MODELS

DEVELOPED
Hydrological model is a powerful tool for studying rainfall-runoff relationship. A hydrological model organizes various hydrological quantities and catchment parameters for transforming rainfall into runoff. So far researchers have concentrated on models based on unit hydrograph theory. These models are lumped in nature, do not take into account spatial and temporal variations in rainfall and physiographic characteristics of a drainage basin.

The natural processes are very complicated. Many factors affect the runoff process which makes rainfall-runoff relationship more complex. Different aspects of the hydrologic transformation process have been widely investigated to understand the complexity involved but till now no perfect model or methodology is available for universal applications to predict runoff characteristics of small to medium sized typical watersheds having typical hydrologic and physiographic characteristics. However researchers are trying to find out the best suited method for runoff prediction. Mathematical modeling of hydrologic processes provides a tool by means of which one can study the runoff process. Various methodologies and available techniques are studied and it is decided that the investigation of the watershed response be carried out in two phases. In the first phase the flood water movement overland surface is considered and in the second phase the subsequent flow in the stream is considered. It is concluded that the dynamic approaches are best to account for the physical processes associated with the runoff mechanism of the watershed. Among the various dynamic approaches the kinematic wave theory is best suited to the prevailing conditions.
4.1 Simplified Hydrological Model

Simplified Hydrological Model uses parameters which are related to the physical characteristics of the watershed in terms of the soil and vegetation cover. Model assumes three vertical tanks, representing Surface reservoir, soil reservoir and Ground water reservoir respectively. The model works on a daily basis. It uses SCS (Soil Conservation Services) method. The concept of curve numbers from the given table number 4.2 and 4.3 which depends on the land cover and soil type used in the model. Moisture accounting is continuously performed from initial conditions given to the model. At each day moisture updating of the unsaturated zone (soil reservoir) is done by computing infiltration through the SCS runoff equation. Recharge to the saturated zone (Ground water reservoir) is done using the concept of field capacity.

This model operates with daily rainfall and monthly means potential evapotranspiration input. The model utilizes a soil moisture accounting procedure that is based on two linear reservoir (surface and soil) and saturated zone. (Ground water reservoir)

4.1.1 Explanation Of Model:

1) The hydrological system of the region is expressed in the form of three vertical tanks. The first tank represent surface reservoir, second tank symbolize the soil reservoir and third tank stand for ground water reservoir.

2) WMR (Weighted Mean Rainfall) is calculated for the basin and used in the model.

3) If rainfall is not occurred no runoff will be formed. Secondly if rainfall value is less than initial abstraction, runoff will be zero.

4) If rainfall is more than initial abstraction, runoff will occur.
5) Here for runoff calculation, SCS (Soil Conservation Services) method is used. Runoff will depend on rainfall (P), Initial abstraction (IA), and Maximum Potential retention (S) and soil moisture (SM) can be found with the help of curve number. Here IA is assumed to be equal to 20% of S.

6) There are four type of land cover identified in the catchment.

   a) Agricultural land,
   b) Open Scrub
   c) Forest land
   d) Water body

Now, with the help of land record total sum of all land has been summarized, from the table no. 4.2 and 4.3 given in the SCS method, value of curve number (CN) is found for each land cover and for land type. A composite curve number found for the region. Runoff will depend on soil moisture at the beginning of each rainfall. There are 3 moisture conditions, Antecedent Moisture Condition-I, II and III. Initially the value of Curve Number for different category given for AMC-II. Before calculating runoff for particular rainfall, status of soil at particular rainfall is to be found out. AMC conditions are indicted in terms of rainfall, if rainfall for previous five days is less than 35 mm than AMC-I condition adopted and curve number accordingly modified. Same way if rainfall for previous 5 days is between 36 and 53 mm than the value given in the table is for AMC-II condition. If rainfall in previous five days is more than 53 mm than AMC-III is adopted and curve number for AMC-II selected from table and modified for AMC-III condition.
7) With the help of CN value, corresponding Maximum Potential Retention found out from the relation given

\[ S = 25.4 \left( \frac{1000}{CN} \right)^{-10} \]  

(1)

8) Initial Soil Moisture (ISM) in soil is to be assumed in the beginning.

