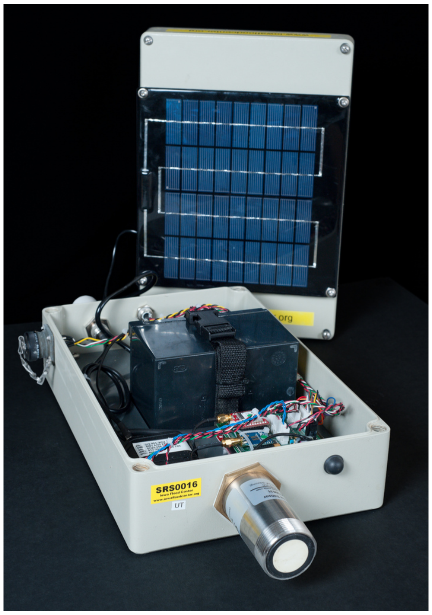
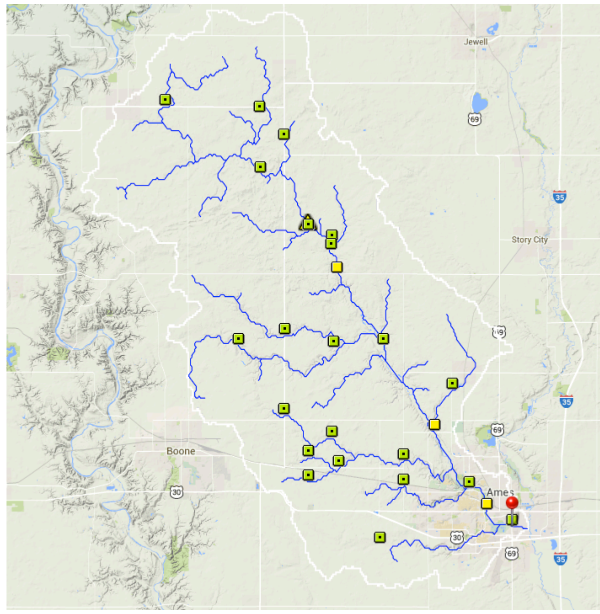
PILOT PROJECT FOR A HYBRID ROAD-FLOODING FORECASTING SYSTEM ON SQUAW CREEK

Final Report

IHRB Project TR-642





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About IIHR- Hydroscience & Engineering and The Iowa Flood Center

IIHR – Hydroscience & Engineering is a unit of the College of Engineering at the University of Iowa. It is one of the nation's oldest and premier environmental fluids research and engineering laboratories. IIHR seeks to educate students on conducting research in the broad fields of river hydraulics, sediment transport, and watershed processes. IIHR has 60 faculty members and research engineers at the Ph.D. level, 15 postdoctoral scholars, and about 108 M.S. and Ph.D. graduate students. IIHR's 43 staff members include administrative assistants (including grants accounting and reporting support), IT specialists, and machine/electrical shop engineers.

The Iowa Flood Center (IFC) was established and funded by the State of Iowa in spring 2009 and commenced July 1, 2009. It is housed by IIHR-Hydroscience & Engineering on The University of Iowa campus. The IFC's overarching objective is to vastly improve flood monitoring and prediction capabilities in Iowa. This is accomplished through a variety of activities with the common goal of improving the transfer of the latest research, information, and technologies into the hands of the appropriate agencies, policy makers, citizens, and other stakeholders.

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16. Abstract

A network of 25 sonic stage sensors were deployed in the Squaw Creek basin upstream from Ames Iowa to determine if the state-of-the-art distributed hydrological model CUENCAS can produce reliable information for all road crossings including those that cross small creeks draining basins as small as 1 sq. mile. A hydraulic model was implemented for the major tributaries of the Squaw Creek where IFC sonic instruments were deployed and it was coupled to CUENCAS to validate the predictions made at small tributaries in the basin. This study demonstrates that the predictions made by the hydrological model at internal locations in the basins are as accurate as the predictions made at the outlet of the basin. Final rating curves based on surveyed cross sections were developed for the 22 IFC-bridge sites that are currently operating, and routine forecast is provided at those locations (see IFIS). Rating curves were developed for 60 additional bridge locations in the basin, however, we do not use those rating curves for routine forecast because the lack of accuracy of LiDAR derived cross sections is not optimal. The results of our work form the basis for two papers that have been submitted for publication to the Journal of Hydrological Engineering. Peer review of our work will gives a strong footing to our ability to expand our results from the pilot Squaw Creek basin to all basins in Iowa.

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PILOT PROJECT FOR A HYBRID ROAD-FLOODING FORECASTING SYSTEM ON SQUAW CREEK
Final Report

Highway Research Board

Iowa Department of Transportation

TR-642

For the period January 1st, 2012 to October 1st, 2014

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EXECUTIVE SUMMARY

The distributed hydrological model CUENCAS has been implemented for the Squaw Creek basin to test the predictability of flooding in small tributaries in the catchment. The long-term goal, and the umbrella for this pilot project, is to create a real-time road-flood forecasting system that is reliable enough to produce actionable results for state and local agencies responsible for maintaining road safety during extreme flooding events. The novel aspect of this objective is that it is expected that the model can produce reliable information for all road crossings including those that cross small creeks draining basins as small as 1 sq. mile.

The Iowa Highway Research Board sponsored project TR-642 to investigate the development of a "Hybrid Road-Flooding Forecasting System" that combined real-time observation of stage at multiple locations in the watershed using sonic-state-sensors developed by the Iowa Flood Center and a state-of-the art hydrological model capable of simulating streamflow hydrographs for all locations in the river network draining a catchment. At the beginning of this project the hydrological model CUENCAS had only been verified for large-scale basins (i.e. larger than 250 km^2). A network of 25 sonic-sensors in the Squaw Creek was proposed for this new test.

The instruments (shown in Figure 1) were built and installed during the spring and summer of 2012 and they serve a double purpose. First, they provide real-time information of site-specific stream conditions that can be visualized in the Iowa Flood Information System (IFIS), and second; they collect information that can be used to validate the predictions made by CUENCAS.

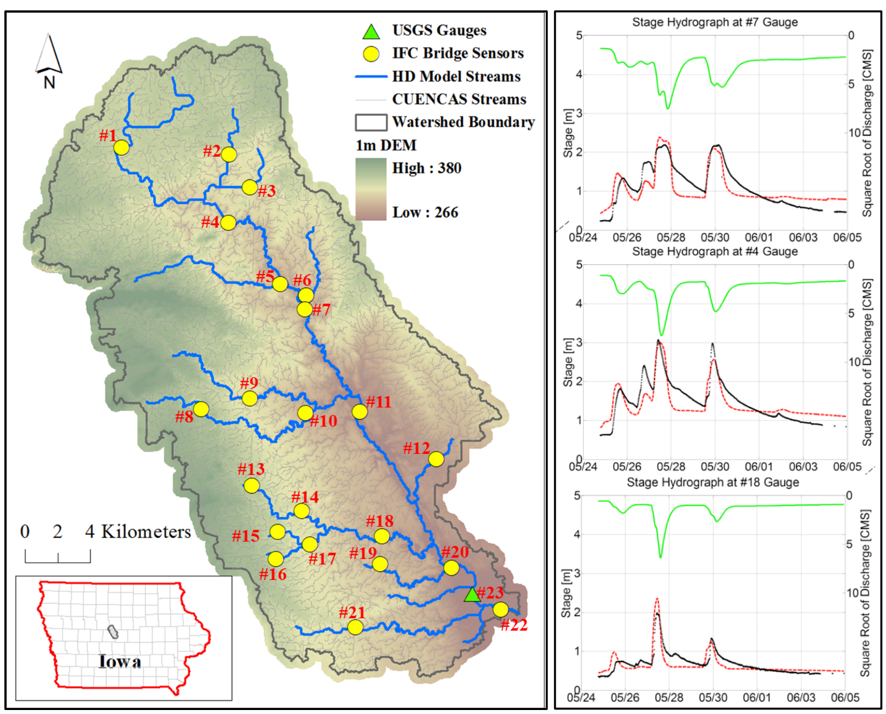

Figure 1. Squaw Creek basin upstream from Ames, Iowa showing (left) the location of sonic-sensors and (right) measurements made by the instruments (dots) and modeled by by the hydrological model (green lines) and the coupled hydraulic model (red).

The original idea that was proposed to The Iowa Highway Research Board was that data collected by the sensors over multiple small and large flood events could be used to validate the assumptions made by the hydrological model regarding the space-time distribution of flow velocities in the basin. In particular, it was hypothesized that peak-flow time of arrival was a measure that should be accurately predicted by the hydrological model if the velocity function assumed was correct. There were two difficulties that precluded a complete test of the hypothesis. First, the small number of flood events that was recorded during the duration of the project, 2012 was unusually dry year in Iowa and therefore no significant flood events were recorded. The spring of 2013 brought stronger storms and at least one significant flood event to the basin, but a dry summer and fall seasons followed it. Second, the precision in prediction of timing of peak flow arrivals was not accurate enough to produce reliable conclusions after simulating one single major event.

A decision was made in the fall of 2012 to look for an alternative form of model validation that relied on the information that was collected across the basin during the largest flood event in Spring 2012. To this end, we developed tools to couple CUENCAS to a one-dimensional hydraulic model (similar to HEC-RAS) to translate discharges into stages that could be compared to the measurements made by the sonic-sensors. Several GIS tools needed to be developed to accomplish this goal but the resulting coupling provided an unequivocal signal that the good performance of CUENCAS at the basin outlet coincided with the accuracy of the model at internal locations as small as 1 square miles.

The outcome of this project can be summarized as follow: **1)** 25 sonic sensors were deployed in the Squaw Creek basin. **2)** 22 sonic sensors continue operating and collecting information in the basin (3 instruments had to be brought back to the lab because of deployment issues). **3)** The hydrological model CUENCAS was implemented and tested in the basin and validated at the outlet and at internal locations. **4)** A hydraulic model was implemented for the major tributaries of the Squaw Creek where IFC sonic instruments were deployed. **5)** Final rating curves based on surveyed cross sections were developed for the 22 IFC-bridge sites that are currently operating, and routine forecast is provided at those locations (see IFIS). **6)** Rating curves were developed for 60 additional bridge locations in the basin, however, we do not use those rating curves for routine forecast because the lack of accuracy of LiDAR derived cross sections is not optimal. **7)** We have demonstrated that the predictions made by the hydrological model at internal locations in the basins are as accurate as the predictions made at the outlet of the basin.

The results that are expanded in this report form the basis for two papers that have been submitted for publication to the Journal of Hydrological Engineering (Ms. No. HEENG-2348 and Ms. No. HEENG-2342). The papers have been well received by the peer-review process and we are currently addressing some moderate and minor concerns of the reviewers. We have attached the abstracts of the two papers in an appendix. Peer review of our work will give a strong footing to our ability to expand our results from the pilot Squaw Creek basin to all basins in Iowa.

INTRODUCTION

According to the National Weather Service, more than half of the fatalities attributed to flash floods are people swept away in vehicles when trying to cross an intersection that is flooded. As little as two feet of water (60 cm) can carry away most SUV-sized vehicles. In fact, using a national 30-year average, more people die yearly in floods, 127 on average, than by lightning (73), tornadoes (65), or hurricanes (16) (e.g. Plate 2002). Efforts are underway to improve prediction of the likelihood of roads to be inundated after heavy storms, however, the rapid rise of waters on small and medium size creeks requires accurate forecasting capabilities that are beyond the current state-of-the-art. This lack of predictive capabilities limits the ability of local authorities to close and monitor dangerous roads with enough anticipation to avoid loss of human lives and property damage.

Figure 2. Road inundated by a flash flood.

