International Conference "Water, Environment, Energy and Society" (WEES–2009)
New Delhi, 12–16 January 2009

Modeling Urban Flooding from Storm Sewers Using Dynamic Wave Theory

Biswajit Mukhopadhyay[1], Aniruddha Dutta and Fariborz Nouri
Hydrology and Hydraulics Group, Infrastructure Division
Walter P. Moore & Associates, Inc.
3131 Eastside Street, Houston, Texas 77098, USA
E-mail: [1]biswajitm@sbcglobal.net

Chitranjan Kaushik
Luminous Engineering & Technology Services Pvt. Ltd.
43 Community Center, Naraina Industrial Area, Phase-I
New Delhi - 110 028, INDIA
E-mail: chitranjank@luminous.com

ABSTRACT: Traditionally, urban storm sewers, meant to convey excess runoff from impervious covers efficiently, are designed with the applications of rational formula of hydrology with all of the conceptual limitations of the method. Typically the system is sized such that the hydraulic grade line remains at or below ground surface and thereby the design means to provide a sense of safety. However, this method does not offer the opportunity to evaluate any potential of overland flooding from inadequacy of the system particularly where surcharge occurs by tail water condition. In contrast, hydraulic models utilizing dynamic wave theory, of newly designed or existing systems offer several advantages in this regard. With these models, the overland flow components can be incorporated and hence the degree of surface flooding can be more reasonably assessed. Second, the time dependence of the downstream boundary condition normal during a storm event can be accounted for. Temporal variations of both rainfall and water surface elevation at the outfall control the volume available in the network elements and hence the hydraulics of the system. Most systems which are designed by rational method when modeled with the applications of dynamic wave theory seem to be prone to produce overland flooding. The primary reason for this is the fact that with rational method the entire volume of the system is assumed to be available for peak design flow. However, during an unsteady state flow condition, from both theoretical and practical standpoint, a significant amount of storage of the network system is unavailable to a peak discharge. A few case studies are presented to emphasize the importance of evaluation of storm sewer systems with dynamic wave modeling to assess and reduce the risk of urban flooding. However, differences in results are obtained when commonly available software are used for such purposes. The differences stem from the variations in the computational algorithm used in these software. This poses problems not only in interpretation of the results but also in applying those in forming opinion about improvements of the systems that appear to be inadequate in mitigation of urban flooding.

INTRODUCTION

From the beginning of urbanization, such as the days of Indus Valley civilization (~2000 B.C.), one of the key issues civil engineers constantly face is the proper design of urban drainage systems to convey the storm water falling on the developed areas efficiently so that life is not disrupted from overland flooding. The most obvious hydrologic consequence of urbanization of a watershed or a part of it is the increase in runoff rate and volume. The peak of a runoff hydrograph for any storm event increases due to increase in the impervious cover of the land under consideration. The principal purpose of a storm sewer system, which is usually a network of underground pipes, is to carry this excess runoff in an urbanized area so that overland flooding is prevented. Delleur (2003) has discussed the evolution of storm sewer systems since antiquity of the Indus Valley, Mesopotamian, and Greek civilizations to the modern systems such as the storm sewer system of Paris.

Most of the twentieth century storm sewer design in major urban areas relied on methods that would provide appropriate sizing of the storm sewers (pipes). The basic purpose of proper design of urban storm sewers or storm water drainage system is to remove the excess runoff from urbanized areas as fast as possible so that potential of overland ponding is reduced and the negative influence on transportation

[1]Conference speaker

and structures is minimized. For this, traditionally and typically, urban storm sewers are designed by the application of rational formula. Although application of rational formula or simply 'rational method' in the design of an urban storm sewer system is a very simplistic approach, it is still widely used even in some urbanized areas where drainage and flooding issues are serious and complex. However, with the rational method the impact of the design on urban flooding can not be properly ascertained due to three serious limitations that are inherent in the method. The first limitation of the rational method is that with this method the overland flooding depths cannot be calculated when the storm sewers are surcharged above ground levels. The second one is that the method ignores the storage within the storm sewers and hence suffers from both over and under estimation of the pipe sizes. The third limitation is that with this method a fixed downstream water surface elevation is used for the calculation of the Hydraulic Grade Line (HGL) of the system. In reality, the downstream boundary is a time-dependent phenomenon which in many cases controls the surcharge condition of the network system.

The underlying theme, taking preponderance over other considerations in the design of a modern storm sewer system in a highly urbanized watershed, is to make the system efficient in removing runoff from developed lands to systems established to mitigate any potential downstream impact as quickly as possible to minimize localized flooding. For this matter, typical and traditional way of designing storm sewer systems using rational formula is giving place to models that use the conservative form of Saint Venant Equations for analysis and design of pipe flows (e.g. see Schmitt *et al.*, 2004). Incorporation of overland flow components and time dependent boundary conditions in modeling urban storm sewer systems are two critical elements in the applications of such models. These factors until now have received less emphasis in the literature as well as in practical design dealing with urban flooding. The general form of the momentum equation (the dynamic wave equation) considers local inertial, convective inertial, pressure, gravity, and frictional forces that act upon a fluid in motion, offering the advantage of coupling pipe flow with overland flow. This allows a more accurate assessment of overland flooding when the pipes are surcharged. When a storm sewer system becomes pressurized and the hydraulic grade line rises above the elevation of the inlets, overland ponding begins. If the overland flow paths are not accounted in the model, the calculations will not give the accurate picture of the flow and overland flooding conditions.

Urban flooding often is not caused by over bank topping of the streams receiving the storm sewer outfalls, but by the routing of overland drainage where sufficient analysis has not been provided to ensure that the flow path is adequate for the intended sheet flow. In other words, the downstream boundary conditions of the storm sewer systems control the surcharge conditions of the main trunk and the collectors, forcing the surface system to either store (flood) the excess, or provide an overland route to the receiving stream. For this reason, the time dependent downstream boundary conditions should also be incorporated in the models using the general form of dynamic wave equation. Thus, overland routing of runoff due to inadequacy of the storm sewer systems, but not due to bank overtopping of the receiving streams, can be accurately analyzed.

The objective of this paper is to show that (1) potential urban flooding from an existing storm sewer system, originally designed using rational method, can be evaluated by modeling the system using dynamic wave theory and this assessment typically leads to show that a system designed by rational method requires improvements for reduction of potential overland flooding from surcharged conditions of the system; (2) new systems designed using rational method should be modeled and if necessary re-designed in the same way to ensure its adequacy in reducing overland flooding potential. The basic theories of both the methods are reviewed. A case study is presented to illustrate the effect of urbanization on the volume of storm water runoff to emphasize the importance of proper design of storm sewers with the growth of urbanization.

HYDROLOGICAL CONSEQUENCE OF URBANIZATION

The consequence of urbanization on flood frequency is not well understood. Urbanization may very well cause a stream flow under pre-development period with p_1 Annual Exeedance Probability (AEP) to become a flow with p_2 AEP during post development period, where $p_2 > p_1$. However, the increase in runoff volumes and the lower return periods of flow due to urbanization can only be detected by a statistical analysis of stream flow records of pre-urbanized and post-urbanized conditions. In many instances, such data may not exist and in those cases detection of alteration of flow regime is not subtle. For example, the effect of urbanization on flood frequency was well illustrated by Novotny *et al.* (2001) with a study of a 36.7 km^2 watershed located in central Wisconsin. In 1960, the watershed was a rural mix of agricultural and forested lands, in 1985 the watershed was 20–25%

urbanized, and in 1988, the watershed was 40% urbanized with the dominant land use being residential. Novotny *et al.* (2001) showed that the 100 year (0.01 AEP) flood in the predevelopment stage corresponds to the 10 year (0.10 AEP) high flow in the 1985 condition, the 3- to 4- year high flow in the 1988 condition, and would become the annual high flow when the watershed is fully developed.

