	95.125	-2.172	-6.5	3.0	-0.648	-1.9	3.0	-6.5
24.0	2.438	25.5	95.075	-2.222	-2.2	1.0	-0.698	-0.7	1.0	-2.2
22.0	2.784	27.5	94.729	-2.568	-5.1	2.0	-1.044	-2.1	2.0	-5.1
19.0	3.079	30.5	94.434	-2.863	-8.6	3.0	-1.339	-4.0	3.0	-8.6
16.0	3.054	33.5	94.459	-2.838	-8.5	3.0	-1.314	-3.9	3.0	-8.5
13.0	3.282	36.5	94.231	-3.066	-9.2	3.0	-1.542	-4.6	3.0	-9.2
10.0	2.951	39.5	94.562	-2.735	-8.2	3.0	-1.211	-3.6	3.0	-8.2
7.6	2.402	41.9	95.111	-2.186	-5.2	2.4	-0.662	-1.6	2.4	-5.2
7.0	2.259	42.5	95.254	-2.043	-1.2	0.6	-0.519	-0.3	0.6	-1.2
5.0	1.125	44.5	96.388	-0.909	-1.8	2.0	0.615	0.0	0.0	0.0
4.5	-0.075	45.0	97.588	0.291	0.0	0.0	1.815	0.0	0.0	0.0
					79.3	42.0		23.1	24.0	59.7

Summary Data

Bankfull Width =	42.0	m
Hydraulic Mean Depth=	1.89	m
Wet Width =	24.0	m
Wet Mean Depth =	0.96	m
Ybed =	2.49	m



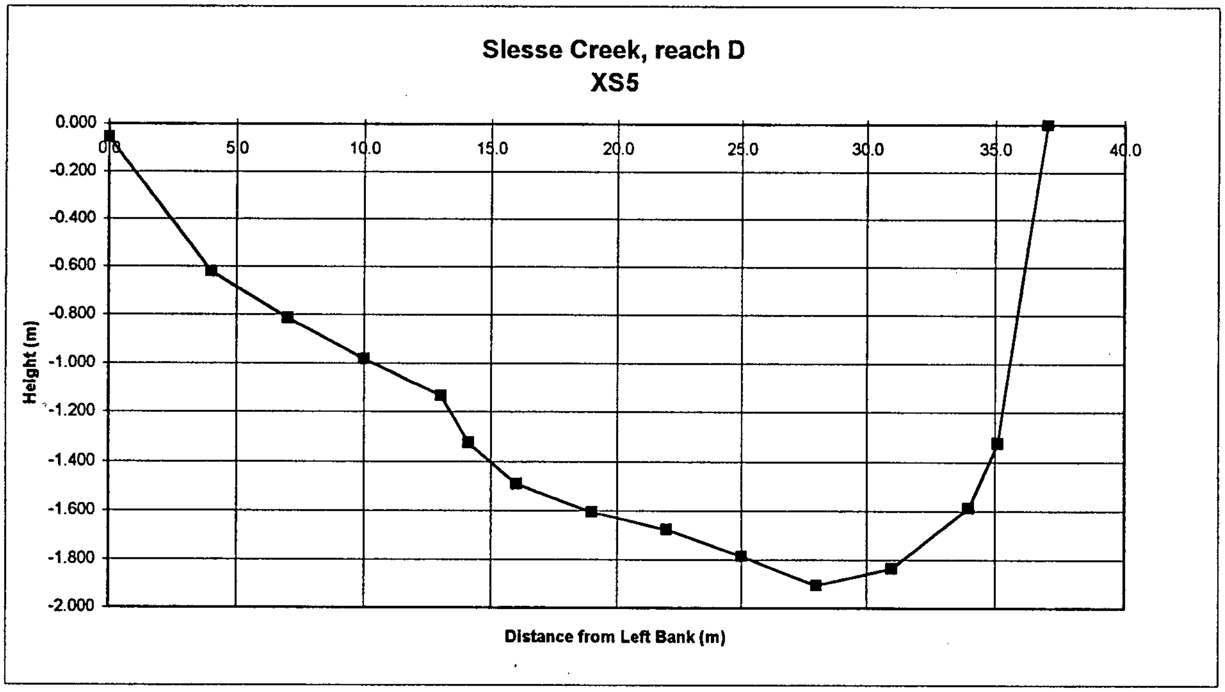
Cross Section # 5

Level Elevation:	94.959	m		FS	from BF
Bankfull FS (BF):	0.784	m	water elev (WE)	2.108	-1.324

distance (m)	FS (m)	D from LB	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
38.0	0.838	0.0	94.121	-0.054			1.270			
34.0	1.405	4.0	93.554	-0.621	-2.5	4.0	0.703	0.0	0.0	0.0
31.0	1.596	7.0	93.363	-0.812	-2.4	3.0	0.512	0.0	0.0	0.0
28.0	1.766	10.0	93.193	-0.982	-2.9	3.0	0.342	0.0	0.0	0.0
25.0	1.919	13.0	93.040	-1.135	-3.4	3.0	0.189	0.0	0.0	0.0
23.9	2.108	14.1	92.851	-1.324	-1.5	1.1	0.000	0.0	0.0	0.0
22.0	2.273	16.0	92.686	-1.489	-2.8	1.9	-0.165	-0.3	1.9	-2.8
19.0	2.388	19.0	92.571	-1.604	-4.8	3.0	-0.280	-0.8	3.0	-4.8
16.0	2.459	22.0	92.500	-1.675	-5.0	3.0	-0.351	-1.1	3.0	-5.0
13.0	2.569	25.0	92.390	-1.785	-5.4	3.0	-0.461	-1.4	3.0	-5.4
10.0	2.688	28.0	92.271	-1.904	-5.7	3.0	-0.580	-1.7	3.0	-5.7
7.0	2.618	31.0	92.341	-1.834	-5.5	3.0	-0.510	-1.5	3.0	-5.5
4.0	2.372	34.0	92.587	-1.588	-4.8	3.0	-0.264	-0.8	3.0	-4.8
2.9	2.108	35.1	92.851	-1.324	-1.5	1.1	0.000	0.0	0.0	0.0
1.0	0.784	37.0	94.175	0.000	0.0	0.0	1.324	0.0	0.0	0.0
					48.2	35.1		7.7	19.9	34.0

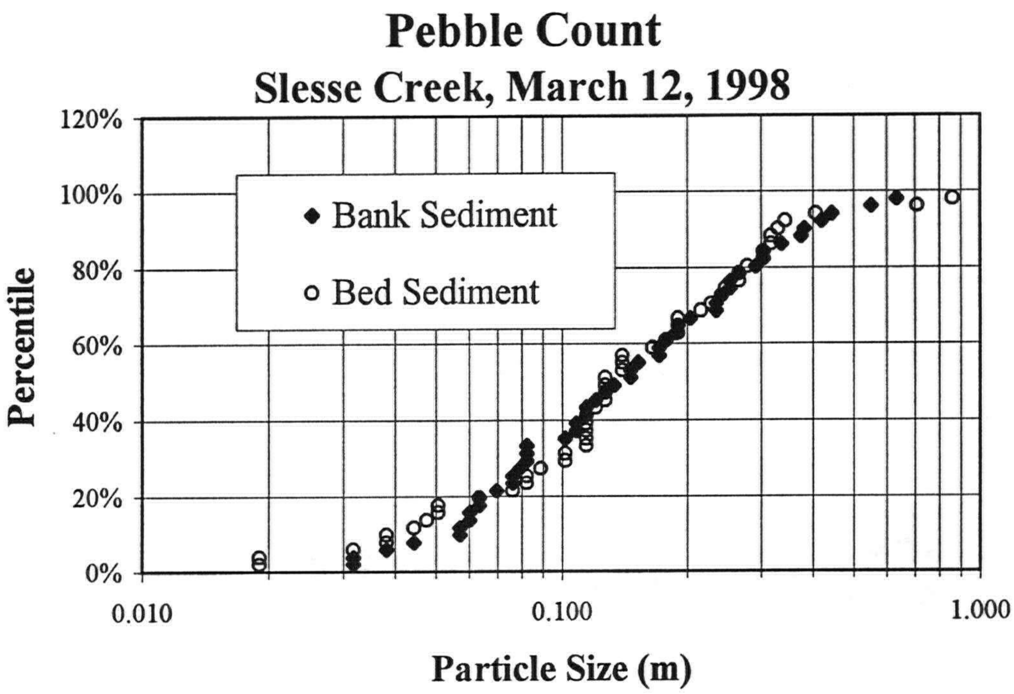
Summary Data

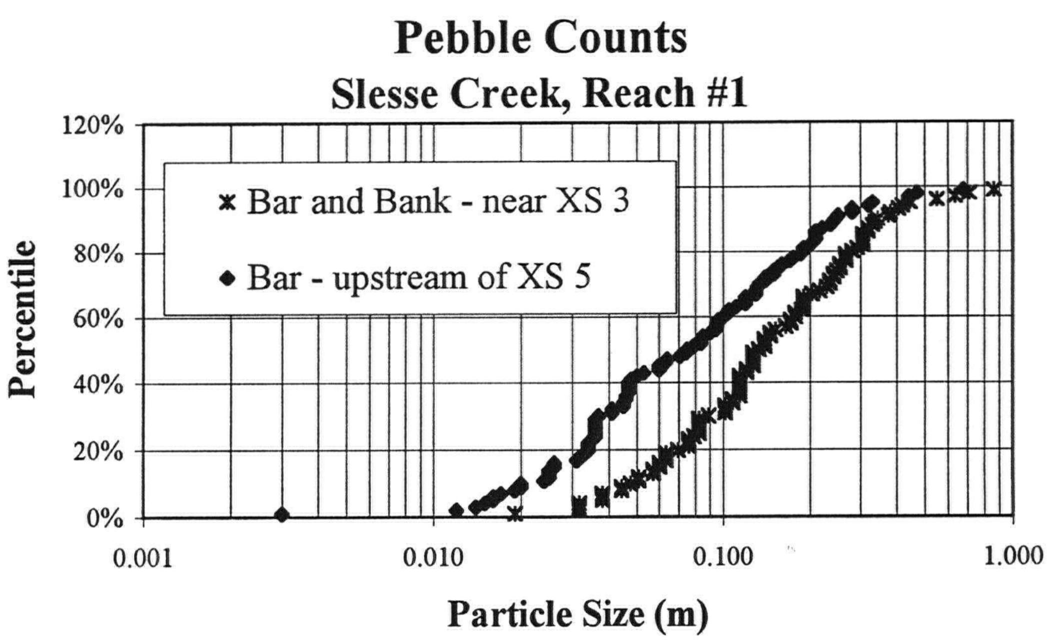
Bankfull Width =	35.1	m
Hydraulic Mean Depth=	1.37	m
Wet Width =	19.9	m
Wet Mean Depth =	0.38	m
Ybed =	1.71	m

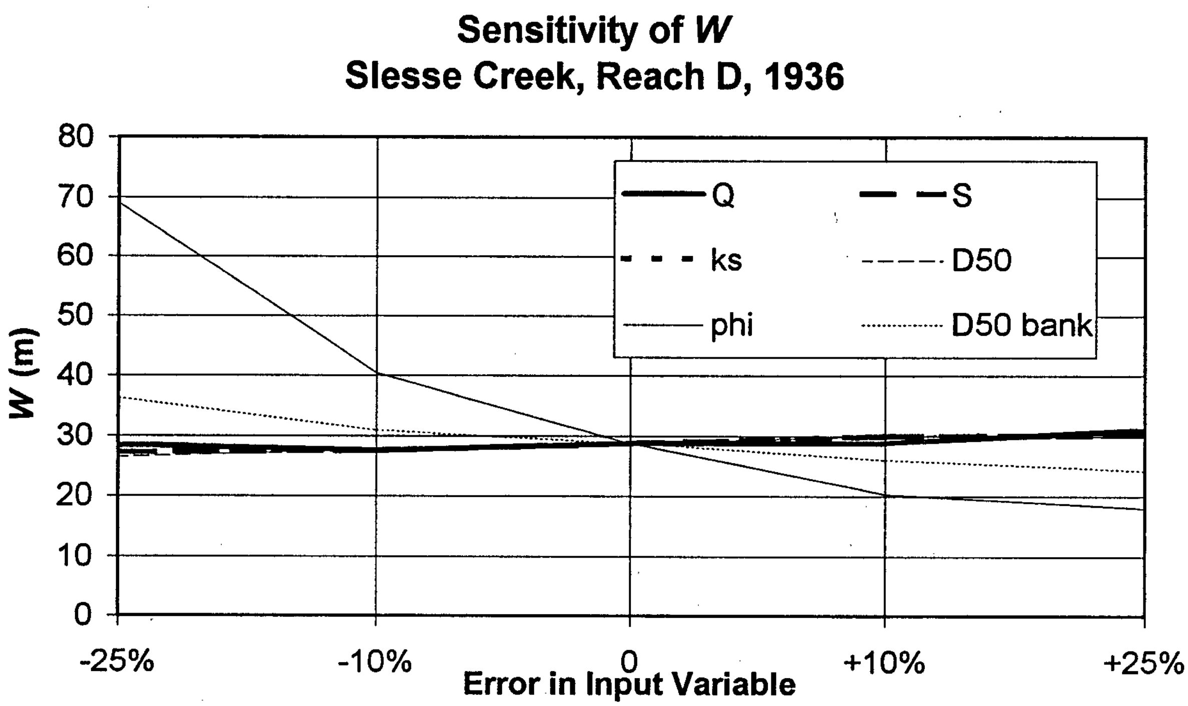


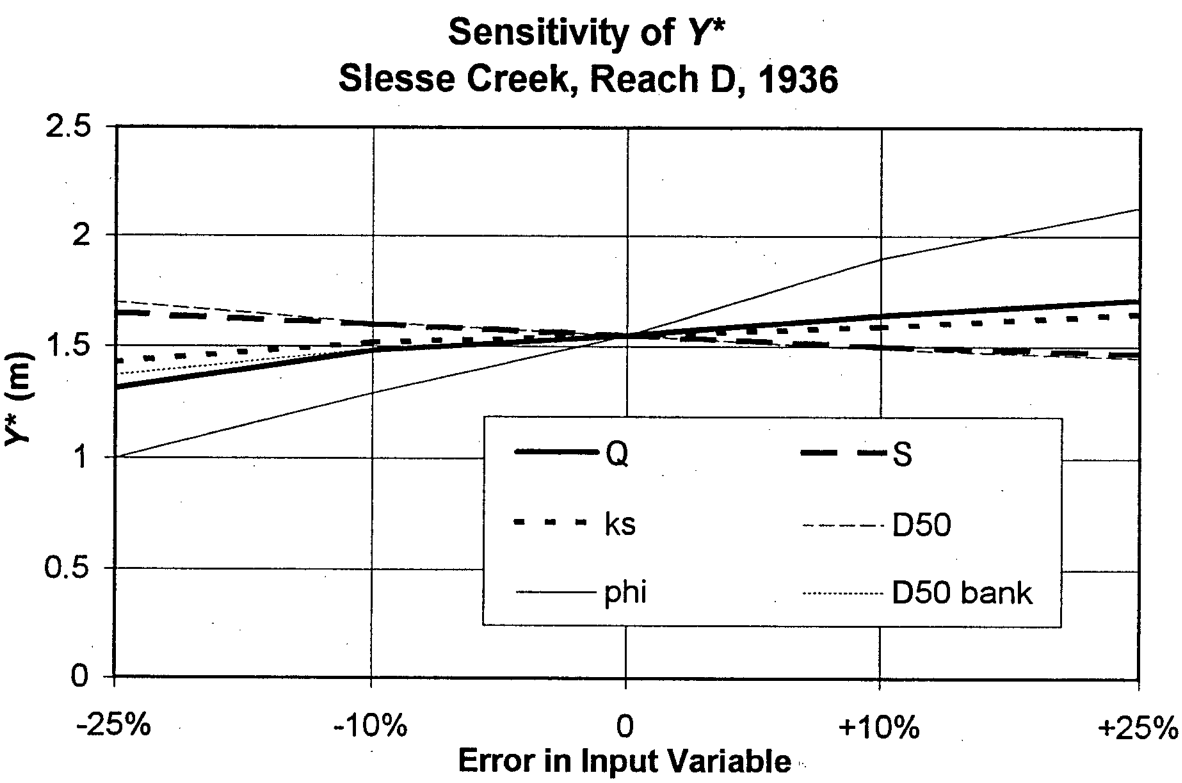
Size of Channel

X-section	Pool w/ over 1	Riffle w/chute 2	Riffle 3	Pool 4	Glide 5	Avg
Wbf	38	51	38	42	35	41
Ybf	1.20	1.07	1.48	1.89	1.37	1.40
W wet	15	23	17	24	20	20
Y wet	0.69	0.47	0.42	0.96	0.38	0.59
Ybed	1.97	1.63	1.95	2.49	1.71	1.95






Sensitivity of W
Slesse Creek, Reach D, 1936
80
70
60
50
40
30
20
10
0
W (m)
Q
S
ks
D50
phi
D50 bank
-25%
-10%
0
+10%
+25%
Error in Input Variable


Sensitivity of Y*
Slesse Creek, Reach D, 1936
2.5
2
1.5
1
0.5
0
Y* (m)
Q
S
ks
D50
phi
D50 bank
-25%
-10%
0
+10%
+25%
Error in Input Variable

APPENDIX B

SHOVELNOSE CREEK

APPENDIX B.1 SHOVELNOSE CREEK - AIRPHOTOS

Air photos are listed in the table below. Due to the small scale of the 1964 photos, no measurements were possible.

Shovelnose Creek Airphotos

Year	Roll Number	Picture Numbers	Scale
1964	BC5106	231-233	~ 1:40,000
1974	BC5583	233-235	1:12,700
1994	BCC94144	166-168	1:21,000

APPENDIX B.2

Input Data and Reach Analysis
Shovelnose Creek
Reach A

Data Collected
Oct 28 - Oct 31, 1997

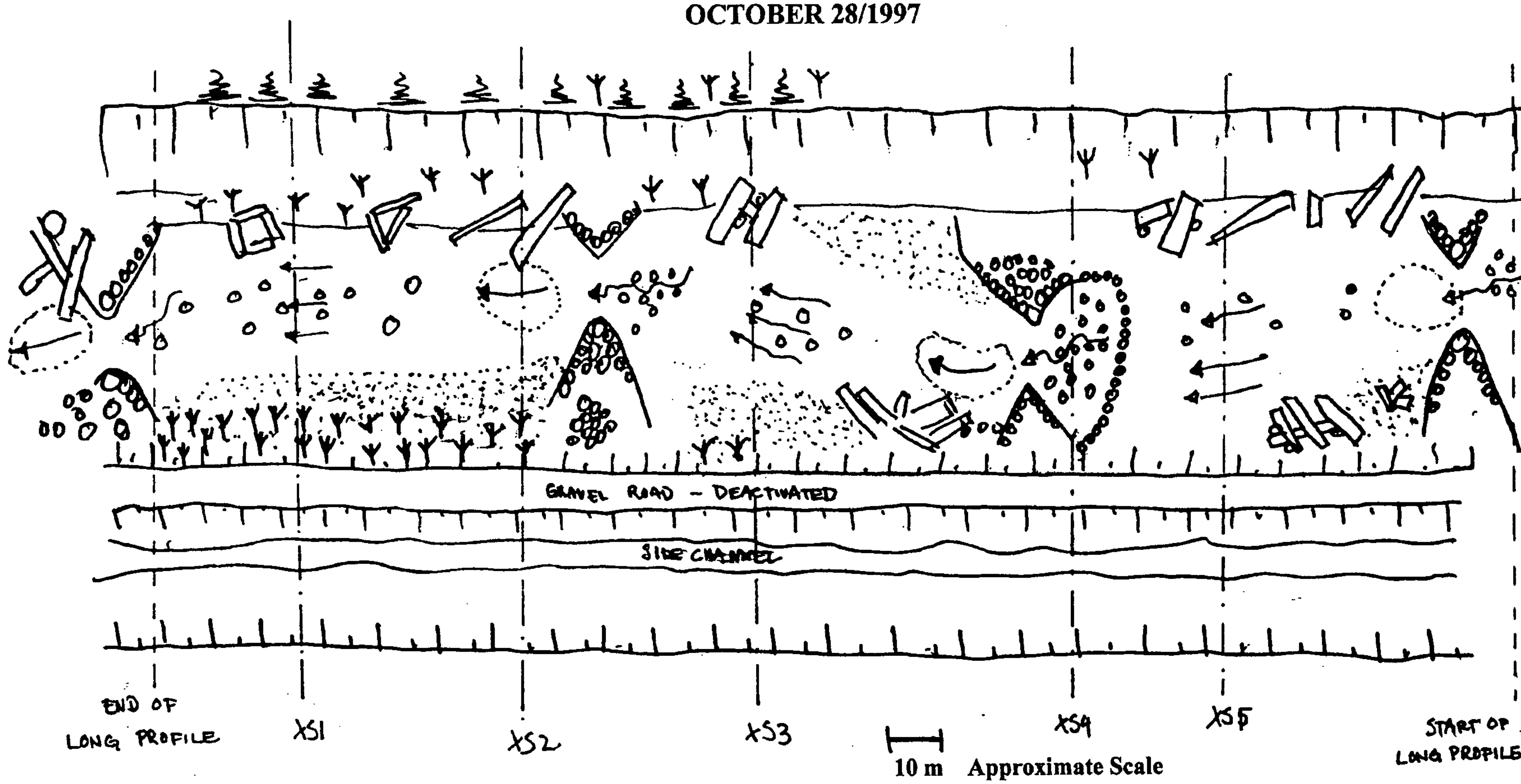
By
Bruce MacVicar
and Stephane Daoust

Summary Data

W =	36	m	Pbed =	20.0	m
Y =	0.95	m	Ybed =	1.22	m
S =	0.0045		Pbank =	16.1	m
			Rh =	0.90	m

Q =	45	m^3/s				
	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	0.077	0.115	0.149	0.292	0.340	m
D50bank =	0.265	m				
D50bulk =	0.0023	D90bulk=	0.021	m		
ks =	0.84	m				
	E-B	Brownlie				
Gb =	1.7	31.1	kg/s			

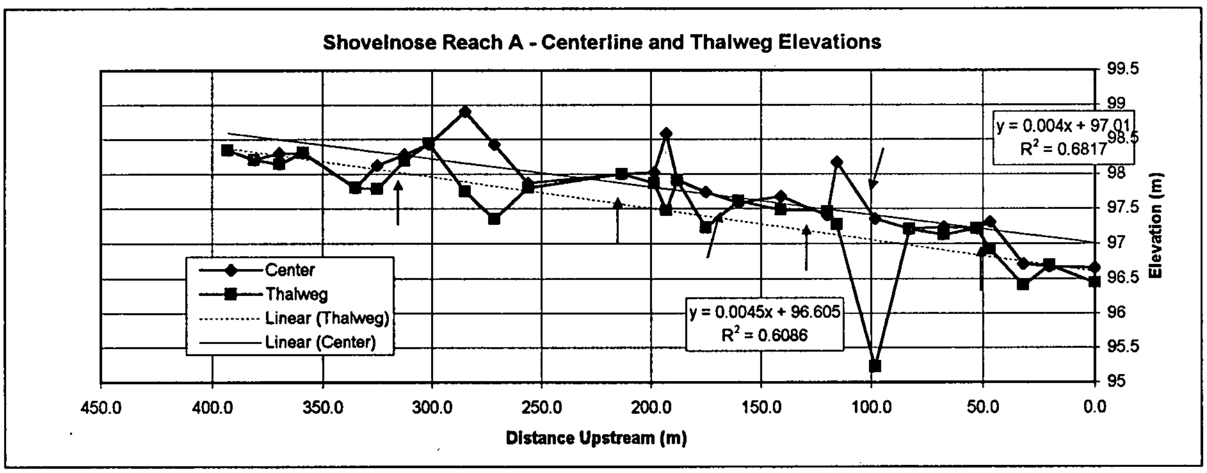
SHOVELNOSE CREEK - REACH A
OCTOBER 28/1997



