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Determination of flood inundation area in Cedar River using calibrated and validated 1D and 1D/2D model

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ABSTRACT

In 2008 flooding occurred over a majority of Iowa, damaging homes, displacing residents, and taking lives. Estimated damages to the state totaled $10 billion. One of the city affected by the flood was Waverly City, USA which is the Cedar River flow in side of it. One-dimensional/two-dimensional (1D/2D)-Mike Flood hydrodynamic model and 1D-HecRAS hydraulic model were developed for flood analyses in Waverly City. For developing both models, channel geometry was obtained from bathymetric surveys and combined with surface topography obtained from Light Detection and Ranging surveys. Roughness parameters were estimated using land use data from the National Land Cover Dataset and aerial photos. Calibration of roughness on floodplain and channels represents an important issue to determine flood inundation area. Both models were calibrated and validated with 2008 flood. The riverbed was validated with high water levels measured after flood. The floodplain was validated with photos, which was taken during flood from airplane. After calibration and validation, results were compared to choose the best one. Return period flood discharge were calculated and run with best inundation model for determine flood inundation maps. These maps can be used by residents and planners in Waverly City to help make informed decisions about potential risk from floods.

Keywords: hydrodynamic, calibrated, flood, Cedar River

Kalibre edilmiş ve doğrulanmış 1D ve 1D/2D model kullanılarak Cedar Nehri taşkın yayılım alanlarının tespiti

ÖZ

2008 yılında Iowa eyaletinin büyük bir bölümünde meydana gelen taşkın birçok konut ve işyerinin hasar görmesine ve birçok can kaybına yol açmıştır. Meydana gelen toplam hasar $10 milyar olarak tahmin edilmektedir. Taşkından etkilen şehirlerden biriside Cedar Nehri'nın tam ortasından aktığı Waverly dir. Waverly için, yüksek çözünürlüklü 1 boyutlu ve 2 boyutlu modelin birlikte çalıştırılması ile elde edilmiş hidrodinamik model geliştirilmiştir. Modelin geliştirilmesi amacıyla kanal geometrisi, batimetrik arazi çalışması ile topoğrafi datası Lidar (Light Detection and Ranging) yüzey topoğrafisi temini çalışmaları ile oluşturulmuştur. Pürüzlülük katsayıları ise "National Land Cover Dataset" arazi kullanım haritaları ve hava fotoğraflarından yararlanılarak tahmin edilmiştir. Hidrodinamik taşkın modeli, 2008 yılı taşkını ile kalibre edilmiştir. Nehir yatağının kalibrasyon ve doğrulaması taşkın sonrasında ölçülen maksimum su seviyeleri kullanılarak yapılmıştır. Taşkın yatağındaki yayılımın kalibrasyonu ve doğrulaması ise taşkın esnasında uçaktan çekilmiş olan fotoğrafların mukayesesi ile gerçekleştirilmiştir. Yapılan doğrulamalar sonucu en iyi model seçilmiş, bu model kullanılarak gelmesi muhtemel taşkınların yayılım alanları hesaplanmıştır. Elde edilen taşkın yayılım haritaları ile Waverly şehir plancılarının ve sakinlerinin potansiyel taşkın riskini bilerek buna göre kararlar almaları konusunda yardımcı olacağı düşünülmektedir.

Anahtar Kelimeler: hidrodinamik, kalibre, taşkın, Cedar Nehri

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1. INTRODUCTION

Regulation of natural processes, such as hurricanes, forest fires, and periodic floods, become “natural disasters” when human populations are adversely affected.

River “disasters” occur when water increases sharply and the river overflows the flat terrain (the floodplain) surrounding the riverbed as a result of torrential rains, melting snow cover and the uncontrolled discharge of water from existing dams. However, flood disasters are not only caused by meteorological events. Industrialization and urbanization have increased the diversity and intensity of human activities in river basins. This situation disrupts the hydrological balance of the entire basin and ultimately leads to loss of life and property as a large number of flood “disasters” occur. Larger and more frequent flood disasters have occurred in recent years because of increased settlement in river basins, the opening of new roads and establishment of new facilities which have altered the terrain, and the use of unsuitable agricultural methods which have degraded the soil and destroyed forests and grasslands.

In addition, even when a river is not designated as a flood risk, the development of infrastructure along the river may increase the risk. The usage of the river bed for infrastructure has led to the creation of new risk management plans. In order to develop the most accurate plans, it is necessary to create flood maps of various scenarios. These maps illustrate the flood areas, flood routes, and possible water retention regions. Risk management plans can implement this information to determine what precautions should be taken where. As the climate shifts, however, flood studies become increasingly important [1-11].

A high-resolution coupled one-dimensional/ two-dimensional (1D/2D) hydrodynamic model and 1D model of Waverly City, was developed. To develop the model, channel geometry was obtained from bathymetric surveys and combined with surface topography obtained from Light Detection and Ranging (LiDAR) surveys. Different return period discharges were calculated in accordance with "Bulletin 17 B" methods using HEC-SSP software. Roughness parameters were estimated using land use data and aerial photos.

The model was calibrated using measured 2008 flood high water surface elevations and aerial photos at river and floodplain. 2008 flood was analyzed using coupled one-dimensional/ two-dimensional (1D/2D) hydrodynamic model and 1D model. Results were compared and determined the best inundation model which was fit with real inundated area in 2008. Flood inundation maps of different return period were determined for entire study area and especially Waverly city center by analyzing with best inundation model.