There is no straightforward method for evaluation of the changes in the form of flood frequency distribution of an area caused by urbanization (Hall, 1984). Furthermore, urbanization usually occurs gradually over a period, and for this reason, it is difficult to separate pre-development and post-development records as two distinct data sets. Thus, unless urbanization takes place within a relatively short period, establishment of the nature of flood frequency distribution for a watershed under its standard state requires certain manipulation of the data. Various workers developed different techniques of estimation of mean annual flood and changes in the magnitude of higher return period floods for urbanizing watershed (e.g. Carter, 1961; Martens, 1968; Anderson, 1970; Espey and Winslow, 1974; Hollis, 1975, etc). These works attempted to show that with urbanization, the flood frequency distribution of a watershed changes.

To illustrate the effects of urbanization on storm water runoff volume we have examined historical flow records of Buffalo Bayou, Harris County, Texas. The City of Houston, the forth largest metropolis in the United States, is located within Harris County which along with the surrounding counties, have an intricate network of bayous due to their proximity to the Gulf of Mexico. The outfalls of the city's storm sewer systems are on these bayous and their tributaries. Thus, the bayous within Harris and surrounding counties basically act as open drains to receive the flows form storm sewers. Consequently, the historical records of flows through these bayous provide an excellent opportunity to assess the effect of urbanization on generation of excess storm water runoff. It should be noted at this point that these bayous do not have well defined sources, their headwaters are mostly located within the city or county limits. In other words, these are not like some major rivers which flow through certain urbanized areas but have their sources in some remote locations far away from the cities and thereby pose a far more complex problem for detection of effects of urbanization on their flows.

Figure 1 shows the three major watersheds that cover most of the City of Houston. These three watersheds are drained by three major bayous. Of these, we have examined the historical flow records along Buffalo Bayou whose watershed encompasses the downtown area and other major urbanized centers. The gauging stations, maintained by the United States Geological Survey (USGS), from where flow data are obtained are also shown in Figure 1.

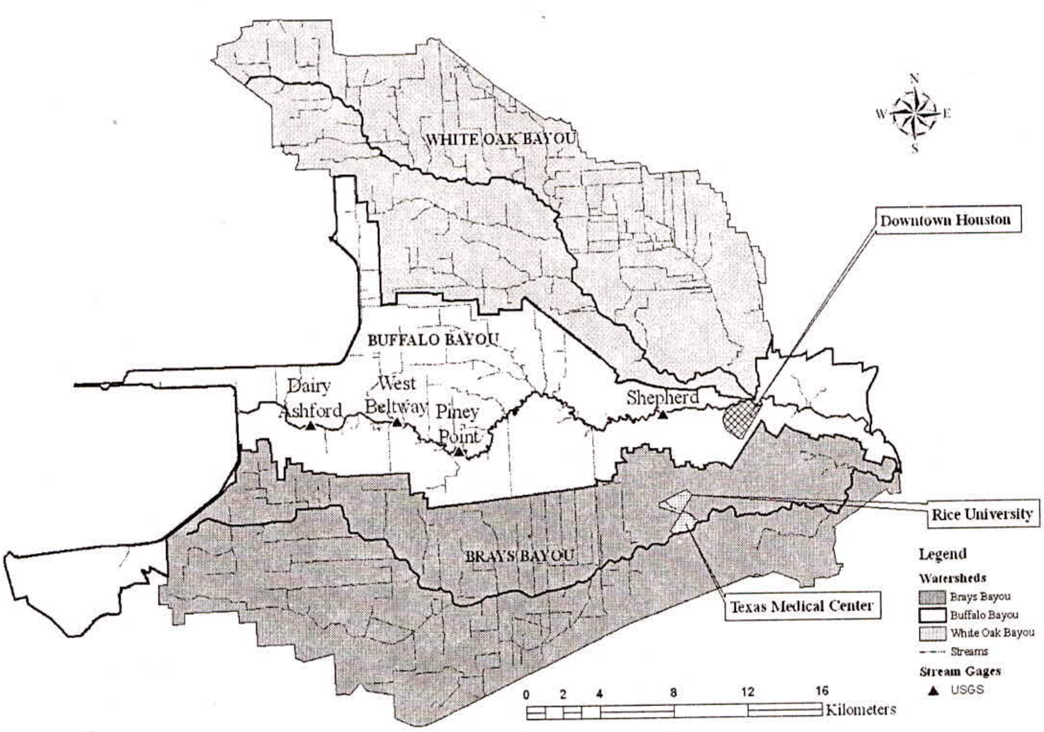

Fig. 1: Three highly urbanized watersheds in Houston area, Texas

Mukhopadhyay *et al.* (in press) determined the flood warning stages at these gauging stations for the design frequency storms with 0.01 and 0.02 AEP. These warning stages are not necessarily the bank elevations, since in an urban area, flooding often occurs due to the tail water effect on the outlets of various storm sewers and minor tributaries that outfall to the subject stream. The surcharge in these drains due to the backwater effects of the receiving stream can cause flooding much earlier than when overtopping occurs in the major channel. Thus, these warning stages were established based on hydrologic and hydraulic modeling of the Buffalo Bayou watershed and statistical analysis of stream gage and stage data. The discharge hydrographs and the rating curves at these gauging stations are presented in Figures 2 and 3 respectively. Form the rating curves, the flow values corresponding to the warning stages are calculated and are marked on Figure 2. Table 1 presents the number of occurrences of flows exceeding these warning stages, called Potentially Flood Producing Flows (PFPF), for the first and second halves of the period of record.

Since there are gaps in the time series records, the numbers of occurrences are translated to percentages (Table 2).

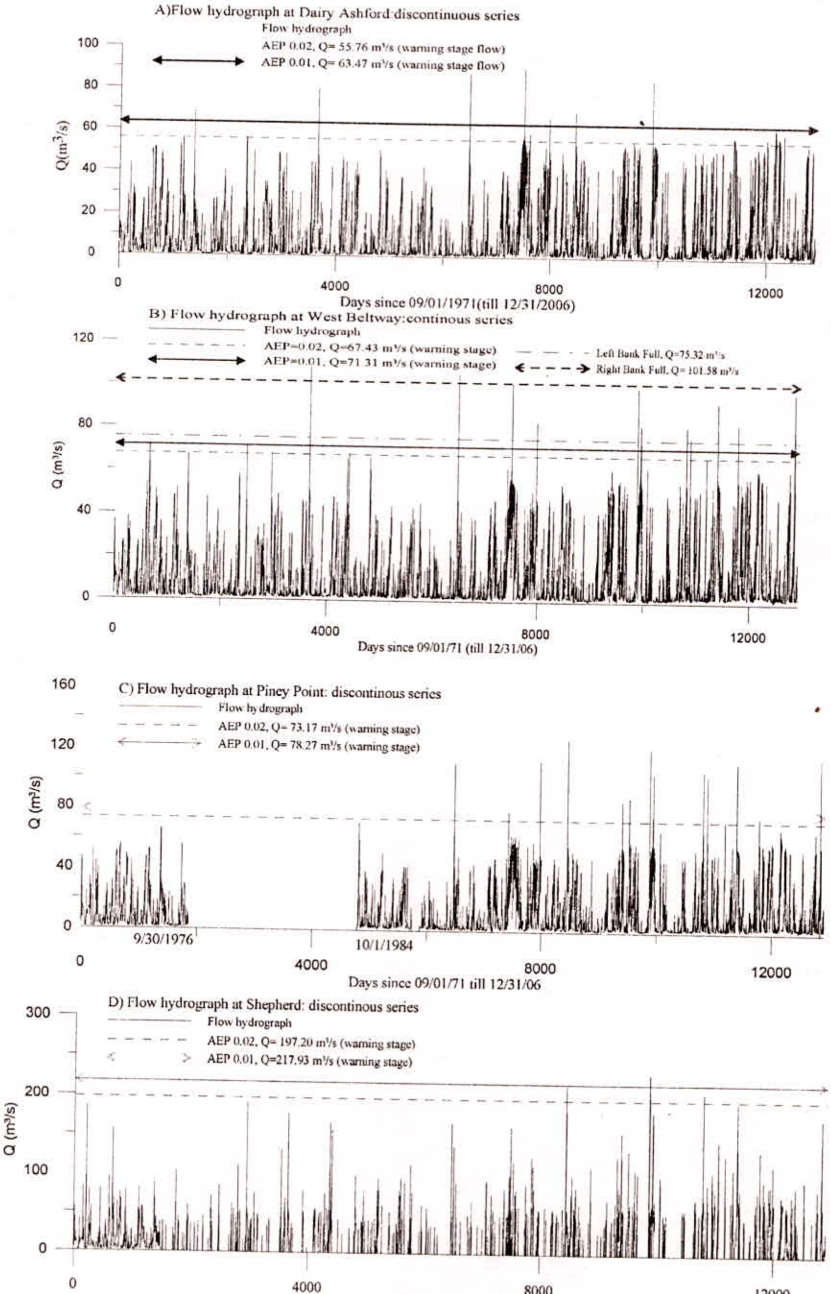

Fig. 2: Stream flow hydrographs at four gauging stations along Buffalo Bayou (see Figure 1)

