STELLINGEN BEHOOREN BIJ HET PROEFSCHRIFT VAN H.E.J. BERGER, GETITELD "FLOW FORECASTING FOR THE RIVER MEUSE"

1. Omdat metingen tijdens calamiteiten als hoogwaters niet te herhalen zijn, is het van het grootste belang deze nauwkeurig te doen en de meetresultaten tezamen met gegevens omtrent de exacte ligging en de soort meetuitrusting te bewaren.

2. Hoewel er verschillende omschrijvingen van het begrip effectieve neerslag in omloop zijn, bestaat er toch een allesomvattende definitie: Effectieve neerslag is dat deel van de neerslag dat ten goede komt aan dat deel van de hydrologische kringloop dat men bestudeert.

3. De gevolgen van veranderingen in ondermeer landgebruik, klimaat en rivierprofielen tijdens de volgende decennia dienen bij de vaststelling van de gewenste hoogten van rivierdijken betrokken te worden.

4. Strikt genomen is ijsgang een waterkwaliteitsprobleem. Hij kan echter grote gevolgen kan hebben voor de waterkantiteit.

5. Voor hydrologen zou het leven een stuk eenvoudiger zijn indien de landsgrenzen zouden overeenkomen met de waterscheidingen.

6. Bij veel wetenschappers is de angst voor neerlandismen bij het opstellen van wetenschappelijke teksten in de Engelse taal zo groot dat hierdoor een gekunsteld soort Engels ontstaat.
7. Aluminiumoxide is in vergelijking tot magnesiumoxide een geschikter dragemateriaal voor nikkelkatalysatoren indien kleine hoeveelheden lithium- en kaliumcarbonaat aanwezig zijn, omdat het een sterkere interactie vertoont met alkaliverbindingen waardoor afdekking van het nikkeloppervlak met alkaliverbindingen wordt tegengegaan.  

8. De verkeerscongestie in steden zou kunnen worden beperkt door naast richtingsborden met de aanduiding centrum ook borden te plaatsen met als opschrift "aanvang centrum".

9. Veel spelletjes zouden interessanter kunnen zijn als bij de keuze van de dobbelsteen de ontwerper zich niet zou hebben beperkt tot het regelmatig zesvlak.

10. Een overeenkomst tussen sport en wetenschap is dat zij beide met politiek te maken hebben, hoezeer zij dat ook betreuren.

11. Het begrip mijlpaal dient met de invoering van het S.I.-stelsel vervangen te worden door het begrip kilometerpaal.

Flow Forecasting for the River Meuse

H.E.J. Berger
Flow Forecasting for the River Meuse

Afvoervoorspelling voor de Maas

Proefschrift ter verkrijging van de graad van doctor aan de Technische Universiteit Delft, op gezag van de Rector Magnificus, prof. drs. P.A. Schenck, in het openbaar te verdedigen ten overstaan van een commissie aangewezen door het College van Dekanen op 24 maart 1992 te 16.00 uur door Hermanus Everhardus Johannes Berger, geboren te Enschede, civiel ingenieur.
Dit proefschrift is goedgekeurd door de promotor, prof. dr. ir. J.C. van Dam

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Preface

Flow forecasting is one of the most important and investigated subjects in river hydrology, as exceptional discharges and water levels cause damages. For most rivers a flow forecasting model has been developed. Only for those rivers where circumstances are not optimum such a model is missing, for example due to lack of money or measurements or due to the character of the river. One of those rivers has been the Meuse, where a complex system (heterogeneous, heavily influenced by man, rapid response of the runoff to precipitation) in combination with a lacking international data supply had caused the flow forecasting model to function badly. It was felt in the Institute of Water Management and Waste Water Treatment (Dienst Binnenwateren/RIZA, nowadays RIZA), by dr. J. de Jong, ir. F.H.M. van de Ven and ing. H.W.J. van der Valk, that a more thorough study (in combination with an improved international cooperation) could lead to a successful model. For that purpose I was engaged in 1986. From the very beginning the ultimate goals have been a reliable model and a PhD-thesis. This dissertation forms the realisation of the second goal.

The work was done under the supervision of a commission that has been established just for that purpose. The chairman was prof.dr.ir. J.C. van Dam of the Delft Technical University. Partly or for the whole period since 1986, the members have been: prof.dr.ir. A. van der Beken of the Brussels Free University, prof.dr.ir. F. de Troch of the State University of Ghent, J.H. Gerritsen (Rijkswaterstaat-Directie Limburg), ir. M. Adriaanse, ir. G.J. Broer, ir. W.J. de Lange, dr.ir. J. Leentvaar, ir. W. Silva, and dr.ir. F.H.M. van de Ven (all Rijkswaterstaat-RIZA). The last person mentioned has been functioning as a daily supervisor.

The study has been executed in the office of RIZA, the same office in which the models are used. This has resulted in the incorporation of many practical aspects in the models. Especially the hoogwatergroep (flood-group) and the berichtencentrum (information centre) have contributed with a lot of practical information.

The models and this thesis could not have been completed without hydrologic data and information. Sources have been the Belgian Ministry of Public Works, the Service de la Navigation at Liège, The Royal Meteorologic Institute of Belgium, The Waterschap Roer en Overmaas and Rijkswaterstaat-Directie Limburg (especially S. Bastings), to mention only the most important.

A number of students and my colleague Mrs. ing. A.L. Mugie have contributed to the work. Results of all of them have been used directly or indirectly in this thesis.

After the modelling, the flood forecast model was made user-friendly with the help of the software house of Information Systems. The work executed by drs. H. Nieboer will undoubtedly contribute to a successful use of the model. In chapter 12 some of the aspects are reflected that are important for a successful real-time model.

I would like to express my gratitude to all the persons and organisations mentioned above for their contribution. This also applies to the persons who have made comments on the draft texts.

Special thanks I owe to B. Jansen, for his drawings, drs. J.H.W. Bouman for the translation of the first seven chapters and the grammatical corrections in the other chapters and Mrs. T. van Keulen for retyping the first seven chapters.

Herbert Berger
Front page: The Meuse near Maashees (fig. 1.1) during the July 1980 flood. Source: Wiek Ernst Luchtfotografie, Kerkrade

Directoraat-Generaal Rijkswaterstaat

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Introduction

The river Meuse has to deal with relatively high floods and low baseflows, due to its character as a rain-fed river. Both the floods and the low flows may cause damage to shipping, agriculture, inhabitants, etcetera. The damage can be reduced significantly if it is known what discharge and water levels will occur. Therefore a forecasting model is indispensable. This thesis describes the development of a flood forecasting model and a low flow forecasting model.

The basis for any hydrologic model has to be the hydrologic knowledge of the catchment. For the Meuse, this knowledge was almost absent where it concerned the foreign part of the catchment. Therefore it has been decided to have a systems analysis precede the model development. This has caused a tripartition of this thesis.

In the first part the results of the systems analysis are described (chapters 1 to 7). After a concise description (chapter 1) attention is paid to the origins and causes of floods and droughts (chapter 2 and 3 respectively). Furthermore it is elucidated how the Meuse basin got its present form (chapters 4 and 5). Afterwards the subcatchments as well as the river are described in some detail (chapters 6 and 7).

A short introductory chapter into modelling (chapter 8) precedes the second and third parts of this thesis.

The second part deals with the flood forecasting (chapters 9 to 12). In chapter 9 a motivation is given to subdivide the flood forecasting model into modules for the subcatchments (which are dealt with in chapter 10) and modules for the river (chapter 11). However, the flood forecasting model is more than a sum of the equations of the modules. For example the starting procedure, the way to handle missing data, the user-friendliness of the computer model and the way of data gathering are important in modifying the mathematical equations to a real-time model in a real-world situation. Those aspects of the model are described in chapter 12, in which also the first experiences in real-time are shown.

In the third part the low flow forecasting model is presented (chapter 13). For modelling the Maastricht-St.-Pieter discharge 10 subcatchments have been distinguished, of which the depletion curves as well as the influence of rainfall on the baseflow have been determined. A rainfall simulation model is described, which is used to incorporate the influence of (future) rainfall on the baseflow.

The second and third parts can be considered to be the core of this thesis.

In writing this thesis it has been attempted to use the hydrologic terms and definitions of Hooghart et al. (1986). In general it could be done satisfactorily, although in some places adaptations were inevitable.

For geographical objects the local names will be used in this thesis, so French names in the part where the French language is spoken, etcetera. In general an English translation is added when the name is used for the first time. This principle does not make the thesis completely consistent with respect to geographical names. For example the spelling of Brussels depends on the context of the sentence. The names of the rivers Scheldt, Meuse and Rhine are kept in English.

Many measuring stations with a combined name are used in this thesis, for example Ampsin-Neuville. For reasons of clarity a hyphen has been added between the separate words.

In the Netherlands the main measuring station is situated at the village of Borgharen.
In the Netherlands the main measuring station is situated at the village of Borgharen. Nowadays it is the station of Borgharen-Dorp (Borgharen-Village), in the past it was Borgharen-Beneden (Borgharen Downstream) and before the construction of the Borgharen weir it was Maastricht-Hoofdsluis (Maastricht-Main Lock). Whenever in this thesis the discharge at the measuring station Borgharen is mentioned, the calculated or measured discharge at or near this village is meant.
Chapter 1. A bird’s-eye view of the Meuse

The Meuse is a river with a length of 874 km from the source in France to the Hollands Diep (Dutch Deep). Its catchment has an area of about 33,000 km² (fig. 1.1).

The Hollands Diep and the adjoining Haringvliet are artificial lakes separated from the sea by means of a dam. At the beginning of the Hollands Diep the Meuse flows together with the Nieuwe Merwede (New Merwede), an important Rhine branch. Because of the confluence, and because the water in the Hollands Diep and the Haringvliet flows very slowly, the Hollands Diep is generally chosen as the mouth of the Meuse. If, however, the Hollands Diep and the Haringvliet are rated as belonging to the Meuse basin, too, its catchment area is enlarged by 3000 km² and the river is almost 60 km longer.

In this report Lith will often be chosen as the lower boundary, on the one hand because below Lith no discharge values are known and on the other because below Lith tidal influences are present and consequently that stretch lies beyond the one of the discharge forecast to be developed. Up to Lith the river has a length of 812 km and a contributing area of 29,370 km².

For a number of rivers Mahon (1982) has collected a series of numbers relating to several rivers in the world. Part of the numbers is given in table 1.1, supplemented with data about the Meuse. Neither in the world nor in Europe does the Meuse take a striking position.

<table>
<thead>
<tr>
<th>river</th>
<th>station</th>
<th>basin area [10³ km²]</th>
<th>specific discharge [mm/day]</th>
<th>specific critical discharge [mm/day]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amazon</td>
<td>Obidos</td>
<td>4688</td>
<td>2.76</td>
<td>3.9</td>
</tr>
<tr>
<td>Meuse</td>
<td>Borgharen-Dorp</td>
<td>21</td>
<td>0.93</td>
<td>5.9</td>
</tr>
<tr>
<td>Meuse</td>
<td>Lith</td>
<td>29</td>
<td>0.73</td>
<td>4.2</td>
</tr>
<tr>
<td>Rhine</td>
<td>Rees</td>
<td>160</td>
<td>1.23</td>
<td>3.6</td>
</tr>
<tr>
<td>Rhône</td>
<td>La Mulatière</td>
<td>50</td>
<td>1.81</td>
<td>5.9</td>
</tr>
<tr>
<td>Seine</td>
<td>Paris</td>
<td>44</td>
<td>0.33</td>
<td>-</td>
</tr>
<tr>
<td>Volga</td>
<td>Volgograd</td>
<td>1350</td>
<td>0.54</td>
<td>2.2</td>
</tr>
<tr>
<td>Weser</td>
<td>Intschede</td>
<td>38</td>
<td>0.73</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Remark: by specific critical discharge is meant the specific discharge which on an average is exceeded once in two years

The main tributaries of the Meuse are: Chiers, Viroin, Semois, Lesse, Sambre, Ourthe, Roer, Niers and Dieze. In fig. 1.2 the height of the river above sea level is shown as a function of distance. Therefore it can be concluded that the Ourthe, Vesdre and Amblève have a great slope and that the Belgian Meuse has a relatively steep slope.

In the relief map (fig. 1.3) it is seen that the Ardennes have a strikingly high level in the catchment. Consequently the annual precipitation is greatest there (fig. 1.4). As a result of the impermeability of the soil the Ardennes contribute considerably to a flood wave.

As far as the hydrologic properties are concerned the Meuse can roughly be split into three parts:
Figure 1.1 The catchment of the Meuse
Figure 1.2 Longitudinal profile of the Meuse and its main tributaries
1. The Upper reaches (Meuse Lorraine or Lotharingian Meuse), from the source at Pouilly-en-Bassigny to the mouth of the Chiers. Here the catchment is lengthy and narrow, the gradient is small and the major bed is wide. Because of that the discharge up to the mouth of the Chiers has a comparatively calm course.

2. The central reaches of the Meuse (Meuse Ardennaise or Ardennes Meuse), leading from the Chiers to the Dutch border near Eijsden. In that section the main tributaries viz. Viroin, Semois, Lesse, Sambre and Ourthe join the Meuse. Here the Meuse transects rocky stone, resulting in a narrow river and a great slope. The poor permeability of the catchment and the steep slope of the Meuse and most of the tributaries contribute to a fast discharge of the precipitation. The contribution of the area to flood waves is great, the contribution to low flows is small.

3. The lower reaches of the Meuse, corresponding to the Dutch section of the river. The lower reaches themselves may again be split into the stretches from Eijsden to Maasbracht and from Maasbracht to the mouth. In the former part the slope is still relatively great. For the greater part the river has no weirs here. In the section the Meuse has no dikes. For those reasons the stretch above Maasbracht is occasionally reckoned as part of the Meuse Ardennaise, which in that case flows from Sedan to Maasbracht. It may be remarked that the stretch that forms the border with Belgium is called the Grensmaas (Border Meuse) in the Netherlands, and Gemeenschappelijke Maas (Common Meuse) in Flanders.

Below Maasbracht the river is provided with weirs to make it navigable. The main tributaries are the Roer, Niers and Dieze. In the Roer reservoirs are found, providing a certain minimum discharge. From Boxmeer the river is a typical lowland stream, with summer dikes, flood plains and winter dikes.
Figure 1.3 Relief map of the Meuse basin. Source: Van der Made (1972)
Figure 1.4 Mean annual precipitation in the Meuse basin. Source: Van der Made (1972)
Chapter 2. Floods

Introduction

A flood arises when in a short period of time the amount of precipitation in the catchment is high. There will have to be a minimum amount of 30 - 40 mm within a few days, dependent among other things on the snowmelt and the baseflow. As far as the Netherlands are concerned the discharge at Borgharen-Dorp is normative. In the Netherlands the term "flood" in the Meuse is formally used if the discharge is greater than the critical discharge. Table 2.1 gives the exceeding frequencies for the discharge at Borgharen-Dorp. In fig. 2.1 the chances of monthly occurrences are given graphically.

In this chapter the consequences of floods will be described in brief. The flood of 1926 will be elucidated in some detail. In conclusion the contributions of canals during floods and types of circulations will be dealt with.

<table>
<thead>
<tr>
<th>exceeding frequency [1/year]</th>
<th>discharge [m$^3$/s]</th>
<th>remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^4$</td>
<td>4370</td>
<td></td>
</tr>
<tr>
<td>$10^3$</td>
<td>3580</td>
<td></td>
</tr>
<tr>
<td>$10^2$</td>
<td>3000</td>
<td>greatest known discharge (1-1-1926)</td>
</tr>
<tr>
<td>$5 \times 10^1$</td>
<td>2790</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1450</td>
<td>critical discharge</td>
</tr>
<tr>
<td></td>
<td>230</td>
<td>mean discharge</td>
</tr>
<tr>
<td></td>
<td>110</td>
<td>mean summer discharge (May-Oct.)</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>smallest known discharge</td>
</tr>
</tbody>
</table>

Sources: Anonymous (Jaarboek der waterhoogten van 1985); Van Mulken (1983).

The consequences of floods

The Meuse Lorraine has a wide river valley. There are hardly any buildings in the valley. Villages and farms are located along the edges of the valley, a few metres above the river valley. In the case of a flood the valley, which is some kilometres wide, is inundated. A great amount of flood water is stored in the inundated valley, where the water flows very slowly. Through those phenomena the discharge at Stenay (fig. 1.1) will hardly ever exceed 550 m$^3$/s.

Further downstream far fewer inundations take place.

At the moment improvement projects are taking place in the Belgian Meuse, aimed at decreasing flood problems in Belgium (chapter 5, chapter 7). It is hoped that before long most of the flood problems will have been solved there.

The tributaries in the Ardennes rapidly respond to great amounts of precipitation (chapter 6), which means that the water level can rise fast. Inundations of a limited extent
Figure 2.1 Distribution of frequencies per month of the discharges at 8 a.m. (8-h discharges) at Borgharen, 1911-1960. Source: Anonymous (1985 d)
Photo 2.1 From top to bottom: The Julianakanaal, the Borgharen weir, the Bosscheveld flood way and the Zuid-Willemsvaart (July 1980). Source: Wiek Ernst Luchtfotografie, Kerkrade
occur. However, in the summer of 1980 the flood caused many caravans placed near the water to be carried downstream.

In the Netherlands many problems arise in particular in the stretch without dikes. In the area, upstream from Boxmeer (near Sambeek, cf. chapter 7), there are several villages and hamlets in the flood plains of the Meuse. The river quays in Venlo and Roermond, too, give rise to problems in the case of a flood.

In former years especially floating ice used to cause problems (chapter 5). A dam of ice used to be formed that could be pushed up so high that the river water flowed over the dikes. In the Dutch Meuse ice has become a rare phenomenon, but in Belgium and France floating ice would occur regularly in the eighties. However, as a result of increasing heat releases the chance of floating ice is diminishing.

The chance of inundations has further been decreased by the digging of the Bergse Maas (Berg Meuse) (1904). Previously the Meuse flowed out into the Waal at Woudrichem. Many floods occurred because of the confluence (chapter 5).

The photos 2.1 to 2.3 illustrate the effects of a great discharge in the Dutch Meuse.
<table>
<thead>
<tr>
<th>station</th>
<th>length&lt;sup&gt;(1)&lt;/sup&gt; [km]</th>
<th>area [km²]</th>
<th>maximum discharge [m³/s]</th>
<th>time of occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neufchâteau</td>
<td>66</td>
<td>862</td>
<td>260</td>
<td>28 Dec. 1925 10h</td>
</tr>
<tr>
<td>Maxey s/Meuse</td>
<td>81</td>
<td>1493</td>
<td>285</td>
<td>28 Dec. 1925 16h</td>
</tr>
<tr>
<td>Commercy (Vignot)</td>
<td>75</td>
<td>2540</td>
<td>311</td>
<td>29 Dec. 1925 21h</td>
</tr>
<tr>
<td>St. Mihiel</td>
<td>39</td>
<td>1415</td>
<td>358</td>
<td>30 Dec. 1925 08h</td>
</tr>
<tr>
<td>Verdun</td>
<td>39</td>
<td>6367</td>
<td>700</td>
<td>1 Jan. 1926 15h</td>
</tr>
<tr>
<td>Bazeilles</td>
<td>39</td>
<td>750</td>
<td>925</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Bazeilles(+Chiers)</td>
<td>39</td>
<td>7705</td>
<td>1040</td>
<td>2 Jan. 1926 16h</td>
</tr>
<tr>
<td>Sedan</td>
<td>415</td>
<td>7831</td>
<td>1280</td>
<td>31 Dec. 1925 17h</td>
</tr>
<tr>
<td>Mézières</td>
<td>415</td>
<td>9180</td>
<td>1500</td>
<td>31 Dec. 1925 18h</td>
</tr>
<tr>
<td>Monthermé</td>
<td>505</td>
<td>10390</td>
<td>1640</td>
<td>31 Dec. 1925 12h</td>
</tr>
<tr>
<td>Monthermé(+Semois)</td>
<td>505</td>
<td>12555</td>
<td>1900</td>
<td>31 Dec. 1925 16h</td>
</tr>
<tr>
<td>Givet</td>
<td>535</td>
<td>15439</td>
<td>2280</td>
<td>31 Dec. 1925 24h</td>
</tr>
<tr>
<td>Anseremme</td>
<td>535</td>
<td>16603</td>
<td>2320</td>
<td>1 Jan. 1926 01h</td>
</tr>
<tr>
<td>Anseremme(+Lesse)</td>
<td>597</td>
<td>20229</td>
<td>2900</td>
<td>1 Jan. 1926 08h</td>
</tr>
<tr>
<td>Namur</td>
<td>615</td>
<td>20366</td>
<td>2950</td>
<td>1 Jan. 1926 11h</td>
</tr>
<tr>
<td>Namur(+Sambre)</td>
<td>627</td>
<td>22221</td>
<td>3000</td>
<td>1 Jan. 1926 15h</td>
</tr>
<tr>
<td>Huy</td>
<td>666</td>
<td>2260</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Huy(+Hoyoux)</td>
<td>695</td>
<td>22221</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Liège</td>
<td>707</td>
<td>3000</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Liège+(Ourthe)</td>
<td>720</td>
<td>3000</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Visé</td>
<td>742</td>
<td>3000</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Maastricht</td>
<td>761</td>
<td>3000</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Meuseik</td>
<td>767</td>
<td>3000</td>
<td>3000</td>
<td>1 Jan. 1926 17h</td>
</tr>
<tr>
<td>Boxmeer</td>
<td>812</td>
<td>29370</td>
<td>2800</td>
<td>4 Jan. 1926 13h</td>
</tr>
</tbody>
</table>

<sup>(1)</sup> The distance to the source has been filled in for the situation of 1990. The distance in 1926 is likely to have been somewhat greater.

<sup>(2)</sup> Anonymous (1985 d), however, gives: Roermond: 2 January 1926 at 23 h.


The Flood of 1926

On 1 January, 1926 the Meuse had its greatest discharge ever known: 3,000 m³/s at Borgharen-Dorp (As a comparison: At Lobith the Rhine got to deal with a discharge of 13,000 m³/s, which for the river has also been the greatest value of the century). Between 6 December 1925 and 19 January 1926 the total volume discharged at Visé was 3.9 km³ (Vereerstraeten, 1969).

The flood could occur after a curious coincidence:
1. a great amount of snowfall in the period between 24 November and 20 December
1925,
2. a sudden rise of the temperature after 20 December 1925. The snow started melting on a large scale,
3. an abnormally great amount of rainfall after 21 December throughout the catchment of the Meuse, and
4. the heavy rains at the end of December accompanied as it were the flood northwards. Vereerstraeten came to the conclusion that the melting of snow was not the main cause of the flood wave. The consequence of the flood wave was that in the Netherlands the flood messages (hoogwaterberichtgeving) were instituted (De Ronde, 1931).

It is striking that the times of peak discharges are considerably influenced by the discharge of the tributaries (table 2.2). The discharges mentioned in the table should be used with some prudence. The peak discharge, for one thing, is unlikely to increase by ten percent in the case of an increase of the contributing area between Mézières and Monthermé of half a percent.

In 1926 the maximum discharge for the Meuse Lorraine was 460 m$^3$/s, so the rate of 550 m$^3$/s was not reached.

If the same weather conditions occurred in the present situation the wave travel times
and peak discharges would be different, among other things because, through the extension of flow profiles, there would be fewer inundations and because the Meuse has become shorter because of cut off bends.

<table>
<thead>
<tr>
<th>Type of circulation</th>
<th>mean number of occurrences per year</th>
<th>the chance that the GWL occurs on the peak day and the previous five days [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>year</td>
</tr>
<tr>
<td>WZ</td>
<td>15.2</td>
<td>35.3</td>
</tr>
<tr>
<td>WS</td>
<td>3.3</td>
<td>11.4</td>
</tr>
<tr>
<td>SWZ</td>
<td>1.6</td>
<td>7.9</td>
</tr>
<tr>
<td>NWZ</td>
<td>4.5</td>
<td>7.7</td>
</tr>
<tr>
<td>TRM</td>
<td>3.9</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28.5</td>
</tr>
</tbody>
</table>

**WZ** West circulation, low pressure zone over Central Europe
Depressions and intermediate highs move to Eastern Europe via the British Isles, the North Sea and the Baltic. The central low is situated near Iceland. The Azores high has a tail end via Spain to the Alps. The precipitation is protracted and plentiful, alternating with bright spells, with frontal thunderstorms in summer.

**WS** Southern west circulation (low pressure zone)
The frontal zone has moved a long way to the south. The depressions move eastward across France and Southern Germany. The "steering" low lies northwest of the British Isles. The rainfall is abundant, in winter there may be heavy snowfall over Northern Germany and the Netherlands.

**SWZ** Southwest circulation, low pressure zone over Central Europe
Fronts move to Northern Russia via the English Channel and the North Sea. The "steering" area of low pressure lies between Ireland and Iceland. Secondary depressions may move eastward across the British Isles or the Bay of Biscay. In winter, too, the precipitation falls as rain, which can be very violent. In summer the quantities of precipitation are smaller.

**NWZ** Northwest circulation, cyclonal over Central Europe.
Between a high west of the Bay of Biscay and a depression over Scotland, the Northern North Sea and Scandinavia active fronts move across the British Isles and via the North Sea to Southeastern and Eastern Europe. The precipitation is heavy, partly in the shape of showers, in winter as snow.

**TRM** Trough over Central Europe
The trough situation is characterised by a very low air pressure, far behind a cold front of a depression. North of the Baltic lies a high. Fronts move to the southeast across Western and Central Europe. The precipitation comes down in showers, in winter as snow. Most precipitation falls over Eastern Europe.

Source: Van der Spek (1985)
The contributions of canals in cases of floods

Normally the discharge via the Julianakanaal (Juliana Canal) and the Albertkanaal (Albert Canal) amounts to approximately 16 and 9 m³/s respectively (chapter 7). In the case of a flood that discharge is not much greater. Neither is it possible to make those canals contribute significantly (some hundreds of m³/s) to the discharge of the Meuse, since apart from several constructive problems it is impossible physically, too. In a canal the slope only numbers some centimetres per kilometre, which keeps the discharge moderate. As a comparison, the slope in the Grensmaas amounts to almost 50 cm per kilometre (Anonymous, 1985 b).

The relation between floods and types of circulation

The chance of the occurrence of floods has a definite correlation with atmospheric types of circulation (weather types). If it is known whether a specific type of circulation will occur it will also be possible to pronounce upon the chance of the occurrence of a flood within the given period.

Applying Bauer’s Grosswetterlage System Van der Spek (1985) drew the conclusion that 5 out of 29 possible types of circulation (Grosswetterlagen, GWL) were responsible for 69% of the floods having occurred at Borgharen (which he defined as: mean daily discharge at Borgharen greater than or equal to 1,000 m³/s, or an increase of the discharge of more than 400 m³/s in 5 days or other considerable discharges). In table 2.3 the chances of a flood in a definite type of circulation have been given (period of reference 1947-1981).

It is striking that most circulations mentioned point to currents from the west. Currents from the southwest have a greater chance of floods than those from the northwest. It will also be caused by the fact that in case of southwest circulations the rain zone will more or less accompany the flood wave.
Chapter 3. Low flows

3.1 THE OCCURRENCE OF LOW FLOWS

After a period of little if any precipitation the discharge of the Meuse is likely to decrease so strongly that the discharge drops below 50 m³/s at Monsin (north of Liège, before the branching off of the Albertkanaal). In the Netherlands such a situation is formally called low water. In table 3.1 the periods are given in which the discharge at Monsin was less than 50 m³/s.

From the table it can be deduced that 1976 was a particularly dry year. In § 3.4 the year will be dealt with to some more detail.

<table>
<thead>
<tr>
<th>Year</th>
<th>Number of days</th>
<th>Year</th>
<th>Number of days</th>
</tr>
</thead>
<tbody>
<tr>
<td>1911</td>
<td>3</td>
<td>1955</td>
<td>33</td>
</tr>
<tr>
<td>1919</td>
<td>4</td>
<td>1959</td>
<td>90</td>
</tr>
<tr>
<td>1921</td>
<td>142</td>
<td>1962</td>
<td>20</td>
</tr>
<tr>
<td>1928</td>
<td>2</td>
<td>1963</td>
<td>3</td>
</tr>
<tr>
<td>1929</td>
<td>5</td>
<td>1964</td>
<td>89</td>
</tr>
<tr>
<td>1933</td>
<td>23</td>
<td>1967</td>
<td>23</td>
</tr>
<tr>
<td>1934</td>
<td>115</td>
<td>1969</td>
<td>36</td>
</tr>
<tr>
<td>1935</td>
<td>25</td>
<td>1970</td>
<td>2</td>
</tr>
<tr>
<td>1938</td>
<td>35</td>
<td>1971</td>
<td>101</td>
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<td>1973</td>
<td>90</td>
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<tr>
<td>1942</td>
<td>12</td>
<td>1974</td>
<td>43</td>
</tr>
<tr>
<td>1943</td>
<td>35</td>
<td>1975</td>
<td>22</td>
</tr>
<tr>
<td>1944</td>
<td>21</td>
<td>1976</td>
<td>167</td>
</tr>
<tr>
<td>1945</td>
<td>10</td>
<td>1977</td>
<td>24</td>
</tr>
<tr>
<td>1946</td>
<td>6</td>
<td>1978</td>
<td>22</td>
</tr>
<tr>
<td>1947</td>
<td>106</td>
<td>1979</td>
<td>27</td>
</tr>
<tr>
<td>1949</td>
<td>77</td>
<td></td>
<td></td>
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<tr>
<td>1952</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1953</td>
<td>34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1954</td>
<td>9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source: De Wildt (1983a)

In the case of low flows Monsin is generally taken as the reference station instead of Borgharen-Dorp because the discharge at the latter is no longer the undivided Meuse discharge, since in the periods of low flows the Albertkanaal, the Zuid-Willemsvaart (Southern William Canal) and the Julianakanaal extract a considerable percentage of water. so that at Borgharen-Dorp the Meuse only receives a part of the Monsin discharge (fig. 3.1). However, the drawback for choosing Monsin is that for that location no discharge is known. The local discharge there is calculated by means of the one for Borgharen-Dorp. In doing so a
Figure 3.1 Average discharges through the canals near Maastricht according to several studies
certain extraction for the canals has been taken. The value has increased through the years. For some years the discharge at Monsin was determined by means of flow rates through locks.

3.2 ARTIFICIAL EXTRACTIONS AND DISPOSALS

Certainly during low flow conditions the total discharge volume of the Meuse is influenced by artificial extractions and disposals. Much of the water extracted from the Meuse is discharged into the river afterwards. However, there are also extractions that are not followed by discharges into the river. An inventory of the extractions from and discharges into the river will follow below. The extractions in the Netherlands are somewhat more specified than those in other countries.

- Along the Meuse there are a number of power stations and factories using Meuse water for cooling. After using it they discharge the water back into the Meuse, though with a higher temperature. Thus there is some additional evaporation because of which the discharge of the Meuse decreases. The main dischargers of heat are: the nuclear power-station at Chooz (France), the nuclear power-station of Tihange (near Amssin-Neuveille), the Claus power-station (near Maasbracht), the Meuse power-station (near Buggenum, downstream from Roermond) and the Amer power-station near Geertruidenberg on the Amer. All the stations with the exception of the Buggenum one have cooling towers. They are mainly used as a means to discharge their heat, in spite of the small discharges, without causing the temperature to rise too much. If compared with other quantities extracted artificially the total quantity of water to be evaporated may be called small and amounts to less than about 1 m³/s in most cases.

- The drinking-water supply of Brussel extracts water: On an average 3 m³/s (Derycke et al., 1982). The extraction takes place near Tailfer (fig. 7.5).

- The canals around Liège and Maastricht extract a considerable flow rate from the total discharge of the Meuse. In several studies it has been attempted to estimate the flow rates (Van Craenenbroeck et al., 1986, Laurent et al., 1986, Meulenberg 1986, Zuiderveen Borgesius, 1981). The values estimated show a relatively close resemblance (fig. 3.1). The discharge into the canals especially takes place by way of lockages, on account of which the flow is strongly non-stationary. The effect is even reinforced by the fact that on Sundays and holidays there are hardly any lockages.

In 1979 from July to October the daily discharges were determined (measured and/or calculated) for a great number of places in the area considered. In fig. 3.2 the results for several places have been given.

- For this investigation the average values are taken:
  - Lanaye Locks: 8 m³/s
  - Albertkanaal upstream from Lanaken: 9 m³/s
  - Zuid-Willemsvaart (border): 14 m³/s
  - Julianakanaal at Bunde: 16 m³/s

- The DSM factory situated near Sittard and Geleen extracts approximately 1.5 m³/s from the Julianakanaal downstream from Bunde (Zuiderveen Borgesius, 1981). In the future the extraction may be doubled. About 75% of the extraction is drained into the Grensmaas.

- The Kanaal Wessem-Nederweert (Wesselem-Nederweert Canal) discharges approximately 1 m³/s into the Meuse for the greater part of the year, but in dry periods it may turn into an extraction of 2 m³/s.
Figure 3.2 Mean discharges for the various canals in the period from July to October 1979. Data supplied by the Ministère des Travaux Publics, Belgium
- The Dutch drinking-water companies of Andel and the Biesbosch extract 1 and 2.5 m³/s respectively. In the future it may increase to from 2 to 4 m³/s and from 6 to 16 m³/s respectively (Zuiderveen Borgesius, 1981).

- In Dutch Limburg and Noord-Brabant drinking-water extraction mainly takes place from groundwater. Possibly two riverbank infiltration projects are also going to play an important part in the future. The estimates are: Bank Infiltration Roosteren approximately 1 m³/s, Bank Infiltration Maaskant from 1 to 2 m³/s.

For lack of data and due to a continually altering situation the list makes no pretensions to completeness. A number of conventional power stations and steel works in Belgium have not been entered into the list. Tributaries, too, have been left aside. The use of water by cooling towers is calculated by assuming an evaporation of 0.6 m³/s for every 1000 kW power of the station (Zuiderveen Borgesius, 1981). Apart from that it is not realistic to assume that in periods of very low discharges the power stations will be operating to capacity, because low flows especially occur in summer and the highest use of energy occurs in winter.

3.3 CONSEQUENCES OF AND MEASURES DURING LOW FLOWS IN THE NETHERLANDS

When the discharge at Monsin amounts to less than 50 m³/s the normal distribution of water can no longer be continued. With discharges greater than 50 m³/s a discharge of approximately 20 m³/s is extracted from the Meuse via the Albertkanaal (almost half of it returns to the Meuse before Maastricht). In the Netherlands water is transported to Belgium by way of the feeding culvert and the Bosscheveld lock in the Zuid-Willemsvaart (approximate total of 14 m³/s). One more part (approximately 16 m³/s) flows to Maasbracht by way of the Julianakanaal and returns to the Meuse there. The discharge that is left flows on through the Grensmaas.

If the discharge is going to fall short of 50 m³/s it is insufficient to comply with the wishes of shipping, agriculture, industry, drinking-water supply and the minimum ecological need of the Grensmaas. The problems are most pressing upstream from the mouth of the Roer, as the reservoir in that river produces a certain minimum discharge (approximately 10 m³/s, cf. also § 6.8).

