JOURNAL OF THE AMERICAN WATER RESOURCES ASSOCIATION
JUNE AMERICAN WATER RESOURCES ASSOCIATION 2006

AN INTEGRATED ONE-DIMENSIONAL AND TWO-DIMENSIONAL URBAN STORMWATER FLOOD SIMULATION MODEL[1]

Xing Fang and Dehui Su[2]

ABSTRACT: Flash flooding is the rapid flooding of low lying areas caused by the stormwater of intense rainfall associated with thunderstorms. Flash flooding occurs in many urban areas with relatively flat terrain and can result in severe property damage as well as the loss of lives. In this paper, an integrated one-dimensional (1-D) and two-dimensional (2-D) hydraulic simulation model has been established to simulate stormwater flooding processes in urban areas. With rainfall input, the model simulates 2-D overland flow and 1-D flow in underground stormwater pipes and drainage channels. Drainage channels are treated as special flow paths and arranged along one or more sides of a 2-D computational grid. By using irregular computation grids, the model simulates unsteady flooding and drying processes over urban areas with complex drainage systems. The model results can provide spatial flood risk information (e.g., water depth, inundation time and flow velocity during flooding). The model was applied to the City of Beaumont, Texas, and validated with the recorded rainfall and runoff data from Tropical Storm Allison with good agreement.
(KEY TERMS: drainage; modeling; stormwater management; surface water hydrology; hydraulics; urban water management.)

Fang, Xing and Dehui Su, 2006. An Integrated One-Dimensional and Two-Dimensional Urban Stormwater Flood Simulation Model. Journal of the American Water Resources Association (JAWRA) 42(3):713-724.

INTRODUCTION

Since the stormwater produced flood is one of the most severe and frequent natural disasters in the world, flood prevention and mitigation has a long history of study in both hydrology and hydraulics. With increasing population and build up of urban areas, the hydrological and hydraulic properties of these areas have been greatly changed, leading to increased flood hazard and damage (Espey *et al.,* 1966). Consequently, the study of flood mitigation is still important. High intensity and short duration precipitation associated with thunderstorms can generate the large volumes of surface runoff (stormwater), which combined with flat terrain and low gradient drainage systems lead to severe urban flooding – flash flooding. For example, coastal areas in southern Texas face frequent threats of urban flash flooding. In fact, a recently unprecedented event occurred in Houston and the surrounding areas starting June 5, 2001. Tropical Storm Allison created one of the most devastating rain events in the history of the United States, simultaneously altering the normal lives of more than 2 million people: Allison caused 22 fatalities, 95,000 damaged vehicles, 73,000 damaged residences, 30,000 stranded residents in shelters, and over US$5 billion in property damage (Bedient and Holder, 2001).

The physical process from rainfall to runoff/flood is complicated. First, the excess rainfall forms a surface overland flow (sheet flow). The overland flow is typically treated as two-dimensional (2-D) flow from high elevation to low elevation, eventually entering man made or natural drainage systems including (e.g.,

[1]Paper No. 04111 of the *Journal of the American Water Resources Association* (JAWRA) (Copyright © 2006). **Discussions are open until December 1, 2006.**

[2]Respectively, Associate Professor, Department of Civil Engineering, Lamar University, Beaumont, Texas 77710-0024; and Doctoral Graduate at Lamar University; currently Engineer, Costello, Inc., 9990 Richmond Avenue, Suite 450, North Building, Houston, Texas 77042 (E-Mail/Fang: xing.fang@lamar.edu).

stormwater pipes, ditches, rivers, and bayous). The flow in pipes/channels/ditches is typically treated as one-dimensional (1-D) flow. Therefore, the stormwater runoff process includes 1-D flow, 2-D flow, and transition flow from 2-D to 1-D. The overall objective of this study is to seek a hydraulic method instead of the traditional hydrological method for simulating rainfall and stormwater/ runoff processes that result in urban floods. The model presented here combines/integrates the 1-D and 2-D hydraulic methods and simulates the complex flood forming and developing processes over the land surfaces and in the ditches/channels/ streams. Such a model can be used to obtain detailed spatial flood information that can be a new tool in flood risk mapping; it is very useful in the decision making for flood prevention and mitigation for many agencies [e.g., Federal Emergency Management Agency (FEMA)].

For a long time, the study of stormwater runoff/flood phenomenon used the systems approach at the scale of a basin or watershed, and mainly belonged to hydrological study (McCuen, 1998). The watershed system is described by a set of mathematical relationships and physical parameters (e.g., watershed area, land slope, soil property, and vegetation coverage). The relationships between rainfall and runoff are usually quantified by numerous empirical equations or models as a function of watershed parameters [e.g., the U.S. Department of Agriculture-Soil Conservation Service (USDA-SCS) curve number method (USDA-SCS, 1972), and the Clark unit hydrograph method (Clark, 1945)]. These hydrological methods have been also summarized and integrated into software for public use, such as the HEC-1 and HEC-HMS by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACE, 1981, 2000). These hydrological methods are the black box approach, which estimates surface runoff from rainfall input for a whole basin, and absorbs the complex hydrodynamic flow process into a handful of parameters that are difficult to specify. The disadvantage of this method is that it does not provide any detailed spatial information of water surface elevation and flood depth variation with time within a basin. However, with the social and economic development of urban areas, flood prevention and mitigation need detailed spatial information of surface runoff. For example, the National Flood Insurance Program could use flood risk maps with floodwater depth distribution for each street and building in an urban area during rainstorms. Hydrological methods cannot meet these specific requirements.

One-dimensional hydraulic methods have been used to study flood in river valleys for a long time. A flood occurs when a river is unable to hold the water in its main channel. Using the discharge process or peak discharge estimated from hydrological models as an input parameter, the 1-D open channel flow model is used to simulate the water surface variation and delineate flood plains around a river. For example, HEC-RAS has been widely used to delineate the regulatory flood plain zone of 100-year or 500-year flood around a river (Roberson *et al.,* 1998).

Several 2-D hydraulic models were developed and used in shallow rivers and flood plains. While these simulation models can obtain spatial flood information, they are still restricted to certain flood types such as dam breaking floods or floods in a shallow river valley. Numerous studies about 2-D dam break models have been published [e.g., Chow and Ben (1973), Katopodes and Strelkoff (1978), and Jha *et al.* (2000)]. However, the urban stormwater flood is different from dam breaking floods or river overflowing floods, and is usually caused by local high intensity rainfall and handicapped (e.g., low gradient) drainage systems. Besides using traditional hydrological methods as a primary tool, in recent years 2-D models have been employed to simulate some urban stormwater flood problems. Iwasa and Inoue (1980) and Toda *et al.* (2001) applied a two 2-D numerical model for urban flood simulation in Japan. Cheng and Qiu (2001) summarized the urban flood simulation techniques in China when a 2-D model was used to simulate the stormwater produced floods for the Great Tianjing City in northeast China, and the model included the simulation of flow in stormwater pipes. The Delft Hydraulics' package Delft-FLS is a finite difference model that is a fully coupled 1-D and 2-D system for flood simulation with rectangular computational grids (Laguzzi *et al.,* 2001).

