# THE STUDY OF GIS-BASED HYDROLOGICAL MODEL IN HIGHWAY ENVIRONMENTAL ASSESSMENT

by

Weizhe An

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This dissertation was presented

by

Weizhe An

It was defended on

March 30, 2007

and approved by

Dr. Rafael G. Quimpo, Professor, Dept. of Civil and Environmental Engineering

Dr. Ronald D. Neufeld, Professor, Dept. of Civil and Environmental Engineering

Dr. Jeenshang Lin, Associate Professor, Dept. of Civil and Environmental Engineering

Dr. William Harbert, Associate Professor, Dept. of Geology and Planetary Science

Dissertation Director: Dr. Rafael G. Quimpo, Professor
Dept. of Civil and Environmental Engineering

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Weizhe An, Ph.D.

University of Pittsburgh, 2007

Highway construction often causes substantial adverse environmental effects, both during and after the construction phases. To assess the impact of highway construction on surrounding environment and mitigate its adverse influences, I-99 Environmental Research is being conducted. This study is a component of I-99 Environmental Research and mainly focuses on hydrological modeling of the highway construction watersheds.

Recent research in basic hydrological theories and related fields were reviewed. Several Geographic Information System (GIS) techniques in hydrological modeling were discussed. Recent GIS-based hydrological applications are mostly applied to large, natural watersheds. However, the watersheds in this research are very small and their topographic characteristics are severely changed by construction. There are a few difficulties in applying the GIS-based watershed models in the project. Several improvements were made to apply GIS-based watershed model to highway watersheds using Watershed Modeling System (WMS). The WMS model employs Soil Conservation Service Unit Hydrograph (SCS UH) to generate hydrograph and has the shortcoming of predicting earlier peak time and higher peak discharge.

To overcome the WMS weakness, a new model - Highway Watershed Model (HWM) was developed. The HWM model uses a new type of unit hydrograph, the Linear Exponential Unit Hydrograph (LEUH) in generating runoff from rainfall. Dimensionless Unit Hydrograph (DUH) in LEUH consists of linear rising part and exponential recession part. HWM has the ability to describe different watersheds using different LEUHs, which reflect the watershed unique hydrologic response characteristic. In both WMS and HWM models, an attempt was also made to find out the relationship between antecedent

moisture condition (AMC) and curve number (CN). The diagram fitting of AMC-CN from HWM is better than that from WMS. It is recommended that this issue may be studied further using additional instrumentation to measure the time variation of soil moisture conditions. Although the WMS is widely used, HWM produces more reliable and better results than WMS in the studied watershed. The peak discharge and peak time are difficult to simultaneously model perfectly.

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| | | |
|---|---|---|
| $f_p$ | = | Infiltration capacity into soil |
| $f_\infty$ | = | Minimum or ultimate value of $f_p$ (at $t = \infty$) |
| $f_o$ | = | Maximum or initial value of $f_p$ (at $t = 0$) |
| $t$ | = | Time |
| $K$ | = | Hydraulic conductivity, Muskingum proportionality coefficient, Coefficient of contraction, flow velocity coefficient. |
| $k$ | = | Recession constant |
| $s_e$ | = | Initial effective saturation |
| $\theta_e$ | = | Effective porosity |
| $\Psi$ | = | Wetting front soil suction head |
| $P$ | = | The incremental precipitation depth, excess rainfall |
| $S$ | = | The potential maximum retention, watershed slope, watercourse storage |
| CN | = | The Curve Number |
| $e_v$ | = | Evaporation loss rate |
| $A$ | = | Watershed area, channel cross section area |
| $e_d$ | = | Evaporation rate |
| $k_{\text{cov}}$ | = | Evaporation conversion factor |
| $V$ | = | Volume of water |
| $D$ | = | Water depth |
| $I^*$ | = | Rainfall excess |
| $I$ | = | Inflow rate |
| $Q$ | = | Outflow rate |
| $n$ | = | Manning's roughness coefficient |

| | | |
|---|---|---|
| $R$ | = | Hydraulic radius |
| $C$ | = | Conversion constant, Muskingum coefficient |
| $a_i$ | = | The area draining through a grid square per unit contour length |
| $\beta_i$ | = | The local surface angle |
| $\tan \beta_i$ | = | The local surface slope |
| $\lambda_i$ | = | Index of hydrological similarity |
| $T_0$ | = | The lateral downslope transmissivity when the soil is just saturated |
| $S_i$ | = | Local storage deficit |
| $m$ | = | Model parameter controlling the rate of decline of transmissivity with increasing storage deficit in TOPMODEL |
| $q_i$ | = | The downslope subsurface flow rate per unit contour length |
| $q_p$ | = | Peak discharge |
| $T_p$ | = | Time of rise |
| $t_p$ | = | Lag time |
| $t_r$ | = | Duration of effective rainfall |
| $T_c$ | = | Time of concentration |
| $U$ | = | Unit hydrograph |
| $X$ | = | Muskingum weighting factor |
| $T_{lag}$ | = | The lag time |
| $L$ | = | Watershed length |
| $g$ | = | Acceleration of gravity |
| $h_i$ | = | Water pressure head before or after the contraction part |
| $T_r$ | = | Relative peak time |
| $K_r$ | = | Recession constant |
| $BS$ | = | Watershed slope |
| $AOFD$ | = | Average overflow distance |
| $MSL$ | = | Maximum stream length |
| $MSS$ | = | Maximum stream slope |
| $NSTPS$ | = | The number of integer steps for the Muskingum routing |
| APD | = | Antecedent precipitation depth |

| | | |
|---|---|---|
| $R$ | = | Pipe radius, the recharge rate in TOPMODEL |
| $c_k$ | = | Kinematic wave celerity |
| $\theta$ | = | The wetted angle in pipe flow |
| $E$ | = | Specific energy |
| $B$ | = | Flume width |
| $CWV$ | = | The channel water velocity |

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# 1.0 INTRODUCTION

## 1.1 BACKGROUND

Hydrology is the scientific study of water and its properties, distribution, and effects on the earth's surface, soil, and atmosphere (McCuen 1997). Water circulation in the air, land surface and underground constitutes hydrologic cycle. The cycle has no beginning or end. Hydrology researchers are often faced with problems of runoff prediction, contaminant concentrations, water stages, etc. Due to the great spatial and temporal variability of watershed characteristics, precipitation patterns, contaminant transport rules, and the number of variables involved in the physical processes, rainfall-runoff relationship is one of the most complex hydrologic phenomena. Prediction of hydrological process is extremely important to water resources engineering. Figure 1 illustrates the main hydrological processes at a local scale.

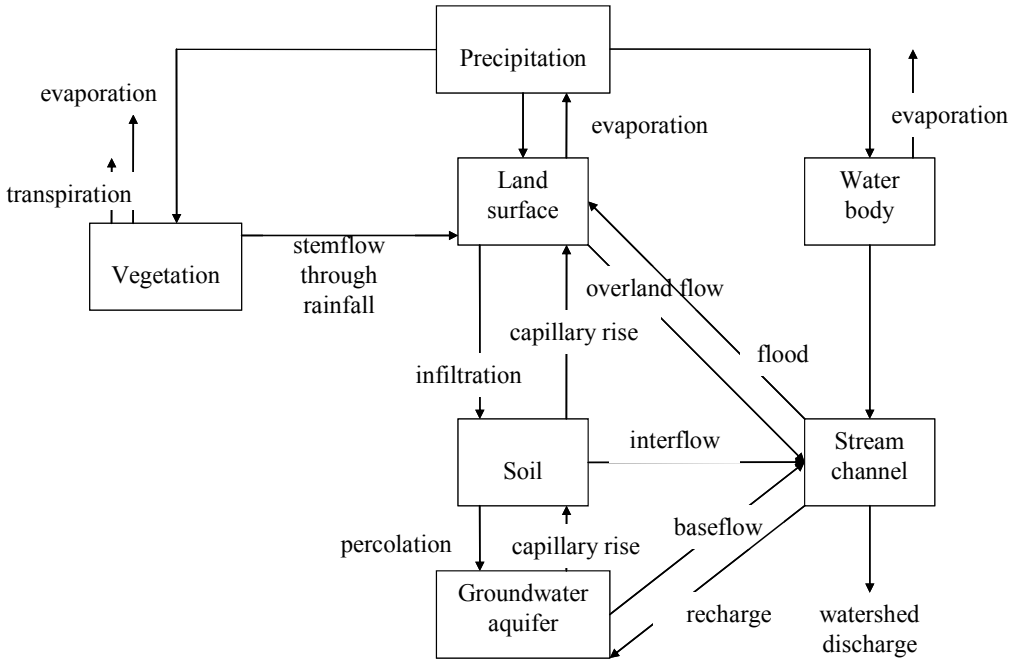



**Figure 1.** Main hydrological processes at a local scale

(Ward, 1975, Modified)

In recent years, in many places of the world, the processes of rapid population growth, highway construction, urbanization, and industrialization have increased the demand for clean water. Simultaneously, these processes also greatly change the hydrological conditions and have negative influences on storm water quantity and quality. For water quantity, urbanization increases the percentage of impervious area in a watershed, thus the surface runoff in a post-development area becomes greater than in that in pre-development area. Consequently, the base flow and interflow in post-development area is significantly reduced. Furthermore, the discharge flow time pattern changes, i.e. the peak discharge increases and peak time become shorter.

Besides quantity, water quality is affected by a combination of natural and human factors. Natural factors affecting water quality include precipitation intensity and amount, geology, soil types, topography and vegetation cover, etc. Meybeck et al. (1989) provided a detailed review of this topic. Most of these factors can be, and have been, affected by humans; for example, changes in river discharge due to construction, abstraction, urbanisation or impounding; discharges from industry, agriculture or sewerage, etc (Meybeck M, et al. 1989).

To protect communities from adverse environmental disturbance, the evaluation of the influence of urbanization or construction is urgently needed for the corresponding watershed areas.

## 1.2 PROBLEMS STATEMENT

The Pennsylvania Department of Transportation (PENNDOT) is constructing the U.S. Route 220/I-99/State Route (S.R.) 6220 project that is a part of a large effort to extend I-99 to I-80 at Bellefonte. Highway construction is often the cause of substantial adverse environmental effects, both during the construction phase and during the operational phase. To appraise the environmental influence of highway construction, monitoring and evaluating the effectiveness of various mitigation techniques implemented are being done during the construction of I-99 to enable improved management of corridor resources.

The research also includes developing enhanced capabilities to predict impacts and identifying suitable mitigation measures for future highway constructions.

The research includes four parts. The first part is the evaluation of approved erosion and sediment controls to determine best management practice. Intimately tied to the first part is the prediction of the runoff as a result of the rainfall on the construction project. This is the second part of the project. The third part is to monitor and assess the wetland hydro-biological indicators for land use planning in the highway. The last part is to evaluate the effectiveness and sustainability of stream restoration, rehabilitation, and relocation projects as part of mitigation for road construction. This study focuses on the second part.

## 1.3 OBJECTIVE AND SCOPE

The objective of this research is to build a runoff prediction model using GIS techniques for a watershed affected by highway construction. The model will be designed to predict runoff at several outlets in the watershed based on rainfall, land use, soil type, detention pond location, stream distribution, water velocity, and basin slope, etc. For efficiency and cost effectiveness, runoff prediction method and local applicability will be verified. A successful model can reduce expensive monitoring instrumentation and personnel training cost in future projects.

The scope of this research will involve numerical computer simulations and field data collection, assimilation and analysis. A GIS-based rainfall-runoff model will be developed using the Watershed Modeling System (WMS) platform and calibrated to simulate the hydrology and hydraulics phenomena along the stream system draining the selected constructed basin near I-99 highway. To compensate for the shortcomings the WMS model, a new model -- Highway Watershed Model (HWM) is developed. The results from WMS and HWM will be analyzed and compared. The relationship between antecedent moisture condition (AMC) and curve number (CN) will be investigated.

Highway watershed is more difficult to model than natural mountainous watersheds due to high human disturbance. The watershed elevation is altered by human construction and hence the stream directions are changed. The alteration makes elevation changes too mild (on the highway surface, for example) or too steep (at the mountain cut, for example). This research will discuss the methods dealing with these human interventions in the watershed.

In applying GIS-based model, some GIS source data and watershed characteristics requirements must be available. Unfortunately, not all of them are available to this project. A trade-off to deal with GIS-based model and traditional conceptual model applied in the practical watershed will be discussed. This is also a reason to develop HWM model. Also, surface flow data and ground flow data will be collected to calibrate the model.

## 1.4 LITERATURE REVIEW

Extensive studies have been done on watershed modeling. Generally, models can be divided into two broad categories: physically based model (distributed model) and conceptually based models (lumped model). A recent review on rainfall-runoff modeling is given by O'Loughlin et al. (1996). Singh (1988) provides a general survey of most of the techniques available for modeling hydrological systems at that time.

Physical-based models are one type of models that are based on physical laws and known initial and boundary conditions. Presently, quite a few physically based models have been developed and applied. Physically-based models are normally run with point values of precipitation, evaporation, soil moisture and watershed characters as primary input data and produce the runoff hydrographs. They are generally accurate, but difficult to use. Many of the assumptions in these models cannot be satisfied in practice. Users must determine a huge number of parameters, which are often difficult to obtain. In regions where precipitation data series are available but runoff data are scarce, a deterministic rainfall-runoff model is a good tool.

Kavvas et al. (2004) presented a new model Watershed Environmental Hydrology Model (WEHY) to the modeling of hydrologic processes in order to account for the effect of heterogeneity within natural watersheds. The parameters of the WEHY are related to the physical properties of the watershed, and they can be estimated from readily available information on topography, soils and vegetation/land cover conditions (Chen et al. 2004a). The parameters can be obtained from GIS database of a watershed without resorting to a fitting exercise. The model was applied to the Shinbara-Dam watershed and has produced promising runoff prediction results (Chen et al. 2004b).

Liang (2003) presented two improvements on the three-layer variable infiltration capacity (VIC-3L) model, which is also originally developed by Liang. The VIC model is a macro-scale hydrologic model that solves full water and energy balances. It can be applied to various watershed sizes ranging from small watersheds to continental and global scale. One improvement of the research is to include the infiltration excess runoff generation mechanism in the VIC model by considering effects of spatial sub-grid soil heterogeneities on surface runoff and soil moisture simulations. The other is to consider the effects of surface and groundwater interaction on soil moisture, evapotranspiration, and recharge rate. The two improvements are tested by comparing the modelled total runoff and groundwater table with observed values at the watershed of Little Pine Creek, Etna, Pennsylvania. Results show that the new version of VIC simulates the total runoff and groundwater table very well.

To investigate the influence of urban pavement and traffic on runoff water quantity, Cristina et al. (2003) developed a kinematic wave model which accurately captured the significant aspects of typical urban runoff. The impacts of the paved urban surface and traffic were examined with respect to the temporal distribution of storm water runoff quantity. The kinematic wave theory gave predictions of the time of concentration that were more accurate than other more common methods currently in use.

Campling et al. (2002) developed TOPMODEL, a semi-distributed, topographically based hydrological model, and applied it to continuously simulate the runoff hydrograph of a medium-sized (379km$^2$), humid tropical catchment. The researcher found that water tables were not paralle to the surface topography. To increase the weighting of local storage deficits in upland areas, a reference topographic index $\lambda_{REF}$ was introduced into the TOPMODEL structure. Not confined in deterministic modeling, this research also assessed the performance of the model with randomly selected parameter sets and set simulation confidence limits by using generalized likelihood uncertainty estimation (GLUE) framework. The model simulated the fast subsurface and overland flow events superimposed on the seasonal rise and fall of the base-flow very well. It was also found that there was increased uncertainty in the simulation of storm events during the early and late phase of the season.

Conceptual-based lumped models are well known for their simplicity. They are also applied widely by many researchers. Fontaine (1995) evaluated the accuracy of rainfall-runoff model simulations by using the 100-year flood of July, 1, 1978 on the Kickapoo River, in southwest Wisconsin as a case study. The accuracy of a simple analysis is compared to that of an elaborate, labor-intensive analysis. The more elaborate modeling approach produces more accurate results. Fontaine concluded that the error in the precipitation data used for calibrating the model appears to be the primary source of uncertainty.

Lidén (2000) did an analysis of conceptual rainfall-runoff modeling performance in different climates. It was found that the magnitude of the water balance components had a significant influence on model performance. Beighley (2002) presented a method for quantifying spatially and temporally distributed land use data to determine the degree of urbanization that occurs during a gauge's period of record. Madsen (2002) presented and compared three different automated methods for calibration of rainfall-runoff models.

Besides building mathematical models, researchers have developed object-oriented software to model rainfall-runoff relationship. Garrote et al. (1997) presented a software

environment for real-time flood forecasting using distributed models. The system, Real-time Interactive Basin Simulator (RIBS), provides an object-oriented framework for implementation of a class of distributed rainfall-runoff models satisfying certain formal requirements. RIBS manages process organization and data handling; facilitation of user interaction and result visualization and provision of access to model structure, hydrologic processes, and model inference.

Other widely used packaged software include Watershed Modeling System (WMS, developed by Environmental Modeling Research Laboratory, Brigham Young University), Storm Water Management Model [SWMM, developed by U. S. Environmental Protection Agency (EPA)], Hydrologic Engineering Center Hydrological Modeling System (HEC-HMS, developed by U. S. Army Corps of Engineering) and Hydrological Simulation Program -- FORTRAN (HSPF, developed by U. S. EPA).

## 1.5 LAYOUT OF THIS DISSERTATION

The dissertation is organized as follows. Chapter Two contains basic theories on watershed modeling; both physical-based and conceptual-based model theories. Chapter Three reviews the procedures for developing the GIS-based model -- WMS model, which the author investigated in detail but found to be inadequate. Chapter Four gives an overview of the studied watershed. Chapter Five illustrates some difficulties in the model building. Chapter Six presents a case study on the selected watershed and the modeling results. Chapter Seven develops the new model -- Highway Watershed Model (HWM), an alternative to the WMS model. Chapter Eight gives the modeling results of HWM; it also analyzes and compares the results from WMS and HWM. The last chapter gives the conclusions of this research and recommendations to future study.

# 2.0 THEORY AND METHODOLOGY

For many years, hydrologists have attempted to understand the transformation of precipitation to runoff, in order to forecast stream flow for purposes such as water supply, flood control, irrigation, drainage, water quality, power generation, recreation, and fish and wildlife propagation.

Since the 1930s, numerous rainfall-runoff models have been developed to simulate hydrologic cycle. As shown schematically in Figure 1, water precipitates from cloud to land surface; water evaporates from the land surface to become part of atmosphere. Precipitation may be intercepted by vegetation, become overland flow on the ground surface, infiltrate into the ground, flow through the soil as sub-surface flow and discharge into streams as surface runoff. Some of the intercepted water and surface runoff returns to the atmosphere through evaporation. The infiltrated water may percolate deeper to recharge groundwater. Groundwater may rise near to the land surface through capillary or be evaporated through vegetation root.

A watershed is a region of land where water drains down slope into a specified body of water, such as a river, lake, ocean or wetland. A watershed includes both the waterway and the land that drains to it. A watershed boundary is determined by its topographic characteristics. Ridges, hills and mountains often play the role of delineating a watershed from other watersheds.

Based on the description above, the main hydrological processes can be modelled by the following four modules. Each module can be simulated by several methods. This proposal considers only some representatives of each module.

1. Precipitation loss module;

2. Surface water flow module;
3. Channel flow module;
4. Base flow module;

## 2.1 PRECIPITATION LOSS MODELING

Rainfall-runoff model computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, transpired and subtracted from the precipitation. The loss can be broadly categorized into infiltration (down loss) and evaporation (up loss).

Infiltration from watershed area can be computed by the Horton Equation, Green-Ampt model and the Soil Conservation Service (SCS) curve number (CN) method (SCS, 1972).

The Horton model is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. The model describes the infiltration capacity as a function of time as:

$$f_p = f_\infty + (f_o - f_\infty) \cdot e^{-kt} \tag{2.1}$$

where:

$f_p$ = infiltration capacity into soil, mm/hr,

$f_\infty$ = minimum or ultimate value of $f_p$ (at $t = 0$), *mm/hr*,

$f_0$ = maximum or initial value of $f_p$ (at $t = 0$), *mm/hr*,

$t$ = time from beginning of storm, *hr*,

$k$ = decay coefficient, $hr^{-1}$.

This equation describes the exponential decay of infiltration capacity evident during heavy storms. Required parameters are $f_0$, $f_\infty$ and $k$. The actual values of $f_0$, $f_\infty$ and $k$ depend on the soil, vegetation, and initial moisture content. These parameters can be estimated using results from field infiltration-meter tests for a number of sites of the watershed and for a number of antecedent wetness conditions. If it is not possible to use field data to find estimates of $f_0$, $f_\infty$ and $k$, the guidelines given by the U.S. Environmental Protection Agency can be used (Huber et al, 1988).

The Green-Ampt equation (Green et al, 1911) for infiltration rate is

$$f(t) = K\left(\frac{\psi \cdot \Delta\theta}{F(t)} + 1\right) \qquad (2.2)$$

F(t) is the cumulated infiltration and can be expressed as

$$F(t) = K \cdot t + \psi \cdot \Delta\theta \ln\left(\frac{F(t)}{\psi \cdot \Delta\theta} + 1\right) \qquad (2.3)$$

where $\Delta\theta = (1 - s_e) \cdot \theta_e$;

$s_e$ = initial effective saturation, dimensionless, $0 \leq s_e \leq 1$;

$\theta_e$ = effective porosity, dimensionless, $0 \leq s_e \leq 1$;

$K$ = hydraulic conductivity, *m/hr*;

$t$ = infiltration time, *hr*;

$\Psi$ = wetting front soil suction head, *cm*;

The cumulated infiltration can be calculated by successive substitution using Equation 2.3. The infiltration parameters are given by Rawls et al (1983).

The Soil Conservation Service (SCS) curve numbers (CN) describe the surface's potential for generating runoff as a function of the soil type and land use on surface. Curve numbers range between $0 < CN \leq 100$, with 0 as the theoretic lower limit describing a surface that absorbs all precipitation, and 100 the upper limit describing an impervious surface such as asphalt, or water, where all precipitation becomes runoff. The

method computes the excess precipitation ($P_e$) generated for an incremental depth of precipitation falling on an area using the following relationship

$$P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad (2.4)$$

where $P$ = the incremental precipitation depth, inch;

$S$ = the potential maximum retention, inch.

The value $S$ is related to the curve number by

$$S = \frac{1000}{CN} - 10 \qquad (2.5)$$

where CN = the Curve Number, which is defined by SCS.

Equation 2.4 applies only for P ≥ 0.2S, otherwise all the precipitation is assumed lost to infiltration. The normal antecedent moisture conditions (AMC II) CN value is defined and tabulated by SCS based on different soil type and land use. For dry conditions (AMC I) or wet conditions (AMC III), equivalent curve numbers can be computed by

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)} \qquad (2.6)$$

and

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)} \qquad (2.7)$$

As a result, the SCS method provides the depth of excess precipitation resulting from a given depth of precipitation falling over an area during a specific time interval. The range of antecedent moisture conditions for each class is shown in Table 1.

**Table 1.** Classification of antecedent moisture condition (AMC) for the SCS method of rainfall abstractions (SCS, 1972)

| AMC Group | Total 5-day antecedent rainfall (inch) | |
|---|---|---|
| | Dormant season | Growing season |
| I | Less than 0.5 | Less than 1.4 |
| II | 0.5 to 1.1 | 1.4 to 2.1 |
| III | Over 1.1 | Over 2.1 |

In this research, SCS abstraction method is employed to obtain excess rainfall, which is used to generate hydrograph in later procedures. As we can see from above discussion, several parameters are needed in Horton model, such as, minimum or ultimate infiltration $f_\infty$, maximum or initial infiltration rate $f_0$, and decay coefficient $k$. More parameters are needed in Green-Ampt equation. These parameters are not available. In contrast, the only parameter in SCS CN method is the curve number, which can be found in National Engineering Handbook (SCS, 1972) based on soil type and land use. The land use and soil type are relatively easy to determine. The environmental research is performed according to PennDOT's need. PennDOT requires a pragmatic model, which they can operate, transfer to other projects, or make revision after the model has been built. Thus, SCS CN method is selected according to project's practical situation.

Evaporation is considered as a loss "of the top." That is, evaporation is subtracted from rainfall depths prior to calculating infiltration. Thus, subsequent use of the symbol $i$ for "rainfall intensity" is really rainfall intensity minus evaporation rate. The loss rate is computed by

$$e_v = A \cdot e_d / k_{\text{cov}} \qquad (2.8)$$

where:

$e_v$ = evaporation loss rate, $ft^3/day$,

$A$ = surface area at the water level in the unit, $ft^2$,

$e_d$ = evaporation rate, *inch/day*, and

$k_{\text{cov}}$ = evaporation conversion factor.

The values of $e_d$ should be supplied for each interval of the simulation period.

## 2.2 DISTRIBUTED SURFACE WATER MODELING

The distributed surface water modeling is based on differential equations that allow the flow rate and water level to be computed as functions of space and time, rather than of

time alone as in the lumped models. The conceptual view of distributed surface runoff is illustrated in Figure 2. Each watershed surface is treated as a nonlinear reservoir. Inflow comes from precipitation and upstream watersheds. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, *Q*, occurs only when the depth of water in the "reservoir" exceeds the maximum depression storage, *dp*, in which case the outflow is given by Manning's equation.

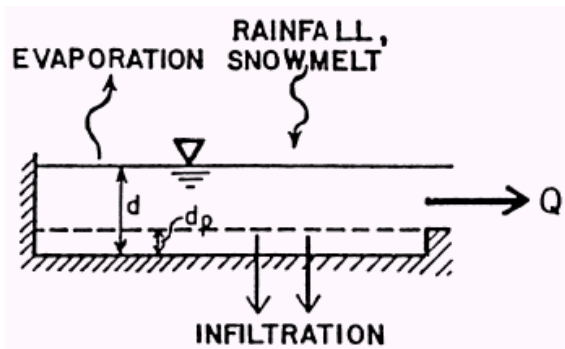



**Figure 2.** Conceptual view of surface runoff

Depth of water over the sub-watershed (*d* in *inches*) is continuously updated with time (*t* in seconds) by solving numerically a water balance equation over the sub-watershed.

The non-linear reservoir is established by coupling the continuity equation with Manning's equation. Continuity may be written for a sub-area as:

$$\frac{dV}{dt} = A\frac{dD}{dt} = A \cdot i^* - Q \qquad (2.9)$$

where $V = A \cdot D$ = volume of water on the sub-area, $ft^3$,

$D$ = water depth, *inch*,

$t$ = time, *sec*,

$A$ = surface area of sub-area, $ft^2$,

$i^*$ = rainfall excess = rainfall/snowmelt intensity minus evaporation/infiltration rate, *inch/sec*,

$Q$ = outflow rate, *cfs*.

The outflow is generated using Manning's equation

$$Q = \frac{1}{n} \cdot D \cdot R^{2/3} \cdot S^{1/2} \qquad (2.10)$$

where

$n$ = manning's roughness coefficient,

$D$ = depth of depression storage, *ft*,

$R$ = hydraulic radius, $R = Area / WetPerimeter$, *ft*

$S$ = sub-watershed slope, *ft/ft*.

Equations (2.9) and (2.10) may be combined into one non-linear differential equation that may be solved for one unknown, the depth, *d*. This produces the non-linear reservoir equation

$$\frac{dD}{dt} = i * - \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2} \qquad (2.11)$$

Equation (2.11) is solved at each time step by means of a simple finite difference scheme (Huber et al, 1988).

## 2.3 LUMPED SURFACE WATER MODELING

The lumped surface water modeling is accomplished by the unit hydrograph method. The unit hydrograph of a watershed is defined as a hydrograph resulting from one inch spatially-uniform excess rainfall over the watershed at a constant rate for an effective duration. Several types of unit hydrographs, such as Snyder unit hydrograph, Clark unit hydrograph, SCS dimensionless unit hydrograph, have been developed.

The SCS dimensionless hydrograph is a synthetic unit hydrograph in which the discharge is expressed as a ratio of q to peak discharge $q_p$ and the time as the ratio of time $t$ to the time of rise of the unit hydrograph, $T_p$. Figure 3 (a) shows the SCS dimensionless hydrograph. The values of $q_p$ and $T_p$ may be estimated using a model of a triangular unit

hydrograph, which is shown in Figure 3 (b), where the time is in hours and the discharge in $m^3/s.cm$ or *cfs/in* (Source: Soil Conservation Service, 1972).

The Soil Conservation Service suggests the time of recession may be estimated as $1.67T_p$. $q_p$ can be expressed as

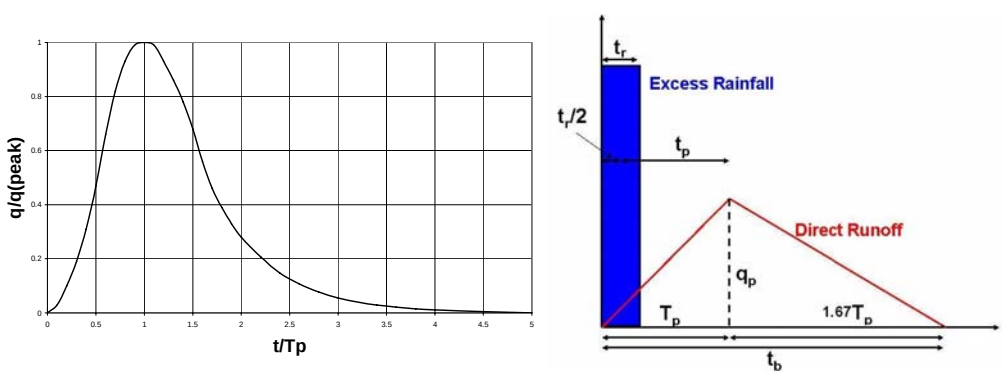
$$q_p = \frac{CA}{T_p} \qquad (2.12)$$

where $C$ = 2.08 (483.4 in the English system) and $A$ = the drainage area in square kilometers (square miles).

The time of rise $T_p$ can be expressed in terms of lag time $t_p$ and the duration of effective rainfall $t_r$.

$$T_p = \frac{t_r}{2} + t_p \qquad (2.13)$$

The lag time $t_p$ can be approximately calculated by $t_p \cong 0.6T_c$, where $T_c$ is the time of concentration of the watershed.




(a) (b)

**Figure 3.** Soil Conservation Service synthetic unit hydrograph
(a) Dimensionless hydrograph (b) triangular unit hydrograph
(Source: Soil Conservation Service, 1972)

Once the unit hydrograph has been determined, it may be applied to find the direct runoff and streamflow hydrograph. For calculation convenience, the time interval used in

defining the excess rainfall hyetograph ordinates should be the same as that for which the unit hydrograph was specified. The discrete convolution equation

$$Q_n = \sum_{m=1}^{n \le M} P_m \cdot U_{n-m+1} \qquad (2.14)$$

can be used to yield the direct overland runoff hydrograph,

where $P$ = The excess rainfall;

$U$ = The unit hydrograph;

$n$ = The $n$th time interval (recording runoff and unit hydrograph);

$m$ = The $m$th time interval (recording rainfall);

$M$ = The total number of time interval (recording runoff);

## 2.4 DISTRIBUTED CHANNEL ROUTING

Similar to surface water distributed model, the distributed channel flow model is based on partial differential equations (the Saint-Venant equations for one-dimensional flow) that allow the flow rate and water level to be computed as functions of space and time. They describe the passage of a flood wave down a section of reach both in space and time. On the contrary, the lumped model does not use the Saint-Venant equations and only considers time factor for solutions.

The Saint-Venant equations include the continuity equation and momentum equation. In complete form, the Saint Venant equations are (Chow et al, 1988)

Continuity equation: $$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (2.15a)$$

Momentum equation: $$\underbrace{\frac{1}{A} \cdot \frac{\partial Q}{\partial t}}_{\text{Local acceleration term}} + \underbrace{\frac{1}{A} \cdot \frac{\partial}{\partial x} \cdot \left( \frac{Q^2}{A} \right)}_{\text{Convective acceleration term}} + \underbrace{g \frac{\partial y}{\partial x}}_{\text{Pressure force term}} - \underbrace{g S_0}_{\text{Gravity force term}} - \underbrace{g S_f}_{\text{Friction force term}} = 0 \qquad (2.15b)$$

The Saint Venant equations have three simplified forms, i.e. dynamic wave model, diffusion wave model, and kinematic wave model. Each of them defines a one-dimensional distributed routing model.

The dynamic wave model considers all the acceleration and pressure terms in the momentum equation. The accounted terms are labeled in Equation 2.15b.

The diffusion wave model neglects the local and convective acceleration terms but incorporates the pressure terms. The simplified momentum equation of diffusion wave model is

$$\underbrace{g\frac{\partial y}{\partial x}}_{\text{Pressure force term}} - \underbrace{g\left(S_0 - S_f\right)}_{\text{Gravity force term}} = \underbrace{0}_{\text{Friction force term}} \qquad (2.16)$$

The kinematic wave model is the simplest of the distributed model. It neglects the local acceleration, convective acceleration and pressure terms in the momentum equation. The simplified momentum equation of kinematic wave model is

$$\underbrace{g\left(S_0 - S_f\right)}_{\text{Gravity force term}} = \underbrace{0}_{\text{Friction force term}} \qquad (2.17)$$

It assumes $S_0 = S_f$ and the friction and gravity forces balance each other.

Solutions to the distributed model can be found in many references (Fread 1973, Chow et al. 1988).

## 2.5 LUMPED CHANNEL ROUTING

Level pool routing method, linear reservoir model and the Muskingum method belong to lumped flow routing. The Muskingum method is a commonly used hydrologic routing

method for handling a variable discharge-storage relationship. The model considers two components of storage, wedge and prism. During the advance of a flood wave, inflow exceeds outflow, producing a wedge of storage. During the recession, outflow exceeds inflow, resulting in a negative wedge. The prism is formed by a volume of constant cross section along the length of prismatic channel. Figure 4 shows the prism and wedge storage in a channel reach.

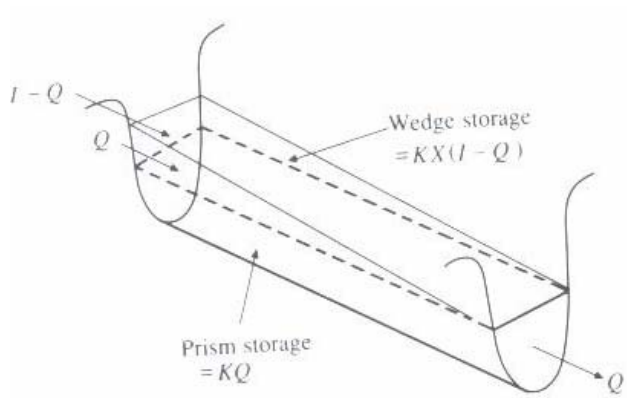



**Figure 4.** The prism and wedge storage in a channel reach
(Chow et al, 1988)

The total storage is the sum of two components

$$S = KQ + KX(I-Q) \qquad (2.18a)$$

which can be rearranged to be the storage function

$$S = K[XI + (1 - X)Q] \qquad (2.18b)$$

Equation 2.16b represents a linear model for routing flow in streams.

The channel outflow is expressed as

$$Q_{j+1} = C_1 I_{j+1} + C_2 I_j + C_3 Q_j \qquad (2.19)$$

where $C_1$, $C_2$, $C_3$ are Muskingum coefficient and defined as

$$C_1 = \frac{\Delta t - 2KX}{2K(1-X) + \Delta t} \qquad (2.20a)$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1-X) + \Delta t} \qquad (2.20b)$$

$$C_3 = \frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t} \qquad (2.20c)$$

## 2.6 TOPOGRAPHICALLY BASED BASE-FLOW MODELING

In areas that have much vegetation or consist of sandy soils, base-flow becomes important in total water balance. The representative topographical based base-flow model is TOPMODEL (Beven, 1995). Figure 5 illustrate the term definitions of TOPMODEL in a flow strip.

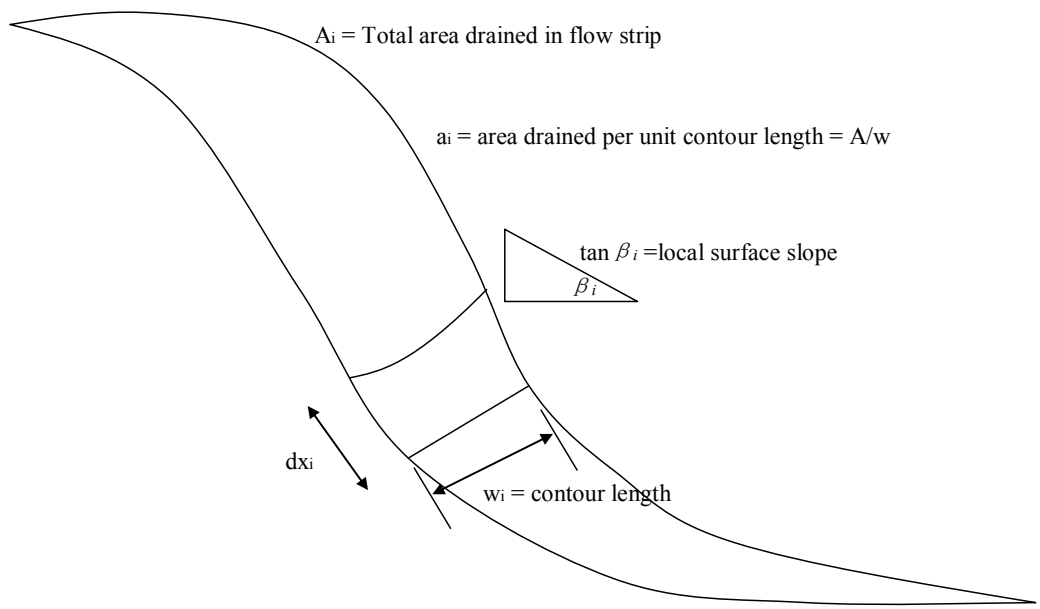



**Figure 5.** Definition sketch for TOPMODEL flow strip
(Kirkby, 1997, Modified)

TOPMODEL uses four basic assumptions to relate local downslope flow from a point to discharge at the watershed outlet.

Assumption 1: The dynamics of the saturated zone are approximated by successive steady state representations.

Assumption 2: The recharge rate $r$ ($m/h$) entering the water table is spatially homogeneous.

Assumption 3: The effective hydraulic gradient of the saturated zone is approximated by the local surface topographic gradient $\beta$.

Assumption 4: The distribution of downslope transmissivity $T_0$ ($m^2/h$) with depth is an exponential function of storage deficit.

TOPMODEL uses the distribution of the topographic index as an index of hydrological similarity, which indicates the propensity of landscape areas to become wet.

$$\lambda_i = \ln\left(\frac{a_i}{\tan\beta_i}\right) \qquad (2.21)$$

where $a_i$ = the area draining through a grid square per unit contour length;

$\tan\beta_i$ = the local surface slope;

Under assumption 1 and assumption 2, the downslope subsurface flow rate per unit contour length $q_i$ ($m^2/h$) is:

$$q_i = ra_i \qquad (2.22)$$

where $r$ = the recharge rate;

Under assumption 3 and assumption 4, $q_i$ ($m^2/h$) is also:

$$q_i = T_0 e^{-Si/m} \tan\beta_i \qquad (2.23)$$

where $T_0$ = the lateral downslope transmissivity when the soil is just saturated;

$S_i$ = local storage deficit;

$m$ = model parameter controlling the rate of decline of transmissivity with increasing storage deficit.

By combining Equation (2.22) and Equation (2.23), the local soil moisture deficit $S_i$ can be derived:

$$S_i = -m\ln\left(\frac{ra_i}{T_0\tan\beta_i}\right) \qquad (2.24)$$

The mean watershed storage deficit $\overline{S}$ is obtained by integrating Equation (2.24) over the entire area $A_i$ of the watershed:

$$\overline{S} = \frac{1}{A}\sum_i A_i\left(-m\cdot\ln\frac{ra_i}{T_0\cdot\tan\beta_i}\right) \qquad (2.25)$$

where $A_i$ = the fractional area of the topographic index class $i$.

Assuming that the water table recharge and the soil transmissivity are spatially constant, then *ln r* and *ln* $T_0$ are eliminated from Equation (2.25) and $S_i$ is expressed as:

$$S_i = \overline{S} + m \cdot \left( \overline{\lambda} - \ln \frac{a_i}{\tan \beta_i} \right) \qquad (2.26)$$

where $\overline{\lambda}$ is the areal average of the topographic index:

$$\overline{\lambda} = \frac{1}{A} \cdot \int_0^A \ln \left( \frac{a_i}{\tan \beta_i} \right) \cdot A_i \cdot dA \qquad (2.27)$$

At each topographic index class $\lambda_i$, unsaturated and saturated zone fluxes are modeled. Figure 6 shows the schematic diagram of the representation of the local storage deficit, $S_i$, for different topographic indices.

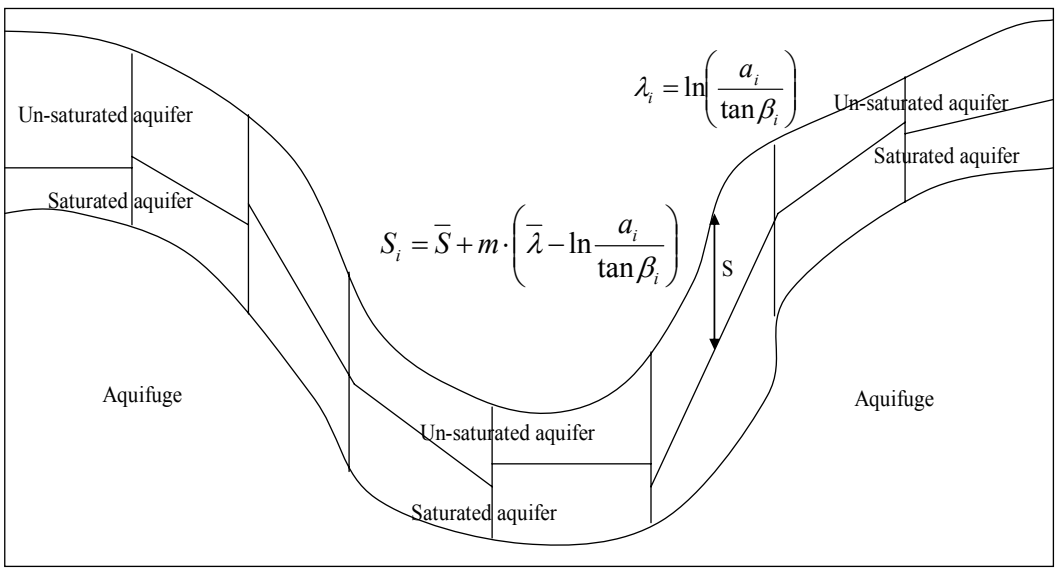



**Figure 6.** The schematic diagram of the representation of the local storage deficit for different topographic indices (Campling, 2002, Modified)

The vertical drainage $q_v$ from the unsaturated store at any point *i* is controlled by the local saturated zone deficit $D_i$, which depends on the depth of the local water table (Beven and Wood, 1983):

$$q_v = \frac{S_{uz}}{D_i \cdot t_d} \qquad (2.28)$$

where $S_{uz}$ = the storage in the unsaturated zone;

$t_d$ = time delay constant that introduces longer residence times to cater for deep water table levels.

The watershed flux of water entering the water table, $Q_v$ is calculated by assuming the $q_v$ of each topographic index class:

$$Q_v = \sum_i q_v A_i \qquad (2.29)$$

Output from the saturated store is represented by the base-flow term, $Q_b$, which can be calculated using a subsurface storage deficit-discharge function of the form:

$$Q_b = Q_0 \cdot e^{-\overline{S}/m} \qquad (2.30)$$

where $Q_0 = A \cdot e^{-\overline{\lambda}}$ is the discharge when $\overline{S}$ is zero.

The watershed average deficit $\overline{S}$ is updated by subtracting the unsaturated zone recharge and adding the base-flow from the previous time step:

$$\overline{S}_t = \overline{S}_{t-1} + [Q_{bt-1} - Q_{vt-1}] \qquad (2.31)$$

The initial base-flow $Q_0$ and the initial root zone storage deficit $S_{r0}$ are input at the start of the modeling.

Experience in modeling the Booro-Borotou watershed in the Cote d'Ivoire (Quinn, 1991), and watersheds in the Prades mountains of Cataluna, Spain, suggests that TOPMODEL will only provide satisfactory simulations once the watershed has wetted up. In many watersheds that tend to receive precipitation in short, high intensity storm, or receive low precipitation, the soil seldom reach a "wetted" state, and the response may be controlled by the connectivity of any saturated downslope flows. Short, high intensity storm may lead to the production of infiltration excess overland flow, which is not usually included in TOPMODEL. The watersheds in I-99 Environmental Research often receive such kind of rainfall. Some assumptions are violated in the studied watershed, so the TOPMODEL is not used in the studied watershed.

Another reason not to use TOPMODEL is that many parameters are difficult to obtain in I-99 Project. For example, one of the key parameters in TOPMODEL is the index of hydrological similarity $\lambda_i$ of each grid square, which depends on the area drained per unit contour length $a_i$ and the local surface slope $\tan\beta_i$. To determine $\lambda_i$ of each grid square, $a_i$ and $\tan\beta_i$ of each grid square are needed. This includes tremendous calculation, which is not practical for I-99 Project.

## 2.7 SIMPLER LUMPED BASED-FLOW MODELING

A simpler lumped model -- the exponential recession model, is available to describe the base-flow. The recession model has been used to explain the drainage from natural storage in a watershed (Linsley et al, 1982). It defines the relationship of $Q_t$, the base flow at anytime $t$, to an initial value as:

$$Q_t = Q_0 k^t \qquad (2.32)$$

where:

$Q_0$ = initial base-flow (at time zero),

$k$ = recession constant.

The base-flow is illustrated in Figure 7. The shaded region represents base-flow in this figure; the contribution decays exponentially from the starting flow. The total flow is the sum of the base-flow and the direct surface runoff.

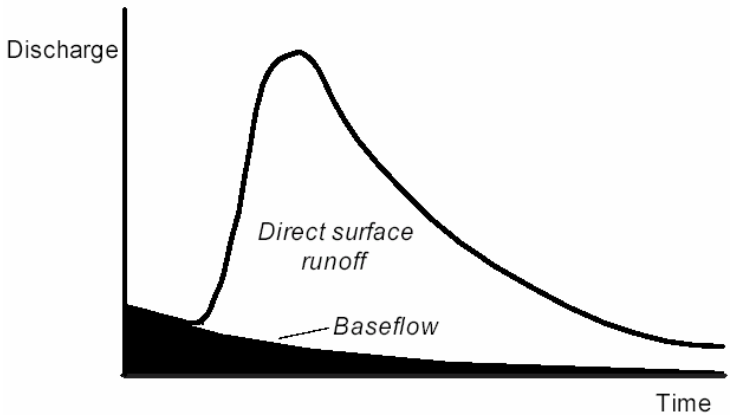



**Figure 7.** Initial base-flow recession

$k$ is defined as the ratio of the base-flow at time $t$ to the base-flow one day earlier. The starting base-flow value $Q_0$ is an initial condition of the model. It may be specified as a flow rate ($ft^3/s$), or it may be specified as a flow per unit area ($ft^3/s/mile^2$).

## 2.8 GIS-BASED WATERSHED MODELING

With the development of computer science, hydrological models have been combined with Geographic Information System (GIS) technology. GIS is a special type of information system in which the data source is a database of spatially distributed features and procedures to collect, store, retrieve, analyze, and display geographic data. In other words, a key element of the information used by utilities is its location relative to other geographic features and objects (Shami, 2002). It combines spatial locations with their corresponding various information.

GIS is a class of concepts instead of one product. There are many kinds of GIS data, which are supported by different software packages. They may not be compatible with each other. Shape files represent city, park and airport using point feature. They represent road, river and pipe using polylines features. They also represent watershed, lake and country using polygon features. This is not always the case. In large scale, for example, a

city or an airport is often represented by polygons. A feature object comprises an entity with a geographic location, typically describable by points, arcs, or polygons.

On the contrary, Grid files represent everything using equal dimensional pixels. In a certain scale map, a large object is represented using more pixels; while a small object is represented using fewer pixels.

Digital Elevation Models (DEM) are regular grid data structures that contain two-dimensional arrays of elevations where the spacing between elevations is constant in the x and y directions. In this manner, it is like a grid file. The resolutions of DEM are normally $30 \times 30$ square meters or $90 \times 90$ square meters.

Triangulated Irregular Networks (TIN) is another type of digital elevation map. A TIN is built from a series of irregularly spaced points with elevations that describe the surface at that point. In contrast to DEM, the elevation points are irregularly distributed. From these points, a network of linked triangles is constructed. Adjacent triangles, sharing two nodes and an edge, connect each other to form a surface. A height can be calculated for any point on the surface by interpolating a value from the nodes of nearby triangles. In addition, each triangle face has a specific slope and aspect. TIN can be used for visualization, as background elevation maps for generating new TIN or DEM, or perform basin delineation and drainage analysis.

A TIN file is flexible in representing different variation terrain. If a watershed elevation varies too much in a small area, more points are needed for accuracy purpose. However, if a watershed elevation is very flat in a large area, fewer points can be used to save storage space. On the contrary, DEM always represents a watershed using identical-sized square pixels.

In essence, a raster file representing elevation is similar to DEM and Grid. A raster file is also a regular grid data structure that contains a two-dimensional array of elevations. However, since raster files, grid files and DEM files are developed by

different agencies, they are processed digitally differently. Vector files represent objects similar to shape files. Vector files may also represent rivers using polyline features and represent watersheds using polygon features. However, vector files and shape files are processed digitally computer differently.