9) Saturation Capacity (SC) of the soil also found in the beginning once only and utilized in the further calculation. In the starting of model, first value of S is calculated and with the help of relation between S and ISM given below, SC found out.

\[ SC = \frac{S}{(1 - ISM)} \]  

(2)

Where,

- \( S \) = Potential Maximum Retention in mm
- \( ISM \) = Initial Soil Moisture in fraction

10) Now, using the value of S and P in the formula,

\[ DR = \frac{(P - IA)^2}{(P - IA + S)} \]  

(3)

- \( DR \) = Direct runoff in mm
- \( P \) = Precipitation in mm
- \( IA = 0.2*S \) = Initial abstraction in mm

The Direct Runoff can be calculated for the given catchment.

11) The value of Direct Runoff coming is in millimeters which can be converted in to cumecs.

12) The runoff value is either reach at the outlet of catchment or that amount is stored in check dam.
13) After finding runoff value, rest of rainfall will either evaporate or infiltrate into soil. Amount of water entered in soil can be found by using following equation,

\[ SM = P - DR - PE \]  

(4)

Where PE is potential evapotranspiration.

14) The second tank is soil, here if no rainfall occurred than there is no income of water but soil moisture is lost at the rate of actual evapotranspiration.

15) Now in soil, water moves in the direction of gravity, soil particles hold the water against the force of gravity termed as field capacity. This field capacity depends on soil type. Sandy soil permits water to percolate downward, while in clayey soil, particles do not allow the water to pass as fast as sandy soil.

16) If moisture in soil, due to update of each rainfall and loss due to actual evapotranspiration, crosses the moisture at field capacity, percolates further and enters into ground water reservoir.

17) If soil moisture due to rain or irrigation is less than soil moisture at field capacity, water from soil lost at the rate of actual evapotranspiration. Here water will not percolate downward. i.e. There is no income to next tank.

18) The recharge value for the Ground Water reservoir can be found using following equations,

\[ Recharge \ R = (SM - SM_{fc}) \times (SM/SC) \times (CR) \]  

(5)

Where, 

- \( R \) = recharge from soil reservoir (mm)
- \( SM \) = Updated Soil moisture in the soil (mm)
- \( SM_{fc} \) = Soil moisture at filed capacity (mm)
- \( SM/SC \) = Rate of filling the soil reservoir
CR = Coefficient of recharge (mm/day)  
(Equivalent to coefficient of permeability)

19) There is a flow in the river after rainfall occurrence. Which indicate the interflow in the river i.e. delayed flow for finding the recession constant, a pair of runoff value on recession limb selected. Runoff Q(t), plotted on x axis and on same graph Q(t+1) runoff after unit time interval (here one day), plotted on y axis. Find the recession constant by taking best fit line among the selected runoff event. $k = \frac{Q(t+1)}{Q(t)}$  

20) The recession constant value is used to find the interflow/base flow in the stream.

21) In third tank, Incoming water is from soil reservoir in term of recharge. The rate depends on the ratio of SM/SC. The permeability of soil also governs the recharge into Ground water reservoir.

22) Here, Initial Ground Water Table is assumed, the value of recharge is added, the formula for finding out the interflow/base flow.

$$GRL = \frac{84.6 \times BF}{(A(1 - K_B))}$$  

where BF is the base flow, if base flow is known to us than GRL can be found out. A is the catchment area under consideration. $K_B$ is recession constant.

23) Here daily recharge value is quantified. In catchment area there is a seasonal water fluctuation data available. So, total daily recharge value summed for the season.
4.1.2 Assumptions Made In The Model:

1. Rainfall (P) occurred on the catchment is uniform.

2. In SCS method assumption is made that direct runoff is proportional to amount of rainfall remain after satisfying initial losses.

3. The above said ratio is equivalent to the ratio of actual retention to maximum potential retention.
4. Top soil is considered as storage tank, which overflows when the capacity exceeds. Zone of aeration is assumed to be a second tank having bottom outlet with raise portion which indicates field capacity. The third tank with side outlet indicates ground water reservoir with water table.

5. The soil is assumed to be of uniform characteristics.

6. The land in the catchment is divided in four categories. Agricultural, forest, open scrub & water body.

7. By calculating the area of different categories and finding the curve number for it there by composite curve number is found. (CN)

8. For initialization of model, Initial moisture content in the soil is assumed as 1.5%.

9. The average value of Potential Evapotraspiration (PE) has been taken equal to evaporation observed at Dhola Kuva, obtained from Maize Research Centre, Godhara and Machhan Nalla Irrigation Scheme Project site.