2. METHODOLOGY

Flow of water in the natural environment can be approximated using numerical methods. The governing equations fluid flow are developed using continuity and the equations of motion. The resulting relationships, known as the Navier-Stokes equations, can be applied in three dimensions to solve complex fluid flows. Simplified one-dimensional and two-dimensional relationships, known as the St. Venant equations, are applied where a more complex description of flow is not necessary. Due to computational limitations, hydraulic models have typically solved the 1D St. Venant equations, which are computationally efficient, but cannot accurately model complex topography. Recent advances in computational capacity have made 2D solvers more feasible. A 2D model can accurately model complex topography, but is not as computationally efficient as a 1D model, and has difficulty modeling in-channel structures. Advantages of both types of models can be combined by coupling 1D and 2D models.

When developing a hydraulic model, uncertainty must be accounted for. Uncertainty in a hydraulic model can come from a number of sources. Errors in data collection can come from instrument error, resolution of collected data, and collection methods. Further uncertainty in simulation results can arise from assumptions made during model development, such as the choice of mesh resolution, methods of modeling structures, and methods of calibration.

2.1. One-Dimensional (1D) Flow Models

The most widely used approach to modeling fluvial hydraulics has been 1D finite difference solutions of the full Saint-Venant Equations [12]. The Saint-Venant Equations are based on conservation equations of mass and momentum for a control volume, as shown in differential form in Equations 1 and 2.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}(uQ) + gA\left(\frac{\partial h}{\partial x} - S_0\right) + gAS_f = 0 \qquad (2)$$

Where Q is discharge, A is cross-sectional flow area, u is longitudinal flow velocity, h is flow depth, So is bed slope, and Sf is friction slope. 1D solutions of the full Saint-Venant Equations are derived based on several assumptions: the flow is one-dimensional, the water level across the section is horizontal, the streamline curvature is small and vertical accelerations are negligible, the effects of boundary friction and turbulence can be accounted for using resistance laws analogous to those for steady flow conditions, and the average channel bed slope is small so the cosine of the angle can be replaced by unity [13].

Widely available software such as MIKE11 and HEC-RAS use the general form of the section-averaged Navier-Stokes equations. The basic forms of the equations used in MIKE11 are shown in Equations 3 and 4.

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q \tag{3}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\alpha\frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial h}{\partial x}+\frac{gQ|Q|}{C^2AR}=0 \tag{4}$$

Where Q is discharge, x is longitudinal channel distance, A is cross-sectional area, q is lateral inflow, t is time, h is flow depth, C is the Chezy coefficient and R is the hydraulic radius.

HEC-RAS has a similar approach except Manning's roughness is used to calculate friction losses instead of the Chezy coefficient [14]. The unsteady equations are solved by HEC-RAS using a four-point implicit scheme which requires that spatial derivatives and functions are evaluated at an interior point $(n+\theta)\ \Delta t$ [14]. Thus, values at the next time step are required for all terms in the general 1D equations. A system of simultaneous equations results from the implicit scheme. The effect of the implicit scheme allows information from anywhere within the reach to influence the solution. This discretization scheme requires much more computational effort than an explicit scheme, but it has improved numerical stability. Von Neuman stabilities analyses conducted by Fread (1974) [15] and Liggett and Cunge (1975) [16] found that the four-point implicit scheme is unconditionally stable for $0.5< \theta <1.0$ [14].

MIKE 11 also utilizes an implicit scheme, but uses a six-point Abbott scheme in solving the general Saint-Venant Equations [17]. Computations are performed on a grid consisting of alternating discharge, Q, and water level, h points. Simulation times depend on the number of computational nodes, but are typically completed in several minutes. Computational efficiency is one of the major advantages of employing a one-dimensional numerical scheme.

An inherent assumption of 1D finite difference river modeling is that flow velocities are perpendicular to cross-sections. Additionally, water surface elevations are assumed constant for entire cross-sections. For river reaches containing backwater areas or naturally occurring diversion channels, these assumptions are frequently violated. For out-of-bank flow, interaction with the floodplain results in highly complex fluid movement with at least two-dimensional properties. Flow at the channel-floodplain transition has been shown to develop a three-dimensional flow field due to intense shear layers [12].

Development of a one-dimensional hydraulic model requires user discretion in defining model geometry. Bates and De Roo (2000) [12] found that subjectivity of cross-section placement is an important contributor to the overall accuracy of a 1D hydraulic model. In addition to directly determining overbank reach lengths, placement of cross-sections must be executed so that changes in conveyance due to expansions or contractions are accurately captured.

2.2. Two-Dimensional (2D) Flow Models

To overcome the limitations of 1D models while maintaining practicality, 2D codes have been developed by depth-averaging, rather than section-averaging, the Navier-Stokes equations. The depth-averaged Navier-Stokes equations are referred to as the Saint-Venant shallow water equations. Two-dimensional numerical schemes can be classified as either full solutions to the shallow water equations or simplified approximations in which the inertia terms are omitted from the controlling equations (often referred to as zero-inertia schemes). Zero-inertia models are acceptable prediction tools when validation data are sparse and contain error. Neglecting inertia terms requires the assumption that flow over inundated areas is a slow, shallow phenomenon. Zero-inertia schemes can justify this assumption and provide results that are accurate at the reach scale, but local inaccuracies will occur [18].

