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Discharge estimation in a backwater affected meandering river

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Abstract

Variable effects of backwaters complicate the development of rating curves at hydro-metric measurement stations. In areas influenced by backwater, single-parameter rating curve techniques are often inapplicable. To overcome this, several authors have advocated the use of an additional downstream level gauge to estimate the longitudinal surface level gradient, but this is cumbersome in a lowland meandering river with considerable transverse surface level gradients. Recent developments allow river flow to be continuously monitored through velocity measurements with an acoustic Doppler current profiler (H-ADCP), deployed horizontally at a river bank. This approach was adopted to obtain continuous discharge estimates at a cross-section in the River Mahakam at a station located about 300 km upstream of the river mouth in the Mahakam delta. The discharge station represents an area influenced by variable backwater effects from lakes, tributaries and floodplain ponds, and by tides. We applied both the standard index velocity method and a recently developed methodology to obtain a continuous time-series of discharge from the H-ADCP data. Measurements with a boat-mounted ADCP were used for calibration and validation of the model to translate H-ADCP velocity to discharge. As a comparison with conventional discharge estimation techniques, a stage-discharge relation using Jones formula was developed. The discharge rate at the station exceeded $3300 \text{ m}^3 \text{ s}^{-1}$. Discharge series from a traditional stage-discharge relation did not capture the overall discharge dynamics, as inferred from H-ADCP data. For a specific river stage, the discharge range could be as high as $2000 \text{ m}^3 \text{ s}^{-1}$, which is far beyond what could be explained from kinematic wave dynamics. Backwater effects from lakes were shown to be significant, whereas the river-tide interaction may impact discharge variation in the fortnightly frequency band. Fortnightly tides cannot easily be isolated from river discharge variation, which features similar periodicities.

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1 Introduction

Discharge is the phase in the hydrological cycle in which water is confined in channels, allowing for an accurate measurement compared to other hydrological phases (Herschy, 2009). Reliable discharge data is vital in research focusing on a broad range of topics related to water management, including water allocation, navigation, and the prediction of floods and droughts. Also, it is crucial in catchment-scale water balance evaluations. Hydrological studies relying on rainfall-runoff models require continuous discharge series for model calibration and validation (e.g. Beven, 2001; McMillan et al., 2010).

Discharge estimates are conventionally obtained from a rating curve model, using water level data as input, and a limited number of discharge measurements for calibration. Despite a number of techniques available to account for unsteady flow conditions, water agencies often assume an unambiguous relation between stage and discharge. Both steady and unsteady rating curve models are prone to uncertainties, related to interpolation and extrapolation error and seasonal variations of the state of the vegetation (Di Baldassarre and Montanari, 2009). Changes in the stage-discharge relations frequently occur due to variable backwater effects, rapidly changing discharge, overbank flow, and ponding in areas surrounding the channel (Herschy, 2009). From discharge uncertainty assessment for the River Po, Di Baldassarre and Montanari (2009) showed that the use of a rating curve can lead to error in discharge estimates averaging 25.6%. In this contribution we show that error in estimates from traditional single gauge rating curves can be even higher, confirming the need for alternative approaches.

Single-valued rating curves can produce biased discharge estimates, especially in highly dynamic rivers and streams. In terms of the momentum equation, this bias is the result of temporal and spatial acceleration terms, and the pressure gradient term, which all have to be neglected to justify an unambiguous relation between stage and discharge. River waves featuring such unambiguous relation are termed kinematic. When the pressure gradient term is retained, but the acceleration terms can be neglected, the

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momentum balance appears as a convection-diffusion equation that can be solved to yield a non-inertial wave as a special type of diffusion wave (Yen and Tsai, 2001). Several formulas have been developed aiming to obtain discharge from parameters that can readily be derived from water level time-series. Among these, the Jones formula is the most well-known, in which the surface level gradient term is approximated using the kinematic wave equation.

Variable backwater is one of the principle factors that cause an ambiguous stage-discharge relation. Backwater from one or several downstream elements such as tributaries, lakes, ponds or dams, complicates rating curve development at hydrometric gauging stations (Petersen-Overleir and Reitan, 2009), causing curved longitudinal surface level profiles for a constant and uniform river discharge. Tides superimposed on river discharge can produce subtidal water level variations (Buschman et al., 2009), with periods of a fortnight or longer, which may not immediately be recognized as phenomena controlled by the tidal motion. Potentially, water level setup by river-tide interactions can cause backwater effects beyond the point of tidal extinction (Godin and Martínez, 1994).

Recently, approaches have been developed to account for backwater effects, using a twin gauge approach to obtain estimates of the longitudinal water level gradient. Such ratings are developed based on records of stage at a base gauge and the fall of the water surface between the base gauge and a second gauge downstream (Hersch, 2009). Considering the water level gradient to be a known variable and with the assumption of a steady state, the terms representing the pressure gradient and spatial acceleration in the momentum equation can be resolved (Dottori et al., 2009). The application of formulas using simultaneous stage measurements was criticised by Koussis (2010). Dottori and Todini (2010) refuted most of the criticism by Koussis (2010), but acknowledged that in lowland areas with a small bed level gradient, the occurring water level gradient can drop below the measuring accuracy of the level gauge. Dottori and Todini (2010) estimate the minimum distance between the gauges to be in between 2000 and 5000 m when the bed slope is 1×10^{-5} . Neither Koussis (2010)

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nor Dottori and Todini (2010) considered an additional drawback that arises from lateral water level gradients, which can be considerable especially in meandering rivers characterised by a high sinuosity. In high-curvature river reaches, level gauges on opposite sides of each of the two cross-section would be needed to infer the longitudinal water surface gradient. We conclude that the twin gauge approach to discharge measurements is suboptimal in lowland meandering rivers, which are most susceptible to backwater effects.