10. Initial Base Flow (IBF) is assumed as zero.

11. Initial Abstraction (IA) is assumed to be equal to 20 percent of Maximum Potential Retention (S)

12. Model is initialized by considering value of AMC-II, as value of curve number given for this condition. Then after AMC conditions have been applied as per its variations.
4.1.3 Analysis Of Model:

As described in data collection various hydrological, meteorological, agricultural, topographical etc. data related to area under study for development of model. Various information directly or derived is required to be incorporated.

Initially the curve number decided on the basis of the land record collected from region for year 2004. As far as curve number is concern, it is applied to other years by modifying it with respect to water bodies. There was not much variation in the value of curve number because the basic structure of region is not changed drastically. (Reference Larry W. Mays, “Water Resources Engineering”). Once the model is ready as explained next step is the trial and error procedure for calibrating the model. In model initial moisture content, field capacity, actual evapotraspiration, and curve number decided from the data obtained for the region by judgment.

In this model most sensitive parameters are Coefficient of Recharge and Recession constant. Coefficient of Recharge (CR) is decided by trial and error such that the simulated value falls nearer to the observed value. Recession Constant (Kₜ) is decided from the observed runoff of the region which is further modified by trial and error from good fitting of simulated values with observed values. Number of trials shows that the value of Coefficient of Recharge varies between 1 to 4.5 from past years to recent years. The value of Recession Constant varies between 0.5 to 1. The model is designed to run for any year.

4.1.4 Parameters Of SHM Model:

SHM model works with various parameters. Main parameters amongst all these are Rainfall (P), curve number (CN), Potential Evapotraspiration (PE), Initial Base Flow (IBF), Initial Abstraction (IA) Maximum Potential Retention (S), and Antecedent Moisture Conditions (AMC).
These parameters values are taken either from available data, worked out or assumed. Many parameters values are fixed by trials. Following table gives list of various parameters of SHM Model.

**Table - 4.1 : Parameters of Simplified Hydrological Model**

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>DISCRPTION</th>
<th>SYMBOL</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>Precipitation</td>
<td>P</td>
</tr>
<tr>
<td>2</td>
<td>Direct Runoff</td>
<td>DR</td>
</tr>
<tr>
<td>3</td>
<td>Potential Evapotraspiration</td>
<td>PE</td>
</tr>
<tr>
<td>4</td>
<td>Actual Evapotraspiration</td>
<td>AE</td>
</tr>
<tr>
<td>5</td>
<td>Soil Moisture</td>
<td>SM</td>
</tr>
<tr>
<td>6</td>
<td>Initial Abstraction</td>
<td>IA</td>
</tr>
<tr>
<td>7</td>
<td>Initial Soil Moisture</td>
<td>ISM</td>
</tr>
<tr>
<td>8</td>
<td>Initial Base Flow</td>
<td>IBF</td>
</tr>
<tr>
<td>9</td>
<td>Maximum Potential Retention</td>
<td>S</td>
</tr>
<tr>
<td>10</td>
<td>Saturation Capacity</td>
<td>SC</td>
</tr>
<tr>
<td>11</td>
<td>Coefficient Of Recharge</td>
<td>CR</td>
</tr>
<tr>
<td>12</td>
<td>Curve Number</td>
<td>CN</td>
</tr>
<tr>
<td>13</td>
<td>Base flow Recession Constant</td>
<td>$K_b$</td>
</tr>
<tr>
<td>14</td>
<td>Rate Of Filling The Soil Reservoir</td>
<td>$\frac{SM}{SC}$</td>
</tr>
<tr>
<td>15</td>
<td>Area Of Basin</td>
<td>A</td>
</tr>
<tr>
<td>16</td>
<td>Field Capacity</td>
<td>FC</td>
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### Table - 4.2 Recession Constants For Different Year

<table>
<thead>
<tr>
<th>YEAR</th>
<th>$K_B$</th>
<th>YEAR</th>
<th>$K_B$</th>
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<td>1994</td>
<td>0.96</td>
<td>2001</td>
<td>0.73</td>
</tr>
<tr>
<td>1995</td>
<td>0.88</td>
<td>2002</td>
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</tr>
<tr>
<td>1996</td>
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<td>2003</td>
<td>0.93</td>
</tr>
<tr>
<td>1997</td>
<td>0.69</td>
<td>2004</td>
<td>0.57</td>
</tr>
<tr>
<td>1998</td>
<td>0.89</td>
<td>2005</td>
<td>0.86</td>
</tr>
<tr>
<td>1999</td>
<td>0.68</td>
<td>2006</td>
<td>0.74</td>
</tr>
<tr>
<td>2000</td>
<td>0.77</td>
<td>2007</td>
<td>0.82</td>
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</table>
### Table - 4.3 Trial Input Values SHM

