Comparing different methods of bed shear stress estimates in complex flow fields; the case of Megech River, Ethiopia

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Abstract

Bed shear stress is a fundamental variable in river studies to link flow conditions to sediment transport. It is, however, difficult to estimate this variable accurately, particularly in complex flow fields. This study compares shear stress estimated from the log profile, the depth-slope product and outputs from a two-dimensional hydraulic model. Vertical velocity profile observations from Megech River (one of the main water sources flowing into Lake Tana, upper Blue Nile Basin, Ethiopia) using SEBA Mini current meter M1attached with SEBA Signal counter Z6-SEBA HAD under typical field conditions are used to evaluate the precision of different methods for estimating local boundary shear stress from velocity measurements. A comparison of the shear stress distributions derived using the two-dimensional hydraulic model and with those estimated using the 1D reach-averaged equation (i.e. the depth-slope product) shows a close correspondence. Mean shear stresses determined using local depth and mean channel slope are roughly comparable to those determined for the same data using local predictions of both depth and energy slope. As the overall mean shear stress provides a useful index of flow strength, this comparison suggests a good level of confidence in using the reach averaged one-dimensional equation, for which data can easily be collected from cross sectional surveys. However, the variance of the modelled shear stress distribution shows some differences to that calculated using the mean channel slope. Although such numerical models are limited to channel types adhering to model assumptions and yield predictions only accurate approximately 20–30%, they can provide a useful tool for river-rehabilitation design and assessment, including spatially diverse habitat heterogeneity and sediment transport studies.

Key words: shear stress, hydraulic model, velocity profile

1. INTRODUCTION

Shear stress is an important parameter in hydraulics and river engineering which provides an index of fluid force per unit area on the stream bed and is related to sediment transport and deposition in many theoretical and empirical treatments of sediment transport. The rate of sediment transport in a channel depends on the shear stress τ. But for typical range of flows in most rivers the shear stress τ rarely exceeds the critical value τ_c for initiation of sediment transport and for flows for which shear stress exceeds τ_c most transport models shows that the relationship between τ and transport is strongly non-linear. Due to this non-linearity, a small error in τ can lead to very large errors in estimated transport rate. Moreover, it is only the portion of the total shear stress called the grain stress which acts on the movable grains to produce transport. Estimation of this portion of the stress is possible but approximate. A further thing which makes sediment transport prediction complicated is the spatial variability in shear stress. Shear stress tends to vary across and along the channel. Although the total shear stress on a cross section can be determined, prediction of transport based on this average value is inaccurate as shear stress tends to vary laterally even in straight reaches with relatively simple cross sections (Carson and Griffiths, 1987;Paola, 1996;Nicholas, 2000;Ferguson, 2003;Wilcock, 1996).

Velocity profiles are often used as an indirect method to determine mean boundary shear stress in natural rivers (Wilcock 1996). While several methods are available to determine the time averaged local boundary shear stress (e.g. Biron et al. 2004, Dietrich & Whiting 1989), this study employs the theoretical log-law, the simplifying cross-section averaged one-dimensional hydraulic equations and prediction from a 2D hydraulic model. Based on the field velocity and flow depth measurements, average shear stresses for natural streams were investigated. The differences between one dimensional approach (depth-slope product), outputs from 2D hydraulic model and shear stress calculated from vertical velocity profile measurements were examined.

2. FIELD WORK SITE

The Megech River, which is about 75 km long, has a drainage area of about 850 km^2 and an average annual discharge of 11.1 m^3/s. This river is one of the main water sources flowing into Lake Tana from the north. Lake Tana represents a major hydrological system in the upper Blue Nile Basin and it is the largest lake in Ethiopia with a surface area of 3012 km^2 and a volume of 28 km^3 at its long term mean elevation of 1786 m.a.s.l. The highest elevation of the Megech watershed is around 2991 m above mean sea level, in its north eastern part. Four major tributaries join the Megech River: two from the right bank and the other two from the left. The Megech catchment is characterized as mountainous, wedge-shaped with mean catchment slope of (3.2%). The catchment of the Megech River is highly vulnerable to sheet, rill and gully erosions. During a field visit made in 2006 by groups of professionals responsible for the design of the Megech Dam, it was observed that new gullies which directly ran into the Megech River were being formed as a result of the increased agricultural activities performed in the catchment such as cultivation of steep slopes steep area farming and intensive grazing. The proposed new dam currently under construction in Megech River is located southeast of Azezo and Tewodros airport about 3 km to the right of Gondar-Bahir Dar main road and Megech River crossing.

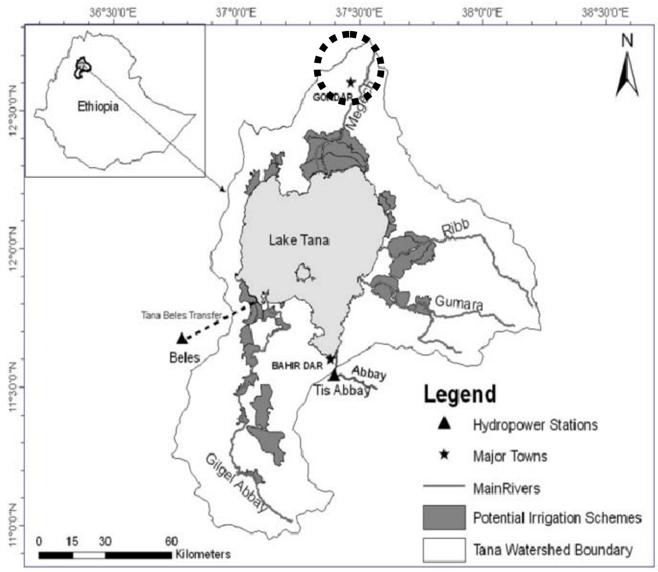

Figure 1: Location map of the Lake Tana catchment showing major inflowing Rivers including Megech River; Fieldwork site is circled by dashed line. (Taken from Megech Dam Feasibility Report, Volume 1)

It was evident during the fieldwork visit that the Megech River in general and the study reach in particular are also characterized by serious bank erosion and mass movement. The eroded and transported sediment from the upstream highlands, bed and banks of the river ultimately reaches Lake Tana. Although an estimate is not available and research related to sediment transport and rates of bank erosion have not yet been carried out for River Megech, there is visual evidence that the contribution to Lake Tana sedimentation may be significant. On the other hand, farm landowners complain of bank erosion and channel shifting, which cause the loss of some parts of their valuable land. The bank erosion problem seems to be triggered by unmanaged in-stream gravel mining in some parts of the river. The mining activity on the right bank of the river appears to lower the bottom of the channel, making the bank almost vertical and unstable. Siltation in the downstream reaches of the major rivers including Megech is one of the causes of overbank flow and flooding (SMEC, 2008a).