In an ideal situation (i.e. with unlimited resources), every road-river intersection would be monitored on a continuous basis using electronic stream-level measuring devices. Measurements would be reported in real-time to a central system for public notification, thus greatly reducing the problem of road hazard by flooding. This ideal situation is impractical since every 10 miles of road intersect, on average, five streams that can potentially flood. A network of thousands of such monitoring devices would be necessary to create a fail-safe system. In contrast, the United States Geological Survey provides monitoring of river discharge at just over 150 locations in the entire state of Iowa using standard gauging technologies with a cost of about $10,000 per year each.

On the opposite end of possibilities, a distributed flood-forecasting mathematical model capable of highly accurate predictions (i.e. with errors on the order of 1%) could replace the need for a network of observations by making predictions of flooding in all the intersections of roads and streams in a river network. However, the level of accuracy of current hydrologic models is much lower (~ 50% error) precluding their use as a sole forecasting tool of road conditions. In addition, the architecture of standard hydrologic models precludes the ability of forecasting flood levels on small tributaries. As an example, the National Weather Service provides routine stream level forecasts for about 100 locations in the state of Iowa. These forecasting locations usually correspond to large cities or highly populated regions, but provide no information on small creeks or the multiple intersections of roads and streams.

PROBLEM STATEMENT

As an alternative to traditional flood forecasting models we present the design, implementation and evaluation of a hybrid flood forecasting system that combines real-time stream level observations with a state-of-the-art distributed hydrologic models called CUENCAS-HM. The system will, over time, provide accurate predictions of flooding potential for each and every road/stream intersection in a river basin. The observation component of the system is accomplished with a stream-level sensing device, which uses ultrasound technology to measure the distance from the bridge deck to the stream water surface. The device is designed for installation under the deck of a bridge. The hydrologic model provides a faithful representation of the waterways in a river basin and does not rely on calibrated parameters. However, it depends on the accurate description of travel times along the channels of the river networks. In the following section we describe the instrument, the hydrologic model and how the information collected by the instruments improves the performance and accuracy of the hydrologic model.

METHODOLOGY: THE HYBRID ROAD-FLOODING FORECASTING SYSTEM

Our vision is a paradigm shift in stream-water-level observation and stream-flow modeling. First, in regards to stream-level observation, our framework favors the observation of multiple small creeks inside the river basin of interest, rather than merely at the outlet (i.e. we are replacing time for space when it comes down to observations). Second, in regards to stream-flow modeling, we take a step away from lumped calibrated models (e.g. the Sacramento model used by the National Weather Service). Instead, we favor a physically-based distributed model that represents the full extent of the river network as it exists on the terrain (e.g. Mantilla and Gupta 2005), and that are forced with high resolution space-time rainfall fields (Krajewski et al. 2011). The advantage of using such detailed models is that observations at internal locations of the river network (e.g. arrival time of flood crest) can be used to improve the description of the velocity function that describes water movement in the channels of the river network. The next subsections present a more detailed description of the instrument and the hydrologic model.

BRIDGE MOUNTED STREAM-LEVEL RECORDING INSTRUMENT

Our team at the Iowa Flood Center (IFC) has developed an electronic automated sensor that measures stream water height (stage) and transmits this measurement automatically and frequently to a central location. The sensor is placed under bridges and uses a sonar signal to measure the distance from the water surface to the sensor. Data from the sensor, and other known parameters at each site, are used to determine stream flow and thus flood stage. The Iowa Flood Center has deployed about 200 of such sensors throughout Iowa (check the Iowa Flood Information System for their locations).

Description of The Distributed Hydrological Model

The recent development of accurate digital elevation models (DEMs) in the last decade (see USGS National Elevation Dataset at http://ned.usgs.gov/) has fostered construction of detailed models of landscape connectivity, thereby facilitating the implementation of hydrological distributed models capable of predicting flow hydrographs, and hence peak flows, for all the streams in a network.

Mantilla and Gupta (2005) developed a decomposition of the landscape into channel-links and hillslopes determined by the river network. As illustrated in Figure 3, changes in mass on a channel-link, are determined by incoming fluxes from upstream channel-links, $q_1(s,t)$ and $q_2(s,t)$, the lateral runoff from the hillslope, $R(s,t)$, and outgoing discharge, $q_i(s,t)$. For water stored on a hillslope $V(s,t)$, changes in mass are dictated by the difference between incoming precipitation, $P(s,t)$, and outgoing evapotranspiration, $E(s,t)$, and runoff. This representation of the landscape gives rise to a system of coupled non-linear ordinary differential equations (ODEs), in which a self-similar river network determines the connectivity of the landscape units (e.g. Reggiani et al. 2001). Constitutive relationships (determining the intensity of the fluxes) for hillslope units can be derived from the micro-scale Richards equations, as proposed by Duffy (1996), and by integrating the Navier-Stokes equations over a channel-link, as demonstrated by Kean and Smith (2005). These detailed models of water fluxes require a description of the velocity of water in the different locations of the river network. Mantilla (2007) developed an equation to describe the changes of velocity in space and time of water in the channels of the river network (q) and the local upstream area A, which can be parameterized with direct observations of water movement (i.e. $\bar{v} = v_o q^{\lambda_1} A^{\lambda_2}$). This parametric is equivalent to the solution of the momentum equation in the Saint-Venant equations.

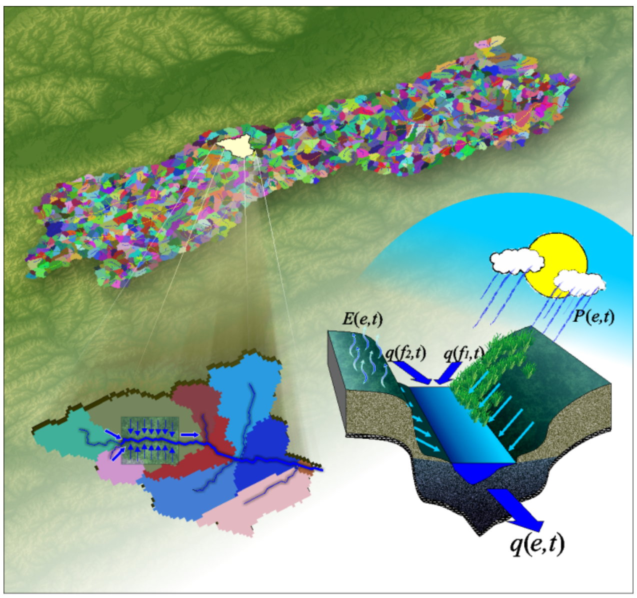

Figure 3. Schematic decomposition of landscape.

The flow transport in river networks in CUENCAS-HM is governed by a system of ODEs that uses the mass conservation equation for a link, e, (Mantilla et al., 2006) as follows:

$$\frac{dS(e,t)}{dt} = a_e R(e,t) + q(f_1,t) + q(f_2,t) - q(e,t)$$

where $S(e,t)$ is the storage in the link at time$t$, a_e is the total hillslope area it is draining into, $R(e,t)$ is the runoff intensity per unit area from the hillslope, $q(f_1,t)+q(f_2,t)$ are the flow from the two upstream tributaries joining the link e, and $q(e,t)$ is the discharge at the outlet of the link

The channel storage, $S(e,t)$, and discharge, $q(e,t)$ can be written as $S(e,t)=l_e w_e d_e(t)$ and

$$q(e,t)=v_e(t)w_e d_e(t)=v_e(t)C_A$$

where w_e is the mean width of the link, $d_e(t)$ is the mean channel depth, C_A is the link average cross sectional area, $v_e(t)$ is the flow velocity, and l_e is the link length. Combining them gives,

$$S(e,t)=\frac{q(e,t)l_e}{v_e(t)}$$

Letting $v_e(t)=v_0 q^{\lambda_1}A^{\lambda_2}$ where v_0 is the initial velocity, λ_1 and λ_2 are the scaling exponents.

The channel storage, $S(e,t)=\frac{1}{v_0}q(e,t)^{1-\lambda_1}A^{\lambda_2}l_e$ is a function of discharge; then, equation 7 becomes,

$$\frac{dq(e,t)}{dt}=K\big(q(e,t)\big)[a_h R(e,t)+q(f_1,t)+q(f_2,t)-q(e,t)]$$

where $K\big(q(e,t)\big)=\frac{v_0 q(e,t)^{\lambda_1}A^{\lambda_2}}{(1-\lambda_1)l_e}$

A simplified version of the runoff production from the hillslope is given by,

$$\frac{dS_p}{dt}=R_c p(t)-q_{pl}$$

$$\frac{dS_s}{dt}=(1-R_c)p(t)-q_{sl}$$

where $R(e,t)=q_{pl}+q_{sl}$ and $q_{pl}=\frac{v_h L_e}{a_h}S_p$ and $q_{sl}=\frac{v_h L_e}{a_h\times 290}S_s$

R_c is the runoff coefficient, $p(t)$ is the rainfall time series, q_{pl} is the surface storage, ET is the evapotranspiration, q_{sl} is the subsurface storage, v_h is the velocity of the hillslope [m/s], S_p is the storage volume from the surface [km^3], a_h is the hillslope area draining to the link [km^2] and S_s is the storage volume from the subsurface [km^3]. The link-based mass conservation equation 10 forms a system of 3M non-linear ODEs, where M is the number of links in the networks.

MERGING INFORMATION FROM INSTRUMENTS WITH MODEL

Our originally proposed idea for merging information from the instruments with the mathematical model was a process of direct comparison of the predicted flood-crest arrival-time at the locations where instruments are installed (i.e. bridges) with the prediction given by the model. The parameters λ_1 and λ_2 can be adjusted to provide an accurate representation of the movement of water in the river network. The initial values of the parameters were taken from regional studies, such as those performed on the Cedar River basin to understand the genesis of the floods of 2008 (Krajewski and Mantilla 2009). However, it became apparent after data was collected in 2013 that the information of travel times was not enough to constrain the model. In fact, we discovered that significant differences in the model configuration led to small differences in the basin response at larger scales, therefore, a change in the methodology involving the coupling of the CUENCAS-HM with a one dimensional hydraulic model that was developed by IIHR as part of the thesis work of Chi Chi Choi (Choi 2013) to provide stronger constraints to the model and to perform a fair evaluation of the forecasting abilities of the hydrological model at smaller scales.

SPECIFIC OBJECTIVES AND TASKS

We propose to design and implement a hybrid flood-forecasting system on the Squaw Creek basin in central Iowa. This system will combine state-of-the-art distributed hydrologic models with real-time stream level observations using new bridge-mounted sensors. By the end of the two-year project, the system will provide accurate predictions of flooding potential for each and every road/stream intersection in the basin. The specific activities that will be performed as part of the proposed work are as follows:

Task 1. Sensor assembly and deployment:

- Construct 25 wireless ultrasound stream-level sensors.
- Select locations for sensor installation. The selection process will include site visits with the engineers and hydrologists to verify that the sites meet all the criteria mentioned in this report.
- Deploy the 25 sensors at the selected and approved locations.
  - Develop a web-based map interface for accessing sensors real-time information (similar to the Iowa Flood Information System, http://ifis.iowafloodcenter.org/ifis/en/).
- Collect and analyze information from the sensors.

Task 2. Hydrologic model implementation and refinement:

- Refine the hydrologic model implementation for Squaw Creek to find an appropriate set of initial stream water velocity parameters
- Perform continuous adjustments to the hydrologic model as sensors collect information.
- Evaluate model performance on ungauged locations.

Task 3. Submit appropriate material for publication in engineering and scientific journals.