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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<th></th>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Actual Evapotranspiration</td>
<td>AE</td>
<td>mm/Day</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Initial Soil Moisture</td>
<td>ISM</td>
<td>%</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
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<tr>
<td>3</td>
<td>Field Capacity</td>
<td>FC</td>
<td>%</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Base Flow Recession Constant</td>
<td>KB</td>
<td>1/Day</td>
<td>0.9</td>
<td>0.85</td>
<td>0.88</td>
<td>0.98</td>
<td>0.5</td>
<td>0.98</td>
<td>0.68</td>
</tr>
<tr>
<td>5</td>
<td>Curve Number</td>
<td>CN</td>
<td>Number</td>
<td>81.53</td>
<td>81.53</td>
<td>80</td>
<td>80</td>
<td>81.7</td>
<td>81.7</td>
<td>80</td>
</tr>
<tr>
<td>6</td>
<td>Coefficient Of Recharge</td>
<td>CR</td>
<td>Number</td>
<td>0.5</td>
<td>1.8</td>
<td>1.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>PARAMETERS</th>
<th>ABRIVATION</th>
<th>UNIT</th>
<th>2000</th>
<th>2001</th>
<th>2003</th>
<th>2005</th>
<th>2006</th>
<th>2007</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Actual Evapotranspiration</td>
<td>AE</td>
<td>mm/Day</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>Initial Soil Moisture</td>
<td>ISM</td>
<td>%</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>Field Capacity</td>
<td>FC</td>
<td>%</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Base Flow Recession Constant</td>
<td>KB</td>
<td>1/Day</td>
<td>0.8</td>
<td>0.98</td>
<td>0.7</td>
<td>0.65</td>
<td>0.72</td>
<td>0.68</td>
</tr>
<tr>
<td>5</td>
<td>Curve Number</td>
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<td>1</td>
<td>4.22</td>
<td>2.2</td>
<td>3.1</td>
<td>4.1</td>
</tr>
</tbody>
</table>
4.1.5 Flow Chart For SHM Model

```
START

P, PE, AE, ISM, IBF, IA, FC, GWL, CN, A, KB, DAYS

35<P5<=53 OR P5=0

P5<=35

P5>53

S= 25.4*((1000/CN)-10) AND IA=0.2*S

SC=(S/(1-ISM)) AND SMFC=SC*FC

DR= ((2-IA)^2)/(P-IA+S))

SM=P-DR-PE

IF SM<=SMFC

R=(SM-SMFC)*(SM/SC)

BF=(R*A*(1-Kb))/86.4

TR=BF+DR

STOP

Figure 4.2: Flow Chart For SHM Model
```
4.2 HEC-RAS Modeling System

HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking, multi-user network environment. The system is comprised of a Graphical User Interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities. HEC-RAS software contains three one-dimensional hydraulic analysis components: (i) steady flow water surface profile computation; (ii) unsteady flow simulation; and (iii) movable boundary sediment transport computations. In addition to the three hydraulic analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed. HEC-RAS incorporates some advanced techniques, like mixed flow regime, dam break analysis and etc. The results of this model can be applied in floodplain management and flood insurance studies. The version 3.1.3 of HEC-RAS is used in this study for Steady and Unsteady Flow Water Surface Profile calculations.

4.2.1 Capabilities of HEC-RAS

Brunner et.al. (2001), described that, HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. Major hydraulic capabilities of HEC-RAS are mentioned below.

4.2.2 Steady Flow Water Surface Profiles

This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a single river reach, a dendritic system, or a full network of channels. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regime water surface profiles.
The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations include mixed flow regime calculations (i.e., hydraulic jumps), hydraulics of bridges, and evaluating profiles at river confluences (stream junction).

The effects of various obstructions such as bridges, culverts, weirs, spillways and other structures in the flood plain may be considered in the computations. The steady flow system is designed for application in flood plain management and flood insurance studies to evaluate floodway encroachments. Also, capabilities are available for assessing the change in water surface profiles due to channel improvements, and levees.

4.2.3 Unsteady Flow Simulation

This component of the HEC-RAS modeling system is capable of simulating one-dimensional unsteady flow through a full network of open channels. The unsteady flow equations solver was adapted from Dr. Robert L. Barkau’s UNET model. This unsteady flow component was developed primarily for subcritical flow regime calculations.