Longitudinal Profile - Shovelnose Creek, Reach A

Data gathered on Oct 30/1997 by Bruce MacVicar (level) and Stephane D'aoust (rod)

High	Mid	Low	Level elevation = Thalweg	Dist to inst	dist		m Center	Thalweg		
2.455	2.260	2.065	2.260	39	393.0		98.345	98.345	BS P1#4	0.605 m
2.530	2.395	2.260	2.395	27	381.0		98.21	98.21	Hi	100.605 m
2.383	2.305	2.227	2.460	15.6	369.6		98.3	98.145		
2.325	2.300	2.275	2.300	5	359.0		98.305	98.305		
2.900	2.800	2.710	2.800	-19	335.0		97.805	97.805		
2.630	2.480	2.340	2.815	-29	325.0		98.125	97.79		
2.535	2.325	2.120	2.420	-41.5	312.5		98.28	98.185		
2.440	2.180	1.915	2.160	-52.5	301.5		98.425	98.445		
2.040	1.700	1.350	2.860	-69	285.0	dyke	98.905	97.745		
2.590	2.180	1.765	3.250	-82.5	271.5		98.425	97.355		
3.230	2.740	2.250	2.805	-98	256.0		97.865	97.8		
1.990	1.830	1.665	1.830	32.5	213.5		97.995	97.995	FS rock	1.77
1.895	1.805	1.720	1.960	17.5	198.5		98.020	97.865	BS	0.990 m
1.305	1.245	1.185	2.350	12	193.0	dyke	98.580	97.475	Hib	99.825 m
1.945	1.910	1.875	1.910	7	188.0		97.915	97.915		
2.115	2.085	2.055	2.600	-6	175.0		97.740	97.225		
2.350	2.245	2.140	2.210	-21	160.0		97.580	97.615		
2.345	2.145	1.945	2.340	-40	141.0		97.680	97.485		
2.720	2.420	2.110	2.360	-61	120.0		97.405	97.465		
1.980	1.650	1.325	2.550	-65.5	115.5	dyke	98.175	97.275		
2.890	2.470	2.065	4.600	-82.5	98.5		97.355	95.225		
3.110	2.620	2.130	2.620	-98	83.0		97.205	97.205		
3.160	2.590	2.025	2.700	-113.5	67.5		97.235	97.125		
3.250	2.610	1.970	2.610	-128	53.0		97.215	97.215		
3.190	2.520	1.850	2.900	-134	47.0	dyke	97.305	96.925		
2.875	2.800	2.725	3.100	-15	32.0		96.710	96.410	FS on SD	2.52
2.970	2.835	2.705	2.810	-26.5	20.5		96.675	96.700	BS	2.205 m
3.060	2.850	2.590	3.065	-47	0		96.660	96.445	Hic	99.510 m



Slope C/L = 0.0040

Thalweg = 0.0045

Discussion

C/L slope is influenced by going over the point deflectors. Thalweg profile is incomplete due to inability to measure pool depths in most cases, but provides good average slope in intermediate sections.

Conclusion:

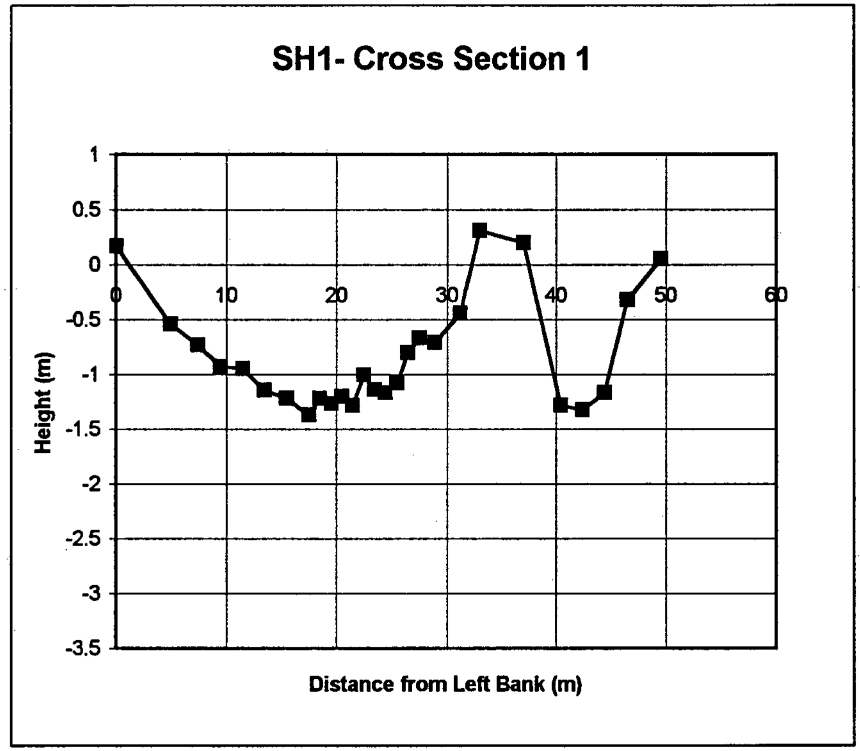
So = 0.0045

Cross Section # 1

Level Elevation: 100.665 m — Located approx 10 m upstream of bridge (right bank looking upstream)

Bankfull reading: 1.03 m — water elev 1.871

distance (	Data reading (	D from LB	elevation	from BF	Y*W
-3	0.975	49.5	99.69	0.055	
0	1.35	46.5	99.315	-0.32	0.96
2	2.2	44.5	98.465	-1.17	2.34
4	2.355	42.5	98.31	-1.325	2.65
6	2.315	40.5	98.35	-1.285	2.57
9.5	0.83	37	99.835	0.2	-0.7
13.5	0.72	33	99.945	0.31	-1.24
15.3	1.47	31.2	99.195	-0.44	0.792
17.6	1.74	28.9	98.925	-0.71	1.633
19	1.7	27.5	98.965	-0.67	0.938
20	1.835	26.5	98.83	-0.805	0.805
21	2.11	25.5	98.555	-1.08	1.08
22	2.2	24.5	98.465	-1.17	1.17
23	2.17	23.5	98.495	-1.14	1.14
24	2.035	22.5	98.63	-1.005	1.005
25	2.315	21.5	98.35	-1.285	1.285
26	2.235	20.5	98.43	-1.205	1.205
27	2.3	19.5	98.365	-1.27	1.27
28	2.25	18.5	98.415	-1.22	1.22
29	2.4	17.5	98.265	-1.37	1.37
31	2.25	15.5	98.415	-1.22	2.44
33	2.175	13.5	98.49	-1.145	2.29
35	1.975	11.5	98.69	-0.945	1.89
37	1.96	9.5	98.705	-0.93	1.86
39	1.765	7.5	98.9	-0.735	1.47
41.5	1.57	5	99.095	-0.54	1.35
46.5	0.86	0	99.805	0.17	-0.85
					31.943



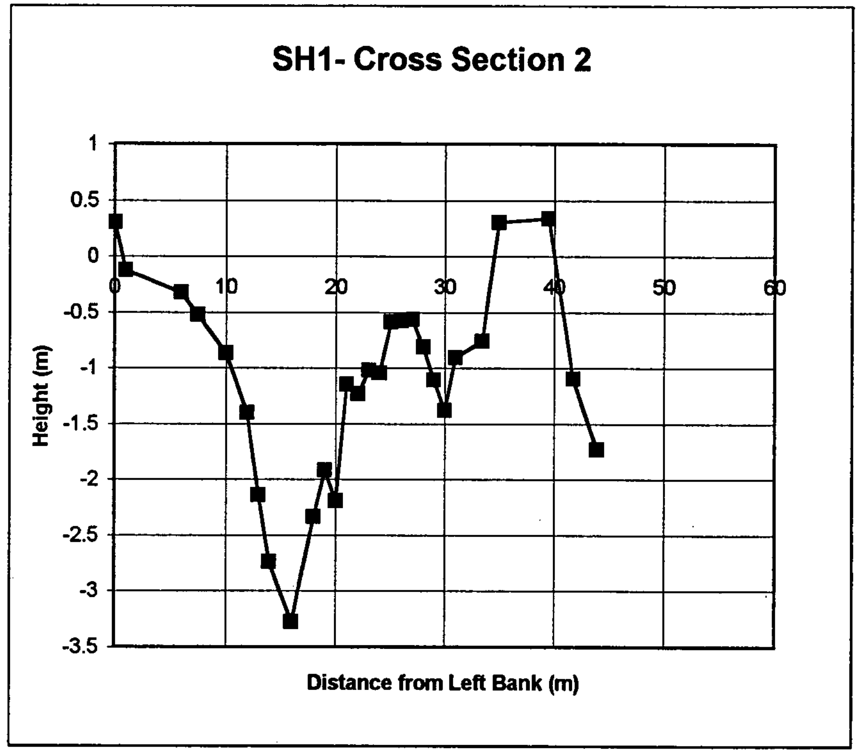
Bankfull Width =	30.5 m
Bankfull Depth =	0.86 m
Road Width =	33 m
Road Depth =	1.08 m
Pbed =	19.4 m
Ybed =	1.17 m

Cross Section # 2

Level Elevation: 101.578 m — located 5 to 10 m downstream of bridge (right side looking upstream)

Bankfull reading: 1.22 m — water elev 2.05

Data distance (	reading (	D from LB	elevation	from BF	Y*W
-6	0.915	0	100.663	0.305	
-5	1.345	1	100.233	-0.125	-0.125
0	1.545	6	100.033	-0.325	-1.625
1.5	1.745	7.5	99.833	-0.525	-0.7875
4	2.09	10	99.488	-0.87	-2.175
6	2.625	12	98.953	-1.405	-2.81
7	3.36	13	98.218	-2.14	-2.14
8	3.96	14	97.618	-2.74	-2.74
10	4.5	16	97.078	-3.28	-6.56
12	3.555	18	98.023	-2.335	-4.67
13	3.14	19	98.438	-1.92	-1.92
14	3.415	20	98.163	-2.195	-2.195
15	2.365	21	99.213	-1.145	-1.145
16	2.455	22	99.123	-1.235	-1.235
17	2.24	23	99.338	-1.02	-1.02
18	2.265	24	99.313	-1.045	-1.045
19	1.805	25	99.773	-0.585	-0.585
20	1.795	26	99.783	-0.575	-0.575
21	1.785	27	99.793	-0.565	-0.565
22	2.03	28	99.548	-0.81	-0.81
23	2.33	29	99.248	-1.11	-1.11
24	2.6	30	98.978	-1.38	-1.38
25	2.13	31	99.448	-0.91	-0.91
27.5	1.98	33.5	99.598	-0.76	-1.9
29	0.915	35	100.663	0.305	0.4575
33.5	0.88	39.5	100.698	0.34	1.53
35.8	2.315	41.8	99.263	-1.095	-2.5185
38	2.95	44	98.628	-1.73	
39.9	2.83				-40.5585
42	1.79				
44.5	0.525				



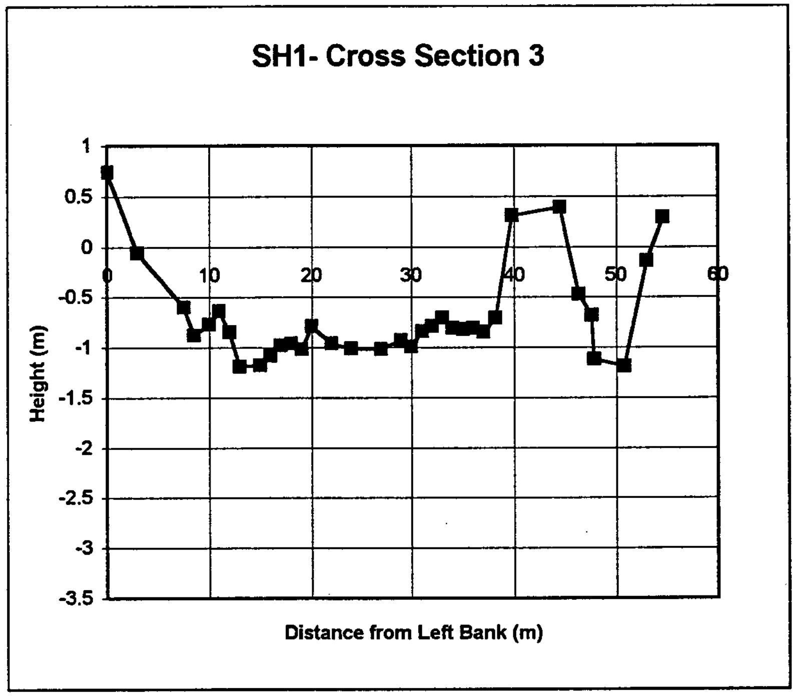
Bankfull Width =	33.7 m
Bankfull Depth =	1.19 m
Road Width =	35 m
Road Depth =	1.44 m
Pbed =	14 m
Ybed =	2.12 m

Cross Section # 3

Level Elevation: 101.578 m — Located approx 50m downstream of 1st set of wing deflectors (downstream of bridge)

Bankfull reading: 1.97 m — water elev 2.57

Data distance (	reading (	D from LB	elevation	from BF	Y*W
-4	1.228	0	100.35	0.742	
-1	2.031	3	99.547	-0.061	-0.183
3.5	2.57	7.5	99.008	-0.6	-2.7
4.5	2.85	8.5	98.728	-0.88	-0.88
6	2.74	10	98.838	-0.77	-1.155
7	2.61	11	98.968	-0.64	-0.64
8	2.82	12	98.758	-0.85	-0.85
9	3.16	13	98.418	-1.19	-1.19
11	3.15	15	98.428	-1.18	-2.36
12	3.05	16	98.528	-1.08	-1.08
13	2.95	17	98.628	-0.98	-0.98
14	2.93	18	98.648	-0.96	-0.96
15	2.99	19	98.588	-1.02	-1.02
16	2.76	20	98.818	-0.79	-0.79
18	2.93	22	98.648	-0.96	-1.92
20	2.98	24	98.598	-1.01	-2.02
23	2.99	27	98.588	-1.02	-3.06
25	2.9	29	98.678	-0.93	-1.86
26	2.96	30	98.618	-0.99	-0.99
27	2.81	31	98.768	-0.84	-0.84
28	2.76	32	98.818	-0.79	-0.79
29	2.68	33	98.898	-0.71	-0.71
30	2.78	34	98.798	-0.81	-0.81
31	2.79	35	98.788	-0.82	-0.82
32	2.78	36	98.798	-0.81	-0.81
33	2.82	37	98.758	-0.85	-0.85
34.2	2.68	38.2	98.898	-0.71	-0.852
35.8	1.66	39.8	99.918	0.31	0.496
40.5	1.58	44.5	99.998	0.39	1.833
42.4	2.44	46.4	99.138	-0.47	-0.893
43.6	2.65	47.6	98.928	-0.68	-0.816
43.9	3.09	47.9	98.488	-1.12	-0.336
46.8	3.16	50.8	98.418	-1.19	-3.451
49	2.11	53	99.468	-0.14	-0.308
50.5	1.68	54.5	99.898	0.29	0.435



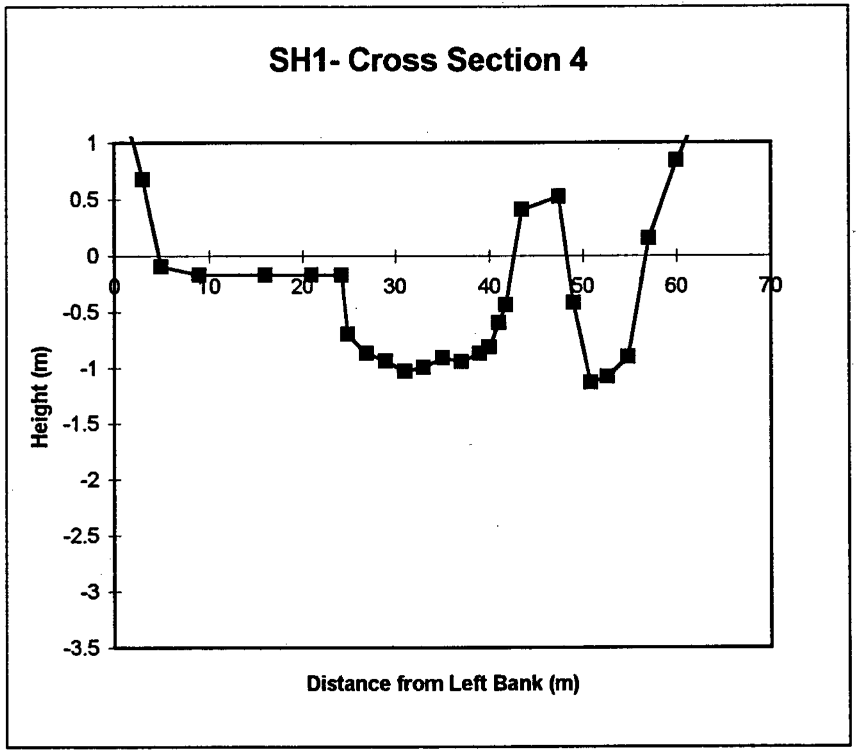
Bankfull Width = 36.4 m
Bankfull Depth = 0.85 m
Road Width = 38 m
Road Depth = 1.30 m
Pbed = 25 m
Ybed = 0.99 m

Cross Section # 4

Level Elevation: 100.255 m — Located approx 50m downstream of 1st set of wing deflectors (downstream of bridge)

Bankfull reading: 2.11 m — water elev 1.74 -1.74

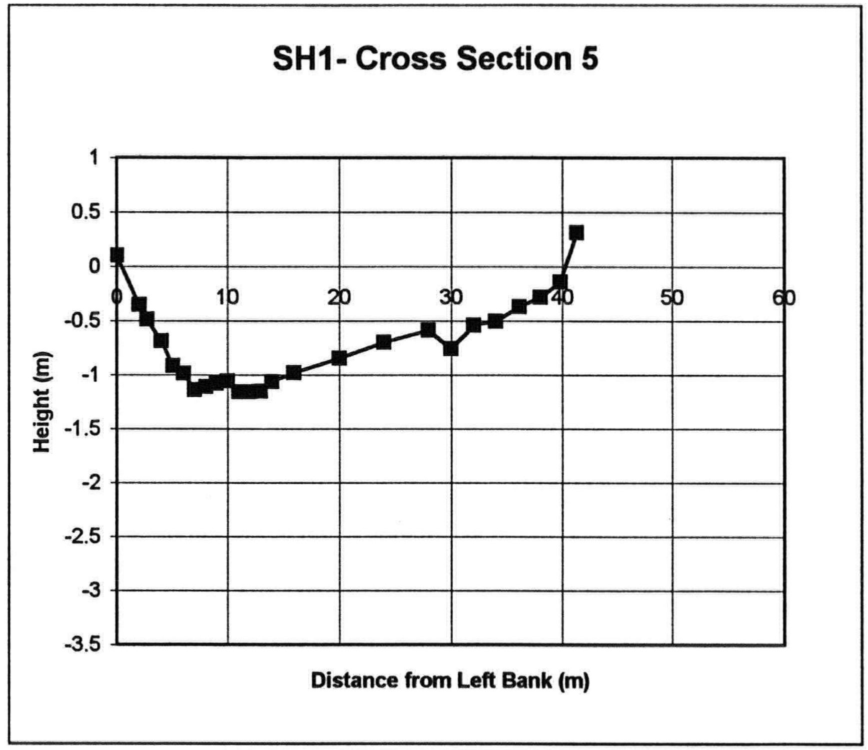
Data distance (	reading (	D from LB	elevation	from BF	Y*W
1	0.73	63	99.525	1.38	
4	1.27	60	98.985	0.84	-2.52
7	1.96	57	98.295	0.15	-0.45
9.2	3.015	54.8	97.24	-0.905	1.991
11.4	3.195	52.6	97.06	-1.085	2.387
13.2	3.245	50.8	97.01	-1.135	2.043
15	2.53	49	97.725	-0.42	0.756
16.6	1.59	47.4	98.665	0.52	-0.832
20.6	1.7	43.4	98.555	0.41	-1.64
22.3	2.55	41.7	97.705	-0.44	0.748
23	2.71	41	97.545	-0.6	0.42
24	2.925	40	97.33	-0.815	0.815
25	2.98	39	97.275	-0.87	0.87
27	3.053	37	97.202	-0.943	1.886
29	3.02	35	97.235	-0.91	1.82
31	3.105	33	97.15	-0.995	1.99
33	3.14	31	97.115	-1.03	2.06
35	3.05	29	97.205	-0.94	1.88
37	2.98	27	97.275	-0.87	1.74
39	2.81	25	97.445	-0.7	1.4
39.7	2.28	24.3	97.975	-0.17	0.119
43	2.28	21	97.975	-0.17	0.561
48	2.28	16	97.975	-0.17	0.85
55	2.28	9	97.975	-0.17	1.19
59	2.21	5	98.045	-0.1	0.4
61	1.435	3	98.82	0.675	-1.35
62	1.1	2	99.155	1.01	-1.01
64	0.63	0	99.625	1.48	-2.96



Bankfull Width =	38.5 m
Bankfull Depth =	0.49 m
Road Width =	41 m
Road Depth =	0.83 m
Pbed =	16 m
Ybed =	0.93 m

Cross Section # 5
Level Elevation: 100.965 m — Located approx 45m downstream of second set of wing deflectors (downstream of bridge)
Bankfull reading: 1.16 m — water elev 1.59

Data distance (	reading (	D from LB	elevation	from BF	Y*W
-2	1.06	0	99.905	0.1	
0	1.51	2	99.455	-0.35	-0.7
0.7	1.65	2.7	99.315	-0.49	-0.343
2	1.85	4	99.115	-0.69	-0.897
3	2.08	5	98.885	-0.92	-0.92
4	2.15	6	98.815	-0.99	-0.99
5	2.3	7	98.665	-1.14	-1.14
6	2.27	8	98.695	-1.11	-1.11
7	2.24	9	98.725	-1.08	-1.08
8	2.22	10	98.745	-1.06	-1.06
9	2.32	11	98.645	-1.16	-1.16
10	2.32	12	98.645	-1.16	-1.16
11	2.31	13	98.655	-1.15	-1.15
12	2.225	14	98.74	-1.065	-1.065
14	2.14	16	98.825	-0.98	-1.96
18	2.005	20	98.96	-0.845	-3.38
22	1.86	24	99.105	-0.7	-2.8
26	1.75	28	99.215	-0.59	-2.36
28	1.92	30	99.045	-0.76	-1.52
30	1.7	32	99.265	-0.54	-1.08
32	1.665	34	99.3	-0.505	-1.01
34.1	1.53	36.1	99.435	-0.37	-0.777
36	1.44	38	99.525	-0.28	-0.532
37.8	1.3	39.8	99.665	-0.14	-0.252
39.3	0.85	41.3	100.115	0.31	0.465



Bankfull Width =	39 m
Bankfull Depth =	0.73 m
Road Width =	41.3 m
Road Depth =	1.14 m
Pbed =	24 m
Ybed =	0.82 m

Size of Channel

	Glide	Pool	Glide	Riffle	Glide	
X-section	1	2	3	4	5	Avg
Wbf	31	34	36	39	39	36
Ybf	0.86	1.19	0.85	0.49	0.73	0.82
Wroad	33	35	38	41	41	38
Y road	1.08	1.44	1.30	0.83	1.14	1.16
Pbed	19	14	25	16	24	20
Ybed	1.17	2.12	0.99	0.93	0.82	1.20

Discussion

Bankfull widths are consistent due to the construction of a road/berm to separate the mainstem from a constructed growndwater channel. Bankfull depths are also consistent in the glides, and depth changes in riffles and pools is evenly accounted for, giving good confidence in the result. The road depth is interesting for calculating the shear stress during floods.