Shape files, grid files, Triangulated Irregular Network (TIN) and Digital Elevation Model (DEM) files are supported by ArcGIS (Environmental Systems Research Institute, 1999). DEM files, Raster files and Vector files are supported by IDRISI (Eastman, 1999). DEM files, shape file and another type of TIN files are supported by Watershed Modeling System (Environmental Modeling Research Laboratory, Brigham Young University, 1999). TIN files used in WMS are incompatible with TIN files used in ArcGIS. DEM files used in ArcGIS are realized through grid files and are incompatible with DEM files used in IDRISI and WMS.

GIS-based hydrological models utilize readily available digital geospatial information more expediently and more accurately than manual-input methods. Also, the development of basic watershed information will help the user to estimate hydrologic parameters. After obtaining adequate experience in GIS-generated parameters, users can make parameter estimation more efficiently. GIS-based hydrological models may use different GIS data as different layers. To make different GIS data display and work in the correct location, coincident spatial referencing is needed for different layers.

HEC-GeoHMS was developed by U.S. Army Corps of Engineers, Hydrological Engineering Center (U. S. Army Corps of Engineers, Hydrologic Engineering Center, 2003). HEC-GeoHMS links GIS tool (ArcView3.2) and hydrologic model (Hydrologic Modeling System - HMS). HEC-GeoHMS combines the functionality of ArcInfo programs into a package that is easy to use with a specialized interface. With the ArcView capability and the graphical user interface, the user can access customized menus, tools and buttons instead of the command line interface in ArcInfo. The hydrologic algorithms in the model are the same as HEC-HMS. First, HEC-GeoHMS imports DEM data and fills sinks in the data. Second, it generates flow direction and

streams based on DEM data. Then the following procedure is to delineate watershed and sub-watershed boundaries. The newly generated files are stored in separate layers. The pertinent watershed characteristics can be extracted from the source DEM data and the generated stream and boundary data. After these processes, a HEC-HMS schematic map and project can be exported. Other parameters, such as meteorological, routing and infiltration parameters, need to be set before the project runs. In fact, HEC-GeoHMS prepares the input file and schematic map for HEC-HMS. By using GIS data, detailed watershed characteristics are obtained automatically for the HEC-HMS model. However, the source GIS files, such as DEM file, are not generally available and difficult to generate from the beginning.

Quimpo et al. (2003) develop a quasi-distributed GIS-based hydrologic model (QD-GISHydro). The model consists of several separate modules, which process data describing the spatial variation of watershed properties, and compute the runoff time series at the watershed outlet. Data processing and visualization is handled primarily by GIS software IDRISI32, while self-written computer programs perform the bulk of the computations. The model is designed to operate as simply and generally as possible, requiring only four external data sets as input (DEM, land coverage, soil coverage, and incremental precipitation depths) to create all other data needed to compute the direct runoff for the watershed under study. The model is able to deal with non-uniform excess rainfall for each watershed pixel. Land use and soil type files are available for most of United State watersheds. Incremental precipitation depth files can be created by the user easily. Unfortunately, DEM file is not always available and the model can only process DEM file as source elevation information. Users cannot build a research model without source DEM file.

WMS was developed by Environmental Modeling Research Laboratory of Brigham Young University. It is flexible in creating a practical model using various input source data, or even creating a model from the scratch. WMS is able to use Shape file, ArcInfo Grid file, DEM or TIN source data to create watershed delineation. In case of none of the source file is available, users can import aerial photographs or even scanned watershed

maps as referenced spatial data. Although some GIS characteristics are missing in this case, users are able to build a flexible, practical model for watersheds where source data are not adequate. GIS data such as land use and soil type files can be created by the user or imported from readily available data. These techniques greatly enhance the applicability of WMS in watershed modeling and make WMS not only a research package, but a pragmatic tool in watershed modeling project.

Another GIS environmental modeling package is Better Assessment Science Integrating Point and Non-point Sources (BASINS). BASINS was developed by the U. S. Environmental Protection Agency (EPA). It takes advantage of developments in software and data management technologies and uses the ArcView3.X as the integrating framework to provide the user with a comprehensive watershed management tool. In this manner, it is like HECGEOHMS. However, BASINS focus on point and non-point pollutant modeling instead of watershed modeling.

# 3.0 GIS-BASED HYDROLOGICAL MODEL - WMS APPROACH

As discussed in Chapter Two, WMS is flexible in creating a practical model using various input source data, or even creating a model from the beginning without any topological GIS data. It may also be used for dealing with constructed watersheds. In this environmental impact of highway assessment research, WMS was tested because it has many desirable features. This chapter will discuss the features of WMS model. The author has devoted a lot of effort in examing whether it is suitable for the project.

## 3.1 WMS INTRODUCTION

WMS provides a platform for using various models, such as HEC-1, National Flood Frequency Program (NFF), HEC-RAS (River Analysis System), Hydrological Simulation Program - FORTRAN (HSPF), and CE-Qual-W2, etc. Each model is designed for particular purpose. HEC-1 was developed by The Hydrologic Engineering Center (HEC), U.S. Army Corps of Engineering. It performs flood hydrograph computations associated with a single recorded or hypothetical storm. The main purpose in Task B of I-99 project is to model the surface water in the construction site watershed. Therefore, model HEC-1 in the WMS package is mainly employed in this research.

## 3.2 COORDINATE SYSTEM SETTING AND CONVERSION

A digital image is very important in building WMS model. An image consists of a collection of pixels, each of which has its own value. The resolution of the pixel

determines the area and detail represented in the image. Images in WMS are used to derive data such as pipes, streams, confluences, land use, and soil type files, etc. It also provides a background map to the watershed. In order to use the image to represent proper length, area and orientation, the user must geo-reference the image. Geo-referencing an image defines x and y coordinates so that distances, areas and orientations computed from the image are correct.

To geo-reference the image correctly, users need to know the coordinates of two or three points on the image. The coordinates can be the Universal Transverse Mercator (UTM) system, the geographic system, state plane or the local system. Once the coordinate of two or three points are determined, all other points in the map can be determined according to the relative position of these points. Different coordinate systems can be converted to each other if necessary. Geo-referencing is extremely important if multiple function layers are imported to represent the same region. In this situation, correct geo-referencing guarantees the coincidence of the same points, lines and polygons in the different layers.

## 3.3 SOURCE DATA IMPORTING AND CREATING

Digital Elevation Models (DEM) is a commonly used digital elevation source and an important part of using WMS for watershed characterization. Many agencies provide DEM data. These include the Meteorological Resource Center (http://www.webgis.com), U.S. Geology Survey (http://gisdata.usgs.net), GeoCommunity/GIS Data Depot (http://www.gisdatadepot.com), Pennsylvania Spatial Data Access (http://www.pasda.psu.edu), etc. Triangulated Irregular Networks (TIN) is another type of digital elevation map. TIN can be used for visualization, as background elevation maps for generating new TIN or DEM, or perform basin delineation and drainage analysis in WMS.

After DEM or TIN files are imported into WMS, users can modify the elevation value of a certain point in WMS. TIN files are not widely published on the web. Therefore, it is more difficult to find the proper TIN file than DEM file. Fortunately, TIN and DEM can be converted to each other. Theoretically, users can also create TIN file from the beginning manually. However, since watershed normally consists of thousands of points, creating TIN files from scratch is not realistic.

## 3.4 WATERSHED DELINEATION

Both DEM and TIN are used as elevation information files to delineate a watershed. First, flow directions for individual DEM cells or TIN vertices are created. By creating an outlet on a down stream branch, stream network and watershed boundary are generated based on flow direction and stream threshold. After the watershed is delineated, the basin's characters, such as area, basin's slope, maximum flow distance, etc., can be calculated automatically by WMS. If the user is not satisfied with the automatically generated streams or watershed boundaries, he can add streams and outlet manually. Also, users are free to combine adjacent sub-watersheds into one and divide a big sub-watershed into several. However, all operations must have solid practical basis instead of only imagination. An example of watershed delineation using DEM and TIN file is shown in Figure 8 and Figure 9, respectively.

Although it is convenient to generate watershed delineation using DEM or TIN files, there are some limitations. In a real watershed, there may be some spikes or pits. A spike is a cell whose elevation is higher than the surrounding cells. A pit is a cell whose elevation is lower than the surrounding cells. Flat terrains, flat channels also exist in reality. While a spike is generally not a serious problem in hydrologic analysis, a pit or flat element on the other hand can be. This can be problematic because the water flows into the pit and never flows out. Thus, the flow path is discontinued. The model cannot determine the flow direction if flat terrains are encountered. In order to perform any type of hydrologic analysis, pits and flat elements must be removed. WMS program has tools

to remove a small amount of error elements by changing their values through interpolation.

However, these tools are only useful when a few errors happen. If huge numbers of pits, flat terrains congregate together to form a big pond, their elevation values cannot be altered though interpolation. Thus, they cannot be removed easily. Actually, they should NOT be altered if they form a big pond. Once they are altered, the model can not reflect the real watershed characteristics. While pits and flat terrains are very common in reality, WMS cannot deal with it very well. Sometimes, users need to find out other methods to solve this problem.

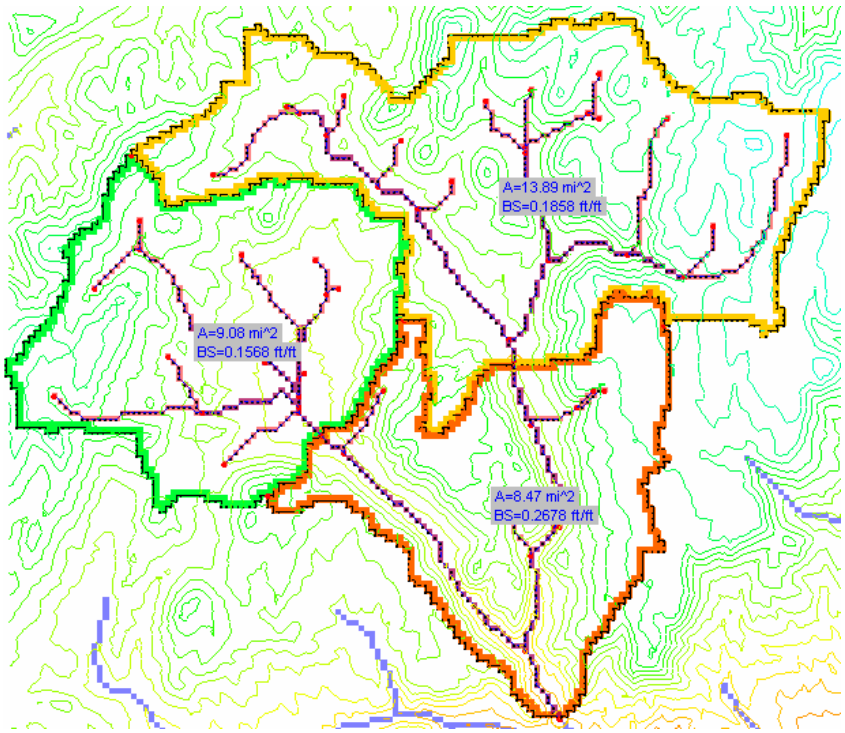



**Figure 8.** An example of watershed delineation using DEM

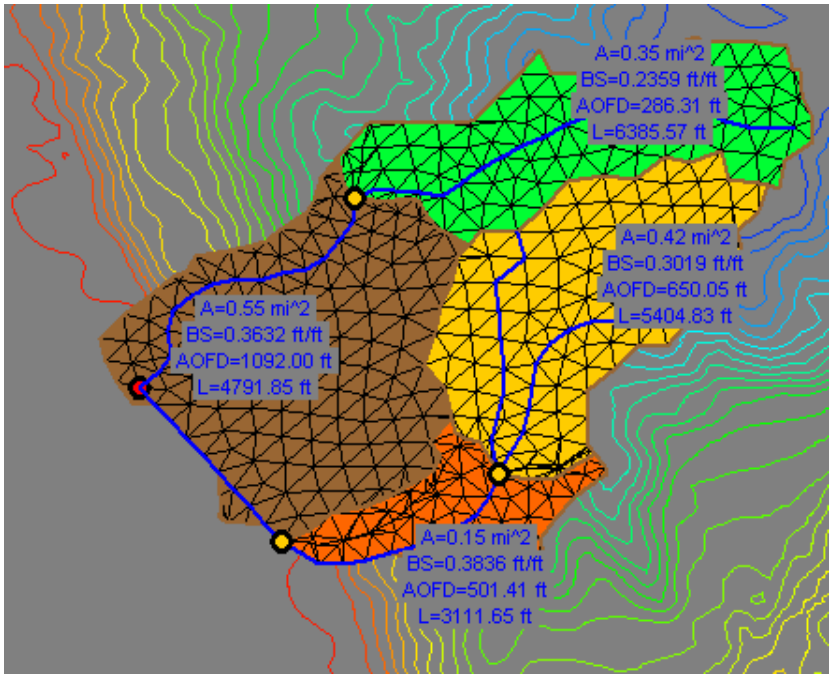



**Figure 9.** An example of watershed delineation using TIN

## 3.5 DRAINAGE COMPONENT EDITING

As stated above, automatic delineation is not enough even for identifying a watershed. Some model components must be edited manually.

Streams can be generated automatically or drawn by users. Automatic streams are generated from DEM or TIN files. If the automatically generated streams are not enough, users need to draw the stream manually. Streams are abstracted to be polylines in WMS. Several types of polylines are employed in WMS model. They are generic, stream, pipe, lake and ridge. After drawing a polyline using draw line tool, user must assign it to be stream type. The drawing direction of a stream must from downstream to upstream. Other types of lines also should be assigned to proper line type.

The basins boundaries are polygon-shaped. Polygons are categorized into the following types: generic, lake, reservoir and drainage boundary. Polygons cannot be

drawn directly in WMS. User needs to draw a closed polyline, assign it to be generic line, and then build a polygon and assign it to be a drainage boundary type.

The outlet is a conceptual point connecting upstream flow and downstream flow. Points are categorized into generic, drainage outlet and route point. The point type should be assigned to a drainage outlet. One outlet must be accompanied with a basin to form the correct watershed model schematic structure. A reservoir works as water storage or flood detention structure in the watershed. Although an actual reservoir has a certain area and shape, its function can be abstracted into an outlet in WMS model. The reservoir's characteristics are input into outlet routing strategies. Thus, an outlet contains not only channel routing, but also reservoir routing strategies. A reservoir is not required in every outlet. If an outlet does not have a reservoir, only channel routing method is defined for the outlet.

## 3.6 IMPORTING AND CREATING OF LAND USE AND SOIL DATA

Land use file is a polygon schematic map overlapped onto the watershed boundary. It has land use data properties and is defined in Land Use Layer. Similarly, soil data file is also a polygon schematic map overlapped onto the watershed boundary. It has soil data properties and is defined in Soil Data Layer. Land use and soil type files are used to calculate the composite curve number of sub-watershed. Most publicly available land use and soil data files are in Shape file format. First, users should import them into WMS; then connect the Shape files with their database to obtain land use and soil digital information. Finally, the Shape files must be converted to feature objects, which are actually used in WMS. Land use is classified into 20 types, while soil type is classified into four types by SCS (1972). The relationship between curve number and land use, soil type is described in Chapter Two. Figure 10, Figure 11, Figure 12, and Figure 13 are illustrations of land use, soil type map and their corresponding tables, respectively. The polygons in Figure 10 and Figure 12 represent different land use and soil type patches.

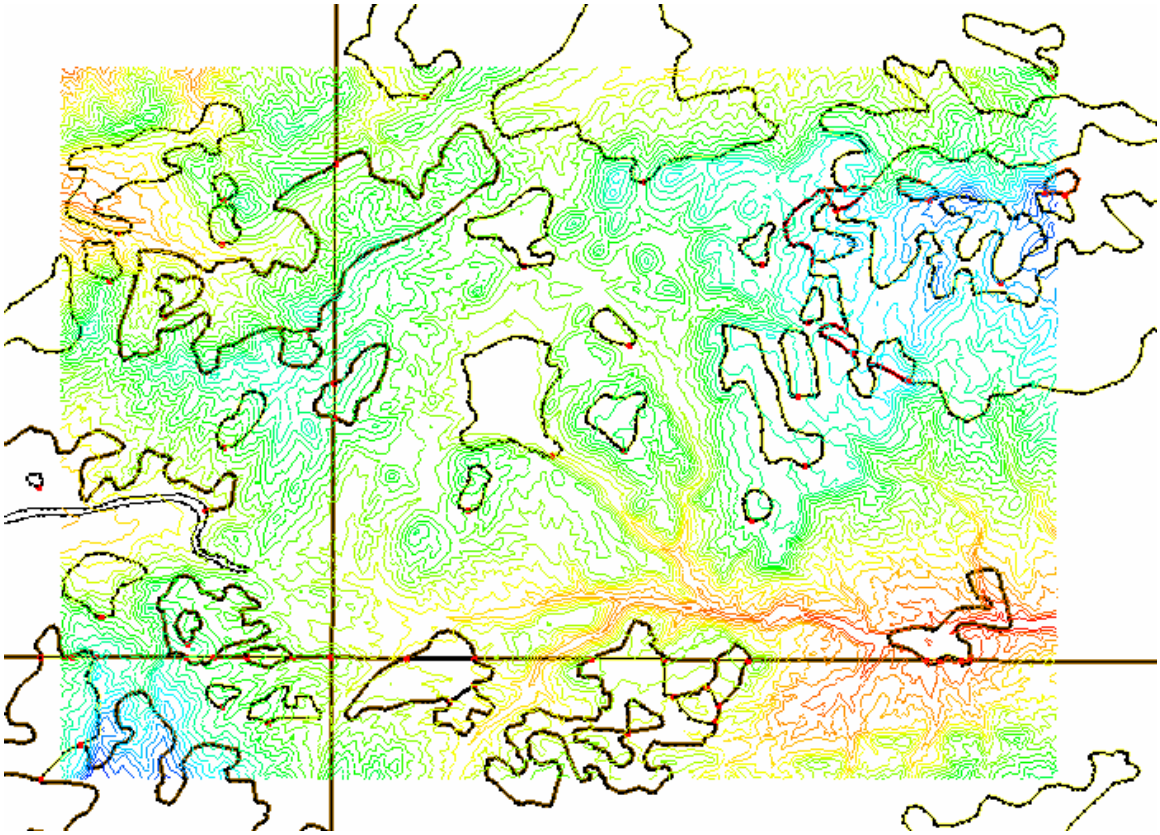


**Figure 10.** Illustration of land use map

Attributes

| | AREA | PERIMETER | L_RICHUT_ | L_RICHUT_I | LUCODE | LEVEL2 |
|---|---|---|---|---|---|---|
| 1 | 0.00001 | 0.01577 | 960 | 42 | 42 | EVERGREEN FOREST LAND |
| 2 | 0.00007 | 0.04109 | 959 | 42 | 42 | EVERGREEN FOREST LAND |
| 3 | 0.00015 | 0.08092 | 958 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 4 | 0.00004 | 0.03276 | 957 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 5 | 0.00003 | 0.02251 | 956 | 33 | 33 | MIXED RANGELAND |
| 6 | 0.00013 | 0.04703 | 955 | 33 | 33 | MIXED RANGELAND |
| 7 | 0.00004 | 0.02518 | 954 | 21 | 21 | CROPLAND AND PASTURE |
| 8 | 0.00031 | 0.10722 | 953 | 43 | 43 | MIXED FOREST LAND |
| 9 | 0.00021 | 0.06161 | 952 | 33 | 33 | MIXED RANGELAND |
| 10 | 0.00017 | 0.06353 | 951 | 43 | 43 | MIXED FOREST LAND |
| 11 | 0.00010 | 0.05931 | 950 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 12 | 0.00009 | 0.03868 | 949 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 13 | 0.00014 | 0.08247 | 948 | 21 | 21 | CROPLAND AND PASTURE |
| 14 | 0.00006 | 0.03269 | 947 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 15 | 0.00012 | 0.04243 | 946 | 32 | 32 | SHRUB & BRUSH RANGELAND |
| 16 | 0.00013 | 0.05665 | 945 | 32 | 32 | SHRUB & BRUSH RANGELAND |

Help 959 records. OK

**Figure 11.** Attribute table for land use map

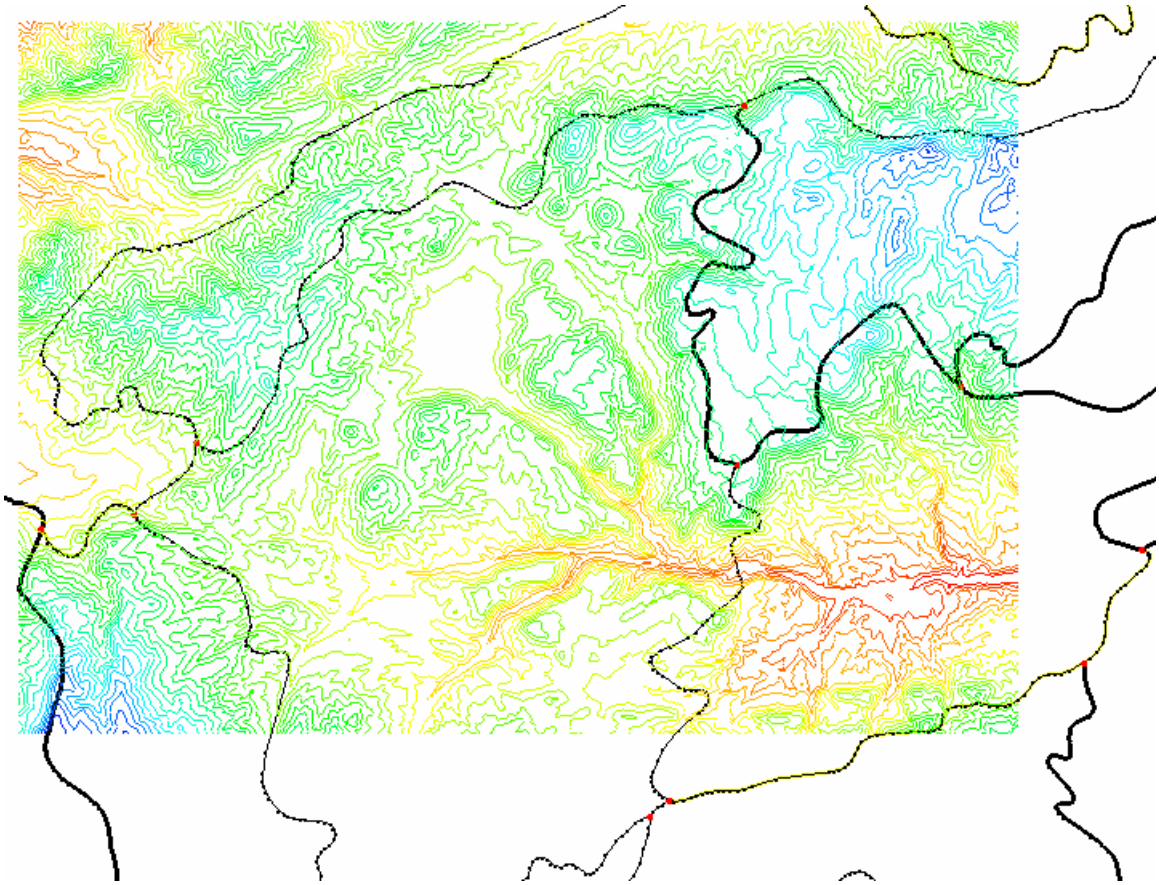


**Figure 12.** Illustration of soil type map

Attributes

| | AREA | PERIMETER | MUID | HYDGRP |
|---|---|---|---|---|
| 1 | 0.015 | 1.161 | UT755 | B |
| 2 | 0.015 | 0.853 | UT516 | C |
| 3 | 0.002 | 0.338 | UT517 | B |
| 4 | 0.007 | 0.639 | UT512 | C |
| 5 | 0.010 | 0.718 | UT509 | B |
| 6 | 0.005 | 0.721 | UT525 | B |
| 7 | 0.004 | 0.528 | UT526 | B |
| 8 | 0.010 | 0.681 | UT758 | B |
| 9 | 0.009 | 0.911 | UT519 | B |
| 10 | 0.003 | 0.421 | UT757 | C |
| 11 | 0.009 | 0.583 | UT511 | B |
| 12 | 0.013 | 0.928 | UT528 | B |
| 13 | 0.010 | 0.685 | UT513 | C |
| 14 | 0.006 | 0.551 | UT529 | C |
| 15 | 0.010 | 0.471 | UT756 | B |
| 16 | 0.006 | 0.424 | UT755 | B |
| 17 | 0.005 | 0.358 | UT570 | D |
| 18 | 0.007 | 0.562 | UT520 | B |

Help 89 records. OK

**Figure 13.** Attribute table for soil type map

The rectangular area inside is the un-delineated watershed DEM file. Normally, the coverage area of the land use and soil type file is much bigger than studied watershed.

If land use and soil type files are not available, the user can create them manually. First, the user must create land use and soil type coverages in WMS separately. For each type of coverage, land use and soil type polygons are created using polygon creating method described in Section 3.5. After creating polygons, a database table regarding land use and soil type must be filled to connect the digital information with the polygon spatial area. The database table works the same way as a database when Shape format soil and land use file are available.

## 3.7 WATERSHED INFORMATION AND CALCULATION METHOD

The watershed information required for modeling in WMS include four main groups.

### 3.7.1 Basin data

Basin data contain the basin's elementary characteristics. The basin's name identifies it from other basins. A descriptive name is helpful in building and running the model. The basin's area is always calculated by WMS if the geo-referencing information is correct. If the basin's area is incorrect, user should modify the geo-reference information instead of changing area directly. If a hydrograph is known for a given basin, there is no need to compute a synthetic hydrograph. This hydrograph can be input by defining the hydrograph using the XY Series Editor. The XY Series Editor is a general purpose editor for entering curves or pairs of lists of data. If the user wants to optimize the modeled hydrograph using an observed hydrograph, the observed hydrograph can be input also using XY Series Editor.

### 3.7.2 Precipitation

Several options of precipitation can be defined. One commonly used precipitation method is basin average precipitation. With this method, a time distribution can be entered via the XY Series Editor. Several standard storm distributions can be loaded automatically from this editor. The standard storm distributions often can not satisfy the user's requirement. In this case, distributions can be created by the user directly. An average precipitation is also input to account for total rainfall.

The basin average method is convenient in small watersheds, where the rainfall can be regarded as spatially uniform. However, rainfall is normally spatially non-uniform in large watersheds, where multiple rain gages are used. A gage may be a storm total and/or temporal distribution recording station type. Recording stations allow for a continuous rainfall accumulation to be entered. The storm total station only allows for a single rainfall value for the event. Figure 14 shows a mix use of storm total and temporal distribution recording station. Gage_A and Gage_B are temporal distribution recording stations, which are "imaginary" gages and do not participate building the Thiessen network, which is used in building the rain gage coverage polygons. Gage1 to Gage 10 are storm total stations and they compose a Thiessen network. The temporal distribution of Gage1 to Gage10 is determined by nearest recording station.

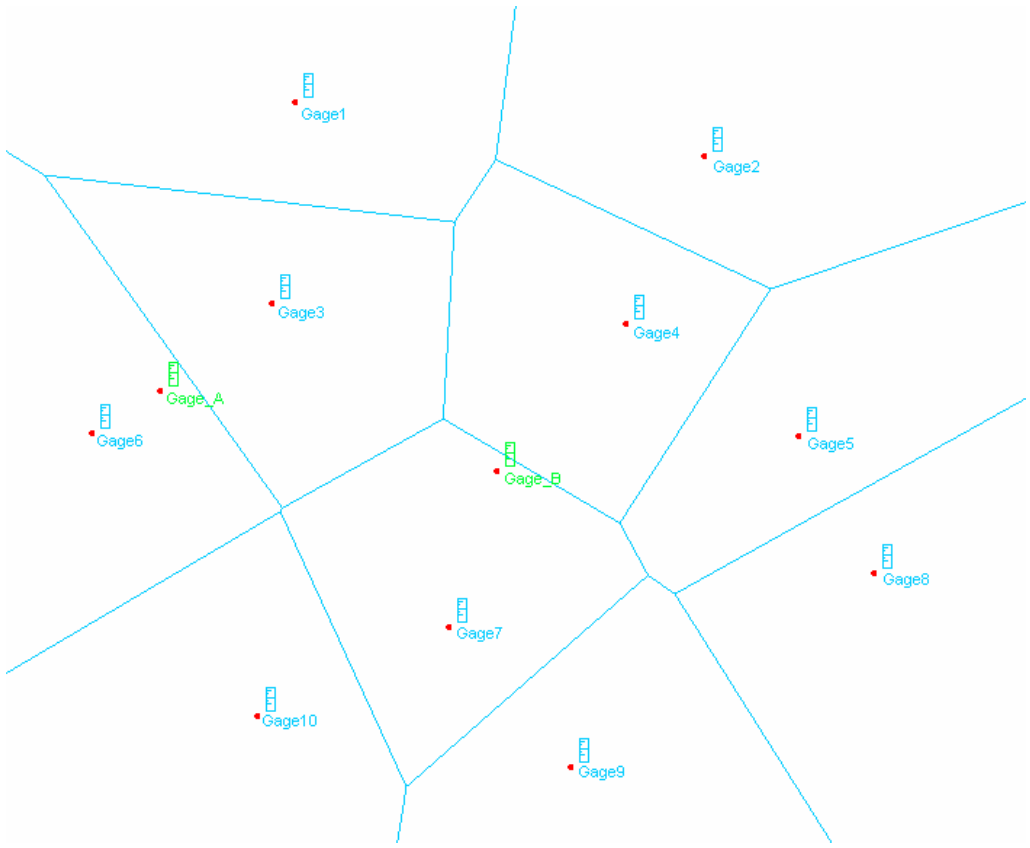



**Figure 14.** A mix use of storm total and temporal distribution recording station

### 3.7.3 Loss method

WMS provides several options in loss method modeling. The uniform loss method is the simplest method. It uses an initial value and a uniform value to define infiltration losses. The Horton loss method uses the starting value of the loss coefficient and the exponential factor as the main parameters. The Green-Ampt method employs initial effective saturation, effective porosity, hydraulic conductivity and wetting front soil suction head to compute infiltration rate and accumulative infiltration. SCS curve number loss method is used in this research. These methods are described in detail in Chapter Two.

### 3.7.4 Unit hydrograph method

Several unit hydrograph methods, such as Snyder unit hydrograph, Clark unit hydrograph and SCS dimensionless unit hydrograph are available in WMS model. The SCS unit hydrograph is used in this research. The most important parameter in SCS is the lag time. Several different equations have been published to determine the lag time of a basin. User also can define the lag time equation himself. Many of them use some of the geometric attributes computed automatically. Eq. 3.1 is used to calculate lag time defined by SCS.

$$T_{Lag} = L^{0.8} \times \frac{[(\frac{1000}{CN} - 10) + 1]^{0.7}}{1900 \times \sqrt{S}} \qquad (3.1)$$

where $T_{Lag}$ = the lag time;

L = the watershed length;

CN = curve number;

S = watershed slope in percent.

## 3.8 CHANNEL AND RESERVOIR ROUTING

The hydrographs from the upper basins would be combined with the lower basin hydrograph at the watershed outlet. Routing parameters should be determined to compute lag and attenuation on the upper basin hydrographs before adding them to the lower hydrograph.

The Muskingum method is often used in channel routing. The method is dependent primarily upon the following factors: the number of integer steps for the routing, Muskingum K coefficient in hours and Muskingum X coefficient. The algorithm is explained in Chapter Two. The integer step number and K coefficient are determined by water flow velocity in the channel.

A reservoir is placed at an outlet to model water storage and retention. Although the reservoir does not occupy area in the model, its characteristics can be defined using a table. The important reservoir routing data are elevation-discharge relationship, elevation-storage relationship and initial condition. Using these data, reservoir routing can be performed without knowing reservoir's area.

## 3.9 JOB CONTROL SETTING

Most of the parameters required for HEC-1 model are defined for basins, outlets and reservoirs. However, there are some global parameters that control the overall simulation and are not specific to any basin or reach in the channel. These parameters are defined in WMS job control module.

A model's title, ID, author and short description can be filled in the first part. Then, the modeling event starting date and calculation unit should be filled. The most important data here are the computational interval and the number of hydrograph ordinates. They determine how long the resulting hydrograph should be displayed. If the display time is too short, the hydrograph will not be shown completely. If the display time is too long, the hydrograph shown is to rough and many details are missed. Proper computational interval and the number of hydrograph ordinates are needed to get good modeling results. They can be estimated by observing similar outflow records.

As stated in 3.7.1, the user has the option to optimize unit hydrograph and loss rate parameters in the modeling so the calculated hydrograph will match the observed hydrograph. User also can optimize routing parameters using observed inflow and outflow hydrographs and a pattern lateral inflow hydrograph for the routing reach. They are also set in job control module.

## 3.10 DISPLAY OUTPUT

The model results are detailed hydrographs at each outlet. At the outlet where reservoir is located, both hydrographs flowing into and flowing out can be displayed. The display formats can be diagram, text format, Excel format, etc. For direct view, a diagram is often used. For comparing with an observed hydrograph, Excel format is often used.

## 3.11 EVALUATION OF PARAMETER INFLUENCE

No model is perfect. In order to evaluate if the model is satisfactory, several important modeling characteristics should be compared with the observed hydrograph. They are the total volume of the runoff, the number of peaks, each peak time and each peak discharge value. Different modeling parameters influence the modeling results in different ways. Table 2 lists the relationship between some important model parameters and model results.

**Table 2.** The relationship between important model parameters and model results

| Influences / Parameters | | Increase Parameters (Decrease - Opposite Influences) | | |
|---|---|---|---|---|
| | | Total Volume | Peak Time | Peak Discharge |
| Basin | CN | Increase | No Change | Increase |
| | Slope | Increase | Early | Increase |
| | Area | Increase | No Change | Increase |
| | Precipitation | Increase | No Change | Increase |
| | Watershed Length | Decrease | Later | Decrease |
| Routing | Water Velocity | Increase | Early | Increase |
| | No. of Steps | Decrease | Later | Decrease |
| | K | Decrease | Later | Decrease |
| Reservoir | Initial Elevation | Increase | Early or No Change | Increase |
| | Storage | No Change | Later | Decrease |

It is strange that the number of peaks of the discharge seldom changes with the above parameters. Actually, the number of peaks of the discharge is mostly controlled by precipitation pattern, i.e. the number of peaks and distribution of the precipitation. After surface flow and channel routing, the number of peaks of the discharge will be fewer than the number of peaks of the precipitation. However, large peaks in precipitation separated by long time intervals will be reflected in the peaks of the discharge.

The above table indicates the parameters and model results in one single sub-watershed. Generally, the hydrograph comes from several sub-watersheds, outlets and reservoirs, which have several sets of parameters. In this case, the model results are more complicated to anticipate.

# 4.0 I-99 ENVIRONMENTAL RESEARCH OVERVIEW

## 4.1 PROJECT INTRODUCTION

The Pennsylvania Department of Transportation (PENNDOT) is constructing the U.S. Route 220/I-99/State Route (S.R.) 6220 project that is a part of extending I-99 to I-80 at Bellefonte. Figure 15 shows the I-99 project location. Figure 16 shows the Environmental Impact Study (EIS) area boundary.

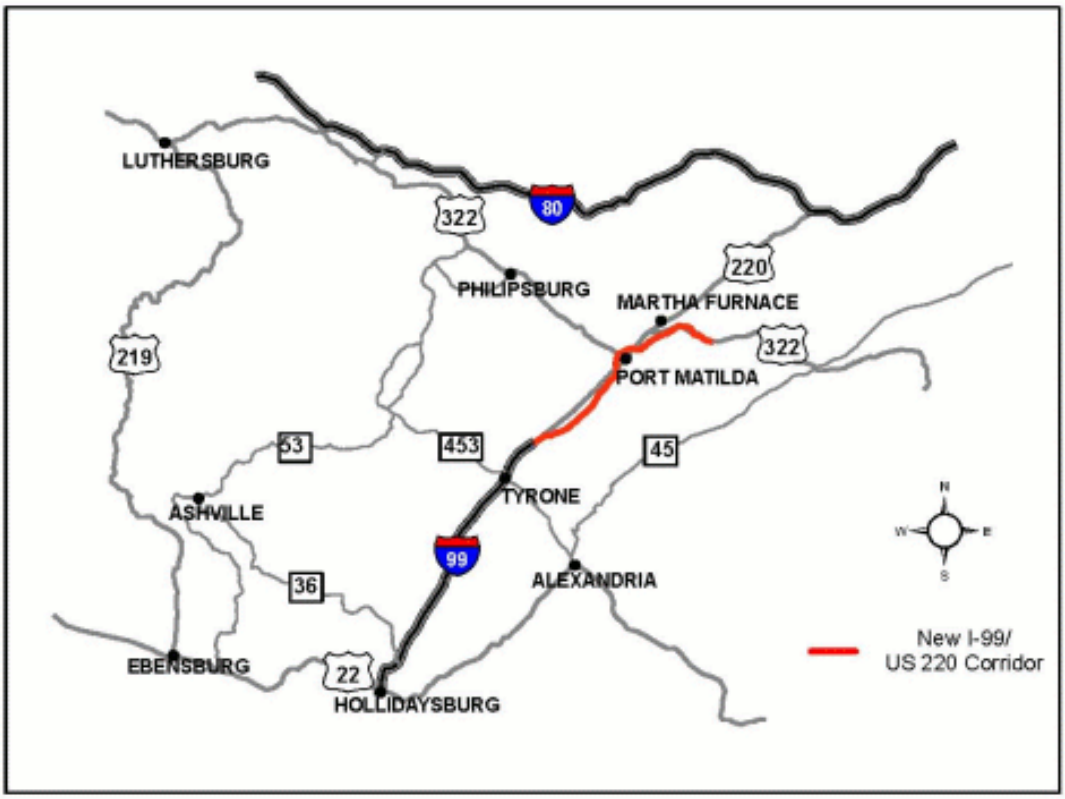



**Figure 15.** The I-99 project location

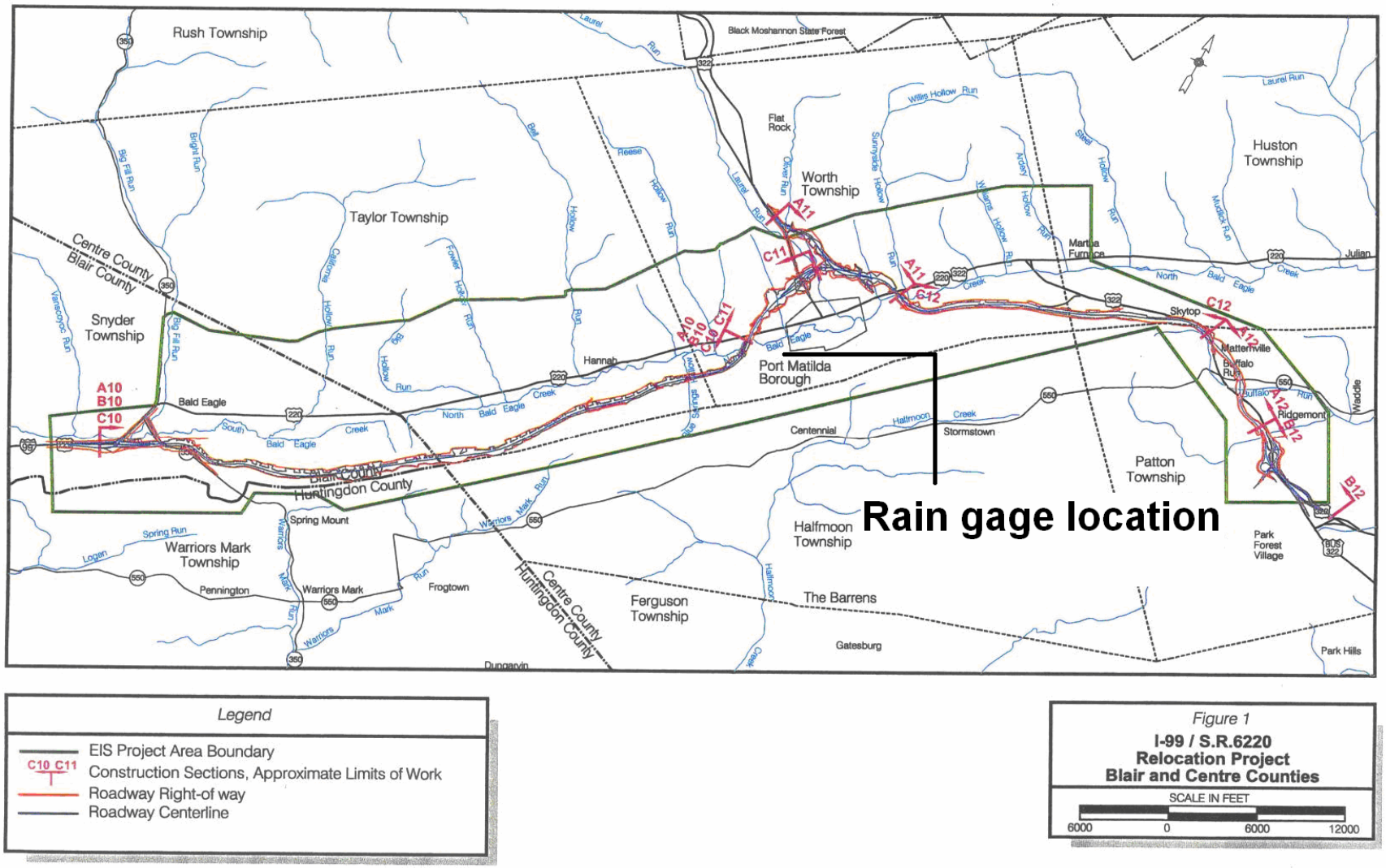



**Figure 16.** The Environmental Impact Study (EIS) area boundary

Highway construction often causes substantial adverse environmental effects, both during and after construction. Construction-induced impacts include soil erosion resulting from clearing, grubbing and earth movement and rainfall runoff. Relocation of streams, direct and indirect impacts to wetlands, and encountering hazardous wastes are also the construction environmental impacts. Highway constructions may also unintentionally damage site of archeological and culture significance. During the operation phase, highways affect the environment through the introduction of pollutants in storm water runoff, permanent changes in land use and resulting ecological consequences.

The objective of the I-99 environmental research is to monitor and evaluate the effectiveness of various mitigation techniques implemented during the highway construction to provide an improved management of the highway, and to develop enhanced capabilities to predict impacts and identify suitable mitigation measures for future highway projects in Pennsylvania (Quimpo, 2004). The project will provide immediate benefits in helping to minimize the construction and operational impacts of I-99. It will provide long term benefits by developing Best Management Practices (BMPs) for Pennsylvania highways, and through the development of models that can be used throughout Pennsylvania to predict construction impacts and mitigation success. These models will provide for greater accuracy and reliability in future designs, while reducing the cost of expensive field and laboratory investigations.

## 4.2 PROJECT COMPONENTS

### 4.2.1 Task A: Evaluation of erosion and sediment controls

As stated in Chapter One, this environmental research includes four tasks. The primary focus of Task A is to continue the research on Best Management Practices (BMPs). The key issue for this task is to conduct field reconnaissance, monitoring of field sites under

normal and high rainfall, and assess the results in a final report. The research includes the following work:

1. Periodic site visits with professional observations, reporting and digital pictures.
2. Digital photographs will be taken and logged with narrative. Digital pictures will be stored electronically to be retrieved as needed for study and comparison, incorporated into a GIS database, and incorporated into the final project report. GIS mapping will be linked to a database including modeling results and to digital photographs.
3. The field crew will identify selected samples that to be analyzed for turbidity analysis, trace heavy metals and filtered COD. Field measurements of pH and temperature will be taken. Such studies provide new insight into the behavior of BMPs subjected to runoff containing residuals typical of motorized vehicle traffic.
4. A qualitative risk-based evaluation method will be prepared. The evaluation aims at the relative environmental impacts associated with the ineffectiveness or failure of the BMPs on downstream receiving waters, including wetlands and streams. The method will address modes, likelihood, and consequences of failure, and will consider reliability, longevity, costs, environmental benefits and liability.

### 4.2.2 Task B: Hydrologic monitoring and modeling

Task B is the prediction of the quantity of runoff on the complex construction project. The primary focus of Task B is to perform monitoring and modeling to evaluate the hydrologic regime associated with the highway cut area, the down slope wetlands, and the mechanism used to maintain groundwater flow to the wetlands, i.e. infiltration galleries. The model, which is to be calibrated using the field monitoring data, will be used to evaluate, explain and predict the effectiveness of the infiltration galleries in maintaining groundwater flow to the down slope wetlands. The objective is to develop a model that can be applied at the I-99 project to reliably predict future hydrologic impacts to the wetlands and thereby reduce costly monitoring efforts. The model may then be used in the design of other projects with similar conditions. Runoff prediction techniques, for the specific local construction site, will be built and verified. To reduce expensive

monitoring instrumentation at other projects, the model should be portable, which means the procedures can be used at other construction sites with adjustments for site-specificity. The portability requirement is an important criterion in the selection and choice of model.

As part of the monitoring effort included in Task B, water level recorders were installed at several locations. The recorders are classified into three categories. The first category, which is called Well Logger, is used to record the ground water table changes. The second category, Deep Ecotone, is used to record the sub-surface flow in the down stream watershed. The last category, Shallow Ecotone, is employed to obtain water depth in the watershed outflow flume. Ecotone is the brand of the instruments. The Shallow Ecotone instrument is a Parshall flume. Figure 17 is an illustration of a Parshall flume, which has a contraction in the middle.

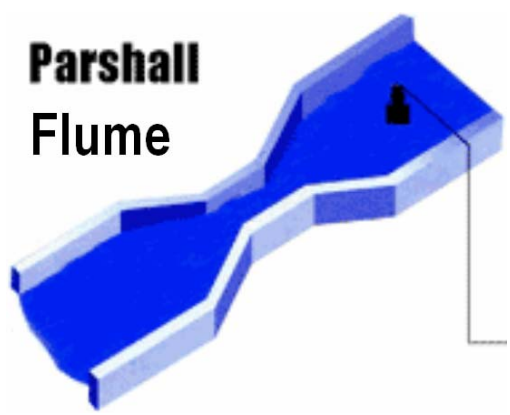



**Figure 17.** An illustration of a Parshall flume

When water flows through the flume, the contraction forces the water flow from sub-critical status to critical flow status. At the critical depth, the specific energy is minimized and discharge is uniquely related to the water depth. Equation (4.1) shows the specific energy expression.

$$E = h + \frac{Q^2}{2gA^2} \tag{4.1}$$

where:

$E$ = specific energy,

$h$ = water depth,

$Q$ = discharge,

$A$ = cross section area,

$g$ = gravity acceleration.

To get the minimum of $E$, Equation (4.1) is differentiated and assigned to be zero.

$$\frac{dE}{dh} = 1 - \frac{Q^2 B}{gA^3} = 0 \qquad (4.2)$$

where:

$B$ = flume width.

Equation (4.2) can be solved by substituting flow velocity $V = Q/A$ and water depth $h = A/B$.

$$\frac{dE}{dh} = 1 - \frac{V^2}{gh} = 0 \qquad (4.3)$$

Then the flow velocity and discharge can be calculated by

$$V = \sqrt{gh} \qquad (4.4)$$

$$Q = B \cdot \sqrt{g \cdot h^3} \qquad (4.5)$$

Theoretically, the discharge can be calculated from depth directly. In practice, the relationship of water depth and discharge is calibrated and provided by manufacturer. Once the relationship between water depth and discharge is determined, we do not need to get discharge from the instrument. Instead, we get water depths and convert them into outflow discharges in the hydrological analysis using the relationship. The Well Logger, Deep Ecotone and Shallow Ecotone instruments automatically record water levels at six-hour intervals, one-hour intervals and one-hour intervals, respectively. The data will be used in Task B for calibrating the model and evaluating model prediction results, and in task C for assessing wetland conditions and key wetland indicators. The locations of monitoring instruments for Watershed One and Watershed Two are illustrated in Figure 36 and Figure 38, respectively.

The Environmental Impact Study (EIS) is about 10 miles long and 1 mile wide. Because the EIS area does not belong to one watershed, it is divided into several sections.

Along the highway, length marks are coded from 0 to 535. One unit mark's distance interval is 100 ft. Hydrological modeling is performed in the selected two watersheds, which are at about the length mark of 185 and 400. For purposes of Task A, ponds were constructed downstream of the highway at several locations. The corresponding ponds in the two watersheds are SB111 (around length mark 400) and SB10, SB11 (around length mark 185). These two watersheds were selected because PENNDOT constructed special underground filter galleries in the watersheds. Also, Ecotones were installed at these two watersheds' outlets. As stated above, one of the objectives of this project part is to reduce instrumentation and data collecting expenses. Due to high cost of buying and installing the instruments, only the two test watersheds have Ecotone recorders. The model built on these two watersheds will be applied to other watersheds with some site specific modifications.

A key feature of this highway watershed design is the inclusion of infiltration galleries under the roadway which catch intercepted groundwater from areas upslope. Runoff from newly impervious surfaces resulting from construction would be collected and routed to storm water management ponds. This slows down runoff production through reduced transport rate and also improves its quality through detention in the ponds, catching the first flush.

Runoff from undisturbed land is channeled directly to receiving streams as clean water. This is achieved by routing the clean runoff directly to downstream outlet without passing the intermediate watersheds. A rain gage was installed in Port Matilda, a town near the watersheds. Because the watershed is not too large, only one rain gage was used. The location of rain gage is illustrated in Figure 16.

The model focuses on the hydrologic phenomenon of a system designed to incorporate an infiltration stratum under the roadway. This project presents a unique opportunity to test the new concept in design to minimize adverse impacts on the environment, particularly with respect to hydrologic regimes in highway cut areas. The uniqueness lies in the minimum disruption of the hydrologic regimes prior to construction.

