6.1 VARIABLE PARAMETER DIFFUSION MODEL

The results of routing with the variable parameter diffusion model are presented in Table 6.1 for Narmada and Table 6.2 for Cauvery. For a visual comparison, the observed and routed hydrographs are presented in Figure 6.1 for two floods each of Narmada and Cauvery. A perusal of Table 6.1 shows that as far as the peak flow prediction is concerned, the routing is successful in some floods and not very good in some others. The prediction of time of occurrence of flood seems to be fairly all right in all floods except in floods NA2 and NA6. Table 6.2 shows that the Cauvery floods have been routed fairly well by this model.

In view of the fact that the $Q_p \alpha$ and $Q_p \bar{C}$ curves used in this routing have been drawn by a trial process, one should not be too optimistic of the resulting solutions. On this score, it can now be said that the data $Q_p \alpha$ and $Q_p \bar{C}$ prepared for both Narmada and Cauvery can be taken to be fairly representative of the respective reaches. The results are encouraging in the respect that they prove the possibility of preparing the data curves even in the absence of flood plain inundation details.

A comparison of these results with those of storage routing methods can be found in the proceedings of the National Symposium on Hydrology [82]. In this study a comparison of the results of the VPD method with those of Muskingum methods is presented later, when a general comparison of all the methods is made in Section 7.3.
<table>
<thead>
<tr>
<th>Flood Number</th>
<th>Observed peak outflow (cumecs)</th>
<th>Computed peak outflow (cumecs)</th>
<th>Error in peak flow (per cent)</th>
<th>Standard deviation of errors in Q (per cent)</th>
<th>Error in time of occurrence (hours)</th>
<th>Error in mass conservation (per cent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA1</td>
<td>44880</td>
<td>45540</td>
<td>-1.49</td>
<td>28.71</td>
<td>-1</td>
<td>10.02</td>
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<tr>
<td>NA2</td>
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<td>27750</td>
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<td>14.26</td>
<td>2</td>
<td>12.27</td>
</tr>
<tr>
<td>NA3</td>
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<td>37330</td>
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<td>13.91</td>
<td>0</td>
<td>12.55</td>
</tr>
<tr>
<td>NA4</td>
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<td>15265</td>
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<td>NA6</td>
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<td>5.62</td>
<td>43.77</td>
<td>36</td>
<td>38.83</td>
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<tr>
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<tr>
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<td>23120</td>
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<td>17.21</td>
<td>6</td>
<td>16.93</td>
</tr>
<tr>
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<td>20545</td>
<td>21762</td>
<td>16.59</td>
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<td>4.91</td>
</tr>
<tr>
<td>NA10</td>
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<td>39782</td>
<td>15.02</td>
<td>46.03</td>
<td>6</td>
<td>35.21</td>
</tr>
<tr>
<td>Flood Number</td>
<td>Observed peak outflow (cumecs)</td>
<td>Computed peak outflow (cumecs)</td>
<td>Error in peak flow (per cent)</td>
<td>Standard deviation of errors in Q (per cent)</td>
<td>Error in time of occurrence of peak (hours)</td>
<td>Error in mass conservation (per cent)</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------------------------</td>
<td>-------------------------------</td>
<td>-------------------------------</td>
<td>---------------------------------------------</td>
<td>---------------------------------------------</td>
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<tr>
<td>CY1</td>
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<td>2564</td>
<td>0.00</td>
<td>17.17</td>
<td>-4</td>
<td>-11.63</td>
</tr>
<tr>
<td>CY2</td>
<td>2331</td>
<td>2398</td>
<td>-2.90</td>
<td>29.01</td>
<td>0</td>
<td>-23.83</td>
</tr>
<tr>
<td>CY3</td>
<td>638</td>
<td>716</td>
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<td>13.25</td>
<td>-4</td>
<td>-7.19</td>
</tr>
<tr>
<td>CY4</td>
<td>908</td>
<td>980</td>
<td>-7.35</td>
<td>47.27</td>
<td>-7</td>
<td>5.47</td>
</tr>
<tr>
<td>CY5</td>
<td>4140</td>
<td>4460</td>
<td>-7.80</td>
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<td>2</td>
<td>1.17</td>
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<td>0.64</td>
</tr>
<tr>
<td>CY7</td>
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<td>9.96</td>
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<td>-8.50</td>
</tr>
<tr>
<td>CY8</td>
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<td>17.64</td>
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<td>7.71</td>
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<td>3871</td>
<td>17.80</td>
<td>22.87</td>
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<td>6.98</td>
</tr>
<tr>
<td>CY10</td>
<td>1925</td>
<td>1726</td>
<td>10.40</td>
<td>15.97</td>
<td>1</td>
<td>7.39</td>
</tr>
</tbody>
</table>

Table 6.2 Routing with VPD method - Cauvery
Figure 6.1
Figure 6.1 Observed and computed hydrographs (VPD method)
6.2 ROUTING WITH IMPULSIVE METHOD OF NARMADA

6.2.1 Grid size of the implicit scheme

The grid size of the implicit scheme is defined by the values adopted for $\Delta x$ and $\Delta t$ in the finite difference scheme formulated in the x-t plane. Obviously these values must be as low as possible for the truncation errors to be minimum. Also, for an implicit routing to be accurate, the grid size must satisfy the condition that $\Delta x/\Delta t \approx V_w$. This condition may impose a strong restriction on the choice of the length of subreaches and the time of routing periods. Obviously small subreach lengths resulting in a large number of space sections, with routing periods of low values of time have to be adopted. This would necessarily increase the computer storage and the computational costs.

However, the effect of the grid on the real time routing conditions may be influenced by other extraneous factors like irregularity of geometry and also by the assumed hydraulic characteristics of the reach. With one of the objectives of this study being to economise the computation without serious loss of accuracy, it was decided to proceed with the aim of choosing large grid sizes, to make the best use of the advantage of the implicit scheme, viz. unconditional stability.

In order to get a feel of the performance of routing as related to the grid size, the flood NA4 was tested for implicit routing with two different $\Delta x$ values. The routing was carried out as follows.

i. The Manning's $n$ value was assumed to be 0.040.

ii. The reach was divided into 12 subreaches (i.e., 13 space sections; Table 4.5) and the routing period was kept at three hours.
ill. The reach was divided into 3 subreaches (i.e., 4 space sections; Table 4.6) and the routing period was kept again at three hours.

The routings were carried out and the resulting computed hydrographs are shown in Figure 6.2. It is seen that both the computed hydrographs compare well with the observed one. The peak is predicted better in the 4 section routing, while the 13 section routing presents a better accuracy in the agreement in the rising limb portion of the hydrograph. If a margin could be allowed for observational errors, it may be concluded that both approaches give nearly the same results.

In the above test, only Δx values were varied and Δt value was three hours in both cases. In order to confirm that large grid sizes also would route fairly accurately, the procedure was further carried out adopting 13 and 4 sections for three different routing periods viz. one hour, three hours and six hours. Flood NA4 with a low peak discharge of 15,130 cumecs and flood NA3 with a comparatively high peak discharge of 34,169 cumecs were tested.

In order to reduce the routing errors to a minimum, in each of these cases the Manning's n was optimized adapting the Influence Coefficient Algorithm. The results obtained are presented in Tables 6.3 and 6.4 for the two floods. In all six grid sizes were tested. They were (13 sections, 1 hour), (13 sections, 3 hours), (13 sections, 6 hours), (4 sections, 1 hour), (4 sections, 3 hours) and (4 sections, 6 hours). Table 6.3 compares the results of routing with these six grid sizes for flood NA4. It is interesting to note the following:

i. Manning's n optimizes to more or less same values in all the three cases involving 13 sections.

ii. In the 4 section routing also the parameter is optimized to very nearly the same values in 1 hour and 3 hours.
Figure 6.2 Comparison of 13 and 4 sections (Flood NA4)

Legend
- - - - Observed
- - - - Computed (13 sections)
- - - - Computed (4 sections)

Figure 6.2 Comparison of 13 and 4 sections hydrograph (Flood NA4)
Table 6.3 Comparison of different grid sizes in implicit routing - Flood NA4, Narmada

