The Assessment of Stream Discharge Models for an Environmental Monitoring Site on the Virginia Tech Campus

by

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ABSTRACT

In the Spring of 2012, hydraulic data was collected to calibrate three types of discharge models: stage-discharge, single-regression and multi-regression index velocity models. Unsteady flow conditions were observed at the site \( \frac{\partial H}{\partial t} = 0.75 \text{ cm/min} \), but the data did not indicate hysteresis nor variable backwater effects on the stage-discharge relation. Furthermore, when corrected with a datum offset \( \alpha \) value of -0.455, the stage-discharge relation \( r^2 \) was equal to 0.98. While the multiple regression index velocity models also showed high correlation \( r^2 = 0.98 \) values, high noise levels of the parameter index velocity \( V_i \) complicated their use for the determination of discharge. Because of its reliability, low variance and accessibility to students, the stage-discharge model \[ Q = 5.459(H-0.455)^{2.487} \] was selected as the model to determine discharge in real-time for LEWAS. Caution should be used, however, when applying the equation to stages above 1.0m. The selected discharge model was applied to ADCP stage \( H \) data collected during three runoff events in July 2012. Other LEWAS models showed similar discharge values (coefficient of variation = 0.14) while the on-site weir also produced similar discharge values. Precipitation estimates for July 19 and 24 rain events over the Webb Branch watershed were derived from IDW interpolated rain data and rainfall-runoff analyses from this data yielded an average ratio of 0.23, low for the urbanized watershed. However, since the three LEWAS models were very similar, and the on-site weir showed a lower value to LEWAS, it was concluded that any error in the ratio would be attributed to the precipitation estimate, and not the discharge models developed in this study.
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1. INTRODUCTION

A LabVIEW Enabled Watershed Assessment System (LEWAS) has been under development in the Department of Engineering Education at Virginia Tech since 2008. LEWAS is designed to advance undergraduate education through real-time stream monitoring and to provide a viable platform for water quality research in small urban watersheds. Figure 1.1 depicts the relationships between and function of the various disciplines that comprise the LEWAS research group.

The LEWAS lab, with its suite of stream and weather monitoring equipment, will serve the educational needs of the local and global community, providing weather and water quality data to educators and researchers within Virginia Tech and elsewhere.

Real-time mass flux and water quality assessments require accurate stream discharge measurements. Errors in mass flux measurements (sediment delivered, nutrients delivered, etc.) are a function of the accuracy of the discharge measurement. Therefore, in order for the LEWAS station to produce research quality data, it is crucial to utilize the most accurate means of discharge measurement currently available, and one that meets the particular needs of LEWAS outdoor site.

The primary goal of the LEWAS research group is to develop a model that delivers discharge and other environmental data in real-time from the LEWAS site on the Virginia Tech Campus.
1.1 BACKGROUND

1.1.1 STROUBLES CREEK WATERSHED – A SHORT HISTORY

Stroubles Creek Watershed (Figure 1.2) was first colonized by the Draper’s Meadow community in 1740 and in 1798 became the town of Blacksburg, VA. Mostly agricultural, the community lived close to the three springs that still feed Stroubles Creek today. In 1872, the Virginia Agricultural and Mechanical College was established and became an impetus for more population growth and urbanization in Blacksburg. The last half century has seen the most significant increase in urbanization for the township (Younos 2010).

In 1937, a dam was erected where Main and Webb branches met and created what is known as the duck pond (Younos 2010). It separates Stroubles Creek Watershed into separate sections, the Lower and Upper Stroubles. Upper Stroubles, containing the Virginia Tech campus and the Town of Blacksburg, VA, is much more urbanized than Lower Stroubles, which is largely agricultural and forest.

Figure 1.2. Stroubles Creek watershed (approx. 25 km²) and LEWAS Lab location. Modified from: Yagow (2006) Used under fair use, 2013.
1.1.2 URBANIZED WATERSHED WATER QUALITY AND QUANTITY ISSUES

Urban watersheds, like the upper Stroubles, have unique hydrologic issues. One very pronounced effect of urban watersheds is the changes in hydrology urban streams experience. In urban watersheds, runoff is drastically increased due to larger areas of impervious surfaces (Wenger et al. 2009). Flood events from urban areas are of shorter duration than those of agricultural areas. The combination of these effects results in more runoff being conveyed through streams in a shorter period of time. This translates to higher shear stresses, bank erosion, and habitat degradation.

Urban areas also experience unique water quality impairments (Wenger et al. 2009). Low dissolved oxygen levels from high BOD concentrations, pesticides and nutrients from lawn applications, sediments from construction areas, and trash all can be found in urban streams (Lehr et al. 2005). In the upper Stroubles watershed, there have been numerous studies that have investigated water quality issues (Younos 2010). For instance, Knocke (1985) found both suspended sediment as well as conductivity significantly increased during rain events and Gronwald et al. (2008) found that Stroubles Creek was affected by nitrates from lawns, E. Coli, and other physical impairments as well.

The LEWAS monitoring site is located in an urbanized portion of the Stroubles Creek Watershed, the Webb Branch Sub-Watershed, on the Virginia Tech campus, just downstream of the culvert under West Campus Drive (Figures 1.2 and 1.3). This site features state-of-the-art instrumentation that monitors both water quality and quantity. Water quality parameters measured by a Hach Hydromet MS-5 Sonde include pH, temperature, conductivity, oxidation reduction potential (ORP), and turbidity. Water quantity, or discharge, is determined using a Sontek Argonaut – SW Acoustic Doppler Current Profiler (ADCP). These pieces of equipment are utilized to study the unique water quality and hydrologic characteristics of the Webb Branch watershed.
Real-time monitoring will allow the LEWAS lab to establish relationships between water quality parameters and land use changes in real-time. For instance, the effects of construction sites around campus may be monitored for sediment discharges into Webb Branch. Also, the water quality impacts of rain events (i.e., changes in suspended sediment, pH, conductivity, etc.) can be monitored and linked to point and non-point sources, and also to temporal parameters like rain intensity and duration. Long-term land use impacts may also be monitored, providing valuable data for land use planners, as well as town and state engineers. Finally, climate change effects may be studied, providing a window for researchers to examine the long term effects of this phenomenon on the urban environment.

1.2 PURPOSE OF THE LEWAS LAB

Real-time monitoring has a wide array of applications including natural resource policy and research, as well as public health and safety. The U.S. Geological Survey (USGS) states that real-time monitoring increases resource management effectiveness (Zeigler 2010). Estuaries like the Chesapeake Bay are heavily impaired by pollutants, impacted are ecosystems, the fishing industry, public health and recreation. Armed with better information through real-time monitoring, resource managers can make better informed decisions with these new real-time monitoring tools and protect these vital services.

The LEWAS Lab, along with other research groups such as the StREAM Lab (Dept. of Biological System Engineering, Virginia Tech) and Christensen et al. (2000) from the USGS, are eager to exploit real-time monitoring for its ability to collect data on a very high resolution time
scale. The following application examples from these research groups such as those mentioned are designed to illustrate which areas the LEWAS Lab may apply its resources towards. It is not intended to be an exhaustive review of real-time stream monitoring or an in-depth analysis of the potential applications for the LEWAS Lab.

1.2.1 REVIEW OF REAL-TIME WATER QUALITY MONITORING WORK

Further downstream of Stroubles Creek from the LEWAS Lab, the StREAM Lab operates an array of weather and stream monitoring equipment. This equipment includes water quality datasondes, groundwater monitoring wells, a full weather station, and on-site cameras (Thompson et al. 2012). The StREAM Lab endeavors to bring researchers together to conduct multidisciplinary research. Some of this research includes stream restoration, floodplain nutrient cycling, stream sediment monitoring and analysis, and watershed management (BSE 2012). Like the StREAM Lab, the LEWAS Lab could provide a multidisciplinary platform for researchers involved in similar research areas.

The USGS operates 7600 stream gages, providing both real-time and archived discharge data for a variety of uses. These uses include engineering projects such as dams and bridges, stream restoration projects, habitat assessments, water quality programs, and water resource allocations (Norris 2009). Real-time discharge measurements could be used to calibrate flash flooding models in urban watersheds. This would increase the accuracy of flood models and enable city officials to make better decisions during critical times. The LEWAS lab could enter into collaborations with Town of Blacksburg (TOB) engineers.
to create better flood forecasting models for the residents of this area. While TOB currently monitors five sites for storm sewer discharge and eight sites for precipitation, the LEWAS Lab could enhance their capabilities and modeling efforts.

Maintained by the USGS, The National Real-Time Water Quality program provides real-time data for sites all across the US. Water Quality parameters include temperature, dissolved oxygen, pH, conductivity, dissolved oxygen, turbidity, and stream discharge. This information is available on their website for a variety of applications to natural resource managers and researchers (http://nrtwq.usgs.gov/). The LEWAS lab, located in a small urbanized watershed (approx. 3 km²) will disseminate real-time data through the web to researchers and educators around the world, bringing high resolution data to researchers, teachers, students and institutions who cannot afford the price of equipment and installation.

While it provides real-time data to researchers, the USGS also conducts real-time research of its own, and at the same time informs natural resource policy. In Kansas (Christensen 2001), USGS researchers are utilizing standard sonde parameters (temperature, dissolved oxygen, pH, conductivity, dissolved oxygen, and turbidity) to calculate other water quality constituents such as alkalinity, suspended sediment concentration (SSC), nitrates and even fecal coliform. Similar studies can be initiated at the LEWAS Site. Correlating water various quality parameters in Webb Branch would provide interesting research while also providing instruction on how water parameters are related. One particular area of research in this direction will be to correlate turbidity measurements to suspended sediment concentrations (SSC). This would provide LEWAS the capability to monitor SSC’s in real-time. Suspended sediment in rivers and streams is natural, but can be problematic if excess amounts
are introduced into the stream system by land disturbances. In rural areas, these disturbances can take the form of deforestation and agriculture while in urban areas construction sites and bank erosion contribute mostly to sediment stream input (Wagner et al. 2007).

Virginia Tech maintains an office for Erosion and Sediment Control. As its name implies, its objectives are to control sediment from entering the Stroubles Creek stream system. This sediment input is largely due to the many construction projects occurring on campus (Figure 1.4). The LEWAS station positioned on the Webb Branch could monitor the sediment contributions in the Webb Branch Watershed and evaluate the effectiveness of erosion and sediment measures being implemented there by the university.

The USGS (Rasmussen and Survey 2009) provides a detailed procedure for calculating sediment concentrations and loads from turbidity and discharge measurements. LEWAS could provide sediment concentrations and loads in real-time. A simple regression between turbidity and SSC must first be established. Turbidity is measured in the stream while SSC measurements would take place in the laboratory. Figure 1.5 shows the regression from the USGS study. With an $R^2$ value of 0.99 for the regression, along with other validation methods, the authors cogently argue the viability of this method.

There are, in fact, many avenues and collaborations the LEWAS Lab may pursue in real-time weather and stream monitoring. And with its strategic location in an urban watershed, the LEWAS Lab has enormous potential to drive forward real-time environmental monitoring research as well as to provide innovative curricula that will enhance environmental sustainability education.
1.2.2 STATEMENT OF THE PROBLEM

While the LEWAS Lab has many exciting possibilities in its future, there is a fundamental component that must be settled before any meaningful water quality analysis is performed: an accurate and reliable method for the determination of stream discharge at the LEWAS site. The determination of stream discharge is fundamental in that it provides context to other water quality parameters. For instance, during first flushes a high concentration of pollutants is found in the first part of the rising limb on the hydrograph (Deletic 1998). An example of first flush data is Figure 1.6, collected in Mantes-la-Ville, France from storm water sewers. In this figure, a high concentration of sediment is clearly visible in the very first stages of the hydrograph. Accurately measuring pollutant load during this crucial period requires a very accurate method for discharge determination. Since LEWAS will be investigating various kinds of pollutant loads, determining discharge accurately is pursued as the first priority.

1.2.3 SELECTING THE RIGHT DISCHARGE METHOD FOR THE LEWAS SITE

As stated earlier, the key goal of this research is to explore various methods for the determination of discharge, and decide which method would best serve our research needs. There are two main categories to choose from. 1) Stage-discharge methods, which calculate stream flow from the stage, or height of the free surface from the channel bed, utilize an array of tools including pressure sensors and gas bubblers to determine stage; and 2) Acoustic Doppler Current Profilers (ADCP) technology, which uses sound waves to determine the velocity of small, suspended particles in the water column. ADCP instruments are also known as velocimeters. Upward looking ADCP directs these sounds waves upwards towards the free surface, allowing for the measurement of velocity along the entire height of the water column, making accurate
determination of the mean vertical velocity possible. Horizontal ADCP (H-ADCP) directs these waves cross the channel, elucidating the horizontal, rather than the vertical stream velocity distribution.

One of challenges the LEWAS site poses is its proximity to the Old Ice Pond (Figure 1.7). Since stage-discharge relations may become compromised by backwater effects, it becomes a very important consideration when developing a discharge model. The other challenge is that the LEWAS site experiences rapidly changing, unsteady flow, which may affect the stage-discharge relation by inducing hysteresis in the curve. Selecting a discharge model at the LEWAS site must bear these considerations firmly in mind.

