HYDRAULICS OF EAST CREEK, BRITISH COLUMBIA: A HEADWATER STREAM WITH HIGH RELATIVE ROUGHNESS

by

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Abstract

Headwater streams provide important aquatic habitat, link hillslope sediments with lowland systems, and are an extensive and important component of the fluvial system. Headwater streams are characterised by steep gradients, poorly sorted sediments, large roughness elements and often exhibit diverse morphologies, described as step-pool, rapid or riffle-pool. Despite their importance, headwater streams have received relatively little attention compared to lowland systems and analysis has centred on approaches developed lowland rivers assuming hydrostatic pressure distributions and low channel gradients. In order to investigate the applicability of these methods in a small headwater streams, two hydrodynamic models were applied.

The 1-dimensional model, HEC-RAS and 2-dimensional model, River 2D were use to characterise the hydraulics of East Creek, British Columbia. Two reaches, one with riffle-pool morphology, the other with rapid morphology were studied. Both reaches exhibit high relative roughness. The hydrodynamic models were calibrated to observed water surface data by adjusting the roughness coefficient employed by the model and the results were compared to observed depths and velocities. Two bedload transport formulae were used in conjunction with the models to provide an indirect test of the hydraulic results.

Both hydrodynamic models were able to replicate the observed conditions with a reasonable degree of accuracy, although extensive calibration was required, particularly in HEC-RAS and the roughness coefficient incorporated many resistance components (e.g grain roughness, form roughness). Similar results were found in certain areas with River 2D. When applied predicatively, neither model accurately captured the variation in flow resistance and thus calculated results deviated from the observed conditions. It was also found that River 2D was unable to model very low flows were form resistance from large bed material was high. Results of the sediment transport analysis were also poor, but further consideration of both hydrodynamic and bedload transport model resolution may provide more reliable results.
Table of Contents

Abstract..........................................................................................................................ii
Table of Contents ..........................................................................................................iii
List of Tables ..................................................................................................................v
List of Figures .................................................................................................................vii
Nomenclature ................................................................................................................xii
Acknowledgements ........................................................................................................xiv

1.0 Introduction ..............................................................................................................1
  1.1 Objectives .............................................................................................................3
  1.2 Thesis Outline .....................................................................................................3

2.0 Literature Review .....................................................................................................5
  2.1 Hydraulics of Headwater Streams ......................................................................5
    2.1.1 Open Channel Flow ...................................................................................5
    2.1.2 Hydrostatic Pressure Distribution .............................................................6
    2.1.3 Flow Resistance .........................................................................................7
  2.2 Sediment Transport ..........................................................................................11
    2.2.1 Bedload Transport Formulae ..................................................................14
  2.3 Hydrodynamic Modelling ....................................................................................15
    2.3.1 HEC-RAS ...............................................................................................15
    2.3.2 River 2D .................................................................................................17

3.0 Methodology ..........................................................................................................20
  3.1 Site Description ..................................................................................................20
  3.2 Water Surface Profiles .......................................................................................22
  3.3 Discharge and Velocity Measurements .............................................................27
  3.4 Topography .........................................................................................................30
  3.5 Sediment and Grain Size Analysis ....................................................................32
  3.6 Data Analysis ......................................................................................................33
    3.6.1 ArcMap ....................................................................................................33
    3.6.2 River 2D .................................................................................................34
    3.6.3 HEC-RAS ...............................................................................................35
List of Tables

Table 3.1: Principle characteristics of the rapid and riffle-pool reaches

Table 3.2. Typical water surface fluctuation, thus uncertainty, in staff gauge readings

Table 3.3. Surface and sub-surface characteristic grain size

Table 4.1. Error and standard deviation of River 2D results as a percent of the observed depth and velocity. Velocity was measured at 0.4 times depth from the bed. Outliers were not included as when depth and velocity get small near the banks, a small absolute difference generates a large error

Table 4.2. Summary of difference between observed and calculated flow depth in the Rapid reach. Difference between observed and calculated water surfaces, thus flow depth, was used as the primary calibration for the models. Difference is calculated as (observed depth) – (calculated depth) and % difference = 100*(observed depth – calculated depth)/(observed depth). Negative values indicate the calculated depth is greater than the observed depth, hence flow resistance is over predicted. Highlighted cells identify the calibrated flow

Table 4.3. Summary of difference between observed and calculated flow depth in the riffle-pool reach. Values are calculated as described in Table 4.2. The 0.85 m³/s and 1.29 m³/s flow are compared to crest gauge data and are biased, thus not directly comparable with the other flows. Highlighted cells identify the calibrated flow
Table 5.1. Observed and calculated sediment transport quantities using the Wilcock and Crowe (2003) formula. Hydraulic data is from River 2D and HEC-RAS. Calibrated $D_{50}$ is a $D_{50}$ that gave a good match between observed and calculated transport for the 25 November event with the River 2D data.

Table 5.2. Observed and calculated sediment transport quantities using the Meyer-Peter and Muller formula. Results based on surface and subsurface grain size distribution (GSD) are compared.
List of Figures

Figure 2.1. A) While the flow is turbulent a temporally averaged velocity can be assumed. B) Velocity profiles near large roughness elements may be irregular and non-logarithmic, but spatially averaged profiles can be assumed logarithmic. C) Depth averaged velocity assumed in 1D and 2D forms of the conservation equations. ................................................................. 6

Figure 2.2. Momentum balance for open channel flow where $F$ is hydrostatic force, $L$ is the distance between adjacent cross-sections, $\tau_o$ is bed shear stress, $u$ is averaged velocity, $Y$ is flow depth and $Z$ is bed elevation. .................................................. 8

Figure 3.1. East Creek is a small headwater stream located in the Malcolm Knapp Research Forest ................................................................. 21

Figure 3.2. Rapid reach showing the location of staff and crest gauges (dots) and fixed cross sections (x’s). The reach was modelled from cross-section 6 – 20. A channel spanning sediment trap is located between cross-sections 19 and 20. Air photos where taken from a 10m pole held above the stream. Scale bar is indicative only. ........................................... 24

Figure 3.3. Riffle-pool reach showing the location of staff and crest gauges (dots) and fixed cross sections (x’s). The reach was modelled from cross-section 30 – 44. A channel spanning sediment trap is located at cross-sections 39. Air photos where taken from a 10m pole held above the stream. Scale bar is indicative only. ........................................... 25

Figure 3.4. A) and B) rapid reach at low and near bank-full flow respectively; C) and D) riffle-pool reach at low and near bank-full flow respectively. Bank-full flow $\approx 0.5 \text{ m}^3/\text{s}$. ................................................................. 26
Figure 3.4 cont. E) and F) step-pool reach at low and near bank-full flow respectively. Bank-full flow = 0.5 m³/s. Modelling was not undertaken in the step-pool reach due to complexity and extreme spatial variability of the channel topography.

Figure 3.5. Discharge-pressure (equivalent to stage) rating curve. A piecewise relationship was fit to the data.

Figure 3.6. Hydrograph for East Creek, British Columbia. Observed discharges are shown with black dots. $Q_2$ is estimated at 2.0 m³/s.

Figure 3.7 Rapid reach A) showing the resolution of survey data. Resolution is similar in the riffle-pool reach. B) River 2D bed topography with 0.1m contours. The section shown can be seen in Figure 3.4 A). Note the log visible in the photo and River 2D bed map. Large roughness elements are not included.

Figure 3.8. Surface and subsurface grain size distribution for the rapid and riffle-pool reaches.

Figure 3.9. Smoothed cross-section topography. The dashed line represents model topography used in HEC-RAS and River 2D.

Figure 3.10. Location of rapid reach HEC-RAS cross-sections. Cross-section spacing was similar in the riffle-pool reach.

Figure 4.1. A) Observed versus calculated water surface elevation for the riffle-pool reach. The outlier is due to woody debris, which was not included in the model. B) Depth and, C) velocity at Cross-Section 35. All results are for a calibrated River 2D discharge of 0.62 m³/s. Observations are shown as open diamonds in B) and C).
Figure 4.2. A) Calculated water surface profile and, B) Observed depth versus HEC-RAS calculated depth for the riffle-pool reach. The model is calibrated to the 0.62 m$^3$/s discharge. Observations are shown as open diamonds. The outlying point at 36m upstream (also seen as outlier in B)) is due to woody debris, which was not included in the model.

Figure 4.3. Comparison of spatial variation of roughness coefficients in the rapid reach for the 1D and 2D models. HEC-RAS (1D) was calibrated to 0.62 m$^3$/s and 0.04 m$^3$/s flows. River 2D was calibrated to 0.62 m$^3$/s. Note that $k_s$ is plotted on a logarithmic axis.

Figure 4.4. Comparison of spatial variation of roughness coefficients in the riffle-pool reach for the 1D and 2D models. HEC-RAS was calibrated to 0.62 m$^3$/s and 0.04 m$^3$/s flows. River 2D was calibrated to 0.62 m$^3$/s and 0.13 m$^3$/s flows. Both models required large variation in the roughness coefficient in order to provide good agreement with the observed conditions. Note that $k_s$ is plotted on a logarithmic axis.

Figure 4.5. Riffle–pool reach. Observed versus calculated flow depths. The legend shows modelled flow with calibration flow in brackets. A) River 2D B) HEC-RAS.

Figure 4.6. Rapid reach observed versus calculated flow. The legend shows modelled flow with calibration flow brackets. A) River 2D and B) HEC-RAS.

Figure 4.7. Variation in inverse relative roughness at $Q = 0.62$ m$^3$/s for the rapid reach. If $d/k_s < 0.62$ (shown as dashed line), resistance is calculated via Equation (4.4).

Figure 4.8. Variation in inverse relative roughness for the riffle-pool reach. If $d/k_s < 0.62$ (shown as dashed line), resistance is calculated via Equation (4.4).
Figure 4.9. River 2D $k_s$ and HEC-RAS $k_s$ (calculated from Equation (4.11)) for the rapid reach calibrated to the flow of 0.62 m$^3$/s. Note that River 2D $k_s$ is plotted against the right ordinate on a logarithmic scale. HEC-RAS $k_s$ is highly variable despite constant $D_{84}$. Variability in both HEC-RAS and River 2D $k_s$ is probably associated with form resistance and rapid changes in topography, which cannot be captured by the model, thus artificially high resistance values are required to replicate observed depths.

Figure 4.10. River 2D $k_s$ and HEC-RAS $k_s$ (calculated from Equation (4.11)) for the riffle-pool reach calibrated to the flow of 0.62 m$^3$/s. HEC-RAS $k_s$ is highly variable despite constant $D_{84}$. River 2D $k_s$ can be considered a multiple of $D_{84}$ except between 25 and 38m where it is likely that form resistance is controlling the flow depth.

Figure 4.11. Comparison between staff gauge water surface at 0.38 m$^3$/s and Crest gauges at 0.40 m$^3$/s, the bias reflecting the higher discharge for the crest gauge readings.

Figure 4.12. $k_s/D_{84}$ for the riffle-pool reach calibrated to 0.62 m$^3$/s. Reach average $k_s/D_{84}$ (excluding 25 to 38m) is 4.5.

Figure 4.13. The outcrop on the right bank is included in the model topography but the hydraulics may not be correctly represented.

Figure 5.1. Shear stress distribution calculated with River 2D and the section average shear from HEC-RAS at cross section above pit trap for 1.6 m$^3$/s.