Because the purpose of the current investigation is to provide accurate inundation data at the local scale, a numerical model that solves the full, dynamic Saint-Venant equations was required. Equations 5 through 7 list the Saint-Venant equations. Equation 5 represents continuity and Equations 6 and 7 describe conservation of momentum in the fluid.

$$\frac{\partial h}{\partial t}+\frac{\partial(hU)}{\partial x}+\frac{\partial(hV)}{\partial y}=0 \tag{5}$$

$$\frac{\partial(hU)}{\partial t}+\frac{\partial(hUU)}{\partial x}+\frac{\partial(hVU)}{\partial y}=\frac{\partial(hT_{xx})}{\partial x}+\frac{\partial\left(hT_{xy}\right)}{\partial y}-gh\frac{\partial z}{\partial x}-\frac{\tau_{hx}}{\rho} \tag{6}$$

$$\frac{\partial(hV)}{\partial t}+\frac{\partial(hUV)}{\partial x}+\frac{\partial(hVV)}{\partial y}=\frac{\partial\left(hT_{xy}\right)}{\partial x}+\frac{\partial\left(hT_{yy}\right)}{\partial y}-gh\frac{\partial z}{\partial y}-\frac{\tau_{hy}}{\rho} \tag{7}$$

In the equations above, U and V are depth-averaged velocity components in the x and y directions, respectively, Txx , Txy , and Tyy are depth-averaged turbulent stresses, z is water surface elevation (WSE) and τhx, τhy are the bed shear stresses due to friction. Comparing the Navier-Stokes equations to the depth-averaged Saint-Venant equations, one can see that all horizontal velocity terms have been depth-averaged, thus removing all differential terms in the vertical direction [19].

DHI's MIKE21 software utilizes similar equations to describe the conservation of mass and momentum in two horizontal dimensions, as shown in Equations 8, 9, and 10.

$$\frac{\partial \zeta}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=\frac{\partial d}{\partial t} \tag{8}$$

$$\frac{\partial p}{\partial t}+\frac{\partial}{\partial x}\left(\frac{p^2}{h}\right)+\frac{\partial}{\partial y}\left(\frac{pq}{h}\right)+gh\frac{\partial \zeta}{\partial x}+\frac{gp\sqrt{p^2+q^2}}{C^2h^2}-\frac{1}{\rho_w}\left[\frac{\partial}{\partial x}(h\tau_{xx})+\frac{\partial}{\partial y}(h\tau_{xy})\right]=0 \tag{9}$$

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial y}\left(\frac{q^2}{h}\right)+\frac{\partial}{\partial x}\left(\frac{pq}{h}\right)+gh\frac{\partial \zeta}{\partial y}+\frac{gq\sqrt{p^2+q^2}}{C^2h^2}-\frac{1}{\rho_w}\left[\frac{\partial}{\partial y}(h\tau_{yy})+\frac{\partial}{\partial x}(h\tau_{xy})\right]=0 \tag{10}$$

Where h is water depth, d is time varying water depth, ζ is surface elevation, p and q are flux densities in x- and y-directions, C is Chezy resistance, g is acceleration of gravity, ρw is the density of water, x and y are Cartesian coordinates, t is time and τxx, τxy, and τyy are the components of effective shear stress [17].

Two-dimensional hydraulic models have a number of advantages over one-dimensional models. Cook and Merwade [20] found that it is reasonable to assume that inundation extent can be more accurately predicted using 2D simulations, as influences of topographic and geometric features are more accurately represented. This advantage is especially apparent when modeling flood events in urban environments [21]. Two-dimensional models are more appropriate where observations and predictions are spatially distributed, such as flood maps, whereas 1D models are preferable for point measurements of stage or discharge [22]. Reduced computational efficiency compared to 1D models is the primary disadvantage of 2D models. A balance has to be struck between spatial resolution and computation time [22]. Two-dimensional models are also limited in their ability to accurately simulate structures such as bridges and weirs [23], as they are not designed to model pressurized flows. A potential method for reducing computation time and adequately simulating structures is to simulate the stream channel, where flow is typically longitudinal, using a 1D model and only model flow in the floodplain two-dimensionally.

2.3. Coupled (1D/2D) Flow Models

Modeling of urban flooding has presented several challenges to using typical one- and two- dimensional numerical codes [24]. One-dimensional numerical models are unable to resolve complex floodplain flow fields and require post-processing to produce realistic flood extents. Two-dimensional numerical models are unable to model structural elements that may produce super-critical or pressurized flow conditions. Consequently, recent urban flood modeling efforts have been focused on dynamically coupling one- and two-dimensional models to avoid these limitations [23],[24]. A one-dimensional numerical model of the river channel complimented by a two-dimensional model of the floodplain provides improvements in hydraulic modeling accuracy and computational efficiency. If an entire river reach is modeled using a one-dimensional model, then computational nodes within that portion of the two-dimensional mesh will not become active, improving computational efficiency.

MIKE FLOOD has been developed to accommodate several types of links between one-dimensional MIKE 11 and two-dimensional MIKE 21. These include the standard link, lateral link, and structure link as shown in Figure 1.

