A VISUAL MODEL FOR PREDICTING STREAM RESPONSE OF ALLUVIAL GRAVEL-BED RIVERS

by

Nerissa Yarmila Moscote

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ABSTRACT

In this thesis, a visual optimization model, *RiverMod*, has been developed to predict the channel geometry of alluvial gravel-bed rivers. The model is an extension of a previous model developed by Millar and Quick (1993b). The model is based on equilibrium theory, and also includes a bank stability analysis. The adjustment of the dependent variables of a river reach is quantified according to changes incurred by the independent variables. The primary dependent variables are channel width, depth and slope, and the primary independent variables are the discharge, flow resistance parameters, sediment size distribution and bank stability parameters. RiverMod, written in Visual Basic, provides a user interface that promotes clear and simple model usage for multiple runs with different sediment transport and flow resistance equations.

The theory behind the Millar and Quick (1993b) model is discussed, as well as Millar's (2000) meandering-braiding transition and its incorporation into RiverMod. Four sediment transport equations and four flow resistance equations are included as part of the model. The model consists of fixed-channel-slope and variable-channel slope versions. The fixed slope version is equivalent to an experiment where the slope is fixed, and the channel width, depth and sediment transport rate adjust to the discharge. The variable slope version more closely approximates natural stream conditions. Each of these models can be applied to streams with cohesive or non-cohesive bank sediment.

The model is applied to two gravel-bed rivers, located in British Columbia. The bank stability of both rivers has been decreased due to logging along the banks. RiverMod is used to quantify the impact of such a disturbance by analyzing past and present channel conditions. Restoration methods are suggested based on model output.

This thesis provides a full description of theory underlying the model development and a description of model usage; therefore, this thesis is a complete user manual for the RiverMod optimization model.

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List of Symbols

A	=	cross-sectional area
A	=	function of grain diameter (Ackers and White)
a	=	empirical coefficient
C	=	function of grain diameter (Ackers and White)
c	=	soil cohesion
C_x	=	constant (Keulegan)
d	=	depth
D	=	sediment grain size
D_{35}	=	thirty-fifth percentile bed surface grain diameter
D_{50}	=	median bed surface grain diameter
d_{50}	=	median sub-surface grain diameter
D_{50bank}	=	median bank grain diameter of non-cohesive sediment
D_{90}	=	ninetieth percentile bed surface grain diameter
D_a	=	average bed surface grain diameter
D_{gr}	=	grain diameter (Ackers and White)
D_x	=	characteristic grain diameter, where x is % of sediment finer
F	=	fall parameter (Einstein-Brown)
f	=	Darcy-Weisbach friction factor
F_{gr}	=	sediment mobility number (Ackers and White)

F_r = Froude number

FS_H = factor of safety for mass failure

FS_τ = factor of safety for fluvial erosion

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

G_b = sediment transport capacity

g_b^* = dimensionless transport rate

G_{bcalc} = sediment transport capacity

G_{bf} = bankfull sediment transport capacity

G_{gr} = sediment transport (Ackers and White)

H = vertical bank height

H = Hammer number

H_{crit} = maximum stable value of H,

I = input of sediment at upstream end of channel

i_b = specific bed load transport rate (dry weight) (Gomez and Church, 1989)

K_B = component of the reducing multiplier (Meyer-Peter and Müller)

K_G = component of the reducing multiplier (Meyer-Peter and Müller)

k_s = equivalent roughness

ℓ = distance along valley axis

L = distance along meander arc length

m = function of grain diameter (Ackers and White)

n = function of grain diameter (Ackers and White)

n = Manning's roughness coefficient

n_{bank} = Manning's roughness coefficient for the bank sediment

n_{bed} = Manning's roughness coefficient for the bed sediment

N_s = dimensionless stability number

O = output of sediment at downstream end of channel

P_{bank} = bank perimeter

P_{bed} = wetted perimeter of bed

Q = discharge

Q_{bf} = bankfull discharge

q_s = sediment discharge

r_c = radius of curvature

R_h = hydraulic radius

S = sediment storage

S = channel slope

S^* = transition slope

SF = shear force

s_s = specific gravity of sediment (assumed = 2.65)

S_v = valley slope

U^* = shear velocity

U,u = mean velocity

W = width of channel

X	=	sediment flux (Ackers and White)
Y	=	depth of channel
Δ	=	change
Φ	=	maximum deviation angle along channel
Φ_i^*	=	transport parameter (Einstein-Brown)
ϕ'	=	modified friction angle
ϕ_{50}	=	shear stress ratio (Parker, Klingeman and McLean)
γ	=	unit weight of water
γ_s	=	unit weight of sediment
γ_t	=	saturated unit weight of soil
λ	=	meander wavelength
μ	=	dynamic viscosity
ν	=	kinematic viscosity
θ	=	bank angle
ρ_f	=	fluid density
ρ_s	=	sediment density
τ	=	shear stress
τ^*	=	dimensionless bed shear stress
τ_{bank}	=	mean bank shear stress
τ_{bed}	=	mean bed shear stress
τ_{crit}	=	critical shear stress for fluvial erosion of bank sediment

τ_{d50} = bed shear stress for the median sub-surface grain size (Parker, Klingeman and McLean)

ω^* = dimensionless bedload transport rate (Parker, Klingeman and McLean)

ξ = sinuosity

ξ_c = sinuosity calculated using the sine-generated curve

ψ = ratio of forces acting on a particle

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Chapter 1

INTRODUCTION

1.1 INTRODUCTION

This project is an extension of a hydraulic geometry model developed by Millar and Quick in 1991, 1993 and 1998. Millar developed "a static analytical model based on previous optimization formulations (Millar (1991), Chang (1980), White *et al.* (1982))". The model is an optimization model which incorporates the effects of bank-stability on the hydraulic geometry of alluvial gravel-bed rivers.

The original model, written using Basic, is not very flexible, is difficult to use and is therefore used primarily as a research tool. The computer program described in this thesis, named RiverMod, was written using Microsoft Visual Basic. It combines four separate, related models and includes several new features. It is visual, user-friendly and includes:

(i) Four sediment transport relations

(ii) Four flow resistance equations

(iii) A planform model

(iv) A user-interface for input and output of data

(v) Compatibility with Microsoft Excel

(vi) The ability to save, rewrite and group files

RiverMod applies to single-thread alluvial gravel-bed rivers with mobile beds that have the capacity to modify their channel dimensions (Millar, 1994). The assumption that the model treats the river as an equilibrium system forms the basis for the optimization model.

1.2 Uses of the Model

The model has two primary applications: It has proven to be both an effective teaching and research tool. The following is a list of possible uses:

(a) River adjustments: RiverMod can help interpret river adjustments by helping researchers and students gain a further understanding into the behaviour of rivers and also predict future changes.

(b) Activities that impact rivers: The model can aid in monitoring the effects of disturbances such as urbanization, agricultural development, the construction of dams and reservoirs and logging on river channel geometry. The effects of natural processes such as wild fire or climate change can also be studied.

(c) Sensitivity Analysis: The model can be used to run a sensitivity analysis in order to determine how sensitive a variable is to a particular activity.

(d) Calibration: The program can also be calibrated to the observed geometry. This can assist in the determination of a variable which is hard to measure with field evaluations.

1.3 EQUILIBRIUM

RiverMod analysis can be applied to a reach of a river considered to be in equilibrium. For the purposes of this study, a channel will be considered to be in equilibrium when the following conditions are met (Millar, 1994):

(i) "The mean hydraulic geometry of the channel reach remains unchanged over an appropriate time scale for which a steady-state equilibrium can be assumed.

(ii) There is no net erosion or deposition along the reach.

(iii) Any perturbations from equilibrium geometry will be offset and the equilibrium geometry restored."

1.4 TEMPORAL AND SPATIAL SCALES

A reach of a river can be said to be in equilibrium when continuity is conserved during specific temporal and spatial scales.

Three principal time scales should be considered: Engineering time, which usually spans between 100 and 200 years; geomorphic, or graded time, which is usually less than 10,000 years and; geologic time, which is usually greater than 10,000 years.

For example, over a geologic time frame, the slope may appear to be constantly changing i.e. adjusting toward equilibrium. But in the short-term, the slope will appear to have reached a state of approximate equilibrium (see Figure 1.1).

Adjustments needed to achieve grade usually occur over several decades or over hundreds of years. RiverMod analysis applies to engineering time scales over which a reach or a river can be considered to be in an approximate equilibrium.

Equilibrium of a specific system depends on the spatial scale. Generally, an entire river will not be in equilibrium, but certain reaches of the river may be. The term "reach of a river" is usually used because different reaches of the same river may exhibit different channel patterns and therefore different ultimate graded conditions at the same point in time.

1.5 Independent and Dependent Variables

In order to study the way in which rivers adjust their form and dimensions, the variables defining the hydraulic geometry of the channel must be identified and the independent and dependent variables must be specified (Hey, 1982). An independent variable is imposed externally on the system; a dependent variable is determined by the value of the independent variables (Vanoni, 1975).

MacVicar (1999) defines the independent variables as those imposed on the reach, namely: climate, geology, runoff, vegetation type, vegetation density, and relief. Climate, geology and runoff determine the discharge, sediment size and sediment yield. Bank stability is controlled by the vegetation type and density and relief determines the valley slope. At the decade time scale, the independent variables become: discharge of water, sediment discharge, sediment size, bank stability (bank vegetation and soil properties) and valley slope.

The dependent variables are hydraulic geometry variables which respond to changes in the imposed independent variables. The primary dependent variables can be considered to be width, depth and bed slope. Other secondary dependent variables, which can be determined from the primary ones are velocity, sinuosity, meander arc length, maximum depth and height of bedforms (MacVicar, 1999). Figure 1.2 is a statistical model of a river system which illustrates the relationship between the dependent and independent variables (Hey, 1976 as cited by Hey, 1982).

Any significant change in the independent variables will produce a new regime channel geometry which corresponds to the modified values of the controlling variables. When the channel is again stable, it will be defined by the new independent variables (Hey and Thorne, 1986).

When using RiverMod to analyze a river reach, the water and the sediment discharges constitute primary independent variables. The channel slope, width and depth constitute primary dependent variables (Vanoni, 1975).

RiverMod attempts to quantify channel changes. The independent variables are input into the model and the corresponding dependent variables are estimated. In this way changes in both the dependent and independent variables of the river system can be simulated.

1.5.1 Adjustment of the Dependent Variables

The dependent variables are mutually dependent and a river may adjust its width, depth and slope simultaneously (Henderson, 1966).

There are three main governing equations for a river system associated with the independent variables. They are the flow resistance equation, the sediment transport equations and a bank stability relation (Henderson, 1966). Although there are an infinite number of combinations of width, depth and slope values for the given independent variables, Gilbert (1914) has shown through flume experiments that these may still be tending toward an optimum determined by the independent variables. The optimum is assumed here to represent an equilibrium condition.

Figure 1.3 is a schematic representation of two solution curves illustrating the theory behind Gilbert's (1914) flume experiments. In Figure 1.3(a) discharge and slope are held constant and an optimum width develops which corresponds to the maximum sediment transporting capacity, similar to Gilbert's flume experiments, described above (Gilbert, 1914). In Figure 1.3(b) discharge and sediment transport are held constant and the optimum is now at the minimum slope. This second case mimics the behaviour of most natural rivers over engineering time scales (Millar, 1994).

When a channel responds to a disturbance or a change in the independent variables, generally, the fastest changing dependent variables are the depth and the width. The slope is the slowest of these three variables to change (Booth, 1990). Figure 1.4 illustrates the dependent variables of the river initially fluctuating about the equilibrium condition, until a large change is imposed. During adjustment to a new, post-disturbance equilibrium, the channel passes through a transient adjustment phase. The non-equilibrium transient phase could persist for years, or decades. Figure 1.4 also shows that at any point during the pre-disturbance stage, the dependent variables may not be in constant equilibrium, but may be in equilibrium in a statistical or average sense.

1.6 STREAM IMPACTS AND CHANNEL RESPONSES

The river system, viewed as an ideal watershed, can be divided into three zones of erosion, transport and deposition (see Figure 1.5). The transport of sediment is continuous and any changes , or disturbances, which affect any reach within the catchment may affect the river downstream (Kondolf, 1997).

Occasionally, cessation of the disturbance will return the stream back to its natural conditions. However, more often, "although riparian pressures have been removed, continued disturbance on a watershed scale may limit the ecological conditions and override the influence of channel adjustments" (Magilligan and MacDowell, 1997).

The following sections discuss the independent variables, stream impacts and subsequent channel responses .

(a) Peak flow discharge: Channels adjust to the dominant discharge, which is at or near bankfull discharge (about the 2-year flood), when most sediment transport occurs (Werritty, 1997).

(b) The sediment load and calibre: The sediment supply and the size of the bed material will likely be affected by any change that produces an instability in the flow regime (Hey and Thorne, 1986).

(c) Bank stability: The bank type (cohesive or non-cohesive) determines whether the channel is more susceptible to fluvial erosion or mass failure. Both fluvial erosion and mass failure may be functions of parameters such as vegetation (Millar, 1994; MacVicar, 1999).

(d) The valley slope: The valley slope is considered relatively constant in engineering time scales; therefore, channel changes and impacts will only be discussed as they pertain to bank-full discharge, the sediment load and calibre, and the bank stability.

1.6.1 Urbanization

Urbanization near a stream often signifies more impervious areas in the vicinity of the stream. Typical examples of urban development include the construction of paved streets and sidewalks, sewers, houses and other buildings, and parking lots. Surface cover might initially increase the sediment load during construction (Dunne and Leopold, 1978) and later decrease the availability of sediment and increase the amount of water reaching the stream, causing increased channel erosion or stream incision (Macklin and Lewin, 1997).

Although there may be an increase in the volume of runoff reaching the stream, many tributaries intended to carry runoff toward a channel, may get buried or eliminated. This might result in the disappearance of many smaller channels that under natural conditions assisted in keeping both sediment and runoff distributed. Rock Creek located in the Washington, D.C. Area, provides an example of such an event. Figure 1.6 shows the drainage net for this creek in 1913 and 1964, both before and after modern urbanization (Dunne, and Leopold, 1978).

Figure 1.7 shows a graph of channel cross-section vs. drainage area for both rural and urban streams in Pennsylvania. It shows that rural streams have, on average, smaller cross-sections than their urban counterparts of similar size (Dunne and Leopold, 1978). Another recent study comparing rural and urban catchments in southeastern Pennsylvania found that, overall, urban streams were wider and straighter than their rural counterparts (Pizzuto, *et al.,* 2000).

The bankfull sediment load may increase or decrease depending on the type of development. During the development stages, effects such as land clearing may temporarily increase the sediment load, but once development is completed, if the area is well paved, the sediment load may decrease. The median grain-size (D_{50}) is usually utilized as an indicator of the distribution. It is a very difficult parameter to predict because the nature in which it is affected depends very highly on the type of development and the D_{50} of any new materials introduced to the system, or any materials removed from the system. Therefore in most cases it is assumed to remain constant (Personal

Communication, Millar, 1998; Human Impacts on River Systems (CIVL 598) lecture notes - University of British Columbia).

The effect of urbanization on bank stability is also difficult to predict. Post-development stabilization of the banks would increase the bank stability. Without it, with the removal of riparian vegetation it would decrease.

1.6.2 Logging and Agriculture

In forested drainage basins, trees are normally cut down in order to use the wood or for farming or grazing to take place (Macklin and Lewin, 1997).

(a) Logging: The most obvious effect of logging activity is the increase in sediment load conveyed into the channel. Large amounts of disturbed soil from increased activity, coupled with the upheaval of tree-root systems, cause channel instability by increasing the occurrence of mass soil movements and landslides, which can then increase the sediment load (Dunne and Leopold, 1978). The sediment size would probably remain unchanged, since any incoming sediment would consist of a larger quantity of the same source.

In most areas affected by logging, bank stability is decreased. Buffer strips of adequate size, left adjacent to the streambanks, might reduce the impact of deforestation. Logging usually reduces the amount of large woody debris (LWD) in a channel, further reducing the bank stability, since LWD can partially armour banks, add to channel roughness and act as a sediment trap.

(b) Agriculture: Land is often cleared of trees to facilitate cultivation or grazing; therefore, the initial stages of agricultural development may have similar effects on a channel as logging. If land clearance spreads to steeper slopes, gullying and mass wasting can increase the proportion of coarse material entering the channel (Clark and Wilcock, 2000), and therefore change the grain-size distribution of the stream. This can adversely affect those species of wildlife dependent on the stream.

Magilligan and McDowell (1997) conducted a study which investigated the geomorphic adjustment following the removal of cattle grazing. They observed that in those catchments where cattle grazing had been removed for at least 10 years the bankfull and low flow widths had narrowed and the bed had been remobilized into more pool area.

1.6.3 Dams and Reservoirs

The primary effect the construction of a dam would have on a channel is the interruption of the sediment transport of the river. The bedload and all or some of the sediment load is trapped upstream of the dam in the reservoir. The water released by the dam may then erode the bed and banks downstream of the dam, decreasing the bank stability and reducing the total sediment load of the system. Figure 1.8 shows the predicted natural yield of a river and the actual yield after dam construction. The actual yield is markedly less than the predicted natural value (Kondolf, 1997).

Increased erosion could possibly change the particle size of the bed. In a gravel-bed river this could signify larger grains downstream of the dam. This can threaten salmonid spawning sites if the bed becomes so coarse that the fish can no longer move the gravel (Kondolf, 1997).

1.6.4 Mineral Extraction

(a) Instream Gravel Mining: Instream gravel mining - the extraction of gravel directly from the stream bed - may cause channel incision upstream and downstream of the mining site, due to sediment shortage, coarsening of the bed, and lateral channel instability.

(b) Metal Mining: Before mining legislation, waste from mine sites was discharged directly into the nearest channel, altering the entire fluvial system. Changes include: hindering vegetation growth along the banks, negatively affecting the bank stability; changes in channel pattern over short periods of time (Macklin and Lewin, 1997); and the presence of toxic materials in the fluvial system due to the extraction and processing of metal ores for up to thousands of years (Macklin, 1996).

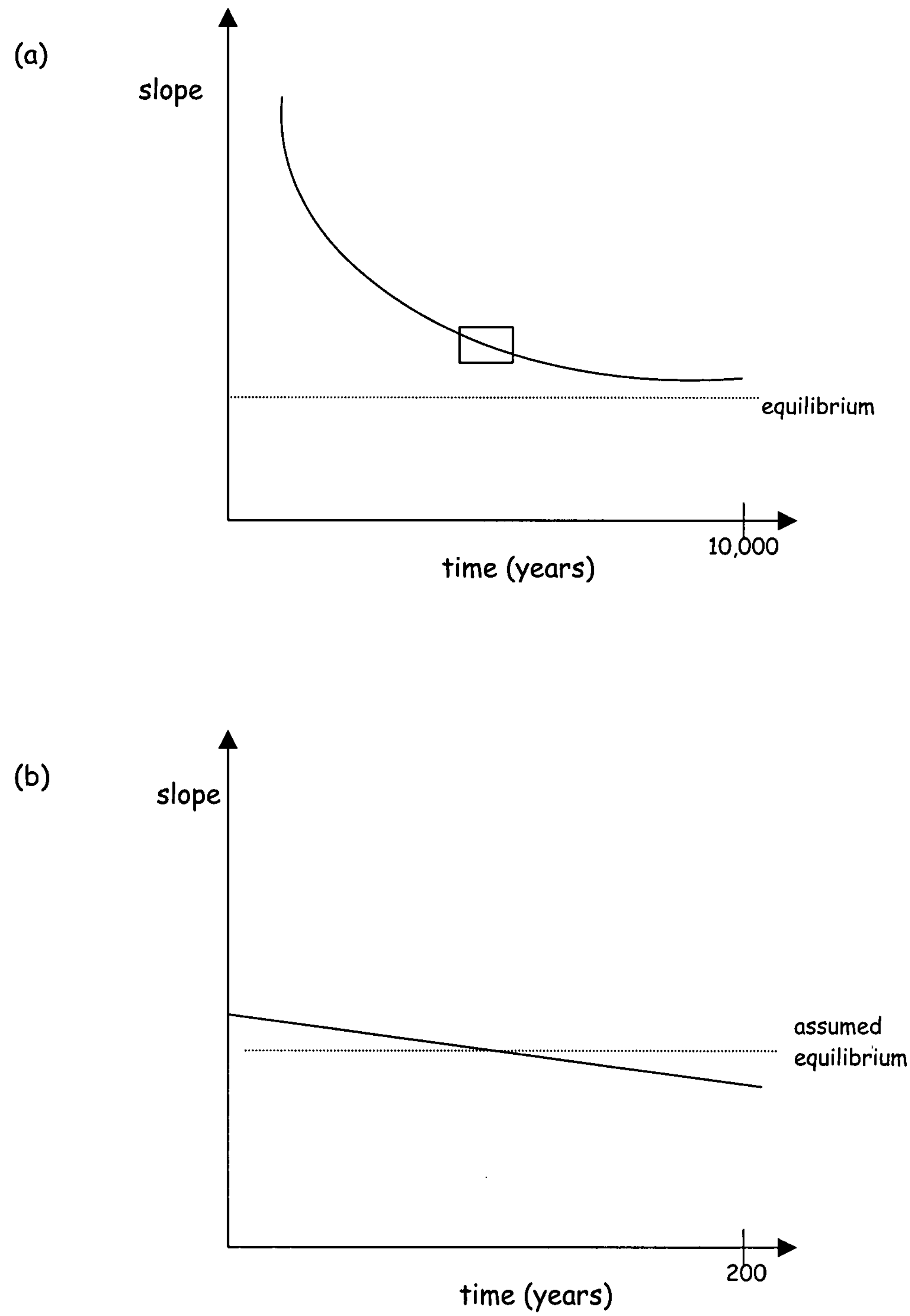


Figure 1.1. Schematic representation of the graded condition in two time scales. (a) The dotted line represents the graded condition the river is trying to adjust to through changes in its slope. (b) This is a close-up of the box shown in (a). The scale is much and over 200 years the river may be said to be in equilibrium (represented by the dotted line).

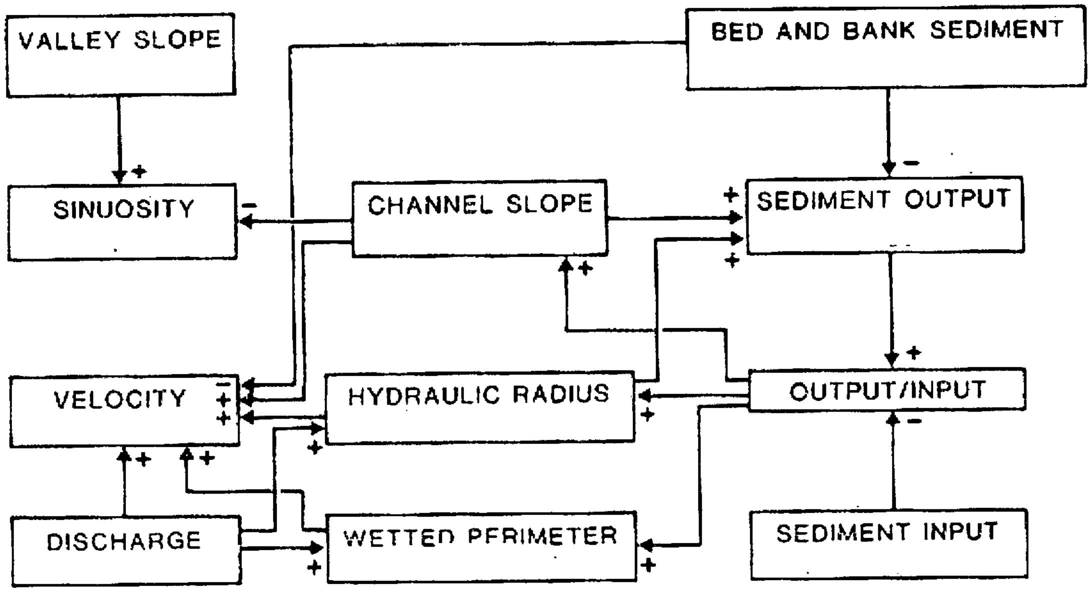


Figure 1.2. Statistical model of a river system illustrating how the dependent variables are influenced by the independent variables. Valley Slope, Bed and Bank Sediment, Discharge and Sediment Input are the independent variables (located in the four corner boxes). A "+" indicates the two variables are directly related. A "-" indicates the two variables are inversely related (Hey, 1976 as cited by Hey, 1982).