Figure 5.2. Flow duration curve for the two modelled events showing significant sediment transporting discharges (>0.5 m$^3$/s). Although the peak discharge of the December event was higher, it was a more flashy flow.
Figure 5.3. Sediment transport rate, $q_b \, [g/m/s]$ for A) $0.5 \, m^3/s$ B) $1.0 \, m^3/s$. The data was plotted as $\log(q_b)$ showing transport rates $>0.1 \, g/m/s$. The sediment trap is located at the very downstream extent of the modelled reach (left of plot).
Nomenclature

\( a \)  Coefficient (Colebrook-White formula)
\( A \)  Cross-sectional area
\( \alpha \)  Velocity weighting coefficient
\( \beta \)  Bed slope angle (Figure 2.2)
\( \beta' \)  Momentum correction factor (Equation (2.9))
\( B_r \)  Channel shape coefficient
\( c \)  Coefficient (Colebrook-White formula)
\( C \)  Chezy resistance coefficient
\( C_i \)  Coefficient proportional to representative grain size associated of \( i \)th Percentile.
\( C_s \)  Non-dimensional Chezy coefficient
\( d \)  Hydraulic mean depth
\( D_{x0} \)  \( D_{20} \) of surface material
\( D_i \)  Grain size associated with \( i \)th percent finer e.g. \( D_{84} \)
\( E \)  Expansion or contraction coefficient
\( \varepsilon \)  Coefficient
\( f \)  Darcy-Weisbach resistance coefficient
\( f_i \)  Fraction of \( i \)th sediment size range in surface sediment
\( F \)  Hydrostatic force
\( g \)  Gravitational acceleration
\( h_e \)  Energy head loss
\( H \)  Flow depth at a point
\( \kappa \)  von-Karman’s constant
\( k_s \)  Nikuradse’s roughness length
\( K \)  Conveyance
\( K_b/K_g \)  Shear stress reduction factor (Meyer-Peter and Muller)
\( L \)  Distance between cross-sections
\( m \)  Mass
\( n \)  Manning’s roughness coefficient
\( v \) Eddy viscosity coefficient
\( p_i \) Fraction of \( i \)th sediment size range in bedload
\( q \) Discharge per unit width
\( q_b \) Bedload transport rate
\( Q \) Discharge
\( Q_2 \) 2-year return period flow
\( Q_b \) Bedload transport
\( Q_b/Q \) Flow reduction factor (Meyer-Peter and Muller)
\( R \) Hydraulic radius
\( R_{eg} \) Grain Reynolds number
\( \rho \) Fluid density
\( \rho_s \) Sediment density
\( s \) standard deviation of bed elevation
\( S_f \) Friction slope
\( S_o \) Bed slope
\( t \) Time
\( \tau_c \) Critical shear stress
\( \tau_o \) Bed shear stress
\( \theta \) Shield's dimensionless shear stress
\( \tau_{ri} \) Reference shear stress
\( \phi \) \( \tau/\tau_{ri} \)
\( u \) Average velocity in the downstream direction
\( \bar{u} \) Shear velocity
\( U \) Depth-averaged velocity in x-direction
\( V \) Depth-averaged velocity in y-direction
\( W \) Width
\( W^* \) Dimensionless transport rate
\( X_S \) Cross-section
\( \bar{Y} \) Distance from water surface to centroid of cross-section area
\( Y \) Average water depth
\( Z \) Channel invert depth
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1.0 Introduction

Legislative requirements and increasing awareness of the importance of riparian vegetation and in-stream habitat diversity (Walker et al, 2004) mean that effort to utilise and manage water resources in a sustainable manner are being made. Watersheds are being studied as holistic units. Mountain and headwater streams provide valuable habitat for benthic invertebrates and fish, transport hillslope sediments to the lowland river system, and contribute significantly to the geomorphology of the entire fluvial system (Montgomery and Buffington, 1997). Further, flooding and debris flows present serious risk to many new and existing developments. Headwater streams are characterised by steep gradients and poorly-sorted sediments. Despite their importance in the fluvial system, large and rapid spatial and temporal variations in fluid and channel boundary conditions means characterisation of their hydraulics is difficult. Typical morphologies of such streams include cascade, step-pool and riffle-pool (Montgomery and Buffington, 1997).

Watercourse morphology represents a complex relationship between sediment supply, flow resistance, transport capacity and external influences (e.g. bedrock outcrops, woody debris or human intervention) (Montgomery and Buffington, 1997). During the 2004/05 winter, detailed coupled depth, velocity, discharge and water surface measurements were collected for a range of discharges, from low flow to above bank-full, at East Creek, British Columbia. East Creek is a low order stream displaying diverse morphologies and two study reaches were investigated. The first is described as rapid morphology (Zimmerman and Church, 2001); the second, riffle-pool morphology (Montgomery and Buffington, 1997). Slope and relative roughness ($D_{80}/d$, were $d$ is hydraulic depth) of the rapid reach are 0.024 and 0.53 respectively, while the riffle-pool reach is less steep and has lower relative roughness, namely 0.02 and 0.32 respectively. Two hydrodynamic models, HEC-RAS and River 2D, were applied to East Creek in order to investigate the capabilities of such models in steep streams with diverse morphologies, and provide
hydraulic information for sediment transport, bed stability and channel morphology studies.

The rapid increase in personal computer processing speed means that 1-, 2- and even 3-dimensional (1D, 2D and 3D) hydrodynamic models are becoming common place in industry and research applications. Although, the relative ease in achieving results does not necessarily translate to accurate results, and understanding of the assumptions and limitations is essential. Hydrodynamic models use numerical techniques to solve the equations of conservation of mass, energy and momentum. HEC-RAS is a 1D model and solves the conservation equations between consecutive downstream cross-sections and flow conditions are averaged across the section. River 2D is a 2D model solving the conservation equations in the x- and y-directions assuming a depth-averaged vertical velocity profile. In order to solve these simplified conservation equations a flow resistance model is required to quantify energy loss due to turbulence. In shallow flows resistance is primarily generated by interaction between the channel boundary and fluid, and is typically quantified with Manning’s or Kuelegan type equations (Section 2.1), which were derived for steady flow conditions, assuming mild channel slopes and hydrostatic pressure distribution (Henderson, 1966). Despite many attempts to describe the flow resistance in streams with high relative roughness and steep channel morphologies (e.g. Thompson and Campbell, 1979; Jarrett, 1984; Wiberg and Smith, 1991; Rosport, 1997; Curran and Wohl, 2002; Lee and Ferguson, 2002; Aberle and Smart, 2003), these conditions do not conform to the assumptions inherent in resistance equations and the results remain semi-empirical (Aberle and Smart, 2003).

In order to model sediment transport and channel morphology, hydrodynamic models must be able to accurately characterise flow hydraulics. Sediment transport and channel morphology can present risk to developments in and near water courses, can influence flood water levels and flood hazards, and impact aquatic habitats and water quality. The Meyer-Peter and Muller (1948) and Wilcock and Crowe (2003) bedload transport formulae were applied to the riffle-pool reach as an indirect test of the modelled hydraulics. Calculated versus observed transport comparisons were unreliable, which
was attributed to both the calculated hydraulics and bedload transport formulae, and ultimately the challenges of working in headwater streams where topography and hydraulics are highly spatially and temporally variable.

While the performance of the hydrodynamic models was not ideal, it is suggested that they were capable of calculating spatially and temporally distributed data in headwater streams on the same order of accuracy as the observed data.

1.1 Objectives

The aim of this study is to characterise the hydraulics of East Creek, British Columbia, and provide suitable information for sediment transport, channel stability, river morphology or aquatic habitat studies. East Creek is a headwater stream and exhibits three morphologies, riffle-pool, rapid and step-pool. Both 1- and 2-dimensional hydrodynamic models are applied to provide varying spatial resolution of the hydraulic characteristics.

Poorly sorted sediments, large relative roughness and high gradient channel morphologies provide analytically and numerically challenging conditions for hydrodynamic models and this study investigates the applicability of such models to the described conditions.

1.2 Thesis Outline

The 2-dimensional hydrodynamic model, River 2D and 1-dimension model, HEC-RAS were applied to East Creek to investigate their performance in a headwater stream with large roughness elements and provide hydraulic data for sediment transport and channel stability investigations. Representation of flow resistance is of primary importance to the capabilities of hydrodynamic models. Chapter 2 presents background
to the resistance equations employed by River 2D and HEC-RAS and discusses their application to headwater streams. Sediment transport and bedload transport formulae are outlined and relevant formulation of the hydrodynamic models is presented.

Chapter 3 gives details of data collection methods and application of the hydrodynamic and bedload transport models. In Chapter 4, results of the hydrodynamic modelling are presented and, accuracy and reliability is discussed. The results of the hydrodynamic models were then used with the bedload transport formulae of Meyer-Peter and Muller (1948) and, Wilcock and Crowe (2003). Results and discussion of this analysis are presented in Section 5. Finally, a summary of the results is presented in Chapter 6 and possible further work is outlined.
2.0 Literature Review

2.1 Hydraulics of Headwater Streams

Traditionally, river research focused on larger lowland systems (Aberle and Smart, 2003) and was driven by flood hazard mitigation, irrigation, navigation, and hydropower requirements. In such locations channel gradient is low, bed-sediment is typically well-sorted and relative roughness is low. Relative roughness is defined as the ratio of grain size to flow depth, \( D/d \), where \( D \) is the representative grain size and \( d \) is hydraulic depth. Under these conditions, assumptions such as uniform or gradually varied flow, hydrostatic pressure distribution with depth, mild channel slope, and uniform logarithmic velocity profiles allow 1- and 2-dimensional (1D, 2D) analytical approaches such as conservation of mass, energy and momentum to be applied with reasonable success.

Headwater streams, however, have steep-gradients, high relative roughness, poorly-sorted bed sediments and spatially diverse channel morphologies. Sands through boulders may co-exist and large roughness elements may protrude above the free surface, causing wavy water surfaces, tumbling, supercritical flow and hydraulic jumps. Montgomery and Buffington (1997) suggest that such conditions typically occur in streams with slopes greater than 0.01 m/m. Under these conditions the assumptions made in derivations of open channel flow equations are stretched, although they are still often applied (e.g. Jarrett, 1984; Lee and Ferguson, 2002) and spatial and temporal resolution of the analysis becomes increasingly important.

2.1.1 Open Channel Flow

Open channel flow is often analysed using conservation equations; conservation of mass, energy and momentum. These are the basis of most hydrodynamic models including River 2D and HEC-RAS. Derivation of these equations can be found in many Fluid
Mechanics texts and the particular forms employed in River 2D and HEC-RAS are presented in Section 2.3. The primary assumptions in the 1D and 2D forms of these equations relate to the vertical pressure distribution and description of flow resistance (Steffler and Blackburn, 2002; USACE, 2004).

### 2.1.2 Hydrostatic Pressure Distribution

If pressure distribution is hydrostatic, little or no vertical acceleration occurs and streamlines are parallel or gradually varied, allowing solution of simplified energy and momentum equations. Open channel flow is almost always fully-rough turbulent and as such the velocity-at-a-point fluctuates, however it is assumed that the velocity fluctuates about a mean value and time-averaged velocity is used. Further, the vertical velocity profile is assumed logarithmic and generated by boundary roughness. If relative roughness is high, velocity profiles often deviate from this logarithmic profile (e.g. Wiberg and Smith, 1991; Byrd et al, 2000) (see Figure 2.1.B). However, velocity profiles measured by Byrd et. al. (2000) in North Boulder Creek, Colorado at locations with relative roughness ($D_{50}/d$) ranging from 0.2 to 0.5 suggest that spatially-averaged velocity profiles can be assumed logarithmic.

![Figure 2.1](image)

**Figure 2.1.** A) While the flow is turbulent a temporally averaged velocity can be assumed. B) Velocity profiles near large roughness elements may be irregular and non-logarithmic, but spatially averaged profiles can be assumed logarithmic. C) Depth averaged velocity assumed in 1D and 2D forms of the conservation equations.


2.1.3 Flow Resistance

The morphology of a river is a manifestation of a complex relationship between temporal and spatial variations in sediment supply, discharge and flow resistance (Grant, 1997). As discharge increases, the force on bed sediments increases; at some point the forces will be large enough to move sediment. The point at which this occurs depends on the size, shape, availability and arrangement of sediment. As sediments move, the resistance to flow changes and certain morphologies generate more resistance than others (e.g. Davies and Sutherland, 1983; Hassan and Reid, 1990). The channel form is derived in part through this feedback process.

Flow resistance is a description of the interaction between the channel boundary and the fluid. This relationship is typically described via a momentum balance, for example Manning’s equation, or by a description of the velocity profile, for example Keulegan’s (1938) equation. However, assumptions in the derivation of these methods make their application to high gradient streams with poorly sorted sediments and large roughness elements unreliable. Despite many attempts to describe the flow resistance in streams with high relative roughness and steep channel morphologies (e.g. Thompson and Campbell, 1979; Jarrett, 1984; Wiberg and Smith, 1991; Abrahams et al, 1995; Rosport, 1997; Curran and Wohl, 2002; Lee and Ferguson, 2002) the results are semi-empirical and there is no standard flow resistance equation for high gradient streams (Aberle and Smart, 2003). The following discussion provides background on the assumptions inherent in common flow resistance equations and examines the approaches of previous authors to describe flow resistance in high gradient streams.