Selecting the best discharge model is extremely important to the LEWAS goals and research opportunities outlined in this chapter. Whichever research endeavors LEWAS chooses to undergo, accurate and reliable discharge measurements will be a critical to the success of all of them.
2. **LITERATURE REVIEW**

2.1 **PURPOSE**

The purpose of this review, much like the purpose of this thesis, is two-fold. Its primary purpose is to research how previous index-velocity and stage-discharge relationship studies have been performed in the past, and how those may inform our current investigation into the determination of discharge at our site. There are many practical considerations critical to the establishment of a reliable index-velocity relation such as site location, the acoustic environment of a site, averaging intervals, model validation, etc. The literature also contains information on stage-discharge relationships, another discharge model that will be explored in this work. Considerations like site location, steady/unsteady flow, hysteresis, floodplain analysis, backwater effects, and model types are all crucial to the establishment of a reliable and accurate discharge model.

The literature, especially from the US Geological Survey (USGS) database, contains valuable guidelines and recommendations for the development for both methods. Since the final discharge equation will serve as the foundation for future water quality and land use studies, following the recommendations of the literature is crucial for the production of reliable and accurate discharge values.

The secondary purpose of this review is to examine where there may exist gaps in the literature. If a novel approach discovered here can be both identified and proven to be the most reliable and accurate method, then that approach will be selected for the LEWAS Lab. However, if an established method proves to satisfy these requirements better than any other, then it will be chosen over any novel approach. Again, the purpose of this review is to identify the best candidate models that could provide us reliable data, regardless of whether that method is novel, or well established.
2.2 ADV AND ADCP TECHNOLOGY: PRINCIPLES OF OPERATION

The term ADV (Acoustic Doppler Velocimeters) applies to all instruments that utilize acoustic signals to characterize the velocity of fluid flow. An instrument may take a point velocity measurement (much like a Marsh McBirney), or it can measure the vertical or horizontal velocity profile of a stream, as is the case with ADCP (Acoustic Doppler Current Profilers). Two common methods for the determination of velocity by acoustic means are time-of-travel and the Doppler shift method. The former method uses transmitted signals in diagonal paths relative to the flow, and by measuring the time between transmission and receipt of the signal, known as the travel time, the velocity of the fluid can be calculated (Laenen and Smith 1982). The Doppler shift method measures the frequency shift from the transmitted to received signal. When reflecting an acoustic signal, moving particles will induce a frequency shift on that signal. By measuring the frequency shift from the particle reflection, the velocity of those particles can be known. The fluid velocity is then inferred to be equal to the velocity of the particles (Sontek 2003).

The following equation illustrates the relationship between velocity of the fluid and the frequency shift.

\[ F_d = -2F_0 \frac{V}{C} \]  

(2.1)

Where:

- \( F_d \) is the shift in frequency between transmitted and received signal (s\(^{-1}\))
- \( F_0 \) is the frequency of transmitted sound (s\(^{-1}\))
- \( V \) is the velocity of the moving fluid (m/s)
- \( C \) is the speed of sound (m/s)

2.2.1 THE SONTEK ARGONAUT SW

The Sontek Argonaut SW is a sensitive instrument able to detect changes in stream velocity to a very precise level (0.001 m/s) (Sontek 2003). It is positioned on the bottom of the stream channel with its acoustic transducers pointing upwards towards the water surface. It operates by
transmitting acoustic signals and then receiving the echoes of those signals from moving particles in the water column (Figure 2.1). By utilizing Doppler effect principles, the velocity of those moving particles is calculated and equated to the velocity of water moving past the instrument.

![Figure 2.1 Sontek Argonaut ADCP. From: Sontek 200. Used under fair use, 2013](Figure 2.1 Sontek Argonaut ADCP. From: Sontek 200. Used under fair use, 2013)

The Argonaut has the capability to measure the vertical velocity profile of the stream, dividing the water column into various cells (Figure 2.1). Measuring the velocity within each cell, the instrument can return either the value of each cells in the array, or the average value of all cells, the vertically averaged velocity. This latter quantity is referred to in this thesis as the index velocity ($V_i$). When the Argonaut is set to output the average vertical velocity, or index velocity ($V_i$), the cell size is automatically adjusted to accommodate 10 cells within the water column. $V_i$ is the average of these ten cells below the water surface.

There is a minimum distance, called the blanking distance (0 to 0.07m from the Argonaut), where the Argonaut cannot measure the velocity of the water column (Figure 2.1). This is a result of the transducers not being able to receive signals so close to where that signal is being
transmitted (Sontek 2003). Velocity data contained within that blanking distance is not included into that averaging integration. During low stages, this may introduce a bias into the data.

Stage (H) is also measured by the Argonaut though acoustic signals. The center beam in Figure 2.1 produces an acoustic signal that reflects from the water surface. This reflection is clearly visible to the instrument and through a speed of sound calculation, the height, or stage (H) of the water surface is known.

2.3 ADV AND ADCP TECHNOLOGY: MODERN APPLICATIONS

ADV instruments can measure point velocities, as is the case with the Sontek FlowTracker ADV, or they can measure the velocity distribution profile of the stream, as is the case with the Sontek Argonaut SW ADCP. In stream flow applications, point velocity ADVs like the Sontek FlowTracker operate much like a Marsh McBirney, taking point velocity measurements across a stream cross section in order to calculate stream discharge.

ADCP instruments can measure discharge directly, as in ADCP boat measurements (González-Castro 2007; Simpson 2001), or indirectly, as in the index velocity method (IVM) (Sloat and Hull 2004). ADCP boat measurements look downwards towards the channel bed and collect bathymetric and velocity profile data. The instrument is moved across the stream in order to obtain a 2-D velocity profile image for the entire cross-section. These data can then be analyzed to determine, among other things, stream discharge (Simpson 2001). This method is widely used in tidal streams rivers where unsteady flow conditions exist (Ruhl et al. 2005; Sloat and Gain 1995). On the other hand, the index velocity method utilizes stream velocity as an independent parameter to predict stream discharge through regression equations, and does not measure discharge directly (Huang 2004; Le Coz 2008; Levesque and Oberg 2012). Unlike ADCP boat technology that moves across the stream channel, this method utilizes stationary side-looking or upward looking ADCP to measure the horizontal or vertical velocity profile at only one location of the cross section (Huhata and Ward 2003; Le Coz 2008). The index velocity method (IVM), which will be discussed more in detail later in this chapter, uses velocity profile data and relates it to the cross-sectional averaged velocity ($V_m$) in a simple or multi-regression linear model. Discharge ($Q$) can then be determined by multiplying $V_m$ times the cross sectional area (A) of the stream.
2.4 STAGE-DISCHARGE RELATIONSHIPS

Stage-discharge relationships have been widely and successfully utilized to measure discharge in rivers and streams (Morlock et al. 2002). However, in some instances stage-discharge curves may show two different discharges for the same stage. This ambiguity is known as hysteresis and results from unsteady flow at the measurement cross section. Streams in small urban watersheds, such as the Webb Branch Watershed, can experience unsteady, rapidly changing flow resulting in hysteresis. Hysteresis can be corrected using techniques such as the Jones Formula. However, the Jones formula doesn’t function well in areas of where backwater conditions are present (Hidayat 2011).

Because it is situated in an urban area, LEWAS site is subject to rapidly changing, unsteady flow. Moreover, it is positioned roughly 50 m upstream of the Old Ice Pond (Figure 1.7), where backwater conditions maybe prevalent particularly during very intense rains. Researchers have argued that complex flow situations such as those containing back water and hysteresis effects are unsuitable for stage-discharge relationships and that the use of velocimeters are best for these complex flow conditions (Morlock et al. 2002). Several researchers have successfully utilized upward looking or bottom mounted velocity meters to characterize discharge in rapidly changing flow situations (Huang 2004; Nihei and Sakai 2006).

Despite these drawbacks, stage-discharge relationships have been used for over a hundred years to measure stream discharge (Schmidt 2002). Successfully implemented by the USGS, stage has been a reliable predictor of discharge in many situations across the US. Although stage is not the only parameter which affects discharge, there are many cases where the stage-discharge equation is uniquely valued. In these situations, it is often convenient to model the relationship using a Chezy or Manning equation or a weir equation. The former case ensures a uniquely valued relationship by assuming steady, uniform flow. The latter case assumes critical flow, a condition where stage and discharge are always uniquely valued. Either way the two approaches result in a very similar equation form.

The following equation is the most basic form:

\[ Q = cH^b \]  \hspace{1cm} (2.2)
Where:

- $H$ is the stage (m)
- $Q$ is the stream discharge ($m^3/s$)
- $c$ and $b$ are coefficients

If Eqn. 2.2 is taken as the logarithm, the following linear equation results.

$$\log Q = \log c + b \log H \quad (2.3)$$

### 2.4.1 SITE LOCATION, SECTION CONTROL AND THE MODIFIED STAGE-DISCHARGE EQUATION

There are several practical considerations when establishing a proper stage-discharge relation. First, a proper site must be selected, and it should be a uniform prismatic channel. A prismatic channel indicates that the slope of the channel bed does not change, and the sides remain parallel for as long as possible (Rantz and others 1982). Downstream of a gaging site, a section control is normally constructed so that the stage-discharge relationship does not change, that is it becomes permanent, and is also uniquely valued. A section control can be a rock ledge, a weir or any apparatus that “stabilizes and regulates the flow past the gaging station” as Herschy (1993a) puts it. The flow at the gaging station is literally controlled by the downstream apparatus. Therefore the stream discharge at the gaging station can be more directly related to the height above the station control than to the gage height at the gaging station itself. In these cases, which are very common, a correction factor ($\alpha$), called the height of zero flow must be applied to the gage height or stage ($H$) in Eqns. 2.2 and 2.3 in order to arrive at the height above the section control (Braca 2008; Herschy 1993a; Herschy 1995; Kennedy 1984; Rantz and others 1982). This correction factor ($\alpha$), is the elevation of gaging site datum subtracted from the elevation of the section control datum. Very commonly, the section control datum is higher than the gaging site datum, producing a negative $\alpha$ value. The situation can also be reversed, where the gaging site datum is higher than the section control datum (Herschy 1995).
This leads to the modified versions of Eqns. 2.2 and 2.3, Eqns. 2.4 and 2.5.

\[ Q = c(H + \alpha)^b \]  
(2.4)

\[ \log Q = \log c + b \log(H+\alpha) \]  
(2.5)

Where:

- \( H \) is the stage (m)
- \( Q \) is the stream discharge (m\(^3\)/s)
- \( \alpha \) is the datum offset.
- \( c \) and \( b \) are coefficients.

In some cases, where no artificial or natural section controls are present or discernible, the existence of an unknown control exerting its influence on the gaging station site can be evidenced in a log-log plot of discharge (\( Q \)) vs. stage (\( H \)) (Figure 2.2). The sign of \( \alpha \) can even be deduced by the direction of concavity in the curve. A concave down shape indicates that the section control datum is higher than the gaging site datum while concave down shape suggests the opposite (Herschy 1995). However, the magnitude of \( \alpha \) must also be determined. One common way to find \( \alpha \) is by trial and error. One simply adjusts the \( \alpha \) value until the log-log plot of \( Q \) vs. \( H \) is a straight line. This can be performed tediously by hand, or a more modern approach may be used such as an optimization algorithm that maximizes the \( r^2 \) value of Eqn. 2.5 by adjusting the \( \alpha \) value.
2.4.2 UNSTEADY FLOW

Researchers and hydrologists, both in the USGS and elsewhere, became aware of certain conditions where stage-discharge relationships became unreliable and inaccurate. Variable backwater is one of these conditions and is present when stage gaging sites are just upstream of a lake, pond, or in stream structure such as a weir. Another condition which can adversely affect the stage-discharge relation is unsteady flow. Unsteady flow is present in urban environments where the flow can change rapidly. The following section will explain why stage-discharge relationships become inaccurate in the presence of these phenomena.

Stage is the simplest and cost effective way to make discharge measurements. However there exist conditions, such as unsteady flow and variable backwater, where stage and discharge are not single or uniquely valued (Petersen-ØVerleir 2006). These conditions, produced by very dynamic stream conditions (i.e. rapidly changing flow), can lead to biases in the stage-discharge curves (Di Baldassarre and Montanari 2009; Hidayat 2011). Unsteady flow conditions include the presence of acceleration and pressure gradients which make the relationship

Figure 2.2 Datum positions and curve concavity. From: Herschy (1995)

Figure 2.3 Physical processes and their effects on stage-discharge curve. (Stage vs. Discharge) From: Herschy (1995)
between stage and discharge multivalued, and much more complicated than uniform, steady flow and weir equations imply (Chaudhry 2007; Chow 1959; Di Baldassarre and Montanari 2009).

During storm events, a flood wave can propagate down a river or stream, producing unsteady flow (Chaudhry 2007). As a flood wave approaches a cross section, the velocity of that wave (the celerity) tends to increase the measured cross section velocity (Chaudhry 2007; Petersen-Øverleir 2006). However, as the wave passes, backwater effects work to decrease the stream velocity. This result is that the rising limb will have a greater discharge and cross-sectional averaged velocity at the same stage than the falling limb. This results in looping in the stage-discharge relationship, which is considered direct evidence of unsteady flow (Figure 2.3). Loopled discharge relationships are difficult to model, mainly because they are not single valued, but also because the extent of looping is variable, as it is a function of rainfall intensity among other things.