Conclusions

Wbf = 36 m

Ybf = 0.82 m

1. Roughness Estimates Based on Flow and Bankfull Measurements

n =	0.048	= W*Y/Q*Rh^(2/3)*S^(1/2)
f =	0.184	= 8*9.81*(W*Y)^2*Y*S/Q^2

2. Roughness Estimates from Developed Empirical Relations

Manning's roughness			
Strickler	n =	0.029	= 0.041 D50^(1/6)
		0.032	= 0.038 D90^(1/6)
Chow	n =	0.045	- from qualitative estimate
Limerinos	n =	0.051	= (0.113*Y^(1/6))/(1.16+2.00log(Y/D84))
Bray	n =	0.040	= 0.0593 D50^0.179
		0.042	= 0.0495 D90^0.160
		0.040	= 0.104*S^0.177
Jarrett	n =	0.051	= 0.39*S^0.38*R^(-0.16)

Friction Factor			
	ks =	0.782	ks = 6.8*D50
		1.022	ks = 3.5*D84
		1.054	ks = 3.1*D90
	avg	0.953	
Keulegan	f =	0.205	= (2.21 + 2.03log(Y/ks))^(-2)
Colebrook	f =	0.206	= (2.03log(12.2*Y/ks))^(-2)
Bray	f =	0.172	= (0.248 + 2.36log(Y/D50))^(-2)
		0.202	= (1.26 + 2.16log(Y/D90))^(-2)
		0.165	= (1.36 (Y/D50)^0.281)^(-2)
		0.182	= (1.78 (Y/D90)^0.268)^(-2)
		0.124	= (- 2.32 - 2.20log(S))^(-2)
		0.130	= (0.696*S^(-0.256))^(-2)
Kellerhals	f =	0.113	= (2.30 (Y/D90)^0.25)^(-2)

3. Equivalent Roughness

ks =	0.83	=Y/10^(((f)^(-0.5)-2.21)/2.03)
		Where f value is from step 1
	0.85	Where f value is from step 2

Sediment Transport

Constants

Specific Weight =	9810	kg/m^2*s^2
Density =	1000	kg/m^3
g =	9.81	m/s^2
nu =	0.000001	m^2/s
s =	2.65	sim

Preliminary Calculations

Shear =	52	N
SFbank =	0.43	dim
bed shear =	37	N
Sheilds =	0.051	dim
Power =	63	Nm/s

Einstein-Brown

gb * =	0.001	dim
F1 =	0.82	dim
gb =	0.086	kg/ms
Gb =	1.7	kg/s

Brownlie (81)

Rg	345
Lauren	0.026
tau*o	0.044
omega	2
Fgo	1.69
q	1.50
q*	4342
Y	0.88
f	0.103
Fg calc	7.26
C	1037
gb	1.56
Gb	31.1

Input Data and Reach Analysis
Shovelnose Creek
Reach B

Data Collected
Oct 28 - Oct 31, 1997

By
Bruce MacVicar
and Stephane Daoust

Summary Data

W =	36	m	Pbed =	27.0	m
Y =	0.87	m	Ybed =	0.99	m
S =	0.0031		Pbank =	9.2	m
			Rh =	0.83	m

Q =	45	m^3/s				
	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	sand	0.007	0.071	0.169	0.27	m
D50bank =	0.265	m				
D50bulk =	0.0023	90bulk	0.021	m		
ks =	0.28	m				
	E-B	Brownlie				
Gb =	68.6	27.7	kg/s			

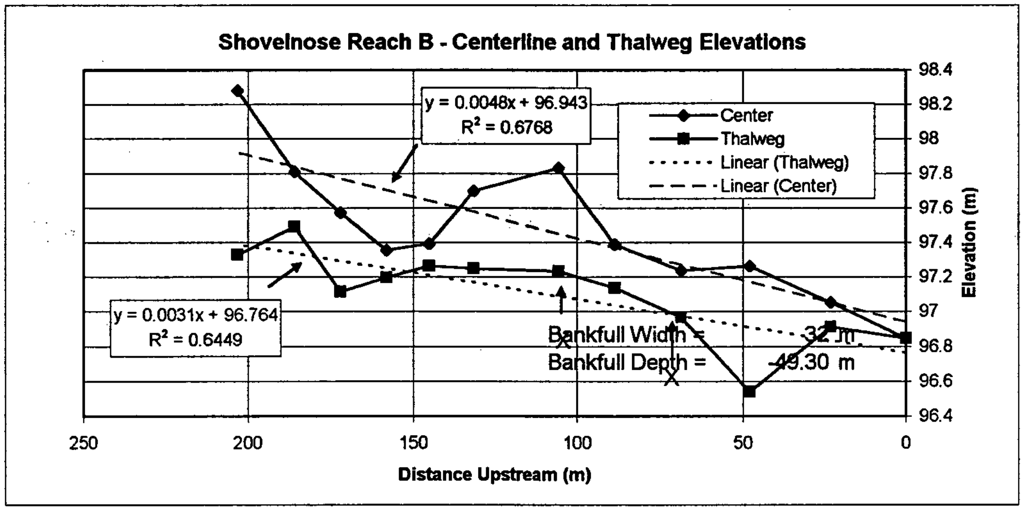
Note: Y adjusted to match discharge value due to low confidence in bankfull marks

Longitudinal Profile - Shovelnose Creek, Reach B

Data gathered on Oct 30/1997 by Bruce MacVicar (level) and Stephane D'aoust (rod)

Level elevation = 99.5 m

High	Mid	Low	Thalweg		X	Center	Thalweg
1.66	1.218	0.788	2.17	87.2	203.1	98.282	97.33
2.05	1.692	1.348	2.008	70.2	186.1	97.808	97.492
2.212	1.925	1.649	2.383	56.3	172.2	97.575	97.117
2.358	2.143	1.935	2.302	42.3	158.2	97.357	97.198
2.254	2.109	1.96	2.238	29.4	145.3	97.391	97.262
1.883	1.802	1.724	2.253	15.9	131.8	97.698	97.247
1.72	1.667	1.617	2.266	-10.3	105.6	97.833	97.234
2.247	2.111	1.975	2.362	-27.2	88.7	97.389	97.138
2.302	2.262	1.83	2.532	-47.2	68.7	97.238	96.968
2.582	2.235	1.903	2.961	-67.9	48	97.265	96.539
2.913	2.448	1.988	2.587	-92.5	23.4	97.052	96.913
3.229	2.652	2.07	2.652	-115.9	0	96.848	96.848



Slope	C/L =	0.0048
	Thalweg	0.0031

Discussion

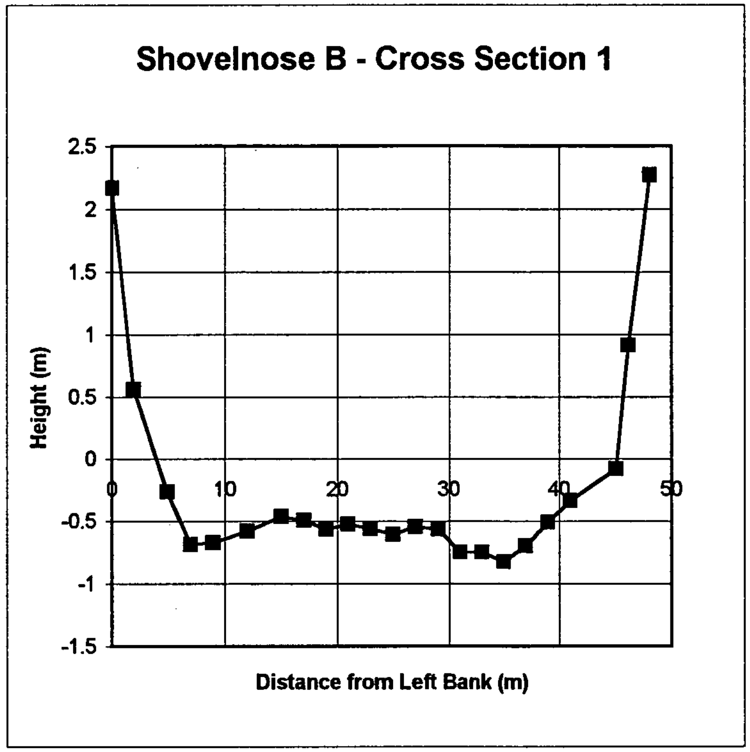
Centerline slope appears highly variable and unreliable. Thalweg slope appears to have repeating features, although the slope of the large riffle was not taken into account.

Conclusion:

So = 0.0031

Cross Section # 1
Level Elevation: 99.03 m
Bankfull reading: 1.27 m (est.)

Data					
distance	reading (	D from L	elevation	from BF	Y*W
-3	-0.9	0	99.93	2.17	
-1	0.711	2	98.319	0.559	1.118
2	1.536	5	97.494	-0.266	-0.798
4	1.957	7	97.073	-0.687	-1.374
6	1.943	9	97.087	-0.673	-1.346
9	1.852	12	97.178	-0.582	-1.746
12	1.731	15	97.299	-0.461	-1.383
14	1.763	17	97.267	-0.493	-0.986
16	1.837	19	97.193	-0.567	-1.134
18	1.795	21	97.235	-0.525	-1.05
20	1.83	23	97.2	-0.56	-1.12
22	1.875	25	97.155	-0.605	-1.21
24	1.813	27	97.217	-0.543	-1.086
26	1.834	29	97.196	-0.564	-1.128
28	2.017	31	97.013	-0.747	-1.494
30	2.017	33	97.013	-0.747	-1.494
32	2.098	35	96.932	-0.828	-1.656
34	1.966	37	97.064	-0.696	-1.392
36	1.781	39	97.249	-0.511	-1.022
38	1.603	41	97.427	-0.333	-0.666
42	1.348	45	97.682	-0.078	-0.312
43.1	0.36	46.1	98.67	0.91	1.001
45	-1	48	100.03	2.27	4.313



Bankfull Width = 41 m
Bankfull Depth = 0.55 m

Terrace Width = 48 m
Terrace Depth = 2.87 m

Pbed = 28 m
Ybed = 0.65 m

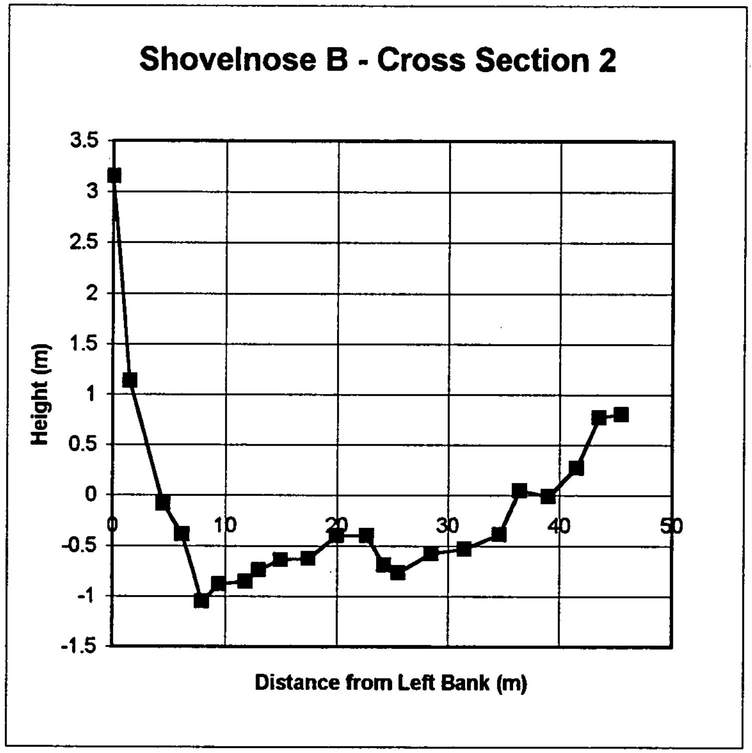
Cross Section # 2

Level Elevation: 99.5 m

Bankfull reading: 1.303 m

Data

distance	reading (	D from L	elevation	from BF	Y*W
-0.5	-1.852	0	101.352	3.155	
1	0.168	1.5	99.332	1.135	1.7025
4	1.386	4.5	98.114	-0.083	-0.249
5.7	1.695	6.2	97.805	-0.392	-0.6664
7.5	2.354	8	97.146	-1.051	-1.8918
9	2.184	9.5	97.316	-0.881	-1.3215
11.3	2.158	11.8	97.342	-0.855	-1.9665
12.5	2.047	13	97.453	-0.744	-0.8928
14.5	1.947	15	97.553	-0.644	-1.288
17	1.931	17.5	97.569	-0.628	-1.57
19.5	1.707	20	97.793	-0.404	-1.01
22.2	1.705	22.7	97.795	-0.402	-1.0854
23.7	1.995	24.2	97.505	-0.692	-1.038
25	2.068	25.5	97.432	-0.765	-0.9945
28	1.884	28.5	97.616	-0.581	-1.743
31	1.834	31.5	97.666	-0.531	-1.593
34.2	1.69	34.7	97.81	-0.387	-1.2384
36	1.257	36.5	98.243	0.046	0.0828
38.5	1.312	39	98.188	-0.009	-0.0225
41	1.028	41.5	98.472	0.275	0.6875
43	0.53	43.5	98.97	0.773	1.546
45	0.5	45.5	99	0.803	1.606



Bankfull Width = 31 m
Bankfull Depth = 0.60 m

Terrace Width = 46 m
Terrace Depth = 3.64 m

Pbed = 26.7 m
Ybed = 0.66 m

Size of Channel

	Glide	Glide		
X-section	1	2	Avg	Note
Wbf	41	31	36	
Ybf	0.55	0.60	0.57	
W terrac	48	46	47	
Y terrace	2.87	3.64	3.26	
Pbed	28	27	27	
Ybed	0.65	0.66	0.66	

Discussion

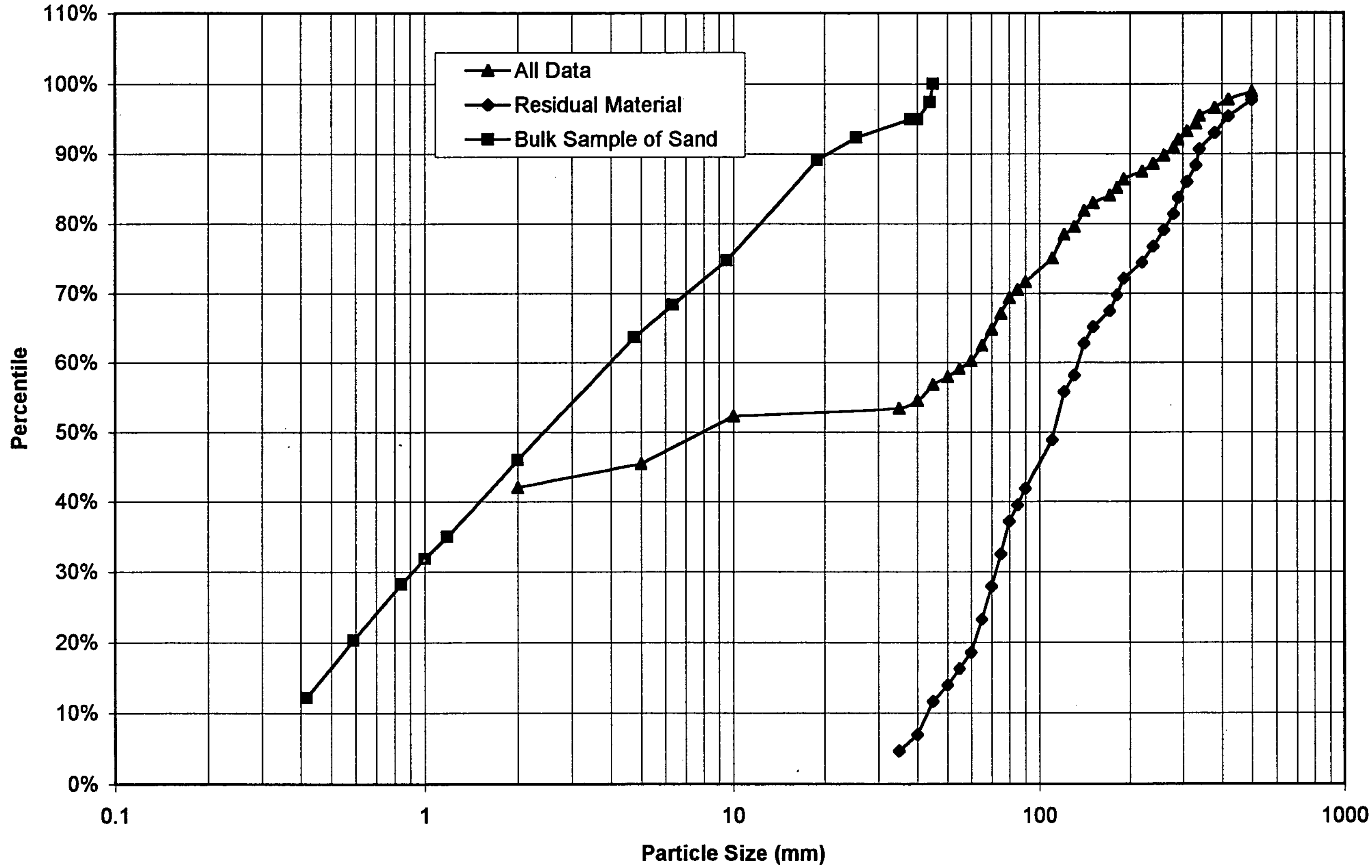
Bankfull depth appears to be very low relative to reach 1 and 3, but marks are unreliable and bed is highly variable, making measurement unreliable. Of greater significance might be that the terrace depth is greater than 1 or 3, which would make sense because the slope is lower in this reach.

Conclusions

Wbf = 36

Ybf = 0.57

Shovelnose B - Pebble Counts



1. Roughness Estimates Based on Flow and Bankfull Measurements

n =	0.034	= W*Y/Q*Rh^(2/3)*S^(1/2)
f =	0.098	= 8*9.81*(W*Y)^2*Y*S/Q^2

2. Roughness Estimates from Developed Empirical Relations

Manning's roughness			
Strickler	n =	0.018	= 0.041 D50^(1/6)
		0.031	= 0.038 D90^(1/6)
Chow	n =	0.045	- from qualitative estimate
Limerino	n =	0.043	= (0.113*Y^(1/6))/(1.16+2.00log(Y/D84))
Bray	n =	0.024	= 0.0593 D50^0.179
		0.040	= 0.0495 D90^0.160
		0.037	= 0.104*S^0.177
Jarrett	n =	0.045	= 0.39*S^0.38*R^(-0.16)

Friction Factor			
	ks =	0.048	ks = 6.8*D50
		0.592	ks = 3.5*D84
		0.837	ks = 3.1*D90
	avg	0.492	
Keulega	f =	0.136	= (2.21 + 2.03log(Y/ks))^(-2)
Colebroo	f =	0.136	= (2.03log(12.2*Y/ks))^(-2)
Bray	f =	0.037	= (0.248 + 2.36log(Y/D50))^(-2)
		0.180	= (1.26 + 2.16log(Y/D90))^(-2)
		0.036	= (1.36 (Y/D50)^0.281)^(-2)
		0.169	= (1.78 (Y/D90)^0.268)^(-2)
		0.098	= (- 2.32 - 2.20log(S))^(-2)
		0.107	= (0.696*S^(-0.256))^(-2)
Kellerhal	f =	0.105	= (2.30 (Y/D90)^0.25)^(-2)

3. Equivalent Roughness

ks =	0.28	=Y/10^(((f)^(-0.5)-2.21)/2.03)
		Where f value is from step 1
	0.28	Where f value is from step 2

Sediment Transport

Constants

Specific Weight =	9810	kg/m^2*s^2
Density =	1000	kg/m^3
g =	9.81	m/s^2
nu =	1E-06	m^2/s
s =	2.65	sim

Preliminary Calculations

Shear =	29	N
SFbank =	0.22	dim
bed shear =	26	N
Sheilds =	0.232	dim
Power =	38	Nm/s

Einstein-Brown

gb * =	0.500	dim
F1 =	0.81	dim
gb =	2.540	kg/ms
Gb =	68.6	kg/s

Brownlie (81)

Rg	345
Lauren	0.026
tau*o	0.044
omega	2
Fgo	1.76
q	1.25
q*	3618
Y	0.85
f	0.096
Fg calc	7.26
C	822
gb	1.03
Gb	27.7

Input Data and Reach Analysis
Shovelnose Creek
Reach C

Data Collected
Oct 28 - Oct 31, 1997

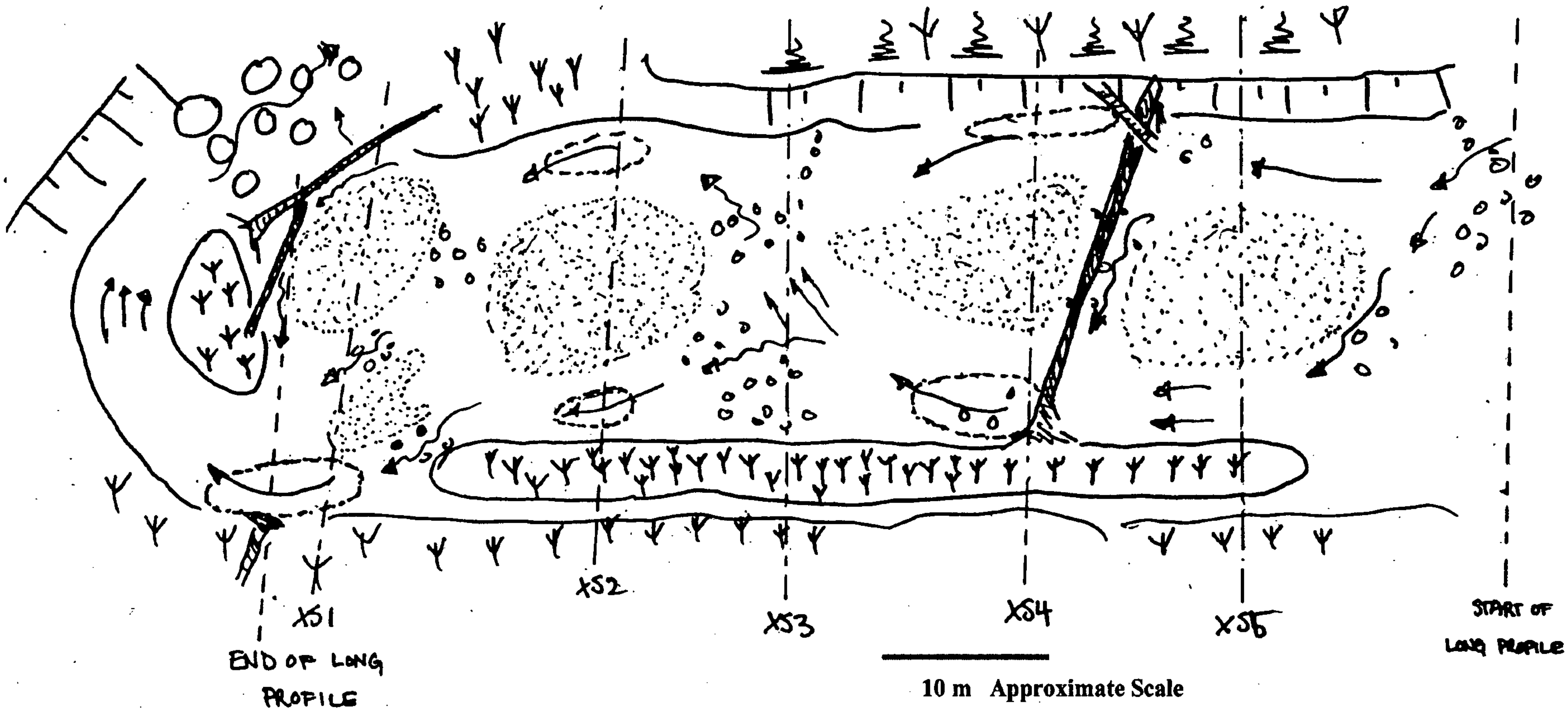
By
Bruce MacVicar
and Stephane Daoust

Summary Data

W =	30	m	Pbed =	25.0	m
Y =	0.87	m	Pbank =	5.3	m
S =	0.0043		Ybed =	0.95	m
			Rh =	0.82	m