### 4.2.3 Task C: Monitoring and assessment of wetland hydro-biological indicator

Task C will focus on the wetlands down slope of the cut areas in Section C10 (Figure 16), and mitigation wetlands. The environmental document compiled for the project through the National Environmental Policy Act (NEPA) process will be utilized to provide the initial baseline documentation of resources in the project area. A GIS database will be compiled of project corridor wetlands. Data will be organized to provide a clear illustration of each wetland's position in the landscape based upon watershed, topographic setting, and relationship to the new highway and other anticipated land use changes. The database will include information concerning those features that may influence function and value. These data includes wetland type, topographic setting and geologic conditions, hydrologic regime, soil type, adjacent land uses and soil types, dominant vegetation species, habitat value, water quality issues, and uniqueness. Photographs of each wetland, wetland delineation data forms, and other relevant supporting information will be linked to this database.

Each monitoring wetland will be subject to periodic field surveys to assess parameters affecting function and value. The following parameters will be monitored at the frequencies listed.

1. Water chemistry;
2. Hydrologic conditions;
3. Vegetation community;
4. Birds;
5. Amphibians and reptiles;
6. Mammals;
7. Benthic macro-invertebrates;
8. Soil chemistry;
9. Soil condition;
10. Adjacent land use and including dominant vegetation community

The initial survey on the selected wetlands will include collection of data on the various animal classifications identified to provide a broad spectrum of baseline data on the community composition and population levels. The subsequent field monitoring will focus on the benthic macro-invertebrates and amphibians, which are key bio-indicator groups.

An overall assessment of function and value will be completed for each monitored wetland for pre and post-construction conditions, and for post-construction mitigation wetlands in accordance with the appropriate methodology. These results will then be analyzed utilizing multiple linear regression, ordination techniques, or other appropriate, ecologically-valid statistical analyses to determine the relative importance of the monitored parameters in explaining observed changes in functions and values, by wetland type and setting, over time.

The results of the previous efforts will be synthesized to provide a regional framework for predicting construction impacts on wetlands by type and setting based on key indicators. An important component of these analyses will be the development of a regional model that will predict the impact of construction on wetland species diversity, relative abundance of domain classification, and the performance of uncommon or unique species. This model will be developed for use as a tool in assessing impacts of future highway projects utilizing key indicators and standardized measures of diversity.

#### 4.2.4 Task D: Evaluation of stream restoration, rehabilitation and relocation

The objective of Task D is to assess the effectiveness of stream restoration, stabilization, and relocation practices for highway construction and their ability to sustain a complex, ecologically diverse healthy stream system over the long term. Highway constructions often impact streams directly and indirectly. These construction projects typically require some type of mitigation, which includes stream bank stabilization, stream relocation, stream restoration, and possibly mitigation elsewhere to compensate for impacts of the project.

# 5.0 DIFFICULTIES IN MODEL BUILDING

## 5.1 DIFFICULTIES STATEMENT

This research focuses on Task B, the prediction of runoff on the complex construction site. Hydrological modeling is performed in the selected two watersheds, which are approximately at about the stations of 400 (SB111, Watershed One) and 185 (SB10, SB11, Watershed Two) in the construction project's alignment. Due to the following features, this construction watershed is difficult to model.

1. The watershed areas are very small. The area of Watershed One is about 0.085 sq mile (54.40 acres). The area of Watershed Two is about 0.072 sq mile (46.08 acres). Although DEM files are available online, they cover large areas, normally in 10 to 100 sq miles. The resolution of the DEM is $30 \times 30$ sq meters. The studied watersheds are only a few points in this kind of DEM file. Detailed watershed delineation cannot be performed in such coarse DEM files. Since DEM files are coarse, the converted TIN files also cannot be used. The same problems exist in the downloaded land use and soil type files. Land use and soil type files are even in larger scale, usually in 100 to 1000 sq miles. If the online land use and soil type files are used, probably the whole watershed falls in only one type of soil and land use.

2. The watershed geomorphological characteristics and land uses have been changed recently by construction. The online data do not reflect the updated elevation and land use changes. For example, during or after the construction, land use in the watershed land use changes from uncultivated land or forest to paved highway. Streams are created or covered. Stream flow directions are changed. Elevations, hence the water flow channel,

are also changed due to construction. The nature stream is sinuous and coarse. The water flow velocity in nature watershed is slow. Constructed channels or streams are generally straight and smooth; hence, water velocity is greatly increased. These changes cannot be reflected simultaneous on the downloading GIS data.

3. The watershed has ponds and flat areas. As stated in Chapter Three, pits or flat elements can be a problem because the water flows into the pit and never flows out. The model cannot determine the flow direction if a flat area is encountered. WMS has tools to remove a small amount of error elements by changing their values through interpolation. However, these tools are useless if huge amount of pits, flat points congregate together to form a big pond. Even if they are removed, the model can not reflect the real watershed characteristics.

4. In the highway construction, an infiltration gallery under the roadway is constructed to catch the infiltration water from the construction area. Runoff from new impervious surfaces resulting from construction would be collected and routed to storm water management ponds. An underground pipe is also installed to conduct water from upstream clean area directly to downstream outlet without passing the ponds. Underground filtration gallery, detention pond and underground pipe are much different from regular stream and must be modeled using a suitable model component.

As stated in Chapter Three, after obtaining the topographic GIS data, watershed delineation is the next procedure in building the hydrological model. The above problems mainly influence the watershed delineation. After watershed is properly delineated, the following procedures, such as drainage component editing, basin parameters estimation, channel and reservoir routing parameters estimation, and job control setting are irrelevant to the topographic GIS data. Although land use and soil type data are also categorized as GIS data, they can be created manually easily. The reason is land use and soil type data are featured (polygon or polyline) GIS data, while topographic GIS data are grid data. Creating featured GIS data can be accomplished by drawing a few polygons and assign them certain properties. Creating topographic GIS data needs to assign thousands of

pixels different values. This cannot be accomplished manually. The main difficulty in the model is the availability and accuracy of topographic GIS data.

## 5.2 WATERSHED DELINEATION FOR A LARGE NATURAL WATERSHED

To illustrate the procedure of delineating a large natural watershed, Little Pine Creek Watershed (LPCW) is employed here. LPCW covers about 15.02 $km^2$ (5.8 $mile^2$) and is located mostly with Shaler and O'Hara Townships in north central Allegheny County, Pittsburgh, Pennsylvania. LPCW was selected because it contains no significant detention pond or other storage. Little Pine Creek is a tributary of Pine Creek, which flows into the Allegheny River at the town of Etna. The DEM data files were obtained from Pennsylvania Spatial Data Access (PASDA) website.

### 5.2.1 Importing DEM data

DEM data should be imported into WMS. The imported DEM data is shown in Figure 18.

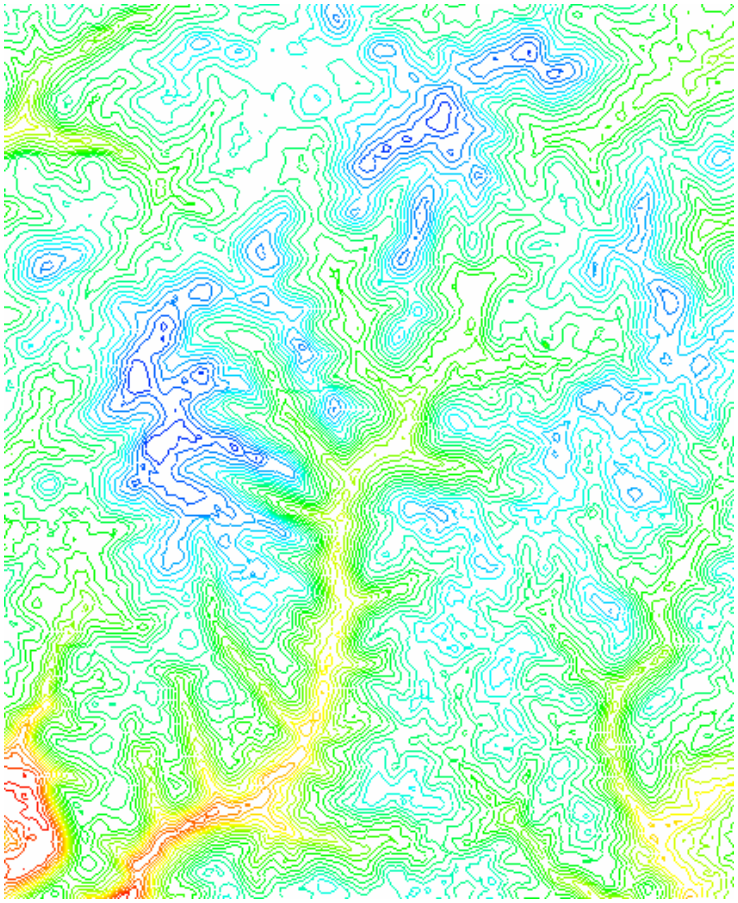


**Figure 18.** The imported DEM data for LPCW

### 5.2.2 Computing flow direction

The flow direction should be computed in WMS. The flow direction form a network of streams on top of the DEM. WMS computes flow direction for individual DEM cells and creates streams based on these directions. Figure 19 shows a flow direction of LPCW. Figure 20 shows a stream network of LPCW.

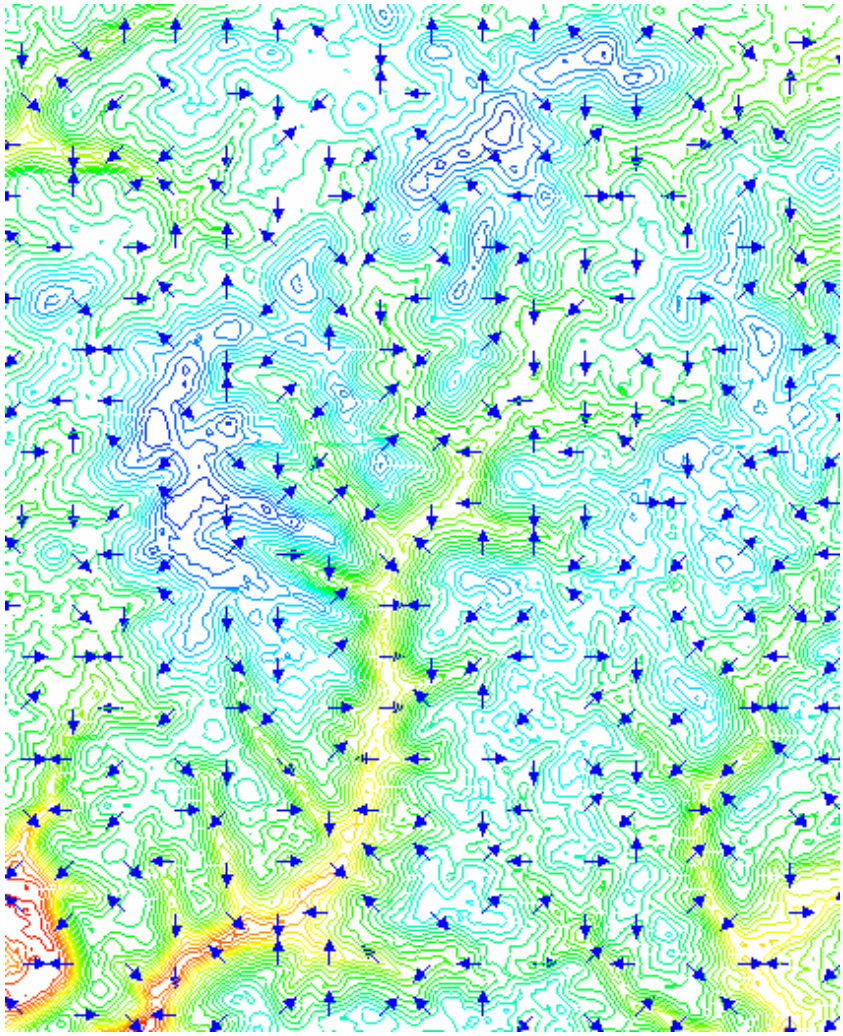


**Figure 19.** The flow direction of LPCW

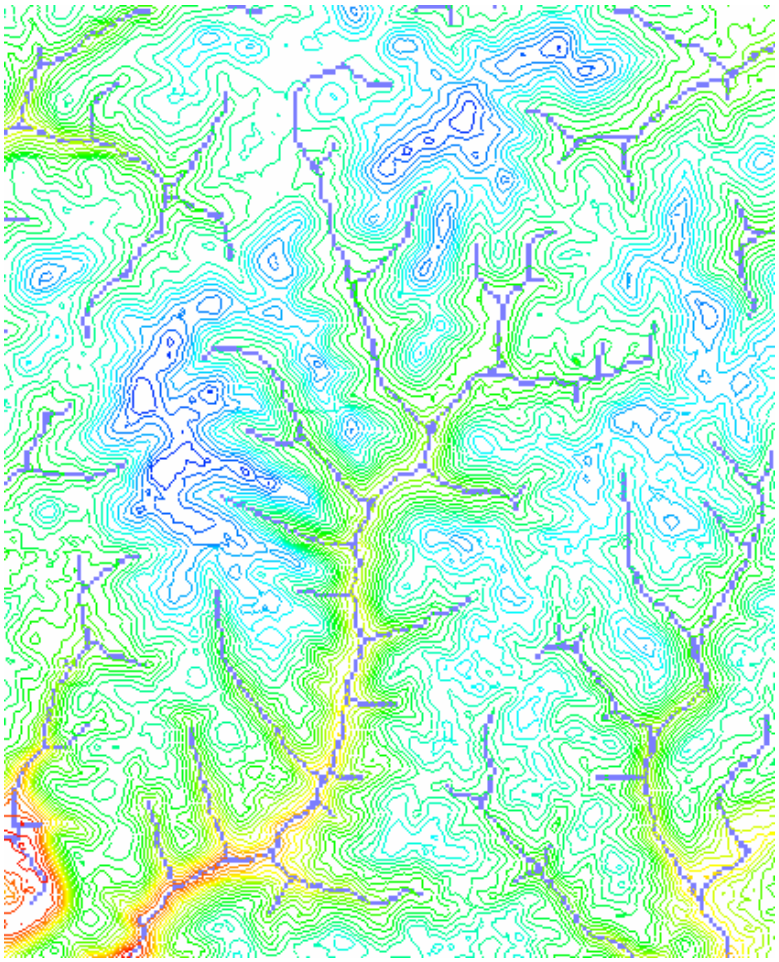


**Figure 20.** The stream network of LPCW

### 5.2.3 Determining the outlet point and stream feature arcs

The outlet point is determined according to project need. Theoretically, it can be any point in the watershed. In this example, a point in the stream near the lower left corner is selected. By choosing a point on a lower stream branch, a watershed outlet is defined. WMS then uses the flow direction and accumulation data from Section 5.2.2 to convert the streams network into stream feature arcs. Figure 21 is the converted stream feature arcs in LPCW.

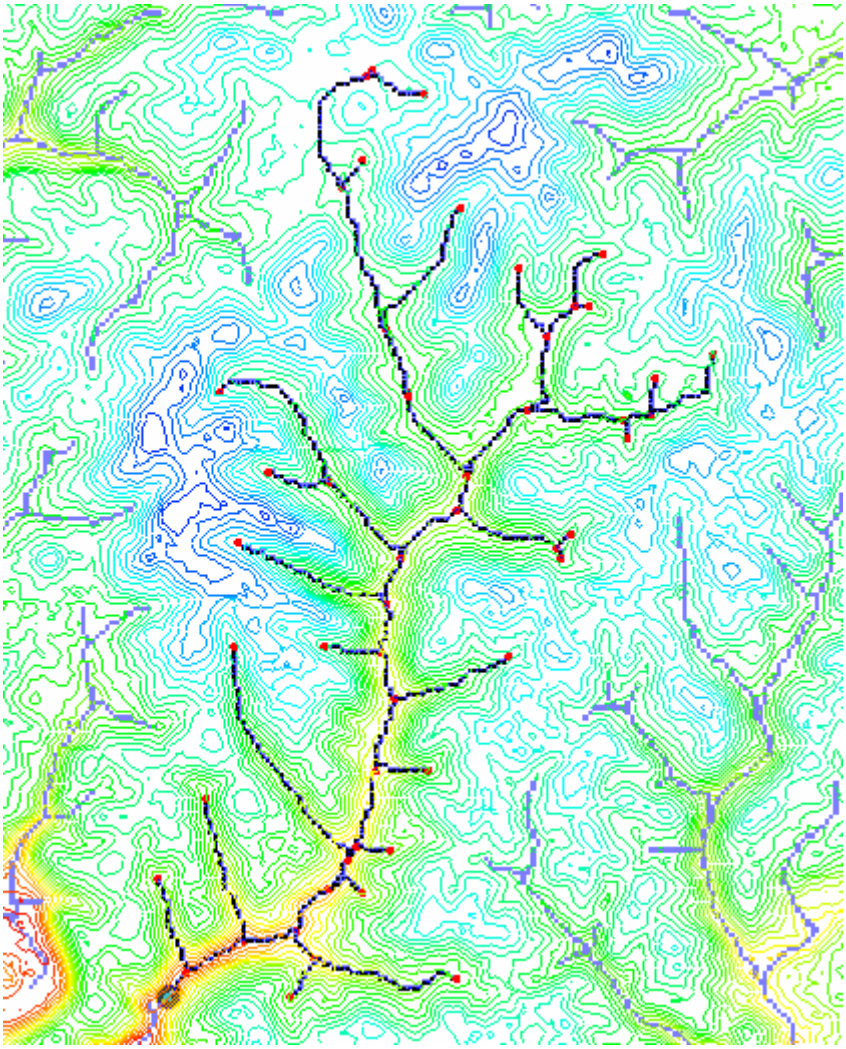


**Figure 21.** The converted stream feature arcs in LPCW

### 5.2.4 Defining the watershed boundary

The stream feature arcs can be used to define the basin boundaries. Because the DEM contains elevation data, the model can be used to calculate the basin properties such as watershed area, slope, and average overflow distance, etc. It is not just a schematic of the watershed. Figure 22 show the delineated watershed boundary and some of the basin properties,

where *A* = watershed area; *BS* = watershed slope;

*AOFD* = average overflow distance; *L* = watershed length;

*MSL* = maximum stream length; *MSS* = maximum stream slope;


**Figure 22.** The delineated watershed boundary and some of the basin properties

### 5.2.5 Creating sub-watersheds

In order to create sub-watersheds, additional drainage outlets need to be defined. The new drainage outlets must be on the stream. After several nodes or vertices along the stream arcs are defined into drainage outlets, the same method defining watershed boundaries can be used again to define sub-watersheds. Figure 23 shows the re-defined sub-watershed boundaries and watershed properties. Due to space availability, Figure 23 only shows two properties of the watershed boundaries. However, all properties in Figure 22 can be shown for each sub-watershed.

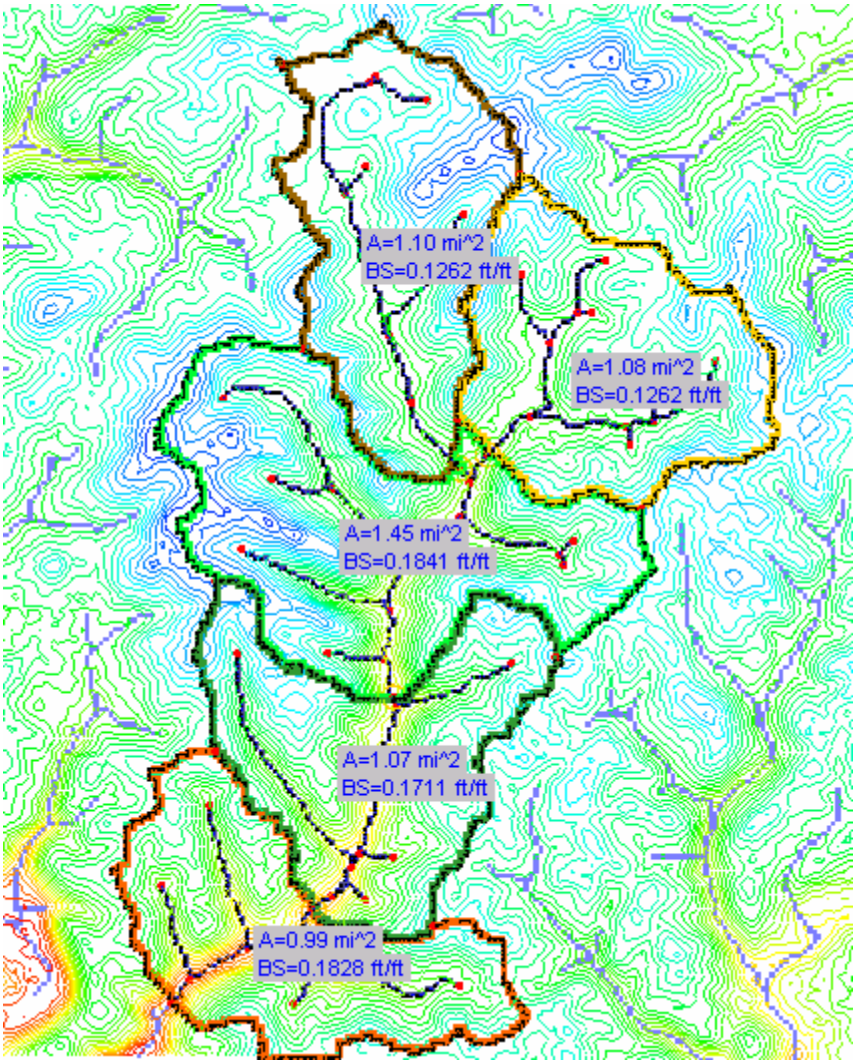



**Figure 23.** The re-defined sub-watershed boundaries and watershed properties

## 5.3 I-99 DEM DATA GENERATION

As stated in Section 5.1, the resolution of downloaded DEM file is too coarse compared to the small watershed area. We created a 2×2 *sq ft* DEM file for Watershed SB10-11 from MicroStation DGN file provided by Pennsylvania Department of Transportation (PENNDOT). The DGN file only contains contour poly lines map. There is no label, polygon, or points in the file. The elevation information of each contour line is stored in the poly line's element information table, but not displayed in the map. The MicroStation

DGN format contour line file can be converted into DEM file through the following procedures.

1. DGN file is imported into ArcGIS using *Add Data* function. The DGN file is shown in Figure 24.

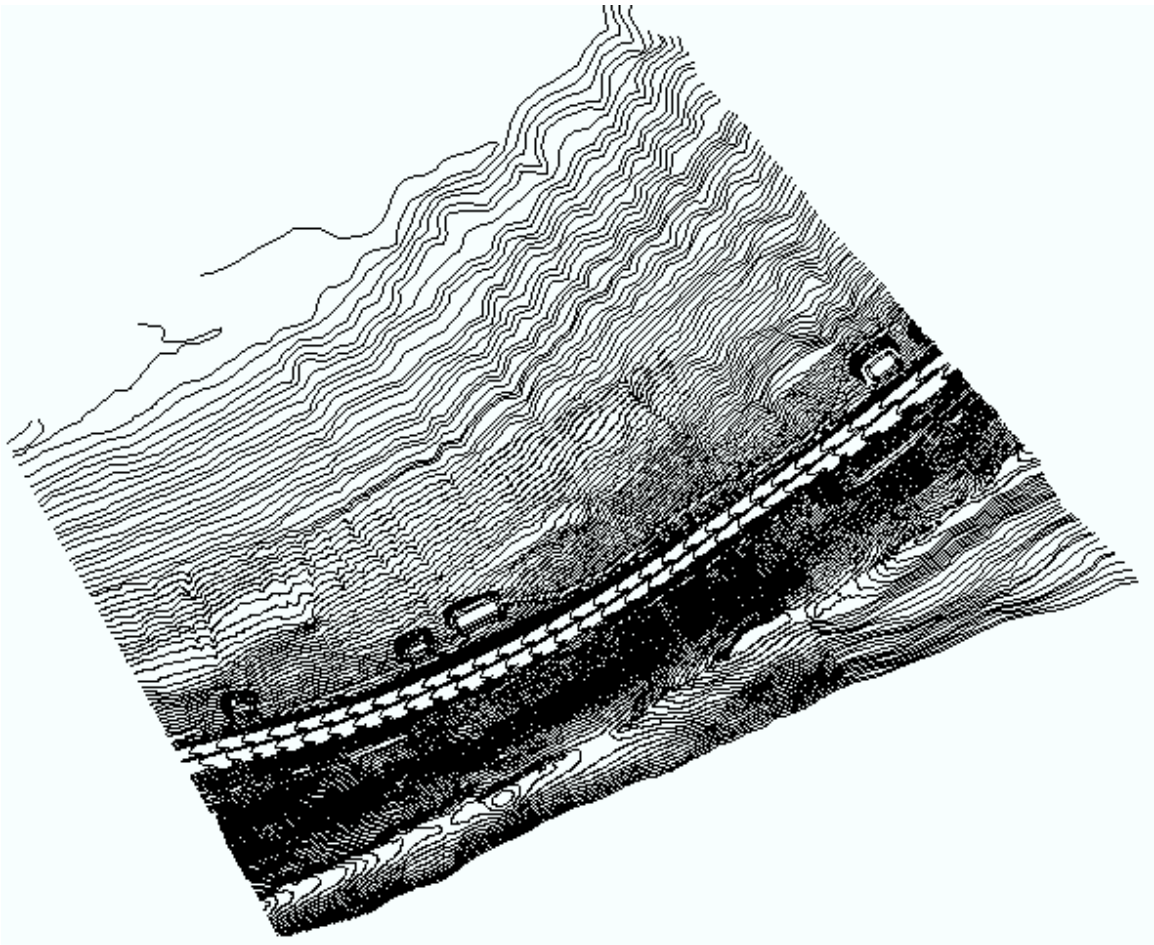


**Figure 24.** The DGN file view in ArcGIS

2. DGN file is converted into TIN file in 3D Analyst, an extension of ArcGIS. The function to be used is *Create TIN From Features…* The TIN file is shown as in Figure 25.

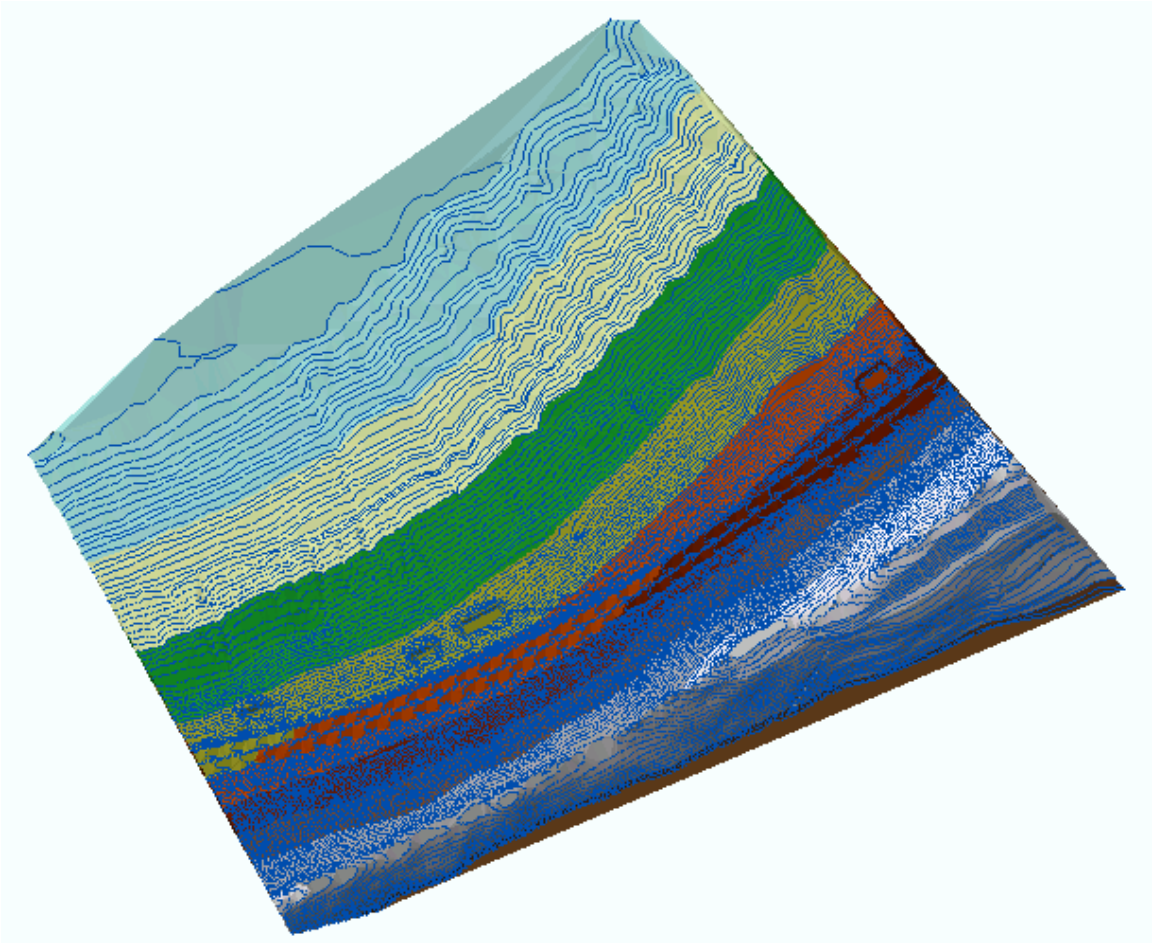


**Figure 25.** The TIN file view in ArcGIS

3. TIN file is converted into GRID file in 3D Analyst using the function of *TIN To Raster*... Resolution of the GRID file can be assigned in this procedure. For example, we can assign it as 2×2 *sq ft* or 5×5 *sq ft*. The GRID file is shown as in Figure 26.

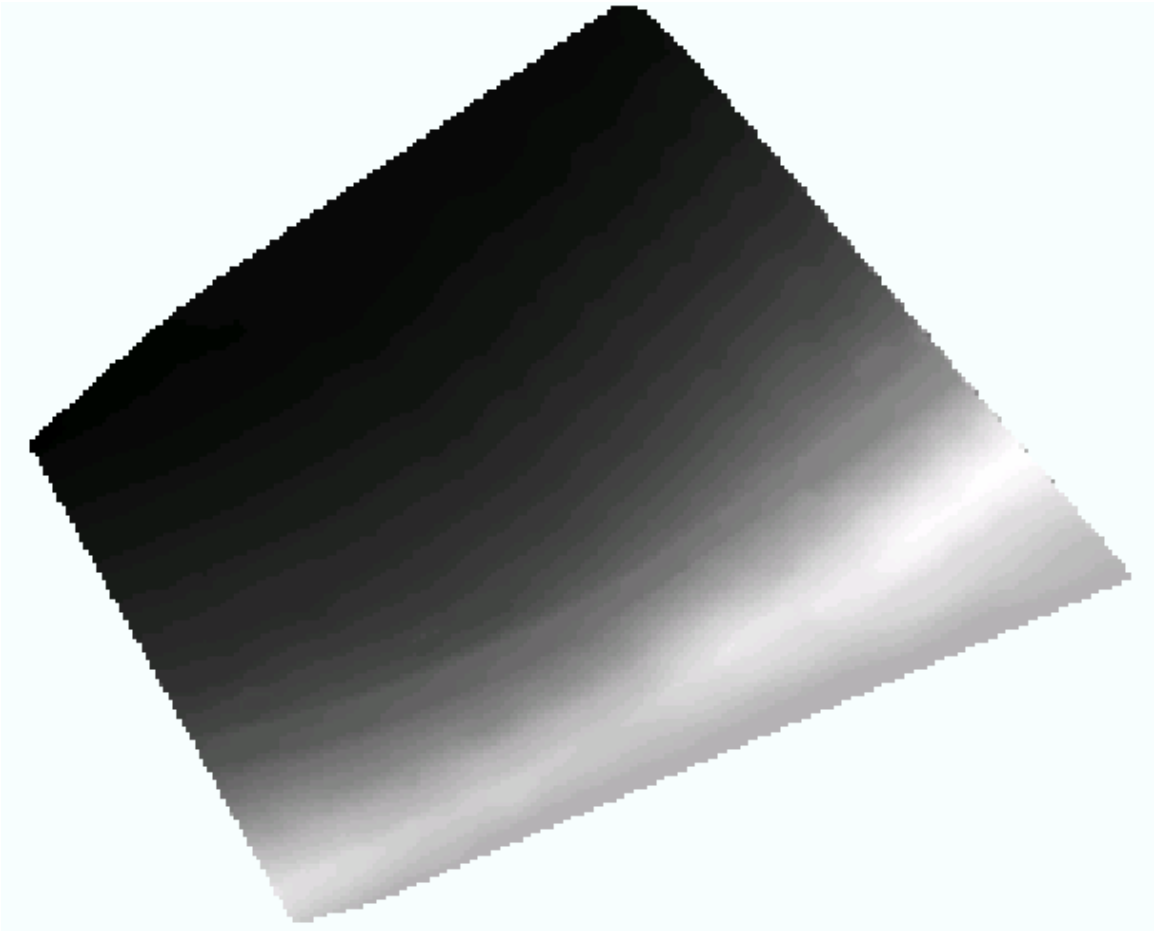


**Figure 26.** The GRID file view in ArcGIS and WMS

4. Start WMS, turn to the GIS module, enable the ArcObject function, add ArcGIS GRID file into WMS using *Add Data* function. The GRID file appearance in WMS is the same. It is also shown in Figure 26.

5. The GRID file is converted into DEM format using *Convert To DEM* function. The DEM file is shown in Figure 27.

After DEM file is obtained, the watershed delineation can be performed in WMS. This is illustrated in the following section - Section 5.4.

## 5.4 WATERSHED DELINEATION IN I-99 BASED ON DEM DATA

As illustrated in Section 5.3, resolution of 2×2 *sq ft* DEM file is derived from PENNDOT MicroStation DGN file for Watershed SB10-11. Figure 27 shows the original DEM file in WMS. Figure 28 shows the DEM file with the flow direction, the stream networks, and stream feature arcs in an enlarged scale. Although these procedures work fine, the generation of watershed is a big problem. The real watershed is disturbed by construction. Many man-made channels and underground pipes were built to drain water. The watershed elevation is also altered to form a shape showed in Figure 38. The automatically generated watershed boundary is shown in Figure 29.

The WMS tutorial states "Watershed delineation from DEMs is straightforward and relatively simple, provided the project area is not entirely flat or completely dominated by manmade structures (you can't expect the DEM method to work if there is no relief in the DEM elevations themselves)." (Environmental Modeling Research Laboratory, Brigham Young University, 2004).

However, I-99 project area DEM file contains many flat areas and pits. It is almost dominated by manmade structures, like channels, pipes, and highways. Figure 29 also shows the DEM with flat and pit cells. The shaded areas are flat or pit cells.

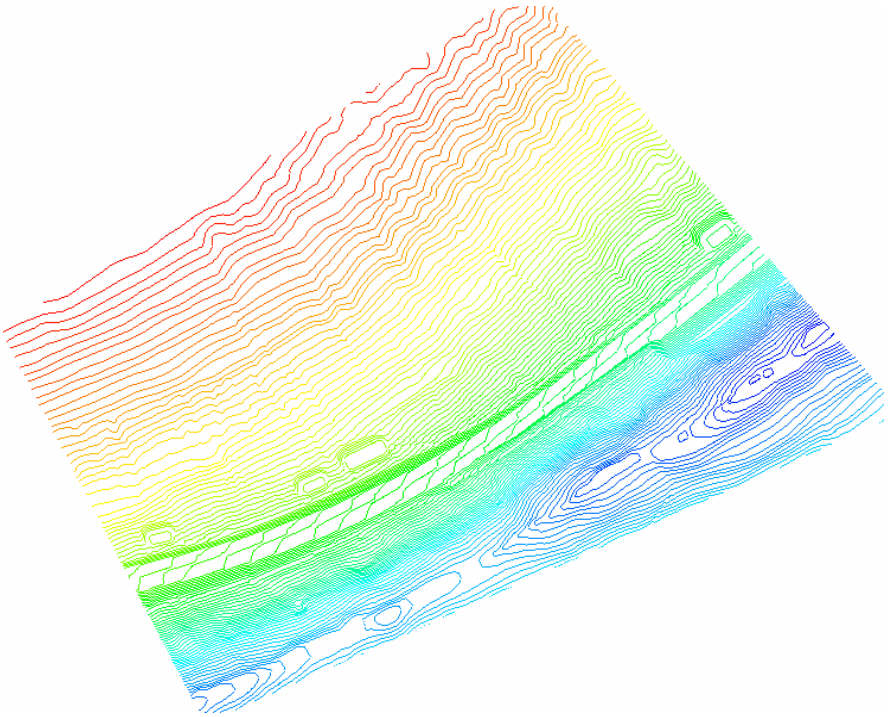


**Figure 27.** The DEM file for I-99 Environmental Research

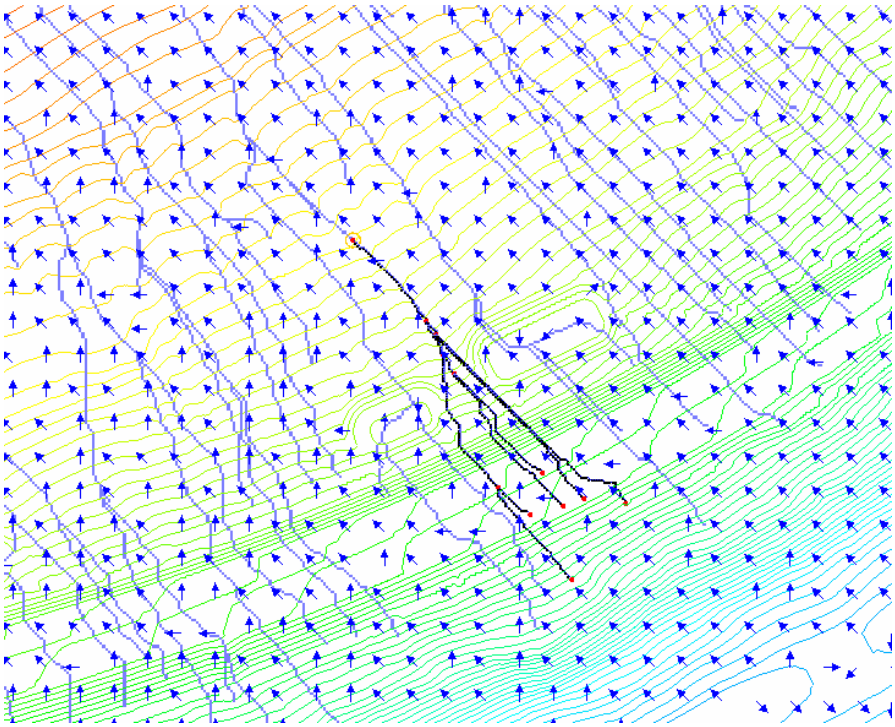


**Figure 28.** The DEM file with the flow direction, the stream networks, and stream feature arcs

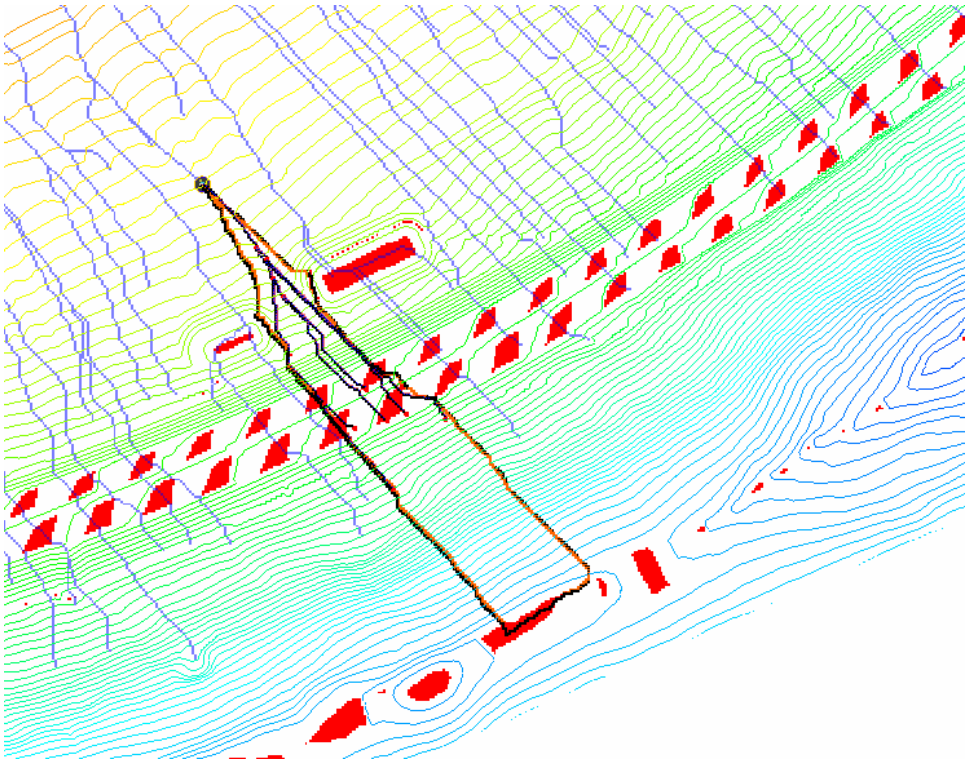


**Figure 29.** The automatically generated watershed boundary and DEM with flats and pit cells

Trimming a profile like the actual watershed to form a proper watershed shape does not solve this problem. The inner flat and pit cells remain unchanged. Actually the man-made structures are not reflected in the DEM file yet. The watershed delineation does not coincide with the actual one. Figure 30 shows the original DEM file for Watershed SB10-11. Figure 31 shows the DEM file with flow direction, stream networks, and the stream feature arcs. Figure 32 shows the automatically generated watershed boundary, the flat, and pit cells. The shaded area are flats or pits cells. In comparison, the LPCW DEM file contains no flat or pit cells.

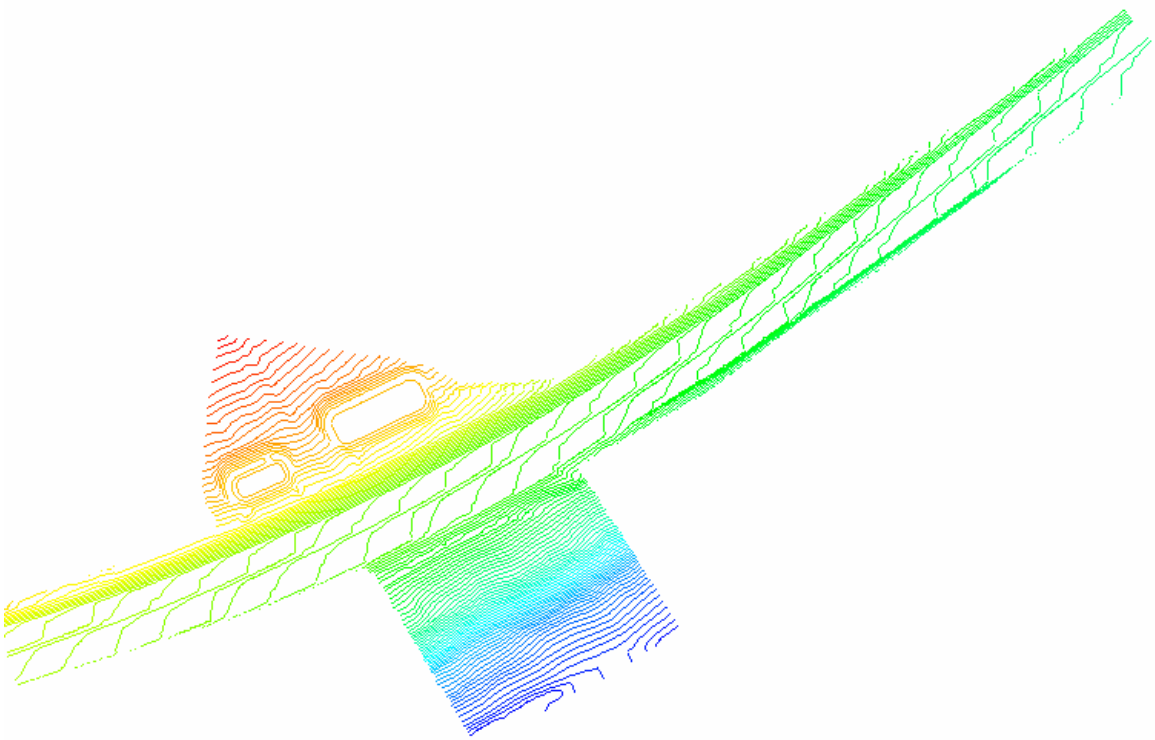


**Figure 30.** The original DEM file for Watershed SB10-11

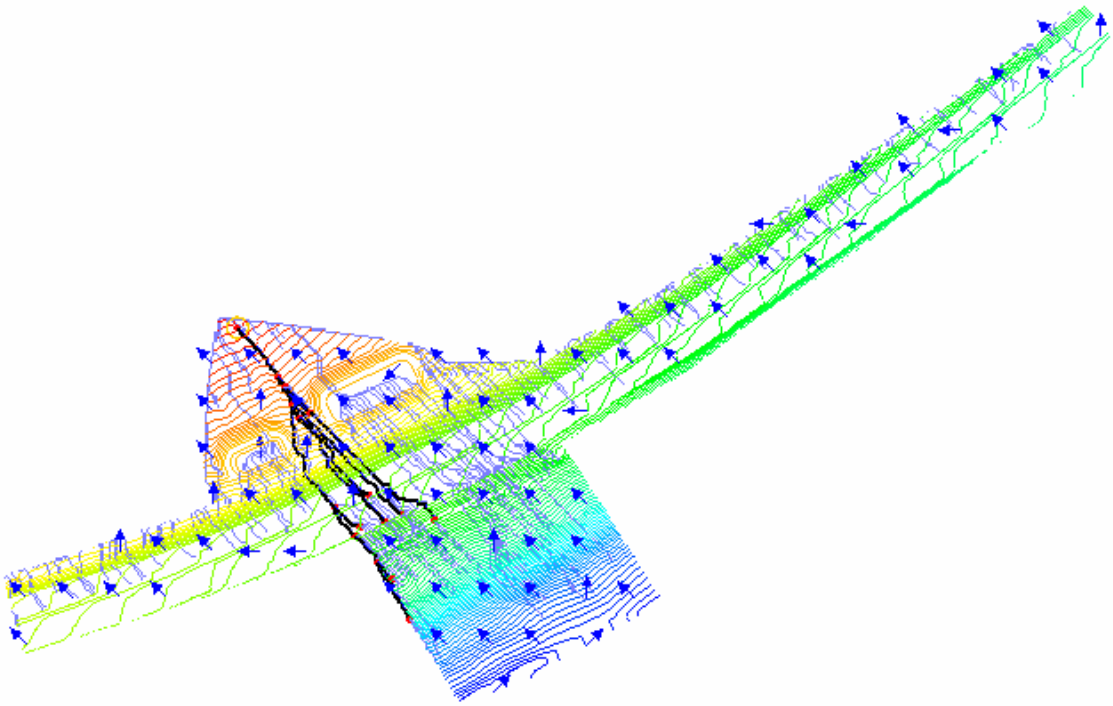


**Figure 31.** The DEM file with the flow direction, the stream networks, and stream feature arcs for Watershed SB10-11

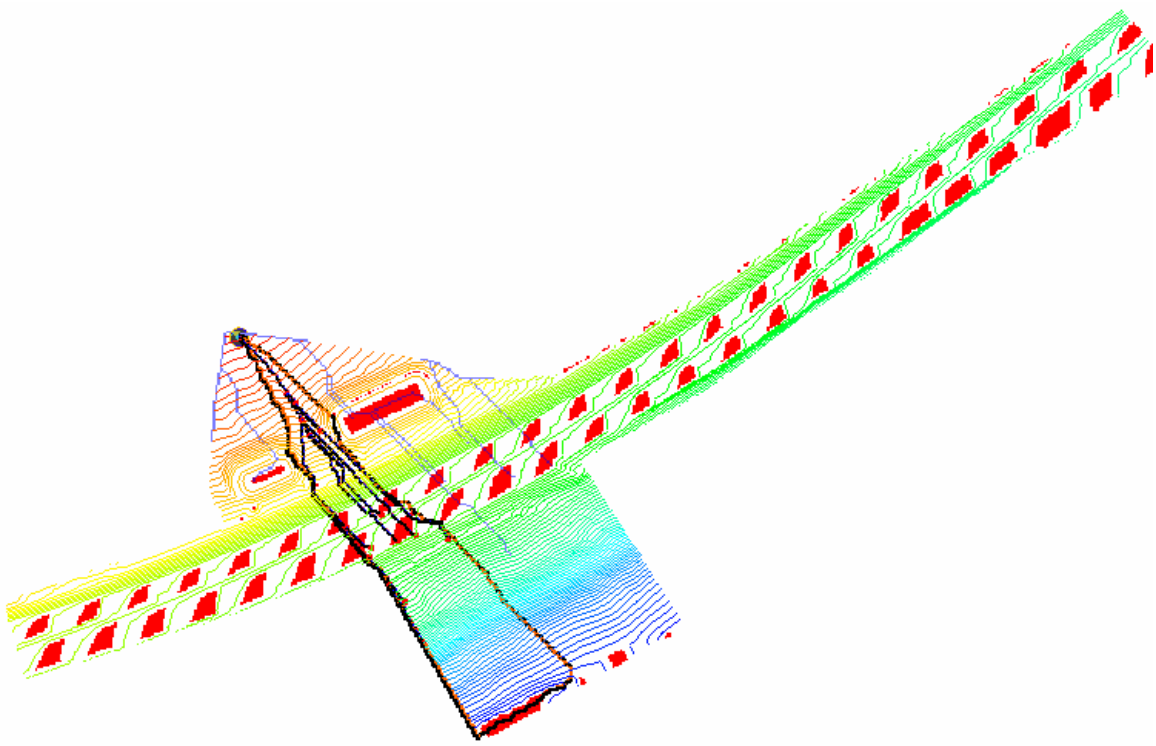


**Figure 32.** The automatically generated watershed boundary, flats, and pit cells for Watershed SB10-11

## 5.5 TOPOGRAPHICAL ANALYSIS BASED ON TIN DATA

Although the structure of TIN file is different from DEM, they represent the same reality. If there are flat or pit cells in DEM, there will be flat triangles or pit cells in corresponding TIN. If the watershed delineation is incorrect in DEM, the delineation is also incorrect in corresponding TIN. Figure 33 is the TIN file for Watershed SB10-11. Figure 34 is the downstream part of TIN file in detail. Figure 35 shows the TIN file with pit cells and flat triangles.