3. FIELD DATA COLLECTION

In order to be able to employ a hydraulic model to simulate flow within the surveyed section of the Megech River, a brief but intensive field measurement campaign was undertaken. The surveyed reach is approximately 102.5m long and 34m wide and is bounded on the true left by a steep sloped bank. There is no clearly defined river bank on the true right of the river (Figure 2). Reach topography was surveyed in detail using Leica TPS1200+ total station and more than 2000 elevation points were recorded in a regular grid of 0.5m lateral and 2.5m longitudinal resolution. A digital elevation model was constructed from these data for use by the hydraulic model (Figure 2).

4. VELOCITY & DEPTH MEASUREMENT

Velocity and depth data provide a basis for evaluating how well the hydraulic model predicts flow conditions for various flow patterns. Therefore, field observation of depth and velocity were made at 0.5m interval across ten cross sections for depth and four cross sections for velocity. In all cases, flow depth was measured using stadia rod to a resolution of +/-1cm and point velocity was measured using SEBA Mini current meter M1attached with SEBA Signal counter Z6-SEBA HAD with a time measurement accuracy of 0.01s and impulse frequency of 40 impulse/s. Despite the small diameter of the propeller (50 mm), its physical dimensions limited the resolution of the profile and constrained the number of points obtainable in some areas of low water depth. A minimum of 3 and maximum of 9 measurements per velocity profile were obtained by varying elevation above the bed at regular intervals of 0.1Y to 0.9Y, where Y is the flow depth. A 9 mm diameter depth setting wading rod was used to position the propeller and a base plate was attached with the rod for resting on the river bed. The 10 flow depth measurement cross sections were placed in such away that they passed through different types of flow features within the channel (e.g. pools and riffles). Two cross sections from the upstream part of the reach and eight cross sections from the downstream section were chosen.

The section at the inlet of the reach is divided into a number of smaller segments. Local depth Y_i measured directly and local mean velocity U_i taken to be the velocity measured at 0.4Y above the bed were used to compute the discharge. Discharge through each segment is estimated by:

$$q_i = Y_i * U_i \tag{1}$$

and the total discharge is estimated by:

$$Q = \sum q_i * b_i \tag{2}$$

Where b is measured width of the channel. The discharge at the time of measurement was 0.35m^3 s^{-1}.

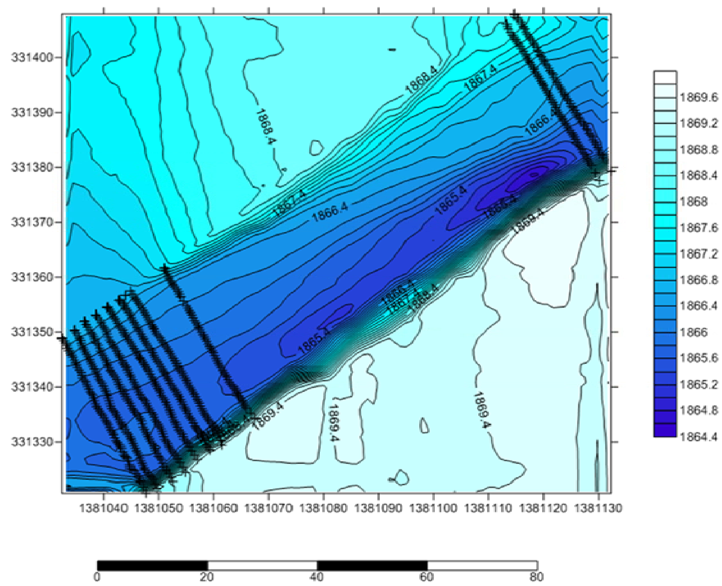

Figure 2: Topographic map of the study area with measurement locations.

5. THE TWO-DIMENSIONAL HYDRODYNAMIC MODEL

The model used is a two-dimensional hydraulic model which solves the depth-averaged shallow-water form of the Navier-Stokes equation in conservative form. The model uses a Godunov-type finite volume method. This method balances all the fluxes entering and leaving each cell, using explicit time integration. Accuracy of second-order is attained through variable extrapolation approach based on van Leer's Monotonic Upstream Schemes for Conservation Laws (MUSCL) (Van Leer, 1977) and a simple but robust approximate HLL solver due to (Harten et al., 1983). Modern finite volume schemes of the Godunov type achieve higher than first-order accuracy by reconstructing the cell interface data from cell centre values and using flux or slope limiters to preserve monotonicity (Mingham and Causon, 1998). The model equations can be expressed in conservative law form as:

$$\partial U/\partial t + \partial F/\partial x + \partial G/\partial y = S \quad (3)$$

Where:

$$U = \begin{pmatrix} \Phi \\ \Phi u \\ \Phi v \end{pmatrix}; \qquad F = \begin{pmatrix} \Phi u \\ \Phi u^2 + \Phi^2/2 \\ \Phi uv \end{pmatrix}; \qquad G = \begin{pmatrix} \Phi v \\ \Phi uv \\ \Phi v^2 + \Phi^2/2 \end{pmatrix}$$

And where $\Phi = gh; h = \text{water depth}$; g= acceleration due to gravity; u and v = depth averaged velocity components in the x- and y- directions; **F** and **G** = convective fluxes in the x- and y- direction; and **S** = vector of source terms, which would normally include bed slope, friction losses, and Coriolis forces as well as turbulent transport effects.

The assumptions made in deriving these equations and within the solution procedure make the model applicable to steady or unsteady flow simulations. Flow regimes can also be subcritical or supercritical as well as gradually varying or discontinuous flow. The data required to run the model consists mainly of topographic data (DEM) describing the channel geometry, boundary conditions, channel-bed roughness coefficients (Chezy) and turbulent diffusivity constant. Bathymetry data were collected in the form of XYZ coordinates at 2.5 m stream-wise and 0.5 m lateral resolution. The typical means of applying boundary conditions in the model is to define the discharge at the upstream boundary, which is represented with a piecewise linear hydrograph, and the slope at the downstream boundary. So, at the upstream reach of the channel the total discharge was specified as an input to the model. The downstream boundary condition in the model is set equal to the average bed slope for the channel as a whole.

