重.25　A STUDY ON UNMANNED / AUTOMATIC RIVER DISCHARGE OBSERVATIONAL TECHNOLOGY AND ITS ACQUIRING ACCURACY

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Abstract : For the purpose of developing automatic water discharge measurement system, authors conducted experimental measurements in two different flow fields with several devices, such as 1) non-contact current meter with electromagnetic wave for measuring water velocity at water surface, 2) Acoustic Doppler Current Profiler (ADCP) traverse measurement for measuring the velocity index, 3) the echo sounder fixed on H-type steel for monitoring the river-bed fluctuations, and 4) the ADCP traverse measurement mounted on a tethered boat for bathymetry measurement as well as for verification purposes. In this study, authors examined the observational results of flood events obtained in actual river with verifying the observed values as well as explain about the hydraulic phenomena observed by the several instrumentations at the river channel, which involve considerable amounts of the riverbed deformation. Finally authors discuss about the possibility of the automatic water discharge measurement system based on the observed results.

Key words : Automatic discharge measurement, non-contact current meter, ADCP measurement, velocity index

1. Introduction

Water discharge measurements in river, as well as storage of the observed values, are very fundamental in terms of designing the river-infrastructure, such as levee, dam, and etc. For conducting the measurement, the current meter incorporating with a depth sounder has been employed in many countries, especially in continental rivers. In the case of Japan, float-type measurements, which established in 1940s, have employed since then, because character of Japanese rivers is highly unsteadiness, loose boundary, and, above all, rough water surface. Recently, many devices have developed with different principles, such as acoustic, video images, erector-magnetic, and etc. Those are classified as fixed/non-fixed type measurements. Firstly, representative of the non-fixed type measurements is acoustic Doppler devices with a profiling technique loaded on boats, which are the only tool capable of water discharge measurement without any hydraulic assumption. Secondly, the fixed type measurements, which are usually automatic measurement without human labors, can be listed as non-contact current meter with erector-magnetic[1) 2)], Particle Image Velocimetry (PIV) and Space-Time Image Velocimetry (STIV) with video images[3)4)], Horizontal-Acoustic Doppler Current Profiler (H-ADCP)[5)], and etc. Especially concerning about the non-contact current meter as well as technique with video images, there was a skepticism whether those instruments measure speed of water or speed of surface ripples generated by many different disturbance including wind. Recently it has confirmed that those instruments obtain appropriate velocity values based on direct comparison with ADCP[6)7)8)]. Anyhow, it has been acknowledged that they could measure water-velocity as operational purposes at points wherever assigned by each device. As for considering about the water discharge values using the fixed type water velocity measurements, additional information such as water depth, as well as an index velocity are still necessary to be taken into account.

The river-bed elevation changing during flooding is unlikely monitored, though few have been reported, such as 1) Echo sounder or ADCP mounted on the boat with traversal measurement, 2) the ground penetrated radar[9] and 3) Ring methods[10]. As for the first one, not many active changing of river-bed have been observed, probably active river-bed changing as well as rough water surface occurs simultaneously, since depth measurements with acoustic type mounted on the boat in such as water surface condition have not conducted because of danger as well as difficulty. Therefore, not many observational results have been reported. As for the second one, not many observational results introduced even after Costa et al. 2000, probably because of less availability of instrumentation. For the third one, many important results were introduced; though it has characteristics of measuring only minimum bed elevation, instead of timely change.

The velocity index is the correction coefficient to estimate depth averaged water velocity from velocity at Water Surface. Polatel 2006 investigated about the velocity indices with velocity measurement with PIV technique in the experimental flume as well as with LES with different shape of roughness[11]. The study reported that the velocity indices vary between 0.659 and 0.910 in smooth bed, as well as between 0.850 and 0.908 in roughed river bed. On the other hand, Yorozuya et al. 2010a focused attention to variation of the index incorporated with non-dimensional shear stress based on traverse measurements using ADCP in actual field. The study indicates that the index varies widely in the different section, though it merges to 0.85 when water stage rises with observed range of non-dimensional shear stress in between 0.2 and 1.0. In addition, Muste et al. 2008 mentioned that the velocity index of 0.85 is widely accepted in hydraulic community[3]. As similar methodology, Nihei and Kimizu 2008 introduced DIEX method[5]. With computational technique, this method interpolates and extrapolates un-measured values in vertical as well as cross sectional direction. Estimating the values in vertical direction is a sort of determination of the velocity index. This method has characteristics of employing constant roughness as well as river bed elevation.

Idealistically, the automatic system supposed to be the system without human labor. In the case of the water discharge measurement, the water velocity, the velocity index, and the water depth are prerequisite information. As explained in previous paragraph, only velocity measurement is already acknowledged as the automatic system. In addition, the monitoring as well as estimating of bathymetry in a whole section are still not fully developed. Therefore, the authors do not intend to show the comprehensive conclusion in this paper; rather, we will introduce the technique currently developed through the observational results, a limitation usage, and accuracies of the system. In this study, authors have set up and conducted flood discharge measurement with the instruments, such as 1) non-contact current meter with electromagnetic type for measuring water velocity at water surface, 2) ADCP traverse measurement for measuring the velocity index, 3) the echo sounder fixed on H-type steel for monitoring the river-bed fluctuations, and 4) the ADCP traverse measurement mounted on a tethered boat for bathymetry measurement as well as for verification purposes.

To apply the technique of an automatic water discharge system in nationwide, probably four different types need to be considered, such as 1) high energy water flow with very active river bed fluctuation, 2) water flow involving bedload with slight river bed fluctuation 3) mild flow with few river bed fluctuation, and 4) flow with tidal influences sometimes involving salt wedge. The current study mainly focus on first and second types of flow, since the non-contact current meter has a strength points when velocity is very high, as well as even when water surface condition is very rough. The weak points are that the non-contact current meter cannot measure the velocity when water velocity is less than 0.5m/s. Moreover, the measurement of the flow with tidal influences has difficulty since the velocity index cannot be determined easily. From this regard, the third and fourth flow types require other instrumentation from different principal, e.g., Horizontal ADCP. Since this project focuses on the non-contact current meter, this study discuss about the flow of first and second types. In this project, two different flow sections were selected, such as Tone river with mild slope as well as Fuji river with very steep.

Finally as one of the outputs of this project, the authors proposed a technical guide line to assist river engineers to apply the system.

2. Methodology

2.1 Site specification in two different channel characteristic

To develop the automatic water discharge system, this project prepared two different channel characteristic, such as a middle reach of Tone river, as well upper reach of Fuji river. The observational site of Tone River has a bed slope of about 1/4,000, bed material of 1 mm with d_{50}, and a maximum flow capacity of 20,000m^3/s. This section had a compound channel with a 300m long floodplain and a 300m long main channel.