<table>
<thead>
<tr>
<th>Routing intervals</th>
<th>Optimized Manning's n (x 10^-2)</th>
<th>Computed peak flow (cumecs)</th>
<th>Error in peak flow (per cent)</th>
<th>Standard deviation of error in peak flow</th>
<th>Computed peak depth (m)</th>
<th>Error in peak depth (per cent)</th>
<th>Standard deviation of errors in peak depth</th>
<th>Error in occurrence in peak depth (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13 sections:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.182789</td>
<td>15475</td>
<td>-2.280</td>
<td>12.635</td>
<td>13.047</td>
<td>-1.217</td>
<td>6.289</td>
<td>5</td>
</tr>
<tr>
<td>3</td>
<td>0.181548</td>
<td>15607</td>
<td>-3.152</td>
<td>12.801</td>
<td>13.108</td>
<td>-1.691</td>
<td>6.422</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>0.175282</td>
<td>15455</td>
<td>-2.134</td>
<td>12.715</td>
<td>13.037</td>
<td>-1.140</td>
<td>6.302</td>
<td>6</td>
</tr>
<tr>
<td>4 Sections:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.119708</td>
<td>14871</td>
<td>1.711</td>
<td>11.270</td>
<td>12.771</td>
<td>0.923</td>
<td>5.487</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>0.127246</td>
<td>14883</td>
<td>1.657</td>
<td>11.570</td>
<td>12.777</td>
<td>0.876</td>
<td>5.756</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>0.165017</td>
<td>15128</td>
<td>0.013</td>
<td>12.464</td>
<td>12.889</td>
<td>0.007</td>
<td>6.054</td>
<td>6</td>
</tr>
</tbody>
</table>

Observed peak flow = 15,130 cumecs
Observed peak depth = 12.890 m
Table 6.4 Comparison of different grid sizes in implicit routing - Flood NA3, Narmada

Observed peak flow = 34,679 cumecs  
Observed peak depth = 20.260 m

<table>
<thead>
<tr>
<th>Routing Intervals</th>
<th>Optimized Manning’s $n$ $(x 10^{-3})$</th>
<th>Computed peak flow (cumecs)</th>
<th>Error in peak flow (per cent)</th>
<th>Standard deviation of percentage error in discharge (m)</th>
<th>Computed peak depth (m)</th>
<th>Error in peak depth (per cent)</th>
<th>Standard deviation or errors in depth (hours)</th>
<th>Error in occurrence in peak depth (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13 Sections:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.102652</td>
<td>37833</td>
<td>-9.095</td>
<td>6.984</td>
<td>21.246</td>
<td>-4.867</td>
<td>3.477</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0.102774</td>
<td>37533</td>
<td>-8.230</td>
<td>8.229</td>
<td>21.153</td>
<td>-4.408</td>
<td>4.027</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>0.104190</td>
<td>38031</td>
<td>-9.666</td>
<td>7.201</td>
<td>21.360</td>
<td>-5.429</td>
<td>3.594</td>
<td>0</td>
</tr>
<tr>
<td>4 Sections:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Failed</td>
<td>35608</td>
<td>-2.679</td>
<td>16.177</td>
<td>20.551</td>
<td>-1.436</td>
<td>8.536</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>0.174078</td>
<td>35508</td>
<td>-2.733</td>
<td>14.950</td>
<td>20.557</td>
<td>-1.466</td>
<td>7.921</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>0.169806</td>
<td>35627</td>
<td>-2.733</td>
<td>14.950</td>
<td>20.557</td>
<td>-1.466</td>
<td>7.921</td>
<td>6</td>
</tr>
</tbody>
</table>

...
routing. The value of in 6 hours routing is higher than in the other two and is nearer to the values in the 13 section routing.

iii. In the 13 section routing there is not much difference in the values of standard deviation of percentage errors in discharge, as also in the 4 section routing. However, the 4 section routing produces slightly lower values of standard deviation of percentage errors in discharge.

iv. The error in peak flow is very low in all cases and is lower in 4 section routing.

v. The observations regarding the peak flow error and standard deviation hold good in analysing these factors with reference to the depth of flow also.

Figures 6.3 and 6.4 compare the 1 hour, 3 hours and 6 hours routing or the 13 section approach and for the 4 section approach respectively for the flood N.W. They show, as found from Table 6.3, that the accuracy of routing is similar amongst the different grid sizes used. In fact, except for the rising limb in all other parts of the hydrographs, the 4 section routing itself seems to be superior. One may infer that with the Manning’s n optimized, even the 4 section approach may give suitable results. A perusal of Table 6.4 confirms the above inference, particularly with respect to the prediction of peak flow. However, the standard deviation of errors in 4 section routing shows higher values. There also seems to be a higher error in predicting the time of peak flow. The two effects may be interrelated. However, the maximum error in the prediction of time of occurrence of peak is equal to six hours, which is only equal to the routing period.

It may now be accepted that if the prediction of the peak flow is the main criterion, the 4 section approach itself is sufficient,
Figure 6.3 Comparison of 13 section routing (Flood NAA)

Legend
- Observed
- x x x 1 Hour routing
- o o o 3 Hours routing
- △ △ △ 6 Hours routing

Figure 6.4 Comparison of 4 section routing

Legend
- Observed
- x x x 1 Hour routing
- o o o 3 Hours routing
- △ △ △ 6 Hours routing
saving enormous computer storage and time. Adopting larger numbers of sections may not always satisfy the requirements of greater accuracy because the more the sections and survey work, the more the chances of committing observational errors. Dividing a reach into just a few subreaches may itself suffice where geometric data reliability is questionable, if Manning's $n$ is properly chosen.

6.2.2 Optimizing the parameters

Having found that the 4 section approach in implicit routing provides results of accuracy similar to the ones obtained in 13 section approach, it was decided to use a (4 sections, 6 hours) grid size to test the routing of the floods in Narmada. This grid size would require the minimum computer storage and would be least costly of all the grid sizes tested in Section 6.2.1 above.

However, it was decided to optimize the model parameters instead of routing the floods for arbitrary values of the parameters so that (i) errors in routing could be minimized and (ii) the optimized parameters can be used for developing a relationship to enable their identification for future floods (as was done in the case of KGSE methods in Chapter 5). The model parameters chosen to be optimized were the Manning's $n$ and the power of $R$ in the friction slope equation

$$S_f = \frac{n^2 Q^2}{A^2 R^{4/3}} \quad \ldots \ (6.1)$$

The parameter Manning's $n$ is necessarily a function of discharge and distance. Thus an implicit scheme should provide for $n$ as $f(Q, x)$ in the computation program. However, this may pose a practical difficulty. For example, the value of $n$ in the Narmada is found to vary with discharge and distance as in Table 6.5. These values were determined in a backwater study conducted in the reach, see Reference [83]. It is seen that the value varies from 0.0210.
### Table 6.5 Variation of n in Narmada reach

<table>
<thead>
<tr>
<th>Reach chainage km</th>
<th>Q in 1,000 cumecs</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.2</td>
</tr>
<tr>
<td>56.693 - 59.741</td>
<td>0.052</td>
</tr>
<tr>
<td>59.741 - 68.885</td>
<td>0.064</td>
</tr>
<tr>
<td>68.885 - 87.173</td>
<td>0.070</td>
</tr>
<tr>
<td>87.173 - 108.509</td>
<td>0.040</td>
</tr>
<tr>
<td>108.509 - 227.686</td>
<td>0.099</td>
</tr>
<tr>
<td>227.686 - 300.838</td>
<td>0.095</td>
</tr>
<tr>
<td>300.838 - 325.405</td>
<td>0.069</td>
</tr>
</tbody>
</table>

Chainage 56.693 km - Mortakka
Chainage 325.405 km - Garudeshwar
to 0.099 and the variation is not uniform either with reference to the discharge or with reference to the distance along the reach.

Introducing such variations in discharge form in the implicit model program will consume more storage space and computer time. It should also be seen that these values of \( n \) may not route the floods because, the parameter \( n \) in the implicit model is not merely the Manning's \( n \) but it is a parameter which should adjust itself for the irregularities in the channel section. Thus, one will have to arrive at the range of \( n \) values to be used in routing often by a trial process. For example, in this study, the floods could not be routed when many of the \( n \) values noted in Table 6.5 were used. In fact as already mentioned in Section 4.5.1 the routing was successful only for \( 0.035 \leq n \leq 0.050 \). So, it is clear that for any routing, particularly for the one with large grid sizes, it is better to optimize the \( n \) for the whole reach and the whole flood, to obtain the possible accuracy in routing.

It was also felt that the routing accuracy can be improved if another parameter could be optimized. The power of hydraulic radius \( R \) in Equation (6.1), being empirical in nature, was considered for optimization. As the Manning's \( n \) occurs as \( n^2 \) in Equation (6.1) the computer program was written so as to optimize this \( n^2 \), hereafter denoted as \( S_n \). The other parameter, the power of \( R \) is denoted as \( p_R \). These two parameters were then optimized for minimum errors in implicit routing using a finite difference scheme employing 4 sections in the reach and six hours for routing period.