In the TIN file, a cell at the left-up is defined as the watershed outlet. Figure 35 also shows the flow direction and pit cells in this watershed, which indicates many flows go to pits and sink. They do not flow to the outlet.

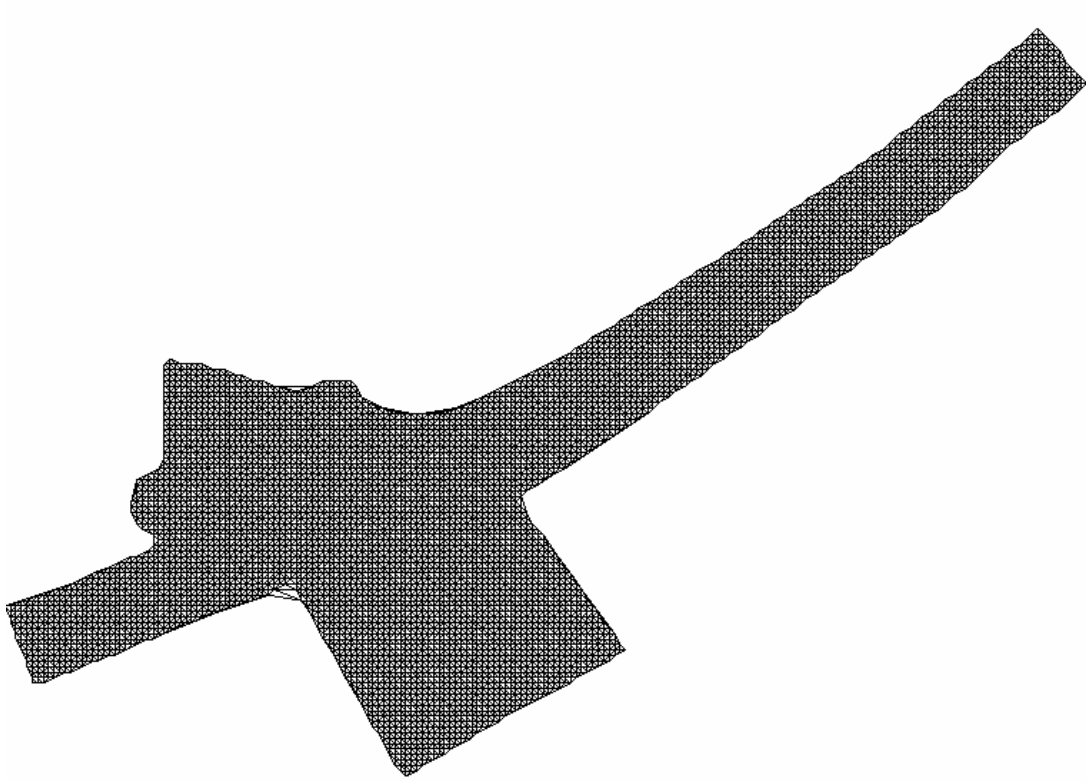


**Figure 33.** The TIN file for Watershed SB10-11

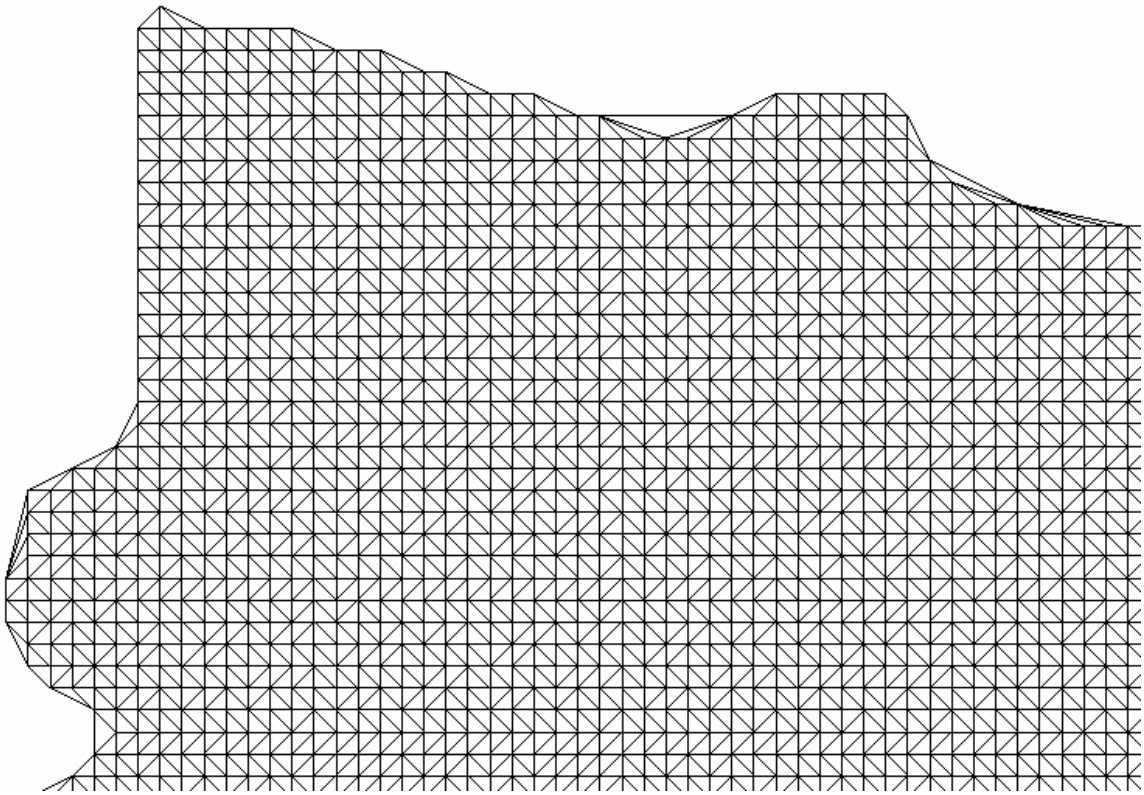


**Figure 34.** The downstream part of TIN file in detail

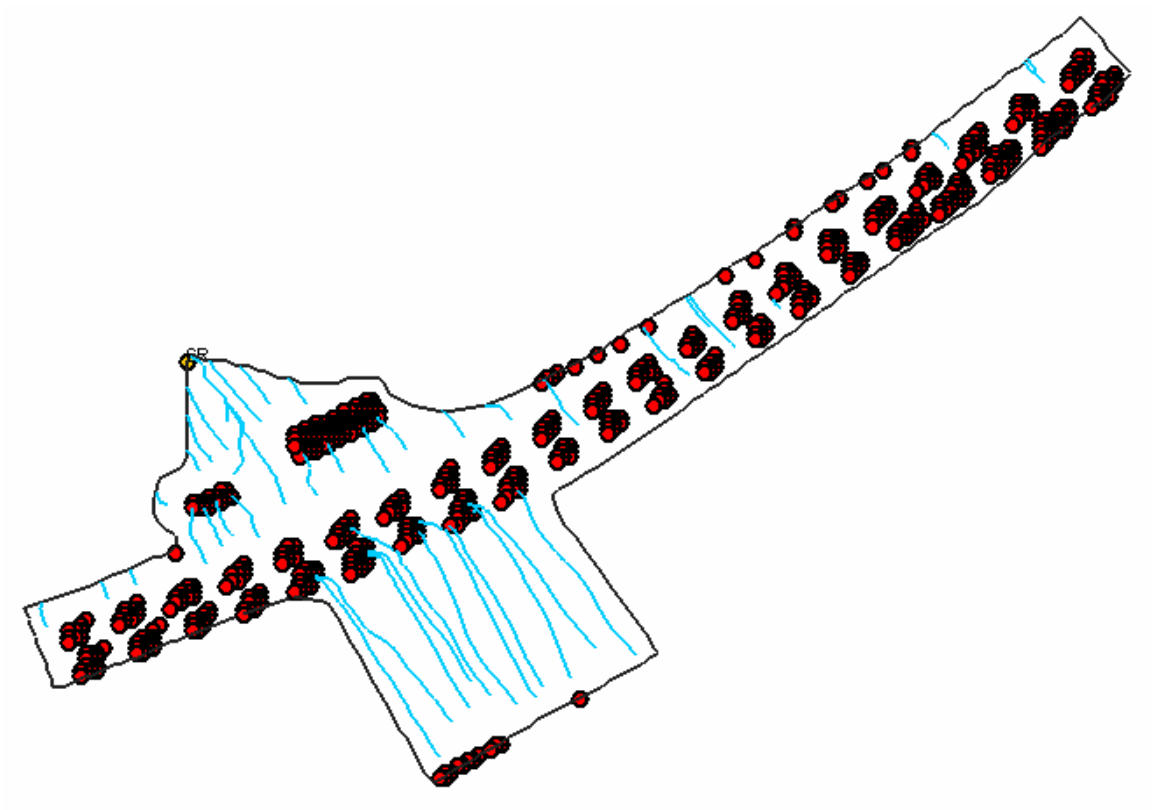



**Figure 35.** The TIN file with pit cells, flat triangles, and flow direction in Watershed SB10-11

Soil type and land use data are featured GIS data. They can be created by drawing a few polygons and assigning certain properties. Soil type and land use data can be viewed in Figure 36 and Figure 38. Although these figures are mainly used for illustrating schematic layout of the watersheds, the background gray polygons are soil type and land use data files.

As stated in Section 5.1, to create topographic GIS data (DEM or TIN), one needs to assign different values to thousands of pixels. This cannot be accomplished manually. Although GIS data are good resources, relying solely on topographic GIS data could be misleading. To overcome these difficulties, some measures will be taken in building the model.

# 6.0 I-99 ENVIRONMENTAL RESEARCH CASE STUDY

Hydrological modeling is performed on the selected two watersheds. These are approximately at about the stations of 400 (SB111, Watershed One) and 185 (SB10, SB11, Watershed Two) in the construction project's alignment. Because the topographic GIS data are not applicable in delineating the watershed boundary, a schematic watershed boundary is drawn based on real watershed boundary. Information related to elevation, such as watershed slope and stream slope, was also input manually based on field survey. Other information, such as watershed area, watershed length, and stream length were calculated in WMS automatically.

After the watershed is delineated, the remaining procedures in runoff modeling, such as drainage component editing, basin parameters estimation, channel and reservoir routing parameters estimation, and job control setting are independent of the GIS data and can be implemented without any problems.

The watersheds' contour maps, structure layout maps, and station numbers were obtained from Pennsylvania Department of Transportation (PENNDOT). The ponds characteristics were collected from ponds designers. The characteristics of the structures, such like pipe diameter, location, and slope, are obtained from the designers. The land use and soil type are obtained by our field survey. Measured runoff data were obtained from Ecotone flumes of AWK Consulting Engineers, Inc. Measured rainfall data were obtained from raingages of Skelly and Loy, Inc.

## 6.1 MODEL ASSEMBLY FOR WATERSHED ONE

Watershed One has an area of 0.085 sq mile (54.4 acres). A scanned map was referenced to the local coordinate system by registering the map. Three points were selected as base points. The first point is at the right edge of Pond SB111; it has the coordinate of (586.8 ft, 474 ft). The second point is at the left boundary; it has the coordinate of (430 ft, -288 ft). The third point is at the ridge of the watershed; it has the coordinate of (2040 ft, -913.6 ft). The origin is selected to be the center of a north arrow in the map. These points can be selected arbitrarily as long as their relative positions are correct. Figure 36 shows the schematic layout for Watershed One. The background map is hidden for clear display purpose. The main layer displayed is the drainage layer, which shows the sub-watersheds, streams and outlets position. The gray segment lines and polygons under the drainage layer are land use and soil type layer, which are inactive in the display.

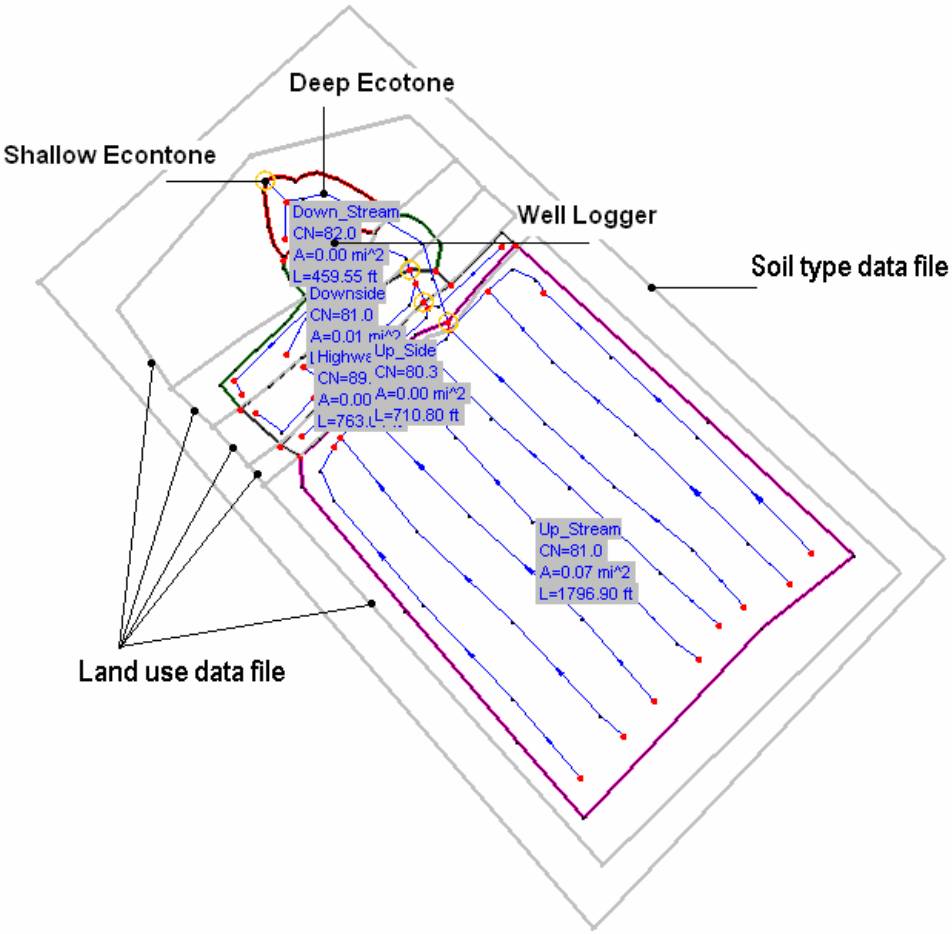



**Figure 36.** Schematic layout for Watershed One

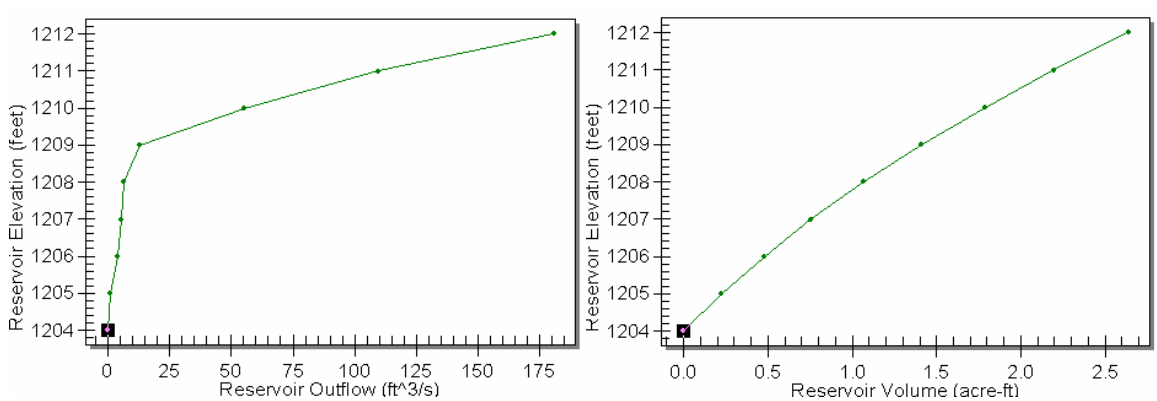
Based on the topography, re-built channel and the flow of water in the watershed, the whole watershed is divided into five sub-watersheds, Up_Stream, Up_Side, Highway, Down_Side, Down_Stream. From the overall view, the water flows from south-east to north-west. As it name indicated, Up_Stream is located in the upper part of the watershed and occupies about 75% area of the whole watershed. Most of Up_Stream is undisturbed forest, so water collected from this part is clean. The clean water flows through a lateral channel, and then goes to the most downstream outlet by an underground pipe, without passing the intermediate downstream watershed. The underground pipe is modeled as water diversion in WMS. Up_Side is located at the upper side of the highway. Because the highway pavement is higher than other land, the overland water from this part flows backward to the highway base channel. Then it is routed to the detention pond through an underground pipe. The watershed Up_Side is disturbed by construction and is close to the highway, so water from this part is dirty. The sub-watershed Highway collects dirty water from the road surface. On the highway, every 450 ft interval distance, there are two catch basins used to collect dirty water. The catch basins conduct water from the highway to the detention pond through an underground pipe. The sub-watershed Down_Side is located at the lower side of the highway. For the same reason, water from this part is also dirty water. The difference of this sub-watershed from Up_Side is that water flow to a highway ditch and then flows to the detention pond directly, without flowing backward. The pond, SB111, is at the bottom of sub-watershed Down_Side. Dirty water from the three sub-watersheds stays in the pond for sedimentation purpose before flowing downstream. The last sub-watershed, Down_Stream, is located at the bottom of the whole watershed. Since it is far from highway and most area is covered by vegetation, water from this part is clean and flows directly to the final outlet. The delineated watershed and sub-watersheds are assigned as the drainage layer for use in later modeling.

Each sub-watershed has an outlet, which is used to route its hydrograph to downstream. The outlet names of the sub-watershed are UpStrm, UpSide, Hiway, DnSide and Final, respectively. Because water from Up_Stream flows directly to the Final outlet, a water diversion and return is used in UpStrm routing. UpSide and Hiway outlet routings use pipe routing instead of open channel routing method. The DnSide outlet uses regular

channel routing. The Final outlet has no routing since it is at the end of the whole watershed.

The watershed's land use layer is divided into five parts and four categories. The four categories are Up_Stream, Highway, Down_Stream and Highway_Sides. As their names indicate, they are mainly distributed in the corresponding sub-watersheds, but not always strictly coincident. For example, the Up_Side sub-watershed may have small part of forest, which is the land use of Up_Stream. The Up_Side and Down_Side sub-watersheds share the same land use category -- Highway_Sides. The curve numbers of the corresponding land use are selected from National Engineering Handbook (SCS, 1972). The composite curve number of a sub-watershed is calculated based on area weight of different land use inside the sub-watershed. The soil type layer of the watershed is set to unique, type D -- loam, due to its small area. Another reason to use unique soil type is that curve number can be adjusted in land use layer design.

The detention pond SB111 is modeled as a reservoir. Its characteristics are obtained from pond designer and are displayed in Figure 37. The reservoir is accompanied with the outlet -- DnSide. Since there is no heavy rainfall in previous days before each modeled rainfall event, an initial condition of bottom elevation (SB111: 1204 ft) is employed in modeling. All basin areas are calculated by WMS.




(a) Elevation - outflow relationship (b) Elevation - volume relationship

**Figure 37.** Elevation - storage - outflow relationship of SB111

Because the watershed's area is small, spatially uniform rainfall is used in the model. Average precipitation and rainfall time distribution is input into the model. For the unit hydrograph, SCS dimensionless method is employed. The most important parameters in this calculation are watershed length, SCS curve number and watershed slope. The watershed lag time is calculated using Equation 3.1. Watershed length and SCS curve number can be calculated by WMS using drainage, land use and soil type layers, since the coordinate system is already set.

## 6.2 MODEL ASSEMBLY FOR WATERSHED TWO

The procedure for constructing the second watershed is the same. The second watershed has an area of 0.072 sq mile (46.08 acres). Three points were selected as base points to register the map. The first point is at the right edge of boundary; it has the coordinate of (1675 ft, 470 ft). The second point is at the left boundary; it has the coordinate of (-1440 ft, -1235 ft). The third point is at the up ridge of the watershed; it has the coordinate of (285 ft, -1190 ft). The origin is selected to be the center of a north arrow in the map. Figure 38 shows the schematic layout for Watershed Two.

The whole watershed is divided into seven sub-watersheds, Up_Stream, Right_Highway1, Right_Highway2, Right_Highway3, Right_Highway4, Left_Highway, Down_Stream. From the whole watershed's view, the water flows from south-east to north-west. Up_Stream is located in the upper part of the watershed. Water collected from this part is clean water. The clean water flows through a lateral channel, and then goes to the most downstream outlet by an underground pipe directly. The underground pipe is modeled as water diversion in WMS. Right_Highway1 to Right_Highway2 is the right part of the highway. Left_Highway is the left part of the highway. These five sub-watersheds collect dirty water from the highway. On the highway, every 450 ft interval distance, there are two catch basins used to collect dirty water. The catch basins conduct water from the highway to the detention ponds through an underground pipe. The two detention ponds, SB10 and SB11, are at the bottom of sub-watershed Left_Highway and

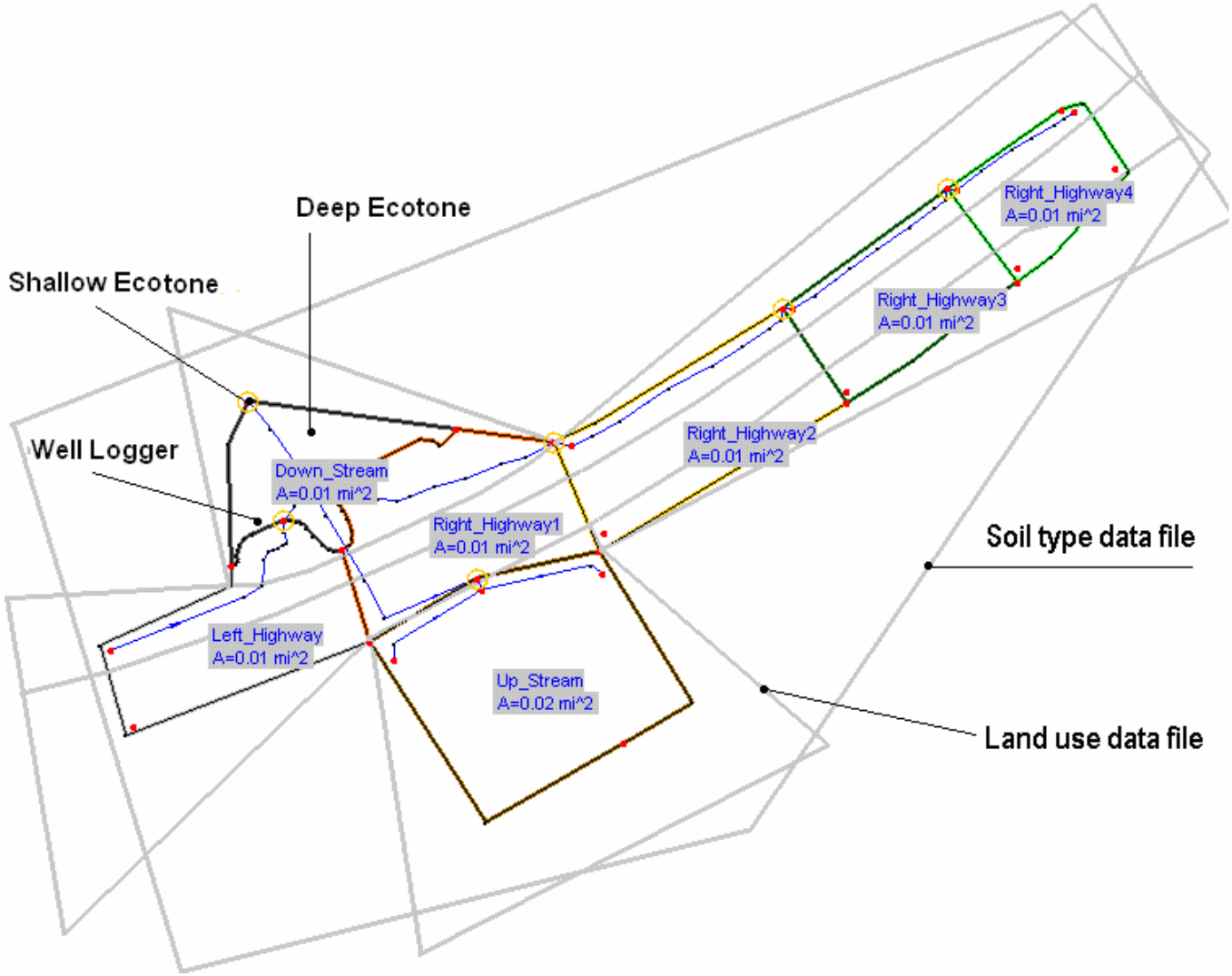
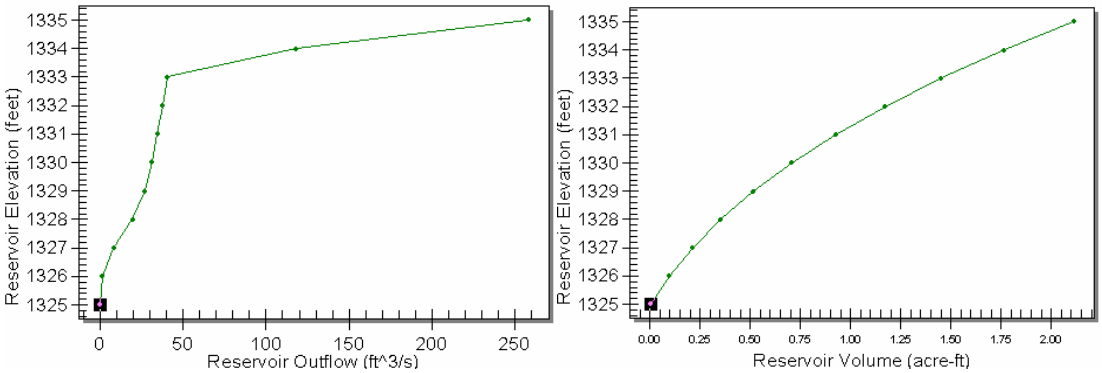



**Figure 38.** Schematic layout for Watershed Two

Right_Highway1, respectively. Their characteristics are obtained from pond designer and are displayed in Figure 39 and Figure 40. The last sub-watershed, Down_Stream, is located at the bottom of the whole watershed. Water from this part is clean water and flows directly to the final outlet.

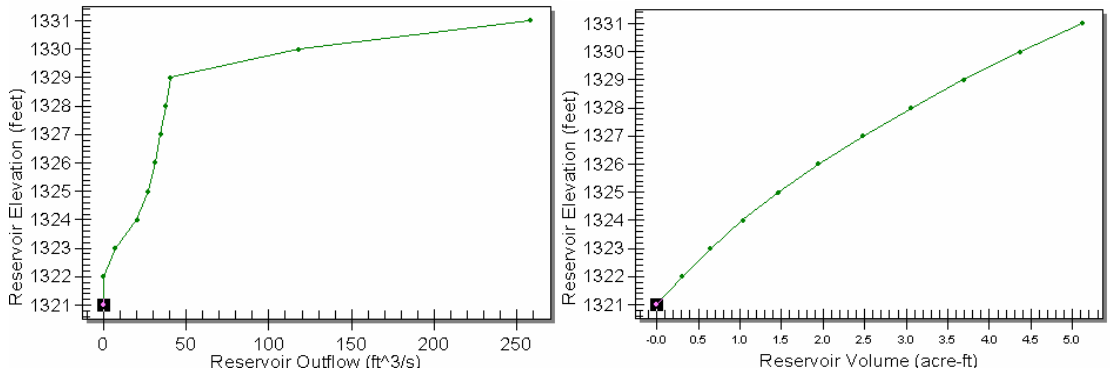
Each sub-watershed has an outlet. The outlet names of the sub-watershed are UpStrm, Left, Right1, Right2, Right3, Right4, and Final, respectively. Because water from Up_Stream flows directly to the Final outlet, a water diversion and return is used in UpStrm routing. Right1 to Right4 and Left outlets use regular channel routing. The Final outlet has no routing since it is at the end of the whole watershed.

As stated in Section 6.1, the watershed's land use layer is divided into five parts and four categories. The four categories are Up_Stream, Highway, Down_Stream and Highway_Sides. The curve number calculation is the same as in Watershed One. The soil type layer of the watershed is set to unique, type D -- loam, due to its small area. The two detention ponds are modeled as reservoirs. The reservoirs are accompanied with outlets Right1 and Left. Since there is no heavy rainfall in previous days before each modeled rainfall event, an initial condition of bottom elevation (SB10: 1325 ft; SB11: 1321ft) is employed in modeling. Spatially uniform rainfall is used in the model. SCS dimensionless unit hydrograph method is employed.




(a) Elevation - outflow relationship (b) Elevation - volume relationship

**Figure 39.** Elevation - storage - outflow relationship of SB10




(a) Elevation - outflow relationship (b) Elevation - volume relationship

**Figure 40.** Elevation - storage - outflow relationship of SB11

## 6.3 MODELED RAINFALL EVENTS

For this research, eight significant stormwater events during the period from Oct 2005 to Sept 2006 were analyzed. Figure 41 (a) to Figure 41 (h) show the bar graph for the eight stormwater events.

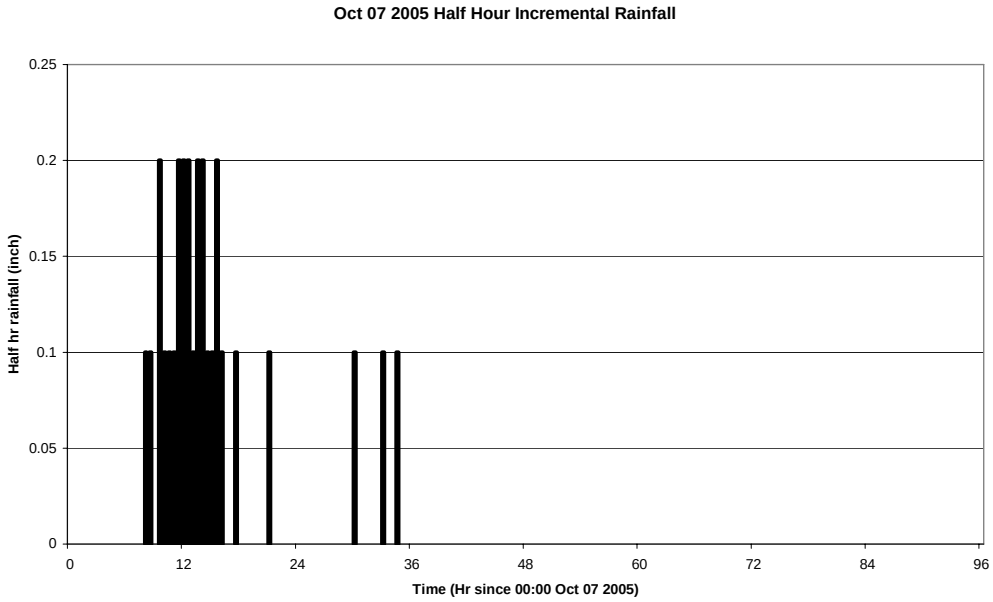



**Figure 41.** Half hour incremental rainfall for eight storm events

(a) From Oct 07 to Oct 10 2005 (Total rainfall depth = 2.90 inch)

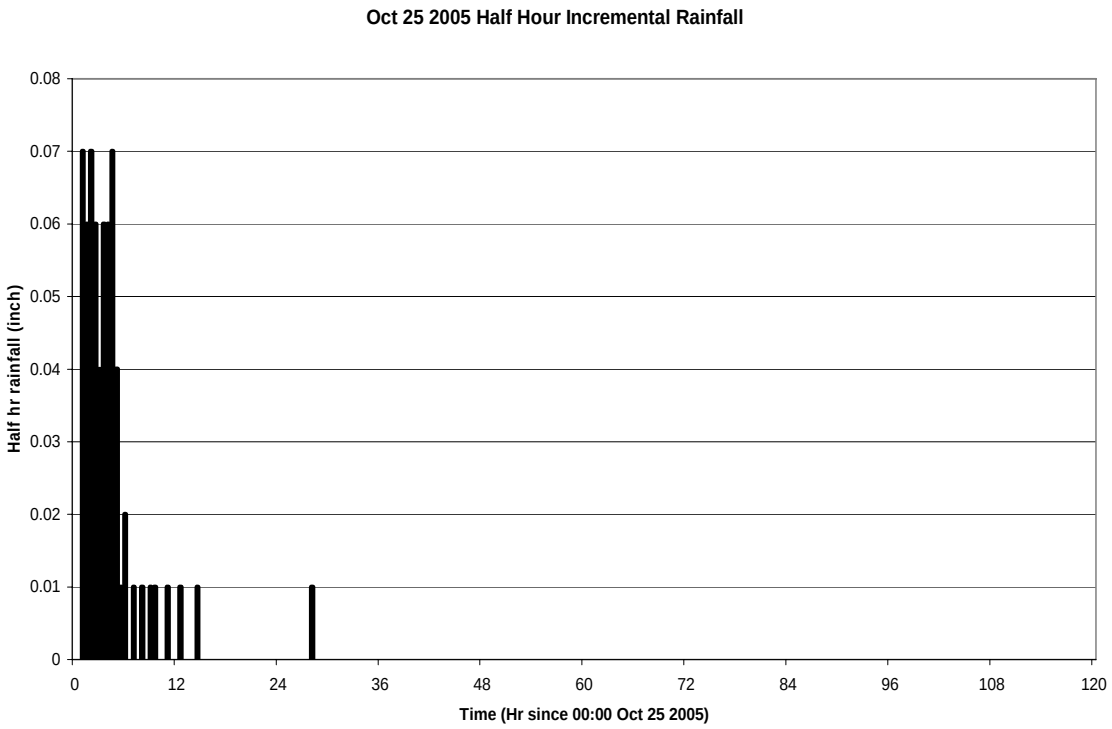



**Figure 41.** (b) Half hour incremental rainfall from Oct 25 to Oct 29 2005

(Total rainfall depth = 0.64 inch)

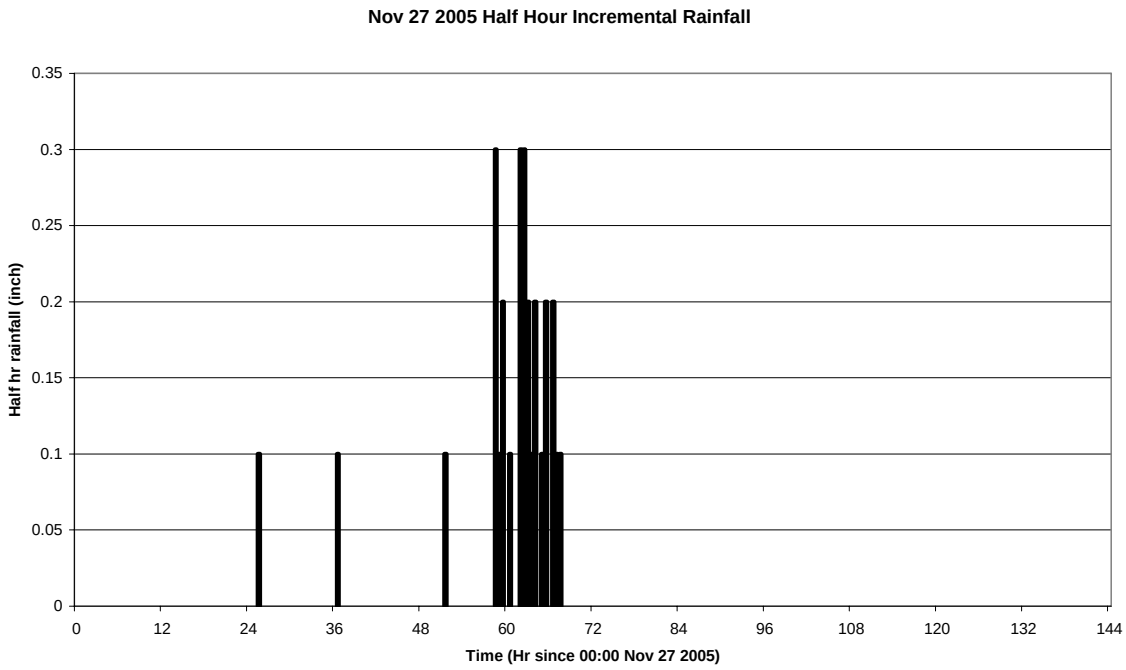



**Figure 41.** (c) Half hour incremental rainfall from Nov 27 to Dec 02 2005

(Total rainfall depth = 2.80 inch)

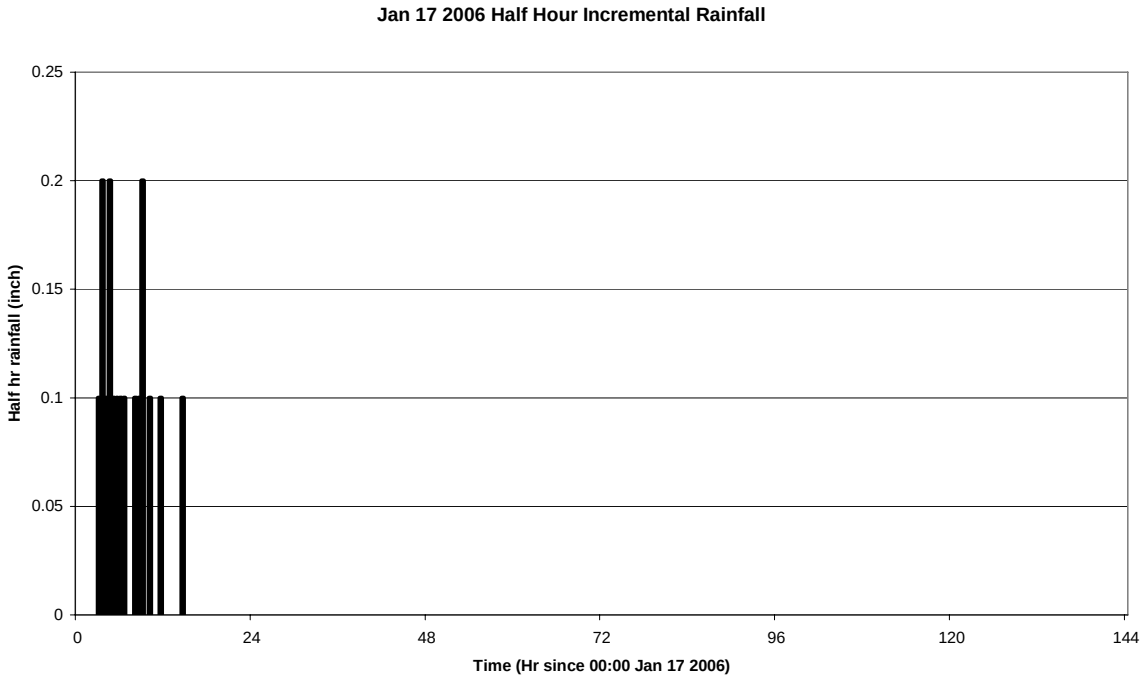



**Figure 41.** (d) Half hour incremental rainfall from Jan 17 to Jan 22 2006

(Total rainfall depth = 1.70 inch)

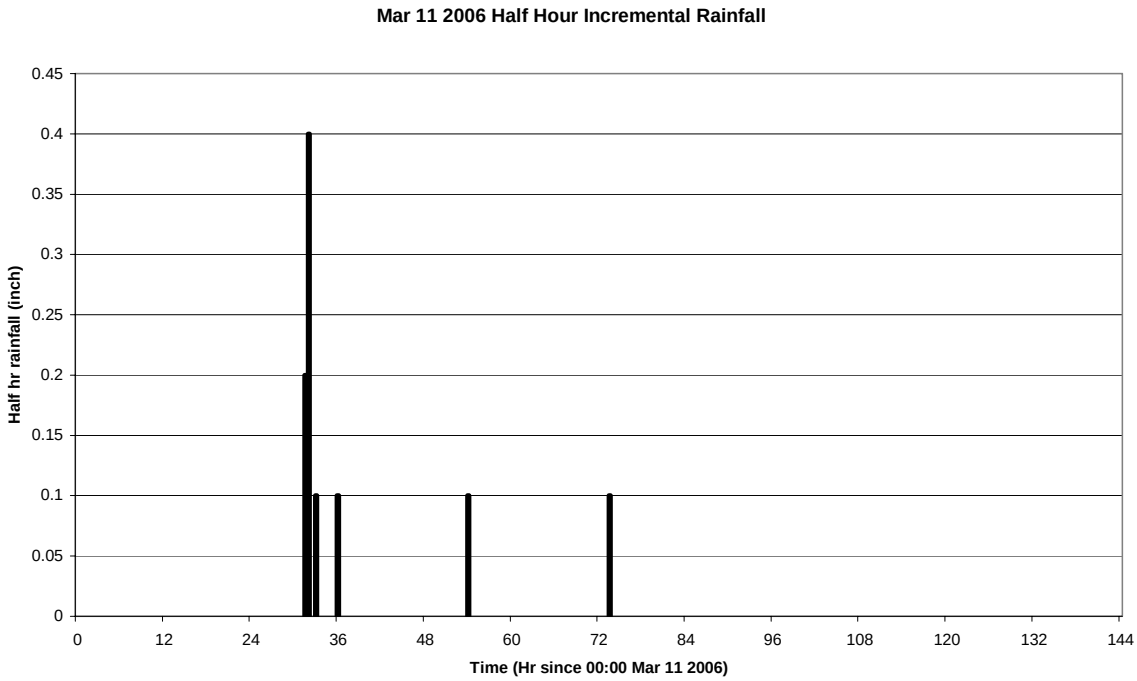



**Figure 41.** (e) Half hour incremental rainfall from Mar 11 to Mar 16 2006

(Total rainfall depth = 1.00 inch)

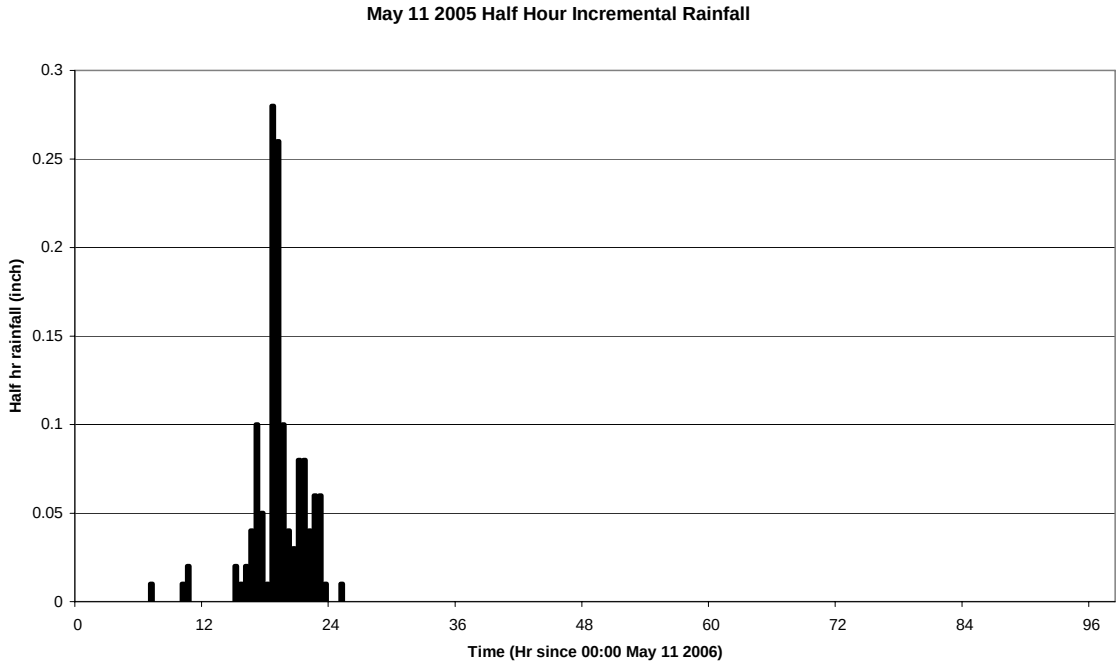



**Figure 41.** (f) Half hour incremental rainfall from May 11 to May 14 2006

(Total rainfall depth = 1.34 inch)

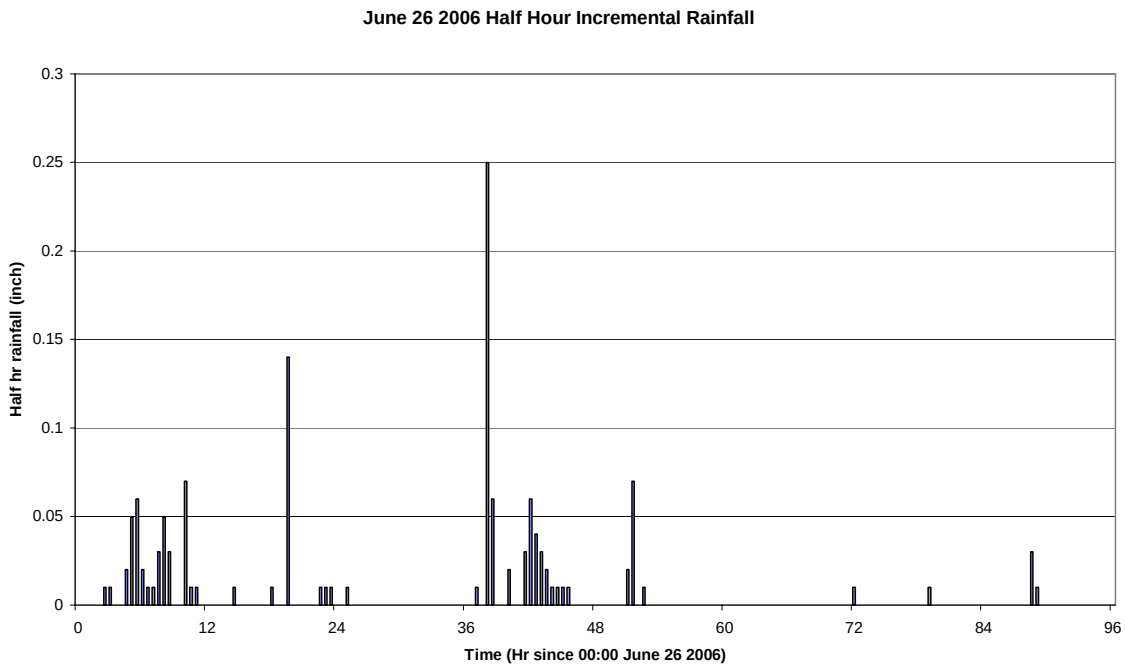



**Figure 41.** (g) Half hour incremental rainfall from June 26 to June 29 2006

(Total rainfall depth = 1.31 inch)

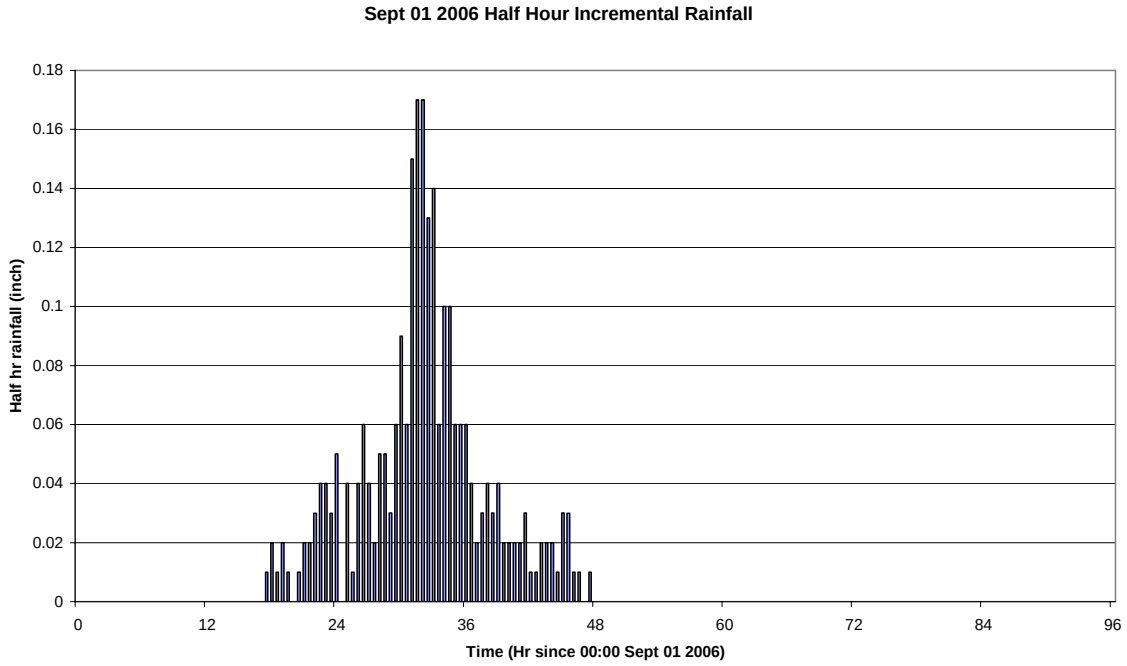



**Figure 41.** (h) Half hour incremental rainfall from Sept 01 to Sept 04 2006 (Total rainfall depth = 2.55 inch)

## 6.4 PARAMETER SELECTION AND MODEL CALIBRATION

The most important criteria for comparing model predictions to measured values are the total volume of the runoff, peak time and peak discharge values. The relationship between some important model parameters and model results are discussed in Section 3.11.