RESULTS

DESCRIPTION OF THE SQUAW CREEK BASIN

We selected the Squaw Creek, which is upstream from Ames, IA (42.011°N 93.596°W), for this study. Squaw Creek Watershed (SCW) is located in central Iowa where it drains about 602 km² and includes parts of Boone, Hamilton, Webster and Story Counties. It drains into the South Skunk River at Ames, Iowa and ultimately discharges into the Mississippi River in southeast Iowa. There are a total of 5,143 km streams in the basin, including ephemeral and perennial river pathways. The hydrologic model CUENCAS includes every one of those streams in its configuration. Of those streams, about 227 km form the perennial channel network, which is modeled by the 1D-SVE models (Figure 4). The latter number agrees with the previously reported network by Wendt (2007). The drainage has been transformed from slowly draining wetlands and depressions into rapidly draining ditches and farm tiles (Squaw Creek Watershed Planning Committee 2004). Schilling et al (2008) and Jha et al (2010) provide additional background on watershed characteristics, as well as information on hydrologic, water quality, and biological monitoring data. Twenty-two bridge-mounted stage sensors that are operated by the Iowa Flood Center (IFC) and a USGS streamflow gauge (#05470500) for river stage and discharge measurements (Figure 4) are located in the watershed. The watershed recently experienced severe flooding in August, 2010; a peak discharge of 634 m^3/s and peak stage of 5.53 meter was recorded at the USGS streamflow gauge (#05470500); and the return period of the flood was estimated to be between the 100 and 500-year flood interval (Barnes, 2012).

Figure 4 shows the basin boundaries and river network delineated using CUENCAS-GIS. The Squaw Creek is an order-8 network. Only the higher order streams are selected for the 1D-HD model implementation for two reasons: 1) the hydrodynamic conditions that grant the implementation of 1D-HD models, including flood plain interactions and backwater effects from downstream constrictions, typically occur in larger streams and 2) many distributed hydrological models include a routing component for the channels in the networks, and we want to demonstrate how to connect the channel network where SVE are solved with the network that uses simplified routing methods. Routing in hydrological models typically entails simplified kinematic wave equations (Whitham, 2011), ODE routing using non-linear reservoirs (Mein et al., 1974; Green, 1979), or Muskingum routing schemes (McCarthy, 1938; Nash, 1959; Ponce, 1979). The delineated river network (HD Model Streams), the river network (CUENCAS-HM Streams), and the adjacent hillslope area extracted by CUENCAS-GIS and the digital elevation model (DEMs) are the only inputs required by the tools that are presented in this paper. CUENCAS-GIS uses the classical D8 algorithm (O'Callaghan & Mark, 1984) to determine drainage pathways.

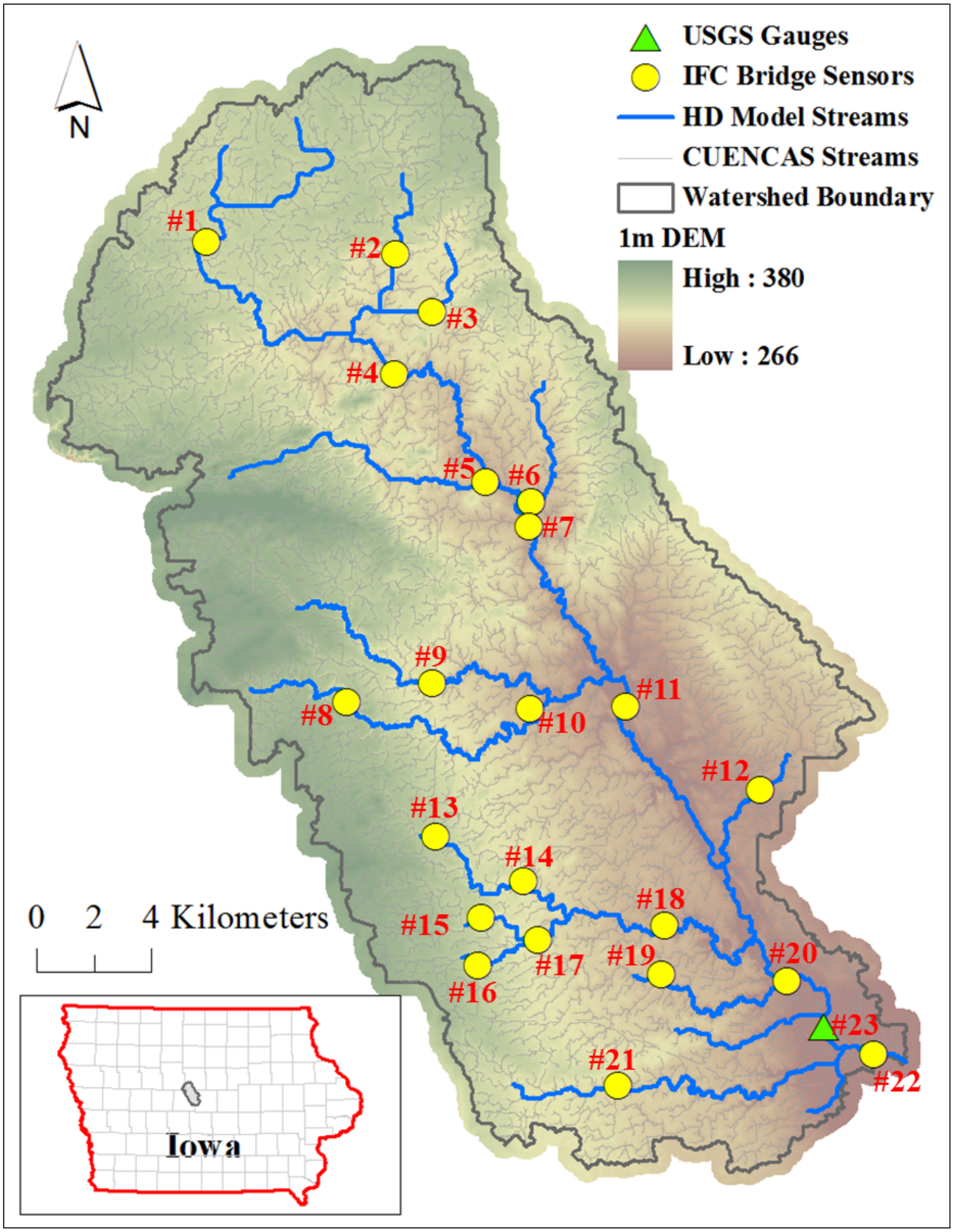

Figure 4. Squaw Creek basin upstream from Ames, Iowa.

TASK 1: DEVELOPMENT AND DEPLOYMENT OF SONIC-STREAM-LEVEL SENSORS

Sensors were installed in the spring of 2012. A total of 35 locations were selected as candidates for site installations but only 22 met the requirements for installation of the instrument. The three remaining instruments have been installed in the neighboring stream of Big Creek. This decision was made after multiple attempts to find suitable bridges within the basin where the instruments could be secured. In the following sections we give some details of the sensor specifications and performance measures and we show of the data that has been collected.

Specs & Device Modifications

Second-generation sensors use the Senix Corporation model TSPC-21S-485 ultrasonic sensor. Both sensors are potted in 303 stainless steel and are IP-68, NEMA-4X rated, and operate in 0-100% humidity over a temperature range from -40 to +70 °C. The sensor has a conical shaped beam pattern and beam width of 12 degrees.

The sensor has internal temperature sensors that are used by the sensor's internal electronics to compensate for changes in the speed of sound with temperature. We also poll and transmit the Senix sensor's internal temperature and transmit it along with the computed distance back to the database. However, the sensor's internal temperature measurement may not reflect the true ambient temperature, due to overheating heating from the sun or heat radiating from the bridge.

The sensor also has adjustable sample rates with built-in averaging algorithms. All sensors are pre-programmed at the IFC before deployment to average the distance over 15 samples taken 500 miliseconds apart. The average calculated distance computed by the Senix sensor is the value transmitted to the IFC database.

Spec	TSPC-15S-485	TSPC-21S-485
DC current @	40 mA max	40 mA max
10-30VDC input	10 in (25.4 cm)	12 in (30.5 cm)
Deadband	240 in (610 cm)	400 in (1016 cm)
Optimum Range	360 in (914 cm)	600 in (1524 cm)
Maximum Range	0.006768	0.013536
Resolution	(0.1719 mm)	(0.3438 mm)
Measurement rate	500 ms	500 ms
Performance	Observed accuracy ~1% of target distance	Observed accuracy ~1% of target distance

Deadband is the small distance near the sensor face within which distance cannot be measured. Optimal range is the range of target distances recommended for optimal performance in varying environmental conditions. The electronic components of the sensor are shown in Figure 5.

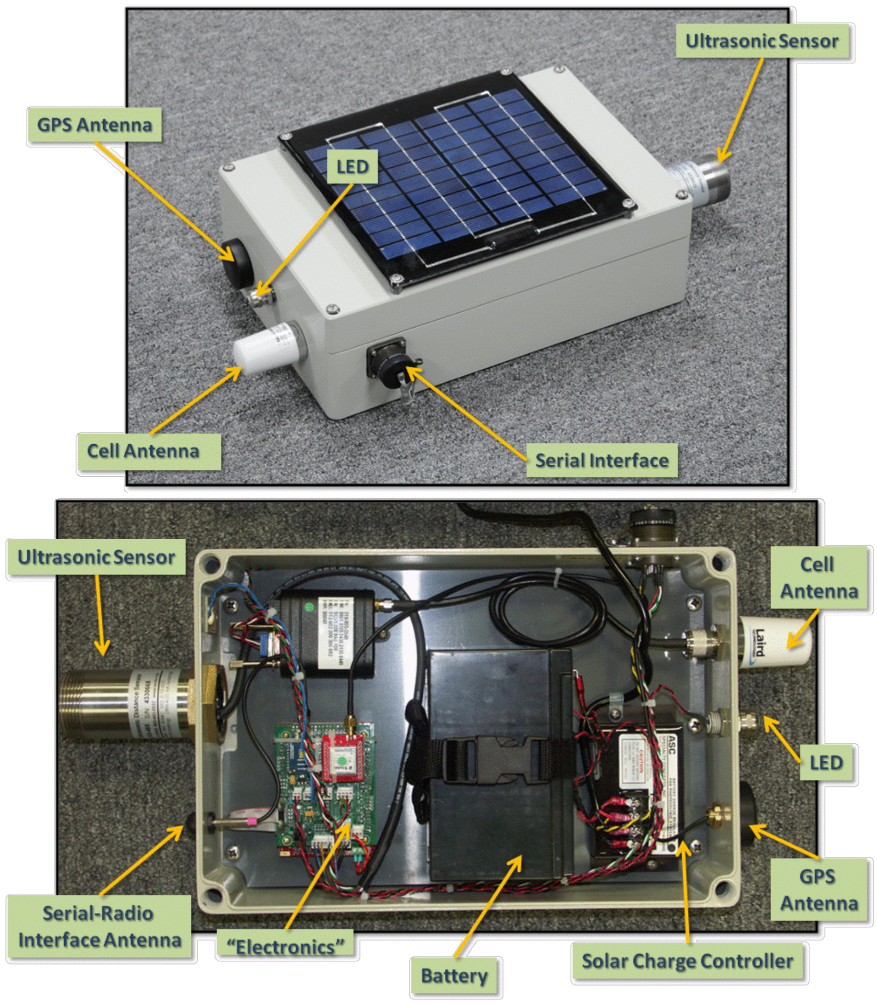

Figure 5. Sonic stream stage sensor. For more photos, visit http://www.flickr.com/photos/ifc-iihr/collections/72157625905373906/.

There have been few design changes since the sensor was designed and deployed in 2010. The changes/upgrades incorporated after the initial 50 include the installation a longer-range transducer, firmware update with additional user settings/options, the ability to choose between CDMA and GSM cell networks, an upgrade to a more efficient solar charge controller, and the removal of underutilized local RF communication capability. The subsequent 127 constructed sensors, including those installed in Squaw Creek, have not had any design changes, and the first 50 were updated where appropriate (i.e., for the locations where the typical distance to the water table was too long for the small and weaker original sensor, model TSPC-15S-485).

Since the original development of bridge sensor, several improvements have been made to the system to enhance its overall performance. These improvements addressed key elements in the sensors' operation, power consumption, data integration, and the maximum operating distance.