(a)

(b)

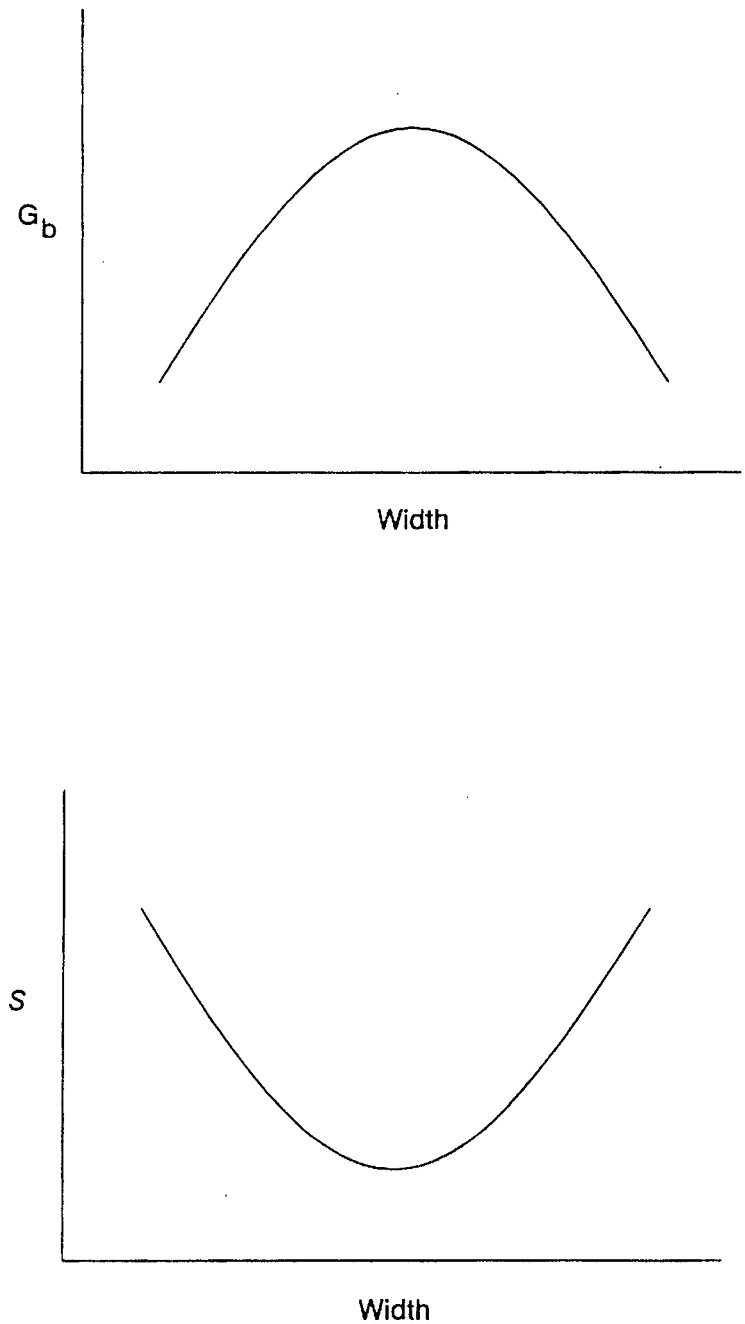


Figure 1.3. A schematic representation of two solution curves showing optimal geometry. (a) Discharge and slope are held constant and an optimum width develops which corresponds to the maximum sediment transporting capacity. (b) Discharge and sediment load are held constant and the optimum occurs at the minimum slope (Millar, 1994).

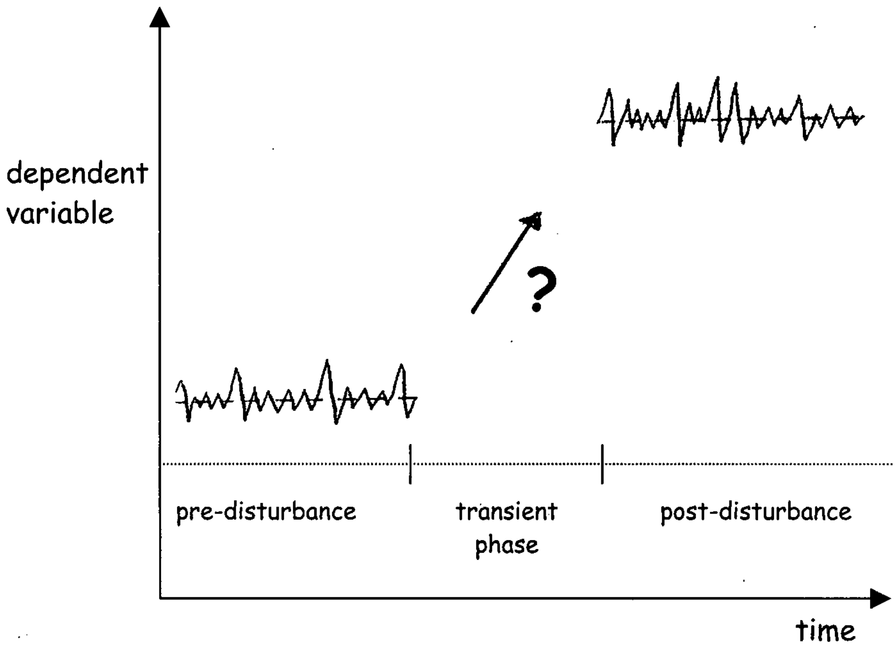


Figure 1.4. Illustration of a dependent variable vs. time. The equilibrium pre-disturbance stage, transient phase and post-disturbance stage are visible. The fluctuations about the equilibrium condition can be seen in both the pre- and post-disturbance stages. It is difficult to predict the precise path of the transient phase.

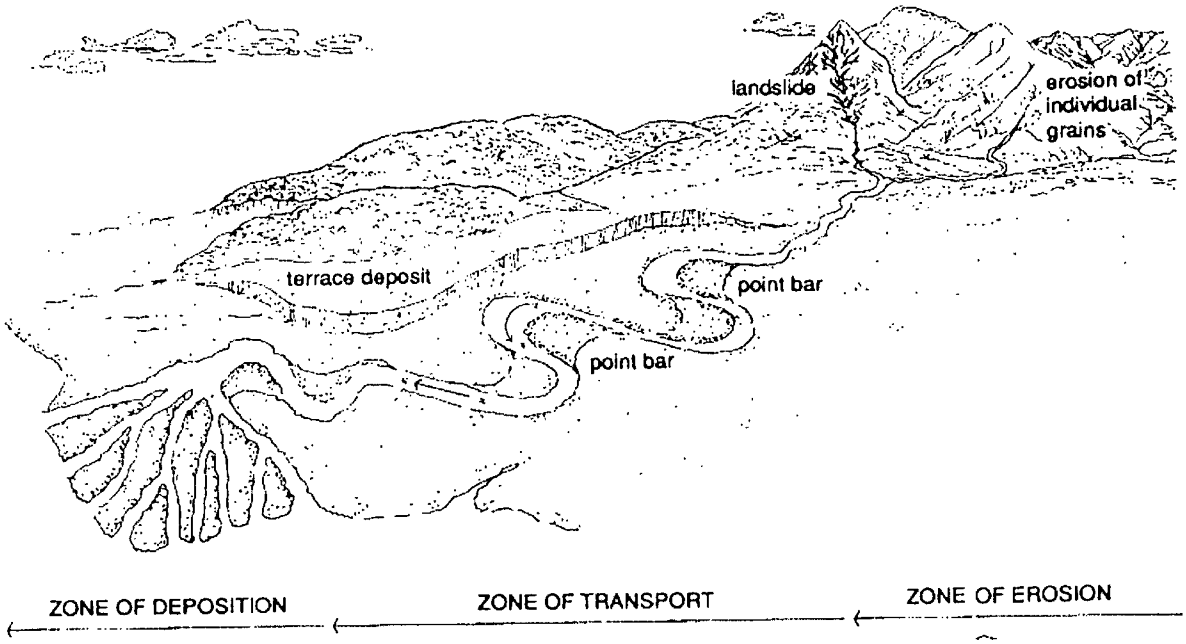


Figure 1.5. Idealized watershed showing three zones of erosion, transport and deposition (Kondolf, 1997).

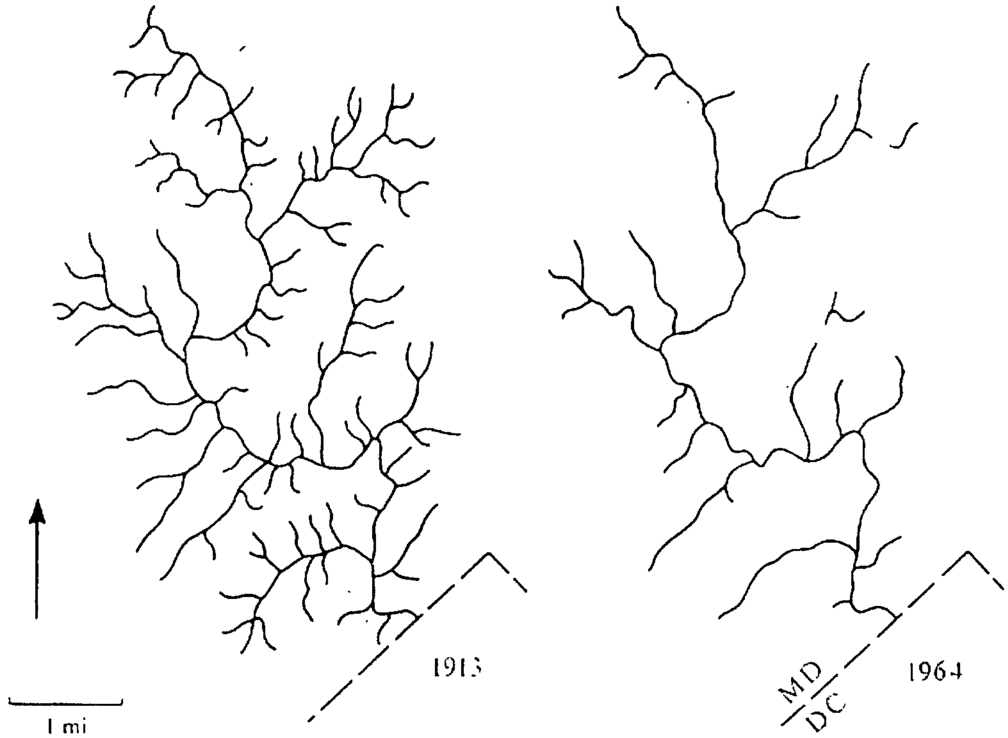


Figure 1.6. Drainage net of Rock Creek in 1913, before modern urbanization and in 1964 after modern urbanization (Dunne and Leopold, 1978).

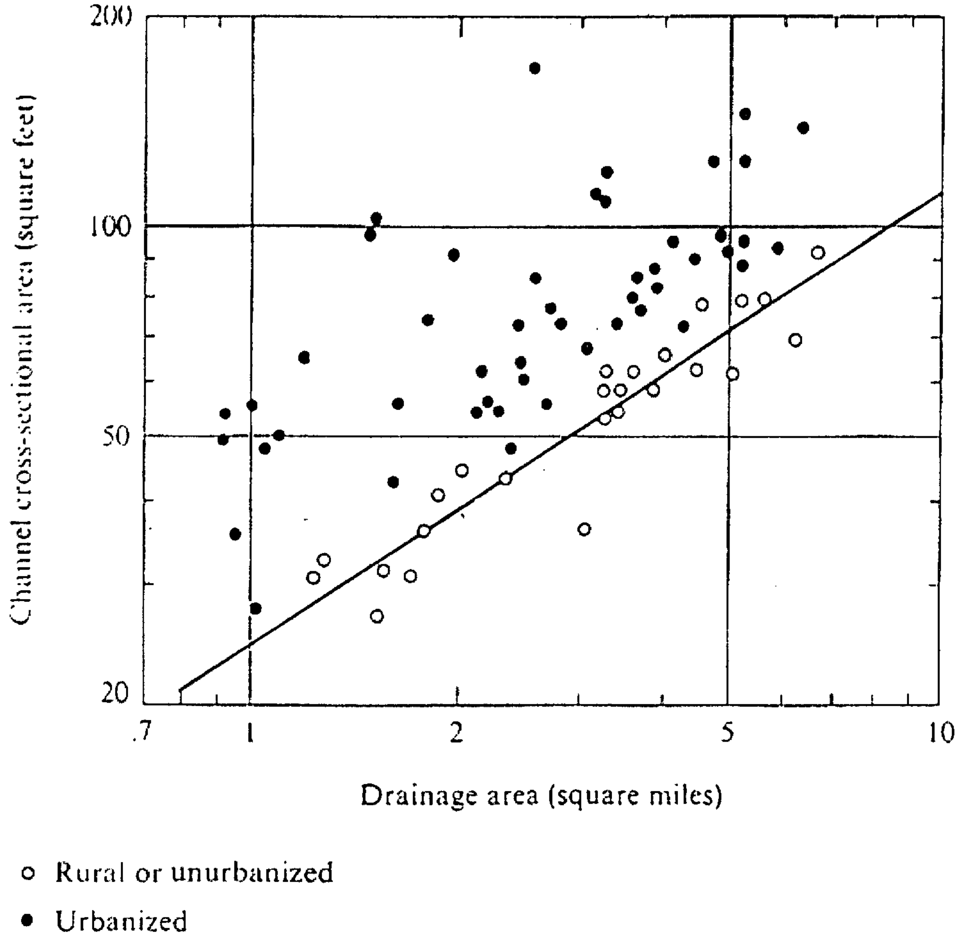


Figure 1.7. Channel cross-sectional area at bankfull vs. drainage area of both rural and urban streams in Pennsylvania (Dunne and Leopold, 1978).

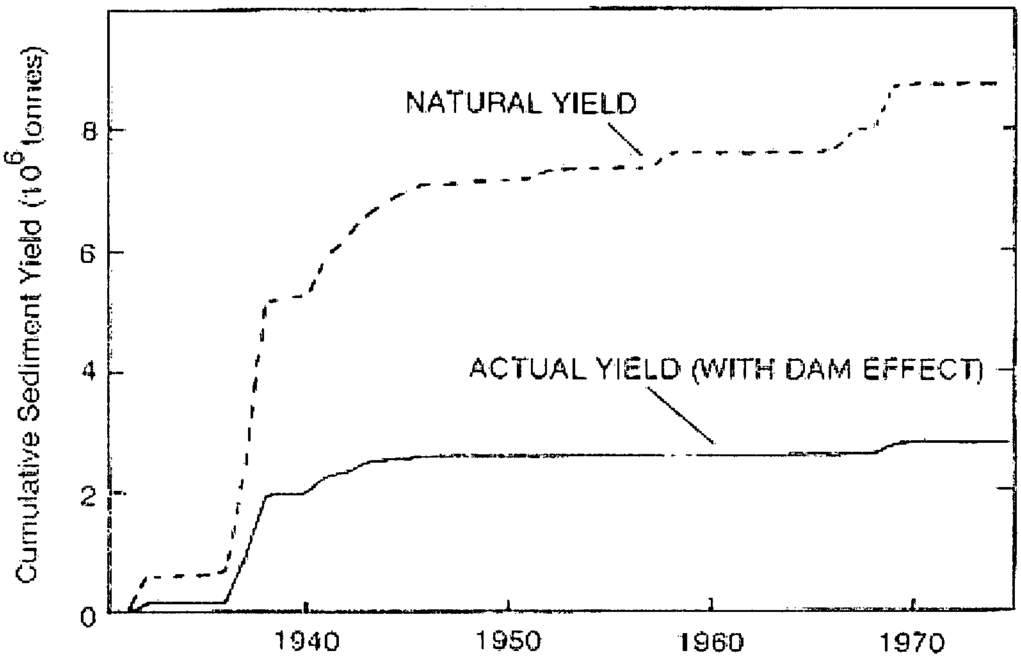


Figure 1.8. Reduction in sediment supply from the catchment of the San Luis Rey River due to the contruction of the Henshaw Dam (Kondolf, 1997).

Chapter 2

MODEL FORMULATION

In this chapter the analytical basis for RiverMod is discussed, and the algorithms used to determine sediment transport, flow resistance, bank stability and planform geometry are formulated.

2.1 THEORETICAL BASIS

The model presented in this thesis is an extension of a previous model developed by Millar and Quick (1993, 1998) and Millar (2000). It models the hydraulic geometry of an alluvial gravel-bed channel. The main contribution of this model, as compared with previous ones, is the fact that it assesses the bank stability of the channel.

The model operates under the assumption that a river will tend toward an equilibrium geometry. The dependent variables adjust according to the independent variables in order to reach an optimum geometry which satisfy the discharge, the bank stability and the bedload constraint. The channel is assumed to have a trapezoidal cross-section (see Figure 2.1).

2.1.1 Extremal Hypothesis

There are seven channel geometry equations mentioned in Millar and Quick (1993b) and eight principal dependent variables. The equations are for flow resistance, continuity, velocity, mean bank shear stress, mean bed shear stress, bank stability and sediment transport. The variables are bed width (W) or perimeter (P), channel depth (Y), channel slope (S), friction factor (f), mean velocity (u), mean bed shear stress (τ_{bed}), mean bank shear stress (τ_{bank}), and bank angle (θ)

As there are more variables than there are equations to solve for the variables, an additional relation is necessary. An "extremal hypothesis" is introduced which states that "the channel geometry will adjust until the sediment transport capacity of the channel is equal to the value supplied from upstream" (Millar and Quick, 1993b). The extremal hypothesis allows an optimum solution to be obtained.

The main function of the model is to solve the seven equations with the extremal hypothesis in order to obtain an optimum channel geometry, performed by a computer routine.

2.1.2 Constraints

There are three constraints that must be satisfied in order to find an optimum hydraulic geometry for a given channel. These are bedload, discharge and bank stability constraints. The bedload constraint ensures that the channel is in equilibrium, and the sediment load is transported through the reach without net deposition or erosion (Millar, 1994). The discharge constraint ensures the discharge capacity of the channel is equivalent to the bankfull discharge (Q_{bf}), given by:

$$UA = Q_{bf} \tag{2.1}$$

where U is average velocity, A is cross-sectional area and Q_{bf} is bankfull discharge. The bank stability constraint ensures that the banks are stable with respect to both mass failure and fluvial erosion.

2.2 SEDIMENT TRANSPORT

Hans Albert Einstein defined a bedload formula as "an equation linking the rate of bedload transportation with the properties of the grain and of the flow causing the movement" (Einstein, 1942). Bedload is fluvial sediment transported along the bed of a channel by rolling, sliding or saltation. It is very difficult to accurately measure the bedload transport rate of gravel-bed rivers, and we are generally forced to rely on predictive relationships.

At present, no universal bedload transport equation exists (Reid *et al.*, 1997). Einstein (1950) sums up one main reason: "sediment movement and river behaviour are inherently complex natural phenomena involving a great many variables". So far, the development of bedload transport equations has relied on empirical and experimental work, mostly carried out in flume studies employing uniform bed materials (Reid *et al.*,

1997) even though most natural gravel-bed rivers tend to have non-uniform bed material size distributions. Another obstacle in the quest for a universal bedload transport formula is the fact that the existing formulae can only be tested with field data (Reid *et al.*, 1997).

RiverMod contains four established, well-known sediment transport equations. Einstein-Brown (1950), Meyer-Peter and Müller (1948), Ackers and White (1973) and Parker, Klingeman and McLean (1982). These particular equations were included in RiverMod because:

1. The parameters required as input variables are usually readily available, or easy to measure;
2. The parameters they require are similar to each other, and therefore it is easy for the user to use more than one equation with available data;
3. The equations are all well-established and well known and have each been included in several comprehensive reviews of bedload transport equations;
4. The user can compare the different answers obtained by using the different sediment transport relationships;
5. The conditions under which a particular equation was derived might be similar to the conditions of the river reach in question.

In the fixed slope model within RiverMod, the bedload transport equation is used to determine the sediment transport capacity (in kg/s or lbs./s) of the stream. In the variable slope model, the sediment transport capacity becomes an input variable and it is used in conjunction with the selected transport equation to determine the slope of the stream.

Gomez and Church (1989) expertly summarize the general assumptions, which are usually followed when applying bedload transport formulae;

(i) that the flow, sediment properties and bedload transport rate are constant for the period in question;

(ii) that the bedload transport rate is a unique function flow and sediment parameters;

(iii) that the maximum amount of bedload is being transported.

2.2.1 Einstein (1942), Einstein-Brown (1950)

Einstein originally developed an empirical bedload equation in 1942, which he replaced in 1950 with an analytical version (Gomez and Church, 1989). Also in 1950, Rouse, Boyer and Laursen modified Einstein's 1950 function and named it after Brown, author of Chapter XII of the book *Engineering Hydraulics* (Rouse (ed), 1950, as cited in ASCE Task Committee, 1971).

Most bedload transport equations are based on the theory that transport is a function of the excess of a flow quantity above the threshold value for initiation of motion of the sediment particles in a channel bed. The Einstein formula is an important exception to this trend. Based on numerous experiments on bedload, Einstein concluded that "a distinct condition for the beginning of transportation does not seem to exist"

(Einstein, 1942). There is a continuous relationship between bedload transport intensity and flow intensity (Reid *et al*, 1997).

The original equation was developed as an empirical relation based on the probability of particle movement (Gomez and Church, 1989).

$$F = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gD^3\rho_f(\rho_s - \rho_f)}} - \sqrt{\frac{36\nu^2}{gD^3\rho_f(\rho_s - \rho_f)}} \tag{2.2}$$

$$g_b{}^* = \frac{1}{F}\frac{q_s}{(\rho_s - \rho_f)g_b{}^*}\sqrt{\frac{\rho_f}{\rho_s - \rho_f}}\frac{1}{g^{1/2}D^{3/2}} \tag{2.3}$$

$$\Psi = \frac{\rho_s - \rho_f}{\rho_f}\frac{D}{SR_h} \tag{2.4}$$

Where F is a parameter for settling velocity, g_b* is the dimensionless transport rate, ψ is the ratio of the forces acting on a particle, ν is kinematic viscosity, g is gravitational acceleration, ρ_f is fluid density, ρ_s is sediment density, D is the sediment grain size, q_s is sediment discharge, S is channel slope and R_h is hydraulic radius.

Note that the dimensionless bed shear stress (τ*) and ψ are inversely proportional to each other. The dimensionless transport rate (g_b*) is calculated using:

$$if\ \tau^* \leq 0 \qquad g_b^* = 0 \tag{2.5}$$

$$if\ 0 < \tau^* < 0.093 \qquad g_b^* = 2.15e^{-0.391/\tau^*} \tag{2.6}$$

$$if\ \tau^* \geq 0.093 \qquad g_b^* = 40(\tau^*)^3 \tag{2.7}$$

The sediment transport capacity (in kg/s) is determined using:

$$G_{b_{calc}} = P_{bed} F S_s \rho g_b^* \sqrt{g(S_s - 1)D_{50}^3} \tag{2.8}$$

2.2.2 Meyer-Peter and Müller (1948)

The Meyer-Peter and Müller formula is an extension of the Meyer-Peter formula derived in 1934 (Gomez and Church, 1989). It is an empirical law of bedload transport based on experimental data. This formula is exclusively for the movement of bedload and suspended load is not considered (Meyer-Peter and Müller, 1948).

$$i_b = \frac{\gamma}{\gamma_s - \gamma}\left[\frac{\left(\frac{Q_B}{Q}\right)\left(\frac{K_B}{K_G}\right)^{3/2}\gamma S - 0.047\left(\frac{(\gamma_s - \gamma)}{\gamma}\right)D}{\left(\frac{0.25}{\gamma}\right)\left(\frac{\gamma}{g}\right)^{1/3}}\right] \tag{2.9}$$

$$K_G = \frac{26}{D_{90}^{1/6}} = \frac{1}{n'} \qquad (2.10)$$

$$K_B = \frac{u}{Y^{2/3} S^{1/2}} = \frac{1}{n} \qquad (2.11)$$

where i_b is specific bedload transport rate, γ is the unit weight of water, γ_s is the unit weight of the sediment, Q_B/Q is a reducing multiplier which accounts for the amount of discharge acting on the bedload, K_B/K_G is a reducing multiplier which accounts for form resistance, Y is the depth of the channel, D_{90} is the ninetieth percentile bed surface grain diameter, u is mean velocity, n is Manning's roughness coefficient and n' is Manning's grain roughness coeffcient.

Meyer-Peter and Müller concluded that the initiation of motion is dependent on a certain magnitude of the shear stress. They also utilize two different grain diameters in their calculations, D_{90} and D_a. This is to account for the difference in composition between the grain composition of the stationary bed and that of the mobile bed.

The description of the procedure used by RiverMod to calculate the sediment transport capacity follows. First, the grain Manning's n (n'), is calculated using:

$$n' = \frac{D_{90}^{1/6}}{26} \qquad (2.12)$$

Next the dimensionless bed shear is determined, applying:

$$\tau^{*\prime} = \left(\frac{n'}{n}\right)^{3/2} \left(\frac{\tau}{\gamma D_a (s_s - 1)}\right) \tag{2.13}$$

Then, the value of τ^* is evaluated, in order to determine the dimensionless bedload (g_b^*) using:

$$if\ \tau^{*\prime} \leq 0.047 \quad g_b^* = 0 \tag{2.14}$$

$$if\ \tau^{*\prime} > 0.047 \quad g_b^* = 8(\tau^{*\prime} - 0.047)^{3/2} \tag{2.15}$$

Equation 2.9 can be reduced to Equation 2.15 by assuming K_B/K_G = 1 for planar beds and Q_B/Q = 1 for wide channels and by substituting values of γ_s = 26000 N/m^3, γ = 9800 N/m^3 and g = 9.8 m/s^2 (Gomez and Church, 1989).

And finally, the sediment transport capacity (in kg/s) can be determined using:

$$G_{b_{calc}} = P_{bed} S_s \rho g_b^* \sqrt{g(S_s - 1)D_a^3} \tag{2.16}$$

2.2.3 Ackers and White (1973)

Ackers and White developed a new framework for the analysis of transport data (Ackers and White, 1973). It is based on both dimensional analysis and physical arguments. They derived three key dimensionless parameters F_{gr} (sediment mobility number), D_{gr} (grain diameter) and G_{gr} (sediment transport) (Ackers and White, 1973). Their formulae can be seen below.

$$i_b = G_{gr}\gamma_s\left(\frac{u}{u_*}\right)^n D_{gr} \qquad (2.17)$$

$$G_{gr} = C\left[\left(\frac{F_{gr}}{A}\right) - 1\right]^m \qquad (2.18)$$

$$D_{gr} = D\left(\frac{g(S_s - 1)}{\nu^2}\right)^{1/3} \qquad (2.19)$$

$$F_{gr} = \frac{u_*^n}{\left[gD(S_s - 1)\right]^{1/2}}\left(\frac{u}{\sqrt{32}\log(10Y/D)}\right)^{1-n} \qquad (2.20)$$

(Gomez and Church, 1989). Where u* is shear velocity, and A, C, m and n are all dimensionless functions of the grain diameter. For coarse sediments (D_{gr} > 60), where only bedload is being transported, the four parameters are all constant:

n = 0; A = 0.170; m = 1.50; C = 0.025

For mixed transport ($1 \leq D_{gr} \leq 60$), parameters are calculated using:

$$n = 1 - 0.56\log D_{gr} \qquad (2.21)$$

$$A = \left(0.23/D_{gr}^{1/2}\right) + 0.14 \qquad (2.22)$$

$$m = \left(9.66/D_{gr}\right) + 1.34 \qquad (2.23)$$

$$C = 10^{\left(2.86\log D_{gr} - (\log D_{gr})^2 - 3.53\right)} \qquad (2.24)$$

During field analysis, Ackers and White concluded that the D_{35} bedload value was the "most appropriate when making predictions of total sediment load" (Ackers and White, 1973). The Ackers and White method for predicting bedload transport is useful for grain sizes larger than 0.04 mm. It seems to be fairly accurate when predicting initiation of motion, although the authors caution against its use in unsteady flow conditions.