Many stage-discharge methods attempt to model unsteady flow, the most widely used models being the Jones formula, the slope method, and the storage method (Neely and Bingham 1986; Petersen-Øverleir 2006). But applying these methods can be data intensive. Along with stage, parameters like the rate of change of slope, rate of change of stage, and wave celerity must also be measured through complicated measurement techniques. This makes unsteady flow measurement impractical because of the amount of resources involved (Petersen-Øverleir 2006). Furthermore, the amount of assumptions one must ultimately make undermines confidence in these models.

2.4.3 VARIABLE BACKWATER

Another condition that can produce unreliable stage-discharge relationships is a phenomenon called variable backwater. This can also induce a hysteretic stage-discharge curve. Variable backwater is caused when variable downstream elements such as stream confluences, lakes, and tidal forces have an influence on the upstream gaging station. The downstream element will cause the slope of the water surface to be less than without backwater. And the amount the water slope changes is related to discharge (Petersen-Øverleir and Reitan 2009). Therefore discharge is related to both stage and the rate of change in water slope. For this reason, twin gaging systems have been utilized to measure discharge in backwater affected areas, where both the slope and
stage can be measured simultaneously (Braca 2008; Herschy 1993a; Kennedy 1984; Rantz and others 1982). Just as in modeling unsteady flow, the effect can be complicated and take up valuable resources.

2.5 DISCHARGE MODELS UTILIZING ACOUSTIC DOPPLER VELOCIMETERS

Acoustic Doppler Current Profiler technology delivers both stage and stream velocity profiles (Huhta and Ward 2003). Since there are several methods that utilize ADCP, it is important to discern which technique delivers the most accurate measurement of flow for our site. The first and simplest model to be considered is the Index Velocity Method (IVM). This model is a simple regression between mean stream velocity (determined using the velocity area method) and the vertically averaged velocity at center channel (i.e., index velocity using the Sontek upwards looking V-ADCP).

Hidayat (2011) assessed the current methods that employ H-ADCP technology. H-ADCP velocity measurements were correlated with discharge measurements taken by boat mounted ADCP profilers. Along with IVM, the Vertical Profile Method (VPM), and the semi-deterministic, semi-stochastic method (DSM) were evaluated by these means. The authors found that the DSM method was accurate to within 5% while the IVM was accurate to within 30%. The question exists whether VPM or DSM could deliver better results than the IVM for our particular site as it did for the authors of this article.

There are some important factors to consider, however, when applying the techniques found in Hidayat et al. 2011 to our site. The LEWAS group employs the upward looking (V-ADCP) Sontek Argonaut SW which measures the vertical velocity profile at center channel. Hidayat et al. 2011 used the H-ADCP, again a horizontal or sideward-looking device. Also, the LEWAS group measures discharge via the velocity area method using the Sontek Flow Tracker ADV (Acoustic Doppler Velocimeter) while their group employed a boat mounted ACDP profiler. The ADV can only measure velocity at single points while an ADCP boat mounted profiler measures the velocity distribution of the entire water column. While boat mounted ADCP systems are best for determining discharge, the small size of our stream prohibits their use. Given these differences in methodologies, as well as the complex theoretical calculations that would be
needed to adapt the sideward looking method to our upward looking model, it was decided that IVM was best because it has proven effectiveness when used with upward looking V-ADCP’s in rapidly changing and backwater flow conditions (Nihei and Sakai 2005 and Huang 2004). Also, since the IVM is essentially a simple or multi-regression model, the error associated with the calculated discharge can be more easily found than with other more complicated theoretical models.

2.5.1 THE INDEX VELOCITY METHOD

The index velocity method has been employed successfully as a reliable discharge model under unsteady flow and variable backwater conditions (Levesque and Oberg 2012; Ruhl et al. 2005; Smith et al. 1971). Nihei and Sakai (2006) investigated the efficacy of applying a downward mounted ADCP on the small, urban Oohori river in Japan affected by variable backwater (Figure 2.4). Their aim was to illuminate the differences in the velocity profile between the rising and falling limbs. They found that the velocity profile distribution did indeed change from the rising and falling limbs and furthermore, they found looping in the plot of stage (H) vs. discharge (Q) as well as depth averaged velocity (V) vs. discharge (Q). Figure 2.5 clearly shows hysteretic looping in both the H-Q and V-Q curves.

A question one might ask is why would using index velocity in place of stage be more effective during unsteady flow/variable backwater conditions as many have reported? The answer is because depth averaged velocity (V) (i.e. index velocity, \( V_i \)) is uniquely related not to the stream discharge (Q), but to the mean-channel velocity (\( V_m \)), even during unsteady flow and backwater conditions. Consider again what occurs during unsteady flow. The velocity on the rising limb (let’s say at an arbitrary stage, h) is greater than the velocity of the falling limb at that same

**Figure 2.4** Field Site and Location of measuring stations. *From: Nihei and Sakai (2006). Public domain, 2013*
stage, h. Therefore, the relation between velocity and stage will change

as the flood wave passes. Figure 2.6 shows this phenomenon in-situ using stream velocity data collected on the Oohori River (Nihei and Sakai 2006). Notice that the vertical velocity distributions are very different between the rising and falling limb. It is precisely these differences in velocity distributions that create the looping effect seen on the plot of depth averaged velocity ($V$) vs. stage ($H$) in Figure 2.5. In contrast, a plot of mean-channel velocity ($V_m$) vs. index velocity ($V_i$) would not produce this same looping if taken at the same site and duration of time. In fact $V_m$ will always be uniquely related to $V_i$, whether it is at the rising or falling limb because $V_m$ will change in a very similar way to $V_i$ as the flood wave passes. $V_m$ and $V_i$ are not separate parameters, but are in fact different samples or perspectives of the same parameter. And this is why their relationship remains unique throughout unsteady flow and backwater conditions.

**Figure 2.5** Stage–Discharge (H-Q) and Velocity–Discharge (V-Q) Rating Curves (Stns. 1 and 2). From: Nihei and Sakai (2006)

Public domain, 2013.
The simple regression form of the index velocity model is the following (Levesque and Oberg 2012):

\[ V_m = zV_i + m \]  \hspace{1cm} (2.6)

Where:

- \( V_m \) is the cross-sectional averaged velocity (m/s)
- \( V_i \) is the vertically or horizontal averaged velocity profile (m/s)
- \( z \) and \( m \) are regression coefficients

Stream discharge (Q) can then be determined by multiplying the cross sectional area (A) of the stream by the mean-channel velocity (\( V_m \)) that was determined as a function of index velocity.

\[ Q = V_m A \]  \hspace{1cm} (2.7)

Where:

- Q is the stream discharge (m\(^3\)/s)
- \( V_m \) is the cross-sectional averaged velocity (m/s)
- A is the cross sectional area (m\(^2\))
The index velocity method therefore uses \( V_i \) measurements, not stage, as the independent predictor of \( V_m \). And because the index velocity method will always produce a unique relationship between \( V_i \) and \( V_m \), Hidayat (2011) employed an H-ADCP and successfully characterized the flow regime in a backwater water affected site using the index velocity method (IVM). Levesque and Oberg (2012) also agree that IVM is useful in unsteady flow and variable backwater conditions, and state that velocity measurements are becoming increasingly widespread at USGS gaging stations.

2.5.2 INDEX VELOCITY MODELS

The simple regression form of the index velocity model was introduced in Eqn. 2.6. It utilizes only one parameter, index velocity (\( V_i \)) to predict mean-channel velocity. However, Levesque and Oberg (2012) have found that multiple regression relationship utilizing stage (\( H \)), index velocity (\( V_i \)) and \( V_i \ast H \) as independent parameters explained data in many situations, including tidal situations. It takes the following form:

\[
V_m = eV_i + fH + gV_i \ast H + i
\]  

(2.8)

Where:

- \( V_m \) is the cross-sectional averaged velocity (m/s)
- \( V_i \) is the vertically or horizontal averaged velocity profile (m/s)
- \( H \) is the stage (m)
- \( e, f, g, \) and \( i \) are regression coefficients.

It is important to keep in mind that both the single and multi-regression models are empirical, and not based on hydraulic theory. While hydraulic models that utilize real time stream velocity measurements do exist (Hidayat 2011), the index velocity method is widely adapted for its convenience and simplicity. Therefore, a method that could both increase the accuracy of the index velocity method and retain its simplicity and convenience may find useful applications. The following section describes an outline to modify the index velocity method so that the parameters \( H \) and \( V_m \) are related in such a manner that is more commonly found in nature.
Modifying the Index Velocity Method

While the multi-regression equation has been used successfully in many circumstances, it does not reflect a theoretical or hydraulic relationship between \( V_m \) and \( H \). According to the Chezy or Manning equations, \( V_m \) and \( H \) are not related in a non-linear manner. Therefore, another form of the multi-regression model not found in the literature may more accurately reflect their relationship. By adding an exponent (b) to the H term, Eqn 2.8 will then contain a non-linear relationship between \( H \) and \( V_m \), as found in the Chezy and Manning equations. Furthermore, if there is evidence in the data to suggest that \( V_m \) is more closely related to \((H-\alpha)^b\), then the term could be modified further to \((H-\alpha)^b\). The following equation is the proposed model that incorporates these modifications, and a comparison with other models, including Eqn 2.9, would determine if a more accurate and reliable index velocity model could result from this process.

As a counterpoint to the above argument, the Manning and Chezy equations assume steady flow, which is known not to exist at the site. Because of this reason, it is not known whether or not this modification will produce a better multi-regression model. This question will be explored further in the Data Analysis and Findings chapter.

\[
V_m = eV_i + f(H-\alpha)^b + gV_i*(H-\alpha)^b + i \quad (2.9)
\]

Where:

- \( V_m \) is the cross-sectional averaged velocity (m/s)
- \( V_i \) is the vertically or horizontal averaged velocity profile (m/s)
- \( H \) is the stage (m)

where \( b, e, f, g, \) and \( i \) are regression coefficients.
3. Proposed Study

3.1 Purpose

The purpose of this study is to develop a reliable and accurate method for the determination of stream discharge at the LEWAS outdoor site. The primary application of the discharge model will be to create a reliable and accurate hydrograph for LEWAS.

3.2 Research Objectives

1. Analyze the data collected in this study and recommend the most time-efficient method to collect hydraulic data at the LEWAS Site.

2. Determine whether hysteresis or backwater affects the stage-discharge rating curve.

3. Determine which model discussed in the literature review is best in terms of accuracy, simplicity, noise level and applicability for educational purposes. This will become the discharge model LEWAS will use to construct hydrographs of future runoff events.

4. Construct pilot hydrographs by applying the selected model to data collected during three runoff events in July 2012. Calculate the runoff-rainfall ratio (ROR) for each rain event.

5. Validate the selected model and determine if the data it produces is reasonable by comparing the results to other models and other gauging sites.

3.3 Definition of Variables

\[ H \text{ = Stage (m)} \]
\[ A \text{ = Cross sectional area (m}^2\text{)} \]
\[ V_m \text{ = mean-channel velocity (m/s)} \]
\[ V_i \text{ = Index Velocity, or vertically averaged stream velocity at center channel (m/s)} \]
\[ Q \text{ = Stream Discharge measured at LEWAS Site (m}^3\text{/s)} \]
4. METHODOLOGY

4.1 OVERVIEW

At the LEWAS Site in the Spring of 2012, stream cross-section points and hydrologic data were collected in order to construct a model for the determination of stream discharge in real-time. The stream cross-section was measured using a laser level and leveling rod. Index velocity was measured using the Sontek Argonaut-SW ADCP instrument while point velocity measurements were taken using the Sontek Flowtracker ADV (Figure 4.1).

Once the final location of the Sontek Argonaut SW ADCP was selected in the stream channel (see Selecting the location of the Sontek Argonaut-SW at the LEWAS Site), stream transect survey points were collected in order to obtain discharge stream geometry information (see Constructing the Stream Cross-section).

After the stream cross-section was constructed, two main categories of hydrologic data were collected simultaneously during rain events by two separate technicians: the ADV and ADCP Technicians. First, measurements of discharge (Q) and mean-channel velocity (\(V_m\)) were the result of applying the velocity area method (Herschy 1993b; Herschy 1995) to various point velocity measurements made by the ADV Technician using the Sontek FlowTracker ADV. Details of this procedure are clearly explained in Appendix A: The Velocity Area Method. Secondly, the real-time ADCP parameters, stage (H) and index velocity (\(V_i\)), were collected by the ADCP Technician using the Sontek Argonaut-SW ADCP (see Collecting the real-time ADCP parameters stage (H) and index velocity (\(V_i\))). Table 4.1 summarizes the types of data collected at the LEWAS Site.
Table 4.1. Summary of Data Types

<table>
<thead>
<tr>
<th>Data</th>
<th>Method</th>
<th>Instrument</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stream Cross-section</td>
<td>Engineering Survey</td>
<td>Field Rod and</td>
</tr>
<tr>
<td>(X,Y Points)</td>
<td>(Harrelson et al. 1994)</td>
<td>Laser Level</td>
</tr>
<tr>
<td>Stream Discharge (Q)</td>
<td>Velocity Area Method</td>
<td>Sontek Flow</td>
</tr>
<tr>
<td>X-Section Avg. Vel. (V_m)</td>
<td>(Herchey 1993b)</td>
<td>Tracker-ADV</td>
</tr>
<tr>
<td>Stage (H)</td>
<td>LEWAS Site Methodology</td>
<td>Sontek Argonaut</td>
</tr>
<tr>
<td>Index Velocity (Vi)</td>
<td></td>
<td>SW-ADCP</td>
</tr>
</tbody>
</table>

By calibrating the real time parameters stage (H) and index velocity (Vi) to either discharge (Q) or mean-channel velocity (V_m), the LEWAS Lab will possess the ability to produce stream discharge in real-time through its LabVIEW enabled network.