Standard links are explicit and are able to link ends of a MIKE 11 branch with a MIKE 21 computational mesh. These types of links allow model boundary conditions to be controlled by a rating curve, which is useful when modeling unsteady conditions. The discharge contribution from a MIKE 11 branch affects the continuity and momentum equations in the MIKE 21 cell when linked with a standard link [17].

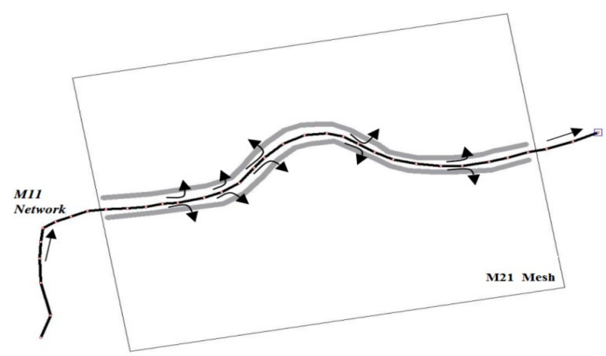

Figure 1. Lateral link structure between one-dimensional MIKE 11 and two-dimensional MIKE 21 [17]

The link requires the MIKE 11 branch be one time step behind the MIKE21 mesh; therefore a discharge predictor is utilized for the time step n + 1/2, as shown in Equation 11.

$$\frac{\partial Q^{n+1/2}}{\partial t} = -\left(gA\frac{\partial H^n}{\partial x} + \frac{Q^n|Q^n|}{AC^2R}\right) \quad (11)$$

Where Q is discharge, t is time, g is acceleration of gravity, A is cross-sectional area, H is water level, x is longitudinal distance, C is the Chezy coefficient, and R is hydraulic radius. This predictor assumes that the roughness coefficient is controlling the flow. Lateral linking of a MIKE 11 branch to a MIKE 21 mesh allows water to enter the floodplain laterally from the river channel. The linking method is explicit. The flow exchanged between the two models is controlled by a structural relationship such as a weir equation. Since one-dimensional hydraulic models like MIKE 11 do not consider cross-channel flow, momentum cannot be conserved across this type of link [17].

Structural links are used to incorporate the effects of structural elements such as dams and bridges. This linking procedure is the most stable coupling method due to its implicit nature. The function of the link is to utilize the momentum calculated through a MIKE 11 branch to modify the momentum in adjacent MIKE 21 cells in order to represent the hydraulic effects of the structure [17]. Conservation of momentum is not guaranteed, so emphasis is placed on interrogating simulation results.

3. MODEL APPLICATION

3.1. Study Area

Waverly City is located in northeast Iowa along the banks of the Cedar River in USA, is chosen as a study area (Figure 2). It lies between the longitudes 92°30' W and 92°25' W and the latitudes 42°48' N and 42°40' N. The river reach within the study area is approximately 23 km in length, running from northwest to southeast through the watershed. The entire study area covers 145.5 square kilometers. Waverly city itself covers 23 square kilometers of the study area. The study area includes 5 bridges and 1 dam.

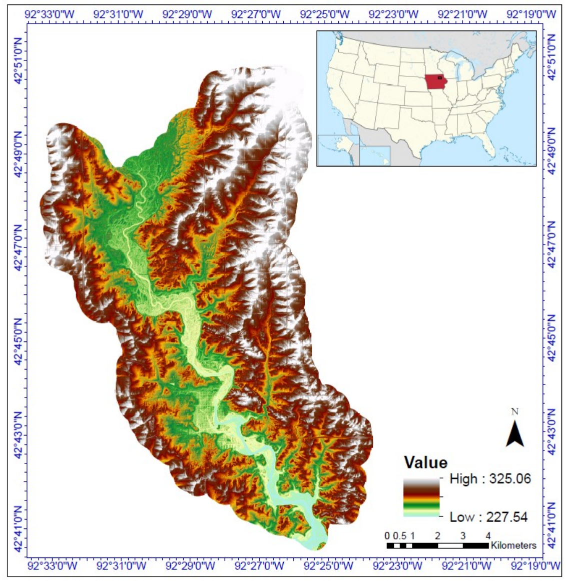

Figure 2. Study area

3.2. Flow Data

United States Geological Survey (USGS) stream gage 05458300 is located of the bridge crossing at Horton Road. The drainage area at the USGS gage is 1547 square kilometers. Stage-discharge and peak discharges have been recorded continuously at the gage since 2001. The largest flood on record occurred in 2008 with 1489.46 cms (52600 cfs) value. However, the number of peak discharges isn't enough to hydrological analyses. Therefore, historical peak discharges were calculated using "Area-Weighted Estimates for Ungaged Sites on Gaged Streams" methods. USGS stream gage 05458500-Jenesville which is located 19.5 km downstream of USGS stream gage 05458300-Waverly.

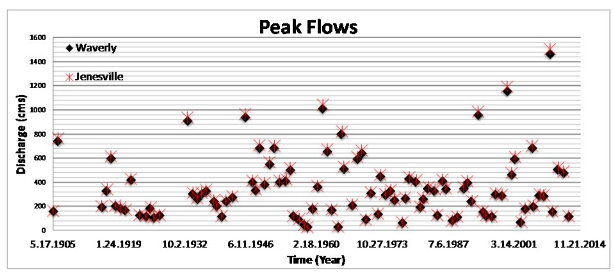

Figure 3. Artificial discharge of waverly

Flood discharges values were calculated using these artificial historical peak flows (Figure 3). Bulleten17B was chosen as a calculation method for flood discharges in HEC-SSP software (Table 1).