The 1863 Maas Tractaat (Meuse Treaty) (with a supplement in 1873) does not allow a decrease of the discharge to Belgium by way of the Zuid-Willemsvaart. In the Maas Tractaat, which is still operative in 1991, the Netherlands and Belgium agree that the Netherlands are obliged to deliver a definite quantity to Belgium via the Zuid-Willemsvaart (in practice 14 m³/s). In turn Belgium is obliged to make part of it (in practice 5 m³/s) return to the Netherlands at Lozen (cf. chapter 5).

Measures can be taken at Dutch weirs and locks. In this context the following points are being considered:

- The buffer capacity of the Dutch headwater reaches can be enlarged by increasing the headwater level.
- The weirs in the Dutch Meuse have a certain loss through leakage (30 m³/s in 1976, except for Borgharen and Lith, source: Zuiderveen Borgesius (1981). It should be noted that over the time the loss through leakage varies considerably). Most problematic is the situation in the headwater reach of Linne (fig. 7.5). Water is supplied through the Grensmaas and the Julianakanaal. It may be remarked that the water in the Julianakanaal can flow southward if the water at the locks of Born and Maasbracht is pumped back. The extractions from the Linne headwater reach are
formed by the Kanaal Wessem-Nederweert, the locks at Heel (approximately 8 m³/s, Zuiderveen Borgesius (1981)), the locks at Linne and from February 1977 onwards by the cooling towers of the Claus power station at Maasbracht. In dry periods the discharge by way of the Poirée parts of the weirs of Roermond, Belfeld and Sambeek can be restricted by sealing the weir with cotton waste and coal ashes. The cotton waste can be applied by divers. Immediately afterwards the coal ashes are dumped right upstream from the weir in order to stop the remaining leaks. The Stoney parts of the weirs can be sealed by means of sandbags. In that way the leak is reduced to approximately 2 m³/s. In connection with the construction of a hydro-electric power station the Poirée shutters and the Stoney slides of the Linne weir have been provided with a rubber sealing, so that the leakage always remains small.

The different construction of the Grave weir necessitated a different solution there. The height of the I-profiles in which the shutters are placed is larger than the thickness of the shutters. In the remaining room wooden beams can be pressed.

In the weirs of Borgharen and Lith leakage is restricted to some cubic metres per second.

- The discharge via the locks can be limited by locking ships only when the lockage chamber is filled with ships to its maximum capacity.
- One more measure of restricting the discharge through the locks is the so-called siphoning lockage. In that process water is exchanged between the chambers. The loss of water of the lock is reduced by some 40%.
- The discharge can still be decreased by restricting the permitted draught of ships.
- The water can be pumped back to the higher section.
- The water supply of the Kanaal Wessem-Nederweert (Wessem-Nederweert Canal) can be maintained by pumping at Panheel.

From the measures it can be deduced that shipping has the highest priority. However, if the pumps in the Julianakanaal are switched on too late the discharge in the Grensmaas will temporarily be very small, which will have harmful consequences for the ecological environment.

The measures are such that their execution will need some time of preparation. For example, not in all the relevant locks there are pumps. It takes a week at least to install them, which among other things is a good reason for having a good low flow forecast.

3.4 THE LOW FLOWS OF 1976

The summer of 1976 was exceptional in many respects: long, sunny and hot and extremely dry (meteorologically, the spring as well as the summer and the autumn were the driest in the past 200 years). The previous years, too, had been drier than normal (with the exception of the autumn of 1974). At the start of the low flow period the groundwater reserves had already been lower than normal. In that summer the discharge mainly came from the groundwater reserves, on account of which the measured discharge had the nature of a depletion hydrograph. In table 3.2 the discharges of some stations are given. Table 3.3 shows that in that summer the open water evaporation was much greater than the precipitation (The open water evaporation has been calculated according to the Penman method).

In the Netherlands the greatest problems presented themselves upstream from Linne. More downstream the discharge was increased by the Roer, some smaller brooks and by
TABLE 3.2 THE DISCHARGES OF SOME STATIONS IN 1976

<table>
<thead>
<tr>
<th>Month 1976</th>
<th>Stenay</th>
<th>Chooz</th>
<th>Borgharen-Dorp</th>
<th>Lith</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>77</td>
<td>127</td>
<td>239</td>
<td>325</td>
</tr>
<tr>
<td>February</td>
<td>122</td>
<td>161</td>
<td>261</td>
<td>350</td>
</tr>
<tr>
<td>March</td>
<td>43</td>
<td>85</td>
<td>117</td>
<td>182</td>
</tr>
<tr>
<td>April</td>
<td>21</td>
<td>54</td>
<td>64</td>
<td>113</td>
</tr>
<tr>
<td>May</td>
<td>18</td>
<td>36</td>
<td>28</td>
<td>69</td>
</tr>
<tr>
<td>June</td>
<td>10</td>
<td>24</td>
<td>18</td>
<td>37</td>
</tr>
<tr>
<td>July</td>
<td>8</td>
<td>17</td>
<td>17</td>
<td>20</td>
</tr>
<tr>
<td>August</td>
<td>5</td>
<td>13</td>
<td>2</td>
<td>13</td>
</tr>
<tr>
<td>September</td>
<td>8</td>
<td>15</td>
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<td>21</td>
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<tr>
<td>October</td>
<td>13</td>
<td>20</td>
<td>4</td>
<td>34</td>
</tr>
<tr>
<td>November</td>
<td>18</td>
<td>30</td>
<td>23</td>
<td>72</td>
</tr>
<tr>
<td>December</td>
<td>93</td>
<td>130</td>
<td>144</td>
<td>208</td>
</tr>
</tbody>
</table>

groundwater. The discharge through the locks was smaller in that stretch owing to a more limited drop at weirs.

Throughout the year the Roer maintained a discharge of 10 m³/s at least. The storing capacity of the reservoirs had decreased to 20% of the normal capacity at the end of the low flow period.

The headwater reach of Borgharen received a minimum of 9 to 10 m³/s because of leakage in the Visé weir (liquidated meanwhile) (Zuiderveen Borgesius, 1977). Moreover there was a supply through the Jeker, Berwinne and Voer, the discharge of the lock of Lanaye and the afflux of groundwater. The discharge from the headwater reach mainly occurred through the Grensmaa, the Julianakanaal, the Zuid-Willemsvaart and through the extraction at DSM. The discharge through the Borgharen weir was kept to a minimum (approximately 1.5 m³/s).

TABLE 3.3 PRECIPITATION AND OPEN WATER EVAPORATION IN 1976 AT BEEK

<table>
<thead>
<tr>
<th>month</th>
<th>precipitation 1976 [mm]</th>
<th>open water evaporation 1976 [mm]</th>
<th>mean precipitation [mm]</th>
<th>mean open water evaporation [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>87</td>
<td>11</td>
<td>57</td>
<td>7</td>
</tr>
<tr>
<td>February</td>
<td>31</td>
<td>16</td>
<td>60</td>
<td>19</td>
</tr>
<tr>
<td>March</td>
<td>25</td>
<td>48</td>
<td>68</td>
<td>45</td>
</tr>
<tr>
<td>April</td>
<td>18</td>
<td>82</td>
<td>61</td>
<td>79</td>
</tr>
<tr>
<td>May</td>
<td>32</td>
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<td>June</td>
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<td>125</td>
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<tr>
<td>July</td>
<td>60</td>
<td>144</td>
<td>88</td>
<td>121</td>
</tr>
<tr>
<td>August</td>
<td>18</td>
<td>117</td>
<td>75</td>
<td>100</td>
</tr>
<tr>
<td>September</td>
<td>96</td>
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<tr>
<td>October</td>
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<td>33</td>
<td>63</td>
<td>31</td>
</tr>
<tr>
<td>November</td>
<td>83</td>
<td>11</td>
<td>84</td>
<td>13</td>
</tr>
<tr>
<td>December</td>
<td>57</td>
<td>2</td>
<td>55</td>
<td>6</td>
</tr>
</tbody>
</table>

33
consultation with the Belgian authorities the discharge through the Zuid-Willemsvaart could be decreased to 8 m³/s (in reality approximately 12 m³/s). For the Julianakanaal first locking limitations became operative, afterwards pumping plants were to be set into action at Born and Maasbracht.

The headwater reach of Linne received water from the locks at Maasbracht and at Panheel and from the Grensmaas. Near Maasbracht the discharge from the Grensmaas was from 8 to 9 m³/s. Zuiderveen Borgesius estimates that half of it came from the brooks Geul, Ur (effluent of the DSM-factory), Bosbeek, Groenbeek, Geleenbeek and Molenbeek; the contribution of groundwater was estimated to be about 30% of the discharge; and the discharge at Borgharen (only through leakage) to 1 to 2 m³/s. The discharge leaving the headwater reach arose because of: the Kanaal Wessem-Nederweert (pumping-engine at the Panheel lock) (3 m³/s), the pumps at Maasbracht for the Julianakanaal, the Lateraal Kanaal (lock at Heel) and the Meuse (weir and lock at Linne). The discharge by way of the Lateraal Kanaal (Lateral Canal) could be decreased by limiting the opening hours of the lock. During the closed periods shipping could use the former route by way of Roermond (through two locks, both with a smaller drop). Normally the discharge through the Heel lock amounts from 6 to 8 m³/s, through the Linne one from 2 to 3 m³/s. It should be noted that in the headwater reach no serious problems occurred, on account of:
- a margin between the normal headwater level and the critical level (NAP + 20.90 m as opposed to NAP + 20.10 m),
- a large storage of water in the gravel pits,
- decreased shipping, and
- the measures taken.

In Roermond the Roer (minimum 10 m³/s) joins the Meuse (minimum 3 m³/s). The leakage through the weirs higher upstream could be reduced by sealing them.
Chapter 4. The origin of the Meuse basin and the Meuse

4.1 INTRODUCTION

The subdivision of the Meuse basin into Meuse Lorraine, Meuse Ardennaise and Dutch Meuse is based on geology. The three parts mentioned clearly have a different geological history.

In this chapter the origination of the Meuse basin will be described as well as the origin of the river. That description is very concise, both chronologically and geographically. It has been tried to focus the attention on those aspects which are relevant to the hydrologic system.

4.2 THE MEUSE BASIN IN A GEOLOGICAL PERSPECTIVE

In geographical history there have been several periods in which big mountain ranges were developed and afterwards were demolished again by erosion. Since the Cambrian three of those formations of mountains have occurred: the Caledonian, the Hercynian and the Alpine one. The cause of the formation of mountains lies in the tectonics of plates (Anonymous, 1985a).

The Caledonian Mountains rose during the Cambrian (table 4.1) between present Wales and Norway. Large parts of Europe remained under the sea level, although mountains were formed there, too. Notably in the Ardennes there is a Caledonian fold belt, at present cropping out in several places. The Stavelot Massif in the Amblève and Vesdre catchments may be mentioned as well as the Rocroi Massif in the south of the Viroin catchment and the adjoining valley of the Meuse. West of them was the Brabant Massif extending from London by way of the (Belgian) province of Brabant to Maastricht. The rock deposited in the Cambrian hardly allows any water to pass (Goossens, 1984).

During the Devonian sedimentation in the present Meuse area took place especially through the erosion of the Caledonian mountains and the formation of limestone by marine life. Several parts of the Meuse basin jutted out from the surface of the sea during the Devonian. In several places in the Ardennes Devonian deposits crop out. In the palaeozoic limestone sediments karstic caves came into being through the dissolution of limestone in later periods.

In the Carboniferous the formation of the Hercynian Mountains began. The mountains extended from the Appalachians in North America (then forming a unity with Western Europe) across the south of Ireland, Britain, the larger part of France, the south of Belgium and parts of Germany and Czechoslovakia. The coastline was just south of the line formed by the Meuse and Sambre. The disintegrated products were carried north and deposited in shallow seas.

During the formation of the Hercynian or Varistic Mountains the Ardennes region became subjected to pressure, the Brabant Massif causing counterpressure. The formation of the Namur syncline (through which now the Sambre and the Meuse flow) and the Condroz anticline were important for the Meuse basin. Apart from folds there also came faults. Like the folds their orientation is west and east; in the Ardennes, however, their orientation is from south-west to north-east. The largest fault is the Midi Heave, in which the Dinant basin was shoved across the Namur basin. The heave is over 200 km long and runs from the North of
### TABLE 4.1 THE GEOLOGICAL ERAS

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<tr>
<th>Eon</th>
<th>Era</th>
<th>Period</th>
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<td>Archaic or Azoic</td>
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France to Aachen by way of Charleroi, Namur and Liège. Near Namur it is called the Condroz Heave, near Liège the Eifel Heave. As a side effect a number of faults came into existence in Dutch Limburg and the adjoining German border region. However, most faults appeared in the region in the Tertiary.

Metamorphism (recrystallisation through physical phenomena) under high pressure and at high temperatures only occurred in a few places in the Ardennes. Consequently the influence on the hydrologic system is small.
Figure 4.1 Europe at the end of the Carboniferous Period. Source: Goossens (1984)

A diagram of the situation at the end of the Carboniferous Period is shown in fig. 4.1. In the same figure the present form of Europe has been drawn.

During the Permian the Hercynian Mountains were disintegrated at a great pace. The sediment, mostly of a red colour, is only to be found to a small degree in the Meuse basin, because the soft sediment has been eroded away in many places. Between Coo and Robertville Permian sedimentations are still found in the wide valley (the Malmédy Graben). In that period north of the present Meuse basin salt was formed on a large scale through the evaporation of seawater. Owing to the shifting of parts of the crust across the earth the present Meuse basin passed the equator in the Permian and has since lain on the northern hemisphere.

During the Triassic the Hercynian Mountains had been eroded away to such an extent that the greater part of Central and Western Europe had reached the sea level again. In that period the Meuse basin regularly changed from land into sea. There were several kinds of sedimentations. As those sediments have a deep location or have been eroded away in the meantime they are rather insignificant for hydrology.

During the Jurassic the Brabant Massif was an island. Clay as well as calcareous clay and sand were deposited in the surrounding sea. Nowadays lime and marl crop out in the north of Lorraine, a part of the Paris basin. The sediment dates from the Dogger epoch.

The Meuse Lorraine basin also possesses the very permeable stratum from the Dogger period. However, it is covered with a stratum of very impermeable clay of Woëvre (Oxfordian, a subepoch of the Malm) with very porous calcareous soil (also Oxfordian) on top. The sediments crop out in the part of the Meuse basin lying in Lorraine (with the exception of the Meuse valley itself). The cropping out is brought about by the Alpine
formation of mountains, which pushed up Lorraine but especially the Alsace. As a result the upper strata have been eroded away (fig. 4.2) (Picard, 1982).

During the Cretaceous inundation of the Meuse basin occurred again, leading to the formation of limestone. In the limestone no speleological formation took place. In the Northern Meuse basin the sediment lies over 1000 m under the surface.

In the Tertiary the Alpine upheaval took place. Through the consequential sideway pressure the Ardennes lowland plateau was also lifted, cf. fig. 4.3. The elevation has not been finished yet.

The Netherlands were situated in a sinking basin. In the Dutch Meuse basin a great number of faults was formed running from northwest to southeast. Owing to the descent the basis of tertiary deposits is nowadays to be found at great depths, up to 2000 m between Eindhoven and 's-Hertogenbosch in the Centrale Slenk (Central Graben). The deposits are several kinds of sand and of clay, alternately of a marine and a continental origin. In the Roerslenkdal (Roer Valley Graben), the extension of the Centrale Slenk in a southeasterly direction, peat is deposited. The deposition took place mainly in the southeasterly corner of the Roerslenkdal, west of Köln. Nowadays the deposits are extracted as brown coal for the production of electricity.

The Pleistocene is the epoch of the glacial periods. Before that time there had been glacial periods indeed, yet the glacial periods of the Pleistocene have been highly significant for the present shape of Northwestern Europe and of the Meuse basin.

Since in the glacial periods water was stored in the form of ice in great quantities, in those times the sea level was much lower than in the so-called interglacial periods. Consequently the eroding activity of rivers was much greater. Moreover great quantities of meltwater and debris were discharged during the glacial periods.

At the end of the glacial period the sea level rose and the great discharges decreased. The debris carried along could be deposited in the eroded valleys so that the so-called alluvial plains came into being. A new incision took place in the next glacial period.

Through the falling of such graben as the Centrale Slenk the Meuse and the Rhine could get rid of their debris in them. In the south (Limburg) the deposit consists of gravel for the greater part, more to the north (from Noord-Brabant) with more sand, clay and loam (Zagwijn et al., 1975). At present a great number of strata is used for collecting water.

In the Saline the lower stretches of Meuse and Rhine are bent from north to west by an ice cap pushing southward. The ice cap came approximately halfway through the Netherlands. The westbound direction of the lower stretches of Meuse and Rhine continues at the moment (§ 4.3).

Because of the great quantities of ice falling masses of air came into being over the ice cap, and as a result also areas of high pressure and stormy winds. In the dry plains and near the heaps of rubble from glaciers the winds loosened grains of loam and sand. In places where the material was deposited loess and surface sand were formed. Windborne sand is to be found in Noord-Brabant. Due to the smaller size of its grains loess was carried further southward and is to be found as far as in South France. In general the strata are no thicker than a few metres, though in places they are up to 20 m.

In the Holocene the Meuse virtually found its present course. In the river valley in the Netherlands, which is a few kilometres wide, some fine-grained material is still being deposited. In the delta the strip has a width of some dozens of kilometres.
Figure 4.2 Highly simplified geological profile between Commercy and Strasbourg. Source: Picard (1982)
Holocene deposits are furthermore found in the Meuse valley between Neufchâteau and Charleville-Mézières and in the lower course of the Chiers, both of which can rapidly receive and pass on river water and on that account play an important part in the hydrologic system.

4.3 THE ORIGIN OF THE MEUSE

Considering the present course of the Meuse it is a striking thing that in the Ardennes the river flows through hard rock. Especially south of Namur it is conspicuous that the river does not adapt itself to the geological structure, but bores its way right through it. There are various hypotheses on the manner in which the present course through the Ardennes came into being. Presumably the two hypotheses that will be treated here are both right, but in different places (Goossens, 1984).

There is less uncertainty about the origination with respect to the other parts of the present course.

The French and the Meuse Ardennaise will be dealt with first, the attention mainly being focused on the latter. Subsequently the origination of the Dutch Meuse, which is of a later date, will come up.

In the first hypothesis on the origin of the Meuse Ardennaise, the antecedent hypothesis, it is supposed that the Meuse found its course when the Ardennes had not been elevated yet. In Belgium the river had about the same course as it has nowadays, but consequently ran through the Ardennes low plateau. When halfway through the Tertiary the plateau began to ascend the river cut itself into it. It is supposed that the incision was equally fast as the elevation of the plateau so that the original course was maintained. The hardest rocks such as the Caledonian rocks of the Rocroi Massif in the north of France are indicative of it. If the Meuse eroded those stones less fast than the elevation occurred a threshold would be formed so that the Meuse had to follow a different track.
Figure 4.4 Decapitation of the Meuse. Source: Goossens (1984)

The second hypothesis is the so-called penetrating hypothesis. It departs from the idea that the present course came into existence after many penetrations.

A penetration is the phenomenon that a river changes the direction of its course because it breaks through a catchment border and thus ends up in the valley of another river (or tributary). A significant cause is the receding erosion of the river (or tributary), on account of which the erosion can assume great proportions in the upper reaches, too. Thus the section between two rivers can more or less be eaten away. Finally the river breaks through the remaining section and has got a different course.

A well-known example is the penetration of the Moselle near Toul (fig. 4.4).

At first the upstream part of the Moselle used to be a Meuse tributary. East of the Moselle ran the Meurthe, belonging to the Rhine catchment. Because of the receding erosion of a Meurthe tributary the area between that tributary and the Moselle grew smaller and smaller. Moreover, the upper reaches of the Moselle got blocked up by great quantities of rubble that had landed in the valley during the Mindel glacial period. Ultimately the upstream stretch of the Moselle ran into the Meurthe, so that the Meuse lost a considerable part of its catchment area.
From the occurrence in the Meuse basin of sediment that could only have come from the present Moselle basin it has been possible to deduce that the penetration has actually taken place. The penetration is known as the decapitation of the Meuse.

At Toul there remained a low point in the new catchment borders. On that account a country road was constructed here in Roman times. In the nineteenth century the Canal de la Marne au Rhin (Marne-Rhine Canal) was built there.

Furthermore the Meuse had to give up territory to the Aisne, which took over a long strip on the west side of the catchment area. That penetration, too, may be looked upon as proven.

The penetrating hypothesis assumes that initially Meuse and Scheldt flowed out together into the sea southwest of Brussel. From Sedan the Meuse is supposed to have had a different track. By way of the present little tributaries Virigne and Goutelle the Meuse is thought first to have flow more to the east and then via Nouzonville and the Petite Helpe to the west (fig. 4.5). There is rather much certainty about the former part of the track, among other things because of the wide valleys of the two small rivers. Of the latter there are only indications.

Through a number of receding erosions the Meuse is thought to have changed its course. The first is a receding erosion of a tributary of the Viroin, which made the Semois change its course. Subsequently a tributary of the new Semois is assumed to have penetrated

![Figure 4.5 Possible course of the Meuse at the end of the Miocene. Source: Goossens (1984)](image_url)
the Meuse at Nouzonville, on account of which the Meuse began flowing northwards. Through a penetration of a small tributary of the Bar the Meuse started following the track by way of Charleville-Mézières. Via the Gete the Meuse flowed out into the sea.

In the meantime the Basse Sambre extended its territory with e.g. the Eau d'Heure, which was lost by the Dijle. The Basse Meuse penetrated the Mehaigne and the Hoyoux through receding erosion near Liège. A small tributary of the Mehaigne and Hoyoux penetrated the Samson and finally a small tributary of the Hoyoux penetrated the Sambre and Haute Meuse. The series of penetrations is explained from the fact that they are all situated in the syncline of Namur, which consisted of relatively soft material. Nowadays the Namurian shales have totally been eroded away.

The Haute Ourthe was penetrated by the Ourthe.

According to the penetrating theory the Meuse is assumed to have found the greater part of its present track at the end of the Tertiary.

Afterwards the Haute Sambre, too, joined the catchment of the Basse Sambre.

It is not (yet?) possible to ascertain whether the Meuse Ardennaise has been formed in accordance with either of the hypotheses or with a combination of the two. For geologists the origination of the Meuse remains an interesting field of investigation.
In the Quaternary the lowest part of the course was formed.

Below Liège the Meuse ran on to Aachen and Jülich, where it flowed out into the Rhine. The Rhine ran onwards in a northwesterly direction, flowing out into the sea after some dozens of kilometres (fig. 4.7).

In the Elster glacial period that situation was changed. Great quantities of rubble blocking the lower stretch were carried along by the meltwater. In Noord-Brabant and Limburg the earth’s crust tipped over resulting in faults and a system of horsts and graben being formed (fig. 4.8). Probably the process also caused the fact that the Meuse broke through the system of horsts and graben and started its northward course. The Meuse and Rhine became two separate rivers.

The rubble was then deposited in the new estuary (fig. 4.9). The western part of the territory is known as the Kempisch Plateau (Campine Plateau). For the greater part the eastern section was again eroded away by Meuse and Geul. The rubble consisted both of gravel and of sand.

In the subsequent glacial periods the heap of rubble was again incised as a result of the erosion by the lower sea level and a fresh stratum of rubble could be deposited downstream. In this way there arose terraces resulting from several glacial periods, the highest of which is the oldest. The terraces are to be found close to the present Grensmaas.

After the Elster glacial period the fault of the western Rhine Graben sank no more. The adjoining fault (Centrale Slenk) did sink further. Because of the coarser nature of the gravel it was not eroded away afterwards, such at the cost of the nearby sandy soil. That is the reason why the Kempisch Plateau has a higher position than the surrounding sandy soil.
Figure 4.8 The system of faults in the Meuse delta. Source: Anonymous (1985a)

Figure 4.9 The Meuse pushes the Rhine eastward. Source: Goossens (1984)
Figure 4.10 The course of Meuse and Rhine during and after the Saline. Source: Goossens (1984)

In the Saline the Drenthe ice cap pushes the course of Meuse and Rhine westward (fig. 4.10).

4.4 CONSEQUENCES FOR HYDROLOGIC MODELLING

Geological history has great consequences for the discharge regime of the Meuse.

In its upper course the Meuse flows through rather permeable calcareous soils and alluvial deposits. The gradient is restricted by the Ardennes lying downstream. For that reason the contribution of the upper stretch to the greatest discharge at Borgharen-Dorp will in many cases be relatively small during floods.

From the Chiers the catchment is situated on impermeable aged rocks. Consequently the gradient of the river and of the tributaries is great. The two facts combined cause a fast runoff, resulting in great flood discharges and small low flows.

Downstream the Meuse flows through its own deposits. Owing to the present high position of the sea level the course is very even. The deposits are rather permeable. Therefore the contributions of the delta area to flood discharges are relatively limited.
Chapter 5. History of hydraulic-engineering works and shipping in the catchment of the Meuse

5.1 INTRODUCTION

For man a river has a multiple of functions. First of all it looks after the discharge of superfluous water. It also creates the possibility of traffic by means of ships. A river, however, also constitutes an obstacle to traffic on land, so that it can also get a military significance. Provided river water is sufficiently present and of a good quality it can be used for the production of drinking water and the irrigation of fields. River water can also be used for fishing, recreation and the generation of electricity. Moreover, rivers play an important part in discharging waste products. Apart from those useful functions a river may also threaten man, e.g. in case of floods.

In this chapter the history of man's interference with the Meuse will be analysed insofar as it had consequences for the hydrology of the basin. Here shipping has had a considerable influence. In that connection the natural depth (or shallowness) of the river has mattered a great deal.

Up to the nineteenth century shipping took place in natural waterways (§ 5.2). In the nineteenth century a system of shipping canals was effected. The canals have also been used for irrigation (§ 5.3). In the twentieth century the building of canals continued, but especially the existing waterways have been enlarged for the benefit of shipping (§ 5.4). Apart from shipping especially floods have played a part in the realisation of the present hydrologic situation of the river (§ 5.5 and 5.6).

For a description of the present situation the reader is referred to chapters 6 and 7.

In this chapter a great many geographical names are used, most of which are represented in fig. 5.1.

5.2 SHIPPING UP TO THE TIME OF THE FRENCH EMPIRE

In the first century B.C. the Romans conquered Gaul and with it the Meuse basin (Breuer, 1969). From written texts it can be gathered that at any rate from 44 B.C. the Romans navigated the Meuse (Latin: Mosa). However, there probably was some navigation in the Meuse before the arrival of the Romans, too. Fish, cranes, geese and swans were traded. Compared with the Moselle (Latin: Mosella, little Meuse) the trade was of a small size, which was caused by the banks rising steeply and the rather unimportant tributaries. Apart from being relevant to commerce the Meuse was also of military importance for provisioning and for the defence of the Empire. Forests and hills were obstacles to traffic on land, so that shipping was more important.

A favourable circumstance for the equal character of the discharge was the presence of dense forests in the region: practically the whole area south of the Kempen (Campine) was wooded. On that account the discharges had higher minima and lower maxima than nowadays. Possibly the climate also played an important role (damper).

In Roman times ships had only very modest sizes. For that reason the Meuse and the Sambre were still navigable further upstream. In the bigger ships smaller ones were carried along, which could be deployed in the upper course of the river.
In one day approximately 30 km could be covered by boat upstream. Mainly as a result the cities of Maastricht, Liège, Huy, Namur and Dinant, lying at about that distance from each other, came into existence. Important cities were to be found in places where roads crossed the Meuse (Verdun, Maastricht).

After the fall of the Roman Empire commerce lived on. At the time of Charlemagne’s Empire the region had become prosperous owing to the brisk trade. However, the Vikings were becoming a threat. The region was protected against the invasions of the Vikings by fortifying the harbours and building a fleet. On Charlemagne’s death the Empire disintegrated and was weakened to such an extent that the Vikings could ransack the region. Maastricht, Liège, Huy and Namur and some more cities were forayed. Of the first two cities little was left. Commerce diminished considerably through the invasions.

In the tenth century after the expulsion of the Vikings commerce increased again and cities began flourishing once more. Trade was conducted with Britain: silver and other metals were exported, pewter and wool were imported. Goods were transshipped from river craft to sea-going vessels at Tiel (on the Waal). Trade grew, there were contacts with Scandinavia, Bohemia, Russia and even Iceland. Weaving was of special importance.

In that century and the next few ones roads were also greatly improved and many bridges were built across the Meuse. In the 12th and 13th centuries the building of bridges and the rise in population increased strongly. Shipping was lagging behind. At the time the trade with Britain was often carried on along roads via Vlaanderen (Flanders). The lag of shipping may partly be explained by the change in the hydrologic properties of the area. In that case the cause would have to be found in deforestation. The deforestation is thought to have contributed to a greater number of inundations and lower summer discharges. The result of it might also have been the origination of islands and clayish river deposits. It is contradictory to that explanation that the deforestation took place during a longer period and in a relatively small area. Possibly the change of climate may also have influenced the discharge regime.

Ships were moved forward in several ways. Upstream the best-known way was towing. One or more horses moved along the bank (tow path) and pulled the ship (track boat) on. For centuries the tow paths along the Meuse were of a poor quality, were inundated when there was a flood and regularly changed banks. In some stretches there even was no tow path.

Apart from being towed, ships were also moved on by rowing and near the mouth by sailing.

Downstream the ship could simply be carried on by the current. Then horses were taken on board.

Upstream the depth of the Meuse decreased more and more, for which reason goods were transshipped into smaller vessels (Venlo, Roermond, Mézières). The big ships (hoogmasten, tall-masters) were mainly to be found below Venlo. When the water level was higher they could also move further upstream as far as Maastricht and Liège. When the water level sank fast they were often harbour-bound in those towns for months: even then there was a need for a low flow forecast!

Since Roman times plans had been made to connect the rivers with one another by means of a network of canals. After the invention of the lock (1438) the plans could be realised. In 1580
the Governor of the Spanish Netherlands tried to construct a canal between the Meuse and the Scheldt (on the stretch of the present Albertkanaal (Albert Canal)). However, only two miles were constructed. In 1613 a new attempt failed through the opposition of the Northern Netherlands (United Provinces).

In 1626 the construction of a Rhine Meuse Scheldt connection was started (from Rheinberg via Geldern, Venlo, Lozen, Antwerpen to Oostende). The canal was named after the Spanish crown princess Isabella Eugenia and therefore it was called Fossa Eugenia. At the same time numerous fortifications were built. The project was opposed both by the Northern Netherlands (especially for military reasons) and by the independent bishopric of Liège (mainly for commercial reasons). After the conquest of the canal region by the Northern Netherlands the construction of the canal was stopped (1628).

Afterwards several attempts were made to build a waterway between Liège and Antwerpen, so that the dependence on the arbitrariness of the Northern Netherlands, having the control of the mouth of the Meuse, would be put an end to. All the attempts proved to be doomed from the start.

In the 17th century trade comprised a wide range of goods: salt, coleseed, butter, cheese, lard, herring, stockfish (from the North to Liège), coal, ironware, alum, potash (from Liège), wine, ore, oil, grain, embroidery (from the south to Liège), wood, granite and marble (from the south to Liège and northward).

The shipping in the Meuse Lorraine was reduced considerably in the seventeenth century. The cause was in the unfavourable situation in the valley in France: far from Paris and turned in a north south direction. The Meuse did have a use as part of the fortifications (Verdun, Mézières, Sedan, Stenay). Plans to construct a network of canals in the east of France proved too expensive: a great number of locks would have to be built, because on technical grounds the maximum height of lift was from 1 to 2 m and problems had to be dealt with concerning the water supply for those canals. Yet the Meuse Lorraine was partly improved: e.g. near Sedan a canal was dug to cut off a bend in the Meuse.

5.3 THE CONSTRUCTION OF CANALS IN THE NINETEENTH CENTURY

During the Napoleonic Empire shipping in the Meuse increased since trade barriers in the form of frontiers disappeared. Up to 1810 the Northern border of the Napoleonic Empire was near the large Dutch rivers.

By order of Napoleon Bonaparte the building of the Canal du Nord (Northern Canal) was begun in 1808, which was to form a connection between the Scheldt (Antwerpen), the Meuse (Venlo) and the Rhine (Neuss). The trace of the canal was chosen in such a way that the whole of the waterway would be within the borders of the French Empire. The feed of the section between Antwerpen and Venlo would be done by means of a feeder. The feeder was to take in water from the Meuse near Maastricht and flow out into the canal at the catchment boundary between Scheldt and Meuse (Lozen). When some years later the Kingdom of Holland was annexed the need for the waterway ceased and its construction was stopped (1810). Only the stretch between Lozen and Beringe as well as the feeder had been finished.

After Napoleon’s fall the Kingdom of the Netherlands came into existence in 1815. At the time the shipping possibilities in the Meuse were considered so small that King Willem I decided to build a canal from Maastricht to ’s Hertogenbosch. The canal was named after the king and called the Zuid-Willemsvaart (Southern William Canal). For the section above Lozen
use was made of the feeder of the Canal du Nord, which was enlarged. However, in Maastricht the canal was led through the city. Beyond Lozen the canal follows the Canal du Nord as far as Weert, where it branches off to 's Hertogenbosch, flowing out into the Dieze there. Its length was 122 km; 20 locks were built. The Zuid-Willemsvaart had no connection with Liège. In Maastricht it cut through the fortifications and in some places it was only 7 m wide. The canal was fed by means of lock 20 (called "hoofdsluis", main lock), lying on the Meuse. It soon appeared that the lock in Maastricht could feed the canal only insufficiently because of the high sill in the lock.

In 1830 the Belgian war of independence started. The stronghold of Maastricht was under Dutch control and so was the feed of the Zuid-Willemsvaart. The Dutch cut off the canal in order to have a sufficient supply of water in the fortifications. To get sufficient water for the Belgian part of the Zuid-Willemsvaart, the Belgians opened an inlet below Maastricht on their own territory (Hocht). In 1839 Belgium's independence became a fact. After some time the Zuid-Willemsvaart was opened again.

In the period from 1843 to 1846 the Kanaal Bocholt-Herentals was built. By using the Zuid-Willemsvaart and the rivers Kleine Nete, Rupel and Scheldt, Antwerpen had now become accessible to shipping. Both the feed of the canals mentioned and the irrigation of the reclaimed moorland led to a strong increase of the demand for water, which could hardly be complied with by the inlets of the lock 20 and of Hocht together. The Kanaal Bocholt-Herentals forked off to Turnhout (afterwards extended to Antwerpen), to Beverlo and to Hasselt (fig. 5.2). The system of canals is known as the Kempische Kanalen (Campine Canals) (Craenenbroeck et al., 1985).

The successful irrigations were imitated in the Netherlands (Olivier Dz., 1859), where the Kanaal naar Eindhoven (Canal to Eindhoven) and the drainage canal to Kessel were built (fig. 5.1).

The canal between Herentals and Antwerpen (the Schelde-Maaskanaal, Scheldt-Meuse Canal) was built in the period from 1846 to 1859. The connection increased the shipping traffic with Antwerpen.