The description of the procedure used by RiverMod to calculate the sediment transport capacity follows. The dimensionless grain diameter (D_{gr}) is first calculated, using Equation 2.19. Then the sediment mobility number (F_{gr}) A, C, m and n are calculated using Equations 2.20-2.24. Next the sediment flux (X) is calculated using:

$$X = \frac{S_s D_{35} G_{gr}}{y\left(\frac{U*}{U}\right)^n} \qquad (2.25)$$

G_{gr} is determined using Equation 2.18 and U*, the shear velocity and calculated using:

$$U* = \sqrt{gR_h S} \qquad (2.26)$$

And finally, the sediment transport capacity (in kg/s) can be determined using:

$$G_{b_{calc}} = XQ \qquad (2.27)$$

2.2.4 Parker, Klingeman and McLean (1982)

The Parker, Klingeman and McLean formula takes the form:

$$i_b = 1400\left(\frac{\omega^* g^{1/2} Y S^{2/3}}{\gamma_s}\right) \quad (2.28)$$

$$if\ \phi_{50} < 0.95 \qquad \omega^* = 0 \quad (2.29)$$

$$if\ 0.95 \le \phi_{50} \le 1.65 \quad \omega^* = 0.0025\exp\left[14.2(\phi_{50} - 1) - 9.28(\phi_{50} - 1)^2\right] \quad (2.30)$$

$$if\ \phi_{50} > 1.65 \qquad \omega^* = 11.2\left(1 - \frac{0.822}{\phi_{50}}\right)^{9/2} \quad (2.31)$$

$$\tau_{d_{50}} = \frac{\tau}{\gamma d_{50}(s_s - 1)} \quad (2.32)$$

$$\phi_{50} = \frac{\tau_{d_{50}}}{0.0876} \quad (2.33)$$

(Gomez and Church, 1989). Where ϕ_{50} is a shear stress ratio, ω^* is the dimensionless bedload transport rate, τ_{d50} is the bed shear stress for the median sub-surface grain size, τ is shear stress, d_{50} is the median sub-surface grain diameter and s_s is the specific gravity of the sediment.

This formula was the first to account for an armour layer and its effect on bedload transport rates, which is present in almost all gravel rivers (see Figure 2.2).

Bakke *et al.* (1999) define the layer termed "armour" or "pavement" as the layer extending to "the bottom of the largest surface particle". The "subarmour" or "subpavement" was identified as the layer just below the pavement of equal thickness.

Based solely on field work, the empirical relations for the dimensionless bedload transport (ω^*) were derived from bedload measurements from Oak Creek (Parker *et al.*, 1982), a small, steep gravel-bed stream in Oregon. Parker *et al.* (1982) theorized that the armour layer varies with shear stress and sediment load and that for conditions exceeding the critical stress of the pavement, the bedload grain size distribution of the pavement is approximately the same as that of the subpavement, so that:

$$D_{50} \sim d_{50}$$

They developed the "equal mobility hypothesis", which states that surface coarsening develops to render all size fractions equally mobile (Parker *et al.*, 1982) once the critical condition for movement was exceeded (Gomez and Church, 1989). If an equilibrium condition develops, a balance has been achieved between the bed material supply and the transporting capacity of the reach (Bakke *et al.*, 1999).

The description of the procedure used by RiverMod to calculate the sediment transport capacity follows. First, the shield's stress for median diameter of the subpavement (τ_{d50}) is calculated using Equation 2.32. Next the shear stress ratio (ϕ_{50}) is determined, using Equation 2.33. Then, Equations 2.29 - 2.31 are used to compute the dimensionless bedload transport (ω^*). And finally, the sediment transport capacity (in kg/s) can be determined using:

$$G_{b_{calc}} = \omega * \tau_{d_{50}} P_{bed} S_s \rho \sqrt{g(S_s - 1) d_{50}^3} \qquad (2.34)$$

2.2.5 Discussion of Sediment Transport Equations

Table 2.1 summarizes the sediment transport equations discussed in the preceding sections.

Table 2.1. Bedload transport formulae used by RiverMod (adapted from Gomez and Church, 1989)

Formula	Particle Size Range (mm) employed in derivation	Comments
Meyer-Peter and Müller (1948)	0.40 - 28.65: uniform sediments, mixtures and light-weight materials	"should be used for pure bedload transport. Solid material rolling or jumping along the bed of a river" (Meyer-Peter and Müller, 1948)
Parker, Klingeman and McLean (1982)	0.60 - 102.0: natural mixture (field data)	"restricted to small-to-medium-sized paved gravel bed streams with steep or moderate slopes, which are not dominated by sand load" (Parker et al., 1982)
Ackers and White (1973)	0.04 - 4.94: uniform sediments and light-weight materials.	dimensional analysis and physical arguments were used to develop this general function (Ackers and White, 1973)
Einstein (1942, 1950)	0.785-28.65: uniform sediments and light materials	restricted to steady, uniform flow, plane bed conditions (Gomez and Church, 1989)

In 1989 Gomez and Church conducted an extensive review of 12 well-known sediment transport formulae for gravel-bed rivers. The review tested each equation with four sets of river data and three sets of flume data with maximum consistency in all tests conducted. The survey illustrated the fact that very different results can be obtained, depending on which sediment transport equation is used.

Figure 2.3 shows a comparison between observed and calculated data using the data from the Elbow River, a gravel-bed channel. It also demonstrates that any sediment transport equation used will perform better if the conditions of the reach in question are similar to the conditions under which the particular formula was derived. These predictive relationships should be used in conjunction with another form of analysis, including, but not limited to, laboratory experiments, the use of air photos and field work.

2.3 FLOW RESISTANCE

Many equations have been proposed to relate average velocity in open channels to flow resistance (Hey, 1979). RiverMod uses four flow resistance relationships to establish the average velocity of the flow through and iterative process, ultimately determining the width and slope of the channel. The four relationships are Manning (1891), Composite Manning (Horton, 1933), Keulegan (1938), and Jarrett (1984).

Because each channel has its own characteristic bedforms, profiles, flow resistance and sediment transport, it is important to use a resistance equation which has been designed for use with a similar channel to the study channel (Bathurst, 1978).

In order to obtain the velocity and ultimately the channel dimensions, the bankfull discharge, channel slope. grain size distribution and roughness coefficient (except for Jarrett (1984) Equations) are needed (Bathurst, 1997).

2.3.1 Manning (1891)

The Manning formula, which was presented in 1891, is the most widely used of all uniform-flow formulae for open-channel flow computations (Chow, 1959). It is known in many countries under several combinations of the names Gauckler, Hagen, Manning and Strickler (Williams, 1970). It can be written as:

$$U = \frac{1}{n} R_h^{2/3} S^{1/2} \tag{2.35}$$

where n is Manning's roughness coefficient.

In order to use the Manning formula, one must determine an adequate value for Manning's roughness coefficient, n, because its value cannot be directly measured. This value is normally estimated either from experience, by using an empirical relationship or from documented cases (Bathurst, 1982). Table 2.2 supplies average values of Manning's n for numerous types of natural streams.

Table 2.2. Values of Manning's roughness coefficient n for natural streams (adapted from Chow, 1959)

Type of Channel and Description			n
I Top width < 30 m	(a) Streams on a plain	(i) clean, straight, full stage, no riffles or pools	0.020
		(ii) same as above, but more stones and weeds	0.024
		(iii) clean, winding, some pools and shoals	0.027
		(iv) same as above, but some weeds and stones	0.030
		(v) same as above, lower stages, more ineffective slopes and sections	0.032
		(vi) same as (iv) but more stones	0.034
		(vii) sluggish reaches, weedy, deep pools	0.047

		(viii) very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.067
	(b) Mountain streams, no vegetation on banks, usually steep	(i) bottom: gravel, cobbles, and a few boulders	0.027
		(ii) bottom: cobbles with large boulders	0.034
II Flood plains	(a) Pasture, no brush	(i) short grass	0.020
		(ii) high grass	0.024
	(b) Cultivated areas	(i) no crop	0.020
		(ii) mature row crop	0.024
		(iii) mature field crop	0.027
	(c) Brush	(i) scattered brush, heavy weeds	0.034
		(ii) light brush and trees, in winter	0.034
		(iii) light brush and trees, in summer	0.040
		(iv) medium to dense brush, in winter	0.047
		(v) medium to dense brush, in summer	0.067
	(d) Trees	(i) dense willows, summer, straight	0.101
		(ii) cleared land with tree stumps, no sprouts	0.027
		(iii) same as above, but with heavy growth of sprouts	0.040
		(iv) heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.067
		(v) same as above, but with flood stage reaching branches	0.081
III Top width > 30 m	(a) Regular section with no boulders or brush		.....
	(b) Irregular rough section		.....

In order to use documented cases as reliable sources of values for Manning's n, one must first consider the factors affecting it, such as channel surface roughness, vegetation, irregularities, alignment, silting and scouring, and obstructions (Chow, 1959).

The user inputs a value for Manning's roughness coefficient (n) and the model calculates the velocity using Equation 2.35

2.3.2 Composite Manning (Horton, 1933)

Originally developed by Horton (1933), this equation is useful when determining the dimensions or velocity of a channel whose bed and banks display differing degrees of roughness (Horton, 1933).

An equivalent roughness is derived using:

$$n = \sqrt{\frac{\left(P_{bed} n_{bed}^2 + P_{bank} n_{bank}^2\right)}{\left(P_{bed} + P_{bank}\right)}} \qquad (2.36)$$

where n_{bank} and n_{bed} are Manning's roughness coefficients for the bank and bed respectively, P_{bank} is the wetted perimeter of the bank and P_{bed} is the bed perimeter.

Horton (1933) assumed the velocity was constant over the area and found that the values of both n_{bed} and n_{bank} vary with depth. Figure 2.4 shows the variation in equivalent n with depth for rectangular and trapezoidal channels.

The user inputs a value of n for the banks and one for the bed and the model calculates a composite value for n using Equation 2.36 and velocity using Equation 2.35.

2.3.3 Keulegan (1938)

Keulegan (1938) applied principles already established for use in circular pipes to turbulent flow in open channels and derived a formula for velocity distribution and hydraulic resistance.

The Keulegan flow resistance equation, together with the Darcy-Weisbach equation, is one of the most well-known and widely-used equations for open channels. The two equations are given below:

$$f = 2.03\log\left(\frac{12.2R_h}{k_s}\right) \tag{2.37}$$

$$U = f\sqrt{8gR_hS} \tag{2.38}$$

where f is the friction factor and k_s is the equivalent roughness height. Although k_s is actually a measure of the effect of any roughness element on the flow (Bathurst, 1982).

Originally, the value of k_s was determined experimentally for each channel type. Others have since set it equal to a certain grain-size diameter, commonly D_{50}, D_{65} and D_{90} (Bray, 1982). Most studies of k_s find that it is dependent on the grain-size distribution of the sediment. For non-uniform conditions, k_s is usually expressed as:

$$k_s = C_xD_x \tag{2.39}$$

where C_x is a constant, D_x is the characteristic grain diameter of which x percentage of sediment is finer (Millar, 1999). As shown in Table 2.3, below, values of k_s can vary greatly.

Table 2.3. Summary of "best-fit" k_s values derived from reach-averaged hydraulic geometry (Millar, 1999)

Source	k_s
Bray (1980, 1982)	$6.8D_{50}$ $3.5D_{84}$
Charlton et al.. (1978)	$3.5D_{90}$
Griffiths (1981)	$5.0D_{50}$
Hey (1979)	$3.5D_{84}$
Leopold et al. (1964)	$3.9D_{84}$
Limerinos (1970)	$3.2D_{84}$
Millar (1999)	$2.9D_{84}$

In spite of the great uncertainty associated with these values, according to Millar (1999) they "still represent the best approach to estimating velocity in ungauged gravel-bed rivers."

The user inputs the value of k_s, which is the equivalent roughness height for the reach. The model, then calculates f using Equation 2.37 and uses this value to determine the velocity using Equation 2.38.

2.3.4 Jarrett (1984)

The Jarrett equation was developed to predict Manning's roughness coefficient, n, for high-gradient streams, with a slope greater than 0.002 (Jarrett, 1984). It is as follows:

$$n = 0.39 R_h^{-0.16} S^{0.38} \quad (2.40)$$

Jarrett (1984) found that for uniform flow, Equation 2.40 could be substituted directly into Manning's velocity equation (Equations 2.35).

Jarrett (1984) found that most guidelines available for high-gradient streams do not take into account how the roughness elements change, with a change in gradient and therefore depth. As part of Jarrett's study, 21 natural streams were used, which represented a wide range of channel type, width, depth, slope, roughness and bed-material size. Figure 2.5 shows the relationship between Manning's roughness coefficient and hydraulic radius, from which Jarrett deduced that the roughness decreases with flow depth (Jarrett, 1984).

Equation 2.40 should be used for natural streams with:

1. stable bed and banks;
2. slopes from 0.002 - 0.04;
3. hydraulic radii which do not include wetted perimeter of bed particles, and;
4. little or no suspended sediment (Jarrett, 1984).

The model calculates a value of n using Equation 2.40 and then input into Equation 2.35.

2.4 BANK STABILITY

There are two types of erosion affecting the banks of gravel-bed rivers: mass failure and fluvial erosion. The precise way in which the banks and consequently the river geometry are affected depends on the type of erosion as well as the sediment composition of the banks. This thesis deals with either strictly non-cohesive or cohesive bank sediment.

A soil is considered cohesive if its properties are influenced primarily by clay and silt size particles, where cohesive forces exist between particles. Sediment whose properties are influenced by sand and gravel size particles are referred to as cohesionless, or non-cohesive (Craig, 1992).

2.4.1 Non-Cohesive Bank Sediment

Mass failure of non-cohesive bank sediment occurs when the bank angle (θ) exceeds the friction angle (ϕ), which is not subject to fluid shear stresses. The factor of safety is given by:

$$F_s = \frac{\tan\phi}{\tan\theta} \tag{2.41}$$

where τ_{bank} is the mean bank shear stress, γ is the unit weight of water, S_s is the specific gravity of the sediment, and D_{50bank} is median bank grain diameter.

Loosening of individual clasts through weathering can reduce the friction angle (Lawler *et al.*, 1997). Oversteepening of the banks due to erosion of the bank toe can increase the bank angle. Both of these processes may cause mass failure in banks composed of non-cohesive sediment. Assessing the stability of undrained and drained banks is similar with the addition that failure may result from an increase in pore water pressure under submerged conditions (Thorne, 1982).

The flow of water and sediment in the main river channel produces shear stresses on the bed and banks. If this stress, which is proportional to the velocity gradient, is increased sufficiently, fluvial entrainment of the sediment will occur (Thorne, 1982). In order for a particular grain to remain stable, its frictional forces must resist gravity and the forces exerted by the fluid. The limiting bank stability is:

$$\frac{\tau_{bank_c}}{\tau_{bed_c}} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \tag{2.42}$$

where τ_{bankc} and τ_{bedc} are the critical shear stresses acting on a grain located on the bank and bed respectively (Chow, 1959; Henderson, 1966; as cited by Millar, 1994). In the original relation, ϕ can only achieve a maximum of 40°, but if the effects of bank vegetation and consolidation are included, ϕ can be replaced with an *in situ* value (ϕ') which can reach a maximum of 90° (Millar, 1994). The final relation is:

$$\frac{\tau_{bank}}{\gamma(S-1)D_{50_{bank}}} \le k \tan\phi' \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi'}} \tag{2.43}$$

This equation is used in order to determine whether the banks of a stream composed of non-cohesive bank sediment are stable with respect to both mass failure and fluvial erosion which accounts for the effects of bank vegetation and consolidation of the bank sediment (Millar, 1994).

2.4.2 Cohesive Bank Sediment

In order to assess the stability of a channel bank with respect to mass failure, the motivating and resisting forces acting on the bank must be balanced. The motivating force is due to the weight of the soil and the resisting force is composed of the cohesion (c) and internal friction angle (ϕ) of the soil (Millar, 1994). Mass failure occurs when the critical bank height is reached (Lawler *et al.*, 1997). The factor of safety with respect to mass failure (FS_H) must be greater than or equal to one. It is calculated using:

$$FS_H = \frac{H_{crit}}{H} = \frac{N_s c}{H \gamma_t} \tag{2.44}$$

where H is the vertical bank height; H_{crit} is the maximum height at which the banks are stable; γ_t is the saturated unit weight of soil; and N_s is a dimensionless stability number, given by:

$$N_s = 3.83 + 0.052\,(90 - \theta) - 0.0001\,(90 - \theta)^2 \tag{2.45}$$

where θ is the bank angle (Millar and Quick, 1998).

There are several ways mass wasting can occur. The mode of bank failure depends on the bank angle, material properties such as grain size and soil cohesion and the location of the groundwater table or channel water level.

Fluvial erosion of cohesive banks usually signifies parcels of soil, not individual grains are being eroded. This is because of the strong cohesive forces that exist between the grains (Lawler *et al.*, 1997). Erodibility is a function of many factors, including mineralogy, particle size, temperature of the water and moisture content of the soil. For example, hard, dry banks are more resistant than wet banks, which are more easily eroded (Thorne, 1982).

In order for the channel banks to be considered stable, the factor of safety with respect to fluvial erosion (FS_τ) must be greater than or equal to one. It is calculated using:

$$FS_\tau = \frac{\tau_{crit}}{\tau_{bank}} \tag{2.46}$$

where τ_{bank} is the mean bank shear stress and τ_{crit} is the critical bank shear stress (Millar and Quick, 1998). τ_{bank} can be calculated using:

$$\frac{\tau_{bank}}{\gamma YS} = SF_{bank}\left[\frac{(W + P_{bed})\sin\theta}{4Y}\right] \tag{2.47}$$

where :

$$SF_{bank} = 1.766\left(\frac{P_{bed}}{P_{bank}} + 1.5\right)^{-1.4026} \tag{2.48}$$

2.4.3 Effect of Vegetation on Bank Stability

There have been many studies conducted to determine the influence of riparian vegetation on the bank stability of gravel-bed rivers (Hey and Thorne, 1986; Millar and Quick, 1993; Beeson and Doyle, 1995; Millar, 2000).

Beeson and Doyle's (1995) study of four streams in southern British Columbia compared bank erosion in vegetated and non-vegetated channel bends and found that riparian vegetation decreases bank erosion. They also concluded that the denser the vegetation, the more effective it was at reducing bank erosion and therefore increasing bank stability.

The theory presented by Millar and Quick (1993b) for channels with non-cohesive banks, was tested using published data values as input, and the output was compared to the observed channel geometry. The modified friction angle (ϕ') was used to quantify the influence of bank vegetation and 40° is a reasonable value for non-vegetated banks (Millar, 2000). Millar and Quick (1998) tested the theory for channels with cohesive banks and concluded that bank vegetation has an effect on the critical shear stress (τ_{crit}), which has a large influence on the stable channel width.

RiverMod incorporates a measure of bank vegetation by utilizing the internal bank friction angle (ϕ') as an indicator of bank vegetation strength (Millar and Quick, 1993). It increases with increased density of bank vegetation (Millar, 2000).

2.5 PLANFORM GEOMETRY

Rivers are often classified based on their planform geometry. Braided channels are those with relatively stable alluvial islands and therefore two or more separate channels (Leopold and Wolman, 1957). A meandering river has a single winding and sinuous channel. The planform of a river is dependant on the bed slope and discharge. For a given discharge, meanders will occur on smaller slopes than braids (Leopold and Wolman, 1957).

River slopes can adjust through the process of meandering. As a river creates meanders and the sinuosity of those meanders increases, the channel slope decreases. Figure 2.6 shows three streams with increasing sinuosity from straight to meandering. The valley slope is built-up over time through aggradation. The channel slope adjusts to a steady-state condition.

Figure 2.7 is a flowchart showing the computations in the planform model. RiverMod determines the planform geometry of the river by comparing the bedslope to the valley slope and the transitional slope. If the bedslope is greater than the valley slope, the channel is unstable and is said to be aggrading. If the bedslope is less than the valley slope, the channel is stable and is either braided or meandering. The transition slope is determined using:

$$S^* = \frac{Y}{W} F_r \tag{2.49}$$

where W is the river width and F_r is the Froude number (Parker, 1976, as cited by Millar, 2000). The transition slope is compared to the bed slope to determine if the river is braided or meandering:

$$S > S^* = \text{braided} \tag{2.50a}$$

$$S < S^* = \text{meandering} \tag{2.50b}$$

If the river is meandering, the sinuosity (ξ) and meander wavelength (λ) are determined using empirical relations developed by Leopold and Wolman (1960). These empirical relations ordinarily use a meander wavelength multiplier between 10 and 12, the model uses a default value of 11.

The model also utilizes Langbein and Leopold's (1966) sine-generated curve to calculate values for the maximum deviation angle along the channel (Φ) and the radius of curvature (r_c). Langbein and Leopold (1966, as cited by Thorne, 1997), found that the sine-generated curve resembled an idealized river, by approximating the path of least resistance in flowing around a bend. The sine-generated curve is defined by:

$$\theta(\ell) = \Phi \sin\left(\frac{2\pi\ell}{L}\right) \tag{2.51}$$

where θ is the channel deviation angle, Φ is the maximum value of θ, ℓ is the distance along the valley axis and L is the distance along the meander arc length (see Figure 2.8).

An iterative technique is used to solve the sine-generated curve for Φ, such that the river sinuosity (ξ_c) determined using θ and Φ is equal to the sinuosity (ξ) determined from the bedslope and valley slope. Ultimately the maximum deviation angle is calculated and presented as output (See Figure 2.7). Equation 2.51 can be used to graph the idealized planform geometry of the river.

The idealized meander parameters are useful for river restoration, or attempting to return a channel to its natural form. The planform geometry analysis may assist the redesign of sections of channel that have been straightened or channelized (Millar and MacVicar, 1997).

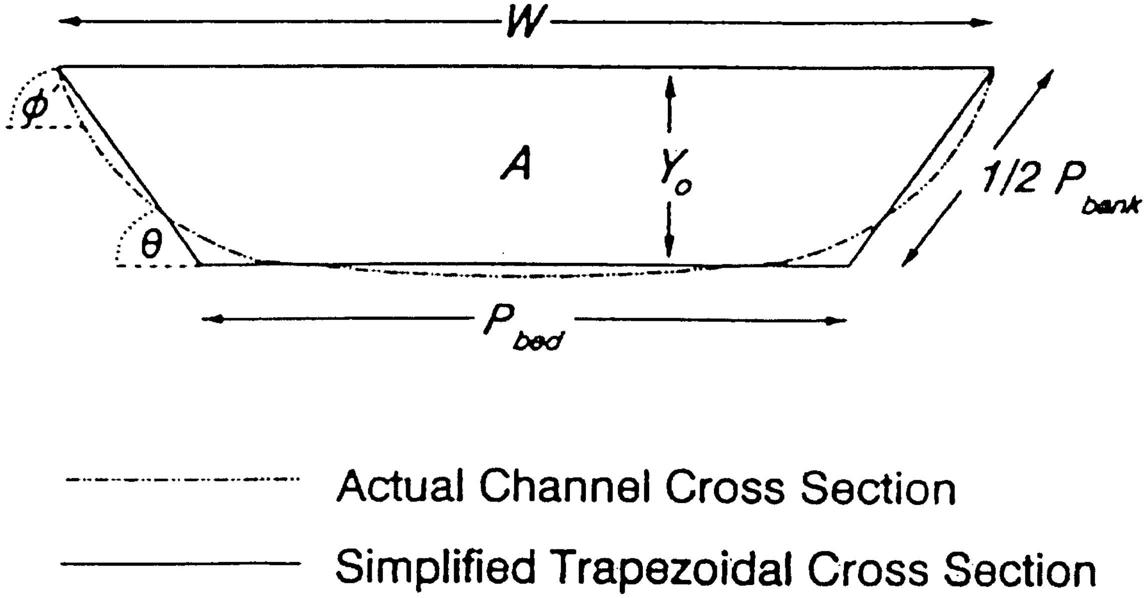


Figure 2.1. Definition Sketch of a simplified trapezoidal channel for Millar and Quick (1993) model (MacVicar 1999).

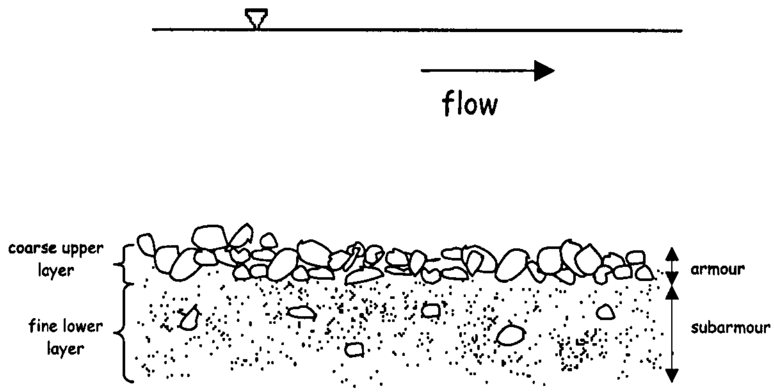


Figure 2.2. A Schematic representation of the armour layer in a gravel-bed channel.