Both current hydrological methods and 1-D or 2-D hydraulic models have some limitations (as mentioned above) in simulating floods produced by stormwater within an urban area; therefore, an integrated 1-D and 2-D flood simulation model was used in this study. This paper presents not only research development and in-depth investigation for some theoretical questions and numerical simulation techniques of the 1-D and 2-D stormwater flood simulation model (e.g., 1-D and 2-D integration, irregular grid application, grid division, simulation of storm pipe system), but also application of the model to simulate the flood in Beaumont, Texas, due to the Tropical Storm Allison.

NUMERICAL MODEL DEVELOPMENT FOR STORMWATER FLOOD

To simulate the complex flooding process in urban areas, the proposed model differs from traditional numerical models in handling governing equations, computational grid division, and integration of 1-D and 2-D flows. The model includes the following characteristics.

• The 1-D and 2-D flows are simulated and handled separately. For overland flow, the simplified 2-D shallow water equations are employed. Rainfall input minus rainfall loss is taken as the source in the continuity equation. For the 1-D ditch/channel flow, a 1-D unsteady open channel flow model is used. The model integrates 2-D flow with 1-D flow simulation into a seamless integration model.

• Irregular computational grids are adopted. The study area for flood process simulation is divided into computation grids (Figure 1). Since urban areas typically include a complex drainage channel/ditch and stormwater pipe system, as well as complex topographic settings including roads, streets, public buildings, and homes, it is difficult to simulate combined 2-D and 1-D flows with the traditional 2-D regular (rectangular) grids. The model developed in this study uses irregular grids for overland flow simulation including triangular grids (e.g., grids abh, bce, bef, bfg, and bgh in Figure 2), rectangular grids (e.g., most of grids in Figure 1 and the grid abcd in Figure 2), and polygon grids with any number of sides (several special grids in Figure 1), therefore it can be applied to any complex topography and drainage systems. The intersection point of any two grid sides is called as a node or computational node, for example, nodes a to h in Figure 2.

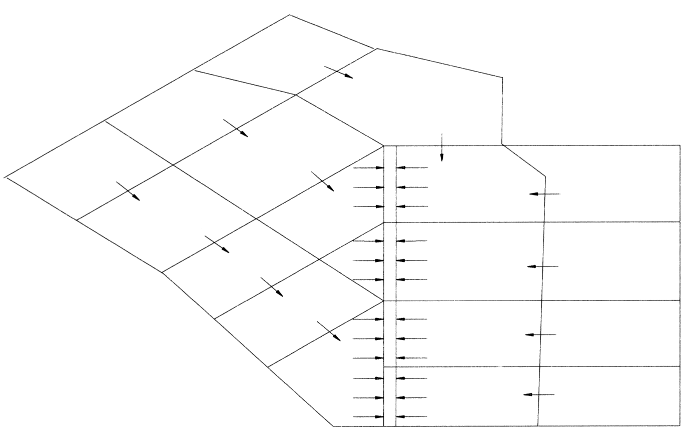

Figure 1. Irregular Grids for 2-D Surface Flow and Ditch Arrangement for 1-D Flow.

• The 1-D model for small channels and ditches (with typical dimensions smaller than a grid) is coupled with the 2-D model to form an integrated (mixed) model. To adopt the complex drainage channel system, small channels and ditches are always arranged along one or more sides of grids for 2-D simulation (Figures 1 and 2). The 2-D and 1-D flows are simulated separately for grid and sides but integrated when the transitional flows from 2-D converging into 1-D are simulated. In this way, stormwater crossing a side of a grid and flowing into an adjacent grid (which is 2-D flow) or flowing into a ditch on the side (it becomes 1-D flow) can be simulated. Stormwater can also flow downstream along a side (e.g., sides ab, bc, bf, and bh in Figure 2) if that side is modeled as a ditch. In the proposed model, any side of a grid is treated as a unique flow path; therefore, the side of a grid is also called a path or flow path.

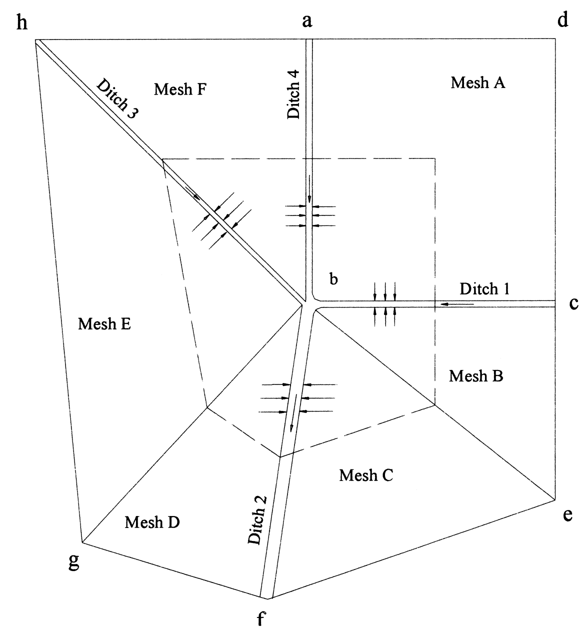

Figure 2. Irregular Grids of Polygons and Linkage With Ditches (1-D flow).

• For urban areas, the overland flow usually first enters inlets and stormwater pipes and then converges to drainage channels and ditches. Since the stormwater pipe arrangement in an urban area can be very complex, it is still impractical to simulate every inlet and each pipe flow in the combined 2-D and 1-D flow model, so a simplified method is used. When a grid contains inlets and stormwater pipes, the pipe arrangements are represented or simplified as five typical types as shown in Figure 3. Type (a) and (e) arrangements are an "I" shape, in which underground pipes pass through only one side or two opposite sides of a grid and collect stormwater from inlets at the same time. In the model, inlets are simplified or conceptualized as one inlet for one grid, which is located at the center of the grid. Type (b), (c), and (d) arrangements are "L," "T," and "+" shapes, in which underground pipes pass through two, three, and four adjacent sides of a grid and collect stormwater from the inlet (Figure 3). If any grids near the border of an urban or rural area do not contain stormwater inlets, the surface flow is assumed to exchange with flow from adjacent grids (Figure 2) or flow directly into an adjacent channel (Figures 1 and 2). If any grids contain stormwater inlets, the model assumes that the surface flow always flows into the inlet, but does not flow from the surface into adjacent grids or channels, and this is usually true for urban areas with relatively flat terrain. In Figure 4, a cross-sectional view shows the flow path from surface overland flow into the inlet, then an underground stormwater pipe, and finally a drainage channel. While this complex drainage setting is in the proposed model, it has never been included in previous models. The vertical dashed lines in Figure 4 indicate the location of sides (or paths) of grids. Since the drainage channel is exactly located on one side of a grid, this side will handle and simulate the 1-D flow in the channel. In the numerical model, the dimensions of a grid are typically much larger than actual channel dimensions, therefore the scale for channel dimensions in Figure 4 is greater than the scale for the grid (for illustration purpose only). Figure 4 also shows a small levee along both sides of the channel, which indicates that the surface flow enters the inlet, and not the channel. Important structures and facilities (e.g., sluice gates and pump stations that may operate during a flood in urban areas, may also be simulated in the model).