Q =	45	m^3/s				
	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	0.033	0.044	0.056	0.075	0.097	m
D50bank =	0.265	m				
ks =	0.26	m				
	E-B					
Gb =	2.8	kg/s				
	mature bank	oung alder				
ϕ ' =	52	40	degrees			
θ =	43	degrees				

SHOVELNOSE CREEK - REACH C
OCTOBER 29/1997

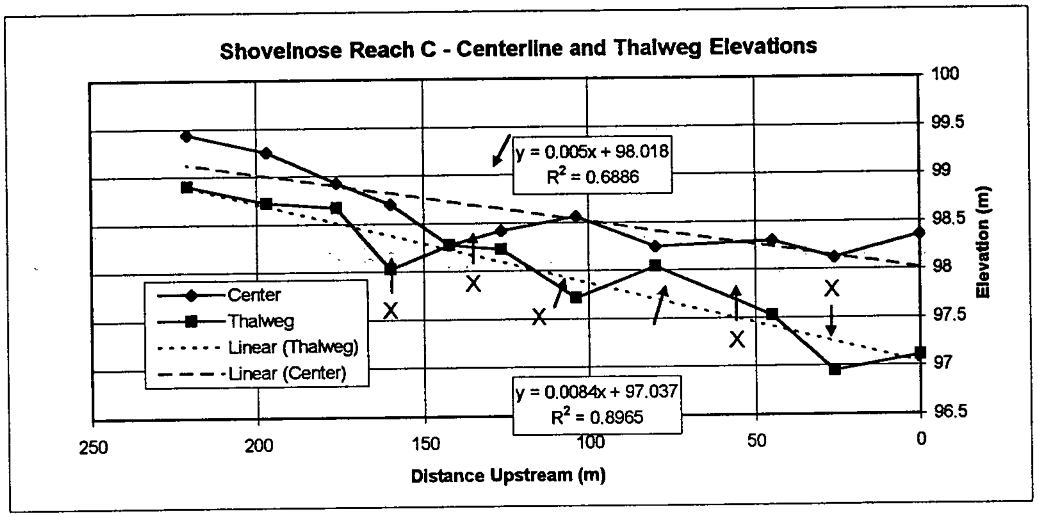


Longitudinal Profile - Shovelnose Creek, Reach C

Data gathered on Oct 30/1997 by Bruce MacVicar (level) and Stephane D'aoust (rod)

Level elevation = 100.26 m

High	Mid	Low	Thalweg	X		Center	Thalweg
1.35	0.82	0.29	1.36	106	221	99.44	98.9
1.43	1.01	0.61	1.54	82	197	99.25	98.72
1.63	1.33	1.02	1.59	61	176	98.93	98.67
1.785	1.56	1.335	2.23	45	160	98.7	98.03
2.125	1.99	1.855	1.99	27	142	98.27	98.27
1.897	1.838	1.781	2.035	11.6	126.6	98.422	98.225
1.758	1.699	1.646	2.545	-11.2	103.8	98.561	97.715
2.178	2.015	1.825	2.22	-35.3	79.7	98.245	98.04
2.315	1.96	1.615	2.735	-70	45	98.3	97.525
2.59	2.14	1.7	3.31	-89	26	98.12	96.95
3.74	1.9	2.59	3.15	-115	0	98.36	97.11



Slope C/L = 0.0050
Thalweg 0.0084
Bartop = 0.0043

Discussion

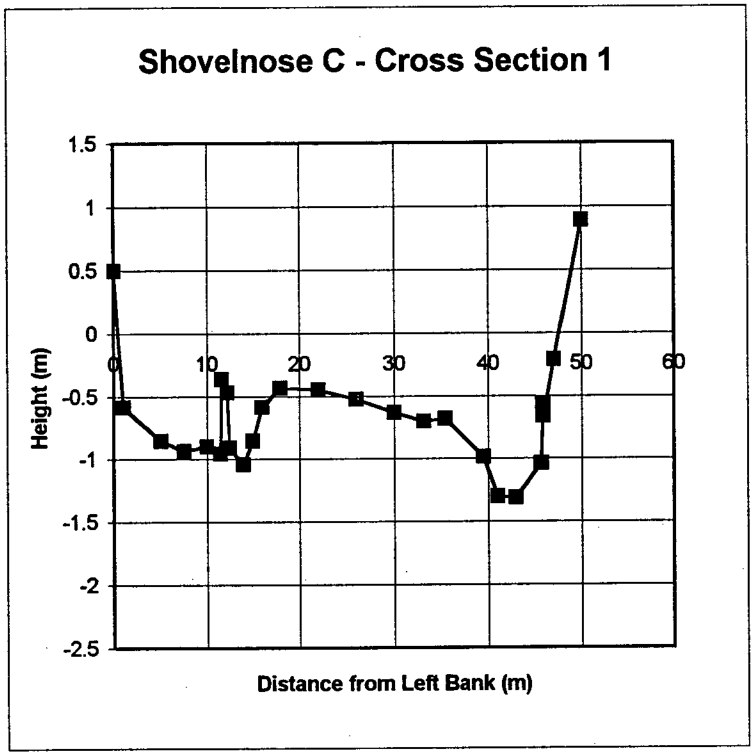
Both centerline and thalweg slopes are affected by the alluvial fan at the upper end and the debris jam at the lower end
The bartops appeared to be the only regular repeating feature and their slope is taken as representative.

Conclusion:
So = 0.0043

Cross Section # 1
Level Elevation: 99.96 m
Bankfull reading: 1.17 m

Data

distance	reading (	D from L	elevation	from BF	
0	0.677	0	99.283	0.493	
1	1.76	1	98.2	-0.59	-0.59
5	2.024	5	97.936	-0.854	-3.416
7.5	2.108	7.5	97.852	-0.938	-2.345
10	2.07	10	97.89	-0.9	-2.25
11.5	2.128	11.5	97.832	-0.958	-1.437
11.6	1.538	11.6	98.422	-0.368	-0.0368
12.2	1.643	12.2	98.317	-0.473	-0.2838
12.5	2.084	12.5	97.876	-0.914	-0.2742
14	2.212	14	97.748	-1.042	-1.563
15	2.028	15	97.932	-0.858	-0.858
16	1.756	16	98.204	-0.586	-0.586
18	1.608	18	98.352	-0.438	-0.876
22	1.62	22	98.34	-0.45	-1.8
26	1.698	26	98.262	-0.528	-2.112
30	1.802	30	98.158	-0.632	-2.528
33.2	1.876	33.2	98.084	-0.706	-2.2592
35.5	1.857	35.5	98.103	-0.687	-1.5801
39.5	2.159	39.5	97.801	-0.989	-3.956
41	2.472	41	97.488	-1.302	-1.953
43	2.482	43	97.478	-1.312	-2.624
45.7	2.208	45.7	97.752	-1.038	-2.8026
45.9	1.745	45.9	98.215	-0.575	-0.115
45.91	1.835	45.91	98.125	-0.665	-0.0066
47	1.385	47	98.575	-0.215	-0.2344
50	0.28	50	99.68	0.89	2.67



Bankfull Width =	47	m
Bankfull Depth =	0.78	m
Terrace Width =	50	m
Terrace Depth =	1.57	m
Pbed =	33.2	m
Ybed =	0.78	m

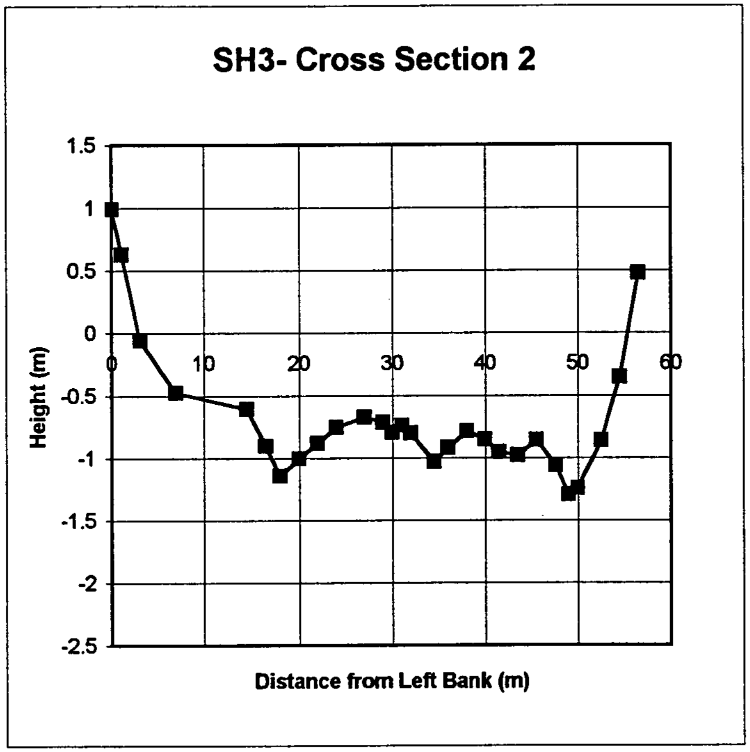
Cross Section # 2

Level Elevation: 99.96 m

Bankfull reading: 0.99 m

Data

distance	reading (	D from L	elevation	from BF	
-1	0	0	99.96	0.99	
0.1	0.36	1.1	99.6	0.63	0.693
2.1	1.05	3.1	98.91	-0.06	-0.12
6	1.467	7	98.493	-0.477	-1.8603
13.5	1.601	14.5	98.359	-0.611	-4.5825
15.5	1.897	16.5	98.063	-0.907	-1.814
17	2.136	18	97.824	-1.146	-1.719
19	1.999	20	97.961	-1.009	-2.018
21	1.875	22	98.085	-0.885	-1.77
23	1.745	24	98.215	-0.755	-1.51
26	1.667	27	98.293	-0.677	-2.031
28	1.705	29	98.255	-0.715	-1.43
29	1.79	30	98.17	-0.8	-0.8
30	1.734	31	98.226	-0.744	-0.744
31	1.794	32	98.166	-0.804	-0.804
33.5	2.021	34.5	97.939	-1.031	-2.5775
35	1.909	36	98.051	-0.919	-1.3785
37	1.776	38	98.184	-0.786	-1.572
39	1.842	40	98.118	-0.852	-1.704
40.5	1.947	41.5	98.013	-0.957	-1.4355
42.5	1.974	43.5	97.986	-0.984	-1.968
44.5	1.85	45.5	98.11	-0.86	-1.72
46.6	2.054	47.6	97.906	-1.064	-2.2344
48	2.288	49	97.672	-1.298	-1.8172
49	2.235	50	97.725	-1.245	-1.245
51.5	1.855	52.5	98.105	-0.865	-2.1625
53.5	1.345	54.5	98.615	-0.355	-0.71
55.5	0.515	56.5	99.445	0.475	0.95

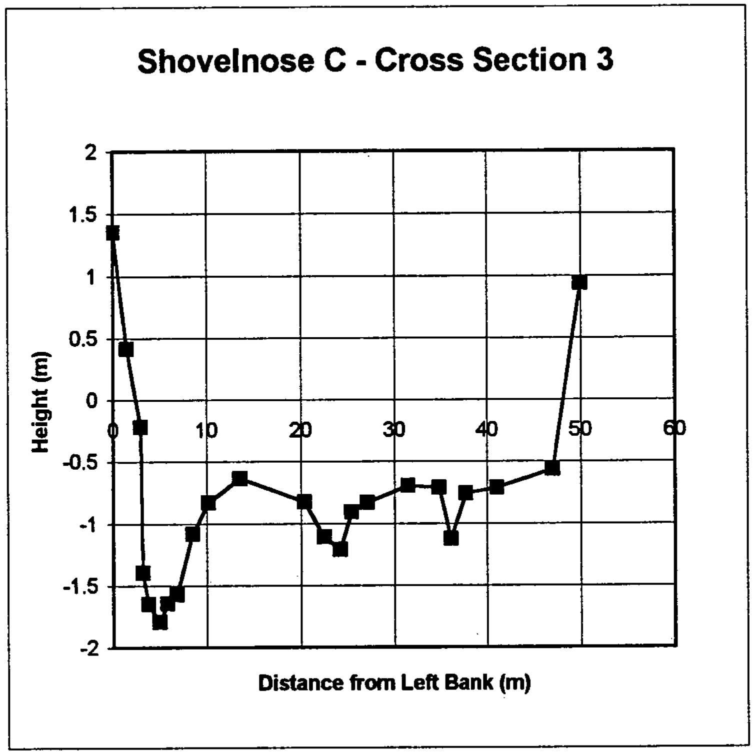


Bankfull Width =	53	m
Bankfull Depth =	0.79	m
Terrace Width =	56.5	m
Terrace Depth =	1.70	m
Pbed =	25.5	m
Ybed =	0.92	m

Cross Section # 3
Level Elevation: 100.26 m
Bankfull reading: 0.985 m

Data

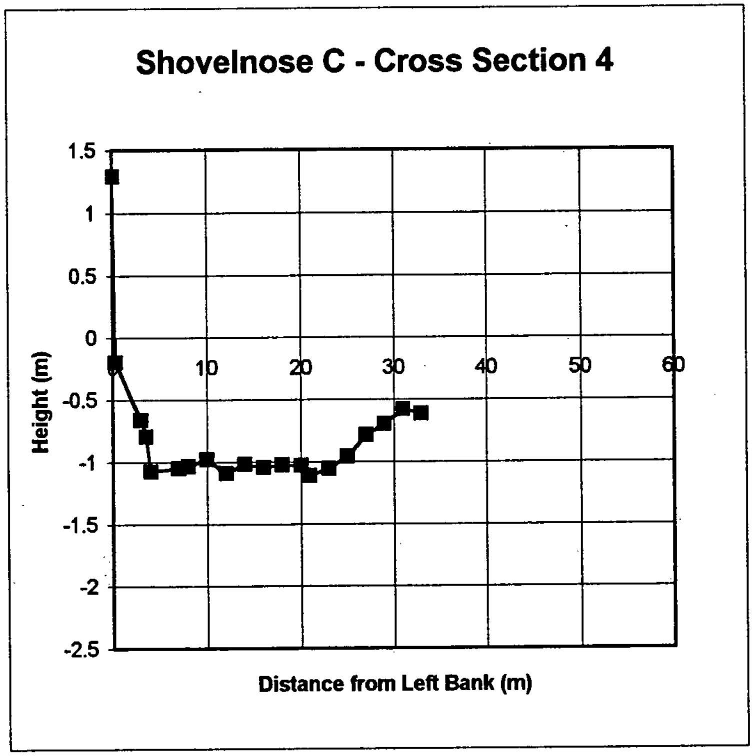
distance	reading (	D from L	elevation	from BF	
0	-0.365	0	100.625	1.35	
1.5	0.575	1.5	99.685	0.41	0.615
3	1.205	3	99.055	-0.22	-0.33
3.2	2.382	3.2	97.878	-1.397	-0.2794
3.8	2.637	3.8	97.623	-1.652	-0.9912
5	2.78	5	97.48	-1.795	-2.154
5.8	2.63	5.8	97.63	-1.645	-1.316
6.8	2.557	6.8	97.703	-1.572	-1.572
8.5	2.069	8.5	98.191	-1.084	-1.8428
10.2	1.82	10.2	98.44	-0.835	-1.4195
13.5	1.62	13.5	98.64	-0.635	-2.0955
20.4	1.813	20.4	98.447	-0.828	-5.7132
22.5	2.096	22.5	98.164	-1.111	-2.3331
24.2	2.198	24.2	98.062	-1.213	-2.0621
25.4	1.896	25.4	98.364	-0.911	-1.0932
27.1	1.82	27.1	98.44	-0.835	-1.4195
31.5	1.683	31.5	98.577	-0.698	-3.0712
34.8	1.7	34.8	98.56	-0.715	-2.3595
36.1	2.11	36.1	98.15	-1.125	-1.4625
37.7	1.745	37.7	98.515	-0.76	-1.216
41.1	1.7	41.1	98.56	-0.715	-2.431
47	1.55	47	98.71	-0.565	-3.3335
50	0.05	50	100.21	0.935	2.805



Bankfull Width =	47	m
Bankfull Depth =	0.82	m
Terrace Width =	50	m
Terrace Depth =	2.05	m
Pbed =	23.3	m
Ybed =	1.03	m

Cross Section # 4
Level Elevation: 100.26 m
Bankfull reading: 0.85 m

Data					
distance	reading (	D from L	elevation	from BF	
1	-0.44	0	100.7	1.29	
1.3	1.05	0.3	99.21	-0.2	-0.06
4	1.514	3	98.746	-0.664	-1.7928
4.5	1.647	3.5	98.613	-0.797	-0.3985
5	1.93	4	98.33	-1.08	-0.54
8	1.9	7	98.36	-1.05	-3.15
9	1.885	8	98.375	-1.035	-1.035
11	1.83	10	98.43	-0.98	-1.96
13	1.945	12	98.315	-1.095	-2.19
15	1.87	14	98.39	-1.02	-2.04
17	1.897	16	98.363	-1.047	-2.094
19	1.879	18	98.381	-1.029	-2.058
21	1.881	20	98.379	-1.031	-2.062
22	1.964	21	98.296	-1.114	-1.114
24	1.91	23	98.35	-1.06	-2.12
26	1.812	25	98.448	-0.962	-1.924
28	1.634	27	98.626	-0.784	-1.568
30	1.55	29	98.71	-0.7	-1.4
32	1.43	31	98.83	-0.58	-1.16
34	1.47	33	98.79	-0.62	-1.24



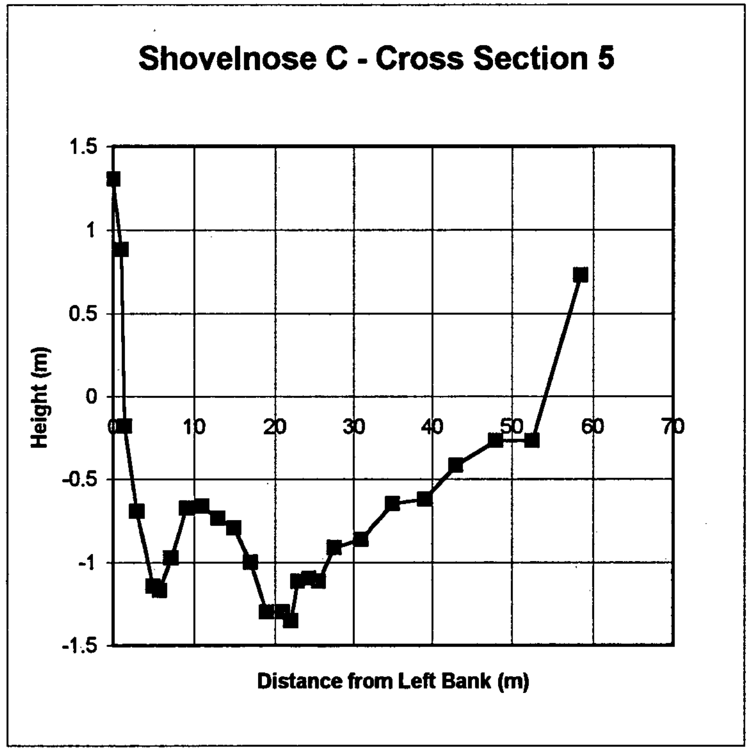
Bankfull Width = 47 m
Bankfull Depth = 0.64 m
Terrace Width = 50 m
Terrace Depth = 1.89 m
Pbed = 23 m
Ybed = 1.10 m

Cross Section # 5

Level Elevation: 100.51 m

Bankfull reading: 0.99 m

Data distance	reading (	D from L	elevation	from BF	
3	-0.313	0	100.823	1.303	
4	0.108	1	100.402	0.882	0.882
4.4	1.17	1.4	99.34	-0.18	-0.072
5.9	1.685	2.9	98.825	-0.695	-1.0425
7.9	2.135	4.9	98.375	-1.145	-2.29
8.7	2.16	5.7	98.35	-1.17	-0.936
10.1	1.963	7.1	98.547	-0.973	-1.3622
12	1.665	9	98.845	-0.675	-1.2825
14	1.653	11	98.857	-0.663	-1.326
16	1.728	13	98.782	-0.738	-1.476
18	1.784	15	98.726	-0.794	-1.588
20	1.99	17	98.52	-1	-2
22	2.29	19	98.22	-1.3	-2.6
24	2.29	21	98.22	-1.3	-2.6
25	2.345	22	98.165	-1.355	-1.355
25.9	2.108	22.9	98.402	-1.118	-1.0062
27.3	2.089	24.3	98.421	-1.099	-1.5386
28.5	2.106	25.5	98.404	-1.116	-1.3392
30.5	1.903	27.5	98.607	-0.913	-1.826
34	1.855	31	98.655	-0.865	-3.0275
38	1.64	35	98.87	-0.65	-2.6
42	1.61	39	98.9	-0.62	-2.48
46	1.41	43	99.1	-0.42	-1.68
51	1.26	48	99.25	-0.27	-1.35
55.5	1.26	52.5	99.25	-0.27	-1.215
61.5	0.26	58.5	100.25	0.73	4.38



Bankfull Width = 52.2 m
Bankfull Depth = 0.73 m
Terrace Width = 58.5 m
Terrace Depth = 1.86 m
Pbed = 19.4 m
Ybed = 1.10 m

Size of Channel

	Pool	Riffle	P	R	P		
X-section	1	2	3	4	5	Avg	Note
Wbf	47	53	47	47	58	50	consistent - due to sizing of channel from flood
Ybf	0.65	0.83	1.38	1.19	1.58	1.13	gradually increasing - due to poor bankfull indicators
W alder	n/a (1)	31.0	13.8	19.5	24.6	22.2	very inconsistent
Y alder	n/a	0.83	0.44	-0.011	0.31	0.39	
W bartop	30.0	n/a	25.6	n/a	31.0	28.9	
Y bartop	0.31	n/a	0.31	n/a	0.30	0.31	
W water	16.5	31.0	13.7	23.0	31.6	23.2	
Y water	0.33	0.25	0.39	0.23	0.30	0.30	
W terrac	50.0	56.5	50.0	50.0	58.5	53.0	
Y terrace	1.57	1.70	2.05	1.89	1.86	1.81	
Pbed	33.2	25.5	23.3	23	19.4	25	
Ybed	0.78	0.92	1.03	1.10	1.10	0.99	

Discussion

Bankfull estimates were poor due to lack of good indicators. The best indicators were high water marks from trapped vegetation, but these are unreliable according to references. Alder growth was also unreliable as water was observed flowing through the small trees in some sections even at the intermediate flows observed at the time of observation. Large trees were confined to the high banks. The high banks were assumed to be a product of the Squamish flood. This can explain the consistency found in bankfull width observations. It is hypothesized that the flow in a typical river of this width can be shown to be much greater than more reliable estimates of the flow in Shovelnose Creek.
The most consistent relation was found to be the area of flow below bartop measurements. Due to this observed consistency and the observed regularity of pools, riffles and bars, it is assumed that the dominant discharge of Shovelnose formed these patterns and that they will lead to a reliable estimate of the bankfull size of the channel.