On the other hand, the observational site of Fuji river has river characteristics, such as a bed slope of about 1/200, bed material between 30 and 210mm, and a river width of about 250m including a main channel of 120m as well as a flood plain of 130m. At upstream of this site, many mountains with slope failure locate, as well as Sabo works have conducted inside of this catchment area. Because of sediment supplies, as well as steep bed slope, frequent river bed elevation change is recognized.

2.2 Non-contact current meter

With a same principal, there are two types of non-contact current meter, such as a horn antenna as well as a parabola antenna, which uses the Doppler effect to identify the surface water velocity with electromagnetic wave. One with a horn antenna is a portable type with specification of 24.15Ghz as well as a half power angle of about 12 degree. Similarly, one with a parabola antenna is a fixed type with specification of 10.525Ghz as well as a half power angle of about 10 degree. In the section of Tone River, seven sensors out of ten of the fixed type are located in the main channel. On the other hand, the section of Fuji River has eight cross sections in total width of 120m in the water-flow section. Single non-contact current meter with a portable type was employed observing frequency of 1Hz in 5 minutes. Thereafter the observation is conducted continuously at another section, until it measured at eight different points within one hour. Using those data, timely interpolated values were employed for determining value at an assigned time. In the current study, 8 section were setup to monitor the water flow velocity facing to downstream side.

Other two different characteristic need to be mentioned for operational purposes. Firstly, the system with the parabola include an automatic data transfer system after averaging data and outputting in every 10minutes. On the other hand, the system with the horn antenna outputs raw data with different time interval by the user's choice. Secondary, because of those two different types about outputting, the case with the parabola sometimes includes error data, which cannot be easily determined the reason. In this case, few data modification is required before obtained discharge value. On the other hand, error data with the horn antenna can be neglected with conducting high frequent measurement.

2.3 River-bed monitoring system

River-bed elevation change was monitored by two echo sounders fixed with oblique angle on H-type steels located on middle of water flow. The echo sounders have specification of 200 kHz, with time interval of 0.2 seconds. 10 seconds measurement was conducted with every 10 minutes, and saved as one single value after averaging them. The beam of echo sounder should be located high enough to illuminate the spot which is outside of wake flow generated by the H-type steel itself; at the same time, it should be low enough to frequent measurement, since measurements cannot start unless a transducer sinks to the water. Therefore, the location of the echo sounders should be carefully decided considering designing the magnitude of flooding. For this set up, the targeted discharge is about 1 time in a year for the echo sounders in the main channel. Also, in this study, river-bed elevation change was located upstream side of bridge.

2.4 Determination of velocity index as automatic system

To convert surface velocity to averaged vertical velocity, float coefficient is usually applied. In the case of an automatic measurement system, velocities at certain depth are measured. For example, non-contact current meters or PIV/STIV always measure velocity at water surface. On the other hand, H-ADCP takes measurements at varying points depending on water stage. To be used for any type of devices, velocity index α, instead of float coefficient, needs to be introduced. In this paper, the authors derived α starting from the logarithmic law as follows:

$$\frac{u(z)}{u_*} = \frac{1}{\kappa}\ln\left(\frac{z}{k_s}\right) + A_r \tag{1}$$

where u(z) is velocity at the water depth of z, u_* is shear velocity, k (= 0.4) is the von Karman constant, ks is Nikuradse's sand roughness and Ar (=8.5) when the bottom is rough enough. When Eq. (1) is applied to an actual river, ks and Ar include some uncertainty since those parameters are determined in the flat bed condition. Instead of relying on an uncertain assumption, the authors modified Eq. (1) by using measurable values as follows:

$$\frac{u(z)}{u_*} = \frac{1}{\kappa}\ln\left(\frac{z}{\beta h}\right) + \frac{U_{\beta h}}{u_*} \tag{2}$$

where h is water depth, β is relative water depth ranging between 0 and 1, and $U_{\beta h}$ is velocity measured by a device. When β is equal to 1, $U_{\beta h}$ is velocity at water surface. Eq. (2) is composed of measured values except k. Even if k changes with unsteady flow condition, k = 0.4 is used since the variance is small enough[12]. After integrating Eq. (2) from river bed to water depth and dividing it by $U_{\beta h}$, then a correction coefficient will appear as:

$$\alpha = \frac{U}{U_{\beta h}} = 1 - \frac{(1+\ln\beta)}{k \cdot U_{\beta h}} \cdot \sqrt{ghI} \tag{3}$$

where U is averaged vertical velocity, I is free surface slope. As the equation shows, the velocity index can be determined once velocities at a certain point as well as free surface slope are measured. Even if Eq. (3) is considered as reasonable, α should be compared with the actual velocity distribution for verification purposes, since a technique for free surface slope measurement in actual rivers has not completely established yet. Comparison between ADCP measurement and Eq.(3) will be discussed in the following section.

In this paper, the next equation was also derived for future use,

$$U_{\beta h} = \frac{U}{\alpha} = \frac{(1+\ln\beta)}{(1-\alpha)\cdot k}\sqrt{ghI} \tag{4}$$

Basically, this equation is similar to Chezy's equation, though the shape of its coefficient term is different. The paper uses this term to apply α.

2.5 Estimation of velocity index from ADCP measurement

The velocity index can be determined by actual velocity measurement by ADCP. Firstly the logarithmic velocity profile with Eq.(1) is determined by observed velocity value in vertical direction applying the least square method to find u*, ks, and Ar assuming k is constant. Secondary, assuming that velocity distribution at upper part follows the logarithmic profile, the velocity at upper unmeasured zone extrapolated. Similarly, assuming velocity distribution at lower part of water depth followed the power low, the velocity at bottom unmeasured zone extrapolated as well. Finally, ratio of mean velocity taken an averaging in vertical including observed/estimated velocity and extrapolated velocity at water surface, the velocity index can be determined. For the future use, the estimated velocity at water surface will be used to compare the velocity obtained by non-contact current meter.

2.6 Measurement of true discharge

ADCP measurements mounted on the tethered ADCP plat form was proposed by Michel et al. 2003[13]. However, it was not easy to apply them in a mountainous area (e.g., most of Japanese rivers) because of rough water-surface vibration; therefore, a research group including authors have developed water discharge measurement system with ADCP, which is capable enough even in a severe measurement condition. Based on the knowledge as well as technique, authors conducted ADCP measurement with RTK-GPS, and a high speed river boat[14]. The Work Horse ADCP with 1,200 kHz by Teledyne RD instruments were employed with commands showing in Table1. Additionally, ADCP measurements were conducted in downstream side from bridge.