Discharge can be estimated from flow velocity, which bears a much stronger relation to discharge than the water surface. Gordon (1989) was among the first to estimate discharge from a boat-mounted acoustic Doppler current profiler (ADCP), which soon after became a standard means of estimating discharge accurately. ADCP surveys are costly and are carried out merely occasionally. Recent developments allow horizontal profiles of flow velocity to be continuously monitored by a horizontal acoustic Doppler current profiler (H-ADCP). The H-ADCP is typically deployed at a river bank, measuring a horizontal velocity profile across a channel. The acquired data can then be used to estimate discharge, predicting cross-section integrated velocity from the array data of flow velocity.

Several methods are available to convert H-ADCP data to discharge. In the Index Velocity Method (IVM), H-ADCP velocity estimates are averaged and regressed directly against discharge from boat-mounted ADCP measurements (Simpson and Bland, 2000). Nihei and Kimizu (2008) adopted a deterministic approach, assimilating H-ADCP data with a two-dimensional model of the velocity distribution over a river cross-section. In the velocity profile method (VPM) described by Le Coz et al. (2008), total discharge is inferred from theoretical vertical velocity profiles, made dimensional with the H-ADCP velocity measurements across the section, extrapolated over the river width. Hoitink et al. (2009) combined elements of the IVM and VPM methods, using a boundary layer model to calculate specific discharge from a point measurement of velocity, and a regression model to relate specific discharge to total discharge. Sassi et al. (2011) elaborated on the work of Hoitink et al. (2009) by embedding a more

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remained relatively constant during the measuring period, amounting to 0.3° and 0.01° , respectively. The measurement protocol for the H-ADCP consisted in 10 min bursts at 1 Hz every 30 min.

The H-ADCP used in this study is a three-beam instrument with angles between beams of 25° and an acoustic beam width ϕ of 1.2° . The H-ADCP was installed at a distance $d = 7.9$ m below the mean water level, with the transducer head at $x = 74.4$ m from the shore. Because the H-ADCP was deployed looking slightly upward, the H-ADCP measured a volume-averaged velocity at elevation z_c , which is calculated from:

$$z_c = \begin{cases} -d + \tan(\theta)(n-x) & \text{if } d + \eta > \tan(\phi/2 + \theta)(n-x) \\ -d + \tan(\theta)(n-x) + \Delta z & \text{otherwise} \end{cases} \quad (1)$$

where θ is pitch, n is cross-channel coordinate, with the origin at the river bank and η is water level variation. Δz is the level difference between the centroid of the ensonified water area and the central beam axis. This correction accounts for the lowering of the centroid of the ensonified water volume if the main lobe intersects with the water surface at low water (Hoitink et al., 2009).

Conventional boat-mounted ADCP measurements were periodically taken at the cross-section where the H-ADCP was deployed to establish water discharge through the river section. Eight campaigns were carried out spanning low and high flow conditions. The campaign consisted of transects in front of the H-ADCP for determining hydraulic parameters (referred to as “par”) and transects carried out about 20 m upstream to cover the whole river section for calibrating the discharge computation (referred to as “cal”). Each transect measurement spanned over about two hours. The boat was equipped with a 1.2 MHz RDI Broadband ADCP measuring in mode 12, a DGPS compass and an echosounder. The ADCP measured a single ping ensemble at approximately 1 Hz with a depth cell size of 0.35 m. Each ping was composed of 6 sub-pings, separated by 0.04 s. The range to the first cell center was 0.865 m. The boat speed ranged between 1 and 3 m s^{-1} .

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Depth estimates from the ADCP bottom pings were used to construct a local depth map. The range estimation from the four acoustic beams was corrected for pitch, roll, and heading of the ADCP, and referenced to the mean water level. Bathymetry data were also collected using a single beam echosounder for validation. Water levels were measured using pressure transducers in Melak at the H-ADCP station, in Lake Jempang, and in Muara Kaman at the confluence of River Kedang Rantau with the Mahakam, downstream of the Makaham lakes area.

3 Methods to estimate discharge

3.1 Flow structure

The design of an appropriate discharge estimation method requires information about the local flow structure, which is discussed in the present section based on the boat-mounted ADCP surveys. The ADCP velocity measurements were projected into normalized (β, σ) coordinates. The normalized spanwise coordinate β was obtained by normalizing the distance from the bank to the maximum width within that survey. The normalized vertical coordinate σ was obtained from:

$$\sigma = \frac{H + z}{H + \eta} \quad (2)$$

where H is water depth, z is normal distance from the bed. The mesh size of the coordinate was $\Delta\beta = 0.025$ and $\Delta\sigma = 0.05$. Turbulence fluctuations were removed by taking the mean over the repeated velocity recordings for each grid cell within a survey. Velocity profiles from boat-mounted ADCP measurements were then averaged over

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3.2 Semi-deterministic semi-stochastic method