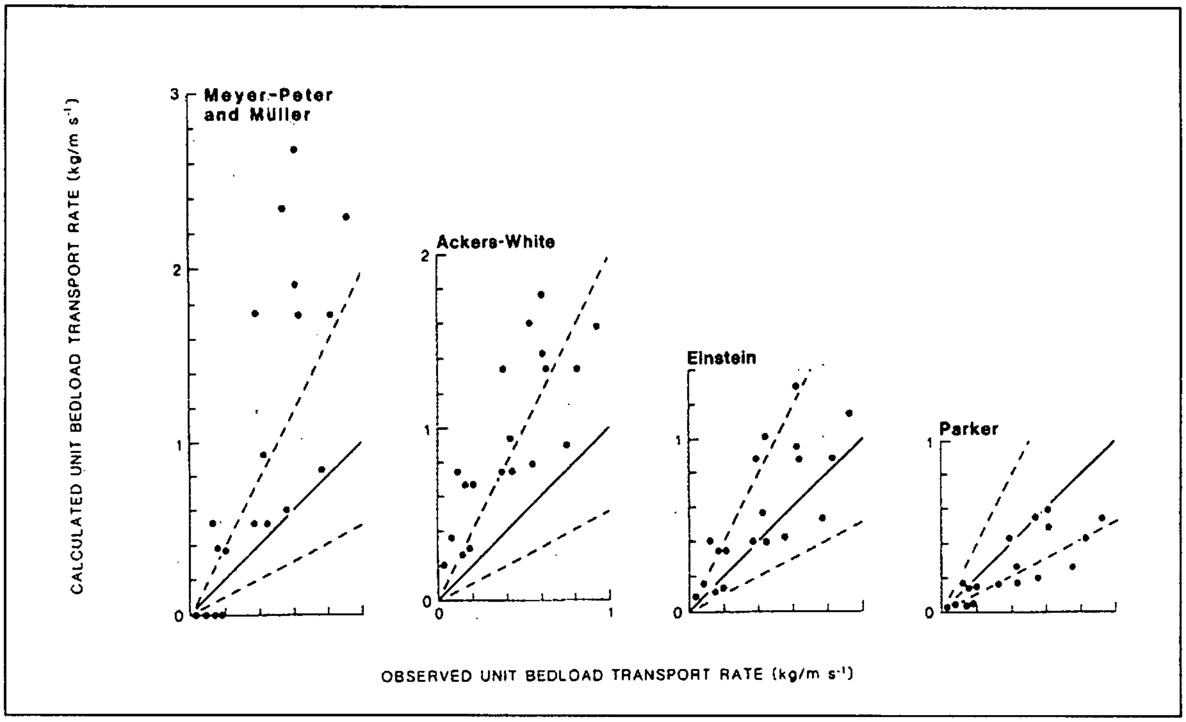


Figure 2.3. Comparison between observed and calculated data for Meyer-Peter and Müller, Ackers and White, Einstein, and Parker, Klingeman and McLean formulae and Elbow River data (Gomez and Church, 1989).

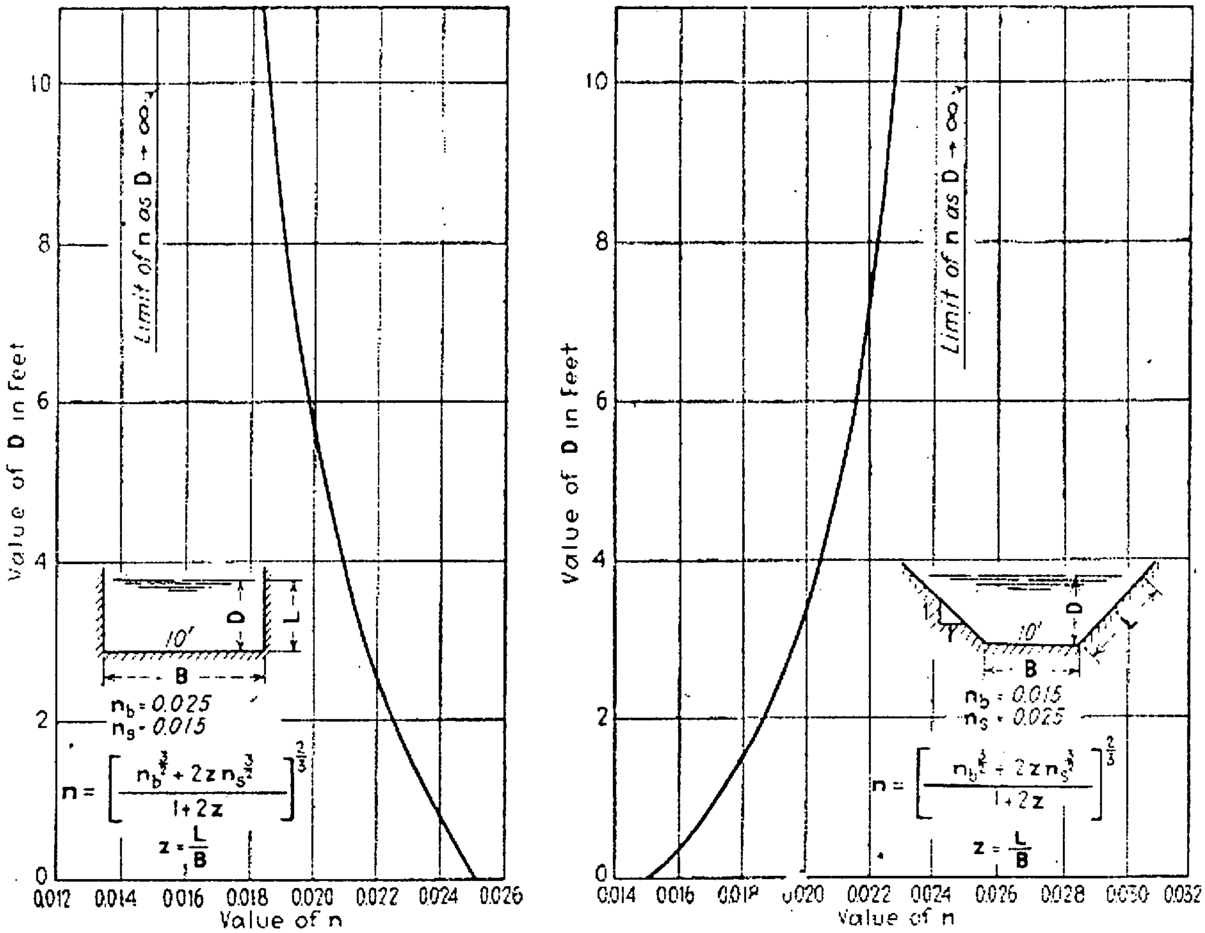


Figure 2.4. Relation between grain-size diameter (D) in feet and Manning's roughness coefficient (n) in imperial values for rectangular and trapezoidal channels in which the bottom and sides have different roughness characteristics (Horton, 1933).

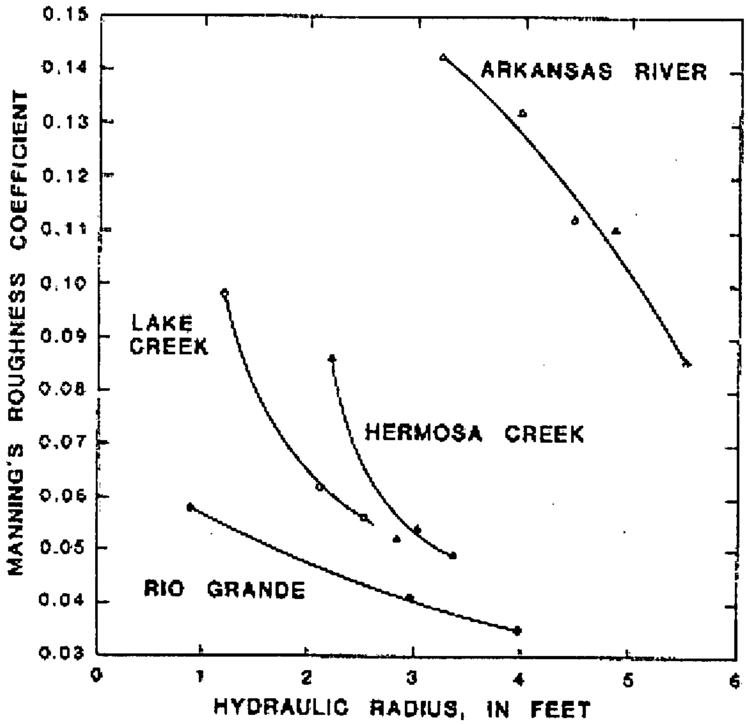


Figure 2.5. Relation of Manning's Roughness Coefficient to Hydraulic Radius for four of the streams Jarrett used in his study (Jarrett, 1984).

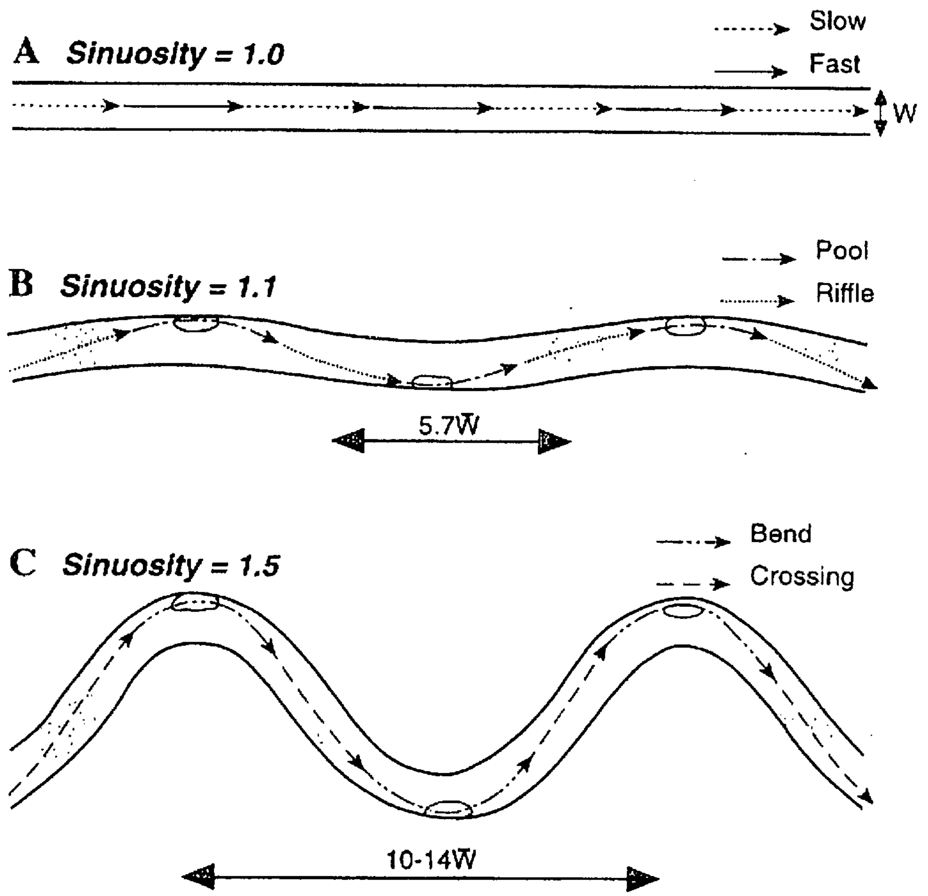


Figure 2.6. Rivers of increasing sinuosity from a straight channel (A) to a meandering channel (C) (Thorne, 1997).

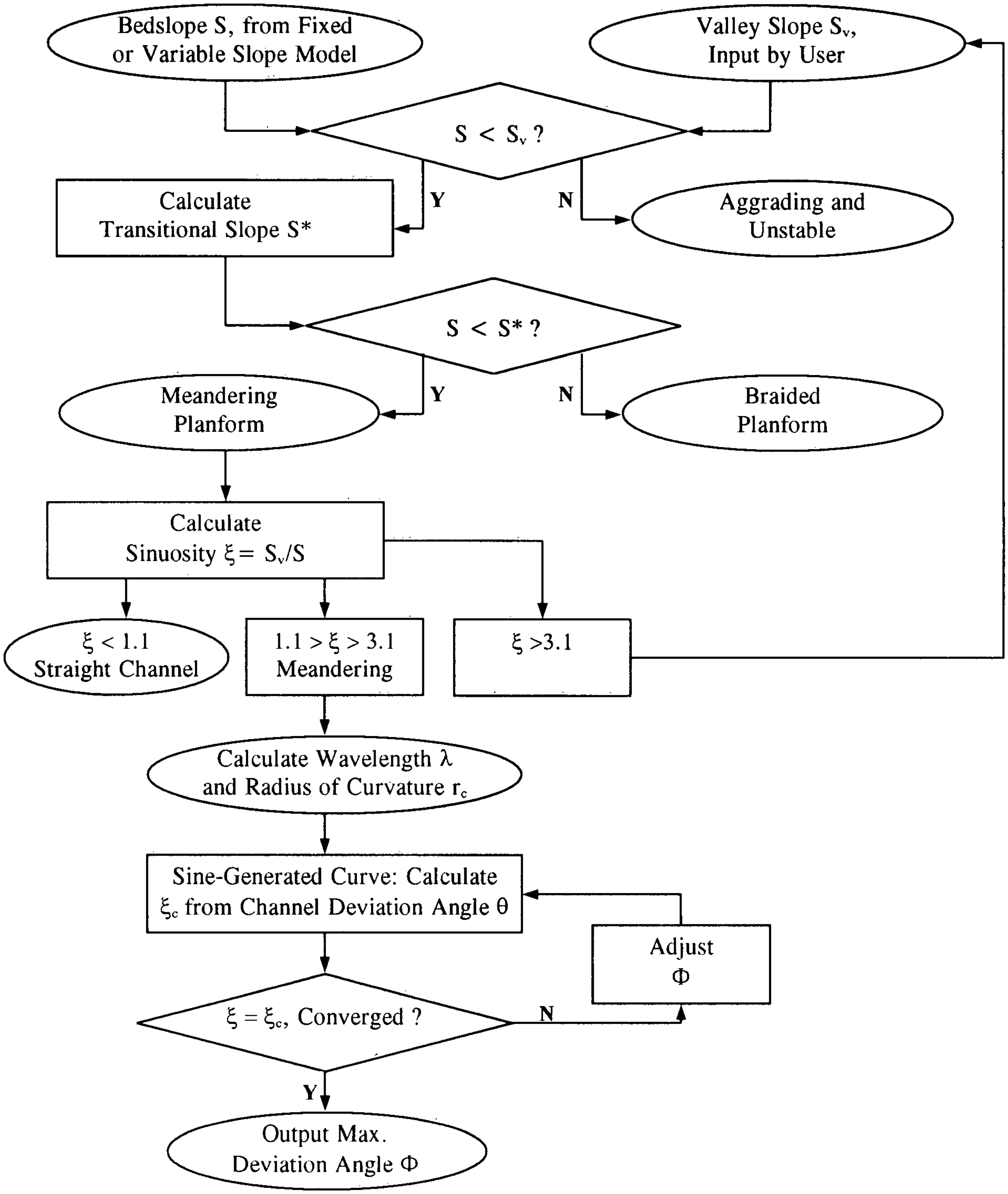


Figure 2.7. Flowchart for the Planform Model.

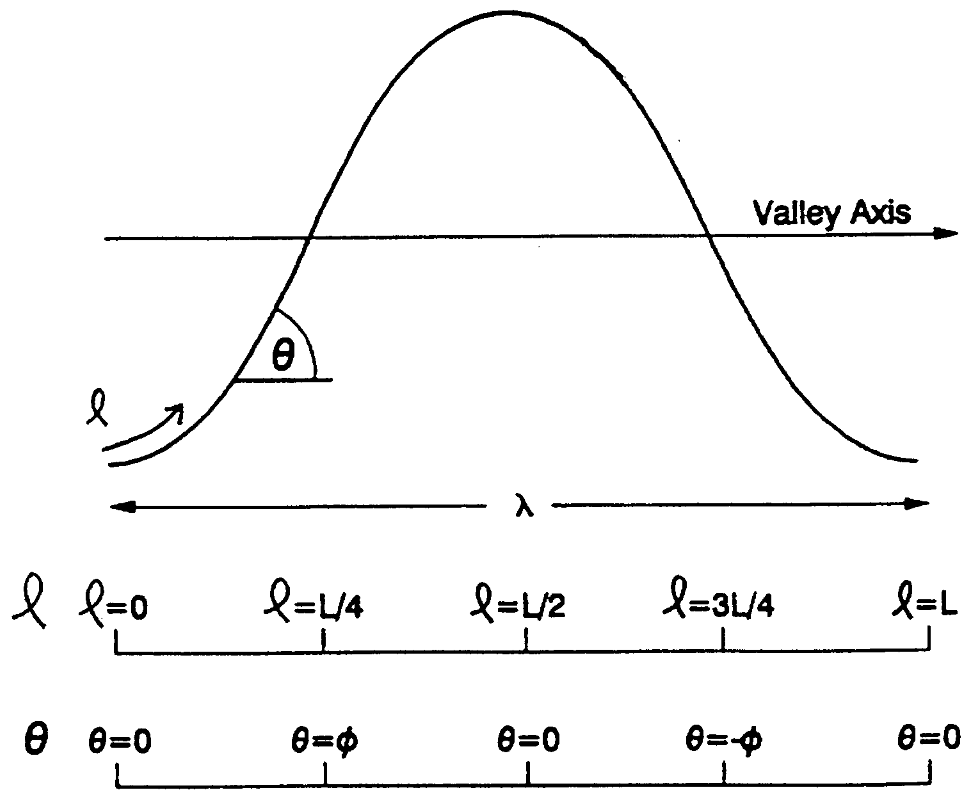


Figure 2.8. Definition sketch of one meander arc.

Chapter 3

TYPE OF ANALYSIS

3.1 INTRODUCTION

There are two versions of the original (Millar and Quick, 1993b) model and therefore two versions of RiverMod: a fixed-channel-slope model and a variable-channel-slope model. Each has been formulated for channels with either non-cohesive or cohesive banks.

3.2 FIXED-CHANNEL-SLOPE MODEL

The fixed-channel-slope version of RiverMod is equivalent to an experiment where the slope is fixed, and the channel width, depth and sediment transport rate adjust to the discharge. It is useful for examining the effect of bank stability on the channel geometry (Millar, 1994).

Discharge and bank stability are the only two constraints that need to be satisfied in the fixed-channel-slope version of the model. Since the value of the slope is input by the user, it is treated as an independent variable. The bedload constraint is only used in the variable-slope version (Millar, 1994).

Figure 3.1 is a flowchart for the Millar and Quick (1993b) fixed-slope model. The model starts by selecting a trial value of the bed perimeter (P_{bed}), bank perimeter (P_{bank}) and the bank angle (θ). These are the primary dependent variables. The other dependent variables can be calculated from these three. Only one of the three primary dependent variables is varied at a time. Once the discharge constraint is satisfied, the channel dimensions are calculated and then the bank stability constraint is assessed. Finally the optimal solution is found at the point of maximum sediment transport (Millar, 1994). Details regarding the dependent and independent variables are discussed in Chapter 4.

The fixed-channel-slope model has been formulated for channels with non-cohesive bank sediment and channels with cohesive bank sediment. The discharge constraint is the same for both of these channel types; however, each channel type has its own bank stability constraint (see Section 2.4).

Millar (1994) noted that in channels with non-cohesive bank sediment, the bank vegetation tended to increase ϕ' and therefore bank stability. In channels with cohesive bank sediment, bank vegetation tended to increase τ_{crit}.

3.3 Variable-Channel-Slope Model

The variable-channel-slope model more correctly approximates the behaviour of a natural gravel-bed river, since the slope is variable.

In the variable-channel-slope version of the model, channel slope (S) is treated as a dependent variable. The bankfull sediment discharge capacity (G_{bf}) is considered an independent variable. This version is similar to the fixed-slope version, with an additional bedload constraint, which requires the bedload transporting capacity of the channel to be equal to the sediment load (G_b) (Millar and Quick, 1993b).

Figure 3.2 is a flowchart showing the steps that make up the variable-channel-slope model. It is similar to Figure 3.1, except that the channel slope is varied for trial values of P_{bed}.

A strong relationship between the dependent variables is noticed when using the variable-slope model. The final output of the model are the dependent variables: width, depth and slope. They would change in response to changes the independent variables caused by alterations in the stream environment. As the independent variables change, the dependent variables will also adjust in accordance to the new conditions.

The dependent variables adjust their values at different rates. According to Booth (1990) the depth and width of the channel adjust at a much quicker rate than the slope. Even though they adjust at different rates, the width, depth and slope of a river are all inter-related and it is unlikely one will change without the others also modifying their values.

For channels with non-cohesive channel banks, Millar (1994) notes that if the width of the channel changes, the depth changes in order to satisfy continuity and the slope adjusts to satisfy the bedload constraint.

For channels with cohesive channel banks, the model can be used to determine whether a stream is bank-height or bank-shear constrained. In general, only one of these bank stability constraints is active at a time. Millar (1994) documented the change in the active bank stability constraint by varying Q_{bf}, keeping G_{bf} constant and then varying G_{bf}, and keeping Q_{bf} constant. In both cases, the remaining independent variables were kept constant for both investigations. The results are shown in Table 3.1

Table 3.1. Results of Millar (1994) active bank stability constraint investigation.

Bank-full Discharge (Q_{bf}) in m^3/s	Bankfull Sediment Transport Capacity (G_{bf}) in kg/s	Active Bank Stability Constraint
$Q_{bf} < 100$	$G_{bf} > 5$	bank-shear constrained
$100 < Q_{bf} < 250$	$3 < G_{bf} < 5$	both bank-height and bank-shear constrained
$Q_{bf} > 250$	$G_{bf} < 3$	bank-height constrained

Whichever bank stability constraint is not active, does not seem to affect the channel geometry (Millar, 1994).

In summary, the fixed-channel-slope and variable-channel-slope models show that the bank stability greatly affects the geometry of the channel. ϕ' is useful as a single parameter for assessing the effects of vegetation on the bank stability in streams with non-cohesive banks. In streams with cohesive banks, τ_{crit} can be used for the same purpose.

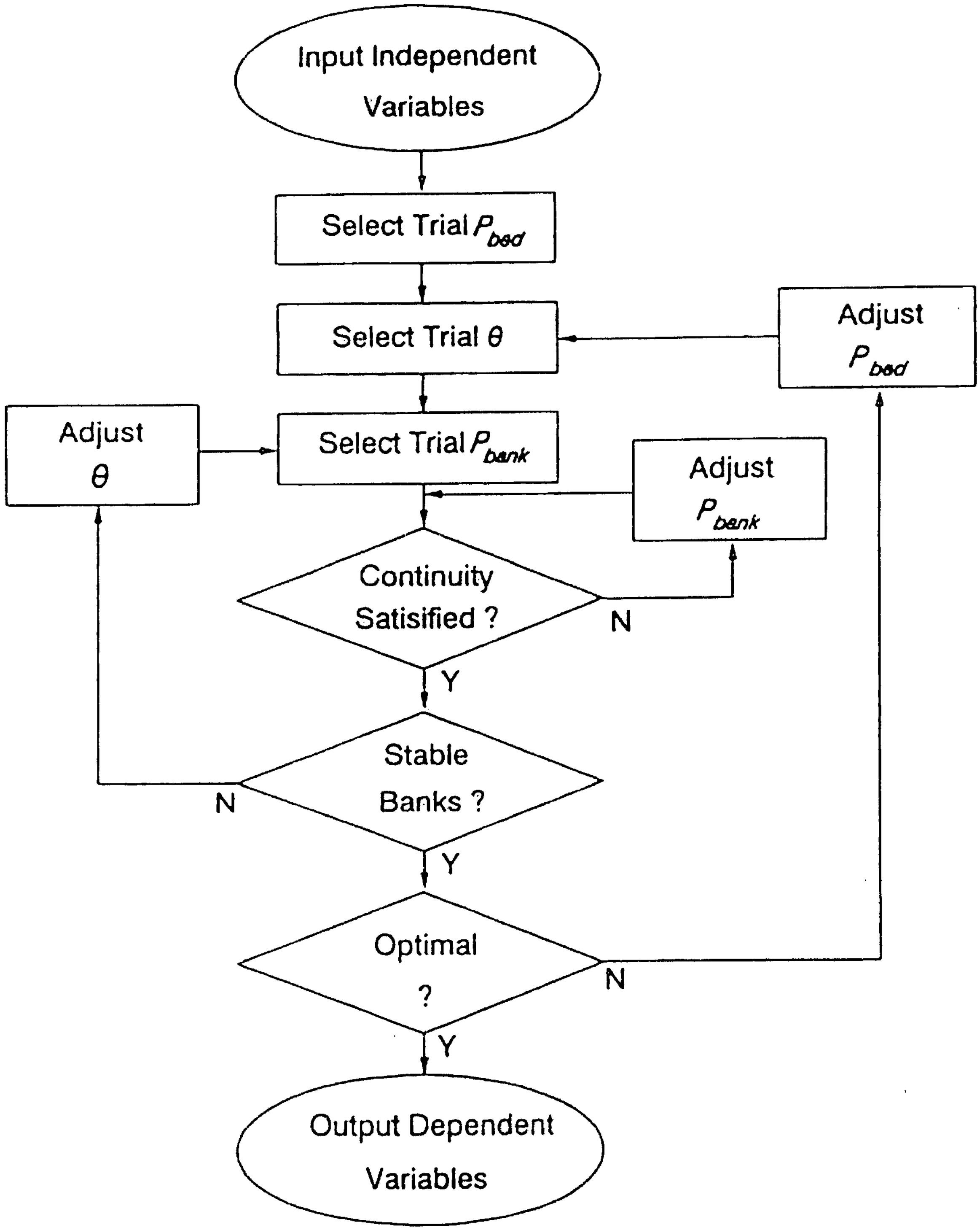


Figure 3.1. Flowchart for Millar and Quick (1993b) Fixed-Slope Model (MacVicar, 1999).