About 1850 the Kanaal Liège-Maastricht was built, linking up with the Zuid-Willemsvaart in Maastricht. The Canal, which lay on the left bank, was 25.6 km long and had 6 locks. It was fit for ships up to 450 tons. Near Visé a transverse canal to the Meuse was built with a view to the mines lying on the right bank of the Meuse.

The inlet of the Kanaal Liège-Maastricht was in the control of Belgium. To irrigate the Kempen much water was led into the canal. A weir in the Meuse below the inlet of the canal made it possible whatever the discharges were. Then the water was conducted to the appropriate place via the Zuid-Willemsvaart. In that way the shipping in the Meuse below Maastricht got to deal with even lower water levels. But also in the Zuid-Willemsvaart shipping ran into problems on account of the high water velocities. The discharges were yet increased by the deepening of the inlet at Hocht. The problem could be reduced by discharging a greater flow rate at night than in the daytime.

At the time of the United Netherlands the flourishing industry near Liège wanted to have a connection with the mines in Luxembourg. A connection was planned between the Meuse and the Moselle. The canal was to start at Liège and to run south via the Ourthe. Between Buret (Belgium) and Hoffelt (Luxembourg) a tunnel was to be dug. In Luxembourg the canal was to follow the rivers Woltz, Wiltz and Sûre, and end into the Moselle at Wasserbillig. In 1827 the construction of the canal was started, which would have to admit ships up to 60 tons. At the time of the Belgian separation from the Netherlands (and from Luxembourg) the construction was stopped. Near Liège and Wasserbillig a small section of the canal had been
Figure 5.2 The network of canals below Liège about 1859. Source: Olivier Dz. (1859)

finished as well as 1300 m of the tunnel. Subsequently the northern part of the canal was completed up to Comblain-au-Pont, but only for ships up to 40 tons.

Nowadays the canal can still be navigated near Angleur, but only for ships with a tonnage of up to 300 tons. Remnants of the remaining part of the canal can be observed in the Ourthe valley.

In the years between 1824 and 1828 the Belgian (then Dutch) part of the Sambre was canalised. Before that time the tonnage of ships was sometimes limited to under 9 tons. In the stretch of 95 km 22 locks were built. Ships of 300 tons were to sail in it, but only a canal for ships up to 200 tons was realised.

Between 1826 and 1835 the French section of the Sambre was canalised and as a connection a canal to the Oise was constructed: the Canal de la Sambre à l'Oise (Sambre-Oise Canal).

In the period 1827-1832 the Canal de Charleroi à Bruxelles (Charleroi-Brussels Canal) was built. On the coming into service of the canal a shipping connection between Liège and Antwerpen had been realised via Charleroi, also making use of the Antwerpen-Brussel waterway.
As a shipping route the Belgian Meuse was unsatisfactory. It was tried to improve the Meuse by improving the tow paths, the bank revetments and the deepening of the waterway to 1.50 m (1838). After the realisation the course of shipping was still disappointing. Plans were begun for the construction of a parallel canal between Namur and Liège. After serious consideration it was decided not to build the parallel canal, but to canalise the Meuse itself between Liège and the French border (First Meuse Canalisation). The start was made in Liège in 1853 and from there the work went on upstream. It should be possible to admit ships of 600 tons. In 1865 Namur was reached. At the time the number of steamships and towboats was increasing considerably. As a result it was decided to build larger locks upstream from Namur (100 m instead of 57 m) and to enlarge the downstream locks up to those sizes. After the canalisation ships with a tonnage of 1350 tons could sail the Meuse, provided they had a load of up to 1150 tons, because the draught was only 2.20 m instead of the usual 2.50 m. In 1880 the French border was reached.

The discharge of water via the Zuid-Willemsvaart had already resulted in several conflicts between the Netherlands and Belgium. In 1863 to terminate the conflict a treaty was concluded for a settlement of withdrawals of water from the Meuse (Anonymous, 1863). A supplement was published in 1873 (Anonymous, 1874). The treaty settled the following points:
- The construction of a new inlet (feeding culvert of Bosscheveld; still used)
- An agreement regarding withdrawals from the Zuid-Willemsvaart
- An agreement to improve the Meuse waterway between Visé and Venlo for the benefit of shipping.

Using the old Hocht inlet was no longer permitted. For the new inlet exact rules have been established about what minimum discharge has to be supplied by the Netherlands, including discharge formulas and coefficients. The discharge formulas turned out to result in a greater discharge than had been established in the treaty. At the same time a maximum of the permitted discharge to be supplied was fixed, by establishing a maximum of the flow rate permitted. Belgium was obliged to return part of the water supplied to the Netherlands via Lozen. In table 5.1 some values are given.

The connection of the French heavy industry on the southern edge of the Ardennes with Reims and Paris was realised by the construction of the Canal des Ardennes (1835).

<table>
<thead>
<tr>
<th>Water mark above the sailing level</th>
<th>to be let in at Maastricht [m³/s]</th>
<th>to be passed at Lozen [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water mark below the sailing level</td>
<td>10</td>
<td>2</td>
</tr>
<tr>
<td>15 Oct. - 20 June</td>
<td>7.5</td>
<td>1.5</td>
</tr>
<tr>
<td>21 June - 14 Oct.</td>
<td>6</td>
<td>1.5</td>
</tr>
</tbody>
</table>

- Sailing level means a 70 cm draught between Maastricht and Venlo (then the discharge would be approximately 80 m³/s; the value is not in the treaty)
- The Netherlands were allowed to discharger even more until a maximum velocity of 25 to 27 cm/s was reached. Belgium was to make the exceeding quantity flow back to the Netherlands via Lozen.
Starting from the Meuse below Sedan the canal runs to the Aisne through the Bar valley and as a parallel canal it follows the Aisne before it ends in it.

The mining of ore in the French Ardennes was insufficient for the local industry so that they started bringing it there from the Luxembourg/Longwy region. Coal was brought from the Charleroi/Liège region. From 1837 to 1845 the Meuse was improved between the French Belgian border and Sedan (possibly even to Verdun), a navigable depth of 1.10 m was realised.

Between 1837 and 1853 the Canal de la Marne au Rhin (Marne-Rhine Canal) was built, connecting the heavy metal industry area of Lorraine with the rest of France. The Meuse was crossed near the town of Troussay. Both in the Meuse Ardennaise and in the Canal de la Marne au Rhin the traffic was busy. In 1868 it was decided to realise a good connection between those waterways by means of a canal.

The war with Germany (1870-1871) thwarted the plans. France lost the region around Metz with its mines and the Moselle shipping route to Germany. However, the canal remained desirable: in that way especially Belgian coal could be transported to the industrial area near Nancy. In 1874 the construction of the Canal de l'Est (Eastern Canal) was started, which was also to connect Scheldt, Meuse and Rhine on the one hand and Saône and Rhône

Figure 5.3 Canal de l'Est near Stenay. Source: Vuillaume
on the other. Part of the Meuse was canalised and part of it was provided with parallel canals. The latter are mainly found in the southern part. The canalisation shortened the waterway by 80 km to 272 km. In all 59 locks were built, with the dimensions of 48.30 by 5.70 m below Verdun and 38.5 by 5.20 m above it (a draught of 1.8 m, suitable for ships of 300 tons). There are four tunnels in the canal. Furthermore it was also of military significance: it would have to be an obstacle to aggressors from the east. Figure 5.3 gives a representative picture of the Canal de l'Est. For a further description the reader is referred to chapter 7.

In 1880 a connection had come into existence for ships of 300 tons from the Canal de la Marne au Rhin to Maastricht and from the latter town it was possible to sail on by way of the Zuid-Willemsvaart.

5.4 EXPANSION IN THE TWENTIETH CENTURY

In the course of years the shipping in the Zuid-Willemsvaart and the Kempische Kanalen became busier and busier, which caused long delays at the locks. In 1913 the voyage from Maastricht to 's Hertogenbosch took a fortnight.

Before that time a Dutch-Belgian committee had already started an investigation in
order to improve the connection between Liège and the mouth of the Meuse by making the river navigable from Visé to Venlo. In the report published in 1912 fourteen weirs were planned, seven of which between Maastricht and Maasbracht (Kemper et al., 1912). Since in that case the transport from the Walloon industrial areas would take place by way of Rotterdam Belgium demanded a better connection between the mouths of Scheldt and Rhine. The Great War and laborious negotiations made the Netherlands determine in 1915 to start the canalisation of the Meuse as far as it concerned their own territory. The weir lying furthest upstream was built at Linne (photo 5.1), so that the Grensmaas kept a free flow. Weirs were also built at Roermond, Belfeld, Sambeek and Grave (photo 5.2). The navigable depth was 2.80 m with a possibility to deepen it to 3 m. At the same time the construction of the Kanaal Wessem-Nederweert (connection between the Zuid-Willemsvaart and the Meuse) was begun and the Maas-Waal-kanaal (Meuse-Waal Canal) between Mook and Nijmegen. The canalised Meuse and the Maas-Waal-kanaal were to admit ships up to 2000 tons, the Kanaal Wessem-Nederweert ships up to 600 tons. Actually it was expected that the greater part of the shipping to and from Rotterdam would not make use of the Grave lock but of the Maas-Waal-kanaal. That is the reason why the Grave lock was built on a smaller scale than the other locks (136 m long instead of 260 m). After the realisation in 1929 Maasbracht became a large point of transshipment for the coal mined in Limburg. The Zuid-Willemsvaart became less important.

After the end of the Great War plans for the canalisation of the Meuse were brought forward again. As an alternative the enlargement of the Zuid-Willemsvaart was proposed, combined with the construction of the Kanaal Maasbracht-Neerpoeren.

However, in the Netherlands it was decided to build a canal between Maastricht and Maasbracht, the Julianakanaal (Juliana Canal). Thus a good connection with the Dutch mining district would be realised. Moreover the canal would have to increase the trade with Belgium and France. Building a canal on the Dutch territory would have the advantage that problems with Belgium could be prevented and moreover only three locks would be sufficient (in which the Limmel guard lock at the entrance to the canal, which is only closed in the case of a flood, has not been included). The new lock to be built at Borgharen (completed in 1928, photo 5.3) was to keep up the water level in the Maastricht headwater reach. After the Julianakanaal came into service in 1935 Maasbracht soon lost its transshipment function to the harbours built in the canal.

The connection with the Zuid-Willemsvaart was improved by the construction of the Bosscheveld lock. As a result part of the flood way of that name was lost. As a compensation a diversion canal was dug with a fixed weir in it. The diversion is situated parallel to the Borgharen weir.

In the Kanaal Liège-Maastricht a connection by means of a lock with the Meuse was built off St. Pieter. The passage via the Maastricht canals, which took two days, could be avoided now.

Opposite page:
Photo 5.2 The Grave weir under construction. Source: Rijkswaterstaat: Schlingemann (1928)
Since the building of the Julianakanaal the shipping in the Grensmaas below Borgharen has strongly decreased.

The Kempische Kanalen were enlarged so that they became suitable for ships up to 600 tons (so-called Kempeners or Campinois) in 1928. The Lanaye lock was also enlarged. The Zuid-Willemsvaart was also to admit 600 ton ships. Strikingly enough the locks in the Kanaal Liège-Maastricht were not enlarged.

The Wilhelminakanaal lying in Dutch Noord-Brabant was opened in 1923. In 1938 Eindhoven got a connection with the Wilhelminakanaal by means of the Beatrixkanaal. Afterwards the Kanaal naar Eindhoven was closed for shipping.

As a reply to the Julianakanaal to be built by the Netherlands Belgium decided to build a large canal from Liège to Antwerpen for the benefit of shipping, but also for the irrigation of the Kempen. In Belgium a waterway completely on their own territory had been preferred, as in the Netherlands. The construction of the canal, called after King Albert I, was started in 1930 and was finished in 1940. The canal starts at Liège and follows the Kanaal Liège-Maastricht as far as Lanaye. There it bends off westwards and leads to Antwerpen by way of Genk. From Hasselt to Kwaadmechelen the canal follows the trace of the Kempische Kanalen. West of Herentals the trace of the Kanaal Antwerpen-Herentals is followed. The Albertkanaal is suitable for ships up to 2000 tons. It only has 6 locks (apart from the guard lock near Monsin). The first normal lock is near Genk. For that reason the water level of the canal near Lanaye has become as much as 9.75 m higher than in the former situation. The Albertkanaal also had a military significance, which is still to be observed near Eben-Emael in the shape of bunkers. Initially the remaining part of the Kanaal Liège-Maastricht was not improved. Near Monsin a connection with the Meuse has been constructed.

Together with the construction of the Albertkanaal the Belgian Meuse was improved starting from Liège and going upstream. When at the beginning of World War II work had to be stopped the building had progressed as far as Neuville-sous-Huy.

Through the construction of the Kanaal Briedgen-Neerharen a connection was made between the Albertkanaal and the Belgian part of the Zuid-Willemsvaart.

The Netherlands thought that Belgium acted contrary to the 1863 treaty, because in the case of the Albertkanaal and the Kanaal Briedgen-Neerharen it could withdraw water from the Meuse in a way other than with the inlet. Belgium denied acting contrary to the treaty and argued that if Belgium had violated the treaty the Netherlands had likewise done so by constructing the Borgharen weir, the Bosscheveld lock and the Julianakanaal. The verdict of the International Court of Justice in 1937 caused hardly any change in the situation created. The verdict was based on these facts: no feeding culverts had been built and Belgium had a right to build a national waterway and feed it with water from the Meuse. On account of the higher water level at the entrance of the feeding culvert of the Zuid-Willemsvaart the Netherlands could not but always deliver 10 m³/s to the Zuid-Willemsvaart. Moreover, the water supply to the Julianakanaal should never result in the Grensmaas running dry or in the impossibility to deliver 10 m³/s to the Zuid-Willemsvaart any longer.

In 1940 the curious situation had arisen near Maastricht that both in Belgium and in the Netherlands 2000 ton ships could be found, only near the border there was a short canal of 3 km with a maximum capacity of 450 tons ("Bouchon de Lanaye"/"Stop van Ternaalen", stop of Lanaye). Not before the late fifties and the early sixties was a large lock (ships up to 2000 tons) built at Lanaye and a direct connection with the Meuse made on the downstream side of the lock (1961). The remaining part of the Kanaal Liège-Maastricht was filled up after some time.

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As a result of the elimination of the Bouchon de Lanaye it was no longer necessary to execute the construction of a canal between Briegden and Maastricht.

After World War II the shipping in the Julianakanaal was so busy that an extension of the waterways became necessary. Parallel locks were built. The Roosteren locks were eliminated and the canal section was joined to the Maasbracht one, so that the Maasbracht locks had to be renewed. The canal was also widened. For the section below Roosteren it was sufficient simply to heighten the dikes. In 1967 the work was finished.

In the early seventies near Heel the Lateraal Kanaal (Lateral canal) was built, so that the waterway along the Dutch Meuse was shortened by 5 km.

The enlargement of the Canal system Antwerpen-Brussel-Charleroi for ships up to 1350 tons was executed simultaneously with the enlargement of the Sambre below Monceau near Charleroi. The number of locks (still dating from 1829) in the section of the river was decreased from 13 to 8, the length of the waterway was reduced from 86.2 to 52.6 km and the locks were enlarged to 110 by 12.5 m. The improvement was wound up in the late sixties. Upstream from Charleroi the weirs and locks in the Sambre were not renewed (Anonymous, 1986).

Photo 5.3 The Borgharen weir
In 1957 Belgium resumed the improvement of the Belgian Meuse by replacing the Neuville-sous-Huy weir by the weir of Ampsin-Neuville.

At the time Belgium, in cooperation with France, wanted to realise a waterway from the North Sea to the Mediterranean, which would be accessible to ships of the same tonnage everywhere.

However, the size of the French heavy industry in the northeastern region was decreasing in those days. At the same time the navigability of the Moselle was improved. An enlargement of the Canal de l'Est could not be justified by France. Consequently waiting for French plans was in vain and ultimately in 1973 Belgium decided to execute a plan of its own for the Belgian Meuse only. The five weirs between Namur and Neuville-sous-Huy have been replaced by only two new ones. Below Namur the locks are being enlarged to 200 m by 12.5 m (for ships up to 9000 tons, coasters). The new weirs were finished in 1983. The enlargement of the locks will probably have been finished in the early nineties.

In 1980 the Visé weir was replaced by the Lixhe one. Like the Monsin and the Namur weirs it contains a hydro-electric power station.

Indeed, above Namur the weirs are also being renewed, but in view of the great scenic value of the Meuse valley the number of weirs will not be decreased. What is done is that the locks will be adapted to fully loaded ships of 1350 tons.

The depth of the Belgian Meuse is being increased from 5 m to 8 m.

Above Namur the operations are supposed to have wound up in the early nineties. Then the second canalisation of the Meuse will have been completed and the improvement of the Sambre above Charleroi will be started.

As far as shipping is concerned the French Meuse is a backward region as compared with the Netherlands and Belgium. Since the canalisation the only essential alteration has been increasing the capacity below Givet, which town is now to be reached by ships up to 1350 tons, too. Therefore Givet is an important point of transshipment, a part played by Venlo, Roermond and Mézières in the past.

5.5 INUNDATIONS IN THE PAST

Through the centuries floods have brought many disasters. Descriptions of many floods have been preserved (Gottschalk, 1971, 1975, 1977). Initially describing them was not very systematic, afterwards the floods and their consequences were described by several towns in the so-called stadsrekeningen (municipal accounts).

The flood of (about) 858 is the oldest about which some information has come down to us. On account of heavy rainfall inundations occurred in Liège and other towns.

As far as it is known inundations were only brought about by heavy rainfall in that and the following centuries. Only in a single case is the melting of snow mentioned as a secondary cause. In the so-called "small glacial period" (approximately from 15th to the 18th or 19th century) there regularly used to be floods caused by the melting of ice and snow.

The best-known storm surge (i.e. coming from the sea) in the Middle Ages is the St. Elisabethsvloed (St. Elisabeth's flood) raging on 19 and 20 November 1421, which caused the drowning of the Grote of Zuidhollandse Waard (Great or South Holland Holm). The facts that at the turn of the year 1421 - 1422 the Rhine and the Meuse had floods, that there was another flood in 1424 and moreover the inadequate rebuilding after the floods, were fateful for the Grote Waard. It resulted in the origin of the Biesbosch or Bergse Veld. The distance between the mouth of the Meuse near Woudrichem and the sea was shortened by
approximately 40 km. The St. Elisabethsvloed had only little influence on shipping.

To illustrate the consequences of floods and the measures taken the 1642 flood has been chosen. At Venlo the highest water level known up to that time was reached (Amsterdams Peil + 19.62 m). A great damage was done in the towns of Namur, Huy, Liège and Maastricht. At Huy mainly the discharge of the Hoyoux was the cause of it. 150 houses were washed away and only few inhabitants survived the disaster. In Liège several bridges were washed away. In Maastricht the city gate was blocked with all the dung present in the city, but the water could not be stopped. At Wijk, lying opposite Maastricht, the town wall collapsed although the water level was falling at the moment. Had the disaster occurred at the moment of the highest water level, only some dozens of people would have survived. The cause of the flood was not mentioned, but it is known that in Germany floods occurred because of snowfall changing into continuous rain in that period.

The highest flood of this century occurred 1 January 1926. The flood has been described in chapter 2.

5.6 CAUSES OF AND MEASURES AGAINST INUNDATIONS

During the first centuries of our era the Meuse at Heusden ran westward and flowed into the sea there. At an unknown moment in the next few centuries the course of the Meuse near Heusden was bent northwards and the confluence of Meuse and Waal occurred near Woudrichem (Loevestein). Both the cause and the time are unknown. The inundation of

![Figure 5.4 The Beersche Overlaat (Beersch flood way)](image-url)

(about) 858 may have scourged the country to such a degree that they closed the land west of Heusden for the river. However, it is still doubtful whether the shift came about artificially. It is also doubtful if the branch to Woudrichem already existed when most of the Meuse water was discharged by way of its old course. It is also possible that the confluence of Meuse and Waal had not taken place entirely until the end of the thirteenth century. It is deduced from the changing picture of the inundations and reports of the construction of locks and dams
about 1273.

The cause of the confluence of Maas and Waal near Heerewaarden is not known either. Presumably it had a natural cause, although man may have lent his assistance for the benefit of shipping and defence.

To decrease water levels during flood waves many flood ways were dug throughout the centuries. The largest is the Beersche Overlaat (Beersch flood way) (fig. 5.4). The flood way started near Cuyk and Grave (upper inlet and lower one respectively) and ended near the mouth of the Dieze. The flood way came about through the centuries.

Apart from the Beersche Overlaat the Baardwijkse Overlaat was to be found in the surroundings and there were also flood ways at Bokhoven and Vlijmen. Near Heerewaarden there were flood ways as well, from which water could flow from the Meuse to the Waal and vice versa.

Some more flood ways were to be found west of Roermond, between Ohé and Maasbracht (the Overlaat van Contelmo), near Maastricht (the Overlaat van Bosscheveld) and between Heugem and Itteren (Heugemse Overlaat). The last mentioned flood way started near Heugem (south of Maastricht on the right bank), passed Wijk on the eastern side and via Limmel and Borgharen it led to Itteren, where it ran into the Meuse.

Up to 1856 there was a connection between the Meuse and the Waal near St. Andries, the "Gat van St. Andries" (St. Andries’ Hole). In the year mentioned the hole was closed. The connection for shipping was maintained through the construction of a lock.

During flood waves the water would occasionally flow from one river into the other by way of the nearby Heerewaardense Overalten (Heerewaarden flood ways). The water mostly ran from the Waal to the Meuse because normally the water level in the Waal was higher than that in the Meuse.

However, the Heerewaardense Overalten proved to have nothing but disadvantages. At a flood in the Waal the water in the Waal started falling because part of the discharge was led off via the Meuse. On account of that the gradient and the consequent velocity became smaller, so that sedimentation occurred and, as a result, shallows. In winter ice was regularly carried along so that an ice dam would be formed here. Then the water would rise so strongly that inundations were caused regularly.

Hardly being able to discharge its own water because of the small gradient, the Meuse had to deal with an additional discharge, which was greater than its own. Clearly the water level at the confluence of Meuse and Waal (at Woudrichem) was heightened by the flood way instead of being lowered. The flood ways caused sustained high water levels in the Meuse, so that the adjoining areas got into problems concerning their drainage.

The problems were enlarged by two natural processes. In due course the Waal had come to discharge an ever growing part of the Rhine water (23/24 part). After the building of the Pannerdensch Kanaal (Pannerden Canal) (1707) that share has strongly been reduced. At the Spijkse Conventie (Convention of Spijk) (1745) a division of the discharge to be aimed at was fixed: 2/3 for the Waal and 1/3 for the Nederrijn (Lower Rhine) and the IJssel. In the subsequent decades several operations were performed to realise the aim.

The ways to carry the water downstream from the confluence worsened because of silting up, the formation of shoals and the overgrowth in the Biesbosch, and the silting up of the Merwede. The digging of the Nieuwe Merwede (1860) solved the problem. Now the Waal could discharge its own water to a sufficient degree and would no longer be dependent on the Meuse.

In 1883 the separation of the Meuse and the Waal was decided on. Near Heusden the course of the Meuse would come to be westwards, as it used to be. For that purpose the
Bergse Maas (Berg Meuse) was dug, which was to flow from Heusden to the Amer. The stretch between Heusden and Woudrichem was closed near Andel and henceforth it was called the Andelse Maas or the Afgedamde Maas (Dammed Meuse). The Bergse Maas was opened in 1904 (Bongearts, 1909). Consequently the water levels in the Meuse became much lower and a lock had to be built in the Dieze.

The Heerewaardense Overlaten could be closed in 1904 by means of the construction of a higher dike and of the digging of the Nieuwe Merwede (New Merwede) and the Bergse Maas (Berg Meuse). Now the Beersche Overlaat had become superfluous in part. Below Heerewarden the Meuse could convey the discharge which now ran through the flood way. However, the stretch between Grave and Heerewarden had not been improved and had a capacity insufficient for the discharge of a flood wave. That appears from measurements and calculations of the 1880 flood. The discharge of the Meuse was estimated at 2700 m³/s, of which 940 m³/s ran through the Beersche Overlaat. The other flood ways in Noord-Brabant could be closed indeed.

It appeared that in the case of a corresponding flood wave the maximum water level at Heusden had become 1.50 m lower.

The 1926 flood caused a disaster. It initiated the improving of the Meuse below Grave (Lely, 1926). The activities were started in 1931. The gradient was increased by cutting off bends. The minor bed was deepened. Groynes were shortened and lowered for a faster discharge of the water.

The result was that even with normal discharges the water level would sink. In order to maintain the water level sufficiently the Lith weir was built (completed 1936). Together with it another problem was solved, viz. that the draught below the Grave weir was too small. Below Lith the water level was lowered, too, so that the lock of St. Andries, which had also become too shallow, had to be renewed.

In 1942 the activities were completed. At the same time the Beersche Overlaat was closed.

At Maastricht the flow profile was too narrow for a flood wave to pass. The Heugemse Overlaat provided a necessary contribution to the discharge. When there were plans for the construction of the Julianakanaal, the flood way would have to be crossed at Limmel, which would be unacceptable. As a solution it was chosen to widen the Meuse at Maastricht, by digging a shipping channel east of the Meuse. The damming of the flood way near Wijk followed in 1932 (fig. 5.5). Nowadays the upstream part of the flood way is still used as a "groene rivier" (grassy river). The part below Limmel is still a flood way and is called the Overlaat bij Borgharen-Itteren (photo 5.4).

As a result of the construction of the Lateral Kanaal between Heel and Buggenum the flood way west of Roermond was closed down in 1971.

In the Netherlands some more small operations were executed after the canalisation and improvement of the Meuse: cutting a bend near Wessem (1939), near Buggenum (1952), near Neer (1957), displacing the Meuse bed near Eijsden (1969-1970) and cutting a bend near Boxmeer (1972-1978).

In the Netherlands and Belgium the extraction of gravel near the Meuse started in 1950. Initially the extraction was performed near the Grensmaas. The resulting gravel pits mainly affect the course of the flood waves. One more effect is the sinking of the bottom in the river. Meanwhile the sinking seems to have come to a standstill.
In France the bed was improved from Neufchâteau to the departmental border of the Vosges. The improvement took place in the late seventies. The objective of the improving operations along a stretch of 30 km was the increase of the capacity of the bed.

In Belgium the Meuse has been adapted for the benefit of shipping since last century. Recently flood problems have been an additional reason. To decrease the problems the Belgian Meuse is being widened and regulated. Though a century ago the aim was making the waterway 1.50 m deep, the bottom is now being realised at 5 to 8 m below the normal water level. The weirs have been adapted to such a level. In Belgium it is hoped that after the completion of the operations they will be released from flood problems in the Meuse. Calculations have shown that in case of a flood wave similar to that of 1926 the water levels in Namur will be 2.30 m lower.

Figure 5.5 The closing of the Heugemse Overlaat (Heugem flood way)
5.7 CONSEQUENCES FOR THE HYDROLOGIC SYSTEM

As soon as it was technically and economically possible man tried to bend the Meuse to his will as much as possible. Particularly shipping has played an important part here. Through expansion and considerations of efficiency new canals were built and the existing waterways were improved. It seems as if the improvements take place faster and faster. With the increase of large works to be regulated electronically the discharge of the Meuse is becoming more and more unnatural. As an example the strongly changing discharges of the Lixhe power station may be mentioned, which have a disturbing influence on the aquatic environment of the Grensmaas.

On account of the feed of canals reaching far beyond the catchment the discharge of the Meuse is being decreased. At times of drought it may have great economic consequences.

In order to avoid flood problems several measures have been taken. As instances may be mentioned: widening and deepening the Meuse, removing bends etc. Most changes are aimed at discharging the water faster from the area. Flood problems have also been decreased by smoothening peaks by means of digging gravel pits and, in the past, flood ways.

Reservoirs, which also play a part in the reduction of floods, are beyond the scope of this chapter.
Chapter 6. The tributaries

6.1 INTRODUCTION

The greater part of the discharge of the Meuse is supplied by its tributaries. Ground water, precipitation and artificial extractions constitute the discharge to a smaller extent.

The Meuse has a great number of tributaries, varying greatly in their sizes. The largest is the Ourthe, with a contributing area of 3,626 km². The smallest might be a ditch in a meadow on the bank of the Meuse. In this chapter attention will mainly be paid to the larger tributaries, which are the most important as far as the discharge modelling is concerned. In this case it regards the tributaries with a contributing area of over 500 km² upstream from Lith. The treatment will be done from the south to the north. After that a number of smaller tributaries will come up in brief.

On the next few sections a number of remarks can be made.

The local precipitations mentioned have been calculated by means of Thiessen polygons (§ 10.3.2) and for a great number of stations. However, the number of stations and their positions are not very important here and therefore they have not been stated.

The maximum discharges mentioned are not momentaneous peak discharges but maximum values of the mean daily discharges, unless mentioned otherwise.

It has proved to be impossible to stick to the same periods of reference for both the discharges and precipitations, among other things because the data have been provided by various sources. The runoff coefficients have been calculated by dividing the specific mean discharge by the mean precipitation intensity. The runoff coefficients should be treated with due prudence because the reference periods are not identical. The runoff coefficients over the year have been calculated by means of annual precipitations and mean annual discharges.

In general discharges in the tributaries are determined with the help of stage-discharge curves. For great discharges the calibration of the curve is based on a wintry situation, when the overgrowth is slight. Hence the data concerning the flood discharges of July 1980 are less accurate than those regarding the flood waves in winter. The occurring deviations may be illustrated by means of the Borgharen-Dorp discharge: on the basis of the discharge curve the momentaneous peak discharge of the July 1980 flood was 2,526 m³/s, but the discharge measurements showed that the peak discharge amounted to only 2,200 m³/s (Anonymous: Jaarboek der waterhoogten van 1981, published 1983).

For most rivers a length will be given. The value should be considered arbitrary, since it is not always possible to establish the position of the source univocally.

As sources of data on precipitation and discharges annuals of various institutes have been used among other things. On account of the limited amount of time for making the systems analysis, the data of the precipitation during flood waves have not been entered.

The wave travel times have not come direct from literature. For the stretch downstream from Chooz they have been derived from the flood modelling, for the stretch upstream from Chooz from longitudinal profiles and the current velocities as calculated by the Agence Rhin-Meuse (not published). As a result the wave travel times mentioned for the stations upstream from Chooz are less accurate than those downstream from Chooz. Moreover, it should be taken into account that for the separate floods the wave travel times may deviate significantly from the times mentioned.
6.2 THE CHIERS

The Chiers is one of the large tributaries of the Meuse. It springs near Differdange (Luxembourg) at approximately NAP + 339 m. The mouth is near Rémilly-Aillicourt at approximately NAP + 149 m (fig. 6.1). The river is 144 km long. The contributing area amounts to 2,222 km², of which 1,967 km² is above the measuring station of Carignan. The mean gradient is 1 \(10^{-3}\).

The southernmost parts of the Ardennes lie in the northwest of the catchment of the Chiers. The valley in the downstream part of the Chiers is very permeable due to the calcareous soil dating from the Dogger. Close to its mouth there is an alluvial stratum. The southern part of the area contains many impermeable clay soils and loamy soils. In the north and the east much sandstone and limestone is found.

The main tributaries of the Chiers are the Ton, the Crusnes, the Othain and the Loison. Of them the Ton and the Loison for the greater part lie in the very impermeable areas mentioned, so that the discharges of the rivulets may become small in dry periods.

In the greater part of the catchment the mean annual precipitation lies between 800 and 1,000 mm (table 6.1). Only along the edges other values are found: 700 mm in the south and 1,200 mm in the north.

---

*Figure 6.1 The catchment of the Chiers*
<table>
<thead>
<tr>
<th>Month</th>
<th>Mean Discharge ($m^3/s$)</th>
<th>Areal Precipitation [mm]</th>
<th>Runoff Coefficient [-]</th>
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</thead>
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<tr>
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<td>58</td>
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<td>65</td>
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<td>June</td>
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<td>70</td>
<td>0.36</td>
</tr>
<tr>
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<td>76</td>
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<tr>
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<td>77</td>
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<td>September</td>
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<td>74</td>
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<td>78</td>
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<td>November</td>
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<td>78</td>
<td>0.37</td>
</tr>
<tr>
<td>December</td>
<td>35.7</td>
<td>82</td>
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</tr>
<tr>
<td>Year</td>
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<td>859</td>
<td>0.50</td>
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</tbody>
</table>

At times of flood waves the water from the whole catchment is practically supplied equally, the north relatively supplying a somewhat greater share. The reason is the smaller permeability in the south than in the north, but in the north the slopes are steeper and the precipitation is greater.

The celerity in the Chiers amounts to over 100 km/day. It is much more than in the Meuse above Sedan. The peak of the wave in the Chiers mostly reaches Sedan sooner than the one in the Meuse Lorraine. Table 6.2 gives a survey of the hydrographs of some floods.

In the dry summer of 1976 the Chiers still had a minimum discharge of over 4 m$^3$/s. The result was that the discharge of the Meuse near Sedan was almost doubled. In that respect 1976 was rather normal (Zumstein et al., 1978).

The low flow discharges of the Chiers are influenced by the draining of the iron mines. The mines were closed down in the eighties. From an investigation it appears that the discharges will decrease (Anonymous, 1987). It may have serious consequences for ditches that are mainly fed by effluents (decrease up to 100%). For the Chiers as a whole the decrease is estimated to be 0.4 m$^3$/s on an average, and in periods of low flows it may be 0.3 m$^3$/s.

The amount of the low flow discharge of the Chiers is usually of the same magnitude as that of the Meuse above the mouth of the Chiers.
### TABLE 6.2 FLOOD WAVES IN THE CHIERS AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Jan./Feb. 1980</th>
<th>Carignan</th>
<th>Borgharen</th>
<th>July 1980</th>
<th>Carignan</th>
<th>Borgharen</th>
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<td>3.8</td>
<td>896</td>
<td>3.6</td>
<td>19</td>
</tr>
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<td>4</td>
<td>87</td>
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<td>1105</td>
<td>4.5</td>
<td>20</td>
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<td>4.6</td>
</tr>
<tr>
<td>12</td>
<td>101</td>
<td>4.4</td>
</tr>
</tbody>
</table>

disch. = discharge [m³/s]  
s.d. = specific discharge [mm/day]

#### 6.3 THE SEMOIS

At NAP + 411 m the Semois springs near Arlon. It flows west and receives the Rulles and Vierre. A few kilometres before its mouth the Semois crosses the French border. The mouth is near Monthermé at approximately NAP + 138 m (fig. 6.2). Its catchment amounts to 1,358 km². The river has a length of 167 km.

Its catchment is lengthy. The main tributaries flow out into the river in the upstream part. It is striking that the tributaries mentioned spring at a higher level than the Semois itself. The highest point in the catchment is at NAP + 508 m, near the source of the Rulles. The lowest point is near the mouth, at NAP + 138 m. Three quarters of its catchment are to be found between 340 and 460 m above NAP.

The upstream course of the Semois lies in a relatively even area being part of Belgian Lorraine. That part of the Paris Basin has many sloping strata, where finally the Cambrian crops out in the north. The strata have a varying permeability (Debbaut, 1989).