In the 1700’s Antoine de Chezy derived a flow resistance equation based on field data from the River Seine. Chezy’s equation can be derived analytically from a momentum balance assuming steady, uniform flow.
Since the flow is steady and uniform there is no change in momentum, hence the net force on the control volume is zero (see Figure 2.2). If the pressure distribution is hydrostatic and the shape of cross-sections 1 and 2 are identical, $F_1$ is equal in magnitude but opposite in direction to $F_2$ and they act through the same line of action. Thus, $F_1$ and $F_2$ have no net effect on the control volume (e.g. Massey, 1996). Hence, flow resistance at the channel boundary balances the weight of water and Chezy’s equation can be written as

$$u = C\sqrt{RS_o}$$

(2.1)

where $u$ is average velocity in the downstream direction, $C$ is Chezy’s resistance coefficient, $R$ is hydraulic radius and $S_o$ is bed slope. For non-uniform flow conditions it can be shown that Equation (2.1) is still valid if bed slope is replaced by friction slope, $S_f$.
In the case of uniform flow, \( S_f = S_o \). Robert Manning (1816 - 97) presented empirical relations that allowed physical descriptions to be assigned to the resistance coefficient (Massey, 1996) and Manning's equation is written as

\[
u = \frac{1}{n} R^{2/3} S_f^{1/2}
\]  

(2.2)

In theory, by scaling by \( R \), Manning's resistance coefficient, \( n \), should remain constant for a given cross-section (Hey, 1979). In practice however, since flow is rarely (if ever) uniform and channel conditions vary between control volume boundaries and with depth at a given cross-section, Manning's \( n \) often becomes a calibration parameter that varies with discharge (e.g. Lee and Ferguson, 2002).

In 1932, Nikuradse published experiments with uniform sand grains glued to a pipe wall. He found that the resistance coefficient can be related to the relative roughness and if the channel boundary is hydraulically rough, flow resistance is independent of kinematic viscosity (Keulegan, 1938). Gravel-bed rivers are typically hydraulically rough. Keulegan (1938) integrated the Prandtl-von Karman law and used Nikuradse's result to derive an expression for the depth-averaged velocity (Equation (2.3)), assuming gradually varied, fully turbulent flow and a fixed, hydraulically rough boundary.

\[
\frac{u}{u_\kappa} = B_r + \frac{1}{\kappa} \log \frac{R}{k_s}
\]  

(2.3)

where \( u_\kappa \) is shear velocity, \( B_r \) is a channel shape coefficient, \( \kappa \) is von Karman's constant of turbulent exchange and \( k_s \) is Nikuradse's roughness length. Keulegan assumed that shear stress was uniformly distributed about the channel boundary and shear stress at the free boundary could be ignored (Hey, 1979). Equation (2.3) is often rearranged to

\[
\frac{1}{\sqrt{f}} = c \log \frac{aR}{k_s}
\]  

(2.4)
which is known as the Colebrook-White formula, were $f$ is Darcy-Weisbach's friction factor, $c$ is a function of von-Karman's constant and $a$ is a function of channel shape. Keulegan (1938) found that $a$ varies from 14.8 for circular pipes to 11.09 for wide open-channels. Also, $\kappa$ is typically considered a constant of 0.4 although values as low as 0.18 have been reported for flows with high relative roughness (Aberle and Smart, 2003). Despite this it is commonly assumed that $a$ and $c$ are constants and $k_s$ is used as a calibration parameter. Nikuradse defined $k_s$ equal to the $D_{50}$ of the uniform sand in his experiments. However, when applied to rivers, $k_s$ is found to equal some multiple, $C_r$, of a characteristic grain size, $D_r$ (i.e. $k_s = C_rD_r$), for example Thompson and Campbell (1979) found $k_s = 4.5D_{50}$, while Hey (1979) reported $k_s = 3.5D_{84}$ and Millar (1999) presents “best fit” results from several authors ranging from $k_s = 3.2D_{84}$ to $k_s = 3.9D_{84}$.

Further, selection of a representative grain size is quite arbitrary (Robert, 1990) and that a wide range of multipliers have been suggested is not surprising. Aberle and Smart (2003) present two sediment mixtures, both have the same $D_{50}$ (16.7mm) but the standard deviation of the bed elevation, $s$, of the two mixtures is different (8.2mm and 5.4mm) due to exposure. Further flow resistance is generated by channel morphology, as well boundary roughness. Bedforms (e.g. micro-topography, bars, riffles, pools etc) cause flow acceleration and deceleration, thus pressure distributions are non-hydrostatic (Einstein and Barbarossa, 1951; Robert, 1990; Wiberg and Smith, 1991; Millar, 1999) which generates flow resistance known as form resistance. Resistance generated by surface sediments is known as grain resistance. While grain and form resistance are not independent and their relationship is non-linear, Einstein and Barbarossa (1951) suggested that resistance components could be assessed individually with reasonable accuracy and they used the analogy of pipe flow where wall resistance is separated from resistance generated by pipe-fittings (e.g. expansions, contractions, bends and valves). The significance of this separation is that both form and grain components hinder flow and thus determine mean velocities and flow depth, while only grain resistance contributes to sediment entrainment and transport.
Most researchers have focussed on applying forms of Manning’s equation (e.g. Jarrett, 1984) or Keulegan’s equation (e.g. Lee and Ferguson, 2002). In high gradient streams, large roughness elements, poorly sorted sediments, steep slopes and a wavy free surface, thus non-uniform channel boundary and flow conditions mean that these equations become simply an empirical relation (Aberle and Smart, 2003).

Thompson and Campbell (1979) considered blocking of flow by large roughness elements, Wiberg and Smith (1991) partitioned total stress into fluid and form drag components based on a spatially averaged vertical velocity profile. Aberle and Smart (2003) used a logarithmic approach but related resistance to the standard deviation of the surface elevation above a datum. And Rosport (1997) included energy loss from the free surface, which is a function of Froude-number. Despite the variety of approaches, there is currently no standard flow resistance equation, applicable to headwater streams (Aberle and Smart, 2003).

2.2 Sediment Transport

River morphology is a representation of spatial and temporal variation of sediment movement and is the result of a feedback interaction between fluvial hydraulics and the channel boundary. An understanding of sediment transport processes is essential for assessment of aquatic habitats, river restoration, catchment management and design of engineered structures in fluvial environments. Flow resistance and boundary shear stress are essentially the same question, the primary difference is the point of view, where the former considers force on the fluid, and the later considers force on the boundary, although as outlined in Section 2.1.3, the entire boundary shear is not effective at moving sediment.

River load can be classified by three components; dissolved load, wash load and bed-material load. Wash load consists of material that is so fine (typically <0.062mm) that once it enters the channel it is quickly entrained and held in suspension by physical
forces. Bed-material load is defined as sediments found in appreciable quantities in the bed (Knighton, 1998) and hence is responsible for channel morphology. The following discussion concerns entrainment, transport and deposition of bed-material sediment.

Bed-material transport studies typically include bed load sampling, morphological mapping, tracer studies, sediment budgets, bedload rating curves or bedload transport equations (see Hicks and Gomez, 2003). While transport equations are generally considered unreliable (Gomez and Church, 1989), they offer the benefit that they can be applied predicatively and hence lend themselves to applications where modifications to the flow regime are proposed. Combined with a hydrodynamic model, many difficult and interesting sediment transport and fluvial morphology questions could be investigated.

Entrainment of bed material is generally considered a function of bed shear stress, $\tau_0$, such that sediment is entrained once $\tau_0$ exceeds a critical shear stress, $\tau_c$. In 1936, Shields presented a comparison of dimensionless shear stress, $\theta$, and grain Reynolds number, $Re_g$, for uniform sediments and found that for hydraulically rough flow, which natural rivers typically are, the dimensionless shear stress is constant. Despite considerable scatter about the data, Shields found a value of about 0.06, however other authors have proposed values from 0.01 to more than 0.1 (Church et. al., 1998).

$$\theta = \frac{\tau_c}{g(\rho_s - \rho)D_i}$$ (2.5)

Equation (2.5) is derived from a simple force balance and neglects several details of sediment entrainment. The parameters in Equation (2.5) are gravitational acceleration, $g$, sediment density, $\rho_s$, fluid density, $\rho$, and $D_i$ is the grain size of interest. Many factors confound the assessment of critical shear stress, not least of which is that sediment entrainment is essentially a stochastic process dependent on the turbulent shear stress at the boundary and sediment is entrained by turbulent flow bursts (Buffington and Montgomery, 1997). Since turbulence is a difficult phenomenon to quantify, it is
approximated in uniform flow by time averaged boundary shear. Further, initiation of sediment motion is not well-defined and different methods to estimate initiation of motion and hence critical shear have been adopted. In flume experiments, critical shear stress is typically defined by observations of sediment movement. Since entrainment is probabilistic however, a particle may be entrained over a wide range of flows. Typically, general motion of the bed material is defined but this is a subjective measure. An alternative approach is to estimate the critical shear stress from measurements of sediment transport rates at a range of discharges above the critical discharge and extrapolate back to a reference shear stress where a small sediment transport rate is observed (Parker et al., 1982).

In gravel-bed rivers with poorly-sorted sediments the interaction between sediment and flow is complicated by hiding, bed armouring and structure. That is to say, fine sediments are sheltered from the main flow by larger particles so that the critical shear stress increases above that expected for uniform sediments. Conversely, larger sediments have increased exposure and are entrained more easily than in a uniform mixture, thus the critical stress decreases. These effects contribute to the scatter about observed dimensionless shear stress estimates.

As the range of sediment sizes increases, Parker et al. (1982) proposed the hypothesis of equal mobility where, at flows capable of moving most available sediment, the bedload grain size distribution is approximated by the substrate distribution. Between the two limits of Shields' approach and Parker's equal mobility lies the hypothesis of partial mobility in which larger sediments move infrequently and their representation in bedload, p, is less than their proportion in the surface sediment, fi, such that p/fi <1. Consideration of fractional transport rates allowed Parker (1990) and Wilcock (2001) to develop bedload transport formula that consider the mobility of each grain size range individually and thus consider hiding and exposure. While the surface-based approach is a step forward, these approaches still cannot consider surface structure, which significantly increases the stability of gravel-bed streams under low transport conditions.
(Church et al, 1998; Hassan and Church, 2000). In order to consider surface structure, descriptors other than grain size distribution and averaged flow conditions are required.

2.2.1 Bedload Transport Formulae

Bedload transport formulae are based on the idea that a relationship exists between hydraulic conditions, the sediments present and the sediment transport rate (Gomez and Church, 1989). However, there are many factors, primarily related to the temporal and spatial resolution and accuracy of observations in real rivers that confound this relationship for example local shear stress and description of sediment and surface structure. Hassan and Church (2000) comment that in low transport regimes, transport rate is “exceedingly sensitive” to bed surface grain size and structure. Consequently, transport equations are often considered unreliable and the results variable.

Two aspects of bed load transport equations make them appealing: 1) if reliable, minimal field observations are required, and 2) they can be used to predictively to study the effects of proposed changes to the sediment and/or fluvial regime. Many sediment transport equations have been proposed but essentially they attempt to relate hydraulic characteristics to sediment transport rates. One of four hydraulic characteristics is typically used: 1) bed shear stress, 2) discharge, 3) stream power or, 4) a stochastic function of sediment transport (Gomez and Church, 1989).

Parker (1990) recognised sediment transport in gravel-bed streams as a function of the grains available for transport i.e. the surface sediment were the surface may be significantly coarser that the bulk sediment mixture. Parkers’ formulation considered entrainment per grain size range. Wilcock and Crowe (2003) developed a surface-based transport equation capable of predicting transient transport conditions based on the formulations of Proffitt and Sutherland (1983) and Parker (1990) (Wilcock and Crowe, 2003).
2.3 Hydrodynamic Modelling

Field data collection, suitable to describe the spatial and temporal variation of conditions at an investigation site, can be both technically challenging and expensive. Modelling can allow increased spatial and temporal resolution of the conditions based on comparatively limited field data. Increasing computer processing power has made the application of hydrodynamic models relatively common.

Models capable of calculating 1-, 2- and 3-dimensional (1D, 2D and 3D) fluid hydraulics are available. In general, 1D hydrodynamic models only calculate variation in the downstream direction and properties are cross-section averaged; 2D models calculate depth-averaged properties in both downstream and across-stream directions; while 3D models calculate hydraulic properties in downstream, across-stream and vertical directions. The dimensionality of a model chosen depends on the application and level of detail required. Although 1D models provide a simplified representation of the system, 3D models require significant computational power and may have numerical stability problems. The 1D model, HEC-RAS and 2D model, River 2D, were applied to East Creek to describe the spatial and temporal variation of hydraulics and to investigate the application of these models to numerically and analytically challenging conditions.