To avoid computational difficulties associated with very shallow flow, the model requires a minimum flow depth for momentum calculations to be specified. Based on this minimum depth the model handles the problem of some areas being wet while others are dry and transitions between the two. In this study the effect of the minimum flow depth for momentum calculations is also investigated by varying its value in the simulation and fixing other variables that influence model output like roughness parameters R_c and R_d. The model used in this study uses a Keulegan type roughness estimator of the form:

$$C = R_c\left[\log\left(Y/R_d\right)\right] \quad \textbf{(4)}$$

Where R_d is the effective roughness height, a value of which is usually required when the Keulegan or Strickler equation is specified and R_c is calibration parameter. It is a better descriptor of resistance than indices such as Manning's n because it tends to remain constant over a wider range of flow depths than does n (Steffler and Blackburn, 2002). There is no table of generally accepted values for the effective roughness as there are for Manning's n values. For resistance due primarily to bed material roughness, a starting estimate of this parameter can be taken as 1-3 times the largest grain diameter (Steffler and Blackburn, 2002) and calibration to observed water surface elevations gives the final values. In this study R_d was initially set to be equal to the minimum flow depth for momentum calculations.This is done so as to get a smooth, continuous and non-negative resistance value for any flow depth. R_d values greater than the minimum flow depth set in the model for momentum calculations would result in negative roughness at some locations where flow depth is shallower, which is unrealistic. R_c is used as a calibration constant. A number of model runs were conducted, each using a different value of R_c and R_d. Depth predictions for each model run were compared with measurements of flow depth across different cross sections within the channel. On the basis of this comparison between measured and predicted flow depths, optimal values of R_c and R_d were identified.

The initial condition for all model runs was a dry channel bed with total water discharge specified at the upstream model boundary. The flow pattern in the channel was allowed to develop until a steady-state solution was obtained, which takes in the order of 7 to 8 hours. A final solution was considered to have been obtained when the inflow and outflow discharges had been equal to each other for at least 8 hours.

6. MEASURED VELOCITY PROFILES

The vertical velocity profiles for steady, uniform, subcritical flow in a wide, straight channel with total roughness dominated by skin friction may be logarithmic over the whole flow depth (Wilcock, 1996). The logarithmic relation is the most widely used (Wilcock, 1996;Robert, 1997;Lawless and Robert, 2001) and models local bed shear stress by relating shear velocity and average velocity with height (Schlichting and Gersten, 2000). It can be used to map spatial patterns of shear stress and roughness height at sub-reach scale. It is based on the assumption that the velocity profile in the lower portion (15-20%) of an open channel flow has a logarithmic structure. However, it is reported to be not valid for complex flows as the velocity profile may not be logarithmic (Biron *et al.*, 2004). Previous researchers (Wilkinson, 1984;Whiting and Dietrich, 1989;Williams, 1995) all observed uncertainties in fitting a logarithmic profile to velocity data. Moreover considerable deviation of velocity profiles from the logarithmic was observed in shallow flows, having a variation near the tops of the roughness elements (Wiberg and Smith, 1991). Errors in measurement of flow velocity and height above the bed can highly influence the results.

To assess this, measured velocities within the study reach with a known bed material size were inspected for their shape and fitted with the logarithmic function:

$$u/u_* = 1/\kappa \, ln[z/z_o] \quad \textbf{(5)}$$

Where u is the flow velocity and u_* is the shear velocity, which is computed by dividing the bed shear stress by fluid density under square root $(\tau_o/\rho)^{0.5}$, z is the height above the bed, κ is the von Karman's constant (taken to be 0.4) and z_o is the characteristic roughness height (height above bed where velocity goes to zero). The above equation only applies to the bottom 20 percent of the water column, above a near-bed roughness region and below an outer turbulent region (Wilcock, 1996).

Most vertical velocity profiles measured in the Megech River had highly correlative fits with a logarithmic profile over the whole flow depth (Figure 3 to 6). However, Individual logarithmic profiles were significantly different in velocity magnitudes and velocity gradients as a function of depth. The deepest areas near the true left bank of the upstream section (cross sections 1 and 2) had more points in their velocity profiles allowing better resolution of the flow regions. It should be noted that due to short sampling times and the location of some velocity measuring sites in the vortex shading zone downstream of big boulders, measured velocities showed high fluctuations over short lateral distances.