The nature of the river changes below the mouth of the Vierre. The river flows
Figure 6.2 The catchment of the Semois

through the Ardennes Massif with rocks of a palaeozoic origin. Consequently the gradient of that part of the river is greater than in the even area, $1.7 \times 10^3$ as opposed to $1.1 \times 10^3$. The mean gradient is $1.5 \times 10^3$.

Approximately 40% of the area is covered with forests, 4/5 of which consisting of deciduous forests.

In the sixties the plan was considered to form a large reservoir in the Semois near Dohan (contents: 1 milliard m$^3$). The reservoir was to form part of a number of measures which would result in a minimum discharge of the Meuse of 90 m$^3$/s. However, the socio-economic consequences of the reservoir were strongly negative. The reservoir has never been built.

In 1970 a small reservoir was built in the tributary Vierre (Barrage de la Vierre). The water is mainly used for the generation of electricity. The reservoir is too small to cause a significant increase of the low flows in the Meuse; its capacity amounts only to $1.3 \times 10^9$ m$^3$. That is also shown by the discharge data published in Anonymous (Annuaire Hydrologique de Belgique/Hydrologisch Jaarboek van België).

The dual nature of the catchment also has consequences for the distribution of precipitation. On account of orographic effects the mean annual precipitation is significantly higher in the west than in the east: 1,250 mm (Bouillon) as opposed to 1,000 mm (Arlon). In the extreme west less precipitation is collected than in Bouillon (Membre: 1,170 mm) (Bultot et al., 1968). On an average the annual precipitation for the catchment of the Semois is $1,139$ mm (table 6.3).

The Belgian Ministère des Travaux Publics has a measuring station for discharges.
at Membre, the Service de la Navigation at the nearby Haulmé. From the discharge measured the IRM (Institut Royal Météorologique de Belgique) arrives at the conclusion that in dry periods the discharge at Membre is halved in a period of 15 to 20 days. At the end of August 1976 the Vierre no longer carried any water, not even below the reservoir.

In times of much precipitation the discharge may rise strongly. The mean daily discharges in February and July 1980 have been given in table 6.4.

From the relations between the discharges of Borgharen and Membre it appears that the Semois can supply a considerable contribution to a flood wave. The contribution to the peak discharge at Borgharen was greater than 10%. However, the Semois does not always contribute greatly, as appears from the flood of July 1980: at the moment of the peak at Borgharen the contribution of the Semois to the discharge at Borgharen is approximately 5%. The increase of the discharge at Membre is relatively small in that period. The increase of the discharge in the Semois during a flood is not so great and fast as in some other rivers in the Ardennes.

**TABLE 6.3 THE SEMOIS AT MEMBRE**

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
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<tr>
<td>January</td>
<td>46.2</td>
<td>112.9</td>
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</tr>
<tr>
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<td>47.8</td>
<td>95.2</td>
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<td>March</td>
<td>35.2</td>
<td>80.7</td>
<td>0.95</td>
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<tr>
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<td>28.3</td>
<td>71.9</td>
<td>0.83</td>
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<tr>
<td>May</td>
<td>19.5</td>
<td>80.9</td>
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<tr>
<td>June</td>
<td>13.7</td>
<td>83.2</td>
<td>0.35</td>
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<tr>
<td>July</td>
<td>12.4</td>
<td>90.3</td>
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</tr>
<tr>
<td>August</td>
<td>11.1</td>
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<td>94.9</td>
<td>0.26</td>
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<tr>
<td>October</td>
<td>18.0</td>
<td>83.3</td>
<td>0.47</td>
</tr>
<tr>
<td>November</td>
<td>30.1</td>
<td>121.0</td>
<td>0.52</td>
</tr>
<tr>
<td>December</td>
<td>47.4</td>
<td>126.3</td>
<td>0.81</td>
</tr>
<tr>
<td>Year Total</td>
<td>26.7</td>
<td>1139.2</td>
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</tr>
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</table>

maximum discharge at flood February 1980: 203 m³/s
maximum discharge at flood July 1980: 80 m³/s
maximum discharge at flood February 1984: 318 m³/s
minimum discharge at low flow 1976: 0.54 m³/s
TABLE 6.4 FLOOD WAVES IN THE SEMOIS AND IN THE MEUSE

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
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<td>11</td>
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</table>

| day              | disch.    | s.d.             |
| 17               | 13        | 0.9              |
| 18               | 10        | 0.7              |
| 19               | 14        | 1.0              |
| 20               | 48        | 3.4              |
| 21               | 140       | 9.8              |
| 22               | 92        | 6.4              |
| 23               | 33        | 2.3              |
| 24               | 18        | 1.3              |

disch. = discharge [m³/s]
s.d. = specific discharge [mm/day]

6.4 THE VIROIN

One of the medium-sized basins is the Viroin basin. The catchment area amounts to 593 km², of which 554 km² lies above the measuring station of Treignes. The water is supplied via the Eau Blanche and the Eau Noire. Below the confluence of the two rivers the river is called Viroin. (fig. 6.3).

The source of the Eau Noire is taken as the source of the Viroin. It is situated in France near Rocroi. The greater part of the catchment is found in Belgium, although the mouth is in France again, viz. near Vireux.

The southern section of the catchment forms part of the Rocroi Massif, with hard and impermeable Cambrian rock. The northern section and the central part contain rock from the Devonian, combined with limestone, for instance.

The source lies at NAP + 359 m, the mouth at NAP + 106 m. The highest point of the catchment is at NAP + 388. The river - Eau Noire and Viroin taken together - is only 57 km long. The gradient is 2 10⁻³.

The mean annual precipitation for the basin is 940 mm (table 6.5). Most precipitation falls in the southwest of the catchment. The Forges station has a mean annual precipitation of 1,088 mm (Bultot et al., 1971). The mean discharge amounts to 6.9 m³/s. In periods of much precipitation the discharge may rise very strongly (table 6.6). 43% of the catchment area consists of forests, 90% of which is deciduous forest.

73
TABLE 6.5 THE VIROIN AT TREIGNES

<table>
<thead>
<tr>
<th>River: Viroin</th>
<th>Measuring station: Treignes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area above measuring station: 554 km²</td>
<td>Wave travel time Treignes-Borgharen: 20 h</td>
</tr>
</tbody>
</table>

The areal precipitation has been determined for the area above Vierves, which is about 96 % of the area above Treignes

<table>
<thead>
<tr>
<th>Month</th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>10.6</td>
<td>83.4</td>
<td>0.66</td>
</tr>
<tr>
<td>February</td>
<td>14.4</td>
<td>75.3</td>
<td>0.84</td>
</tr>
<tr>
<td>March</td>
<td>11.1</td>
<td>65.9</td>
<td>0.82</td>
</tr>
<tr>
<td>April</td>
<td>8.3</td>
<td>64.6</td>
<td>0.60</td>
</tr>
<tr>
<td>May</td>
<td>6.3</td>
<td>71.2</td>
<td>0.43</td>
</tr>
<tr>
<td>June</td>
<td>4.2</td>
<td>76.8</td>
<td>0.26</td>
</tr>
<tr>
<td>July</td>
<td>4.3</td>
<td>79.1</td>
<td>0.26</td>
</tr>
<tr>
<td>August</td>
<td>1.9</td>
<td>88.2</td>
<td>0.10</td>
</tr>
<tr>
<td>September</td>
<td>1.3</td>
<td>76.8</td>
<td>0.08</td>
</tr>
<tr>
<td>October</td>
<td>3.6</td>
<td>70.6</td>
<td>0.24</td>
</tr>
<tr>
<td>November</td>
<td>7.0</td>
<td>97.7</td>
<td>0.34</td>
</tr>
<tr>
<td>December</td>
<td>10.8</td>
<td>90.0</td>
<td>0.58</td>
</tr>
<tr>
<td>Year</td>
<td>6.9</td>
<td></td>
<td>0.42</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>939.6</td>
</tr>
</tbody>
</table>

maximum discharge at flood February 1980: 69 m³/s
maximum discharge at flood July 1980: 140 m³/s
maximum discharge at flood February 1984: 103 m³/s
minimum discharge at low flow 1976: 0.21 m³/s

In one of the branches of the Eau Noire, the Ry de Rome, there is a reservoir with a capacity of 2.2 10⁶ m³. The purpose of the reservoir, which is called after the rivulet, is to provide the surroundings with drinking-water and industrial water. According to the Ministère des Travaux Publics the capacity of the reservoir finished in 1974 is sufficient for supplying 8,600 m³/day (0.1 m³/s) during two dry years. The reservoir has a contributing area of only 10 km² (Anonymous, 1985c).

In the area calcareous soil is found showing features of karst. Because of it the discharges in the Eau Noire and Eau Blanche are essentially decreased in places. The Viroin is partly fed in that way. Near the mouth the Viroin flows through schists of a Cambrian origin so that it may be assumed that the greater part of the discharge flows out into the Meuse via the measuring station.
Figure 6.3 The catchment of the Viroin

| TABLE 6.6 FLOOD WAVES IN THE VIROIN AND IN THE MEUSE |
|---------------------------------|--------|--------|--------|--------|--------|--------|--------|
| Feb. Treignes Borgharen 1980    |        |        |        |        |        |        |        |
| day  | disch. | s.d.   | disch. | s.d.   | day  | disch. | s.d.   | disch. | s.d.   |
| 1    | 27     | 4.2    | 582    | 2.4    | 17   | 13     | 2.0    | 568    | 2.3    |
| 2    | 24     | 3.7    | 573    | 2.3    | 18   | 10     | 1.6    | 509    | 2.1    |
| 3    | 56     | 8.7    | 896    | 3.6    | 19   | 14     | 2.2    | 504    | 2.0    |
| 4    | 69     | 10.8   | 1105   | 4.5    | 20   | 48     | 7.5    | 965    | 3.9    |
| 5    | 57     | 8.9    | 1369   | 5.6    | 21   | 140    | 21.8   | 1930   | 7.8    |
| 6    | 51     | 8.0    | 1440   | 5.9    | 22   | 92     | 14.3   | 2000   | 8.1    |
| 7    | 41     | 6.4    | 1364   | 5.5    | 23   | 33     | 5.1    | 1340   | 5.4    |
| 8    | 29     | 4.5    | 1233   | 5.0    | 24   | 18     | 2.8    | 917    | 3.7    |
| Feb. Treignes Borgharen 1984    |        |        |        |        |        |        |        |
| day  | disch. | s.d.   | disch. | s.d.   |       |        |        |        |
| 5    | 33     | 5.1    | 1332   | 5.4    |       |        |        |        |
| 6    | 38     | 5.9    | 1261   | 5.1    |       |        |        |        |
| 7    | 103    | 16.1   | 1835   | 7.5    |       |        |        |        |
| 8    | 78     | 12.2   | 2466   | 10.0   |       |        |        |        |
| 9    | 64     | 10.0   | 2236   | 9.1    |       |        |        |        |
| 10   | 40     | 6.2    | 1912   | 7.8    |       |        |        |        |
| 11   | 25     | 3.9    | 1592   | 6.5    |       |        |        |        |
| 12   | 18     | 2.8    | 1360   | 5.5    |       |        |        |        |

*disch. = discharge [m³/s]*
*s.d. = specific discharge [mm/day]*
6.5 THE LESSE

The Lesse may be considered a typical Ardennes river. It springs at NAP +403 m near Libramont. The rivers Gembes, Lomme, Wimbe and close to its mouth the Ywenne flow out into the Lesse. The mouth is at a level of NAP + 89 m just below the weir of Anseremme, 3 km south of Dinant. The length of the river is 83 km. Its highest point is at NAP + 585 m in the east of the catchment of the Lomme. The measuring station of the Ministerie van Openbare Werken is at Gendron, a few kilometres upstream from the mouth. A survey of the catchment of the Lesse and the adjoining catchment of the Houille is given in fig. 6.4.

In the catchment calcareous soil is found, originating from the Devonian and the Carboniferous. The calcareous soil shows karst features. Underground parts of the river are found in the Wamme (between On and Eprave), in the Lesse (near Belvaux and Han) and moreover in some more smaller tributaries. In figure 6.4 those sections are indicated by means of dotted lines.

--- boundary of the catchment
--- measuring station (water level and/or discharge)
○ precipitation station
□ indication of place
--- in part underground course
--- border

Figure 6.4 The catchment of the Lesse

76
TABLE 6.7 THE LESSE AT GENDRON

<table>
<thead>
<tr>
<th>Month</th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>25.1</td>
<td>83.2</td>
<td>0.61</td>
</tr>
<tr>
<td>February</td>
<td>33.9</td>
<td>76.8</td>
<td>0.82</td>
</tr>
<tr>
<td>March</td>
<td>25.0</td>
<td>65.1</td>
<td>0.78</td>
</tr>
<tr>
<td>April</td>
<td>20.4</td>
<td>65.7</td>
<td>0.61</td>
</tr>
<tr>
<td>May</td>
<td>15.1</td>
<td>75.6</td>
<td>0.41</td>
</tr>
<tr>
<td>June</td>
<td>8.5</td>
<td>79.7</td>
<td>0.21</td>
</tr>
<tr>
<td>July</td>
<td>11.1</td>
<td>83.9</td>
<td>0.27</td>
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<tr>
<td>August</td>
<td>5.9</td>
<td>90.2</td>
<td>0.13</td>
</tr>
<tr>
<td>September</td>
<td>4.1</td>
<td>78.4</td>
<td>0.10</td>
</tr>
<tr>
<td>October</td>
<td>8.1</td>
<td>70.1</td>
<td>0.24</td>
</tr>
<tr>
<td>November</td>
<td>14.3</td>
<td>95.8</td>
<td>0.29</td>
</tr>
<tr>
<td>December</td>
<td>24.8</td>
<td>89.8</td>
<td>0.56</td>
</tr>
<tr>
<td>Year</td>
<td>16.3</td>
<td></td>
<td>0.41</td>
</tr>
<tr>
<td>Total</td>
<td>954.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

maximum discharge at flood February 1980: 119 m³/s
maximum discharge at flood July 1980: 202 m³/s
maximum discharge at flood February 1984: 157 m³/s
minimum discharge at low flow 1976: 0.71 m³/s

Near its source the gradient of the Lesse is great and towards the mouth it decreases more and more. The mean gradient amounts to approximately 5 x 10⁻³.

Approximately 40% of the catchment is covered with forests. The proportion between deciduous forests and softwoods is about 7:3. There are no reservoirs in the catchment.

The mean annual precipitation amounts to 967 mm, which is over 150 mm less than the precipitation in the Semois catchment (table 6.7). The precipitation is greatest in the south (1,250 mm, Paliseul) and smallest in the west (800 mm, Beauraing). Near the highest point of the catchment the annual precipitation amounts to 1,120 mm (St. Hubert) (Bultot et al., 1973).

Up to their flowing out into the Lesse most of the tributaries have the same kind of course as the Lesse, regarding their length and their gradient. The result is that a shower which suddenly occurs throughout the area is likely to have a great peak discharge (the instantaneous hydrograph has a high peak value). The effect is also to be observed during the flood of 1980 (table 6.8). The momentaneous peak discharge amounted to 228 m³/s. Twenty-four hours before the discharge had only been 95 m³/s, twenty-four hours afterwards it was 149 m³/s. From table 6.8 it appears that the 1980 wave described was not an exception in the rapid rise of the discharge.

When investigating the low flows the IRM reached the conclusion that with a
discharge of over 10 m³/s the discharge is halved in 10 days and with smaller discharges in 25 days. (Bultot et al., 1973). On account of the absence of reservoirs the low flow discharge may sink to very low values (table 6.7).

**TABLE 6.8 FLOOD WAVES IN THE LESSE AND IN THE MEUSE**

<table>
<thead>
<tr>
<th>Feb. 1980</th>
<th>Gendron</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
<td>1</td>
<td>45</td>
<td>3.0</td>
</tr>
<tr>
<td>2</td>
<td>38</td>
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<td>3</td>
<td>86</td>
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<td>4</td>
<td>116</td>
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<td>5</td>
<td>119</td>
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<td>6</td>
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<td>7</td>
<td>100</td>
<td>6.6</td>
</tr>
<tr>
<td>8</td>
<td>79</td>
<td>5.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>July 1980</th>
<th>Gendron</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
<td>17</td>
<td>51</td>
<td>3.4</td>
</tr>
<tr>
<td>18</td>
<td>43</td>
<td>2.8</td>
</tr>
<tr>
<td>19</td>
<td>45</td>
<td>3.0</td>
</tr>
<tr>
<td>20</td>
<td>91</td>
<td>6.0</td>
</tr>
<tr>
<td>21</td>
<td>202</td>
<td>13.3</td>
</tr>
<tr>
<td>22</td>
<td>158</td>
<td>10.4</td>
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<tr>
<td>23</td>
<td>103</td>
<td>6.8</td>
</tr>
<tr>
<td>24</td>
<td>65</td>
<td>4.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Feb. 1984</th>
<th>Gendron</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
<td>5</td>
<td>83</td>
<td>5.5</td>
</tr>
<tr>
<td>6</td>
<td>79</td>
<td>5.2</td>
</tr>
<tr>
<td>7</td>
<td>157</td>
<td>10.3</td>
</tr>
<tr>
<td>8</td>
<td>147</td>
<td>9.7</td>
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<tr>
<td>9</td>
<td>143</td>
<td>9.4</td>
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<tr>
<td>10</td>
<td>113</td>
<td>7.4</td>
</tr>
<tr>
<td>11</td>
<td>84</td>
<td>5.5</td>
</tr>
<tr>
<td>12</td>
<td>67</td>
<td>4.4</td>
</tr>
</tbody>
</table>

| disch. = discharge [m³/s] |
| s.d. = specific discharge [mm/day] |

6.6 THE SAMBRE

As compared with the other tributaries the Sambre is special in being the only river that admits of shipping over a greater length.

The source of the Sambre is in France, north of Garmouzet, at NAP + 210 m. Its mouth is in Namur at NAP + 77 m. The length of the river amounts to approximately 184 km. The main tributaries are the Helpe Mineure, the Helpe Majeure, the Solre, the Thure and the Eau d'Heure. The first four tributaries flow out into the upper course of the Sambre, the last into the central course.

The catchment is 2,863 km² in size, 80% of which is above Charleroi. The mean gradient amounts to 7 × 10⁻⁴. In the 85 km long Belgian stretch the gradient is 5 × 10⁻⁴ on an average. A survey of the catchment of the Sambre and of some nearby areas is given in figure 6.5.
Figure 6.5 The catchments of the Sambre and some nearby rivers
Geologically, the catchment lies in Palaeozoic rock, the Carboniferous sediments of which are of special importance for the mining of coal in the region. The mining has been of great importance for the development of the network of waterways.

The Sambre is an important waterway. Most ships in the Meuse sail into the Sambre in Namur. In Charleroi there is an important connection with Bruxelles by way of the Canal de Charleroi à Bruxelles. Upstream from Charleroi the Sambre is only to be navigated by small ships. Via the Canal de la Sambre à l'Oise there is a connection with the French network of waterways. It is noted that because of the canal the catchment boundary of Sambre and Oise is no longer univocal. In general the catchment boundary is supposed to be near Oisy.

From approximately 20 km from the source the Sambre has been canalised. The discharge course of the river is hardly natural owing to the presence of many weirs and locks. The discharge is further influenced by man through artificial withdrawals and discharges.

Especially below Charleroi the banks of the river are formed by a fixed construction with steep slopes. There are hardly any flood plains. Owing to shipping the bed profile is so large that there are hardly any inundations at times of floods. The striking construction has consequences for the flood wave propagation, since due to the widening of the midstream the peaks are hardly attenuated.

The weirs below Monceau (near Charleroi) are operated by electric motors. The weirs have such generous dimensions that even in cases of rather great flood waves they keep exercising their damming function. Above Monceau there are mainly unmechanised weirs, i.e. the horizontally placed beams have to be removed and placed by muscular strength. The weir elements are removed with a much smaller discharge than below Monceau.

As the water levels are influenced by weirs everywhere, it is not possible to determine the discharge using a water level only. With some weirs the discharge can be determined with the help of the position of the weir, though. However, most of those observations are not so frequent - once daily - and lack the accuracy of the other discharges presented in this chapter.

Once daily the measuring of the discharge is performed at the weir of Namur-Salzinnes (photo 6.1). From an investigation it has appeared that the discharges are not representative of the mean daily discharge (Canaparroz, 1989). However, for the investigation after flow forecasting it is desirable to have an approximation of the discharges occurring. By means of the data on a great number of tributaries of the Sambre a relatively simple relation can finally be found between the Sambre discharge and the discharges of the Molignée and the Mehaigne (Canaparroz, 1989). The relation reads as follows:

$$Q_{Namur} = 7.466Q_{Moha} + 10.370Q_{Warnant} - 5.358$$

(6.1)

where:

$$Q_{Namur} = \text{discharge of the Sambre at Namur} \quad [\text{m}^3/\text{s}]$$
$$Q_{Moha} = \text{discharge of the Mehaigne at Moha} \quad [\text{m}^3/\text{s}]$$
$$Q_{Warnant} = \text{discharge of the Molignée at Warnant} \quad [\text{m}^3/\text{s}]$$

The relation has been established by utilising multiple linear regression analysis and both for small and for great discharges it appears to suffice reasonably. Only peak discharges are to be established to a limited extent because the investigation has been performed by means of data on a daily basis. Consequently it is not permitted to calculate the peak discharges with the aid of the formula of hourly values and momentaneous peak discharges. In the event of the extrapolation of the relation to great discharges some prudence should be observed.
Mainly below Charleroi shipping is intensive. As a result the locks need much water, more than can be supplied in a natural way in dry periods. As an addition to the discharge in dry periods a complex of reservoirs has been built in the Eau d'Heure. The complex is intended to maintain the discharge in the Sambre at a minimum of 5 m³/s. The additional objectives are: a. decreasing the consequences of industrial and household pollution in the Sambre by means of dilution, and b. enlarging the low flow in the Meuse. The complex has no function in the supply of drinking water so that recreation in the reservoir is allowed (Anonymous, 1985c).

The complex of reservoirs in the Eau d'Heure consists of several reservoirs (fig. 6.6). The Eau d'Heure reservoir (17.9 10⁶ m³) has a central position. The reservoir is smaller than originally planned (47 10⁶ m³) in order to spare the village of Cerfontaine.

In order still to have a sufficiently great stock of water another reservoir has been built in a nearby valley, the Plate Taille, with a capacity of 67.8 10⁶ m³. The position of the reservoir is 43 m higher than that of the Eau d'Heure reservoir. The valley in which the reservoir is situated has the hydrologic drawback that the natural supply of water is small. For that reason the reservoir has to be fed artificially. For that purpose pumps (turbines) have been built, which can pump the water from the Eau d'Heure to the Plate Taille. On financial grounds it is only done at night. When the water is needed again, energy is generated, which on the same grounds is only done by day.

On account of the seasonal supply and the daily pumpings the level of the water in the
Eau d’Heure fluctuates considerably. In order to maintain its touristic function three preliminary dams have been built. The function of the three dams is keeping the water level in the preliminary reservoirs as constant as possible. The difference between the water level of the preliminary reservoirs and the Eau d’Heure is approximately 6 m. The capacities of the preliminary reservoirs are respectively: $1.2 \times 10^6$ m$^3$ (Falemprise), $1.1 \times 10^6$ m$^3$ (Ry Jaune) and $0.8 \times 10^6$ m$^3$ (Feronval). For the Eau d’Heure proper $14.8 \times 10^6$ m$^3$ is left.

The complex of reservoirs was built in the seventies of this century. The first to be completed was a preliminary dam (1975), the last was the Plate Taille dam (1980).

It took many years to fill the complex of reservoirs. That is the reason why a transitional period of several years elapsed between the situation with the natural discharge of the Eau d’Heure and the ultimate artificial situation. The discharge data published most recently date from 1981, a year in which the reservoirs were not completely filled yet. An analysis of those discharges is likely to lead to hardly any result and is therefore left out.

In France there are some more small reservoirs. The largest of them is the reservoir of Lac du Val Joly in the Helpe Majeure.

The mean annual precipitation is 825 mm. The southern part of the catchment receives more precipitation than the north. At Namur-Salzinnes the discharge is from 25 to 28 m$^3$/s on an average (table 6.9). The measured value is from Vereerstraeten (1975).

During the 1984 flood the greatest discharge in the Sambre amounted to between 300 and 400 m$^3$/s (table 6.10). It was approximately 15% of the Meuse discharge at Borgharen.
### TABLE 6.9 THE SAMBRE AT NAMUR

River: Sambre  
Measuring station: Namur-Salzinnes  
Area above measuring station: 2863 km²  
Wave travel time Namur-Salzinnes - Borgharen: 7 h  
Reference period precipitation: Jan.1951-Dec.1975  

<table>
<thead>
<tr>
<th></th>
<th>measured mean discharge [m³/s]</th>
<th>estimated mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>estimated runoff coefficient [-]</th>
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</thead>
<tbody>
<tr>
<td>January</td>
<td>33.1</td>
<td>68.9</td>
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<td>0.45</td>
</tr>
<tr>
<td>February</td>
<td>41.7</td>
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<td></td>
<td>0.56</td>
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<td>March</td>
<td>37.0</td>
<td>58.3</td>
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<td>23.8</td>
<td>67.0</td>
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</tr>
<tr>
<td>June</td>
<td>17.7</td>
<td>73.4</td>
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<td>0.22</td>
</tr>
<tr>
<td>July</td>
<td>18.8</td>
<td>75.5</td>
<td></td>
<td>0.23</td>
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<td>0.14</td>
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<td>October</td>
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<td>November</td>
<td>22.0</td>
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<tr>
<td>December</td>
<td>36.3</td>
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<td>0.44</td>
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<tr>
<td>Year Total</td>
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<td>24.9</td>
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<td>0.33</td>
</tr>
<tr>
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</tr>
</tbody>
</table>

- estimated maximum discharge at flood February 1980: 160 m³/s  
- measured maximum discharge at flood February 1980: 414 m³/s  
- estimated maximum discharge at flood July 1980: 380 m³/s  
- measured maximum discharge at flood July 1980: 300 m³/s  
- estimated maximum discharge at flood February 1984: 280 m³/s  
- measured maximum discharge at flood February 1984: 340 m³/s

**Note:**  
1. With the flood discharges mentioned the second figure is only significant to a small extent  
### TABLE 6.10 FLOOD WAVES IN THE SAMBRE AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Feb. 1980</th>
<th>Namur</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>e.d.</td>
<td>e.s.d.</td>
</tr>
<tr>
<td>1</td>
<td>80</td>
<td>2.5</td>
</tr>
<tr>
<td>2</td>
<td>70</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>120</td>
<td>3.5</td>
</tr>
<tr>
<td>4</td>
<td>160</td>
<td>5.0</td>
</tr>
<tr>
<td>5</td>
<td>160</td>
<td>5.0</td>
</tr>
<tr>
<td>6</td>
<td>140</td>
<td>4.0</td>
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<tr>
<td>7</td>
<td>120</td>
<td>3.5</td>
</tr>
<tr>
<td>8</td>
<td>90</td>
<td>2.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>July 1980</th>
<th>Namur</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>e.d.</td>
<td>e.s.d.</td>
</tr>
<tr>
<td>17</td>
<td>40</td>
<td>1.0</td>
</tr>
<tr>
<td>18</td>
<td>30</td>
<td>1.0</td>
</tr>
<tr>
<td>19</td>
<td>40</td>
<td>1.0</td>
</tr>
<tr>
<td>20</td>
<td>210</td>
<td>6.5</td>
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<tr>
<td>21</td>
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<td>11.5</td>
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<td>23</td>
<td>150</td>
<td>4.5</td>
</tr>
<tr>
<td>24</td>
<td>90</td>
<td>2.5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Feb. 1984</th>
<th>Namur</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>e.d.</td>
<td>e.s.d.</td>
</tr>
<tr>
<td>5</td>
<td>50</td>
<td>1.5</td>
</tr>
<tr>
<td>6</td>
<td>60</td>
<td>2.0</td>
</tr>
<tr>
<td>7</td>
<td>130</td>
<td>4.0</td>
</tr>
<tr>
<td>8</td>
<td>280</td>
<td>8.5</td>
</tr>
<tr>
<td>9</td>
<td>270</td>
<td>8.0</td>
</tr>
<tr>
<td>10</td>
<td>180</td>
<td>5.5</td>
</tr>
<tr>
<td>11</td>
<td>110</td>
<td>3.5</td>
</tr>
<tr>
<td>12</td>
<td>40</td>
<td>1.0</td>
</tr>
</tbody>
</table>

e.d. = estimated discharge [m³/s]
e.s.d. = estimated specific discharge [mm/day]
m.d. = measured discharge [m³/s]
m.s.d. = measured specific discharge [mm/day]
6.7 THE OURTHE

Of all the subcatchments the Ourthe has the largest area (3,626 km² at Angleur). It is a typically Ardennes river and therefore has great discharges rising fast. Through its nature and situation (close to the Dutch border) the Ourthe is also the most important Meuse tributary for flood forecasts.

In its upper course the Ourthe consists of two branches: the Ourthe Occidentale and the Ourthe Orientale, uniting near Nisramont. Near Comblain-au-Pont the Amblève joins the Ourthe and near Angleur the Ourthe also receives the Vesdre. Counting from the source of the Ourthe Occidentale the Ourthe is approximately 175 km long.

From a hydrologic point of view the tributaries Amblève and Vesdre are so important that separate sections will be devoted to them. Two reasons may be mentioned for it:
1. the discharges can be greater than those of many other Meuse tributaries, and
2. the mouths of both tributaries are to be found near the mouth of the Ourthe.

Moreover, a very practical point underlies the subdivision: the measuring station in the Ourthe near its mouth does not always produce reliable values of the discharges as a result of the changing positions of the weir elements of the Grosses Battes weir (at Angleur).

This section will first treat the tributaries Vesdre and Amblève. Then the Ourthe upstream from Tabreux and finally the remaining part of the catchment will be dilated on.

6.7.1 THE VESDRE

The Vesdre springs at NAP + 626 m east of Eupen. The mouth in the Ourthe is near Chênée (a Liège suburb) and Angleur at NAP + 62 m. The main tributaries are the Hoegne and the Gileppe. The river is approximately 72 km long (fig. 6.7).

For the main part the catchment lies on rocks of Palaeozoic origin. In some places they are cracked and carrying water. However, the discharge through the rocks is very small.

A special part in the hydrologic system is played by the Hautes Fagnes (High Moors). The Vesdre catchment only comprises the northern section of the Hautes Fagnes, a comparatively flat area at a level of 600 m and rather limited in size. The Hautes Fagnes also drain into the Amblève and to a small extent into the Roer, too. The Hautes Fagnes have a relatively high moisture content. It is caused by the fact that the rocks are covered with a stratum of reasonably permeable silex clay (clay with flint) with a layer of peat on top, though eroded clay in some places. That part of the catchment responds relatively slowly to the precipitation fallen. The highest point in the Hautes Fagnes is Botrange (NAP + 692 m), which is also the highest point in the whole Meuse basin. Botrange is part of the catchment boundaries of the Vesdre and the Amblève.

The larger part of the Vesdre catchment has steep slopes, so that with heavy showers short and high peaks of discharge arise. The result is inundations in the river valley.

A flood wave has soon disappeared from the area because the gradient of the river amounts to 8·10⁻³.

One more consequence of the fast runoff is the shortage of water in summer. To diminish the shortages of water two large reservoirs have been built in the Vesdre area. The subject will be returned to afterwards.

Of the area 44% is wooded, 2/3 of which consisting in coniferous woods.

The mean annual precipitation is 1,104 mm. The distribution of the precipitation within the area is connected with its high position. 15% of the precipitation falls on days when there is at least 20 mm. An analytic calculation of the IRM has shown that at a level of
550 m the snow cover will be higher than 26 cm once in two years on an average (Bultot et al., 1975).

Apart from differences through natural causes the Vesdre also differs from the nearby rivers on account of the presence of two large reservoirs.

Near Eupen in the Vesdre lies the Vesdre reservoir, with a capacity of $25 \times 10^6$ m$^3$. Finished in 1949 the reservoir has primarily been constructed to supply the waterworks and industries in the catchment and in Liège with a quantity of water of approximately 75,000 m$^3$/day (0.87 m$^3$/s) (Anonymous, 1985c, Derycke et al., 1982). A secondary aim is reducing flood waves in the Vesdre by blocking great discharges. The catchment area above the reservoir amounts to 106 km$^2$.

In the tributary Gileppe there is another large reservoir with a capacity of $26 \times 10^6$ m$^3$. Originally it dates from 1876. In 1971 the dam was heightened. The contributing area is 54 km$^2$. The reservoir is used for the same purposes as the Vesdre reservoir and is also to be capable of supplying 75,000 m$^3$/day.

Together the two reservoirs can block the water coming from 24% of the Vesdre catchment. Here it should be noted that that part actually comprises the section of the area richest in precipitation (Anonymous, 1985c).

It may be remarked that the contributing areas of the two reservoirs are enlarged artificially through extracting water from nearby tributaries.

Table 6.11 gives the precipitation of the area and the discharges measured. Table 6.12 shows the development of the discharges during three flood waves. As the peak of the wave discharged in July 1980 occurred earlier than in the other tributaries a longer period has been
mentioned for it.

The reservoirs may have a significant influence on the runoff of the Vesdre. In the next few paragraphs the influence will be analysed.

First the Vesdre discharges at Verviers and Chaudfontaine are compared. The areas of the two catchments amount to 333 km² and 677 km² respectively. Unfortunately no long-term series of the former measuring station are available so that the investigation must be restricted to the years 1980 and 1981.

The discharges of the Vesdre above Verviers and of the catchment between the measuring stations of Verviers and Chaudfontaine will now be compared. In fig. 6.8 the results as to the mean monthly discharges are given (in the next few paragraphs the line drawn will be dilated upon). From the figure no definite seasonal influence is to be ascertained concerning the control of the reservoirs. Possibly it is due to the small size of the series and the wet summer to which the series applies. However, from the observations it can be deduced that the specific discharge of the downstream stretch is greater than that of the upstream part, but that need not necessarily be caused by the reservoirs.