The watershed area, watershed length, precipitation depth and temporal distribution are important to watershed modeling. They are easy to estimate and errors are not large. However, some important parameters are very difficult to estimate. These include the curve number for each land use, the slope for each sub-watershed, the water flow velocity in Muskingum routing and the initial condition of reservoir. Sometimes, the parameters are different for different rainfall events even for the same watershed. The reservoir elevation-storage and elevation-discharge characteristics are easy to obtain from the

design engineer. However, the reservoir will be partially filled after a period of operation. The reservoir initial conditions are also difficult to estimate.

The model is used to simulate different storm events. To keep the model robust, the watershed characteristics, such as slope, watershed length and area are all the same in different storm events. The curve numbers are not the same for different storm events. This is true because soil moisture content varies before different storm events.

Most parameters are determined by field survey. These parameters include watershed length, watershed area, precipitation depth and temporal distribution, reservoir characteristics and reservoir initial elevation. However, some other parameters cannot be determined accurately solely by field survey. These parameters include curve number (CN), watershed slope, watershed length, water velocity in channel, Muskingum K, Muskingum X. For example, CN, the watershed slope, and watershed length are used to calculate the lag time of a watershed. CN is used to calculate the excess rainfall. Water velocity in channel, Muskingum K, and Muskingum X are used to route hydrograph in channel. CN is determined based on land use, soil type and antecedent moisture condition (AMC). It is difficult to decide the accurate CN by only surveying and looking up tables. The same reasons exist for watershed slope and watershed length. The watershed slope is the average slope of a watershed. However, the watershed is irregular. The slope varies from one part to another part. The average slope is difficult to determine. Water velocity in channel depends on many factors, such as channel slope, roughness, and channel shape. The channels' shape, slope, and roughness in the watershed are very irregular. It is difficult to determine these parameters accurately only by field survey. One solution to determine these parameters is to combine field survey with the trial-and-error method. First, parameters' rough value is estimated by field survey. Then, these parameters are adjusted in order to fit modeled hydrograph to measured hydrograph.

Ecotone water stage recorders were installed at the outlets of Watershed One and Watershed Two. They are used to collect runoff data to calibrate the model. Although two watersheds were selected, only Watershed Two will be discussed. Table 3 shows the

runoff rainfall ratio for each storm event for Watershed Two (SB10-11). Due to some shortcomings of WMS, which are explained in Section 6.7, the modeling results are not all satisfactory. Only two events will be discussed in this dissertation.

**Table 3.** Runoff Rainfall Ratio for Each Storm Event in Watershed Two

| Each Event | Oct 07 2005 | Oct 25 2005 | Nov 27 2005 | Jan 17 2006 | Mar 11 2006 | May 11 2006 | June 26 2006 | Sept 01 2006 |
|---|---|---|---|---|---|---|---|---|
| Rainfall Depth (inch) | 2.9 | 0.64 | 2.8 | 1.7 | 1 | 1.34 | 1.31 | 2.55 |
| Rainfall Volume on the Watershed ($ft^3$) | 485084 | 107053 | 468357 | 284360 | 167270 | 224142 | 219124 | 426540 |
| Runoff Volume of the Watershed ($ft^3$) | 142341 | 77531 | 262528 | 89015 | 34977 | 44466 | 90631 | 121958 |
| Runoff Depth (inch) | 0.8508 | 0.4634 | 1.5693 | 0.5321 | 0.2091 | 0.2658 | 0.5418 | 0.7291 |
| Runoff Rainfall Ratio | 0.2934 | 0.7241 | 0.5605 | 0.3130 | 0.2091 | 0.1984 | 0.4136 | 0.2859 |

## 6.5 PARAMETERS FOR WATERSHED TWO

### 6.5.1 Parameters for Watershed Two, Oct 07 2005 Event

The rainfall distribution and reservoir characteristics are shown before. After several trial modifications of model parameters, a set of reasonable parameters are determined for this watershed and event. The watershed parameters used in Watershed Two, Oct 07 2005 Event are listed in Table 4.

**Table 4.** Watershed parameters for Watershed Two, Oct 07 2005 Event

| | Up Stream | Left Highway | Right Highway1 | Right Highway2 | Right Highway3 | Right Highway4 | Down Stream |
|---|---|---|---|---|---|---|---|
| Area (acre) | 11.52 | 6.4 | 7.68 | 7.04 | 5.76 | 3.84 | 3.84 |
| Composite CN | 65.8 | 80.4 | 75.7 | 78.7 | 78.3 | 81.2 | 65 |
| Basin length (ft) | 785.44 | 843.19 | 844.34 | 932.45 | 744.28 | 575.51 | 663.68 |
| Overland slope (%) | 0.003 | 0.02 | 0.01 | 0.01 | 0.01 | 0.01 | 0.02 |

There are five outlet channel flow routings. Their parameters are listed in Table 5.

**Table 5.** Routing Parameters for Watershed Two, Oct 07 2005 Event

| | WV (ft/s) | NSTPS | AMSKK (hr) | *X* (hr) |
|---|---|---|---|---|
| Up_Stream | N/A, because diversion pipe is used. | | | |
| Left | 0.0016 | 80 | 6.577 | 0.2 |
| Right1 | 0.0016 | 73 | 6.029 | 0.2 |
| Right2 | 0.0016 | 121 | 9.979 | 0.2 |
| Right3 | 0.0016 | 145 | 12.028 | 0.2 |
| Right4 | 0.0016 | 111 | 9.167 | 0.2 |
| Final | N/A, because no routing at the final outlet. | | | |

where: *WV* = The water velocity in the channel;

*NSTPS* = The number of integer steps for the Muskingum routing;

*AMSKK* = Muskingum *K* coefficient in hours for the reach, *K* = Channel Length / WV;

*X* = Muskingum *X* coefficient for the reach.

The curve numbers for each land use type are listed in Table 6.

**Table 6.** CN for each land use for Watershed Two, Oct 07 2005 Event

| Index | Land use name | CN value for Soil Type D |
|---|---|---|
| 0 | Up_Stream | 65 |
| 1 | Highway | 90 |
| 2 | Down_Stream | 65 |
| 3 | Highway_Sides | 70 |

### 6.5.2 Parameters for Watershed Two, Oct 25 2005 Event

Like in Oct 07 2005 Event, rainfall distribution and reservoir characteristics are shown before. The watershed area, basin length, and overland slope are the same as in previous events. The only different parameters are curve numbers. After several trials, a set of reasonable CNs are determined for this watershed and event. Table 7 displays the CN for this event.

**Table 7.** Curve Number for Watershed Two, Oct 25 2005 Event

| | Up Stream | Left Highway | Right Highway1 | Right Highway2 | Right Highway3 | Right Highway4 | Down Stream |
|---|---|---|---|---|---|---|---|
| Composite CN | 98 | 98 | 98 | 98 | 98 | 98 | 98 |

There are five outlet channel flow routings. Their parameters are listed in Table 5. The curve numbers for each land use type are listed in Table 8.

**Table 8.** CN for each land use for Watershed Two, Oct 25 2005 Event

| Index | Land use name | CN value for Soil Type D |
|---|---|---|
| 0 | Up_Stream | 98 |
| 1 | Highway | 98 |
| 2 | Down_Stream | 98 |
| 3 | Highway_Sides | 98 |

## 6.6 WMS RESULTS FOR WATERSHED TWO

The Ecotones installed in the outlet flumes record water depth. A rating curve is used to convert the water depth to the discharge. Figure 42 shows the rating curve for flumes used.

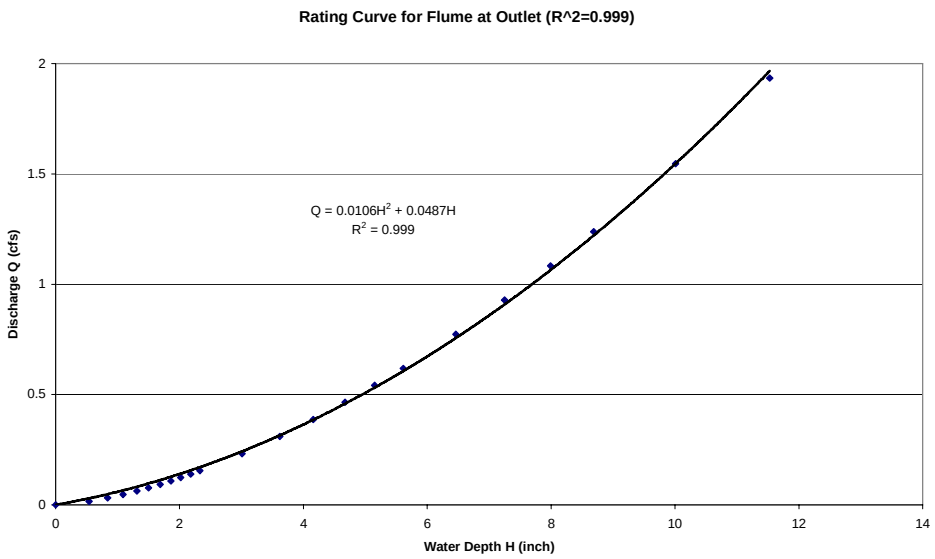



**Figure 42.** The rating curve for Ecotones in the two studied watersheds

### 6.6.1 Watershed Two, Event of Oct 07 2005

Oct 07 2005 Rainfall Event began at 00:00 AM Oct 07 2005 and ended at 23:59 PM Oct 10 2005. The total rainfall depth of this event is 2.90 inches. Figure 41 (a) is the half hour incremental rainfall data. Figure 43 shows the comparison of measured and WMS modeled hydrograph. It is seen that the hydrograph were predicted poorly.

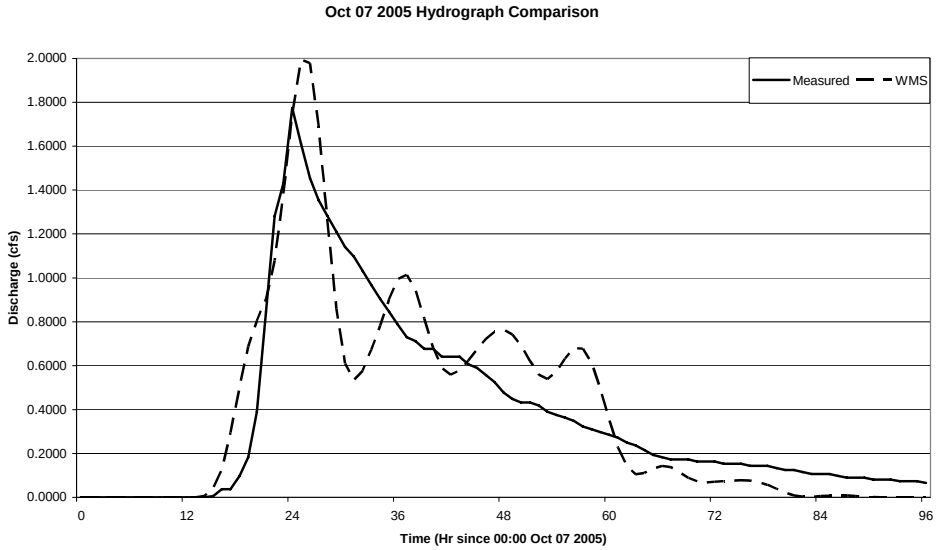



**Figure 43.** Measured and WMS modeled hydrograph for Oct 07 2005 Event

### 6.6.2 Watershed Two, Event of Oct 25 2005

Oct 25 2005 Rainfall Event began at 00:00 AM Oct 25 2005 and ended at 23:59 PM Oct 29 2005. The total rainfall depth of this event is 0.64 inches. Figure 41 (b) is the half hour incremental rainfall data. Figure 44 shows the comparison of measured and WMS modeled hydrograph.

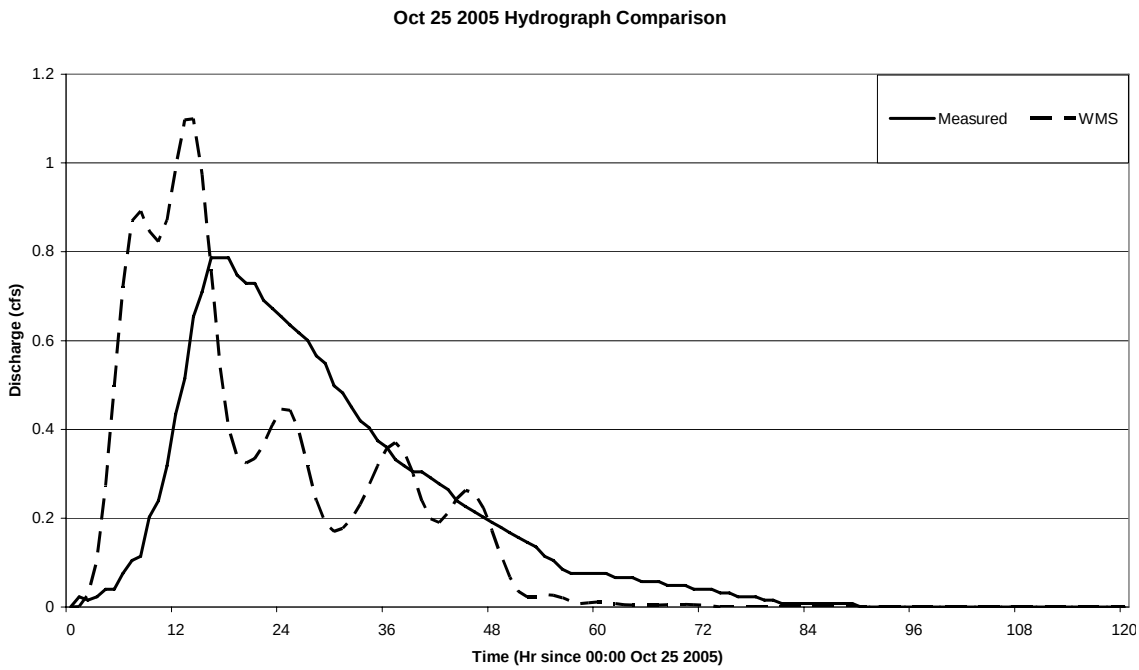



**Figure 44.** Measured and WMS modeled hydrograph for Oct 25 2005 Event

## 6.7 ANALYSIS OF WMS RESULTS

The most important criteria for comparing modeled hydrograph to measured hydrograph are the total volume of the runoff, peak time and peak discharge values. Table 9 shows the comparison of these three criteria for the two events modeled. Based on the project need, a deviation within 15% on runoff total volume and peak discharge is regarded as satisfactory. A deviation within 120 minutes (equal two modeling time intervals) on peak time is regarded as satisfactory. The hydrological modeling object is a watershed, which contains various kinds of land use, soil type, vegetation coverage, irregular surface slope,

complicated stream networks and channels. Although the hydrological theory works well in laboratory, many uncertain factors and unavailable parameters in the watershed may influence the modeling results. Rainfall spatial distribution may also add inaccuracy in the modeling. There is no standard criterion in watershed modeling to evaluate the modeling quality. The criteria adopted in this research were selected based on other researchers experience cited in the literature.

Quinn et al. (1993) modeled the discharge for the River Severn watershed at Plynlimon, Wales. The difference between observed and predicted peak discharge was 19%; the difference in peak time was up to 5 hours. Campling et al. (2002) modeled the River Ebonyi headwater watershed in Nigeria. The difference between observed and calculated total runoff was about 15%; in the peak discharge it was 31%; the deviation of peak times was up to 3 hours. Muzik (1996) modeled Waiparous Creek in the Alberta, Canada. The difference in total runoff was up to 16%; the difference of peak discharges was up to 18%; in peak times up to 2.5 hours. Yue et al. (2000) modeled the Kaifu River basin in Japan. The difference in peak discharges was up to 15%; in peak times up to 3 hours. Whigham et al. (2001) modeled the Namoi River catchment, Australia. The deviation between total runoff was 17%; in peak discharges up to 26%. Based on these, it was decided that 15% deviation would be acceptable. The above criteria were adopted in the rest of the study.

**Table 9.** The comparison of the three criteria in two events for Watershed Two

| | | Total Runoff Volume ($ft^3$) | Peak Discharge (cfs) | Peak Time (min) |
|---|---|---|---|---|
| Oct 07 2005 | Measured | 142341 | 1.77 | 1440 |
| | Modeled | 137675 | 1.99 | 1500 |
| | Deviation | -3.3 % | 12.4 % | 60 min |
| Oct 25 2005 | Measured | 77531 | 0.79 | 1020 |
| | Modeled | 74786 | 1.1 | 840 |
| | Deviation | -3.5 % | **39.2 %** | **-180 min** |

As we can see from Table 9, both deviations of the modeled runoff total volume are accepted within satisfactory; one of two deviations of the peak discharge values is not satisfactory; one of two deviations of the peak times is not satisfactory. As mentioned before, among the three criteria, runoff total volume is easy to model. However, the peak discharge and peak time are difficult to model accurately.

There are several shortcomings of this model. First, although the calculation methods were chosen in each section of the model, the WMS model does not provide calculation details, which may be important to modeling results. For example, if the calculation time interval is changed, the modeling results are also changed a little. This behavior is not expected in modeling. Since the WMS is an integrated model, we do not know the program codes and do not know how the calculation interval influences the modeling results in algorithm.

Second, the model has some restrictions. For example, only one routing method can be assigned to a channel. However, in reality, one channel can have different routing methods. Another example is the maximum limitation for the number of hydrograph ordinates. The maximum limitation is 2000. If we have 5 minutes calculation time interval, the maximum modeling time period is 2000×5 = 10000 minutes = 166.67 hours = 6.94 days. This is a big restriction of the model.

Third, to obtain modeled hydrograph that is close to measured hydrograph, some parameters are selected unrealistic. For example, the water velocity in the channel is set to be 0.0016 ft/sec. This is much slower than real water velocity in the watershed channels. The slope of the sub-watershed is set to be 0.003 - 0.01 %, which is much milder than actual sub-watershed slope. However, from Table 2, we can see that if the water velocity is chosen to be larger, or the slope is to be chosen larger, the modeled peak time will be early than measured; the modeled peak discharge will also be higher than measured. Sensitivity analysis proves this behavior.

Furthermore, some of the modeling results are not satisfactory and need to be improved. As we can see from Table 9, one of two deviations of the peak discharge values is not satisfactory; one of two deviations of the peak times is not satisfactory. The modeled hydrograph shapes do not fit the measured shapes very well.

To compensate for the above shortcomings and get better results, a new model, Highway Watershed Model (HWM) will be developed and presented in Chapter Seven. It will be applied in the project in Chapter Eight.

# 7.0 HIGHWAY WATERSHED MODEL (HWM) DEVELOPMENT

## 7.1 MOTIVATION FOR DEVELOPING HWM

Although WMS is widely used in watershed modeling, it may not be suitable for the present case. Sensitivity analysis indicates the total discharge is mainly related to curve number; the hydrograph shape is mainly related to the watershed slope. Other parameters influence the hydrograph very little. In some cases in WMS, no matter how the parameters are adjusted, the modeled hydrographs still do not match the measured hydrograph very well. This may be the shortcoming of WMS. As discussed in Section 6.8, there are some shortcomings and restrictions in manipulating in WMS. To make the model more flexible and to have full control in its implementation, a new model, the Highway Watershed Model (HWM), was developed by writing completely new programs. Because this model is developed directly for I-99 Environmental Research, the design schematic of the model is especially suitable for the project. HWM module structures are documented in this chapter.

## 7.2 WATERSHED DELINEATION

First, the modeled watershed is divided into several sub-watersheds according to topographical characteristics. Mountain ridges are normally used as sub-watersheds divider. Deep valleys are normally used as watershed channel. The sub-watershed and channel schematic map can be drawn before modeling. This part of work is done manually and is the pre-processing part of the hydrological model component. Figure 45 is the schematic of Watershed One. Figure 46 is the schematic of Watershed Two.

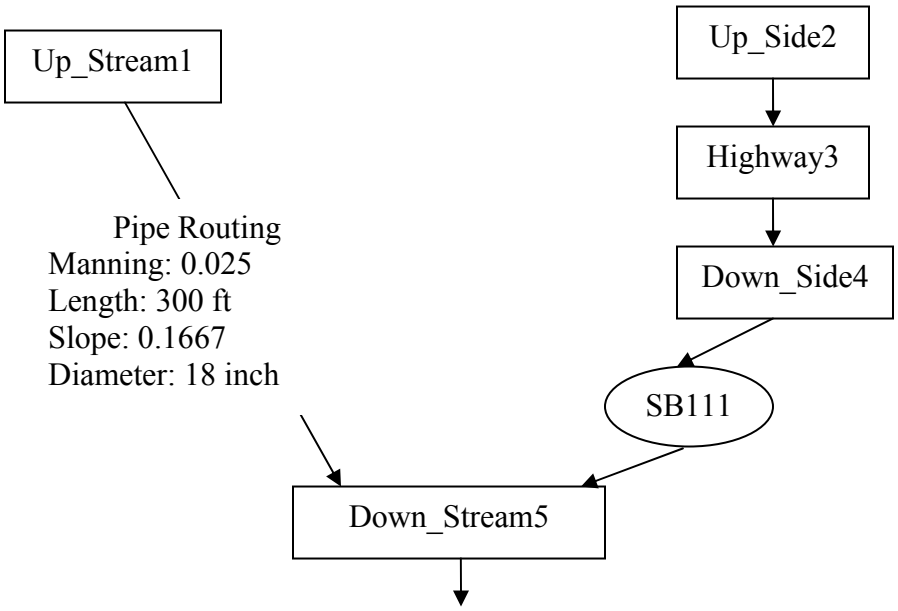



**Figure 45.** Schematic diagram of Watershed One

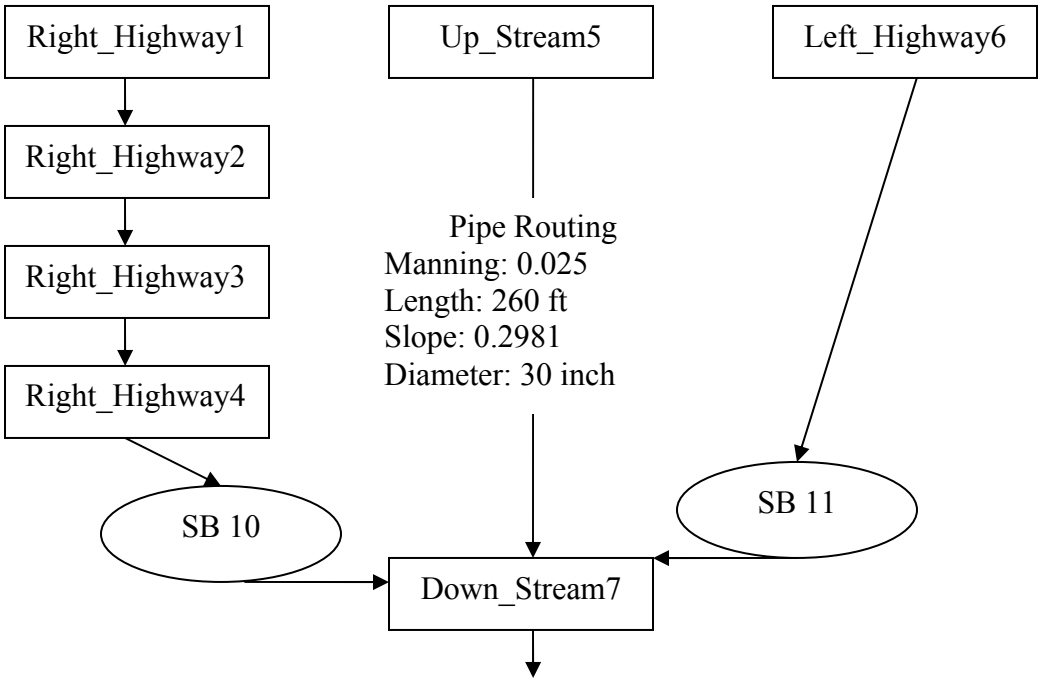



**Figure 46.** Schematic diagram of Watershed Two

## 7.3 EXCESS RAINFALL GENERATION

Not all the rainfall is converted to runoff. Part of rainfall is lost due to infiltration, evaporation, evapotranspiration, etc. For short term rainfall-runoff modeling, the evaporation and evapotranspiration can be assumed to be negligible compared to infiltration. The excess rainfall can be generated from total rainfall using SCS abstraction method, which is described in Chapter Two. HWM utilizes half-hour accumulative total rainfall data to generate incremental excess rainfall data.

## 7.4 SCS AND LE UNIT HYDROGRAPH

The unit hydrograph method is used to produce hydrograph at outlet of each sub-watershed. Excess rainfall from Section 7.3 is used as input in unit hydrograph method. The SCS Unit Hydrograph (SCS UH) is employed in WMS model in Chapter Six. The SCS UH method assumes the dimensionless unit hydrograph (DUH) shapes are all the same for any shaped watershed. The DUH is shown in Figure 3 (a). This assumption does not consider the influence of watershed shape on the DUH, which may be one reason for the unsatisfactory fit. For example, the DUH for a long-narrow watershed is quite different from a square watershed. But in SCS UH method, they are the same.

Although the SCS UH is widely used, it lacks flexibility in describing hydrological response for different shapes of sub-watershed. The profile of dimensionless unit hydrograph is always same, no matter what is shape of the sub-watershed it applies to. This is obviously not suitable and cannot reflect the hydrological response characteristics of the sub-watershed. To solve this problem, a new DUH of linear exponential unit hydrograph (LEUH) is employed.

In SCS UH, the DUH always reaches peak when the relative peak time $T_r = t/T_p = 1$. The DUH always recedes near to zero when $T_r = t/T_p = 5$. The author found this shape of DUH gave much larger peak discharge, early peak time and quick recession. This behavior suggests the idea of increasing the relative peak time $T_r$ and decreasing the recess constant $K_r$ in DUH. In order to depict the DUH in a systematic and easily-modified manner, the author represents the first part of DUH as a linear rise and the second part of DUH as an exponential recession. Thus, the LE DUH method uses two parameters, $T_r$ -- relative peak time and $K_r$ -- relative recessing constant, to describe the watershed unit hydrograph response. Small $T_r$ indicates quick rising while large $T_r$ indicates slow rising. Small absolute value of $K_r$ indicates quick recession while large absolute value of $K_r$ indicates slow recession. Different value of $T_r$ and $K_r$ are able to change the DUH in a great extent and to describe different watershed shape. Figure 47 illustrates four different combinations of $T_r$ and $K_r$.

One problem arises when applying different values of $T_r$ and $K_r$. The area under the DUH is not a constant. As we can see from Figure 47, the area under the fourth DUH is much larger than the first one; it is also larger than the other two DUHs. This induces non-equal of runoff for the same rainfall, which is obviously incorrect. One solution to this problem is to normalize all DUH areas equal to the area under the SCS DUH, which is about 1.33595 and is proved to be correct in water budget. This is accomplished by shrinking or enlarging all the ordinates to a constant for certain DUH, while keeping the abscissas unchanged. For example, the area under the first DUH in Figure 47 is 1. So the ordinates of the first DUH should be multiplied by 1.33595/1, while the abscissas of it will be kept unchanged. The area under the fourth DUH in Figure 47 is 15. So the ordinates of the fourth DUH should be multiplied by 1.33595/15, while the abscissas of it will be kept unchanged. Figure 48 shows the normalized DUH for different $T_r$ and $K_r$.

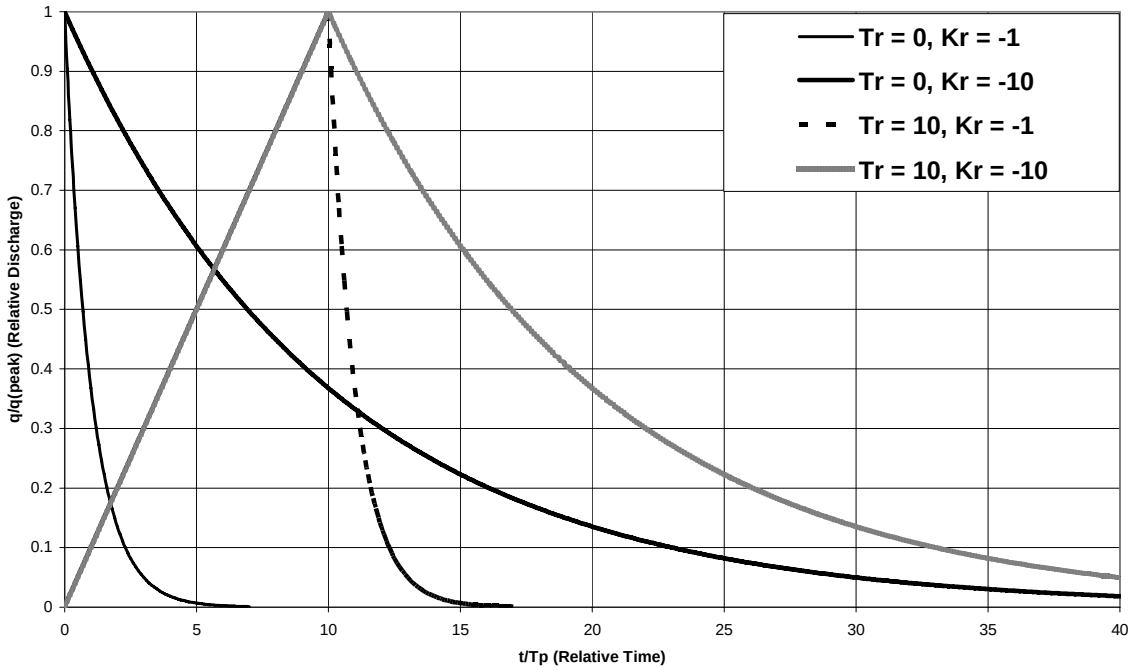



**Figure 47.** Illustration of different combinations of $T_r$ and $K_r$

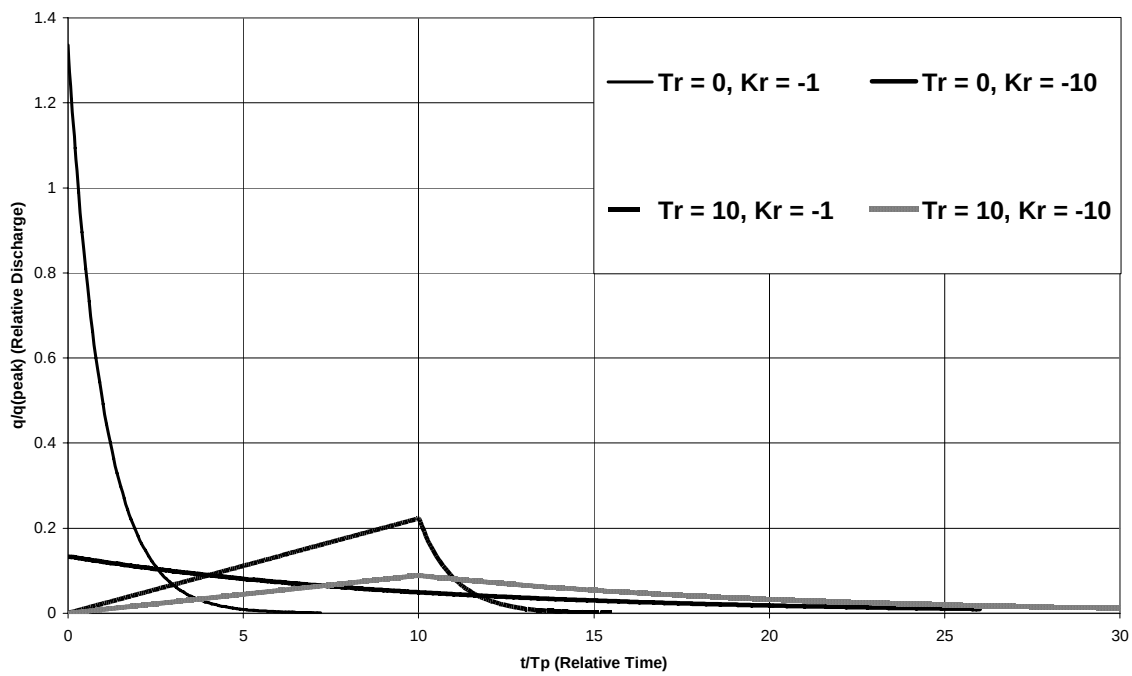



**Figure 48.** Illustration of normalized different combinations of $T_r$ and $K_r$

After normalization, the area under all DUHs is 1.33595. As we can see from Figure 48, the normalized DUHs with different parameters can represent much different unit hydrograph styles.

The DUH in Figure 48 substitutes the DUH in SCS unit hydrograph method. The value of $q_p$ and $T_p$ in LEUH are estimated using the same method as the SCS UH. A simplified model of a triangular unit hydrograph is shown in Figure 3 (b), where the time is in hours and the discharge in $m^3/s \cdot cm$ or *cfs/in* (Soil Conservation Service, 1972).

The parameters used in remaining part of LEUH are also the same as in the SCS UH. The time of recession is approximated as $1.67T_p$. Because the area under the unit hydrograph should be equal to a direct runoff of 1 cm or 1 inch, it can be derived that

$$q_p = \frac{CA}{T_p} \tag{7.1}$$

where $C$ = 2.08 in international unit system (483.4 in English unit system);
$A$ = the drainage area in square kilometers (square miles).

The basin lag time $t_p \cong 0.6T_c$, where $T_c$ = the time of concentration of the watershed. As illustrated in Figure 3 (b), time of rise $T_p$ can be expressed in terms of lag time $t_p$ and the duration of effective rainfall $t_r$

$$T_p = \frac{t_r}{2} + t_p \tag{7.2}$$

The use of LEUH is similar to that of SCS UH, except Figure 48 is employed instead of Figure 3 (a). Due to different values of $T_r$ and $K_r$, the valid length of abscissa in Figure 48 will be much larger than that of Figure 3 (a), which is a constant 5. For example, when $T_r$ = 10 and $K_r$ = -10, the abscissa can be as large as 35.

Time of concentration ($T_c$) is approximately the longest travel time within the basin. In general, the longest travel time corresponds to the longest drainage path. To determine $T_c$, the flow path is broken into segments with the flow in each segment being represented

by some type of flow regime. An equation similar in form to the Manning's equation can be used to calculate the flow velocity.

$$V = K \cdot S^{1/2} \qquad (7.3)$$

where $V$ = flow velocity (*ft/s*);

$K$ = coefficient based on the flow type;

$S$ = slope in percent.

McCuen (1989) and SCS (1972) provided values of $K$ for several flow situations, which are listed in Table 10. With different velocities in different segments, $T_c$ can be obtained.

**Table 10.** Coefficients of velocity (*ft/s*) versus slope (%) relationship for estimating travel velocities

(McCuen 1989; SCS 1972)

| ***K*** | **Land Use / Flow Regime** |
|---|---|
| 0.25 | Forest with heavy ground litter, hay meadow (overland flow) |
| 0.5 | Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow) |
| 0.7 | Short grass pasture (overland flow) |
| 0.9 | Cultivated straight row (overland flow) |
| 1 | Nearly bare and untilled (overland flow); alluvial fans in western mountain regions |
| 1.5 | Grassed waterway |
| 2 | Paved area (sheet flow); small upland gullies |

For different land uses and slopes, the water flow velocity ranges from 0.025 *ft/sec* to 0.894 *ft/sec*.

LEUH is better than SCS unit hydrograph method in that it can describe different sub-watersheds response using different DUHs. For example, some watersheds starts to produce runoff quite quickly, but the recession time is very long. While some watersheds start to produce runoff very slowly, but the recession time is short. This can be modeled and controlled by $T_r$ and $K_r$, but can not be described in SCS UH.

## 7.5 ISOCHRONE UNIT HYDORGRAPH

Besides the SCS and LE unit hydrograph method, a more complicated method, the Isochrone Unit Hydrograph (ISO UH), is also employed in unit hydrograph development. The Isochrone curve is the curve which connects points in the watershed with equal travel time to the outlet. The ISO UH is constructed based on the isochrone curve.

### 7.5.1 Isochrone curve generation

To determine the isochrone curve, detailed flow path distribution, flow path length, channel slope, water velocity in a watershed must be determined. The procedure of isochrone curve generation, which is an example of sub-watershed Right_Highway1, is illustrated in the following.

1. Each patch in the sub-watershed is identified using the project land use and topographical map. The patches are sliced small enough to ensure each patch can be represented by single slope, single flow direction, and single flow velocity coefficient $K$ (in Equation 7.3). The patch identification is illustrated by polygons in Figure 49.
2. The flow path placement is determined based on topographical analysis. The flow paths are displayed in Figure 49 using arrows.
3. Slope and flow velocity coefficient $K$ are calculated based on field survey. They are shown in Figure 49.
4. Flow velocity in each patch is calculated using Equation 7.3. This is illustrated in Figure 49.
5. A travel time interval of 10 minutes is used to divide the isochrone curves.
6. Travel time of each sub-patch (with 10 minutes travel time interval) is calculated.
7. Contributing areas of each sub-patch to each time interval are summed together to obtain the area for each time interval.
8. A table showing the travel time and contributing area is displayed in Table 11.

9. A diagram showing the travel time and contributing area is displayed in Figure 50, which is also called the isochrone diagram.

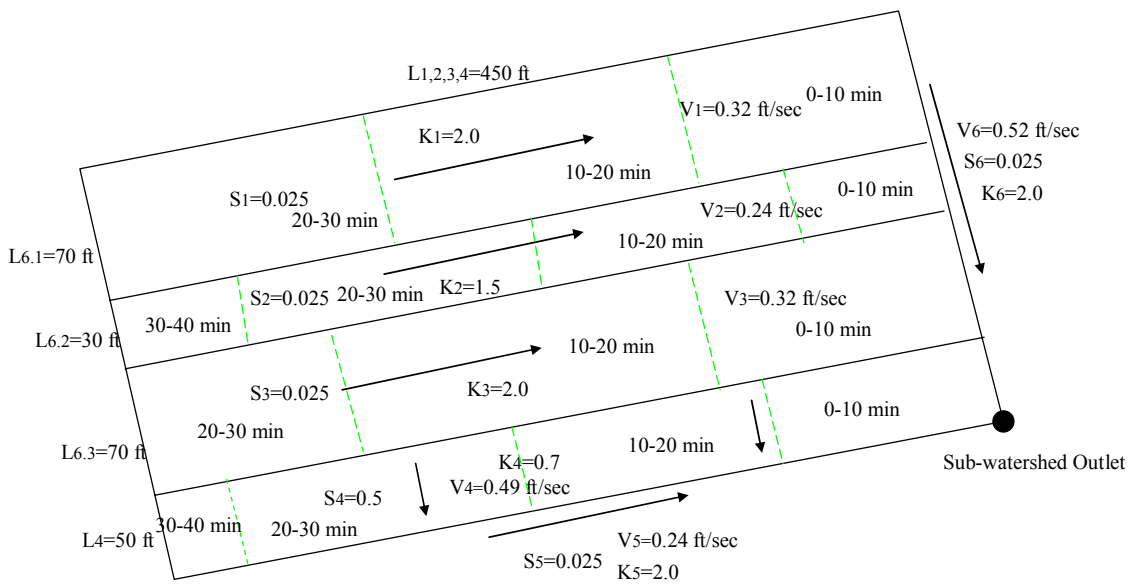



**Figure 49.** Illustration of isochrone curve generation

**Table 11.** The travel time and contributing area for Right_Highway1

| Time (min) | Area (sq ft) |
|---|---|
| 10 | 29801 |
| 20 | 106608 |
| 30 | 34626 |
| 40 | 3127 |

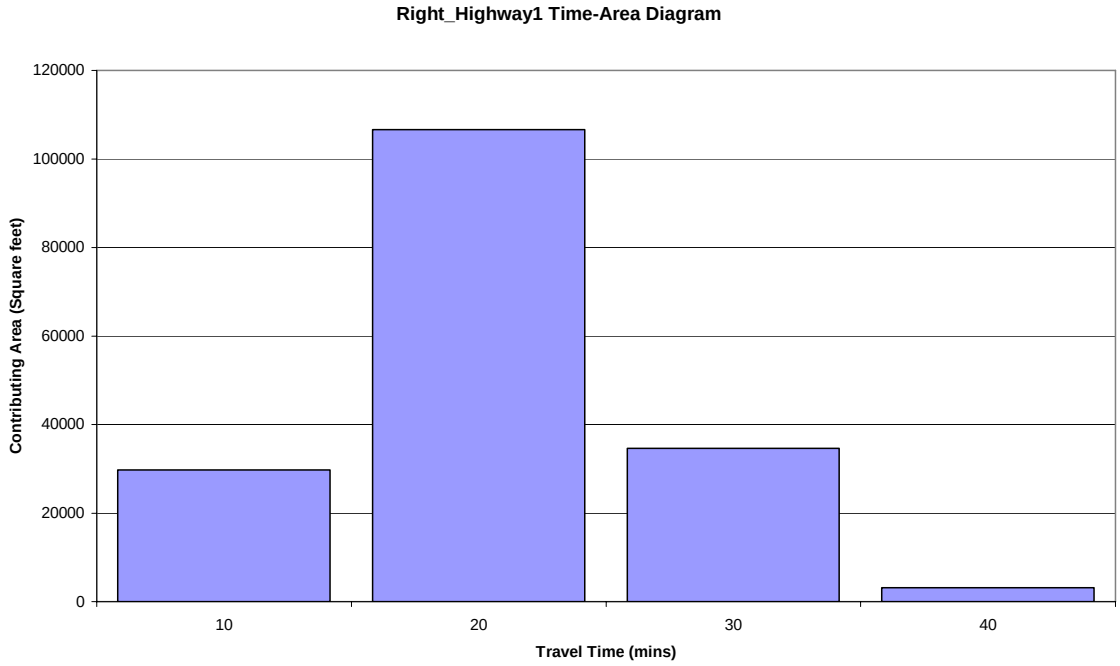



**Figure 50.** The travel time and contributing area diagram for Right_Highway1

The method to generate the isochrone diagrams for the other sub-watersheds is the same as explained above. Figure 51 to Figure 56 show the travel time and contributing area in diagram format.

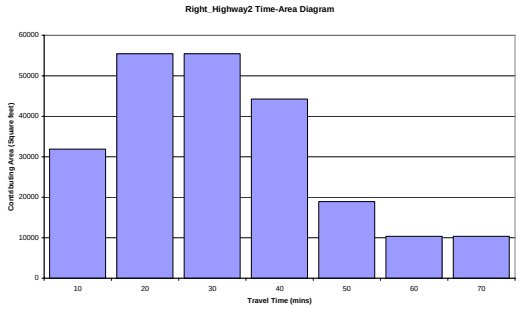



**Figure 51.** The travel time and contributing area diagram for Right_Highway2

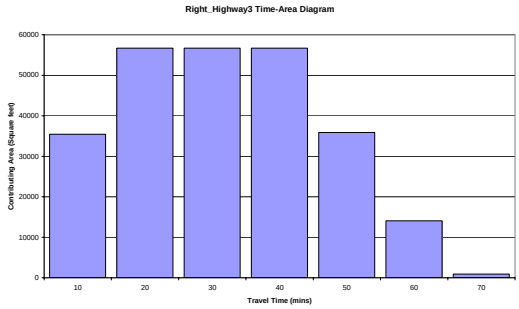



**Figure 52.** The travel time and contributing area diagram for Right_Highway3

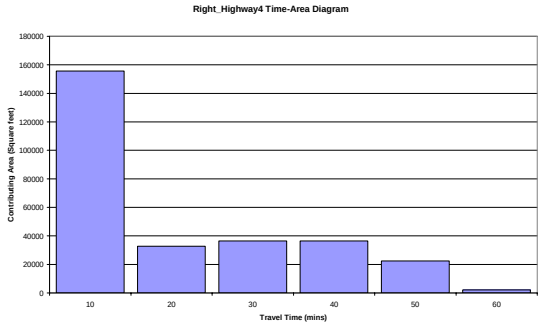



**Figure 53.** The travel time and contributing area diagram for Right_Highway4

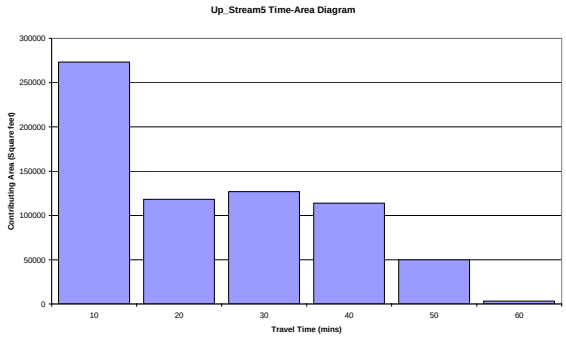



**Figure 54.** The travel time and contributing area diagram for Up_Stream5

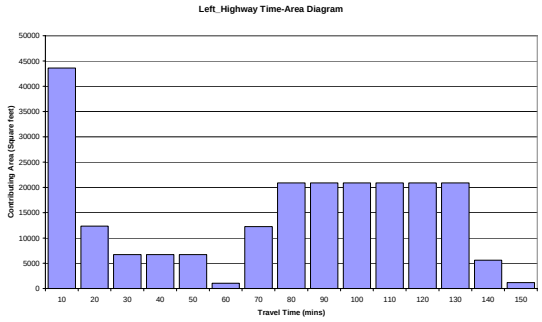



**Figure 55.** The travel time and contributing area diagram for Left_Highway6

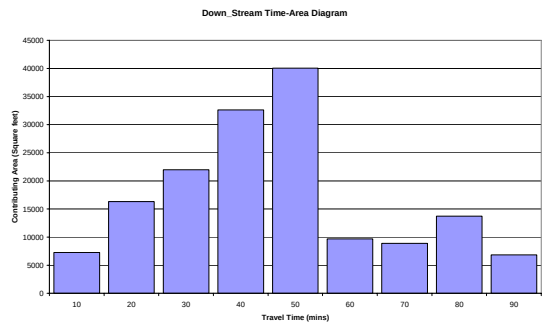



**Figure 56.** The travel time and contributing area diagram for Down_Stream7

### 7.5.2 ISO UH derivation

After the isochrone diagram is obtained, the outflow of the sub-watershed can be obtained by routing the isochrone diagram through a linear reservoir (Clark, 1945; Singh, 1988), whose storage constant is the time of concentration of the sub-watershed $T_c$. $T_c$ can be estimated by multiplying the largest travel time of the sub-watershed by 0.75.

Suppose the average inflow of the sub-watershed is $I_t$ at time interval $t$, the outflow at the beginning ant the end of time interval $t$ is $O_{t-1}$ and $O_t$, then the average outflow of at time interval t is $\overline{O}_t = \frac{O_{t-1} + O_t}{2}$. The outflow of at the end of time interval $t$ can be calculated using Equation 7.4.

$$O_t = C_A \cdot I_t + C_B \cdot O_{t-1} \tag{7.4}$$

where $C_A$ and $C_B$ can be calculated by Equation 7.5 and Equation 7.6.

$$C_A = \frac{\Delta t}{T_c + 0.5 \cdot \Delta t} \tag{7.5}$$

$$C_B = 1 - C_A \tag{7.6}$$

where $T_c$ = time of concentration;

$\Delta t$ = time interval of the unit hydrograph.

Equation (7.4), Equation (7.5), and Equation (7.6) can also be derived from Equation (2.18), Equation (2.18), and Equation (2.18) by assigning $X$ = 0. $I_t$ in cubic foot per second is calculated by $I_t$ = 1 (*inch*) × contributing area of time t ($ft^2$) / 12 (*inch/ft*) / 600

(*sec*). $O_t$ ($t$ = 0) can be assumed to be 0. With this information, we can obtain the average outflow at any time, which is the ordinate of the ISO UH.

## 7.6 RUNOFF GENERATION AT EACH SUB-WATERSHED

Once the unit hydrograph has been determined, it may be applied to find the direct runoff and stream flow hydrograph. For calculation convenience, the time interval used in defining the excess rainfall hyetograph ordinates should be the same as that for which the unit hydrograph was specified. The discrete convolution equation is described in Equation 2.14.

## 7.7 CHANNEL AND PIPE ROUTING

As shown in Figure 45 and Figure 46, runoff obtained at each sub-watershed needs to be routed downstream. Muskingum routing, which is described in Chapter Two, is used in channel routing.

To improve the modeling results, kinematic wave routing is applied in pipe routing. This is the case in Watershed One, runoff from Up_Stream sub-watershed to Down_Stream and in Watershed Two, runoff from Up_Stream sub-watershed to Down_Stream. The diameter of the pipe used in Watershed One and Watershed Two is 18 inches and 30 inches, respectively. Although Watershed One is mentioned here, it will not be modeled due to water diversion problem, which is described in Section 6.4.

The kinematic wave model is one of the distributed models. It neglects the local acceleration, convective acceleration and pressure terms in the momentum equation. The kinematic wave model is defined by Equation 2.15(a) and Equation 2.17.

The assumptions and approximations made in kinematic wave theory is that $Q$ is assumed to be a function of $x$ alone. This means that $S_0 = S_f$ and the other three slope terms (secondary terms) in Equation (2.15b) are negligible. Thus, the bed slope is assumed to be large enough. The water wave is assumed long and flat enough so that the change in depth and velocity with respect to distance ($\frac{\partial y}{\partial x}$ and $\frac{\partial v}{\partial x}$) and the change in velocity with respect to time ($\frac{\partial v}{\partial t}$) are negligible when subtracted from $S_0$ in Equation (2.12b).

Henderson (1966, p 364) showed that the value of the secondary terms are small if the channel slope is about 10 feet per mile (0.189%) or more. Gunaratnam and Perkins (1970, p 45) also found the similar behavior. In the project, the slopes of the two steep underground pipes are 16.67% and 29.81%, which is much larger than 0.189%. This validates the kinematic wave application.