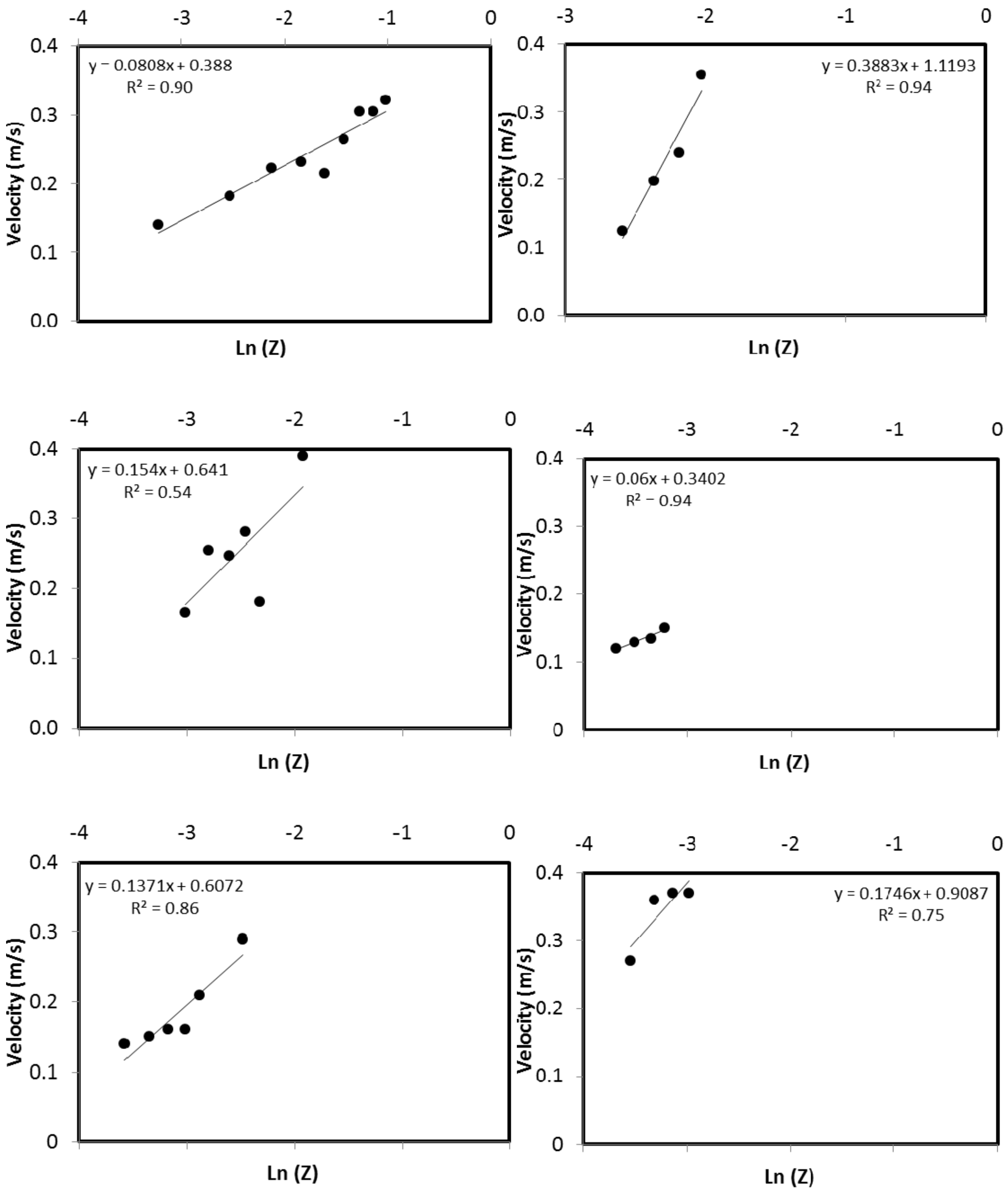

Figure 3: Vertical velocity profiles for cross section 1

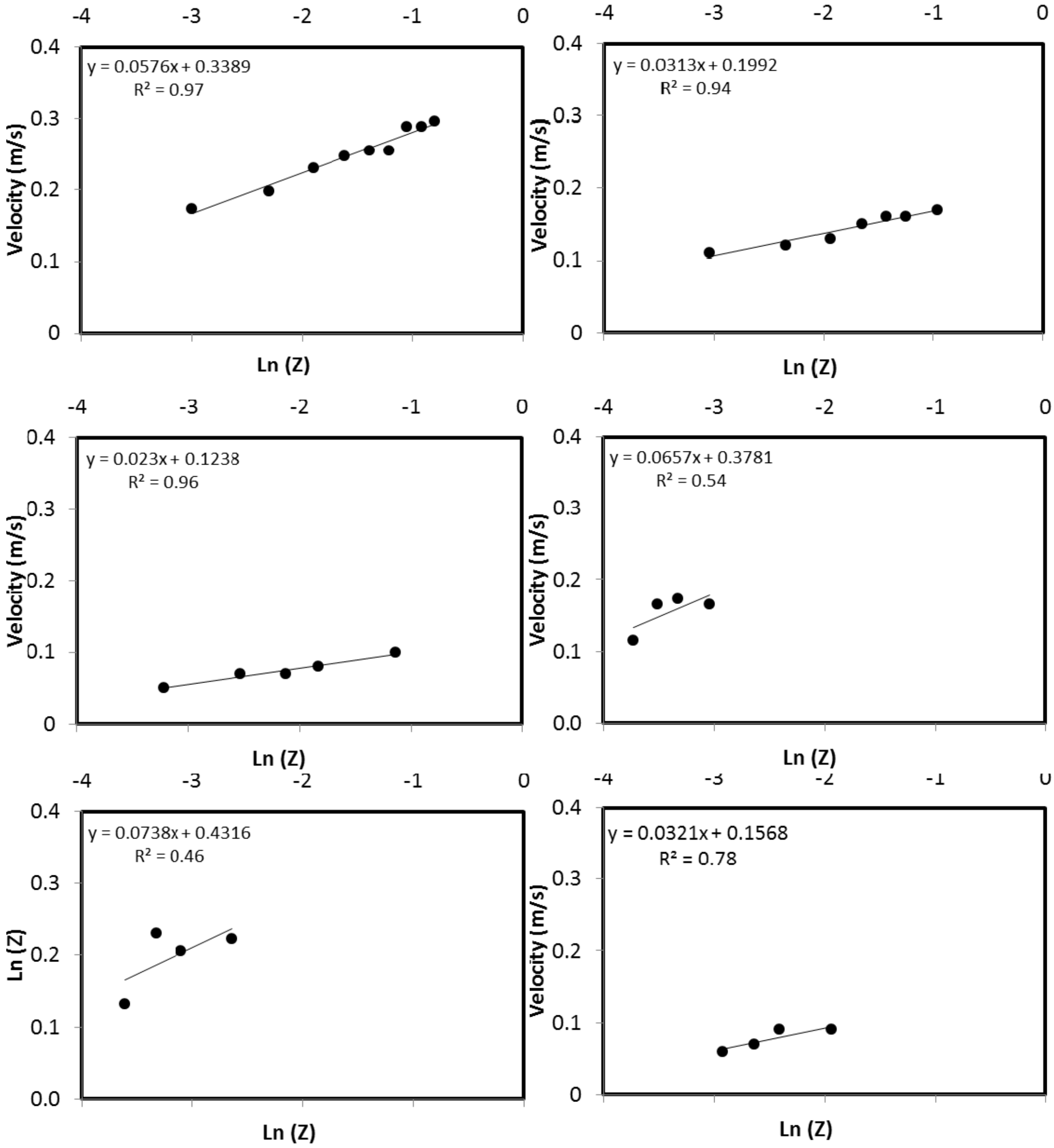

Figure 4: Vertical velocity profile for cross section 2

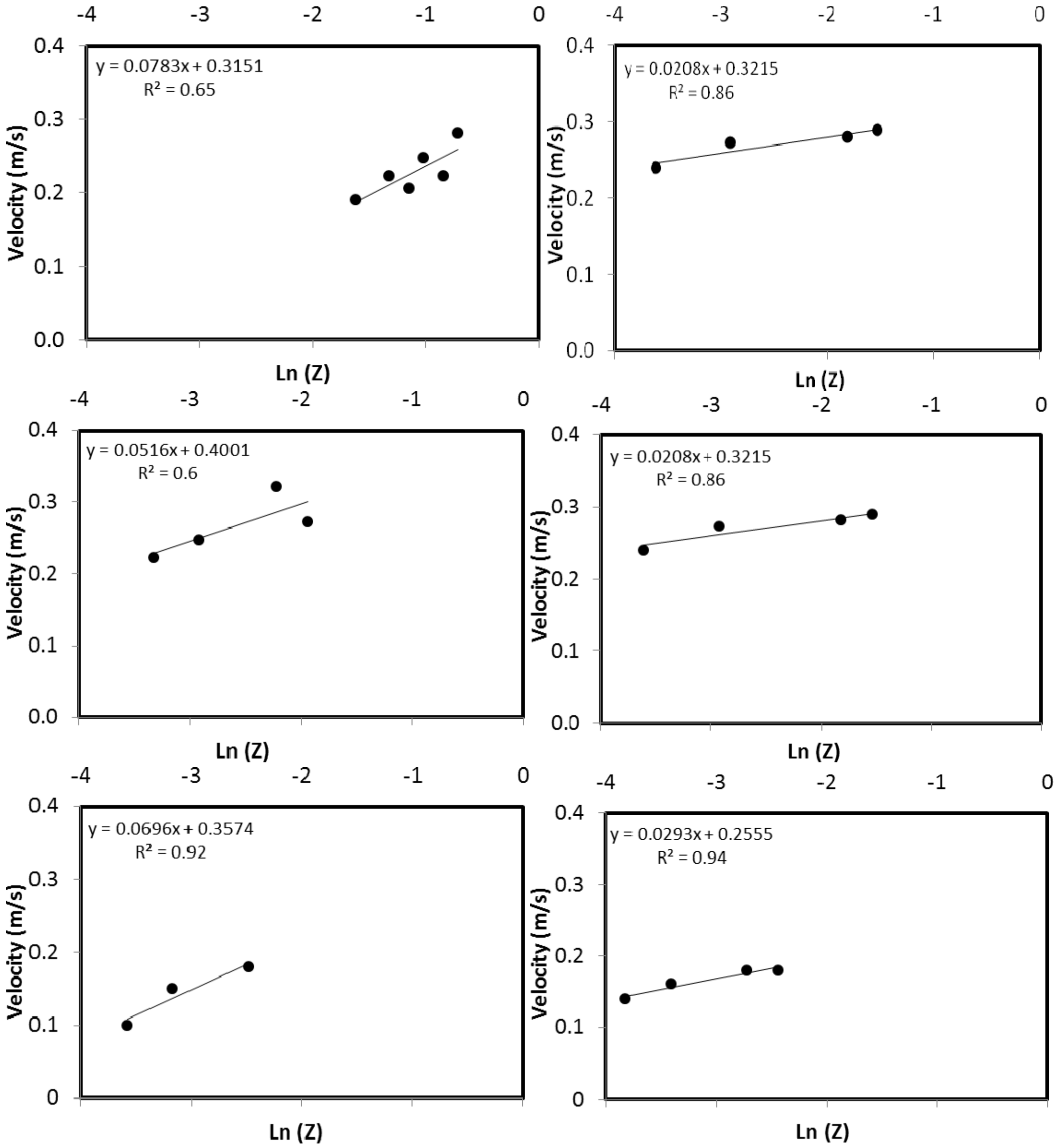

Figure 5: Vertical velocity profile for cross section 42

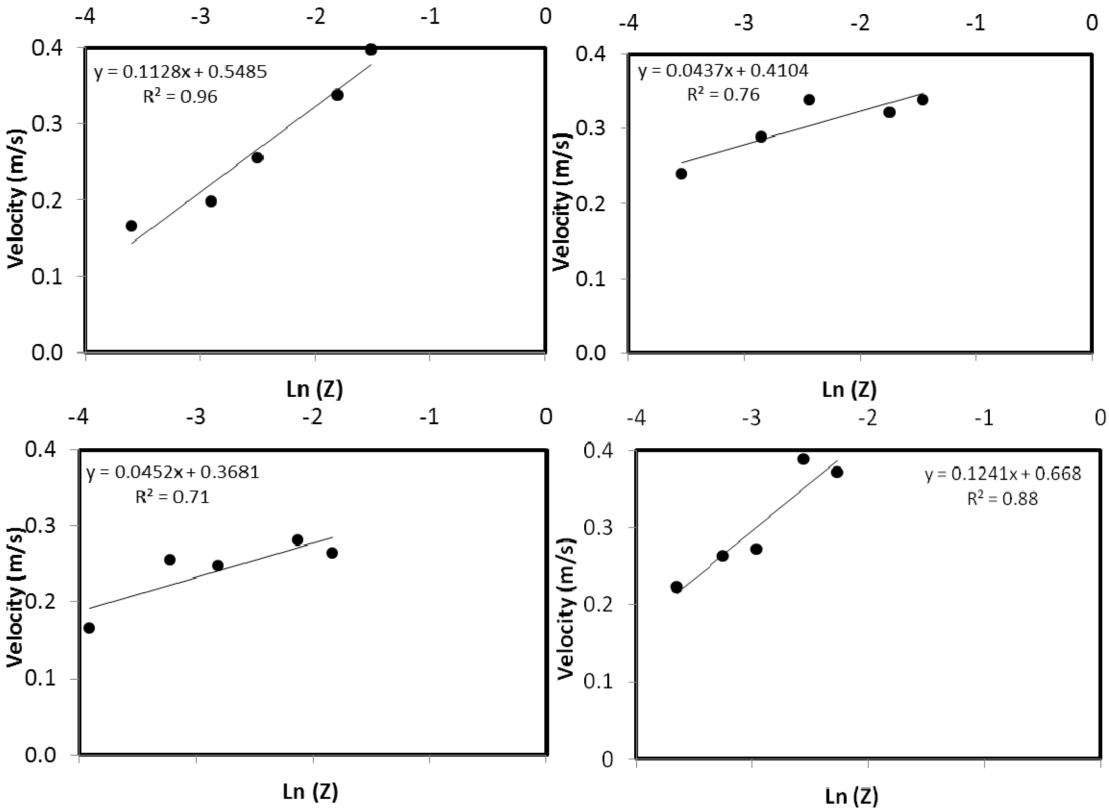

Figure 6: Vertical velocity profile for cross section 41

7. SHEAR VELOCITY ESTIMATION

In this study shear velocity (u_*) was computed using two different methods:

A) Using the whole velocity profile and fitting a straight line through the relationship between u and Ln (z); this estimate of u_* is termed u_{*p} and;
B) Using depth averaged velocity (U) assuming that flow depth h>>z_o (Wilcock, 1996;Smart, 1999). This estimate of u_* is termed u_{*h}.

In the first method (method A) the slope of the least square line fitted to (u, ln (Z)) is seen to be u_*/κ from the derivative of $\partial u/\partial(\ln(z))$. In the second method (method B) z_o was calculated once as a global constant for all cross sections using $3D_{84}/30$ (Whiting and Dietrich, 1990;Wilcock, 1996), where D_{84} = 11cm that gives z_o = 0.011m. D_{84} value was obtained from a previous feasibility study report of Megech Dam and this estimate of u_* is termed u_{*h}. If the velocity profile follows a logarithmic form, both velocities u_{*h} and u_{*p} must be strongly correlated. Figure 7A shows the relationship between u_{*h} and u_{*p} computed from the whole velocity profile measured on Megech River. It can be seen that there were large differences between the results of the methods for estimating u_* from field observations.

Wilcock (1996) used replicate measurements to evaluate the precision of each of the above methods. The velocity profile approach offered the least precision of the above two methods based on the standard errors to represent the uncertainty, although it has the important advantage that an independent estimate of z_o is not required. The estimate based on the depth average velocity offered the highest accuracy, again based on the standard error of replicate observations to measure the uncertainty (this may be due to the fact that depth average velocity can be measured more accurately than the value of velocity at any particular depth z) (Wilcock, 2001); however, it requires an independent estimate of z_o.