A few minor modifications to the original system have allowed for an overall reduction in power consumption of the complete system. This reduction of power usage allows the system battery to maintain its charge with fewer hours of sunlight each day. This is particularly important on bridges without a southern exposure or bridges with dense riparian zones that limit sunlight through the day. The new systems also allow the IFC to dynamically change the sample rate from five minutes to several hours between samples. During winter months when streams are frozen and daylight hours are shorter, the sensor can be placed remotely into an ultra-low power state to maintain battery charge.

Allowing the database to communicate with the system to insure that the stage measurements transmitted through cellular link have been received properly has increased data integration. This ensures that all measurements are received accurately and in their entirety for display on IFIS. The new systems also allow for the use of two different cellular providers that use the two different cellular technologies (CDMA and GSM/GPRS). This allows for installation in locations where only one form is supported. The first generation sensors used only GSM/GPRS technology, which is not supported statewide, or in some areas allows only limited use of the cellular network.

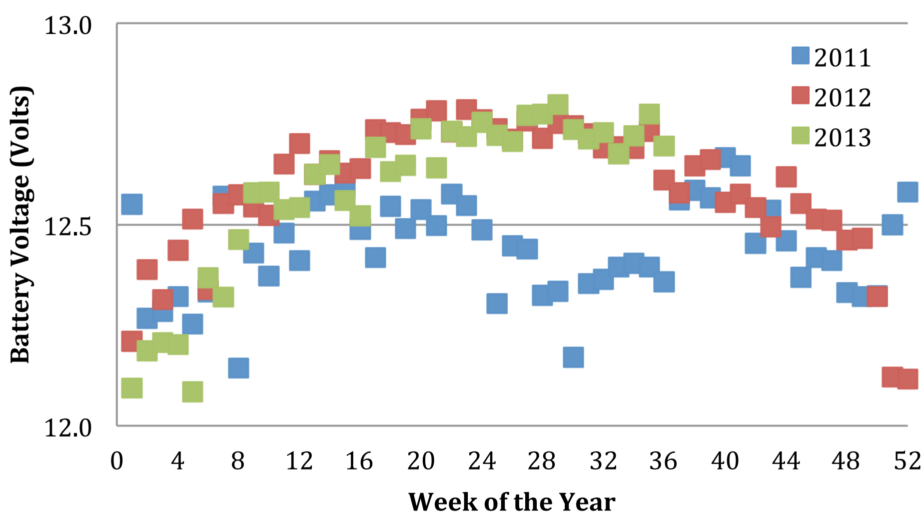

Figure 6. Sonic Sensor's Battery charge as a function of time of the year.

The first-generation sensor, while adequate for most installations, lacked the capability to measure bridges that were taller than 20 feet. This meant that only peak flows could be captured when the water was within 20 feet of the bridge deck. Later generations now use an ultrasonic sensor with a maximum range of 40 feet. This has allowed for installation on all but the tallest bridges in the state.

Performance Evaluation

The IFC bridge sensors are autonomous devices, meaning that they are equipped with their own power sources and relay data automatically. There are three major aspects of the sensors' operation: (1) providing adequate power; (2) ensuring frequent and reliable communication; and (3) adequate accuracy of the collected data. Power is consumed by the ultrasonic sensor and cell phone data transmit. A 12V battery that is recharged by a solar panel with a custom-built charge regulator provides the power. During the first winter in 2010/2011, we identified some of the recharging problems during cloudy winter days when exposure to sunlight is limited. We improved the system by redesigning the charge regulator, tilting the solar panel for better exposure to the sun, and adding larger panels in few locations. This seems to have fixed the problems. We demonstrate this in Figure 6. It shows the network-averaged minimum weekly battery voltage. It is clear that it is well over the nominal value of the required 12V.

Regarding communication, the sensor on-board computer is programmed to wake up the cell phone model to call the IFC server for sending of the collected data. While the overall logistics of this function are pretty complicated, we have figured out the nuances of the cell phone data plans, the priority structure of the individual communications, and the optimal structure of the data packets. For example, if the data packets are too short, the transmissions get low priority, but when they are too long, the total cost per month might exceed the contract plan. Our efforts ensured that we lose very few packets. The calls are made every 15 minutes, which is a compromise between frequent sampling and the sensor's energy budget. In fact, as an experiment, during 2012, we operated with data sampling every five minutes (Figure 7).

Not all locations have good cellular communication. There was a period when a sensor or groups of sensors were unable to transmit due to local cell tower issues. Service is typically only interrupted for a couple of hours, or in extreme cases, a couple of days. There have been a few installations where service was never restored. In those few instances, the sensor was swapped out with a sensor with a different cellular service provider (typically from AT&T to Verizon). The compact all-in-one design allows for quick removal and replacement of a bridge sensor, with typical replacement taking about one hour.

The communication performance of the system is illustrated in the plots below. They show the number of transmissions delayed more than the given period, averaged over a week. The number is generally low and decreasing as we go from 2011 to 2012 (changes of the Internet Service Provider and modem type).

The last issue is the data quality. There were two instances when the ultrasonic sensor would not provide consistent repeatable measurements due to a supplier-manufacturing defect.

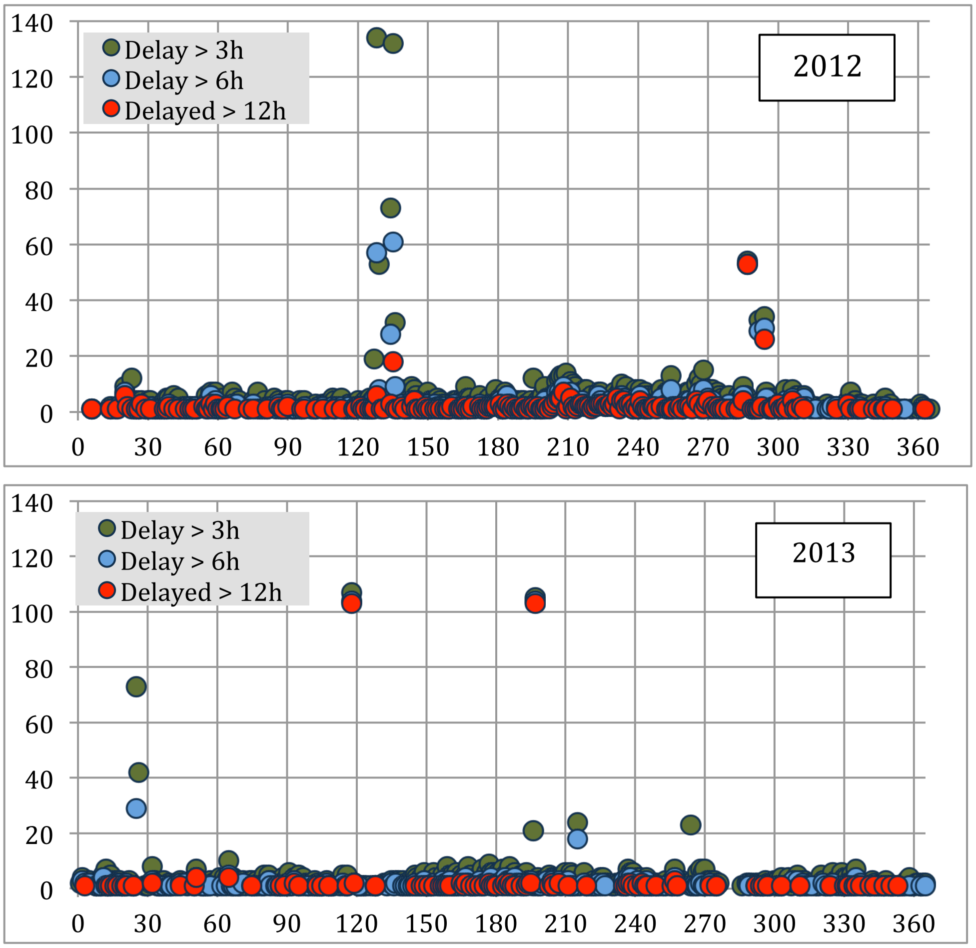

Figure 7. Sonic Sensor's transmission delay in 2012 and 1013.

The majority of the sensors are located on small streams with little or no flow during dry periods. Some of these streams have vegetation that grows in the streambed during dry periods, which can skew stage measurements. This, coupled with the fact that most installations are not on the mainstream channel (pillars and deck drains limit installation locations), makes it difficult in some circumstances for the sensor to measure extremely low water events.

There were a couple of sensors that indicated moisture in the box due to sealing issues. All sensors are now submerged for several days and checked for leaks before deployment and all sensors have an electronic moisture indicator inside the enclosure that warns of water infiltration.

Last spring we experienced a new test for the sensors: submergence by floodwaters. In at least two well documented cases this past spring, the bridge sensors ceased sending data. Our investigation revealed that flash flooding submerged the sensors. When the floodwaters subsided, the sensors resumed sending correct data on their own, without any need for repairs. This is a testimony to the rigorous procedure of testing the sensor enclosure prior to their field deployment.

A sample of data collected by the sonic sensors

Stage data collected by the stream sensors in the Squaw Creek confirms the reliability of stream sensors presented in the previous sections. Figure 8 show plots of stage measurements made between July 1st 2012 and September 30th 2013 at the 22 sites. In the section that follows we will illustrate how this data was used in conjunction with the hydrological model to improve flood-forecasting capabilities in small streams. Labels in Figure 8 correspond to those shown in Figure 4.

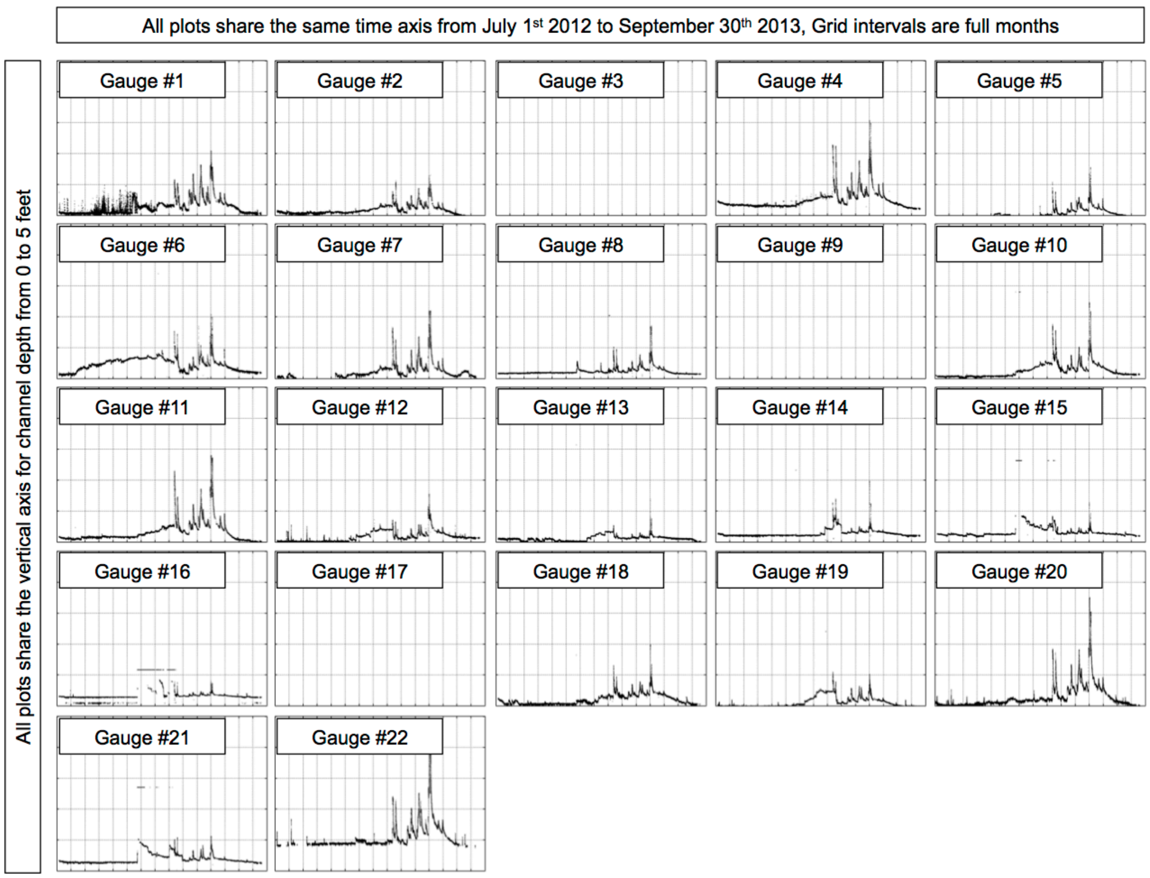

Figure 8. Data collected at 22 sites in Squaw Creek.