Governing Equations for 2-D Overland Flow

Modified 2-D shallow water equations (Hromadka *et al.,* 1987; Ligget, 1994) are employed as follows for overland flow simulation:

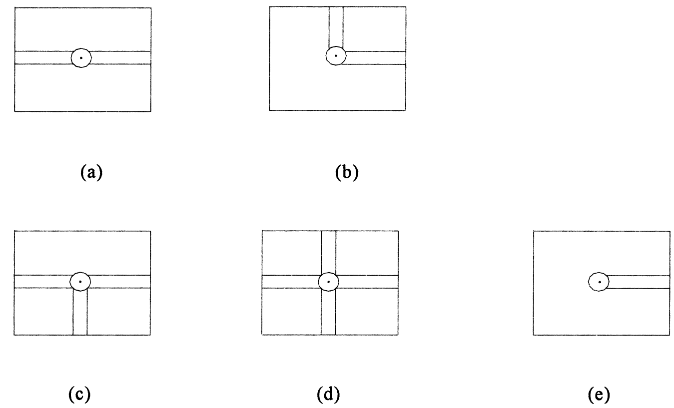

Figure 3. Simplified Arrangements of Stormwater Pipe in Grids.

$$\frac{\partial H}{\partial t} + \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = q \tag{1}$$

$$\frac{\partial q_x}{\partial t} + \frac{\partial (uq_x)}{\partial x} + \frac{\partial (vq_x)}{\partial y} + gH\frac{\partial Z}{\partial x} + g\frac{n^2 u\sqrt{u^2+v^2}}{H^{1/3}} = 0 \tag{2}$$

$$\frac{\partial q_x}{\partial t} + \frac{\partial (uq_y)}{\partial x} + \frac{\partial (vq_y)}{\partial y} + gH\frac{\partial Z}{\partial y} + g\frac{n^2 v\sqrt{u^2+v^2}}{H^{1/3}} = 0 \tag{3}$$

where q is the source term as net rainfall intensity (m/s); H is the water depth (m); Z is the water surface elevation (m), ground elevation plus the water depth H; q_x and q_y are discharges per unit width in the x and y directions (m^2/s); u and v are velocities in the x and y directions, respectively (m/s); g is the gravity acceleration (m/s^2); and n is overland flow Manning's roughness coefficient. Friction slopes are approximated by Manning's formula for 2-D shallow water flow (Ligget, 1994). The equations are applied to a study

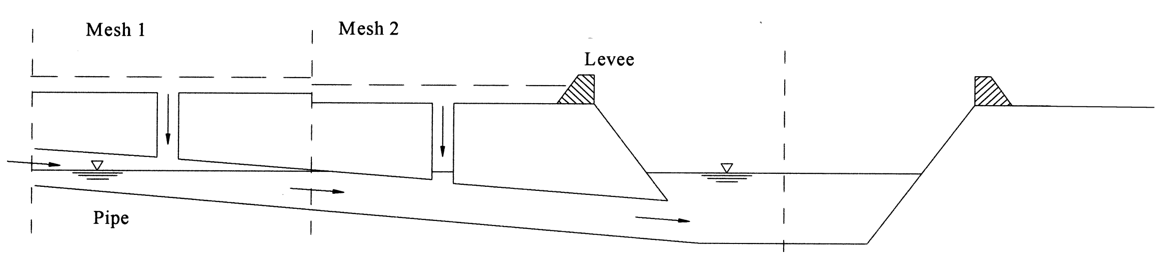

Figure 4. Schematic Showing 2-D Overland Flow Entering Into Stormwater Pipes and Then Draining to a Ditch.

area composed of 2-D computational grids (Figures 1 and 2); therefore, 2-D control-volume and finite difference forms of the above equations are used. Water level and water depth are computed at the center of a grid or control volume. The interchange of flow between grids is solved using the difference form of the continuity equation for a grid, which can be written as

$$H_i^{T+2DT} = H_i^T + \frac{2DT}{A_i}\sum_{k=1}^{N} Q_{ik}^{T+DT} l_{ik} + 2DT q^{T+DT} \quad (4)$$

where l_{ik} and Q_{ik} are the length (m) and the discharge (m^2/s) per width of the kth side of the grid i; N is the number of sides for the grid i; T and DT are the time and time steps; and A_i is the surface area (m^2) of the grid. Since 2-D irregular computational grids are used in the model as the control volumes, Q_{ik} is replaced for q_x and q_y to present the discharge per width perpendicularly crossing any side of a grid.

Typically, the overland sheet flow varies slowly along its flow path and over time on relatively flat terrain. Since the convection terms in Equations (2) and (3) can be small compared to the resistance and gravity terms, they are ignored in this model. The finite difference representation of the momentum equation for the flow crossing the kth side of the grid i is

$$Q_i^{T+2DT} = Q_{ik}^{T-DT} - 2gh_{ik}DT\frac{Z_{i2}^T - Z_{i1}^T}{DL_{ik}} - 2gDT\frac{n^2 Q_{ik}^T\left|Q_{ik}^T\right|}{h_{ik}^{7/3}} \quad (5)$$

where h_{ik} and DL_{ik} are the average water depth (m) and the distance (m) between centers of adjacent grids $i1$ and $i2$ along the left side and right side of the kth path (side) of a grid i, respectively; and Q_{ik}/h_{ik} replaces u and v in Equations (2) and (3). The similar momentum Equation (5) is repeated for any sides of a grid with any number of sides (Figures 1 and 2).