4.2 STUDY AREA

The LEWAS Lab is located on the outlet of the heavily urbanized Webb Branch subwatershed on the Virginia Tech Campus (Figure 4.2). Upstream of LEWAS, the Webb St. (TOB) discharge monitoring site is also shown in the figure. Urbanized streams such as the Webb Branch very often experience rapidly changing flow conditions. That is, stream discharge or flow changes rapidly with respect to time. Because of the location of LEWAS in an urbanized watershed, rapidly changing discharge is an issue that needs serious attention during discharge measurements. When making
measurements of discharge, care must be taken to ensure that it changes minimally during the course of its measurement.

There are other considerations that must be addressed at the LEWAS Site when making discharge measurements. Being a narrow channel at 1.5 m across (Figure 4.3), any discharge measurement using downward-looking, boat mounted ADCP technology is not possible. This type of technology is limited to wider stream channels or rivers because of its inability to measure velocity profiles close to the banks. Therefore, only wading rod mounted point velocity measurements (e.g., Sontek FlowTracker ADV) are possible when computing discharge that is contained within the stream channel.

4.2.1 SELECTING THE LOCATION OF THE ARGONAUT ADCP AT THE SITE

As with any environmental sampling method, the location and environment of the Sontek Argonaut SW ADCP within the stream is vital to the collection of any data as well as any subsequent data analysis. The instrument’s original location, named Location 1, was slated to be directly adjacent to the culvert underneath West Campus Drive (Figure 4.4). This location was not satisfactory for various reasons. First, the instrument requires a straight section of stream that is as long as possible. This is to ensure that flow is uniform and constant over the area occupied by the beams, as in Figure 4.5 (Sontek 2003). Instead of straight flow lines, the flow at Location 1 was circular and erratic, and the shape of the channel in this location was irregular. These factors would have made the
calibration of index velocity ($V_i$) to either mean-channel velocity ($V_m$) or discharge ($Q$) impossible because the velocity profile directly over the Argonaut (i.e., the velocity profile within the instrument’s sampling space) would change constantly at the same discharge. Also, its proximity to straight, concrete edges would reflect the sound waves that emanate from the Argonaut and decrease the signal to noise ratio (Sontek 2011). Location 2 (Figure 4.6), roughly 12m downstream of Location 1, is much straighter, more uniform, and its flow lines parallel at base flow. In relation to constant, straight, and uniform flow, Location 2 satisfied the requirements of the Argonaut much better than did Location 1.

However, there were other considerations such as minimum flow depth. The Argonaut requires a minimum flow depth of 0.3m (1ft) in order to perform its velocity profiling correctly. While Location 2 was a straight, constant flow region, it barely reached the required minimum depth and during very low flows, the Argonaut would not function properly. It was noticed that the channel in this location was lined with concrete. The decision was then made to remove this concrete from the bottom of the channel in order increase the depth of that section. When the conditions at Location 2 satisfied all the requirements for the Argonaut, the final decision was made to locate the Argonaut there. In the Spring of 2012, hired contractors permanently installed the Argonaut ADCP in the center of the channel, along with the Hydrolab MS-5 Sonde, at Location 2.

The culvert seen in Figure 4.4 is not the only input to the LEWAS ADCP location. In Figure 4.6, it can be seen that another pipe inputs into the location (the photo is taken from the top of culvert
This pipe is connected to the storm water system that runs along West Campus Drive. It is unknown whether the runoff that drains into this pipe is totally derived from the Webb Branch watershed area, or if it is exported from another drainage area. This brings up another important issue. Since most of the runoff entering the site is collected through storm sewer systems, it is possible that the sewer system is exporting runoff from Webb Branch to another system. Given that the drainage system is engineered, the watershed boundary created by topologic maps (i.e. LIDAR, Arc GIS) may not necessarily be the actual area that drains into the LEWAS Site.

Also seen in Figure 4.4 within the box culvert is a v-notch weir. This weir will be used to validate the discharges computed by LEWAS just downstream from it. This comparison should be performed cautiously however because the TOB Storm water pipe that empties into Webb St lies between the weir and the LEWAS discharge monitoring location. The LEWAS site should therefore show higher discharges during runoff events.

4.3 DATA COLLECTION

4.3.1 CONSTRUCTING THE STREAM CROSS-SECTION

Constructing the stream cross-section was performed according to Harrelson et al. (1994) in the Spring 2012 exactly where the measuring tape traverses the channel at Location 2 in Figure 4.3, and where the ADCP unit resides in Figure 4.6. To ensure the stream cross-section encompassed most runoff events, the span of the stream cross-section was done as widely as possible (14 m) which corresponded to a 1.46 m vertical change in elevation. The datum (Elevation = 0m) was set at the deepest point of the channel. Figure 4.7 shows the results of this cross-section. The axes are oriented to an observer facing downstream (Left Bank is x=0 m, Right Bank is x=14 m). The channel itself is very narrow, exactly 169 cm across. High-resolution survey measurements (every 5cm) were made inside the channel in order to provide high station density for discharge accuracy (Figure 4.7). Each survey measurement of either the channel or floodplain corresponds to the horizontal location of verticals. The survey is very important because it delineates the horizontal spacing of verticals. Low vertical resolution, or wide spacing, could result in inaccurate discharge measurements.
4.3.2 COLLECTING POINT VELOCITY MEASUREMENTS WITH THE FLOWTRACKER ADV

The first step in collecting point velocity measurements was to create the station pattern from the survey cross-section data. Point velocity stations were placed at the midpoint between each vertical. In the channel, 33 verticals were placed, creating the potential for 32 stations. However, since 32 stations took too much time to measure, 22 stations were used instead. To reduce the number of stations from 32 to 22, subsections of low velocity areas were consolidated. Appendix B: Station Patterns illustrates this process. Stations 23-27, and 28-32 were consolidated into stations 21 and 22 respectively while stations 1 and 2 were eliminated because at this location

Figure 4.7 Illustration of the stream cross-section at the final ADCP location.

Figure 4.8. Taking point velocity ADV measurements.
it was not possible to take velocity measurements due to bank interference with the ADV unit.

These 22 stations were marked by electrical tape on survey tape that was stretched across the stream channel, each end attached to a pin pushed into the ground. To measure point velocity, the technician would align the wading rod of the ADV with the black mark on the tape (Figure 4.8). A point velocity measurement was taken at each station and recorded in the FlowTracker’s memory (for operating procedures of the FlowTracker ADV, see Sontek (2009)).

In order to arrive at discharge, the velocity area method was applied to the point velocity measurements according to Herschy (1993b). Measurements were made at 60% of the depth.

4.3.3 THE SONTEK ARGONAUT SW: COLLECTING ADCP PARAMETERS

Collecting the ADCP parameters stage (H) and index velocity (Vi).

The real-time parameters stage (H) and index velocity (Vi) were collected using the Sontek Argonaut-SW and a Lenovo Windows 7 Laptop connected via a RS 232 cable and an RS232-serial port to USB adapter. Using the laptop as an interface with the Argonaut, the ADCP Technician (see Figure 4.9) would obtain stage (H) and index velocity (Vi) data through the software program View Argonaut underneath a rain umbrella that shielded the computer equipment from rain.

Deploying and Recording the Argonaut ADCP Data

To deploy the Argonaut SW ADCP, please refer to Sontek (2009). During deployment, stream cross-section data were not entered in the software since a MATLAB program was created to work with the LabVIEW environment that can calculate cross sectional area more precisely. Therefore the only output recorded in the field was the velocity in the x-direction (V1/X/E), which is referred to as index velocity (Vi) in this study. In order to arrive at the Real-Time module in View Argonaut, the operator first selected and ran the View Argonaut icon, and then when the menu appears, Real-Time Measurements was selected. This brought the operator to the
real-time module screen. Stage (m) and V1/X/E (cm/s) (i.e. index velocity (Vi)), values were recorded from this screen.

While the ADV Technician was taking a point velocity measurement at a particular station, H and Vi values from the ADCP were recorded next to that station (see Table 4.2). The purpose of collecting the Argonaut data in this manner was so that data were not collected in the shaded blue region found in Table 4.2. These stations are located very close to the Argonaut ADCP and its acoustic beams; the presence of the ADV operator affects the ADCP readings while he was in that location. Data outputted from the real-time module were ignored and not recorded while the ADV operator was in that region. The sampling interval of 30s for ADCP was selected because the ADV operator usually spent an average of 30s at each station.

In the field, the ADV operator took measurements across the channel and the Vi and H readings from the ADCP were entered next to the station number. However, data gaps appear in the data sheet (Table 4.2). This is because the ADV operator would spend the least time possible at each station and occasionally, an ADCP output would not be ready. The ADV operator, attempting to make discharge measurements as quickly as possible, would sometimes make a point velocity measurement in less than 30s. This would make the overall number of ADCP readings less than total number of point velocity measurements, or stations in the stream cross-section.

The data presented in Table 4.2 are an example of a typical sampling event taken on 5/15/2012. It contains all ADCP measurements made during each of the three discharge measurements, including average Vi and H values for each discharge measurement.

**Precision of data reported by ADV and ADCP Instruments.**

The Sontek Flowtracker ADV has a precision level for point velocity measurements of +/- 0.001 m/s and is accurate to within +/- 1% of the actual stream velocity (Sontek 2009). The Sontek Argonaut ADCP reports the level of precision for index velocity to be +/- 0.001 m/s and +/- 0.001 m for stage (Sontek 2009).
While the ADCP may actually detect differences in stage to the millimeter in laboratory settings, environmental conditions at the LEWAS Site may prevent such precise measurements (Hession 2012). It is more reasonable therefore, to change the stage precision level from 0.001m to 0.01m. Velocity precision levels (ADV and ADCP) will also be changed from 0.001m/s to 0.01 m/s. Turbulence, especially at very high velocities, will render velocity measurements to the third decimal place meaningless. All subsequent calculations (e.g. discharge) will reflect these precision values.

Table 4.2 ADCP data collected on 5/15/2012.
Bringing the ADV point velocity and ADCP stage data together to compute discharge

The mean stage for each discharge measurement from Table 4.2 was used together with point velocity measurements from the ADV to calculate discharge. Table 4.3 displays the results of these velocity area calculations for each of the three discharge measurements collected on 5/15/2012. Discharge (Q), mean-channel velocity (V_m), index velocity (V_i), and stage (H) are tabulated in Table 4.3 for the 5/15/2012 sampling event.

Table 4.3. Final summary for data collected on 5/15/2012

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Q (m^3/s)</th>
<th>V_m (m/s)</th>
<th>V_i (m/s)</th>
<th>A (m^2)</th>
<th>H (m)</th>
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<td>0.70</td>
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<td>0.71</td>
<td>0.59</td>
</tr>
<tr>
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<td>0.03</td>
<td>0.12</td>
<td>0.71</td>
<td>0.59</td>
</tr>
</tbody>
</table>

For eleven sampling dates, the data was collected and processed in the manner shown above. Table 4.4 displays the data collected over the duration of the collection study. Each sampling date contains multiple discharge measurements.

Table 4.4. Final summary for data collected over the duration of the collection study

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Q (m^3/s)</th>
<th>V_m (m/s)</th>
<th>V_i (m/s)</th>
<th>A (m^2)</th>
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Table 4.4 Summary of data collected over entire collection study. Data is listed in chronological order.

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<th>Sampling Date</th>
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<th>Q (m$^3$/s)</th>
<th>V$_m$ (m/s)</th>
<th>V$_i$ (m/s)</th>
<th>A (m$^2$)</th>
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</table>
5. **DATA ANALYSIS AND FINDINGS**

5.1 **OVERVIEW**

In this chapter, the research questions and objectives in the proposed study are addressed. The first question was addressed through a re-examination of the methodology. The methodology was investigated for time-effectiveness in order to minimize discharge sampling duration and the error associated with rapidly changing flow. The error associated with the averaging interval for each point velocity measurement was also examined.

The second objective was to determine which of the models discussed in the Proposed Study and Literature Review best fit the needs of the LEWAS Lab. Applying statistical criteria, each model was examined carefully for soundness and reliability. A final model was selected based on these statistical criteria and applicability towards education.

The next objective was to determine the runoff to rainfall ratio (ROR) of the Webb Branch subwatershed during three rain events on 7/19/2012 and 7/24/2012 using the recommended discharge model.

The final objective was to validate the selected model by comparison to other models developed in this chapter as well as to the Webb St. gauging station and the on-site v-notch weir.

5.2 **ADDRESSING SAMPLING ERROR AT THE LEWAS SITE**

Discharge sampling error ($\sigma_{se}$) at the LEWAS Site is derived from two sources. The first source of error is rapidly changing discharge ($\sigma_{cf}$): Discharge changes during a discharge measurement will induce an error into its value; the longer the measurement duration, the longer the discharge will change, and the larger $\sigma_{cf}$ will be. The second source of sampling error is the ADV averaging interval ($\sigma_{av}$). Each point velocity measurement requires an averaging interval, or sampling time, and a longer sampling time (or number of samples ($n$)) will tend to be more accurate than a shorter interval. These two sources of error are inversely related. That is, a longer averaging interval, or larger $n$, will result in a lower $\sigma_{av}$, but because of rapidly changing
flow, the longer averaging interval will also result in a larger $\sigma_{cf}$. This relationship was further explored in the following sections.