TASK 2: HYDROLOGIC MODEL IMPLEMENTATION AND REFINEMENT

The original plan for the hydrological model implementation was to use the information on flood arrival time collected at the 25 newly gauged sites to constrain the values in the velocity function in CUENCAS. This task was delayed in 2012 by an unusually dry year where streamflow at the outlet of Squaw Creek stayed close to zero almost the entire year and only rose to discharges in the order of 400 cfs, ten times lower to the mean annual flood for this site = 4100 cfs (Figure 9). The mean annual flood is a good surrogate for the bankfull discharge, which means that the very small fluctuations that occurred in 2012 were well contained within the channels.

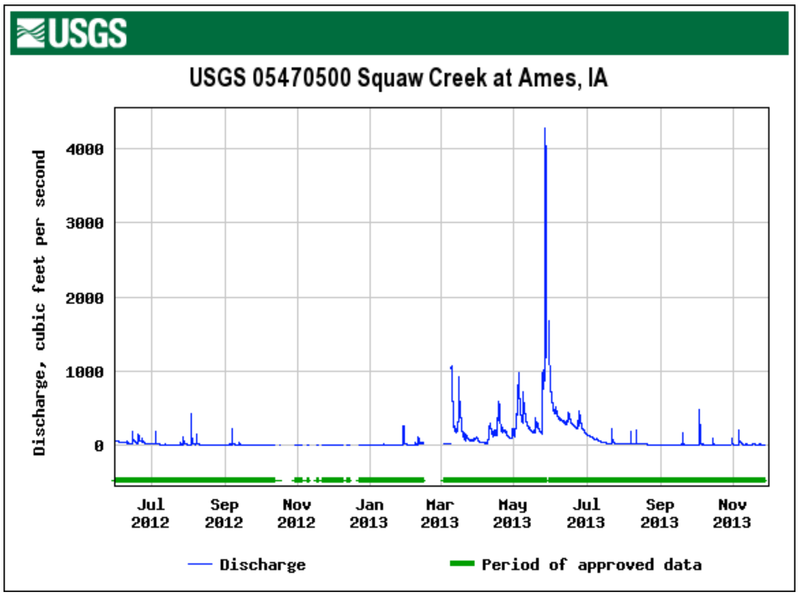

Figure 9. Streamflow fluctuation at the outlet of Squaw Creek in 2012 and 2013, the period of record of newly installed sonic sensor in the basin.

In contrast, the spring of 2013 brought heavy precipitation to central and Eastern Iowa and one significant event was recorded in the basin. All the newly installed instruments in the basin recorded the event and collected valuable information across multiple scales. In a wet year, flows raise regularly above 1000 cfs. In 2010, for example flows were above 1000 cfs over 9 times in the year, and needless to remind that one of them included the record flood of August 12th. The small sample of events recorded in 2012 and 2013 lead us to rethink our strategy for using the information from the sonic stream sensors. A decision was made in the fall of 2013 to couple our hydrological model (that provides estimates for streamflow at all locations in the river network) with a one-dimensional hydraulic model recently developed in IIHR. The model is equivalent to HEC-RAS in the sense that it solves the Saint-Venant Equations but with the added advantage that it can be implemented in dendritic river networks and that flows can enter the channels laterally in between cross sections in a flexible and automatic manner.

The goal of the following sections is to document the development of a physical-based 1D-SVE PDEs solver that can be applied to calculate flood routing in river networks and that can be coupled with the ODE-based hydrological model introduced by Mantilla and Gupta (2005). The CUENCAS framework is divided into two sets of tools, and we will refer to them as follows in this document by using: 1) CUENCAS-GIS when we refer to the set of tools that extracts and analyzes the river network morphology and 2) CUENCAS-HM when we refer to the set of tools for modeling flow in river networks. First, we describe the 1D-SVE model and the corresponding set of GIS-based geo-processing tools that can simulate 1D unsteady flow through a dendritic river network. The 1D-SVE code has been implemented as a set of Matlab™ libraries, which provides portability and ease of use for applications ranging from classroom exercises to complex engineering applications. We subsequently give details of the ODE setup of the CUENCAS-HM hydrological model. The coupled hydrologic and hydraulic (H-H) model is then implemented and tested for a realistic rainfall event in the Squaw Creek upstream from Ames, IA. The runoff field generated by CUENCAS-HM is used as inflow (discharge) to the 1D-SVE model. The coupled H-H model takes advantage of both stage and discharge data for model validation.

COUPLED H-H MODEL'S DESCRIPTION AND DATA PREPARATION

The Hydraulic Model – 1D Saint-Venant Equation (SVE) Solver

The governing equations for the one-dimensional, unsteady, open-channel flow, known as one-dimensional Saint-Venant equations (1D-SVE), can be written as the continuity equation,

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_{lat} = 0$$

and the momentum equation,

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(\frac{\beta Q^2}{A}\right)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f\right) = 0$$

where β = momentum correction factor, Q = discharge [m^3/s], A = flow area, g = gravitational acceleration [m/s^2], q_{lat} = net lateral inflow per unit length of channel [m^2/s], h = elevation of water surface measured from a horizontal datum [m], S_f = frictional slope, t = time [s], and x = distance measured along stream centerline [m].

In the 1D-SVE code developed as part of the present work, the standard four-point weighted Preissmann scheme (1961) is used to solve the dynamic wave form of the 1D-SVE. The channel/floodplain interaction of the hydraulic routing was embedded in the modified 1D-SVE (Fread et al., 1976). There are two major assumptions associated with this approach. First, the water surface elevation is assumed to be the same across the channel and the floodplain. Second, the friction slopes in the channel and the floodplain are assumed to be equal. The reach lengths of the channel and the floodplain can be different, but they are assumed to be equal in the present study because of the following reasons: 1) The delineation of the floodplain flow path lines is humanly subjective; 2) The river cross sections within the in-channel and their dog-leg alignment over the floodplain are not easy to be automated, and our geo-processing (GIS) tools (see full description in 'Model Setup and Model Parameters Selection' section) does not include this capability; 3) The channel-floodplain interaction are complex and multi-dimensional.

The modified forms of the 1D-SVE (see Eqs. (1) and (2)) that include the channel/floodplain interaction (Fread et al., 1976 and 78) are given as the continuity equation for the channel,

$$\frac{\partial A_c}{\partial t}+\frac{\partial Q_c}{\partial x_c}=0$$

the momentum equation for the channel,

$$\frac{\partial Q_c}{\partial t}+\frac{\partial\left(\frac{{Q_c}^2}{A_c}\right)}{\partial x_c}+gA_c\left(\frac{\partial h_c}{\partial x_c}+S_{fc}\right)=0$$

where x = displacement in the main flow direction [m]. The continuity equation for the floodplain,

$$\frac{\partial A_f}{\partial t}+\frac{\partial Q_f}{\partial x'_f}=0 \quad \frac{\partial Q_f}{\partial t}+\frac{\partial\left(\frac{{Q_f}^2}{A_f}\right)}{\partial x'_f}+gA_f\left(\frac{\partial h_f}{\partial x'_f}+S_{ff}\right)=0$$

where x' = displacement in the floodplain direction [m]

The subscript, "*c,*" denotes the variables pertaining to the river channel and the subscript, "*f,*" denotes the variables pertaining to the floodplain. For full details of the numerical algorithms, readers are referred to Fread et al., (1976).

Two internal boundaries conditions are imposed to solve flow in confluences. The first is continuity at the junction node. The second is the stage at all nodes coming into and exiting the junction are the same. Since the 1D-SVE is written in Matlab, we use the Matlab built-in function (mldivide, \) to solve the systems of linear equation Ax=C, where A is stored as sparse matrix format. The discretized form of 1D-SVE and the boundary conditions are used to build the system of linear equations. The Newton Raphson method is used to solve the full systems of equations of the 1D-SVE (AX=C, where X is the vector column $(\Delta Q_i, \Delta h_i)$for the 2N unknowns, Q_i^{n+1} and h_i^{n+1} for i=1, 2…, N, where N is the number of computation nodes). First, a set of initial values are assigned to the unknowns $Q_i^{n+1}$and h_i^{n+1} and the iteration (k) is equal to 1. The coefficient of the matrix on the left hand side (A) and the residual column (C) on the right hand side is filled with the calculated values based on the initial values of the unknowns. The corrections $(\Delta Q_i, \Delta h_i)$ obtained from column (x) are the solution of the system of linear equations (Ax=C). The new values of the unknowns are calculated as $Q_i^{n+1}$and $h_i^{n+1}$for the next iteration (k+1) are expressed as follows:

$$(Q_i)_{k+1}=(Q_i)_k+(\Delta Q_i)_k$$

$$(h_i)_{k+1}=(h_i)_k+(\Delta h_i)_k$$

The iterative procedure terminates until the maximum value of corrections $\Delta Q_i, \Delta h_i$ column is reduced to the assigned threshold values.

Coupling the Hydrologic and Hydraulic Models

Ideally, hydraulic models use the measured discharge hydrograph at the basin outlet as a boundary condition, which limits the uncertainty of boundary conditions to stage-discharge relationships. However, the availability of measured data is sparse in ungauged basins. This restricts the use of the hydraulic model for a complex river network. To compensate for the data scarcity, the simulated discharge hydrographs from the hydrological model are used as the inflow boundary conditions of the hydraulic model. In this study, the hydrological model, CUENCAS-HM, which is primarily written in Java, simulates a realistic rainfall-runoff event. The runoff generated from CUENCAS-HM is used as the inflow (discharge) of the cross-section based 1D-SVE solver. The flow transport in river networks in CUENCAS-HM is governed by a system of ODEs that uses the mass conservation equation for a link, *e*, (Mantilla et al., 2006). Since the spatial distribution of the river networks and the storage-discharge relationship of the ODEs systems used in CUENCAS-HM differ from the PDEs systems used in the 1D-SVE solver, geo-processing tools (Choi et al., 2014; Choi, 2013) are used to convert the tributary inflows from the ODEs systems into inflows for the 1D-SVE solver.

MODEL IMPLEMENTATION DETAILS FOR THE SQUAW CREEK BASIN

The first step in coupling the hydrologic and hydraulic models is to identify how the river networks in the distributed hydrologic model relate to the network modeled by the hydraulic model. A short river segment within the watershed is used to illustrate the steps necessary to assess the linkage. A requirement for linking the model is that the channel centerlines in both models coincide relatively well. This condition can be easily achieved by burning the channel centerlines from the hydraulic model into the digital elevation model (5-meter DEM) used for river network generation. As a result, the hillslope runoff generated from CUENCAS-HM can be matched as inflows for the cross-section based hydraulic model via automatic geo-processing tools. The process of linking the models is shown in Figure 10.