In order to use kinematic wave method, kinematic wave celerity should be obtained. Chow (1988) presented that kinematic wave celerity can be expressed as

$$c_k = \frac{dQ}{dA} \tag{7.7}$$

We have

$$Q = \frac{1.49}{n} S^{1/2} \cdot A \cdot R^{2/3} \tag{7.8}$$

where $R$ = the hydraulic radius;
$c_k$ = kinematic wave celerity;
$Q$ = pipe discharge;
$n$ = Manning coefficient;
$A$ = flow cross section area;
$S$ = pipe slope;

For circular pipe,

$$R = \frac{A}{P} = \frac{A}{\theta \cdot r} \qquad (7.9)$$

where $\theta$ = the wetted angle indicated in Figure 57, $r$ is the pipe radius.

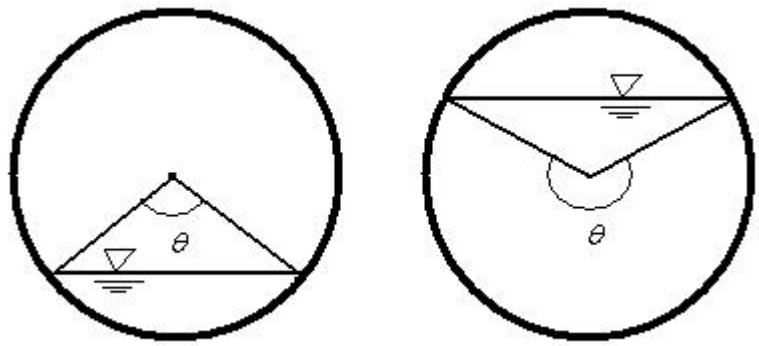



**Figure 57.** Pipe cross section illustration

Put Equation (7.9) into Equation (7.8), we get

$$Q = \frac{1.49}{n} \cdot S^{1/2} \cdot A \cdot \left(\frac{A}{\theta \cdot r}\right)^{2/3} = \frac{1.49 \cdot S^{1/2}}{n \cdot r^{2/3}} \cdot A^{5/3} \cdot \theta^{-2/3} \qquad (7.10)$$

where

$$A = \frac{(\theta - \sin\theta) \cdot r^2}{2} \qquad (7.11)$$

Put (7.11) into (7.10), we get

$$Q = \frac{1.49 \cdot S^{1/2}}{n \cdot r^{2/3}} \cdot \left[\frac{(\theta - \sin\theta) \cdot r^2}{2}\right]^{5/3} \cdot \theta^{-2/3} \qquad (7.12)$$

All variables in Equation (7.12) should be known except $\theta$ . Given any discharge at the pipe inlet, $\theta$ can be determined by trial and error. Theoretically, the flow can submerge the pipe. However, in this project reality, such a large storm almost never happens. So it is not necessary to consider the submerged situation.

The kinematic wave can be obtained by

$$c_k = \frac{dQ}{dA}$$

$$= \frac{1.49 \cdot S^{1/2}}{n \cdot r^{2/3}} \cdot \left[ \frac{5}{3} \cdot A^{2/3} \cdot \theta^{-2/3} - \frac{2}{3} \cdot A^{5/3} \cdot \theta^{-5/3} \cdot \frac{1}{r^2 \cdot (1-\cos\theta)/2} \right] \qquad (7.13)$$

After $\theta$ is determined, $c_k$ can be obtained from (7.13).

The routing procedure calculates the outlet discharge using inlet discharge. Suppose the inlet discharge is $Q_{in}(t)$, which can be represented by discrete input $Q_{in}(t_i)$. Then the outflow discharge can be calculated as $Q_{out}(t_i+T_i) = Q_{in}(t_i)$, where $T_i = L/c_{ki}$ is the kinematic wave travel time in the pipe or channel (Chow, 1988). For each $Q_{in}(t_i)$, $c_{ki}$ and $T_i$ can be calculated through above method.

## 7.8 RESERVOIR ROUTING

If the flow passes through a pond or reservoir, the reservoir routing is needed to run the model. The level pool reservoir routing and linear reservoir routing is employed in HWM.

Level pool routing is a procedure for calculating the outflow hydrograph from a reservoir with an assumed horizontal water surface, given its inflow hydrograph and storage outflow characteristics (Chow 1988). The time horizon is broken into intervals of $\Delta t$, indexed by $i$, that is, $t = 0, \Delta t, 2\Delta t, \ldots\ldots, i\Delta t, (i+1)\Delta t, \ldots\ldots$ The continuity equation for the reservoir is

$$\frac{dS}{dt} = I(t) - Q(t) \qquad (7.14)$$

where $S$ = the reservoir storage, $I(t)$ = reservoir input discharge at time $t$ and $Q(t)$ = reservoir output discharge at time $t$.

Integrating Equation (7.14) at $i$-th time interval, we got

$$\int_{S_i}^{S_{i+1}} dS = \int_{i\Delta t}^{(i+1)\Delta t} I(t)dt - \int_{i\Delta t}^{(i+1)\Delta t} Q(t)dt \qquad (7.15)$$

The inflow values at the beginning and end of the $i$-th time interval are $I_i$ and $I_{i+1}$, respectively. Similarly, the corresponding values of the outflow are $Q_i$ and $Q_{i+1}$. If the variation of inflow and outflow over the interval is approximate linear, the change in storage over the interval, $S_{i+1}$ - $S_i$ can be found by re-writing Equation (7.15) as

$$S_{i+1} - S_i = \frac{I_i + I_{i+1}}{2} \cdot \Delta t - \frac{Q_i + Q_{i+1}}{2} \cdot \Delta t \qquad (7.16)$$

The values of $I_i$ and $I_{i+1}$ are input discharges and are known. The value of $Q_i$ and $S_i$ are known at the $i$-th time interval from calculation during the previous time interval. Multiplying Equation (7.16) through $2/\Delta t$ and re-arranging the results, we can isolate the two unknowns, $Q_{i+1}$ and $S_{i+1}$.

$$(\frac{2S_{i+1}}{\Delta t} + Q_{i+1}) = (I_j + I_{j+1}) + (\frac{2S_j}{\Delta t} - Q_j) \qquad (7.17)$$

In order to calculate the outflow, $Q_{j+1}$, a storage-outflow function relating $\frac{2S}{\Delta t} + Q$ and $Q$ is needed. The method for developing this function using elevation-storage and elevation-outflow relationship is shown in Figure 58. The elevation-storage relationship and elevation-discharge relationship can be derived from reservoir design data. The value of $\Delta t$ is taken as the time interval of the inflow hydrograph. For a given value of water surface elevation, the value of storage $S$ and discharge $Q$ are determined [Figure 58 (a) and Figure 58 (b)], then the value of $2S/\Delta t+Q$ is calculated and plotted on the horizontal axis of a graph with the value of the outflow $Q$ on the vertical axis [Figure 58 (c)].

In routing the flow through time interval $i$, all terms on the right side of Equation (7.17) are known, and so the value of $2S_{i+1}/\Delta t+Q_{i+1}$ can be computed. The corresponding value of $Q_{j+1}$ can be determined from the storage-flow function $2S/\Delta t+Q$ versus $Q$. To set up the data required for the next time interval, the value of $2S_{i+1}/\Delta t$ - $Q_{i+1}$ is calculated by

$$(\frac{2S_{i+1}}{\Delta t} - Q_{i+1}) = (\frac{2S_{i+1}}{\Delta t} + Q_{i+1}) - 2S_{i+1} \qquad (7.18)$$

The computation is then repeated for subsequent routing periods.

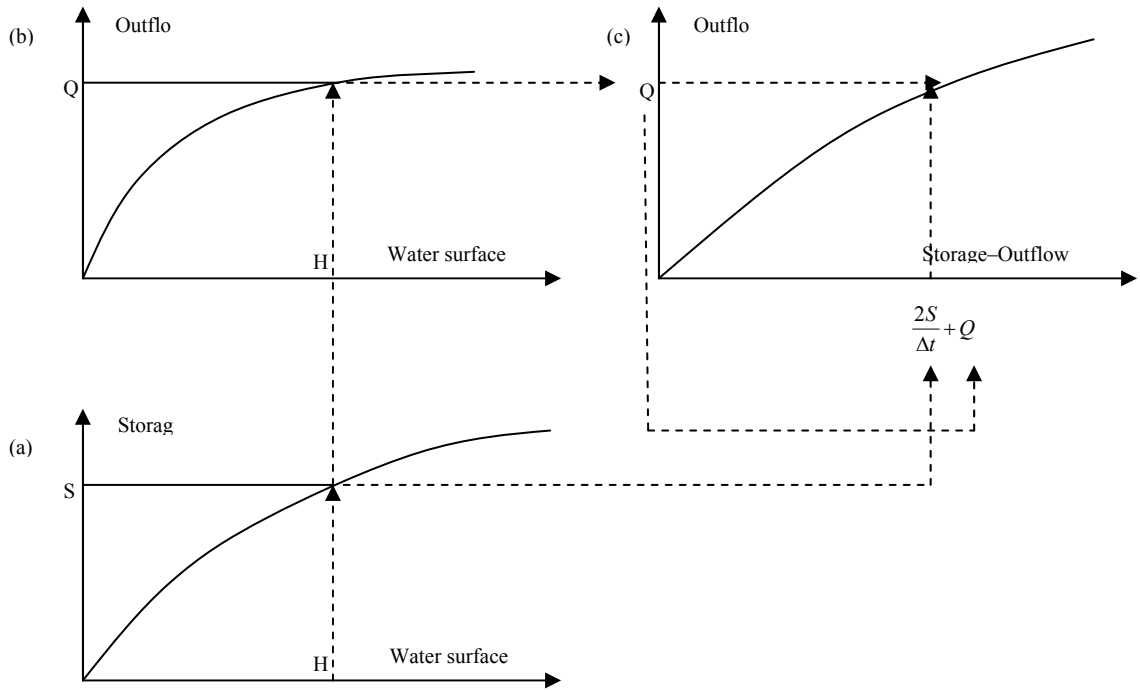



**Figure 58.** Development of the storage-outflow function for level pool routing on the basis of storage-elevation and elevation-outflow curves
(Chow, 1988)

Linear reservoir routing is the simplified Muskingum routing, which is discussed in Chapter Two. If we take Muskingum parameter $X = 0$, we obtain linear reservoir routing method.

Although the algorithm of reservoir routing is different from channel routing, it is regarded as similar procedure as channel routing. That is, if we encounter a reservoir routing, we should consider it any another "channel routing" to process the model.

## 7.9 HYDROGRAPH ADDITION

The hydrograph after channel routing is still separate from other sub-watershed's hydrograph. In order to obtain the total hydrograph at the final outlet, we should add the routed hydrograph to the proper downstream routed or un-routed hydrograph. This is done by simply adding discharges at the same time for different hydrographs.

Channel routing and hydrograph addition are operated interactively according to the schematic of the whole watershed. The hydrograph from upstream sub-watershed is routed to the downstream sub-watershed. The routed hydrograph is added onto the un-routed hydrograph at the downstream sub-watershed. If routed hydrographs from more than one sub-watershed meet at the same sub-watershed, all the routed hydrographs should be added onto the un-routed hydrograph at the downstream sub-watershed. The added hydrographs are then routed further downstream as in the previous routing procedure. As Section 7.7 states, reservoir routing can be considered as another "channel routing". This routing and adding procedure is repeated until all the hydrographs are added to the most downstream outlet.

According to the above description, different watershed schematics induce different routing and adding order. Because the studied watersheds are fixed in schematic, the routing and adding procedures are fixed in the model. If a different watershed is studied, the structure of the model will change. The fixed structure model is easy to use, but less flexible. All parameters are input by a text file. To execute the model, the user only needs to run the master program of the model. Another version of the same model is flexible but difficult to use. The user needs to run the routing and adding program individually and interactively, but the order of routing and adding can be changed anytime. The user needs more hydrological knowledge and familiarity with the watershed to run the flexible model.

## 7.10 ROUTING AND ADDING ORDER

Based on Figure 46, the modeling order for Watershed Two is executed as follow.

1. Hydrograph from sub-watershed 1 (H1) is routed to sub-watershed 2 outlet. The routed hydrograph is RH1;
2. RH1 is added to hydrograph at sub-watershed 2 (H2). The added hydrograph is AH2;
3. AH2 is routed to sub-watershed 3 outlet to become RAH2;
4. RAH2 is added to hydrograph at sub-watershed 3 (H3). The added hydrograph is AH3;
5. AH3 is routed to sub-watershed 4 outlet to become RAH3;
6. RAH3 is added to hydrograph at sub-watershed 4 (H4). The added hydrograph is AH4;
7. AH4 is routed to Pond SB 10 to become RAH4;
8. RAH4 is routed to sub-watershed 7 outlet through Pond SB 10 to become RRAH4;
9. Hydrograph from sub-watershed 5 (H5) is routed to sub-watershed 7 outlet. The routed hydrograph is RH5;
10. Hydrograph from sub-watershed 6 (H6) is routed to Pond SB 11 to become RH6;
11. RH6 is routed to sub-watershed 7 outlet through Pond SB 11 to become RRAH6;
12. Routed hydrograph from upstream watershed or ponds RH5, RRAH4 and RRAH6 is added to hydrograph at sub-watershed 7 (H7). The added hydrograph is AH7 and is the final hydrograph for the whole watershed.

Similarly, based on Figure 45, the modeling order for Watershed One can also be determined.

# 8.0 APPLICATION OF HWM IN I-99 PROJECT

Basic information on the project is shown in Chapter Four. The model schematic of Watershed Two is shown in Section 7.2 and Section 7.9. The rainfall bar diagrams are displayed in Section 6.3. The ponds characteristics are shown in Section 6.1 and Section 6.2. This chapter only shows the modeling results.

The methods to determine parameters are similar to WMS model. Some parameters are determined only by field survey. These parameters include watershed length, watershed area, precipitation depth and temporal distribution, reservoir characteristics and reservoir initial elevation. The other parameters are determined by both field survey and trial and error method. These parameters include curve number (CN), watershed slope, watershed length, water velocity in channel, Muskingum $K$, Muskingum $X$. As discussed in Section 7.4, HWM has extra parameters other than WMS. These include time of concentration ($T_c$) for each sub-watershed, relative peak time ($T_r$) and relative recess constant ($K_r$) for the Linear Exponential Unit Hydrograph (LEUH).

## 8.1 PARAMETERS FOR WATERSHED TWO, ALL EVENTS

HWM has the advantage of putting all parameters in one text file, which can be adjusted easily. After several trial modifications of model parameters, several sets of reasonable parameters are determined for Watershed Two. Table 12 shows all event parameters for Watershed Two.

**Table 12.** Important parameters used in specific storm events

| | Oct 07 2005 | Oct 25 2005 | Nov 27 2005 | Jan 17 2006 | Mar 11 2006 | May 11 2006 | June 26 2006 | Sept 01 2006 |
|---|---|---|---|---|---|---|---|---|
| Sub-watershed Right_Highway4 | | | | | | | | |
| Weighted CN | 80 | 98 | 94 | 89 | 90 | 85 | 93 | 80 |
| $T_c$ (min) | 30 | 40 | 40 | 60 | 20 | 40 | 60 | 60 |
| Sub-watershed Right_Highway3 | | | | | | | | |
| Weighted CN | 80.7 | 98 | 94 | 89 | 90 | 85.7 | 93 | 80 |
| $T_c$ (min) | 50 | 70 | 70 | 90 | 40 | 70 | 70 | 70 |
| Sub-watershed Right_Highway2 | | | | | | | | |
| Weighted CN | 80.9 | 98 | 94 | 89 | 90 | 85.9 | 93 | 80 |
| $T_c$ (min) | 50 | 70 | 70 | 90 | 40 | 70 | 70 | 70 |
| Sub-watershed Right_Highway1 | | | | | | | | |
| Weighted CN | 80.9 | 98 | 94 | 89 | 90 | 85.9 | 93 | 80 |
| $T_c$ (min) | 40 | 60 | 60 | 80 | 30 | 60 | 40 | 40 |
| Sub-watershed Up_Stream | | | | | | | | |
| Weighted CN | 65 | 98 | 68 | 80 | 66 | 65 | 96 | 85 |
| $T_c$ (min) | 40 | 60 | 60 | 80 | 30 | 60 | 60 | 60 |
| Sub-watershed Left_Highway | | | | | | | | |
| Weighted CN | 81.7 | 98 | 94 | 89 | 90 | 86.7 | 93 | 80 |
| $T_c$ (min) | 100 | 150 | 150 | 200 | 100 | 150 | 150 | 150 |
| Sub-watershed Down_Stream | | | | | | | | |
| Weighted CN | 70 | 94 | 75 | 87 | 72 | 69 | 70 | 60 |
| $T_c$ (min) | 60 | 90 | 90 | 120 | 60 | 90 | 90 | 90 |
| Routing Right_Highway4--Right_Highway3 | | | | | | | | |
| $K$ (min) | 20 | 20 | 20 | 40 | 20 | 20 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| Routing Right_Highway3--Right_Highway2 | | | | | | | | |
| $K$ (min) | 20 | 20 | 20 | 40 | 20 | 20 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| Routing Right_Highway2--Right_Highway1 | | | | | | | | |
| $K$ (min) | 20 | 20 | 20 | 40 | 20 | 20 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| Routing Right_Highway1--SB10 | | | | | | | | |
| $K$ (min) | 30 | 30 | 30 | 60 | 30 | 30 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| Routing Up_Stream--Down_Stream | | | | | | | | |
| $K$ (min) | 30 | 30 | 30 | 40 | 30 | 30 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| Routing Left_Highway--SB11 | | | | | | | | |
| $K$ (min) | 20 | 20 | 20 | 40 | 20 | 20 | 20 | 20 |
| $X$ | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| | | | | | | | | |
| $T_r$ (dimensionless) | 12 | 14 | 13 | 18 | 10 | 17 | 10 | 12 |
| $K_r$ (dimensionless) | 12 | 14 | 14 | 18 | 10 | 17 | 10 | 12 |

## 8.2 HWM RESULTS FOR WATERSHED TWO

The Ecotones installed in the outlet flumes record water depth. A rating curve is used to convert the water depth to the discharge and shown in Figure 42. Detailed information on all rainfall events can be found in Section 6.3 and Figure 41. Figure 59 to Figure 66 shows the comparison of measured and HWM modeled hydrograph.

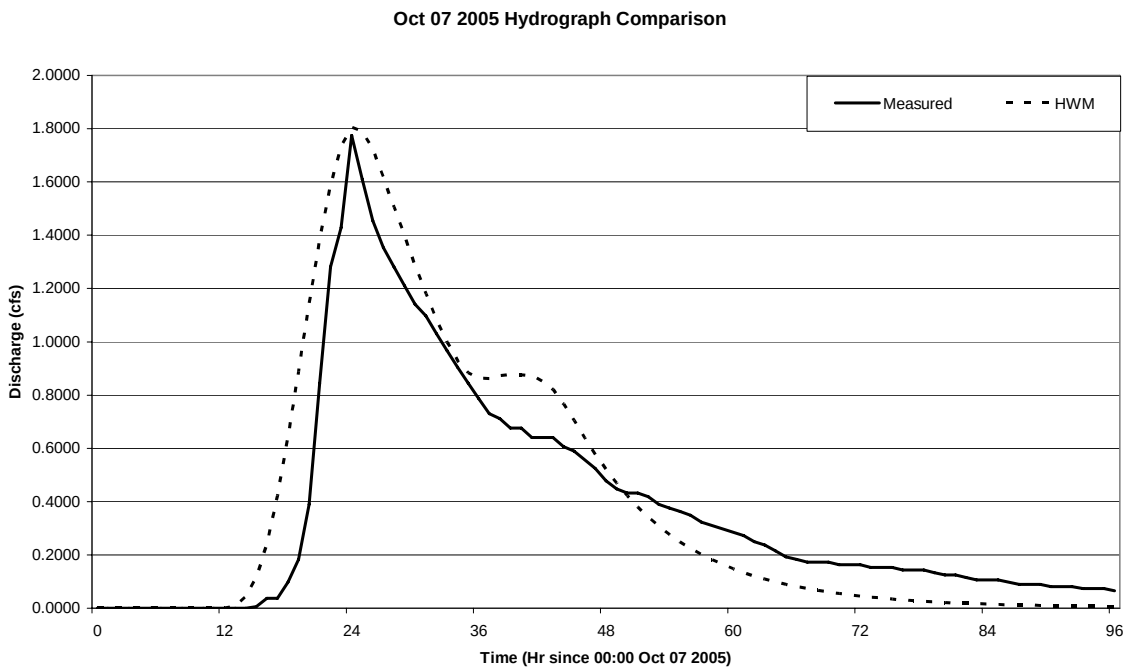



**Figure 59.** Measured and HWM modeled hydrograph for Oct 07 2005 Event

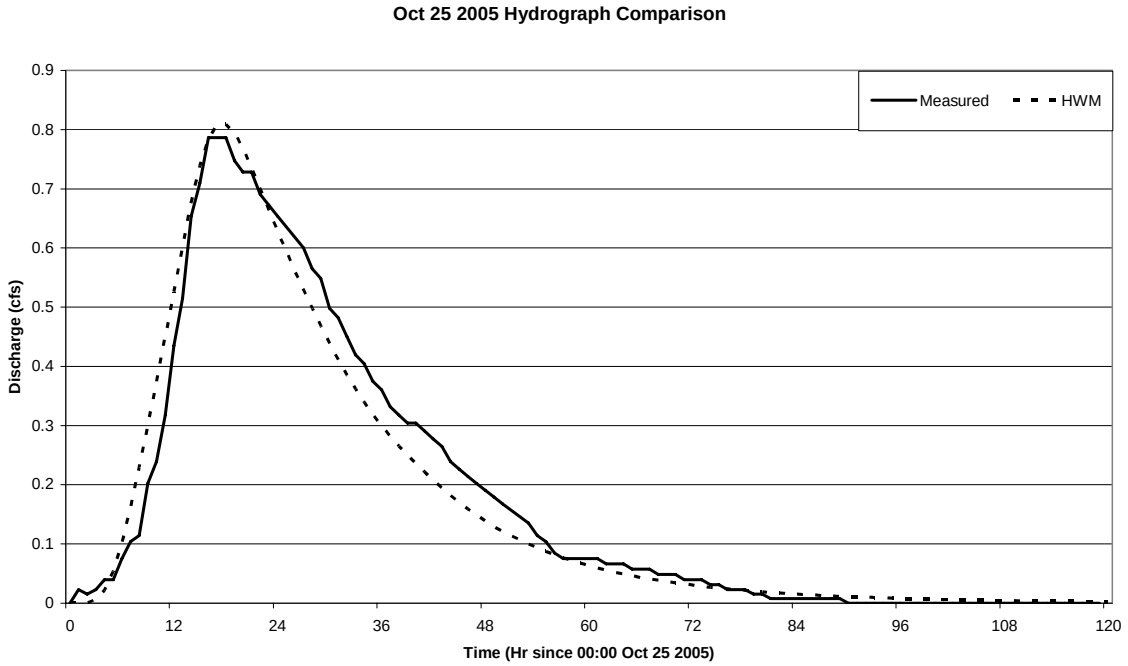



**Figure 60.** Measured and HWM modeled hydrograph for Oct 25 2005 Event

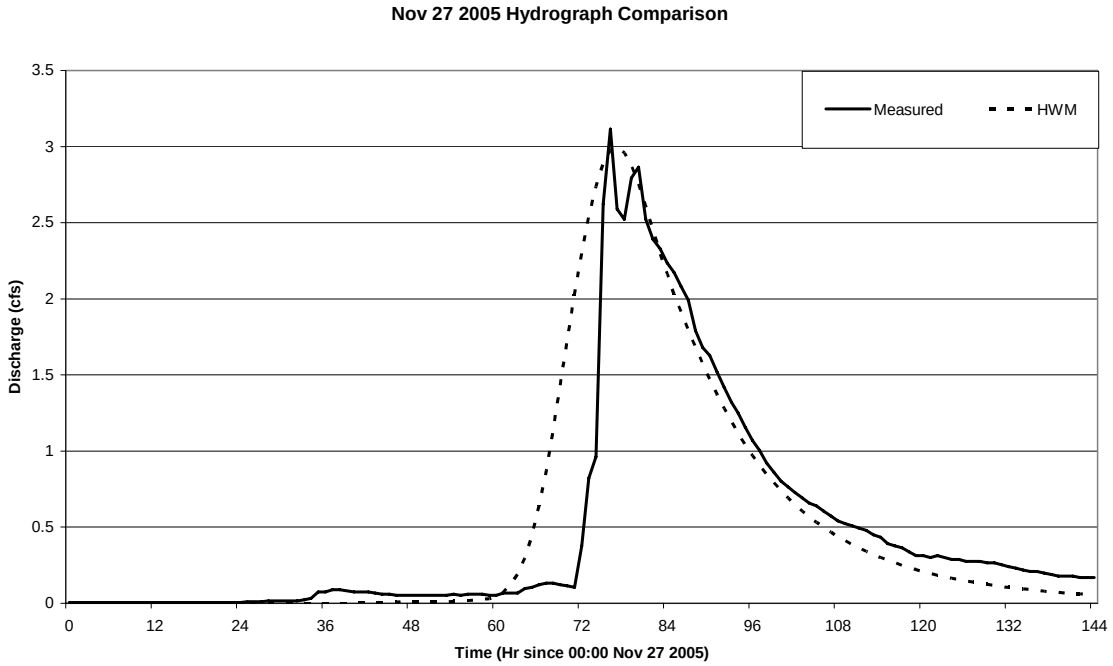



**Figure 61.** Measured and HWM modeled hydrograph for Nov 27 2005 Event

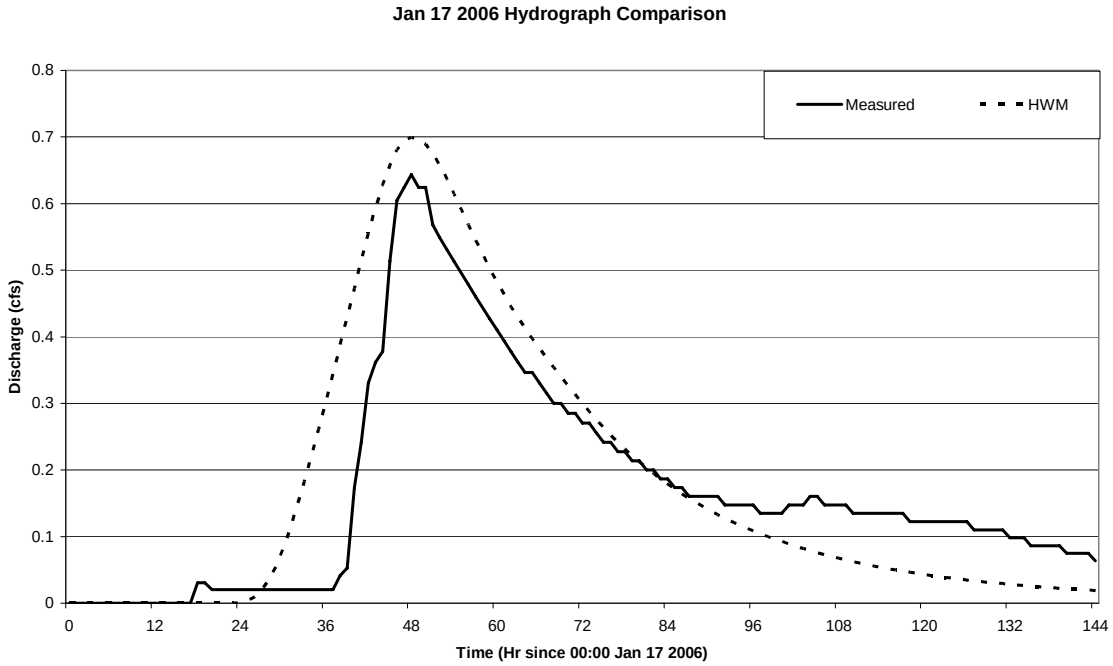



**Figure 62.** Measured and HWM modeled hydrograph for Jan 17 2006 Event

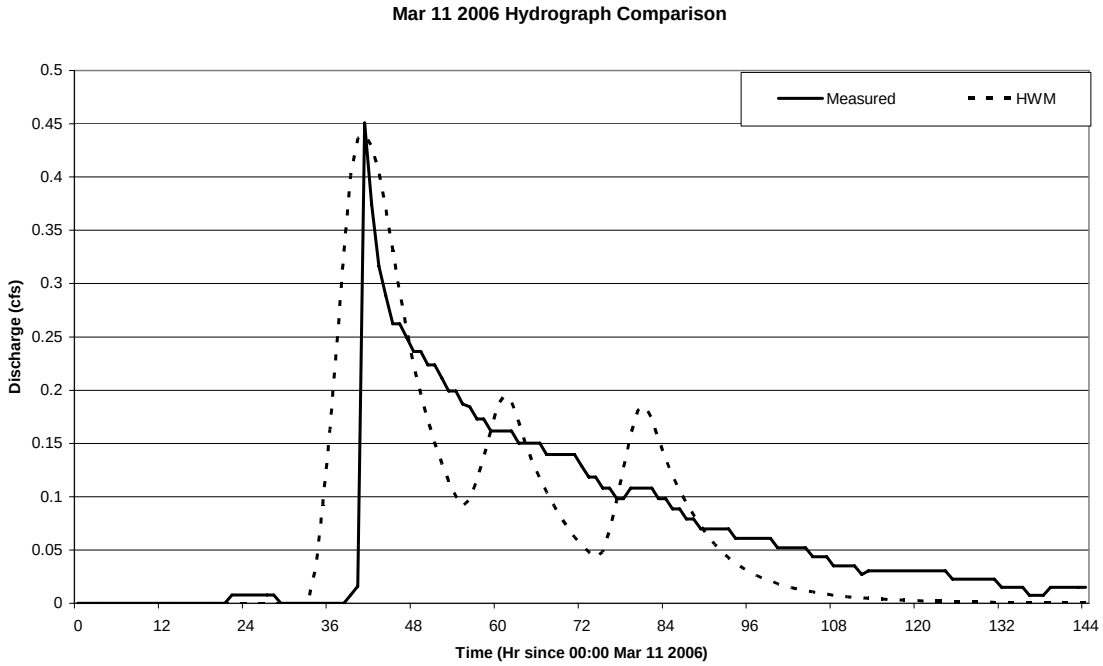



**Figure 63.** Measured and HWM modeled hydrograph for Mar 11 2006 Event

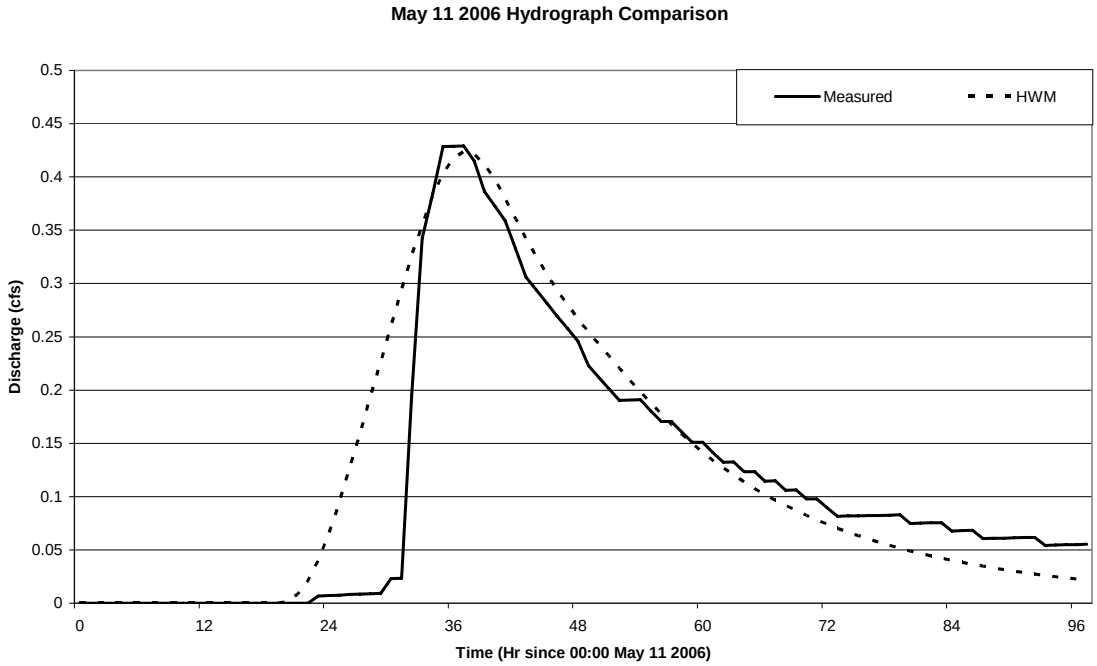



**Figure 64.** Measured and HWM modeled hydrograph for May 11 2006 Event

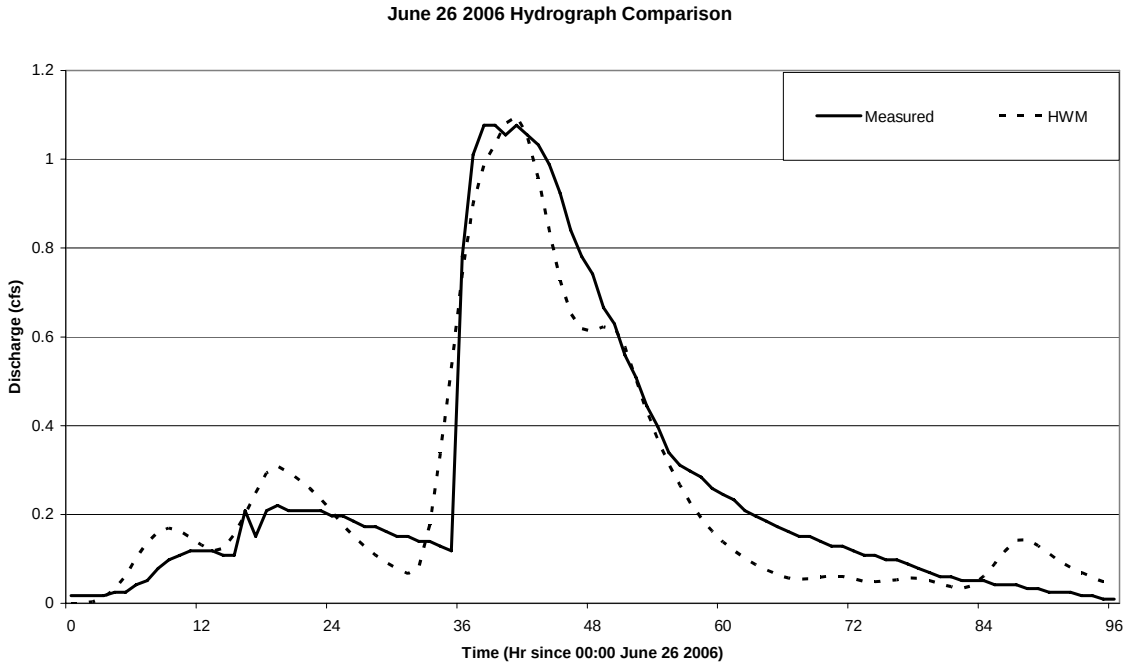



**Figure 65.** Measured and HWM modeled hydrograph for June 26 2006 Event

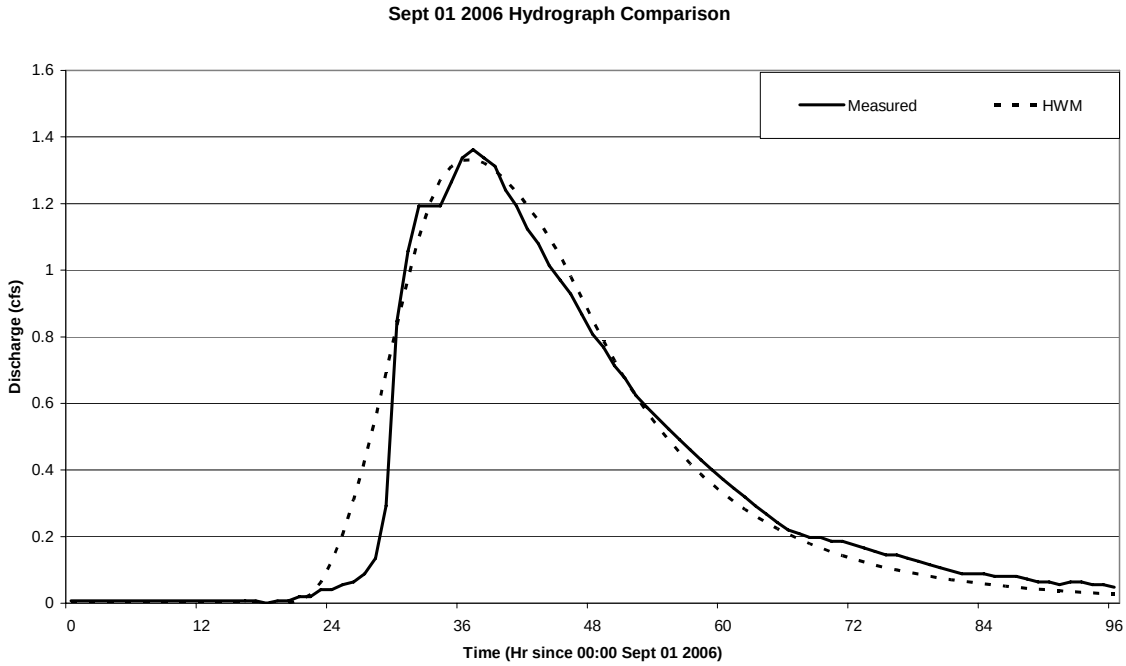



**Figure 66.** Measured and HWM modeled hydrograph for Sept 01 2006 Event

## 8.3 ANALYSIS OF HWM RESULTS

As defined in Section 6.7, the most important criteria for comparing modeled hydrograph to measured hydrograph are the total volume of the runoff, peak time and peak discharge values. Table 13 shows these three criteria were satisfied. Based on the project need, a deviation within 15% on runoff total volume and peak discharge is regarded as satisfaction. A deviation with 120 minutes (equal two modeling time intervals) on peak time is regarded as satisfactory.

**Table 13.** The comparison of the three criteria for Watershed Two

| | | Total Runoff Volume ($ft^3$) | Peak Discharge (*cfs*) | Peak Time (*min*) |
|---|---|---|---|---|
| Oct 07 2005 | Measured | 142341 | 1.77 | 1440 |
| | Modeled | 152055 | 1.80 | 1440 |
| | Deviation | 6.82 % | 1.69 % | 0 min |
| Oct 25 2005 | Measured | 77531 | 0.79 | 1020 |
| | Modeled | 74596 | 0.81 | 1080 |
| | Deviation | -3.79 % | 2.53 % | 60 min |
| Nov 27 2005 | Measured | 262528 | 3.12 | 4560 |
| | Modeled | 273125 | 3.00 | 4620 |
| | Deviation | 4.04 % | -3.85 % | 60 min |
| Jan 17 2005 | Measured | 89015 | 0.64 | 2880 |
| | Modeled | 90542 | 0.70 | 2880 |
| | Deviation | 1.72 % | 9.37 % | 0 min |
| Mar 11 2005 | Measured | 34977 | 0.45 | 2460 |
| | Modeled | 35800 | 0.44 | 2460 |
| | Deviation | 2.35 % | -2.22 % | 0 min |
| May 11 2005 | Measured | 44466 | 0.43 | 2220 |
| | Modeled | 45542 | 0.42 | 2220 |
| | Deviation | 2.42 % | -2.33 % | 0 min |
| June 26 2005 | Measured | 90631 | 1.08 | 2460 |
| | Modeled | 86472 | 1.10 | 2460 |
| | Deviation | -4.59 % | 1.85 % | 0 min |
| Sept 01 2005 | Measured | 121958 | 1.36 | 2220 |
| | Modeled | 124077 | 1.33 | 2220 |
| | Deviation | 1.74 % | -2.21 % | 0 min |
| Average Absolute (Percent) Deviation | | 3.43 % | 3.26 % | 15 min |

As can be seen from Table 13, all deviations of the modeled runoff total volume, peak discharge value, and peak discharge time are acceptable. These results are much better than model results in WMS.

The average absolute percent deviation of the runoff volume deviations is 3.43 %, which is less than the deviation upper limit, 15 %. “Absolute” means all negative deviations values are converted to positive before taking the average. The absolute percent deviation of the peak discharge is 3.26 %, which is also within satisfactory limit.

The average absolute deviation (not percent) for peak discharge is 15 minutes, which is also satisfactory.

To compare the results, the measured and modeled runoff volumes are shown in Figure 67. A trend line is added to see the modeled runoff volume deviation. As can be seen from Figure 67, the perfect fit line should be $Y = X$, with $R^2 = 1$, while the actual fit line is $Y = 1.0312\ X$, with $R^2 = 0.9971$. Similarly, scatter plot for measured and modeled peak discharge is shown in Figure 68. The perfect fit line should be $Y = X$, with $R^2 = 1$, while the actual fit line is $Y = 0.9829\ X$, with $R^2 = 0.997$. For peak time modeling, Figure 69 shows the scatter diagram for measured and modeled peak time. The perfect fit line should be $Y = X$, with $R^2 = 1$, while the actual fit line is $Y = 1.0062\ X$, with $R^2 = 0.9993$.

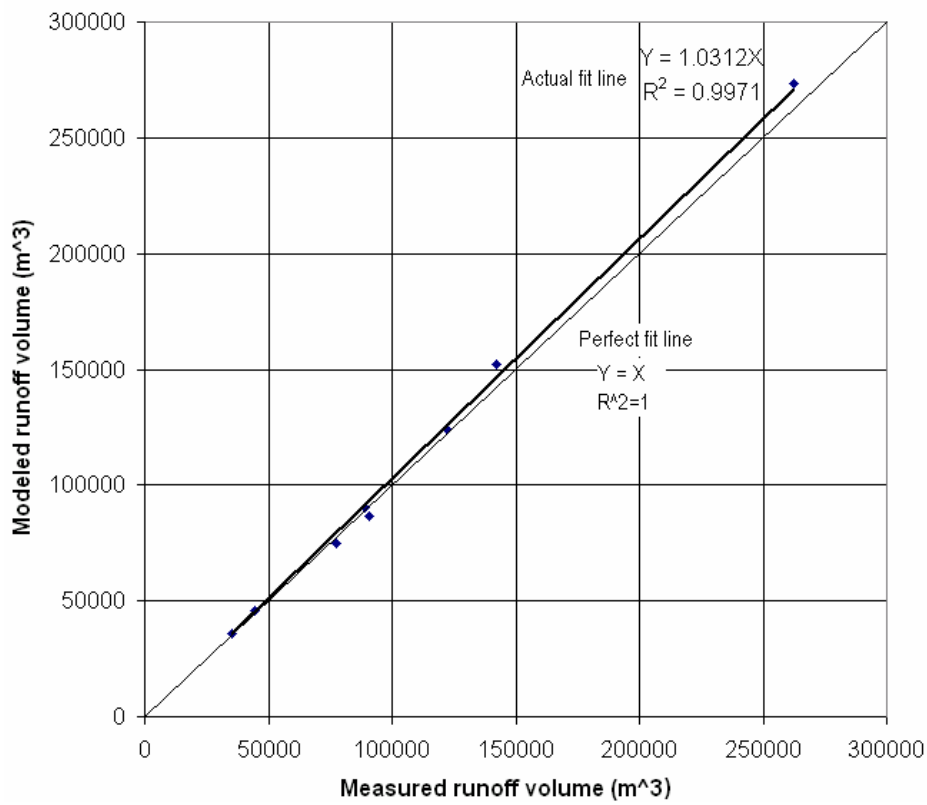



**Figure 67.** Scatter plot for measured and modeled runoff volume

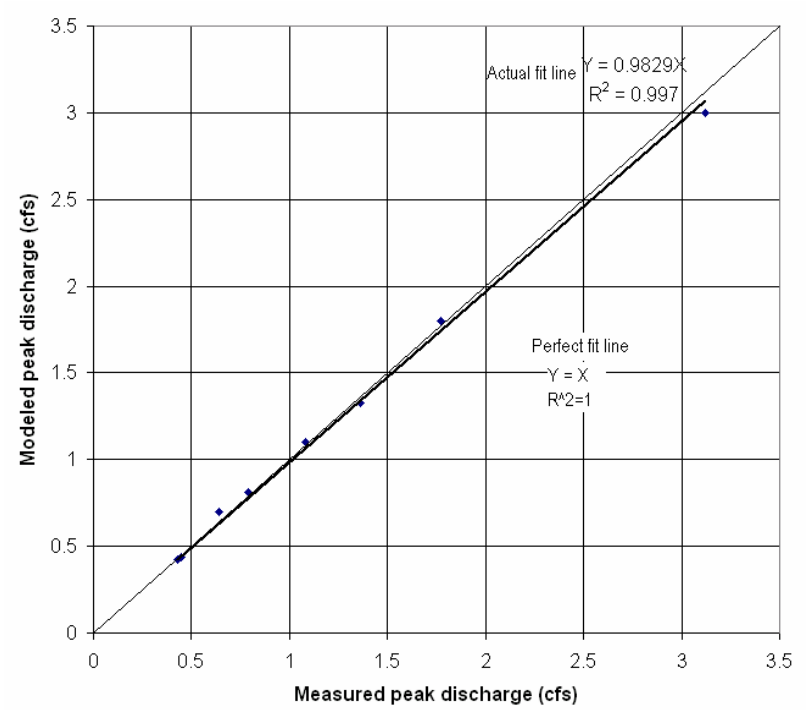



**Figure 68.** Scatter plot for measured and modeled peak discharge

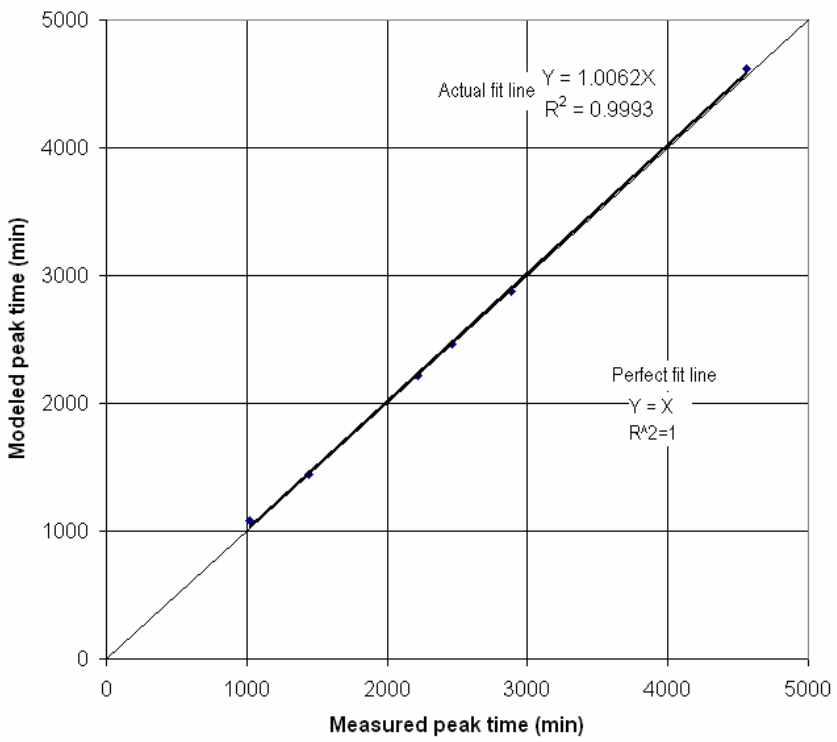



**Figure 69.** Scatter plot for measured and modeled peak time

## 8.4 RELATIONSHIP BETWEEN CN AND AMC

Based on the length of the modeling period, we can categorize our model as an event model instead of a continuous model. An event model simulates storm events one by one. The modeling period ranges from several hours to a few days. In contrast, continuous model simulates storm events over a long time period. The modeling period ranges from several days to a few months. While the event modeling is sufficient for project need, there is another reason to build event modeling in this research. The models WMS and HWM are designed mainly for event modeling. The main runoff generation method that we used is the Curve Number (CN) method. The runoff volume, peak discharge, peak time is mainly controlled by CN. In different storm events, the CN for a certain sub-watershed is different because of different Antecedent Moisture Conditions (AMC). In the modeling, CN for each sub-watershed is selected based on AMC and is fixed once the model procedure starts. If multiple events are modeled continuously, CN needs to be adjusted for different events. This can not be accomplished in WMS or HWM.

Even in events modeling, the choice of CN is the most difficult task. As stated in Section 2.1, CN can be found in National Engineering Handbook (SCS, 1972) based on soil type and land use. For different AMC, Soil Conservation Service (1972) also classified AMC into three types: normal condition (AMC II), dry condition (AMC I) and wet condition (AMC III). The classification of AMC is shown in Table 1, Section 2.1. Equivalent CN can be computed by Equation (2.6) and Equation (2.7).

However, in the research we found only three kinds of AMC were not sufficient to describe various antecedent moisture conditions. Among the parameters used in the models, CN is explicitly related to AMC. How to determine CN value becomes a big problem in the modeling. To determine if CN values change in a systematic way, an attempt was made to find out the relationship between CN and AMC. In the studied watershed, some sub-watershed land uses are paved driveway. Their CN values change

very little with AMC. Sub-watershed Up_Stream is large and mostly natural land. Thus, Up_Stream is chosen to analyze the CN-AMC relationship. It is difficult to measure the soil moisture condition. After many attempts, two-day antecedent precipitation depth (APD), four-day APD and seven-day APD are examined as indicators of AMC.