Table 1 Commands of ADCP

Bottom truck	BM5: 5
Pings for Bottom	BM3: 3
Pings for Water	WP3: 3
Type of band	WB0: Broad band
Distance for first blank	WS25: 25 cm
Mode of measurement	WM12: high speed mode
Number of layer	WN40: 40
Thickness of layer	WS25: 25 cm

2.7 Determination of water discharge

Table 2 Cases of measurement system

method	velocity	velocity index	River bed
Case 1	Raw	constant(0.85)	constant
Case 1#	Modified	constant(0.85)	constant
Case 2	Raw and modified	constant(0.85)	update
Case 3	Raw and modified	updated by ADCP	updata

To compare different ways of determining water discharge, traverse measurement with ADCP, as well as three different methods were compared as summarized in Table 2. The discharges in the each section were calculated by multiplying velocity from non-contact current meters, the velocity index, and water depth; then, discharge in each sections were summed for obtaining total discharge. In the table, each term indicates as follows: “raw” for velocity obtained is employed the values obtained by non-contact current meters; “constant (0.85)” for velocity index is constant value: "measured by ADCP" for velocity index is employed the values obtained at previous ADCP measurements; “constant” for river bed employed bathymetry survey before flooding, while “update” employed the bathymetry which was obtained at previous ADCP measurements.

3. Results

To illustrate general ideas of the current study as well as observational section, flow pattern, and etc, Fig.1 indicates one of examples of observational results, such as velocity distribution, river bed elevation, and location of bridge pier and echo sounders. The horizontal, vertical, secondary vertical axes indicate distance from the left river bank in m, water velocity in cm/s, and elevation for riverbed and water surface in m, respectively. Among the marks in this figure, the filled circles are for WS velocities converted by the ADCP measurements; vertical-dotted lines for section boarders with section number from 1 to 8; triangles for section averaged values of the filled circles; open-circles for WS velocities measured with non-contact current meters; diamonds for locations of the echo sounder, such as B1 and B2; BP for locations and width of the bridge pier; solid curve for river bed obtained by the ADCP measurements; a broken line for river bed surveyed few years ago; and a dotted line for water surface elevation.

As this figure indicates, the velocity distributions (filled circles) were widely and locally distributed with affecting by the bridge piers, as well as by local turbulence. Especially, flow behind bridge pier is concaved, though concaved parts is not exactly located at that of bridge pier. This can be explained as the flow in this river section has few angles from left to right. Therefore, not only location of the concave by the wake flow, but also location of sediment deposition also occurred at the same place. As comparison between triangles as well as open-circles indicates, both of them are not so exactly identical but few difference within acceptable range, though few sections might has slight difference. For example, left/right most section; section 1 and 8, have edge of flood flow. Since velocity distributes widely in the section with local geometry, representativeness of the point measurement of open-circle in the section is not

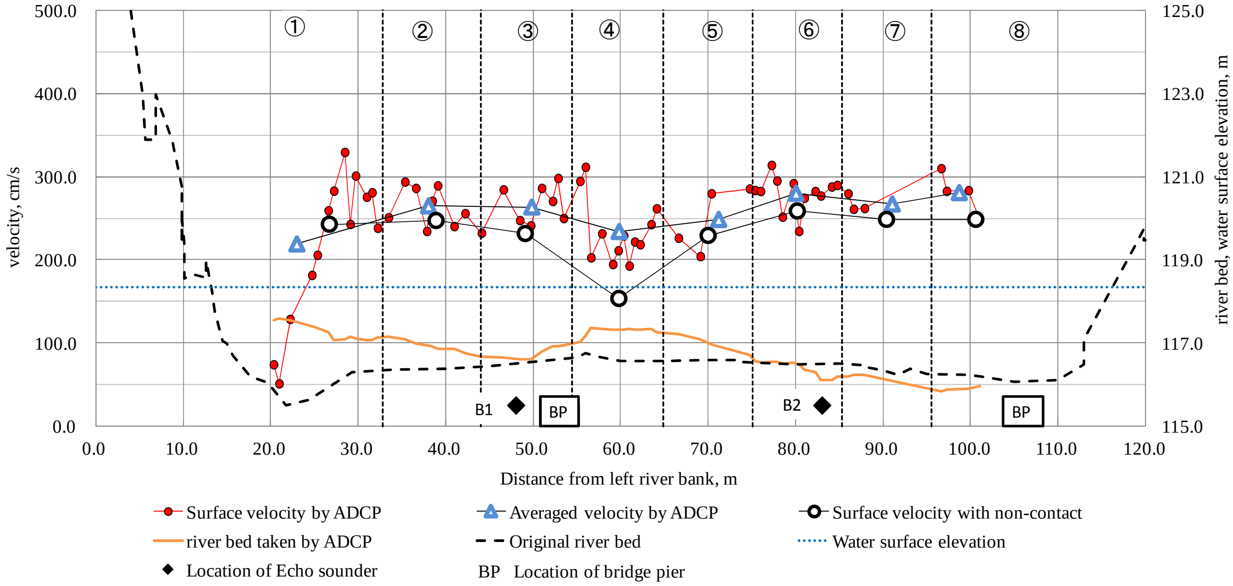

Fig. 1. Cross sectional distribution of velocity and river bed

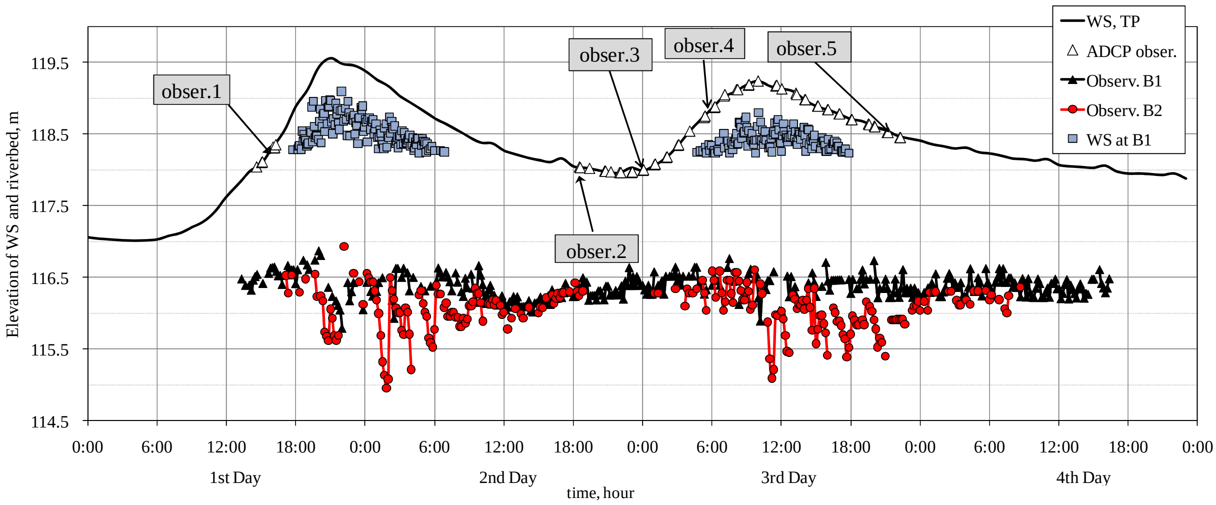

Fig. 2. Time series of river bed elevation, water surface elevation, and timing of ADCP measurements