The Influence Coefficient Algorithm was adapted to optimize these parameters also. This optimization procedure requires to fix the gradient along which the search will proceed. For fixing this gradient one has to impart a unit change to the parameter (See Appendix 2). It was found during the attempt that it was very difficult to arrive at this value for both the parameters simultaneously. Both parameters being very sensitive to routing, the optimization often failed in two
or three function evaluations itself. Hence it was decided to optimize one parameter only at a time.

The parameter $S_n$ was optimized first. Its initial estimate was taken as $1.6 \times 10^{-3}$ (corresponding to $n = 0.04$). The unit change to be imparted to this parameter ranged from 10% to 16% of the initial value and one has to make a large number of trials before arriving at this gradient. Of the ten floods chosen for routing, two viz., NA1 and NA8 could not be routed for any value of $n$ at all. Of the remaining eight floods NA10 was not amenable for optimization. However, it was reserved as a test flood. The remaining seven floods were tested for this optimization procedure. The sum of the squares of errors in discharge was minimized and the stopping criterion was a tolerance of 0.001 for the standard deviation in percentage errors in discharge. Table 6.6 provides an idea of the performance of the Influence Coefficient Algorithm in optimizing $S_n$ for the flood NA3. The reduction in standard deviation is noted.

The parameter $P_R$ was taken up for optimization with the initial estimate of 1.33333 (which is as in Manning’s equation). For this procedure the value of $S_n$ was arbitrarily kept at $1.6 \times 10^{-3}$. Table 6.7 presents how the optimization was proceeded with for the flood NA3. The standard deviation values did reduce; nevertheless the reduction is not as much as in optimizing $S_n$ with $P_R = 1.3333$.

As the third step, the parameter $P_R$ was optimized using the optimized values of $S_n$ for the respective floods. This procedure did reduce the values of standard deviation albeit to a very small extent only, vide Table 6.8 for flood NA3.

6.2.3 Identification of parameters $S_n$ and $P_R$

The parameters $S_n$ and $P_R$ optimized as explained in the foregoing Section were then related to peak inflows, $Q_p$ in the reach.
### Table 6.6 Optimization of $S_n$ Flood NA3, Narmada

<table>
<thead>
<tr>
<th>Iteration Number</th>
<th>$S_n (x 10^{-3})$</th>
<th>$PR$</th>
<th>Standard deviation in the $S$ errors in $Q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.600000</td>
<td>4.0/3.0</td>
<td>15.685</td>
</tr>
<tr>
<td>1</td>
<td>1.668668</td>
<td>4.0/3.0</td>
<td>15.144</td>
</tr>
<tr>
<td>2</td>
<td>1.696307</td>
<td>4.0/3.0</td>
<td>14.959</td>
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<tr>
<td>3</td>
<td>1.698343</td>
<td>4.0/3.0</td>
<td>14.949</td>
</tr>
<tr>
<td>4</td>
<td>1.698004</td>
<td>4.0/3.0</td>
<td>14.951</td>
</tr>
<tr>
<td>5</td>
<td>1.698086</td>
<td>4.0/3.0</td>
<td>14.950</td>
</tr>
</tbody>
</table>

### Table 6.7 Optimization of $P_R$ with $S_n = 1.6 \times 10^{-3}$ - Flood NA3, Narmada

<table>
<thead>
<tr>
<th>Iteration Number</th>
<th>$S_n (x 10^{-3})$</th>
<th>$P_R$</th>
<th>Standard deviation in the $S$ errors in $Q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.600000</td>
<td>1.33333</td>
<td>15.685</td>
</tr>
<tr>
<td>1</td>
<td>1.600000</td>
<td>1.30975</td>
<td>15.105</td>
</tr>
<tr>
<td>2</td>
<td>1.600000</td>
<td>1.30811</td>
<td>15.078</td>
</tr>
<tr>
<td>3</td>
<td>1.600000</td>
<td>1.30831</td>
<td>15.081</td>
</tr>
<tr>
<td>4</td>
<td>1.600000</td>
<td>1.30827</td>
<td>15.080</td>
</tr>
</tbody>
</table>
Table 6.8 Optimisation of $P_N$ with optimised values of $S_n$ Flood Narmada

<table>
<thead>
<tr>
<th>Iteration Number</th>
<th>$S_n$ (x $10^{-3}$)</th>
<th>$P_R$</th>
<th>Standard deviation of the % errors in $Q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.698062</td>
<td>1.33333</td>
<td>14.950</td>
</tr>
<tr>
<td>1</td>
<td>1.698062</td>
<td>1.33264</td>
<td>14.938</td>
</tr>
<tr>
<td>2</td>
<td>1.698062</td>
<td>1.33215</td>
<td>14.932</td>
</tr>
<tr>
<td>3</td>
<td>1.698062</td>
<td>1.33184</td>
<td>14.927</td>
</tr>
<tr>
<td>4</td>
<td>1.698062</td>
<td>1.33155</td>
<td>14.923</td>
</tr>
<tr>
<td>5</td>
<td>1.698062</td>
<td>1.33137</td>
<td>14.920</td>
</tr>
<tr>
<td>6</td>
<td>1.698062</td>
<td>1.33124</td>
<td>14.918</td>
</tr>
</tbody>
</table>

...
Such a relationship will provide a means to identify the routing parameters for a future flood. Tables 6.9 to 6.11 show the inflow peaks and the respective optimized parameters. Polynomials could be fitted to these sets of values; the fitted curves and their relevant equations are shown in Figure 6.5. In each of these curves the regression was made by grouping the various points into two clusters - (i) points at low peak flows which show a tendency to slope down from high n values to comparatively low n values and (ii) points at high peak flows which tend to almost constant or slightly rising values of n. The peak flow of 21,000 cumecs is the point beyond which the values of n do not change much.

6.2.4 Parameter and peak inflow

Though it is true that the accuracy of routing is improved when the parameters $S_n$ and $P_R$ are optimized, the improvement is found to be only marginal. For example in flood NA3 (Tables 6.6 to 6.8) the standard deviation values improved from 15.685 to 14.918. It was also found that the error in peak flow improved from 3.63 % for initially estimated values of $S_n$ and $P_R$ to 2.73 % for optimized values of $S_n$ and $P_R$. Similar results were found in other floods also. Though the improvement is not substantial, it should be noted that the optimization procedure is necessary if one has to have an equation relating the parameter with $Q_p$ for identifying the parameter for a future flood. The curves obtained in this study should serve that purpose well.

In Figure 6.5 two observations are noteworthy. It is generally known that Manning's n varies with Q; for low values of Q, n is high and vice versa. Similar relationship seems to hold good for optimized n and peak inflow Q also. However, optimized n is not strictly Manning's n but is a lumped parameter taking care of Manning's n as also computational and probably geometry errors.
### Table 6.9 Optimized values of $S_n$ for $P_R = 4/3$

<table>
<thead>
<tr>
<th>Flood Number</th>
<th>Inflow peak (cumecs)</th>
<th>Optimized $S_n \times 10^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA2</td>
<td>21000</td>
<td>1.600000</td>
</tr>
<tr>
<td>NA3</td>
<td>37000</td>
<td>1.698080</td>
</tr>
<tr>
<td>NA4</td>
<td>14000</td>
<td>1.650388</td>
</tr>
<tr>
<td>NA5</td>
<td>28470</td>
<td>1.637550</td>
</tr>
<tr>
<td>NA6</td>
<td>12480</td>
<td>1.822950</td>
</tr>
<tr>
<td>NA7</td>
<td>16000</td>
<td>1.649355</td>
</tr>
<tr>
<td>NA9</td>
<td>18300</td>
<td>1.600000</td>
</tr>
</tbody>
</table>

### Table 6.10 Optimized values of $P_R$ for $n = 0.040$

<table>
<thead>
<tr>
<th>Flood Number</th>
<th>Inflow peak (cumecs)</th>
<th>Optimized $S_n \times 10^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA2</td>
<td>21000</td>
<td>1.257133</td>
</tr>
<tr>
<td>NA3</td>
<td>37000</td>
<td>1.308270</td>
</tr>
<tr>
<td>NA4</td>
<td>14000</td>
<td>1.389080</td>
</tr>
<tr>
<td>NA5</td>
<td>28470</td>
<td>1.306630</td>
</tr>
<tr>
<td>NA7</td>
<td>16000</td>
<td>1.292561</td>
</tr>
<tr>
<td>NA9</td>
<td>18300</td>
<td>1.252060</td>
</tr>
</tbody>
</table>
Table 6.11 Optimized values of $P_R$ for optimized $S_n$ values