Table 1: Number of Occurrences of Flood Warning Flows along Buffalo Bayou

Stations	1971–1989		1990–2006		PFPF (m^3/s)	
	0.02 AEP	0.01 AEP	0.02 AEP	0.01 AEP	0.02 AEP	0.01 AEP
Dairy Ashford	3	3	57	5	55.76	63.47
West Beltway	1	1	1	13	67.43	71.31
Piney Point	1	2	1	15	73.17	78.27
Shepherd	0	0	4	2	197.2	217.93

Table 2: Percentages of Flood Warning Flows along Buffalo Bayou

Stations	*1971–1989*		*1990–2006*		*PFPF (m^3/s)*	
	0.02 AEP	*0.01 AEP*	*0.02 AEP*	*0.01 AEP*	*0.02 AEP*	*0.01 AEP*
Dairy Ashford	0.023%	0.023%	0.442%	0.039%	55.76	63.47
West Beltway	0.008%	0.008%	0.008%	0.101%	67.43	71.31
Piney Point	0.008%	0.015%	0.008%	0.116%	73.17	78.27
Shepherd	0.000%	0.000%	0.031%	0.015%	197.2	217.93

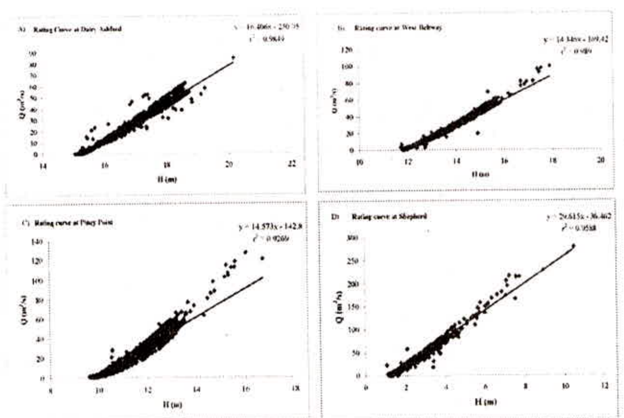

Fig. 3: Rating curves of the four gauging stations shown in Figure 2

The simple quasi-statistical analysis presented above, without recourse to elaborate time series analysis, shows that in the Buffalo Bayou watershed, which has been experiencing continued urbanization indeed there has been an increase in PFPF since 1990. These PFPF are generated primarily from storm water runoff. This demonstration signifies the importance of the proper design and assessment of storm sewer systems in major urbanized areas. In one hand there is increasing concerns amongst engineers and planners about urban flooding. On the other hand, departure form traditional way of designing storm sewers using rational method is taking at a slower pace.

TRADITIONAL METHOD OF STORM SEWER DESIGN

The rational method is a simple design tool, not an analytical method. The method using rational formula in the design of a storm sewer system is a steady state approach. Hall (1984) points out that the rational method does not take into account variations of the followings with time: (1) The rate of change of rainfall intensity (2) Flow velocity and discharge (3) Temporary storage in the sewer systems (4) The rate of increase in the area contributing to the sewer system. Most of the design methods for storm sewers, involving rational formula, have attempted to address the last item through a concept by which each sewer in a network is designed individually and independently, except the computation of sewer flow time for the purpose of rainfall duration (to calculate rainfall intensity) for the next sewer. The time of concentration for each of the flow paths is the sum of the inlet time plus the upstream sewer flow time. This concept can be seen form an example of a storm sewer system serving an urban residential-commercial area (Figure 4).

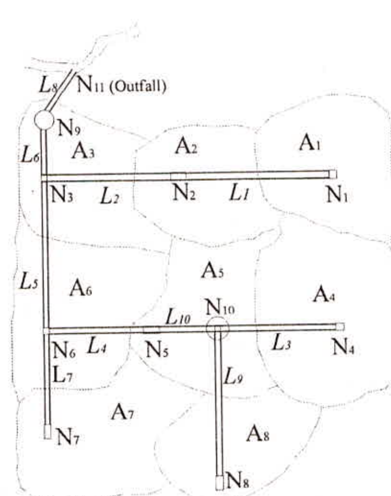

Fig. 4: A hypothetical storm sewer system serving eight catchments. Node N_{10} is a manhole and Node N_9 is a junction box. All other nodes represent inlets

The drainage area of interest is divided into sub-areas or catchments that are small enough for the contributing area to be directly proportional to time during the time of concentration. Figure 4 shows two such sub-areas served by a storm sewer pipes L_1 and L_2. The time of concentration for sub-area A_1 is TC_1 and this area contributes to the storm sewer through inlet, N_1. Sub-area A_2 has time of concentration TC_2 and runoff from this area is captured through inlet, N_2. The travel time from N_1 to N_2 is TT_1. Until time TT_1, only area A_1 contributes to the flow. By time TC_1, area A_1 contributes fully. Area A_2 begins to contribute from TT_1. Up to time TC_1, the total area that contributes to the flow is given by $A_1 + A_2(TC_1 - TT_1)/TC_2$. At time $TC_2 + TT_1$, which is the time of concentration of the composite area $A_1 + A_2$, the entire area upstream of L_2 contributes to the flow. This illustration shows how the contributing area increases with time according to rational method. From this it can be seen that when rational formula is applied to the design of urban drainage systems, the time of concentration is estimated from the sum of the time of flow in the sewer pipe and a time of entry. The time of flow is computed with the assumption that for the given discharge based on a particular design storm frequency, the pipe flows full and the velocity remains constant throughout the run of the pipe. The time of entry is an allowance for the time taken for water from the most remote point in the sub-area to the inlet serving that sub-area.

The time-area concept enumerated above, when is applied to the design of a storm sewer system the design of the pipes is carried out from the most upstream sub-area. In Figure 4, pipe L_1 will carry the flow Q_1 which is given by $Q_1 = C_1 i_1 A_1$. In this case i_1 is a function of TC_1. But the pipe downstream of N_2 must have the capacity to convey the flow from both sub-areas. This flow designated by Q_2, will be calculated on the basis of the total area ($A_1 + A_2$) and time of concentration which is the sum of $TC_2 + TT_1$. Thus, for each sewer, the design peak discharge is calculated using the rational formula given by Eqn. 1 (Yen and Akan, 1999),

$$Q_P = i_k \sum_{j=1}^{n} C_j A_j \qquad \ldots (1)$$

where, i_k is the design rainfall intensity of the kth sewer in a network, C is the runoff coefficient, and A is the area. The subscript j represents the jth sub-area upstream to be served by the kth sewer line. The summation sign implies inclusion of all sub-areas upstream of the sewer being designed. Each sewer has its own design i because each sewer has its own flow time of concentration and design storm. The only information needed from upstream sewers for the design of a current sewer is the upstream flow time for the determination of the time of concentration.