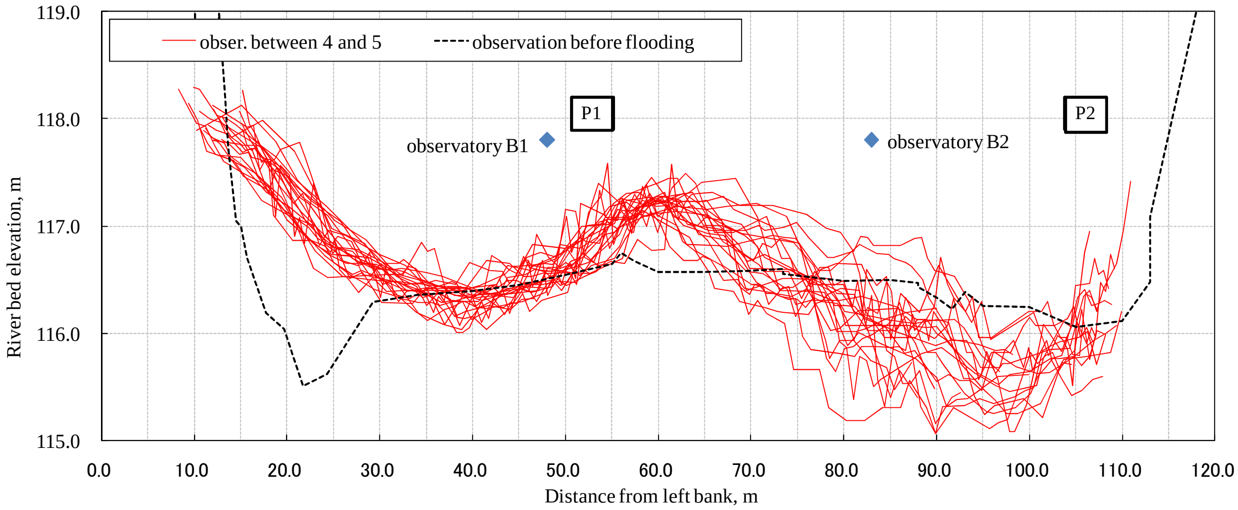

Fig. 3.　River bed elevation at different timing using ADCP

easy to satisfy. Fortunately, the value at section 8 is appropriate, since almost no flow was acknowledged around area of 120m because of another bridge pier. In addition, open-circle in the section 4 is also different compared with triangle, possibly observation period is different in an unsteady wake flow from the bridge pier. For obtaining the discharge values for both unmeasured area, the information of original riverbed with broken line is used to determine the area of stream. Firstly a point of contact is determined with the dashed line and the broken line; thereafter, the straight line links in between the point of contact and the edge of the river bed obtained by ADCP. The area among those three lines is determined as the area of stream. Representative velocity at this area is defined by multiplying 0.707 to observed values at right/left most edge. Thereafter, discharge can be determined[15].

One of the most significant characteristics of the current study is an active vibration of river bed elevation, which is generally acknowledged in mountainous are. Here, the Fig.2 shows timely change of riverbed elevation observed by Echo sounder at observatory of B1 and B2 indicated in Fig. 1, water surface elevation at left bank and observatory B1, and timing of traverse measurement with ADCP. During this observation, two flood waves passed through the section. As Fig.2 shows, water surface elevation started to increase at 6:00 of first day, it reached at the first peak around 21:00 with increasing about 2.5m in water depth. After water surface elevation decreases till 118.0m, next peak came with slightly less discharge compared with first peak. The water discharges are about 1,000m3/s and 800 m3/s for the first and second peaks, respectably. The monitoring of the river-bed elevation by observ.B1 started after WS elevation increased about 117.7m, while observ.B2 started about 118.5m. At depression time between two flood peaks, the transducer of observ.B2 quit monitoring, since WS elevation is too low. Thereafter, it started monitoring again after increasing of water depth.

As for observ.B1, sediment deposition continued till the peak of flooding. After peak discharge pass, river bed elevation decreased about 1m with vibrating

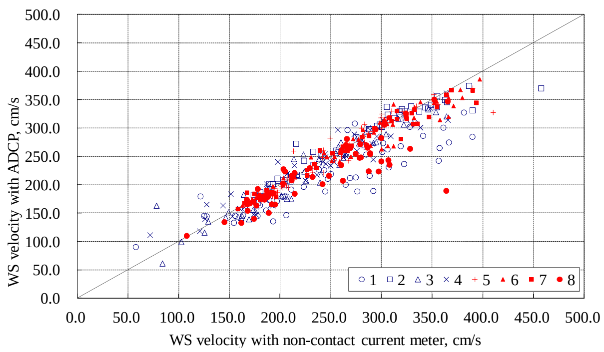

Fig. 4. Comparison of the WS velocity between ADCP and non-contact current meter

within the similar range till the WS elevation decrease. Starting at 12:00 of 2nd day, sediment deposition continues resulting about 50cm accumulation in next 12 hours with decreasing of water depth. After next flood came, it shows similar trend with the first one; thereafter, it came back to the initial level. As for observ.B2, the relation between WS and riverbed elevation shows similar trend with that of observ.1. Slightly different characteristic is that magnitude of vibration of river bed obtained by B2 is higher than B1. Common characteristic for both of them is that minimum river bed height appears not at the peak of water surface elevation but after few hours. Authors expected that area of flow section have simple-increasing function with WS elevation, such as degradations/aggradations corresponding with increasing/decreasing of water discharge; however, actual phenomenon in this case was not so simple, e.g., bed forms was developed and pass though the observational section.

Previous figures describes about river bed fluctuation at the two points. In this paragraph, cross sectional changes of riverbed obtained by ADCP traverse measurements are discusses with Fig.3, whose horizontal axis shows distance from left bank, and vertical axis shows river bed elevation. A dashed line is a result of a cross sectional survey before flooding, while solid lines are results obtained by ADCP, whose timings are in between obser.4 and obser.5 indicated in Fig. 2. Also, location of observatory B1 and B2 as well as two bridge piers are illustrated in this figure.