In the semi-Deterministic semi-Stochastic Model (DSM) developed by Hoitink et al. (2009) and Sassi et al. (2011), time-series of single point velocity u_c , measured at the relative height σ_c , are translated into depth-mean velocity U according to:

$$U = F u_c \quad (5)$$

where

$$F = \frac{\ln\left(\frac{H}{\exp(1+\alpha)}\right) - \ln(z_0)}{\ln(\sigma_c H) + \alpha \ln(1 - \sigma_c) - \ln(z_0)} \quad (6)$$

Herein, α accounts for sidewall effects that retard the flow near the surface by means of secondary circulations and z_0 is the apparent roughness length. The value of α is obtained from:

$$\alpha = \frac{1}{\sigma_{\max}} - 1 \quad (7)$$

where σ_{\max} is the relative height where the maximum velocity occurs. To estimate σ_{\max} we closely followed the approach of Sassi et al. (2011) by repeatedly fitting a logarithmic profile starting with the lowermost three ADCP cells, adding successively a velocity cell from the bottom to the top for each fit. σ_{\max} is determined from the development of the goodness of fit which decreases once the cell above σ_{\max} is included. Figure 5 illustrates that cross-river profiles of α do not show a systematic variation between $0.2 < \beta < 0.9$. We adopt a constant value of $\alpha = 0.28$, which results in $\sigma_{\max} = 0.78$.

The determination of the effective hydraulic roughness length z_0 is fundamental in the approaches by both Hoitink et al. (2009) and Sassi et al. (2011). The value of z_0 is obtained as:

$$z_0 = \frac{H}{\exp\left(\frac{\kappa U}{u_*} + 1 + \alpha\right)} \quad (8)$$

where κ is the Von Karman constant and u_* is the shear velocity. Values of u_* coincide with the slope of the linear regression line of $u(\sigma)$ against $(\ln(\sigma) + 1 + \alpha + \alpha \ln(1 - \sigma)) / \kappa$ (Sassi et al., 2011). Figure 6 shows that values of z_0 change over width and are consistent at each β location for each ADCP campaigns in the range $\beta > 0.4$. The geometric mean of z_0 at each β location over all boat-mounted transects in front of the H-ADCP (par) were taken for further computation, processing only the H-ADCP data in the range $\beta > 0.4$.

Specific discharge q is obtained from $q = UH$, where U is depth mean velocity estimates from H-ADCP measurements. Discharge Q is obtained from:

$$Q(t) = f(\beta)Wq(\beta, t) \quad (9)$$

where W is the river width, $f(\beta)$ is constant amplification factor obtained from the total discharge of each “cal” campaign divided by the product Wq from the “par” campaigns. Profiles of f remain constant up to $\beta = 0.8$ during the two calibration campaigns (Fig. 7). The mean value of f at each β location was taken for discharge computation.

3.3 Index velocity method

We estimated discharge from the H-ADCP data based on the widely used IVM approach (Simpson and Bland, 2000; Le Coz et al., 2008), which is more straightforward than the approach by Sassi et al. (2011). The sectionally integrated velocity obtained from the boat-mounted ADCP is regressed against the width-averaged along channel velocity (i.e. the index velocity) obtained with the H-ADCP. Figure 8 shows the regression results.

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3.4 Stage-discharge relation

To investigate the degree in which discharge at Melak station can be captured by a rating curve, Jones' formula was applied, which reads:

$$Q = Q_{\text{kin}} \left\{ 1 + \frac{1}{cS_0} \frac{\partial h}{\partial t} \right\}^{1/2} \quad (10)$$

where Q_{kin} is the kinematic discharge, c is wave celerity, S_0 is bed slope, and $\partial h/\partial t$ is rate of water level change in time t (Petersen-Overleir, 2006). The celerity c was estimated from $c = \sqrt{gd}$ (Liggett and Langley, 1998), where g is gravitational acceleration and d is hydraulic mean depth, according to $d = A/b$. Herein, A is river cross sectional area and b is river width. The bed slope was estimated from the Mahakam River bed level profile derived from SRTM data by van Gerven and Hoitink (2009). Q_{kin} was calculated by Manning formula:

$$Q_{\text{kin}} = \frac{1}{n} S_0^{1/2} AR^{2/3} \quad (11)$$

where n is Manning roughness coefficient, R is hydraulic radius obtained from the ratio between A and the wetted perimeter of the river cross-section. The Manning coefficient was estimated based on an evaluation of the river geometry and composition, following a standard empirical technique provided by Gore (2006). The details of channel evaluation to determine n are presented in Table 1.

4 Results and discussion

Table 2 shows the validation results. Discharge estimates obtained by applying the method by Sassi et al. (2011) differed less than 5% from the accurate estimates obtained from the boat surveys, whereas discharge estimates from the IVM feature a bias

up to 23%. Figure 9 shows time-series of the absolute and relative difference between Q_{DSM} and Q_{IVM} , which indicate that the validation results represent the medium to high flows well. During low flows, Q_{DSM} and Q_{IVM} can deviate much more, both in a relative and in an absolute sense. Unfortunately, a planned validation survey during the low flow was cancelled due to technical problems, which could have shed more light on the validity of the low-flow discharge estimates. Regarding the medium to high flows, the larger bias introduced by the index velocity method is due to the fact that the H-ADCP is monitoring flow at a relative depth that changes with the river stage, which challenges the constancy of the conversion factor to calculate discharge from the index velocity. The obtained results highlight the merits of applying the more elaborate procedure advocated by Hoitink et al. (2009) and Sassi et al. (2011), which in this case reduces the difference with the reference measurements from typically 20% to less than 5%.