2.3.1 HEC-RAS

HEC-RAS (USACE, 2004) is a 1D model capable of steady and unsteady flow analysis including subcritical, supercritical and mixed flow scenarios. In this work only steady flow conditions have been computed hence the following discussion is limited to this aspect of the model. The analysis is based around the solution of 1D equations of conservation of mass, energy and momentum.
In HEC-RAS, conservation of energy is applied between adjacent cross-sections and solved using the standard step iterative method. The energy equation is described in HEC-RAS as

\[ Y_2 + Z_2 + \alpha_2 \frac{u_2^2}{2g} = Y_1 + Z_1 + \alpha_1 \frac{u_1^2}{2g} + h_e \]  

(2.6)

where \( Y \) is the average water depth, \( Z \) is the channel invert elevation, \( \alpha \) is a velocity weighting coefficient, \( g \) is gravitational acceleration and the subscripts 1 and 2 refer to consecutive cross-sections. Energy head loss, \( h_e \), is defined as

\[ h_e = L S_f + E \left( \frac{\alpha_2 u_2^2}{2g} - \frac{\alpha_1 u_1^2}{2g} \right) \]  

(2.7)

where \( E \) is an expansion or contraction coefficient and \( S_f \) is friction slope

\[ S_f = \frac{Q n^2}{AR^{2/3}} \]  

(2.8)

Equation (2.8) is simply Manning’s equation rearranged to make \( S_f \) the subject where \( Q \) is discharge, \( n \) in Manning’s coefficient, \( A \) is cross-section flow area, \( R \) is the hydraulic radius.

In East Creek, short steep riffles produced shallow rapid flow that, from visual observation, appear supercritical. HEC-RAS is capable of mixed flow analysis (subcritical and supercritical flow conditions). In order to calculate mixed flow conditions a specific force form of the momentum equation is applied. The steady flow momentum equation is defined as

\[ \frac{Q_i^2 B_i}{g A_i} + A_i \bar{v}_i + \frac{A_i + A_2}{2} L S_o - \frac{A_i + A_1}{2} L S_f = \frac{Q_i^2 B_i}{g A_i} + A_i \bar{v}_i \]  

(2.9)
where $\beta$ is a momentum correction factor, $\bar{Y}$ is the distance from the water surface to the centroid of area of the cross section and $S_0$ is average channel slope. When applied to short channel reaches it is assumed that the friction force and weight of water are small compared to the dynamic and hydrostatic terms and the specific force equation is defined as

$$\frac{Q_2^2 \beta}{gA_2} + A_2 \bar{Y}_2 = \frac{Q_1^2 \beta}{gA_1} + A_1 \bar{Y}_1$$

(2.10)

To calculate the mixed flow profile, HEC-RAS starts at the downstream boundary condition and calculates a subcritical profile. If a subcritical solution cannot be found the program defaults to the critical depth. The program then calculates a supercritical profile (an upstream boundary condition must also be defined) working downstream from where a critical depth was assumed. The supercritical profile is calculated until a cross-section is found with valid sub- and supercritical solutions. The specific force is calculated and the solution with the greatest specific force is assumed correct. A hydraulic jump is assumed between the subcritical and supercritical cross sections (USACE, 2004).

### 2.3.2 River 2D

River 2D is a 2D depth-averaged velocity model capable of computing steady and unsteady subcritical and supercritical open channel flow. Local flow depth, $H$, and specific discharge, $q$, in the $x$- and $y$-directions are calculated at each node by solving the 2D, depth-averaged St Venant Equations, namely conservation of mass (Equation (2.11)) and conservation of momentum in the $x$-direction (Equation (2.12)) and $y$-direction.

$$\frac{\partial H}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial t} = 0$$

(2.11)
\[
\begin{align*}
\frac{\partial q_x}{\partial t} + \frac{\partial}{\partial x} (Uq_x) + \frac{\partial}{\partial y} (Vq_x) + \frac{g}{2} \frac{\partial}{\partial x} (H^2) = gH(S_{ax} - S_{fx}) + \frac{1}{\rho} \frac{\partial}{\partial x} (H\tau_{xx}) + \frac{1}{\rho} \frac{\partial}{\partial y} (H\tau_{xy})
\end{align*}
\] 

(2.12)

where \( g \) is gravity, \( \rho \) is density of water, \( x \) and \( y \) are cartesian coordinates in the horizontal plane, \( t \) is time, and \( U \) and \( V \) are depth-averaged velocities in the \( x \) and \( y \) directions respectively. Velocity, flow depth and discharge per unit width are related by:

\[
U = \frac{q_x}{H} \quad \text{and} \quad V = \frac{q_y}{H}.
\]

\( S_{ax} \) is bed slope and \( S_{fx} \) is friction slope, where \( S_{fx} = \frac{\tau_{ax}}{\rho g H} \).

In order to solve Equation (2.12), models are required to represent bed shear, \( \tau_{ax} \), and transverse shear, \( \tau_{xx} \) and \( \tau_{xy} \). The friction slope is modelled with a 2-dimensional Chezy equation

\[
S_{fx} = \frac{\tau_{ax}}{\rho g H} = \sqrt{\frac{U^2 + V^2}{g HC_s}} U
\] 

(2.13)

where

\[
C_s = 5.75 \log \frac{12H}{k_s}
\] 

(2.14)

and \( k_s \) is Nikuradse's roughness length. Transverse turbulent shear is modelled with a Boussinesq type eddy viscosity formulation, where \( \tau_{xy} \) is given by

\[
\tau_{xy} = \nu_t \left( \frac{\partial U}{\partial y} + \frac{\partial V}{\partial x} \right)
\] 

(2.15)

The eddy viscosity coefficient, \( \nu_t \), is assumed composed of a constant, a bed generated term, and a transverse shear component.
\[ v_i = \varepsilon_1 + \varepsilon_2 \frac{H\sqrt{U^2 + V^2}}{C_r} + \varepsilon_3 H^2 \sqrt{2 \frac{\partial U}{\partial x} + \frac{\partial U}{\partial y} + \frac{\partial V}{\partial x} + \frac{\partial V}{\partial y}} \]  

(2.16)

where \( \varepsilon_1, \varepsilon_2, \) and \( \varepsilon_3 \) are coefficients. Derivations of Equations (2.11) through (2.16) assume that the pressure distribution is hydrostatic. This limits its applicability to gradually varied flow and bed gradients less than about 10% (Ghanem, 1995; Steffler and Blackburn, 2002). The two reaches modelled at East Creek have average slopes of 1.5% and 2.5%, respectively. While large roughness elements cause local accelerations and rapidly varied flow the topography is modelled excluding these features to provide spatially averaged results, which are assumed sufficiently uniform and gradually varied (Byrd et al, 2002). It is also assumed that coriolis and wind forces are negligible, which is considered reasonable for river flows (Ghanem, 1995; Steffler and Blackburn, 2002).

In 2D models flow depth is a variable and the spatial extent of the water surface may be unknown, hence wet computational nodes may become dry and dry nodes wet. This situation causes computational difficulties. Two common approaches to solve this problem are: 1) remove dry areas from the computation domain, or 2) solve the equations everywhere irrespective of wet and dry. River 2D employs the second approach and Equation (2.11) is replaced with a groundwater continuity equation and a continuous free surface is calculated with positive depths calculated for surface flow and negative depths for groundwater.

To solve Equations (2.11) through (2.16) River 2D employs an implicit finite element solution method based on a Petrov-Galerkin formulation, which provides good numerical stability in mixed flow regimes and wetting and drying of cells (Steffler and Blackburn, 2002).
3.0 Methodology

The analyses in this thesis are based on field data collected at East Creek, British Columbia, over the winter of 2004/05. Detailed coupled discharge, velocity and water surface profile data were collected for 7 discharges between October 2004 and January 2005. Topographic, bed material and bedload transport data were also collected. The following sections describe the data collection methods employed.

3.1 Site Description

East Creek rises in the University of British Columbia Malcolm Knapp Research Forest. It is a low order stream that contributes to the Fraser River via Spring Creek and the Alouette River. A 1km study reach extending upstream from the confluence with Spring Creek has been set up to investigate sediment transport, channel stability, surface structure and channel morphology in high gradient streams. Within the 1km study reach three distinct morphologies, described as riffle-pool, rapid and step-pool, are present. The classification scheme of Montgomery and Buffington (1997) was used except the title "plane-bed" was replaced by "rapid" to avoid confusion with sand-bed literature (Zimmerman and Church, 2001). For the purpose of this thesis three sub-reaches, one in each morphology, were delineated to undertake hydrodynamic modelling. Due to the challenges encountered in the rapid and riffle-pool reaches, no analysis was undertaken in the step-pool reach and aside from Figure 3.4 the step-pool reach is not considered in this thesis.

Each sub-reach is approximately 60m long (15-20 channel widths). Table 3.1 describes the primary characteristics of each reach and Figure 3.4 shows the morphology and flow conditions in each reach.
Table 3.1: Principle characteristics of the rapid and riffle-pool reaches

<table>
<thead>
<tr>
<th>Morphology</th>
<th>Reach Length [m]</th>
<th>Reach Average Slope[^1] [m/m]</th>
<th>Bankfull Relative Roughness[^2] (D_s/d)</th>
<th>Observed Discharge [m^3/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rapid</td>
<td>79</td>
<td>0.024</td>
<td>0.53</td>
<td>0.04 - 0.62</td>
</tr>
<tr>
<td>Riffle-pool</td>
<td>64</td>
<td>0.02</td>
<td>0.32</td>
<td>0.04 - 0.62</td>
</tr>
</tbody>
</table>

[^1]: Calculated from linear regression of the thalweg elevation
[^2]: d is the hydraulic depth ((cross sectional area)/(surface width)) calculated with HEC-RAS

Figure 3.1. East Creek is a small headwater stream located in the Malcolm Knapp Research Forest

The study reach elevation is approximately 150m above sea level. The 1km^2 catchment is subject to occasional snowfalls; however, stream flow is dominated by rainfall-runoff processes with an average annual rainfall on the order of 2500mm. The catchment is predominantly forested with a mixture of clear-cut, thinned and 70-120yr old stands; the
result significant of logging and fire in the 1930’s. Common species include coniferous Douglas Fir, Western Red Cedar and Western Hemlock (www.mkrf.forestry.ubc.ca, downloaded May 2005).

3.2 Water Surface Profiles

Water surface profiles were collected for a range of discharges, from low flows to above bank-full flows. Staff gauges were installed approximately every 5m along the channel and are located above and below morphological features so as to identify breaks in water surface slope. For example, in the riffle-pool reach, staff gauges were located at the crest and toe of riffles and one or two gauges were located at mid-riffle locations. Pools were gauged similarly. Several staff gauges are visible in Figure 3.4.

A digital video camera was used to record the water surface along the reach and 10 second records of the water surface at the staff gauges. Review of the video was used to estimate a time averaged staff gauge reading, provide estimates of the water surface between staff gauges and form a record of the flow conditions. Due to the turbulent nature of the flow, the water surface was wavy and fluctuated at a point. The magnitude of these fluctuations depended on the local topography and discharge, but typically time-averaged water surface estimate at each staff gauge is considered accurate to ±0.005m. A stilling-well could improve staff gauge readings.

Table 3.2. Typical water surface fluctuation, thus uncertainty, in staff gauge readings.

<table>
<thead>
<tr>
<th>Discharge [m$^3$/s]</th>
<th>Staff Gauge Accuracy [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.04</td>
<td>±0.005</td>
</tr>
<tr>
<td>0.13</td>
<td>±0.005</td>
</tr>
<tr>
<td>0.33</td>
<td>±0.005</td>
</tr>
<tr>
<td>0.62</td>
<td>±0.01</td>
</tr>
</tbody>
</table>
Crest-stage gauges, which record the peak water level, were made to US Geological Survey guidelines (see Appendix 1) and installed next to the staff gauges at the crest and toe of clearly defined morphologic units. The resolution of the water surface with the crest gauges was not as high as with the staff gauges but debris lines were used to check and/or improve peak water surface estimates. The location of crest and staff gauges is shown in Figures 3.2 and 3.3. Utilising a stage-discharge rating curve, the peak discharge and corresponding water surface was estimated.
Figure 3.2. Rapid reach showing the location of staff and crest gauges (dots) and fixed cross sections (x's). The reach was modelled from cross-section 6 - 20. A channel spanning sediment trap is located between cross-sections 19 and 20. Air photos where taken from a 10m pole held above the stream. Scale bar is indicative only.
Figure 3.3. Riffle-pool reach showing the location of staff and crest gauges (dots) and fixed cross sections (x's). The reach was modelled from cross-section 30 - 44. A channel spanning sediment trap is located at cross-sections 39. Air photos where taken in from a 10m pole held above the stream. Scale bar is indicative only.
Figure 3.4. A) and B) rapid reach at low and near bank-full flow respectively; C) and D) riffle-pool reach at low and near bank-full flow respectively. Bank-full flow $\approx 0.5 \text{ m}^3/\text{s}$
Figure 3.4 cont. E) and F) step-pool reach at low and near bank-full flow respectively. Bank-full flow $\approx 0.5$ m$^3$/s. Modelling was not undertaken in the step-pool reach due to complexity and extreme spatial variability of the channel topography.