The hydraulic calculations for cross-sections, bridges, culverts and other hydraulic structures that were developed for the steady flow component are incorporated into the unsteady flow module. Additionally, the unsteady flow component has the ability to model storage areas and hydraulic connections between storage areas as well as between stream reaches.

4.2.4 UNET model for unsteady flow simulation element of HEC-RAS:

HEC-RAS model uses Barkau's (1995) UNET model for one-dimensional unsteady open-channel flow simulation. Barkau (2001), described that, UNET
can simulate flow in single reaches or complex networks of interconnected channels. Many types of in-channel hydraulic controls such as bridges, weirs and culverts can be modeled. Exchange of flow over levees with storage areas can be simulated.

UNET simulates one-dimensional unsteady flow through a network of open channels. One element of open channel flow in networks is the split of flow into two or more channels. Second element of a stream network is the combination of flow; which is termed the dendritic problem. In this problem, flow from each tributary is dependent only on the stage in the receiving stream. Figure 3.1 illustrates a dendritic channel system, including a full network. The system shown includes flow bifurcations, a crossing canal, a four-node junction and a storage area. UNET has the capability to simulate flow in such a system. Another capability of UNET is the simulation of storage areas; e.g., lake-like regions that can either provide water to, or divert water from, a channel.

Figure 4.3 Sample flow network.
Brunner (2001), described the methodologies used in performing the one-dimensional flow calculations within HEC-RAS. The physical laws used are principle of conservation of mass and conservation of momentum.

**Principle of mass conservation (continuity):**

"The net rate of flow into the volume is equal to the rate of change of storage inside the volume."

Considering control volume, continuity equation derived is given below

\[
\frac{\partial A_t}{\partial t} + \frac{\partial Q}{\partial x} - q_l = 0
\]

where, \(Q(x,t)\) = flow at the mid-point of control volume having length \(\Delta x\).

\(A_t\) = total flow area at the mid-point of control volume storage.

\(q_l\) = Lateral inflow entering the control volume per unit length

**Principle of conservation of momentum:**

"The net rate of momentum entering the volume (momentum flux) plus the sum of all external forces acting on the volume is equal to the rate of accumulation of momentum." Barkau (1995) considered three forces: (i) pressure (ii) gravity and (iii) boundary drag or friction force. He has obtained the final form of momentum equation:

\[
\frac{\partial Q}{\partial t} + \frac{\partial QV}{\partial x} + gA \left( \frac{\partial Z}{\partial x} + S_f \right) = 0
\]

where, \(V\) = velocity

\(g\) = acceleration due to gravity

\(A\) = cross-sectional area

\(Z\) = elevation of water surface

\(S_f\) = friction slope
While applying this unsteady flow equations within HEC-RAS, they assumed a horizontal water surface at each cross-section normal to the direction of flow; such that the exchange of momentum between the channel and the floodplain was negligible and that the discharge was distributed according to conveyance, i.e.

\[ Q_c = \Phi Q \]  

(10)

Where, \( \Phi = \frac{K_c}{K_c + K_f} \)

Where, \( k_c \) = conveyance in the channel

\( k_f \) = conveyance in the flood plain

With these assumptions, the one-dimensional equations of motion can be combined into a single set:

**Continuity equation:**

\[ \frac{\partial A_T}{\partial t} + \frac{\partial (\Phi Q)}{\partial X_c} + \frac{\partial [(1 - \Phi)Q]}{\partial X_f} - q_l = 0 \]

Momentum equation:

\[ \frac{\partial Q}{\partial t} + \frac{\partial (\Phi^2 Q^2 / A_c)}{\partial X_c} + \frac{\partial [(1 - \Phi)^2 Q^2 / A_f]}{\partial X_f} + gA_c \left[ \frac{\partial Z}{\partial X_c} + S_{xc} \right] \]

\[ + gA_f \left[ \frac{\partial Z}{\partial X_f} + S_{xf} \right] = 0 \]  

(11)

Where, c and f refer to the channel and flood plain

These equations are approximated using implicit finite differences, and solved numerically using Newton-Raphson iteration technique.
4.2.5 Boundary Conditions:

For a reach of river there are \( N \) computational nodes, which bound \( N-1 \) finite difference cells. From these cells \( 2N-2 \) finite difference equations can be developed. Because there are \( 2N \) unknowns (\( \Delta Q \) and \( \Delta Z \) for each node), two additional equations are needed. These equations are provided by the boundary conditions for each reach for subcritical flow are required at the upstream and downstream ends. For supercritical flow, boundary conditions are only required at the upstream end.