<table>
<thead>
<tr>
<th>Flood Number</th>
<th>Inflow peak (cumecs)</th>
<th>Optimized $P_R$</th>
<th>$S_n \times 10^{-3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA2</td>
<td>21000</td>
<td>1.257133</td>
<td>1.600000</td>
</tr>
<tr>
<td>NA3</td>
<td>37000</td>
<td>1.331244</td>
<td>1.698062</td>
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<tr>
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<td>14000</td>
<td>1.403272</td>
<td>1.650388</td>
</tr>
<tr>
<td>NA5</td>
<td>28470</td>
<td>1.333333</td>
<td>1.635500</td>
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<tr>
<td>NA6</td>
<td>12480</td>
<td>1.403930</td>
<td>1.823601</td>
</tr>
<tr>
<td>NA7</td>
<td>16000</td>
<td>1.304356</td>
<td>1.649355</td>
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<tr>
<td>NA9</td>
<td>18500</td>
<td>1.252060</td>
<td>1.600000</td>
</tr>
</tbody>
</table>
Figure 6.5 Polynomials for Implicit method
The second observation relates to \( Q \) Vs \( S \) curve forming two parts - a descending part till \( Q_p = 21,000 \) cumecs and a flatter part for \( Q_p > 21,000 \) cumecs. This may be interpreted as follows. The river acts as a simple channel up to 21,000 cumecs of flow as \( n \) varies clearly with \( Q_p \). The river flows overbank beyond 21,000 cumecs. The flow with flood plains inundated more or less makes the velocity constant even for higher discharges. The value of \( n \) being related to velocity, remains nearly constant beyond 21,000 cumecs of flow. This argument may look tenuous, but it is interesting to compare Figure 6.5 with Figures 5.5 to 5.7 obtained for the KGSE-2C, 3C and C3 models. In Figures 5.5 to 5.7 the Muskingum storage coefficient \( K \) is indirectly related with \( Q_p \) for this same reach and floods. These figures show that beyond 22,000 cumecs of flow, the flood plains influence the flow (see Section 5.2.4).

6.2.5 Discussion on results of implicit routing in Narmada

The floods were then routed employing the parameters identified from Figure 6.5 for their respective inflow peaks. The results are presented in Table 6.12. This table presents the results of all floods (used for calibration as well as for testing) for reasons explained earlier in Section 5.2.5. This table can be read with Figure 6.6 which presents in graphical form the results of routing the floods NA4, NA5, NA7 and NA9, and the following inferences can be made.

(i) The three alternatives of optimization, viz. optimizing \( n \), keeping \( P_R \) constant at 4/3, optimizing \( P_{R'} \) keeping \( n \) constant at 0.040 and optimizing \( P_R \) at the optimized values of \( n \), present more or less the same results for every flood. Any improvement in one or the other is only marginal. Hence, considerable labour can be saved if \( n \) alone is optimized.

(ii) A comparison of errors in peak flow prediction shows that this error is less than 4% in four of the eight floods routed. It is 7.63% in one other flood. These are fairly accurate predictions.
Table 6.12 Results of routing by implicit method, Narmada

<table>
<thead>
<tr>
<th>Flood Number</th>
<th>Observed outflow peak (cumecs)</th>
<th>Type of Optimization</th>
<th>Computed outflow peak (cumecs)</th>
<th>Error in peak flow (per cent)</th>
<th>Standard deviation of $%$ errors in $Q$ (per cent)</th>
<th>Error in mass conservation (per cent)</th>
<th>Error in time of occurrence of peak (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA2</td>
<td>21960</td>
<td>$S = n^2$</td>
<td>22383</td>
<td>- 1.93</td>
<td>15.757</td>
<td>-0.168</td>
<td>0</td>
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<tr>
<td></td>
<td></td>
<td>$P_R$</td>
<td>22696</td>
<td>- 3.35</td>
<td>14.427</td>
<td>-0.363</td>
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<tr>
<td></td>
<td></td>
<td>$S + P_R$</td>
<td>22696</td>
<td>- 3.35</td>
<td>14.425</td>
<td>-0.363</td>
<td>6</td>
</tr>
<tr>
<td>NA3</td>
<td>34679</td>
<td>$S$</td>
<td>35627</td>
<td>- 2.73</td>
<td>14.951</td>
<td>-1.193</td>
<td>12</td>
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<tr>
<td></td>
<td></td>
<td>$P_R$</td>
<td>35792</td>
<td>- 3.21</td>
<td>15.081</td>
<td>-1.171</td>
<td>12</td>
</tr>
<tr>
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<td></td>
<td>$S + P_R$</td>
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<td>- 2.70</td>
<td>14.917</td>
<td>-1.200</td>
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<tr>
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<td>15130</td>
<td>$S$</td>
<td>15108</td>
<td>+ 0.15</td>
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<td></td>
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<td>$S + P_R$</td>
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</tr>
<tr>
<td>NA5</td>
<td>31516</td>
<td>$S$</td>
<td>27014</td>
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<td>19.788</td>
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<td>12</td>
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<td>$P_R$</td>
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<td>$S + P_R$</td>
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<td>29.639</td>
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<td>NA7</td>
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<td>S_n</td>
<td>14321</td>
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<td>8.322</td>
<td>1.776</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P_n</td>
<td>14622</td>
<td>5.69</td>
<td>8.106</td>
<td>1.697</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S_{n+R}^P</td>
<td>14554</td>
<td>6.13</td>
<td>8.136</td>
<td>1.714</td>
<td>0</td>
</tr>
<tr>
<td>NA9</td>
<td>20545</td>
<td>S_n</td>
<td>20333</td>
<td>1.03</td>
<td>9.370</td>
<td>-2.304</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>P_n</td>
<td>20568</td>
<td>0.86</td>
<td>7.681</td>
<td>-2.183</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>S_{n+R}^P</td>
<td>20423</td>
<td>0.59</td>
<td>7.815</td>
<td>-2.733</td>
<td>-6</td>
</tr>
<tr>
<td>NA10</td>
<td>39206</td>
<td>S_n</td>
<td>24375</td>
<td>37.93</td>
<td>32.383</td>
<td>-0.630</td>
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<td></td>
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<td>P_n</td>
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<td></td>
<td>S_{n+R}^P</td>
<td>24377</td>
<td>37.82</td>
<td>32.380</td>
<td>-0.630</td>
<td>12</td>
</tr>
</tbody>
</table>

***
Figure 6.6 Observed and computed hydrographs at Garudeshwar (Implicit method)
Of the floods which show a higher error in predicting peak flow, the flood of September, 1975, i.e., flood NA5, (Figure 6.6b) seems to have been routed much less accurately by the implicit method when compared to the storage routing methods (see Figure 5.8c). A look at the observed hydrograph shows that there is a sharp rise to peak flow (from 8,000 cumecs to 32,000 cumecs nearly in just 30 hours). It is possible that when a grid size of 4 sections and 6 hours is employed, the acceleration factors are not properly defined because of the large size of the grid. This was checked by routing the flood for 13 sections and 3 hours. Figure 6.7 compares the solution obtained by routing with 13 sections and 4 sections keeping the routing intervals at 3 hours holding to be equal to 0.040. The improvement in the solution by the 13 section approach is evident in this figure; the importance of low values of $\Delta x$ should not be ruled out, at least for steep rising floods.

It may be recalled that in Section 6.2.1, it was concluded that in order to restrict the influence of observational errors in survey data, $\Delta x$ may have high values with a properly chosen Manning's $n$. In the light of the present observation, $\Delta x$ has to be chosen in such a way as to avoid excessive observational errors and at the same time it has to be properly chosen as to account better for acceleration terms in sharply rising hydrographs.

iii) Though optimizing $n$ value may be sufficient in this reach, generally, optimization of both $n$ and $P_R$ may be necessary in particular cases. For example the flood of August, 1976 (flood NA6) failed to optimize $P_R$ with $n$ kept at 0.040, but $P_R$ could be optimized when tried with the optimized values of $n$. This would mean that the initial estimate of the parameter also plays a part in the optimization procedure. More study and trials are indicated in this regard.

iv) Two floods, viz, floods of August 1976 (flood NA6) and August 1979 (NA 10) show very high values in all the error parameters considered. The former is a multi-peaked flood with violent fluctuations
Figure 6.7 Comparison of 13 and 4 section hydrograph (Flood NA5)
in the peak flow. Despite high values of errors obtained in the implicit routing of this flood, the method adopted in this study cannot be faulted. This flood has behaved in the same manner in all the methods adopted. For a comparison of the routing by the implicit model with that by the 3C model vide Figure 6.8a. The flood of August 1979 (flood NA10) was not used in optimization but was used for testing the routing with the identified parameters. The results are not at all encouraging. However, a comparison of this result with results from other methods shows that this flood has produced higher values of errors in all methods. Figure 6.8b compares the hydrographs computed by the implicit model with that computed by the 3C model. The reason for this discouraging results should lie elsewhere and this is discussed further in general comparison of all the methods as elaborated in Section 7.4.