3.3 Discharge and Velocity Measurements

During the winter of 2004/2005, seven complementary measurements of discharge, velocity, flow depth, and water surface profile were recorded in the rapid and riffle-pool reaches. Since East Creek is a low order stream, the hydrology is quite flashy and discharge increased and decreased rapidly in response to rainfall events (see Figure 3.6). However, to collect one set of complementary measurements (discharge, depth, velocity, and water surface) in both the rapid and riffle-pool reaches typically took 2 hours. Water level change observed at a control staff-gauge was typically less than 0.01m during the sampling period and thus considered steady within the error of discharge and water surface measurements. Further, there are no tributaries between the two reaches and discharge is considered equivalent in both reaches.
Discharge was measured using the velocity-area method at cross sections 12 and 19 in the rapid reach and cross sections 35 and 39 in the riffle-pool reach. The average of the four estimates was considered the true discharge. One-dimensional velocity was measured at 0.4 times depth from the bed with an electromagnetic velocity meter. Two 20-second-averaged velocities were recorded at each position. If they were equivalent, the velocity was recorded. If not, a third 20-second-averaged velocity was recorded and the average of the three measurements was used. This occurred occasionally at high flows or behind large roughness elements where the flow was unstable. Depth and velocity were recorded so that no partial area contained more than 10% of the total flow (WMO, 1980), which required measurements every 0.1 to 0.2m across the channel depending on the discharge.

A pressure-discharge rating curve (Figure 3.5) was developed. Discharge from the velocity-area measurements, salt-dilution method in the step-pool reach and, low flow measurements estimated by recording the time to collect a known volume at a culvert outfall were used. A pressure transducer located at the top of the rapid reach provided a continuous 10-min-averaged pressure record. A piecewise relationship was fit to the data and combined with the pressure record to give a hydrograph for the site (Figure 3.6). A flow frequency analysis was undertaken using data from Seymour River (Environment Canada gauge 08GA077) and Coquitlam River (Environment Canada gauge 08MH141). Comparison between hydrographs showed similar timing and relative magnitude of flow events and the 2-year return period flow, $Q_2$, for East Creek was calculated as 2.0 m$^3$/s.
Figure 3.5. Discharge-pressure (equivalent to stage) rating curve. A piecewise relationship was fit to the data.

Figure 3.6. Hydrograph for East Creek, British Columbia. Observed discharges are shown with black dots. $Q_2$ is estimated at 2.0 m$^3$/s.
3.4 Topography

In 2003, 75 channel cross-sections were marked out along the 1km study reach and are located by re-bar pins. Cross-sections 6 to 20 and 30 to 39 are pertinent to this study and are shown in Figures 3.2 and 3.3. A benchmark was not easily accessible so the location of cross-section 1 was identified with a hand held GPS and compass. All other pins were located from this cross section. Consequently, the absolute locations of the pins are not highly accurate, however it is only the relative locations that are of real interest. The location of the cross-section pins was stored in a “*.gsi” file on the total station and all subsequent surveys have been recorded relative to these pins. All surveying was undertaken with a Leica total station.

A baseline channel survey was undertaken in mid August 2004, focusing on changes channel gradient with the objective of capturing plan- and bedform scale features. Large roughness elements were not included in the survey. Over 3500 topographic data points were collected in the two reaches, giving a typical point density of 5/m². Flow data was collected from October 2004 to January 2005 and while the channel topography has changed during this time the bed was considered fixed throughout the analysis, based on the August survey. The channel was re-surveyed in January 2005 at a point density of 7/m² in order to quantify changes in the topography (areas of scour and deposition).

Abrupt changes in topography, large roughness elements and undercut banks are both difficult to represent with a topographic survey and challenging for hydrodynamic models. For the most part these high-resolution features are not included in the bed map and must be considered when reviewing the results.
Figure 3.7  Rapid reach A) showing the resolution of survey data. Resolution is similar in the riffle-pool reach. B) River 2D bed topography with 0.1m contours. The section shown can be seen in Figure 3.4 A). Note the log visible in the photo and River 2D bed map. Large roughness elements are not included.
3.5 Sediment and Grain Size Analysis

Surface sediments were sampled in February 2005 using the Wolman (1954) method. Three Wolman counts of approximately 300 stones each were conducted in riffle-pool reach to investigate spatial variation in surface sediments. Similarly, in the rapid reach two surveys were undertaken. The areal extent of each survey was based on a visual assessment of variation in surface grain size. However, the differences in grain size distribution (GSD) between surveys were not significant and were combined to give one GSD per reach. Surface sediment data was collected only once so it was not possible to comment on the temporal characteristics; hence it was assumed that the surface GSD was constant throughout the investigation.

Figure 3.8. Surface and subsurface grain size distribution for the rapid and riffle-pool reaches.
Table 3.3. Surface and sub-surface characteristic grain size

<table>
<thead>
<tr>
<th>Surface</th>
<th>Sub-surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>D_{16}</td>
<td>14</td>
</tr>
<tr>
<td>D_{50}</td>
<td>42</td>
</tr>
<tr>
<td>D_{64}</td>
<td>83</td>
</tr>
</tbody>
</table>

In 2003, pit traps were installed to measure bedload transport. The traps span the active channel width and the material excavated to install the traps was sampled to calculate a sub-surface GSD. Pit traps are located at the downstream extent of the modelled reaches. The traps were emptied after competent flow events and transport quantities and GSD were calculated.

### 3.6 Data Analysis

Stream hydraulics and sediment transport were assessed using one-dimensional (1D) and two-dimensional (2D) hydrodynamic models. HEC-RAS (version 3.1.2 April 2004), developed by the U.S Army Corps of Engineers, was used for the 1D analysis, while River 2D (version 0.90, September 2002) developed at the University of Alberta was used for the 2D assessment. This section provides information on how the models were set up and run. For relevant details on the formulation and numerical schemes see Section 2.3. Results and discussion of the model performance is given in Section 4.0.

#### 3.6.1 ArcMap

Analysis, organisation and visualisation of spatial data was undertaken with ArcMap 8.3, which is a GIS programme developed by ESRI Inc. Aerial photos of study site were taken from a 10m pole. The camera was attached to a gimble so that it pointed downwards and these photos were rectified in ArcMap. Survey data was overlain on top
of the photos. From this map topographic features could be identified, results were related to physical features and distances were measured.

### 3.6.2 River 2D

In River 2D, topography is saved in a text file with a "*.bed" extension in which each line contains a unique identity number, the coordinates of one topographic point (x, y, z) and a roughness value described by Nikuradse’s roughness length, \( k_s \). Two model domains were prepared, one for the rapid reach, one for the riffle-pool reach. The bed and banks were defined with the August 2004 topographic survey and the above bank areal extent was increased with data from a 2003 survey (see Figure 3.7) as it was assumed the topography in this area had not changed. The model uses a triangulated irregular network (TIN) to represent the surface. Breaklines were included at the crest and toe of the channel banks to capture the abrupt change in gradient. As discussed in Section 3.4, large roughness elements are not included in the topography. Figure 3.9 shows how the rapidly varied topography was modelled.

![Smoothed cross-section topography](image)

**Figure 3.9.** Smoothed cross-section topography. The dashed line represents model topography used in HEC-RAS and River 2D.
A no-flow boundary was defined above the expected highest water level or highest available topographic data, which ever was least and a computation mesh was created with a uniform node spacing of 0.3m, which gave around 4500 nodes per reach. The mesh was then smoothed and refined to give a mesh quality index >0.4. A few unstructured tests of alternative meshes were tried initially but when this mesh was found to give stable runs no further adjustments were made.

Flows were then modelled, beginning with uniform roughness throughout the reach. The model was calibrated by adjusting the flow resistance parameter, $k_s$, until the modelled longitudinal water surface profile matched the measured profile. Due to uncertainty in the measured discharge and water surface elevation an error of $\pm10\%$ of the flow depth was desired although this was not always achieved. Error was calculated as

$$\%\;\text{error} = \frac{H_{\text{obs}} - H_{\text{calc}}}{H_{\text{obs}}} \times 100$$

(3.1)

where $H$ is the flow depth. As well as a bed resistance model, 2D models also require a transverse shear model and this can also be used to calibrate the model. In River 2D, three coefficients, $\varepsilon_1$, $\varepsilon_2$, and $\varepsilon_3$ are used to weight the three assumed components of eddy viscosity. The default values of 0, 0.5 and 0.1 respectively were used in all runs.

3.6.3 HEC-RAS

HEC-RAS is a 1D model that calculates water surface profiles by solving the equations of continuity of mass, energy and, momentum using a standard step iteration scheme (USACE, 2004). While the model has many empirical and analytical methods to help represent complex situations, the basic computation requires the definition of cross-sections, distance between cross-sections and flow resistance parameter, Manning’s $n$. 

35
The rapid and riffle-pool reaches were modelled separately. The rapid reach was modelled from cross section 6 to 20 and the riffle-pool reach from cross-section 30 to 39. These cross sections were used and further cross sections were then extracted from the River 2D TIN and were spaced approximately every 1-2m to capture changes associated with bedforms, for example bars or riffles (see Figure 3.10). HEC-RAS describes each cross section by a minimum of three components, left over bank, main channel and right over bank and requires Manning's $n$ and downstream distance for each. Each cross section was defined so that only the main channel was active. Downstream distances were measured in ArcMap.

Figure 3.10. Location of rapid reach HEC-RAS cross-sections. Cross-section spacing was similar in the riffle-pool reach.
During calibration runs the following warning was often returned: "The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections." Conveyance, $K$, is defined as

$$K = \frac{1}{n} AR^{\frac{3}{2}}$$  \hspace{1cm} (3.2)

where $A$ is cross sectional flow area and $R$ is hydraulic radius. Since the user specifies $n$, a large change in $K$ indicates a large change in area and suggests the water surface may not be accurately resolved and flow may be rapidly varying. To avoid this error the auto-interpolation feature was used to create cross sections spaced every 0.2m. Generally, sections were not required at this resolution but as computation time was 1-2 seconds there was no need to optimise their spacing.

The model was calibrated by adjusting Manning's $n$ until the modelled water surface matched the observed water surface. Calibration commenced with a uniform roughness value and was increased if the modelled surface was below the observed surface. Conversely, roughness was decreased if the calculated water surface was above the observed surface. Calibration aimed to match the flow depth within ±10% although this was not always achieved.

HEC-RAS was run to calculate mixed flow, which allows both sub- and super-critical conditions. Discharge and upstream and downstream water surface must be entered as initial conditions and observed water surface values and discharge were entered.
3.6.4 Sediment Transport Analysis

In order to extend the hydrodynamic model results to sediment transport applications and investigate the models further, the results were used in conjunction with the bedload transport formulae of Meyer-Peter and Muller (1948) and, Wilcock and Crowe (2003).

Two small flood events, large enough to generate sediment transport were selected. The flood peaks occurred on 25 November 2004 and 10 December 2004 with peak flows of 1.3 $\text{m}^3/\text{s}$ and 1.6 $\text{m}^3/\text{s}$ respectively. Sediment transport data was available from channel spanning pit traps that were emptied on 5 November, 28 November, 8 December and 22 December 2004. Only sediment quantities are currently available, as the grain size analysis has not been completed. During the November and December events, 246kg and 252kg respectively were collected in the trap.