Table 1. Return period discharge

Return Period (Year)	Discharge (cms)	0,05 Confidence Limit	0,95 Confidence Limit
500	1631.4	2447	1439
200	1401.8	2040	1241
100	1232.5	1746	1093
50	1064.4	1465	947
25	899.8	1113	754
10	681.5	863	609
5	513.8	627	462
2	282.6	332	255

3.3. Land Use Data

Manning's n values were determined using National Land Cover Dataset and aerial photos. Land use data of study area is shown in Figure 4. %70 of study area is an agricultural lands and cover with corn, soybeans and oats. Overbank is completely cover with dense forest and inland ponds. Waverly City Center, 20% of the entire study area, is approximately 30 km^2.

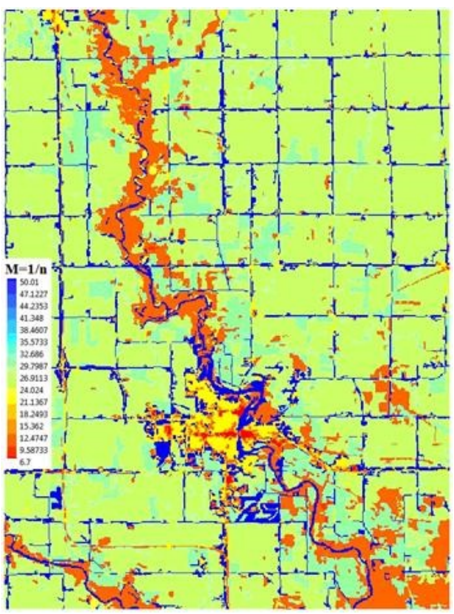

Figure 4. Land use data

3.4. Topographic Data (Topografya Datası)

Topographic data is the most important input of all hydrodynamic models. Cross Section of 1D model and floodplain mesh of 2D model was created with Digital Elevation Model (DEM). In this study, DEM was created from 1m*1m LIDAR data. Floodplain was sensitive but riverbed was poor because of reflection from water. This reason, generated DEM was modify with river bathymetry which was created from field survey. Differences of DEM sensitivity before and after modification was shown in Figure 5.

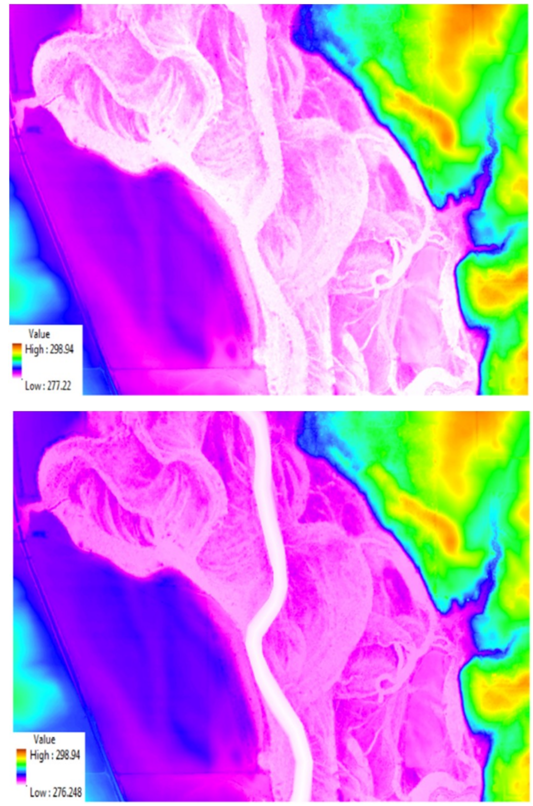

Figure 5. Differences of DEM sensitivity

4. CALIBRATION AND VALIDATION

4.1. Calibration of Model

After hydraulic and hydrologic infrastructure of models were completed , the model was calibrated with 2008 Flood for accurately representing of study area. For this purpose, the maximum water surface elevations of models and the maximum water surface elevation of 2008 flood were compared with each other for obtaining an accurate model (Figure 6) . Each models default parameters, river and floodplain roughness were changed for reach same inundated area and water surface elevation with 2008 flood.