To find bankfull depth
- assuming that Sheilds criterion will give a reliable estimate of the depth above the bar during dominant discharge

		X1	X3	X5		X2	X4	Avg
Given	D50 =	0.0305	0.0435	0.0515				
	So =	0.004	0.004	0.004	- for bartop slope			
	s =	2.65	2.65	2.65	- assumed specific gravity of sediment			
	X =	0.035	0.035	0.035	- from Parker			
	Y (abv ba	0.44	0.63	0.74	= X * (s-1) * D50 / So			
	Y	0.75	1.02	0.84		0.83	0.89	0.87
	W	47.5	47.0	52.2		53.0	46.0	49.1

Discussion

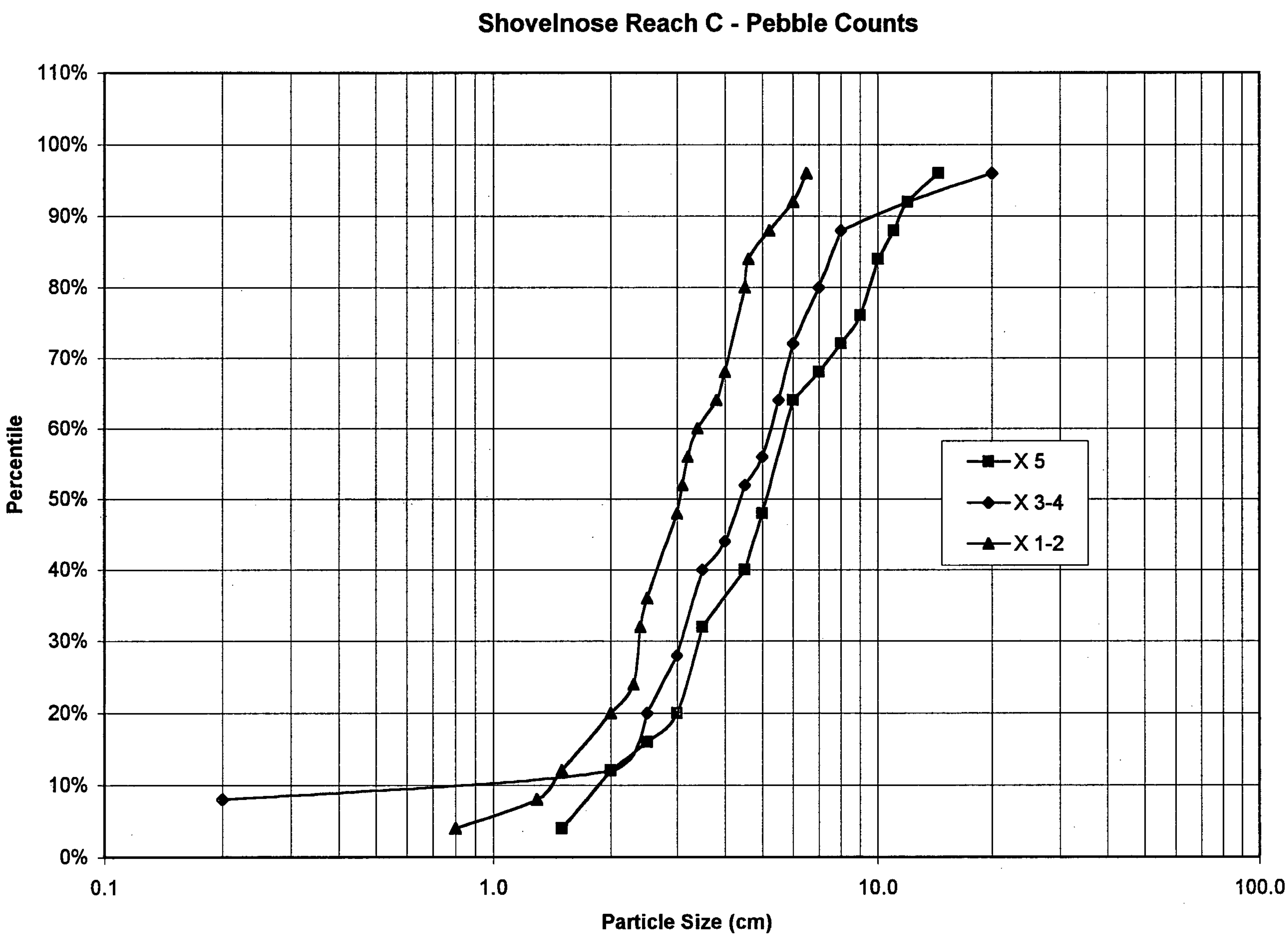
more consistent resuts. Difference in averages attributable to differences in roughness.
Alders are growing on a lot of that width, making it too rough for significant flow, even at high flows
Bankfull width is not representative of the width being chosen by the stream. For that width, the alder free width of the channel will be used. It is felt that this is most representative of the width that is used to pass current bankfull flows. Flow through the alders is small even though there may be significant depth. Flow in the small channel through the alders is not considered.

W aldfre	30.0	38.5	24.0	23.5	32.1	29.6

Conclusion

The width and depth representative of the Squamish flood flows are as follows:
Wflood = 53.0 m
Yflood = 1.81 m

The width and depth representative of current flows are as follows:
Wrep = 29.6 m
Yrep = 0.87 m



Shovelnose Reach C - Pebble Counts
Percentile
110%
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.1
1.0
10.0
100.0
Particle Size (cm)
X 5
X 3-4
X 1-2

1. Roughness Estimates Based on Flow and Bankfull Measurements

n =	0.033	= W*Y/Q*Rh^(2/3)*S^(1/2)
f =	0.093	= 8*9.81*(W*Y)^2*Y*S/Q^2

2. Roughness Estimates from Developed Empirical Relations

Manning's roughness			
Strickler	n =	0.024	= 0.041 D50^(1/6)
		0.026	= 0.038 D90^(1/6)
Chow	n =	0.045	- from qualitative estimate
Limerino	n =	0.034	= (0.113*Y^(1/6))/(1.16+2.00log(Y/D84))
Bray	n =	0.034	= 0.0593 D50^0.179
		0.034	= 0.0495 D90^0.160
		0.040	= 0.104*S^0.177
Jarrett	n =	0.051	= 0.39*S^0.38*R^(-0.16)

Friction Factor			
	ks =	0.299	ks = 6.8*D50
		0.263	ks = 3.5*D84
		0.301	ks = 3.1*D90
	avg	0.287	
Keulega	f =	0.098	= (2.21 + 2.03log(Y/ks))^(-2)
Colebroo	f =	0.099	= (2.03log(12.2*Y/ks))^(-2)
Bray	f =	0.091	= (0.248 + 2.36log(Y/D50))^(-2)
		0.091	= (1.26 + 2.16log(Y/D90))^(-2)
		0.101	= (1.36 (Y/D50)^0.281)^(-2)
		0.097	= (1.78 (Y/D90)^0.268)^(-2)
		0.120	= (- 2.32 - 2.20log(S))^(-2)
		0.127	= (0.696*S^(-0.256))^(-2)
Kellerhal	f =	0.063	= (2.30 (Y/D90)^0.25)^(-2)

3. Equivalent Roughness

ks =	0.26	=Y/10^(((f)^(-0.5)-2.21)/2.03) Where f value is from step 1
	0.25	Where f value is from step 2

Sediment Transport

Constants

Specific Weight =	9810	kg/m^2*s^2
Density =	1000	kg/m^3
g =	9.81	m/s^2
nu =	1E-06	m^2/s
s =	2.65	sim

Preliminary Calculations

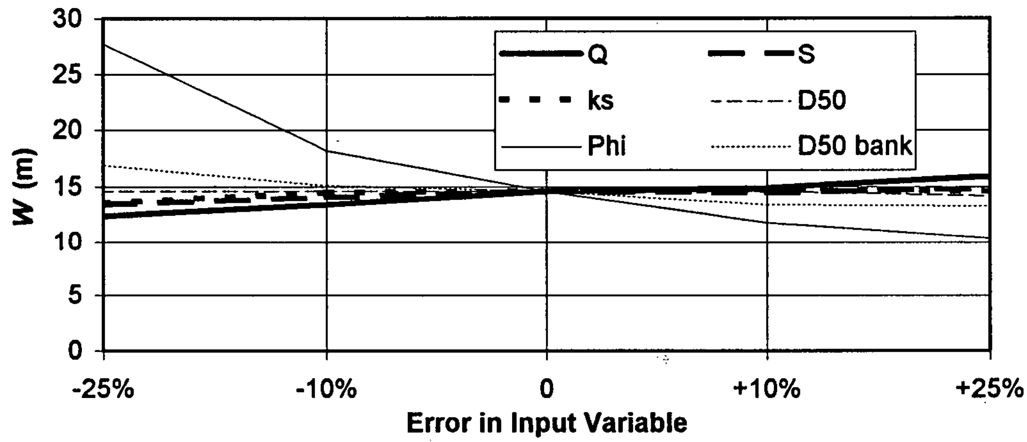
Shear =	40	N
SFbank =	0.14	dim
bed shear =	38	N
Sheilds =	0.053	dim
Power =	65	Nm/s

Einstein-Brown

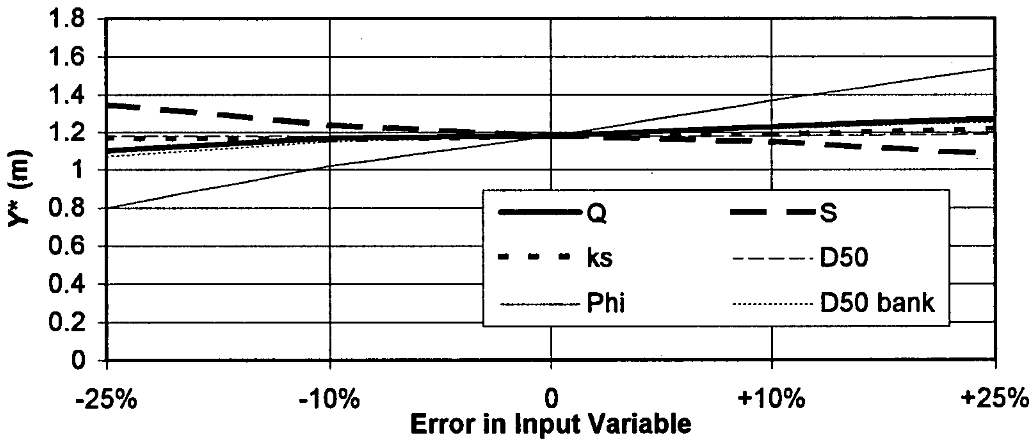
gb * =	0.001	dim
F1 =	0.82	dim
gb =	0.112	kg/ms
Gb =	2.8	kg/s

APPENDIX B.3 - SHOVELNOSE CREEK SENSITIVITY ANALYSIS

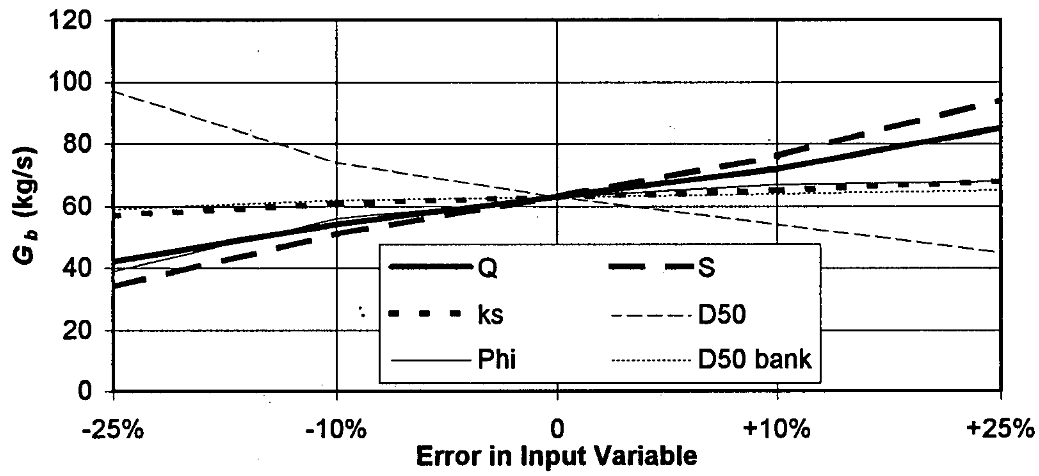
Sensitivity of W - Shovelnose Creek 1974



Sensitivity of Y^* - Shovelnose Creek 1974



Sensitivity of G_b - Shovelnose Creek 1974



APPENDIX C

HARRIS CREEK

APPENDIX C.1 HARRIS CREEK - AIRPHOTOS

Air photos are listed in the table below. Photos for the upper watershed in 1952 are not shown.

Harris Creek Air Photos

Year	Roll Number	Picture Numbers	Scale
1952	A5904	31-35	1 : 21,300
1970	BC7264, BC7074	32-34, 219-221	1 : 18,500
1980	BC80083, BC80082	84-86, 196-198	1 : 22,000
1984	BC84083	60-61, 142-144	1: 6,400
1992	BC92032, BC92031	10-12, 19-21	1 : 23,000

APPENDIX C.2 - HARRIS CREEK SURVEY DATA AND ANALYSIS

Input Data and Reach Analysis
Harris Creek
Reach H4

Data Collected
April 18 -19, 1998

By
Bruce MacVicar
and Stephane D'Aoust

Summary Data

W =	51	m	Pbed =	44.0
Y =	2.0	m	Pbank =	8.1
S =	0.0032		Ybed =	2.15
			Rh =	1.9

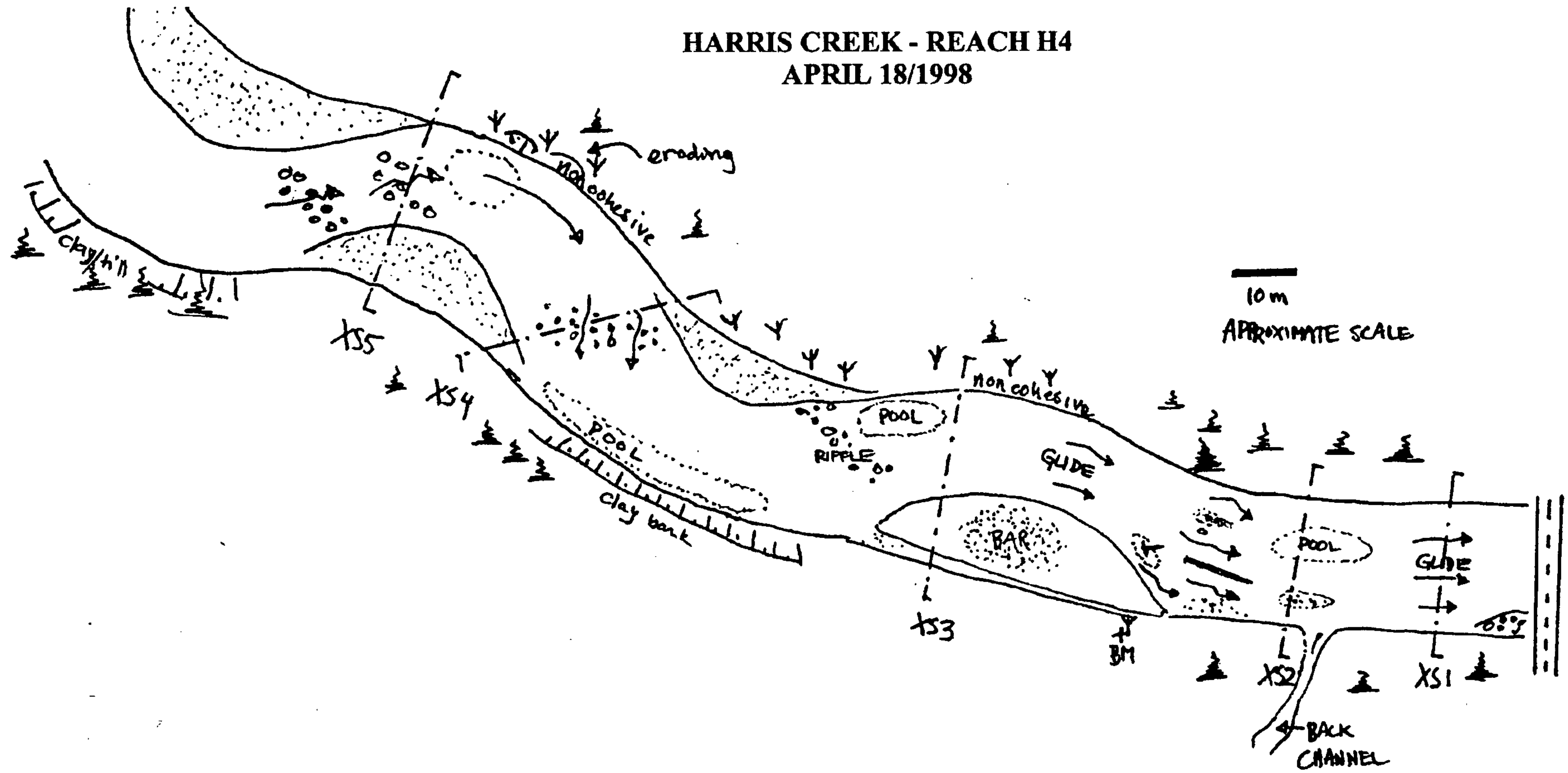
Q = 250 m^3/s

	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	0.049	0.065	0.084	0.110	0.135	m

ks = 0.44 m

Gb = 22.6 kg/s

HARRIS CREEK - REACH H4
APRIL 18/1998

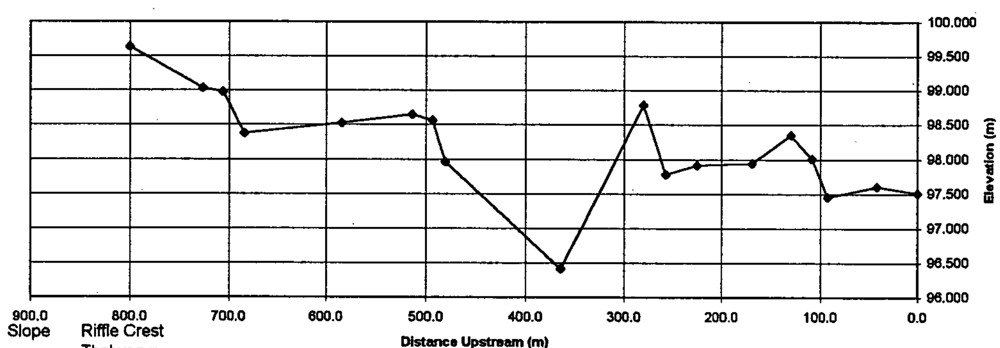


Longitudinal Profile - Harris Creek Reach H4

		BS	HI	FS	ELEV
BM		0.082	100.082		100.000
TP				1.580	
	1	2.525	101.027		98.502
				1.640	
	2	2.765	102.152		99.387
		-0.800	99.2		100.000 from BM
				1.531	
	3	0.884	99.386		97.669

	Elevation					Distance											
Note	TP elev	Level H	HI	Thalweg	Thalweg	Level D	Low	Mid	High	Dist 1	Dist 2	from leve	Thalweg	Slope			
	100	0.082	100.082	2.580	97.502	108.5	2.040	2.580	3.125	54	54.5	108.5	0.0				
XS1	100	0.082	100.082	2.480	97.602	108.5	2.145	2.480	2.815	33.5	33.5	67.0	41.5				
XS2	100	0.082	100.082	2.628	97.454	108.5	2.547	2.628	2.710	8.1	8.2	16.3	92.2	*			
	100	0.082	100.082	2.075	98.007	108.5						0.0	108.5				
	100	0.082	100.082	1.725	98.357	108.5	1.617	1.725	1.833	10.8	10.8	21.6	130.1				
	100	0.082	100.082	2.140	97.942	108.5	1.840	2.140	2.445	30	30.5	60.5	169.0				
	100	0.082	100.082	2.170	97.912	108.5	1.590	2.170	2.755	58	58.5	116.5	225.0				
	98.502	2.525	101.027	3.245	97.782	364	2.320	3.245	3.780	92.5	53.5	107.0	257.0	*	0.0020		pool
	98.502	2.525	101.027	2.245	98.782	364	1.820	2.245	2.665	42.5	42	84.5	279.5			0.0028	riffle
	98.502	2.525	101.027	4.610	96.417	364						0.0	364.0				
	98.502	2.525	101.027	3.070	97.957	364	2.485	3.070	3.655	58.5	58.5	117.0	481.0				
	98.502	2.525	101.027	2.465	98.562	364	1.815	2.465	3.115	65	65	130.0	494.0				
	98.502	2.525	101.027	2.385	98.642	364	1.640	2.385	3.140	74.5	75.5	150.0	514.0				
	99.387	2.765	102.152	3.625	98.527	726	2.915	3.625		71		142.0	584.0				
	99.387	2.765	102.152	3.780	98.372	726	3.570	3.780		21		42.0	684.0	*	0.0014	0.0016	pool
	99.387	2.765	102.152	3.175	98.977	726	1.824		2.023			19.9	706.1				
	99.387	2.765	102.152	3.125	99.027	726						0.0	726.0				
	99.387	2.765	102.152	2.515	99.637	726	2.145	2.515	2.885	37	37	74.0	800.0			0.0035	riffle
															Average	0.0032	

Longitudinal Profile, Harris Creek Reach H4



Slope Riffle Crest
Thalweg =

Discussion

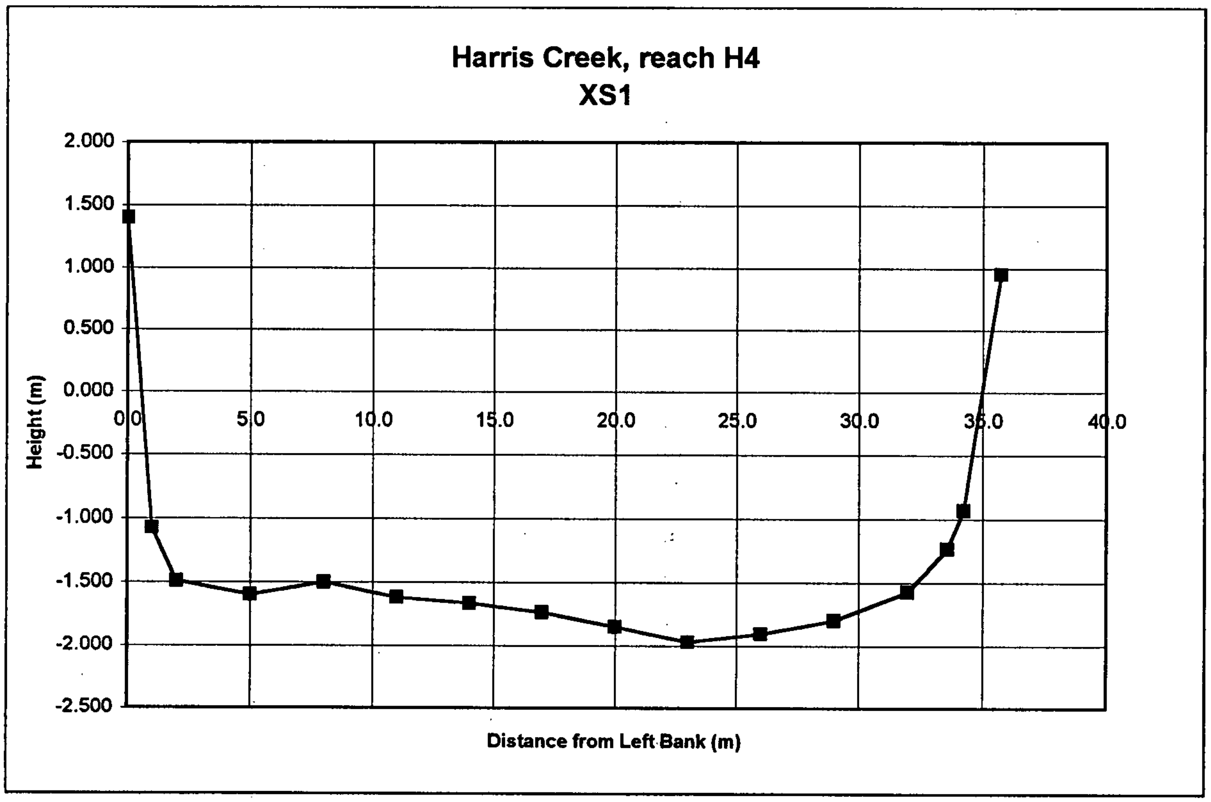
pool and riffle slopes calculated were significantly different.
use riffle crests as the best estimate
large scour hole in middle of reach due to resistant banks, ponding effect created, this section ignored.