It also relies on the assumption that equation 5 holds throughout the flow depth; however, this can be evaluated based on the observations used to determine the depth-averaged velocity. Figure 7B shows the relationship between the shear velocities estimated from the depth-averaged velocity calculated as the mean of vertical velocity profiles (Pasternack *et al.*, 2006) and flow velocity measured at 0.4 of the depth of flow above the bed. The two shows high degree of correlation with an r^2 value of nearly 0.9, although differences between the true depth-averaged velocity and the velocity measured at an elevation of 0.4 of the flow depth may be of the order of 5% for flows characterized by high relative roughness (Byrd *et al.*, 2000).

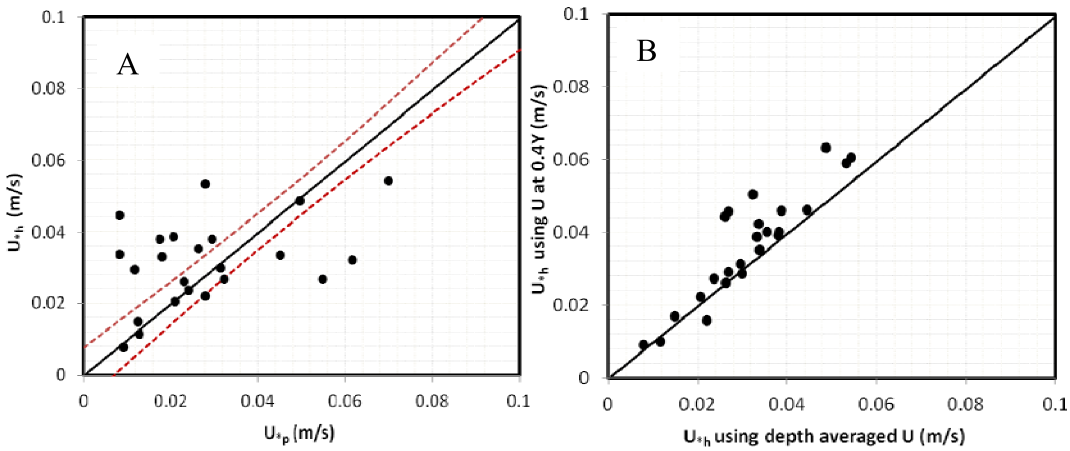

Figure 7: Comparison of u* estimation methods using field measurements A) u_{*p} using the vertical velocity profile and u_{*h} using the depth averaged velocity and global z_o (B) u_{*h} using depth averaged velocity and u_{*h} using velocity at 0.4Y. Solid line is 1:1 while dashed lines are 95% confidence limit estimates.

8. MODEL DEPTH PREDICTIONS

Model depth predictions were compared against cross sectional data for quantitative analysis and spatial patterns. Model results replicate flow depths measured in the field at different sections of the river including where flow is divided by channel bars although the highest depths are under predicted. The possible reason for this under prediction is that the low points are missed from the DEM used for simulation. This is clearly indicated in figure 8.

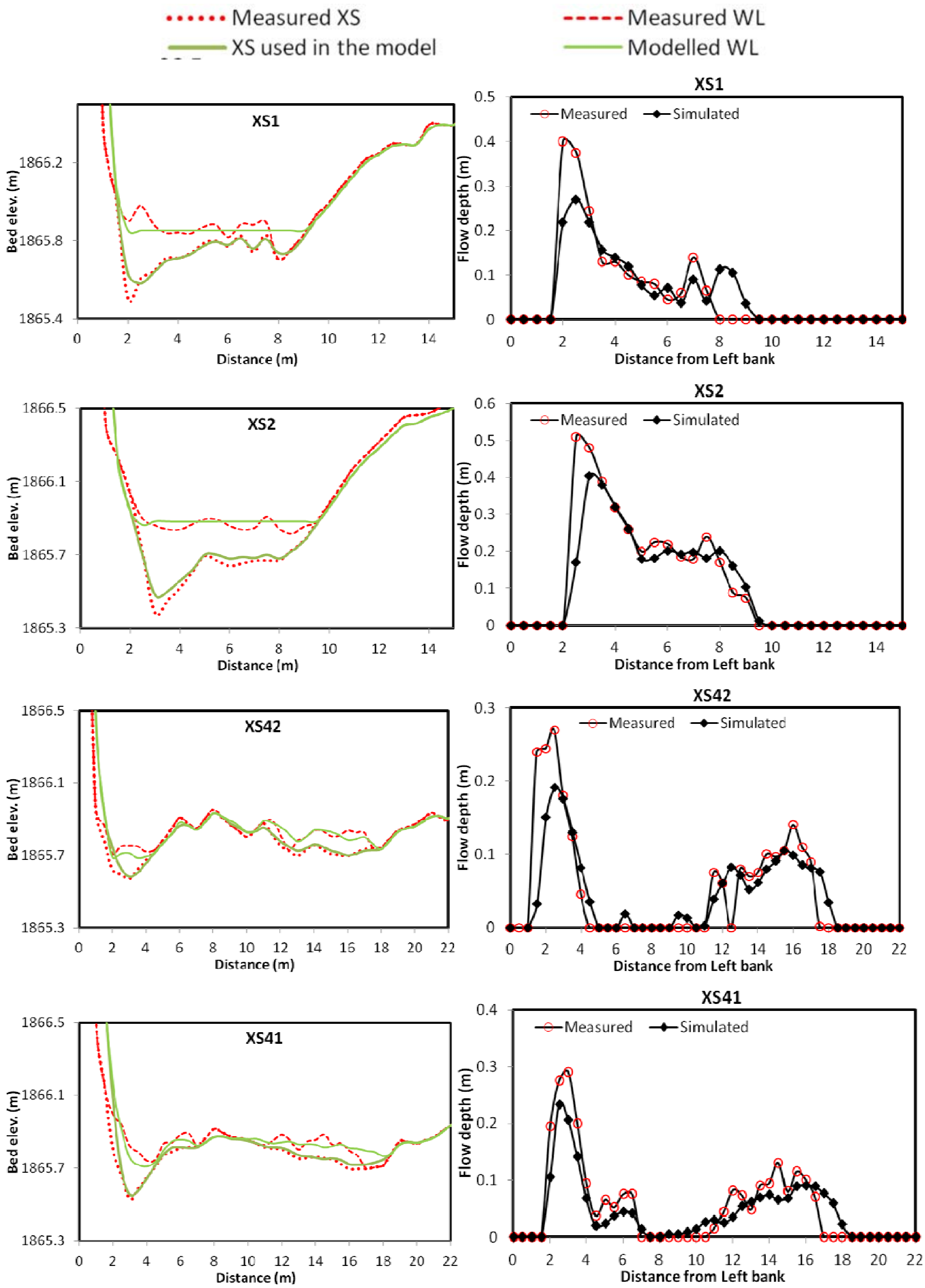

Figure 8: Predicted and measured flow depth along the channel of the study reach.