Governing Equations for 1-D Flow in Ditches and Rivers

For 1-D open channel flow in drainage ditches/channels/rivers, the differential continuity and momentum equations are simplified as

$$\frac{\partial(hb)}{\partial t} + \frac{\partial Q}{\partial x} = bq' \quad (6)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gA\left(S_f - S_0\right) = 0 \quad (7)$$

where q' is the source term as net rainfall intensity (m/s) plus lateral flow per reach/side length from 2-D overland grids; b or $b(h, t)$ is the surface width (m) of channel cross section; A or $A(h, t)$ is the flow cross-sectional area (m^2); $h(t)$ is the water depth in the channel (m); S_f is the friction slope, and S_0 is the channel bed slope. For rectangular ditches, the above equations can be simplified in the difference form

$$Q_{kd}^{T+DT} = Q_{kd}^{T-DT} - 2gh_{kd}DT\frac{Z_{k2}^T - Z_{k1}^T}{DL_{kd}} - 2gDT\frac{n^2 Q_{kd}^T\left|Q_{kd}^T\right|}{h_{kd}^{7/3}} \quad (8)$$

where Q_{kd} is the discharge (m^2/s) per channel width on the kth path of a grid (e.g., path ab and bc for the grid abcd in Figure 2); h_{kd} is the average water depth (m) of upstream and downstream nodes of the kth path (e.g., computational nodes a and b on the special path ab, Figure 2); Z_{k1} and Z_{k2} are the water levels (m) at the upstream node $k2$ and the downstream node $k1$ of the kth path; and DL_{kd} is the ditch length (m) (path length) between nodes of the special path.

The overland surface flow will either converge directly into a ditch or enter a stormwater pipe and then flow into ditches (Figure 4). The transitional flow from land to these ditches is the key integration of 1-D and 2-D hydraulics methods. The difference representation of the continuity equation of such special 1-D flow paths (sides) with a rectangular cross section can be given as

$$H_{di}^{T+2DT} = H_{di}^T + \frac{2DT}{A_{di}}\left(\sum_{k=1}^{M} Q_{ik}^{T+DT} b_{ik} + \sum_{j=1}^{2M} Q_{ij}^{T+DT} L_{ij}/2\right) + 2DT q_{dt}^{T+DT} \quad (9)$$

where H_{di} is the average water depth (m) at the node; A_{di} is the bottom area (m^2) of the ditch around the node; and $\sum Q_{ik}\, b_{ik}$ is the summation of discharges (m^3/s) due to flow exchange among ditches. As an example, in Figure 2, discharge converging and leaving node **b** from the ditches **ab, hb, cb**, and **bf** are included in this computation. $\sum Q_{ij}\, L_{ik}/2$ is the summation of discharges (m^3/s) due to flow exchange between the ditch and adjacent grids (Figure 2). L_{ij} is the length (m) of a ditch connecting to the node. For

1-D channel flow, initial water depth and elevation at each node are first given as very small values. The momentum equation is solved to determine discharge between channel reaches at time step $T+DT$, and then water depth and level at grid nodes for the time step $T+2xDT$ are solved by the continuity equation. Equation (9) has to be solved after discharge from adjacent 2-D grids has been computed as discussed before.

Governing Equations for 1-D Flow in Stormwater Pipes

During an urban flood, the flow in a stormwater pipe can be of two types: open channel and pressure pipe flow for small and large runoff inputs from grids (overland flow), respectively. The Preissmann slot approach (Franz and Melching, 1997) is used here to deal with the flow in pipes

$$\frac{1}{g}\frac{\partial Q_d}{\partial t} = -A_d \frac{\partial H_d}{\partial l_d} - \frac{n_d^2}{A_d}\frac{Q_d|Q_d|}{R^{4/3}} \tag{10}$$

where Q_d is the discharge (m^3/s) in the pipe between adjacent grids; H_d is the average water depth (m) in the pipe; l_d is the pipe length (m); n is the pipe roughness coefficient; A_d is the flow area (m^2) in the pipe; and R is the hydraulic radius (m). The difference representation is

$$Q_i^{T+DT} - Q_i^{T-DT} = -2DT \times g \times A_i \frac{Z_{i2}^T - Z_{i1}^T}{DL_i}$$

$$-2DT \times g \times \frac{n^2 Q|Q|}{A_i^{7/3}} \tag{11}$$

where A_i is the flow area (m^2) in the pipe, and Z_{i1} and Z_{i2} are the water levels (m) in the pipe. When the pipe is full, the flow will become pressure flow, and the continuity equation given as

$$H_{i1}^{T+2DT} = H_{i1}^T + \frac{2DT}{A_{i1}}\sum_{k=1}^{N} Q_{ik}^{T+DT} + 2DTq^{T+DT} \tag{12}$$

where A_{i1} is the bottom area (m) of the pipe. When H_{i1}^{T+2DT} is larger than the pipe diameter, pressure flow will occur; otherwise it is still open channel flow (Figure 4).

Integration of 1-D and 2-D Flows

The stormwater pipes are simplified (Figure 3) and arranged among grids, receiving overland flow from surrounding grids. The ditches and rivers are arranged along the grid sides and receive flow either directly from the grids, or from stormwater pipes. By the flow exchange among grids, pipes and ditches or rivers, the 1-D and 2-D models are integrated. The simplified broad weir flow method is employed to calculate the flow from grid to ditch. The exchange discharge between a grid and stormwater pipe is calculated in the following process. The discharge generated by rainfall for each grid is calculated by the Rational equation. The water volume balance in the pipe is

$$V_j^{T+DT} = V_j^T + DT\left(\sum Q_{di}^{T+DT} + Q_{jl}^{T+DT}\right) \tag{13}$$

where V_j^{T+DT} and V_j^T are the volume (m^3) of water inside the pipe at time step $T+DT$ and T, and Q_{di}^{T+DT} is the discharge (m^3/s) between adjacent pipes. Q_{jl}^{T+DT} is the rainfall generated discharge (m^3/s) from grid j, and has a positive value when the discharge flows into the pipe; otherwise it has a negative value (flooding on grid j since flow cannot get into the pipe). If V_{Mj} as the total volume of the pipe as a function of pipe size is given, V_j^{T+DT} is compared with V_{Mj} to determine whether flow from a grid can reach the stormwater pipe or cause surface flooding. The volume of water that cannot flow into a pipe will be used to determine inundation water depth for the grid. Since the stormwater pipe arrangement is very complex, it is still impractical for the current model to simulate flow at every inlet and each pipe. Several simplified but possible arrangements for stormwater pipes (see Figure 3) are used to account for flow exchange between grid and pipe.

Model Coding and Testing

The model was coded in FORTRAN language. The model was tested by comparing numerical simulations with theoretical results for several case studies, which included overland flows over various sloping planes and flow interactions between overland flows and pipe or channel flows (Su, 2003; Su and Fang, 2003). The model solves unsteady flow equations that are capable of handling reverse flow in channel and pipe systems and overflow from the ditch system into the overland grids. The model simulates surcharge from the pipe system into the overland grids when pipe capacity is insufficient to handle the runoff generated by a heavy thunderstorm (Su, 2003). The computational time step (DT) is very small (1.5 seconds), while simulation results can be reported at specified intervals, e.g., every five or ten minutes; and simulation solutions of governing equations are typically stable due to the small time step (Cheng and Qiu, 2001). With a high speed personal computer and the small time step, even when the model was applied to a city the size of Beaumont, the time required to run the model is reasonable (e.g., 15 minutes for a 12-hour rainfall simulation period).