H-ADCP measurements at Melak station revealed a complex stage-discharge relation that was highly hysteretic (Fig. 10). Hysteresis is generally related to flood wave propagation. For the same water level, higher discharge during rising stage and lower discharge during falling stage are known to occur, resulting in distinctive loops in stage-discharge relationships (Petersen-Overleir, 2006). At Melak station, the range of discharges that can occur for a specific stage can span over more than $2000 \text{ m}^3 \text{ s}^{-1}$, which is exceptionally large in comparison with the maximum discharge of $3370 \text{ m}^3 \text{ s}^{-1}$. Such variation can be considered far beyond the rising stage and falling stage explanation. Compared to standard rating curves for different hydraulic conditions (Hersch, 2009), the stage-discharge relation in Fig. 10 will reflect the presence of variable backwater effects, looping due to changing discharge, and multiple looping due to overbank flow and ponding. Radar images showed vast areas in the Mahakam Lakes Region to become inundated during high flows (Hidayat et al., 2011). Part of the complexity in the stage-discharge relation can be explained from the subharmonics generated by river-tide interactions (Buschman et al., 2009). At low discharges in August 2009, the tidal signal is clearly visible in the discharge series. Due to the flat terrain of the middle and lower Mahakam, tidal energy propagates up to the Mahakam lakes area, where much

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of the tidal energy is dissipated. Subharmonics such as the fortnightly tide MS_f may extend beyond the lakes region, but cannot be readily isolated from river discharge variation as discharge variation features fortnightly variation both in the presence and in absence of a tidal influence.

5 The wide loops in the stage-discharge plots are the result of the geographical complexity of the region where Melak station is located, experiencing a flashy discharge from upstream and backwater effects from downstream. The flashy discharge regime relates to high rainfall rates in large parts of the catchment upstream of Melak, which dominates the moderating effect of the rain forest. The backwater effects are caused
10 both by the lakes and a number of tributaries, all affecting the water level profile. Lake emptying and filling processes contribute to retarding and accelerating the river flow velocity. Figure 11 illustrates the lake emptying and lake filling influencing water levels and discharge upstream. At the start of lake emptying, when the lake level was still high, water stage in Melak was relatively high for a relatively low discharge. When the
15 lake level dropped, the backwater effect was reduced and discharge increased while water stage kept decreasing until the point that discharge was sufficiently high to make water stage follow the trend in the discharge time-series. The opposite mechanism took place during lake filling as shown in Fig. 11 (bottom panel). Water stage records downstream of the Mahakam lake area (Muara Kaman) indicate that some peaks of
20 water level were shaved by the lake filling and emptying mechanism.

The discharge obtained from the stage-discharge relation using Jones formula is merely a rough estimate of discharge at Melak station, indicating the range of discharge variation. It did not capture the detailed discharge dynamics as revealed by the H-ADCP measurements. This can be related to a wide variety of reasons. The
25 Froude number takes a value around 0.01, which legitimizes neglecting the spatial and temporal acceleration terms in the momentum equation, validating the non-inertial wave approximation. The key assumption used to derive the Jones formula is the applicability of the kinematic wave equation to deal with the surface gradient term in the non-inertial wave equation. There is no theoretical justification for this, and it is

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well-known that the kinematic wave equation cannot capture discharge dynamics in a backwater effected river reach (e.g. Tsai, 2005). The Jones formula is just one of a series of formulas available to predict discharge from time-series of a single level gauge (Henderson, 1966; Di Silvio, 1969; Fread, 1975; Lamberti and Pilati, 1990; Perumal and Ranga Raju, 1999), all aiming to improve the original Jones' formula. Dottori et al. (2009) systematically reviewed those formulas, explicitly mentioning they are best applicable when flow conditions are quasi kinematic. None of these approaches will be capable of reproducing the wide loops occurring at Melak station, which underlines the importance of monitoring additional information besides stage at a single section. Considering the ease of deployment of H-ADCPs, they are a promising alternative to the dual gauge approach advocated by a.o. Dottori et al. (2009).

5 Conclusions

Flow measurements using a 600 kHz H-ADCP were carried out at a 300 m wide cross section of the Mahakam River in Melak, 40 km upstream of the Mahakam lakes area. Conventional boat-mounted ADCP measurements were periodically taken to establish water discharge through the cross section. We followed a recently developed semi-deterministic, semi-stochastic method to convert the H-ADCP to discharge, and compared the results with those obtained from the standard index-velocity method and a rating curve model. The new method was found to reduce the difference with discharge estimates from the boat-mounted ADCP surveys from around 20% to less than 5%, based on three validation surveys. The continuous time-series of discharge showed that the validation data were representative for medium to high flows. A stage-discharge model based on Jones's formula captured only a small portion of the discharge dynamics, which was attributed to the invalidity of the kinematic wave assumption. A discharge range of about $2000 \text{ m}^3 \text{ s}^{-1}$ was established for a particular stage in the recorded discharge series, which is about 60% of the peak discharge and therefore exceptionally large. The large range of discharge occurring for a given stage was

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attributed to multiple backwater effects from lakes and tributaries, floodplain impacts and effects of river-tide interaction, which generate subharmonics that cannot readily be isolated from river discharge oscillations.