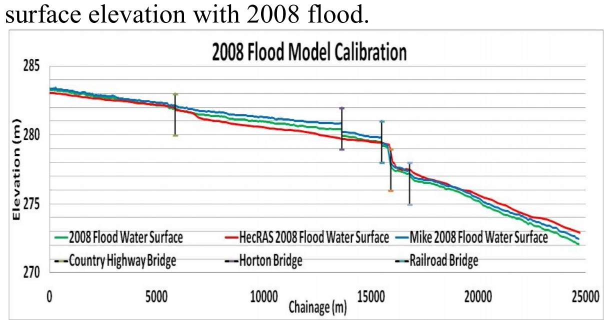

Figure 6. 1D and Coupled model calibration using highwater levels on river bed

4.2. Validation of Model

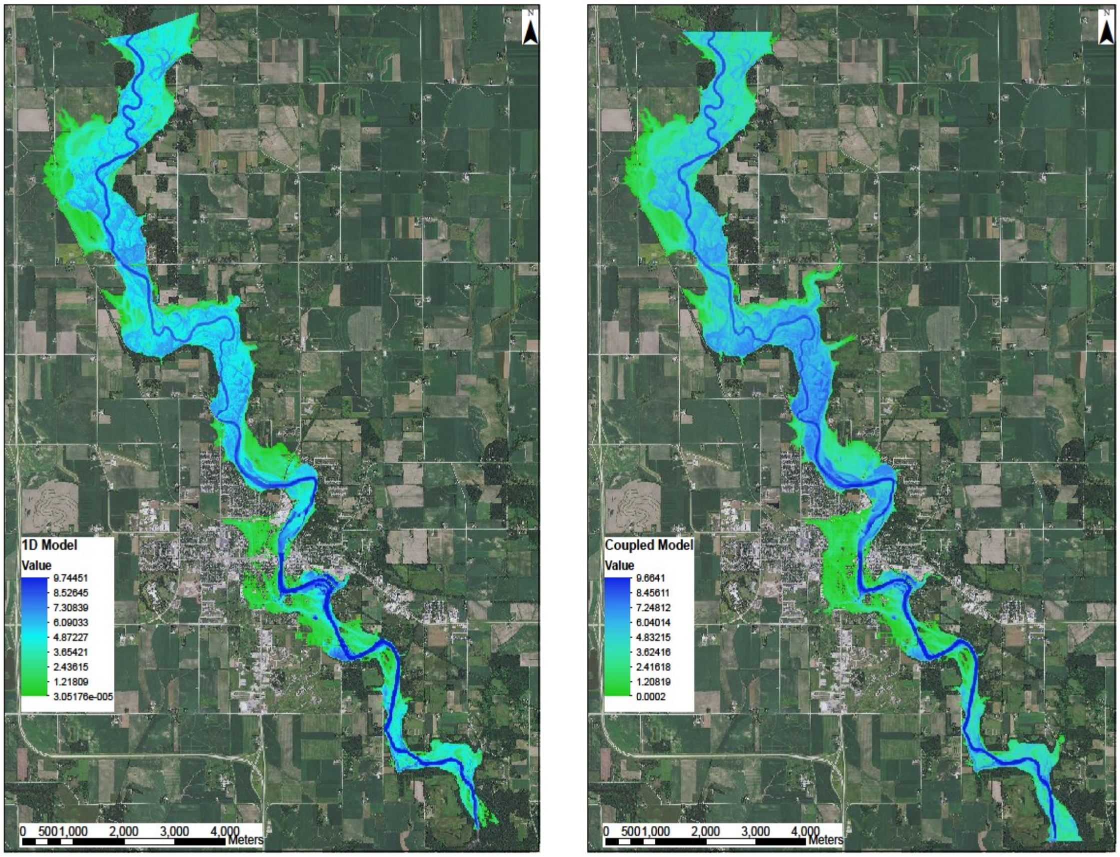

Figure 7. 2008 flood inundation maps using Calibrate 1D and Coupled Model

2008 flood inundated area was determined using calibrated HecRAS and Mike Flood Model (Figure 7). When the results were compared, it was observed that Mike Flood Model has extremely same result with real (Figure 8-10).

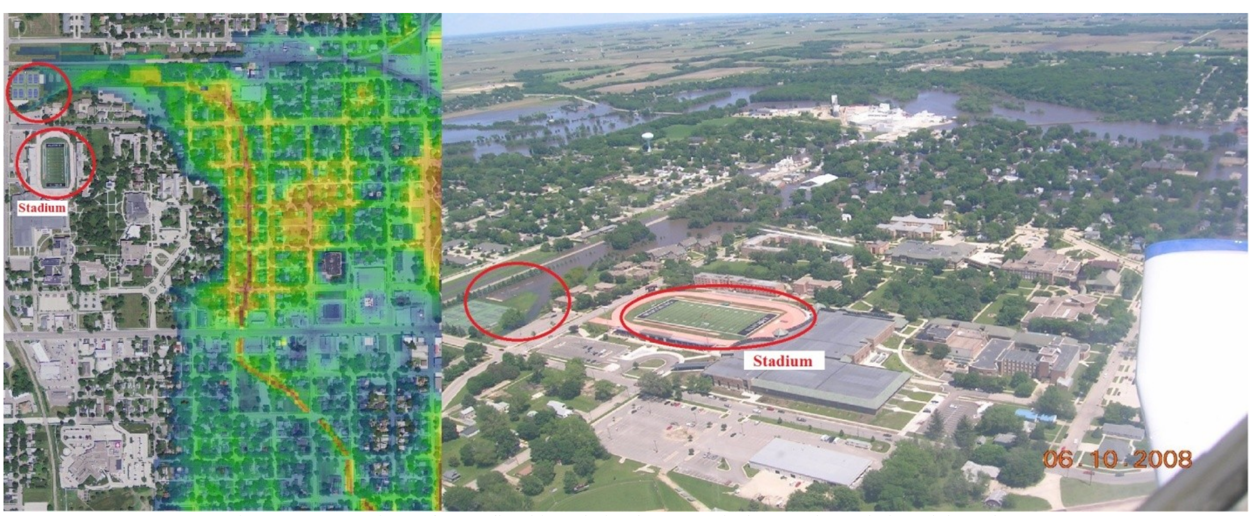

Figure 8. Comparison of 2008 flood inundation between airphoto and coupled model result-1