Figure 5. Location of the Study Area – The City of Beaumont and Surrounding Areas Near the Gulf of Mexico (modified from http://www.mapquest.com, and the map is vertically distorted and not to scale).

THE MODEL APPLICATION FOR THE CITY OF BEAUMONT

Beaumont is an important industrial and cultural city in southeastern Texas. Thirty miles (48 km) north of the Gulf of Mexico (Figure 5), the city lies in a relatively flat area of the coastal plain region, where frequent rainstorms or hurricanes from the Gulf of Mexico generate large volumes of surface stormwater (runoff) during a short period of time. For example, between October 18 and 28, 1994, the study area experienced severe flooding, where approximately 14,000 people were evacuated from their homes and 20 people lost their lives. The total losses were estimated at $700 million (*Beaumont Enterprise*, October 27, 1994). Previously, on June 5, 2001, Tropical Storm Allison made landfall in the Greater Houston and Beaumont areas. The rainfall in Beaumont was as high as 24.6 inches (0.625 m) within 24 hours and over 40.4 inches (1.03 m) in five days. The maximum rainfall intensity was 6.26 in/hour (0.159 m/hr) at 4:00 a.m., June 6, 2001, occurring at water alert gage 5800 of Jefferson County Drainage District 6 (DD6, 2001, unpublished report). About 48 percent of Beaumont is located east of Highway 96 (69) and drains directly into the Neches River. Storm runoff in the western part of Beaumont flows southward, and converges into the Hillebrandt Bayou, which drains approximately 52 percent of the city. Therefore, the Hillebrandt Bayou watershed, containing about 15,000 acres (60.7 km^2), was chosen as the study area for the model application (Figure 6). There are five water alert stations within the study area (2000, 2100, 2200, 2600, and 2700) with rainfall and water level gages set by the DD6 (DD6, 1986, unpublished report) from which data was collected for model calibration and validation.

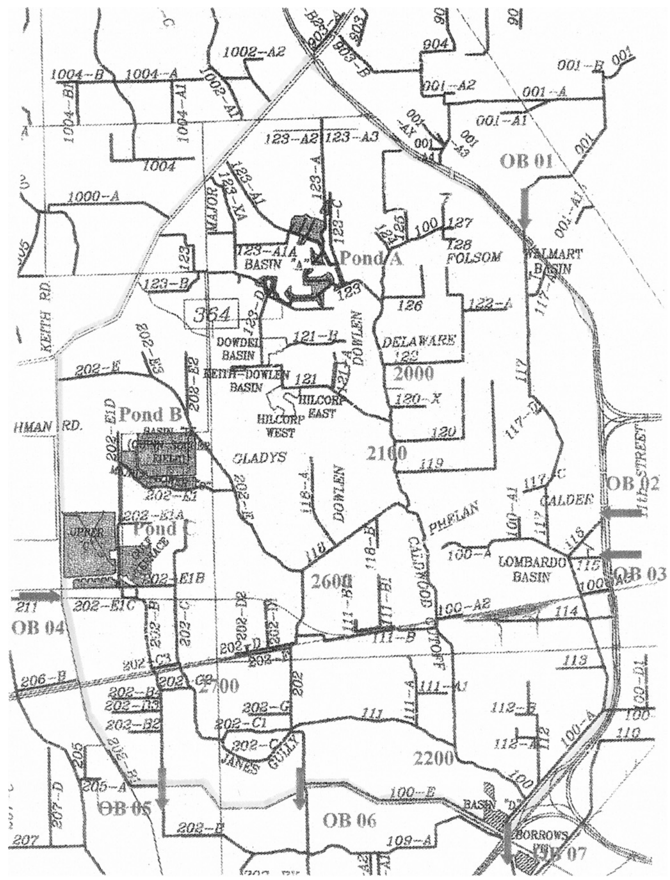

Figure 6. The Study Area Showing the Drainage Ditch System, Detention Ponds, Study Boundary (yellow lines) and Seven Open Boundaries (OB - entrances and exits) and Water Alert Gage Stations (modified from a base map from DD6, 1986, unpublished report).

Grid Division and Data File Preparation for Model Application

The total study area is divided into 786 grids with 794 nodes and 1,578 sides, including five open boundaries and three detention ponds (Figure 6). Details for grid division are given by Su (2003). For example, each ditch in the study area is always taken as one grid side to simulate 1-D flow. There are 268 special grid sides used as drainage ditches or channels. For the total 786 grids, lengths of grid sides are arranged from 300 ft (91 m) to 2,200 ft (670 m), and areas of grids are arranged from 20,000 ft^2 (5 acres, 1,858 m^2) to 2,400,000 ft^2 (55 acres, 223,000 m^2). Detention/retention ponds are the important structures in urban flooding mitigation. There are three existing detention ponds (ponds A, B, and C in Figure 6) in the study area. Detention pond A is composed of eight small ponds and 19 channels with a total excavation volume of 25.6 acre-ft (31,564 m^3), and plan area of 100 acres (0.4 km^2). Detention pond B is a solitary pond with a plan area of approximately 150 acres (0.61 km^2). Detention pond C is separated into two parts (upper C and lower C). The upper C is about 100 acres and lower C is about 140 acres (0.57 km^2). Each of the three detention ponds consists of several grids, and each could have a different bottom elevation and surface area. Each detention pond has its own release structures, but only a simple rectangular weir is used for model simulation since the model does not try to simulate the release of floodwater from a pond that typically occurs days after a rainstorm stops.

The information for each grid, such as average ground elevation, rainfall loss coefficient, and overland roughness coefficient, was collected and summarized. For ditches, the cross-section geometry, bottom elevation, and slope were obtained. For the stormwater collection pipe network, the pipe arrangement, pipe size, slope, inlet, and outlet were obtained from the DD6 (1986, unpublished report), and then simplified and added to the computational grids.

Model Results and Comparison With Recorded Data

The model developed was specifically applied to simulate urban flooding for Beaumont, Texas, and for the Tropical Storm Allison in June 2001. The rainfall data for the model application were obtained from the five water alert stations in the study area. Table 1 gives hourly rainfall depths on June 7, 2001, for the five stations (2000, 2100, 2200, 2600, and 2700). Thiessen polygons were developed for each station (Su, 2003), and each polygon gave unique rainfall hyetographs with a time interval of one hour for model simulation. The rainfall data starting from 1:00 a.m. on June 7, 2001, were used for a 24-hour simulation. Each of the 786 grids was designated as one of three land use types: residential, agriculture, or commercial, which included paved streets and parking lots. The runoff coefficients were 0.4, 0.4, and 0.7, and were based on a storm sewer inventory developed in November 1980 (Kohler and Kohler Engineers and Surveys, 1981, unpublished report). Overland flow roughness coefficients for grids are 0.08, 0.06, and 0.05 for residential areas, agriculture and undeveloped areas, and commercial areas, respectively. For drainage ditches with consideration of head losses through bridges and culverts, Manning's coefficient was calibrated as 0.035. For open boundary entrances 01 to 05 in Figure 6, the contributing watershed area for each ditch (outside of the study area) was used to estimate input discharge by the Rational equation.