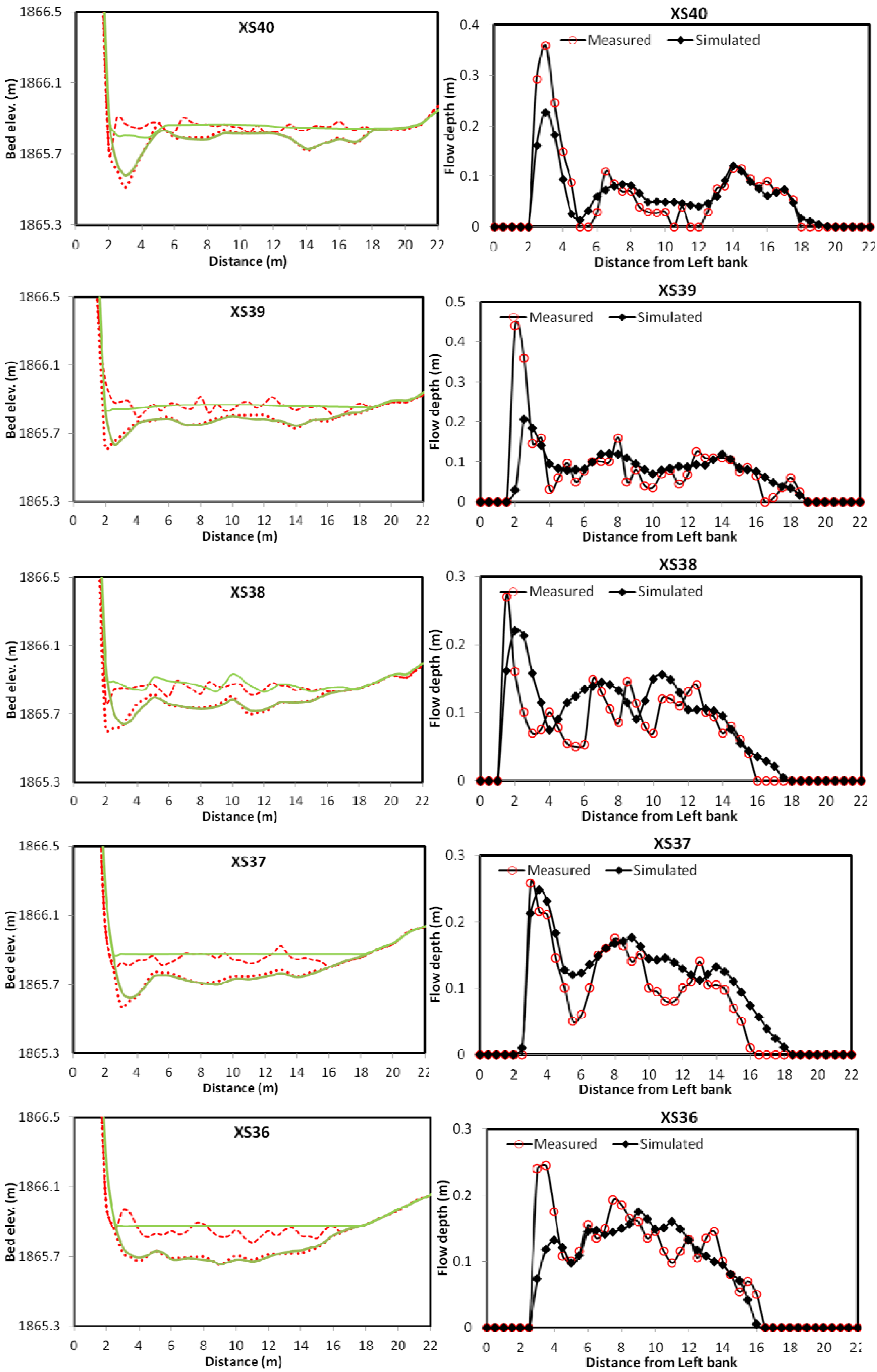

Figure 9 (Cont.): Predicted and measured flow depth along the channel of the study reach.

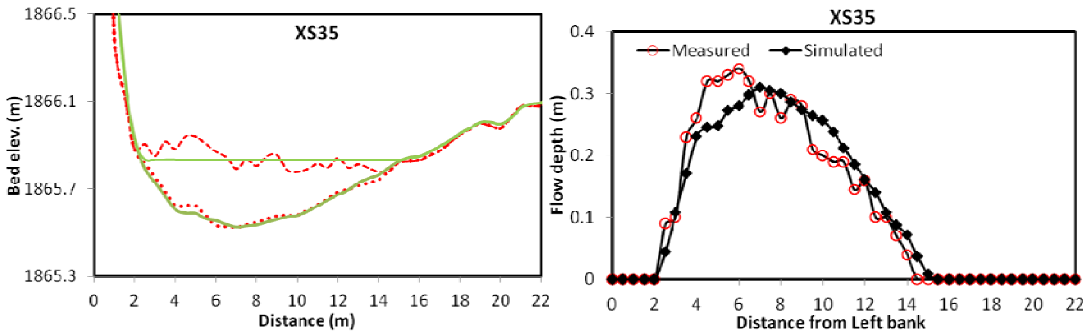

Figure 10 (Cont.): Predicted and measured flow depth along the channel of the study reach.

Table1 summarizes the results of the sensitivity analysis for point values of simulated and measured flow depth conducted using different values of the roughness depth R_d and the roughness constant R_c using the structural analysis (Webster, 1997). The coefficient of determination for the relationship between measured and simulated flow depth is relatively insensitive to various combinations of R_c and R_d values. However, the best fit lines between measurements and model predictions suggest an optimum roughness depth between 0.01 and 0.04 at lower resistance (low values of the constant R_c).

The results presented above for low flow conditions illustrate that there is a high level of correspondence between the measured and simulated depths in most areas of the study reach and generally the model replicates the patterns observed in the field data. Differences between model results and field data can be attributed to a wide source of errors and uncertainty. One uncertainty might be the presence of small scale topographic variability between the measured and DEM extracted cross sections that made the model unable to capture flow depths at the deepest sections. From the comparison figure it appears that depth prediction error was attributable to error in the DEM and was not primarily an error of the 2D model itself since in some sections where there is no significant difference in bed elevations of measured and DEM extracted cross sections, the model replicates the measured flow depth. Pasternack *et al.*,(2004) addressed the issue of DEM accuracy in terms of topographic survey (resolution and accuracy) and DEM generation methodology. In this study, the bed was surveyed with a resolution of 1 point every 1.21 m^2, which appeared to be in the higher side as compared to those specified to capture typical gravel bed morphology(Brasington and Richards, 2000;Brasington *et al.*, 2000). It might be possible to reduce error of 2D model predictions at individual nodes by having higher point densities. Second, it was very difficult to accurately predict the discharge due to extreme shallowness of the flow in one of the channels. Although the current meter used for measuring flow velocity was particularly suitable for shallow flows, the coarse bed material made use of the current meter difficult or impossible in some places.