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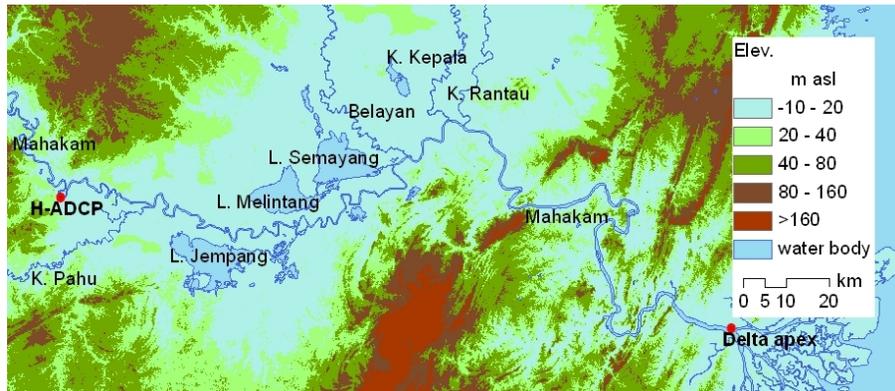


Fig. 1. Location of H-ADCP discharge station in the Mahakam River, plotted on a digital elevation model obtained from Shuttle Radar Topographic Mission (SRTM) data.

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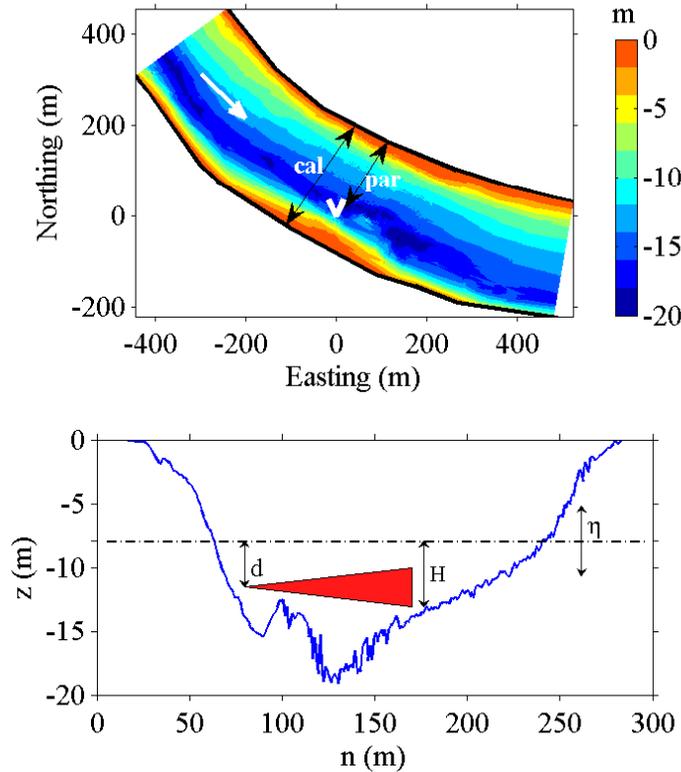


Fig. 2. Top: bathymetry at Melak discharge gauging station. The arrow indicates flow direction, V indicates the location where the H-ADCP was deployed, double arrows indicate locations of boat-mounted ADCP transects. Bottom: channel cross-sectional profile at the station. The shaded area indicates cross-section of the H-ADCP conical measuring volume.

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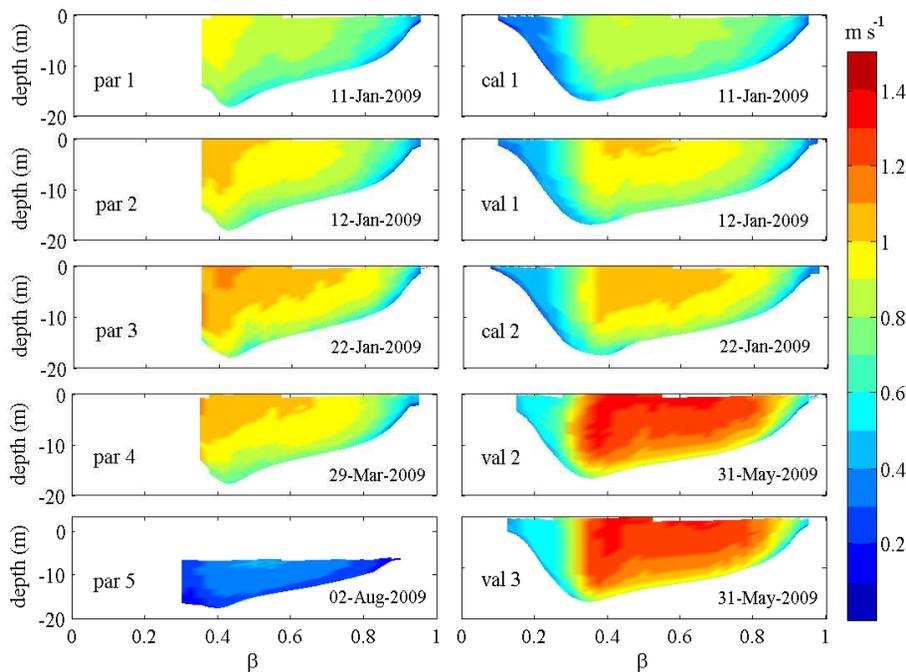


Fig. 3. Streamwise velocity spatial structure over the cross-section during boat-mounted ADCP campaigns. Transects labelled “par” were taken in front of the H-ADCP to obtain hydraulic parameters, while the ones labelled “cal” were taken 20 m upstream to cover the whole channel width for calibration and validation.