**TABLE 6.11 THE VESDRE AT CHAUDFONTAINE**

<table>
<thead>
<tr>
<th>River: Vesdre</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measuring station: Chaudfontaine</td>
</tr>
<tr>
<td>Area above measuring station: 677 km²</td>
</tr>
<tr>
<td>Wave travel time Chaudfontaine-Borgharen: 5 h</td>
</tr>
<tr>
<td>Reference period precipitation: Jan.1951-Dec.1975</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>13.1</td>
<td>91.3</td>
<td>0.57</td>
</tr>
<tr>
<td>February</td>
<td>14.1</td>
<td>81.9</td>
<td>0.62</td>
</tr>
<tr>
<td>March</td>
<td>13.1</td>
<td>75.1</td>
<td>0.69</td>
</tr>
<tr>
<td>April</td>
<td>11.1</td>
<td>74.9</td>
<td>0.57</td>
</tr>
<tr>
<td>May</td>
<td>7.5</td>
<td>86.6</td>
<td>0.34</td>
</tr>
<tr>
<td>June</td>
<td>5.3</td>
<td>97.9</td>
<td>0.21</td>
</tr>
<tr>
<td>July</td>
<td>8.1</td>
<td>112.0</td>
<td>0.29</td>
</tr>
<tr>
<td>August</td>
<td>6.9</td>
<td>115.4</td>
<td>0.24</td>
</tr>
<tr>
<td>September</td>
<td>4.7</td>
<td>86.4</td>
<td>0.21</td>
</tr>
<tr>
<td>October</td>
<td>6.7</td>
<td>80.0</td>
<td>0.33</td>
</tr>
<tr>
<td>November</td>
<td>8.7</td>
<td>99.1</td>
<td>0.34</td>
</tr>
<tr>
<td>December</td>
<td>13.9</td>
<td>102.9</td>
<td>0.53</td>
</tr>
<tr>
<td>Year</td>
<td>9.4</td>
<td></td>
<td>0.40</td>
</tr>
<tr>
<td>Total</td>
<td>1103.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Maximum discharge at flood February 1980: 64 m³/s
- Maximum discharge at flood July 1980: 177 m³/s
- Maximum discharge at flood February 1984: 141 m³/s
- Minimum discharge at low flow 1976: 2.0 m³/s
### TABLE 6.12 FLOOD WAVES IN THE VESDRE AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Feb. 1980</th>
<th>Chaudfontaine</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
<td>8</td>
<td>12</td>
<td>1.5</td>
</tr>
<tr>
<td>9</td>
<td>16</td>
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<td>10</td>
<td>110</td>
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<td>11</td>
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<td>14</td>
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<td>28</td>
<td>2.3</td>
</tr>
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<td>124</td>
<td>15.8</td>
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<td>148</td>
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<tr>
<td>22</td>
<td>81</td>
<td>10.3</td>
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<td>5.2</td>
</tr>
<tr>
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<td>25</td>
<td>3.2</td>
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<table>
<thead>
<tr>
<th>July 1980</th>
<th>Chaudfontaine</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
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<td>48</td>
<td>6.1</td>
</tr>
<tr>
<td>6</td>
<td>54</td>
<td>6.9</td>
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<tr>
<td>7</td>
<td>141</td>
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<tr>
<td>8</td>
<td>99</td>
<td>12.6</td>
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<td>9</td>
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<td>7.8</td>
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<td>4.0</td>
</tr>
<tr>
<td>12</td>
<td>26</td>
<td>3.3</td>
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</table>

<table>
<thead>
<tr>
<th>Feb. 1984</th>
<th>Chaudfontaine</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>disch.</td>
<td>s.d.</td>
</tr>
<tr>
<td>5</td>
<td>48</td>
<td>6.1</td>
</tr>
<tr>
<td>6</td>
<td>54</td>
<td>6.9</td>
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<tr>
<td>7</td>
<td>141</td>
<td>18.0</td>
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<td>8</td>
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<td>9</td>
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<td>7.8</td>
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<td>10</td>
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<td>11</td>
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<td>4.0</td>
</tr>
<tr>
<td>12</td>
<td>26</td>
<td>3.3</td>
</tr>
</tbody>
</table>

With the data on the discharge it will now be tried analytically to arrive at a quantitative value of the flow rate withdrawn from the natural discharge. In doing so seasonal influences will be neglected.

The point of departure is the natural discharge of the Vesdre above Verviers, i.e. the discharge that would occur in the absence of the reservoir. That natural discharge is called $Q_n^{Ver}$.

Assume the natural discharge of the downstream stretch is equal to a factor $b$ times its natural discharge of the upstream part, so $b Q_n^{Ver}$.

However, in the upstream section there are reservoirs by which part of the natural discharge of the river is withdrawn. The quantity withdrawn will partly be restored to the Vesdre above the highest measuring station of Verviers. That quantity is not to be established by means of the Verviers and Chaudfontaine data and will consequently be left aside. The discharge withdrawn from the natural course and not restored to the Vesdre above Verviers is

---

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called \( a \).

Part of the discharge withdrawn \( a \) is likely to return to the Vesdre between Verviers and Chaudfontaine. With the factor \( c \) that part is called \( ca \).

The real discharges at Verviers and Chaudfontaine amount to:

\[
Q_{\text{Ver}}^v = Q_n^{\text{Ver}} - a \tag{6.2}
\]

\[
Q_{\text{VC}}^{CN} = b Q_n^{\text{Ver}} + ca \tag{6.3}
\]

where:

- \( Q_{\text{Ver}}^v \) = discharge at Verviers \([L^3T^{-1}]\)
- \( Q_{\text{VC}}^{CN} \) = discharge at Chaudfontaine originating from the area between Verviers and Chaudfontaine \([L^3T^{-1}]\)
- \( Q_n^{\text{Ver}} \) = natural discharge at Verviers \([L^3T^{-1}]\)
- \( a \) = discharge withdrawn from the natural course and not restored to the Vesdre above Verviers \([L^3T^{-1}]\)
- \( b \) = ratio of the natural discharge at Verviers and that at Chaudfontaine \([-]\)
- \( c \) = ratio of the discharge that is returned between Verviers and Chaudfontaine and the total discharge withdrawn \([-]\)

The natural discharge is unknown, but with the two equations it can be eliminated, so that:

\[
Q_{\text{VC}}^{CN} = b Q_{\text{Ver}}^v + a(b+c) \tag{6.4}
\]

If all the suppositions had been right all the points in fig. 6.8 would have formed one straight line. Presumably the suppositions are acceptable (the proof of the correctness of the figure, however, cannot be deduced from the line).

The line drawn has been determined by means of linear regression. The equation of the line found is:

\[
Q_{\text{VC}}^{CN} = 1.23 Q_{\text{Ver}}^v + 0.97 \tag{6.5}
\]

From that line \( b \) can be determined. If \( c \) is known, \( a \) can be established, too.

Seeing the position of the area to be serviced, the value of \( c \) is not likely to deviate strongly from 0.5. If \( c = 0.5 \), the equations yield:

\[
Q_{\text{Ver}}^v = Q_n^{\text{Ver}} - 0.56 \tag{6.6}
\]

\[
Q_{\text{VC}}^{CN} = 1.23 Q_n^{\text{Ver}} + 0.28 \tag{6.7}
\]

The change of the discharge turns out to be rather insensible to variations of \( c \).

From the numbers and the figure it appears that the discharge is only slightly influenced by the reservoirs.
Figure 6.8 Relation between the mean monthly discharges of the Vesdre catchment upstream from Verviers and the area between Verviers and Chaudfontaine for the years 1980 and 1981

It may be concluded that the reservoirs in the Gileppe and the Vesdre can hold and give out a great volume of water. In time the discharge to be given out is rather constant (to be derived from the needs). A low flow model for the Meuse will have to contain that constant component in the discharge, although it only concerns some per cent of the Maastricht-St.-Pieter discharge.

Flooding
Apart from their functions for the water supply the two reservoirs also have a task in the protection against floods. They should limit the discharges of the Vesdre. With relation to the discharges in the river Meuse those reservoirs have no task.

From considerations of the area it is to be deduced that in the case of floods (1980, 1984) the supply has a maximum in the order of 50 m³/s.

The main purpose of the reservoirs in the Vesdre catchment is to supply water in dry periods. Consequently a reservoir will never be empty. By that fact the capacity to receive flood waves is limited. However, even if 90% of the Gileppe is filled a great flood wave such as the 1984 one can completely be stored. For the reservoir near Eupen the figure is somewhat lower.

The water of a flood wave can be stored for a shorter or a longer time. At the flood in July 1980 the volume of the water stored additionally in the Vesdre reservoir was again discharged after a short time, viz. as soon as the water level in the Vesdre had fallen below
the flood level. It was done in connection with the possibility of the occurrence of a new flood wave. The hold-up of the discharge lasted 12 to 16 hours. The maximum percentage of fullness was 99.5% (Laurent et al., year unknown).

It could be noticed that the volume stored was discharged at the very moment when the water level in the Vesdre had fallen, but the Meuse was still dealing with its peak discharge. So a reservoir that reduces flood waves may also result in an enlargement of the peak elsewhere.

During the flood the Gileppe reservoir did not reach the maximum degree of fullness. The discharge was limited to 0.8 m³/s, whereas the supply was approximately 10 m³/s.

It may be asked if it is necessary to know the control of the reservoir in order to arrive at a good flood forecast. The question will be dilated upon in appendix 1. The Vesdre reservoirs will be used as examples there. Here only the conclusion will be mentioned: the fact that 25% of the catchment of the Vesdre receives its discharge in an artificial way has as a result that the influence of the reservoirs must be taken into account if an equally good flood forecast is desirable as for the other tributaries. Here both a reduction of the discharge and an increase should be taken into account. In general the quantity of water withdrawn or added artificially cannot be deduced from discharge figures by simple means. That is to say that calibration may produce a relatively bad discharge forecasting model and that if the control is altered a good model may arrive at incorrect results.

6.7.2 THE AMBLÈVE

The Ourthe is fed by the Amblève to a considerable extent. With a catchment of 1,052 km² the Amblève comprises nearly 30% of the total catchment of the Ourthe.

The Amblève springs at NAP + 584 m near Rivage. The mouth in the Ourthe near Comblain-au-Pont is at NAP + 108 m. The river is approximately 88 km long. The measuring station is situated 8 km above the mouth at Martinrive (fig. 6.9).

With the exception of the upper course the gradient is 5 \(10^{-3}\). A curiosity is formed by the Cascade of Coo, below the mouth of the Salm. Here over a length of 70 m the bed sinks as much as 12 m.

The Amblève has a number of tributaries showing great similarities from a hydrologic point of view. The Lienne, Salm, Eau Rouge, Warche and Recht all flow out into the "central course" of the Amblève, spring between 450 m and 650 m, are approximately 20 km long and for the greater part their catchments have rock of a Caledonian origin as their basis. Because of that structure of the catchment of the Amblève the river responds to precipitation very fast so that a great peak discharge is likely (tables 6.13 and 6.14). For an example the 1984 flood is mentioned, when the momentaneous peak discharge was 314 m³/s. 24 hours before the discharge had been 84 m³/s, 24 hours later it was 196 m³/s. The precipitation fell especially in the 24 hours before the peak passed Martinrive. In the 24 hours after it there were only some millimetres of precipitation.

In the greater part of the catchment the mean annual precipitation is between 1,000 and 1,100 mm, but in the higher northeastern region it increases greatly to nearly 1,500 mm near Botrange. The mean annual precipitation amounts to 1,104 mm (Bultot et al., 1973).

38% of the catchment is covered with wood, of which 9/10 are coniferous woods and 1/10 deciduous woods.
Figure 6.9 The catchment of the Amblève

There are three reservoirs in the area and a complex of reservoirs. All the reservoirs have been built for the generation of electricity (Anonymous, 1985c).

Two reservoirs are found in the catchment of the Warche. Most upstream lies the Butgenbach reservoir, with a capacity of $11 \times 10^6$ m$^3$ and a contributing area of 72 km$^2$.

Further downstream is the Robertville reservoir, with a capacity of $7.7 \times 10^6$ m$^3$ and a contributing area of 118 km$^2$, which includes the part above Butgenbach. The Butgenbach power station can make use of a fall of 154 m, the greatest to be realised in Belgium. The Robertville reservoir dates from 1929, the Butgenbach one from 1932.

In the system of reservoirs especially that of Butgenbach serves as a dynamic reservoir: its water level may strongly vary. The reservoir has a water control depending on the seasons. In the months of April and May it is totally full ($11 \times 10^6$ m$^3$) and in October and November the reservoir is minimally filled ($3 \times 10^6$ m$^3$). That means that in the summer half-year the downstream discharge is increased by an average of 0.5 m$^3$/s and in the winter half-year it is decreased by the same amount compared with the natural situation.

Apart from generating electricity the reservoirs also have a function in the reduction of floods. The flood discharge is blocked and at a later point of time it is discharged again.

A part of the discharge is used for the water supply of the town of Malmedy. About the Butgenbach reservoir it is known that in a few weekends a year it discharges additional
water for the sake of anglers and white-water canoeists.

In the Amblève near Lorcé there is a small reservoir for the generation of electricity. The Heid de Goreux reservoir has a capacity of only 50 $10^3$ m$^3$.

Near the Coo falls lies an energy accumulating station, intended to supply energy during peak hours. For that purpose one low-lying and two high reservoirs have been built near the river, with capacities of $4 \times 10^4$, $4.5 \times 10^4$ and $8.5 \times 10^4$ m$^3$ respectively. At times when electricity is hardly in demand the water of the lower reservoir is pumped up to the higher reservoirs. The complex of reservoirs has no function in the reduction or the supply of water.

### TABLE 6.13 THE AMBLÈVE AT MARTINRIVE

<table>
<thead>
<tr>
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<td><strong>areal precipitation [mm]</strong></td>
<td><strong>runoff coefficient [-]</strong></td>
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<td></td>
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<td>Total</td>
<td></td>
<td></td>
<td>1104.4</td>
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- maximum discharge at flood February 1980: 109 m$^3$/s
- maximum discharge at flood July 1980: 234 m$^3$/s
- maximum discharge at flood February 1984: 249 m$^3$/s
- minimum discharge at low flow 1976: 2.0 m$^3$/s

93
TABLE 6.14 FLOOD WAVES IN THE AMBLÈVE AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Feb.</th>
<th>Martinrive</th>
<th>Borgharen</th>
<th>July</th>
<th>Martinrive</th>
<th>Borgharen</th>
</tr>
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<td>s.d.</td>
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<td>917</td>
<td>3.7</td>
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<table>
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<tr>
<td>day</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>12</td>
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</tbody>
</table>

disch. = discharge [m³/s]  
s.d. = specific discharge [mm/day]

6.7.3 THE OURTHE UPSTREAM FROM TABREUX

The Ourthe consists of two branches, the Ourthe Occidentale and the Ourthe Orientale. In its upper course the former springs at approximately NAP + 500 m near Libramont, the latter at approximately NAP + 520 m near Beho. At present the confluence is in the Nisramont reservoir. Up to Tabreux many more tributaries flow out into the Ourthe (fig. 6.10). The Ourthe Occidentale is approximately 45 km long, the Ourthe Orientale is approximately 35 km. Between Nisramont and Comblain-au-Pont the water covers a distance of approximately 90 km.

The highest point of the catchment is at NAP + 658 m, near the source of the Martin-Moulin. The catchment above Tabreux has an area of 1,597 km². Palaeozoic rock especially from the Cambrian and the Devonian is found here.

The region has a mountainous character, which appears from the distribution of altitudes in the region. It is also to be deduced from the gradient of the river. Up to Tabreux the average gradient amounts to 3.7 10⁻³.

Thirty per cent of the area is occupied by woods, 70 % of which consisting in deciduous woods.

The mean discharge of the area is 968 mm. There is a clear correlation between the high position and the annual precipitation. In low-lying places the annual fall is approximately 900 mm, in high places approximately 1,100 mm. The precipitation stations in the central part of the catchment have the greatest correlation with the areal precipitation (Bultot et al., 1972).
The mean discharge amounts to 23.3 m³/s, the mean runoff coefficient is 0.48 (table 6.15).

The quantities of precipitation and the relatively small runoff coefficient indicate that the importance of the Ourthe in the case of flood waves is not caused by those components. It is the regional properties (impermeable grounds, great gradient in the river, steep slopes in the landscape) that are the causes of the fast increasing and great discharges.

The flood of 1980 provides an example of the fast changing course of the discharges (table 6.16). At a definite moment the discharge rose by 15 m³/s per hour.

The minor bed cannot cope with flood discharges, so that inundations regularly occur in the valley. In places the river may become over 100 metres wide. The influence of storage, however, remains relatively small.
TABLE 6.15 THE OURTHE AT TABREUX

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
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</thead>
<tbody>
<tr>
<td>January</td>
<td>37.2</td>
<td>85.3</td>
<td>0.73</td>
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<tr>
<td>February</td>
<td>43.0</td>
<td>75.9</td>
<td>0.97</td>
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<tr>
<td>March</td>
<td>30.0</td>
<td>65.4</td>
<td>0.76</td>
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<tr>
<td>April</td>
<td>28.7</td>
<td>66.0</td>
<td>0.76</td>
</tr>
<tr>
<td>May</td>
<td>16.8</td>
<td>76.0</td>
<td>0.41</td>
</tr>
<tr>
<td>June</td>
<td>10.8</td>
<td>81.7</td>
<td>0.23</td>
</tr>
<tr>
<td>July</td>
<td>13.1</td>
<td>86.8</td>
<td>0.27</td>
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<td>93.9</td>
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<td>8.0</td>
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<td>October</td>
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<td>November</td>
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<td>93.7</td>
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<tr>
<td>December</td>
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<td>Year Total</td>
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<table>
<thead>
<tr>
<th></th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>968.4</td>
</tr>
</tbody>
</table>

maximum discharge at flood February 1980: 153 m³/s
maximum discharge at flood July 1980: 242 m³/s
maximum discharge at flood February 1984: 247 m³/s
minimum discharge at low flow 1976: 0.64 m³/s

The Nisramont reservoir, completed 1958, has a capacity of 3 · 10⁶ m³. The purpose of the reservoir is the supply of water (Derycke et al., 1982) and the generation of electricity (Anonymous, 1985c). The influence of the reservoir on the discharge at Borgharen is small.

6.7.4 THE DOWNSTREAM SECTION OF THE OURTHE CATCHMENT

The downstream section of the Ourthe catchment is relatively small, it only covers 8% of the total catchment of the Ourthe. The contribution of precipitation falling in the area to the discharge is likely to be small, although the hilly region can actually supply a fast rising contribution to the discharge.

The main hydrologic function this part of the catchment has is the transport of water coming from higher parts. The transit is fast: in approximately 2 hours the water flows from the confluence with the Amblève to the mouth at Angleur (at NAP + 58 m). The flood peak attenuation is small in the stretch (appendix 2).

Near Angleur a canal is found parallel to the Ourthe, its length is 2.5 km. The canal comes to an end in the Meuse. The discharge through the canal is small.

96
TABLE 6.16 FLOOD WAVES IN THE OURTHE AT TABREUX AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Feb. 1980</th>
<th>Tabreux</th>
<th>Borgharen</th>
<th>July 1980</th>
<th>Tabreux</th>
<th>Borgharen</th>
</tr>
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<td>day</td>
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<td>s.d.</td>
<td>disch.</td>
<td>s.d.</td>
<td>disch.</td>
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<td>80</td>
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<table>
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<td>s.d.</td>
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6.8 THE ROER

The Roer catchment shows a striking dual division. The south of the area is very hilly, whereas the north is rather flat.

The source of the Roer is in the Hautes Fagnes. In its upper reaches the river flows through the Ardennes and then through the Eifel. The downstream course is in the Niederrheinisches Flachland. The mouth in the Meuse is at NAP + 1.5 m off Roermond. The river is 207 km long (Anonymous: Deutsches Gewässerkundliches Jahrbuch).

The tributaries Urft and Kall are entirely in the Eifel. The Inde only has its upper course in the Eifel. The Wurm flows practically entirely in the flat part of the catchment (fig. 6.11).

The dual character of the river also appears from its gradient. In its upper course it initially amounts to $3 \times 10^3$ and subsequently it increases to over $4 \times 10^3$. In the Niederrheinisches Flachland (Lower Rhine Plain) the value diminishes again and in the final stretch it amounts to only $6 \times 10^4$.

The upstream part lies in a region with an impermeable soil of a Palaeozoic origin. The downstream area lies in the Roerslenkdal (Roer Graben Valley) and is filled with Pleistocene sediment. The graben runs from Bonn to Weert and there it joins the Centrale Slenk (Central Graben). In the southwest the graben is bounded by the Feldbissbreuk (Feldbiss Fault) and in the northeast by the Roerrandbreuk (Roer Edge Fault) and Peelrandbreuk (Peel...
Edge Fault). The graben is approximately 25 km wide. In and near the eastern side of the graben brown coal is mined in open quarries. They have a depth of up to 300 m, which will be up to 600 m in the future. In order to keep the quarries dry they are drained, around 1990 the extracted quantity of water amounted to $1.2 \times 10^9$ m$^3$ per year (nearly 40 m$^3$/s). Because the discharge of the drainage mainly takes place via the Erft, a Rhine tributary, the discharge is smaller than in the natural situation. The extractions will be so great that in the future a sinking of the ground water level may be expected even west of the Meuse.

---

boundary of the catchment
■ measuring station (water level and/or discharge)
□ indication of place
--- border

---

Figure 6.11 The catchment of the Roer

98
TABLE 6.17 THE ROER AT STAHL

<table>
<thead>
<tr>
<th>River: Roer</th>
<th>Measuring station: Stah</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area above measuring station: 2105 km²</td>
<td>Reference period precipitation and discharge: Nov.1961-Oct.1985</td>
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</table>

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
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</thead>
<tbody>
<tr>
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<td>57</td>
<td>0.54</td>
</tr>
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<td>23.7</td>
<td>67</td>
<td>0.45</td>
</tr>
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<td>25.0</td>
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</tr>
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<td>May</td>
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<tr>
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</tr>
<tr>
<td>Year Total</td>
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<td>0.39</td>
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</table>

maximum discharge at flood February 1980: 76 m³/s
maximum discharge at flood July 1980: 113 m³/s
maximum discharge at flood February 1984: 118 m³/s
minimum discharge at low flow 1976: 10.8 m³/s

Throughout the year the amounts of precipitation in the Ardennes and in the Eifel are on an average greater than in the Niederrheinischs Flachland. The difference is greatest in winter.

Some information on the precipitation and the discharges is given in tables 6.17 and 6.18.

The Roer discharge is considerably influenced by two reservoirs.

The greatest and most important reservoir is the Rurtalsperre (Roer Dam), which is situated on the spot where the Urft discharges into the Roer. The reservoir finished in 1959 has a capacity of 205 10⁶ m³. The purpose of the reservoir is increasing the small discharges and reducing flood waves. Afterwards the improvement of the quality of the water, the generation of energy and the supply of drinking water were added as objectives. The drinking water for the city of Aachen is in part supplied by the Rurtalsperre.

Immediately before the Rurtalsperre the Urfttalsperre is to be found with a capacity of 45.5 10⁶ m³. It serves to decrease flood discharges and to generate electricity.

Together those two reservoirs have a capacity of 250 10⁶ m³, which is about the same amount as of all the reservoirs in the Belgian Meuse area taken together.

In the catchment there are some more reservoirs, among which the Oeletalsperre.

As electricity is only generated there when it is needed the transit discharge is subject to fluctuations. In order to neutralise that effect a number of small reservoirs have been built.
below the Rurtalsperre. With a view to the landscape a few preliminary dams have been constructed near the reservoirs, which prevent the ugly sight of bare banks in the upstream part of the complex.

The amount of the outflow is a function of the degree of filling and of the day of the year. The greatest discharge takes place in December and January with a filled reservoir (60 m³/s), the smallest with a low degree of filling (5 m³/s). In case of calamities the rule is deviated from. It occurred in July 1980 and in March 1988 when 115 - 120 m³/s had to be passed.

There are no reservoirs in the Inde. Therefore flood waves in the Inde will sooner reach the Roer mouth than the flood waves of the Roer itself.

The influence of the two great reservoirs is investigated by comparing the discharges above the reservoirs with the discharges below them. For that purpose have been chosen:
- upstream:  
  a. Monschau (Roer), contributing area 142 km²  
  b. Gemünd (Urft), contributing area 343 km²  
- downstream: Zerkall (Roer), contributing area 787 km²  

Through that subdivision 61% of the catchment above Zerkall is also situated above Monschau or Gemünd.

The influence of the reservoirs can clearly be observed in fig. 6.12, representing the specific mean monthly discharges for the areas mentioned. From the figure it appears that the mean monthly discharges at Zerkall are more constant than those at Monschau and Gemünd.
### TABLE 6.18 FLOOD WAVES IN THE ROER AND IN THE MEUSE

<table>
<thead>
<tr>
<th>Feb. 1980</th>
<th>Stah</th>
<th>Borgharen</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
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<table>
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<td>s.d.</td>
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<tr>
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<td>4.6</td>
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<td>24</td>
<td>107</td>
<td>4.4</td>
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<table>
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<tbody>
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<td>s.d.</td>
</tr>
<tr>
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<td>50</td>
<td>2.1</td>
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<td>7</td>
<td>109</td>
<td>4.5</td>
</tr>
<tr>
<td>8</td>
<td>118</td>
<td>4.8</td>
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<td>9</td>
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<td>4.5</td>
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<td>10</td>
<td>87</td>
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</tr>
<tr>
<td>11</td>
<td>74</td>
<td>3.0</td>
</tr>
<tr>
<td>12</td>
<td>69</td>
<td>2.8</td>
</tr>
</tbody>
</table>

**Floods**

The discharges during floods are analysed on the basis of the 1984 flood. The flood is representative of winter floods. With summer floods the reservoirs pass more water.

The Roer had its peak discharge on 7 February 1984. The mean daily discharge at Monschau amounted to 71 m³/s, at Gemünd 91 m³/s and at Zerkall only 54 m³/s. In fig. 6.13 the discharges of some stations in February 1984 are given. From the figure it is clear that the water stored is discharged in the next few weeks: the discharge at Zerkall is much greater than the discharges of Monschau and Gemünd taken together.

From fig. 6.13 it appears that the flood discharges at Stah are significantly greater than those at Zerkall, and that the peak is more flatly shaped. A few days after the peak the discharge at Stah is mainly determined by the Rurtalsperre.

At the moment of the start of the flood forecast modelling of Roer and Niers, the discharges of those rivers are not available automatically in real-time. In the event of floods the Roer discharge used to be estimated with a constant or as a linear function of the Borgharen discharge. To what extent the approach is acceptable will be analysed in the next few paragraphs.

In fig. 16.4 the discharges at Borgharen and Stah have been given for six flood waves. For each wave 6 discharges have been given: the maximum of the mean daily discharge, the mean daily discharges on the three days before the peak and on the two days after it. The peak discharges at Borgharen have been indicated with an open circle. The optimal linear
relation has been established by means of the least squares method. It reads:

\[ Q_{Stah} = 0.0426 Q_{BAD} + 7.11 \]  

(6.8)

where:

\[ Q_{Stah} = \text{discharge at Stah} \quad [\text{m}^3/\text{s}] \]

\[ Q_{BAD} = \text{discharge at Borgharen-Dorp} \quad [\text{m}^3/\text{s}] \]

If for reasons of simplicity the above relation for establishing the Roer discharge was chosen it will result in errors. The standard deviation of the error calculated for the discharge of the Roer is 19.2 m³/s. Starting from the normal distribution the confidential limits of 95% will be between 37.7 m³/s above and below the regression line calculated (fig. 6.14). In the series given the greatest deviation was 61 m³/s.

If the discharge deviates from the discharge calculated by 37.7 m³/s, with a discharge of 1,500 m³/s at Borgharen, the error in the water level forecast at Roermond will amount to 9 cm as a result. Those errors are not to be neglected, since the forecasts are made with an accuracy of from 5 to 10 cm. If a shift of time is applied with respect to the moments of the discharges of Stah and Borgharen the standard deviation calculated increases.

With a second degree polynomial as a regression curve no significant improvement of the results occurs.

In the analyses the mean daily discharges were the starting point. In an incidental case the momentaneous peak discharges of the Roer may be up to 30% greater than the maximum mean daily discharges. On account of it the deviations are further increased.

![Figure 6.13 Flood discharges in the Roer](image)
Figure 6.14 The relation between the discharges at Borgharen and Stah

Figure 6.15 Low flows in the Roer
The conclusion may be that it is not sufficient to estimate the Roer discharge by means of the Borgharen discharges. A simple model will suffice, though, e.g. only on the basis of precipitation or discharges measured.

**Low flows**
The varied nature of the Roer is easy to observe in the discharge course of the dry year 1976 (fig. 6.15). The baseflow reservoirs of the Urft and of the Roer at Monschau are soon exhausted. During the greater part of the year the Urftalsperre and the Rurtalsperre have a discharge that is greater than the influx. The reservoirs have a capacity sufficient to maintain the discharge for an even longer period. In the figure the weekly cycle is not to be observed in which on Sundays the discharge at Zerkall is significantly smaller than on other days. The periodical decrease of the discharge is hardly to be noticed at Stah any longer.

---

**Figure 6.16 The catchment of the Niers**

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The downstream part of the Roer flows through an even region. The discharge is increased by effluent discharges. The contribution of the ground water is likely to be limited by withdrawals for the benefit of the mining of brown coal.

The contributions of the upstream catchment to the low flow are consequently small. According to the German authorities the contributions of the reservoirs and of the effluent discharges together minimally amount to 10 m³/s. That value was at least equalled in the period for which data were available.

6.9 THE NIERS

With a catchment of 1,358 km² the Niers constitutes one of the larger basins of the Meuse. The Niers springs south of Mönchengladbach at Holzweiler and flows out into the Meuse at Ottersum at NAP + 7 m. The catchment is rather flat and for the greater part it lies below 100 m. The highest point of the catchment is at NAP + 106 m. With the exception of the upper course the mean gradient is 3 \times 10^{-4}. The length of the river is approximately 119 km. The mean annual precipitation is 719 mm (table 6.19). Part of the discharge reaches the Meuse by way of the Nierskanaal. The Nierskanaal is situated west of Geldern and is

<table>
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<th>TABLE 6.19 THE NIERS AT GOCH</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>mean discharge [m³/s]</td>
</tr>
<tr>
<td>January</td>
<td>9.5</td>
<td>64</td>
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<td>February</td>
<td>10.4</td>
<td>41</td>
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<td>68</td>
<td>0.19</td>
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<td>60</td>
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<td>Year Total</td>
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<tr>
<td>maximum discharge at flood February 1980:</td>
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<td>maximum discharge at flood July 1980:</td>
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<td>maximum discharge at flood February 1984:</td>
<td>29 m³/s</td>
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<tr>
<td>minimum discharge at low flow 1976:</td>
<td>1.09 m³/s</td>
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connected with the Meuse near Ooijen (fig. 6.16).

At the measuring station of Goch the mean discharge is 7.04 m³/s. On an average 0.3 m³/s flows into the Meuse via the Nierskanaal.

In a flood discharges of 20 to 30 m³/s can be reached (table 6.20). Considering the size of the catchment it is a low value. The explanation may be found in the even area, the relatively small precipitation and the great infiltration capacity. The contribution to the Meuse discharge is of the order of 2%. If in a flood forecasting model the Niers discharge is schematised into one fixed value, deviations of up to 10 m³/s may occur as compared with the real Niers discharge. With a Meuse discharge of 1,500 m³/s at Borgharen it corresponds with a deviation of 0.02 m. Considering the maximum deviation desirable of the water levels from 0.05 to 0.10 m the error is close to the limit of what may be ignored.

In dry periods the discharge is being reduced to about 1 m³/s. The increase of the Meuse discharge takes place below the stretch with the greatest low flow problems.

<table>
<thead>
<tr>
<th>Feb.</th>
<th>Goch</th>
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<th>Goch</th>
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<tr>
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<td>573</td>
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<td>disch.</td>
<td>s.d.</td>
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<td>1.7</td>
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<td>12</td>
<td>21</td>
<td>1.5</td>
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</table>

disch. = discharge [m³/s]

s.d. = specific discharge [mm/day]

6.10 SMALLER TRIBUTARIES

Apart from the large tributaries the Meuse also has a great number of small tributaries. Separately the discharges of the small tributaries are of hardly any importance, but taken together they constitute a factor not to be neglected in the discharge of the Meuse. In this section the particular aspects of a number of tributaries will be mentioned.

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In the upper course the Mouzon and the Vair flow out into the Meuse. Hydrologically the rivers have the same structure as the Meuse locally has. Therefore the upper course of the Meuse is often considered a hydrologic unity including the Mouzon and the Vair.

The Bar, Vence and Sormonne are three rivulets on the left bank flowing out into the Meuse between Sedan and Charleville-Mézières. In regard of the geological structure they resemble the Chiers very closely.

The Houille, Hermeton, Mollignée, Bocq, Samson and Hoyoux are a series of Belgian tributaries based on Palaeozoic rock, the valleys of which are deeply incised. The greater part of the catchment of the Houille lies in Belgium, but the mouth lies in France, as is the case with the Viroin.

The rivulets have the same kind of discharge regime as the large Ardennes rivers. In those small catchments there is a far greater chance of a great areal precipitation than in the larger areas. On that account the maximum specific discharge may reach far greater values than those of the larger areas. However, it does not apply to all the small catchments taken together, because together they again constitute a large area and, moreover, they are scattered in the Meuse basin.

The Mehaigne and Jeker (French: Geer), lying on the left bank, have a structure quite different from the tributaries on the opposite bank. Their catchments are far more even and contain an aquifer in the Haspengaw Chalk. However, the stratum is covered by a number of strata of a varying permeability with among other things loam, silex clay and loess. Particularly in the Jeker catchment the chalk stratum is used for the collection of water.

The Berwinne and Geul (French: Gueulle) are two little tributaries on the right bank near the Belgian Dutch border. The upper courses are to be found in the Ardennes.

The Swalm is a small tributary strongly resembling the Niers structurally.

The Dieze is a river with a large contributing area, which amounts to approximately 2,800 km². The river arises from the confluence of the Aa and the Dommel and only has a length of a few kilometres itself. The catchment lies in wind-borne sand deposits and is very even. As the mouth of the Dieze lies in a tidal region its catchment is beyond the scope of this thesis.

Finally the Donge flows out on the borderline between Bergse Maas and Amer.

6.11 SIMILARITIES AND DIFFERENCES AMONG THE TRIBUTARIES

In this section the various tributaries described in the preceding sections are compared with each other regarding their behaviour by means of the discharges that have occurred.

Table 6.21 provides a survey of areas and discharges of the large and the small tributaries.

In formulating that table it appeared to be hardly possible to maintain the same periods of reference for all the stations, because then the reference period would only cover three years. In general a period as long as possible has been chosen. However, it should be realised that deviations from the 'real' mean values may also arise from different measuring methods
and ways of calculating. For instance it regularly occurs that in the various successive annuals the initial points of time of the reference periods change irregularly.