CN-AMC relationships from WMS and HWM are analyzed. Table 14 shows Up_Stream sub-watershed CN and AMC for the eight storm events in WMS. It should be noticed that CN in Table 14 only refer to CN in Up_Stream sub-watershed, not the overall weighted CN in the model. Figure 70 shows their relationship in graph format. Since two-day APDs in many events are zero, it may not be a good AMC indicator. Most of early rainfall in seven-day APDs is evaporated by sunshine and vegetation. This may be the reason for the poor fit in Figure 70 (c), whose *R* square value is 0.5394. Figure 70 (b) shows the plotted graph with *R* square value 0.7191 for four-day APD and CN. From Figure 70 (a), Figure 70 (b), and Figure 70 (c), it can be seen that four-day APD may be a better indicator of AMC. The *R* square value is defined in Equation 8.1.

$$R^2 = \frac{SS_R}{SS_T} = 1 - \frac{SS_E}{SS_T} \tag{8.1}$$

where $SS_T = \sum_i (y_i - \overline{y})^2$ is the total sum of square;

$SS_R = \sum_i (\hat{y}_i - \overline{y})^2$ is the explained sum of square;

$SS_E = \sum_i (y_i - \hat{y}_i)^2$ is the residual sum of square;

and $y_i$ = actual value of statistic variables;

$\overline{y}_i$ = average value of statistic variables;

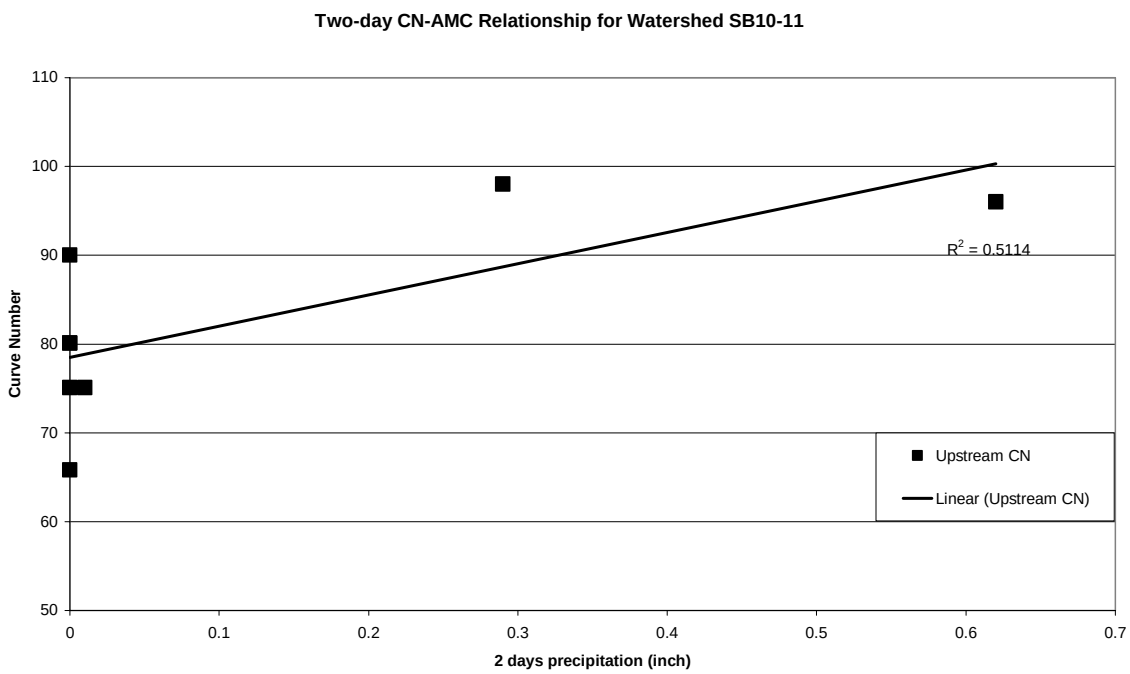
$\hat{y}_i$ = modeled (expected) value of statistic variable.

**Table 14.** Up_Stream sub-watershed CN and AMC for eight events in WMS

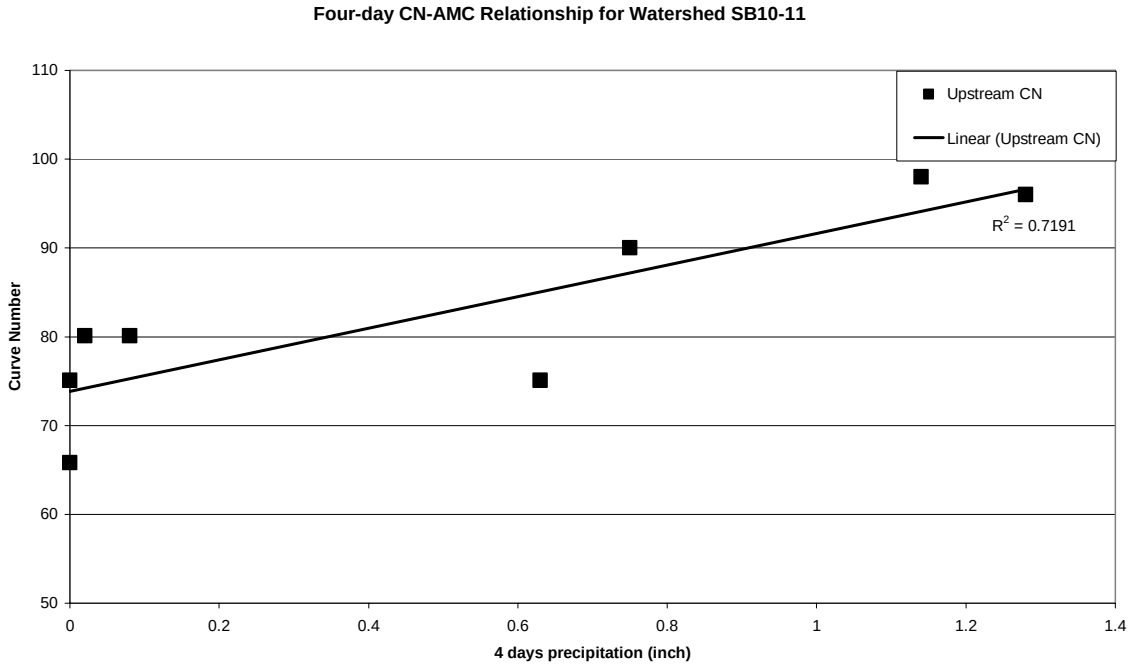
| APD & CN / Date | 2-Day APD | 4-Day APD | 7-Day APD | CN |
|---|---|---|---|---|
| Oct 07 2005 | 0 | 0 | 0 | 65.8 |
| Oct 25 2005 | 0.29 | 1.14 | 1.18 | 98 |
| Nov 27 2005 | 0 | 0.08 | 0.08 | 80.1 |
| Jan 17 2006 | 0 | 0.63 | 0.91 | 75.1 |
| Mar 11 2006 | 0.01 | 0.02 | 0.03 | 80.1 |
| May 11 2006 | 0 | 0 | 0 | 75.1 |
| June 26 2006 | 0.62 | 1.28 | 1.7 | 96 |
| Sept 01 2006 | 0 | 0.75 | 2.27 | 90 |

APD = Antecedent Precipitation Depth;

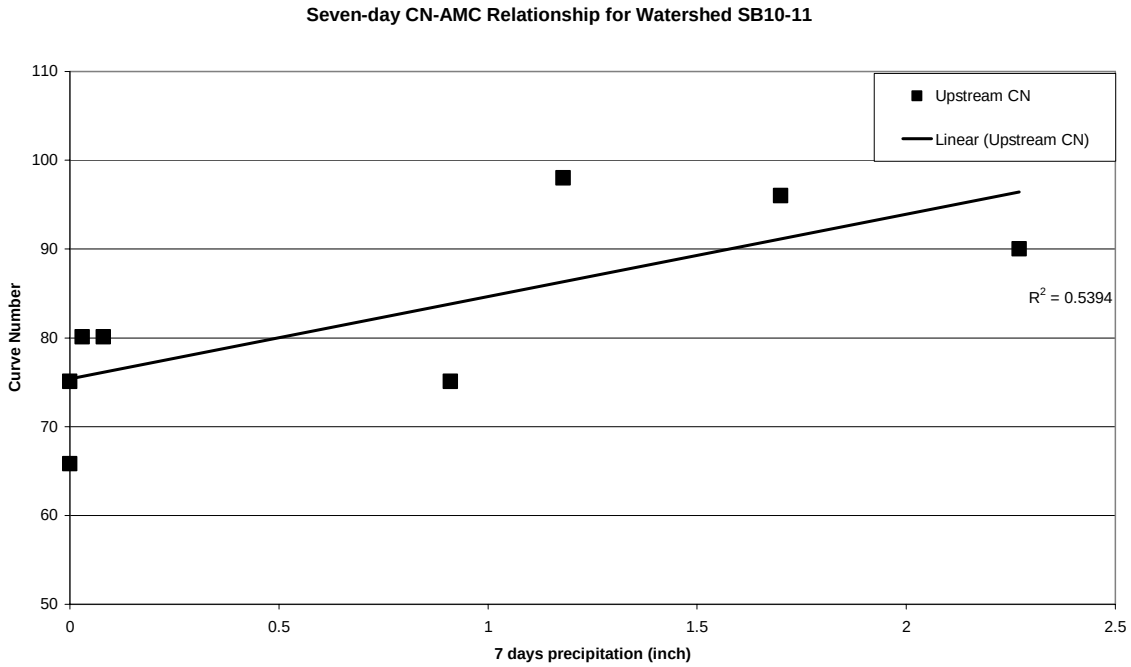
CN = Upstream Sub-watershed Curve Number;




(a) Two-day APD and CN




(b) Four-day APD and CN




(c) Seven-day APD and CN

**Figure 70.** Up_Stream sub-watershed CN and AMC for eight events in WMS

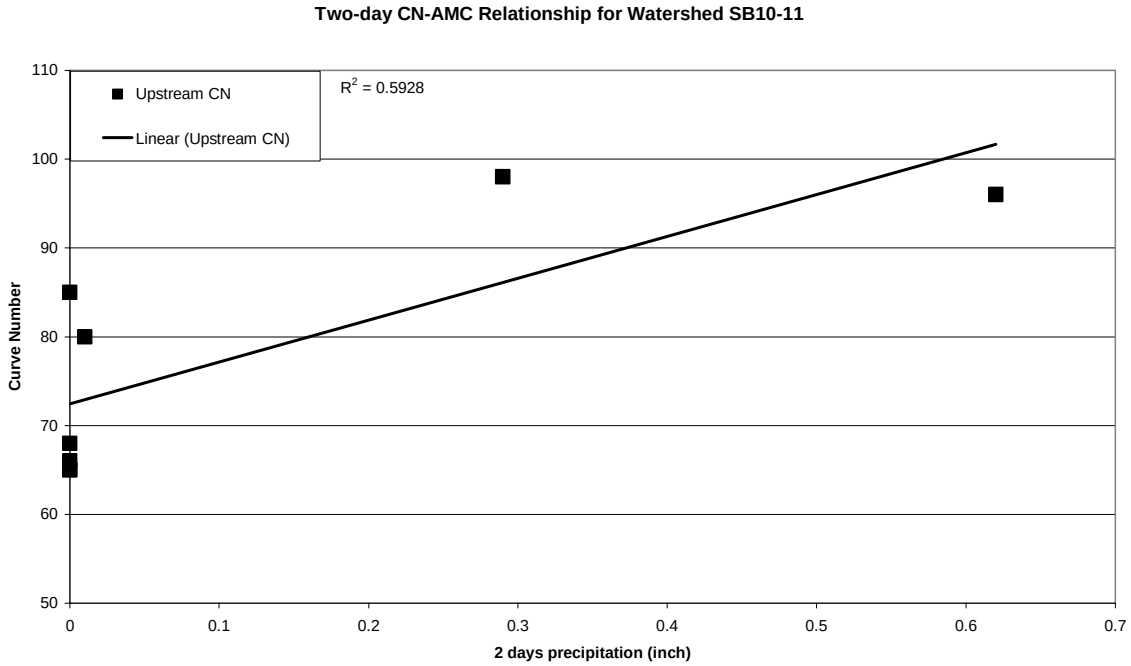
Similar to CN-AMC analysis in WMS model, Up_Stream is chosen to analyze the CN-AMC relationship in HWM. Two-day antecedent precipitation depth (APD), four-day APD and seven-day APD are chosen to be indicators of AMC. Table 15 shows Up_Stream sub-watershed CN and AMC for the eight storm events. Figure 71 shows their relationship in graph format. Since the two-day APDs in many events are zero, it may not be a good AMC indicator. Most of early rainfall in seven-day APDs is evaporated by sunshine and vegetation. This may be the reason for the bad fit in Figure 71 (c), whose *R* square value is 0.6829. In contrast, Figure 71 (b), which is the relationship of four-day APDs and CN, shows very good fit, with *R* square value 0.9858. From Figure 71 (a), Figure 71 (b), and Figure 71 (c), it can be seen that four-day APD may be a better indicator of AMC.

**Table 15.** Up_Stream sub-watershed CN and AMC for eight events in HWM

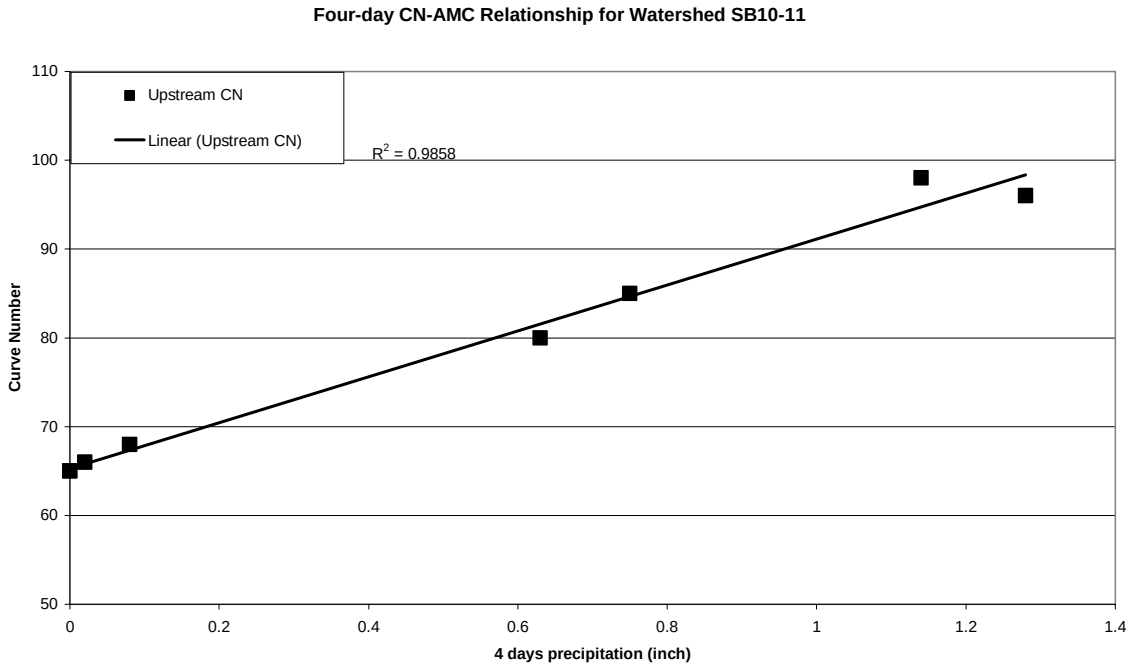
| APD & CN / Date | 2-Day APD | 4-Day APD | 7-Day APD | CN |
|---|---|---|---|---|
| Oct 07 2005 | 0 | 0 | 0 | 65 |
| Oct 25 2005 | 0.29 | 1.14 | 1.18 | 98 |
| Nov 27 2005 | 0 | 0.08 | 0.08 | 68 |
| Jan 17 2006 | 0 | 0.63 | 0.91 | 80 |
| Mar 11 2006 | 0.01 | 0.02 | 0.03 | 66 |
| May 11 2006 | 0 | 0 | 0 | 65 |
| June 26 2006 | 0.62 | 1.28 | 1.7 | 96 |
| Sept 01 2006 | 0 | 0.75 | 2.27 | 85 |

APD = Antecedent Precipitation Depth;

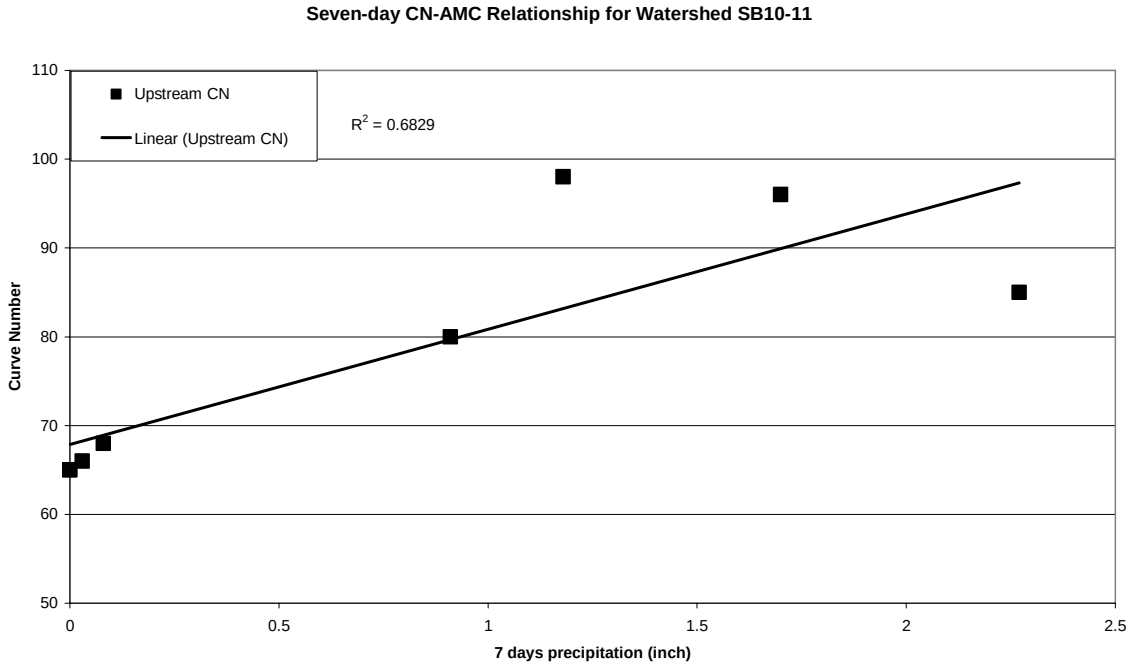
CN = Upstream Sub-watershed Curve Number;




(a) Two-day APD and CN




(b) Four-day APD and CN




(c) Seven-day APD and CN

**Figure 71.** Up_Stream sub-watershed CN and AMC for eight events in HWM

Although Figure 70 (b) is the best fitting among WMS models for CN and AMC, its *R* square value is only 0.7191. The relationship of CN and AMC is not very well defined. The goodness of fit needs to be improved. In contrast, Figure 71 (b), which is the best fitting among HWM models for CN and AMC, shows very good fit, with *R* square value 0.9858. The better fit of CN-AMC relationship in HWM indicates that the CN can be determined accurately by using the fitting curve in future event modeling. This is another strength of HWM over WMS.

## 8.5 COMPARISON WITH WMS

### 8.5.1 Three indicators comparison

Chapter Six and Chapter Eight display results from WMS and HWM. From the direct observation of Figure 43 and Figure 44 with Figure 59 and Figure 60, we can see that the modeled hydrographs from HWM are better than those from WMS. This section further compares the performance of WMS with HWM. Three indicators are used: total runoff volume, peak discharge, and peak time.

From Table 9 and Table 13, it is found that all total runoff volumes from WMS and HWM are satisfactory. The average absolute deviation (AAD) of total runoff volume of HWM is 3.43 %, which is a little larger than that of WMS. A large difference can be observed for peak discharge. One of two modeled peak discharges in WMS is not satisfactory, while all the modeled peak discharges are satisfactory in HWM. The AAD of peak discharge in HWM is only 3.26 %, which is much smaller than that of WMS. A large difference can also be seen for peak time. One of two modeled peak times in WMS is not satisfactory, while all the modeled peak times are satisfactory in HWM. The AAD of peak time in HWM is 15 min, which is much smaller than that of WMS.

Although model structures and development procedures of WMS and HWM are different, the most important parameter that controls the total runoff volume is the curve number (CN). The model calibration is implemented mainly based on net rainfall equality principle, i.e. excess rainfall equals measured total runoff volume. This is the reason that total runoff volume is easy to model.

The peak discharge and peak time are controlled by many factors, such as CN, watershed slope, Muskingum *K*, shape of dimensionless unit hydrographs (DUH), etc. All of these factors can be changed in HWM. However, the shape of DUH is fixed in WMS. This is the main difference between HWM and WMS, and is the motivation to develop HWM. Modeling results show this change is valid and successful. With the total

runoff volume unchanged, the peak discharge and peak time change interactively with the change of those controlling factors. For example, if the watershed slope is increased, the peak discharge will increase, but the peak time is advanced simultaneously. If the $T_r$ [parameter in Linear Exponential Unit Hydrograph (LEUH)] is increased, the peak discharge will decrease, while the peak time is delayed simultaneously. With the application of LEUH, HWM extends the ability of modeling different watershed hydrological responses.

### 8.5.2 Comparison using parameters

The model WMS and HWM can be compared by examining their parameter suitability. As we can see from Table 5, the channel water velocity (CWV) in the channel is set to be 0.0016 *ft/sec*, which is much slower than real CWV. The Muskingum *K* is calculated by *K* = Channel Length / WV. Using the under estimated CWV, the modeled peak discharge and peak time are near satisfaction. However, we know from Equation 7.3 and Table 10 that CWV ranges from 0.025 *ft/sec* to 0.894 *ft/sec* in I-99 project. If we use the calculated CWV from Equation 7.3 and Table 10, the Muskingum *K* will be much smaller than values in Table 5. The modeled peak discharge will be much larger than measured. The modeled peak time will be much earlier than measured. Thus, to obtain good modeling hydrograph results in WMS, parameters are adjusted to unreasonable values. Figure 72 and Figure 73 show the comparison of modeled hydrographs with measured hydrograph with CWV to be 0.1 *ft/sec*, which is about the real value of CWV.

In HWM, LEUH can model different hydrologic responses of a watershed. In each HWM model, real calculated CWV is employed and the modeled results are satisfactory.

From the comparison of CWV in WMS and HWM, we can find the necessity of LEUH method. With appropriate LEUH parameter selection, the peak discharge and peak time can be modeled well using reasonable values of CWV. These results also question the applicability of SCS UH method in the studied watershed. With the calculated CWV, SCS UH method always results in higher peak discharge and early peak time than

measured. Although SCS UH is widely used, it appears to be unsuitable for I-99 highway watershed modeling.

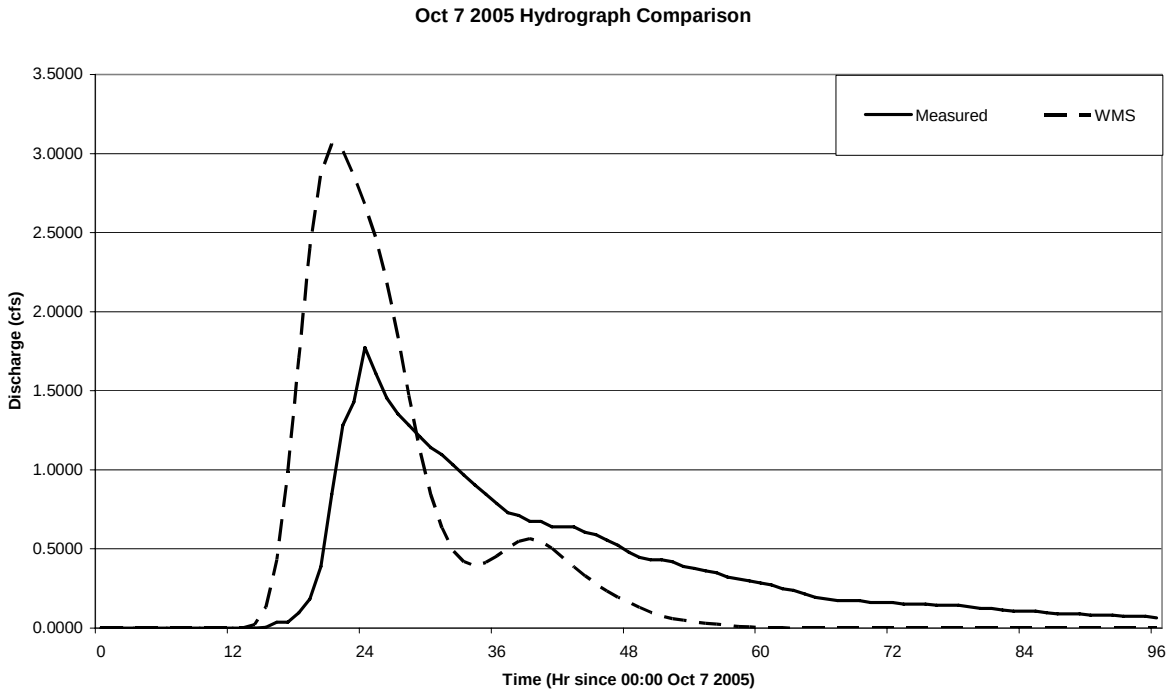



**Figure 72.** Measured and WMS modeled hydrograph for Oct 07 2005 Event with calculated CWV

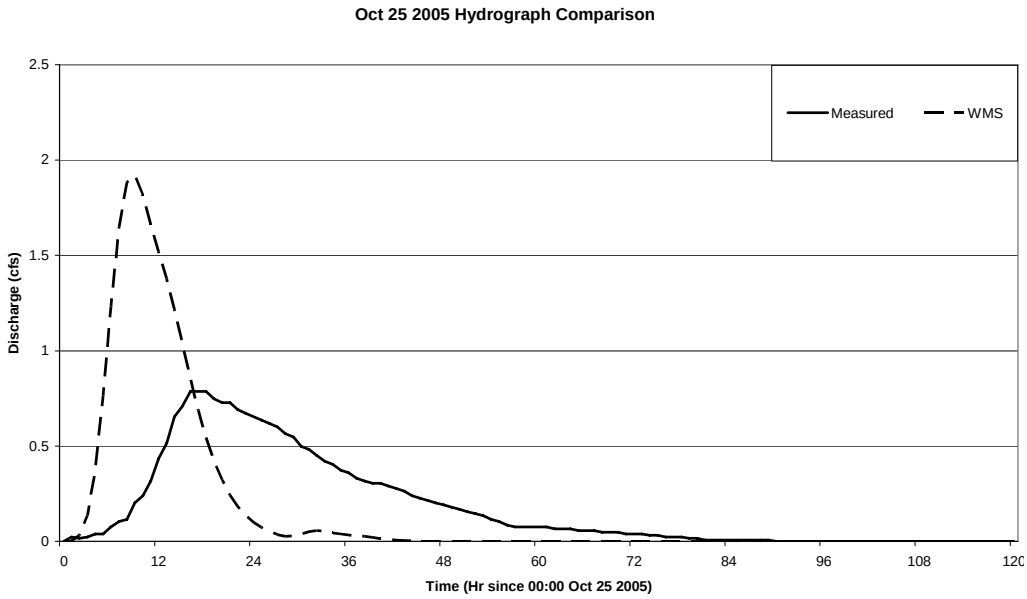



**Figure 73.** Measured and WMS modeled hydrograph for Oct 25 2005 Event with calculated CWV

**Table 16.** The comparison of the three criteria for Watershed Two with calculated CWV

| | | Total Runoff Volume ($ft^3$) | Peak Discharge (cfs) | Peak Time (min) |
|---|---|---|---|---|
| Oct 07 2005 | Measured | 142341 | 1.77 | 1440 |
| | Modeled | 137365 | 3.06 | 1260 |
| | Deviation | -3.50 % | **72.88 %** | **-180 min** |
| Oct 25 2005 | Measured | 77531 | 0.79 | 1020 |
| | Modeled | 74761 | 1.92 | 540 |
| | Deviation | -3.57 % | **143.04 %** | **-480 min** |

As we can see from Figure 72, Figure 73 and Table 16, with calculated CWV, the modeled total runoff volumes do not change much. However, the peak discharges are greatly increased and peak times are greatly advanced. These results show the SCS UH may not be suitable in I-99 highway watershed modeling.

### 8.5.3 Comparison using AMC-CN relationship

Another comparison will be made to evaluate WMS and HWM. The AMC-CN relationships were analyzed in Section 8.4. Although Figure 70 (b) appears to fit the data best compared to Figure 70 (a) and Figure 70 (c), its *R* square value is only 0.7191. In contrast, the best fitting graph in Figure 71 has *R* square value of 0.9858. Thus, the AMC-CN relationship is better represented in the HWM model.

### 8.5.4 Comments on WMS and HWM software packages

The WMS software package used in I-99 Environmental Research was developed by Environmental Modeling Research Laboratory, Brigham Young University. It is a commercial software package and can be used in many situations. WMS has the strength to handle many kinds of GIS data and has graphical user interface (GUI). However, the GIS data in this research is not suitable for WMS. The SCS UH method in WMS is also not suitable for the highway watershed response. Because WMS is an integrated

commercial software package, the details of the calculations in WMS modeling are not displayed to users. We also can not insert our own model ideas, such like LEUH, in WMS.

The HWM package is developed by our research group. Although it does not directly use GIS data, it uses many parameters from GIS data analysis, such as CN, Muskingum *K*, and Muskingum *X*. The applicability of HWM to other watersheds needs to be tested. At least in the I-99 highway watersheds, HWM produces satisfactory modeling results. One of HWM modules utilizes LEUH instead of SCS UH to produce hydrograph. LEUH is shown to be a better unit hydrograph method than SCS UH. Another strength in using HWM is that it is developed by our research group and we know all the calculation details. It is easier for us to change to the most suitable calculation method, which is restricted by an integrated software.

# 9.0 CONCLUSIONS AND RECOMMENDATIONS

## 9.1 CONCLUSIONS

The objective of this research is to build a runoff prediction model using GIS techniques for a watershed affected by highway construction. The model was designed to predict runoff at several outlets in the watershed based on rainfall, land use, soil type, detention pond location, stream network distribution, water velocity, and basin slope, etc. The local applicability, efficiency, and runoff prediction method were verified. Based on the results, the following conclusions can be made.

1. A GIS-based rainfall-runoff model was developed using the Watershed Modeling System (WMS) platform and calibrated to simulate the hydrology and hydraulic behavior along the stream system draining selected watersheds near I-99 highway construction site. Because of GIS data problems, the watershed delineation was carried out manually instead of using GIS-based topographical data. With 15% deviation as accepted criterion, the modeling results of WMS show all total runoff volumes are satisfactory, but the prediction of peak discharge is not satisfactory.

2. To address the shortcomings the WMS model, a new model -- the Highway Watershed Model (HWM) was developed. HWM employs LEUH method instead of SCS UH to generate runoff. LEUH performs better at describing different watershed responses using different dimensionless unit hydrographs. Using the same 15% deviation criterion, all total runoff volume and peak discharges are satisfactory. With 120 minutes (equal two modeling time intervals) deviation as peak time acceptable criterion, all modelled peak

times are satisfactory. It was shown that in I-99 highway watershed modeling, the results from HWM are better than those from WMS.

3. The channel water velocity (CWV) that will produce acceptable results in WMS is much smaller than the calculated velocity. Using a realistic CWV, the peak discharges modeled from WMS are much larger than measured peak discharges. The peak times modeled from WMS also occur earlier than measured peak times. On the other hand, using LEUH, all results are satisfactory, when real calculated CWVs are employed.

4. The relationship between antecedent moisture condition (AMC) and curve number (CN) was investigated. Two-day antecedent precipitation depth (APD), four-day APD, and seven-day APD were tried as indicators of AMC. In both WMS and HWM modeling, the best AMC indicator to fit AMC-CN relationship is the four-day APD. The best $R$ value of linear regression of AMC-CN relationship in WMS is 0.7191, while the best $R$ value of linear regression of AMC-CN relationship in HWM is 0.9858. The relationship between AMC and CN is better represented in HWM model than in WMS model.

## 9.2 RECOMMENDATIONS

Although HWM yields satisfactory results, there are still several improvements that can be made.

1. Due to data availability, only eight storm events were modeled in WMS and HWM. More storm events modeling are needed to further validate the models. Since HWM is a newly developed model and LEUH is a newly developed unit hydrograph, more watershed modeling tests are recommended to demonstrate its usefulness.

2. Because of data availability, only antecedent precipitation depth (APD) is used in discovering CN - AMC relationship. However, CN may change due to many factors, such as temperature, humidity, antecedent sunshine and evaporation, vegetation etc. More

studies should be carried out to explore CN changes according to these comprehensive factors. This may require additional instrumentation to measure changes in soil moisture.

Figure 71 shows the AMC-CN relationship for HWM. Figure 71 (b) is the best fitting graph we can get. However, we are not certain if AMC and CN relationship is really linear. The linear relationship is only an assumption to find out their relationship. More rainfall and soil moisture data are needed to determine a better AMC-CN relationship.

3. In different events modeling of WMS and HWM, $T_c$ and Muskingum $K$ are adapted based on field survey and experience. $T_c$ and Muskingum $K$ may also change with APD, temperature, humidity, antecedent sunshine and evaporation, vegetation etc. More studies are recommended to examine the effect of these factors on $T_c$ and Muskingum $K$.

# APPENDIX A

# FORTRAN PROGRAMS FOR EACH MODULE OF HWM

# PREFACE

The original HWM consists of six modules, which are flexible to model any number of sub-watershed and arbitrary layout. However, it requires keyboard input and complicated layout combination. Under Dr. Quimpo's direction, we re-wrote the program using file input instead of keyboard input. The current model reads all parameters from a file. The final hydrograph can be obtained by double-clicking a master file (batch file). However, the flexibility is reduced. This model can only model seven pre-defined sub-watershed layout situation. If the user wants to model other layouts, the program must be modified. These programs, source codes of the latest version of Highway Watershed Model, are easy to run, but less flexible. We are able to modify the model to accommodate other watersheds configuration. For conciseness, only one program for each module is presented. Detailed explanation of each module and HWM executing procedures are documented in Appendix B.

## A.1 HWM.BAT

```
@echo off
rem **** This Batch file is used to execute all program together. ***

1.0LINEAR_EXP_UH.EXE
1.1EXCESSRAINFALL.exe
1.2EXCESSRAINFALL.exe
1.3EXCESSRAINFALL.exe
1.4EXCESSRAINFALL.exe
1.5EXCESSRAINFALL.exe
1.6EXCESSRAINFALL.exe
1.7EXCESSRAINFALL.exe

2.1HYDRO.exe
2.2HYDRO.exe
2.3HYDRO.exe
2.4HYDRO.exe
2.5HYDRO.exe
2.6HYDRO.exe
2.7HYDRO.exe

A_MUSKINGUM.exe
B_ADD.exe
C_MUSKINGUM.exe
D_ADD.exe
E_MUSKINGUM.exe
F_ADD.exe
G_MUSKINGUM.exe

del HYDRO_4.TXT
ren RHYDRO_4.TXT HYDRO_4.TXT

HH_LEVELPOOL.exe
I_MUSKINGUM.exe
J_KINEMATIC_WAVE.exe

del HYDRO_6.TXT
ren RHYDRO_6.TXT HYDRO_6.TXT

KK_LEVELPOOL.exe
L_ADD.exe

rem Finished!
```

## A.2 LINEAR_EXP_UH.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing
C     we use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ******************************************************************

C     Version: NewUH.F
C     Created by Weizhe An (June/28/06)
C     Considering the limitations of SCS Dimensionless UH method, a new UH is developed
C     under Dr. Quimpo's guidance. The new UH assumes that Dimensionless UH
C     consists two of parts. The first part is linear uprising. The peak time
C     is Tp, as explained in SCS Dimentionless UH method. The second part
C     is exponential decrease from peak to zero. This program generates the Dimensionless
C     UH by calculating 51 points of the UH curve.

C     ******************************************************************
C     Code inputs parameters
C      - Files: Two parameters in file “PARAMETER.txt”
C      - User: Tp, K. They are both read from PARAMETERS.TXT file.
C        Although the theoretical range of Tp is 0-5, the practical range is 0.5-2.
```

```
C       Although the theoretical range of K is 0-Infinity, the practical range
C       is 0.42-0.98.
C     Code output:
C      - Files: UHPROTOTYPE.TXT
C     ****************************************************************
C     UHTp and UHK are the only two parameters used in this program.
C     Y(1:100) and X(1:100) are arrays storing parameters that are read from PARAMETER.TXT
file.
C     t is the pseudo-time, its range is 0-5.
      REAL UHTp, UHK, Y(1:100), t, UH(0:5000), UHArea, ratio
      INTEGER I
      CHARACTER*20 X(1:100)

C     Read all parameters from a file.
      OPEN(30,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 20 I = 1, 100
          READ(30,*,END=23) X(I), Y(I)
20        CONTINUE
23    CLOSE(30, STATUS='KEEP')

      UHTp = Y(87)
      UHK = Y(88)
      UHArea = UHTp/2+UHK
      ratio = 1.33595/UHArea

      DO 21 I = 0, 5000
        t=0.1*I
        IF (UHTp.EQ.0) THEN
          IF (I.EQ.0) THEN
            UH(I) = ratio*1
          ELSE
            UH(I)= ratio*EXP(-(t-UHTp)/UHK)
          ENDIF
        ELSE
          IF (I.LE.UHTp*10) THEN
            UH(I)= ratio*t/UHTp
          ELSE
            UH(I)= ratio*EXP(-(t-UHTp)/UHK)
          ENDIF
        ENDIF
21    CONTINUE

C     Write to UHPROTOTYPE.TXT
      OPEN(10,FILE='UHPROTOTYPE.TXT',STATUS='UNKNOWN')
        DO 22 I=0,5000
          WRITE(10,*) UH(I)
22        CONTINUE
      CLOSE (10, STATUS='KEEP')

      PRINT 98
98    FORMAT('New UH is generated! Please close the window')
      PRINT 99
99    FORMAT('and check the output files.')

      STOP
      END
```

## A.3 EXCESSRAINFALL.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing
C     we use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ****************************************************************

C     Version: EXCESSRAINFALL.F
C     Created by Weizhe An (April/09/06)
C     The program reads the half hour accumulative rainfall txt file and generates
C     a half hour cumulative excess rainfall text file for different sub-watershed
C     because different SBWS have different CNs
C     The RAINFALL.TXT file contains cumulative half-hourly rainfall.

C     ****************************************************************
C     Code inputs:
C      - Files: RAINFALL.TXT
C      - User: Antecedent soil moisture condition 1,2 or 3
C             Curve number, SBWS number, rainfall duration
C     Code output:
C      - Files: EXCESSACCUM.TXT, EXCESS_#.TXT
C     ****************************************************************
```

```
C     RET is the potential max retention, CN is the original input curve number
C     CNN is the corrected curve number for AMC 1,2,3.
C     TEMPPREC and PREVIOUS are temporary variables used to convert EXCESSACCUM to
EXCESSPREC.
C     See the program for details.
      REAL TEMPPREC, PREVIOUS, CN, CNN, RET
C     EXCESSACCUM is temporary variable, then it is written to EXCESSACCUM.TXT.
C     PRECIPITATION is temporary variable, it is read from RAINFALL.TXT.
      REAL EXCESSACCUM, EXCESSPREC, PRECIPITATION, Y(1:100)

C     AMC is the antecedent soil moisture condition, SBWSNUM is the sub-
C     watershed number, PDUR is the storm duration, PN is the interval num
C     of the storm. Because the rainfall is at half-hour intervals, PN = PDUR/0.5 = PDUR*2
      INTEGER AMC, SBWSNUM, PDUR, PN, LINE
      CHARACTER*20 X(1:100)

C     Read all parameters from a file.
      OPEN(30,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 20 I = 1, 100
          READ(30,*,END=21) X(I), Y(I)
20        CONTINUE
21    LINE=I-1
      CLOSE(30, STATUS='KEEP')

C       OPEN(31,FILE='TEST.TXT',STATUS='UNKNOWN')
C           WRITE(31,*) LINE
C         DO 22 I = 1, LINE
C           WRITE(31,*) X(I), Y(I)
C22         CONTINUE
C       CLOSE(31, STATUS='KEEP')

      AMC = Y(2)
      SBWSNUM = Y(3)
      PDUR = Y(4)
      CN = Y(5)

      PN=PDUR*2

C     Conditions for AMC=1 (dry) OR AMC=3 (wet)
C     RET is the potential max retention
C     CNN is the corrected curve number for AMC 1,2,3.
      IF(AMC.EQ.2) CNN=CN*1.0
      IF(AMC.EQ.1) CNN=(4.2*CN)/(10-0.058*CN)
      IF(AMC.EQ.3) CNN=(23*CN)/(10+0.13*CN)
      RET=(1000/CNN)-10

C     Read from RAINFALL.TXT file and write to EXCESSACCUM.TXT
      OPEN(70,FILE='RAINFALL.TXT',STATUS='OLD')
      OPEN(75,FILE='EXCESSACCUM.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.1) OPEN(80,FILE='EXCESS_1.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.2) OPEN(80,FILE='EXCESS_2.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.3) OPEN(80,FILE='EXCESS_3.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.4) OPEN(80,FILE='EXCESS_4.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.5) OPEN(80,FILE='EXCESS_5.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.6) OPEN(80,FILE='EXCESS_6.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.7) OPEN(80,FILE='EXCESS_7.TXT',STATUS='UNKNOWN')
```

```
      IF (SBWSNUM.EQ.8) OPEN(80,FILE='EXCESS_8.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.9) OPEN(80,FILE='EXCESS_9.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.10) OPEN(80,FILE='EXCESS_10.TXT',STATUS='UNKNOWN')
        DO WHILE (PN.GT.0)
          READ(70,*)PRECIPITATION
          CALL EXCESS(EXCESSACCUM,PRECIPITATION,RET)
          WRITE(75,*)EXCESSACCUM
          PN=PN-1
        END DO
      CLOSE (75, STATUS='KEEP')
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C     Read from EXCESSACCUM.TXT file and write to EXCESS_#.TXT
C     EXCESSACCUM.TXT can be used in many watersheds.
C     EXCESS_#.TXT is particular to each watersheds.
      OPEN(75,FILE='EXCESSACCUM.TXT',STATUS='OLD')
        PN=PDUR*2
        PREVIOUS = 0
        DO WHILE (PN.GT.0)
          READ(75,*)TEMPPREC
          IF (PN.EQ.(PDUR*2)) THEN
            EXCESSPREC=TEMPPREC
          ELSE
            EXCESSPREC=TEMPPREC-PREVIOUS
          END IF
          PREVIOUS=TEMPPREC
          IF (EXCESSPREC.LT.0.0001) EXCESSPREC=0
          WRITE(80,*)EXCESSPREC
          PN=PN-1
        END DO

      CLOSE (70, STATUS='KEEP')
      CLOSE (75, STATUS='KEEP')
      CLOSE (80, STATUS='KEEP')

      PRINT 98
98    FORMAT('Calculation finished! Please close the window')
      PRINT 99
99    FORMAT('and check the output files.')

      STOP
      END

C     Subroutine of calculating the rainfall excess
C     Applied Hydrology, Ven Te Chow, ISBN 0-07-010810-2, P148
      SUBROUTINE EXCESS(A,C,B)
            REAL A,B,C
C           C is the precipitation, B is the potential max retention
            IF(C.LT.0.2*B)THEN
            A=0.0
            ELSEIF(C.GE.0.2*B)THEN
            A=(C-0.2*B)**2/(C+0.8*B)
        ENDIF
        RETURN
            END
```

## A.4 HYDRO.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing
C     we use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ******************************************************************
C     Version: HYDRO.F
C     Created by Weizhe An (April/10/06)
C     The program reads the half hour incremental excess rainfall txt file
C     generates half-hour SCS unit hydrograph and generates hydrograph at the outlet
C     at the sub-watershed's outlet.
C     The EXCESS_#.TXT file contains the incremental half-hourly rainfall for the
C     corresponding sub-watershed.

C     ******************************************************************
C     Code inputs:
C      - Files: EXCESS_#.TXT, UHPROTOTYPE.TXT
C      - User: Time of concentration Tc of each sub-watershed.
C              Area of the sub-watershed.
C     Output:
C      - Files: UNITHYDRO.TXT, UH_#.TXT, HYDRO_#.TXT
```

```
C     UNITHYDRO.TXT and UH_#.TXT are both half-hour unit hydrograph, but they are in
different coordinate scale
C     UNITHYDRO.TXT are in natural coordinate, UH_#.TXT are in 5 min interval coordinate. Be
careful.
C     ******************************************************************
C     The following parameters are adopted from Applied Hydrology, Ven Te Chow, ISBN 0-07-
010810-2, Page 229.
C     Tc is the time of concentration, tp is the lag time
C     TTp is the time of rise, tr is the duration of unit hydrograph = 0.5 hr
C     tb is the duration of the direct runoff
C     qp is the unit hydrograph peak discharge
C     PROTOTYPE is the original SCS unit hydrograph, by proportion, not real values
C     UHDUR is the unit hydrograph duration = 0.5 hr = 30 min
C     INTERVAL is the time interval of the prototype unit hydrograph
      REAL Tc, tp, TTp, tr, tb, Area, qp, PROTOTYPE, UHDUR, INTERVAL
C     UNITHYDRO(5000) is a temporary array
C     TEMP is a temporary value
C     FIVE is five minutes in hour
C     EXCESS is the excess rainfall in a certain half hour
C     UNIT is the temporary variable to store the unit hydrograph
      REAL UNITHYDRO(0:5000), TEMP, FIVE, EXCESS, UNIT(0:4200),Y(1:100)
C     SBWSNUM is the watershed number, I and J are the index
C     FLAG is a indicator
C     UNITNUM is the number of unit hydrograph coordinate
      INTEGER SBWSNUM, I, J, FLAG, UNITNUM, LINE
C     HYDROARRAY is the temporary array to store hydrograph
C     HYDRO_ONE_D is the temporary array to store one dimension hydrograph
      REAL HYDRO_ONE_D(0:28800), HYDROARRAY(0:480,0:28800)
      CHARACTER*20 X(1:100)

C     Read all parameters from a file.
      OPEN(50,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 200 I = 1, 100
          READ(50,*,END=201) X(I), Y(I)
200     CONTINUE
201   LINE=I-1
      CLOSE(50, STATUS='KEEP')

      Tc=Y(6)
      SBWSNUM=Y(3)
      Area=Y(7)

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

      tp = 0.6*Tc/60
C     TTp is the time of rise;
C     We are generating half-hour unit hydrograph, so tr = 0.5, tr/2=0.25
      tr = 0.5
      TTp = 0.25 + tp
      tb = 2.67*TTp
C     We use the English unit system. The peak discharge unit is cfs.
      qp = 483.4*Area/TTp
      UHDUR = TTp*5

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C     Based on prototype unit hydrograph, derive the real unit hydrograph
```

```
      OPEN (70,FILE='UHPROTOTYPE.TXT',STATUS='OLD')
      OPEN (71,FILE='UNITHYDRO.TXT',STATUS='UNKNOWN')
        DO 20 I = 0, 5000
          READ (70,*)PROTOTYPE
          WRITE(71,*)PROTOTYPE*qp
20        CONTINUE
      CLOSE (70, STATUS='KEEP')

      INTERVAL = TTp/10
      CLOSE (71, STATUS='KEEP')

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C     Read the random interval unit hydrograph into an array.
      OPEN (71,FILE='UNITHYDRO.TXT',STATUS='OLD')
        DO 21 I = 0,5000
          READ (71,*)UNITHYDRO(I)
21        END DO
      CLOSE (71, STATUS='DELETE')

CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

      IF (SBWSNUM.EQ.1) OPEN (73,FILE='UH_1.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.2) OPEN (73,FILE='UH_2.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.3) OPEN (73,FILE='UH_3.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.4) OPEN (73,FILE='UH_4.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.5) OPEN (73,FILE='UH_5.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.6) OPEN (73,FILE='UH_6.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.7) OPEN (73,FILE='UH_7.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.8) OPEN (73,FILE='UH_8.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.9) OPEN (73,FILE='UH_9.TXT',STATUS='UNKNOWN')
      IF (SBWSNUM.EQ.10) OPEN (73,FILE='UH_10.TXT',STATUS='UNKNOWN')
C     UH_# is the 5 minute interval unit hydrograph for a certain watershed
C     UNITHYDRO is the random interval unit hydrograph for a certain watershed
C     The following codes convert UNITHYDRO to UH_# using linear interpolation
C     Because UNITHYDRO is a random interval unit hydrograph, we set
C     it as a temporary file and do not store it.
        WRITE(73,*)0
        FIVE = 0.083333333
        J = 0
        DO WHILE (J.LE.5000)
        DO 30 J = 0, 5000
          IF (J*INTERVAL.LE.FIVE .AND. (J+1)*INTERVAL.GE.FIVE) THEN
            TEMP = UNITHYDRO(J)+(UNITHYDRO(J+1)-UNITHYDRO(J))*
     *      (FIVE-J*INTERVAL)/INTERVAL
            WRITE(73,*)TEMP
            GOTO 31
          END IF
30        CONTINUE
31        FIVE = FIVE + 0.083333333
        END DO
        WRITE(73,*)0
      CLOSE (73, STATUS='KEEP')

C     The following code convert the excess rainfall in different time
C     into sub-hydrograph and add them together to get hydrograph at the SBWS outlet
      IF (SBWSNUM.EQ.1) OPEN (74,FILE='EXCESS_1.TXT',STATUS='OLD')
```

```
      IF (SBWSNUM.EQ.2) OPEN (74,FILE='EXCESS_2.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.3) OPEN (74,FILE='EXCESS_3.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.4) OPEN (74,FILE='EXCESS_4.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.5) OPEN (74,FILE='EXCESS_5.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.6) OPEN (74,FILE='EXCESS_6.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.7) OPEN (74,FILE='EXCESS_7.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.8) OPEN (74,FILE='EXCESS_8.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.9) OPEN (74,FILE='EXCESS_9.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.10) OPEN (74,FILE='EXCESS_10.TXT',STATUS='OLD')

      IF (SBWSNUM.EQ.1) OPEN (73,FILE='UH_1.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.2) OPEN (73,FILE='UH_2.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.3) OPEN (73,FILE='UH_3.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.4) OPEN (73,FILE='UH_4.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.5) OPEN (73,FILE='UH_5.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.6) OPEN (73,FILE='UH_6.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.7) OPEN (73,FILE='UH_7.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.8) OPEN (73,FILE='UH_8.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.9) OPEN (73,FILE='UH_9.TXT',STATUS='OLD')
      IF (SBWSNUM.EQ.10) OPEN (73,FILE='UH_10.TXT',STATUS='OLD')

      FLAG = 1
      I=0

      DO WHILE (FLAG.LE.6000)
        READ (73,*,END=101) UNIT(I)
        I=I+1
        FLAG=FLAG+1
      END DO
101   UNITNUM=I-1
      FLAG = 1
      I=0
C     We do not know how long the two file are, so we use EOF to
C     determine when to stop reading it.
C     END=LABEL: Specifies a label to branch (jump) to if an
C               END-OF-FILE (EOF) is reached (READing past
C               the end of file).
C     If no EOF, reading will continue...
      DO WHILE (FLAG.EQ.1)
        READ (74,*,end=40) EXCESS
          DO 102 J=0, UNITNUM
            HYDROARRAY(I,J+6*I) = EXCESS*UNIT(J)
102         CONTINUE
        I=I+1
      END DO
40    CLOSE (73, STATUS='DELETE')
      CLOSE (74, STATUS='KEEP')

C     Initialize the 1-D hydrograph
      DO 70 K = 0, 28800
        HYDRO_ONE_D(K)=0
70      CONTINUE
C     Add the 2-D hydrograph together into 1-D hydrograph
      DO 80 K = 0, 28800
      DO 60 I = 0, 480
        HYDRO_ONE_D(K)=HYDRO_ONE_D(K)+HYDROARRAY(I,K)
```

```
60   CONTINUE
     IF ((HYDRO_ONE_D(K).GE.10000).OR.(HYDRO_ONE_D(K).LE.0.0001)) THEN
        HYDRO_ONE_D(K)=0
     END IF
80   CONTINUE

     IF (SBWSNUM.EQ.1) OPEN (75,FILE='HYDRO_1.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.2) OPEN (75,FILE='HYDRO_2.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.3) OPEN (75,FILE='HYDRO_3.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.4) OPEN (75,FILE='HYDRO_4.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.5) OPEN (75,FILE='HYDRO_5.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.6) OPEN (75,FILE='HYDRO_6.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.7) OPEN (75,FILE='HYDRO_7.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.8) OPEN (75,FILE='HYDRO_8.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.9) OPEN (75,FILE='HYDRO_9.TXT',STATUS='UNKNOWN')
     IF (SBWSNUM.EQ.10) OPEN (75,FILE='HYDRO_10.TXT',STATUS='UNKNOWN')

C    Write the 1-D hydrograph to file
     DO 90 K = 0, 28800
     WRITE(75,*)HYDRO_ONE_D(K)
90   CONTINUE
     CLOSE (75, STATUS='KEEP')

     PRINT 98
98   FORMAT('Calculation finished! Please close the window')
     PRINT 99
99   FORMAT('and check the output files.')