5.2.1 SAMPLING ERROR DUE TO RAPIDLY CHANGING FLOW

Rapidly changing flow can produce sampling error on discharge when taking point velocity measurements (Herschy 1993b). At the LEWAS Site the stage was observed to increase and decrease very rapidly during and after intense rain events. Because of this, a time-efficient method for collecting the necessary number of point velocity measurements is necessary. The uncertainty associated with the discharge changing with respect to time, $\frac{\partial Q}{\partial t}$, during the measurement could be mitigated if the discharge measurement duration were reduced.

The stage at the LEWAS Site has been observed to rise and fall very rapidly during rain events. On 6/1/2012, during a major rain event, the stage was recorded to fall from 0.962m to 0.888m during the duration of one discharge measurement, approximately 10 minutes. By inputting the initial and final stage values of every discharge measurement into the following equation (arrived at later in this chapter),

$$ Q = 5.459(H - 0.455)^{2.48} \quad (5.1) $$

the change in discharge during each discharge measurement was calculated. This made possible the quantification of both absolute ($\sigma_{cf}$) and relative error ($r\sigma_{cf}$) with respect to discharge. These error results are tabulated in Table 5.1. The error from rapidly changing discharge presents a real concern, especially at higher flows. Minimizing this error will be very valuable for future sampling events.
Table 5.1 Sampling error for discharge due to rapidly changing flow ($\sigma_{cf}$) and ADV averaging interval ($\sigma_{av}$).

<table>
<thead>
<tr>
<th>Sampling Date</th>
<th>Discharge Meas.</th>
<th>$V_m$ (m/s)</th>
<th>SEM</th>
<th>$Q$ (m$^3$/s)</th>
<th>$\sigma_{cf}$</th>
<th>$\sigma_{av}$</th>
<th>$\Gamma_{cf}$</th>
<th>$\Gamma_{av}$</th>
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<td>0.0044</td>
<td>0.02</td>
<td>0.000</td>
<td>0</td>
<td>0.006</td>
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<td>0.02</td>
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<td>0.02</td>
<td>0.002</td>
<td>13</td>
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<td>0.02</td>
<td>0.000</td>
<td>1</td>
<td>0.005</td>
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<tr>
<td></td>
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<td>0.05</td>
<td>0.0142</td>
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<td>0.000</td>
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<td>0.0042</td>
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<td>0.051</td>
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<td>27</td>
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<tr>
<td></td>
<td>2</td>
<td>0.31</td>
<td>0.0441</td>
<td>0.29</td>
<td>0.074</td>
<td>25</td>
<td>0.083</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.52</td>
<td>0.0542</td>
<td>0.56</td>
<td>0.085</td>
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<td>0.115</td>
<td>21</td>
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<td>6.1.12</td>
<td>1</td>
<td>0.62</td>
<td>0.0531</td>
<td>0.76</td>
<td>0.164</td>
<td>22</td>
<td>0.127</td>
<td>17</td>
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<tr>
<td></td>
<td>2</td>
<td>0.45</td>
<td>0.0490</td>
<td>0.50</td>
<td>0.142</td>
<td>29</td>
<td>0.105</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.33</td>
<td>0.0375</td>
<td>0.33</td>
<td>0.034</td>
<td>11</td>
<td>0.073</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.26</td>
<td>0.0330</td>
<td>0.24</td>
<td>0.033</td>
<td>14</td>
<td>0.061</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.23</td>
<td>0.0282</td>
<td>0.20</td>
<td>0.027</td>
<td>14</td>
<td>0.049</td>
<td>25</td>
</tr>
<tr>
<td>9.1.12</td>
<td>1</td>
<td>0.02</td>
<td>0.0032</td>
<td>0.01</td>
<td>0.000</td>
<td>2</td>
<td>0.004</td>
<td>39</td>
</tr>
</tbody>
</table>
The reduction of point velocity stations

It is possible to reduce the number of stations at the LEWAS site in order to minimize $\sigma_{ef}$. Herschy (1993b) suggests that 20 is the minimum number of stations, or point velocity measurements, used to determine discharge. However, Herschy has applied this method mostly to wider rivers and streams; the stream width at the LEWAS site is only 1.5m across. It is possible then that fewer stations may produce a discharge of comparable accuracy.

To address this issue, a discharge comparison was made using fewer stations to examine whether discharge measurement duration may be reduced by this manner. Currently, 22 stations are used to collect point velocities for the determination of discharge at the LEWAS Site. Using the exiting data set, it is possible to determine the discharge as if fewer point velocity measurements were taken.

Subsections were consolidated from 22 to 16, 9, and 6 stations in the same manner as they were reduced from 32 to 22 stations (see Appendix B: Stations Patterns). The five discharge measurements taken on 6.1.2012 were treated in this manner. Table 5.2 shows the effects on discharge values as the number of station was reduced.

Table 5.2 The number of stations and its effect on final discharge value.

<table>
<thead>
<tr>
<th>Number of Stations</th>
<th>Discharge (Q)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>22</td>
<td>0.76</td>
<td>0.50</td>
<td>0.33</td>
<td>0.24</td>
</tr>
<tr>
<td>16</td>
<td>0.76</td>
<td>0.48</td>
<td>0.33</td>
<td>0.24</td>
</tr>
<tr>
<td>9</td>
<td>0.76</td>
<td>0.48</td>
<td>0.32</td>
<td>0.25</td>
</tr>
<tr>
<td>6</td>
<td>0.72</td>
<td>0.51</td>
<td>0.30</td>
<td>0.28</td>
</tr>
</tbody>
</table>

The data clearly shows that the reduction to nine stations does not significantly affect the discharge calculation. However, the use of six stations departs significantly from the original values calculated from 22 stations.
5.2.2 SAMPLING ERROR DUE TO ADV AVERAGING INTERVAL

The ADV averaging interval is the number of seconds the FlowTracker ADV spends collecting each point velocity measurement. The FlowTracker is capable of taking 10 point velocity measurements per second. When the averaging interval is set to 10s, the FlowTracker will take 100 measurements over that period, and then return the average of those measurements. If the averaging interval is set to 30s, it will return the average of 300 velocity samples. The averaging interval on the FlowTracker ADV can be set from 10s up to 1000s in one-second increments (Sontek 2009).

Before discharge measurements were taken in the spring, it was decided to set the ADV to the minimum averaging interval of 10s in order to minimize the discharge measurement duration. From the perspective of minimizing \( \sigma_{cf} \), this makes good sense, but the ADV manufacturers recommend using a longer averaging time of 30s to 40s, in order to minimize \( \sigma_{av} \) (Sontek 2009). Using a longer averaging interval increases the likelihood of the sample mean representing the true mean of the velocity. That is, the velocity the ADV reports is more likely to be closer to true mean velocity if it has more time to sample the natural velocity distribution. This poses the following questions that must be addressed: Is the sample mean produced by the 30s method closer to the true mean than the 10s method? Is using the 30s averaging interval method a more accurate method than the 10s method? In other words is \( \sigma_{av} \) the same for both methods? In order for the 30s method to be more accurate, its mean and variance must be demonstrably different than the 10s method. If no difference can be detected between them, then the 30s method cannot be deemed more accurate, and their \( \sigma_{av} \) values could also not be deemed different.

On July 15, 2012 an experiment was conducted to answer these questions. Eleven of the twenty four stations were sampled 15 times (n=15) for point velocity using the 30s and 10s methods. That is, each of the eleven stations was sampled 15 times for a duration of 30s, and then another 15 times for a duration of 10s. The eleven stations were chosen to represent the horizontal velocity distribution of the stream. Sampling all 24 stations was not possible due to time limitations. The means and variances for each method (30s and 10s averaging times) at each station were treated in JMP statistical software. The five tests used to test for variance homogeneity include the O’Brien, and for mean equivalence, the Welch Test was used because it
does not assume equal variances. Table 5.3 tabulates the p-values from these statistical analyses ($\alpha = 0.05$). As a reminder to the reader, any p-value below the alpha value of 0.05 is considered to detect a difference either in variance or the mean. Table 5.4 summarizes the conclusions drawn from the statistical test results in Table 5.3.

Table 5.3. Comparison of two sampling methods: p values for mean and variance tests comparing 10s and 30s averaging intervals for point velocity measurements. Red indicates $p<0.05$, or a significant result.

<table>
<thead>
<tr>
<th>Station</th>
<th>Location (cm)</th>
<th>V (m/s) (10s Avg Time)</th>
<th>V (m/s) (30s Avg Time)</th>
<th>Test for Mean</th>
<th>Tests for Variance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mean</td>
<td>std dev</td>
<td>mean</td>
<td>std dev</td>
<td>Welch's</td>
</tr>
<tr>
<td>2</td>
<td>38.5</td>
<td>0.00955</td>
<td>0.00571</td>
<td>0.01631</td>
<td>0.00179</td>
</tr>
<tr>
<td>4</td>
<td>48.5</td>
<td>0.03651</td>
<td>0.01039</td>
<td>0.03424</td>
<td>0.00961</td>
</tr>
<tr>
<td>8</td>
<td>68.5</td>
<td>0.04693</td>
<td>0.01160</td>
<td>0.04767</td>
<td>0.00646</td>
</tr>
<tr>
<td>10</td>
<td>78.5</td>
<td>0.05700</td>
<td>0.00381</td>
<td>0.05633</td>
<td>0.00676</td>
</tr>
<tr>
<td>12</td>
<td>88.5</td>
<td>0.04654</td>
<td>0.01170</td>
<td>0.04701</td>
<td>0.00589</td>
</tr>
<tr>
<td>14</td>
<td>98.5</td>
<td>0.02529</td>
<td>0.00867</td>
<td>0.02747</td>
<td>0.00661</td>
</tr>
<tr>
<td>16</td>
<td>108.5</td>
<td>-0.00599</td>
<td>0.00533</td>
<td>0.00800</td>
<td>0.00611</td>
</tr>
<tr>
<td>18</td>
<td>118.5</td>
<td>-0.00599</td>
<td>0.00533</td>
<td>-0.00730</td>
<td>0.00481</td>
</tr>
<tr>
<td>20</td>
<td>128.5</td>
<td>-0.01051</td>
<td>0.00412</td>
<td>-0.00813</td>
<td>0.00468</td>
</tr>
<tr>
<td>22</td>
<td>143.5</td>
<td>-0.03386</td>
<td>0.00392</td>
<td>-0.01512</td>
<td>0.00563</td>
</tr>
<tr>
<td>24</td>
<td>168.5</td>
<td>-0.00009</td>
<td>0.00041</td>
<td>0.00003</td>
<td>0.00007</td>
</tr>
</tbody>
</table>

Table 5.4. Results summary for method comparison test.

<table>
<thead>
<tr>
<th>Station</th>
<th>Location (cm)</th>
<th>Equivalence Test Results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mean</td>
<td>std dev</td>
</tr>
<tr>
<td>2</td>
<td>diff detected</td>
<td>no diff detected</td>
</tr>
<tr>
<td>4</td>
<td>no diff detected</td>
<td>no diff detected</td>
</tr>
<tr>
<td>8</td>
<td>no diff detected</td>
<td>diff detected</td>
</tr>
<tr>
<td>10</td>
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<td>no diff detected</td>
</tr>
<tr>
<td>12</td>
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<td>diff detected</td>
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<tr>
<td>14</td>
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<td>no diff detected</td>
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<tr>
<td>16</td>
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<tr>
<td>20</td>
<td>no diff detected</td>
<td>no diff detected</td>
</tr>
<tr>
<td>22</td>
<td>no diff detected</td>
<td>no diff detected</td>
</tr>
<tr>
<td>24</td>
<td>no diff detected</td>
<td>diff detected</td>
</tr>
</tbody>
</table>

Table 5.4 clearly shows that there are very few instances where either the mean or the variances are different. In fact, only two of eleven stations show a difference in the mean of the two methods. This leads to the conclusion that at baseflow, both methods produce the same mean. In terms of variance, only three of the eleven stations show a difference in the standard deviation. At baseflow, the natural variation (or true variation) of velocity is not large enough for a
difference to be detected in the two methods. Therefore at baseflow, $\sigma_{st}$ is the same for both the 10s and 30s averaging intervals.