The results enable one to make some conclusions. Generally a large grid size would not sacrifice the accuracy of routing and one can choose and adopt large grid sizes to avoid excessive computational costs. But should a steeply rising flood be envisaged, subreach lengths should be smaller for better accuracy. In fact floods NA1 and NA8 could not be routed by the implicit method even when the reach was divided into 13 sections. These two floods also rise very steeply (the former being the flood with the highest peak flow amongst the ten floods employed in this study) and probably they require shorter $\Delta x$ and $\Delta t$ values and also probably different values of $n$ at low and high flows. Except in these special cases, it should be possible to use large sizes of $x$-$t$ grid in implicit routing.

The improvement in the accuracy of routing by the optimization of Manning’s $n$ is also limited. For better accuracies, probably the cross-sections could be optimized. However, if the routing is sensitive to changes in cross-section, as it is to Manning’s $n$, then first order gradient search method of optimization will not suffice and more powerful techniques will have to be employed entailing higher computational costs.
Figure 6.8 Comparison of routing by implicit and KGSE-3C models
6.3 ROUTING WITH IMPLICIT METHOD IN CAUVERY

6.3.1 Grid size boundary condition and Manning's n

The cross-section details at eleven sections in the reach Kodumudi-Musiri of the river Cauvery were available (Table 4.7). Based on the experience in implicit routing of floods in Narmada, it was decided to use and test long grid sizes. Hence the routing was restricted mostly to that of 4 section approach (Table 4.8) and the 11 section approach was used only for a comparison. The routing interval was maintained at one hour in all routings.

The initial condition was assumed to be steady gradually varied flow and the discharges and the depths at various sections were found by linear interpolation and backwater computation respectively as was done in the case of Narmada, vide Section 4.5.2. The upstream boundary condition was defined by the inflow hydrograph.

A rating curve could not be obtained in a polynomial form for the downstream boundary and a discrete stage-discharge relationship had to be used for the downstream boundary condition. The opportunity was taken up to test an alternative boundary condition (i.e., other than a rating curve). Fread [65] and Amein [84] have mentioned the use of a normal flow equation like Manning's as the downstream boundary condition where a rating curve could not be obtained. Karmegam [14] has also used the normal flow equation as a downstream boundary condition, while routing hypothetical floods in prismatic channels. However, the effect of the use of Manning's or Chezy's equation as a downstream boundary condition in real time flood routing does not seem to have been reported in flood routing literature. So, it was thought apt for this study to physically evaluate the performance of implicit routing with Manning's equation as the downstream boundary condition; accordingly in routing the floods of Cauvery, the Manning's equation (Equation 4.20) was used as the downstream boundary condition.
It is also necessary to choose the Manning's $n$ suitably for the routing to be successful. In the case of Narmada, an optimization procedure was adopted to identify the Manning's $n$ for a flood. In the present problem such a step was not included. The optimization procedure changes the value of $n$ so that the errors in routing will be a minimum. When the Manning's equation itself is used as the downstream boundary condition, it would mean that the boundary condition also would vary along with the change in the value of $n$. This would probably make the optimization more difficult and time consuming. Hence, it was decided to choose a constant value of $n$ for all the floods taking care that for most of the floods this value of $n$ routes with maximum accuracy. Accordingly all the floods were tested with different values of $n$ to arrive at a best value of $n$ to be used for routing in further attempts.

In natural channels, it is justifiable to assume a value of $n$ in the range of 0.030 to 0.045. These values, when used in attempts to route the floods of Cauvery, failed to route. After an elaborate trial process, it was found that the minimum value of $n$ where the routing could be carried out successfully was 0.055. The routing was repeated by increasing the value of $n$ by 0.005. Thus for $n = 0.055$ to 0.075, each flood was routed five times. The results of these routings are presented in Table 6.13. This table is supplemented by Figure 6.9 which compares the performance of routing with different $n$ values for two floods (CY2 and CY5) in Cauvery.

The Manning's $n$ values which could be used in this routing seem to be on the higher side. In routing the floods of Narmada, the Manning's $n$ was found to be around 0.040. One reason for the high value of $n$ for Cauvery may be that the flow in this river is invariably very low, the peak reaching a maximum of around 5,000 cusecs only. It is generally known that low values of flow may show high values of $n$. Possibly low flows and low peaks of Cauvery indicate high values of Manning's $n$. Or it is also quite likely that the topography
Table 6.13 Implicit routing with different $n$ values. Cauvery

<table>
<thead>
<tr>
<th>Flood Number</th>
<th>$n$ value</th>
<th>Downstream peak flow (cumecs)</th>
<th>Error in peak flow (per cent)</th>
<th>Standard deviation of errors in $Q$ (per cent)</th>
<th>Error in time of occurrence of peak (hours)</th>
<th>Downstream peak depth (m)</th>
<th>Error in peak depth (per cent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CY1</td>
<td>0.055</td>
<td>2564</td>
<td>Failed</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>0.060</td>
<td>2317</td>
<td>9.65</td>
<td>36.27</td>
<td>0</td>
<td>4.968</td>
<td>-52.00</td>
</tr>
<tr>
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<td>0.065</td>
<td>2318</td>
<td>9.59</td>
<td>35.99</td>
<td>0</td>
<td>4.808</td>
<td>-17.25</td>
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<td>36.08</td>
<td>0</td>
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<td>2298</td>
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**Table 6.13 (Contd.)**
Figure 6.9  Computed hydrographs for different n values
through which the reach extends may have such soil or rock structure and such vegetation that the values of $n$ are high.

From Table 6.13 it is seen that the peak flow discharges have been predicted well in all the floods and with all values of $n$. The percentage error in peak flow discharges varies from 0.21 to 10.37 in six floods and is slightly on the higher side, 13 to 15% in the remaining two floods. All the values of $n$ have predicted the peak discharge to more or less same degree of accuracy. The standard deviation of errors in discharge decreases with increase in the value of $n$, but the difference in standard deviation values is small particularly when $n \geq 0.065$. The time of occurrence of peak flow has also been predicted well. But the prediction of depth of flow is far off the mark, the percentage error in peak depth varying from 22% to 79%. The failure to predict the depth of flow properly can be attributed to the downstream boundary condition. Manning's equation is valid only for uniform flow and when it is applied to flood flow, produces erroneous solutions.

6.3.2 Extension of downstream boundary

With a view to improve the solutions, the downstream boundary was imaginarily extended and the routing was carried out by applying the Manning's equation as the condition at the extended downstream boundary. This was carried out in two steps. In the first step the last subreach was extended, introducing three more sections. (For definition sketch of the extension of downstream boundary, vide Figure 4.10). The total number of sections then was seven. The cross-section data at the fifth, sixth and seventh sections were assumed to be the same as those of the fourth section (i.e., the section at Musiri, the downstream section in the actual reach Kodumudi-Musiri). Similarly the length and slope of the fourth, fifth and sixth subreaches were assumed to be the same as the third subreach. This has resulted in the original length of the reach 68.283 km being extended to 125.883 km,
The Manning's value of \( n \) was chosen as 0.065. This was chosen because it could be seen from Table 6.13 that generally the standard deviation values decrease as \( n \) increases from 0.055. The decrease, though continuous for \( n > 0.065 \) also, is found to be much less pronounced for values of \( n > 0.065 \). On an average a value of \( n = 0.065 \) may be suitable. Even so, this choice is only arbitrary. But, it can be hoped that good results can be obtained using this value; any error imparted by this choice may be equally influencing different routings. The floods were then routed for this 7 section (4 real and 3 extended section) approach.

In the second step, the reach was further extended by three more sections and hence three more subreaches were added in the same lines as described in the foregoing paragraph. Thus there were 4 real sections and 6 extended sections. The length of the reach had thus been extended to 183.483 km. For the value of \( n = 0.065 \), the floods were routed in this 10 section (4 real and 6 extended section) approach. In these approaches the routing was carried through the total reach (i.e., real reach plus the extended reach) but the required results were monitored at the real downstream boundary, which would then be an intermediate section.

The results of routings thus obtained above, i.e., routing with (i) 4 real sections only, (ii) 4 real sections and 3 extended sections and (iii) 4 real sections and 6 extended sections, are presented in Table 6.14. These results, for two floods CY7 and CY8, are also illustrated in Figure 6.10.

In the 7 section and 10 section approaches of routing by implicit method, the downstream boundary is imaginarily extended based on the following logic. If Manning's equation is employed as a downstream boundary condition, it will not relate to the flood flow situation there. However if it is applied at a place far off the real downstream boundary, the flow may get to near uniform conditions at the extended downstream boundary, because the extended reach is prismatic in shape.
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<th>Error in peak flow (per cent)</th>
<th>Standard deviation of errors in <em>Q</em> (per cent)</th>
<th>Error in time of occurrence of peak (hours)</th>
<th>Downstream peak depth (m)</th>
<th>Error in peak depth (per cent)</th>
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Figure 6.10 Routing with extended sections
Hence the application of Manning's equation, though erroneous in concept, may still serve the purpose. This has been proved in this study.