For the residual regions the discharges have been determined from the condition of continuity. Here the differences in measuring methods, ways of calculating and reference periods may have great consequences, because various discharges are subtracted from one another. Therefore the values given should only be used as an indication. Interpreting a calculated mean specific discharge of the residual regions would result in incorrect conclusions, because its size is strongly influenced by the reference periods and the methods of measuring and calculating at the stations of the areas measured. Those values had better be

<table>
<thead>
<tr>
<th>TABLE 6.21 SURVEY OF THE TRIBUTARIES</th>
</tr>
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<td>-------------------------------------</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>A. Upstream from Stenay</td>
</tr>
<tr>
<td>Meuse at Goncourt</td>
</tr>
<tr>
<td>Mouzon ( Villars)</td>
</tr>
<tr>
<td>Vair (Soulles)</td>
</tr>
<tr>
<td>Residual area upstream from St. Mihiel</td>
</tr>
<tr>
<td>Meuse at St. Mihiel</td>
</tr>
<tr>
<td>Between St. Mihiel and Stenay</td>
</tr>
<tr>
<td>Meuse at Stenay</td>
</tr>
<tr>
<td>B. Between Stenay and Chooz</td>
</tr>
<tr>
<td>Chiers (Carignan)</td>
</tr>
<tr>
<td>Bar (Cheveuges)</td>
</tr>
<tr>
<td>Vence (Lafrangeville)</td>
</tr>
<tr>
<td>Sormonne (Belval)</td>
</tr>
<tr>
<td>Semois (Membre)</td>
</tr>
<tr>
<td>Viroin (Treignes)</td>
</tr>
<tr>
<td>Residual area between Stenay and Chooz</td>
</tr>
<tr>
<td>Meuse at Chooz</td>
</tr>
<tr>
<td>C. Between Chooz and Amspin-Neuville</td>
</tr>
<tr>
<td>Houille (Landrichamps)</td>
</tr>
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<td>Hermeton (Hastière)</td>
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<td>Lesse (Gendron)</td>
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<td>Malignée (Warnant)</td>
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<td>Sambre (Namur)</td>
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<td>Hoyoux (Marchin)</td>
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<tr>
<td>Mehaigne (Moha)</td>
</tr>
<tr>
<td>Taffier*</td>
</tr>
<tr>
<td>Residual area between Chooz and A.-N.</td>
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<tr>
<td>Meuse at Amspin-Neuville</td>
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<tr>
<td>Area (measuring station)</td>
</tr>
<tr>
<td>-------------------------</td>
</tr>
<tr>
<td><strong>D. Between Ampsin-Neuvville and Borgharen</strong></td>
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<td>Ourthe (at Angleur 3626 km²)</td>
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<tr>
<td>- Vesdre (Chaudfontaine)</td>
</tr>
<tr>
<td>- Ambîève (Martinrive)</td>
</tr>
<tr>
<td>- Ourthe (Tabreux)</td>
</tr>
<tr>
<td>Jeker (Kanne)</td>
</tr>
<tr>
<td>Berwinne (Dalhem)</td>
</tr>
<tr>
<td>Albertkanaal (at Briegden)*</td>
</tr>
<tr>
<td>Zuid-Willemsvaart*</td>
</tr>
<tr>
<td>Julianakanaal*</td>
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<tr>
<td>Residual area between A.-N. and Borgh.</td>
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<tr>
<td><strong>Meuse at Borgharen</strong></td>
</tr>
<tr>
<td><strong>E. Between Borgharen en Lith</strong></td>
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<td>Niers (Goch)</td>
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<td>Swalm (Pannenmühle)</td>
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<tr>
<td>Julianakanaal and DSM factory*</td>
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<tr>
<td>Kanaal Wessem-Nederveent*</td>
</tr>
<tr>
<td>Maas-Waal-kanaal*</td>
</tr>
<tr>
<td>Residual area between Borgh. and Lith</td>
</tr>
<tr>
<td><strong>Meuse at Lith</strong></td>
</tr>
</tbody>
</table>

* artificial withdrawal or disposal
b. Ungauged areas. Figures calculated from continuity.
c. Source: Ministère des Travaux Publics, Belgium.
d. Discharges estimated (cf. § 6.6).
f. For the calculation of the specific discharges of Ampsin-Neuville, Borgharen and Lith, the total outflow of the catchment considered has been taken, so inclusive of extractions by canals etcetera.
g. Source: Rijkswaterstaat and Provinciale Waterstaat in Limburg.
h. The reference period of the discharges is 1966-1981. It has been chosen on the one hand to be in keeping with the Belgian periods of reference, on the other because withdrawals such as via canals did not change strongly in the period. (For Borgharen the mean discharge in the period of 1911-1985 amounts to 230 m³/s, for Lith 320 m³/s. Those values have been corrected for the situation of about 1980, i.e. as if the withdrawals had already occurred since 1911.)
i. Source: Landesamt für Wasser und Abfall - Nordrhein-Westfalen, Germany.
Figure 6.17 The hydrographs of the southern large tributaries in February 1980

Figure 6.18 The hydrographs of the northern large tributaries in February 1980
Figure 6.19 The hydrographs of the southern large tributaries in July 1980

Figure 6.20 The hydrographs of the northern large tributaries in July 1980
Figure 6.21 The hydrographs of the southern large tributaries in February 1984

Figure 6.22 The hydrographs of the northern large tributaries in February 1984
### TABLE 6.22 SURVEY OF PEAK DISCHARGES

<table>
<thead>
<tr>
<th>Area (measuring station)</th>
<th>Flood</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>February 1980</td>
</tr>
<tr>
<td></td>
<td>a [m³/s]</td>
</tr>
<tr>
<td>A. Meuse at St. Mihiel</td>
<td>408</td>
</tr>
<tr>
<td>Meuse at Stenay</td>
<td>515</td>
</tr>
<tr>
<td>B. Between Stenay and Chooz</td>
<td></td>
</tr>
<tr>
<td>Chiers (Carignan)</td>
<td>88</td>
</tr>
<tr>
<td>Semois (Membre)</td>
<td>203</td>
</tr>
<tr>
<td>Viroin (Treignes)</td>
<td>69</td>
</tr>
<tr>
<td>Meuse at Chooz</td>
<td>735</td>
</tr>
<tr>
<td>C. Between Chooz and Ampsin</td>
<td></td>
</tr>
<tr>
<td>Houille (Landrichamps)</td>
<td>21</td>
</tr>
<tr>
<td>Hermeton (Hastière)</td>
<td>18</td>
</tr>
<tr>
<td>Lesse (Gendron)</td>
<td>119</td>
</tr>
<tr>
<td>Molignée (Warrant)</td>
<td>7</td>
</tr>
<tr>
<td>Bocq (Evrehailles)</td>
<td>8</td>
</tr>
<tr>
<td>Sambre (Namur)</td>
<td>d</td>
</tr>
<tr>
<td>Hoyoux (Marchin)</td>
<td>6</td>
</tr>
<tr>
<td>Mehaigne (Moho)</td>
<td>12</td>
</tr>
<tr>
<td>Meuse at Ampsin-Neuville</td>
<td>1357</td>
</tr>
<tr>
<td>D. Between Ampsin and Borgh.</td>
<td></td>
</tr>
<tr>
<td>Outsche (Angleur)</td>
<td>347</td>
</tr>
<tr>
<td>- Vesdre (Chaudfontaine)</td>
<td>e</td>
</tr>
<tr>
<td>- Ambîève (Martinrive)</td>
<td>109</td>
</tr>
<tr>
<td>- Outsche (Tabreux)</td>
<td>153</td>
</tr>
<tr>
<td>Jeker (Kanne)</td>
<td>5</td>
</tr>
<tr>
<td>Berwinne (Dalhem)</td>
<td>9</td>
</tr>
<tr>
<td>Meuse at Borgharen</td>
<td>1440</td>
</tr>
<tr>
<td>E. Between Borgharen and Lith</td>
<td></td>
</tr>
<tr>
<td>Geul (Meerssen Papier)</td>
<td>12</td>
</tr>
<tr>
<td>Roer (Stah)</td>
<td>76</td>
</tr>
<tr>
<td>Niers (Goch)</td>
<td>30</td>
</tr>
<tr>
<td>Meuse at Lith</td>
<td>1426</td>
</tr>
</tbody>
</table>

- **a.** greatest mean daily discharge
- **b.** momentaneous peak discharge
- **c.** February 1984: Haulmé instead of Membre
- **d.** value estimated (%6.6). The values measured are: Feb. 1980: 414 m³/s, July 1980: 300 m³/s, Feb. 1984: 340 m³/s.
- **e.** for the flood of July 1980 the discharges of 20/21 July are concerned, as in the case of the other measuring stations

Sources of the data: cf. table 6.21.

Cf. also note 1 (page 114)
determined with the aid of a systems description and the mean specific discharges of nearby regions.

In table 6.21 it is shown that the Ardennes tributaries have the greatest mean specific discharge. It may be explained by the great annual precipitation and the great runoff coefficient.

The specific discharge of the upper course of the Meuse is rather great as compared with those of some tributaries in the lower course of the Meuse. The explanation of the fact is to be found by means of fig. 1.4, from which it appears that the Meuse Lorraine has greater annual amounts of precipitation than large parts of the northern Meuse basin.

An analysis of the discharge frequencies has been omitted, because they are strongly influenced by the periods of reference. In most cases it only amounts to one or two decades.

The relation between mean discharge and momentaneous peak discharge is not univocal. For a great number of tributaries both values have been given for some floods in table 6.22¹.

The water of the flood wave of February 1980 is mainly from the south of the catchment. The flood of July 1980 mainly comes from the Ardennes. With the 1984 flood the Ardennes also play a great part.

From the difference of the values in the columns a and b of table 6.22 the relevance of hourly observations to the investigation of floods can be deduced. With the Chiers mean daily discharges will suffice; the difference between a momentaneous peak discharge and a mean daily discharge appears to be no more than a few cubic metres per second. The need of hourly discharge data of tributaries is greatest with the Amblève: On 7 February 1984 the difference between the momentaneous peak discharge and the mean daily discharge was 65 m³/s.

When comparing the discharge courses of the various tributaries (figs. 6.17 to 6.22) the following things are striking:
- The Chiers slowly responds in comparison with other tributaries. That seems to be contradictory to § 6.2, in which it is stated that the Chiers responds fast in comparison with the Meuse Lorraine.
  It may be stated, however, that the Ardennes tributaries respond much faster than the Chiers and that the Chiers in turn responds faster than the Meuse Lorraine.
- Especially in the Ardennes tributaries a fast rise of great discharges is possible.
- The Roer and the Niers respond far more slowly than the Ardennes rivers.

The discharge courses of the various floods show once more that the Meuse has tributaries with altogether different characters.

¹For the flood of February 1980 a number of discharges with striking values have been mentioned in table 6.22. At Borglaren the Meuse had a mean daily discharge of 1,440 m³/s (the peak discharge passed here at 11.00 h. C.E.T.). That day the Ourthe discharge at Angleur was about 350 m³/s. Both values are not in keeping with the discharge of Ampsin-Neuville on that day, which was 1,357 m³/s. Starting from Angleur and Borglaren it should have been about 300 m³/s less. It may be concluded that at least at one of the stations the discharge mentioned greatly deviates from the mean daily discharges that occurred.

For that flood a great difference between the Sambre discharge mentioned and calculated at Namur can also be observed in table 6.22 (414 as opposed to 160 m³/s). From an investigation as to the amounts of precipitation and discharges of the tributaries in the catchment considered a discharge of 414 m³/s cannot be justified. However, the discharge of 414 m³/s fits in well with the discharges of Chooz and Ampsin-Neuville, but the discharge of 160 m³/s fits in well with the discharge of Chooz and Borglaren-Dorp.
Chapter 7. The river Meuse

7.1 INTRODUCTION

In this chapter the river Meuse will be discussed in more detail. A general description is given in § 7.2. Then § 7.3 will go more deeply into the weirs and their control. The same remarks on the data given are applicable as those mentioned in the introductory section to chapter 6.

7.2 GENERAL

The Meuse springs at NAP + 400 m at Pouilly-en-Bassigny on the Plateau of Langres. The Plateau forms the catchment boundary of basins draining into the Mediterranean (Saône), into the Atlantic (Seine) and into the North Sea. The Plateau is even and lies relatively low. There is no storage of water by way of for example glaciers. That is the reason why the source of the Meuse is not to be fixed clearly to one spot. As a result in two different places there are monuments to indicate the source, lying two kilometres from one another. (The problem that the source of a river is not exactly to be determined is a phenomenon happening frequently. Accordingly in France a kilometre numbering system has been chosen in which the mouth of a river is assigned kilometre marker 1000. From there the numbering goes back. For the Meuse the kilometre marker 1000 is at the bridge of the motorway Breda-Rotterdam across the Hollands Diep.) The distinction hardly matters, so that the term "the source" will be used.

Near the source the Meuse has the character of a calm brook. In the upper course there are some fixed weirs, some of which are adjustable.

Off Troussesy the Canal du Marne au Rhin crosses the Meuse basin and the northern branch of the Canal de l'Est forks off from the former canal. During the greater part of the year the Canal du Marne au Rhin adds approximately 0.4 m³/s to the Meuse discharges; in very dry periods, however, up to 0.2 m³/s may be discharged from the Meuse basin (Zumstein et al., 1978).

Between Troussesy and Sedan the Meuse has the characteristic course of the Meuse Lorraine: a wide and even valley and in it the river and a canal situated parallel (fig. 5.3). Regularly the canal joins the river so that the canal can be fed. Downstream from places where the canal and the river split there are weirs in the Meuse. Below those weirs there is free flow again, in general up to the next common stretch. There are no measurements and calculations of the leakage from the canal into the Meuse.

The discharge capacity of the minor bed is so small that with great discharges the whole valley, which can be several kilometres wide, is inundated. The water stored in that way hardly flows. The flood wave celerity is limited to only 20 km a day. According to the French authorities a discharge of 550 m³/s will hardly ever be exceeded. In the reference period the maximum discharge at Stenay was 530 m³/s (26 February 1970). Some data on the Meuse at Stenay are found in table 7.1.

In the greater part of the Meuse Lorraine the basis is formed by lime. In the even river valley, however, there are alluvial deposits. Both sediments have the capacity to let the water through well as compared with e.g. the Ardennes. The result is that in the course of a period
TABLE 7.1 THE MEUSE AT STENAY

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>75.5</td>
<td>68.1</td>
<td>0.76</td>
</tr>
<tr>
<td>February</td>
<td>103</td>
<td>80.8</td>
<td>0.80</td>
</tr>
<tr>
<td>March</td>
<td>74.0</td>
<td>61.1</td>
<td>0.83</td>
</tr>
<tr>
<td>April</td>
<td>60.5</td>
<td>55.2</td>
<td>0.73</td>
</tr>
<tr>
<td>May</td>
<td>41.6</td>
<td>67.6</td>
<td>0.42</td>
</tr>
<tr>
<td>June</td>
<td>26.6</td>
<td>72.8</td>
<td>0.24</td>
</tr>
<tr>
<td>July</td>
<td>21.6</td>
<td>62.6</td>
<td>0.24</td>
</tr>
<tr>
<td>August</td>
<td>15.7</td>
<td>58.6</td>
<td>0.18</td>
</tr>
<tr>
<td>September</td>
<td>15.3</td>
<td>55.9</td>
<td>0.18</td>
</tr>
<tr>
<td>October</td>
<td>19.3</td>
<td>52.8</td>
<td>0.25</td>
</tr>
<tr>
<td>November</td>
<td>34.9</td>
<td>96.1</td>
<td>0.24</td>
</tr>
<tr>
<td>December</td>
<td>72.0</td>
<td>72.7</td>
<td>0.68</td>
</tr>
<tr>
<td>Year</td>
<td>46.3</td>
<td></td>
<td>0.54</td>
</tr>
<tr>
<td>Total</td>
<td>804.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

maximum discharge at flood February 1980: 515 m³/s
maximum discharge at flood July 1980: 131 m³/s
maximum discharge at flood February 1984: 301 m³/s
minimum discharge at low flow 1976: 3.45 m³/s

of dry weather the supplies of groundwater keep the discharge of the Meuse Lorraine relatively great. It is of great importance for the Dutch low flow.

At Sedan the Chiers joins the Meuse. Below the confluence the Meuse gets a different character. The gradient increases strongly. To a considerable extent the river lies on very impermeable soils. That character is maintained up to Namur. Via the Bar valley the Canal des Ardennes has a connection with the Canal de l’Est. Through the Canal de l’Est an average of approximately 0.3 m³/s disappears to the Aisne basin by means of reservoirs, extractions from rivulets and pumping. The nuclear power station of Chooz uses a discharge of 4.75 m³/s, between half and two thirds of which is returned to the Meuse.

Near Chooz is the most downstream spot where the Canal de l’Est is a parallel canal, and the river level is not influenced by weirs. At that point the last possibility before the Dutch border is found to determine the discharges while only making use of water levels.

At Chooz the Meuse has covered about half of its catchment above Borgharen (fig. 1.1). For many floods, however, the contributions of many flood waves are much smaller than half the peak discharge (table 6.22). The mean specific discharges of Chooz and Borgharen do correspond rather well (tables 7.2 and 7.3).
TABLE 7.2 THE MEUSE AT CHOOZ

River: Meuse
Measuring station: Chooz
Area above measuring station: 10120 km²
Distance to the source: 482 km
Wave travel time Chooz-Borgharen: 16 h
Reference period precipitation: 1951-1970
Reference period discharge: 1953-1980

<table>
<thead>
<tr>
<th></th>
<th>mean discharge [m³/s]</th>
<th>areal precipitation [mm]</th>
<th>runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>234</td>
<td>88</td>
<td>0.70</td>
</tr>
<tr>
<td>February</td>
<td>278</td>
<td>79</td>
<td>0.85</td>
</tr>
<tr>
<td>March</td>
<td>194</td>
<td>67</td>
<td>0.76</td>
</tr>
<tr>
<td>April</td>
<td>155</td>
<td>61</td>
<td>0.65</td>
</tr>
<tr>
<td>May</td>
<td>112</td>
<td>70</td>
<td>0.42</td>
</tr>
<tr>
<td>June</td>
<td>74</td>
<td>78</td>
<td>0.24</td>
</tr>
<tr>
<td>July</td>
<td>63</td>
<td>73</td>
<td>0.23</td>
</tr>
<tr>
<td>August</td>
<td>54</td>
<td>92</td>
<td>0.15</td>
</tr>
<tr>
<td>September</td>
<td>55</td>
<td>79</td>
<td>0.18</td>
</tr>
<tr>
<td>October</td>
<td>71</td>
<td>70</td>
<td>0.27</td>
</tr>
<tr>
<td>November</td>
<td>124</td>
<td>89</td>
<td>0.36</td>
</tr>
<tr>
<td>December</td>
<td>217</td>
<td>100</td>
<td>0.57</td>
</tr>
<tr>
<td>Year</td>
<td>135</td>
<td></td>
<td>0.45</td>
</tr>
<tr>
<td>Total</td>
<td>946</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

maximum discharge at flood February 1980: 735 m³/s
maximum discharge at flood July 1980: 595 m³/s
maximum discharge at flood February 1984: 1199 m³/s
minimum discharge at low flow 1976: 10.2 m³/s

In the Belgian part of the Meuse shipping takes place practically along the whole stretch. As a result the water level is influenced by weirs everywhere in the river. Detailed data of the discharges based on water levels are lacking; only discharges while making use of the positions of the weir elements are available.

Throughout the Belgian Meuse an extension of profiles has been performed. That was done for the benefit of shipping, safety during flood waves and the demand for gravel. Above Namur the Meuse is approximately 100 m wide, between Namur and Borgharen approximately 130 m. Upstream from Namur the Meuse in Belgium lies in a narrow valley, below Namur the valley is generally wider. Between Liège and Borgharen there are extractions and disposals in many places (chapter 3). Some data on the measuring station of the discharges at Borgharen-Dorp are supplied in table 7.3.

In the Grensmaas, the Meuse between Borgharen and Maasbracht, there are no weirs. Here the gradient is approximately 4 10⁻⁴, as a result of which the velocity of the current may be high. As the Meuse lies on gravel here there is a strong exchange with the groundwater (cf. § 11.4.3).
**TABLE 7.3 THE MEUSE AT BORGHAREN**

<table>
<thead>
<tr>
<th>Month</th>
<th>Mean Discharge [m³/s]</th>
<th>Areal Precipitation [mm]</th>
<th>Runoff Coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>459</td>
<td>84</td>
<td>0.75</td>
</tr>
<tr>
<td>February</td>
<td>432</td>
<td>77</td>
<td>0.70</td>
</tr>
<tr>
<td>March</td>
<td>339</td>
<td>65</td>
<td>0.73</td>
</tr>
<tr>
<td>April</td>
<td>274</td>
<td>62</td>
<td>0.62</td>
</tr>
<tr>
<td>May</td>
<td>173</td>
<td>72</td>
<td>0.37</td>
</tr>
<tr>
<td>June</td>
<td>107</td>
<td>79</td>
<td>0.23</td>
</tr>
<tr>
<td>July</td>
<td>87</td>
<td>80</td>
<td>0.20</td>
</tr>
<tr>
<td>August</td>
<td>77</td>
<td>95</td>
<td>0.15</td>
</tr>
<tr>
<td>September</td>
<td>79</td>
<td>77</td>
<td>0.19</td>
</tr>
<tr>
<td>October</td>
<td>115</td>
<td>69</td>
<td>0.28</td>
</tr>
<tr>
<td>November</td>
<td>254</td>
<td>86</td>
<td>0.42</td>
</tr>
<tr>
<td>December</td>
<td>380</td>
<td>95</td>
<td>0.56</td>
</tr>
<tr>
<td>Year</td>
<td>231</td>
<td></td>
<td>0.43</td>
</tr>
</tbody>
</table>

**Total**: 940

**Note**: When calculating the runoff coefficient the starting point was the natural discharge. That means that the discharges mentioned have been increased by 39 m³/s, on account of these extractions: Albertkanaal (approximately 9 m³/s), Zuid-Willemsvaart (approximately 14 m³/s) and Julianakanaal (approximately 16 m³/s).

Near Maasbracht an intricate junction of waterways is found (fig. 7.1). During periods of drought it is one of the critical points as far as maintaining the headwater level is concerned (cf. chapter 3). Downstream from Maasbracht the river slope decreases to 1 x 10⁻⁴. The width of the river increases from 100 to approximately 150 m. Here the flood wave propagation is rather slow. Consequently the flood-peak attenuation is great, which appears if the peak discharges of Borgharen and Lith are compared. In spite of the fact that near Lith the catchment area is nearly 40% greater than that at Borgharen the peak discharges are smaller (table 6.22). Some information on precipitation and discharges is given in table 7.4.
Figure 7.1 The system of waterways between Maasbracht and Roermond

Figure 7.2 The hydrographs of some stations along the Meuse during the flood of February 1980
### Table 7.4 The Meuse at Lith

**River:** Meuse  
**Measuring station:** Lith (weir)  
**Area above measuring station:** 29370 km²  
**Distance to the source:** 812 km  
**Wave travel time Borgharen-Lith:** 3 days  
**Reference period precipitation:**  
  - Belgium and France: 1951-1970  
  - Netherlands: 1951-1980  
  - Germany: unknown  
**Reference period discharge:** 1911-1985

<table>
<thead>
<tr>
<th>Month</th>
<th>Discharge [m³/s]</th>
<th>Precipitation [mm]</th>
<th>Runoff coefficient [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>590</td>
<td>78</td>
<td>0.72</td>
</tr>
<tr>
<td>February</td>
<td>553</td>
<td>70</td>
<td>0.68</td>
</tr>
<tr>
<td>March</td>
<td>458</td>
<td>63</td>
<td>0.70</td>
</tr>
<tr>
<td>April</td>
<td>379</td>
<td>59</td>
<td>0.60</td>
</tr>
<tr>
<td>May</td>
<td>254</td>
<td>70</td>
<td>0.36</td>
</tr>
<tr>
<td>June</td>
<td>170</td>
<td>77</td>
<td>0.22</td>
</tr>
<tr>
<td>July</td>
<td>143</td>
<td>80</td>
<td>0.19</td>
</tr>
<tr>
<td>August</td>
<td>133</td>
<td>90</td>
<td>0.16</td>
</tr>
<tr>
<td>September</td>
<td>138</td>
<td>72</td>
<td>0.20</td>
</tr>
<tr>
<td>October</td>
<td>177</td>
<td>66</td>
<td>0.28</td>
</tr>
<tr>
<td>November</td>
<td>332</td>
<td>82</td>
<td>0.38</td>
</tr>
<tr>
<td>December</td>
<td>476</td>
<td>89</td>
<td>0.51</td>
</tr>
</tbody>
</table>

**Year Total**  
Discharge [m³/s]: 317  
Precipitation [mm]: 895  
Runoff coefficient [-]: 0.41

Maximum discharge at flood February 1980: 1426 m³/s  
Maximum discharge at flood July 1980: 1542 m³/s  
Maximum discharge at flood February 1984: 2231 m³/s  
Minimum discharge at low flow 1976: 0 m³/s

**Note:** When calculating the runoff coefficient the starting point was the natural discharge. That means that for that purpose the discharges mentioned have been increased by 22.5 m³/s on account of these extractions or discharges: Albertkanaal (9 m³/s), Zuid-Willemsvaart (14 m³/s), Kanaal Wessem-Nederweert (addition 1 m³/s), Maas-Waal-kanaal (0.5 m³/s).

With discharges up to approximately 400 m³/s, tidal influences can be noticed below the weir of Lith. The Meuse flows on by way of the Bergse Maas (§ 5.6) and the Amer to the Hollands Diep, where it flows together with a considerable part of the Rhine discharge. In the Haringvliet the water is separated from the sea by means of a dam provided with sluices.

In table 7.5 and in figures 7.2, 7.3 and 7.4 the courses of some flood waves have been given. The Stenay discharge only amounts to a small part of the Borgharen discharge. It is striking that the peak of February 1980 in the Meuse Lorraine passed Stenay at a moment when
Figure 7.3 The hydrographs of some stations along the Meuse during the flood of July 1980

Figure 7.4 The hydrographs of some stations along the Meuse during the flood of February 1984
### TABLE 7.5 FLOOD WAVES IN THE MEUSE

<table>
<thead>
<tr>
<th>Jan./Feb. 1980</th>
<th>Stenay (disch.)</th>
<th>Chooz (disch.)</th>
<th>Borgharen (disch.)</th>
<th>Lith (disch.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>day</td>
<td>s.d.</td>
<td>s.d.</td>
<td>s.d.</td>
<td>s.d.</td>
</tr>
<tr>
<td>1</td>
<td>96</td>
<td>2.1</td>
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**disch.** = discharge [m³/s]  
**s.d.** = specific discharge [mm/day]
Borgharen had completed its peak discharge already some days before. At the flood of July 1980 it is clearly to be seen that the water mainly came from the Belgian part of the Meuse basin. On comparing the floods of July 1980 and of February 1984 it is striking that with the summer flood the flood-peak attenuation was much greater. It may be explained by the steeper form of the wave, the greater bank storage due to the initial lower water levels and the greater friction of the flood plains on account of the vegetation.

7.3 THE WEIRS

In a large part of its stretch the water levels in the Meuse are increased artificially by means of weirs. It is especially done for the benefit of shipping, because in that way the passage is possible for ships with greater draughts. Weirs also play a part in the storage of water, the feeding of the canals, recreation, the supply of industrial water, cooling water and drinking water, the generation of energy etc. The control of weirs is in the hands of weir keepers, who often combine that function with the task of a lock keeper. For every weir there are one or more weir keepers.

In France, in Belgium and in the Netherlands alike there are weirs in the Meuse. For the way they came about the reader is referred to chapter 5. The positions of the Dutch and
Figure 7.5 The position of the Dutch and Belgian weirs in the Meuse
the Belgian weirs in the Meuse are given in fig. 7.5.

The principles according to which in different countries water is dammed cross borders. There also turns out to exist a relation between the date of construction and the type of weir. Through the years the mechanisation and the automation of the operation of the weir have been developed more and more. On that account the weirs will be described on the basis of the dates of construction of the types of the weirs, starting from the oldest weir.

The oldest weirs in the Meuse are found in France. About some of them it is known that they were built before 1600. In most cases those old weirs were built for supplying water to the feeding canals and/or for the benefit of shipping. In most cases the weirs have been constructed as fixed dams, consisting of vertical beams standing side by side held in their places by piles of stones on either side (masonry). Those dams have often been provided with flap gates (hausses) on their tops. Between Troussay and Charleville-Mézières 19 of those weirs are still to be found, and upstream from Troussay another number of them.

On behalf of the construction of the Canal de l’Est a great number of needle weirs were built (barrage à aiguilles type Poirée) (fig. 7.6, photo 7.1). All of them are operated by hand. The weirs have been constructed as follows: a metal gantry stands perpendicular to the river axis. A footbridge is mounted on the gantry. At the upstream side against the gantry square beams², called needles, have been placed, which dam up the water. The needles are nearly vertical. By taking away or placing the needles the passing discharge can be influenced. In case of a flood all the needles are taken away and put on the quay. Then by swinging the gantry down it is put on the bottom of the river. The action is called striking. In table 7.6 some data on the weirs are given. It should be noted that due to the presence of canals the number of locks in the stretch from Troussay to the French-Belgian border is

² some French weirs are (partly) provided with rectangular needles
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Waulsor</td>
<td>radial gates</td>
<td>4×19m</td>
<td>1992</td>
<td>TAW+96.05m</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>up to 1992:</td>
<td>combined weir</td>
<td>398×0.10m, 39×1.40m</td>
<td>TAW+95.43m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>km</td>
<td>name</td>
<td>type</td>
<td>first year of operation</td>
<td>width</td>
<td>headwater level drop</td>
<td>m.d.</td>
<td></td>
</tr>
<tr>
<td>-----</td>
<td>-----------------------</td>
<td>----------------------------</td>
<td>-------------------------</td>
<td>-------------</td>
<td>----------------------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>000</td>
<td>Ansereum</td>
<td>radial gates</td>
<td>1989</td>
<td>4×22.5m</td>
<td>TAW+93.65m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>003</td>
<td>Dinant</td>
<td>radial gates</td>
<td>1988</td>
<td>4×22.5m</td>
<td>TAW+91.40m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>007</td>
<td>Houx</td>
<td>radial gates</td>
<td>1988</td>
<td>4×22.5m</td>
<td>TAW+89.40m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>012</td>
<td>Hain</td>
<td>radial gates</td>
<td>1987</td>
<td>4×22.5m</td>
<td>TAW+87.40m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>017</td>
<td>Tüllen</td>
<td>radial gates</td>
<td>1987</td>
<td>4×22.5m</td>
<td>TAW+84.60m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>022</td>
<td>Rivière</td>
<td>radial gates</td>
<td>1987</td>
<td>4×22.5m</td>
<td>TAW+82.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>027</td>
<td>La Plante</td>
<td>radial gates</td>
<td>1986</td>
<td>4×22.5m</td>
<td>TAW+80.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>032</td>
<td>Amiens</td>
<td>radial gates</td>
<td>1986</td>
<td>4×22.5m</td>
<td>TAW+78.40m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>037</td>
<td>Andenne</td>
<td>radial gates</td>
<td>1986</td>
<td>4×22.5m</td>
<td>TAW+76.40m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>042</td>
<td>Avesnes-Neville</td>
<td>radial gates</td>
<td>1986</td>
<td>4×22.5m</td>
<td>TAW+74.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>047</td>
<td>Juvézy</td>
<td>radial gates</td>
<td>1986</td>
<td>4×22.5m</td>
<td>TAW+71.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>052</td>
<td>Monsin</td>
<td>sliding gates</td>
<td>1986</td>
<td>5×22m</td>
<td>TAW+69.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>057</td>
<td>Herouville sous Argenteau</td>
<td>needle weir radial gates</td>
<td>1986</td>
<td>6×27m</td>
<td>TAW+67.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>062</td>
<td>Lichte</td>
<td>radial gates</td>
<td>1986</td>
<td>6×27m</td>
<td>TAW+65.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>505</td>
<td>NETHERLANDS</td>
<td>sliding gates</td>
<td>1931</td>
<td>3×23m, 1×30m</td>
<td>NAP+44.00m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>510</td>
<td>Borgharen</td>
<td>S. and P.</td>
<td>1929</td>
<td>5×17m</td>
<td>NAP+20.80m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>515</td>
<td>Limé</td>
<td>S. and P.</td>
<td>1929</td>
<td>5×17m</td>
<td>NAP+16.75m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>520</td>
<td>Roemond</td>
<td>S. and P.</td>
<td>1929</td>
<td>5×17m</td>
<td>NAP+14.00m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>525</td>
<td>Beffont</td>
<td>S. and P.</td>
<td>1929</td>
<td>5×17m</td>
<td>NAP+10.75m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>530</td>
<td>Sambekend</td>
<td>sliding gates</td>
<td>1929</td>
<td>18×3.45m, 2×5m</td>
<td>NAP+7.50m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>535</td>
<td>Grave</td>
<td>sliding gates</td>
<td>1929</td>
<td>3×38m</td>
<td>NAP+5.00m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>540</td>
<td>Lath</td>
<td></td>
<td>1929</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### TABLE 7.6 THE WEIRS IN THE MEUSE (continued)

<table>
<thead>
<tr>
<th>km</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>*</td>
<td>With respect to the limited amount of data only weirs downstream from Trousscy have been included</td>
</tr>
<tr>
<td>S</td>
<td>The French hydro-electric power stations have not been included, because they have a small capacity and are often situated between two weirs</td>
</tr>
<tr>
<td>P</td>
<td>Especially in the French reach of the Meuse the drops in water level differ from the headwater levels up- and downstream from those weirs. Therefore those drops are included explicitly</td>
</tr>
<tr>
<td>rev.Poirée s.</td>
<td>Small deviations with respect to the data in the table can be found when other sources are used, even if they originate from the same service</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>km from the source</th>
<th>under construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>S</td>
<td>Stoney slides</td>
</tr>
<tr>
<td>P</td>
<td>Poirée shutters</td>
</tr>
<tr>
<td>rev.Poirée s.</td>
<td>reversed Poirée shutters</td>
</tr>
<tr>
<td>3 x 17 m</td>
<td>3 weirs with a width of 17m each</td>
</tr>
<tr>
<td>m.d.</td>
<td>maximum discharge through hydro-electric power station</td>
</tr>
<tr>
<td>N</td>
<td>Normale, French reference level according to the system IGN69. At the French-Belgian border at Heer-Agimont the French reference level is 1.80 m above that of the TAW</td>
</tr>
<tr>
<td>TAW</td>
<td>Tweede Algemene Waterpassing, Belgian reference level. The reference level of the TAW is situated 2.33 m below that of the NAP</td>
</tr>
<tr>
<td>NAP</td>
<td>Normaal Amsterdams Peil, Dutch reference level</td>
</tr>
</tbody>
</table>
greater than the number of weirs: 59 as opposed to 48.

In Belgium there are several kinds of weirs. The first generation of weirs, almost disappeared in 1990, consists of needle weirs (Poirée) and so-called combined weirs. For half of it, a combined weir consists of a needle weir and for the other half of a flashboard weir of the Chanoine type. The latter much resembles a needle weir in having a footbridge and manual operation. However, the weir has flashboards instead of needles. The flashboards can be tilted and with great discharges they are struck to the bottom in the flow direction. The flashboards are placed downstream from the footbridge. By way of a connecting rod with the footbridge the flaps are kept almost vertical during normal discharges. Each flashboard contains a valve. The flashboards are 1.30 m wide, but between the flashboards there still is a space of 0.10 m. In 1990 the weirs of the first Belgian Meuse Canalisation are only to be found at Hastière, Waulsort and Hermalle sous Argenteau (table 7.6).

With the exception of the weir at Grave the five weirs of the Dutch Meuse Canalisation consist of two sections: one with shutters of the Poirée type and one with Stoney slides (fig. 7.7).