At higher discharges and velocities, however, velocity variation or turbulence may increase and the accuracy of the two methods may in fact differ. In fact, there is evidence in the data that supports this theory. Along with the mean of point velocity data, the ADV returns the standard error of the mean (SEM) of those measurements. For instance, for ten one-second measurements, the ADV will return the standard error of the mean of those ten measurements.

$$SEM = \frac{s}{\sqrt{n}}$$ (5.2)

where: SEM is the standard error of the mean

$s$ is the standard deviation of the mean

$n$ is the number of measurements

Standard error of the mean (SEM) values for all point velocity measurements were averaged for each discharge measurement using the 10s averaging interval. SEM values were used to calculate the 95% confidence interval expressed as absolute $Q$ and also percent of the $Q$. There are displayed as absolute error ($\sigma_{av}$) and relative error ($r\sigma_{av}$) in Table 5.1. In Figure 5.1(a), it can be seen that SEM increases as mean-channel velocity ($V_m$) increases. This indicates that velocity variation did indeed increase as discharge increases, due to increased turbulence. Not surprisingly, this also resulted in an increase in the absolute error in $Q$ ($\sigma_{av}$) as both $V_m$ and $Q$ increased (Table 5.1). However, this error did not increase proportionally as a fraction of $Q$. In fact, the relative error from averaging interval ($r\sigma_{av}$) actually decreased as $Q$ and $V_m$ increased (Figure 5.1b). On the other hand, $r\sigma_{cf}$ shows a slight increase as $V_m$ and $Q$ increase (Figure 5.1c).
Figure 5.1 (a) Standard error of the mean of point velocity measurements (SEM) vs. mean-channel velocity ($V_{m}$). (b) Relative error due to averaging interval ($r\sigma_{av}$) vs. discharge (Q). (c) Relative error due to rapidly changing flow ($r\sigma_{cf}$) vs. discharge (Q).

Figure 5.1 indicates that $r\sigma_{av}$ decreases while the $r\sigma_{cf}$ increases at higher Q values. As a percent of Q, the error from rapidly changing discharge is more significant at higher discharges than that from averaging interval. In other words rapidly changing flow has more of an effect on the overall sampling error at high discharges. Therefore, minimizing the sampling duration time at higher flows would mitigate the largest source of error, $\sigma_{cf}$ and hence minimize the overall sampling error, $\sigma_{se}$. A 10s averaging interval is therefore best when collecting all point velocity measurements at the LEWAS Site. This applies to both base flow and high discharge events.

Whether the 30s and 10s methods differ at higher flows still remains unknown; the data do not support any conclusion regarding this question. The 30s method may in fact be more accurate but the $r\sigma_{av}$ is insignificant compared to $r\sigma_{cf}$ at higher flows. Therefore regardless of whether the 30s method is more accurate or not, measurement duration is the controlling factor in the overall sampling error ($\sigma_{se}$).
5.2.3 FUTURE DISCHARGE SAMPLING RECOMMENDATIONS

In the future, it is recommended that 9 point velocity measurements, or stations be used to determine discharge. The resulting reduction in discharge measurement time will significantly decrease the discharge sampling error, especially at high flows.

The averaging interval of point velocity measurements will remain at 10s. The data indicate that the relative error due to rapidly changing flow ($r_{σ_{cf}}$) is more significant at higher flows than the relative error due to averaging interval ($r_{σ_{st}}$). Measurement duration therefore is the dominant factor controlling sampling error ($σ_{se}$) and must be minimized under all flow conditions.

While the previous section focused on future sampling efforts, the following sections focus on the hydrologic data gathered during 2012 rain events and baseflow conditions. A final discharge model was selected and then applied to an educational module for the CEE 4304 classroom.
5.3 SELECTING THE RIGHT MODEL FOR THE LEWAS LAB

In the Spring and Fall of 2012, 37 baseflow and rain event discharge measurements, along with other hydrologic parameters, were collected over ten rain events in order to obtain a representative sample of the range of flows the LEWAS Site experiences. Table 4.4 displays the results of this sampling effort.

5.3.1 STAGE-DISCHARGE RELATIONSHIPS

In the following section, stage-discharge model concepts from the Literature Review and Proposed Study were applied to the data set collected in Spring 2012. These equations involve the determination of discharge (Q) as a function of stage (H) (i.e. Q = f(H)). From Table 4.4, Figures 5.2a and 5.2b were constructed. [See Appendix C: Model Regression Parameters for detailed statistical outputs of all regression models used in this chapter]

Figure 5.2 (a) Discharge (Q) vs. stage (H) (b) Concave down property of log-log stage-discharge relation.

Figure 5.2a shows the data in an exponential form that could potentially be explained by Eqn. 2.2. However, when the logarithms of discharge and stage are plotted in Figure 5.2b, the resulting curve is non-linear and therefore Eqns. 2.2 and 2.3 cannot explain the data. If Eqn. 2.3 explained the relationship between the discharge and gage height perfectly, Figure 5.2b would result in a perfectly linear regression line. As Figure 5.2b indicates however, the curve has a concave down property and is visibly non-linear. This phenomenon, as was described in detail in the Literature Review, is due to the section control datum differing from the cross section datum. This difference in the section control and cross section datums is called the offset, \( \alpha \). Equations
2.4 and 2.5 incorporate the datum offset, and they are good candidates to fit the data, but first the datum offset, $\alpha$ must be found.

When the log-log scale stage-discharge curve is concave down, $\alpha$ is negative because the section control datum is higher than the cross section datum. However, when the section control datum is lower than that of the cross section datum, $\alpha$ becomes positive, and the curve becomes concave up. Judging from the concave down nature of Figure 5.2b, its $\alpha$ value, or offset, should be negative.

There are cases, such as in natural channels with no artificial controls, that the section control datum is not easily identifiable. In these cases, such as the LEWAS Site, the datum offset ($\alpha$) can be found through an optimization process. In this process, the maximum of the regression coefficient, $r^2$, of Eqn. 2.5 is found by varying the datum offset, $\alpha$. Excel Solver, with its optimization algorithms, was used to determine the alpha value corresponding to the maximum $r^2$ value.

Through this process, the maximum value of $r^2$ ($r^2 = 0.963$ on log-log scale) was found when $\alpha$ was -0.455 m. To illustrate this optimization process, Figure 5.3 plots various negative values of $\alpha$ against their corresponding $r^2$ values, resulting in a maxima at the $\alpha$ value of -0.455 m.

![Figure 5.3](image)

**Figure 5.3.** $r^2$ values for various $\alpha$ values used in Equation 2.5.
Using -0.455m as the alpha value in Eqn. 2.5, log Q vs. log (H - 0.455) was plotted in Figure 5.4b. First of all, notice that the concavity found in Figure 5.2b is absent and also that the r² value for Figure 5.4b is 0.9637 (0.9827 on the original scale - Fig 5.4a). The relationship in Figure 5.4b is clearly more linear than in Figure 5.2b; applying this modification has significantly increased the linearity of the model. Equation 2.4 can now be expressed with known values of the regression coefficients and alpha.

\[ Q = 5.459(H - 0.455)^{2.487} \]  

(5.1)

The residuals plot for both figures show a few outliers but no discernible pattern that would suggest the influence of other significant independent parameters. The residual analysis and r² correlation values indicate that Eqn. 5.1 accounts for the variation in the data in a sound, reliable and complete way.
Hysteresis and Variable Backwater

When examining Figures 5.2 (a,b) and 5.4 (a,b), no discernible looping is visible, indicating that unsteady flow does not significantly affect the stage-discharge relation. Also, the effects of variable backwater as described in Herschy (1995) are not present. Given the remarkably low variance of the datum corrected stage-discharge relation (Fig 5.4), it is not likely that it is being adversely affected by either hysteresis or variable backwater. Furthermore, this finding makes Eqn. 5.1 a very good candidate for the final model for the determination of discharge at the LEWAS Site.
5.3.2 VELOCITY INDEX RELATIONSHIPS

The following section will discuss the index velocity models discussed in the Literature Review and Proposed Study. Again, these models will be compared to the stage-discharge model to determine which model is ultimately best for the site.

Eqn. 2.6 was applied to data in Table 4.4 and resulted in Figure 5.5: the simple linear relation between $V_m$ and $V_i$ seems to roughly represent this data. Intuitively speaking, the intercept in this model should be zero because when the index velocity is zero, so should the mean-channel velocity. The model appears to conform to this reasoning, rejecting the existence of an intercept (intercept $p > 0.05$). It also explains the variance in the data reasonably well, providing an $r^2$ value of 0.912, and by visual inspection, the data appears to generally follow a linear form. However, more data in the upper $V_i$ range is needed to make this description complete. It is possible that more data would reveal a non-linear relation between $V_m$ and $V_i$; a residuals analysis may provide some insight into this question.

When examining the residuals plot, the residuals are not evenly distributed throughout the range of predicted $V_i$, suggesting either a non-linear relationship, or that the presence of outliers are skewing the true slope of the line, or both. Notice the residuals trend as $V_m$ predicted increases: the model tends to over predict $V_m$ at lower values of $V_i$ and then at higher values of $V_i$, it tends to under predict $V_m$. This would indicate that while the model may be affected in part by the outlier, it may also indicate a different kind of relationship between $V_m$ and $V_i$ entirely. Again, more data in the upper discharge range is needed to complete this analysis. This would bring light to the question of whether the highest $V_m$ measurement is truly an outlier, or whether some
non-linear relationship exists between the two parameters. By applying the parameter estimates from Fig. 5.6 to Eqn. 2.6, the final form of the simple linear regression model expressed with its regression coefficients is as follows:

\[ V_m = (0.796)V_i - 0.162 \]  \hspace{1cm} (5.2)

As described in the Literature Review, Levesque and Oberg (2012) have utilized the multiple regression relationship utilizing stage (H), index velocity (\(V_i\)) and \(V_i*H\) as independent parameters to estimate \(V_m\) in many situations (Equation 2.8).

Let us now examine the relationship found in Eqn. 2.8 in Figure 5.6.

All three parameters, as well as the intercept, show very low p-values. This indicates that all four of these parameters estimates are indeed significant and all should be included in the model. Very importantly, the intercept is significant as well as negative and cannot be discounted in the model. This will have important ramifications when applying the model to real-time hydrologic data since low values of \(V_i\) and \(H\) may result in negative values of \(V_m\). However, providing an \(r^2\) value of 0.982, the multi-regression model explains a great deal more of the variance in the data than the single-regression model \((r^2 = 0.912)\). An examination of the residuals shows no discernible pattern as the residuals showed in Figure 5.5. Based on evidence provided by both the residuals plot and
the regression correlation, the multi-regression model is a better fit for the data collected at the LEWAS Site than the single-regression model. However, the presence of an intercept may make baseflow measurements complicated.

*The non-linear nature between stage (H) and mean-channel velocity (Vm)*

It is interesting to note that Eqn. 2.8 implies a linear nature between stage (H) and mean-channel velocity ($V_m$). Therefore, it is worthwhile to investigate whether this is true or not for the data collected at the LEWAS Site. A simple plot of $V_m$ vs. H shows that the relationship is indeed non-linear (Figure 5.7a). Also, the plot of ln ($V_m$) vs. ln (H) in Figure 5.7b is concave down in very similar way that Figure 5.2b is concave down. This could indicate that applying the same datum offset process to the mean-channel velocity data that was applied to the discharge data could yield a more linear shape to the curve. When this method was applied, the mean-channel velocity datum offset, $\beta$, was found to be -0.468 m. Figure 5.7c is a plot of ln($V_m$) vs. ln(H-$\beta$), showing a much more linear shape than Figure 5.7b. All of these plots however, indicate that the relationship between $V_m$ and H is logarithmic, and certainly non-linear.

---

**Figure 5.7** (a)(top left) Non-linear nature of $V_m$ vs. H curve. (b)(top right) Concave down shape of the logarithmic model. (c)(bottom left) Applying the datum offset ($\beta$) modification to the logarithmic model. (d)(bottom right) Summary of fit and parameter estimates for the regression fit in Figure 5.7c
It is possible then, to modify Eqn. 2.8 to include stage (H) values modified by the regression parameter estimate found in Figure 5.7d. Eqn. 2.8 is modified by replacing the H term with \((H-0.468)^{1.96}\) in order to provide a model that accurately reflects the data at the LEWAS Site. This modification reveals the following model, Eqn. 5.3. This model was plotted in Figure 5.8.

\[
V_m = aV_i + b(H-0.468)^{1.96} + cV_i(H-0.468)^{1.96} + d
\]  

(5.3)

where a, b, c, and d are regression coefficients.

**Figure 5.8 (a) Model results for Eqn. 5.3.**
While the relationship between $V_m$ and $H$ is clearly non-linear, the regression model represented by Figures 5.6 and 5.8 are statistically very similar in all manners. The regression coefficients from the regression provide the final forms of the two multi-regression models.

$$V_m = (-1.52)V_i + (1.71)H + (1.57)V_i*H - (0.88) \quad (5.4)$$

$$V_m = (-1.01)V_i + (0.783)(H-\beta)^{1.96} + (0.141)V_i*(H-\beta)^{1.96} - (0.760) \quad (5.5)$$

Since neither model is a better fit for the data, only Eqn. 5.4 will be selected to be applied to real data for the rainfall-runoff analysis.
One of most important considerations of a discharge model is for stage conditions that rise above the floodplain. As Figures 5.9a and 5.9b show, stages above 1.0m start to show a change in wetted perimeter. The substrate begins to change from cobbler and gravel to grass above 0.74m.
and has changed completely to grass above 1.0m. Both of these changes in hydraulic conditions may influence the slope and nature of the stage-discharge and index velocity models. Therefore, any extrapolation above 1.0 m should be viewed with caution, because these relationships will change dramatically above the 1.0 m stage mark.

Since measurements are very difficult, and dangerous, to perform in high flow conditions, it is recommended that down-looking ADCP instruments be used to record the data at high flows. Especially important are the flows that rise above the floodplain, over 1.0 m of stage. In the future, ADCP data collection should be targeted for stages above 0.5 m for safety reasons.