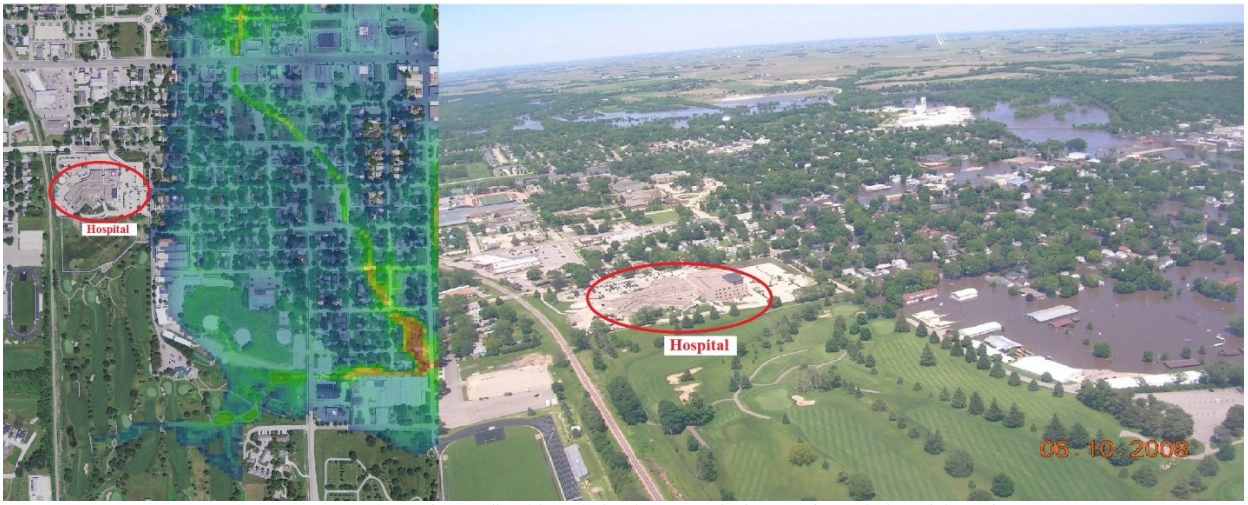

Figure 9. Comparison of 2008 flood inundation between airphoto and coupled model result-2

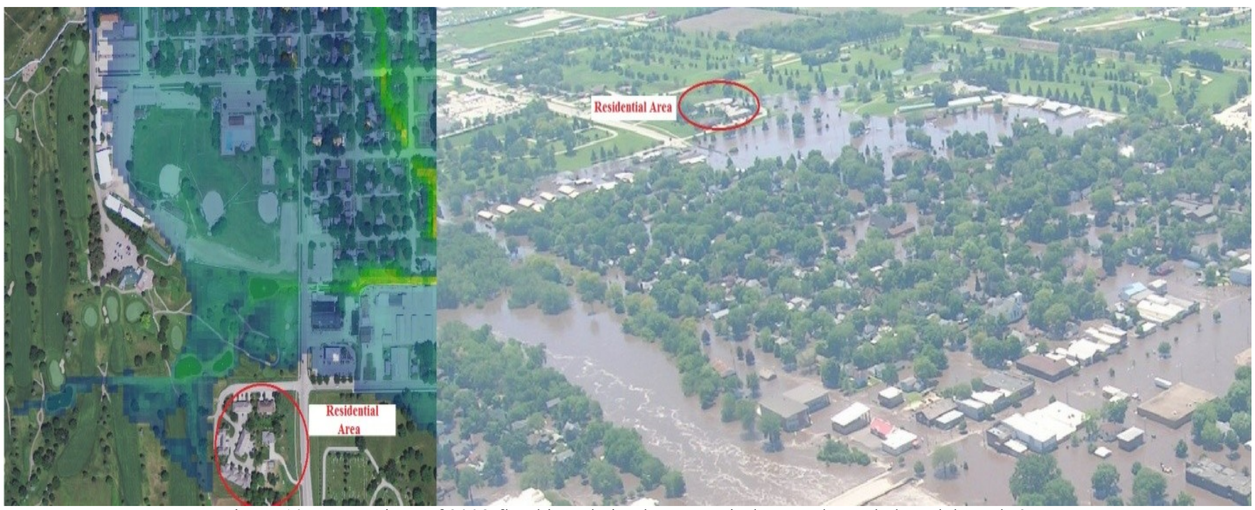

Figure 10. Comparison of 2008 flood inundation between airphoto and coupled model result-3

Between Figures 8 and 10 shows that, aerial photos which was taken during 2008 flood were compared with model results and have been found to give very close to real results.

Calibrated one-dimensional HECRAS model was insufficient for correctly detection of inundated area for urban area. According to both results, all floodplain especially city center (urban area) were correctly representing by one-dimensional/ two-dimensional Mike flood model. Flood inundation maps were carried out for different return period using coupled model after calibration and validation.