Once the peak discharge is computed for a sewer, its size can be computed from application of Manning's equation with the assumption that the pipe flows full for the design flow. For a circular pipe, the minimum required diameter is given by Eqn. 2,

$$D_r = \left[3.208 \frac{n}{k_n} \frac{Q_P}{\sqrt{S_0}}\right]^{\frac{3}{8}} \qquad \ldots (2)$$

where, k_n = 1 for SI unit and 1.486 for English unit. Pipes with other geometrical cross sections can also be derived from this equation by taking the equivalent conveyance with different shapes. The key element to note with the procedures summarized above is that the method completely lacks any consideration for the flow through a network system and its storage capacity (the method assumes that the entire volume of the system is available for peak flow and is independent of time). This fundamental drawback of the method makes it inappropriate as a tool for assessment or modeling of urban flooding. This point is further elaborated in the following section. Furthermore, with rational method no runoff hydrograph is routed through the network and the concept that peak flow occurs at the time of concentration is not supported by a hydrological basis.

URBAN FLOODING AND RATIONAL METHOD

Apart from the conceptual problems of rational method noted above, there is another serious limitation with this method. This limitation is directly related to its applicability in the evaluation of potential of urban flooding from either an existing or a proposed storm sewer system. Usually the adequacy of the design of a storm sewer system using rational method is determined through calculation of the HGL of the system. For this purpose, a Water Surface Elevation (WSE) at the outfall is assumed. From the assumed or given WSE at the downstream boundary of the system, the WSE at various junctions or nodes (e.g., inlets or manholes) upstream are calculated using standard step method or energy balance calculations. In this process, the size of the pipe calculated from Eqn. 2 or its equivalent, is used to calculate the head loss through the pipe and if the HGL is determined to be above a permissible level (usually a set elevation above or below the ground elevation at this node) then the size of the pipe is considered inadequate and a greater size is used until the calculated HGL remains at or below the acceptable level. The fallacy of this approach is this. Suppose the ground elevation at a node N_i is GE_i and the calculated HGL at the same node is HG_i and the calculations yield $HG_i > GE_i$. The calculation assumes that the pressure head at this point can be built up to the level of HG_i. But in reality, as soon as WSE reaches GE_i, the water from the sewer pipe through the node is poured overland and goes through an open channel flow unless the node is pressure-sealed. Thus, the pressure head usually cannot be available to the pipe size used in the calculation of the HGL. Even in case of standard manholes, the force created by the pressure head can exceed the weight force of the manhole cover and can let the water spread over the surface by displacing the manhole cover. Furthermore, when this happens, the total discharge (Q) used for the computation is no longer available to the pipe. The total discharge is divided into two components, namely the overland discharge (Q_o) and pipe flow (Q_p), giving $Q = Q_o + Q_p$. Standard computer programs or software that use rational method for the design of storm sewers lack the capability of solving parallel paths for flow which results when both the underground and the overland segment connect two consecutive nodes. In these situations, application of rational method ends in either of two possibilities:

- The underground pipe size will be increased to the point that HGL is below the acceptable level which means the road or overland conveyance has been completely ignored and the pipe sizes are over estimates; or
- Erroneous results where a water column is assumed to stand vertically at the nodes and this water column extends above the level of ground (e.g. curb or gutter inlet), without spreading over the land or road surface.

The third problem, with regard to overland flooding form inadequacy of storm sewers designed by rational methods is this. As stated above, rational method is a steady state method. In other words, it assumes that when peak flow is conveyed through the storm sewer system, the entire volume of the network is available to the flow and pipe sizes are calculated assuming full flow conditions. When a storm sewer system is designed and placed accordingly, overland flooding can still occur due to the following reason. Almost in all cases of storm events, peak discharge occurs at a certain time after the onset of rainfall. Form the onset of rainfall till the time when peak discharge occurs, the sewer system receives flows and thus during the time of peak discharge, the entire volume of the system cannot be available for the peak discharge to pass. Moreover, due to spatial variation of rainfall different inlets receive differential amounts of inflows and thus have varying capacity available for the peak discharge to convey through the system. These problems can be addressed more accurately by modeling the storm sewer systems using dynamic wave equations which are unsteady state equations of flow.

DYNAMIC WAVE MODELING

The unsteady state hydraulic models that simulate flow in the storm sewers use a modified form of Saint Venant equations. Saint Venant equations include the equations of continuity and momentum for gradually varied, unsteady, and subcritical flow in open channels. In normal situations, most flow in a storm sewer system can be classified as flow with a free surface. However, for high flow conditions during extreme events, a storm sewer may flow full and thus eventually becomes pressurized. These later flow conditions are known as surcharge flows. Several techniques have been developed whereby Saint Venant equations can also be generalized to include pressurized (surcharged) flows (e.g. see James and James, 2000; Rossman, 2006; DHI, 2007).

In Saint Venant equations, the equation of continuity is given by Eqn. 3,

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=r(x,t) \qquad \dots (3)$$

where,

A = Cross sectional area of flow (flow area)
Q = Discharge through the cross sectional area A
t = Time
x = Distance in the direction of flow
$r(x,t)$ = Rate of lateral inflow per unit length of the channel. When lateral inflow is zero, the continuity equation becomes,

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0 \qquad \dots (4)$$

The equation of conservation of momentum can be written in conservative (where the dependent variable is discharge) or non-conservative (where the dependent variable is velocity) forms.

In conservative form, it is given by Eqn. 5,

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\alpha\frac{Q^2}{A}\right)+gA\frac{\partial y}{\partial x}+gA\left(S_f-S_0\right)=0 \qquad \dots (5)$$

where,

α = Velocity distribution coefficient
S_0 = Bottom or bed slope
S_f = Friction slope or loss of energy per unit mass of fluid moving.

The non-conservative form of Eqn. 5 is Eqn. 6,

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+g\frac{\partial y}{\partial x}+g\left(S_f-S_0\right)=-\frac{u}{A}r(x,t) \quad \dots (6)$$

where,

u = Velocity in the principal direction of flow (x-direction) or the local average velocity

y = Depth of water in the y-direction which is perpendicular to the flow direction (x-direction). Actually, y denotes the hydraulic head of water in the conduit (elevation head plus any possible pressure head)

Equation 6 can also be expressed in terms of depth of flow. Neglecting the lateral inflow term, it is given by Eqn. 7 (Singh, 1996),

$$\underset{(1)}{\frac{\partial(uh)}{\partial t}}+\underset{(2)}{\frac{\partial(u^2h)}{\partial x}}+\underset{(3)}{\frac{\partial}{\partial x}\left(\frac{gh^2}{2}\right)}+\underset{(4)}{gh}\underset{(5)}{\left(S_f-S_0\right)}=0 \quad \dots (7)$$

As shown with Eqn. 7, there are five terms in any of the forms of the momentum equation. Each of these terms indicates the nature of the force acting on the fluid in motion. Thus, the momentum equation actually defines the equilibrium relation among the forces. The forces that are defined by the above terms are as follows.

Terms (1) and (2) denote inertial forces. In particular, Term 1 = Local inertial force; Term 2 = Convective inertial force;

Term 3 = Force resulting from pressure gradient over water column element;

Term 4 = The weight force or the gravitational force;

Term 5 = Resistance force created by the channel bed or the frictional force.

These forces are the dynamic mechanisms that govern wave motion. Frequently, two or more of these forces are predominant and the remaining forces are negligible. Consequently, classification of shallow water waves arises from the dominance of particular forces. Accordingly, the wave equation is classified into five main categories (Singh, 1996).

1. *Dynamic Waves*: When all of the five forces (inertial, pressure, gravity, and friction), are important, the resulting water wave is called dynamic wave. These are the most general open channel flow waves.
2. *Diffusive Waves*: The waves in which inertial forces are negligible are termed diffusion waves. These waves are dominated by pressure, gravity, and friction forces.
3. *Kinematic Waves*: When only gravity and friction forces are dominant and inertial and pressure forces are neglected, the resulting waves are called kinematic waves.
4. *Inertial* or *Small Gravity Waves*: The waves which are dominated by inertial and pressure forces are termed as small gravity waves.
5. *Steady Dynamic Waves*: When local inertia or local acceleration is negligible, the general dynamic waves become steady dynamic waves.