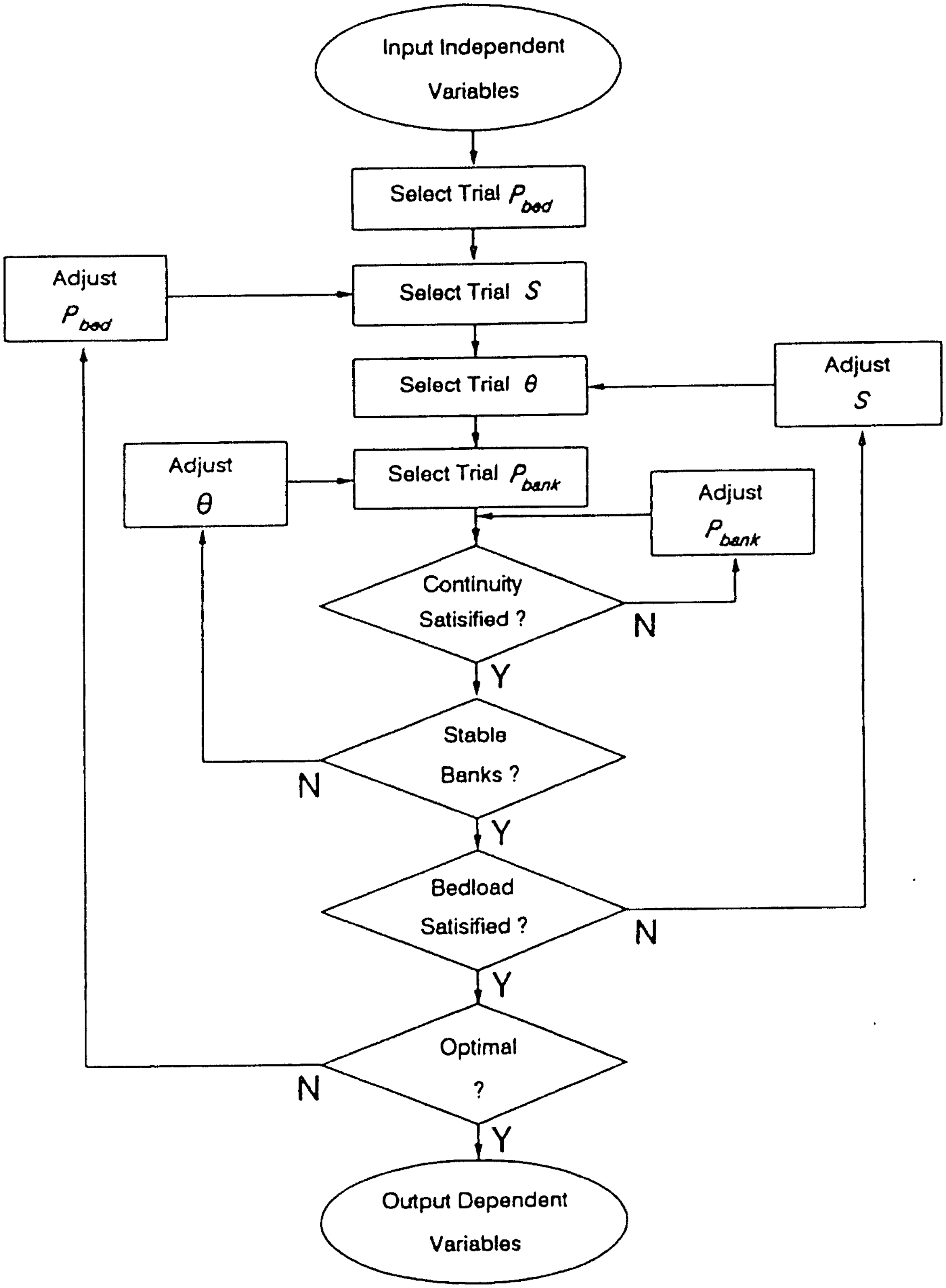


Figure 3.2. Flowchart for Millar and Quick (1993b) Variable-Slope Model (MacVicar, 1999).

Chapter 4

USING RIVERMOD

This chapter functions as a user manual, by guiding the user while running RiverMod. The following sections illustrate a step-by-step process for the successful operation of the model.

4.1 INSTALLATION

RiverMod is installed by inserting the CD and selecting "setup.exe". The installation program will guide the user through the setup process.

4.2 FILE SETUP SCREEN

RiverMod prompts a choice between two options: the user can either open an existing file or create a new record. (see Figure 4.1).

4.2.1 Open an Existing File

The user can choose to open a file saved earlier, or to return to the opening screen by clicking "Cancel". Once the user chooses which file to open, the name of the file will appear on the screen. Clicking "Open" (see Figure 4.2) will open the file and load the Computation Screen.

4.2.2 Creating a New File

The user will be guided through the RiverMod screens, which enable a new file to be created. This sequence is demonstrated in the following sections.

4.3 MODEL SCREEN

Once the user has chosen to create a new file, the Model Screen is loaded (see Figure 4.3). The Model Screen allows the user to select the Model Type, Bank Type and Unit Type.

There are two different model types: the fixed slope model and the variable slope model.

There are two different bank types to choose from. The model has been formulated for gravel-bed rivers with either non-cohesive or cohesive bank sediment.

The model operates using the SI (Système Internationale d'Unités) system of units. However, the user may select between two systems of units: the British Gravitational (BG) system, when working with variables in imperial units, and the SI system when dealing with metric units.

Once the user has selected the model, bank and unit type, the Equation Screen is loaded.

4.4 EQUATION SCREEN

The Equation Screen presents the user with four sediment transport models and four flow resistance equations (see Figure 4.4). The sediment transport models are Einstein-Brown (1950), Ackers and White (1973), Meyer-Peter and Müller (1948) and Parker, Klingeman and McLean (1982). The flow resistance equations are Keulegan (1938), Manning (1891), Composite Manning's (Horton, 1933) and Jarrett (1984). The combination of equations will affect the model's ability to accurately predict channel changes.

4.5 COMPUTATION SCREEN

This screen provides an interface to the nucleus of the model. It is divided into an input section and an output section (see Figures 4.5 and 4.6). The options selected on the Model and Equation Screens determine which variables are needed as input and which will appear as output.

4.5.1 Model Input

(a) <u>Discharge</u>

Bankfull discharge (Q_{bf}) is used in either m^3/s or ft^3/s. Often Q_{bf} is taken to be the discharge with a two-year return period (Q_2).

(b) <u>Roughness Coefficient</u>

The roughness coefficient depends on which flow resistance equation has been selected.

1. Keulegan - equivalent roughness (k_s) in metres or feet.
2. Manning's n - Manning's roughness coefficient (n). It is unitless.
3. Composite Manning's n - Manning's roughness coefficient for the bed of the channel (n_{bed}) and for the channel banks (n_{bank}).
4. Jarrett - this relation calculates a value for Manning's roughness coefficient; therefore no specific roughness input is needed.

(c) Sediment Transport

Each sediment transport relation requires a particular grain diameter to represent the sediment grain-size distribution. The units should be in metres or feet.

1. The Einstein-Brown model requires the median grain diameter of the bedload sediment (D_{50}).
2. The Ackers and White model requires the thirty-fifth percentile grain diameter of the bedload sediment (D_{35}).
3. The Meyer-Peter and Muller model requires both the ninetieth percentile grain diameter (D_{90}) and the average grain diameter (D_a) of the bedload sediment.
4. The Parker, Klingeman and McLean model requires the median sub-surface grain diameter (d_{50}).

(d) Bank Stability

The bank stability constraint has been formulated for both cohesive and non-cohesive bank sediments (Millar, 1994). For channels with non-cohesive bank sediment, the median grain diameter of the bank material (D_{50bank}) and the internal friction angle

(ϕ') are necessary as input data. D_{50bank} is in metres or feet and ϕ' is measured in degrees. For channels with cohesive bank sediment, soil cohesion (c), critical shear stress (τ_{crit}) and the unit weight of bank sediment (γ_t) are needed as input data. Soil cohesion and critical shear stress are measured in Pascals or lb/in^2 (psi), and γ_t is measured in N/m^3 or lb/ft^3.

(e) Channel Slope

If the fixed-slope model is used, the reach-averaged bedslope (S) appears on the input screen as an independent variable. It is unitless and is the ratio of the vertical elevation change of the channel to the horizontal distance along the bed. In the variable-slope version of the model, the slope appears only as an output and is therefore a dependent variable.

(f) Sediment Transport Capacity

If the variable-slope model is used, the sediment transport capacity (G_b) appears as input data, measured in kg/s or lb/s. In the fixed-slope version, it appears as the output data.

4.5.2 Model Output

The surface width (W), mean depth (Y) and bed slope (S) are considered the three primary dependent variables. Surface width and mean depth are calculated in either metres or feet, while bed slope is unitless. The three primary dependent variables appear on the main Computation Screen. Bank height (in metres or feet), bank angle, (in degrees), and sediment transport capacity (in kg/s or lb/s) further describe the geometry of the channel and are considered secondary dependent variables. They may be viewed on the Supplementary Output Screen by clicking "More Details" (see Figure 4.7)

For channels with cohesive bank sediment, two additional output variables appear on the Supplementary Output Screen, are H/H_{crit} and τ_{bank}/τ_{crit}. They are both unitless and further indicators of bank stability.

Table 4.1 shows the minimum and maximum values for each of the variables in order for RiverMod to run properly for both SI and Imperial Units. Some of the limits may extend beyond the normal range for a certain variable in a small gravel-bed river.

Table 4.1. Minimum and maximum values for input variables

Input Variable	SI Units	Imperial Units
Bankfull Discharge, Q_{bf}	0 - 25000 m^3/s	0 - 883,000 ft^3/s
Equivalent Roughness, k_s	0.002 - 1 m	0.0066 - 3.28 ft
Manning's Roughness Coefficient, n	0.01 - 0.15	0.01 - 0.15
Bank Manning's Roughness Coefficient, n_{bank}	0.01 - 0.15	0.01 - 0.15
Bed Manning's Roughness Coefficient, n_{bed}	0.01 - 0.15	0.01 - 0.15
Thirty-fifth Percentile Grain Diameter, D_{35}	0.0001 - 0.5 m	0.00033 - 1.64 ft
Median Grain Diameter, D_{50}	0.0001 - 0.5 m	0.00033 - 1.64 ft
Ninetieth Percentile Grain Diameter, D_{90}	0.0001 - 0.5 m	0.00033 - 1.64 ft
Average Grain Diameter, D_a	0.0001 - 0.5 m	0.00033 - 1.64 ft
Median Sub-surface Grain Diameter, d_{50}	0.0001 - 0.5 m	0.00033 - 1.64 ft
Median Bank Grain Diameter, D_{50bank}	0.0001 - 0.5 m	0.00033 - 1.64 ft
Modified Friction Angle, ϕ'	20 - 90 °	20 - 90 °
Critical Shear Stress, $\tau_{critical}$	0 - 150 Pa	0 - 0.02175 psi
Soil Cohesion, c	0 - 100,000 Pa	0 - 14.5 psi
Unit Weight of Bank Sediment, γ_t	15, 000 - 40,000 N/m^3	95.5 - 254.6 lb/ft^3
Reach-Averaged Bed Slope, S	0.0001 - 0.05	0.0001 - 0.05
Sediment Transport Capacity, G_b	0 - 5000 kg/s	0 - 11000 lb/s

4.5.3 Drop-down Menu

The drop-down menu at the top of the Computation Screen provides options for data manipulation. Each of these options is detailed below.

4.5.3.1 File

(a) Entering Data

Data is entered into the Computation Screen and is grouped into records. This enables the user to save several series together as one file. Each record contains the same sediment transport and flow resistance equations as well as the same model type, bank type and unit type.

The "New", "Next" and "Previous" buttons enable navigation from one record to another.

(b) Saving Data

As data is entered into the Computation Screen, it is automatically saved into a file called "RIVERFILE.DAT". Once the user chooses to save the file, RIVERFILE.DAT is renamed with the user-selected name. Input and output data are saved together as a "DAT" file. The save function operates similarly to other windows-based programs.

(c) Open Existing File

There are two ways to open an existing file. The first is to select this option on the File Setup Screen when the model is loaded. The second is to choose this option from the File menu, on the Computation Screen.

RiverMod's capacity to check for compatibility between saved options and new user-determined options is a powerful and innovative feature that extends previous models.

For example, perhaps the file was originally saved using the Einstein-Brown sediment transport relation, and the user wishes to open it using the Ackers and White sediment transport relation. The program would open the files and all of the data common to both options would appear on the screen. The data not held in common by the chosen equations - in this case, the grain diameter - would not appear on the screen.

4.5.3.2 Edit

(a) Create a New Record

Choosing this option from the Edit Menu enables the user to add a new blank record to the file. The data in the new record is related to the data in the previous record because they share the same model options, but is input to a completely separate model run. The data contained in the text boxes of the new record can be entirely different from the data in previous or subsequent records.

(b) Delete and Clear Current Record

Deleting a record allows the user to delete the current record from the series. The series will then contain one less record.

Clearing a record will leave all the text boxes in the current record blank, but the number of records in the series will remain the same.

(c) Search for a Record

The "Search for a Record" option under the Edit Menu, permits the user to search for a particular record. This can be done by searching for a particular value of any of the variables contained within the series of records.

Using a series of user-friendly screen prompts (see Figure 4.8), the model will locate the first record containing that value. This feature is particularly useful for a user working with a series containing a large number of records.

4.5.3.3 Output

(a) Run

Choosing "Run" from the Output menu executes the optimization model. Before running, the model automatically scans for three potential errors that can occur during data entry:

- empty text boxes;
- non-numeric characters;
- values outside the permissible range.

If an error is located, the program pauses, alerts the user, and allows the user to correct the mistake. The user can then choose the "Run" option again, and the program will proceed with the execution of the optimization model.

There are two slightly different paths the routine can follow, one for the fixed-slope model and one for the variable-slope model. Each is explained in more detail in Chapter 3.

Figure 3.1 shows the fixed-slope version of the optimization model and Figure 3.2 shows the variable-slope version of the model. The two methods are very similar and follow this general procedure (Millar and Quick, 1998):

(i) Input the independent variables.

(ii) The minimum and maximum bounds for P_{bed}, θ and P_{bank} are set.

(iii) The midpoints of P_{bed}, θ and P_{bank} are determined.

(iv) Flow resistance and velocity are calculated, using the equation chose by the user, at the midpoint. The bounds of P_{bank} are adjusted until the discharge constraint is satisfied ($UA = Q_{bf}$) and the flow resistance and velocity values are also adjusted for each new midpoint value of P_{bank}.

(v) Bank stability:

a. For non-cohesive banks, Equation 2.43 must be satisfied for the banks to be stable,

b. For cohesive banks, the bank angle is adjusted until $FS_H = 1$ (Equation 2.44) and $FS_\tau = 1$ (Equation 2.46). If the banks are not stable, the channel is too narrow, the bounds for P_{bed} are reset and the procedure returns to step (iv). If the banks are stable, the routine proceeds to step (vi).

(vi) The bedload transport is calculated using the equation chosen by the user. If the P_{bed} value corresponds to the maximum bed load transport, then the optimum solution has been reached and the dependent variables width (W), depth (Y) and bank angle (θ) are calculated as output. If the P_{bed} value does not represent the optimum solution, the procedure returns to step (iii).

The final solution represents the steady-state equilibrium geometry of the reach (Millar, 1994).

(b) Planform Geometry

This option takes the user to the Planform Screen (see Figure 4.9 and Section 2.5). RiverMod will determine the planform geometry for the record which is open when this option is chosen. This feature of the model is useful for river restoration or design purposes (see Section 4.6).

4.5.3.4 Microsoft Excel

(a) Open Microsoft Excel

Choosing this option from the Output menu opens the spreadsheet program Microsoft Excel (provided it is available on the user's computer). In order for it to function properly, Microsoft Excel must be closed before this feature is executed.

Integration of RiverMod with Microsoft Excel adds power and flexibility, allowing the user to:

- create graphs and charts;

- visually compare the relationship of one variable to another;
- visually compare the results obtained using different equations or model options.

The model will automatically identify the options chosen by the user and transfer them to the Microsoft Excel worksheet. The worksheet will have a column for each variable with headings, which include the name and units of each variable. Figure 4.10 shows an example where the discharge was increased by 25 m^3/s for 5 runs, while all other variables were held constant.

Once the user has opened Microsoft Excel, all of its functions are independent of those of RiverMod. If the user wishes to create graphs or move and delete data, he/she must do so from within the Microsoft Excel program. In order to save any new worksheets, graphs or files created in Microsoft Excel, the user must also do this from within Microsoft Excel and not by using RiverMod.

As soon as RiverMod is closed, Microsoft Excel, opened by RiverMod will also be closed, so any file handling within Microsoft Excel should be done before RiverMod is terminated.

(b) Append to Microsoft Excel Spreadsheet

There are two separate append features built into the model. In order to use these features, Microsoft Excel must already have been opened through RiverMod. Both "append" commands add to the spreadsheet the input and output data, found on both the Computation Screen and the Supplementary Output Screen.

- Append Current Record to Spreadsheet - this feature is useful for importing the data in the current record into the Microsoft Excel Spreadsheet.
- Append All Records to Spreadsheet - this feature allows transfer of the data in all the records into the spreadsheet simultaneously. The "append all" feature can only be executed from Record 1, so that the model can run through the records in order.

4.6 Planform Screen

The user can access the Planform Screen (see Figure 4.9) from the Computation Screen. RiverMod will use the valley slope, entered by the user, in conjunction with a formulation for the transitional slope (S^*) to determine the geometry of the river. If the channel slope is greater than the valley slope, the river is aggrading and unstable. Otherwise, a meandering-braiding transition formulation is used to determine the river planform. Equation 2.49, developed by Parker (1976, as cited in Millar 2000), is used to determine the transitional slope. The transition slope is compared to the bed slope to determine if the river is braided or meandering, using Equations 2.50a and 2.50b.

If the river is meandering, the sinuosity (ξ) and meander wavelength (λ) are determined using empirical relations developed by Leopold and Wolman (1960). These empirical relations ordinarily use a meander wavelength multiplier between 10 and 12. The model uses 11 as the default value, but the user may enter an alternate value.

The model also utilizes Langbein and Leopold's (1966) sine-generated curve (see Section 2.5) to calculate values for the maximum deviation angle along the channel (Φ) and the radius of curvature (r_c).

This information is useful for river restoration, which attempts to return the channel to its "natural" form. This may require the redesign of sections of channel that have been straightened or channelized.

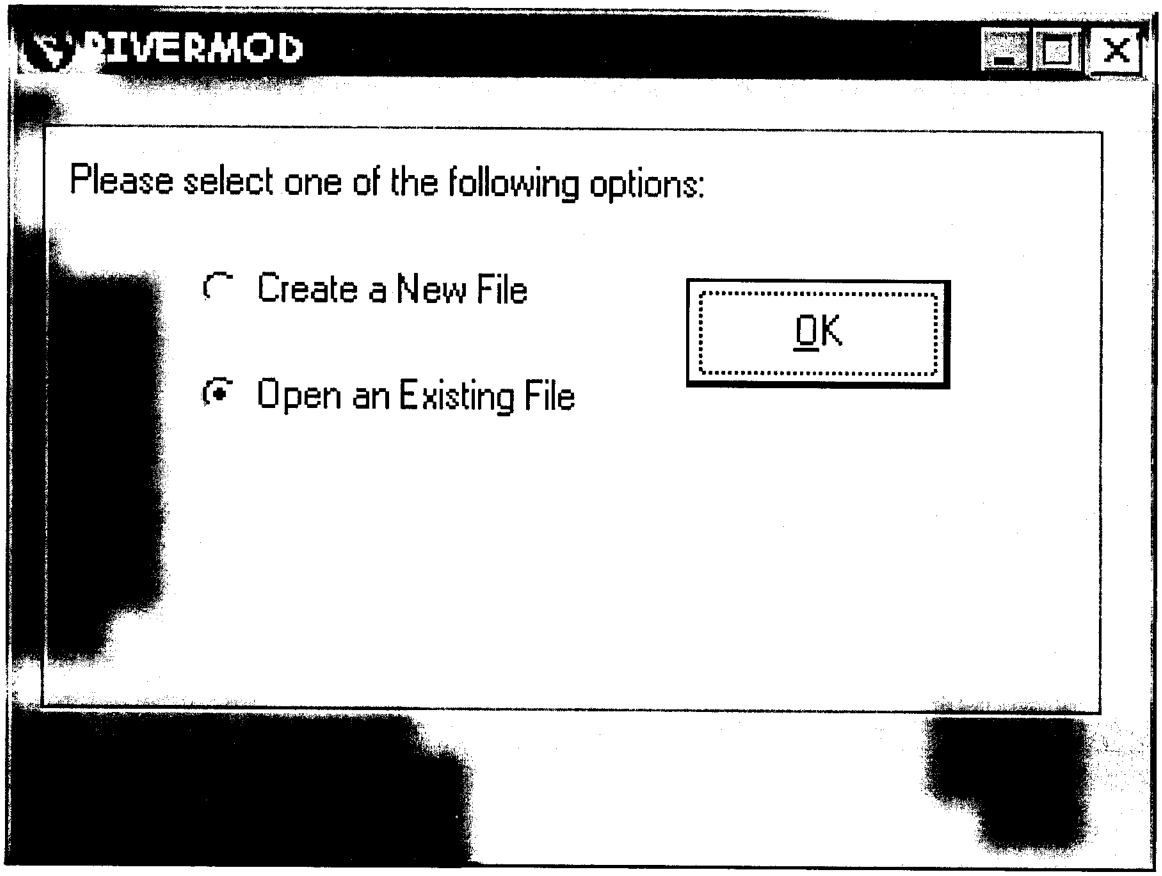


Figure 4.1. File Setup Screen

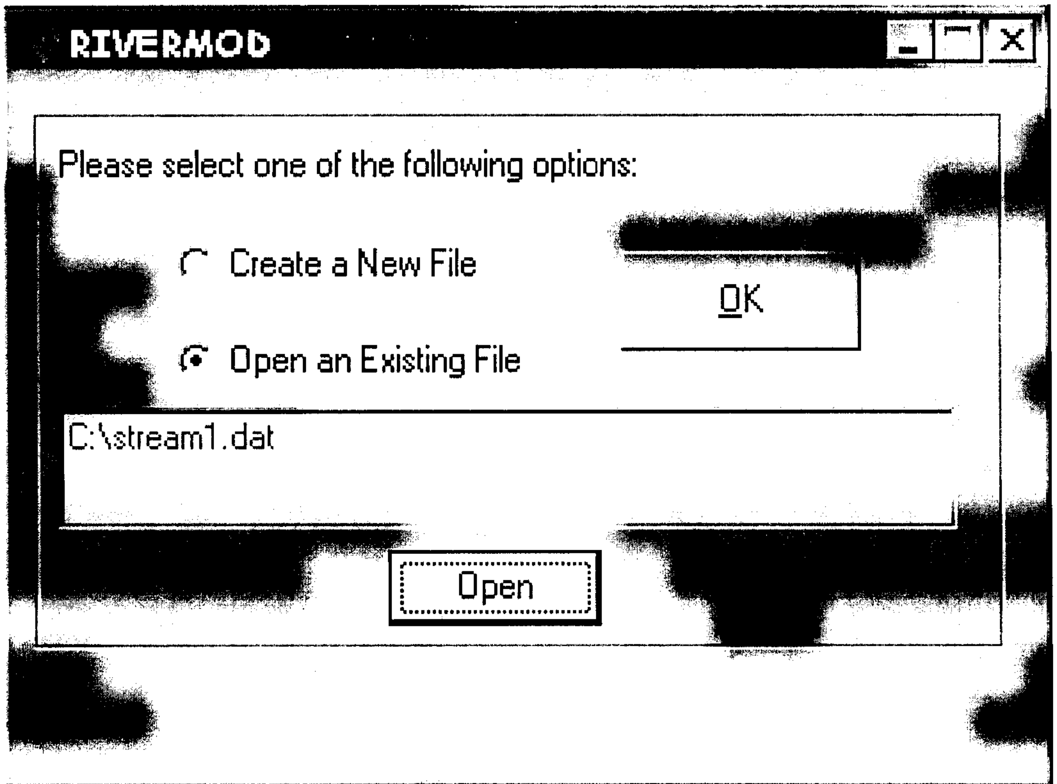


Figure 4.2. File Setup Screen with selected file name.