Fig.3 indicates two sections in this water flow divided by bridge pier. As for the left side water flow, river bed fluctuations during flooding are in between a slip whose width has about 0.5m. On the other hand, the fluctuations are within a slip of about 1.5m for the right side channel. It should be noted that the size of the two slips are almost same as the magnitude of bed form height in Fig.2. Unfortunately, it is not easy to discuss more about river bed fluctuations from both observational results, since observation sections for both of them are separated about 40m. However, still it is very valuable that the magnitude of the river-bed fluctuations were affirmed from both different observational results.

As Fig.1 indicates one of the results, Fig.4 is whole sets of observational results in the current study. Horizontal and vertical axis indicates the WS velocity values obtaiend by non-contact current meter, as well as one estimated by ADCP, respectively. Numbers as caption from 1 to 8 indicates the section number explained in Fig.1. As it shows, overall, those values are almost identical, though slightly different values can be acknowledged. As already explained previously in Fig.1, three section; 1, 4 and 8, might show some differences because of local velocity distribution. As a matter of fact, values of non-contact current meter at section 1 and 8 is slightly higher than that of ADCP; though, values of section 4 fortunately are not so inappropriate. If magnitude of flood increase, size of seciton 1 and 8 increase. In that case, other observational points should be supplimented. Other than those sections, authors could couclude that the non-contact current meter is accurate enough to obtain representative WS velocity value. As another steps for water discharge measurement, velocity index should be discuss to obtaine depth averaged velocity.

Fig.5 and 6 indicate velocity index in horizontal axis, as well as non-dimensional shear stress. Non-dimensional shear stress was estimated using shear velocity obtained by ADCP as slope of logarithmic profile, as well as averaged sediment size as d_{50} = 1mm and 30mm which are reported by the river management office of Tone river and Fuji river, respectively.

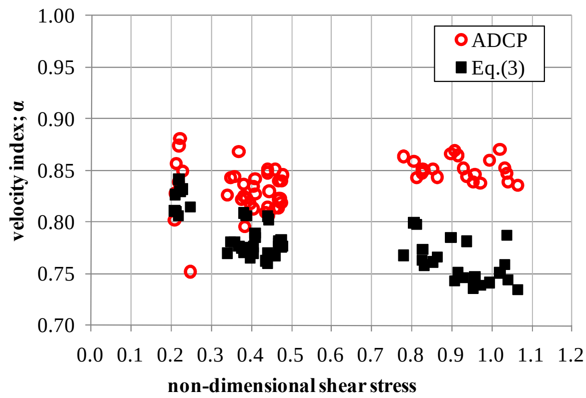

Fig. 5. Relation between non-dimensional shear stress and velocity index by ADCP in Tone River

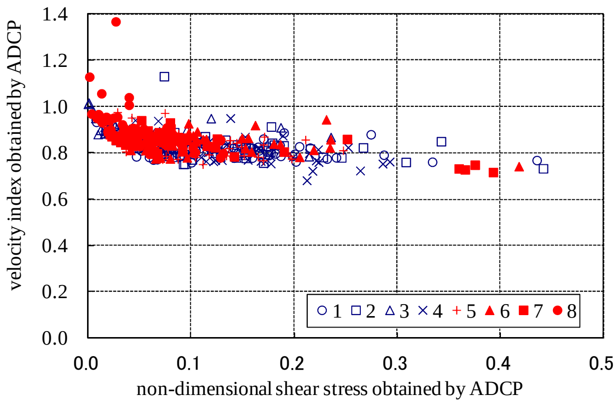

Fig. 6. Relation between non-dimensional shear stress and velocity index by ADCP in Fuji River

Fig. 5 in the case of Tone river shows two correction coefficients α. The open circles indicate α estimated by ADCP, while the squares indicate α estimated by Eq.(3). Fig.5 shows comparison of shear velocities from ADCP and free surface slope. The shear velocities on the horizontal axis were calculated based on the logarithmic profile. There were three groups in each α, whose groups can be classified as a different flooding case. As the α from ADCP indicates, they scattered from 0.75 to 0.88 when the discharge was small, while they group together as the discharge increased; finally, it converged to 0.85. On the other hand, the α by Eq.(3) decreased with increase in non-dimensional shear stress. Even though the α from Eq.(3) looks promising compared with those in other studies[16], the authors let the discussion postpone to further study, since types of bed forms during each observation are not clear.

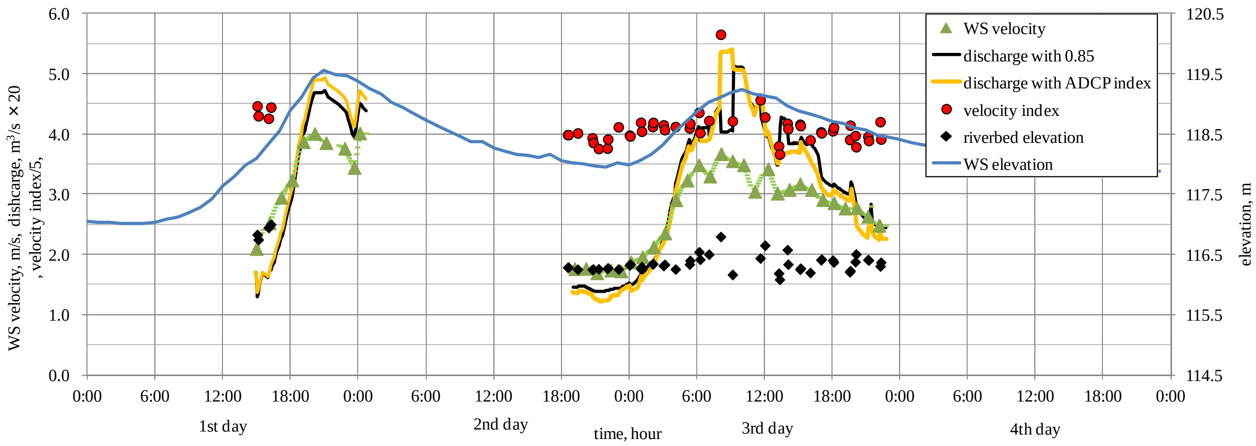

Fig. 7. Time series of river bed elevation, WS elevation, WS velocity, velocity index obtained by ADCP, discharge with velocity index of 0.85 and that by ADCP at section 2

As the Fig. 6 shows, the scattered values are located more when x-axis are smaller. As x-axis becomes larger, range of scattering becomes narrower, as well as deviation of the velocity index becomes smaller. The mostly populated area in this figure is in between 0.8 and 0.9 for velocity index when x-axis is in between 0.0 and 0.1. The secondly most populated area is around 0.8 when y-axis is in between 0.1 and 0.2. As the x-axis increases, it merges to about 0.75 with few populations. Compared with other study, such as Costa et al. 2000, Yorozuya et al. 2010, and Polatel 2006, the number of the current study is slightly different. Detail discussion will be followed in next paragraph.