The shutter part shows great resemblance to the needle weir, especially the gantry against which the damming elements have been placed, at the same time serving as a

*Photo 7.2 The La Plante weir*
footbridge. With great discharges here, too, the gantry is put on the bottom, as is the case with the needle and flashboard weirs. The Poirée shutters have been placed on top of each other in three rows. They are taken down row after row using a crane that moves on the footbridge. When the gantry has been struck to the bottom ships can use the pass formed in that way, the so-called shipping opening.

The other part of the weir consists of the so-called Stoney slides. They consist of two slides put vertically against each other (fig. 7.8). By leaving one slide on the sill and moving the other slide in a vertical direction a great range of nappe heights can be achieved. The greatest nappe height possible is reached as soon as both slides sit on the sill. Although a flow under the slides is also permitted, it never occurs in practice. The operation is done electrically. The Stoney slides are worked from the lock control room. Two or three pairs of Stoney slides 17 m wide have been built side by side. In the weir operation the Stoney slides are used for precision.

The lock at Grave has a different construction (photo 5.2): the gantry cannot be put on the bottom, but it is swung against the bridge for traffic lying over it. The crane can be moved along the bridge. For those reasons the shutter weir is called a reversed shutter weir. Stoney slides are absent in the Grave weir.

![Figure 7.7 The weirs of Belfeld and Maasbracht. Source: Anonymous (1985b)](image)

Most modern locks to be found in the Meuse are provided with tilting flaps. There are many versions. In fig. 7.9 they are reproduced in a simplified and schematic way.

In general those weirs have large slides or gates on which tilting flaps are mounted. With small and middle-sized discharges the large weir elements are placed down on the sill and the control of the weir is only performed through the tilting flaps. With greater discharges the large elements are lifted more or less, thus allowing a flow under the elements. With very great discharges the weir is put out of operation by lifting the elements over the surface of the water. The operation of slides and tilting flaps is done electrically.

At Monsin, Borgharen and Lith the large slides have a flat front, in other places they have a curved shape. The weirs of the former kind have so-called sliding gates, those of the latter kind radial gates.

For tourist reasons it was tried to keep the height of the weir system limited in the section between the French border and Namur. For that reason unlike in the other weirs the driving mechanism has not been installed over the weir elements, but near the surface of the
water (photo 7.2). With great discharges the elements are tilted down to a position just over the water level.

With great discharges ships can sail through the resulting opening in the weirs at Borgharen and Lith, unlike in the other weirs.

For some weirs or their surroundings it is also possible to calculate the discharges. For the Netherlands and Belgium the discharges near Lixhe and Borgharen can be deduced direct from the level of the water. At La Plante, Ampsin-Neuville and Lith the discharges are calculated by way of the water levels, the position of the weir elements and the discharge via the locks and the hydro-electric power station, if present.

**Hydro-electric power stations**

At any weir in the Meuse there is a certain drop and as a result the possibility to generate energy by means of a hydro-electric power station. From a hydraulic point of view these factors are necessary for an economically profitable generation:
- as great a drop as possible, and
- as great a (low) flow as possible.

For those reasons hydro-electric power stations in Belgium are found near large headwater reaches. In the Netherlands the construction of two water-power stations was started after the energy crises (table 7.6).

In Belgium the transit discharge is decided by the electricity companies. However, the headwater level is to be kept within certain limits. On account of that construction it does not become clear to those controlling the water downstream what discharge they may expect, since the electricity company will aim at as great a drop as possible and may make the transit dependent on the momentaneous demand for electricity.
It is striking that the power stations built have capacities that are one and a half times as large as the mean discharges (compare table 7.6 with table 6.21).

In most power stations the turbines have a controllable discharge. In the stations of Andenne-Seilles and Lixhe, however, the turbines can only be put in two positions, on and off. With those stations the result is that the discharge changes in phases and that it is a multiple of the discharge by a single turbine. The Andenne-Seilles station has 3 turbines with a capacity of approximately 75 m³/s each, the Lixhe station has 4 turbines with a capacity of approximately 80 m³/s each. The effects caused by the power station are noticeable over a long distance. For example the effects of the operation of the Lixhe station on the water levels are still clearly to be observed at Borgharen-Dorp.

In the French Meuse about another 40 hydro-electric power stations are found with a smaller capacity, such as water mills. Most of the hydro-electric power stations rather have a cultural-historical value than an economic one (e.g. in Sedan).

Figure 7.9 The weirs with tilting flaps
a: sliding gates, rounded gates
(example: Andenne-Seilles)
b: radial gates
(to be found above Namur)
c: sliding gates, flat gates
(example: Lith)
double arrow: movement of gate
single arrow: movement of tilting flap
(Source: Construction plans and informative material)
Leakage
When a weir is entirely closed, some flow will pass the weir. That flow is called leakage. In general the leakage is greater as the number of damming elements is greater. Especially with needle weirs, combined weirs and at shutter gates the leakage is great. With the shutter weirs in the Netherlands values between 10 and 25 m³/s are known. With weirs with sliding gates leakage can be limited to approximately 1 m³/s. By means of emergency measures leakage can be decreased.

Striking and lifting discharges
For modelling the progress of flood waves it is important to know the striking and lifting discharges. By a striking or lifting discharge is meant that discharge at which the whole weir is put out of operation. If that discharge is known it is clear when the river has a free flow and when e.g. discharge curves can be used. However, the curious fact arises that those discharges are hardly known. Consequently the following figures should be used with some prudence.

- French weirs: if the corresponding discharge at Charleville-Mézières is equal to 250 m³/s. The accuracy of the number is illustrated with the aid of the striking discharge of the weir at L’Alma: it amounts to 280 m³/s, though it is above Charleville-Mézières.
- Belgian weirs, from French border to Namur: 500 - 900 m³/s.
- Belgian weirs, from Namur to Dutch border (excepting Hermalle): 1500-2500 m³/s.
- Hermalle: unknown.
- Dutch weirs: Borgharen 1200 m³/s, Linne 1100 - 1200 m³/s, Roermond 800 m³/s, Belfeld 700 m³/s, Sambeek 800 - 1000 m³/s, Grave 1000 m³/s, Lith 1000 m³/s.

Ice
If ice is going to be formed there are two dangers for weirs: the mechanism may freeze up so that the position of flaps, partitions, needles and slides cannot be changed any longer, and the weir may be damaged by ice-floes. In order to prevent those dangers the weir elements are lifted or struck respectively with ice drift.

Ice drift especially occurs in those parts of the Meuse where little industrial heat is discharged. Below Namur the industrial discharges of heat are so great that the chance of the formation of ice has become much smaller than above Namur. The heat transfer is a fortunate thing for the prevention of inundations. In the past inundations were caused to a considerable extent by the formation of ice dams (chapter 5).

In the Netherlands ice drift is the only situation in which it is not the weir keeper who decides on lifting and striking the weirs, but the so-called ice board, a regional consultative body within Rijkswaterstaat.

Weir operation
The operation of the weirs comprises the whole of the measures that are taken to rule water levels and/or discharges. The control of weirs mainly used to be aimed at the needs of shipping, at present the interests of other users are also taken into account. In times of floods the superfluous surface water should be let through without any obstacle.

Upstream from the weir it is usually tried to maintain a definite water level, the headwater level. In some weirs, however, the headwater level is not constant, but a function of the discharge. It is especially the case in most Belgian weirs. In the case of a greater discharge the headwater level is lowered. In that way the water levels upstream in the headwater reach become less high.
In the Netherlands the headwater level is kept constant at the upstream side of the weir, except for the Grave headwater reach, where the water level at the entrance of the Maas-Waal-kanaal is kept constant.

In all the French weirs the headwater level is constant. In table 7.6 the headwater levels are given when the discharges are small.

The weir keeper is responsible for the control of the weir, and in general he should maintain a definite headwater level as well as possible. In order to cause his downstream colleague to come into action as little as possible a weir keeper will minimise the number and size of the adjustments, so that the course of the discharges is kept regular.

Whereas in the Netherlands and in France the control of the weirs is done by hand, the control in many Belgian weirs is performed by a computer when discharges are normal.

In many weirs the water levels upstream and downstream from the weir can be read digitally. In Belgium those data are dispatched by the B.E.E. (Bestuur Elektriciteit Elektromechanica) via a data network, in the Netherlands it is going to be done via the M.S.W. network (Monitoring Systeem Waterhoogten) of Rijkswaterstaat.

Apart from information on the weir itself and weirs lying upstream, the discharges and water levels at stations in the Meuse and its tributaries, the weather forecast etc. are also utilised. That is necessary in order to gain a somewhat longer lead time since most weirs are unmanned at night.

When performing the control of the weir the experience of the weir keeper is one of the most important aids.

For all the weirs in each country it is necessary for the weir master to inform his downstream colleague of the actions to be executed in his own weir as well as of the relevant information from his upstream colleague.

In order to serve shipping as long as possible in the event of low flows Dutch weir keepers increase the headwater level when low flows are arising.
Chapter 8. Introduction to flow forecasting modelling for the river Meuse

8.1 DEMANDS AND RESTRICTIONS

For a lot of purposes it is desirable to have a forecast of the water level and discharge, for example for shipping, protection against floods, weir operation, (hydro-electric) power station operation, water distribution management, calculation of travel times of substances, calculation of sediment transportation, etcetera. From the point of view of discharge forecasting modelling, three kinds of models can be distinguished:

a. Flood forecasting
b. Low flow forecasting
c. Routine forecasting

The first two types of models are only used during extraordinary situations, whereas the third type is used as an every-day routine.

The flood forecasting may help to take measures to minimise damage caused by high water levels and great discharges (chapter 2).

The low flow forecasting may help to decide what kind of measures could be taken to optimise the important functions of the river and its water (chapter 3).

Routine forecasting can be important for all the reasons mentioned above.

The routine forecasting was excluded at the start of this study. It was done because the main group of users would be shipping, and they were thought not to have a need for a discharge forecast every day, because all the required minimum water depths are maintained with the help of weirs, by maintaining a certain constant headwater level. However, during this study it has become clear that also a routine forecast would be useful. At present routine forecasts would solve several problems, two of which are mentioned here.

First, the draught of ships has increased in time. To realise a passage without trouble the minimum water depth rather than the headwater level is critical. The purpose of weir operation is therefore shifting from a constant headwater level to a constant minimum water depth. The latter can be achieved better if a routine forecast is available.

Secondly, in case of a water quality calamity, a discharge forecast is nearly indispensable. The continuing absence of a routine forecasting is regretted by the author.

The choice of a type of model and its calibration is dependent on demands and restrictions (fig. 8.1).

In many cases the requirements can be made explicit with the help of two parameters: accuracy and lead time. It is, however, often formulated as: as best and as long as possible. For the flood forecast for the Meuse at Borgharen-Dorp a lead time of 24 h and an accuracy as good as possible are desired. For the low flow forecast a lead time of at least a week in case of dry weather flow is desired, the accuracy should be as good as possible.

Various types of restrictions will reduce the number of types of models that can be applied. Already at the start it should be ascertained whether the model can match operational restrictions in the future. For example, an hourly call by means of several telephones with an automatic answering device will not be carried out enthusiastically by the model operators.

Before a good choice of a type of model can be made, a good understanding of the system is essential.
Figure 8.1 Considerations for making a choice of a forecasting model or module

Floods and droughts are caused by quite different mechanisms. A lot of precipitation can cause a flood in a relatively short period, whereas low flows are caused by a long period with little, if any, (effective) precipitation. Therefore it is logical to make a distinction between the models to be used.

In the case of floods the processes involved are of various characters, of which watershed modelling (chapter 10) and river wave propagation (chapter 11) are the most important. The subdivision of the runoff process into so-called modules will be elucidated in chapter 9. Chapter 12 deals with practical problems of the real-time model, such as missing data and the development of the user-friendly computer program FLOFOM.

In case of a drought the most important process for flow forecasting is reservoir depletion. However, in the Meuse catchment the discharge is significantly influenced by man (for the Netherlands mainly by Rijkswaterstaat). As the forecast will be used by Rijkswaterstaat it is considered to be useless to develop a forecast for the Netherlands, because then the actions of Rijkswaterstaat itself would be forecast. It is for that reason that the low flow forecasting has been restricted to the Dutch border, more exactly Maastricht-St.-Pieter. One of the remaining problems will be the influences of the actions taken by the water authorities upstream from the Netherlands on the flow during droughts, which are not clear either. The present human influence (chapter 3) is assumed to be maintained in the future. The modelling will be presented in chapter 13.

8.2 HYDROLOGIC MODELS

One of the topics most investigated in hydrology is the rainfall-runoff relation. It has lead to a great number of rainfall-runoff models, of various types. Not all of those types are suitable for the modelling of the hydrologic processes in the Meuse basin. Here a short discussion will be given of several types of models, and their suitability for operational forecasting for the Meuse. The discussion will be restricted to so-called mathematical models. Due to the modern information technology that type of models has outdated most physical hydrologic models, such as scale models.

Opposite page:
Figure 8.2 Survey of the precipitation stations in the Belgian reach

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In hydrologic modelling, the most important choice to be made is the basis of the equations to be used.

If the model is based on proven physical relations it is a physically based model. If the basis is not present at all, the terms empirical model and black box model are used. Between those extremes the conceptual models are found, which have some physically acceptable basis. That type of model is often a good compromise between theoretical analysis and practical calculation. The limits between the types are not well defined. If a concept or empirically found relations are found to be exact, the model changes its type into a physically based one. (Darcy's equation was based on experiments and therefore empirical, until the correctness was demonstrated using the Navier-Stokes' equation, which changed the type of equation to a physically based one).

Rainfall-runoff models for real-world catchments nearly always contain some conceptual parts.

Another form of classification is the subdivision into linear and non-linear models. It is known that the rainfall-runoff process is non-linear. Two examples of non-linear parts in the rainfall-runoff process are given: the precipitation-effective precipitation relation and the wave velocity in watercourses. Many modellers, however, favour only linear elements in their models because of the easier computation. It is acceptable if the relations are nearly linear in the investigated interval. A practical way of investigation is first to suppose that the system can be considered to be linear. Then an analysis of the calibration residuals may show what non-linear elements should be included in the model.

Rainfall-runoff models are often subdivided in lumped and distributed models. A lumped model neglects the variability within the areal unit and/or time unit used. A distributed model does take into account that variability. It is striking that in several distributed models the variability within the considered unit is neglected, and therefore the model at that scale is lumped (for example the SHE-model neglects spatial variabilities within the area units of 250 x 250 m (Abbot et al., 1986a,b; Bathurst, 1986a,b)).

Another subdivision is that into deterministic and stochastic models. If the input and/or output has a probability density function, the model is called stochastic. If, on the other hand, the input and output are both determined uniquely, the model is called deterministic.

In this thesis most kinds of the models mentioned will be applied.

8.3 AVAILABLE DATA

For the development of the flow forecasting models many types of data have been used from many stations. In the relevant chapters and sections the data will be elucidated. Because many of the stations are used in more than one section, the choice was made here to present the map indicating the locations of the stations (fig. 8.2 and 8.3).

It should be noted that it took a lot of time (more than a year) to obtain all the data desired. That especially applies to time-series with an observation frequency higher than once-per-day.

Opposite page:
Figure 8.3 Survey of the water level and discharge measuring stations

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Chapter 9. Flood forecasting: A subdivision into modules

In hydrology many types of models have been developed with regard to flood forecasting. Differences in catchment properties, users’ needs in terms of accuracy and lead time, available data and available capacity in terms of time, money and equipment have caused a wide variety of types of forecasting models (cf. fig. 8.1, § 8.2).

In the ideal situation the catchment that is studied is well equipped, homogeneous with respect to hydrologic properties, homogeneous with respect to the available data and is not subject to modifications. Furthermore, a long time series for calibration is available, the number of processes involved is small, etcetera. Those are the areas which are very suitable for academic model development. In fact, the catchments which offer the ideal situation are usually small. The Meuse basin does not offer the properties of the ideal situation. Some of the factors that complicate flood modelling are:

- the catchment is situated in five countries,
- there is no coherent system of measuring and collecting data in the different countries,
- the overlapping period of time series is very small,
- it is not de facto clear what data are available in real-time,
- the type of geometric information across borders changes dramatically,
- the influences of some large tributaries (such as the Vesdre) are very important, unlike those of other tributaries (such as the Roer). Some tributaries (such as the Semois) only need a short lead time,
- regular discharge data are available for only few stations along the Meuse,
- the dimensions of the Belgian Meuse and the weirs in the Belgian Meuse have been changed during the last few decades,
- the exact operation of the weirs is not known,
- snow plays a small but significant role in the runoff process, and
- the network of meteorological stations is sparse as compared with the size of the catchments of the tributaries.

The Meuse catchment can therefore be considered to be heterogeneous with respect to various aspects. Therefore the model is split up into several modules, where each module approaches the ideal of a homogeneous unit more. Two cross sectional cuts have been made, resulting in a subdivision:

![Flow scheme for the flood forecasting model](image)

*Figure 9.1 Flow scheme for the flood forecasting model*
a. in tributaries and river modelling, and
b. in countries, i.e. in France, Belgium and the Netherlands.
The subdivision into tributary and river modelling is a logical one, because of the different kinds of processes involved.

The modelling of the tributaries involves the study of the hydrologic cycle from precipitation to subcatchment runoff. The forecast module will need precipitation data, completed with precipitation forecasts, and runoff data, although not all three types are required for all the tributaries. The output will be a discharge forecast. The modelling process is described in chapter 10. There the subdivision into a French, Belgian and Dutch part will be elucidated in some more detail (§ 10.1).

Discharges forecast and measured form the input of the modules concerning the river. The output consists of forecast discharges and water levels. The modelling is described in chapter 11, which starts with a motivation for the subdivision of the river into three parts of the river, corresponding to the country.

In all two times three, so six so-called modules have been developed. The scheme ultimately used for the flood forecasting model is shown in fig. 9.1.
Chapter 10. Flood-forecasting: The tributaries

10.1 A SUBDIVISION INTO THREE PARTS

The character of the tributary runoff changes along the course of the Meuse. The most important tributaries with respect to flood forecasting can be found in the middle course of the river. It is a ground to subdivide the tributaries into a number of types of models, because then the modelling of each tributary can be tailored to the specific demands. Another, at least equally important, ground to make a subdivision is the difference in the available kind and amount of data. The kind of available data is predominantly determined by the authority in charge. Because of those facts it is logical to treat the French, Belgian and Dutch tributaries separately.

It is not a priori clear which tributary should be categorised in which country. In particular it applies to the Viroin and the Semois, which are almost completely situated in Belgium, but discharge into the French Meuse. Because of the long wave travel time from those mouths to the Dutch border and the fact that a measuring station in the Meuse is situated between the mouth and the Dutch border, a less detailed and data-requiring model will suffice compared to that of the tributaries debouching into the Belgian Meuse. Therefore they are treated in a way different from the tributaries which are situated completely in Belgium. The Viroin and the Semois are therefore considered to be French tributaries. It should be noted that discharge measurements are available from the Belgium monitoring system.

The rivers Roer and Niers are situated predominantly in Germany. But because they discharge into the Dutch part of the Meuse, both rivers will be called Dutch tributaries.

In § 11.2 it will be shown that the input for the French part of the river Meuse also needs an input from its most upstream station Montcy-Notre-Dame. The station will be treated in the same way as those on the other French tributaries, and can therefore be seen in this model as another station of a tributary.

The demands for the French tributaries are relatively small, as the wave travel time approaches the lead time desired for the Borgharen-Dorp forecast. However, it does not imply that no forecast is necessary. In the present situation discharge data of the French stations are available in real-time only a limited number of moments per day. If the moment of the last known discharge is many hours before the moment of making the forecast, a discharge forecast for the tributaries is necessary. It is clear that a short-term forecast will suffice.

The discharges of the Belgian tributaries are important in the flood forecast in three ways: They are great in magnitude, the discharges change fast in time and the wave travel time between the mouths and the Dutch border is relatively small. A complicating factor is the fact that also the time between rainfall and runoff is relatively small. All those factors have led to a comprehensive model in which among other things the rainfall-runoff process is simulated.

The Dutch tributaries which are incorporated in the model, Roer and Niers, have both a relatively flat middle and lower course. Those facts cause a relatively smooth hydrograph. In combination with the relatively small magnitude of the discharges of the Dutch tributaries, a relatively simple model will suffice. An essential difference of the Dutch tributaries model compared to that of the French tributaries is the fact that for the Dutch ones a longer lead time is necessary.
The modelling of the French, Belgian and Dutch tributaries will be explained in the next three sections.

10.2 THE FRENCH TRIBUTARIES

In the flood forecast for Borgharen-Dorp the role of the discharges of the tributaries discharging into the French Meuse are of minor importance, as the wave travel time between the French measuring stations and Borgharen is relatively great. Therefore the required lead time is relatively small. The accuracy of the forecast should at least be the same as for a longer term forecast of the Belgian tributaries. Both conclusions justify a relatively simple forecasting model for the French tributaries.

Discharge data for stations on the territory of France which are used for calibration are available in the form of non-equidistant time series which have been collected by the Service de la Navigation de Nancy. The French authorities have made efforts to store only a minimum of data, with the preservation of the original course of the discharge. It was done by omitting discharge data for which the same values could be obtained by linear interpolation. It has led to a significant reduction of the storage capacity needed. However, the way of data storage, in day-numbers starting from 1 January and in thousands of days, is complex and may cause erroneous calculations. For example, 1 March in a leap-year has another day number than in a normal year. Unfortunately, the data differ significantly from the mean daily discharges published. In § 11.2 the difference will be dealt with.

For this investigation the opposite calculation has been made, by making a linear interpolation of the non-equidistant series. After an analysis of the data it was decided to base the model on a time step of 4 hours. For reasons of simplicity, the time step of 4 hours was also used for the Belgian station of Treignes, although an hourly series was available. More information on the French discharge data can be found in § 11.2.

No certainty was obtained about the moments at which discharge data will be available in real-time for the stations on the French territory. Probably the current method, where discharge data for 8.00 h, 12.00 h and 16.00 h local time are determined, will be maintained. The real-time availability of the discharges at especially Montcy-Notre-Dame is considered not to be ensured completely, as it has to be estimated by an employee. Therefore also an alternative module to forecast the discharge at Chooz has been developed (§ 11.2), based on the discharge at Chooz only. The alternative model for Chooz is based on the same expressions as those used for the French tributaries, and the results are therefore given here. It should be remarked that a more extended system of data collection is being studied by the Service de la Navigation.

Precipitation data in France can be provided by the National Direction of Meteorology (Direction de La Météorologie Nationale). For the Meuse basin area the Service Météorologique is responsible. The precipitation network is sparse, compared to the Dutch one. For none of the stations in the French part of the Meuse basin hourly precipitation data are available.

The forecast model will be concentrated on forecasts only a few hours ahead. Because the lead time is small and the complexity of the model should not increase unnecessarily, it has been decided not to incorporate precipitation data. (In retrospect it was a lucky choice, as doubts arose about the moments of the discharge, cf. § 11.2).

The calibration was done with the use of all the available periods. A subdivision of the
TABLE 10.1 IDENTIFICATION OF ARIMA-MODELS FOR THE SEMOIS AT HAULMÉ

<table>
<thead>
<tr>
<th>ARIMA-model</th>
<th>$\sigma_0^2$ [m³/s²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1,0,0)</td>
<td>84.3</td>
</tr>
<tr>
<td>(2,0,0)</td>
<td>18.3</td>
</tr>
<tr>
<td>(3,0,0)</td>
<td>17.9</td>
</tr>
<tr>
<td>(1,1,0)</td>
<td>18.5</td>
</tr>
<tr>
<td>(2,1,0)</td>
<td>18.0</td>
</tr>
<tr>
<td>(3,1,0)</td>
<td>17.8</td>
</tr>
<tr>
<td>(0,1,1)</td>
<td>35.0</td>
</tr>
<tr>
<td>(1,1,1)</td>
<td>17.9</td>
</tr>
</tbody>
</table>

data into a calibration period and a verification one is considered to be inappropriate, as data of only few floods were available. With the help of the SPSS-X-routine ARIMA a Box-Jenkins analysis has been performed.

An ARIMA-model was considered to be suitable for the model, because the data are equidistant in time, and the ARIMA-model will probably be less complex than a physically based or conceptual model.

Several ARIMA-models (p,d,q) have been identified, where p stands for the the number of autoregressive terms, d for the order of integration and q for the number of moving average terms. Results for the Haulmé gauge are shown in table 10.1, in which $\sigma_0^2$ stands for the mean square error.

Table 10.1 shows that the (1,1,0) model is appropriate. Only a slightly smaller mean square error is obtained if more parameters are used. It should be noted that the (1,1,0) model is very suitable for operational flood forecasting, as only two discharge measurements are necessary. In particular moving average models need many more data. Also because of the principle of parsimony the (1,1,0) model has been chosen.

Based on time steps of 4 hours, the (1,1,0) model can be expressed as:

$$Q_{i+4} = (1+\alpha_{d,q})Q_i - \alpha_{d,q}Q_{i-4} + e_{i+4} \quad (10.1)$$

where:

- $\alpha_{d,q}$ = coefficient used in the determination of $Q_{i+4}$, using $Q_i$ and $Q_{i-4}$ [-]
- $Q_i$ = discharge at $i \Delta t$ [m³/s]
- $e_i$ = residual at $i \Delta t$ [m³/s]

In this section $\Delta t = 1$ h.

The forecasting model corresponding to (10.1) is:

$$\dot{Q}_{i+4} = (1+\alpha_{d,q})Q_i - \alpha_{d,q}Q_{i-4} \quad (10.2)$$

where:
\( \dot{Q}_{i+1} = \) discharge forecast made at \( i \Delta t \) with a lead time of \( 4 \Delta t \) \[ \text{m}^3/\text{s} \]

If data are available only at 16.00 h of the previous day and at 8.00 h of the current day the model (10.2) cannot be used directly. If a forecast of 6 hours ahead is required, the model will not be suitable either. Therefore a more general model based on (10.2) is required.

Analysis has shown that a more general model can be derived from (10.2) for \( \alpha_{4,4} \neq 1 \):

\[
\dot{Q}_{i,n} = (1 + \alpha_{m,n})Q_i - \alpha_{m,n} Q_{i-m}
\]

(10.3)

where:

\( m, n \) = positive integer constants
\( \alpha_{m,n} \) = coefficient

It can be derived that the coefficient \( \alpha_{m,n} \) is equal to:

\[
\alpha_{m,n} = \frac{\alpha^m - \alpha^{m+n}}{1 - \alpha^m}
\]

(10.4)

where:

\( \alpha \) = \( \alpha_{1,1} \)

\textbf{Derivation}

Let be given:

\[
Q_{i+1} = (1 + \alpha)Q_i - \alpha Q_{i-1} + e_{i+1}
\]

(10.5)

where:

\[
e_i \sim N(0, \sigma)
\]

(10.6)

To obtain the desired result the properties of eigen vectors and eigen values will be used. (10.5) can be written as:

\[
\overline{w}_{i+1} = \Lambda \overline{w}_i + \overline{b}_{i+1}
\]

(10.7)

where:

\[
\overline{w}_{i+1} = \begin{pmatrix} Q_{i+1} \\ Q_i \end{pmatrix}; \quad \Lambda = \begin{pmatrix} 1 + \alpha & -\alpha \\ 1 & 0 \end{pmatrix}; \quad \overline{b}_{i+1} = \begin{pmatrix} e_{i+1} \\ 0 \end{pmatrix}
\]

(10.8)

Starting from one, repeated use of (10.7) yields:
\[
\bar{w}_{i+1} = \Lambda^i \bar{w}_i + \sum_{j=0}^{i-1} \Lambda^j \bar{f}_{i-j-1}
\]  \hspace{1cm} (10.9)

\(\Lambda^k\) can easily be calculated with the help of eigen values and eigen vectors, because:

\[
\Lambda^k = \mathcal{E} D^k \mathcal{E}^{-1}
\]  \hspace{1cm} (10.10)

where \(\mathcal{E}\) is a basis of eigen vectors, and \(D\) a diagonal matrix containing the eigen values.

The calculation of the eigen values \(\lambda_1\) and \(\lambda_2\) and of the matrix \(D\) gives, with a few intermediate steps:

\[
\begin{vmatrix}
1+\alpha -\lambda & -\alpha \\
1 & 0 -\lambda
\end{vmatrix} = 0 \Rightarrow \lambda_1 = 1 \quad \lambda_2 = \alpha \Rightarrow D = \begin{pmatrix}
1 & 0 \\
0 & \alpha
\end{pmatrix}
\]  \hspace{1cm} (10.11)

The eigen vectors \(\bar{v}\) can be calculated; for \(\lambda_1\):

\[
\begin{pmatrix}
1+\alpha & -\alpha \\
1 & 0
\end{pmatrix} \bar{v}_1 = \lambda_1 \bar{v}_1 \Rightarrow \bar{v}_1 = \begin{pmatrix}
1 \\
1
\end{pmatrix}
\]  \hspace{1cm} (10.12)

and for \(\lambda_2\):

\[
\begin{pmatrix}
1+\alpha & -\alpha \\
1 & 0
\end{pmatrix} \bar{v}_2 = \lambda_2 \bar{v}_2 \Rightarrow \bar{v}_2 = \begin{pmatrix}
\alpha \\
1
\end{pmatrix}
\]  \hspace{1cm} (10.13)

That gives:

\[
\mathcal{E} = \begin{pmatrix}
1 & \alpha \\
1 & 1
\end{pmatrix}
\]  \hspace{1cm} (10.14)

After a few intermediate steps:

\[
\mathcal{E}^{-1} = \frac{1}{1-\alpha} \begin{pmatrix}
1 & -\alpha \\
1-\alpha & 1
\end{pmatrix}
\]  \hspace{1cm} (10.15)

The result can easily be checked with (10.10) for \(k=1\).

Substituting in (10.10) yields, after a few steps:

\[
\Lambda^k = \frac{1}{1-\alpha} \begin{pmatrix}
1-\alpha^{k+1} & -\alpha + \alpha^{k+1} \\
1-\alpha^k & -\alpha + \alpha^k
\end{pmatrix}
\]  \hspace{1cm} (10.16)

It results in:

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\[ Q_{i+1} = \frac{1 - \alpha^{i+1}}{1 - \alpha} Q_i + \frac{-\alpha + \alpha^{i+1}}{1 - \alpha} Q_0 + \sum_{j=0}^{i-1} \frac{1 - \alpha^{j+1}}{1 - \alpha} e_{i-j+1} \]  

(10.17)

The equation shows that the sum of the coefficients before \( Q_i \) and \( Q_0 \) are equal to 1 for all \( \alpha \neq 1 \).

From the theory of eigen values and eigen vectors it can be derived:

\[ Q_i = p_1 \lambda_1^i + p_2 \lambda_2^i + \text{error term} \]  

(10.18)

where \( p_1 \) and \( p_2 \) are constants, as both eigen vectors occur once. It should be noted that in some way \( p_1 \) and \( p_2 \) contain the information of the values of \( Q_1 \) and \( Q_0 \).

The error term, found in (10.17), and the eigen values, found in (10.11), are substituted in (10.18):

\[ Q_i = p_1 + p_2 \alpha^i + \sum_{j=0}^{i-2} \frac{1 - \alpha^{i-j+1}}{1 - \alpha} e_{i-j} \]  

(10.19)

Rewriting the error term yields:

\[ Q_i = p_1 + p_2 \alpha^i + \sum_{j=2}^{i} \frac{1 - \alpha^{i-j+1}}{1 - \alpha} e_j \]  

(10.20)

Applying it to \((i+n) \Delta t\) and \((i-m) \Delta t\) yields:

\[ Q_{i+n} = p_1 + p_2 \alpha^{i+n} + \sum_{j=2}^{i+n} \frac{1 - \alpha^{i+n-j+1}}{1 - \alpha} e_j \]  

(10.21)

\[ Q_{i-m} = p_1 + p_2 \alpha^{i-m} + \sum_{j=2}^{i-m} \frac{1 - \alpha^{i-m-j+1}}{1 - \alpha} e_j \]  

(10.22)

From (10.20), (10.21) and (10.22) the unknowns \( p_1 \) and \( p_2 \) can be eliminated, and therefore the information of \( Q_i \) and \( Q_0 \). Then one equation is left, where \( Q_{i+n} \) is a function of \( Q_i \) and \( Q_{i-m} \) and errors at several moments.

After many intermediate steps the result is:

\[ Q_{i+n} = \left(1 + \frac{\alpha^{m-n}}{1 - \alpha^m}\right) Q_i - \left(\frac{\alpha^m - \alpha^{m+n}}{1 - \alpha^m}\right) Q_{i-m} \]

\[ + \frac{1}{1 - \alpha} \left\{ \frac{\alpha^m - \alpha^{m+n}}{\alpha^m - 1} \sum_{j=0}^{m-1} (1 - \alpha^j) e_{i-m+j+1} + \sum_{j=1}^{n} (1 - \alpha^j) e_{i+n-j+1} \right\} \]  

(10.23)

The variance of the forecast error is:

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\[ \sigma_{m,n}^2 = \left( \frac{1}{1-\alpha} \right)^2 \left( \left( \frac{\alpha^m - \alpha^{m+n}}{\alpha^m - 1} \right)^2 \sum_{j=0}^{m-1} (1-\alpha^j)^2 + \sum_{j=1}^{n} (1-\alpha^j)^2 \right) \sigma_{1,1}^2 \] (10.24)

where:
\[ \sigma_{m,n}^2 = \text{variance of the forecast error if } Q_{i+n} \text{ is forecast, using } Q_i \text{ and } Q_{i-m} \] [m$^6$/s$^2$]

It should be noted that the errors occurring before $(i-m)\Delta t$ do not influence the accuracy of the forecast. That could be expected, as the forecast is only dependent on values at $(i-m)\Delta t$ and $i\Delta t$.

The corresponding forecasting model of (10.23) is:
\[ \hat{Q}_{i+n} = \left( 1 + \frac{\alpha^m - \alpha^{m+n}}{1-\alpha^m} \right) Q_i - \left( \frac{\alpha^m - \alpha^{m+n}}{1-\alpha^m} \right) Q_{i-m} \] (10.25)

The result is in agreement with (10.4). It concludes the derivation.

If $m = n$ then (10.25) reduces to:
\[ \hat{Q}_{i+n} = (1 + \alpha^n) Q_i - (\alpha^n) Q_{i-n} \] (10.26)

Therefore:
\[ \alpha_{m,n} = \alpha^n \] (10.27)

With equation (10.27) the coefficients found for the 4-h forecast can be transposed to a 1 h forecast, or to a 6 or 12 h forecast. That way of model identification will be called the indirect method.