Table1: Results of sensitivity analysis for simulated and measured flow depth using different values of effective roughness R_d

Roughness Values R_d			
R_c	0.01	0.04	0.07
24	r^2 =0.76	0.71	0.74
	B = 0.84	0.9	0.94
15	r^2 = 0.79	0.75	0.74
	B = 0.83	0.94	0.96
5	r^2 = 0.79	0.74	0.74
	B =0.99	1.03	1.05

9. COMPARISON OF BED SHEAR STRESS ESTIMATES

The measured velocity profiles at different cross sections were used directly to determine the boundary shear stress. The shear stress determined using the velocity profile approach and the measured flow depth and 'the depth-slope product' were also compared. The following equations were used to calculate the shear stresses compared.

The first and second approaches were based on the logarithmic law. In the first approach u_{*p} was calculated using the vertical velocity profile and fitting a straight line through a plot of u and Ln (z) using equation 5, then the shear stress, termed in this case τ_p is:

$$\tau_p = u_{*p}{}^2 * \rho \tag{6}$$

In the second approach u_{*h} was calculated using the depth-averaged velocity (calculated as the velocity at 0.4 of the depth of flow) and a global value of z_o using equation 5, then the shear stress, termed in this case τ_h, is:

$$\tau_h = u_{*h}{}^2 * \rho \tag{7}$$

In the third approach the shear stress, termed τ_o, was calculated using the depth-slope product as:

$$\tau_o = \rho g h S \tag{8}$$

In the fourth approach shear stress was calculated using output from the two-dimensional hydraulic model. Figure 9 shows the frequency histogram of the shear stress estimates obtained by utilizing the four methods (equations 6, 7, 8 and the 2D hydraulic model). Generally, there is only a very small similarity in the overall pattern of the shear stress distributions. It is clear that shear stress estimates from the depth-slope product are consistently higher than those estimated from the vertical velocity profiles and the two-dimensional hydraulic model. Moreover, the two-dimensional hydraulic model, gives slightly less average shear stress than the depth-slope product (average shear stress of 12.9 and 15.9 N/m^2 for the hydraulic model and depth-slope product respectively). This value is close to six times that of the third largest estimated shear stress (using the whole velocity profile, 2.03 N/m^2). It is also more than ten times larger than the shear stress estimated using the depth averaged velocity (τ_h). There is much greater shear stress variability associated with the two-dimensional hydraulic model. The standard deviation of the velocity profile approach is closer to that of the depth-slope product (4.8 and 7.6 N/m^2 respectively).

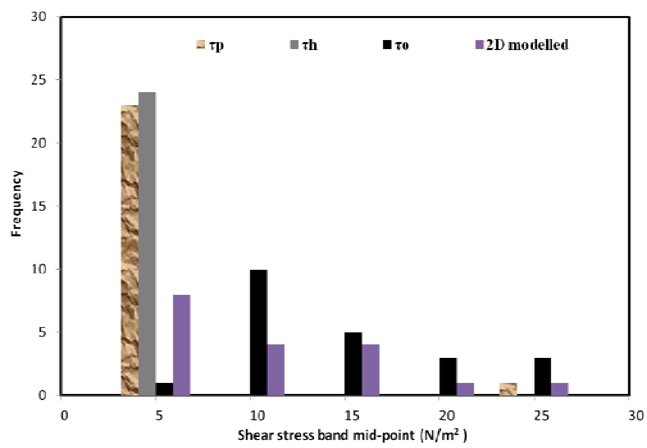

Figure 11: Frequency histogram of shear stress values obtained with different approaches. X-axis refers to the frequency band mid-point.

Given the roughness of the streambed and shallowness of the stream, it was not surprising that the local shear stresses represented only a small proportion of the total shear stress predicted within the reach using the depth-slope product.

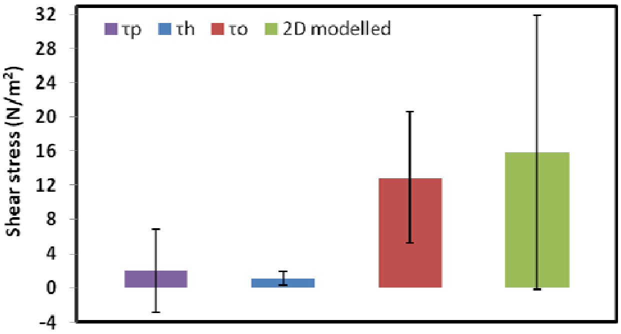

Figure 12: Frequency histogram of shear stress values obtained with different approaches. X-axis refers to the frequency band mid-point.

Assessing which approach is correct is difficult because of the complexity of shear stress measurement in complex bed topographies. Moreover, since single observations are used to determine the shear velocity, an estimate of their uncertainty cannot be made directly. However, the shear stress provided by the depth-slope product is the total shear stress and shear stress estimates by the velocity profile approaches were only a fraction of the total shear stress, which is consistent with the partitioning of shear stress due to high form drag contributed by large bed forms in these types of shallow gravel bed streams (Buffington and Montgomery, 1999;Manga and Kirchner, 2000). Moreover, it is clear from Figure 10 that the reach averaged shear stress equation (τ_o) gives a value which is equivalent to the predictions of the two-dimensional hydraulic model. This is good as the reach averaged equation can be used with ease and straightforward and requires relatively fewer dataset.

10. CONCLUSION

Analysis of the measured velocities in Megech River confirmed that the logarithmic law is valid for the Megech gravel-bed River. Shear velocity derived from the whole velocity profile and depth averaged velocity do not show a close correspondence. The difficulty in obtaining precise u_* estimates from the slope of a velocity profile has also been noted previously (Wilkinson, 1984;Whiting and Dietrich, 1989). Estimate of shear velocity (U^*_h) from the depth averaged velocity requires an independent estimate of z_o. This requires knowledge of the grain-size distribution of the bed. When the whole velocity profile is used to estimate the shear velocity, it is necessary to measure the velocity profile as accurately as possible (e.g., by using multiple observations), although there may still be uncertainty to make calculations of sediment transport rates. There is good agreement between measured and simulated flow depth. A comparison of the shear stress distributions derived using the two-dimensional hydraulic model and with those estimated using the 1D reach-averaged equation (i.e. the depth-slope product) shows a close correspondence between the two. Mean shear stresses determined using local depth and mean channel slope are roughly comparable to those determined for the same data using local predictions of both depth and energy slope. As the overall mean shear stress provides a useful index of flow strength, this comparison suggests a good level of confidence in using the reach averaged one-dimensional equation, for which data can easily be collected from cross sectional surveys.

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