Let the results of routing with extended sections be studied in Table 6.14 along with Figure 6.10. The efficacy of Manning's equation as the downstream boundary condition can be discussed as follows.

When the Manning's equation is applied as the boundary condition at the real downstream boundary, it routes the floods well for discharges but computes erroneously the depths of flow, the error in peak depth varying from 23.56% to 47.25%, with Manning's $n = 0.065$. When the imaginary sections are added and the reach is extended, there is a perceptible improvement in the solution. Standard deviation of percentage errors is reduced, implying better agreement between computed and observed hydrographs. Errors in peak flow in all the three approaches show almost the same values in each flood. There is a good improvement in this regard in flood CY2. The time of occurrence of peak has been predicted well in all the three approaches.

The justification for the extended section approach is clearly seen when the computed peak depths are compared. While in the 4 real section approach, the peak depths were computed with very high errors, the extended sections approach has brought down these errors considerably. If the 4 real and 3 extended section approach is considered, this error is brought down to the range 2.22% to 17.67% with only two floods showing an error of more than 13% in the extended sections approach. Thus the adoption of extended reach in routing with Manning's equation as downstream boundary condition is justified in real time flood routing.

If a reach has to extended, the question arises as to how much the extension should be. In this study the floods were routed (i) by doubling the number of subreaches (nearly doubling the total reach length and (ii) by trebling the number of subreaches (nearly
trebling the total reach length). One may surmise that the longer the extension, the more the chances that the flow may tend to uniform conditions at the downstream boundary.

Table 6.14 shows that 4 real and 6 extended section approach shows minimum standard deviation of percentage errors in discharge in all but one floods. However, a closer examination of the table shows that a 4 real and 3 extended section approach is a clear improvement over the 4 real section approach, and the improvement seen in the 4 real and 6 extended section approach is only marginal. If the computational cost is to be considered, the 4 real and 3 extended section approach has to be preferred. Another point to be noted in the table is with regard to the prediction of depth of flow at the real downstream section. The peak depth has invariably been predicted better by the 4 real and 3 extended section approach.

This observation may be explained with the following lines of argument. The Manning's equation will provide a loop rating curve defined by the rising and falling stages of flood. The uniform flow conditions could be achieved only on the rare occasion when $S_f = S_0$. Hence, there would always be attenuation computed as the flow passes along the extended reach, even if the reach is prismatic. This attenuation should be proportional to the reach length. So the use of Manning's equation could be best only at that location where it is more likely to be a single valued function. This position would therefore vary for different floods. For the floods routed in this study, this position seems to be the one obtained in the 4 real and 3 extended section approach. That is to say, it is approximately double the original length. Perhaps, more trials with the extension of reach length may provide more accurate answers.
It is seen that flood CY2 and flood CY9 have failed in one of the attempts. They failed because at some intermediate time levels in one or more of the extended sections the value of $y$ was predicted in one or more iterations at a value higher than what was provided for in the cross-section data. Perhaps a provision for extrapolation for the cross-section should be envisaged in the routing model program. The same was the case with flood CY10 which could not be fully routed in the 4 real section approach for any value of $n$ between 0.055 and 0.075. However, in the 4 real and 6 extended section approach, the flood has been routed albeit with unsatisfactory results. This fact is further discussed later when a general comparison of all the methods is taken up in Section 7.4.

6.3.3 Reduction in subreach length

Mathematically, if the subreach length $\Delta x$ is small, the routing will be more accurate. This was tested from practical point of view for the river Narmada (Sections 6.2.1 and 6.2.5) and it was found that even large grid sizes of $\Delta x$ and $\Delta t$ routed many floods very well. However, one flood showed a perceptible improvement when the grid size was small. Hence, a similar test was made in Cauvery also, reducing the value of $\Delta x$, to see whether any of the floods substantially improve in solution as to definitely warrant smaller grid sizes.

For this purpose, the reach Kodumudi-Musiri was divided into ten subreaches (see Table 4.7) and the last subreach was extended
reducing the value of $\Delta x$, to see whether any of the floods substantially improve in solution as to definitely warrant smaller grid sizes.

For this purpose, the reach Kodumudi-Musiri was divided into ten subreaches (see Table 4.7) and the last subreach was extended ten times, such that there were 11 real and 10 extended sections. The total reach length amounted to 112.083 km compared to the real reach length of 68.283 km. The floods were then routed for $n = 0.065$ and $\Delta t = 1$ hour. Table 6.15 provides a comparison of the results of the two approaches viz, (i) 7 section (4 real and 3 extended section) approach and (ii) 21 section (11 real and 10 extended section) approach. For an illustration of the performance of the two approaches, results of routing two floods, flood CY1 and CY6 are presented in Figure 6.11.

The following points are noted from Table 6.15. As far as the standard deviation of percentage errors in discharge is concerned it is found that both 21 section and 7 section approaches route the floods almost for the same values. The prediction of peak flow has been within 10% in five out of seven floods. There is nothing to choose between the two approaches, wherein for some floods 21 section approach and for some others 7 section approach route with better accuracy. Regarding the time of occurrence of peak, both approaches are very accurate. But as far as the prediction of the peak depth of flow is concerned the 21 section approach produces an improved result. The error range 2.22% to 17.67% in the 7 section approach has been reduced to a range 1.62% to 8.05%. The conclusion is apparent that $\Delta x$ values must be small for predicting depths of flow accurately, particularly when the downstream boundary condition is not evaluated by a genuine rating curve.

Also, it is seen that flood CY9 which failed in the 7 section routing could be routed in the 21 section approach. But the results are unsatisfactory, even though the depth seems to have been predicted well. The performance of this flood will be further discussed in a general comparison of the routing techniques.
Table 6.15 Implicit routing with 21 and 7 sections, Cavory (n 0.065)

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<tr>
<th>Flood Number</th>
<th>Real + extended sections</th>
<th>Downstream peak flow (cumeecs)</th>
<th>Error in peak flow</th>
<th>Standard deviation of $F$ errors in discharge</th>
<th>Error in time of occurrence of peak (per cent)</th>
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Figure 6.11 Routing with 21 and 7 sections
6.4 CONVEYANCE ROUTING

6.4.1 Conveyance of the channel reach

The implicit routing model can provide accurate solutions if the model parameters are properly identified. This study attempted to identify one model parameter, viz. Manning's n by an optimization technique. It also considered the hydraulic radius R as another model parameter and optimized it, by optimizing its power, which in Manning's equation is equal to 4/3. While this attempt reduced the routing errors, there seems to be still room for improvement.

Optimizing R is only a partial approach in the sense that the friction slope uses cross-section area A indirectly optimized in R but not optimized elsewhere. So, it would perhaps be more beneficial if the cross-section itself is optimized. Karmegam [14] considered the width of the channel, side slope, Manning's n and the power of R as the four model parameters to be optimized. The theory behind the optimization of these parameters can be explained as follows.

The four model parameters can be classified into two categories, (i) physical (bedwidth and side slope) and (ii) hydraulic (Manning's n and power of R). The first group signifies the size of the available waterway, while the second signifies the resistance it will offer to the flow. The size of the waterway can be specified in terms of the area A of the waterway but it is difficult to specify the resistance by a single factor. Nevertheless, it is possible to combine both the area and resistance in the form of Manning's equation.

\[ Q = \frac{A}{n} R^{2/3} S_f^{1/2} \quad \cdots (6.2) \]

or \[ Q = K S_f^{1/2} \quad \cdots (6.3.) \]

where \[ K = \frac{1}{n} AR^{2/3} \quad \cdots (6.4) \]
The term $K$ is known as the conveyance of the channel section. This $K$ is a measure of the carrying capacity of the channel section since it is directly proportional to $Q$, and has the units of discharge.

Optimizing the four model parameters (after Karmegam) would in effect be optimizing the conveyance of the channel, which in turn should produce excellent results in predicting the outflow, as it did for Karmegam [14]. However, it should be noted that the procedure for optimization would be tedious with four parameters and would require powerful optimization algorithms. Karmegam adopted a combination of Influence Coefficient Algorithm with linear programming to obtain the solution. Such a procedure requires enormous storage space in the computer (the linear programming's requirement of space is proportional to the number of time levels to which the routing is taken) and will entail high computational costs.

This study attempted to employ the principle of conveyance of the channel section and evolve a procedure to evaluate it. Routing a flood by implicit model with the conveyance factor obtained as above is termed the conveyance routing in this study.