Fig. 7 shows time series of water surface elevation as well as river bed elevation averaged the observed values in the section 2 obtained by ADCP, WS velocity obtained by the non-contact current meter, discharges within the section, and velocity index. As Fig.1 indicates, the section 2 is one of downstream section from the river bed monitoring B1. Similar with the bed elevation change, the velocity index was estimated with averaging the observed values in the section 2 obtained by ADCP. The discharge with 0.85 obtained by applying the velocity index of 0.85 for obtaining the averaged velocity from the WS velocity by the non-contact current meter, the water depth obtained by the WS elevation as well as the river bed elevation, while the discharge with ADCP index applies the velocity index obtained by ADCP observation.

Fig. 7 indicates that, starting from 18:00 of 2nd day, as water surface increase, firstly the WS velocity increase until 6:00 of 3rd day, secondary it start to fluctuate with range about 25cm until 13:00 of 3rd day, and finally it continuously decreases. On the other hand, river bed elevation changes in this section shows similar trend with obser.B1 indicated in Fig.2, in terms of the shape as well as the magnitude of sand wave. Correspondence between the WS velocity and the riverbed elevation implies that sand wave generated at the bottom of river bed affects to the WS velocity, though WS elevation does not show any similarity, since they are located at the river bank. The velocity index has number in between 0.75 and 0.85 when bed form does not exist. When bed form generated between 6:00 and 15:00 of 3rd day, the velocity index vibrates between 1.1 and 0.73. Those numbers with the bed from are slightly larger than other study. For example, Polatel (2006) conducted experimental studies as well as numerical computation with different types of roughness. In this study, she obtained the velocity index between 0.85 and 0.908 on roughened bed. On the other hand, Hino and Miyanaga (1977) shows the vertical velocity distribution at the different location of sand wave; e.g., trough, crest and in between[17]. They shows that velocity distribution at the crest is almost vertical or sometimes overshooting type which indicates the velocity index around 1.0, while that at the trough is

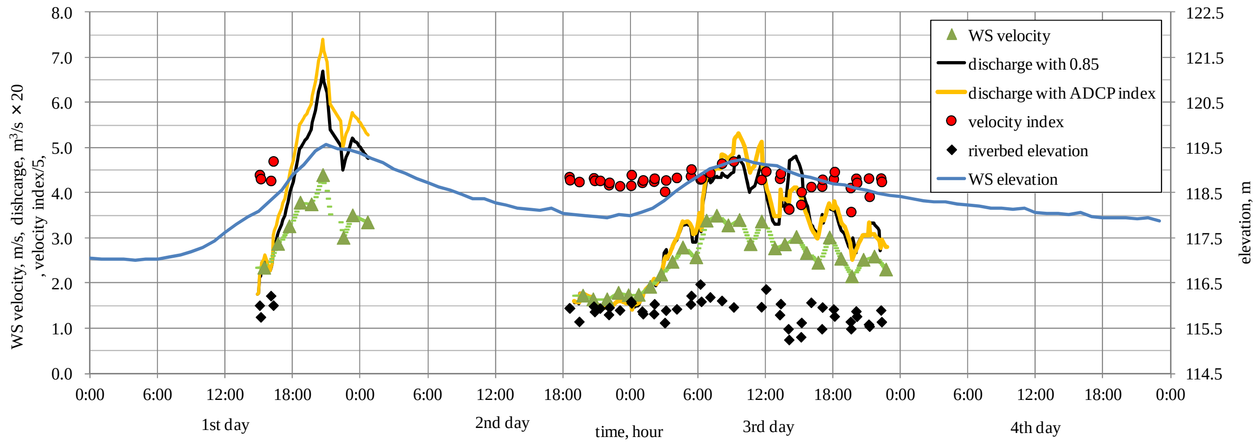

Fig. 8. Time series of river bed elevation, WS elevation, WS velocity, velocity index obtained by ADCP, discharge with velocity index of 0.85 and that by ADCP at section 7

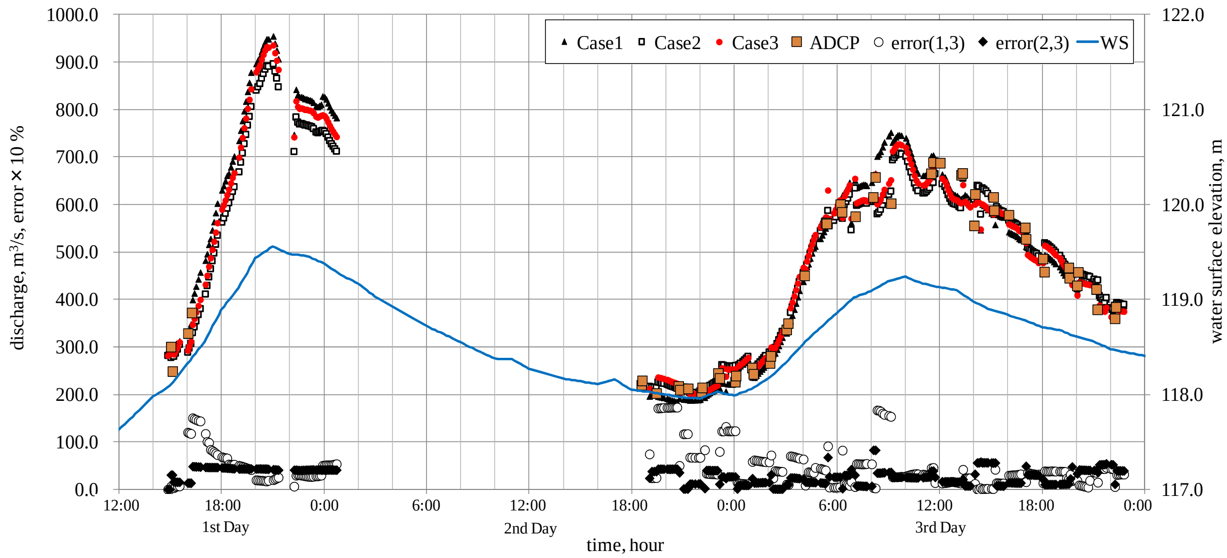

Fig. 9. Time series of water discharges with different methods, water surface elevation, relative errors of Fuji river