A question then obviously arises on to what wave form should be applied to a particular flow condition. The general flow equation or the dynamic wave equation forms the best theoretical basis for a flow model since the full equation of momentum makes it possible to describe all forces affecting the flow conditions. However, difficulties arise when dynamic wave equation is applied to simulate supercritical flow conditions. Dynamic wave equation, which uses full momentum equation including acceleration forces, allows correct simulation of fast transients and backwater profiles. The dynamic flow description should be used where the change in inertia of the water body over time and space is of importance. This is the case when the bed slope is small and bed resistance forces are relatively small (DHI, 2007).

The diffusive wave equation takes into account only the bed friction, gravity force, and the hydrostatic gradient (pressure) terms in the momentum equation. It allows the downstream boundary condition to be taken into account thus enabling simulation of backwater effects. The momentum equation for diffusive wave approximation is given as Eqn. 8,

$$gA\frac{\partial y}{\partial x}+gA\left(S_f-S_0\right) \quad \dots (8)$$

Since the inertia terms are neglected in the diffusion wave equation, it is suitable for backwater analyses where the bed and wall resistance are dominant, and for slowly propagating waves where the change in inertia are negligible.

The kinematic wave approximation is applicable where the flow is chiefly controlled by a balance between the friction and gravity forces. This means that the kinematic wave approach cannot simulate backwater effects. Thus, this description is appropriate for steep pipes without backwater effects. The flow conditions in steep, partially full pipes are mainly established by the balance between gravity forces and friction forces. Consequently, the inertia and pressure terms in the momentum equation are less dominant. Accelerations are comparably small and the flow is almost uniform (since friction slope is equal to bottom slope) so that the kinematic wave approximation is reasonable. In this approximation, the momentum equation is reduced to Eqn. 9,

$$gA\left(S_f - S_0\right) = 0 \qquad \ldots (9)$$

The kinematic wave is independent of the downstream boundary conditions. This implies that disturbances only propagate downstream. The kinematic wave description can therefore only be applied in cases when the flow is independent of the downstream condition which is the case in supercritical flow (Froude's number > 1). For all these reasons, kinematic wave approximation to pipe flows must be used with great caution. However, this approximation works very well for overland (sheet) flow.

From the discussion above, it should be evident that the dynamic wave equation is the most desirable equation that should be used in the simulation of pipe flow except when the flow is supercritical. Other simplification requires a good understanding of the influence of the omitted terms and their significance. Sometimes trial and error approach can be undertaken to establish the differences between various wave approximations.

Two of the most common computer models that implement dynamic wave equations in the solution of flow through a sewer network are SWMM (or XP-SWMM) and MOUSE. SWMM (Storm Water Management Model) has been developed by the US Environmental Protection Agency. The latest version is SWMM 5 (2006) which is a window based application with graphical user interface (GUI). XP-SWMM is proprietary software developed by XP Software Corporation by using the original SWMM code with certain modifications and with modern GUI. MOUSE (Model of Urban Sewers) is also proprietary software developed by the Danish Hydraulic Institute (DHI, 2007). SWMM uses only full form dynamic wave model whereas XP-SWMM or MOUSE offers the choice among dynamic, diffusive, and kinematic waves. In case of supercritical flow, MOUSE reduces the dynamic wave approximation to diffusive wave approximation. Moreover, there is a significant difference in the way SWMM and MOUSE solve Saint Venant Equations. SWMM uses an explicit finite difference scheme (see James and James, 2000; Rossman, 2006) whereas MOUSE uses an implicit finite difference schemes (DHI, 2007). An example presented below illustrates how the results of a calculation may vary depending upon the choice of the solution algorithm. Mark *et al.* (2004) discussed how these models can be used to simulate urban flooding by incorporating the interaction between the buried pipe system, the streets, and the areas flooded with stagnant water. Hsu *et al.* (2000) and Chen *et al.* (2005) integrated hydrologic model to derive inflow hydrographs into the storm sewer systems with SWMM models to simulate underground pipe flows and two-dimensional diffusive wave model to simulate overland flow in order to model inundation from urban storm sewer systems.

DESIGN EXAMPLES

A Hypothetical Case

A hypothetical example simplified from a real-world situation is used to illustrate that storm sewers designed with the applications of rational method should subsequently be evaluated with dynamic wave modeling to determine the flooding potential from the system. Figure 4 shows a newly designed storm sewer system associated with a small residential street project adjacent to a stream. The pipe sizes are designed using rational method with design frequency storm having an AEP of 0.10.

For this purpose we have used the computer model NeoUDSewer originally developed by Dr. James C. Y. Guo of the University of Colorado at Denver. Table 3 provides the hydrologic and hydraulic data used in the design calculations. Following a typical design criterion, the HGL was calculated by setting the WSE of the downstream boundary at the soffit of the outfall pipe. The pipe sizes obtained in this method satisfy the common design standard by keeping the HGL below ground elevation (Table 3).

The system was subsequently modeled using both XP-SWMM (explicit method) and MOUSE (implicit method) with the same design frequency storms under two contrasting conditions. One set of models was produced using a constant WSE at the downstream boundary. In the second set, a time-variable boundary condition was imposed at the downstream end. For both these models hydrographs at inlet locations were determined by hydrologic modeling of the catchments using 0.10 AEP frequency storm for Harris County. The hydrologic models were developed using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) developed by the U. S. Army Corps of Engineers (2006). The results are presented in Table 4. For the constant head boundary condition, results of XP-SWMM modeling show that the system designed by rational method is adequate since no surface ponding is observed with this model. However, the results of MOUSE model shows that at three locations overland flooding is possible.

Table 3: Hydrologic and Hydraulic Data for the Design Example (Figure 4)

Catchment ID	A_1	A_2	A_3	A_4	A_5	A_6	A_7	A_8	
Area	0.344	0.263	0.243	0.304	0.344	0.223	0.668	0.182	
Runoff coefficient	0.7	0.75	8	0.75	0.6	0.85	0.25	0.3	

	Dia	*Node ID*		*Inverts (m)*		*Ground (m)*		*WSE at nodes (m)*	
Link ID/Length	*m*	*From*	*To*	*USIN*	*DSIN*	*US*	*DS*	*From*	*To*
L1 (91.44)	0.4572	N_1	N_2	105.65	105.19	107.62	107.44	106.19	105.77
L2 (76.20)	0.5334	N_2	N_3	105.19	104.81	107.44	106.44	105.77	105.31
L3 (67.06)	0.4572	N_4	N_{10}	105.92	105.59	107.90	107.11	106.23	106.05
L4 (60.96)	0.5334	N_5	N_6	105.37	105.07	106.68	106.44	105.91	105.69
L5 (36.58)	0.5334	N_6	N_3	105.08	104.81	106.44	106.44	105.69	105.31
L6 (12.19)	0.6096	N_3	N_9	104.81	104.69	106.44	105.98	105.31	105.24
L7 (22.86)	0.4572	N_7	N_6	105.20	105.07	107.19	106.44	105.81	105.69
L8 (17.01)	0.6096	N_9	N_{11}	104.75	104.56	105.98	105.80	105.24	104.57
L9 (56.39)	0.4572	N_8	N_{10}	105.87	105.59	107.85	107.11	106.10	106.05
L10 (42.67)	0.4572	N_{10}	N_5	105.59	105.37	107.11	106.68	106.05	105.91

Rainfall intensity (mm/hr), 0.01 AEP, Harris County									
5 min	**15 min**	**20 min**	**60 min**	**120 min**	**180 min**	**360 min**	**720 min**	**1440 min**	
219.40	182.70	157.70	81.00	50.20	37.60	22.60	13.50	8.00	

US = Upstream; DS = Downstream; USIN = Upstream invert; DSIN = Downstream invert.