Cross Section # 1

Level Elevation:	99.386	m		FS	from BF
Bankfull FS (BF):	0	m	ater elev (W	1.236	-1.236

istance (	FS (m)	from L	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
35.0	-1.395	0.0	100.781	1.395			2.631			
34.0	1.076	1.0	98.310	-1.076	-1.1	1.0	0.160	0.0	0.0	0.0
33.0	1.493	2.0	97.893	-1.493	-1.5	1.0	-0.257	-0.3	1.0	-1.5
30.0	1.596	5.0	97.790	-1.596	-4.8	3.0	-0.360	-1.1	3.0	-4.8
27.0	1.504	8.0	97.882	-1.504	-4.5	3.0	-0.268	-0.8	3.0	-4.5
24.0	1.618	11.0	97.768	-1.618	-4.9	3.0	-0.382	-1.1	3.0	-4.9
21.0	1.663	14.0	97.723	-1.663	-5.0	3.0	-0.427	-1.3	3.0	-5.0
18.0	1.734	17.0	97.652	-1.734	-5.2	3.0	-0.498	-1.5	3.0	-5.2
15.0	1.848	20.0	97.538	-1.848	-5.5	3.0	-0.612	-1.8	3.0	-5.5
12.0	1.966	23.0	97.420	-1.966	-5.9	3.0	-0.730	-2.2	3.0	-5.9
9.0	1.901	26.0	97.485	-1.901	-5.7	3.0	-0.665	-2.0	3.0	-5.7
6.0	1.796	29.0	97.590	-1.796	-5.4	3.0	-0.560	-1.7	3.0	-5.4
3.0	1.574	32.0	97.812	-1.574	-4.7	3.0	-0.338	-1.0	3.0	-4.7
1.4	1.236	33.6	98.150	-1.236	-2.0	1.6	0.000	0.0	0.0	0.0
0.8	0.936	34.3	98.450	-0.936	-0.6	0.6	0.300	0.0	0.0	0.0
-0.8	-0.956	35.8	100.342	0.956	0.0	0.0	2.192	0.0	0.0	0.0
					56.8	34.3		14.8	31.0	53.1

Summary Data			
	Bankfull Width =	34.3	m
	Hydraulic Mean De	1.66	m
	Wet Width =	31.0	m
	Wet Mean Depth =	0.48	m
	Ybed =	1.71	m

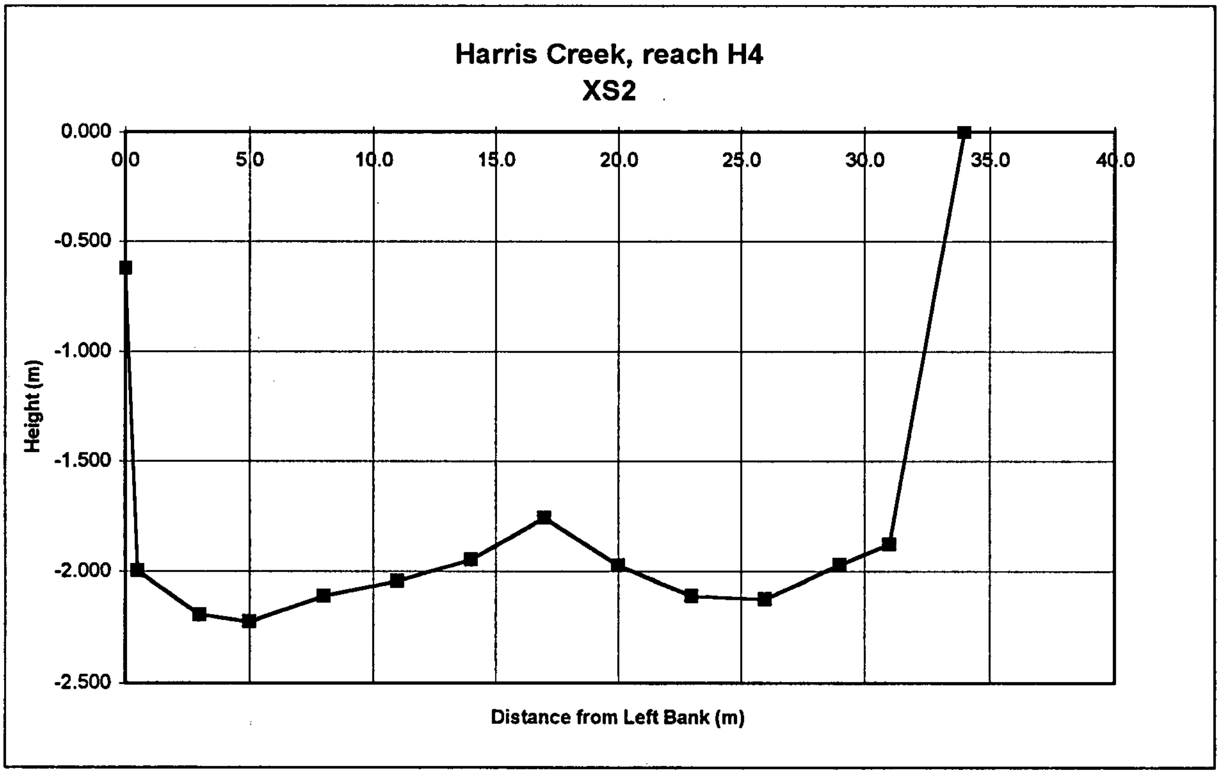


Cross Section # 2

Level Elevation:	99.2	m		FS	from BF
Bankfull FS (BF):	0.321	m	ater elev (W	1.787	-1.466

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
1.0	0.945	0.0	98.255	-0.624			0.842			
1.5	2.319	0.5	96.881	-1.998	-1.0	0.5	-0.532	-0.3	0.5	-1.0
4.0	2.516	3.0	96.684	-2.195	-5.5	2.5	-0.729	-1.8	2.5	-5.5
6.0	2.549	5.0	96.651	-2.228	-4.5	2.0	-0.762	-1.5	2.0	-4.5
9.0	2.431	8.0	96.769	-2.110	-6.3	3.0	-0.644	-1.9	3.0	-6.3
12.0	2.364	11.0	96.836	-2.043	-6.1	3.0	-0.577	-1.7	3.0	-6.1
15.0	2.269	14.0	96.931	-1.948	-5.8	3.0	-0.482	-1.4	3.0	-5.8
18.0	2.082	17.0	97.118	-1.761	-5.3	3.0	-0.295	-0.9	3.0	-5.3
21.0	2.296	20.0	96.904	-1.975	-5.9	3.0	-0.509	-1.5	3.0	-5.9
24.0	2.432	23.0	96.768	-2.111	-6.3	3.0	-0.645	-1.9	3.0	-6.3
27.0	2.446	26.0	96.754	-2.125	-6.4	3.0	-0.659	-2.0	3.0	-6.4
30.0	2.292	29.0	96.908	-1.971	-5.9	3.0	-0.505	-1.5	3.0	-5.9
32.0	2.200	31.0	97.000	-1.879	-3.8	2.0	-0.413	-0.8	2.0	-3.8
35.0	0.321	34.0	98.879	0.000	0.0	0.0	1.466	0.0	0.0	0.0
					62.8	31.0		17.4	31.0	62.8

Summary Data			
	Bankfull Width =	31.0	m
	Hydraulic Mean De	2.03	m
	Wet Width =	31.0	m
	Wet Mean Depth =	0.56	m
	Ybed =	2.03	m



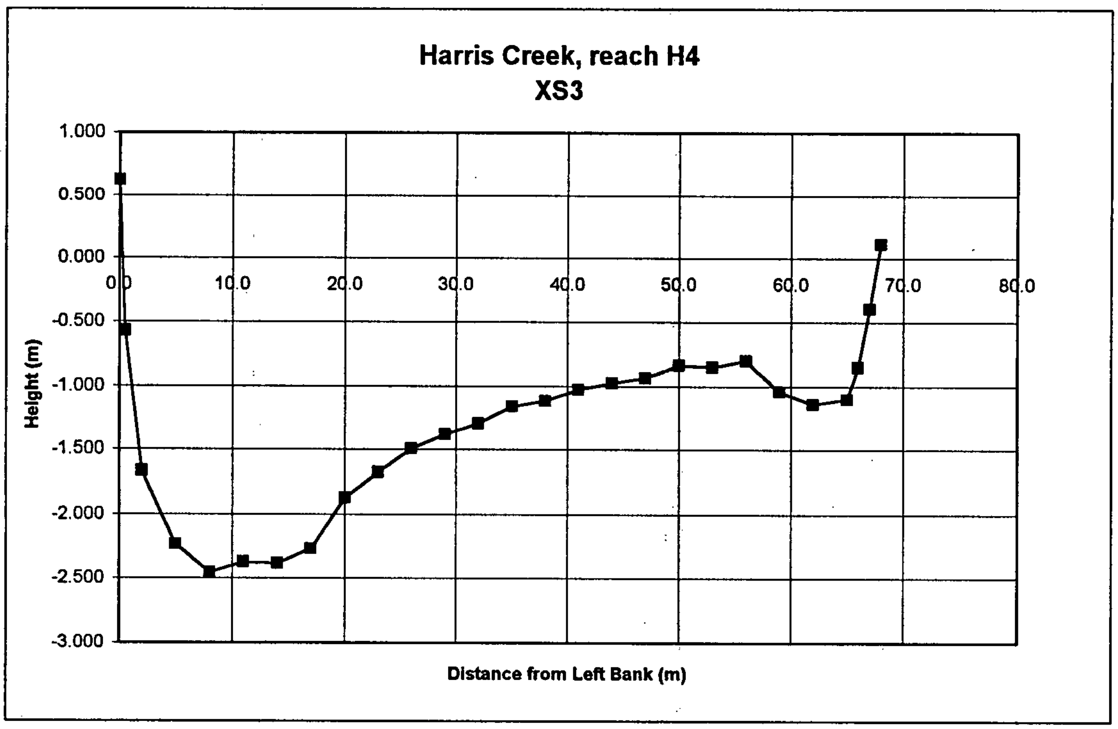
Cross Section # 3

Level Elevation:	100.951	m		FS	from BF
Bankfull FS (BF):	0.625	m	ater elev (W	2.533	-1.908

istance (	FS (m)	from L	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
68.0	0	0.0	100.951	0.625			2.533			
67.5	1.195	0.5	99.756	-0.570	-0.3	0.5	1.338	0.0	0.0	0.0
66.0	2.289	2.0	98.662	-1.664	-2.5	1.5	0.244	0.0	0.0	0.0
63.0	2.859	5.0	98.092	-2.234	-6.7	3.0	-0.326	-1.0	3.0	-6.7
60.0	3.081	8.0	97.870	-2.456	-7.4	3.0	-0.548	-1.6	3.0	-7.4
57.0	2.998	11.0	97.953	-2.373	-7.1	3.0	-0.465	-1.4	3.0	-7.1
54.0	3.007	14.0	97.944	-2.382	-7.1	3.0	-0.474	-1.4	3.0	-7.1
51.0	2.892	17.0	98.059	-2.267	-6.8	3.0	-0.359	-1.1	3.0	-6.8
48.0	2.494	20.0	98.457	-1.869	-5.6	3.0	0.039	0.0	0.0	0.0
45.0	2.3	23.0	98.651	-1.675	-5.0	3.0	0.233	0.0	0.0	0.0
42.0	2.117	26.0	98.834	-1.492	-4.5	3.0	0.416	0.0	0.0	0.0
39.0	2.009	29.0	98.942	-1.384	-4.2	3.0	0.524	0.0	0.0	0.0
36.0	1.922	32.0	99.029	-1.297	-3.9	3.0	0.611	0.0	0.0	0.0
33.0	1.789	35.0	99.162	-1.164	-3.5	3.0	0.744	0.0	0.0	0.0
30.0	1.745	38.0	99.206	-1.120	-3.4	3.0	0.788	0.0	0.0	0.0
27.0	1.655	41.0	99.296	-1.030	-3.1	3.0	0.878	0.0	0.0	0.0
24.0	1.6	44.0	99.351	-0.975	-2.9	3.0	0.933	0.0	0.0	0.0
21.0	1.56	47.0	99.391	-0.935	-2.8	3.0	0.973	0.0	0.0	0.0
18.0	1.462	50.0	99.489	-0.837	-2.5	3.0	1.071	0.0	0.0	0.0
15.0	1.475	53.0	99.476	-0.850	-2.6	3.0	1.058	0.0	0.0	0.0
12.0	1.428	56.0	99.523	-0.803	-2.4	3.0	1.105	0.0	0.0	0.0
9.0	1.67	59.0	99.281	-1.045	-3.1	3.0	0.863	0.0	0.0	0.0
6.0	1.768	62.0	99.183	-1.143	-3.4	3.0	0.765	0.0	0.0	0.0
3.0	1.73	65.0	99.221	-1.105	-3.3	3.0	0.803	0.0	0.0	0.0
2.0	1.475	66.0	99.476	-0.850	-0.9	1.0	1.058	0.0	0.0	0.0
1.0	1.024	67.0	99.927	-0.399	-0.4	1.0	1.509	0.0	0.0	0.0
0.0	0.51	68.0	100.441	0.115	0.0	0.0	2.023	0.0	0.0	0.0
					95.3	67.0		6.5	15.0	35.1

Summary Data

Bankfull Width =	67.0	m
Hydraulic Mean De	1.42	m
Wet Width =	15.0	m
Wet Mean Depth =	0.43	m
Ybed =	2.34	m



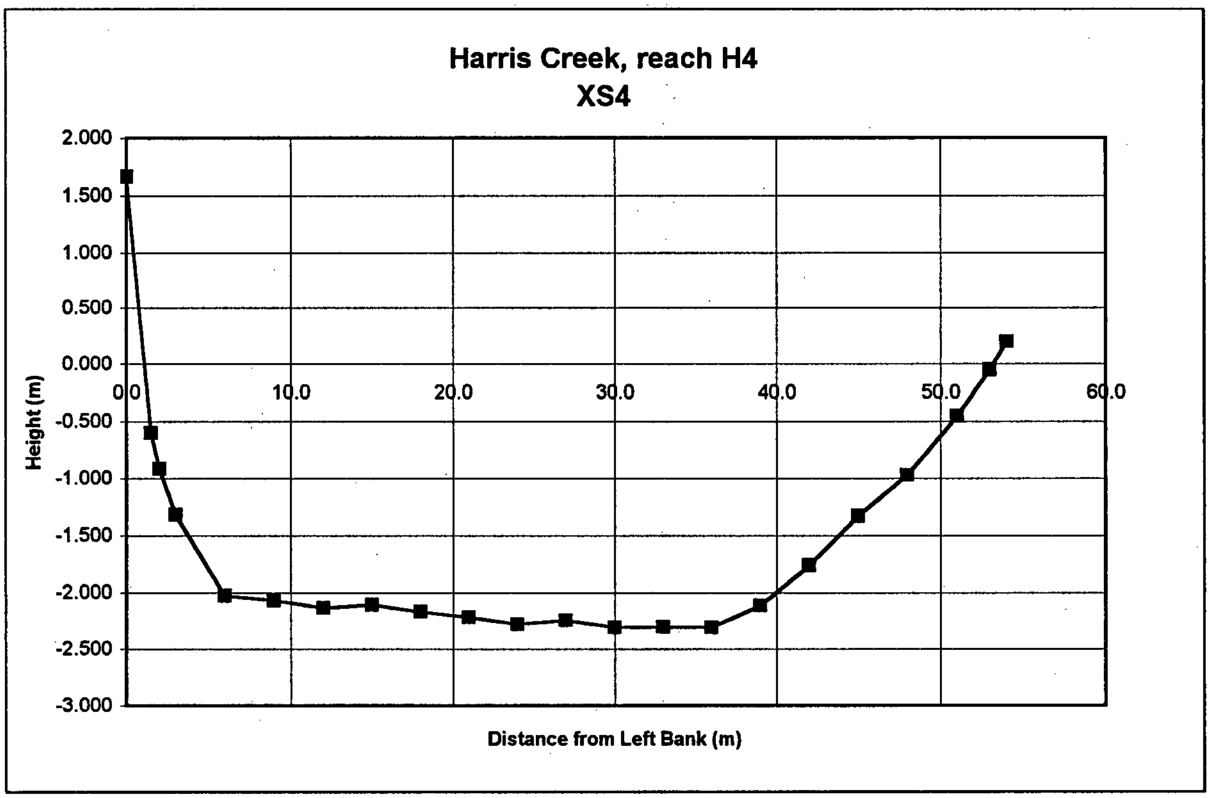
Cross Section # 4

Level Elevation:	100.751	m		FS	from BF
Bankfull FS (BF):	-0.15	m	ater elev (W	1.778	-1.928

istance (	FS (m)	from L	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
0.0	-1.812	0.0	102.563	1.662			3.590			
1.5	0.449	1.5	100.302	-0.599	-0.9	1.5	1.329	0.0	0.0	0.0
2.0	0.767	2.0	99.984	-0.917	-0.5	0.5	1.011	0.0	0.0	0.0
3.0	1.174	3.0	99.577	-1.324	-1.3	1.0	0.604	0.0	0.0	0.0
6.0	1.878	6.0	98.873	-2.028	-6.1	3.0	-0.100	-0.3	3.0	-6.1
9.0	1.918	9.0	98.833	-2.068	-6.2	3.0	-0.140	-0.4	3.0	-6.2
12.0	1.985	12.0	98.766	-2.135	-6.4	3.0	-0.207	-0.6	3.0	-6.4
15.0	1.953	15.0	98.798	-2.103	-6.3	3.0	-0.175	-0.5	3.0	-6.3
18.0	2.019	18.0	98.732	-2.169	-6.5	3.0	-0.241	-0.7	3.0	-6.5
21.0	2.067	21.0	98.684	-2.217	-6.7	3.0	-0.289	-0.9	3.0	-6.7
24.0	2.127	24.0	98.624	-2.277	-6.8	3.0	-0.349	-1.0	3.0	-6.8
27.0	2.094	27.0	98.657	-2.244	-6.7	3.0	-0.316	-0.9	3.0	-6.7
30.0	2.159	30.0	98.592	-2.309	-6.9	3.0	-0.381	-1.1	3.0	-6.9
33.0	2.153	33.0	98.598	-2.303	-6.9	3.0	-0.375	-1.1	3.0	-6.9
36.0	2.156	36.0	98.595	-2.306	-6.9	3.0	-0.378	-1.1	3.0	-6.9
39.0	1.963	39.0	98.788	-2.113	-6.3	3.0	-0.185	-0.6	3.0	-6.3
42.0	1.61	42.0	99.141	-1.760	-5.3	3.0	0.168	0.0	0.0	0.0
45.0	1.181	45.0	99.57	-1.331	-4.0	3.0	0.597	0.0	0.0	0.0
48.0	0.824	48.0	99.927	-0.974	-2.9	3.0	0.954	0.0	0.0	0.0
51.0	0.305	51.0	100.446	-0.455	-1.4	3.0	1.473	0.0	0.0	0.0
53.0	-0.1	53.0	100.851	-0.050	-0.1	2.0	1.878	0.0	0.0	0.0
54.0	-0.35	54.0	101.101	0.200	0.0	0.0	2.128	0.0	0.0	0.0
					95.2	53.0		9.4	36.0	78.8

Summary

Bankfull Width =	53.0	m
Hydraulic Mean De	1.80	m
Wet Width =	36.0	m
Wet Mean Depth =	0.26	m
Ybed =	2.19	m