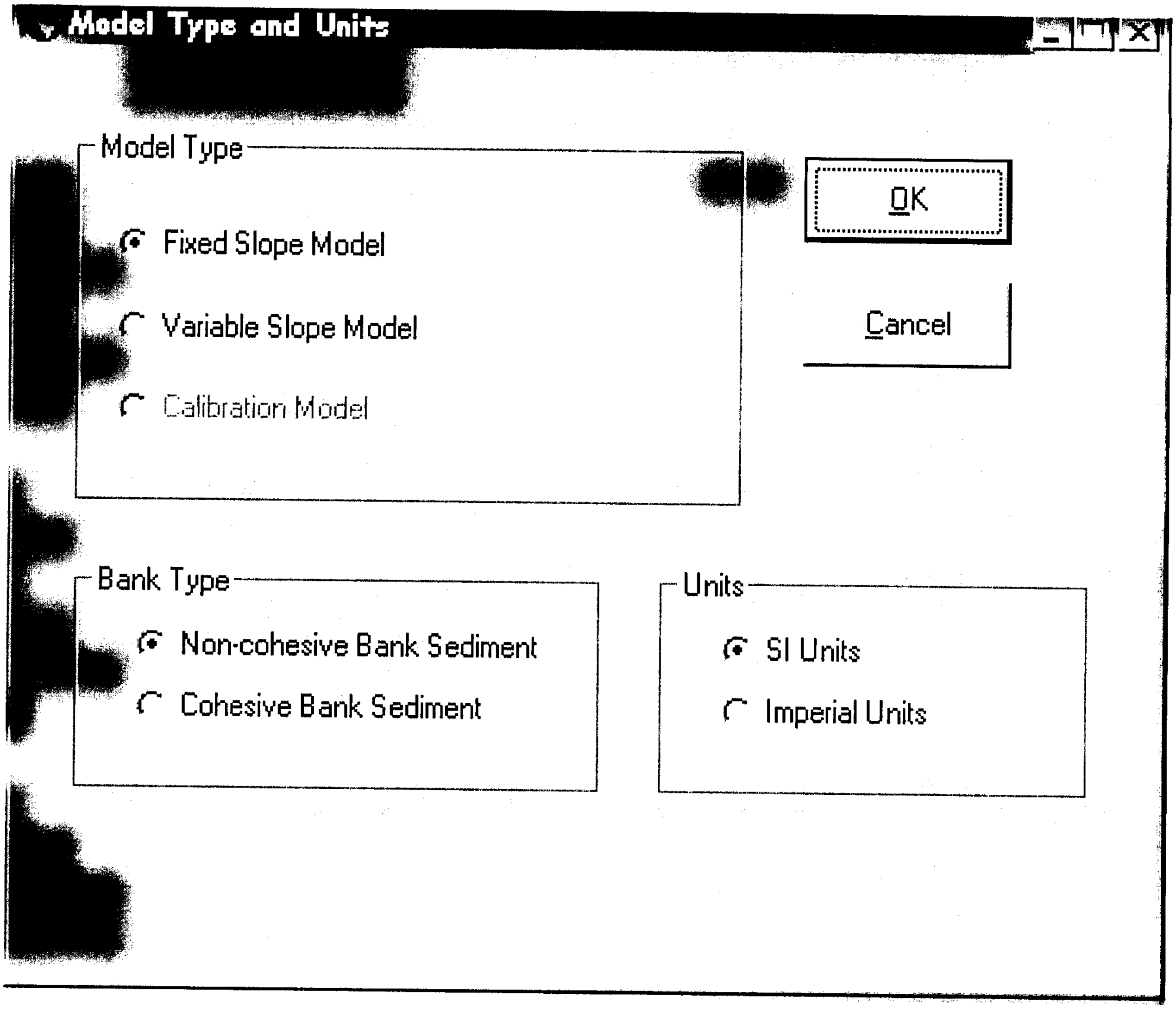


Figure 4.3. Model Screen

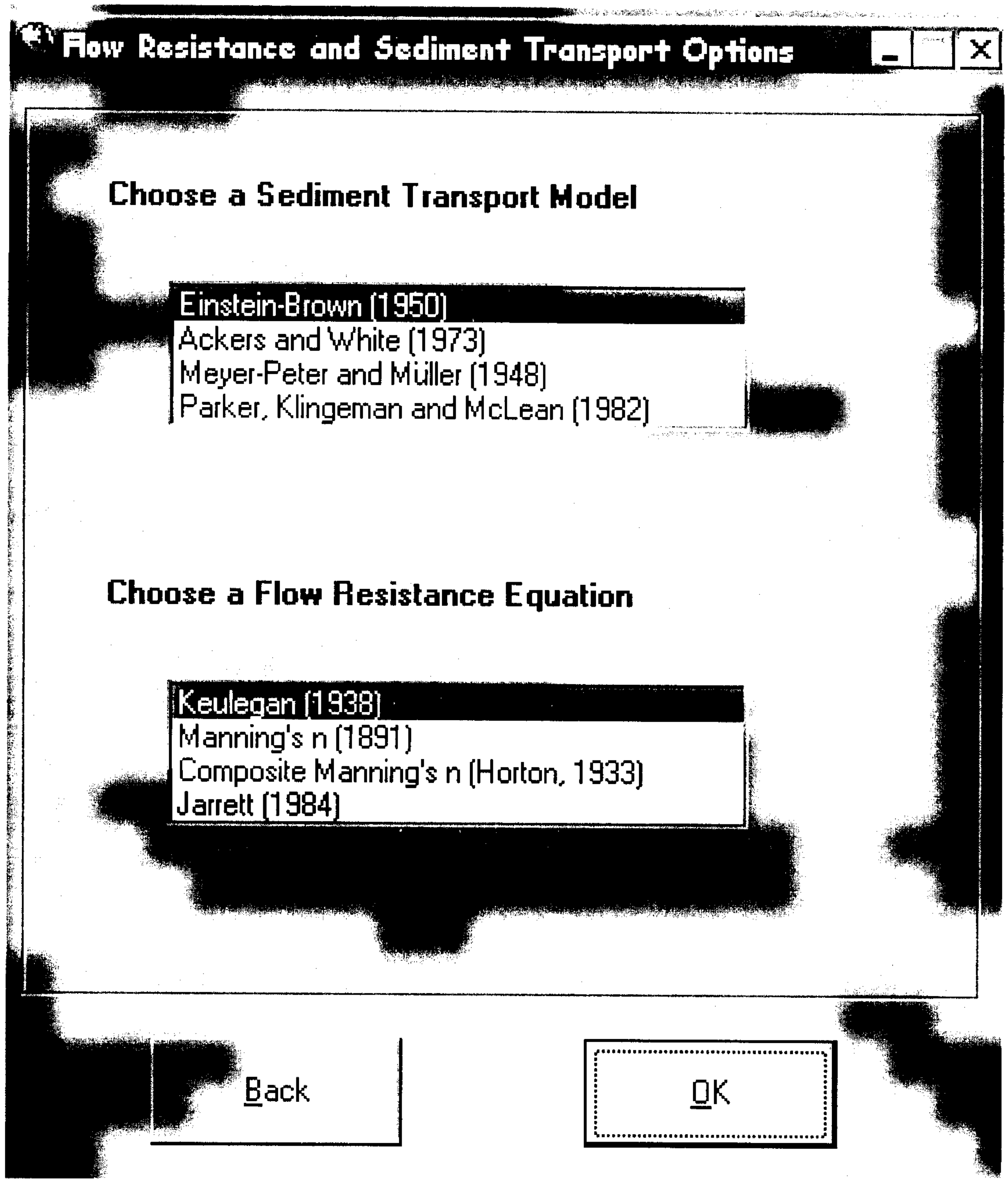


Figure 4.4 Equation Screen

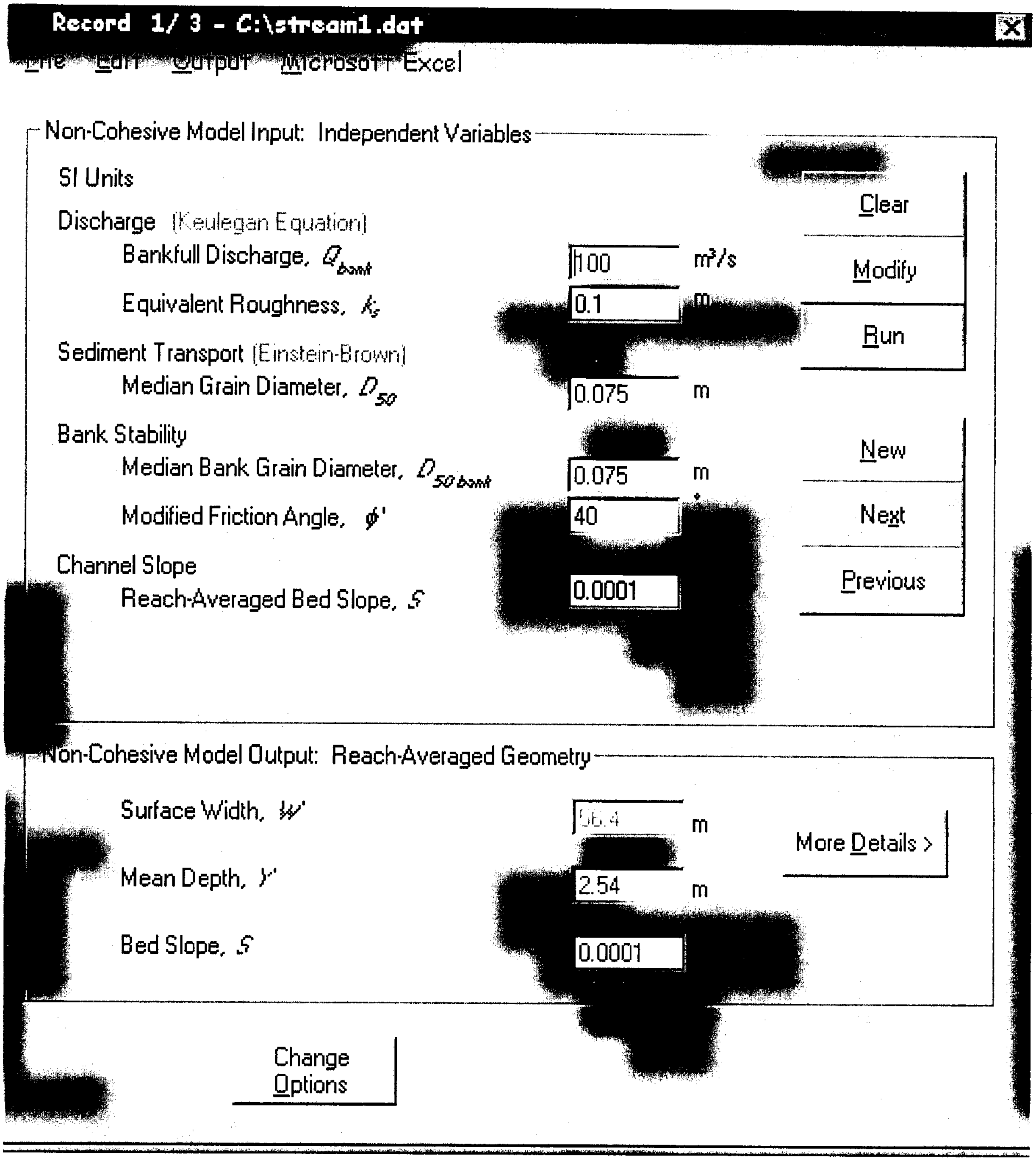


Figure 4.5. Computation Screen for Non-Cohesive Sediment with Einstein-Brown and Keulegan equations in SI Units, using the Fixed Slope Model.

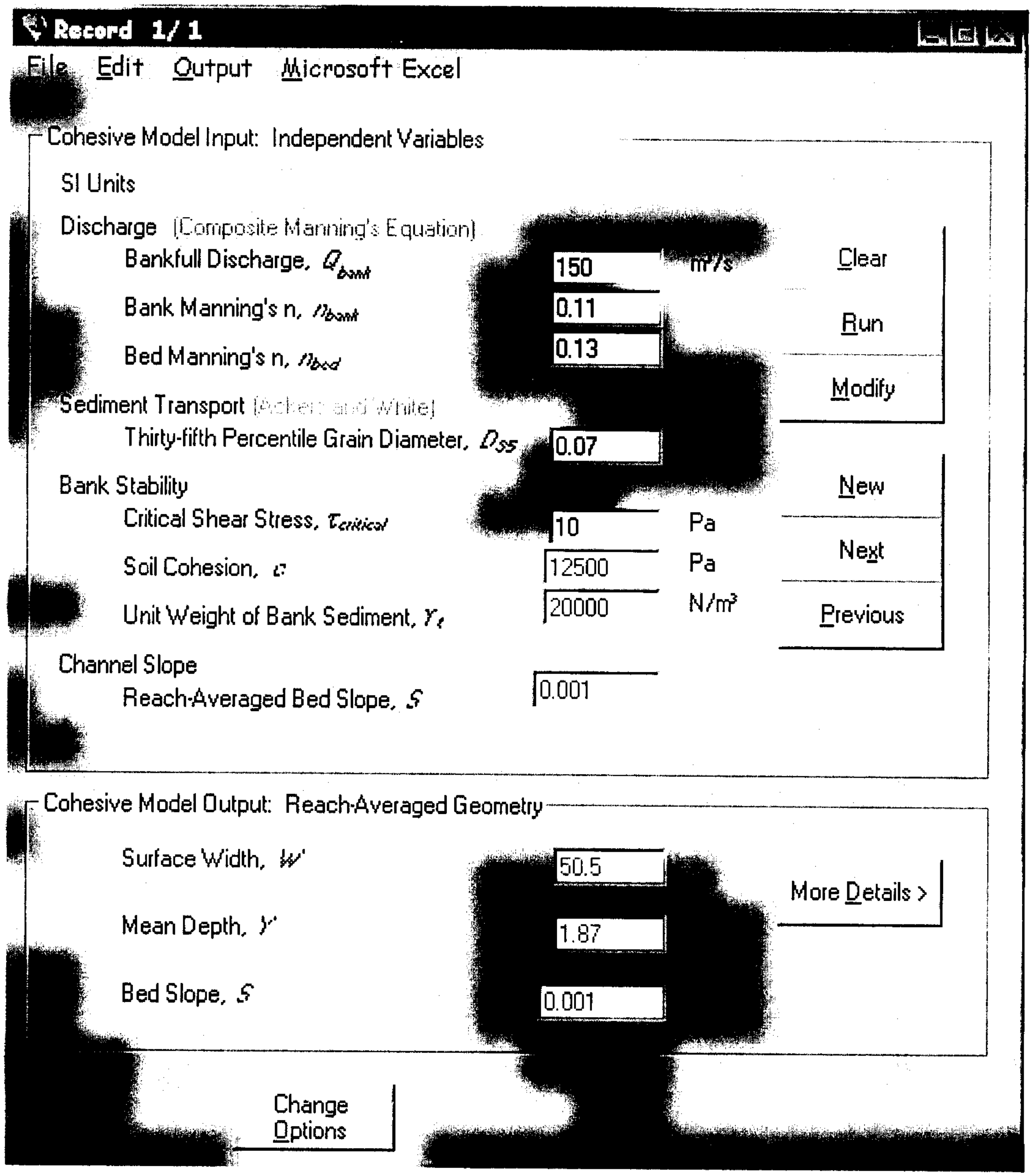
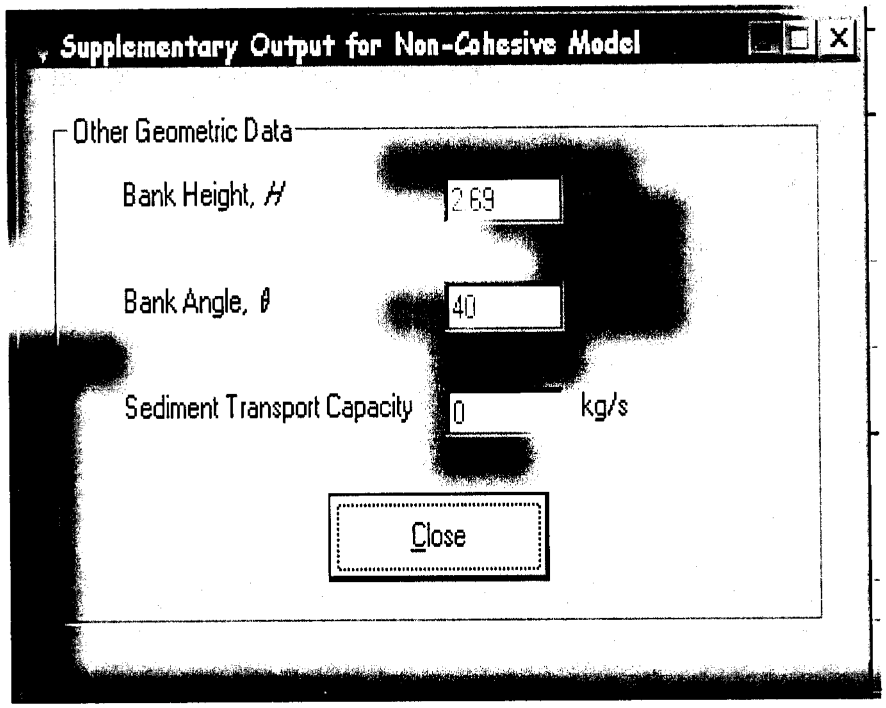


Figure 4.6. Computation Screen for Cohesive Sediment with Ackers and White and Composite Manning equations in SI Units using the Fixed Slope Model.

(a)



(b)

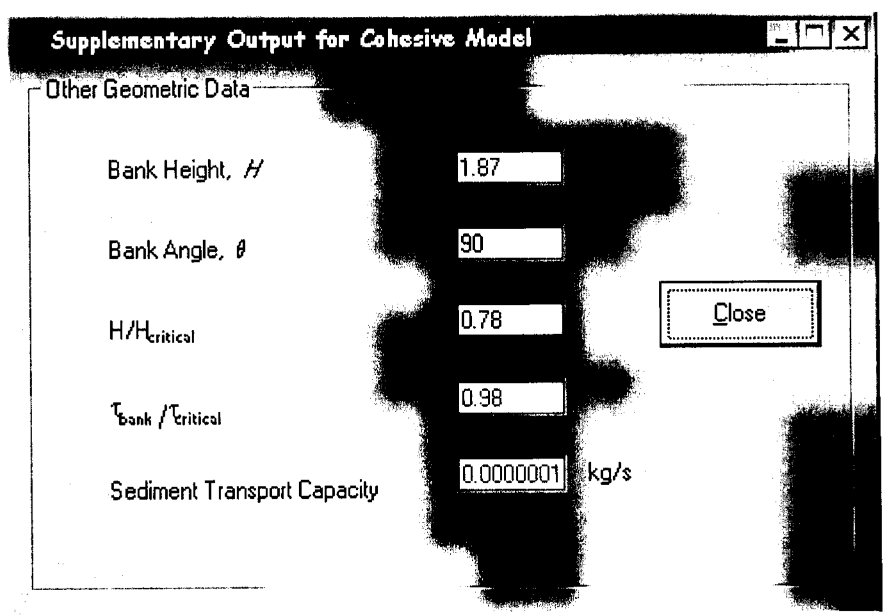


Figure 4.7. Supplementary Output Screen for: (a) Non-Cohesive Sediment Model, with Einstein-Brown and Keulegan equations in SI Units, using the Fixed Slope Model; and (b) Cohesive Sediment Model with Ackers and White and Composite Manning equations in SI units, using the Fixed Slope Model.

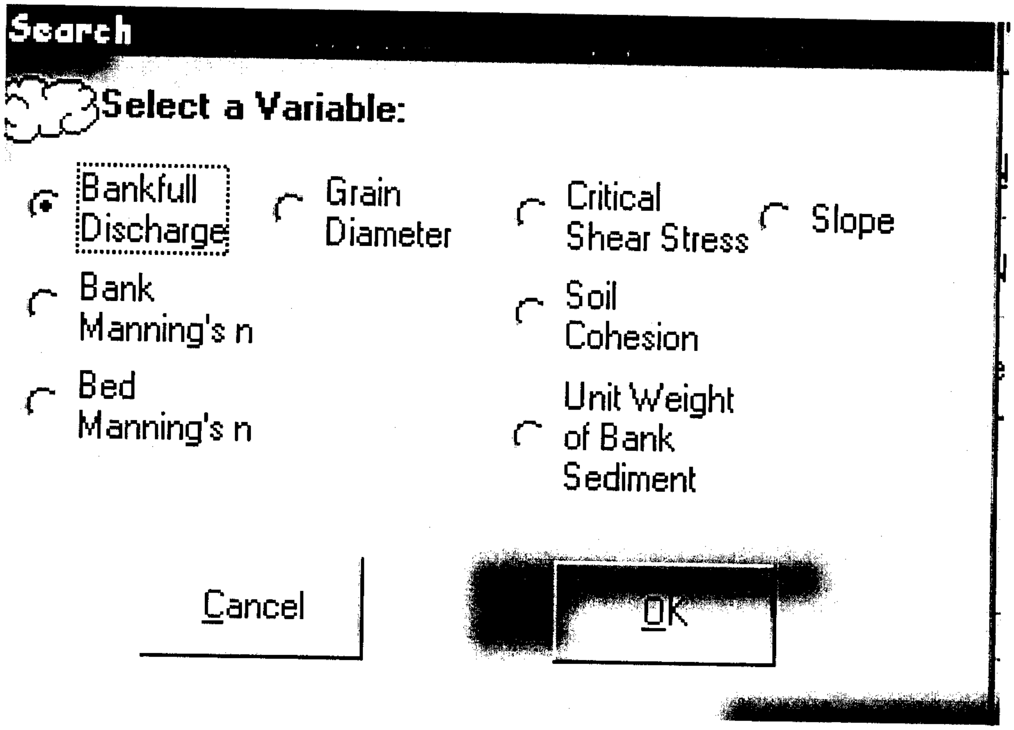


Figure 4.8. Search Screen showing Cohesive Sediment Model Options.

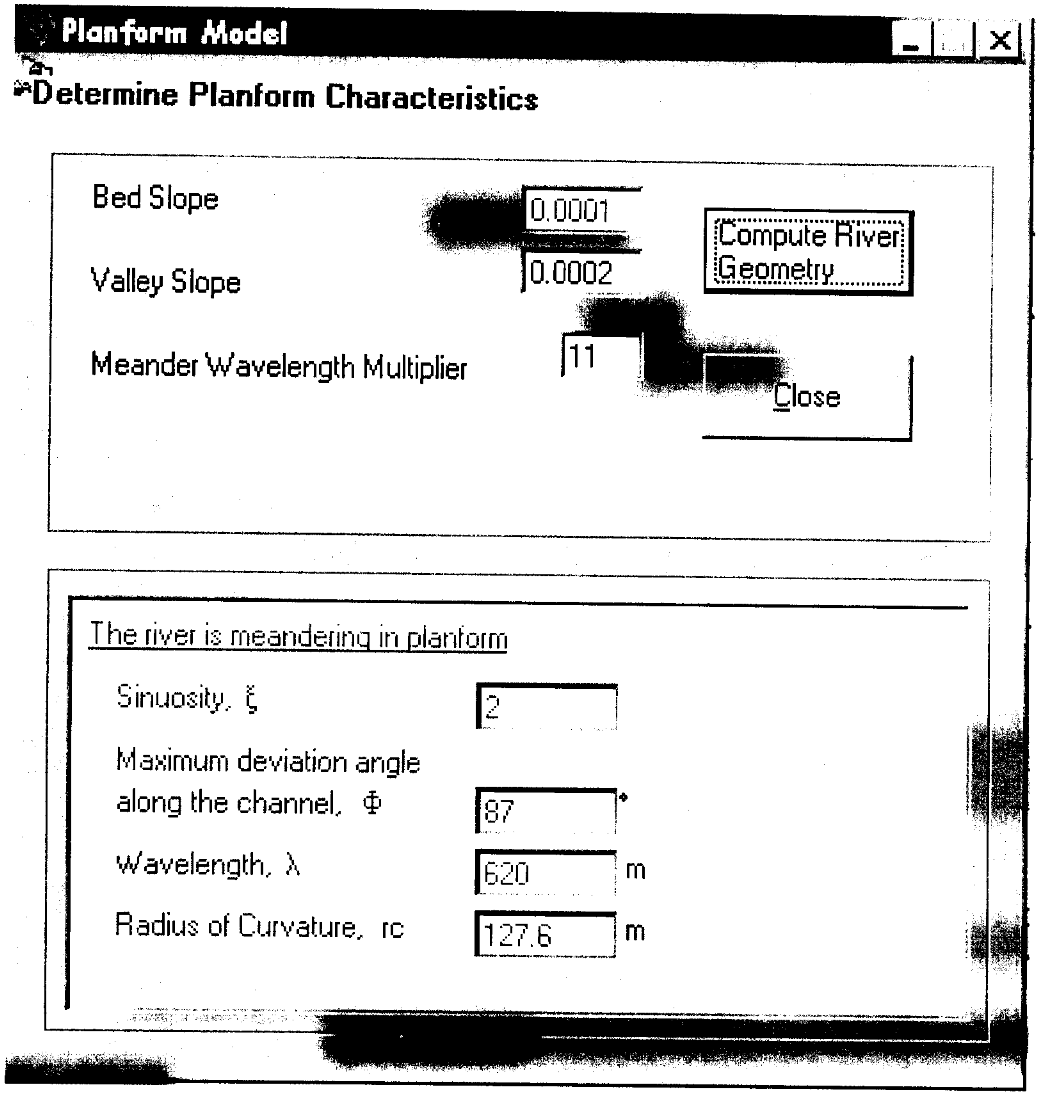
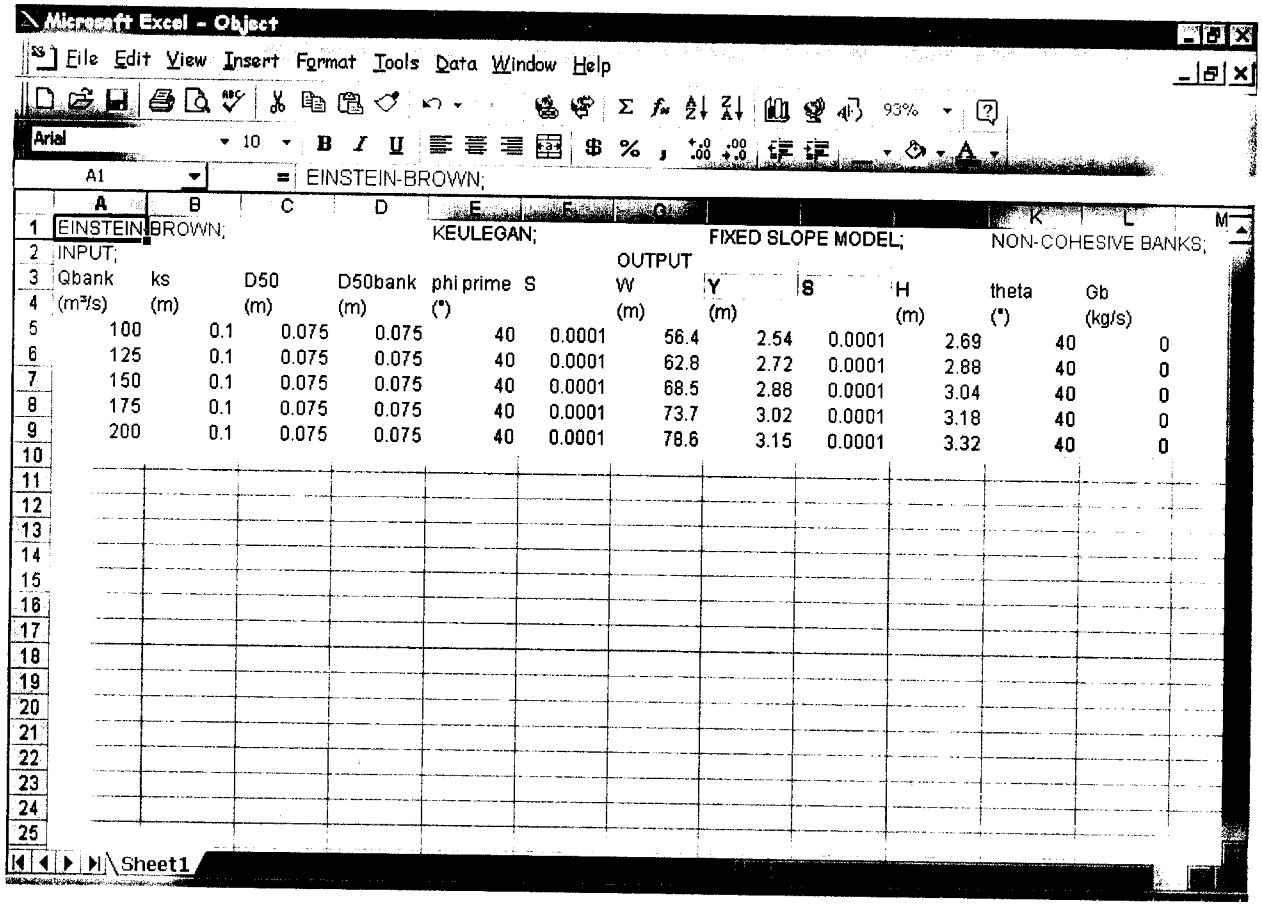


Figure 4.9. Planform Screen. This example shows a meandering stream.