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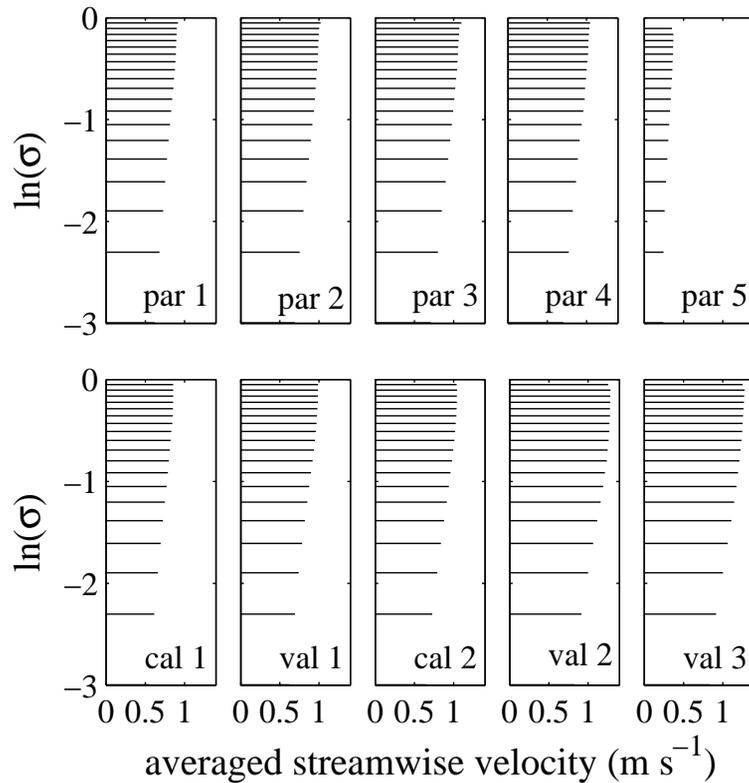


Fig. 4. Velocity profiles averaged over the middle part of the river section ($\beta = 0.35 - 0.65$) during the ADCP campaigns. The length of lines represents velocity magnitude.

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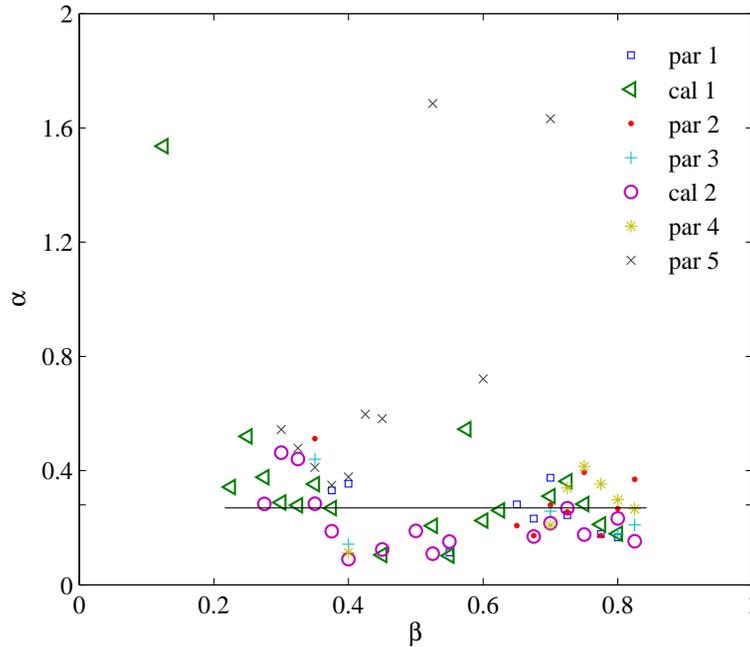


Fig. 5. Profiles of α across the river section for all parameter and calibration surveys. In the conversion model $\alpha = 0.28$ is taken for $\beta > 0.35$.

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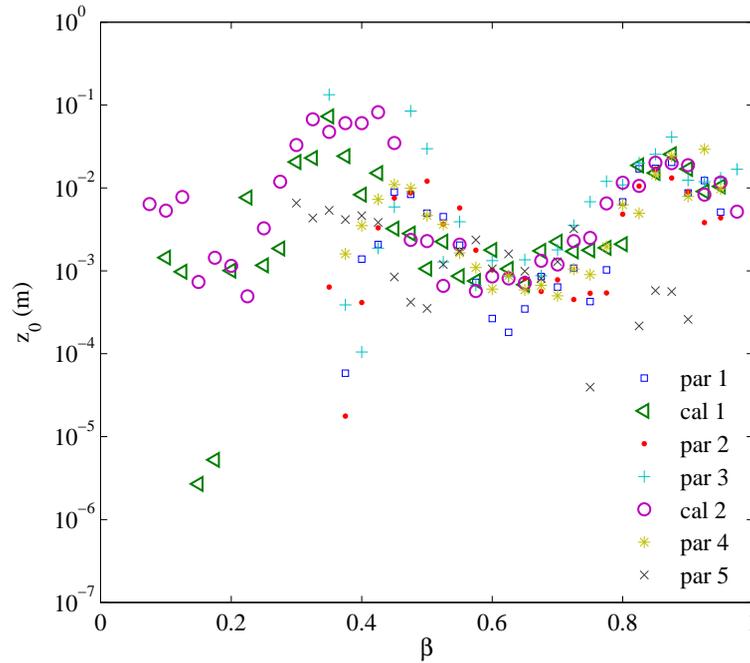


Fig. 6. Cross-river profiles of z_0 for all parameter and calibration surveys.