     STOP
     END
```

## A.5 MUSKINGUM.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing
C     we use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ****************************************************************
C     Version: MUSKINGUM.F
C     Created by Weizhe An (April/11/06)

C     Input: HYDRO_#.TXT, K, X
C     Output: RHYDRO_#.TXT
C     ******************************************************************************
C     This section is for input data
C     K is the storage constant
C     X is the wedge parameter
C     INTERV is rainfall interval (5 min) used in RUNOFF program
C     C0, C1, C2 are coefficients of Muskingum routing
C     I(X) is the Muskingum input, Q(X) is the Muskingum output
C     They are expressed as C1,C2,C3 in the book, Applied Hydrology, Ven Te Chow, ISBN 0-07-
010810-2, P258
      REAL K,X,INTERV,C0,C1,C2,Q(0:2880),I(0:2880),Y(1:100)
      INTEGER INDEX,J
```

```
C     BEFORE, AFTER are file name representatives.
      CHARACTER*20 BEFORE,AFTER, XX(1:100)

C     Read all parameters from a file.
      OPEN(30,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 20 J = 1, 100
          READ(30,*,END=21) XX(J), Y(J)
20        CONTINUE
21    LINE=J-1
      CLOSE(30, STATUS='KEEP')

      SBWSNUM=Y(45)
      K=Y(46)
      X=Y(47)
      INTERV = 0.08333333

C     Select the file to read
      IF(SBWSNUM.EQ.1)THEN
      BEFORE='HYDRO_1.TXT'
      AFTER='RHYDRO_1.TXT'
      ENDIF
      IF(SBWSNUM.EQ.2)THEN
      BEFORE='HYDRO_2.TXT'
      AFTER='RHYDRO_2.TXT'
      ENDIF
      IF(SBWSNUM.EQ.3)THEN
      BEFORE='HYDRO_3.TXT'
      AFTER='RHYDRO_3.TXT'
      ENDIF
      IF(SBWSNUM.EQ.4)THEN
      BEFORE='HYDRO_4.TXT'
      AFTER='RHYDRO_4.TXT'
      ENDIF
      IF(SBWSNUM.EQ.5)THEN
      BEFORE='HYDRO_5.TXT'
      AFTER='RHYDRO_5.TXT'
      ENDIF
      IF(SBWSNUM.EQ.6)THEN
      BEFORE='HYDRO_6.TXT'
      AFTER='RHYDRO_6.TXT'
      ENDIF
      IF(SBWSNUM.EQ.7)THEN
      BEFORE='HYDRO_7.TXT'
      AFTER='RHYDRO_7.TXT'
      ENDIF
      IF(SBWSNUM.EQ.8)THEN
      BEFORE='HYDRO_8.TXT'
      AFTER='RHYDRO_8.TXT'
      ENDIF
      IF(SBWSNUM.EQ.9)THEN
      BEFORE='HYDRO_9.TXT'
      AFTER='RHYDRO_9.TXT'
      ENDIF
      IF(SBWSNUM.EQ.10)THEN
      BEFORE='HYDRO_10.TXT'
      AFTER='RHYDRO_10.TXT'
```

```
      ENDIF

C     File unit 10 is the input file, it contains routing input required by Muskingum.
      OPEN (10,STATUS='OLD',FILE=BEFORE)
C     Initialize the input and output, I() is the Muskingum input, Q() is the Muskingum output.
      DO 5 INDEX=0,2880
        I(INDEX)=0
        Q(INDEX)=0
5     CONTINUE
C     Read data from the runoff txt file
      DO 6 INDEX = 0,2880
        READ (10,*) I(INDEX)
6     CONTINUE
      K=K/60
C     This section is to calculate the coefficients
      C0=(0.5*INTERV-K*X)/(K*(1.0-X)+0.5*INTERV)
      C1=(0.5*INTERV+K*X)/(K*(1.0-X)+0.5*INTERV)
      C2=(K*(1.0-X)-0.5*INTERV)/(K*(1.0-X)+0.5*INTERV)

C     This section is to route and print out the results
      Q(0)=0
      DO 7 INDEX = 1,2880
        Q(INDEX)=C0*I(INDEX)+C1*I(INDEX-1)+C2*Q(INDEX-1)
        IF (Q(INDEX).GE.10000 .OR. Q(INDEX).LE.0.0001) THEN
          Q(INDEX)=0
        END IF
7     CONTINUE

C     File 20 is the output file, it contains Muskingum routing output.
      OPEN (20,FILE=AFTER,STATUS='UNKNOWN')
      DO 8 INDEX=0,2880
        WRITE (20,*) Q(INDEX)
8     CONTINUE

      CLOSE (10, STATUS='KEEP')
      CLOSE (20, STATUS='KEEP')

      PRINT 98
98    FORMAT('Calculation finished! Please close the window')
      PRINT 99
99    FORMAT('and check the output files.')

      STOP
      END
```

## A.6 ADD.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing.
C     We use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ****************************************************************

C     Version: ADD.F
C     Created by Weizhe An (April/11/06)
C     ****************************************************************

C     Q* is the original hydrograph data array, QT is the total hydrograph data array
      REAL Q1(0:2880),Q2(0:2880),Q3(0:2880)
      REAL Q4(0:2880),Q5(0:2880),QT(0:2880),Y(1:100)

C     RHYDRO_* is the routed hydrograph from upstreams.
C     HYDRO is the hydrograph file at the question SBWS.
C     THYDRO is the final total hydrograph
      CHARACTER*20 HYDRO,RHYDRO_A,RHYDRO_B,RHYDRO_C,RHYDRO_D,THYDRO
      CHARACTER*20 X(1:100)
C     OUTLETNUM is the number of the outlets
C     HYDRONUM is the number of routed hydrographs
```

```
C     H is the hydrograph counter when the added hydrograph is input
C     H* is the added input hydrograph num. Z is the index.
C     FINAL is a variable to indicate whether the calculation is the final.
C     If it is final, the output data is at one-hour intervals
C     If it is not, the output data is at 5 minute intervals
      INTEGER OUTLETNUM,HYDRONUM,H,H1,H2,H3,H4,Z,FINAL


      OPEN(30,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 22 I = 1, 100
          READ(30,*,END=21) X(I), Y(I)
22        CONTINUE
21      LINE=I-1
      CLOSE(30, STATUS='KEEP')


C     H is the hydrograph counter when the added hydrograph is input.
      H=0

      FINAL=Y(48)
      OUTLETNUM=Y(49)
      HYDRONUM=Y(50)
      H1=Y(51)

      H=H+1
C       IF(H.EQ.HYDRONUM)GOTO 5
C       PRINT 4
C4      FORMAT('Please enter the next hydrograph')
C       READ(5,*)H2
C       H=H+1
C       IF(H.EQ.HYDRONUM)GOTO 5
C       PRINT 4
C       READ(5,*)H3
C       H=H+1
C       IF(H.EQ.HYDRONUM)GOTO 5
C       PRINT 4
C       READ(5,*)H4
C       H=H+1
C       IF(H.EQ.HYDRONUM)GOTO 5

C     Set the total hydrograph file name.
5     IF(OUTLETNUM.EQ.1)HYDRO='HYDRO_1.TXT'
      IF(OUTLETNUM.EQ.2)HYDRO='HYDRO_2.TXT'
      IF(OUTLETNUM.EQ.3)HYDRO='HYDRO_3.TXT'
      IF(OUTLETNUM.EQ.4)HYDRO='HYDRO_4.TXT'
      IF(OUTLETNUM.EQ.5)HYDRO='HYDRO_5.TXT'
      IF(OUTLETNUM.EQ.6)HYDRO='HYDRO_6.TXT'
      IF(OUTLETNUM.EQ.7)HYDRO='HYDRO_7.TXT'
      IF(OUTLETNUM.EQ.8)HYDRO='HYDRO_8.TXT'
      IF(OUTLETNUM.EQ.9)HYDRO='HYDRO_9.TXT'
      IF(OUTLETNUM.EQ.10)HYDRO='HYDRO_10.TXT'

      IF(H1.EQ.1)RHYDRO_A='RHYDRO_1.TXT'
      IF(H1.EQ.2)RHYDRO_A='RHYDRO_2.TXT'
      IF(H1.EQ.3)RHYDRO_A='RHYDRO_3.TXT'
      IF(H1.EQ.4)RHYDRO_A='RHYDRO_4.TXT'
```

```
      IF(H1.EQ.5)RHYDRO_A='RHYDRO_5.TXT'
      IF(H1.EQ.6)RHYDRO_A='RHYDRO_6.TXT'
      IF(H1.EQ.7)RHYDRO_A='RHYDRO_7.TXT'
      IF(H1.EQ.8)RHYDRO_A='RHYDRO_8.TXT'
      IF(H1.EQ.9)RHYDRO_A='RHYDRO_9.TXT'
      IF(H1.EQ.10)RHYDRO_A='RHYDRO_10.TXT'

      IF(H2.EQ.1)RHYDRO_B='RHYDRO_1.TXT'
      IF(H2.EQ.2)RHYDRO_B='RHYDRO_2.TXT'
      IF(H2.EQ.3)RHYDRO_B='RHYDRO_3.TXT'
      IF(H2.EQ.4)RHYDRO_B='RHYDRO_4.TXT'
      IF(H2.EQ.5)RHYDRO_B='RHYDRO_5.TXT'
      IF(H2.EQ.6)RHYDRO_B='RHYDRO_6.TXT'
      IF(H2.EQ.7)RHYDRO_B='RHYDRO_7.TXT'
      IF(H2.EQ.8)RHYDRO_B='RHYDRO_8.TXT'
      IF(H2.EQ.9)RHYDRO_B='RHYDRO_9.TXT'
      IF(H2.EQ.10)RHYDRO_B='RHYDRO_10.TXT'

      IF(H3.EQ.1)RHYDRO_C='RHYDRO_1.TXT'
      IF(H3.EQ.2)RHYDRO_C='RHYDRO_2.TXT'
      IF(H3.EQ.3)RHYDRO_C='RHYDRO_3.TXT'
      IF(H3.EQ.4)RHYDRO_C='RHYDRO_4.TXT'
      IF(H3.EQ.5)RHYDRO_C='RHYDRO_5.TXT'
      IF(H3.EQ.6)RHYDRO_C='RHYDRO_6.TXT'
      IF(H3.EQ.7)RHYDRO_C='RHYDRO_7.TXT'
      IF(H3.EQ.8)RHYDRO_C='RHYDRO_8.TXT'
      IF(H3.EQ.9)RHYDRO_C='RHYDRO_9.TXT'
      IF(H3.EQ.10)RHYDRO_C='RHYDRO_10.TXT'

      IF(H4.EQ.1)RHYDRO_D='RHYDRO_1.TXT'
      IF(H4.EQ.2)RHYDRO_D='RHYDRO_2.TXT'
      IF(H4.EQ.3)RHYDRO_D='RHYDRO_3.TXT'
      IF(H4.EQ.4)RHYDRO_D='RHYDRO_4.TXT'
      IF(H4.EQ.5)RHYDRO_D='RHYDRO_5.TXT'
      IF(H4.EQ.6)RHYDRO_D='RHYDRO_6.TXT'
      IF(H4.EQ.7)RHYDRO_D='RHYDRO_7.TXT'
      IF(H4.EQ.8)RHYDRO_D='RHYDRO_8.TXT'
      IF(H4.EQ.9)RHYDRO_D='RHYDRO_9.TXT'
      IF(H4.EQ.10)RHYDRO_D='RHYDRO_10.TXT'

      IF(OUTLETNUM.EQ.1)THYDRO='HYDRO_1.TXT'
      IF(OUTLETNUM.EQ.2)THYDRO='HYDRO_2.TXT'
      IF(OUTLETNUM.EQ.3)THYDRO='HYDRO_3.TXT'
      IF(OUTLETNUM.EQ.4)THYDRO='HYDRO_4.TXT'
      IF(OUTLETNUM.EQ.5)THYDRO='HYDRO_5.TXT'
      IF(OUTLETNUM.EQ.6)THYDRO='HYDRO_6.TXT'
      IF(OUTLETNUM.EQ.7)THYDRO='HYDRO_7.TXT'
      IF(OUTLETNUM.EQ.8)THYDRO='HYDRO_8.TXT'
      IF(OUTLETNUM.EQ.9)THYDRO='HYDRO_9.TXT'
      IF(OUTLETNUM.EQ.10)THYDRO='HYDRO_10.TXT'

C     Initialize all variables to be zero.
      DO 20 Z=0,2880
        Q1(Z)=0
        Q2(Z)=0
        Q3(Z)=0
```

```
      Q4(Z)=0
      Q5(Z)=0
20    CONTINUE

C     File unit 1 is the original HYDRO_#.TXT file
C     RHYDRO_* and HYDRO are the original HYDRO_#.TXT file
      OPEN(1,STATUS='OLD',FILE=HYDRO)
      IF(HYDRONUM.GE.1)OPEN(2,STATUS='OLD',FILE=RHYDRO_A)
      IF(HYDRONUM.GE.2)OPEN(3,STATUS='OLD',FILE=RHYDRO_B)
      IF(HYDRONUM.GE.3)OPEN(4,STATUS='OLD',FILE=RHYDRO_C)
      IF(HYDRONUM.GE.4)OPEN(5,STATUS='OLD',FILE=RHYDRO_D)
C     Read the routed hydrograph into an array.
      DO 6 Z=0,2880
      READ(1,*)Q1(Z)
6     CONTINUE

C     If the number of rounted file is greater than 1, more statements need to be executed.
      IF(HYDRONUM.GE.1)THEN
      DO 7 Z=0,2880
      READ(2,*)Q2(Z)
7      CONTINUE
      ENDIF

      IF(HYDRONUM.GE.2)THEN
      DO 8 Z=0,2880
      READ(3,*)Q3(Z)
8      CONTINUE
      ENDIF

      IF(HYDRONUM.GE.3)THEN
      DO 9 Z=0,2880
      READ(4,*)Q4(Z)
9      CONTINUE
      ENDIF

      IF(HYDRONUM.GE.4)THEN
      DO 10 Z=0,2880
      READ(5,*)Q5(Z)
10      CONTINUE
      ENDIF

C     Adding hydrographs
      DO 11 Z=0,2880
      QT(Z)=Q1(Z)+Q2(Z)+Q3(Z)+Q4(Z)+Q5(Z)
11    CONTINUE

C     THYDRO is the final total hydrograph
      OPEN(10,STATUS='UNKNOWN',FILE=THYDRO)
      IF (FINAL.EQ.0) THEN
      DO 30 Z=0,2880
        WRITE (10,*)QT(Z)
30      CONTINUE
      ELSE
      DO 31 Z=0,2880,12
        WRITE (10,*)QT(Z)
31      CONTINUE
```

```
      END IF

      CLOSE (1, STATUS='KEEP')
      IF(HYDRONUM.GE.1) CLOSE (2, STATUS='KEEP')
      IF(HYDRONUM.GE.2) CLOSE (3, STATUS='KEEP')
      IF(HYDRONUM.GE.3) CLOSE (4, STATUS='KEEP')
      IF(HYDRONUM.GE.4) CLOSE (5, STATUS='KEEP')
      CLOSE (10, STATUS='KEEP')

      PRINT 98
98    FORMAT('Calculation finished! Please close the window')
      PRINT 99
99    FORMAT('and check the output files.')
      STOP
      END
```

## A.7 LEVELPOOL.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing.
C     We use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ****************************************************************
C     Version: LEVELPOOL.F
C     Created by Weizhe An (April/16/06)

C     Input: HYDRO_#.TXT
C     Output: RHYDRO_#.TXT
C     ******************************************************************************
C     This section is for input data
C     INTERV is rainfall interval (5 min = 300 sec) used in RUNOFF program
C     I() is the Level pool input, Q() is the Level pool output.
C     ADDINPUT is a middle variable array, it stores two adjacent inflow
C     it is column 4 of Table 8.2.3 in Chow's text book.
      REAL Q(1:2880),I(1:2880),ADDINPUT(1:2880)
C     VARAIBLE1 AND VARIABLE2 are column 5 and 6 of Table 8.2.3 in Chow's text book.
      REAL VARIABLE1(1:2880),VARIABLE2(1:2880)
C     ELE, DIS, STO, REL are temporary arrays to store the reservoir characters
      REAL ELE(1:30),DIS(1:30),STO(1:30),REL(1:30)
```

```
C     S1, Q1 are the initial storage and outflow of the reservoir
C     INITIAL is the initial elevation of the reservoir, INTERV is 5 min = 300 sec
      REAL INTERV,INITIAL,S1,Q1,Y(1:100)
C     INDEX and J are indices, RESNUM is the reservoir number
C     SBWSNUM is the sub-watershed number
C     FLAG is an indicator, RESINTERVAL is the reservoir's character's interval numbers
      INTEGER INDEX,J,SBWSNUM,RESNUM,FLAG,RESINTERVAL
      CHARACTER*20 X(1:100)
      CHARACTER*20 BEFORE,AFTER,DISCHARGE,STORAGE,ELEVATION,RELATIONSHIP
C     The following characters are file name representatives.

      OPEN(31,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 20 J = 1, 100
          READ(31,*,END=21) X(J), Y(J)
20        CONTINUE
21      LINE=J-1
      CLOSE(31, STATUS='KEEP')

      SBWSNUM=Y(69)
      RESNUM=Y(70)
      INITIAL=Y(71)

      INTERV = 300

C     Select the file to read
      IF(SBWSNUM.EQ.1)THEN
      BEFORE='HYDRO_1.TXT'
      AFTER='RHYDRO_1.TXT'
      ENDIF
      IF(SBWSNUM.EQ.2)THEN
      BEFORE='HYDRO_2.TXT'
      AFTER='RHYDRO_2.TXT'
      ENDIF
      IF(SBWSNUM.EQ.3)THEN
      BEFORE='HYDRO_3.TXT'
      AFTER='RHYDRO_3.TXT'
      ENDIF
      IF(SBWSNUM.EQ.4)THEN
      BEFORE='HYDRO_4.TXT'
      AFTER='RHYDRO_4.TXT'
      ENDIF
      IF(SBWSNUM.EQ.5)THEN
      BEFORE='HYDRO_5.TXT'
      AFTER='RHYDRO_5.TXT'
      ENDIF
      IF(SBWSNUM.EQ.6)THEN
      BEFORE='HYDRO_6.TXT'
      AFTER='RHYDRO_6.TXT'
      ENDIF
      IF(SBWSNUM.EQ.7)THEN
      BEFORE='HYDRO_7.TXT'
      AFTER='RHYDRO_7.TXT'
      ENDIF
      IF(SBWSNUM.EQ.8)THEN
      BEFORE='HYDRO_8.TXT'
      AFTER='RHYDRO_8.TXT'
```

```
      ENDIF
      IF(SBWSNUM.EQ.9)THEN
      BEFORE='HYDRO_9.TXT'
      AFTER='RHYDRO_9.TXT'
      ENDIF
      IF(SBWSNUM.EQ.10)THEN
      BEFORE='HYDRO_10.TXT'
      AFTER='RHYDRO_10.TXT'
      ENDIF
C      ************************************

      IF(RESNUM.EQ.1)THEN
      DISCHARGE='DISCHARGE_1.TXT'
      STORAGE='STORAGE_1.TXT'
      RELATIONSHIP='RELATIONSHIP_1.TXT'
      ELEVATION='ELEVATION_1.TXT'
      ENDIF
      IF(RESNUM.EQ.2)THEN
      DISCHARGE='DISCHARGE_2.TXT'
      STORAGE='STORAGE_2.TXT'
      RELATIONSHIP='RELATIONSHIP_2.TXT'
      ELEVATION='ELEVATION_2.TXT'
      ENDIF
      IF(RESNUM.EQ.10)THEN
      DISCHARGE='DISCHARGE_10.TXT'
      STORAGE='STORAGE_10.TXT'
      RELATIONSHIP='RELATIONSHIP_10.TXT'
      ELEVATION='ELEVATION_10.TXT'
      ENDIF
      IF(RESNUM.EQ.11)THEN
      DISCHARGE='DISCHARGE_11.TXT'
      STORAGE='STORAGE_11.TXT'
      RELATIONSHIP='RELATIONSHIP_11.TXT'
      ELEVATION='ELEVATION_11.TXT'
      ENDIF

      FLAG=1
      INDEX=1
      OPEN (30,STATUS='OLD',FILE=ELEVATION)
      OPEN (40,STATUS='OLD',FILE=DISCHARGE)
      OPEN (50,STATUS='OLD',FILE=STORAGE)
      OPEN (60,STATUS='OLD',FILE=RELATIONSHIP)

      DO WHILE (FLAG.EQ.1)
      READ (30,*,END=101) ELE(INDEX)
      INDEX=INDEX+1
      END DO

101   INDEX=1
      DO WHILE (FLAG.EQ.1)
      READ (40,*,END=102) DIS(INDEX)
      INDEX=INDEX+1
      END DO

102   INDEX=1
      DO WHILE (FLAG.EQ.1)
```

```
      READ (50,*,END=103) STO(INDEX)
      INDEX=INDEX+1
      END DO

103   INDEX=1
      DO WHILE (FLAG.EQ.1)
      READ (60,*,END=104) REL(INDEX)
      INDEX=INDEX+1
      END DO

104   PRINT*, ('Calculation is in progress, please wait...')
C     Check if the initial condition is correct.
100   IF (INITIAL.LT.(ELE(1)-0.05).OR.INITIAL.GT.(ELE(INDEX-1)+0.05))
     *   THEN
      PRINT*, 'The initial elevation is out of reservoir range.'
      PRINT*, 'Please enter the initial condition (elevation) again...'
      READ (5,*) INITIAL
      GOTO 100
      END IF

      RESINTERVAL = INDEX - 1

      DO 105 J = 1, RESINTERVAL
C       The IF statement interpolates to get initial outflow and storage.
        IF (INITIAL.GE.ELE(J).AND.INITIAL.LE.ELE(J+1)) THEN
        S1=STO(J)+(STO(J+1)-STO(J))*(INITIAL-ELE(J))/(ELE(J+1)-ELE(J))
        Q1=DIS(J)+(DIS(J+1)-DIS(J))*(INITIAL-ELE(J))/(ELE(J+1)-ELE(J))
        GOTO 106
        END IF
105   CONTINUE

106   PRINT*, ('Calculation is in progress, please wait...')
C     File unit 10 is the input file, it contains routing input required by LEVEL POOL.
      OPEN (10,STATUS='OLD',FILE=BEFORE)
C     Initialize the input and output, I() is the LEVEL POOL input, Q() is the LEVEL POOL output.
      DO 5 INDEX=1,2880
        I(INDEX)=0
        Q(INDEX)=0
5     CONTINUE
C     Read data from the runoff txt file
      DO 6 INDEX = 1,2880
        READ (10,*) I(INDEX)
6     CONTINUE

      ADDINPUT(1)=0
      DO INDEX = 2,2880
      ADDINPUT(INDEX)=I(INDEX)+I(INDEX-1)
      END DO

      VARIABLE1(1)=2*S1/300-Q1
      VARIABLE2(1)=0
      Q(1)=Q1

C     This section is to route and print out the results
      DO INDEX = 2,2880
        VARIABLE2(INDEX)=ADDINPUT(INDEX)+VARIABLE1(INDEX-1)
```

```
      DO 107 J = 1, RESINTERVAL
C        The IF statement interpolates to get outflow.
       IF (VARIABLE2(INDEX).GE.REL(J).AND.
   *   VARIABLE2(INDEX).LE.REL(J+1)) THEN
         Q(INDEX)=DIS(J)+(DIS(J+1)-DIS(J))*
   *      (VARIABLE2(INDEX)-REL(J))/(REL(J+1)-REL(J))
         IF (Q(INDEX).GE.10000 .OR. Q(INDEX).LE.0.0001) THEN
           Q(INDEX)=0
         END IF
       GOTO 108
       END IF
107     CONTINUE

108     VARIABLE1(INDEX)=VARIABLE2(INDEX)-2*Q(INDEX)
     END DO


C     File 20 is the output file, it contains LEVEL POOL routing output.
     OPEN (20,FILE=AFTER,STATUS='UNKNOWN')
     DO 8 INDEX=1,2880
      WRITE (20,*) Q(INDEX)
8    CONTINUE
      WRITE (20,*)  0.
     CLOSE (10, STATUS='KEEP')
     CLOSE (20, STATUS='KEEP')
     CLOSE (30, STATUS='KEEP')
     CLOSE (40, STATUS='KEEP')
     CLOSE (50, STATUS='KEEP')
     CLOSE (60, STATUS='KEEP')

     PRINT 97
97   FORMAT('  ')
     PRINT 98
98   FORMAT('Calculation finished! Please close the window')
     PRINT 99
99   FORMAT('and check the output files.')

     STOP
     END
```

## A.8 KINEMATIC_WAVE.F

```
C     The program, Highway Watershed Model (HWM), is organized as followings.

C     1.  SCS curve number method is used to generate excess rainfall;
C     EXCESSRAINFALL.F
C     Parameters needed to input
C     CN -- Curve number for each sub-watershed
C     AMC -- Antecedent moisture conditions
C     2.  SCS Unit hydrograph is used to generate hydrograph at the outlet
C     HYDRO.F
C     Parameters needed to input
C     Tc -- Time of concentration of the sub-watershed
C     Area -- Area of the sub-watershed
C     3.  Muskingum method is used to route hydrograph in channel
C     MUSKINGUM.F
C     Parameters needed to input
C     K -- Muskingum coefficient
C     X -- Muskingum coefficient
C     4.  Linear reservoir or Level pool method is used to route hydrograph in reservoir
C     LINEARRESERVOIR.F (n = 1, K = time of travel)
C     Parameters needed to input
C     K -- Travel time of the reservoir
C     LEVELPOOL.F
C     Parameters needed to input
C     Initial: Initial condition of the reservoir (elevation)
C     ADD.F
C     No parameters. Only input hydrographs are needed.

C     If X = 0, the Muskingum routing method becomes linear reservoir routing method.
C     LINEARRESERVOIR.F program is similar to MUSKINGUM.F.
C     Because a hydrograph may be routed many times through channel and reservoir,
C     it is very complicated to use different names at every routing. Thus, we use
C     name HYDRO_#.TXT at the outlet of a sub-watershed and routing.
C     We use name RHYDRO_#.TXT after each routing. A hydrograph may be routed many times.
C     Each time, before routing, the hydrograph from RHYDRO_#.TXT must be changed to
HYDRO_#.TXT if needed.

C     ****************************************************************
C     Version: KINEMATIC_WAVE.F
C     Created by Weizhe An (April/11/06)

C     Input: HYDRO_#.TXT, Geometry characteristics of pipe
C     Output: RHYDRO_#.TXT
C     ******************************************************************************
C     This section is for input data
C     K is the storage constant
C     X is the wedge parameter
C     INTERV is rainfall interval (5 min) used in RUNOFF program
C     C0, C1, C2 are coefficient of Muskingum routing
C     I() is the Muskingum input, Q() is the Muskingum output.
C     They are expressed as C1,C2,C3 in the book, Applied Hydrology, Ven Te Chow, ISBN 0-07-
010810-2, P258
      REAL K,X,Q(0:2880),I(0:2880),Y(1:100)
C     VELOCITY is the result kinematic celerity, TOTALTIME = inflow time + travel time
```

```
C     SLP is the slope in interperlation linear function
C     LENGTH is the length of the pipe
      REAL VELOCITY, TOTALTIME(0:2880), TRAVELTIME(0:2880), SLP, LENGTH
      INTEGER INDEX,J
C     BEFORE, AFTER are file name representatives.
      CHARACTER*20 BEFORE,AFTER, XX(1:100)

C     Read all parameters from a file.
      OPEN(30,FILE='PARAMETER.TXT',STATUS='OLD')
        DO 20 J = 1, 100
          READ(30,*,END=21) XX(J), Y(J)
20        CONTINUE
21      LINE=J-1
      CLOSE(30, STATUS='KEEP')

      SBWSNUM=Y(72)
      K=Y(73)
      X=Y(74)
C     Select the file to read
      IF(SBWSNUM.EQ.1)THEN
      BEFORE='HYDRO_1.TXT'
      AFTER='RHYDRO_1.TXT'
      ENDIF
      IF(SBWSNUM.EQ.2)THEN
      BEFORE='HYDRO_2.TXT'
      AFTER='RHYDRO_2.TXT'
      ENDIF
      IF(SBWSNUM.EQ.3)THEN
      BEFORE='HYDRO_3.TXT'
      AFTER='RHYDRO_3.TXT'
      ENDIF
      IF(SBWSNUM.EQ.4)THEN
      BEFORE='HYDRO_4.TXT'
      AFTER='RHYDRO_4.TXT'
      ENDIF
      IF(SBWSNUM.EQ.5)THEN
      BEFORE='HYDRO_5.TXT'
      AFTER='RHYDRO_5.TXT'
      ENDIF
      IF(SBWSNUM.EQ.6)THEN
      BEFORE='HYDRO_6.TXT'
      AFTER='RHYDRO_6.TXT'
      ENDIF
      IF(SBWSNUM.EQ.7)THEN
      BEFORE='HYDRO_7.TXT'
      AFTER='RHYDRO_7.TXT'
      ENDIF
      IF(SBWSNUM.EQ.8)THEN
      BEFORE='HYDRO_8.TXT'
      AFTER='RHYDRO_8.TXT'
      ENDIF
      IF(SBWSNUM.EQ.9)THEN
      BEFORE='HYDRO_9.TXT'
      AFTER='RHYDRO_9.TXT'
      ENDIF
      IF(SBWSNUM.EQ.10)THEN
```

```
      BEFORE='HYDRO_10.TXT'
      AFTER='RHYDRO_10.TXT'
      ENDIF

C     File unit 10 is the input file, it contains routing input required by Kinematic
      OPEN (10,STATUS='OLD',FILE=BEFORE)
C     Initialize the input and output, I() is the Kinematic input, Q() is the Kinematic output.
      DO 5 INDEX=0,2880
        I(INDEX)=0
        Q(INDEX)=0
5     CONTINUE
C     Read data from the runoff txt file
      DO 6 INDEX = 0,2880
        READ (10,*) I(INDEX)
6     CONTINUE

C     This section is to route and print out the results, 5 MIN interval
      LENGTH = 26000
      Q(0)=0
      TOTALTIME(0)=0

      DO 7 INDEX = 1,2880
        CALL GETSOLUTION(I(INDEX), VELOCITY)
C     if no water, totaltime is calculated in another way
        IF (VELOCITY.EQ.0) THEN
        TOTALTIME(INDEX) = TOTALTIME(INDEX-1)+5
        TRAVELTIME(INDEX) = 0
        ELSE
        TOTALTIME(INDEX) = INDEX*5+LENGTH/VELOCITY/60
        TRAVELTIME(INDEX) = LENGTH/VELOCITY/60
        ENDIF
7     CONTINUE

C     INDEX loop is for outflow
      DO 9 INDEX = 1,2880
C     J loop is to determine the nearest outflow value
        DO 11 J = INDEX, 0, -1
          IF ((INDEX*5-TOTALTIME(J)).GE.0) THEN
            SLP = (I(J+1)-I(J))/(TOTALTIME(J+1)-TOTALTIME(J))
            Q(INDEX)=I(J)+SLP*(INDEX*5-TOTALTIME(J))
            IF (Q(INDEX).GE.10000 .OR. Q(INDEX).LE.0.0001) THEN
              Q(INDEX)=0
            ENDIF
            GOTO 9
          ENDIF
11        CONTINUE
9     CONTINUE

C     File 20 is the output file, it contains Muskingum routing output.
      OPEN (20,FILE=AFTER,STATUS='UNKNOWN')
      DO 8 INDEX=0,2880
        WRITE (20,*) Q(INDEX)
8     CONTINUE

      CLOSE (10, STATUS='KEEP')
      CLOSE (20, STATUS='KEEP')
```

```
      PRINT 98
98    FORMAT('Calculation finished! Please close the window')
      PRINT 99
99    FORMAT('and check the output files.')
      END

C     This sub-routine is to calculate the celerity
      SUBROUTINE GETSOLUTION(input, celerity)
C     input is the input discharge, celerity is the water velocity
C     considering kinematic wave, temp123 is the temp variable used.
C     diff is the difference of try and error result and actual discharge
C     min is the minimum difference value
      real input, diff, min, temp1, temp2, celerity, temp3
C     radius is the pipe radius, manning is the manning coeff
C     slope is the pipe slope, coeff and temp is the temp variable
C     theta is the angle used in calculation, result is the temporary result
C     const is the manning constant
      real radius, manning, const, slope, coeff, theta, temp, result
C     This section is to calculate the coefficients
      radius = 1.25
      manning = 0.025
      const = 1.49
      slope = (1432.5-1355.0)/260.0
      coeff = const*slope**0.5/(manning*radius**(0.6667))
      min = input

C     solve the equation by try and error
      DO 10 try = 0, 6.28, 0.01
        temp = (0.5*radius**2)*(try-SIN(try))
        result = coeff*temp**(1.6667)/try**(0.6667)
        diff = ABS(input-result)
        IF (diff.LT.min) THEN
          min = diff
          theta = try
        ENDIF
10      CONTINUE

C     after solved the theta, we can calculate the celerity
      area = 0.5*radius**2*(theta - SIN(theta))
      temp1 = 1.6667*(area/theta)**(0.6667)
      temp3 = 0.6667*(area/theta)**(1.6667)
      temp2 = temp3/(0.5*radius**2*(1-COS(theta)))
      celerity = coeff*(temp1-temp2)

      IF (input.LE.0.001) THEN
        celerity = 0
      ENDIF
      END
```

# APPENDIX B

# USER'S MANUAL OF HWM

## PREFACE

The Appendix B explains the following issues.

1. Functions of each module in HWM;
2. The required input and output files of each module in HWM;
3. The executing procedure of HWM.

Users should be advised that the order placed here is not always the real order of HWM module execution. The real order of HWM changes with the watershed delineation schematic. For I-99 highway watershed modeling, the modules EXCESSRAINFALL.F, HYDRO.F, MUSKINGUM.F, and ADD.F are executed several times. For conciseness, they are only explained one time in this manual.

## B.1 WATERSHED FORMULATION

Before HWM can be utilized, it is first required that the user identify the boundaries of the watershed, separate it into smaller sub-watersheds based on the topographical characteristics, determine the size of each sub-watershed, and identify the path that water will flow through the sub-watersheds to reach the common outlet. It is important to identify the order in which water flows through the watershed. The hydrologic methodology utilized in executing this program must be completed in a step-wise manner in order to achieve accurate results. That is, discharge contributions from various locations throughout the area cannot simply be added together to obtain the total. A complex routing of the data through the adjacent downstream basins or reservoirs is required to obtain the correct estimates of discharge.

The boundary of the watershed can be determined by several methods including utilizing topographic maps or using digital elevation model (DEM) data. In this research, topographic maps were used. Once the watershed boundary is determined, the sub-watershed should be investigated thoroughly so that different parameters can be determined for each sub-watershed. These parameters include the Curve Number (CN), Time of Concentration ($T_c$), and the Antecedent Moisture Condition (AMC). Breaking up the large drainage basin into these smaller sub-areas will make for a much more detailed and accurate model. An example of this can be seen in Figure 36 and Figure 38, where the large area has been divided into the five and seven sub-watersheds. Also note the locations of reservoirs, which will be modeled later.

Based on the general slope and topography of the area, a framework for the direction of water flow can be determined. This framework should be identified using a flow chart. For instance, Figure 45 and Figure 46 show the flow chart for the test watersheds shown in the Figure 36 and Figure 38. This framework must be followed when ordering the routing sequence.

## B.2 EXCESS RAINFALL GENERATION

Program used: EXCESSRAINFALL.F

Once the watershed framework and basic parameters are determined and recorded, the computations can be begun. The first step in this process is determining the amount of excess rainfall that results from a specific storm event. Excess rainfall is defined as the direct runoff from a precipitation event. It is the depth of water that does not infiltrate into the soil, evaporate, or get used by plant life. It flows on the surface of the watershed until it is discharged at the outlet. HWM model utilize SCS abstraction method, which is described in Section 2.1. The program code requires certain input values and gives output values. The input and output file (parameters) are shown in Table 17.

**Table 17.** Input and output of EXCESSRAINFALL.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | AMC, CN, sub-watershed number, precipitation duration | User defined input |
| RAINFALL.TXT | Half-hour incremental rainfall | User defined input |
| EXCESS_#.TXT | Half-hour incremental excess rainfall | Program defined output |

With these data and the computation scheme outlined above the program may compute its output. The HWM is set up so that a separate program is used to compute the excess rainfall for each sub-watershed. Each of these programs opens different parameters, and records its output under a different name. If the watershed contains seven sub-watersheds, five programs will run to compute excess rainfall for each of these. The output will be in half hourly incremental excess rainfall and will be numbered after the corresponding sub-watersheds. From here on, the # symbol will be used to represent the number of the corresponding sub watersheds.

## B.3 DIMENSIONLESS UNIT HYDROGRAPH (DUH) GENERATION

Program used: LINEAR_EXP_UH.F

The theory of LEUH was explained in detail in Section 7.4. Once completed, this portion of the model will generate text files containing the coordinates of the designed DUH of LEUH method. This file will be stored for later use in the following Section B.4. The input and output file (parameters) are shown in Table 18.

**Table 18.** Input and output of LINEAR_EXP_UH.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | Tr, Kr | User defined input |
| UHPROTOTYPE.TXT | DUH abscissas and coordinates | Program defined output |

## B.4 SUB-WATERSHED HYDROGRAPH GENERATION

Program used: HYDRO.F

This program is used to convert the unit hydrograph to the appropriate sub-watershed hydrograph by the event. The input and output file (parameters) are shown in Table 19. In order to determine the actual runoff hydrograph at the outlet of each watershed, the unit hydrographs must be scaled for the appropriate amount of rainfall. Since the Excess Rainfall Hyetograph (ERH) has already been determined for each of the sub watersheds, this process is simple. First, the program ensures that the time coordinates of the two graphs are same so that they are compatible. Based on the discrete convolution equation - Equation 2.14, the values of excess rainfall are multiplied with the corresponding values from the derived unit hydrograph. Each instance of excess rainfall amounts is modeled with its own UH, thus the final hydrograph is sum of each of these products, lagged by the appropriate time based on when the excess rainfall occurred.

Once completed, this portion of the model will generate text files containing the coordinates of the direct runoff hydrographs for each sub watershed. These files will be stored for later use in the program and can be viewed and analyzed at any time.

**Table 19.** Input and output of HYDRO.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | $T_c$, sub-watershed area | User defined input |
| UHPROTOTYPE.TXT | DUH abscissas and coordinates | Program defined input |
| EXCESS_#.TXT | Half-hour excess rainfall | Program defined input |
| HYDRO_#.TXT | Coordinates of the total runoff hydrographs computed at each sub watershed | Program defined output |

## B.5 MUSKINGUM ROUTING

Program used: MUSKINGUM.F

Once the outflow hydrographs have been determined at each sub watershed for a particular storm event, they must be added together in order to determine the total outflow hydrograph at the watershed outlet. However, hydrographs within a watershed cannot simply be added, they must be first routed downstream in order to account for time delays and parameters specific to the channels within each sub-watershed.

This program utilizes the Muskingum Method of channel routing in order to combine hydrographs. This method models a flood wave propagating down a channel. The detailed theory was described in Section 2.5. Based on Muskingum method parameters, an iterative method can be completed in which the inflow hydrograph will produce the coordinates of the routed outflow hydrograph.

Equation 2.19 and Equation 2.20 are executed in this program. The coefficients in the equations are calculated within the code and the initial discharge values are contained

within the input file that was generated from the previous program (HYDRO.f). That is, the inflow values into a specific reach are the outflow hydrograph coordinates from the previous watershed in the schematic.

The HWM is formulated so that there is a separate program for each reach of stream that must be routed. Before a particular watershed is modeled, one must first determine how many times the flow should be routed, and the order of this procedure. Then the programs can be executed in the correct order to ensure that flow is being modeled correctly in an upstream to downstream fashion. Also, after each channel routing, the hydrograph addition must be executed. The input and output file (parameters) are shown in Table 20.

**Table 20.** Input and output of MUSKINGUM.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | Muskingum *K*, Muskingum *X* | User defined input |
| HYDRO_#.TXT | Sub-watershed hydrograph | Program defined input |
| RHYDRO_#.TXT | Coordinates of the routed hydrograph from each sub watershed | Program defined output |

## B.6 HYDROGRAPH ADDITION

Program used: ADD.F

The purpose of ADD.f is to add the routed hydrograph to the direct un-routed hydrograph of the adjacent downstream sub-watershed. The part is relatively simple and its working mechanism is illustrated in Section 7.8. The input of the program is one or more routed hydrographs and an un-routed base hydrograph that needs to be added. The output of the program is the total hydrograph after adding. The input and output file (parameters) are shown in Table 21.

**Table 21.** Input and output of ADD.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | Routed and un-routed hydrographs number | User defined input |
| RHYDRO_#.TXT | The routed total runoff hydrographs from each sub-watershed or from any reservoir | Program defined input |
| HYDRO_#.TXT | Sub-watershed hydrograph before adding | Program defined input |
| HYDRO_#.TXT | Sub-watershed hydrograph after adding | Program defined output |

## B.7 RESERVOIR ROUTING

Program used: LEVELPOOL.F

Often storage ponds are included within a watershed in order to control runoff and sediment load. This part the HWM deals with these ponds by modeling them using the level pool method. Using this method and inflow hydrographs obtained from the previous sub-program, an outflow hydrograph will be determined for the outlet of the ponds. This outlet hydrograph may then also be routed further downstream.

In order to complete this section of the model much information must be obtained and recorded concerning the design of the storage basins. First, a text file containing incremental elevations from the bottom to the top of the pond must be created. The increments should be small enough to produce a detailed representation of the pond. Once this file is in place, other files relating discharge and storage to the selected elevations must be made. Lastly, a file containing the relationship between storage and discharge is created. All of these files have the same number of entries which is equal to the number of elevation increments that were entered. This process of relations is classically a graphical method where the storage at corresponding height is compared to discharge at the same height. This regime of curves used to develop the storage outflow relationships and perform the routing procedure is shown in Section 7.7. All of equation calculations are completed within the program LEVELPOOL.F.

Once it has finished routing the flow through the pond, the routed hydrograph must be added to the hydrograph of the adjacent downstream un-routed hydrograph. To do this the ADD.F program (Section B.6) is implemented. The input and output file (parameters) are shown in Table 22.

**Table 22.** Input and output of LEVELPOOL.F

| File | Input / Output data | Definition |
|---|---|---|
| PARAMETERS.TXT | Pond number, initial condition | User defined input |
| ELEVATION_#.TXT | Pond elevation data | User defined input |
| DISCHARGE_#.TXT | Pond discharge data | User defined input |
| STORAGE_#.TXT | Pond storage data | User defined input |
| RELATIONSHIP_#.TXT | Storage-outflow function in Figure 58 | User defined input |
| HYDRO_#.TXT | Sub-watershed hydrograph | Program defined input |
| RHYDRO_#.TXT | Routed hydrograph after storage pond | Program defined output |

## B.8 KINEMATIC ROUTING

Program used: KINEMATIC_WAVE.F

The function of this module is the same to module B.5, except that this module uses the kinematic wave method, while module B.5 uses Muskingum method. The pipe's geometric characteristics, such like pipe diameter, pipe slope, roughness, are needed in kinematic wave method. Since only one pipe routing is employed in HWM, we integrated the specific pipe's geometric data in the program KINEMATIC_WAVE.F. The detailed theory of kinematic wave is illustrated in Section 7.6. The input and output file (parameters) are shown in Table 23.

**Table 23.** Input and output of HYDRO.F

| File | Input / Output data | Definition |
|---|---|---|
| KINEMATIC_WAVE.F | Pipe diameter, pipe slope, roughness | User defined input |
| HYDRO_#.TXT | Sub-watershed hydrograph | Program defined input |
| RHYDRO_#.TXT | Coordinates of the routed hydrograph from each sub watershed | Program defined output |

## B.9 MASTER FILE

Program used: HWM.BAT

This file is a computer BATCH file instead of a FORTRAN program. It contains no calculations. The task of this file is to execute different HWM modules consecutively without human intervention. To achieve correct modeling results, HWM modules for each sub-watershed must be built and put into the same directory. The correct order of HWM modules must be placed in HWM.BAT file. If the schematic of the watershed is changed, the order of HWM modules in HWM.BAT must be changed accordingly.

The input hydrograph file of channel routing and reservoir routing module is HYDRO_#.TXT. The output hydrograph file of channel routing and reservoir routing module is RHYDRO_#.TXT. If two or more routings are connected, the input file of later routing is the output file of previous routing. Thus, the original HYDRO_#.TXT needs to be deleted and the routing output file RHYDRO_#.TXT needs to be renamed to HYDRO_#TXT to meet the input name format requirement. Since the modeled watershed contains two consecutive routings, the deleting and renaming operations are required in the BATCH file. Fortunately, these operations can be easily accomplished by DOS command *DEL* and *REN*.

## B.10 HWM EXECUTION

With all the HWM modules, HWM.BAT file, and parameter file well established, the HWM execution is simple. All that is needed is to double click the HWM.BAT file. The HWM.BAT executes each module of HWM, and output various text files. The final output file contains coordinates of final outlet hydrograph in hourly interval. For diagram view, user needs to open the final output text file and copy all values into MS Excel Spread Sheet. Although executing HWM using HWM.BAT file seems easier, the preparation works of it are much more than executing manually. Furthermore, once the automatic method is set, it can only model the fixed watershed schematic. If the

watershed schematic is changed, most HWM modules, HWM.BAT file, and parameter file have to be revised to fit the updated schematic.

If the user wants to execute the model manually, the user must determine the order of each module execution. Most parameters are input by keyboard instead of file. In case of consecutive routings, the user must delete and rename the appropriate output text files manually. The preparation works is less than automatic ways, but the total time spends in this manner is much longer than using HWM.BAT file. In model calibration process, user needs to change parameters quite often. Even if only one parameter is changed, the model must be implemented again to obtain new results. Since automatic method reads parameters from files while manual method reads parameters from keyboard, automatic execution can save much time for user, especially in model calibration process. The manual method has merit of flexibility, i.e., if the watershed schematic is changed, all HWM modules and parameter file are kept unchanged. The order of executing HWM modules should be changed, but this order is controlled by user instead of HWM modules and parameter file.

It must be pointed out that the final results, whether using HWM.BAT or executing modules separately are the same. For the same watershed schematic, the order of HWM module execution is also the same. Section 7.9 shows the execution order for Watershed One and Watershed Two. The only difference is that HWM.BAT executes all modules automatically, reads parameters from file, and can only model a fixed watershed schematic. Thus, using HWM.BAT can save much time for user, but it is less flexible.

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