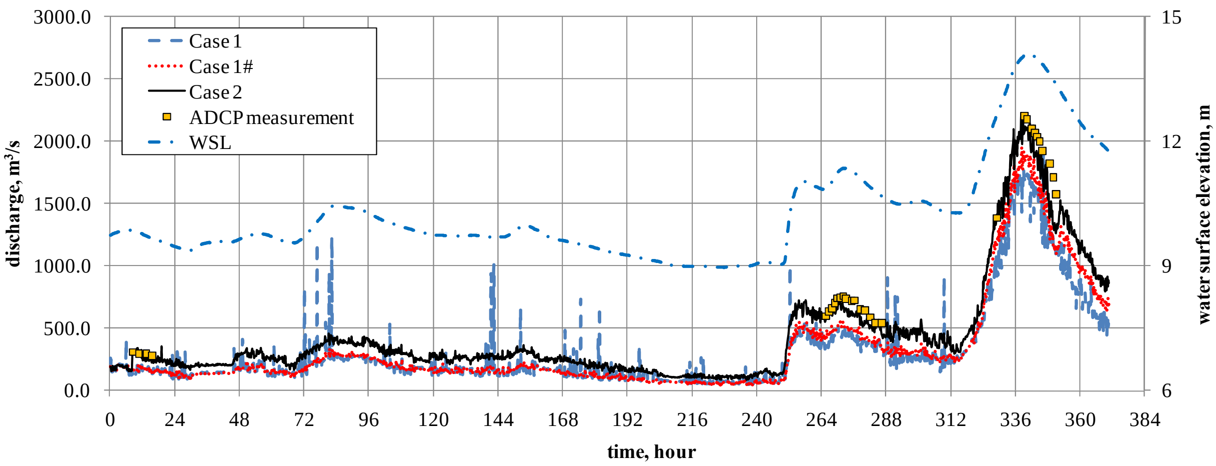

Fig. 10. Time series of water discharges with different methods, water surface elevation, relative errors of Fuji River

sometimes overshooting type which indicates the widely distributed including the reverse flow which has possibility of the velocity index of less than 0.8. Correspondingly, two discharges are directly related to differences of velocity index; for example, they are not much different when the velocity indices are around 0.85, while 24% and 16% difference occurs when the velocity index is 1.13 and 0.73, respectively.

Fig. 8 indicates the variables obtained at section 7 which is downstream of oberv.2 with the same format and contents of Fig.7. As Fig. 8 indicates, the riverbed vibration in this section is more severe compared with Fig.7 but somehow similar with the results of the river bed monitoring B2 in Fig. 2 in terms of shape of wave as well as magnitude of wave height. Actually, both of them have deposition toward the peak of the WS elevation, and higher oscillation during recession phase within the range of 1.5m. In addition, the WS velocity show similar trend with those in Fig.6, but more dynamically vibrate with corresponding to the river bed fluctuation. On the other hand, most of the velocity indices in between 18:00 of 2nd day and 23:00 of 3rd day idles around 0.85 within the range of ±0.05. Once generation of the bed forms starts, the velocity index correspondingly vibrates.

Fig.9 shows time series of water discharges and water surface elevation, as well as relative error. The horizontal axis indicates time in hours starting at 12:00 of the first day ending 24:00 of the third day, the vertical axis of left side indicates discharges/relative error, as well as the water surface elevation. Three discharges explained in Table2, as well as ADCP measurements were conducted at the timing where those dots plotted. Assuming the discharge of case3 is true value, relative errors were calculated for case 1 as error(1,3), as well as for case 2 as error(2,3). Those errors are multiplied by 1,000 showing % with multiplying 10 in the left vertical axis (e.g., 100 in this figure means 10%). As they indicate, measurements started above normal stage of discharge with a magnitude of about 300 m3/s. After one ADCP measurement was conducted, the river bed elevation was incorporated for all cases in table 2.

For the case 2 and 3, another river bed by next ADCP observation is updated. After first four ADCP measurements, no ADCP observation was conducted, which means that no information was updated. After 18:00 of 2nd day, 2 times in 1 hour measurement was conducted.

As Fig.9 indicates, discharge value obtained by ADCP and case 3 are almost identical. Very few different occurs because of differences between WS velocities indicated in Fig.2, as well as treatments of the unmeasured zone; otherwise, we could consider that the case 3 is accurate enough. Concerning about the second flood, all three cases of discharges have common characteristic. The curves of discharge are smooth during rising stage, they started to be zigzag, and keep as it is with being less amplitude as times goes by. Those trends are very similar with the fluctuation of river bed in Fig.2. On the other hand, time series of relative error; error(1,3), indicates that about 0 to 10%, as well as few over 15% appears, while error(2,3) indicates about 0 to 5% with few over 5%.

Fig.10 shows time series of discharge and water surface elevation, as well as discharge results by ADCP measurement. As it indicates, measurements started from the normal stage of discharge with a magnitude of about 300 m^3/s. Around 250 hours, as water surface increased, discharge reached more than 500 m^3/s. Another flooding started from 312 hours, and then reached the maximum discharge of around 2,000 m^3/s at 336 hours. Almost all the discharge measurement systems detected a similar volume of discharge, though several different aspects need to be mentioned as follows. Case 1 has many undesired spikes because of some measurement errors. Also, a 20% less discharge was observed at the maximum peak. On the other hand, Case 1# indicates improvements in terms of eliminating spikes through modification based on Eq.(4). At the maximum peak, a 15% less discharge was observed, which is 5% better than Case 1. Case 2 shows the most accurate trend compared with the ADCP measurements. Though small spikes are still present, the peak discharge is only 5% difference.

4. Guide line proposed by this project

Based on the knowledge explained in this paper as well as other knowledge from previous study, the project proposed the a technical guide line to assist river engineers to apply the system. Here in this paragraph, the authors would like to introduce the guide line with showing the list of content.

(1) Overview of fixed type water velocity measurement
 (1-1) Introduction
 (1-2) Types of fixed type water velocity measurement
 (1-3) Characteristics of fixed type water velocity measurement
 (1-4) Overview of the water discharge measurement system with fixed type water velocity measurement
 (1-5) Verification of the water discharge measurement system with fixed type water velocity measurement
 (1-6) Periodical field survey of bathymetry
 (1-7) Estimation of the velocity index
 (1-8) Modification of wind effect
 (1-9) Arrangement of observatory
(2) Radio non-contact current meter
 (2-1) Selection of observational points
 (2-2) Selection of observational section
 (2-3) Water discharge measurement
  (2-3-1) Determination of water discharge value
  (2-3-2) Accuracy verification of water discharge
 (2-4) Quality assurance of water discharge value
 (2-5) Management of observed value
 (2-6) Maintenance of observatory