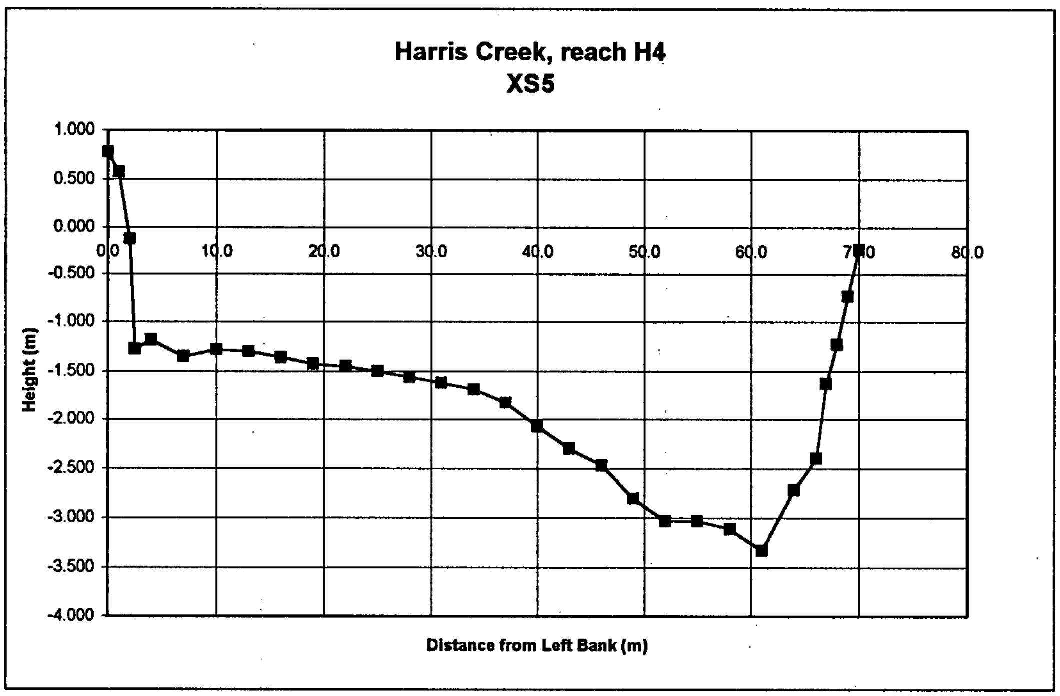


Cross Section # 5

Level Elevation:	102.152	m		FS	from BF
Bankfull FS (BF):	0.113	m	ater elev (W	2.800	-2.687

istance (m	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-1.0	-0.66	0.0	102.812	0.773			3.460			
0.0	-0.46	1.0	102.612	0.573	0.0	0.0	3.260	0.0	0.0	0.0
1.0	0.24	2.0	101.912	-0.127	-0.1	1.0	2.560	0.0	0.0	0.0
1.5	1.39	2.5	100.762	-1.277	-0.6	0.5	1.410	0.0	0.0	0.0
3.0	1.303	4.0	100.849	-1.190	-1.8	1.5	1.497	0.0	0.0	0.0
6.0	1.473	7.0	100.679	-1.360	-4.1	3.0	1.327	0.0	0.0	0.0
9.0	1.402	10.0	100.750	-1.289	-3.9	3.0	1.398	0.0	0.0	0.0
12.0	1.417	13.0	100.735	-1.304	-3.9	3.0	1.383	0.0	0.0	0.0
15.0	1.478	16.0	100.674	-1.365	-4.1	3.0	1.322	0.0	0.0	0.0
18.0	1.543	19.0	100.609	-1.430	-4.3	3.0	1.257	0.0	0.0	0.0
21.0	1.572	22.0	100.580	-1.459	-4.4	3.0	1.228	0.0	0.0	0.0
24.0	1.618	25.0	100.534	-1.505	-4.5	3.0	1.182	0.0	0.0	0.0
27.0	1.673	28.0	100.479	-1.560	-4.7	3.0	1.127	0.0	0.0	0.0
30.0	1.736	31.0	100.416	-1.623	-4.9	3.0	1.064	0.0	0.0	0.0
33.0	1.802	34.0	100.350	-1.689	-5.1	3.0	0.998	0.0	0.0	0.0
36.0	1.938	37.0	100.214	-1.825	-5.5	3.0	0.862	0.0	0.0	0.0
39.0	2.184	40.0	99.968	-2.071	-6.2	3.0	0.616	0.0	0.0	0.0
42.0	2.407	43.0	99.745	-2.294	-6.9	3.0	0.393	0.0	0.0	0.0
45.0	2.577	46.0	99.575	-2.464	-7.4	3.0	0.223	0.0	0.0	0.0
48.0	2.917	49.0	99.235	-2.804	-8.4	3.0	-0.117	-0.4	3.0	-8.4
51.0	3.142	52.0	99.010	-3.029	-9.1	3.0	-0.342	-1.0	3.0	-9.1
54.0	3.143	55.0	99.009	-3.030	-9.1	3.0	-0.343	-1.0	3.0	-9.1
57.0	3.221	58.0	98.931	-3.108	-9.3	3.0	-0.421	-1.3	3.0	-9.3
60.0	3.438	61.0	98.714	-3.325	-10.0	3.0	-0.638	-1.9	3.0	-10.0
63.0	2.83	64.0	99.322	-2.717	-8.2	3.0	-0.030	-0.1	3.0	-8.2
65.1	2.505	66.1	99.647	-2.392	-5.0	2.1	0.295	0.0	0.0	0.0
66.0	1.74	67.0	100.412	-1.627	-1.5	0.9	1.060	0.0	0.0	0.0
67.0	1.343	68.0	100.809	-1.230	-1.2	1.0	1.457	0.0	0.0	0.0
68.0	0.843	69.0	101.309	-0.730	-0.7	1.0	1.957	0.0	0.0	0.0
69.0	0.348	70.0	101.804	-0.235	-0.2	1.0	2.452	0.0	0.0	0.0
					135.0	69.0		5.7	18.0	54.0

Summary Data			
	Bankfull Width =	69.0	m
	Hydraulic Mean De	1.96	m
	Wet Width =	18.0	m
	Wet Mean Depth =	0.32	m
	Ybed =	3.00	m

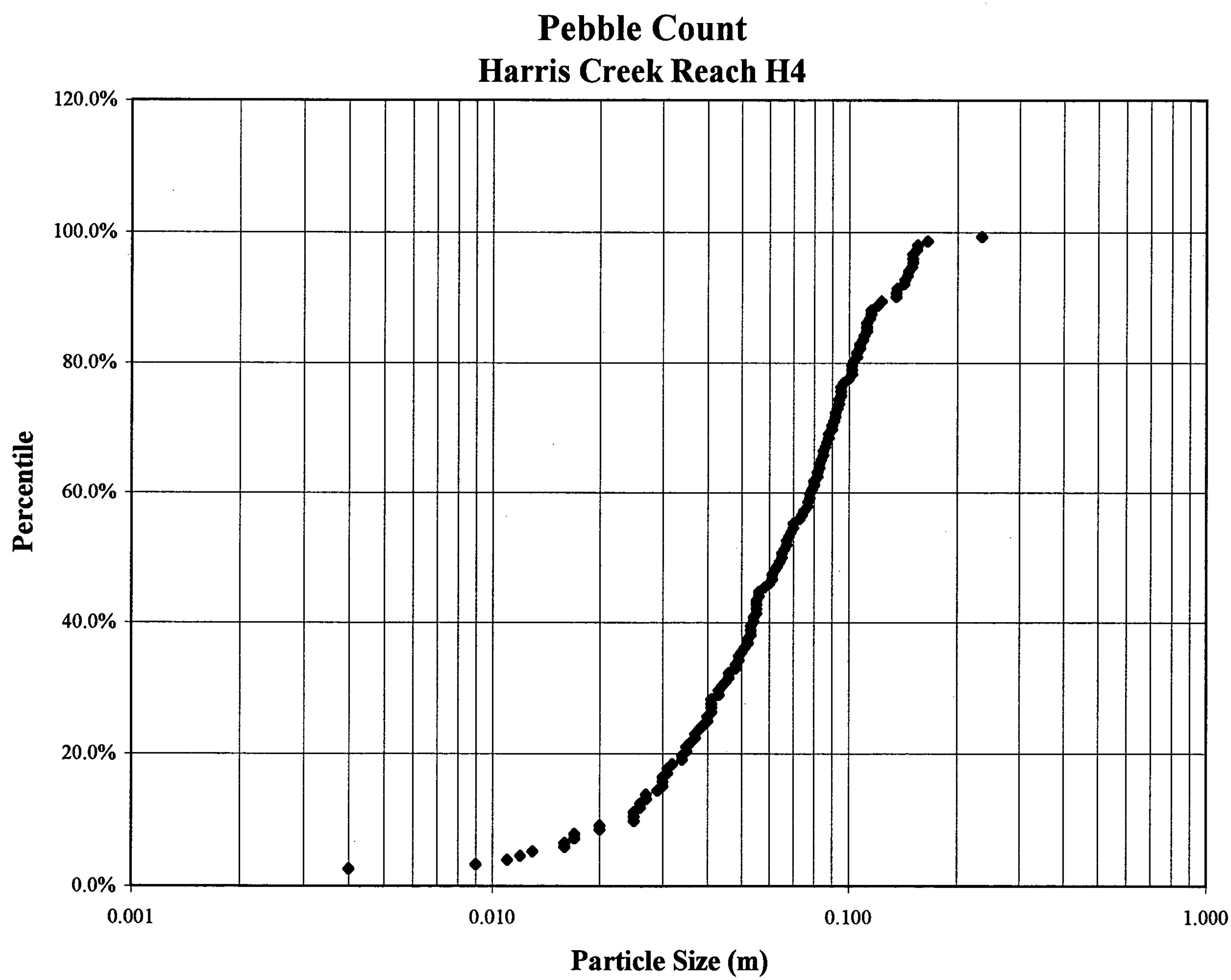


Size of Channel

	Pool w/ ov	iffle w/chut	Riffle	Pool	Glide		
X-section	5	4	3	2	1	Avg	Note
Wbf	69	53	67	31	34	51	
Y*	1.96	1.80	1.42	2.03	1.66	1.77	
W wet	18	36	15	31	31	26	
Y wet	0.32	0.26	0.43	0.56	0.48	0.41	
Ybed	3.00	2.19	2.34	2.03	1.71	2.25	

Discussion

Two types of cross-sections.


Pebble Count
Harris Creek Reach H4
Percentile
120.0%
100.0%
80.0%
60.0%
40.0%
20.0%
0.0%
0.001
0.010
0.100
1.000
Particle Size (m)

Roughness

Estimates from Flow and Bankfull Measurements

	f =	0.078	= 8*9.81*(W*Y)^2*Y*S/Q^2

Empirical Relations

Bray	ks =	0.442	ks = 6.8*D50
Keulega	f =	0.080	= (2.03log(12.2*Y/ks))^(-2)

Sediment Transport

Constants

Specific Weight =	9810	kg/m^2*s^2
Density =	1000	kg/m^3
g =	9.81	m/s^2
nu =	1E-06	m^2/s
s =	2.65	sim

Preliminary Calculations

Shear =	67	N
SFbank =	0.12	dim
bed shear =	64	N
Sheilds =	0.061	dim
Power =	147	Nm/s

Einstein-Brown

gb * =	0.004	dim
F1 =	0.82	dim
gb =	0.514	kg/ms
Gb =	22.61	kg/s

Input Data and Reach Analysis
Harris Creek
Reach H2

Data Collected
April 18 -19, 1998

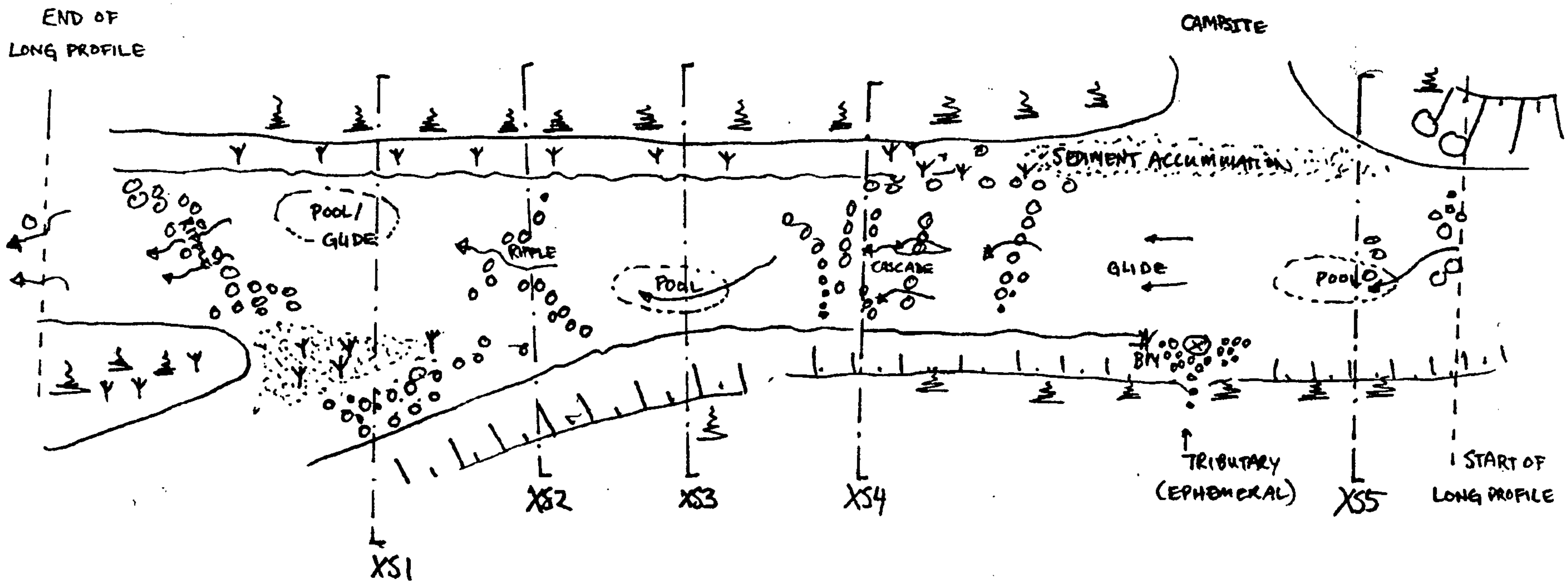
By
Bruce MacVicar
and Stephane D'Aoust

Summary Data

W =	29	m	Pbed =	19.0	m
Y =	1.9	m	Pbank =	10.7	m
S =	0.0069		Ybed =	2.25	m
			Rh =	1.6	m

Q =	120	m^3/s				
	x = 35	x = 50	x = 65	x = 84	x = 90	
Dx =	0.14	0.23	0.33	0.47	0.52	m
D50bulk =	0.017	m	D90bulk	0.070	m	
ks =	1.56	m				
	E-B					
Gb =	1465	kg/s	(of bulk material)			

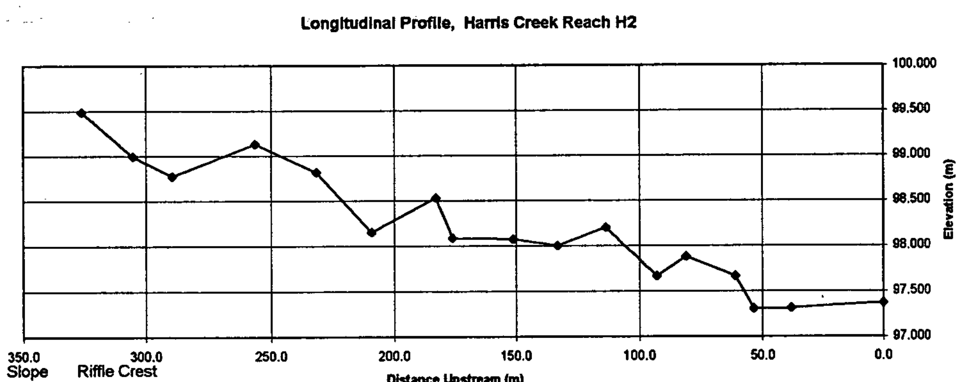
HARRIS CREEK - REACH H2
APRIL 19/1998



Longitudinal Profile

		BS	HI	FS	ELEV	
BM		0.592	100.592		100.000	
TP				0.455		
	2	1.308	101.445		100.137	
		0.632	100.769		100.137	from BM
				1.540		
	3	1.448	101.585		99.229	

	Elevation					Distance										
Note	TP elev	Level H	HI	Thalweg	Thalweg	Level D	Low	Mid	High	Dist 1	Dist 2	from lev	Thalweg	Slope		
	100	0.592	100.592	3.220	97.372	151	2.480	3.220	3.990	74	77	151.0	0.0			
XS1	100	0.592	100.592	3.275	97.317	151		3.275	3.840		56.5	113.0	38.0			
XS2	100	0.592	100.592	3.285	97.307	151	2.800	3.285	3.775	48.5	49	97.5	53.5	*		
	100	0.592	100.592	2.930	97.662	151	2.480	2.930	3.380	45	45	90.0	61.0			
	100	0.592	100.592	2.715	97.877	151	2.365	2.715	3.060	35	34.5	69.5	81.5			
	100	0.592	100.592	2.925	97.667	151	2.635	2.925	3.215	29	29	58.0	93.0			
	100	0.592	100.592	2.390	98.202	151	2.205	2.390	2.580	18.5	19	37.5	113.5		0.0073	
	100	0.592	100.592	2.592	98.000	151	2.503	2.592	2.685	8.9	9.3	18.2	132.8	*		
	100	0.592	100.592	2.520	98.072	151				0	0	0.0	151.0			
	100	0.592	100.592	2.505	98.087	151	2.383	2.505	2.635	12.2	13	25.2	176.2			
	100	0.592	100.592	2.065	98.527	151	1.910	2.065	2.230	15.5	16.5	32.0	183.0		0.0047	
	100	0.592	100.592	2.440	98.152	151	2.150	2.440	2.730	29	29	58.0	209.0			
	100	0.592	100.592	1.775	98.817	151	1.375	1.775	2.180	40	40.5	80.5	231.5			
	100	0.592	100.592	1.465	99.127	151	0.940	1.465	1.99	52.5	52.5	105.0	256.0	*	0.0130	0.0069
	100.137	1.308	101.445	2.675	98.770	265.3	2.553	2.675	2.795	12.2	12	24.2	289.5			
	100.137	1.308	101.445	2.455	98.990	265.3	2.255	2.455	2.655	20	20	40.0	305.3			
	100.137	1.308	101.445	1.960	99.485	265.3	1.650	1.960	2.260	31	30	61.0	326.3			



Slope Riffle Crest
Thalweg =

Discussion
used total slope over four riffle crests

Conclusion:
So = 0.0069

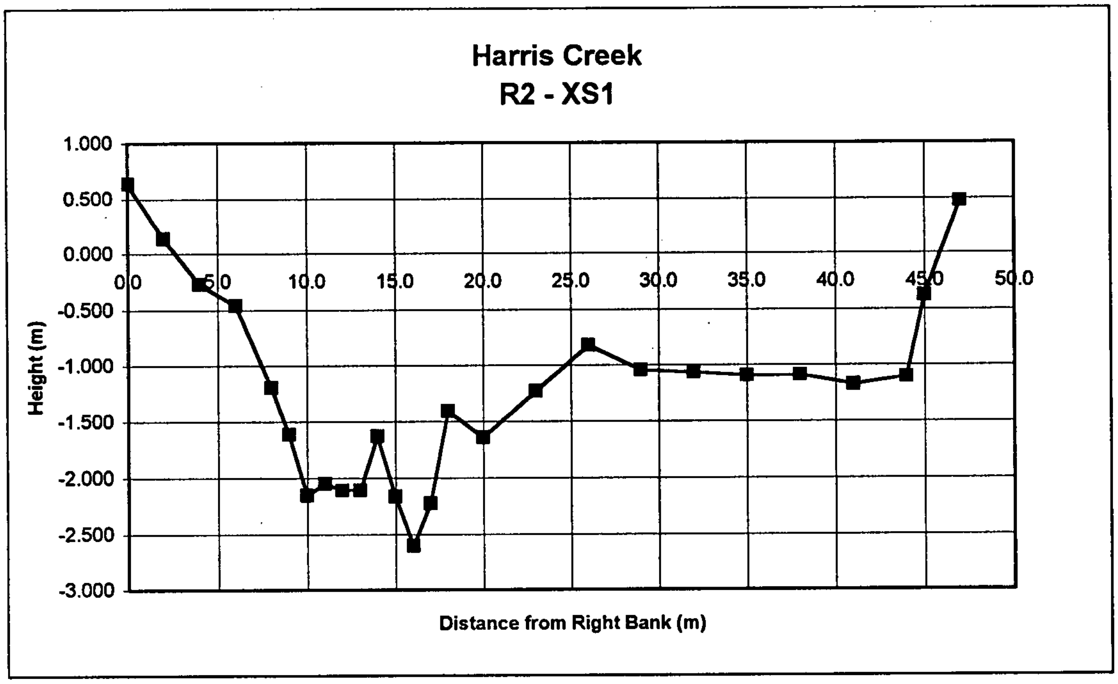
Cross Section # 1

Level Elevation:	101.585	m		FS	from BF
Bankfull FS (BF):	0.387	m	ater elev (W	1.856	-1.469

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-8.0	-0.25	0.0	101.835	0.637			2.106			
-6.0	0.25	2.0	101.335	0.137	0.0	0.0	1.606	0.0	0.0	0.0
-4.0	0.66	4.0	100.925	-0.273	-0.5	2.0	1.196	0.0	0.0	0.0
-2.0	0.847	6.0	100.738	-0.460	-0.9	2.0	1.009	0.0	0.0	0.0
0.0	1.588	8.0	99.997	-1.201	-2.4	2.0	0.268	0.0	0.0	0.0
1.0	2.006	9.0	99.579	-1.619	-1.6	1.0	-0.150	-0.2	1.0	-1.6
2.0	2.542	10.0	99.043	-2.155	-2.2	1.0	-0.686	-0.7	1.0	-2.2
3.0	2.436	11.0	99.149	-2.049	-2.0	1.0	-0.580	-0.6	1.0	-2.0
4.0	2.498	12.0	99.087	-2.111	-2.1	1.0	-0.642	-0.6	1.0	-2.1
5.0	2.494	13.0	99.091	-2.107	-2.1	1.0	-0.638	-0.6	1.0	-2.1
6.0	2.023	14.0	99.562	-1.636	-1.6	1.0	-0.167	-0.2	1.0	-1.6
7.0	2.55	15.0	99.035	-2.163	-2.2	1.0	-0.694	-0.7	1.0	-2.2
8.0	2.993	16.0	98.592	-2.606	-2.6	1.0	-1.137	-1.1	1.0	-2.6
9.0	2.616	17.0	98.969	-2.229	-2.2	1.0	-0.760	-0.8	1.0	-2.2
10.0	1.796	18.0	99.789	-1.409	-1.4	1.0	0.060	0.0	0.0	0.0
12.0	2.034	20.0	99.551	-1.647	-3.3	2.0	-0.178	-0.4	2.0	-3.3
15.0	1.62	23.0	99.965	-1.233	-3.7	3.0	0.236	0.0	0.0	0.0
18.0	1.209	26.0	100.376	-0.822	-2.5	3.0	0.647	0.0	0.0	0.0
21.0	1.435	29.0	100.150	-1.048	-3.1	3.0	0.421	0.0	0.0	0.0
24.0	1.454	32.0	100.131	-1.067	-3.2	3.0	0.402	0.0	0.0	0.0
27.0	1.483	35.0	100.102	-1.096	-3.3	3.0	0.373	0.0	0.0	0.0
30.0	1.477	38.0	100.108	-1.090	-3.3	3.0	0.379	0.0	0.0	0.0
33.0	1.565	41.0	100.020	-1.178	-3.5	3.0	0.291	0.0	0.0	0.0
36.0	1.495	44.0	100.090	-1.108	-3.3	3.0	0.361	0.0	0.0	0.0
37.0	0.77	45.0	100.815	-0.383	-0.4	1.0	1.086	0.0	0.0	0.0
39.0	-0.08	47.0	101.665	0.467	0.0	0.0	1.936	0.0	0.0	0.0
					53.6	43.0		5.8	11.0	22.0

Summary Data

Bankfull Width =	43.0	m
Hydraulic Mean De	1.25	m
Wet Width =	11.0	m
Wet Mean Depth =	0.53	m
Ybed =	2.00	m