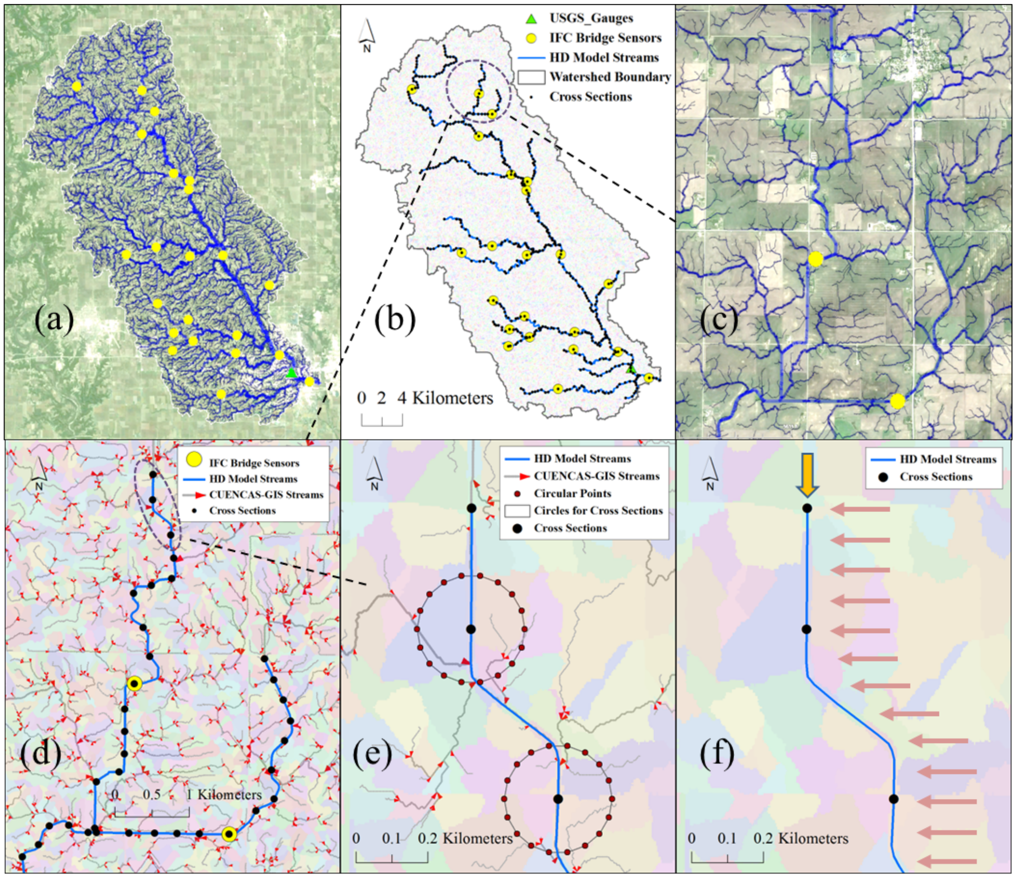

Figure 10. Schematic diagram showing the step-by-step (a to f) procedures of converting inflow from CUENCAS-HM to the 1D-SVE solver.

The basin decomposition made by CUENCAS-GIS yields 34,925 links and the corresponding adjacent hillslopes (LINK-ID), as depicted in Figure 10a, Figure 10b and Figure 10c. The automatically extracted river cross-section locations (points) are overlaid on the landscape partition given by CUENCAS-GIS (Figure 10d) to determine which hillslopes and what river channels drain directly into the channel segments that are defined in the hydraulic model.

In order to correlate the river cross-section locations and the LINK-ID in both models, we use the extracted values from the LINK-ID raster data generated by CUENCAS-GIS that lay within the 300-meter radius circle that surrounds the cross-section points used by the hydraulic model. This is done for all of the cross-section points except for the first upstream cross section (Figure 10e). This buffering method is necessary because the channel centerlines in both model setups do not match perfectly. The river networks used in the hydraulic model are humanly digitized based on LiDAR-derived DEM (1m), while the river network and its adjacent hillslope unit (equivalent to sub-catchment unit in other models) used in CUENCAS are generated based on calculated flow direction grids through the D8 algorithms derived from the resampled DEM (5m). The D8 algorithm is not completely accurate, especially near hydraulic structures such as roadway embankments or culverts, which cannot be well represented by LiDAR-derived DEM. Therefore, the LINK-IDs identified within a larger buffered domain can be traced to compensate for the small mismatch of the channel centerlines in both models. The extracted LINK-ID is then followed downstream to the following cross section, ensuring that the extracted values of the LINK-IDs for all the river cross sections are properly connected. Once the correct LINK-ID of all the cross sections has been identified, the inflows used for the 1D-HD models can be calculated. The inflow at the first river cross section is the discharge hydrograph corresponding to the top LINK-ID, while the lateral inflows along the river segments are equal to the sum of the tributary inflows over the length interval between two consecutive cross sections (Figure 10f). By implementing this approach to all of the streams in the 1D-HD models, the runoff and storage release generated by the distributed hydrologic model (CUENCAS-HM) can be automatically coupled with the cross section based 1D-HD models.

MODEL SETUP AND MODEL PARAMETERS SELECTION

Two significant flood events that occurred on August 2010 and May 2013 in Squaw Creek basin upstream from Ames, IA were selected to validate the accuracy of the coupled H-H models. Since river stage data from IFC bridge-mounted sonic sensors for the 2010 event were not available, we used the 2013 event for model validation and argue that if the coupled H-H model can perform well for multiple validation sites in the 2013 event, then we can expect that it will perform equally well for the 2010 event using the same model parameters and model setup. Measurements of discharge at the USGS station (Ames, #05404220) and measurements of stage at 22 IFC bridge-mounted sonic sensors were obtained to validate our modeling results.

Four data inputs are required to set up the hydraulic model: 1) A set of centerlines with river labeling that will be modeled using the 1D-SVE, 2) a 1 meter LiDAR-based DEM, 3) a map of land-cover to estimate roughness in the flood plain, and 4) the runoff and streamflow space-time field generated by the hydrologic model to be used as the inflow boundary condition. The Iowa Department of Natural Resources (DNR) collected the LiDAR-derived 1-meter DEM topography used in this study. Floodplain roughness coefficients are estimated from the 30-meter resolution land cover dataset (NLCD 2001). A set of geo-processing (GIS) tools are used to automate the cross section generation, river geometry extraction and overbanks locations identification and calculate the required inflows for our coupled H-H model from files generated by the CUENCAS-HM hydrologic model. The cross sections selected follow some standard guidelines: 1) they do not overlap, 2) they intersect with the stream once, 3) they are not placed in river meandering zones, and 4) they do not overlap at junctions. The details of the GIS tools are fully described in a companion paper, which is currently under revision. Our GIS tools generated a total of 486 cross-sections along thirty-three individual river reaches (a river reach refers to the river channel in between two river junctions in the hydraulic model river network). The cross-section spacing is selected within a reasonable range (30 to 500-meter) so that a relatively smooth river bathymetric profile can be obtained. The cross-section width is fixed at 400 meters to include the floodplain topography. The cross-sections are selected to satisfy the following criteria: (1) non-meandering region, (2) at least 40-meter distance to avoid overlapping cross-sections at channel junctions, and (3) at least one cross-section for all the bridge crossings within the basin. The in-channel Manning's coefficients are assigned with values in the range of 0.03 to 0.05 (typically clean, straight channel, Chow, 1959). In this case study, we use a constant value of 0.045 for every stream. The roughness coefficients for the left and right flood plains are the mean value extracted from roughness coefficient grids derived from NLCD 2001 (Manning's n value: 0.02 to 0.15). Figure 11 shows the final result of the selection of cross section along the main tributaries of Squaw Creek.

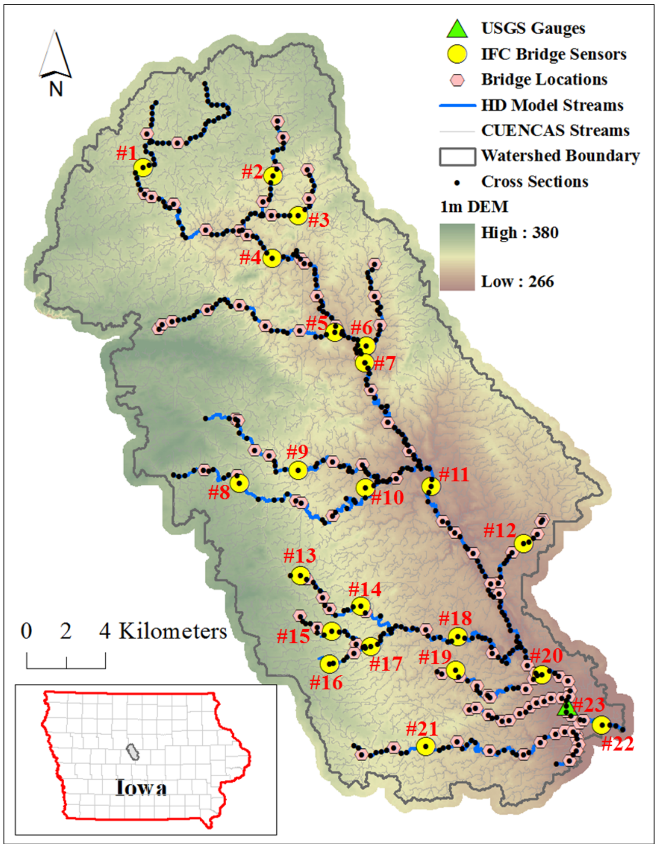

Figure 11. Simulated river networks on the Squaw Creek basin upstream from Ames, IA; the branch numbers (red) and the location of USGS gauges and IFC stations are shown.

Corrections to LiDAR-derived Streambed Profiles and Cross-sections

We investigated the difference in elevation for the channel-inverts obtained from the LiDAR-derived DEM and surveyed cross sections in the model domain (Kyutae Lee, IFC, personal communication, Aug, 2013) for 17 out of 22 measured sites ranges from 0.1 to 3.3 meters (Figure 12). Our results indicate that the LiDAR-derived longitudinal streambed profiles tend to underestimate the depth of the actual streambed profile's bottom. This is expected because laser beams used to produce LiDAR maps cannot penetrate standing water. Therefore, an artificial channelization method is used here to modify the cross-section geometry in order to better approximate reality. This process is accomplished by creating a 20-meter stream buffer polygon. If the channel bathymetries of the selected streams are within the polygons, it will be *deepened* by three artificial values (0.5, 1.0, and 2.0 meters). A cross section example is illustrated here to show the change in the channel geometries before and after the artificial channelization (Figure 12).

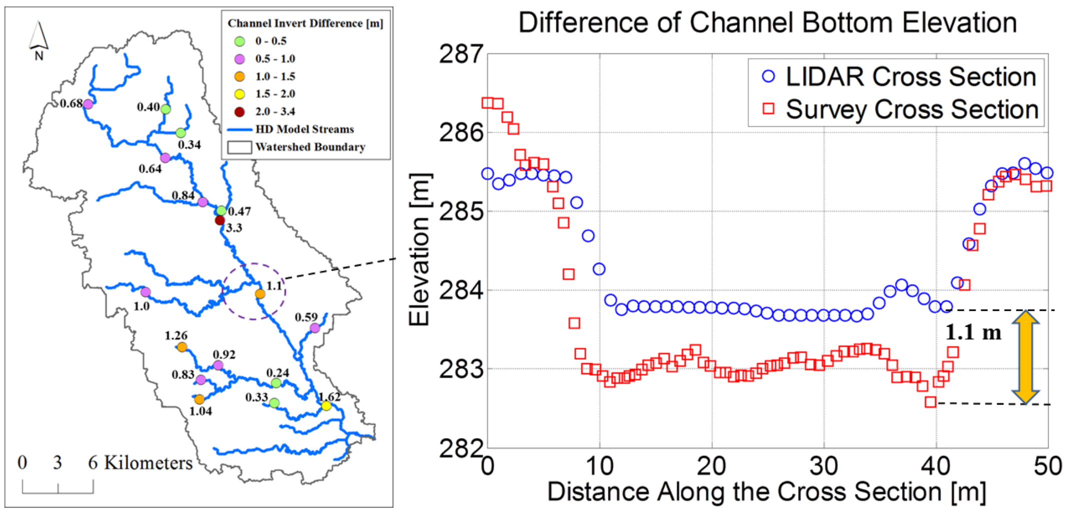

Figure 12. Comparison of LiDAR extracted channel bed elevations and surveyed river cross sections for 17 sites in the Squaw Creek basin upstream from Ames, IA.