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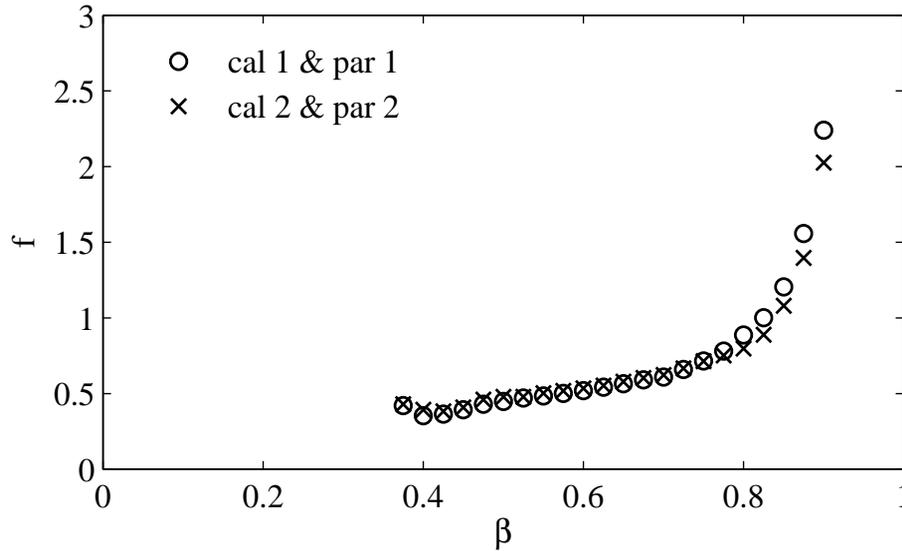


Fig. 7. Amplification factor f obtained for quasi-simultaneous parameter and calibration surveys.

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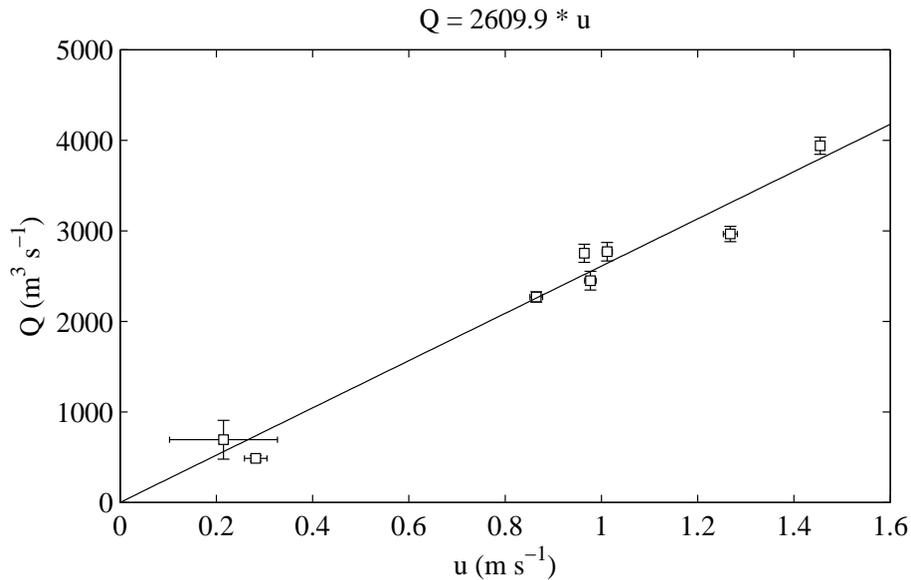


Fig. 8. Discharge (Q) as a function of index velocity (u) measured by the H-ADCP at Melak station. Standard deviations represented by error bars in the plot indicate that flow variations were higher during low flow condition.

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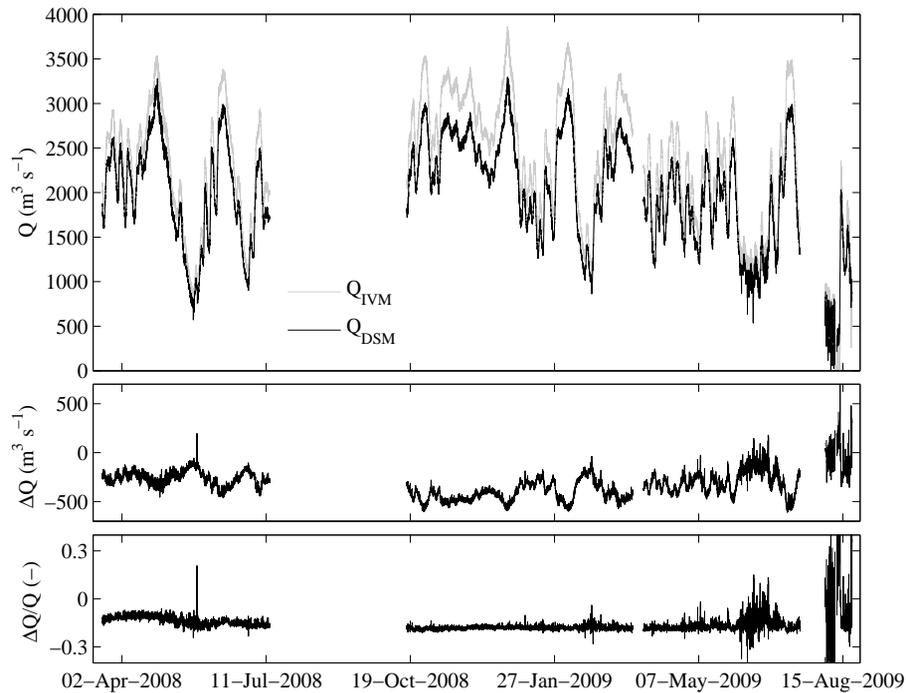