6.4.2 Data accuracy requirements in implicit routing

If the conveyance $K$ of the river reach is to be identified, it is necessary to study the influence of the accuracy of the section parameter bedwidth and side slope, represented in a natural channel by area of flow $A$, and also the influence of hydraulic parameter, Manning's $n$. This study has already optimized Manning's $n$ (see Section 6.2.).

There seems to be no literature which actually describes the degree of accuracy required in the geometric data to be used in implicit routing. Observational errors in geometric data - the reach length, the cross-section details and the bedslope - will impart their
own inaccuracies in the routing. In fact, a highly erroneous data may even fail to route the floods by virtue of the incompatibility of these data with the actual flow occurring in the reach. So it was decided to test the requirements of the accuracy of the geometric data; such an information would help in deciding the identification of a proper conveyance of the river reach.

The procedure adopted in this study for this error (in geometric data) analysis is akin to a sensitivity analysis. Adopting the optimized values of $n$, errors were induced into any one of the geometric data, keeping others unaltered. In a 4 section and 3 hours routing approach, this analysis was done individually at (i) the upstream subreach/section, (ii) the downstream subreach/section, and (iii) all the three subreaches/four sections. The data used were changed successively by 10%, 30%, -10%, -20% and -30% and the error parameters were compared. This exercise was carried out for flood NA4.

An analysis of the results obtained in this study showed that even a variation of ±30% in the length of any subreach or in the lengths of all subreaches had not substantially altered the results. A conclusion could be arrived at that any ordinary observational error in the length alone of the channel will not unduly affect the routing. However, it should be noted that the bed slopes were kept at original values. In a river routing the bed slopes would be calculated with reference to difference in level and length of the subreach, when the alteration of length will compute slope erroneously. Hence, it is more pertinent to study the sensitivity of slope to routing accuracy.

The slopes were altered by just imparting the percentage change to the slope value. Even a 10% alteration in the slope at the downstream subreach had failed to route the flood. The same error imparted in all the subreaches routed the flood with increased inaccuracy. The reduction of slope values at the downstream subreach has affected the routing results resulting in higher error values as
seen in all the error parameters compared in Table 6.16. This table illustrates the effect of change in slope in routing flood NA4 by the implicit method. It may be noted that changes in upstream subreach have not affected the results unduly; certainly not as much as the change in slope at the downstream subreach. It is proper to infer that the geometry at the downstream subreach should be accurately defined to avoid erroneous solutions.

The influence of errors in cross-section details was then tested. The cross-sections were altered by imparting the percentage change to the top widths, keeping the corresponding depths constant as shown in Figure 6.12 for 10% increase. The results of routing with inaccurate cross-sections are presented in Table 6.17. It is again seen that even a 10% error in the cross-section at the downstream section is not tolerated. Evidently if errors are committed in all sections, the accuracy is sacrificed. Here also it is noted that changes in upstream cross-section have a comparatively less pronounced influence on the routing.

Any error committed in the downstream geometry is at once reflected in the routing. This is probably because of the following reason. The downstream boundary condition is a fixed relationship between stage and discharge and is a special condition, unlike a forward or backward characteristic as used in explicit methods which is only a substitute for either the continuity or the momentum equation, vide...
### Table 6.16 Effect of change of slope - Flood NA4

<table>
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<th>Magnitude of change (1)</th>
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<th>Error in peak</th>
<th>Standard deviation</th>
<th>Error in time of occurrence of Peak (hours)</th>
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<td>Flow (m)</td>
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<td>(hours)</td>
</tr>
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<td>(5)</td>
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<td>(cumecs)</td>
<td>(m)</td>
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Observed peak outflow = 15,130 cumecs

Observed peak depth = 12.890 m

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u/s = upstream; d/s = downstream
Table 6.17 Effect of change in top width - Flood NA4

Observed peak flow = 15,130 cumecs  
Observed peak depth = 12.890 m

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u/s = upstream; d/s = downstream
The downstream boundary condition relates $Q$ with $y$ at all time levels. Any error at the downstream cross-section or slope will immediately affect the downstream boundary condition adopted.

The sensitivity test made for assessing the requirement of the accuracy of data has shown that all geometric data must be particularly accurate at the downstream subreach/section and that the routing would be much less sensitive to errors in the upstream subreach/section.

6.4.3 Equation for the conveyance of the reach

The analysis on data accuracy requirements indicates that if the cross-section and conveyance at the downstream boundary can be properly defined, an implicit routing model with conveyance term incorporated in it, instead of cross-section data, can be formulated. The Mortakka-Garudeshwar reach of Narmada was taken up for testing this approach. As was done in the regular implicit model tests, the 4 section approach (Table 4.6) was adopted.

As the first step, the cross-section data at the four sections were studied and plotted in a graph paper for the following relationships.

i. Area of flow $A$ vs depth of flow $y$, and

ii. Section factor $AR^{2/3}$ vs depth of flow $y$.

Then these discrete values were regressed to fit in a best fit polynomial or an exponential curve. The $A$ vs $y$ curve was found to fit a second degree polynomial. The $AR^{2/3}$ vs $y$ curve fitted a straight line in the log-log graph and accordingly it was regressed as an exponential curve. The following are the relevant relationships.
a. $A$ Vs $y$ curve (second degree polynomial)

Section 1
(Upstream section)

\[ A = 26.05 y^2 + 141.42 y + 177.44 \]

Section 2
(Intermediate section)

\[ A = 85.15 y^2 - 1681.05 y + 8689.56 \]

Section 3
(Intermediate section)

\[ A = 7.53 y^2 + 90.92 y - 505.83 \]

Section 4
(Downstream section)

\[ A = 5.35 y^2 + 360.20 y - 521.49 \]

b. $AR^{2/3}$ Vs $y$ curve (Exponential curve)

\[
\begin{align*}
\text{Section 1} & & AR^{2/3} = 54.59 y^{2.39} \\
\text{Section 2} & & AR^{2/3} = 128.36 y^{2.14} \\
\text{Section 3} & & AR^{2/3} = 44.06 y^{2.26} \\
\text{Section 4} & & AR^{2/3} = 70.45 y^{2.20}
\end{align*}
\]

Having established the above equations, the implicit routing model computer program was modified to incorporate these values. The scheme was formulated originally for the implicit routing based on the form of the Saint-Venant Equations (see Section 3.5.4.1)
and
\[ \frac{\partial Q}{\partial x} + \frac{\partial Q_1}{\partial t} = q_1 \quad \ldots \quad (6.5) \]

Now, in view of the relationship \( A \) vs \( y \), no values of \( B \) will be read in the computer program and hence the continuity equation, in the conveyance routing, will be employed in the form

\[ \frac{\partial Q}{\partial x} + \frac{\partial (Q^2)}{\partial A} + gA \frac{\partial h}{\partial x} = q_1 (S_0 - S_f) + q_1 \frac{Q}{A} \quad \ldots \quad (6.6) \]

and the momentum equation can be retained as such (Equation 6.6).

The friction slope is evaluated thus

\[ Z = AR^{2/3} \quad \ldots \quad (6.8) \]

\[ K = Z/n \quad \ldots \quad (6.9) \]

and

\[ S_f = \frac{Q^2}{K^2} \quad \ldots \quad (6.10) \]

where the value of Manning's \( n \) used is the one identified for the flood in Section 6.2.3.

When the four area of flow equations and corresponding 'section factor' equations are used in routing, they should produce the results obtained earlier in Section 6.2.5, subject to some variations imposed by the differences between the discrete values of \( A \) and the continuous values obtained from a regressed equation. This study aimed at one single equation of \( A \) and another for \( AR^{2/3} \) to represent the whole reach, in order to simplify the routing model. Use of any one of the equations of sections 1 to 3 would evidently be a failure as...
they would be affecting the section 4 which is a downstream section. A test of any one pair of these equations did not route the floods at all. This perhaps confirms the earlier finding that the downstream subreach and section should be very accurately defined in an implicit routing model.