The hydrograph for each flood was split into 0.1 $\text{m}^3/\text{s}$ intervals and the duration of each discharge interval was calculated for the period between sediment sampling. Velocity, flow depth, bed shear stress, $\tau_o$, and shear velocity ($u^* = (\tau_o/p)^{1/2}$) were calculated for each 0.1 $\text{m}^3/\text{s}$ interval using both HEC-RAS and River 2D. Since sediment transport primarily occurs at higher flows, both models were calibrated to an observed flow of 0.62 $\text{m}^3/\text{s}$ and it was assumed that the hydraulic conditions where reasonably represented over the range of flows pertinent to sediment transport. Using hydraulic data just upstream of the pit trap the specific sediment transport rate, $q_s$, at the trap was calculated using the bedload transport formulae of Meyer-Peter and Muller (1948) and, Wilcock and Crowe (2003). The total mass of sediment transported during the event, $Q_{bs,\text{total}}$, was calculated as

$$Q_{bs,\text{total}} = q_s W_n T$$  \hspace{1cm} (3.3)

Where $W_n$ is width and $T$ is duration. For the HEC-RAS results, $W_n$ is channel width, for River 2D hydraulic data was output at 0.2m intervals across the channel so that $W_n$ is 0.2.
4.0 Hydrodynamic Modelling of East Creek

Both River 2D and HEC-RAS were capable of replicating water surface profiles and velocities observed at East Creek with reasonable accuracy. However, both models required considerable calibration and accuracy decreased when used to predict conditions outside the calibrated flow. The poor predictive results indicate that the flow resistance models were not able to capture the variation in resistance. Considering the assumptions in their derivation (see Section 2.1) this is not unexpected. River 2D was unable to model very low discharges, as the maximum flow resistance that can be generated by the model is limited. Further, abrupt changes in topography (e.g. root debris causing a channel constriction) and water surface profiles generated by backwater conditions were found to create calibration difficulties highlighting the difficulty to accurately represent the topography and model the hydraulics of these highly spatially diverse morphologies.

4.1 Model Calibration

Calibration was undertaken by comparing observed and calculated water-surface profiles. Generally the water surface is well matched except at Staff-Gauge 37 (the outlying point in Figure 4.1a), which was located behind a tree growing in the stream and causes backwater. The tree was not included in the model topography; hence calibration near the tree was not expected. Table 4.1 shows the error and standard deviation of the calibrated water surfaces as a percent of flow depth. While the mean error is low, the standard deviation is relatively high, particularly for the low flow calibration.

Following water surface calibration, calculated velocities and depths were compared to observed values. A typical River 2D result is presented in Figure 4.1 showing cross-section 35 and it can be seen that the model captures the depth, velocity and water surface trends quite well, including the 2-dimensional distribution. Qualitatively, Figure 4.1 shows that the depth is slightly under predicted, which is probably due to the calibration.
accuracy i.e. the calibrated water surface is slightly high at Cross-Section 35. Also, there is considerable scatter seen in observed velocities, which is probably due to shooting and blocking flow past large roughness elements. Since large roughness elements are not included in the model topography, this diversity is not resolved, but the model does appear to capture the averaged velocity well. By conservation of mass, since the calculated depth is under predicted the calculated velocity is slightly over predicted. Considering the observation cross-sections for a calibrated flow, it appeared that error in depth and velocity was a function of the accuracy of the water surface calibration and channel topography at the cross-section.

Table 4.1. Error and standard deviation of River 2D results as a percent of the observed depth and velocity. Velocity was measured at 0.4 times depth from the bed. Outliers were not included as when depth and velocity get small near the banks, a small absolute difference generates a large error.

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean Error [%]</th>
<th>Standard Deviation [%]</th>
<th>Mean Error [%]</th>
<th>Standard Deviation [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High Flow Calibration (0.62 m³/s)</td>
<td>Low Flow Calibration¹ (0.13 m³/s)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All staff</td>
<td>-3.8</td>
<td>14.3</td>
<td>7.8</td>
<td>16.5</td>
</tr>
<tr>
<td>Rapid</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid Staff</td>
<td>0.5</td>
<td></td>
<td>16.8</td>
<td></td>
</tr>
<tr>
<td>Rapid Gauge</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid Water</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid Surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid Rifflle-Pool</td>
<td>0.5</td>
<td>4.9</td>
<td>7.8</td>
<td>16.5</td>
</tr>
<tr>
<td>Rapid XS 12</td>
<td>-2.7</td>
<td>1.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rapid XS 19</td>
<td>17.7</td>
<td>31.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Velocity</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Velocity Rapid</td>
<td>-13.0</td>
<td>41.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Velocity XS 35</td>
<td>-3.8</td>
<td>29.2</td>
<td>-12.1</td>
<td>58.3</td>
</tr>
<tr>
<td>Velocity XS 39</td>
<td>-6.8</td>
<td>30.9</td>
<td>17.0</td>
<td>28.9</td>
</tr>
<tr>
<td>Depth</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth Rapid</td>
<td>-0.1</td>
<td>16.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth XS 12</td>
<td>10.3</td>
<td>8.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth XS 35</td>
<td>13.6</td>
<td>11.1</td>
<td>12.0</td>
<td>13.0</td>
</tr>
<tr>
<td>Depth XS 39</td>
<td>-9.1</td>
<td>8.19</td>
<td>-16.3</td>
<td>18.6</td>
</tr>
</tbody>
</table>

¹ No low flow calibration was achieved for the rapid reach.
Figure 4.1. A) Observed versus calculated water surface elevation for the riffle-pool reach. The outlier is due to woody debris, which was not included in the model. B) Depth and, C) velocity at Cross-Section 35. All results are for a calibrated River 2D discharge of 0.62 m³/s. Observations are shown as open diamonds in B) and C).
Figure 4.2. A) Calculated water surface profile and, B) Observed depth versus HEC-RAS calculated depth for the riffle-pool reach. The model is calibrated to the 0.62 m³/s discharge. Observations are shown as open diamonds. The outlying point at 36m upstream (also seen as outlier in B)) is due to woody debris, which was not included in the model.
Similar results were achieved at the other observation cross-sections, in both the riffle-pool and rapid reaches, and for other calibration flows (see Table 4.1). HEC-RAS was also able to achieve reasonable agreement between calculated and observed water surface profiles (Figure 4.2); again excluding staff-gauge 37. While reasonable results where achieved with both models, the roughness parameter had to be varied significantly through the reach in order to match the observed water surface (see Figures 4.3 and 4.4).

Figure 4.3 and 4.4 shows that $k_s$ and Manning's $n$ values follow the same trend, although Manning's $n$ is more variable. Nikuradse's roughness length is intended to represent the skin roughness of uniform roughness elements. Since HEC-RAS is a 1D model the roughness parameter must also allow for bed- and plan-form resistance between cross-sections (for example, bars or channel curvature). HEC-RAS cross-sections where located at any significant change in topography, so many of these resistance components are calculated in the energy balance, but the variability suggests Manning's $n$ is not easily described by physical channel characteristics. Further, the water surface was measured at one point in the cross-section and non-uniform water surface across the cross-section caused calibration difficulties for HEC-RAS, particularly in the rapid reach. Both 1D and 2D models required large spatial variation in calibrated roughness coefficients in order to provide good agreement with observed water levels.

It should also be noted that the agreement between the HEC-RAS and River 2D calibrations is partly a function of the calibration method. Manning's $n$ and $k_s$ were increased or decreased as necessary in all areas where the water surface was inaccurate, even though the profile may have been forced by an up- or downstream control. This is seen is the high roughness length between 25 and 38m upstream despite the low relative roughness (Figure 4.9). In this section, it is likely there is a downstream control that is not captured by the model topography rather than bed roughness controlling the surface profile.
Figure 4.3. Comparison of spatial variation of roughness coefficients in the rapid reach for the 1D and 2D models. HEC-RAS (1D) was calibrated to 0.62 m$^3$/s and 0.04 m$^3$/s flows. River 2D was calibrated to 0.62 m$^3$/s. Note that $k_s$ is plotted on a logarithmic axis.

Figure 4.4. Comparison of spatial variation of roughness coefficients in the riffle-pool reach for the 1D and 2D models. HEC-RAS was calibrated to 0.62 m$^3$/s and 0.04 m$^3$/s flows. River 2D was calibrated to 0.62 m$^3$/s and 0.13 m$^3$/s flows. Both models required large variation in the roughness coefficient in order to provide good agreement with the observed conditions. Note that $k_s$ is plotted on a logarithmic axis.
4.2 Predictive Modelling

One of the primary uses of hydrodynamic models is to predict the hydraulics at conditions outside those observed. In order to test the predictive capabilities of the models, a range of flows were modelled and compared to observed water surfaces, depths and velocities. Calibrated River 2D and HEC-RAS models were prepared for both the rapid and riffle-pool reaches for a 0.62 m$^3$/s discharge. A low flow calibration was also prepared although River 2D could not replicate water surfaces below 0.62 m$^3$/s in the rapid reach. Figures 4.5 and 4.6 show observed versus calculated water depths from the two models. If the model was calibrated to a low flow, resistance was over predicted as the flow increases, thus the water surface was too high. Conversely, resistance was under predicted and water surfaces too low if the model was calibrated to a higher flow and applied to low flows. For example, the calculated depth is 73% lower than the observed depth when modelling a discharge of 0.62 m$^3$/s using the 0.13 m$^3$/s calibrated model.

The overall accuracy of the calibrated HEC-RAS water surface was typically better than River 2D, although this is considered a function of the calibration method. River 2D required significantly less calibration effort. When applied predictively, both models over- or under-predicted the water surface with similar degrees of inaccuracy. This variation is quantified in Tables 4.2 and 4.3. The cause of this poor predictive capability is the inability of the flow resistance models to capture the change in resistance with discharge.
Table 4.2. Summary of difference between observed and calculated flow depth in the Rapid reach. Difference between observed and calculated water surfaces, thus flow depth, was used as the primary calibration for the models. Difference is calculated as (observed depth) – (calculated depth) and % difference = 100*(observed depth – calculated depth)/(observed depth). Negative values indicate the calculated depth is greater than the observed depth, hence flow resistance is over predicted. Highlighted cells identify the calibrated flow.

<table>
<thead>
<tr>
<th>Calibrated Flow</th>
<th>0.62 m³/s</th>
<th>0.04 m³/s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Difference [m]</td>
<td>Standard Deviation [m]</td>
</tr>
<tr>
<td>River 2D Modelled Flow [m³/s]</td>
<td>0.13</td>
<td>0.06</td>
</tr>
<tr>
<td>HEC-RAS Modelled Flow [m³/s]</td>
<td>0.13</td>
<td>0.06</td>
</tr>
</tbody>
</table>

River 2D could not be calibrated to this flow.
Table 4.3. Summary of difference between observed and calculated flow depth in the riffle-pool reach. Values are calculated as described in Table 4.2. The 0.85 m$^3$/s and 1.29 m$^3$/s flow are compared to crest gauge data and are biased, thus not directly comparable with the other flows. Highlighted cells identify the calibrated flow.