A	B	C	D	E	F	G	H	I	J	K	L
EINSTEIN-BROWN;				KEULEGAN;			FIXED SLOPE MODEL;			NON-COHESIVE BANKS;	
INPUT;						OUTPUT					
Qbank	ks	D50	D50bank	phi prime	S	W	Y	S	H	theta	Gb
(m^3/s)	(m)	(m)	(m)	(°)		(m)	(m)		(m)	(°)	(kg/s)
100	0.1	0.075	0.075	40	0.0001	56.4	2.54	0.0001	2.69	40	0
125	0.1	0.075	0.075	40	0.0001	62.8	2.72	0.0001	2.88	40	0
150	0.1	0.075	0.075	40	0.0001	68.5	2.88	0.0001	3.04	40	0
175	0.1	0.075	0.075	40	0.0001	73.7	3.02	0.0001	3.18	40	0
200	0.1	0.075	0.075	40	0.0001	78.6	3.15	0.0001	3.32	40	0

Figure 4.10. Microsoft Excel Spreadsheet with values appended from RIVERMOD for an example using Non-Cohesive Sediment with Einstein-Brown and Keulegan equations in SI Units, using the Fixed Slope Model. In this example, the discharge was increased by 25 m^3/s for 5 runs, while all other variables remained constant.

Chapter 5

SAMPLE APPLICATIONS

5.1 INTRODUCTION

The following sections present two examples for using RiverMod. The model is applied to two rivers in British Columbia: Slesse Creek and Narrowlake Creek. Both creeks have been affected by logging, resulting in changes to the bank stability, river width and planform geometry.

5.2 SLESSE CREEK

Slesse Creek is a mountain stream situated in southwestern British Columbia and northwestern Washington State (see Figure 5.1). The upper catchment of the river, located on the U.S. side, is in a protected, pristine forested area. The lower reaches of the river, on the Canadian side, have been subjected to clear-cut logging along the banks and floodplain. As a result of this disturbance on the banks, the stream's geometry has changed with active widths increasing by over 3 times from pre-logging to post-logging conditions (MacVicar, 1999). The morphology has changed from a single channel meandering planform to a wide unstable braided planform (see Figure 5.2).

This river provides an excellent case study for RiverMod. The nearby forestry activity has not affected water and sediment supply, but riparian logging has caused a decrease in the bank stability, due to the loss of bank vegetation. Therefore, the only independent variable to suffer significant change is the internal friction angle, ϕ', which decreased from 73° to 40° from 1936-1993 (MacVicar, 1999).

5.2.1 Investigation

In order to apply RiverMod to Slesse Creek, the input parameters must be defined for past and present values. MacVicar (1999) concentrated his analysis on Reach D of Slesse Creek (see Figure 5.3) because the reach is alluvial, the morphology is relatively homogeneous, fish habitat has been greatly decreased and recent restoration efforts have been focused on this reach (Babikaiff and Associates, 1997 as cited by MacVicar, 1999).

Site surveys and historic aerial photograph analysis have shown that from 1936-1993 the river width has increased from 28 m to 145 m (see Figure 5.2) while the slope has remained constant at around 0.02 (MacVicar, 1999). The constant slope indicates the fixed-channel-slope version of the model should be used.

Table 5.1 shows the reach-averaged channel geometry acquired from air photos (MacVicar, 1999).

Table 5.1. Slesse Creek channel geometry (MacVicar, 1999)

Year	1936	1973	1993
Width (m)	28	21	145
Sinuosity	1.15	1.12	1.06
Slope	0.019	0.020	0.021
Planform	Single thread, wandering	Single thread, wandering	Braided

The following list describes the method for determining input data, as discussed by MacVicar (1999). The input parameters used are shown in Table 5.2.

- Bankfull discharge (Q_{bf}) - Gauge records for Slesse Creek were used to determine the bankfull discharge. For the years 1993 and 1973, the short-term mean of flows was used and for 1936, the long-term mean was used.
- Flow resistance (k_s, n) - No roughness information was available for Slesse Creek. Since the Jarrett (1984) Equation calculates a value of Manning's n (see Equation 2.40) based on hydraulic radius and slope, this option was selected while using RiverMod.
- Sediment size (D_x) - A pebble count was taken, where $D_{50} = D_{50\ bank}$ and the sediment size was assumed to remain constant over the years because no historical particle size distribution information was available.
- Modified internal friction angle (ϕ') - For vegetated banks, ϕ_{1936} and ϕ_{1973} were determined by calibrating RiverMod. This was done by using the known channel geometry for each year and entering varying values of ϕ' until the modelled channel geometry matched the observed channel geometry. For unvegetated banks, $\phi_{1993} = 40°$ was used (Millar and Quick, 1993b).
- Channel slope (S) - S_{1993} was determined through field surveys. S_{1936} and S_{1973} were calculated using sinuosities determined from air photos and by comparison with S_{1993}.

Table 5.2. Input variables for Slesse Creek (adapted from MacVicar, 1999)

Variable	1936	1973	1993
Q_{bf} (m^3/s)	92	67	117
D_{35} (m)	0.11	0.11	0.11
D_{50} (m)	0.133	0.133	0.133
D_a (m)	0.166	0.166	0.166
D_{90} (m)	0.33	0.33	0.33
D_{50bank} (m)	0.133	0.133	0.133
ϕ' (°) calibrated using RiverMod	70	73	-
ϕ' (°) estimated	-	-	40
S	0.019	0.020	0.021

5.2.2 Analysis

The fixed-channel-slope model of RiverMod for rivers with non-cohesive bank sediment was used to analyze Slesse Creek. The selected sediment transport relation was the Einstein-Brown Equation (1950) and the selected flow resistance formula was the Jarrett Equation (1984).

The input values listed above were used to run several tests. All input values were held constant and ϕ' was varied between 40 and 90° in order to establish any influence on river width. This was done for 1936, 1973 and 1993 values. The results can be seen in Figure 5.4. For each of the three years, as ϕ', and therefore bank stability, decreases, the width of the river increases.

For 1993, the ϕ' value was extrapolated to less than 40° because ϕ'=40° resulted in a width of 119 m. The observed width for 1993 was 145 m, which according to RiverMod, corresponds to a ϕ' value of 35°. Figure 5.4 shows the observed geometry for each of the years. For the rest of the analysis, however, ϕ' = 40° was used, the generally accepted minimum value that applies to non-vegetated banks

In order to make restoration recommendations, ϕ' was plotted against width and depth using 1993 values (see Figure 5.5). The Millar (2000) meandering-braiding transition slope equation, was used to determine the value of ϕ' necessary to change the river morphology back to a meandering pattern from its present-day braided pattern. It is given by:

$$S^{*} = 0.0002 D_{50}^{0.61} \phi'^{1.75} Q_{bf}^{-0.25} \tag{5.1}$$

where S^* is the transitional slope, D_{50} is the median grain diameter, ϕ' is the modified internal friction angle and Q_{bf} is the bankfull discharge. Millar (2000) improved on the Parker (1976, as cited by Millar 2000) criterion (see Equation 2.49) by including the effects of bank stability.

D_{50bank} was also plotted against width and depth for the 1993 value (see Figure 5.6). Using the Millar (2000) meandering-braiding transition slope equation, the D_{50bank} necessary to change the stream back to meandering was determined.

5.2.3 Results

The RiverMod results show that the bank stability parameter (ϕ') has effectively decreased from 70° in 1936 to 40° in 1993. This decrease, caused by logging on the floodplain and banks, has led to channel widening. Figure 5.4 shows the variation of ϕ' with W for the bankfull discharge of each test year. River restoration could consist of increasing ϕ' and/or D_{50bank} to achieve a narrower, deeper, more stable, single-thread, channel.

Holding all other variables constant, including D_{50bank} at 0.133, it was established that increasing ϕ' to 57° would decrease the river width to 62.2 m. A similar analysis, which held ϕ' constant at 40°, determined that increasing D_{50bank} to 0.36 m would decrease the width to 47.8. These two width values, calculated using Equation 5.1, were established as the transitional width between a meandering and braided stream according to the above conditions. Either of these two strategies should change the river back to meandering, over time. Riparian planting and re-vegetation of the floodplain is a way of increasing ϕ' and therefore stabilizing the banks (MacVicar, 1999).

This analysis shows the influence the bank stability parameter, ϕ' and D_{50bank} have on W, assuming all other variables are held constant. Several errors are associated with this, since several variables including the sediment size distribution and roughness data were not available for past years. These values were assumed to be the same as the 1993 measured values.

The model predicts a change in planform from meandering in 1936 to braided in 1993. Since this has been determined from air photo analysis (MacVicar, 1999), RiverMod is a good predictor of planform geometry in the case of Slesse Creek and can successfully be used for restoration recommendations.

5.3 Narrowlake Creek

Narrowlake Creek is located in the central interior of British Columbia about 80 km southeast of Prince George (see Figure 5.7). It is a tributary to the Willow River and its floodplain was extensively logged in the 1960's and 1970's (Wilson, 2001). 35% of the watershed was harvested, including 80% of the riparian bank vegetation (Berry, 1996 as cited by Wilson *et al.*, 2001). Air photos from 1946 and 1997 (see Figure 5.8) clearly show that the morphology of Narrowlake Creek has changed from a meandering channel before logging to an almost braided channel after logging.

The change in Narrowlake Creek's planform geometry over time has been forced by logging on the banks. Accompanying the change to a "semi-braided" morphology is a marked change in channel width. Channel width has doubled from 1946 to 1997, increasing from 29 m to 58 m (Wilson, 2001).

Narrowlake Creek is similar to Slesse Creek in that both creeks have undergone a change in planform geometry following logging and disruption of the banks. However, according to Wilson (2001), Narrowlake Creek has experienced a much higher level of disturbance, including the cumulative impacts of road development, stream crossings, vegetation removal and channel avulsions. As a result, sediment supply and upstream conditions of Narrowlake Creek have changed. This is unlike Slesse Creek, where the upper watershed conditions are still pristine.

5.3.1 Investigation

In order to apply RiverMod to Narrowlake Creek, the input parameters must be defined for past and present values. The following analysis was conducted on treatment Reach 3 as described by Wilson (2001).

Historic aerial photographs have shown that from 1946-1997 the river width has increased from 29 m to 58 m while the slope has remained constant at around 0.007 (Wilson, 2001). The constant slope indicates the fixed-channel-slope version of the model should be used.

Table 5.3 shows the reach-averaged channel geometry acquired from air photos.

Table 5.3. Narrowlake Creek channel geometry.

Year	1946	1997
Width (m)	29	58
Slope	0.007	0.007
Planform	Single thread, wandering	Braided

The following list describes the method for determining input data, derived from Wilson (2001). The input parameters used are shown in Table 5.4.

- Bankfull discharge (Q_{bf}) - Gauge records do not exist for Narrowlake Creek. Using gauge information from the Willow River, which Narrowlake Creek flows into, an interpolation was performed in order to determine the bankfull discharge (Personal Communication, Andrew Wilson, 2002).

- Flow resistance (k_s, n) - No roughness information was available for Narrowlake Creek. Since the Jarrett (1984) Equation calculates a value of Manning's n (see Equation 2.40) based on hydraulic radius and slope, this option was selected while using RiverMod.
- Sediment size (D_x) - Sediment size information was only available for 1997. With no historical sediment size data, it was assumed that $D_{1946} = D_{1997}$. Values for both D_{50} and D_{50bank}, were obtained.
- Modified internal friction angle (ϕ') - For vegetated banks, ϕ_{1946} was determined by calibrating RiverMod. This was done by using the model inputs along with the known channel geometry for 1946 and entering varying values of ϕ' until the modelled channel geometry matched the observed channel geometry. For unvegetated banks, $\phi_{1997} = 40°$ was used (Millar and Quick, 1993b).
- Channel slope (S) - S_{1997} was measured through field surveys. S_{1946} was determined using air photos and by comparison with S_{1997}.

Table 5.4. Input variables for Narrowlake Creek (adapted from Wilson, 2001)

Variable	1946	1997
Q_{bf} (m^3/s)	31.1	31.1
D_{35} (m)	0.055	0.055
D_{50} (m)	0.058	0.058
D_a (m)	0.093	0.093
D_{90} (m)	0.088	0.088
D_{50bank} (m)	0.047	0.047
ϕ' (°) calibrated using RiverMod	55	

ϕ' (°) estimated		40
S	0.007	0.007

5.3.2 Analysis

The fixed-channel-slope model for rivers with non-cohesive bank sediment of RiverMod was used to analyze Narrowlake Creek. The selected sediment transport equation was the Einstein-Brown (1950) and the selected flow resistance equation was Jarrett (1984).

The values listed in Table 5.4 were input to RiverMod. ϕ' was varied between 40° and 90° to determine its influence on river width, for 1946 and 1997 data. The results can be seen in Figure 5.8. As ϕ', and therefore bank stability decreases, the width of the river increases.

For 1997, the ϕ' value was extrapolated beyond 40° because 40° resulted in a width of 50.8 m. This is a reasonable approximation of the observed W_{1997} of 58 m. Figure 5.9 shows markers for the observed geometry for each of the curves.

In order to make restoration recommendations, ϕ' was plotted against width and depth for the 1997 values (see Figure 5.10). The Millar (2000) meandering-braiding transition formulation was used to determine the value of ϕ' necessary to change the river morphology back to a meandering pattern from its present-day braided pattern.

D_{50bank} was plotted against width and depth for the 1993 value (see Figure 5.11). Using the Millar (2000) meandering-braiding transition formulation, the D_{50bank} necessary to restore the creek's meandering planform was determined.

5.3.3 Results

RiverMod predicts a meandering channel for 1997. Although the present channel is no longer meandering, it is not quite braided either. Since 1946, the transition slope has been approaching the limit between meandering and braided rivers. The transition slope may approximate conditions necessary for the gradual change from one planform to another.

In Wilson (2001) it is assumed that $\phi' = 70°$ for 1946 and $\phi' = 40°$ for 1997, but RiverMod calibration finds $\phi' = 55°$ for 1946. These refined values give the same results as those obtained by Wilson (2001) which show the creek to be meandering in both years, when actually it is close to braided in 1997. Once calibrated, the model still shows a trend towards a braided river.

According to what can be seen from air photos and field surveys, the river has begun to change from meandering to braided over the 30 year period between 1946 and 1997, primarily due to logging activity. Although RiverMod does not show this same result, the trend towards a braided river is evident.

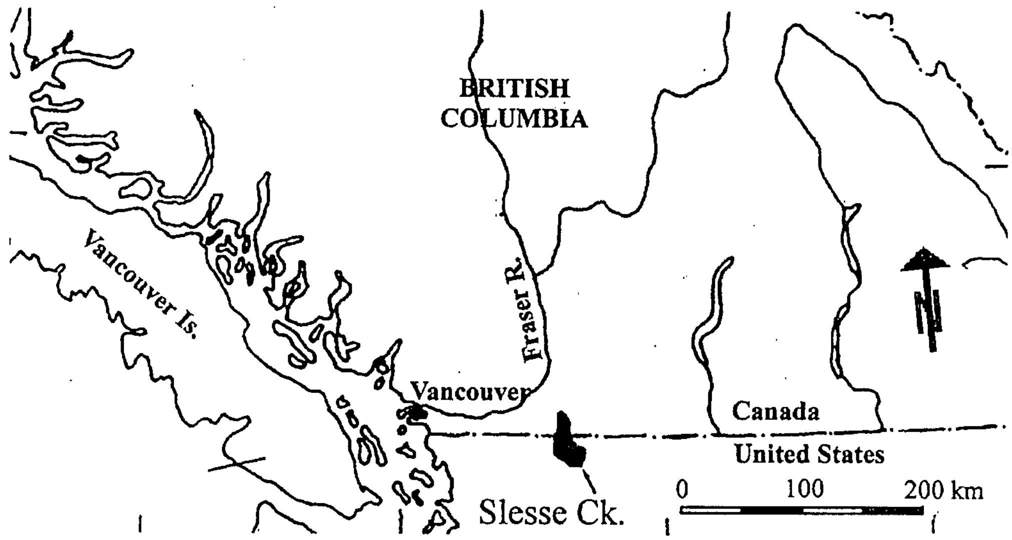
Since the modified friction angle (ϕ') has decreased due to logging and has led to channel widening, any river restoration program should consist of increasing ϕ' and/or D_{50bank} in order to stabilize the banks. These two methods would aid in the development of a narrower, deeper, more stable, single-thread, channel. ϕ' could be increased by riparian planting and re-vegetation of the floodplain. D_{50bank} could be increased by the addition of larger, stabilizing gravel along the banks of the river.

Since RiverMod did not accurately predict the change in planform for Narrowlake Creek, credible values of ϕ' and D_{50bank} were not obtained. According to results obtained by using RiverMod, the bank stability values at the meandering-braiding transition are $\phi' = 34^{\circ}$ and $D_{50bank} = 0.035$ m. These values are lower than the 1997 values, and would most likely decrease the bank stability further, if used in restoration planning.

Errors contributing to this could be the lack of historical sediment size data and roughness values. Since no gauge exists on Narrowlake Creek, the bankfull discharge was extrapolated, and therefore may not be the true bankfull discharge. The level of disturbance on Narrowlake Creek is much higher than that of Slesse Creek (Wilson, 2001). This could account for the higher degree of success in the application of RiverMod to Slesse Creek over Narrowlake Creek. The higher level of disturbance on Narrowlake Creek could also signify a larger level of uncertainty associated with input data. It could also mean that there are more factors influencing the decrease in bank stability than RiverMod can account for.

The model should predict a change in planform from meandering in 1946 to braided in 1997, since this has been determined from air photo analysis (Wilson, 2001), RiverMod is a good predictor of planform geometry in the case of Slesse Creek and has provided useful information in the case of Narrowlake Creek.

(a)



(b)

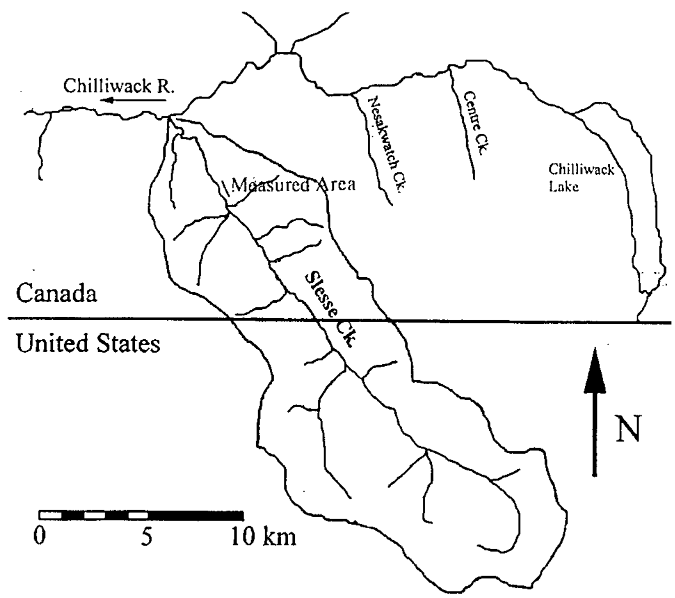


Figure 5.1. (a) Location of Slesse Creek Watershed; (b) Slesse Creek Watershed (MacVicar, 1999)

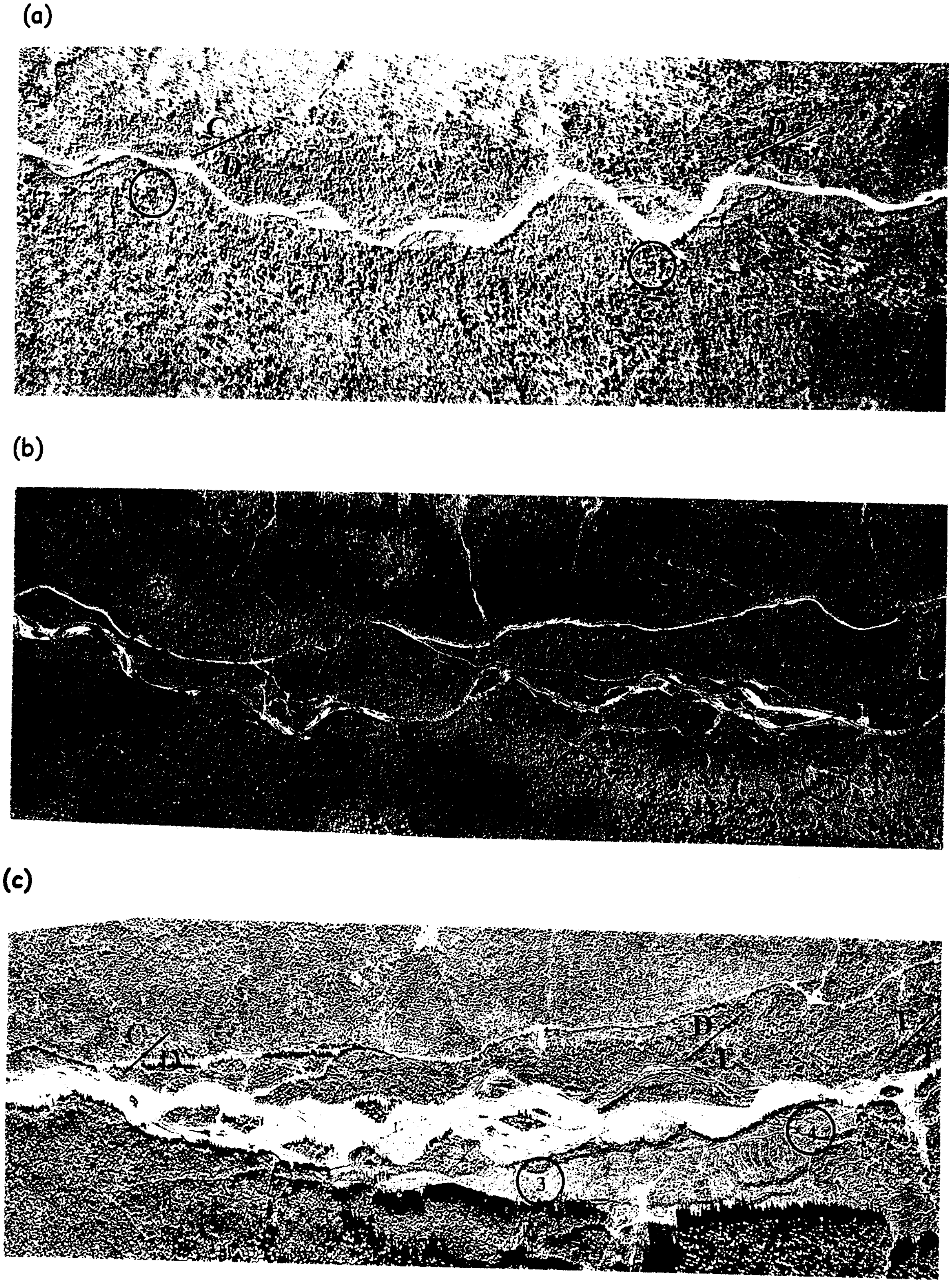


Figure 5.2. Airphotos of Slesse Creek: (a) 1936, 1:22000 scale; (b) 1973, 1:19050 scale; (c) 1993, 1:17650 scale (MacVicar, 1999).

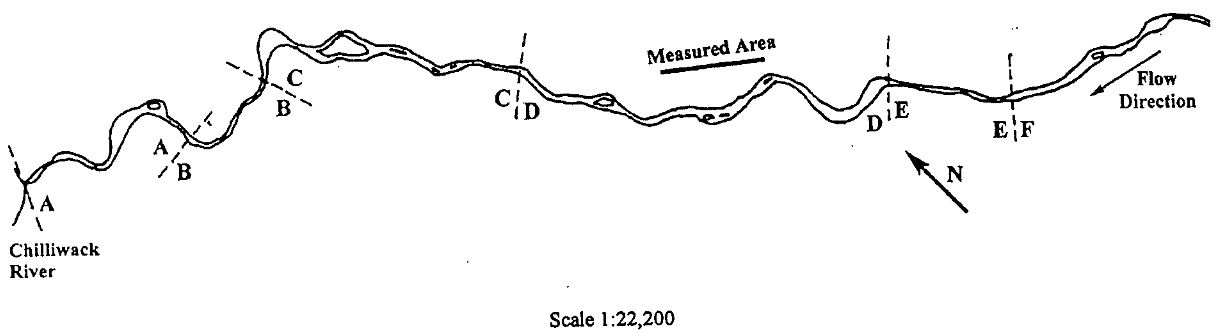


Figure 5.3. Slesse Creek lower watershed, showing study reach D in 1936 (MacVicar, 1999).