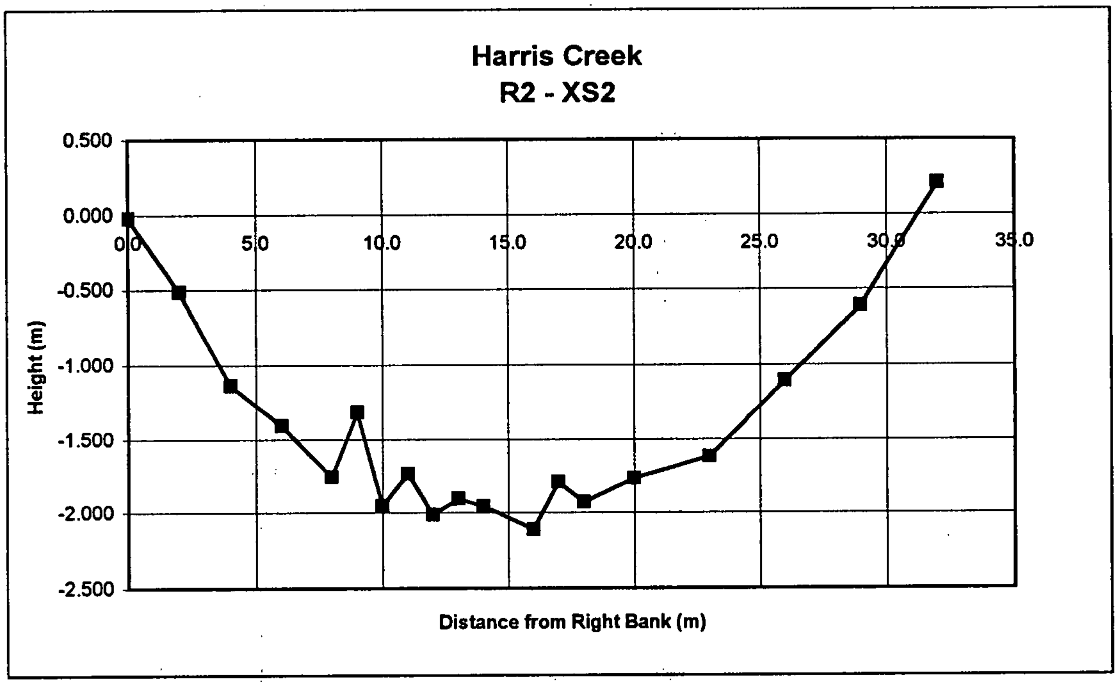


Cross Section # 2

Level Elevation:	101.564	m		FS	from BF
Bankfull FS (BF):	0.154	m	ater elev (W	1.660	-1.506

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-5.0	0.178	0.0	101.386	-0.024			1.482			
-3.0	0.670	2.0	100.894	-0.516	-1.0	2.0	0.990	0.0	0.0	0.0
-1.0	1.290	4.0	100.274	-1.136	-2.3	2.0	0.370	0.0	0.0	0.0
1.0	1.559	6.0	100.005	-1.405	-2.8	2.0	0.101	0.0	0.0	0.0
3.0	1.907	8.0	99.657	-1.753	-3.5	2.0	-0.247	-0.5	2.0	-3.5
4.0	1.470	9.0	100.094	-1.316	-1.3	1.0	0.190	0.0	0.0	0.0
5.0	2.106	10.0	99.458	-1.952	-2.0	1.0	-0.446	-0.4	1.0	-2.0
6.0	1.885	11.0	99.679	-1.731	-1.7	1.0	-0.225	-0.2	1.0	-1.7
7.0	2.167	12.0	99.397	-2.013	-2.0	1.0	-0.507	-0.5	1.0	-2.0
8.0	2.060	13.0	99.504	-1.906	-1.9	1.0	-0.400	-0.4	1.0	-1.9
9.0	2.111	14.0	99.453	-1.957	-2.0	1.0	-0.451	-0.5	1.0	-2.0
11.0	2.264	16.0	99.300	-2.110	-4.2	2.0	-0.604	-1.2	2.0	-4.2
12.0	1.947	17.0	99.617	-1.793	-1.8	1.0	-0.287	-0.3	1.0	-1.8
13.0	2.086	18.0	99.478	-1.932	-1.9	1.0	-0.426	-0.4	1.0	-1.9
15.0	1.923	20.0	99.641	-1.769	-3.5	2.0	-0.263	-0.5	2.0	-3.5
18.0	1.772	23.0	99.792	-1.618	-4.9	3.0	-0.112	-0.3	3.0	-4.9
21.0	1.260	26.0	100.304	-1.106	-3.3	3.0	0.400	0.0	0.0	0.0
24.0	0.765	29.0	100.799	-0.611	-1.8	3.0	0.895	0.0	0.0	0.0
27.0	-0.050	32.0	101.614	0.204	0.0	0.0	1.710	0.0	0.0	0.0
					42.0	29.0		5.3	16.0	29.4

Summary Data			
	Bankfull Width =	29.0	m
	Hydraulic Mean De	1.45	m
	Wet Width =	16.0	m
	Wet Mean Depth =	0.33	m
	Ybed =	1.84	m

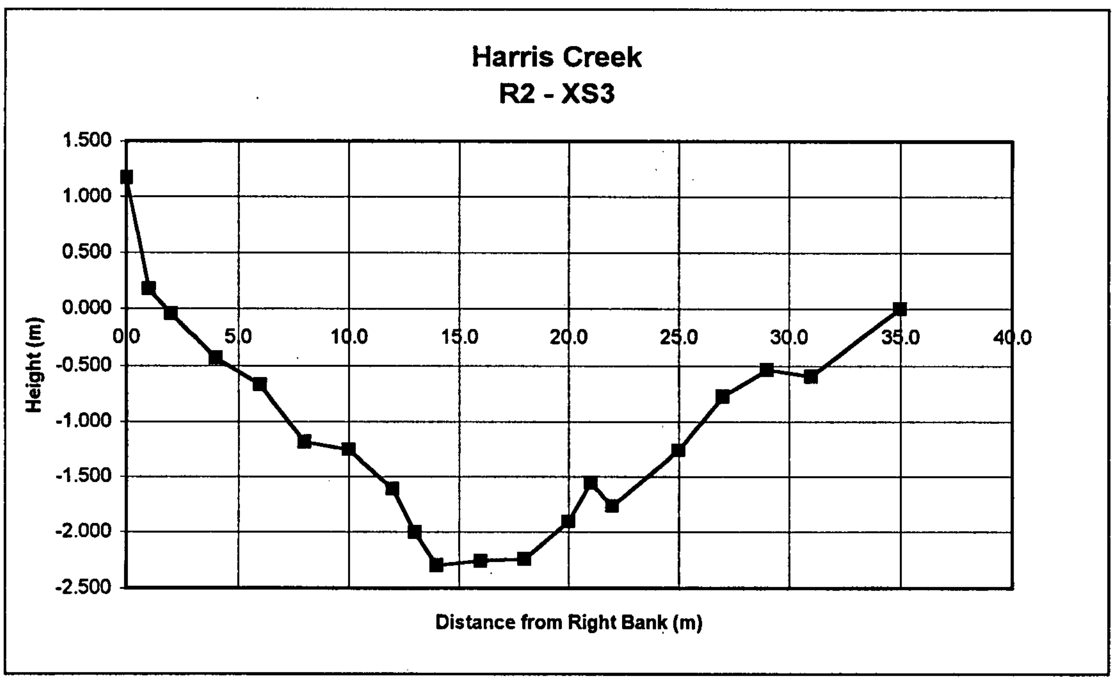


Cross Section # 3

Level Elevation:	100.769	m		FS	from BF
Bankfull FS (BF):	0.176	m	ater elev (W	1.72	-1.544

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-4	-1	0.0	101.769	1.176			2.720			
-3.0	0	1.0	100.769	0.176	0.0	0.0	1.720	0.0	0.0	0.0
-2.0	0.22	2.0	100.549	-0.044	0.0	1.0	1.500	0.0	0.0	0.0
0.0	0.612	4.0	100.157	-0.436	-0.9	2.0	1.108	0.0	0.0	0.0
2.0	0.844	6.0	99.925	-0.668	-1.3	2.0	0.876	0.0	0.0	0.0
4.0	1.366	8.0	99.403	-1.190	-2.4	2.0	0.354	0.0	0.0	0.0
6.0	1.436	10.0	99.333	-1.260	-2.5	2.0	0.284	0.0	0.0	0.0
8.0	1.789	12.0	98.980	-1.613	-3.2	2.0	-0.069	-0.1	2.0	-3.2
9.0	2.185	13.0	98.584	-2.009	-2.0	1.0	-0.465	-0.5	1.0	-2.0
10.0	2.473	14.0	98.296	-2.297	-2.3	1.0	-0.753	-0.8	1.0	-2.3
12.0	2.437	16.0	98.332	-2.261	-4.5	2.0	-0.717	-1.4	2.0	-4.5
14.0	2.416	18.0	98.353	-2.240	-4.5	2.0	-0.696	-1.4	2.0	-4.5
16.0	2.089	20.0	98.680	-1.913	-3.8	2.0	-0.369	-0.7	2.0	-3.8
17.0	1.733	21.0	99.036	-1.557	-1.6	1.0	-0.013	0.0	1.0	-1.6
18.0	1.945	22.0	98.824	-1.769	-1.8	1.0	-0.225	-0.2	1.0	-1.8
21.0	1.442	25.0	99.327	-1.266	-3.8	3.0	0.278	0.0	0.0	0.0
23.0	0.947	27.0	99.822	-0.771	-1.5	2.0	0.773	0.0	0.0	0.0
25.0	0.715	29.0	100.054	-0.539	-1.1	2.0	1.005	0.0	0.0	0.0
27.0	0.768	31.0	100.001	-0.592	-1.2	2.0	0.952	0.0	0.0	0.0
31.0	0.176	35.0	100.593	0.000	0.0	0.0	1.544	0.0	0.0	0.0
					38.4	30.0		5.2	12.0	23.7

Summary Data			
	Bankfull Width =	30.0	m
	Hydraulic Mean De	1.28	m
	Wet Width =	12.0	m
	Wet Mean Depth =	0.43	m
	Ybed =	1.97	m

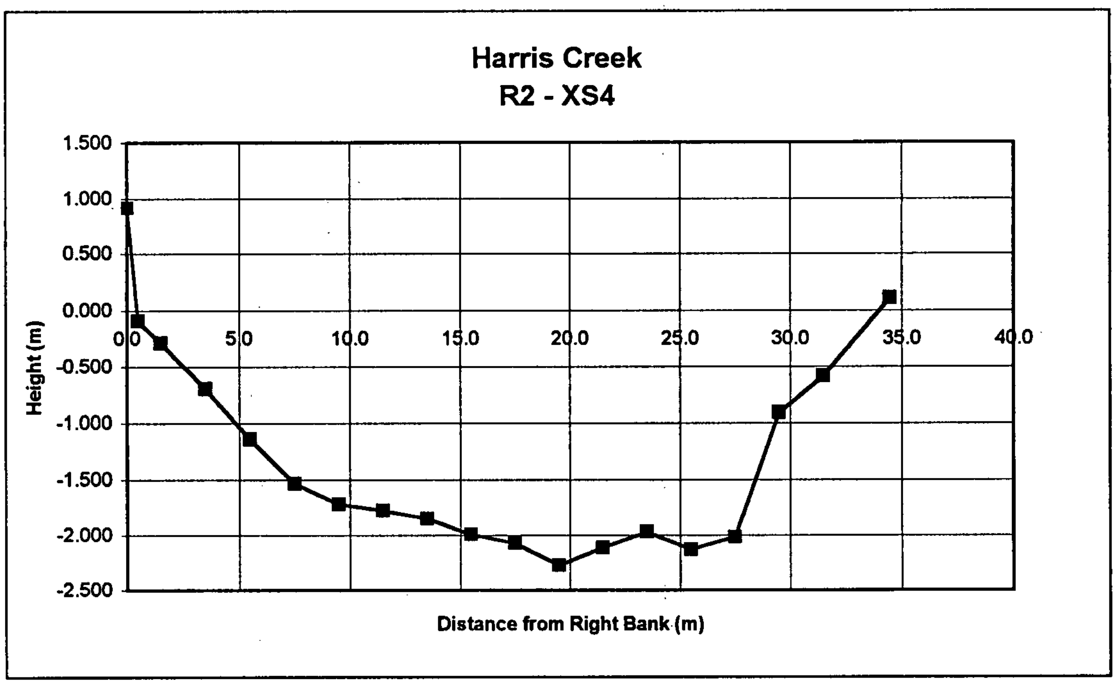


Cross Section # 4

Level Elevation:	101.394	m		FS	from BF
Bankfull FS (BF):	0.01	m	ater elev (W	1.858	-1.848

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-4.5	-0.900	0.0	102.294	0.910			2.758			
-4.0	0.100	0.5	101.294	-0.090	0.0	0.5	1.758	0.0	0.0	0.0
-3.0	0.303	1.5	101.091	-0.293	-0.4	1.5	1.555	0.0	0.0	0.0
-1.0	0.707	3.5	100.687	-0.697	-1.4	2.0	1.151	0.0	0.0	0.0
1.0	1.154	5.5	100.24	-1.144	-2.3	2.0	0.704	0.0	0.0	0.0
3.0	1.545	7.5	99.849	-1.535	-3.1	2.0	0.313	0.0	0.0	0.0
5.0	1.730	9.5	99.664	-1.720	-3.4	2.0	0.128	0.0	0.0	0.0
7.0	1.786	11.5	99.608	-1.776	-3.6	2.0	0.072	0.0	0.0	0.0
9.0	1.858	13.5	99.536	-1.848	-3.7	2.0	0.000	0.0	0.0	0.0
11.0	2.001	15.5	99.393	-1.991	-4.0	2.0	-0.143	-0.3	2.0	-4.0
13.0	2.080	17.5	99.314	-2.070	-4.1	2.0	-0.222	-0.4	2.0	-4.1
15.0	2.280	19.5	99.114	-2.270	-4.5	2.0	-0.422	-0.8	2.0	-4.5
17.0	2.123	21.5	99.271	-2.113	-4.2	2.0	-0.265	-0.5	2.0	-4.2
19.0	1.979	23.5	99.415	-1.969	-3.9	2.0	-0.121	-0.2	2.0	-3.9
21.0	2.138	25.5	99.256	-2.128	-4.3	2.0	-0.280	-0.6	2.0	-4.3
23.0	2.023	27.5	99.371	-2.013	-4.0	2.0	-0.165	-0.3	2.0	-4.0
25.0	0.919	29.5	100.475	-0.909	-1.8	2.0	0.939	0.0	0.0	0.0
27.0	0.596	31.5	100.798	-0.586	-1.2	2.0	1.262	0.0	0.0	0.0
30.0	-0.100	34.5	101.494	0.110	0.0	0.0	1.958	0.0	0.0	0.0
					50.0	32.0		3.2	14.0	29.1

Summary			
	Bankfull Width =	32.0	m
	Hydraulic Mean De	1.56	m
	Wet Width =	14.0	m
	Wet Mean Depth =	0.23	m
	Ybed =	2.08	m



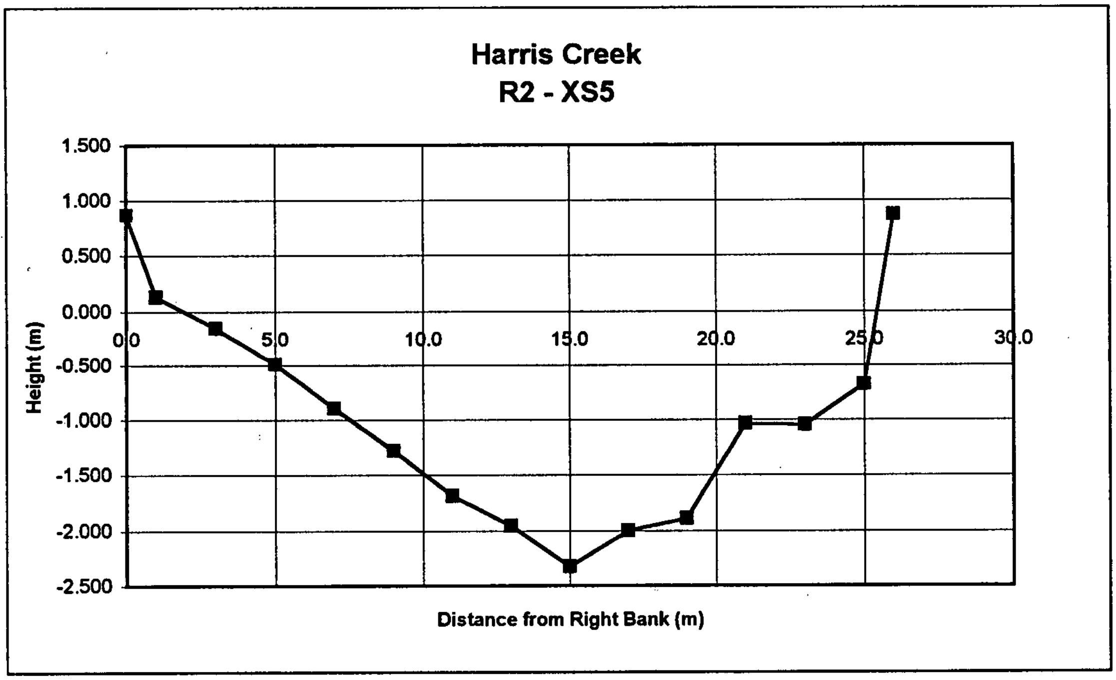
Cross Section # 5

Level Elevation:	101.417	m		FS	from BF
Bankfull FS (BF):	0.362	m	ater elev (W	1.640	-1.278

istance (	FS (m)	from R	elevation	from BF	Y*W	BF W	from WE	Y*W	Wet W	Ybed
-1.0	-0.5	0.0	101.917	0.862			2.140			
0.0	0.232	1.0	101.185	0.130	0.0	0.0	1.408	0.0	0.0	0.0
2.0	0.515	3.0	100.902	-0.153	-0.3	2.0	1.125	0.0	0.0	0.0
4.0	0.848	5.0	100.569	-0.486	-1.0	2.0	0.792	0.0	0.0	0.0
6.0	1.262	7.0	100.155	-0.900	-1.8	2.0	0.378	0.0	0.0	0.0
8.0	1.645	9.0	99.772	-1.283	-2.6	2.0	-0.005	0.0	2.0	-2.6
10.0	2.052	11.0	99.365	-1.690	-3.4	2.0	-0.412	-0.8	2.0	-3.4
12.0	2.32	13.0	99.097	-1.958	-3.9	2.0	-0.680	-1.4	2.0	-3.9
14.0	2.69	15.0	98.727	-2.328	-4.7	2.0	-1.050	-2.1	2.0	-4.7
16.0	2.365	17.0	99.052	-2.003	-4.0	2.0	-0.725	-1.5	2.0	-4.0
18.0	2.255	19.0	99.162	-1.893	-3.8	2.0	-0.615	-1.2	2.0	-3.8
20.0	1.4	21.0	100.017	-1.038	-2.1	2.0	0.240	0.0	0.0	0.0
22.0	1.41	23.0	100.007	-1.048	-2.1	2.0	0.230	0.0	0.0	0.0
24.0	1.04	25.0	100.377	-0.678	-1.4	2.0	0.600	0.0	0.0	0.0
25.0	-0.5	26.0	101.917	0.862	0.0	0.0	2.140	0.0	0.0	0.0
					30.9	24.0		7.0	12.0	22.3

Summary Data

Bankfull Width =	24.0	m
Hydraulic Mean De	1.29	m
Wet Width =	12.0	m
Wet Mean Depth =	0.58	m
Ybed =	1.86	m



Size of Channel

	Pool	Cascade	Pool	Riffle	Pool/split		
X-section	5	4	3	2	1	Avg	Note
Wbf	24	32	30	29	43	29	XS 1 not used
Ybf	1.29	1.56	1.28	1.45	1.25	1.40	
W wet	12	14	12	16	11	13	
Y wet	0.58	0.23	0.43	0.33	0.53	0.42	
Ybed	1.86	2.08	1.97	1.84	2.00	1.95	
W terrac	25	34	37	33		32	

Discussion

Conclusions

Wbf = 29 m

Ybf = 1.4 m

Roughness

From Flow and Bankfull Measurements

f = 0.180 = 8*9.81*(W*Y)^2*Y*S/Q^2

From Empirical Relations

Bray ks = 1.564 ks = 6.8*D50

Keulega f = 0.180 = (2.03log(12.2*Y/ks))^(-2)

Sediment Transport

Constants

			Bulk Sediment Sample
Specific Weight =	9810	kg/m^2*s^2	
Density =	1000	kg/m^3	
g =	9.81	m/s^2	
nu =	0.000001	m^2/s	
s =	2.65	sim	

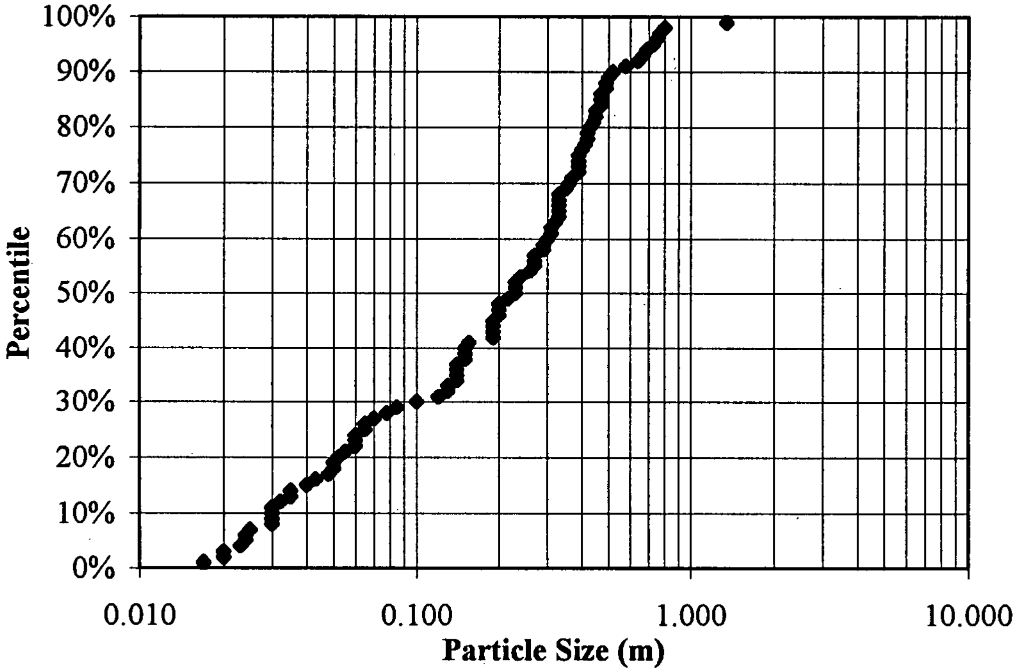
Preliminary Calculations

Shear =	152	N	
SFbank =	0.34	dim	
bed shear =	128	N	
Sheilds =	0.034	dim	0.464
Power =	235	Nm/s	

Einstein-Brown

gb * =	0.000	dim	3.998
F1 =	0.82	dim	0.82
gb =	0.023	kg/ms	77.088
Gb =	0.4	kg/s	1465

Pebble Count
Harris Creek Reach H2



Bulk Sample
Harris Creek Reach H2