<table>
<thead>
<tr>
<th>Calibrated Flow</th>
<th>0.62 m$^3$/s</th>
<th>0.13 m$^3$/s or 0.04 m$^3$/s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean Difference [m]</td>
<td>Standard Deviation [m]</td>
</tr>
<tr>
<td>0.13</td>
<td>0.03</td>
<td>0.01</td>
</tr>
<tr>
<td>River 2D</td>
<td>0.33</td>
<td>0.02</td>
</tr>
<tr>
<td>Modelled Flow [m$^3$/s]</td>
<td>0.44</td>
<td>0.0</td>
</tr>
<tr>
<td>0.62</td>
<td>0.0</td>
<td>0.03</td>
</tr>
<tr>
<td>0.85</td>
<td>0.03</td>
<td>0.04</td>
</tr>
<tr>
<td>1.29</td>
<td>0.03</td>
<td>0.05</td>
</tr>
<tr>
<td>0.44</td>
<td>0.00</td>
<td>0.01</td>
</tr>
<tr>
<td>0.85</td>
<td>0.03</td>
<td>0.02</td>
</tr>
<tr>
<td>1.29</td>
<td>0.01</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Figure 4.5. Riffle-pool reach. Observed versus calculated flow depths. The legend shows modelled flow with calibration flow in brackets. A) River 2D B) HEC-RAS.
Figure 4.6. Rapid reach observed versus calculated flow. The legend shows modelled flow with calibration flow brackets. A) River 2D and B) HEC-RAS.
4.3 Flow Resistance Models

In River 2D, bed resistance is modelled with a two-dimensional Chezy equation, described in the x-direction by

\[ S_{\beta} = \frac{\tau_{mx}}{\rho g H} = \frac{\sqrt{U^2 + V^2}}{gHC_s} U \]  

(4.1)

where \( S_{\beta} \) is friction slope, \( \tau_{mx} \) is bed shear stress in the x-direction, \( g \) is gravitational acceleration, \( \rho \) is density of water, \( U \) and \( V \) are depth averaged velocities in the x- and y-directions respectively and \( C_s \) is a non-dimensional Chezy coefficient, which is defined by a Keulegan type flow resistance equation,

\[ C_s = 5.75 \log \frac{12H}{k_s} \]  

(4.2)

However, \( C_s \) becomes increasingly sensitive to depth and roughness as the roughness ratio \( (H/k_s) \) becomes small, and if \( 12H/k_s < 1 \) Equation (4.2) gives negative values for \( C_s \). Therefore, in River 2D if

\[ \frac{H}{k_s} < \frac{e^2}{12} = 0.62 \]  

(4.3)

Equation (4.2) is replaced by Equation (4.4), which gives a smooth, continuous, non-negative relation for any flow depth, however, there is no physical basis for this equation (Steffler and Blackburn, 2002).

\[ C_s = 2.5 + \frac{30}{e^2} \frac{H}{k_s} \]  

(4.4)
Figures 4.7 and 4.8 show $d/k_s$ for the rapid and riffle-pool reaches respectively. Hydraulic depth, $d$, was used as an average cross-section depth rather than the local depth, $H$, and $d/k_s = 0.62$ was plotted to show the limit of Equation (4.2). Only for the 0.62 m$^3$/s calibration in the riffle pool reach does River 2D use the physically based resistance model (Equation (4.2)). While Equation (4.4) does calculate a variable $C_s$ with changing discharge, there is no reason to expect it to perform particularly well. In steep streams with poorly sorted sediments, Manning’s $n$ varies as discharge increases (Jarret, 1984, Lee and Ferguson, 2002: Aberle and Smart, 2003). Consequently, neither River 2D or HEC-RAS can be used to accurately predicate the conditions observed at East Creek. However, there is some evidence that River 2D can model higher flows in the riffle-pool reach. Flow resistance in steep streams is discussed in Section 2.1.3.

As mentioned in Section 4.2, River 2D was not able to replicate low flow water surfaces. To investigate this, HEC-RAS and River 2D roughness lengths were compared. Flow depth, $H$, Manning’s $n$ and $k_s$ are related by

$$k_s = \frac{12H}{e^m}$$  \hspace{1cm} (4.5)

where $m$ is given by

$$m = \frac{H^{\frac{1}{6}}}{2.5n\sqrt{g}}$$  \hspace{1cm} (4.6)

Equation (4.6) is derived from the relation

$$C = \frac{H^{\frac{1}{6}}}{n}$$  \hspace{1cm} (4.7)
Equation (4.2) valid

Equation (4.2) invalid

Figure 4.7. Variation in inverse relative roughness at Q = 0.62 m³/s for the rapid reach. If \( d/k_s < 0.62 \) (shown as dashed line), resistance is calculated via Equation (4.4).

Figure 4.8. Variation in inverse relative roughness for the riffle-pool reach. If \( d/k_s < 0.62 \) (shown as dashed line), resistance is calculated via Equation (4.4).
which assumes a hydrostatic pressure distribution (e.g. Henderson, 1966) and \( C \) is Chezy’s coefficient \([g^{1/2}]\). Combining Equations (4.2) and (4.5), a relationship for \( C_s, H \) and \( n \) in terms of the formulation used in River 2D is derived

\[
n = \frac{H^{1/6}}{2.5 \sqrt{g \ln 10}^{5.75}}
\]  

(4.8)

From Equation (4.4), as the depth to roughness ratio gets very small, \( C_s \) approaches a minimum of 2.5, which upon substitution into equation (4.8) gives

\[
n = \frac{H^{1/6}}{7.84}
\]  

(4.9)

Since \( H \) is raised to the power 1/6, the relation is relatively insensitive to flow depth and for the depths observed (average hydraulic depth \( \approx 0.25 \text{m} \)) the maximum Manning’s \( n \) that can be achieved is approximately 0.1 s/m^{1/3}. While typical values of Manning’s \( n \) for lowland rivers and artificial channels are around 0.03 to 0.05, the HEC-RAS Hydraulic Reference Manual (2004) gives guiding \( n \) values as high as 0.2 for certain conditions. In the riffle-pool reach, \( n \) values as high as 0.27 were used to calibrate HEC-RAS to the 0.04 m³/s flow, whereas the maximum \( n \) for the 0.62 m³/s calibration was 0.1. Since River 2D uses Equation (4.4), the model is not able to generate enough flow resistance to model the low flow conditions.

In Figures 4.7 and 4.8 it is clear that the River 2D is at or near the limit of the roughness calculation \( (C_s = 2.5) \) for the 0.13 m³/s flow in the riffle-pool reach and 0.62 m³/s flow in the rapid reach. Calibration to lower flows was attempted but the model could not match the observed water surface. In comparison, HEC-RAS calculates energy loss directly for a 1-dimensional form of Manning’s equation
\[ Q = \frac{1}{n} AR^{\frac{3}{2}} S^{\frac{1}{2}} \]  

(4.10)

and is calibrated by adjusting \( n \) directly hence there is no upper limit to the resistance and water surface calibration was able for all observed discharges.

To allow comparison of the calibrations the HEC-RAS Manning’s \( n \) was converted to an equivalent \( k_s \) with Equation (4.11), which is derived for metric units (USACE, 2004) and is equivalent to Equation (4.5).

\[ R^{\frac{1}{16}} k_s = 12.2 \frac{R}{10^{16n}} \]  

(4.11)

Figure 4.9. River 2D \( k_s \) and HEC-RAS \( k_s \) (calculated from Equation (4.11)) for the rapid reach calibrated to the flow of 0.62 m³/s. Note that River 2D \( k_s \) is plotted against the right ordinate on a logarithmic scale. HEC-RAS \( k_s \) is highly variable despite constant D₈₄. Variability in both HEC-RAS and River 2D \( k_s \) is probably associated with form resistance and rapid changes in topography, which cannot be captured by the model, thus artificially high resistance values are required to replicate observed depths.
Figure 4.10. River 2D $k_s$ and HEC-RAS $k_s$ (calculated from Equation (4.11)) for the riffle-pool reach calibrated to the flow of 0.62 m$^3$/s. HEC-RAS $k_s$ is highly variable despite constant $D_{84}$. River 2D $k_s$ can be considered a multiple of $D_{84}$ except between 25 and 38m where it is likely that form resistance is controlling the flow depth.

Calculating $k_s$ for the HEC-RAS low flow calibrations, a reach average $d/k_s$ of 0.16 and 0.13 was calculated for the riffle-pool and rapid reaches respectively, which is significantly lower than the River 2D limit of 0.62 for using Equation (4.2). In a 2D model, very shallow flows near banks and wetting and drying nodes could cause numerical problems if the use of a Keulegan type resistance equation is not limited.

The 0.62 m$^3$/s calibration in the riffle-pool reach was the only observed flood where the depth to roughness ratio was high enough to allow River 2D to use the physically based resistance equation. In the riffle-pool reach peak flood water surface was recorded at five locations with crest gauges. Using the crest gauge data and peak discharge, estimated from the hydrograph, two high flows (0.85 m$^3$/s and 1.29 m$^3$/s) were modelled. Figure 4.11 shows that the relationship between staff gauge and crest gauge readings is not the same at all gauges. Combined with uncertainty in the hydrograph, results from the two high flows cannot be compared directly with the other water surfaces (<0.62 m$^3$/s). The
model error appears biased but does not change between the 0.85 m$^3$/s and 1.29 m$^3$/s flows for the riffle-pool reach calibrated to 0.62 m$^3$/s (Table 4.3). Although certainly not conclusive, this suggests that flow resistance is being modelled reasonably. The error continues to increase for the 0.13 m$^3$/s calibration. Thus, the 0.62 m$^3$/s calibration can be used to estimate the hydraulics of the riffle-pool reach for flows above 0.62 m$^3$/s.

In figure 4.8 we see that the riffle-pool 0.62 m$^3$/s calibration is right on the limit of Equation (4.2) (Equation (4.4) is used between 25 and 38m). In this calibration $k_s/D_{84} \approx 4.5$ (excluding 25 to 38m), hence $k_s \approx 4.5D_{84}$ (see Figure 4.12). Nikuradse originally defined $k_s$ for uniform flow as the mean diameter of a uniform sediment mixture. For non-uniform sediments, $k_s$ is expressed as some multiple of a characteristic grain diameter ($k_s = CD$) where the multiplier is a function of flow resistance components (e.g. bed- and surface-form) included in the analysis (Millar, 1999). Thompson and Campbell (1979) proposed $k_s = 4.5D_{84}$, while Millar (1999) presented results from several studies where “best fit” results ranged from $k_s = 3.2D_{84}$ to $k_s = 3.9D_{84}$. Assuming $k_s = 4.5D_{84}$, Equation (4.3) can be written as

$$\frac{H}{4.5D_{84}} < 0.62$$

(4.12)

Thus, if

$$\frac{D_{84}}{H} > 0.35$$

(4.13)

the non-physically based resistance equation (Equation (4.4)) is used. Consequently, the maximum relative roughness River 2D can model is 0.35. The relative roughness ($D_{84}/d$) of the riffle-pool reach for a discharge of 0.62 m$^3$/s is 0.35. However, since there is considerable variability in the relationship between $k_s$ and $D_{84}$, this limit may not applicable to other streams but can be considered a reasonable indicator.
Figure 4.11. Comparison between staff gauge water surface at 0.38 m$^3$/s and Crest gauges at 0.40 m$^3$/s, the bias reflecting the higher discharge for the crest gauge readings.

Figure 4.12. $k_s/D_{84}$ for the riffle-pool reach calibrated to 0.62 m$^3$/s. Reach average $k_s/D_{84}$ (excluding 25 to 38m) is 4.5.
4.4 Modelling Diverse Topography

Rapid changes in topography were found to cause modelling difficulties. The outcrop shown in Figure 4.13 coincides with the downstream extent of the high $k_s$ values in the riffle-pool reach (at 25m upstream in Figure 4.10) and while this feature is included in the model topography it is likely that the models cannot accurately represent this constriction. Figure 4.1 shows depth and velocity at Cross-section 35. From the observed velocities there is an area of recirculation (negative velocities at 1m from the right bank) behind the outcrop, which is not resolved in River 2D and not considered by HEC-RAS. The influence of this control may well change as discharge varies. This feature and other similar features, for example rip-rap boulders, many of which are not accurately represented in the DEM are probably part of the cause of poor results when the models are used. Representation of these features introduces difficulties both in topographic resolution required and in the capabilities of hydrodynamic models. It is suggested that River 2D cannot accurately capture the hydraulics of bed features with horizontal extent less than about 10 flow depths (Steffler and Blackburn, 2002).

Despite that hydraulics may not be accurately modelled, it is at least interesting that River 2D did not have numerical stability problems with the diverse morphology, transcritical flow, and wetting and drying of nodes; all of which are typically challenging for 2D numerical models.
Figure 4.13. The outcrop on the right bank is included in the model topography but the hydraulics may not be correctly represented.
5.0 Sediment Transport Analysis

While both HEC-RAS and River 2D were able to replicate water surfaces with reasonable accuracy, the objective of this study was to assess the hydraulics and provide information suitable for sediment transport or aquatic habitat studies. However, spatially distributed depth, velocity and shear stress are difficult to measure in the field. Hydrodynamic modelling allows increased spatial and temporal resolution of these characteristics, but direct validation (i.e. observed versus calculated) is limited. In order to assess the hydraulics, observed and calculated sediment transport was used as an indirect test. This test introduces new unknowns such as the accuracy of transport formulae and observed sediment transport rates, but allows investigation of the general performance of the hydrodynamic models and allows investigation of the suitability of bedload transport formulae to such conditions.

Although the intension is that transport equations are universal, at least part of their formulation relies on experimental data and/or field observations, consequently they tend to be semi-empirical. It is recommended that an equation used be derived from similar conditions as experienced at the site of interest. The bedload transport formula of Wilcock and Crowe (2003) presents a thorough and physically based approach to sediment transport, thus it was selected for this analysis. Through application of the formula, limitations were investigated. Sensitivity of the model to surface grain size and inability to accurately define this parameter, including variation with time, mean performance is poor. Consequently the Meyer-Peter and Muller formula was applied, which is based on sub-surface grain size distribution (GSD) and can be considered to represent a temporally averaged sediment conditions.
Table 5.1. Observed and calculated sediment transport quantities using the Wilcock and Crowe (2003) formula. Hydraulic data is from River 2D and HEC-RAS. Calibrated $D_{50}$ is a $D_{50}$ that gave a good match between observed and calculated transport for the 25 November event with the River 2D data.