Preparing Outputs of the Hydrologic Model as Inputs to the Hydraulic Model

Rainfall products from the Hydro-NEXRAD-2 system are used to generate near real-time rainfall maps for Iowa using data from seven radars covering the state (Krajewski et al., 2013) that are used as the inputs for the hydrologic simulation. The selections of the model parameters from CUENCAS-HM are critical in producing accurate inflows for the hydraulic model. Five parameter values are imposed in the CUENCAS-HM simulation: three parameters related to flow routing equations are $v_0 = 0.3$ m/s, $\lambda_1 = 0.2$, and $\lambda_2 = -0.1$, and two parameters related to the runoff production from hillslopes are the runoff coefficient RC and hillslope velocity $v_h = 0.03$ m/s (see Eqs. (9) and (11). The velocity values have been found to appropriately describe flows in Iowa in other studies (Cunha et al., 2012; Ayalew et al., 2014). The parameter RC is spatially uniform, but it is allowed to vary in time. During the May 2013 flood event, RC takes the values [0.1, 0.0, 0.1, 0.6, 0.0, 0.25, and 0.05] during seven corresponding time intervals given by the times [5/24 18:45; 5/25 12:00; 5/26 6:00; 5/26 21:00; 5/27 16:00; 5/28 12:00; 5/30 08:00, and 6/4 22:45]. During the August 2010 event, RC takes values [0.5, 0.7, and 0.7] during three time intervals given by the times [8/8 6:25; 8/9 12:00; 8/10 10:00, and 8/17 2:40]. The selections of the temporally varying runoff coefficients are chosen to approximate the hydrographs. Because we do not want to make this paper a calibration exercise of the hydrological model, we have chosen the simplest model configuration allowed by CUENCAS-HM to ensure that the total flow hydrographs entering into the hydraulic model are a realistic representation. The CUENCAS-HM simulation provides hydrographs for every link in the CUENCAS-GIS river network. This not only allows us to provide hydrographs to the hydraulic model as boundary conditions, but it also provides the hydrograph at the outlet of the basin calculated using hydrologic routing alone. The resulting hydrograph at the outlet, calculated by CUENCAS-HM, allows us to examine the effect of the simplified hydrologic routing scheme.

Simulation Results

Hydrographs simulated by the coupled H-H models and by CUENCAS-HM are compared with observed stage and streamflow hydrographs provided by IFC and USGS sensors for the flood event that occurred on May 2013. The model performance from two models is evaluated based on three statistical measures used to compare the simulated and observed hydrographs: the root mean square error (RMSE), the correlation coefficient (R), and the Nash-Sutcliffe coefficient (NS) (Table 2). We use the square root of discharge predicted from CUENCAS as a surrogate of stage to compare model simulation results to stage measurements and direct discharge values at gauge #23 where streamflow measurement are available.

An initial inspection of the streamflow hydrographs at the basin outlet indicates that the timing of the flood peaks, the peak flow, and the shape of the simulated hydrographs are properly captured in the two models (Figure 13). In the case of the CUENCAS-HM simulated hydrographs, the only opportunity for comparison with data is at locations where streamflow estimates exist. The comparison of the simulated hydrographs between the coupled H-H model and CUENCAS-HM indicates that utilizing the coupled H-H models offers a better estimate than using the CUENCAS-HM alone. The improvement can be attributed to the difference in the flow routing mechanisms in both models. Physical-based equations (1D-SVE) used in the coupled H-H model can reproduce more complex flow dynamic conditions, such as flood plain interactions and backwater effects, which are more likely to occur in the main stem of the river network. At smaller scales, the uncertainties associated with the parameterization of the hydraulic routing model (i.e., cross sections and lateral roughness) are significant enough to be comparable in accuracy to the purely hydraulic routing method.

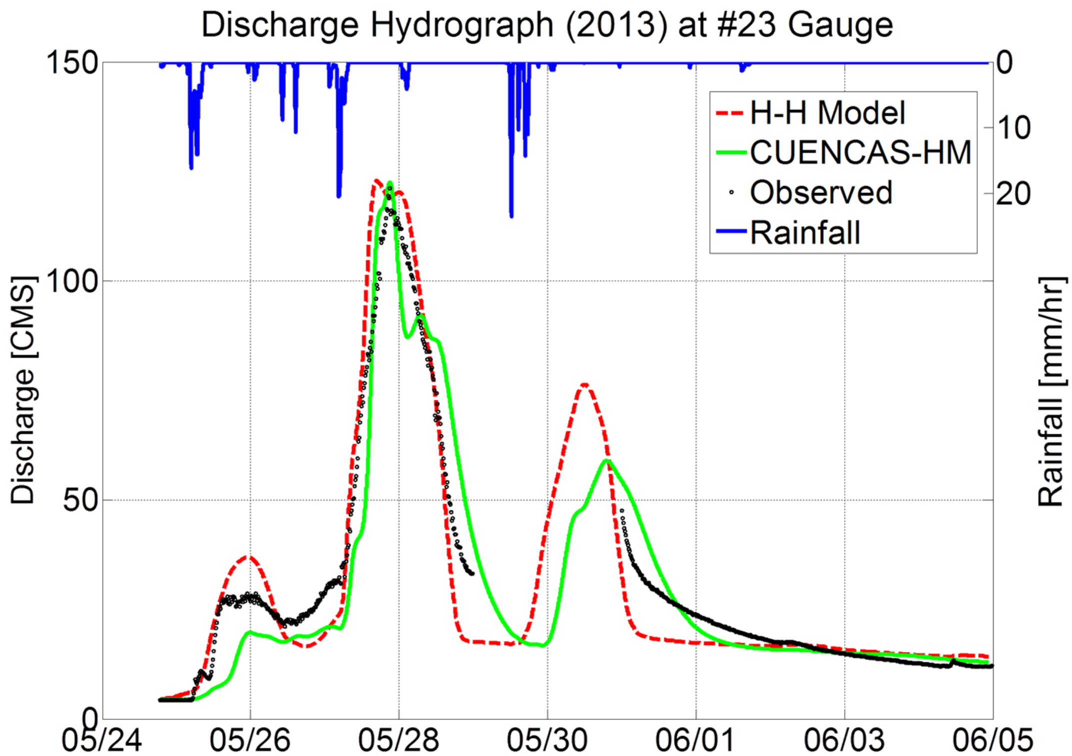

Figure 13. Simulated and observed discharge hydrographs at control point #23 in the Squaw Creek basin upstream from Ames, IA for a flood event that occurred in May, 2013.

Direct comparisons of model performance with respect to data at interior basin locations can only be made for the coupled H-H model. In general, the H-H model matches well the stage hydrographs of the main stem of the river and the tributaries, but there is better performance along the main stem of a river network (locations #7, #11, and #23) than on the tributaries (locations #2, #5, and #21), as shown in Figure 14. The lower accuracy for simulation in small tributaries that drain small sub-basins can be partly attributed to the fact that there is more uncertainty in the computed runoff field provided by the hydrological model. At those scales, hydrological models are more susceptible to errors in the radar derived rainfall field (Mandapaka et al., 2010). In addition, the measurements used to benchmark the model are more prone to error in small tributaries because the range of fluctuation is a lot smaller (i.e., 1 meter versus 4 meters), which also imposes uncertainty in the measured data.

Also in Figure 14 we have plotted (using an inverted axis) the square root of the discharge estimated by CUENCAS-HM. Fluctuations of the square root of discharge are surrogates for stage because the exponent of rating curves is close to 0.5 (Fenton, 2001). These plots allow for indirect comparisons of model performance. In particular, it can be seen in Figure 14 that the timing of peaks predicted by both the coupled H-H models and CUENCAS-HM for the tributaries are close, but the same prediction for the main stem favor the coupled H-H model's prediction (Figure 13). By comparing the correlation coefficients calculated from the two models, the coupled H-H model perform better than

CUENCAS at multiple sites with a higher average correlation coefficient value of 0.81 for the coupled H-H model and 0.64 for CUENCAS (Table 2). Given that the stage-discharge relationships at various locations may not be well represented by the one to one exponential function ($\sqrt{Q} = h$), this indirect comparison can bias our interpretation but a visual inspection of the results corroborates our conclusion that hydraulic routing based on the 1D-SVE is a more appropriate alternative for flow routing in complex flow situations. However, we recognize that more testing needs to be done to make a definitive conclusion.

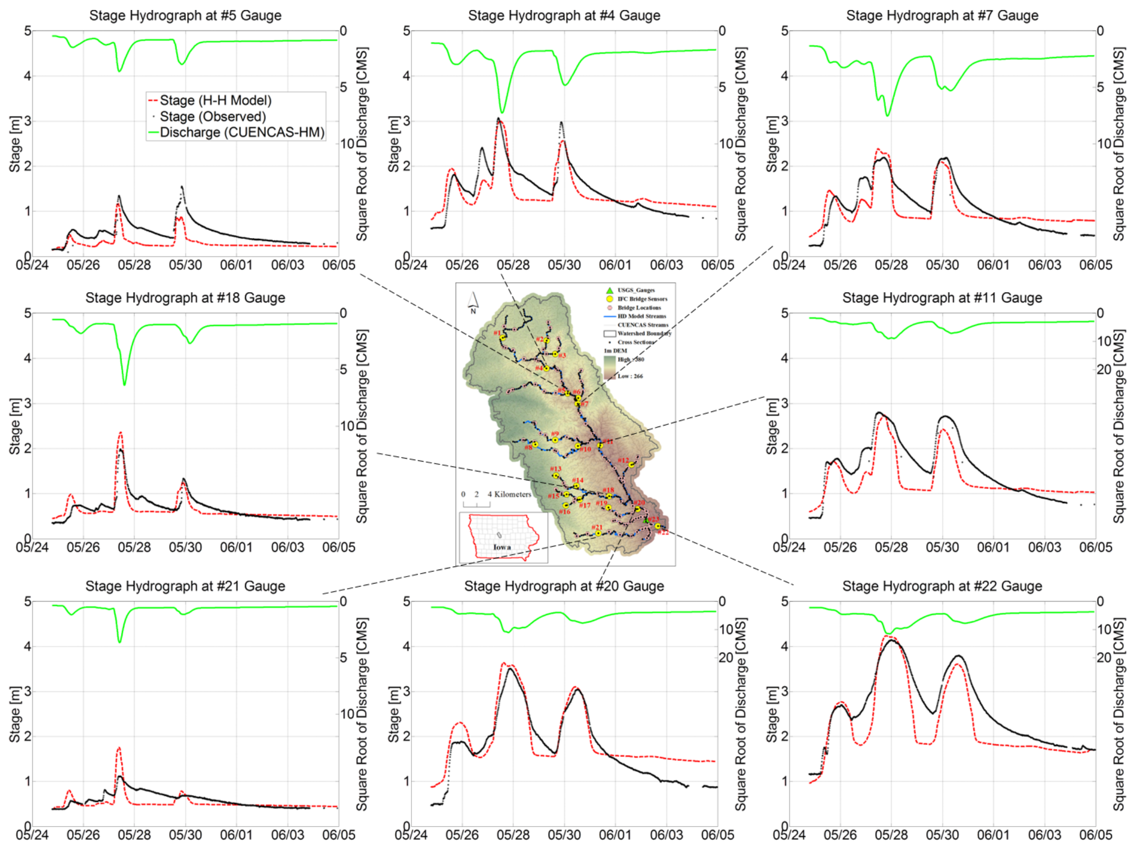

Figure 14. Simulated (red line) and observed (black dot) stage hydrographs at 8 control points in the Squaw Creek basin upstream from Ames, IA.