Table 4: Comparison of Results from MOUSE and XP-SWMM Models (system shown in Figure 4)

	Constant Head Boundary Condition				*Time Dependant Boundary Condition*			
	MOUSE		*XP-SWMM*		*MOUSE*		*XP-SWMM*	
Node ID	*WSE (m) (Maximum)*	*Ponding Depth (m)*	*WSE (m) (Maximum)*	*Ponding Depth (m)*	*WSE (m) (Maximum)*	*Ponding Depth (m)*	*WSE (m) (Maximum)*	*Ponding Depth (m)*
N_1	106.7	–0.92	105.97	–1.66	107.21	–0.41	106.55	–1.08
N_2	106.53	–0.91	105.76	–1.68	107.04	–0.40	106.36	–1.08
N_3	106.31	–0.13	105.56	–0.87	106.83	*0.39*	106.14	–0.30
N_4	107.39	–0.51	106.47	–1.43	107.57	–0.33	106.95	–0.95
N_5	107.12	*0.44*	106.21	–0.47	107.31	*0.63*	106.68	0.00
N_6	106.84	*0.40*	105.92	–0.52	107.13	*0.69*	106.44	0.00
N_7	106.85	–0.33	105.92	–1.26	107.13	–0.05	106.44	–0.74
N_8	107.31	–0.54	106.38	–1.47	107.49	–0.36	106.86	–0.99
N_9	105.75	–0.23	105.40	–0.58	106.45	*0.47*	105.98	0.00
N_{10}	107.29	*0.18*	106.36	–0.75	107.47	*0.36*	106.84	–0.27
N_{11}	105.18	–0.62	105.18	–0.62	106.08	*0.28*	106.08	0.28

Ponding depth = Maximum WSE – Ground Level (Table 3). A negative value indicates that the WSE is below the ground level.

With time-dependent downstream boundary conditions both XP-SWMM and MOUSE models show that there is a potential of overland flooding from the system. However, the two models yield different results in terms of both flooding depth and locations. This has important implications if the design obtained from the rational method needs to be improved based on such assessments.

Case Study 1: Design of a New System

Figure 5 shows the layout of a storm sewer system that is designed along a proposed roadway extension adjacent to Brays Bayou in the City of Houston. This is a highly flood prone area. The initial system is designed using rational method and according to the design criteria set by the Public Works Department of the City of Houston. According to these criteria, a new

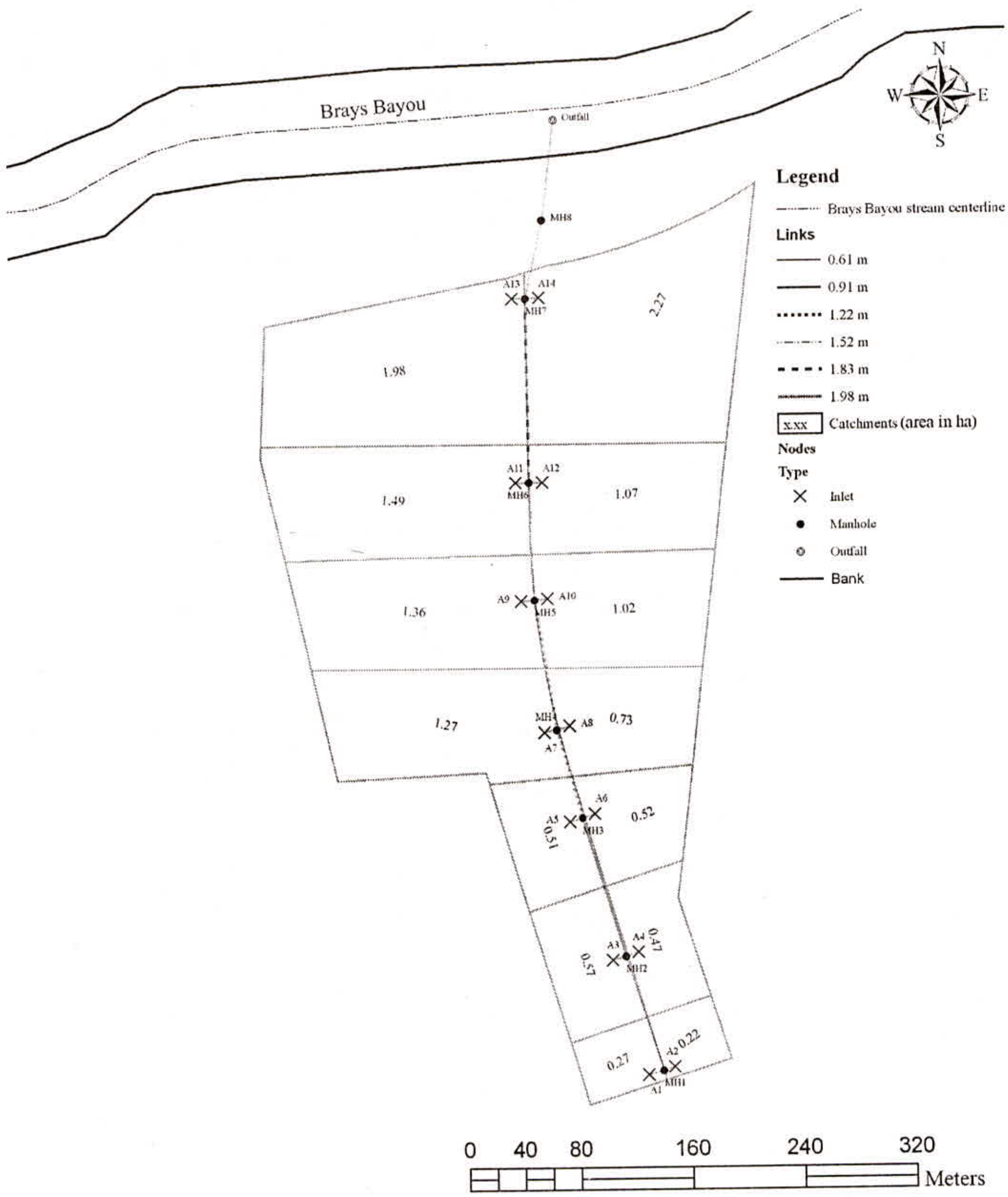

Fig. 5: Plan of a proposed roadway close to Brays Bayou, City of Houston

storm sewer system should be designed with a design frequency storm with 0.5 AEP and the HGL must be at or below the gutter elevation.

The system must also be stressed with a frequency storm with 0.01 AEP and if the HGL exceeds the ground elevation at the right-of-way then the pipe sizes must be adjusted appropriately. The pipe sizes shown in Figure 5 are calculated following these guidelines. The system was designed using a computer model called HOUSTORM which is based on rational method. The City of Houston recommends its use in the design of a new system.

In order to access the overland flooding potential of the system, it was subsequently modeled using MOUSE. Hydrographs at each of the inlets were calculated using HEC-HMS and Natural Resources Conservation Service's (NRCS) curve number method. Two models were run. In the first a constant head boundary at the soffit of the outfall pipe was used. In the second model, a time-dependent downstream boundary condition was used. Since the unsteady state hydraulic model of Brays Bayou is not available at this stage, a time dependent boundary condition is created from the 100 year water surface elevation, obtained from a steady state open channel hydraulics model of Brays Bayou. The peak stage is assumed to be attained by the channel close to the time when peak of the inflow hydrographs from overland occurs. The stage-time curve rises uniformly form the invert of the pipe at the initiation of the storm and recede uniformly with the recession limb of the hydrographs. Figure 6 shows the calculated HGL for the constant head boundary condition. Figure 7 shows the calculated HGL for the time dependant boundary condition. It can be seen from these figures, that with the assumption of a constant head boundary at the soffit of the outfall, the pipe sizes are adequate there is no potential for overland flooding from the storm sewer surcharge.

The second model, with a time-dependent boundary condition, where the WSE at the outfall can attain a greater level, shows that the surcharge of the system is possible. The extent of surcharge of the system depends on the maximum water level attained at the downstream boundary. This example illustrates the importance of incorporation of time dependent boundary condition on the evaluation of adequacy of a design.