Fig. 9. Continuous series of discharge estimates derived from H-ADCP data with the DSM and the IVM. Central and bottom panels offer a comparison between the two approaches to convert H-ADCP data to discharge, where $\Delta Q = Q_{\text{DSM}} - Q_{\text{IVM}}$.

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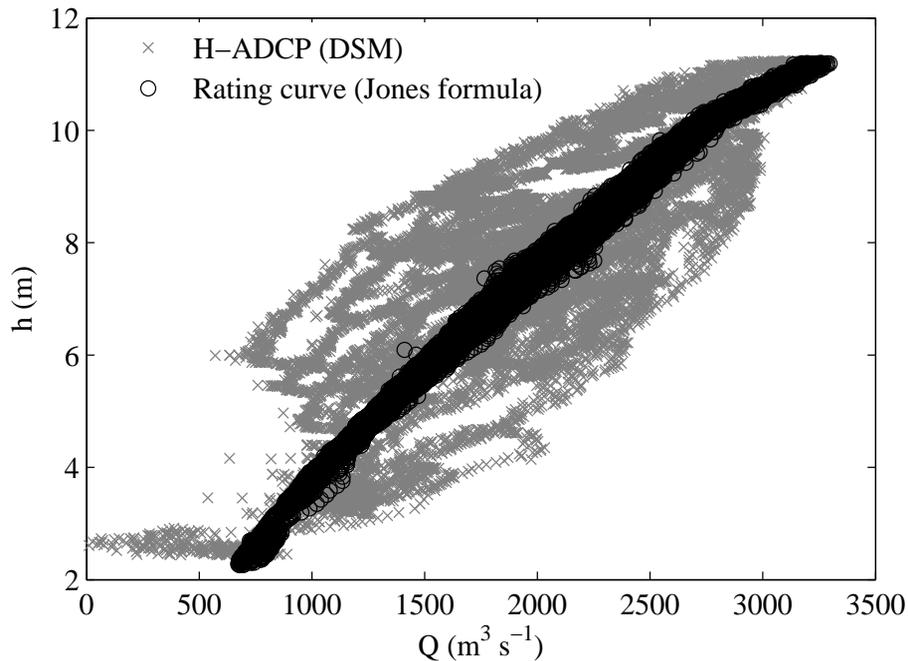


Fig. 10. Water stage and discharge estimates at Melak station, obtained from a rating curve (Jones' formula) and from H-ADCP measurements, applying the DSM. Water stage is with respect to the position of a pressure gauge about 9 m from the deepest part of the river cross-section.

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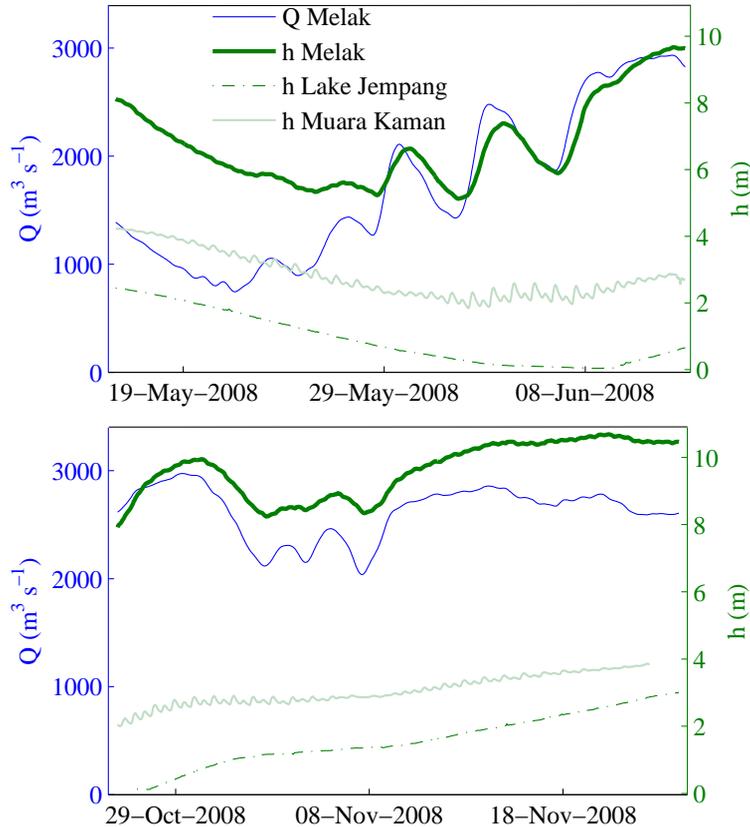


Fig. 11. Water stage and discharge during lake emptying (top) and during lake filling (bottom). Muara Kaman, where the tidal signal was observed during most of the measurement period, is located downstream of the Mahakam lakes area about 170 km from Melak.