We also recognized that both, the coupled H-H model and CUENCAS-HM, fail to reproduce the secondary exponential recession ($Q = Q_0e^{-kt}$) during the falling limb of the simulated flood events. We attribute this to physical processes such as tile drainage that are not being modeled by the hydrological model. The total volumes of the predicted discharge are slightly less than the observed discharge hydrograph at the outlet (Figure 13) during the recession period. One of the possible reasons is that the tile drainage has not been considered in this study, which possibly constitutes the missed outflow volume. The tile drainage (perforated tubes underground) enhances the movement of excess water in the subsurface and lowers the water table for crop production. Since the majority of the land use within the Squaw Creek Watershed is cropland (80% in 2005, Osmond et al., 2012) and it is estimated that 25-35% of all cropland is artificially drained (Schilling et al., 2008), the tile drainage reduces the storage capability of soil and therefore increases the amount of subsurface flow entering the channel after the storm event has ended, which results in an underestimation of discharge hydrographs during the recession period.

The main advantage of using the physical-based coupled H-H model is that it can provide both stage and discharge prediction (i.e., unsteady rating curve) at any intermediate site for a watershed with multiple discharge and stage gauging locations. Since the selection of these site locations are automated in our models, one can insert additional points of interest (e.g., bridge locations in our models) into the model's configuration. Another application is the development of model-based rating curves (Di Baldassarre et al., 2009; Lang et al., 2010; Neppel et al., 2010; Di Baldassarre et al., 2010) that can be used as an alternative to empirically based rating curves or to extend those beyond the range of existent measurements. Although the uncertainties of the hydraulically derived rating curve are complex and site-specific, the proposed framework of the coupled H-H models is capable of providing a rating curve at multiple locations within a certain confidence interval. An example is shown in Figure 15.

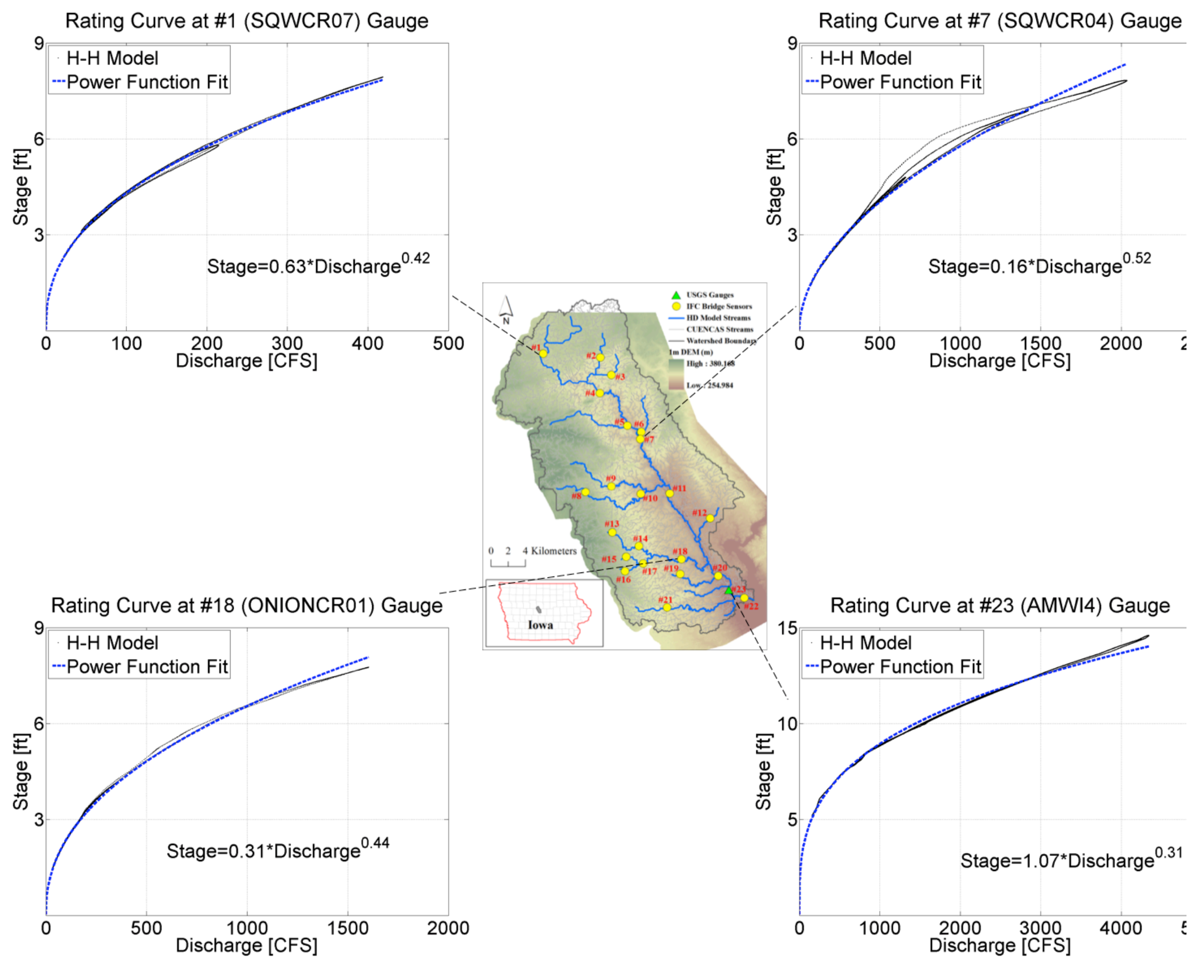

Figure 15. Rating Curves generated using the unsteady 1D-SVE at four select locations in Squaw Creek.

CONCLUSIONS AND OUTCOMES

The original idea that was proposed to The Iowa Highway Research Board was that data collected by the sensors over multiple small and large flood events could be used to validate the assumptions made by the hydrological model regarding the space-time distribution of flow velocities in the basin. In particular, it was hypothesized that peak-flow time of arrival was a measure that should be accurately predicted by the hydrological model if the velocity function assumed was correct. There were two difficulties that precluded a complete test of the hypothesis. First, the small number of flood events that was recorded during the duration of the project, 2012 was unusually dry year in Iowa and therefore no significant flood events were recorded. The spring of 2013 brought stronger storms and at least one significant flood event to the basin, but a dry summer and fall seasons followed it. Second, the precision in prediction of timing of peak flow arrivals was not accurate enough to produce reliable conclusions after simulating one single major event.

A decision was made in the fall of 2012 to look for an alternative form of model validation that relied on the information that was collected across the basin during the largest flood event in Spring 2012. To this end, we developed tools to couple CUENCAS to a one-dimensional hydraulic model (similar to HEC-RAS) to translate discharges into stages that could be compared to the measurements made by the sonic-sensors. Several GIS tools needed to be developed to accomplish this goal but the resulting coupling provided an unequivocal signal that the good performance of CUENCAS at the basin outlet coincided with the accuracy of the model at internal locations as small as 1 square miles.

Our study allowed us to identify several advantages of using coupled H-H models that can simultaneously provide stage and discharge prediction (i.e., unsteady rating curve) in watersheds with multiple discharge and stage gauging locations. First, the coupling creates the possibility for a validation of the hydrological model using stage measurements at internal watershed locations where only stage measurements are available. Second, the spatial and temporal variations of the channel velocities provided by the two different routing mechanisms can be used to provide a theoretical basis for empirically based hydraulic routing methods. Third, the strong constraint of lateral inflows imposed by the hydrological model reduces the possibility of calibration of certain 1D-SVE model parameters such as Manning coefficients. For example, in the implementation presented in this report, it was necessary to impose a channel streambed elevation correction to match observations rather than adjusting the Manning coefficients, which would be outside of the parameter values typically observed in open channels. Lastly we want to highlight the important issue that the coupling is not necessary for CUENCAS-HM to provide forecasts at all locations in the network. The coupling was shown to improve only slightly the performance of the forecasts, however, it also indicated that flood forecasts for certain reaches in the network could benefit from an online coupling of the models. A definitive answer is a left as a future area of research.

The outcomes of this project can be summarized as follow:

1) 25 sonic sensors were deployed in the Squaw Creek basin.

2) 22 sonic sensors continue operating and collecting information in the basin (3 instruments had to be brought back to the lab because of deployment issues).

3) The hydrological model CUENCAS was implemented and tested in the basin and validated at the outlet and at internal locations.

4) A hydraulic model was implemented for the major tributaries of the Squaw Creek where IFC sonic instruments were deployed.

5) Final rating curves based on surveyed cross sections were developed for the 22 IFC-bridge sites that are currently operating, and routine forecast is provided at those locations (see IFIS).

6) Rating curves were developed for 60 additional bridge locations in the basin, however, we do not use those rating curves for routine forecast because the lack of accuracy of LiDAR derived cross sections is not optimal.

7) We have demonstrated that the predictions made by the hydrological model at internal locations in the basins are as accurate as the predictions made at the outlet of the basin.

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APPENDIX A: ABSTRACT OF PAPERS SUBMITTED FOR PEER REVIEW TO JOURNAL OF HYDROLOGICAL ENGINEERING

Implementation of a Hydraulic Routing Model for Dendritic Networks with Offline Coupling to a Distributed Hydrological Model

Chi Chi Choi[1], George Constantinescu[2], and Ricardo Mantilla[3*]

Abstract: We present a new set of tools for solving the one-dimensional Saint-Venant equations (1D-SVE) of flow transport throughout dendritic river networks. The numerical solver is integrated with a set of geo-processing tools, that include automatic cross-section selection, river bathymetry extraction, and selection of model parameters, that facilitate the implementation of the 1D-SVE simulation setup. In addition, GIS-based preprocessing tools are developed to provide a seamless coupling of the hydraulic model to a hydrological model, which provides estimates of surface and subsurface runoff from hillslopes and performs routing in river networks using simplified ordinary differential equations. The hillslope runoff and streamflow generated by CUENCAS are re-distributed as lateral inflows to the channels modeled by the 1D-SVE hydraulic model. The coupling of the hydraulic and hydrologic (H-H) models enables the validation of the hydrological model at internal locations in the basin where stage measurements are made, instead of only at locations where streamflow is estimated. An application of the coupled H-H models is demonstrated in the Squaw Creek watershed, Iowa. Results show that the coupled H-H models serve to validate assumptions in the hydrological model related to the spatial and temporal production of runoff in the watershed and bolster confidence in the estimated discharges at ungauged locations.

CE Database subject headings: Hydraulic models; Hydrology; Geographic information systems; Cross sections; River flow; River systems; Hydrodynamics.

Author keywords: Coupled Hydrologic-Hydraulic models; One-dimensional Saint-Venant equations (1D-SVE); CUENCAS; Geographic information systems (GIS); Cross sections.

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A set of GIS tools for the automatic implementation of 1D hydraulic models and for coupling with distributed hydrological models

Chi Chi Choi[1] and Ricardo Mantilla[2*]

Abstract: We present a set of automated GIS-based geo-processing tools for the implementation of one-dimensional hydrodynamic (1D-HD) models that simulate unsteady open channel flow through a channel network. Our tools aim to streamline the process of preparing river bathymetry and model parameters for the 1D-HD model simulation and are designed to facilitate the coupling of the 1D-HD model with the distributed hydrological model CUENCAS. Our tools automatically identify hillslopes and side tributaries that are connected to the channels simulated by the 1D-HD model. We present two case implementations to illustrate the use of the geo-processing tools and to demonstrate how they simplify the labor-intensive pre-processing tasks of preparing the hydraulic model parameters and inflow boundary conditions. We use the first implementation to illustrate multiple details related to the automated algorithms and caveats of the tools. The second implementation is used to determine differences between our tools and the more manually intensive implementation of a HEC-RAS model that uses HEC-GeoRAS tools. We show that the fully automatic/unsupervised implementation made with our tools is comparable in quality and applicability to the HEC-Geo-RAS tools, with the additional benefit of saving time and requiring less expertise from the operator implementing the software.

CE Database subject headings: Hydraulic models; Hydrology; Geographic information systems; Cross sections; River flow; River systems; Hydrodynamics.

Author keywords: Coupled Hydrologic-Hydraulic models; One-dimensional Saint-Venant equations (1D-SVE); CUENCAS; Geographic information systems (GIS); Cross sections.

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