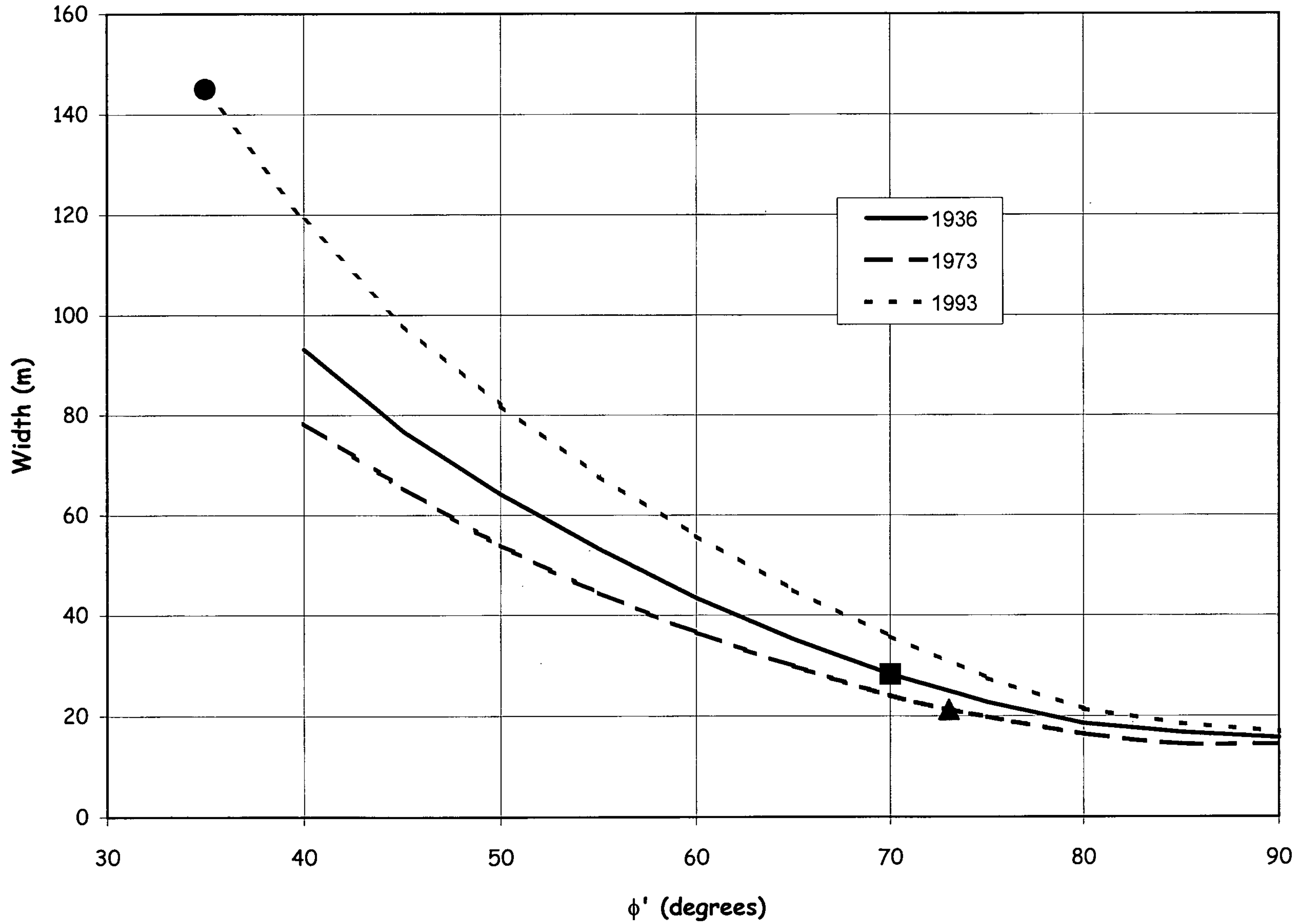


Figure 5.4. Slesse Creek calibration, width vs. modified friction angle. Symbols show actual calibrated geometry for each year. Circle = 1993; Triangle = 1973; Square = 1936.

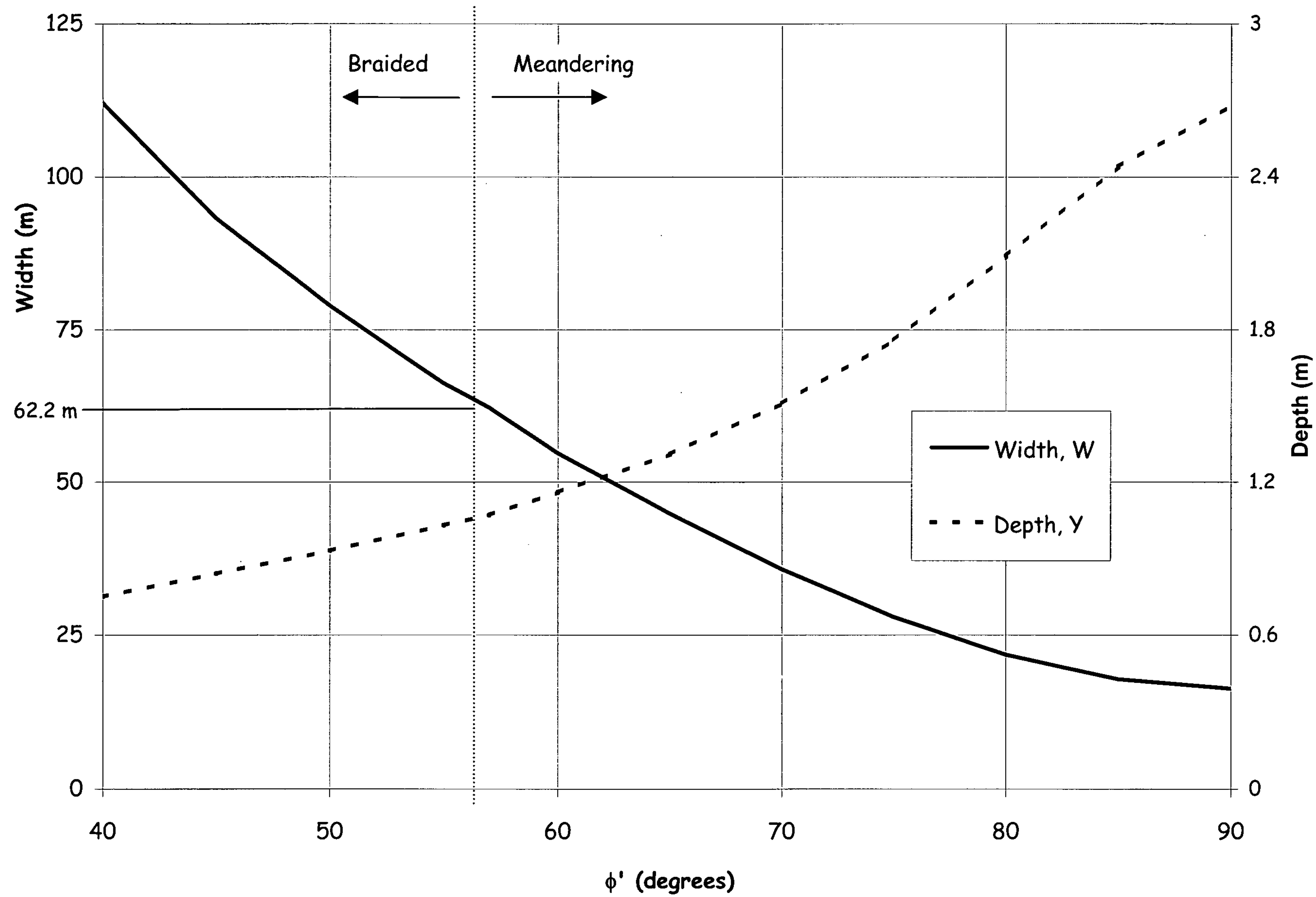


Figure 5.5. Restoration of Slesse Creek, altering ϕ' (D50bank = 0.133m).

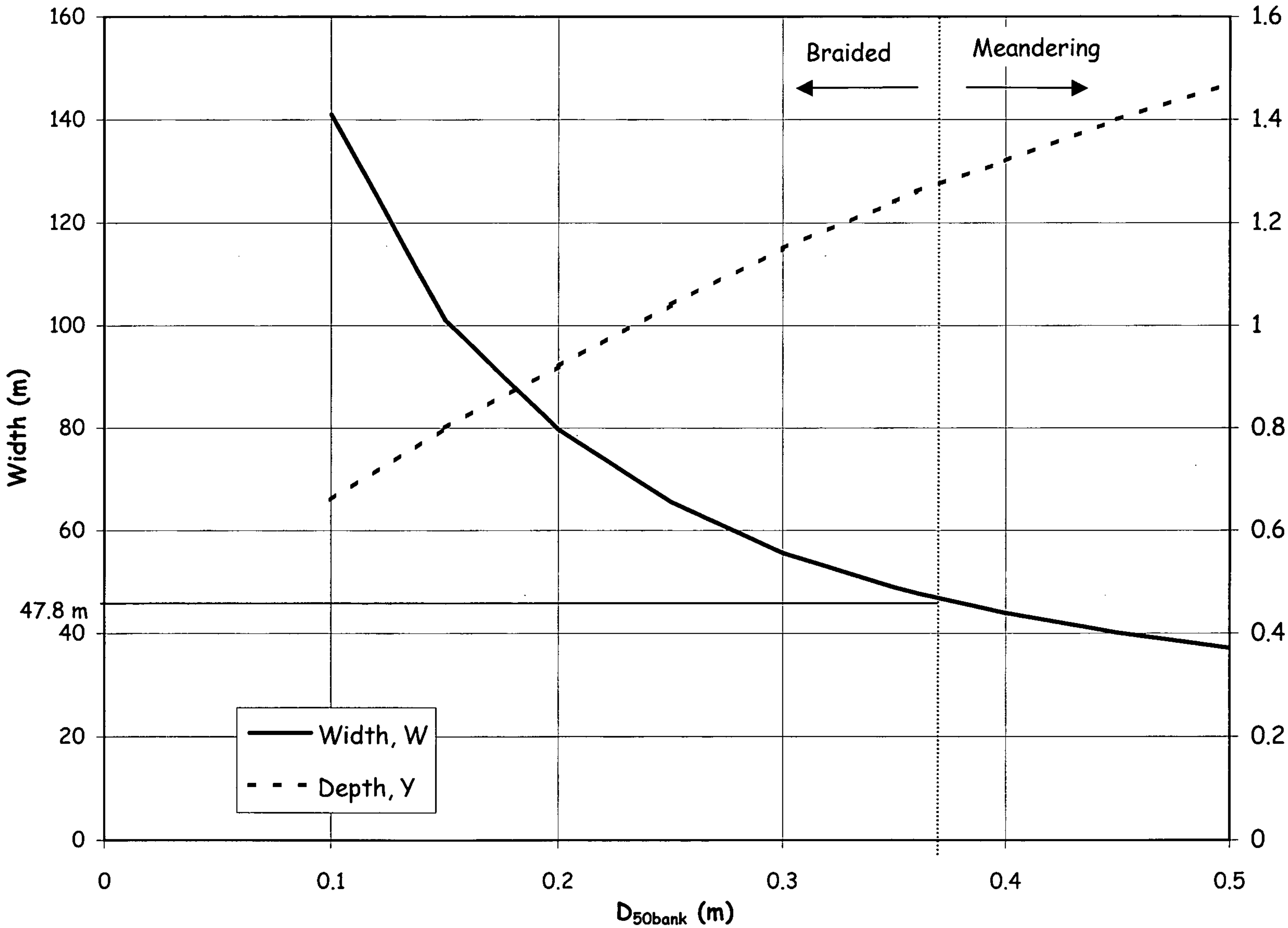


Figure 5.6. Restoration of Slesse Creek, altering D_{50bank} (ϕ' = 40°).

Figure 5.7. Location of Narrowlake Creek Watershed

(a) (b)

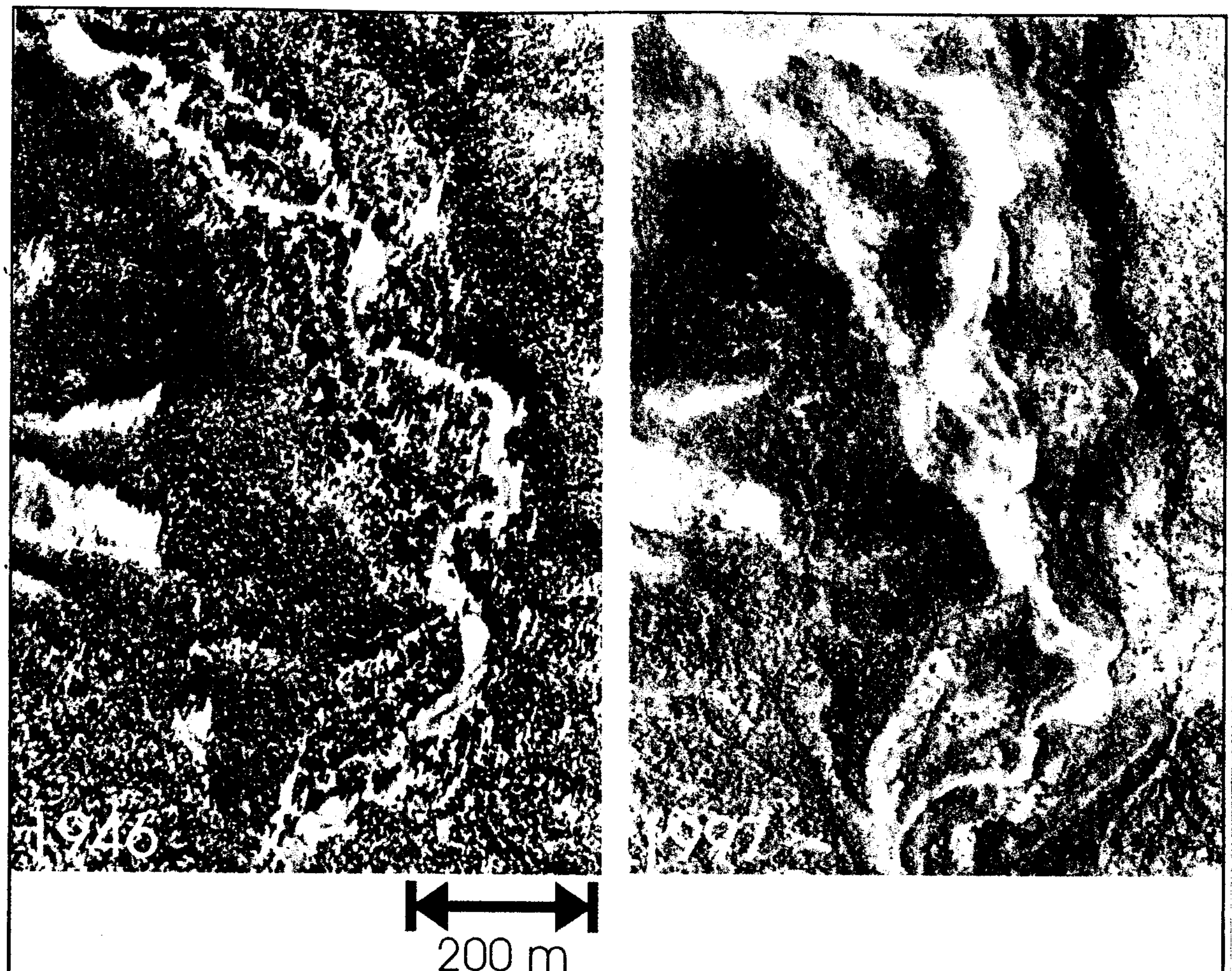


Figure 5.8. Airphotos of Narrowlake Creek: (a) 1946; (b) 1997 (Wilson, 2001).

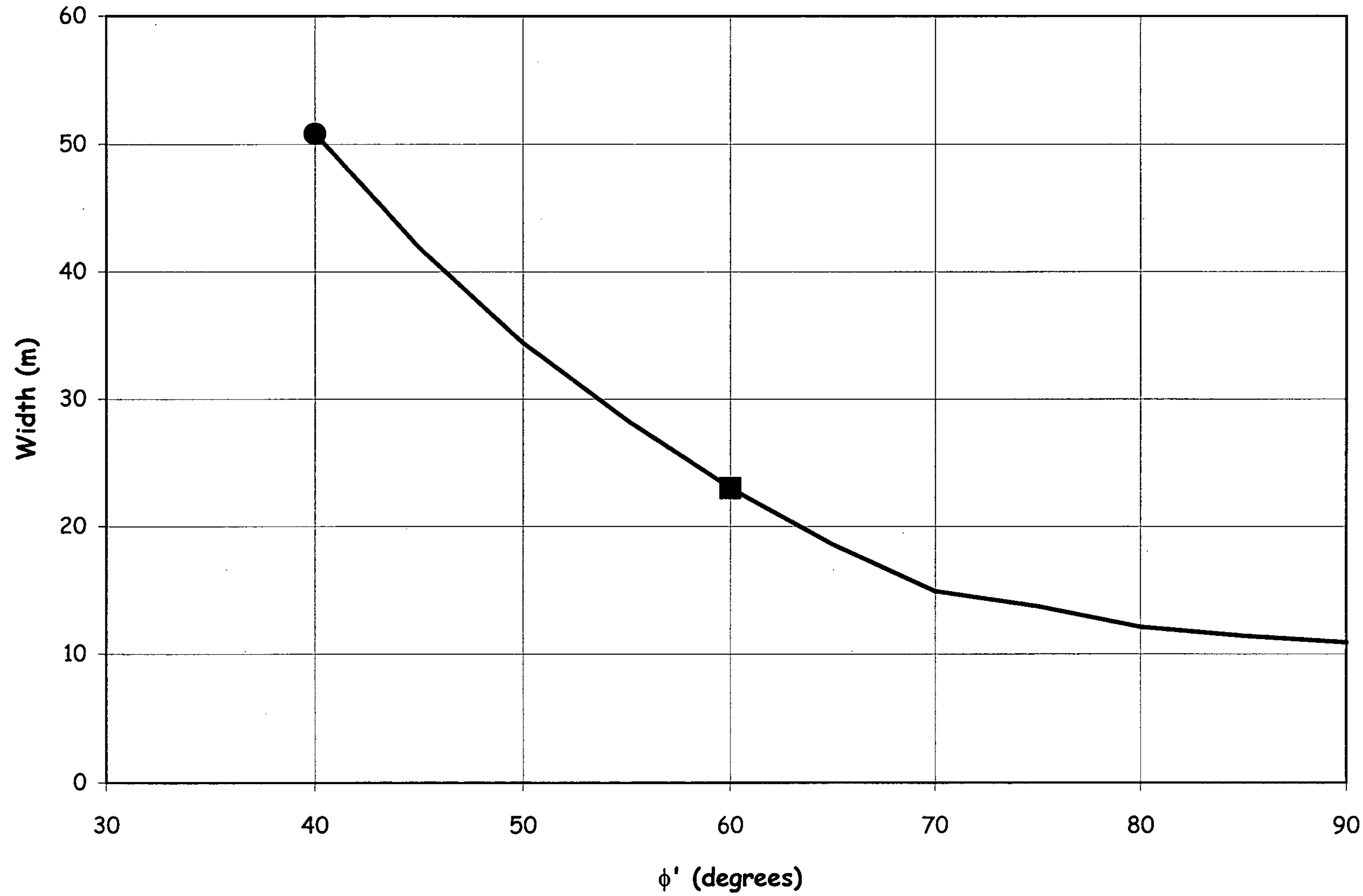


Figure 5.9. Narrowlake Creek calibration, width vs. modified friction angle. Symbols show actual calibrated geometry. Circle = 1997; Square = 1946.

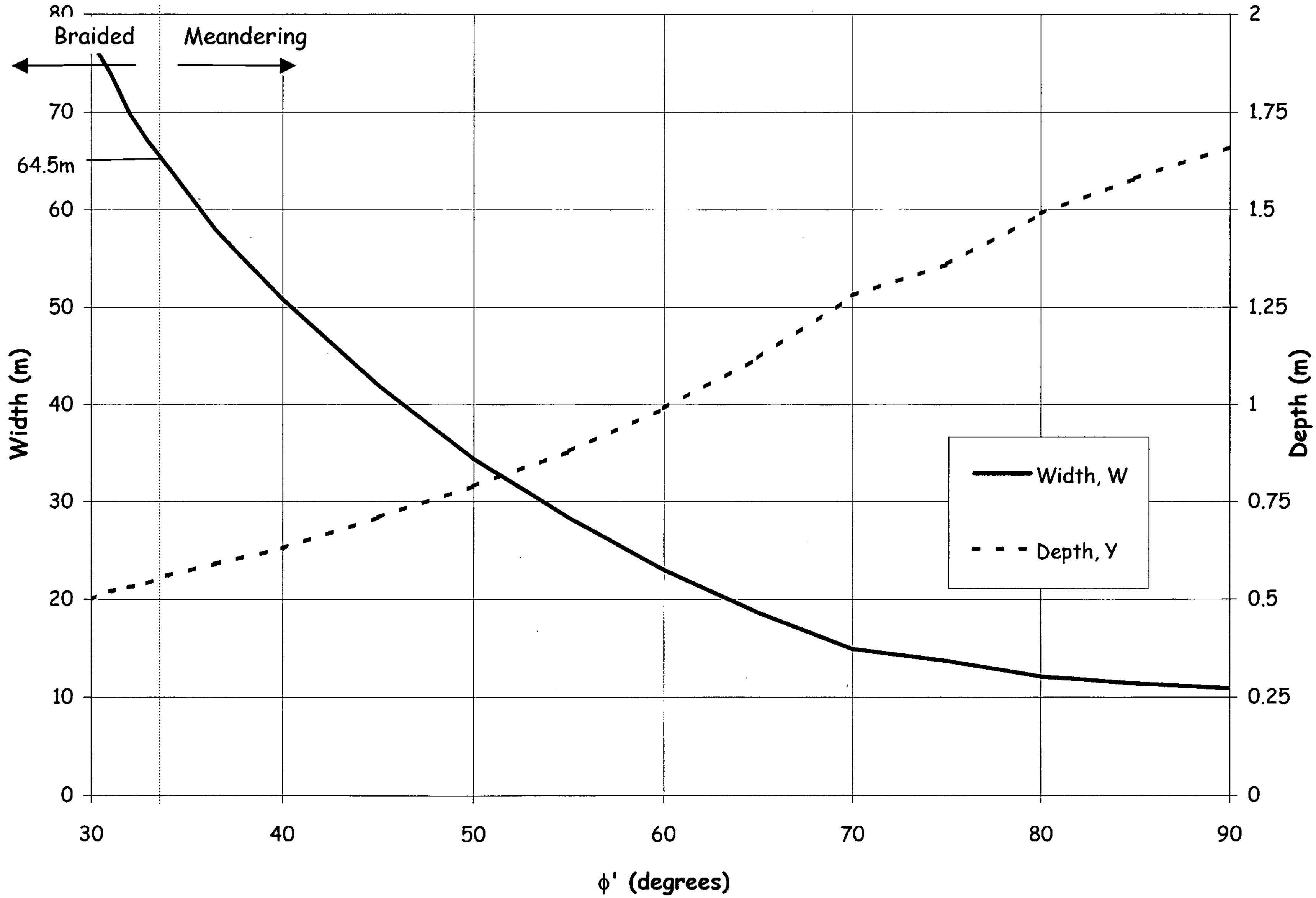


Figure 5.10. Restoration of Narrowlake Creek, altering ϕ' (D50bank = 0.047m).

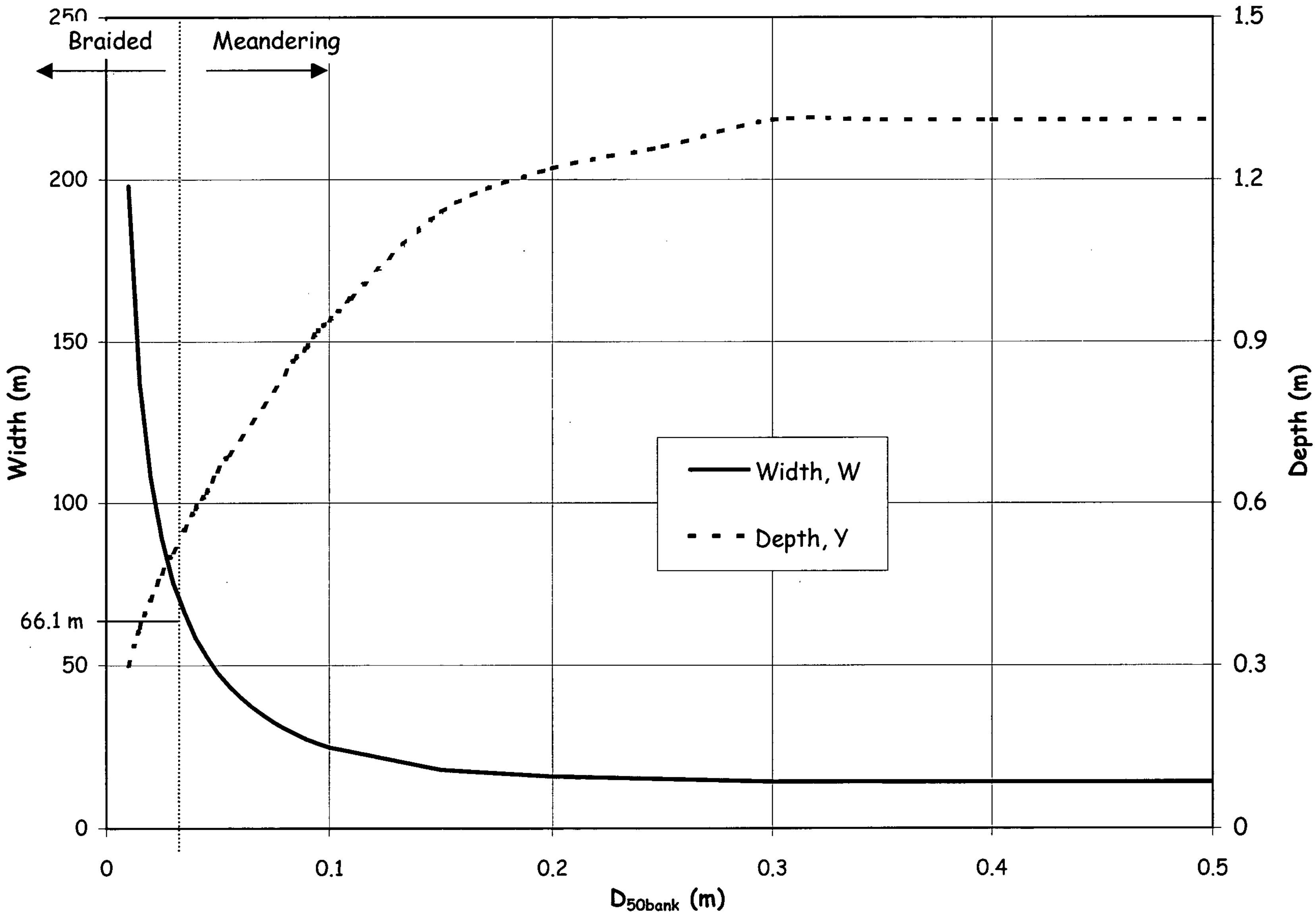


Figure 5.11. Restoration of Narrowlake Creek, altering D_{50bank} (ϕ' = 40°).

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