Encouraged by the inference that errors in sections other than the downstream section may not appreciably affect the routing, provided the downstream section is properly evaluated, an attempt was made to route the floods using the area equation and the section factor equation of the downstream boundary only. Thus for all four sections, the equations used were (to an accuracy of five decimals).

\[
A = 5.34928 y^2 + 360.19850 y - 521.49010 \quad \ldots \quad (6.11)
\]

and

\[
AR^{2/3} = 70.45000 y^{2.21932} \quad \ldots \quad (6.12)
\]

The flood NAA4 was routed in the conveyance routing using Equations (6.11) and (6.12) and the solutions were comparable to observed values. In order to get more accurate results, Equation (6.12) was then slightly altered and the routing repeated. In order to keep the equation in a simple form the power of y was taken as 2.2 in all trials and the coefficient of y was changed in increments of 0.5. The improvement in the routing was decided by the reduction in the values of standard deviations in percentage errors in discharge. A few trial routings arrived at the following equation as best suited for the flood NAA4.

\[
AR^{2/3} = 81 y^{2.2} \quad \ldots \quad (6.13)
\]

Conveyance routing with Equation (6.13) was then tested for all the floods. The results, which are very satisfactory, are presented in Table 6.18.
Table 6.18 Comparison of the results of implicit and conveyance routings

<table>
<thead>
<tr>
<th>Flood Method</th>
<th>Observed peak Flow (cumecs)</th>
<th>Computed peak Flow (cumecs)</th>
<th>Error in peak Flow ($)</th>
<th>Error in peak Depth ($)</th>
<th>Error in time to peak (hours)</th>
<th>Error in $%$ error in Q ($)</th>
<th>Error in depth conservation ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA2 Implicit Conveyance</td>
<td>21960</td>
<td>15.77</td>
<td>22071</td>
<td>15.82</td>
<td>-0.55</td>
<td>-0.3</td>
<td>-12</td>
</tr>
<tr>
<td>NA3 Implicit Conveyance</td>
<td>34679</td>
<td>20.26</td>
<td>35615</td>
<td>20.55</td>
<td>-2.70</td>
<td>-1.5</td>
<td>12</td>
</tr>
<tr>
<td>NA4 Implicit Conveyance</td>
<td>15130</td>
<td>12.89</td>
<td>15615</td>
<td>13.11</td>
<td>-3.21</td>
<td>-1.7</td>
<td>0</td>
</tr>
<tr>
<td>NA5 Implicit Conveyance</td>
<td>31516</td>
<td>19.22</td>
<td>27014</td>
<td>17.66</td>
<td>14.28</td>
<td>8.1</td>
<td>12</td>
</tr>
<tr>
<td>NA6 Implicit Conveyance</td>
<td>19417</td>
<td>14.75</td>
<td>15385</td>
<td>13.01</td>
<td>20.77</td>
<td>11.8</td>
<td>36</td>
</tr>
<tr>
<td>NA7 Implicit Conveyance</td>
<td>15504</td>
<td>12.80</td>
<td>14554</td>
<td>12.62</td>
<td>6.13</td>
<td>1.4</td>
<td>0</td>
</tr>
<tr>
<td>NA9 Implicit Conveyance</td>
<td>20545</td>
<td>15.19</td>
<td>20423</td>
<td>15.16</td>
<td>0.59</td>
<td>0.2</td>
<td>-6</td>
</tr>
<tr>
<td>NA10 Implicit Conveyance</td>
<td>39206</td>
<td>21.67</td>
<td>24377</td>
<td>16.70</td>
<td>37.82</td>
<td>22.9</td>
<td>12</td>
</tr>
</tbody>
</table>
6.4.4 Discussion on the performance of conveyance routing

The performance of the conveyance routing model can be discussed by comparing its results with the solutions obtained from the implicit routing model in which the cross-section data were discretely fed. For convenience sake, the latter model is called the implicit routing model and the former, the conveyance routing model. The performance of these two models is presented in Tables 6.18.

If floods NA6 and NA10 are removed from consideration temporarily, the conveyance routing has predicted the peak flow better in three floods and the implicit routing in three others. In these floods the highest error in peak flow is -10.21%. With regard to predicting the time of occurrence of peak flow, except in flood NA2, the conveyance routing improves the prediction of the implicit routing. It is also found that the conveyance routing shows less values for standard deviation of percentage errors in discharge in all floods except two (NA4 and NA7), in which also it shows only very marginally higher values. In general, it can be concluded that the conveyance routing has been more successful than the implicit routing.

This conclusion is reinforced when a deeper analysis is made on the three floods, NA2, NA3 and NA5. The flood NA5 has been routed well by the storage routing techniques but has been less accurately routed in the implicit method (See Section 6.2.5). The routing was improved by adopting 13 section approach in the implicit routing (Figure 6.7). But even with a 4 section approach it is now found that conveyance routing brings vastly improved results on this flood.

Of the two floods NA2 and NA3, the latter has been predicted with a higher percentage of peakflow error by the conveyance routing. But a perusal of Table 6.18 shows that conveyance has done better in all other error parameters. In fact a look at the table shows that conveyance routing has produced better values of standard deviation
of errors in depth. Evidently the conveyance routing should have solved these two floods more efficiently than implicit routing, not withstanding the comparatively a higher error in peak flow and depth predicted in flood NA3. This is borne out by a comparison of the solutions at every time level in the computer output.

Solution to both the floods NA2 and NA3 by the implicit method had produced oscillations around the true value from the start of the routing period till the time the flood reaches the peak and even a little after the peak has been reached. But these oscillations have been dampened and the curve of the computed hydrographs has been smoothened for these floods in the conveyance routing. This performance is illustrated in Figures 6.13 and 6.14.

The dampening of oscillations can be explained thus: In implicit routing the geometric data are fed into the computer discretely. So when the derivatives of $A$, $B$ and $P$ are computed, they would have their truncation errors. These errors, which are carried forward to every time level may affect the results. But, in conveyance routing, the derivatives of $A$ and $K$ are obtained by differentiating the relevant equations. So, there will be no truncation errors and the derivatives are more accurately obtained and so no oscillations are seen.

Though the conveyance routing seems to have produced improved results in floods NA6 and NA10 also in respect of all error parameters, the prediction is definitely erroneous. It has already been pointed out that they behave erratically in all methods and that they would be discussed when a general comparison of all the methods is made in Section 7.4. This holds good for conveyance routing also.

6.4.5 Computer time and storage space

For an implicit routing model, the data requirement is higher compared to a storage routing model. Apart from flow data, the cross-
Figure 6.13 Comparison of implicit and conveyance routings, Flood NA2

Figure 6.14 Comparison of implicit and conveyance routings, Flood NA3
section data also has to be provided. Bulk of the cross-section data would comprise the area of flow A, the top width of flow B and the wetted perimeter P for a large number of values of y in every section. The number of y values and hence the corresponding A, B and P values required to define the cross-section depends on the irregularities present in the cross-section. The program can be so formulated as to calculate A from B and y, when one can dispense with data of A. Also if the river is large and shallow, $P \approx B$ for a trapezoidal section and so the data for P also need not be read. In this study both the above procedures were followed. Nevertheless sufficient values of y and B have to be read in. In the conveyance routing these values are replaced by just two equations, one for A and one for $AR^{2/3}$. The storage space saved will be substantial if a large number of cross-sections are employed in the reach.

Similarly computer time also will be less in conveyance routing. Compilation and linking time will be marginally less because of less data points. The execution time depends upon the number of time levels to which the routing is taken. The higher the number of time levels, the higher would be the execution time (assuming that at each time level, the Newton-Raphson procedure would take the same number of iterations on an average).

This study has used a large grid size in the implicit and conveyance routing. The number of sections used was only four and the number of time levels to which the routing was taken was around forty. Still there is a perceptible reduction in computer space and time. The computer time required was noted in UPTRON PC-XT computer. The savings in computer storage and computational time can be understood from Table 6.19 for two floods NA3 and NA5.

6.4.6 Conclusions on conveyance routing

The conveyance routing model has routed the floods fairly well. The technique with conveyance routing is as accurate as the
Table 6.19 Computer time and space for implicit and conveyance routing

<table>
<thead>
<tr>
<th>Time level in which routing is taken</th>
<th>Method of routing</th>
<th>Computer space (Bytes)</th>
<th>Compiling and linking time</th>
<th>Execution time</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA3 37</td>
<td>Implicit</td>
<td>136591</td>
<td>6 min 35s</td>
<td>1 min 05 s</td>
</tr>
<tr>
<td></td>
<td>Conveyance</td>
<td>122497</td>
<td>5 min 39s</td>
<td>0 min 40 s</td>
</tr>
<tr>
<td>NA5 41</td>
<td>Implicit</td>
<td>136711</td>
<td>6 min 35s</td>
<td>1 min 12 s</td>
</tr>
<tr>
<td></td>
<td>Conveyance</td>
<td>122817</td>
<td>5 min 39s</td>
<td>0 min 44 s</td>
</tr>
</tbody>
</table>
implicit routing; in fact, it provides much better solution for almost all the floods. The conveyance routing model is simpler than the implicit routing model in that it avoids interpolations of cross-section data and calculates the friction slope by a simple equation. Its computer space requirements and computational costs would be much less than the implicit routing model, particularly when a large number of sections are introduced and the routing is carried through a large number of time levels.