5. RESULTS AND DISCUSSION

5.1. Flood Iundation Maps

Calibrated and validated coupled model was analyzed with different return period and result were show between figure 11-18. What amounts to the flooding was affected of study area was determined. Total inundated areas was shown at Table 2.

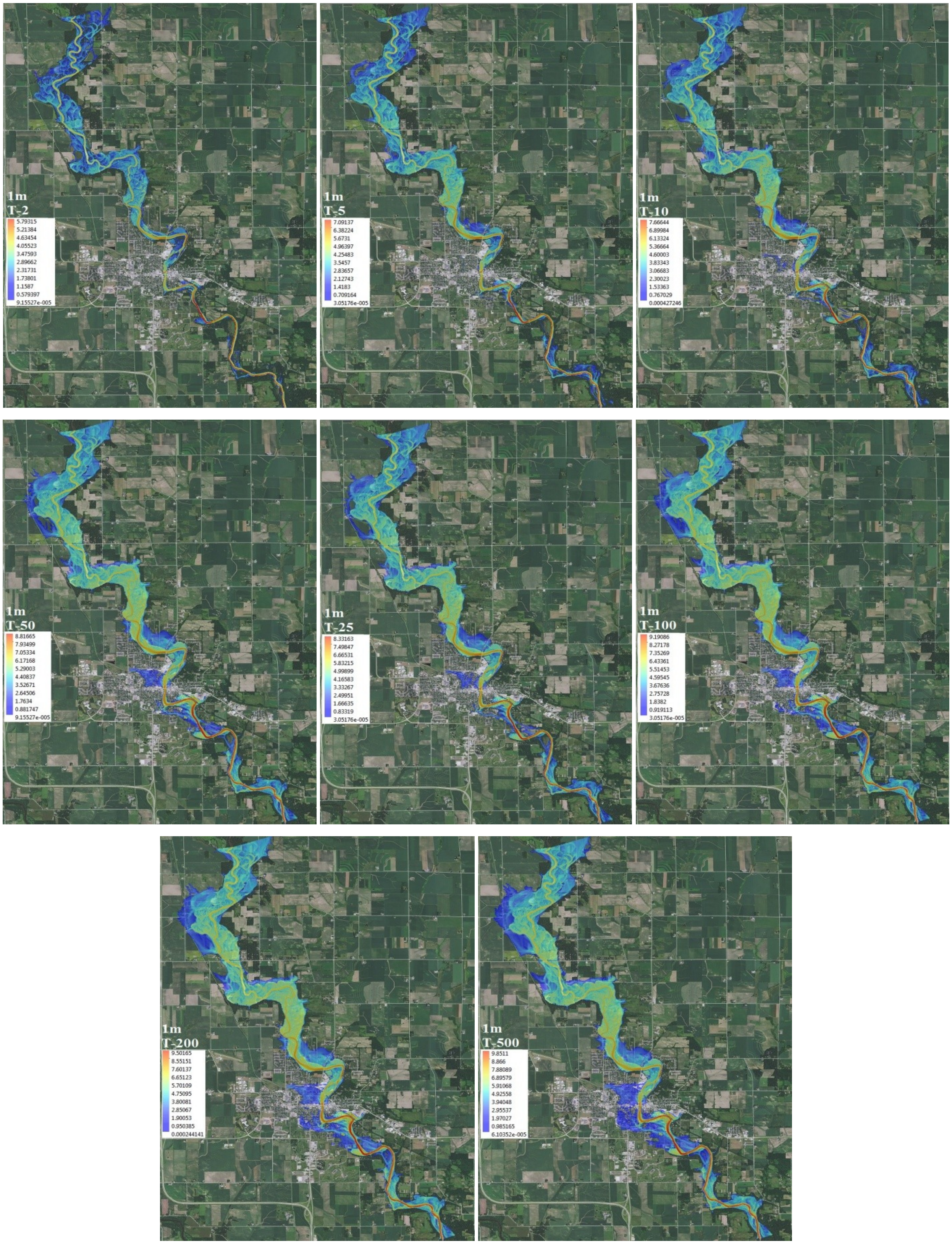

Figure 11-18. Flood inundation maps for different return period

Table 2. Inundated area for different return period

Return Period (Year)	2	5	10	25	50	100	200	500
Inundated area (sq.km)	8.40	8.50	11.30	12.70	13.70	15.00	16.10	17.20

6. CONCLUSION

Calibrated 1D and coupled model analyzed with 2008 flood discharge. model results was compared with aerial photos which is taken during flood. Coupled model gave extremely same result with real inundated area. According to result, 16,41 km^2 land was under flooding during 2008 flood. A significant portion of the Waverly city center was affected by the flood.

Both riverbed and floodplain were designed between upstream and downstream using cross-section for 1D model. The topographic and bathymetric characteristics of surfaces were determined by interpolating between two cross section. Therefore, study area was defined by only cross sections and generated interpolation inputs. As a result, topographic details will not correctly represented between two cross section. This model will lead to some handicap especially in urban areas which has lots of topographic details and changing.

But in coupled model, floodplain (urban area) was represented with 1m*1m elevation grid mesh which was derived from LİDAR. Results was accurately estimated by grid to grid flood and dryness analysis. The coupled models should be preferred while flood analysis in complex topographic areas such as city center and urban areas. This allows to obtain more reliable results are achieved.

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