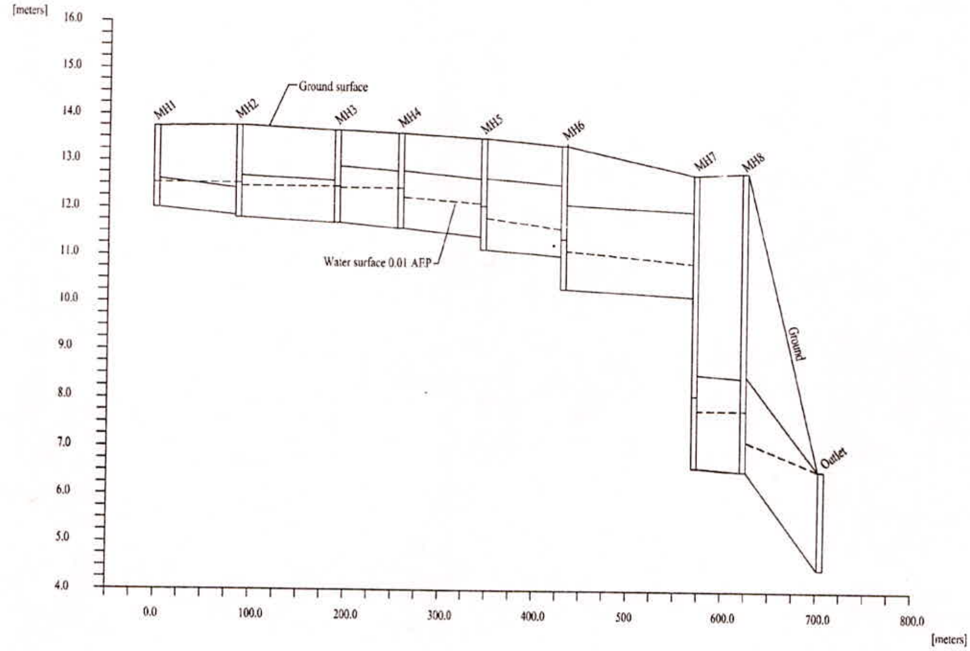

Fig. 6: HGL of the proposed storm sewer system close to Brays Bayou (Figure 5) with constant head boundary at sofit; 0.10 AEP

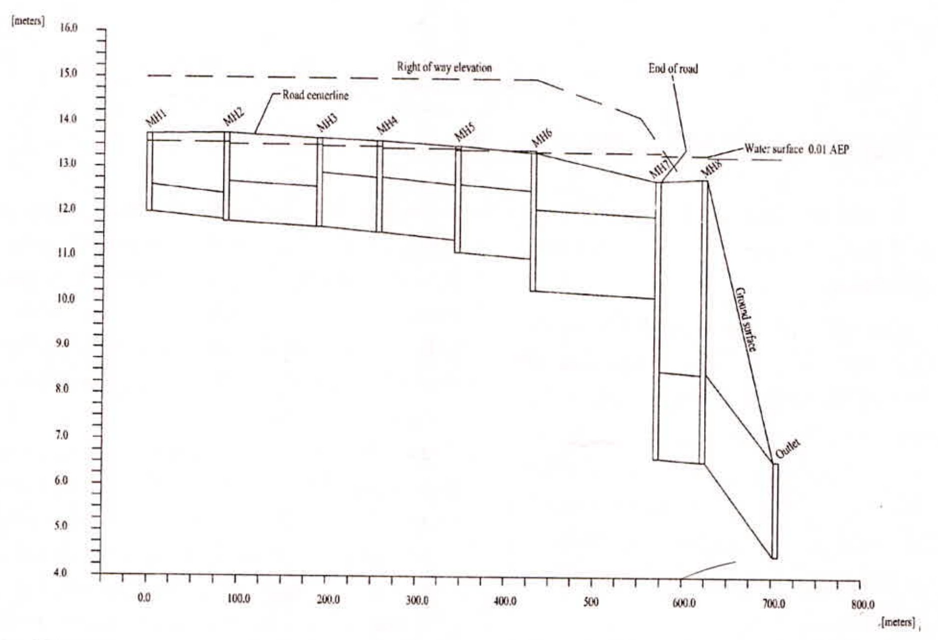

Fig. 7: HGL of the proposed storm sewer system close to Brays Bayou (Figure 5) with time dependant downstream boundary conditions; 0.10 AEP

Case study 2: Analysis of an Existing System

Figure 8 shows an existing storm sewer system in a mixed residential-commercial area within City of Richardson, a satellite city of the highly urbanized City of Dallas, Texas. This system is particularly interesting since it clearly shows how urbanization affects natural drainage system of an area and its consequences. There was a time when Twin Rivers, the natural channel shown in the southwestern part of the area depicted in Figure 8 used to be a tributary of Huffhines Creek, the natural channel shown in the northern part of the area. With development, part of the Twin Rivers was converted to an underground storm sewer system which in turn acted as a main trunk to which lateral systems were connected. If the original design followed local design criteria, then a design storm frequency with 0.04 AEP must have been used in sizing the system using rational method. However, the area over the storm drain system experiences frequent flooding from overland ponding due to surcharge of the storm sewers. All of the catchments, except the one drained by Twin Rivers are virtually impervious (parking lots, roads etc.). This system was modeled using MOUSE. An unsteady state hydraulic model was developed to determine the stage-time curve of Huffhines Creek at the outfall of the system. Hydrologic models using HEC-HMS and NRCS methodologies were developed to calculate the hydrographs of the catchments. These hydrographs were routed through the appropriate inlets. Finally, two consecutive nodes were connected with two links. One link represented the underground pipe and the other overland flow paths as open channels. The cross sectional geometries of these overland flow paths were derived from detailed surface topographic data. There are two ways by which two nodes can be connected by more than one link. Either a weir flow is assumed from one link to the other when the WSE reaches the weir crest or the inverts of the two links can be specified at different heights (e.g. for the overland flow path it is the ground elevation but for the storm sewer it is the invert of the pipe at a manhole or inlet or junction box). In the latter, flow is divided between the two links when WSE in the node reaches ground elevation.

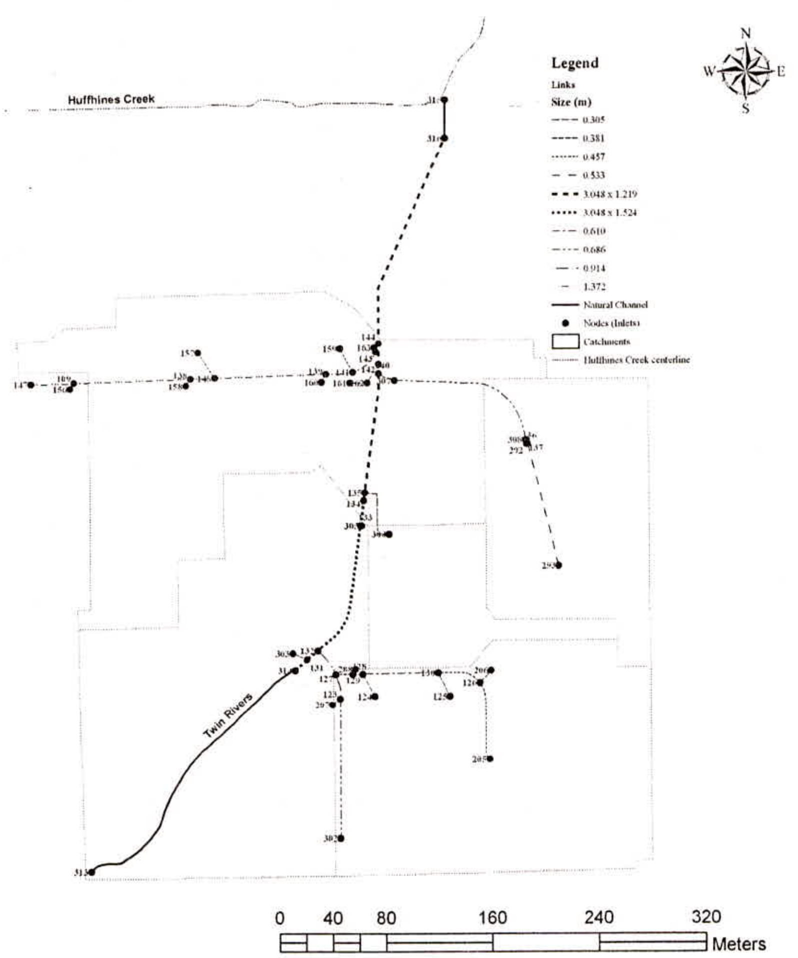

Fig. 8: Layout of an existing storm sewer system in the City of Richardson with chronic overland flooding problem