5. Conclusions and Discussion

In this paper, authors discussed about possibility of the automatic water discharge measurements with three different components, such as water velocity, velocity index, and river bed. In this study, authors examined the observational results of flood events obtained in actual river with verifying the observed values as well as explain about the hydraulic phenomena observed by the several instrumentations. Firstly, the water velocity obtained by non-contact current meter was assured that the measurement was accurate enough as shown in Fig.4. Additionally, correspondences between the bed form and WS velocity with non-contact current meter were recognized as shown in Fig.6 and 7. Secondary, the velocity index obtained by ADCP indicated values in between 0.7 and 1.0 as shown in Fig.5, which is slightly different from previous study. For the further understanding, the velocity index incorporated with bed form was described as shown in Fig.6 and 7 indicating that the vertical velocity distribution were mainly affected by local geometry. Thirdly, river bed elevation observed by ADCP as well as the echo sounders indicate very active river bed fluctuation as well as movement of bed form with amplitude of about 0.5m in the left side of water course, as well as 1.5m in the right side as shown in Fig.2 and 3. Finally, authors compared three different methods for obtaining the water discharge value as indicated in Table 2 with employing the above knowledge. Among those three, major sources of error were related to the river bed elevation as shown in Fig.8. Authors would like to be solved this problem with combining the numerical simulation and the river bed monitoring in the future. However in current study, author's proposal to the automatic system is employing the ADCP traverse measurements among the automatic measurement instruments, though authors do not satisfy yet; since the ADCP measurement is not automatic system. Before conclude this paper, authors would like to keep discussing about the velocity index as well as water discharge measurement system in the following two paragraphs.

To determine the velocity index in the automatic system, Yorozuya et al (2010) introduced a theoretical method combined with observational results, based on two assumptions. Firstly, shear velocity obtained by water depth as well as water surface slope is identical with that obtained by logarithmic law. Actually, u* of the logarithmic profile can be considered as parameter to determine the vertical velocity distribution, e.g., when u* = 0, vertical distribution is constant; therefore, the velocity index is unit. Similarly, as u* is larger incorporating with Ar and ks, vertical velocity distribute wider as well; therefore, the velocity index has some number, though usually 0.85 is applied in

Table 3. Summary of each discharge value

Case	Strengths	weaknesses	Error
1	･very simple, and easy to apply. ･already operated	･many assumptions involves	possibly from 0 to 20%
2	･No need to use expensive instruments (only depth sounder could be enough)	･Human labors are required	possibly from 0 to 10%
3	･Most accurate	･Expensive in terms of human labor as well as instrumentation	very less

Table 3. Optimum method of automatic water discharge measurement with different flow types

flow type	comments
1) high energy water flow with very active river bed fluctuation	non-contact current meter for WS velocity as well as river bed monitoring is necessary; Case 2 in Table3 is most suitable. ADCP measurement may not be able to conduct. Automatic riverbed monitoring system may be helpful.
2) water flow involving bedload with slight river bed fluctuation	non-contact current meter for WS velocity, updating of index velocity as well as river bed monitoring is necessary; Case 3 in Table3 is most suitable. ADCP measurement most likely is able to conduct.
3) mild flow with few river bed fluctuation	non-contact current meter can be applied for WS velocity; Case 1 in Table3 is most suitable. ADCP measurement should be conducted for determining velocity index before regular operation
4) flow with tidal influences sometimes involving salt wedge	velocity index should be examined as function of tidal effect. WS velocity might not be suitable as representative velocity measurements

hydraulics community[3]. Second assumption is related to the size of hydraulics phenomena. As for the shear velocity of water depth/slope, scale of the shear velocity is determined in this few hundred meters. On the other hand, the shear velocity of the logarithmic profile has the scale of several meters. Therefore, the second assumption in this discussion is that two shear velocities in different scale should be identical. In the case of the channel section involving the several different bed forms, the second assumption cannot be conserved, since vertical velocity profile is more sensitively determined by the local bed form. Therefore, the method by Yorozuya et al (2010) has little difficulty to apply it in such as channel characteristic. Actually, as Fig.6 and 7 indicated, the velocity index varies corresponding to riverbed elevation change. In this case, the determination of the velocity index as an automatic discharge system cannot be easily constructed. River bed monitoring system might be one of the indicator, since 1) it can indicates a relative location at the bed form as well as 2) vertical velocity distribution can be estimated as the function of the relative location. As the first estimation, the experimental study, e.g., Polatel 2006 should be carefully studied. Thereafter, field measured data by ADCP measurement should be accumulated, and compared to both the experimental and field studies.

Table 3 shows strengths/weaknesses as well as accuracy of each automatic discharge measurement system, explained in Table 2. As it shows, Case 1 is already developed as automatic system, though still it includes considerable amounts of errors. Case 3 is most accurate, though it requires high cost in terms of labor as well as instrumentation. On the other hand, Case 2 is in between other two methods, which still requires the human labor; though the cost for the instrumentation is much less, since echo sounder can be alternative to ADCP. Concerning about a selection of those three different methods, river administrator need to understand the channel characteristic under their management compensating with their budget and required accuracy. For example, suppose their channel involve considerable amount of river-bed deformation, Case 2 or case 3 should be selected. If the ADCP measurement involves some difficulty or error of 10% can be acceptable, then Case 2 with echo sounder should be selected. On the other hand, suppose their channel has no bed-deformation but complex flow field, e.g., flow affected by tide, the vertical velocity distribution should be carefully investigated; therefore, Case 1 should be selected.

Table 4 indicates an optimum method of automatic water discharge measurement concerning about different flow types. The discussion only about first and second flow is output of current study.

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重.25　無人自動流量観測技術と精度確保に関する研究

研究予算：運営費交付金（一般勘定）
研究期間：平 21～平 23
担当チーム：水災害研究グループ
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【要旨】
非接触型流速計（電波式流速計）を活用して連続的な河川流量を計測する無人自動流量観測システムを構築する技術を確立することを目的として、電波式流速計による表面流速値および実測の水面勾配から電波式流速計の区分断面内平均流速に変換する流速補正係数を算出するための理論モデルを提案した．また，そのモデルを現地の電波式流速計システムに適用し，Acoustic Doppler Current Profiler（ADCP）による流量観測データとの比較検証を，河道特性の異なる二つの観測地点において行った. その結果, 電波式流速計は実用に資する精度を確保していることを確認したものの, 上記モデルにより水面勾配を用いて算出した流速補正係数は ADCP 計測結果から算出されるものと必ずしも一致せず, 水面勾配の計測方法等に課題があることを指摘した．また急流河川における流速補正係数の算出では,河床波の存在に大きな影響を受けるため,水面勾配のような 500m 程度のスケールに支配される物理現象ではなく,数 m 程度の水深スケールに支配される現象であることが理解できた．これらの知見を元に,異なる河道特性に応じた自動流量観測手法を提案した．さらにこれらの知見から自動流量観測に関するガイドラインを作成した.

キーワード：自動流量観測，非接触型流速計，ADCP 計測，流速補正係