The upper limit of 1.0 m affects both the stage-discharge and multi-regression model, and this limit must be taken into account when applying both of them. However, the lower limit of the model must also be taken into consideration. The lowest discharge measurement was taken on 9/1/2012 (Table 4.4). Since this is the lower limit for all models, any stage measurement below 0.538 m will report the < 0.00868 m³/s, which is the predicated discharge value for the stage using Eqn. 5.1.
5.3.4 SELECTING THE FINAL DISCHARGE EQUATION AND CREATING THE HYDROGRAPH

Figure 5.10 (a) Stage vs. time for two rain events on 7/19/2012 (b) Index velocity vs. time for the two July 19 rain events. (c) Stage vs. time for the July 24 rain event (d) Index velocity vs. time for the two July 24 rain events.

Hydrologic data from three runoff events, two on 7/19/2012 and one on 7/24/2012 were collected to evaluate the hydrologic models. Figures 5.10 (a-d) show the raw Argonuat-SW Data from two rain events on 7/19/2012 and another on 7/24/2012. Notice that Fig. 5.10b, \( V_i \) vs. time, has a great deal of noise in the data. Even at baseflow on 7/19/2012, the least turbulent flow, the noise level is very high. The 7/19/2012 baseflow \( V_i \) standard deviation value is 1.84 m/s while the mean is 1.80 m/s, making the signal to noise ratio (SN = \( \bar{x}/\sigma \)) less than one. Fig 5.10b also makes it clear that a high level of noise is present at all velocity ranges. On the other hand, Fig. 5.10a shows very little noise. Its baseline measurement is constant and more predictable. The changes in stage (H) are very clear and defined while in Fig 5.10b, the changes in index velocity (\( V_i \)) are not well defined at all. This ambiguity, or noise level, has very important ramifications.
for which discharge model to choose for the LEWAS Site because any discharge model that will utilize index velocity as an independent parameter will experience a noise level similar in magnitude to it. Furthermore, at baseflow, the discharge should not be fluctuating in any significant manner. Using index velocity to predict discharge may mislead students to believe that discharge fluctuates greatly during baseflow, when those with more experience know that baseflow cannot fluctuate to such a degree that the index velocity equations would indicate. These effects must be considered carefully, because one of the aims of this discharge model is to demonstrate the principles of hydrology clearly and succinctly to students. Delving into turbulence and noise levels may in fact obfuscate, rather than enhance the lesson.

One solution to this problem would be to increase the $V_i$ averaging interval from 1 min to a longer duration. However, applying this strategy would mean the other instruments (water quality sonde and weather station) would also have to increase their respective averaging intervals, since all data must be outputted simultaneously and on the same interval. One the LEWAS goals is to collect data at the highest resolution that is meaningful, and the both the sonde and weather station are perfectly capable of outputting meaningful data every minute. Increasing the $V_i$ averaging interval for the Argonaut ADCP would effectively limit the overall resolution capabilities of the LEWAS platform.

Since the LEWAS Lab will incorporate real-time data into the CEE 4304 classroom, it would be best to present the material to students as simply and tangibly to the student as possible. Using stage, as opposed to index velocity, as the independent parameter would accomplish this goal simply because stream stage is more tangible to the student. It is also visually accessible; photographs of the site during rain events can be shown to correlate with real-time stage measurements increasing and decreasing with time. Then from stage, a stage-discharge equation can be constructed. This progression from photos to real-time stage measurements to hydrographs could be an excellent teaching tool for the CEE 4304 classroom.

In light of these realizations and given the excellent statistical results of the stage-discharge relationship found at the LEWAS Site, it is therefore most practical and best to use stage alone as the independent parameter for the determination of discharge at the LEWAS Lab. Using Eqn.
5.1, discharge was determined for three rain events on 7/19/2012 and 7/24/2012. These hydrographs are displayed in Figures 11a and 11b.

![Figure 5.11](a) Hydrograph of two rain events (July19A and July19B) on 7/19/2012 (b) Hydrograph of single rain event (July24) on 7/24/2012.

### 5.4 APPLICATION OF THE STAGE-DISCHARGE EQUATION

An important application of hydrograph and precipitation data is to compute runoff to rainfall ratios (ROR). The CEE 4304 course intends to use hydrograph data produced by the LEWAS Site as a classroom lesson in computing runoff to rainfall ratios. Using data collected for three separate rain events on 7/19/2012 and 7/24/2012 by the Sontek Argonaut-SW, an example lesson was created that would outline the steps to calculate the ROR. These simple steps are outlined below.

The results of this lesson were then used to estimate the viability of the final stage-discharge equation, especially during high flow events like the July24 rain event. ROR values from two rain events of different magnitudes and gage height maximums were compared in order to examine whether exceeding the 1m stage maximum for the rating curve grossly underestimates discharge at the site. It was found that gross underestimation does not occur but a small amount of underestimation may be present. Further analysis is necessary to validate the discharge model and ROR methodology and can be a future endeavor of the LEWAS Lab.
5.4.1 STEP 1. CALCULATE THE RUNOFF VOLUME OF EACH RAIN EVENT.

To calculate the runoff from each of the three rain events, the constant slope baseflow separation method from McCuen (1989) was applied to the three rain events hydrographs (Figs 5.12a and 5.12b). Table 5.6 displays the runoff amounts from each runoff event.

![Figure 5.12](a) Baseflow separation on hydrographs July19A and July19B. (b) Baseflow separation on hydrograph July24.

5.4.2 STEP 2. CALCULATE TOTAL RAINFALL FOR WEBB BRANCH WATERSHED

The other component in determining the ROR is computing the total volume of rainfall over the Webb Branch subwatershed. First the watershed was delineated in ArcGIS 10, using the LEWAS Site as the pour point. Figure 4.2 shows the delineation against an aerial photograph of the Blacksburg Area.

In order to compute the average depth of rainfall for the Webb Branch from each rain event, an inverse distance weight (IDW) interpolation was performed on rainfall data from five different rain gages in Blacksburg for the three rain events in ArcGIS 10 (Figure 5.13(a-c)). Using raster analysis, the values of interpolated rainfall amounts within the watershed boundary were averaged for each rain event. This value, multiplied by the watershed area (2.78 km²), reveals the total rainfall volume for each rain event. The rain gage locations and rainfall depths for the three rain events are displayed in Table 5.5. It also includes the IDW averaged values of rainfall for the Webb Branch for each of these events.
Figure 13a. IDW rainfall amount interpolation analysis for July19A rain event using five rain gage stations in Blacksburg, VA. Dotted squares represent rain gage locations.
Figure 13b. IDW rainfall amount interpolation analysis for July 19B rain event using five rain gage stations in Blacksburg, VA. Dotted squares represent rain gage locations.
Figure 13c. IDW rainfall amount interpolation analysis for July24 rain event using five rain gage stations in Blacksburg, VA. Dotted squares represent rain gage locations.
Table 5.5 Blacksburg rain gage locations and observed rainfall amounts.

<table>
<thead>
<tr>
<th>Rain Gage Station</th>
<th>July 19A Rain Event</th>
<th>July 19B Rain Event</th>
<th>July 24 Rain Event</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rainfall (mm)</td>
<td>Rainfall (mm)</td>
<td>Rainfall (mm)</td>
</tr>
<tr>
<td>Cedar Run</td>
<td>0.0</td>
<td>0.0</td>
<td>7.4</td>
</tr>
<tr>
<td>CRC/NOAA</td>
<td>0.0</td>
<td>0.8</td>
<td>6.9</td>
</tr>
<tr>
<td>Shenendoah</td>
<td>0.5</td>
<td>0.0</td>
<td>5.6</td>
</tr>
<tr>
<td>Stroubles Mill</td>
<td>1.3</td>
<td>1.3</td>
<td>2.8</td>
</tr>
<tr>
<td>Wyatt Farms</td>
<td>2.0</td>
<td>0.0</td>
<td>7.9</td>
</tr>
<tr>
<td>IDW Webb Mean</td>
<td>0.67</td>
<td>0.15</td>
<td>5.6</td>
</tr>
</tbody>
</table>

5.4.3 STEP 3. COMPUTE THE RUNOFF TO RAINFALL RATIO

The ROR value is computed by dividing the runoff found in Step 1 by the total rainfall volume found in Step 2. Table 5.6 displays the values of the ROR for each of the three rain events.

Table 5.6 Runoff/Rainfall ratio summary for three rain events.

<table>
<thead>
<tr>
<th></th>
<th>July 19A Rain Event</th>
<th>July 19B Rain Event</th>
<th>July 24 Rain Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Runoff (m³)</td>
<td>491</td>
<td>198</td>
<td>3135</td>
</tr>
<tr>
<td>Total Rainfall (mm)</td>
<td>0.67</td>
<td>0.15</td>
<td>5.6</td>
</tr>
<tr>
<td>Watershed Area (km²)</td>
<td>2.78</td>
<td>2.78</td>
<td>2.78</td>
</tr>
<tr>
<td>Total Rainfall (m³)</td>
<td>1861</td>
<td>411</td>
<td>15519</td>
</tr>
<tr>
<td>Runoff/Rainfall Ratio</td>
<td>0.26</td>
<td>0.48</td>
<td>0.20</td>
</tr>
</tbody>
</table>

For rain events July19A and July19B, the ROR values are 0.26 and 0.48, respectively. This increase in the ROR is reasonable for consecutive rain events such as these. As the ground becomes saturated, less precipitation is lost through infiltration, thereby increasing the ROR. While the increase in ROR is reasonable for the consecutive rain events, the absolute values appear, at least initially, to be low for the urbanized Webb Branch watershed (Dymond 2012). However, in a study done in Italy on 21 urban catchments, the mean ROR value was found to be 0.29, the sample having a mean standard deviation of 0.07 (Becciu and Paoletti 2000). The mean ROR from this study indicates that 0.26 value found in the LEWAS study is not totally unreasonable. Also, the mean standard deviation of 0.07 indicates that each catchment shows significant variability in its ROR, which could also be present in the LEWAS ROR value set, but is currently unknown. Therefore, more samples of the LEWAS ROR are needed in order gain a
better estimate of the true mean ROR value, before it can be declared low. While an analysis of these few ROR values is still appropriate, continuous samples of the LEWAS ROR would provide a more robust analysis. The following paragraphs will explore a few possibilities that could explain a low ROR value, and discover the most likely source of error.

Notice that in Figure 5.10c, the stage has reached beyond the 1 m upper limit described earlier. The stage-discharge relationship in Eqn. 5.1 cannot be trusted above 1 m because the relationship between stage and wetted perimeter changes dramatically. The stage during the rain event July24 reached a maximum of 1.079m, above the upper limit of the discharge-rating curve. It is likely that the discharge is being underestimated above this limit because at this stage the change in area with respect to stage $\frac{\partial A}{\partial H}$ increases substantially, indicating that $\frac{\partial Q}{\partial H}$ also increases significantly. However, the July19A rain event does not exceed the 1.0m mark and also exhibits a low ROR (0.26). Therefore, this would not be the most likely source of error.

Other explanations for the low ROR exist such as the possibility of the Webb Branch being a losing stream, or that karst beneath Blacksburg increases infiltration dramatically. The most plausible explanations, however, are that the watershed delineation is overestimating the watershed area and that the IDW precipitation estimate is too large. That is, the precipitation is being overestimated. Notice that in Figures 4.2, the watershed boundary was delineated based on topographic data (Lidar DEM, courtesy of TOB) and does not take into account the complex system of storm sewers and detention ponds that exist in Blacksburg and Virginia Tech. It is possible that the precipitation falling within the watershed boundary is being diverted to adjacent watersheds through the storm sewer network. In fact, it is known that the parking area between Stanger Rd. and West Campus Dr. diverts its runoff to the adjacent wetland area near the Virginia Tech Inn, and not the LEWAS Site (Hester 2012). This would of course decrease the amount of runoff entering the site. With regard to IDW analysis, notice that in Figures 13(a-c), not any of the five rain gauge stations falls within the watershed boundary. Also notice that the variability in rainfall amounts is high in all three rain events. This indicates that a significant error, such as an overestimation, could be present in the IDW mean rainfall for the Webb Branch watershed for those rain events.
5.5 STAGE-DISCHARGE MODEL VALIDATION

One possibility remains that could explain the low ROR; the runoff is being underestimated due to a defective stage-discharge relationship established in the previous section. This section has made an attempt to validate the stage-discharge relation by applying the index velocity models to the same July 19th runoff data and comparing the resulting discharges and runoff coefficients. Comparisons to the TOB Webb St gauging site and the on-site culvert weir were also made.

![Figure 5.14](image)

*Figure 5.14* Hydrographs produced by all three models for July 19 runoff event.

By visual inspection, it is clear from Figure 5.14 that all three LEWAS models are predicting similar discharges. Table 5.7 contains the total runoff and ROR values for the stage-discharge as well as the single and multi-regression index velocity models. [Note: the single-regression intercept was eliminated because the original model produced negative values of discharge. Recall that the regression p-value for the intercept indicated that it was not likely to be anything but zero.]

![Graph](image)
Table 5.7 Total runoff and ROR values for all three models applied to the July 19 rain events.