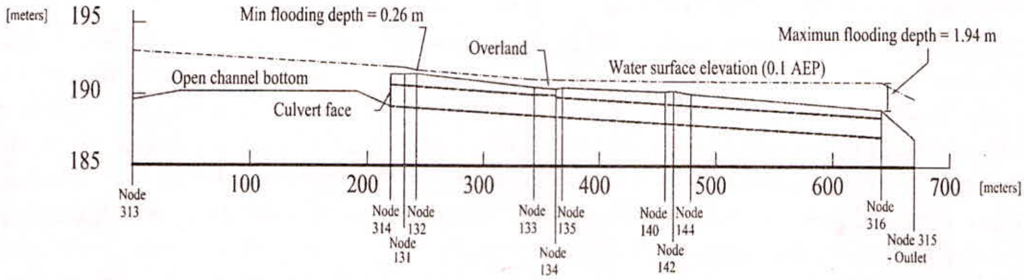

Fig. 9: HGL of the Twin Rivers storm sewer system (Figure 8); 0.10 AEP

Multiple models were run using various storm frequencies ranging from 0.01 to 0.10 AEP. Figure 9 shows that overland flooding in this area can occur from a storm event with 0.10 AEP.

This example illustrates the importance of incurporation of overland flow paths in assessment of flooding potential of a storm sewer system.

SUMMARY

Most of the design models for storm sewers are based on rational formula (Yen and Akan, 1999). These models are not based on any hydraulic consideration except for the very basic equation involved in the determination of the size of the pipes in the sewer system. However, for the purpose of reliable flow simulation, models with highest level of hydraulic sophistication and accuracy are required. Models utilizing dynamic wave theory can be used for this purpose. Commonly these models are used for the evaluation and prediction of a system. Often it is cost prohibitive to use these models for the design of a new system but they can also be used for this purpose. However, in urban areas where overland flooding is a problem, either a proposed design or an existing system should be assessed with dynamic wave modeling to evaluate whether the storm sewer system serves efficiently in abating overland flooding from surcharged flows. An evaluation of an existing system leads to recommendations for its improvements.

In most cases designs based on rational method are found to be inadequate when they are evaluated with dynamic wave models. It is primarily because the rational method assumes that during peak discharge the entire volume of the sewer system is available for flow. But with an unsteady state model both spatial and temporal variations of rainfall are accounted for and hence the storage capacity of the system is a time dependent element.

Specification of time dependent downstream boundary conditions requires unsteady state hydraulic models of the receiving streams. Similarly, time dependent boundary conditions at the upstream boundaries require hydrologic models of the watershed. Thus, design and analysis of storm sewer systems for mitigation of urban flooding from storm sewer surcharges require an integrated approach combining hydrology of the watershed, hydraulics of the streams serving the storm sewers, and topography of the surface. However, commonly available software that implement dynamic wave theory do not produce comparable results. The differences arise from the algorithm used in the solution of the Saint Venant's equations of unsteady flow.

A caveat for the approach presented above should be given at this point. In the models presented above it is assumed that a runoff hydrograph first enters an inlet and then part of this flow is carried by the pipe system and under surcharged conditions, the surcharged volume is carried by overland conveyance such as a road or pavement. Most applications are typically modeled in this way and assume that a free movement of water can occur between the inside of the inlet and overland. In reality the process is in reverse direction because runoff first enters the road segment and part of the runoff then passes through rectangular orifices that represent the curb opening and enters the inlet. The shape and size of this opening can easily restrict the amount of water that enters the inlet. The result is that the calculated pipe sizes may never be utilized if the opening is not large enough which in turn results in HGL over the ground surface being higher than the calculated value. As a matter of fact design of the proper inlet sizing is an altogether different issue. However, interactions of the actual geometries of the inlets, overland flow paths, and sewer pipes can be modeled in more complete and complex manner by creating two nodes which represent the overland segment and the inlet separately and by connecting those by orifices.

REFERENCES

Anderson, D.G. (1970). "Effects of urban development on floods in northern Virginia". *US Geol. Surv., Water-Supply Paper* 2001-C.

Carter, R.W. (1961). "Magnitude and frequency of floods in suburban areas". *US Geol. Surv., Prof. paper 424-B,* pp. 149–59.

Chen, A.S., Hsu, M.H., Chen, T.S. and Chang, T.J. (2005). "An integrated inundation model for highly developed urban areas". *Water Sci. Tech.* IWA, 51 (2), 221–229.

Danish Hydraulic Institute (2007). "Mouse Pipe Flow – Reference Manual". DHI Water & Environment, HØrsholm, Denmark.

Delleur, J.W. (2003). "The evolution of urban hydrology: Past, present, and future". *J. Hyd. Engrg.*, ASCE, 129 (8), 563–573.

Espey, W.H. and Winslow, D.E. (1974). "Urban flood frequency characteristics". *Proc. Am. Soc. Civ. Engrs. J. Hydraul.* Div. 100 (HY2), 279–293.

Hall, M.J. (1984). *Urban Hydrology*, Elsevier Applied Science Publishers, New York, N. Y., 299p.

Hollis, G.E. (1975). "The effect of urbanization on floods of different recurrence interval". *Wat. Resour. Res.*, 11, 431–435.

Hsu, M.H., Chen, S.H. and Chang, T.J. (2000). "Inundation simulation for urban drainage basin with storm sewer system". *J. Hydrol.* 234, 21–37.

James, W. and James, R.C. (2000). "Water systems models [2] Hydraulics: User's guide to. SWMM4 Transport, Extran, and Storage modules". Computational Hydraulics International, Canada.

Mark, O, Weesakul, S., Apirumanekul, C., Aroonet, S.B. and Djordjevic, S. (2004). "Potential and limitation of 1D modeling of urban flooding". *J. Hydrol.* 299, 284–299.

Martens, L.A. (1968). Flood inundation and effects of urbanization in metropolitan Charlotte, North Carolina, *US Geological Survey Water-Supply Paper*, 1591-C.

Mukhopadhyay, B., Nouri, F., Penland, C. and Dutta, A. (in press). "Model flood alert system: A development and application for the Theater District within Downtown Houston, Texas". *J. Hydrol. Engrg.*, ASCE.

Novotny, V., Clark, D., Griffin, R.J. and Booth, D. (2001). "Risk-based urban watershed management under conflicting objectives". *Water Sci. Technol.*, 43 (5), 69–78.

Rossman, L.A. (2006). "Sorm water management model quality assurance report: Dynamic wave flow routing". EPA/600/R-06/097. United States Environmental Protection Agency. Cincinnati, OH.

Schmitt, T.G., Thomas, M. and Ettrich, N. (2004). "Analysis and modeling of flooding in urban drainage systems". *J. Hydrol.*, 299, 300–311.

Singh, V.P. (1996). *Kinematic wave modeling in water resources*, John Wiley & Sons, Inc. New York, N.Y.

U.S. Army Corps of Engineers (2006). "Hydrologic Modeling System HEC-HMS", Hydrologic Engineering Center, Davis, CA.

Yen, B.C. and Akan, A.O. (1999). "Hydraulic design of urban drainage systems". *Hydraulic Design Handbook* (L.W. Mays, Editor in Chief). McGraw-Hill, New York, N.Y.