4.2.5.1 Upstream Boundary Conditions:

Upstream boundary conditions are required at the upstream end of all reaches that are not connected to other reaches or storage areas. An upstream boundary condition is applied as a flow hydrograph of discharge versus time. The equation of a flow hydrograph for reach \( m \) is:

\[
\Delta Q_k^{n+1} = Q_k^n - Q_k^n
\]  

(12)

Where \( k \) is the upstream node of reach \( m \). The finite difference form of Equation 3.5 is:

\[
MUQ_m^* \Delta dQ_k = MUB_m
\]  

(13)

where: \( MUQ_m = 1 \),

\[
MUB_m = Q_1^{n+1} - Q_1^n
\]

4.2.5.2 Downstream Boundary Conditions:

Downstream boundary conditions are required at the downstream end of all reaches, which are not connected to other reaches or storage areas. Four types of downstream boundary conditions can be specified:
4.2.6 Flow hydrograph:

A flow hydrograph may be used as the downstream boundary condition if recorded gauge data is available and the model is being calibrated to a specific flood event. At time step \((n+1)\Delta t\), the boundary condition from the flow hydrograph is given by the finite difference equation:

\[
CDZ_m \Delta Q_n = CDB_m
\]

where:

\[ CDQ_m = 1, \]

\[ CDB_m = Q_{n+1} - Q_n. \]

4.2.7 Normal Depth:

Use of Manning's equation with a user entered friction slope produces a stage considered to be normal depth if uniform flow conditions existed. Because uniform flow conditions do not normally exist in natural streams, this boundary condition should be used far enough downstream from the study area that it does not affect the results in the study area. Manning's equation may be written as:

\[
Q = k(S_f)^{0.5}
\]

where: \(K\) represents the conveyance and \(S_f\) is the friction slope.
4.2.8 HEC-RAS User Interface:

Brunner (1995) provided an introduction and overview of HEC-RAS modeling system. He described method for entering geometry data and cross-sections for entering unsteady flow data, along with boundary conditions, to view graphical and tabular output and some advanced techniques, like mixed flow regime, dam break analysis and etc.

4.2.9 Model Stability:

In order to develop a good unsteady flow model of a river system, it is necessary to understand how and why the solution of the unsteady flow equations becomes unstable. An unstable numerical model is one for which certain types of numerical errors grow to the extent at which the solution begins to oscillate, or the errors become so large that the computations cannot continue. This is a common problem when working with an unsteady flow model of any size or complexity. The following factors will affect the stability and numerical accuracy of the model:

1. Cross-section spacing.
2. Computation time step.
3. Theta weighting factor for numerical solution.
4. Solution iterations
5. Solution tolerances.
7. Weir and spillway submergence factors.

4.2.10 Cross-Section Spacing:

Cross-sections should be placed at representative locations to describe the changes in geometry. Additional cross-sections should be added at locations where changes occur in discharge, slope, velocity and roughness.
Bed slope plays an important role in cross-section spacing. Steeper slopes require more cross-sections. Streams flowing at high velocities may require more cross-sections so as to keep energy difference between two nodes in permissible limit.

### 4.2.11 Computational Time Step:

Stability and accuracy can be achieved by selecting a time step that satisfies the Courant condition:

\[ C_r = \frac{V_w}{\Delta t} \leq 1.0 \]  

(16)

Therefore:

\[ \Delta t \leq \frac{\Delta x}{V_w} \]  

(17)

Where: \( V_w \) = Flood wave speed, which is normally greater than the average velocity.

\( V \) = Average velocity of the flow.

\( \Delta x \) = Distance between cross-sections.

\( \Delta t \) = Computational time step

For most rivers the flood wave speed can be calculated as:

\[ V_w = \frac{dQ}{dA} \]  

(18)

However, an approximate way of calculating flood wave speed is to multiply the average velocity by a factor. Factors for various channel shapes are shown in table below.
Table 4.4 Factors for Computing Wave Speed From Average Velocity

<table>
<thead>
<tr>
<th>Channel Shape</th>
<th>Ratio Vw/V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wide Rectangular</td>
<td>1.67</td>
</tr>
<tr>
<td>Wide Parabolic</td>
<td>1.44</td>
</tr>
<tr>
<td>Triangular</td>
<td>1.33</td>
</tr>
<tr>
<td>Natural Channel</td>
<td>1.5</td>
</tr>
</tbody>
</table>

4.2.11.1 Practical Time Step Selection:

For medium to large rivers, the courant condition may yield time steps that are too restrictive (i.e. a larger time step could be used and still maintain accuracy and stability). A practical time step is:

$$
\Delta t \leq \frac{T_r}{20}
$$

(19)

Where: $T_r = $ Time of rise of the hydrograph to be routed.