The model includes a graphic display of simulated water depths for all 786 grids as well as changes of water levels over time at the five water alert stations. The water depth for each grid in the study area is shown in different colors. During Tropical Storm Allison, on June 7, 2001, severe rainfall lasted eight hours, with its maximum intensity at 6:00 a.m. Simulation results indicated that water depths reached the maximum levels at approximately 7:00 a.m. Figure 7 shows an example of the simulated water depth for each grid at 7:00 a.m. on June 7, 2001, and the changes in water level over a period of time for the five water alert stations. The simulated water depths are arranged from 0.1 ft) (0.03 m) to 5.0 ft (1.52 m). One can see that some grids of the study area had water depths between 0.5 and 1.0 feet (0.15 and 0.30 m), with some depths exceeding 1.0 ft (0.30 m). Flooding depths greater than 0.5 ft (0.15 m) could cause serious property damage. As seen in Figure 7, the water depths at three detention ponds on the west end of Beaumont increased quickly, reaching up to three feet (0.91 m) or more during the peak flooding (Su, 2003).

Figure 8 shows the simulated water levels were in agreement with the recorded data for four ditch gages. The average absolute deviation between simulated and recorded water levels ranged from 0.21 to 0.38 m for the above gages, and standard deviation from the average ranged from 0.14 to 0.30 m. The model can display the change of water depth at any grid during the flood for flood risk information (e.g., Figure 9 traces the simulated water depth change during flooding at Gladys Avenue). The photograph in Figure 9 provides a qualitative/quantitative comparison of the extent of flooding near Gladys Avenue during peak flooding produced by Tropical Storm Allison, and shows that flood depth is about 10 inches (0.25 m)

TABLE 1. Rainfall (in/hr) on June 7, 2001, for Six Water Alert Stations.

Time	2000	2100	2200	2600	2700	5800
1:00 a.m.	0.59	0.59	0.16	0.59	0.51	0.67
2:00 a.m.	1.14	0.51	0.04	0.31	0.24	0.43
3:00 a.m.	0.83	0.31	0.00	0.12	0.28	3.50
4:00 a.m.	1.26	1.02	0.67	1.22	1.22	6.26
5:00 a.m.	0.94	0.35	0.83	0.75	0.91	3.31
6:00 a.m.	3.70	1.46	1.42	1.69	1.57	4.88
7:00 a.m.	3.11	2.13	1.42	2.60	3.03	5.36
8:00 a.m.	0.28	0.39	1.38	0.98	1.02	0.02
9:00 a.m.	0.04	0.00	0.04	0.00	0.04	0.03
Total	11.89	6.76	5.96	8.26	8.82	24.46

Note: Stations 2000, 2100, and 2200 are located in drainage ditch 100 and intersected at Folsom Road, Gladys Avenue, and Washington Boulevard, respectively. Stations 2600 and 2700 are located at ditch 2002 and intersected at Prutzman Road and Landis Road. Above five stations are given in Figure 6. Station 5800 is at the Moore Detention Pond about six miles west of the study area.

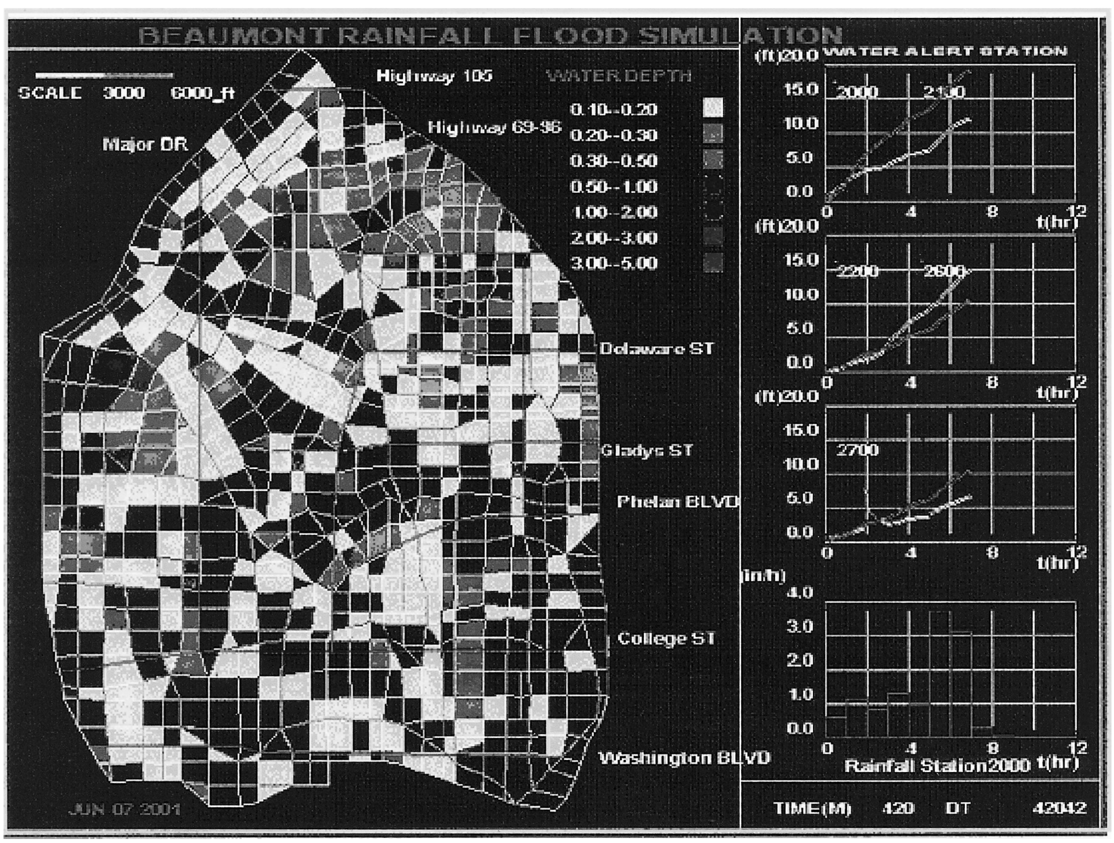

Figure 7. Simulated Water Depth for Each Grid at 7:00 a.m. on June 7, 2001, and Change in Water Level for Five Water Alert Stations (screen display from the model).