<table>
<thead>
<tr>
<th>Event</th>
<th>Transport (kg)</th>
<th>Observed $D_{50}$</th>
<th>Calibrated $D_{50}$</th>
<th>HEC-RAS (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Observed</td>
<td>= 36mm</td>
<td>= 70mm</td>
<td>Observed</td>
</tr>
<tr>
<td>25 Nov 2004</td>
<td>252</td>
<td>14388</td>
<td>284</td>
<td>69403</td>
</tr>
<tr>
<td>10 Dec 2005</td>
<td>246</td>
<td>59072</td>
<td>2646</td>
<td>145505</td>
</tr>
</tbody>
</table>

Results of the Wilcock and Crowe (2003) formula are shown in Table 5.1 and show the strong dependency on shear stress, $\tau_0$, and reference shear stress, $\tau_{ri}$, which is a function of $D_{50}$. The Wilcock and Crowe (2003) transport function is defined as

$$W_i^* = 0.002 \phi^{0.5} \quad \text{for } \phi < 1.35$$

$$W_i^* = 14.1 - \frac{0.849}{\phi}^{4.5} \quad \text{for } \phi \geq 1.35$$

(5.1)

where $\phi = \tau / \tau_{ri}$. Equation (5.1) reveals that at near threshold conditions the dimensionless transport rate, $W_i^*$, is a function of $(\tau_0 / \tau_{ri})^{0.5}$ where

$$\tau_{ri} = \tau_{r50} \frac{D_i}{D_{50}}$$

(5.2)

$$\tau_{r50} = \tau_{rm} g(s-1)D_{sm}$$

(5.3)

$$\tau_{rm} = 0.021 + 0.015 \exp[-20F_s]$$

(5.4)

and $D_i$ is grain size of the $i$th fraction, $\tau_{r50}$ is reference transport rate of surface median grain size $D_{50}$, $D_{sm}$ is the diameter of the mean surface material, $\tau_{rm}$ is Shields number for the surface sediment mixture, $s$ is the ratio of sediment density to water density and $F_s$ is the percent of sand on the bed surface. Thus, for modest changes in $D_{50}$, transport
quantities of 3 orders of magnitude were calculated. Both shear and $D_{50}$ are highly spatially and temporally variable hence a wide range of results is possible. Surface grain size data was collected via a Wolman count conducted on 1 February 2005, following a major flow event (peak discharge = 2.9 m$^3$/s). It is likely that this event significantly changed the surface structure and grain size distribution compared to the surface during the November and December events that have been modelled.

Figure 5.1 shows the shear stress distribution from the two models with a discharge of 1.6m$^3$/s. The two results are quite different and shear stress that is effective at transporting sediment must also be considered. Shear stress calculated with River 2D shows the variation across the channel and includes both grain and bed structure effects where only the grain component is effective in transporting sediment (Einstein and Barbarossa, 1951). This is accounted in Wilcock and Crowe (2003) the bedload transport formula via the hiding function. In comparison, the shear stress calculated by HEC-RAS includes other resistance components such as plan-form and bank resistance as well as grain and micro-topography components, consequently HEC-RAS calculates higher transport rates than River 2D (Table 5.1). Considering the sensitivity of the Wilcock and Crowe (2003) formula to $\tau_0$, this variability introduces significant uncertainty.

![Figure 5.1. Shear stress distribution calculated with River 2D and the section average shear from HEC-RAS at cross section above pit trap for 1.6 m$^3$/s.](image)
If a surface based model were to be applied to a 2-dimensional morphological model, a detailed surface description would be required as an initial boundary condition. Further, the morphologic model would have to update the bed surface condition during a simulation to allow for coarsening or fining of the surface. Careful consideration of the model resolution is required, as the stochastic nature of sediment entrainment must be considered. Following this result, the Meyer-Peter and Muller equation was applied, which is based on subsurface GSD and thus represent a spatially and temporally averaged bed condition. However, as Table 5.2 shows, the results remain poor.

Using the subsurface GSD the River 2D results are several orders of magnitude above the observed transport quantities, comparatively the HEC-RAS results seem more reasonable. The surface GSD was also applied and transport quantities reduced considerably with both models, in fact transport was only predicted with River 2D for flows greater than 1.2 m³/s and not at all with HEC-RAS. At the timescale of a single event as has been modelled here, the surface GSD is a better representation of the sediment available for transport, although even at such a relatively short time scale this parameter cannot be accurately quantified. This result leads to the question of what spatial and temporal scales are appropriate for sediment transport formulae.

Table 5.2. Observed and calculated sediment transport quantities using the Meyer-Peter and Muller formula. Results based on surface and subsurface grain size distribution (GSD) are compared.

<table>
<thead>
<tr>
<th>Event</th>
<th>Observed Transport (kg)</th>
<th>River 2D [kg]</th>
<th>HEC-RAS [kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Surface GSD</td>
<td>Subsurface GSD</td>
</tr>
<tr>
<td>25 Nov 2004</td>
<td>252</td>
<td>0</td>
<td>&gt;43,000</td>
</tr>
<tr>
<td>10 Dec 2005</td>
<td>246</td>
<td>5073</td>
<td>&gt;100,000</td>
</tr>
</tbody>
</table>
Figure 5.2. Flow duration curve for the two modelled events showing significant sediment transporting discharges (>0.5 m$^3$/s). Although the peak discharge of the December event was higher, it was a more flashy flow.

Figure 5.2 shows flow-duration of the two events. Without information about changes in the surface GSD it is unlikely that the Wilcock and Crowe (2003) can consistently predict transport. While, applying the Meyer-Peter and Muller (1948) formula with subsurface GSD requires long durations and equilibrium transport conditions. At East Creek, bedload is trapped above each study reach and suspended sediment is minimal so that sediment transported must be entrained from the bed (and limited bank supply). In order to achieve accurate and reliable results attention should be focused on the resolution with which data can readily collected and the resolution that the bedload transport models are applicable.
5.1 Spatial Distribution of Sediment Transport

In order to examine the spatially distributed hydraulic data calculated with River 2D the sediment transport rate, \( q_b \), was calculated with the Wilcock and Crowe (2003). Due to the limitations discussed in Section 5.0 the results are not expected to be quantitatively correct but Figure 5.3 shows qualitatively the spatial variation of sediment transport rate between discharges of 0.5 m\(^3\)/s and 1.0 m\(^3\)/s. It should be noted that deposition or scour occur where there is a change in transport rate not necessarily were the transport rate is highest or lowest. Applying a similar method as described by Equation (3.3) and calculating the transport at each computational node the change in sediment can be calculated from \( \Delta Q_{b,\text{total}} = \Delta Q_{b,\text{in}} - \Delta Q_{b,\text{out}} \). By converting to volumetric sediment quantity and assuming the sediment is evenly distributed over the computational cell the bed elevation can be updated. The channel topography at East Creek was surveyed in August 2004 and again following the flood season in February 2005. The intention was to compare before and after observed topography with calculated topography, at least areas of scour and deposition, although this analysis has not been completed to date. Intuitively, the results seem reasonable except in the area identified in Figure 5.3. This area coincides with high roughness coefficient, \( k_s \), in the River 2D model caused by form resistance rather than grain resistance. Even though observed depth and depth-averaged velocity matched calculated results well, shear stress is obviously not well represented. The spatial distribution of bedload transport is difficult to quantify.

The bedload trap in the riffle-pool reach is located at the downstream extent of the modelled reach. It interesting to note that transport is not predicted at the trap for the 0.5 m\(^3\)/s flow, while transport is occurring upstream. This result further highlights the need to carefully consider spatial and temporal resolution issues.
Figure 5.3. Sediment transport rate, $q_b$ [g/m/s] for A) 0.5 m$^3$/s B) 1.0 m$^3$/s. The data was plotted as log($q_b$) showing transport rates >0.1 g/m/s. The sediment trap is located at the very downstream extent of the modelled reach (left of plot).
6.0 Conclusions

Both HEC-RAS and River 2D were able to replicate observed water surface profiles, although detailed water surface profiles and extensive calibration were required and to accurately model conditions. In HEC-RAS, Manning’s \( n \) varied significantly in magnitude and rapidly in spatial extent, for example it ranged from 0.07 to 0.25 for the low flow calibration in the riffle-pool reach. This variation is a result of resistance from bedforms, micro-topography and large roughness elements. It also suggests Manning’s \( n \) cannot be readily predicted from channel conditions when resolving flow conditions at this resolution. Roughness length, \( k_r \), used to calibrate River 2D was found to be less variable although form resistance associated with abrupt topographies caused difficulties. For a well-calibrated reach, the calculated depths and velocities agree well with observed values and accuracy appears to be a function of the calibrated water surface and the agreement between modelled and actual topography.

However, when the hydrodynamic models were applied predictively they were unable to accurately represent the variation in flow resistance and hence could not accurately predict flow conditions outside the calibrated flow. If calibrated to low flow conditions, flow resistance was over predicted at higher flows, resulting in low water surfaces. In the rapid reach water depth was 30% lower than observed depths, while flow depths were 25% under predicted in the riffle-pool reach. Conversely, if calibrated to higher flow conditions, flow resistance was under estimated at low flow and calculated water surfaces were too high (30% and 25% greater than observed flow depth in the rapid and riffle-pool reaches respectively).

Many researchers have found that Manning’s \( n \) does not remain constant as discharge varies. And River 2D applies a non-physically based resistance equation when relative roughness is large. It was found that only the 0.62 m\(^3\)/s calibration in the riffle-pool reach used the physically based, Keulegan type, resistance equation (Equation (4.2)). In conditions that allow application of the physically based equation, it was found that
$k_s \approx 4.5D_{84}$, which generally agrees with results found by others (see Millar, 1999). If $k_s = 4.5D_{84}$ it can be shown that physically based equation is only applied if relative roughness ($D_{84}/d$) is less than 0.35. The $0.62 m^3/s$ flow was the highest flow recorded with detailed observations of depth and velocity, however evidence from crest-gauge water-surface data suggest the model accurately predicts water surface elevations at higher flows. Thus, $D_{84}/d < 0.35$ can be considered an indicative limit for the predictive use of River 2D.

While the performance of the hydrodynamic models was not ideal, it is suggested that they are generally capable of calculating spatially and temporally distributed data in headwater streams on the same order of accuracy as readily obtained observed data, but should be used with caution if applied predictively. Further, hydrodynamic models are typically calibrated to water surface observations, which are easily collected, however in steep streams with diverse and rapidly varied topography, agreement between observed and calculated water surfaces does not imply that velocity distribution is well represented. And agreement between observed and calculated depths and velocities does not necessarily indicate shear stress is well represented.

When applied to sediment transport, difficulties with both the hydrodynamic and sediment transport models mean results were poor. The Wilcock and Crowe (2003) formula was found to be very sensitive to spatial and temporal variation in grain size distribution and reference shear stress. Quantification of these parameters in field conditions is difficult, thus a wide range of results is possible. Applying the Meyer-Peter and Muller (1948) formula, similar mismatch between spatial and temporal observations and model ability was found to cause poor results. Sensitivity of the bedload formulae to hydraulic and bed-surface conditions, both of which are highly variable in steep streams, and characterisation of the bed surface to include surface structure must be improved before reliable results can be achieved. Despite the poor results achieved, the approach does provide insight into the sensitivity of parameters and limitations of the models applied.
East Creek has been described by some as a "flume in the field". It's small size means
detailed observations can be collected relatively quickly and easily. Building on the work
presented in this thesis, there is potential to further study hydrodynamic modelling, flow
resistance, and sediment transport formulae in headwater streams where sediments are
poorly sorted in a field environment. Further work should focus on the resolution with
which the models are applicable and with which field conditions can be readily
quantified. Obviously increased resolution requires increased boundary and initial
conditions, which may or may not be achievable. Conversely, both hydrodynamic and
bedload transport models are based on spatially and temporally averaged results and
conditions, hence reduced resolution may be the only solution for reliable results.
References


Appendix A. Crest gauge design (USGS).

Note—Set 8-penny nail at top of measuring stick for flush fit with cap.