However, you may need to use a smaller time step when you have lateral weirs/ spillways and hydraulic connections between storage areas and the river system.

4.3 Lumped Physiographic Model

A basin may be having variations in shape, physiographic parameters i.e. slope and roughness, drainage pattern of channels. For the application of kinematic wave theory to a catchment appropriate geometrical representation of catchment is very essential. Two basic elements (i) over land plane element and (ii) channel flow element are identified for analysis. Lumping of parameters of basin results in to very simplified approach.
The geometrical representation of watershed adopted for this model is simple. A single main channel of the watershed having length equal to the main drainage channel of the watershed is identified. The overland portion on both sides of the channel is the surface which feed in the main channel in perpendicular direction.

The watershed with one drainage channel and two overland planes are represented by one channel and two equivalent rectangular planes representing overland planes known as kinematic wave cascade. The average values of the physiographic parameters, overland portions, channel slopes and roughness are considered for applying to the model.
4.3.1 Formulation of the Model

Fluid flow in any situation can be studied by a set of equations known as Navier-Stoke equations and the continuity equation. All these equations are three dimensional and complicated. It is very difficult to apply them for studying rainfall-runoff relationship of a watershed. Saint Venant (1871) derived following one-dimensional hydrodynamic equations:

**Continuity equation**
\[
\frac{dA}{dt} + \frac{dQ}{dx} = q
\]  

(20)

and momentum equation
\[
\frac{1}{g} \frac{du}{dt} + \frac{u}{g} \frac{du}{dx} + \frac{dy_0}{dx} + S_f - S_0 = 0
\]  

(21)

Where,
- \(Q\) = Discharge
- \(X\) = Distance in the direction of flow
- \(U\) = component of velocity in x direction
- \(Q\) = lateral inflow per unit length
- \(S_0\) = bed slope of the stream
- \(S_f\) = energy slope
- \(G\) = acceleration due to gravity

The procedure of obtaining exact solution of the St. Venant equation is very complicated however using modern computers. It has now become possible to solve these equations by using numerical methods.

Kinematic wave equations are simplified form of the St. Venant equations. In momentum equation the derivatives of the energy and velocity terms are smaller than the gravity and frictional forces.
Lumped physiographic model has two types of elements

Overland flow elements
Subsequent channel flow elements

For overland flow, kinematic wave approximation has been used. Infiltration is computed by Green-Ampt model.

\[ \frac{\partial A}{\partial t} + \frac{\partial q}{\partial x} = q_0 \]  
(22)

\[ Q = ac.A^{mc} \]  
(23)

Where \( A = \) Cross section area of the channel flow
\( Q_0 = \) lateral inflow per unit length of the channel i.e. outflow from the overland plane which is computed by \( q_0 = a.h^m \)
\( ac, mc = \) Kinamatic wave routing parameters which depends on watershed and channel flow characteristics

\( Q_0 = \) Discharge

In channel flows the values of \( ac \) and \( mc \) depends on channel cross section, roughness coefficient of the channel (Manning's N) and longitudinal slope of the channel. The final kinematic wave equations for the channel flow are as given below

\[ \frac{dA}{dt} + \frac{d(ac.A^{mc})}{dx} = q_0 \]  
(24)

Differentiating the second term of the above equation with reference to \( X \)

\[ \frac{dA}{dt} + mc \cdot ac.A^{mc-1}. \frac{dA}{dx} = q_0 \]  
(25)

Here \( A = \) the only dependant variable which is found by solving the same equation. The discharge \( Q \) is then computed by equation (23).
For overland flow routing the second order Lax-Wendroff method have been used. For channel flow routing Mac-Cormack method is used. Mac-Cormack (1969) derived a simple variation of Lax-Wendroff method. This method is widely recognized for the solution of hyperbolic equations. It is second order accurate in space and time and uses a time splitting technique.

4.3.2 Initial and Boundary Conditions

Kinematic wave equations are solved by explicit finite difference method. For overland flows the initial conditions refer to the depth of surface runoff at the instant of beginning of the generation of the runoff by rainfall excess (at t=0).