based on the submerged wheels of the vehicle in the photograph. The simulated water depth is 0.365 ft (0.11 m) in the simulation grid at 11:30 a.m. (Figure 9). Since a street within a grid is typically 6 inches (0.15 m) lower than the surroundings, simulated water depth on Gladys Avenue is 10.4 inches (0.26 m), which is about the same as the water depth in the photograph. Many other photographs that were taken

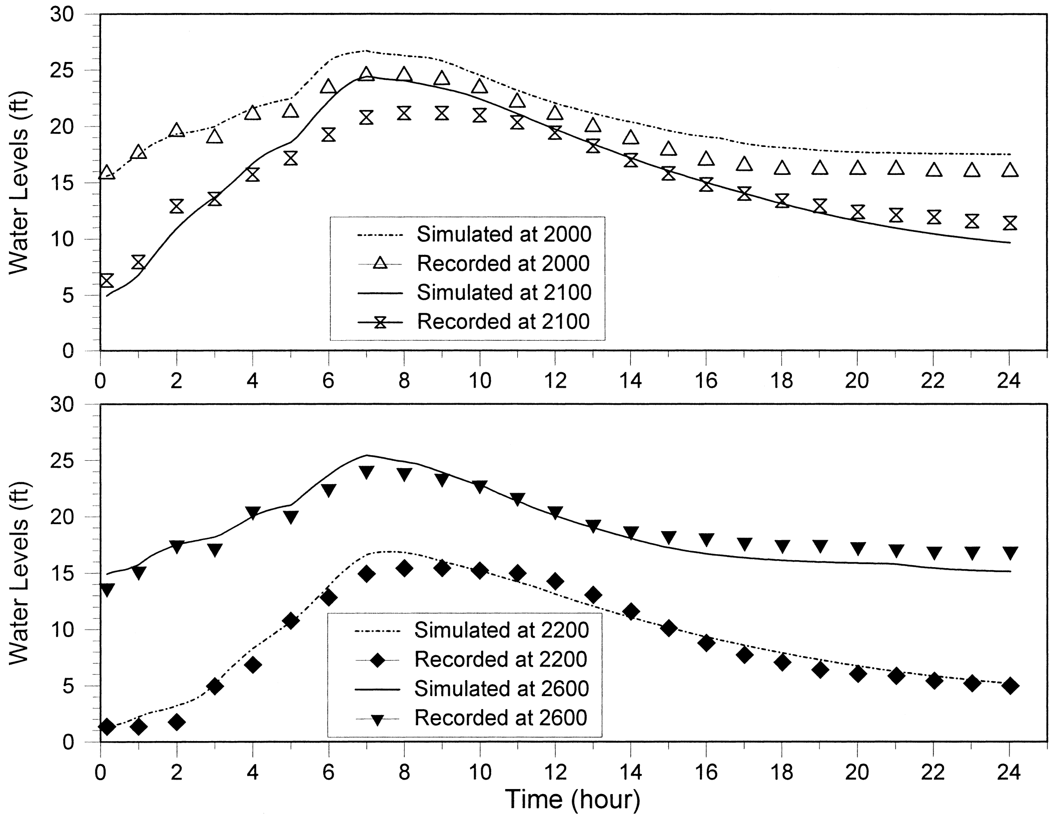

Figure 8. Simulated and Recorded Water Levels at Four Water Alert Stations on Drainage Ditches.

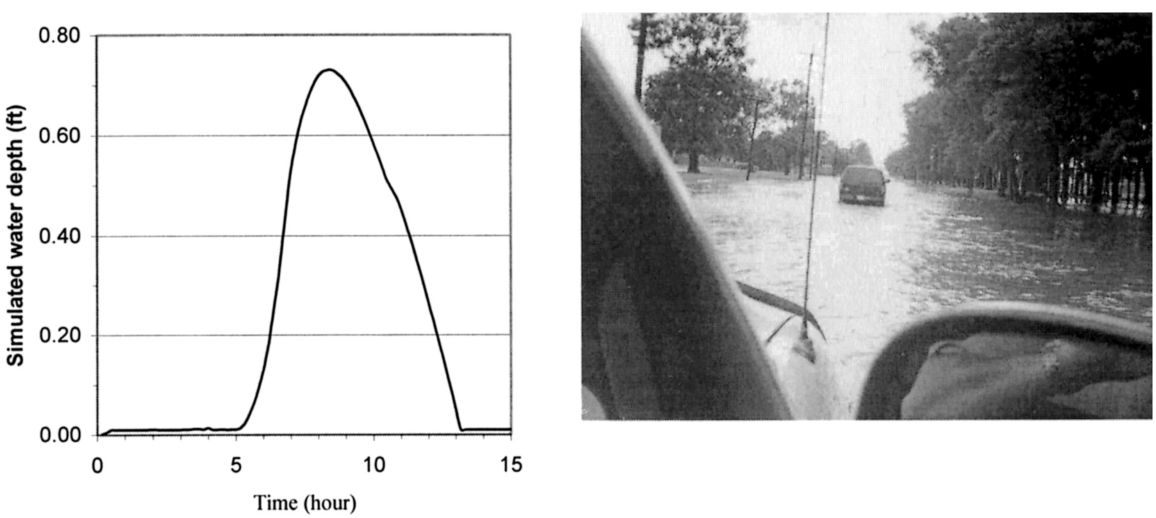

Figure 9. Simulated Change in Water Depth During Flooding and Recorded Flooding at Gladys Avenue Near Marshall Middle School (photograph provided by DD6 Engineer Mr. Doug Canant and taken westwards about 11:00 a.m. on June 7, 2001).

and provided by staff members in the DD6, were used for comparison, and showed a good qualitative agreement between model simulation and observation. One of the advantages of this model is that in addition to floodwater depth, the model can offer other flood risk information, such as inundation time and flow velocity. This additional information is useful for flood loss evaluation as well as evacuation planning. If 0.5 ft (0.15 m) of water depth is used as the flooding standard, the model begins to count and accumulate the time when water depth is over 0.5 ft (0.15 m). Therefore, an inundation time map can be developed for the study area (Su, 2003).

The model was also applied to simulate flooding due to Tropical Storm Kenna on October 28, 2002 (Su, 2003). The rainfall depth during this storm was over six inches (0.15 m) within two hours with rainfall intensity over 4 in/hr (0.10 m/hr) and caused one death in the study area. At the suggestion of DD6, the rainfall data from Kenna was used for the model application to study or check performance for a planned detention pond in mitigation of floods. The model results show that the proposed detention pond could lower the water level 0.7 ft (0.21 m) at alert water station 2100. For further sensitivity analysis, two hypothetical storms were used to project what degree of flooding could be developed in the west end of Beaumont. Detailed results of that study are presented elsewhere (Su, 2003). During Tropical Storm Allison, the most severe rainfall was at gage station 5800, about six miles (9.7 km) west of the City of Beaumont. The eight-hour actual rainfall depth was 24.57 inches (0.62 m) with maximum intensity of 6.3 in/hr (0.16 m/hr) (Table 1), which is quite close to the six-hour probable maximum precipitation of 32 inches (0.81 m) in the Beaumont area. It was found that much more area of west Beaumont could be flooded if the rainfall event at station 5800 was assumed to occur in the city (Su, 2003).