<table>
<thead>
<tr>
<th>Model</th>
<th>July 19A Rain Event</th>
<th>July 19B Rain Event</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Runoff (m³)</td>
<td>Total Runoff (m³)</td>
</tr>
<tr>
<td>Stage-discharge</td>
<td>443</td>
<td>198</td>
</tr>
<tr>
<td>Single Index Velocity</td>
<td>523</td>
<td>195</td>
</tr>
<tr>
<td>Multi Index Velocity</td>
<td>590</td>
<td>344</td>
</tr>
<tr>
<td>Std Dev (m³)</td>
<td>74</td>
<td>85</td>
</tr>
<tr>
<td>Std Dev (% of mean)</td>
<td>14</td>
<td>35</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Runoff/Rainfall Ratio</th>
<th>Runoff/Rainfall Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage-discharge</td>
<td>0.26</td>
<td>0.48</td>
</tr>
<tr>
<td>Single Index Velocity</td>
<td>0.28</td>
<td>0.47</td>
</tr>
<tr>
<td>Multi Index Velocity</td>
<td>0.32</td>
<td>0.84</td>
</tr>
</tbody>
</table>

An examination of Table 5.7 reveals that all three models produce similar ROR values, with the exception of the July 19B multi-regression model value of 0.84. While there is significant difference in the runoff amounts produced, they all produce low ROR values similar to the stage-discharge model. This is evidenced by the % std. dev. of the mean (i.e. coefficient of variation) of 14% for the three LEWAS models. While this does not absolutely validate the stage-discharge model, it does provide evidence to its viability, since three independently constructed models using two independent parameters (H and V_i) have reported very similar discharge values.

One weakness in the above argument is that both types of models were calibrated using the same discharge data, and thus are not completely independent assessments of discharge. This discharge data set could have been compromised due to miscalculation. However, the author has taken the raw data and recalculated the discharge values from scratch and arrived at the same values. Discharge sampling bias also could have been introduced, compromising the discharge outputs of the three LEWAS models. The next section will address whether gross bias exists by comparison to completely independent discharge assessments from TOB Webb St. (Fig. 4.2) and the on-site culvert weir (see Thye (2003) for v-notch weir calibration).
Figure 5.15 shows the Webb St and weir discharges along with the discharge values from the three LEWAS models.

![Figure 5.15 Hydrograph reported by the Webb St gauging station, the culvert weir and LEWAS.](image)

It is very clear from Figure 5.15 that the Webb St station discharge is much lower than that of LEWAS models. The reason for this discrepancy is unclear, especially since the Price parking lot runoff is being diverted to the wetland area and not the LEWAS station. However, before the Price parking lot, there are other inputs between Webb St. and LEWAS; this may account for the discrepancy. A thorough sewer system analysis for the Webb Branch is necessary to completely answer this question.

Comparison to the weir also reveals lower discharge values compared to LEWAS, but the values are reasonably close. It has been observed at the LEWAS site that during high stream flows the storm sewer pipe in Figure 4.6 begins to discharge. Since this pipe is between the weir and
LEWAS gauging station, this additional discharge to the LEWAS station could explain the discrepancy in Figure 5.15. However, it is important to keep in mind that the weir was calibrated in a flume and not at the site (Thye 2003), and may not be correctly calibrated for the real environment. It may be useful to calibrate the weir in the future to provide a more robust independent assessment of discharge.

In conclusion, the steps to compute the ROR have been clearly delineated but have sparked a question: Why are the ROR values for Webb Branch so low for an urbanized environment? Since all three LEWAS models, as well as the on-site weir report similar discharge values, it is very possible that precipitation is being overestimated. Since not a single rain gage used in the IDW estimation is located within the washed boundary, and some many kilometers away, the precipitation estimate may contain significant error. Also, the watershed boundary is not the true boundary, it is itself only an estimate, and this fact can also lead to error in the precipitation estimate. The best way to validate the LEWAS model(s) is to deploy boat mounted ADCP at the site during runoff events and compare the total discharge between the ADCP and LEWAS.
6. CONCLUSIONS

This final chapter will summarize the insights this study has provided into the research objectives outlined in the Proposed Study section of this thesis.

6.1 INSIGHTS TO RESEARCH OBJECTIVES

1. Analyze the data collected in this study and recommend the most time-efficient method to collect hydraulic data at the LEWAS Site.

During future discharge sampling events, it is recommended that 9 stations be used instead of 22. This will reduce the discharge sampling duration and decrease the error associated with $\partial H/\partial t$. The sampling time, or averaging interval, of point velocity measurements will remain at 10s. The data indicate that the relative error due to rapidly changing flow ($r\sigma_{cf}$) is more significant at higher flows than the relative error due to averaging interval ($r\sigma_{av}$). Measurement duration therefore is the dominant factor controlling sampling error ($\sigma_{se}$) and must be minimized under all flow conditions.

2. Determine whether hysteresis or backwater affects the stage-discharge rating curve.

Unsteady conditions complicate the use of stage-discharge relationships and often make the index velocity method the best viable option. However, despite rapidly changing flow conditions, hysteresis or looping was not present in the stage-discharge relationship found at the LEWAS Site.

3. Determine which model discussed in the literature review is best in terms of accuracy, simplicity, noise level and applicability for educational purposes. This will become the discharge model LEWAS will use to construct hydrographs of future runoff events.

The stage-discharge relationship was the best model because of its simplicity, strong statistical results, and its ability to easily convey hydrologic principles in a clear manner to students. The stage-discharge relationship possessed a stronger $r^2$ value than the single-regression index.
velocity relationship (0.98 vs. 0.91) and an equal correlation value to the multi-regression models. Perhaps the greatest difference was that a great deal of noise was found in the raw $V_i$ data, resulting in some negative values of discharge for those models. In contrast, stage showed very little noise, and the determination of flow from the raw stage data was a simple and accurate process. When introducing concepts to students, simplicity is often the best approach.

The model has its limits, however. Stage values over 1.0m will most likely produce underestimated discharge values. Caution should be used when applying any of the three models above this 1.0m stage mark.

No discernible difference was found between the novel multi-regression model and the one found in the literature. For this reason, the novel multi-regression model was not applied to data collected by the Argonaut ADCP.

4. Construct pilot hydrographs by applying the selected model to data collected during three runoff events in July 2012. Calculate the runoff-rainfall ratio (ROR) for each rain event.

The model successfully created hydrographs for the July 19 and 24 rain events. However, the ROR values ($\mu = 0.23$) for those events are low by comparison to other urban watersheds.

5. Validate the selected model and determine if the data it produces is reasonable by comparison to other model outputs and other gauging sites.

The two index velocity models (single and multi-regression) were applied to the July 19 data along with the stage-discharge model. All three models produced similar discharge and ROR values. The on-site weir and TOB Webb St station discharges were also compared against the three LEWAS models. The weir produces lower discharges than LEWAS, but the values are reasonably close. Meanwhile, the TOB Webb St values were much lower than the LEWAS models. Furthermore, since both the weir and TOB values are lower than LEWAS, it is most likely the precipitation estimate is responsible for the low ROR value, and not the discharge models developed in this study.
6.2 FUTURE NEEDS

6.2.1 MODEL STRENGTHENING AND FURTHER VALIDATION

ADCP discharge data in the higher stage ranges (0.75 m to 1.0 m) are needed in order to strengthen all three LEWAS models. Above 1 m, discharge data is needed to describe and calibrate all three LEWAS models for discharges that rise into the floodplain. Model validation requires a completely independent discharge method such as ADCP; at random discharge values, ADCP should be used to QC and validate the LEWAS models.

6.2.2 CORRECTING THE RAINFALL-RUNOFF RATIO (ROR)

More rainfall gages within the Webb Branch watershed should be used to calculate the average rainfall over the watershed. This would decrease the error of the precipitation estimate. Also, a rain event could be chosen to evaluate the ROR that has very little variance amongst the five existing rain stations. A small variance should result in a lower error of the rainfall estimate. It is also recommended that another watershed boundary be delineated that reflected the system of storm sewers (i.e. sewer-shed) within the watershed and not simply the topography of the area.
REFERENCES


APPENDIX A: THE VELOCITY AREA METHOD

The velocity area method is very commonly used to measure discharge in natural channels (Herschy 1993b). Figure A.1 represents a simplified illustration of a natural cross-section. Each vertical, depicted by the vertical black lines and given their locations by survey techniques, divides the stream cross-section into subsections (denoted by their respective areas, $A_1$, $A_2$, $A_n$). Each station is located at the midpoint between each vertical and corresponds to a subsection (1,2,3…n). The black points represent point velocity measurements made at 0.6 the depth (measured from the surface) at each station. Multiplying the velocity measured at each subsection ($V_j$) by the area of that subsection ($A_j$) determines the discharge through each subsection (i.e. $Q_j = V_j \times A_j$). Summating the subsection discharges ($Q_1, Q_2…Q_n$) then reveals the entire stream discharge ($Q$).

![Figure A.1 Illustration of the velocity area method.](image)

The areas of each subsection ($A_j$), as well as the total cross-sectional area ($A$) are determined using the stage ($H$) recorded during each discharge measurement in conjunction with the stream transect data set. As the reader can see from figure 7, each value of stage ($H$) would correspond to a unique value of cross-sectional area ($A$). To obtain this area, each subsection is treated like a trapezoid. That is, the height of each leg of the trapezoid ($l_1, l_2…l_{n-1}$) is the bed elevation (BE) subtracted from the stage ($H$) at each vertical. The width of each section ($w_j$) is the horizontal distance between these two verticals.
For example:

\[ A_3 = \frac{1}{2} (L_3 + L_2)(w_3) \]  

(4.1)

This calculation is then repeated for every subsection.

Total cross-sectional area (A) is calculated by summing the subsection areas:

\[ A = A_1 + A_2 + \ldots + A_n \]  

(4.2)

The total discharge of the stream (Q) is calculated by the following:

\[ Q = Q_1 + Q_2 + \ldots + Q_n = V_1 A_1 + V_2 A_2 + \ldots + V_n A_n \]  

(4.3)

where

- Q is the total stream discharge (m³/s)
- \( V_j \) is the point velocity measurement (m/s)
- \( A_j \) is the area of each subsection (m²)
- \( j = (1, 2, \ldots, n) \)

The cross-sectional average velocity (\( V_m \)) can now be calculated:

\[ V_m = \frac{Q}{A} \]  

(4.4)

where

- Q is the total stream discharge (m³/s)
- \( V_m \) is cross-sectional average velocity (m/s)
- A is the cross-sectional area (\( A_1 + A_2 + \ldots + A_n \)) in (m²)
Figure B.1 Diagram for the location of all stations. Station patterns using 32, 22, 16 and 9 stations were used to calculate discharge measurements.
APPENDIX C: MODEL REGRESSION PARAMETERS

Regression parameters from the model found in the Data Analysis and Findings are tabulated in this appendix. The caption will refer to the equation and figure the regression parameters correspond to. For instance, the regression parameters found in Table C5.1 corresponds to the model described by Equation 5.1 (corresponds to regression found in Figure 5.4a).

Table C5.1 Regression parameters for Equation 5.1
(corresponds to regression found in Figure 5.4a)

<table>
<thead>
<tr>
<th>Parameter Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Term</td>
</tr>
<tr>
<td>Intercept</td>
</tr>
<tr>
<td>ln (H-a)</td>
</tr>
</tbody>
</table>

Summary of Fit

| RSquare | 0.96477 |
| RSquare Adj | 0.963763 |
| Root Mean Square Error | 0.241151 |
| Mean of Response | -2.63714 |
| Observations (or Sum Wgts) | 37 |

Table C5.2 Regression parameters for Equation 5.2
(corresponds to regression found in Figure 5.5)

<table>
<thead>
<tr>
<th>Parameter Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Term</td>
</tr>
<tr>
<td>Intercept</td>
</tr>
<tr>
<td>Vi (m/s)</td>
</tr>
</tbody>
</table>

Summary of Fit

| RSquare | 0.914172 |
| RSquare Adj | 0.91172 |
| Root Mean Square Error | 0.046795 |
| Mean of Response | 0.155 |
| Observations (or Sum Wgts) | 37 |
### Table C5.4 Regression parameters for Equation 5.4
(corresponds to regression found in Figure 5.6)

<table>
<thead>
<tr>
<th>Summary of Fit</th>
</tr>
</thead>
<tbody>
<tr>
<td>RSquare</td>
</tr>
<tr>
<td>RSquare Adj</td>
</tr>
<tr>
<td>Root Mean Square Error</td>
</tr>
<tr>
<td>Mean of Response</td>
</tr>
<tr>
<td>Observations (or Sum Wgts)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Term</td>
</tr>
<tr>
<td>Interception</td>
</tr>
<tr>
<td>Vi (m/s)</td>
</tr>
<tr>
<td>H (m)</td>
</tr>
<tr>
<td>Vi*H</td>
</tr>
</tbody>
</table>

### Table C5.5 Regression parameters for Equation 5.5
(corresponds to regression found in Figure 5.8)

<table>
<thead>
<tr>
<th>Summary of Fit</th>
</tr>
</thead>
<tbody>
<tr>
<td>RSquare</td>
</tr>
<tr>
<td>RSquare Adj</td>
</tr>
<tr>
<td>Root Mean Square Error</td>
</tr>
<tr>
<td>Mean of Response</td>
</tr>
<tr>
<td>Observations (or Sum Wgts)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Term</td>
</tr>
<tr>
<td>Interception</td>
</tr>
<tr>
<td>Vi (m/s)</td>
</tr>
<tr>
<td>c(H-B)^d</td>
</tr>
<tr>
<td>Vi*c(H-B)^d</td>
</tr>
</tbody>
</table>