Hence the initial condition is

\[ q_0(x,0) = 0.0 \]
\[ h_0(x,0) = 0.0 \]

Basin under study is bounded by 'divide', no flow across the divide is possible. Hence upstream boundary conditions adopted for overland flow are:

\[ q_0(0,t) = 0.0 \]
\[ h_0(0,t) = 0.0 \quad \text{for all } t \]

4.3.3 Finite Difference Formulations of Channel Flow Equations

For present study off-centered method is used for channel flow routing. Ponce et al (1979) have derived three fully off-centered methods which are explicit finite difference methods. These methods are classified according to forward and backward space and time steps used. They are as follows:
Sr. Characteristics Method
No.
1 Forward in time and backward in space I
2 Backward in time and forward in space II
3 Backward in time and space III

The criteria of stability and convergence for these three methods is lined out by Ponce (1979, 1986). Methods I and II are conditionally stable and convergent to analytical solution as the Courant number approaches 1.

Method I is stable for Courant number less than or equal to 1. Method II is stable for Courant number greater than or equal to 1. Method I is mirror image of Method II. Method III is unconditionally stable but non-convergent for any values of Courant number. For present study fully off-centered method III is adopted for channel flow routing.

4.3.4 Physiographic Parameters and Their Estimation

The values of following physiographic parameters are required for the application of this model for surface flow simulation:

1. Overland slope
2. Overland roughness
3. Channel bed slope
4. Channel roughness

4.3.4.1 Determination of Overland slope

The overland slope is determined at right angle to the channel which is being fed by the respective overland plane. For determining the overland slope of the basin Grid-contour method is adopted. The method is as follows:

1) Draw horizontal line on topographic or contour map along the main channel from the basin outlet to the farthest upstream point such that the basin is divided into two equal halves.
2) Draw perpendicular lines at suitable uniform interval on the horizontal axis of maximum length.

3) Draw lines parallel to maximum length axis intersecting the perpendiculars drawn on the maximum length axis.

4) Extend all lines to intersect the basin boundary in all directions.

5) The slope of each grid is computed by using following relation:

\[ S_g = \frac{N \cdot C}{L_g} \]

Where, \( S_g \) is Slope of each grid line
- \( C \) = Contour interval
- \( N \) = Number of contour lines that intersects the grid line
- \( L_g \) = Length of the grid line

6) Slopes in horizontal and vertical directions \( S_x \) and \( S_y \) by averaging the slopes of all parallel grid lines in that direction.

7) Mean slope is calculated by following relation:

\[ S_0 = \left( S_x^2 + S_y^2 \right)^{0.5} \]

\( S_0 \) = Mean overland slope, \( S_x \) & \( S_y \) are average grid slopes in X and Y directions respectively.

**4.3.4.2 Determination of Overland roughness**

Manning's roughness coefficient \( N \) is used as overland roughness coefficient and channel roughness coefficient. Average overland roughness coefficient is determined by trial and error.
4.3.4.3 Determination of Channel bed slope

For determining channel bed slope two reference points are marked on the channel on the index map or topographical map. Then the elevation and distance between two marked points are found. The channel bed slope is calculated by following equation:

\[
\text{Channel bed slope} = \frac{\text{Diff. in elevation between two ref. points}}{\text{Distance between them}}
\]

4.3.4.4 Determination of Channel roughness coefficient

Manning's roughness coefficient \( N \) is used as channel roughness coefficient for channel flow routing. From the available information on the channel bed characteristics its value can be fixed up or decided.

Table 4.5 The Lumped Physiographic Parameter Values For Hathmati Basin

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Area</td>
<td>1174 sq. km.</td>
</tr>
<tr>
<td>2</td>
<td>Overland Plane</td>
<td></td>
</tr>
<tr>
<td>(1)</td>
<td>Average Length</td>
<td>12889 m</td>
</tr>
<tr>
<td>(2)</td>
<td>Average Slope</td>
<td>0.018</td>
</tr>
<tr>
<td>3</td>
<td>Channel</td>
<td></td>
</tr>
<tr>
<td>(1)</td>
<td>Average Length</td>
<td>97340 m</td>
</tr>
<tr>
<td>(2)</td>
<td>Average Slope</td>
<td>0.0061</td>
</tr>
<tr>
<td>(3)</td>
<td>Average Roughness</td>
<td>0.036</td>
</tr>
</tbody>
</table>