SUMMARY AND CONCLUSIONS

An integrated 1-D and 2-D hydraulic model has been developed and it simulates the complex flood forming and developing processes over the land surfaces and in the ditches/ channels/streams in an urban area. From the model application, the following results and conclusions can be drawn: (1). integration of 1-D and 2-D methods was demonstrated in this study for modeling rainfall produced floods and the average absolute deviation between simulated and recorded water levels ranges from 0.21 m to 0.38 m for four gages in the model application; (2) the irregular grids simplified the representation of the complex topography and drainage systems in an urban area – regular grids would have required far greater density to adapt to unusual geometries and shapes; (3) integration of the stormwater pipe system with the surface flows was demonstrated in this study; and (4) the simulation results offered detailed spatial flood risk information.

When the model was used for a feasibility study of a proposed detention pond, the model results indicated that the proposed detention pond could lower the water level 0.7 ft (0.21 m) at alert water station 2100. Simulation of stormwater flooding with an integrated model still needs improvement, especially in the automation of data collection and preprocessing of data for model preparation, such as grid division, coding, and input data files. These difficulties could be overcome with the development of a special geographic information system model to preprocess data. Also, the 1-D methods need to be improved for handling natural rivers with complex cross sections and simulating flow through bridges and culverts. The development of user friendly software for the model with easy application in flood risk mapping will be the next research goal.

ACKNOWLEDGMENTS

The authors wish to thank District Engineer Doug Canant, P.E., from DD6 for providing data for the model application in Beaumont, *JAWRA* reviewers for valuable comments, and editorial help from Linda Dousay at the Department of Civil Engineering at Lamar University. The second author acknowledges the Doctoral Scholarship provided to him by the College of Engineering at Lamar University. The study was partially supported by the Lamar University Scholar Award given to Dr. Xing Fang.

LITERATURE CITED

Bedient, B.P. and A.W. Holder, 2001. Summary of the June 8-9 Flood Over Brays Bayou and the Texas Medical Center. *Available at* http://www.floodalert.org/BraysFAS/index.php?sPageID=CaseStudiesAllison&sRadar=KHGX. *Accessed in* March 2006.

Cheng, X.T. and J.W. Qiu, 2001. Development of Urban Flood Simulation Techniques in China. *In:* Theme C, Forecasting and Mitigation of Water-Related Disasters, XXIX International Association of Hydraulic Research Congress Proceedings, Tsinghua University Press, Beijing, China, pp. 393-402.

Chow, V.T. and A.Z. Ben, 1973. Hydrodynamic Modeling of Two-Dimensional Watershed Flow. Journal of Hydraulics Division, ASCE 99(11):2023-2040.

Clark, C.O., 1945. Storage and the Unit Hydrograph. ASCE Transactions, 110:1419-1446.

Espey, W.H., Jr., C.W. Morgan, and F.D. Masch, 1966. A Study of Some Effects of Urbanization on Storm Runoff From a Small Watershed. Report No. 3, Texas Water Development Board, Austin, Texas.

Franz, D.D. and C.S. Melching, 1997. Full Equation Utilities (FEQUTL) Model for the Approximation of Hydraulic Characteristics of Open Channels and Control Structures During Unsteady Flow. Water Resources Investigation Report 97-4037, U.S. Geological Survey, Urbana, Illinois.

Hromadka, T.V., R.H. McCuen, and C.C. Yen, 1987. Comparison of Overland Flow Hydrograph Models. Journal of Hydraulic Engineering, ASCE 113(11):1422-1439.

Iwasa, Y. and K. Inoue, 1980. Numerical Method for Inundating Flood. Annals of Disasters Prevention Research Institute of Kyoto University, No.23, B-2, Kyoto, Japan.

Jha, A.K., J. Akiyama, and M. Ura, 2000. Flux-Difference Splitting Schemes for 2D Flood Flows. Journal of Hydraulic Engineering, ASCE 126(1):33-42.

Katopodes, N. and T. Strelkoff, 1978. Computing Two-Dimensional Dam-Break Flood Waves. Journal of Hydraulic Engineering, ASCE 104(9):1269-1288.

Laguzzi, M.M., G.S. Stelling, and K. Brujin, 2001. A Fully Coupled 1-D and 2-D System Specially Suited for Flooding Simulation. *In:* Theme C, Forecasting and Mitigation of Water-Related Disasters, XXIX International Association of Hydraulic Research Congress Proceedings, Tsinghua University Press, Beijing, China, pp. 36-40.

Ligget, J.A., 1994. Fluid Mechanics. McGraw-Hill, Inc., New York, New York.

McCuen, R.H., 1998. Hydrologic Analysis and Design (Second Edition). Prentice Hall, Englewood Cliffs, New Jersey.

Roberson, J.A., J.J. Cassidy, and M.H. Chaudhry, 1998. Hydraulic Engineering (2nd edition). John Wiley and Sons, Inc. New York, New York.

Su, D.H., 2003. Development of an Integrated One- and Two-Dimensional, Numerical, Urban Rainfall-Flood Simulation Model With Its Applications. Doctoral Dissertation, Department of Civil Engineering, Lamar University, Beaumont, Texas.

Su, D.H. and X. Fang, 2003. Estimating Traveling Time of Flat Terrain by 2-Dimensional Overland Flow Model. The International Symposium on Shallow Flows, Delft University of Technology, The Netherlands, June 16-18, 2003. Delft University of Technology, Delft, The Netherlands, Part II, pp. 287-293.

Toda K., K. Inoue, K. Kuriyama, and O. Maeda, 2001. Inundation Flow Analysis in Urban Areas Considering Streets and Underground Space Effects. *In:* Theme C. Forecasting and Mitigation of Water-Related Disasters. XXXIX International Association of Hydraulic Research Congress Proceedings, Tsinghua University Press, Beijing, China, pp. 416-423.

USACE (U.S. Army Corps of Engineers), 1981 (Revision in 1987). HEC-1 Flood Hydrograph Package, User's Manual. U.S. Army Crops of Engineers, Hydrologic Engineering Center, Davis, California.

USACE (U.S. Army Corps of Engineers), 2000. HEC-HMS Hydrologic Modeling System, User's Manual (Version 2.0). U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California.

USDA-SCS (U.S. Department of Agriculture-Soil Conservation Service), 1972. National Engineering Handbook, Section 4, Hydrology. U.S. Department of Agriculture, Washington D.C.