**Application**

For the Semois at Haulmé the following model was calculated with the help of the SPSS-X-routine ARIMA:
\[ \hat{Q}_{i,4} = 1.88 Q_i - 0.88 Q_{i-4} \quad s^2_{4,4} = 18.5 \text{ m}^6/\text{s}^2 \] (10.28)

where:
\[ s^2_{m,n} = \text{calculated variance of the forecast error if } Q_{i+n} \text{ is forecast, using } Q_i \text{ and } Q_{i-m} \] [m$^6$/s$^2$]

Using (10.24) and (10.27) yields (indirect method):
\[ \alpha = 0.969 \quad s^2_{1,1} = 0.0250 \quad s^2_{4,4} = 0.461 \text{ m}^6/\text{s}^2 \] (10.29)

The model for $m = 6$ and $n = 6$ can be derived, again using (10.24) and (10.27):
\[
\hat{Q}_{i,6} = 1.83 Q_i - 0.83 Q_{i-6} \quad s_{6,6}^2 = 57.8 \text{m}^3/\text{s}^2 \quad (10.30)
\]

and for \( m = 12 \) and \( n = 12 \):

\[
\hat{Q}_{i,12} = 1.69 Q_i - 0.69 Q_{i-12} \quad s_{12,12}^2 = 382 \text{m}^3/\text{s}^2 \quad (10.31)
\]

The results can be compared with calibration results of the original data set. For that purpose not each fourth, but each sixth, respectively each twelfth hourly discharge has been selected. The method will be called the direct method. The results of the direct method are:

\[
\hat{Q}_{i,6} = 1.84 Q_i - 0.84 Q_{i-6} \quad s_{6,6}^2 = 53.3 \text{m}^3/\text{s}^2 \quad (10.32)
\]

\[
\hat{Q}_{i,12} = 1.72 Q_i - 0.72 Q_{i-12} \quad s_{12,12}^2 = 324 \text{m}^3/\text{s}^2 \quad (10.33)
\]

The equations show that the direct and the indirect method result in almost the same model. The small differences between the indirect and the direct determination are considered to be acceptable, as the data set used is substantially different, because of the different time steps. Apparently, the assumption (10.6) is not violated seriously.

### TABLE 10.2 COEFFICIENTS \( \alpha_{m,n} \) FOR THE SEMOIS BASIN

<table>
<thead>
<tr>
<th>( n )</th>
<th>( m )</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>0.969</td>
<td>0.48</td>
<td>0.31</td>
<td>0.23</td>
<td>0.18</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>1.91</td>
<td>0.94</td>
<td>0.62</td>
<td>0.45</td>
<td>0.36</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>2.82</td>
<td>1.39</td>
<td>0.91</td>
<td>0.67</td>
<td>0.53</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>3.70</td>
<td>1.82</td>
<td>1.19</td>
<td>0.88</td>
<td>0.69</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>4.55</td>
<td>2.24</td>
<td>1.47</td>
<td>1.09</td>
<td>0.85</td>
</tr>
</tbody>
</table>

### TABLE 10.3 RATIOS \( s_{n,n}^2/s_{1,1}^2 \) FOR THE SEMOIS BASIN

<table>
<thead>
<tr>
<th>( n )</th>
<th>( m )</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>1</td>
<td>1.25</td>
<td>1.54</td>
<td>1.82</td>
<td>2.13</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>4.88</td>
<td>5.81</td>
<td>6.97</td>
<td>8.12</td>
<td>9.30</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>13.3</td>
<td>15.4</td>
<td>17.9</td>
<td>20.4</td>
<td>22.9</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>27.9</td>
<td>35.8</td>
<td>35.8</td>
<td>40.1</td>
<td>44.5</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>50.0</td>
<td>55.4</td>
<td>61.8</td>
<td>68.4</td>
<td>75.2</td>
</tr>
</tbody>
</table>
Figure 10.1 An example of a forecast based on equations (10.24) and (10.25)

For elucidating purposes the coefficients $\alpha_{m,n}$ have been calculated for the Semois basin (table 10.2). The ratios $s_{m,n}^2 / s_{1,1}^2$ are shown in table 10.3. Both tables apply to the indirect method.

It is shown that the accuracy is predominantly determined by the lead time $n$. If $n > m$ the coefficient $\alpha_{m,n}$ is great. That might lead to erroneous results, the more so because measuring errors (for example measurements in units of centimetres) are not included explicitly. Therefore very small values of $m$ in relation to $n$ (for example $m = 1$ and $n = 5$) should be avoided.

An example of the calculation of the forecast is given in fig. 10.1. In the example $\alpha = 0.969$, $\sigma_{1,1} = 0.68$, $Q_{1,m} = 100$, $Q_{i} = 120$, $i = 12$ and $m = 8$. If the lead time approaches infinity, the discharge forecast will approach a constant value. That is considered to be superior to linear extrapolation, where extremely high or even negative values may be obtained. Of course, the forecast is not developed on behalf of such lead times.

For the stations of Montcy-Notre-Dame (Meuse), Treignes (Viroin) and Chooz (Meuse) the same kind of calculations have been made as for Haulmé. Calibration results are summarised in tables 10.4 to 10.7. It is clear that both methods for the 4 h forecasting model give exactly the same results.

Table 10.6 shows significantly poorer results for the forecast accuracy for the Viroin. Probably the small reaction time of the catchment makes the catchment less suitable for this kind of modelling. Differences in results between direct and indirect modelling are most significant for Chooz. Probably the method used here is not as suitable for Chooz as it is for other tributaries. Because the method of discharge forecasting for Chooz is only a second choice, and an extra type of model is considered not to be worthwhile for a second choice, it has been maintained.

In general the results are considered to be acceptable. The models derived for the 4 h forecast will be used as a basis for other lead times.
### TABLE 10.4 COMPARISON OF RESULTS THAT HAVE BEEN OBTAINED DIRECTLY AND INDIRECTLY FOR THE SEMOIS AT HAULMÉ

<table>
<thead>
<tr>
<th>Time step (h)</th>
<th>Direct</th>
<th>Indirect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\alpha_{n,n}$</td>
<td>$\sigma_{n,n}^2$</td>
</tr>
<tr>
<td>4</td>
<td>0.88</td>
<td>18.5</td>
</tr>
<tr>
<td>6</td>
<td>0.84</td>
<td>53.3</td>
</tr>
<tr>
<td>12</td>
<td>0.72</td>
<td>324</td>
</tr>
</tbody>
</table>

### TABLE 10.5 COMPARISON OF RESULTS THAT HAVE BEEN OBTAINED DIRECTLY AND INDIRECTLY FOR THE MEUSE AT MONTCY-NOTRE-DAME

<table>
<thead>
<tr>
<th>Time step (h)</th>
<th>Direct</th>
<th>Indirect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\alpha_{n,n}$</td>
<td>$\sigma_{n,n}^2$</td>
</tr>
<tr>
<td>4</td>
<td>0.86</td>
<td>22.7</td>
</tr>
<tr>
<td>6</td>
<td>0.84</td>
<td>55.0</td>
</tr>
<tr>
<td>12</td>
<td>0.75</td>
<td>279</td>
</tr>
</tbody>
</table>

### TABLE 10.6 COMPARISON OF RESULTS THAT HAVE BEEN OBTAINED DIRECTLY AND INDIRECTLY FOR THE VIROIN AT TREIGNES

<table>
<thead>
<tr>
<th>Time step (h)</th>
<th>Direct</th>
<th>Indirect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\alpha_{n,n}$</td>
<td>$\sigma_{n,n}^2$</td>
</tr>
<tr>
<td>4</td>
<td>0.75</td>
<td>35.0</td>
</tr>
<tr>
<td>6</td>
<td>0.65</td>
<td>97.4</td>
</tr>
<tr>
<td>12</td>
<td>0.34</td>
<td>521</td>
</tr>
</tbody>
</table>

### TABLE 10.7 COMPARISON OF RESULTS THAT HAVE BEEN OBTAINED DIRECTLY AND INDIRECTLY FOR THE MEUSE AT CHOOZ

<table>
<thead>
<tr>
<th>Time step (h)</th>
<th>Direct</th>
<th>Indirect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\alpha_{n,n}$</td>
<td>$\sigma_{n,n}^2$</td>
</tr>
<tr>
<td>4</td>
<td>0.73</td>
<td>192</td>
</tr>
<tr>
<td>6</td>
<td>0.71</td>
<td>442</td>
</tr>
<tr>
<td>12</td>
<td>0.67</td>
<td>1657</td>
</tr>
</tbody>
</table>

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10.3 THE BELGIAN TRIBUTARIES

10.3.1 INTRODUCTION

In many flood-forecasting models the hydrologic process is simplified to only a few steps (fig. 10.2). The output of one part of the process is the input of the next. The exception is generally the computation of the baseflow, which has been treated separate from the precipitation. The data available for calibration consist at least of precipitation (or rainfall) and discharge data.

The discharge forecasts of the Belgian tributaries are important for the forecast at Borgharen-Dorp. Therefore it is important that the discharge forecasts are accurate. One way to achieve that accuracy is to simulate the process, another is using advanced stochastic models (for example Hino et al., 1986). The latter type, for example ARMAX (Weeks et al., 1987, Van Breusegem et al., 1987), nearest neighbour (Karlsson et al., 1987) or multiple linear regression models, requires the availability, in real-time, of an almost complete series of discharge data. The availability was not guaranteed at the beginning of this study. Therefore it was decided to try to simulate the precipitation-runoff process (indeed during the January 1991 flood the data were not available, cf. chapter 12).

The diagram of fig. 10.2 also outlines the structure of the first parts of these sections. The precipitation process in reality and the way of modelling it are dealt with in § 10.3.2, the calculation of the effective precipitation in § 10.3.3 and the calculation of the direct runoff in § 10.3.4. The theories and calculations of § 10.3.4 more or less form an entity with § 10.3.5, the calibration, in which also the baseflow is dealt with.

The model presented in § 10.3.2 through 10.3.5 is appropriate if the discharge

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**Figure 10.2 Simplified hydrologic process as it is often used in flood-forecasting**

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information is limited. If an extensive amount of information is available, the quality of the forecast can be improved by the model presented in § 10.3.6. The quality can also be improved by incorporating precipitation forecasts (§ 10.3.7). Ungauged tributaries and wave travel times between station and mouth are dealt with in section 10.3.8.

Available calibration data

Precipitation data

In Belgium there exist two hourly precipitation data networks, which both work in real time. The oldest network belongs to the IRM (Institut Royal Météorologique de Belgique), the other to the Ministère des Travaux Publics (nowadays Ministère Wallonne de l'Équipement et des Transports). Both networks are extensive and may be used in real-time.

The precipitation measurement network of the Ministère des Travaux Publics did not become operational until after the first flood which is used for calibration. In order to have one type of series only the data of the IRM have been used.

From other studies it was clear that the areal precipitation is an important data for rainfall-runoff models, and at the same moment difficult to determine. Therefore it has been decided to use data of as many precipitation stations as possible. The only restriction was that a sufficiently long period of data should be available (and that the station should be situated in or near the Meuse basin). The stations of which hourly precipitation data are used, are given in table 10.8.

It is not easy to estimate the accuracy of the measured values. The method used for the measurements changes per station and in time. No publications concerning accuracy of the Belgian hourly precipitation stations have been found.

It is well known that measurement errors can be subdivided into systematic and random errors (cf. Van der Made, 1987). Errors in precipitation measurements do not fit into that strict subdivision. For example, errors due to a certain wind direction are highly correlated in time, but they cannot be called systematic errors. Flood events often occur as a result of a certain meteorological circulation (chapter 2) with a certain prevailing wind direction. Therefore it is possible that during an event data with significant mean errors are used.

Another suspect situation is the case of precipitation in a solid state. It might take some time for that kind of precipitation to melt. Therefore the precipitation is measured much later than it has fallen. It is not clear to what extent the situation may occur in the Belgian Meuse basin. Therefore the true value of the precipitation is not clear. However, it is possible to compare the data of hourly measurements with those of daily measurements (which are published in Anonymous: Annuaire Hydrologique de Belgique/Hydrologisch Jaarboek van België, various volumes). The data do not agree exactly, as different gauges were used. It was not possible to detect which type of gauge gives the best results.

Apart from precipitation measurements some other meteorological information might also be important for a flood forecasting model, such as (potential) evapotranspiration, snow depth, temperature, etc. Those data will be excluded from the models, because the data requirements should be as small as possible, not all the data are available hourly, and moreover the data are strongly dependent on place.

It should be noted that 6-h and 12-h values of the precipitation can also be obtained from the international meteorological network. That network differs from the network presented in table 10.8.
**TABLE 10.8 HOURLY PRECIPITATION DATA AVAILABLE FOR AND USED IN THIS STUDY**

<table>
<thead>
<tr>
<th>Station</th>
<th>Watershed</th>
<th>Coordinates N. latitude</th>
<th>Coordinates E. longitude</th>
<th>Level [TAW+m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rochefort</td>
<td>Lesse</td>
<td>50°10'34&quot;</td>
<td>5°13'28&quot;</td>
<td>193</td>
</tr>
<tr>
<td>Dourbes</td>
<td>Viroin</td>
<td>50°05'55&quot;</td>
<td>4°35'53&quot;</td>
<td>244</td>
</tr>
<tr>
<td>Nadrin</td>
<td>Ourthe</td>
<td>50°09'05&quot;</td>
<td>5°40'55&quot;</td>
<td>405</td>
</tr>
<tr>
<td>Carlsbourg</td>
<td>Lesse</td>
<td>49°53'45&quot;</td>
<td>5°04'47&quot;</td>
<td>408</td>
</tr>
<tr>
<td>Ernage</td>
<td>Sambre</td>
<td>50°35'0&quot;</td>
<td>4°41'14&quot;</td>
<td>159</td>
</tr>
<tr>
<td>St-Hubert</td>
<td>Lesse</td>
<td>50°02'23&quot;</td>
<td>5°24'06&quot;</td>
<td>556</td>
</tr>
<tr>
<td>Bierset</td>
<td>Meuse</td>
<td>50°38'18&quot;</td>
<td>5°26'56&quot;</td>
<td>191</td>
</tr>
<tr>
<td>Spa(Aerodromme)</td>
<td>Vesdre</td>
<td>50°28'24&quot;</td>
<td>5°54'39&quot;</td>
<td>483</td>
</tr>
<tr>
<td>Florennes</td>
<td>Sambre</td>
<td>50°14'21&quot;</td>
<td>4°39'40&quot;</td>
<td>285</td>
</tr>
<tr>
<td>Gosselies</td>
<td>Sambre</td>
<td>50°27'43&quot;</td>
<td>4°26'45&quot;</td>
<td>187</td>
</tr>
<tr>
<td>Acosse</td>
<td>Mehaigne</td>
<td>50°36'11&quot;</td>
<td>5°03'02&quot;</td>
<td>135</td>
</tr>
<tr>
<td>Cerfontaine</td>
<td>Sambre</td>
<td>50°10'04&quot;</td>
<td>4°24'47&quot;</td>
<td>239</td>
</tr>
<tr>
<td>Daverdisse</td>
<td>Lesse</td>
<td>50°01'03&quot;</td>
<td>5°07'17&quot;</td>
<td>295</td>
</tr>
<tr>
<td>Somme-Leuze</td>
<td>Ourthe</td>
<td>50°20'05&quot;</td>
<td>5°22'03&quot;</td>
<td>200</td>
</tr>
<tr>
<td>Bertogne</td>
<td>Ourthe</td>
<td>50°05'03&quot;</td>
<td>5°40'03&quot;</td>
<td>430</td>
</tr>
<tr>
<td>Boussu-lez-Walcourt</td>
<td>Sambre</td>
<td>50°13'40&quot;</td>
<td>4°22'41&quot;</td>
<td>243</td>
</tr>
</tbody>
</table>

*TAW = Belgian Reference level. The reference level lies 2.33 m below NAP*

**Discharge data**

The Belgian Ministère des Travaux Publics (nowadays Ministère Wallonne de l'Équipement et des Transports) possesses an extensive network of hourly gauging stations. As in France, most discharges are computed using a water level-discharge relation. Data are stored on an hourly basis in a data bank. Data of all the stations are collected centrally in real-time.

For the Sambre the discharges at Namur are calculated by means of two water level measurements and the hydraulic properties of the section between the gauges. Due to influences of the weir, the discharge fluctuates significantly, especially for normal and low discharges. The discharge can only be used for floods.

The number of events that can be used for the calibration is very limited, due to the limited length of the available time series. In principle the data set should be subdivided into a calibration set and a verification set. One condition is that both can be seen as representative of the system under investigation. For the Meuse tributaries it means that flood events with and without snowmelt in various months of the year should be available. For some subcatchments, however, only 4 events were considered to be suitable for calibration and/or verification. In those cases all the events have been used for the calibration. Table 10.9 gives the (output) data used.
<table>
<thead>
<tr>
<th>River and station</th>
<th>Calibration periods</th>
<th>Verification periods</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sambre at Namur</td>
<td>12 - 17 March 1979 12 - 16 December 1979 3 - 8 February 1980 4 - 10 February 1984</td>
<td></td>
</tr>
</tbody>
</table>

### 10.3.2 THE AREAL PRECIPITATION

One of the unavoidable steps in rainfall-runoff modelling seems to be the transformation of precipitation of gauges measured into mean areal precipitation. In many models the classic way is followed: Calculate an areal rainfall from point-measurements. Then the computed areal rainfall is the input in the calculation of the (areal) effective precipitation. Nowadays, due to more modern precipitation measurement equipment and a greater computation capacity it is also possible to have a finer distribution of the precipitation process (both in area and in time).

However, a distributed precipitation model is only useful if the following conditions are met:

- Reliable information is available about the distribution of the precipitation over the catchment area.
- The spatial distribution of precipitation is significantly non-homogeneous. The correctness of the conditions has been shown by Hughes et al. (1989). No condition is given about the physiography of the catchment area.

For the subcatchment areas of the Meuse it is not clear to what extent both conditions are met. With only few precipitation gauging stations, per subcatchment area, available and the main interest in frontal zones precipitation, there is no convincing argument to use a distributed precipitation model. Besides, distribution has already taken place by considering subcatchment areas. Therefore, this study will be restricted to the determination of the mean areal precipitation in a subcatchment area.

The calculation of the mean areal precipitation, further called areal precipitation, should preferably satisfy the following conditions if used for operational use. It should:

a. be quick in operational use
b. be robust, that is: give an answer in any case
c. be accurate
d. be objective, that is: it should not have subjective elements in it
e. have the possibility for automatic adaptation to another set of gauging stations f. give statistical information

The first two conditions are restrictive, the others are not.

After elucidating several methods, it will be investigated which method is most suitable for determining the areal precipitation in relation to the flood forecasting model for the Meuse.

**Methods to calculate the areal precipitation**

**Unweighed Mean**

The areal precipitation is simply the mean of the precipitation measurements in the catchment area:

\[
\bar{P} = \frac{1}{N'} \sum_{j=1}^{N'} P_j
\]  

(10.34)

where:

- \( \bar{P} \) = areal precipitation depth estimated [L]
- \( N' \) = number of precipitation stations in the considered catchment area [-]
- \( P_j \) = precipitation depth in station \( j \) measured [L]

The method can be used in a flat area where the correlation between gauge measurements is high. For the Meuse subcatchment areas, the number of stations is very low (as low as zero, even for larger subcatchment areas).

**Thiessen Polygons**

The method and all the following methods assign a precipitation to any point in the catchment areas. The areal precipitation is calculated by integrating the point values found over the area.

For Thiessen Polygons, the precipitation in a point is thought to be equal to the one of the nearest station. Integration over the area gives an expression of the form:
\[
\bar{P} = \frac{\sum_{j=1}^{N} A_j' P_j}{\sum_{j=1}^{N} A_j'}
\quad (10.35)
\]

where:

- \( N \) = number of precipitation stations \([-]\)
- \( A_j' \) = area in the catchment for which the nearest station is \( j \) \([L^2]\)

Note 1.
Rewriting with:

\[
b_j = \frac{A_j'}{\sum_{j=1}^{N} A_j'}
\quad (10.36)
\]

gives:

\[
\bar{P} = \sum_{j=1}^{N} b_j P_j
\quad (10.37)
\]

where:

\[
\sum_{j=1}^{N} b_j = 1 \quad \text{and} \quad b_j \geq 0 \quad j = 1, \ldots, N
\quad (10.38)
\]

Note 2.
The Thiessen Polygons can be considered as a special case of the Reciprocal Distance Method, in which the precipitation at a point \( i \) is estimated by:

\[
\hat{P}_i = \frac{\sum_{j=1}^{N''} \frac{P_j}{d_{ji}^\omega}}{\sum_{j=1}^{N''} \frac{1}{d_{ji}^\omega}}
\quad (10.39)
\]

where:

- \( \hat{P}_i \) = precipitation depth at point \( i \) estimated \([L]\)
- \( N'' \) = number of nearby gauges that should be considered in the equation \([-]\)
- \( d_{ji} \) = distance between the points \( j \) and point \( i \) \([L]\)
- \( \omega \) = constant \([-]\)

For \( N'' = 1 \), the Thiessen Polygons method is obtained. The method needs the estimation of
two parameters, \( N'' \) and \( \omega \). \( N'' \) is dependent on the surface area, gauge density, spatial variation in precipitation, etcetera. The parameter \( \omega \) may be determined by means of optimisation.

Here, the attention will be restricted to the original Thiessen Polygons method.

**Isohyets**
The precipitation in every point is determined by drawing lines of equal precipitations (isohyets) on a map of the catchment area, the gauging stations and the precipitation measured. Knowledge of the weather type, orographic effects, etcetera as well as experience can be used. A computer model would be complex, handwork requires a lot of time and experience.

**Trend Surface Analysis**
Here an analytical function is applied to calculate the precipitation depth as a function of the location. The form of the function has to be chosen in advance. The expression is as follows:

\[
\hat{P}_j = \sum_{i=1}^{i_m} \beta_i g_{ij}
\]

(10.40)

where:
- \( \hat{P}_j \) = precipitation depth at point \( j \) estimated [L]
- \( \beta_i \) = coefficient [-]
- \( g_{ij} \) = a certain function [L]
- \( i_m \) = number of functions \( g \) to be incorporated in the equation [-]

\( g_{ij} \) can be dependent on place or altitude, for example:

\[
\hat{P}_j = \beta_1 1 + \beta_2 x_j^C + \beta_3 y_j^C
\]

(10.41)

where \((x_j^C, y_j^C)\) are the Cartesian coordinates of point \( j \) and the 1 has the dimension of [L].

By minimizing:

\[
\sum_{j=1}^{N} (P_j - \hat{P}_j)^2
\]

(10.42)

the coefficients \( \beta_i \) can be determined.

The coefficients \( \beta_i \) should be calculated for every time step. For a complex depth function, the minimisation might cause calculating problems. Special precautions have also to be taken to avoid negative precipitation values.

If some suppositions are made in advance, statistical information may be obtained. If \( E(P_j - \hat{P}_j) = 0 \), \( E((P_j - \hat{P}_j)^2) = s^2 \) and \( E((P_{j_1} - \hat{P}_{j_1})(P_{j_2} - \hat{P}_{j_2})) = 0 \) for \( j_1 \neq j_2 \) then \( s^2 \) can be estimated by (Clarke et al., 1972):

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\[ s^2 = \frac{1}{N - i_{\text{max}}} \sum_{j=1}^{N} (P_j - \hat{P}_j)^2 \] (10.43)

**Finite Element Method**

The method starts from the creation of a triangular mesh, with the gauging stations in the angular points (fig. 10.3).

![Triangular mesh diagram](image)

○ gauging station

*Figure 10.3 An example of a triangular mesh*

The information of the precipitation depth in a gauge is used in the adjacent triangles. It is done by connecting the precipitation levels in the corners of the triangle on a plane. For every point in the area it leads to a formulation similar to (10.37), with at most three coefficients unequal to zero. Problems occur if a part of the catchment area is not covered by the mesh. The areal precipitations found are influenced by the subjective choice of the mesh.

**Kriging**

The Kriging interpolation method is able to give statistical information about the reliability of the estimates, like the Trend Surface Analysis. In that context, only a brief review of the method will be given; more detailed and extensive information can be found in Delhomme (1978). It is supposed that no drift is present.

Let the function \( Z \) be an intrinsic random function. It means that the random function \( Z \) must satisfy the intrinsic hypothesis:
\[ E(Z(x + d) - Z(x)) = 0 \]  \hspace{1cm} (10.44)

\[ \text{var}(Z(x + d) - Z(x)) = 2 \gamma(d) \]  \hspace{1cm} (10.45)

where \( x \) denotes a point in the area and \( d \) denotes a distance. The function \( \gamma(d) \) is the semi-variogram.

The best linear unbiased predictor \( \hat{z}(x) \) of \( z(x) \) in which \( z \) is a realisation of \( Z \) is:

\[ \hat{z}(x) = \sum_{i=1}^{N} \beta_{i} z(x_i) \]  \hspace{1cm} (10.46)

For reasons of brevity, the index \( x_i \) will be omitted in \( \beta_i \).

If the semi-variogram \( \gamma(d) \) is known, \( \beta_i, i = 1,...N \) may be calculated by minimising the variance of the prediction error \( \hat{z}(x) - Z(x) \), so by minimising:

\[ \text{var}(\hat{z}(x) - Z(x)) \]  \hspace{1cm} (10.47)

Furthermore, it can be derived that (unbiasedness):

\[ \sum_{i=1}^{N} \beta_i = 1 \]  \hspace{1cm} (10.48)

(10.47) and (10.48) give \( N + 1 \) equations (Delhomme (1978), Bastin et al. (1985)):

\[
\begin{align*}
\sum_{i=1}^{N} \beta_i \gamma(d_{ij}) + \mu &= \gamma(d_{jk}) \quad j = 1,...N \\
\sum_{j=1}^{N} \beta_j &= 1
\end{align*}
\]  \hspace{1cm} (10.49)

where \( d_{ij} \) is the Euclidian distance between \( x_i \) and \( x_j \).

The system of \( N + 1 \) linear equations can be solved to obtain \( N \) weights \( \beta_j \) and the Lagrange multiplier \( \mu \). The Lagrange multiplier is introduced to meet the condition of an unbiased estimator.

The semi-variogram \( \gamma(d) \) may be estimated by:

\[ \gamma(d) = \frac{1}{2N(d)} \sum_{i=1}^{N(d)} (z(x_i + d) - z(x_i))^2 \]  \hspace{1cm} (10.50)

where:

\[ N(d) = \text{number of paired points with a mutual distance } d \]  \hspace{1cm} [-]

For determination purposes, the variogram is only estimated for classes of distances, for example 0-5 km, 5-10 km etcetera.

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In the case of short duration precipitation, many realisations of $Z$ are available. Therefore it is sensible to use statistical information from the previous realisations, instead of determining the semi-variogram for only the current realisation.

The simplest way for incorporating the realisations known, would be to take the mean of the semi-variograms. By doing so, the variation of intensity of the different showers is neglected. Therefore, an adaptation of Kriging is used (climatological Kriging, Bastin et al. (1985), Lebel et al. (1987)), using the mean of $\gamma_i'(d)$, where $\gamma'_i(d)$ is defined as:

$$\gamma'_i(d) = \frac{\gamma_i(d)}{\alpha(t)} \quad (10.51)$$

where:

$$\alpha(t) = \frac{1}{N} \sum_{i=1}^{N} \left( z(x_i,t) - \frac{1}{N} \sum_{i=1}^{N} z(x_i,t) \right)^2 \quad (10.52)$$

and:

$$\gamma_i(d) = \text{variogram at time } t$$

It should be noted that the set $\beta_i, i=1\ldots N$ does not change if a semi-variogram is multiplied by a constant.

The semi-variogram calculated is usually fitted into a semi-variogram with an analytical form, for example:

$$\gamma(d) = 1 - \exp(-\kappa d) \quad (10.53)$$

in which $\kappa$ is a constant. The nugget effect may be incorporated in the equation by means of the Dirac function. The nugget effect describes an initial discontinuity in the semi-variogram ($d = 0$), which is caused by measuring errors and/or by the physical characteristics of the process studied.

Climatological semi-variograms have been calculated for the Meuse basin, using 50 day periods with great depths of precipitation in the winter periods of the years 1979 and 1980. In other words: 50 realisations of $Z$ have been used to find the climatical semi-variogram. It is clear that the semi-variogram function will change if the time step used is changed; the coherence of the precipitation will decrease if the time step is reduced.

The calculation of semi-variograms using smaller time steps encounters the problem of zero-precipitation depths. It is possible that during a period within a wet day no precipitation has occurred in any gauge. Then $\gamma'_i(d)$ cannot be determined. It also applies to time steps in which all the precipitation depths measured are the same.

For the stations in table 10.8 the semi-variogram is shown in fig. 10.4, using equation (10.53), a time step of 24 hours and a class length of 5 km.

Tests have shown that the intrinsic hypothesis, more precisely equation (10.45), was heavily violated. Therefore the results, being the set of weights and the statistical information, are not very reliable.
Figure 10.4 Calculated semi-variogram

Radar
At the time this study was carried out, for the Ardennes no operational system of determining the precipitation by radar was available. The possibility of using radar has nevertheless been included in this section, as technical developments are going fast and the technique may become available within a few years. In the United Kingdom several radars are functioning in real-time, the results are promising also if applied to real-world situations (Collier, 1986a,b, Collier et al., 1986, Hill et al., 1987). De Troch et al. (1990) found that radar pictures can contribute to the accuracy of a flood forecast for the river Meuse.

A radar is an instrument that emits wave pulses, and measures the reflected pulses received. If the wave length is chosen in the right bands, precipitation is one of the substances that will reflect the radar pulse. The intensity of the reflection is a measure of the precipitation intensity, and the travel time of the pulse is a measure of the distance between radar and object.

If it is assumed that the wave length of a radar beam is great with respect to the diameter of the drop, the radar equation is valid (Aniol et al., 1979):

\[
\overline{P}_r = \frac{\pi^3}{1024 \ln 2} \frac{P_r H G^2 \Theta_1 \Theta_2}{\lambda^2 d_o^2} |K|^2 X
\]  

(10.54)

where:
\[
\begin{align*}
\overline{P}_r & = \text{power received} & [W] \\
\overline{P}_r & = \text{power emitted} & [W] \\
H & = \text{pulse length (in the air)} & [m]
\end{align*}
\]

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\[ G = \text{antenna gain} \] [-]
\[ \Theta_1 = \text{horizontal beam width} \] [rad]
\[ \Theta_2 = \text{vertical beam width} \] [rad]
\[ \lambda = \text{wave length} \] [m]
\[ d_{o0} = \text{distance radar-object} \] [m]
\[ |K|^2 = \text{constant (0.93 for rain and usual wave lengths)} \] [-]
\[ X = \text{radar reflection} \] [mm²/m³]

\( \bar{P} \) is measured so that \( X \) is the only unknown variable, which can therefore be determined.

The relation between precipitation intensity and radar reflection is:

\[ X = \sum_i n_i D_i^6 \] (10.55)

where:
\[ n_i = \text{number of drops with a diameter } D_i \text{ per unit of content} \] [m³]
\[ D_i = \text{diameter of the drop considered} \] [mm]

Unfortunately the drop distribution is unknown. Therefore equation (10.55) has only small practical use in the forecasting context. In practice, the following equation is used to calculate the precipitation intensity:

\[ X = a_R p^{b_R} \] (10.56)

where:
\[ p = \text{precipitation intensity} \] [mm/h]
\[ a_R, b_R = \text{constants} \]

For the constants \( a_R \) and \( b_R \) many values are proposed. The values have been determined by comparing the reflection with the point depths of precipitation measured. Values for \( a_R \) and \( b_R \) vary considerably, due to (Collier, 1987):

a. variations in raindrop size or hail, snow or melting snow in the radar beam,
b. changes in the intensity of precipitation, in and below the radar beam,
c. variations in the performance of the radar system,
d. attenuation of the radar signal due to heavy rainfall along the beam and rain on the radome, and
e. ground echoes caused by anomalous propagation.

The use of pictures produced by radar is limited.

Not only drops reflect or absorb the radar beam, but also buildings and hills do so. The Uithof flats in Utrecht, for example, can be recognised on the radar-picture of De Bilt as a small strip without precipitation behind the object. Hills may prevent the screening of the areas behind. To avoid that type of absorption and reflection, a radar is usually placed on a high point in the landscape. For technical reasons, the magnitudes of \( \Theta_1 \), and \( \Theta_2 \) are about 1 degree. To prevent a lower beam from being reflected by the earth (which is worse than
absorption), the centre of the radar beam is directed upward, with an angle of about 1 degree. That means that for long distances, part of the radar beam will be above the parts of the clouds which cause precipitation (or other phenomena causing changes of the precipitation intensity such as orographic effects). Then the expressions normally used are no longer valid. As the distance increases, an increasing fraction of the radar beam will not be reflected by any drop. The effect is enlarged by the curvature of the earth’s surface. Joss et al. (1987) propose a full volume scan (scan in three dimensions) and to determine the rainfall intensity for each pixel by taking the reflection for the lowest visible pixel above the ground.

For those reasons, the distance is limited to 70 km for a quantitative interpretation of the picture, and to 200 km for a qualitative one. A condition is a radar with sufficient power.

An important problem occurs when the radar beam approaches the 0 °C-boundary. In the area below the 0 °C-boundary, the air can be filled with melting snow and hail. The layer, which may be several hundreds metres thick, possesses a great reflecting capacity. Therefore the layer is to be seen on the picture as a bright band. Above the 0 °C-boundary the air is filled with small ice particles, which reflect the radar waves much less than normal water drops.

The determination of the relation between reflection and precipitation is difficult. For summer showers the 0 °C-boundary is much higher above the earth’s surface than in winter. It is one of the reasons why most experiments have been executed in summer. In principle it is possible to avoid the bright band by changing the vertical angle of the radar beam (Collier, 1987) or to use several angles (as in De Bilt).

Precipitation that reaches the ground as snow or hail gives much trouble when using a radar picture to determine the corresponding intensities.

Great gradients of air temperatures in the lower air layers may cause reflections, thus sending the radar beam back to the earth’s surface. Unfortunately, the effects (anomalous propagation) of the ground clutter are similar to those of rain. They cannot be removed by simple subtraction of the echo of a dry day. Maybe the slow variations of the echo of the anomalous propagation (compared to those of rain) give a possibility for a solution.

Equation (10.56) appears to be inaccurate, because the raindrop distribution changes from shower to shower, and even during a shower. To have a simple correction, the so-called assessment factor \( AF \) is commonly used, which is defined as:

\[
AF = \frac{\text{precipitation intensity found by radar}}{\text{precipitation intensity derived from point measurements}} \tag{10.57}
\]

When the numerator and the denominator are known, the assessment factor can be calculated. The factor can be used to calculate precipitation intensities near the precipitation gauge.

In the ideal case, the assessment factor would always be equal to 1. In practice, the values vary between 0.1 and 10. However, it would be wrong to blame only the radar for it, because in the determination of the denominator, too, errors are made.

From the preceding text, it may clear that between the introduction of radar pictures and the successful use of radar in operational hydrology, many obstacles must be overcome. That applies especially to flood-causing situations in the Ardennes.

The introduction of a vertical profile and a volume scan, Doppler radar techniques, polarisation diversity, multi-wavelength techniques possibly from space (Joss et al., 1987) may contribute to the improvement of the radar technique. In the future a powerful tool may become available, which gives distributed and statistical information and can be used in all
places and weather conditions. It may even be used successfully in short-term precipitation forecasting (Browning et al., 1989, Einfalt et al., 1990).

Synthesis

Several methods for the determination of areal precipitation have been discussed. Some of them are more suitable than others. The decision which method should be taken is dependent on the kind of catchment, the type of weather that is important and the available data for calibration and in real-time. Delrieu et al. (1988) concluded that for summer conditions around Montréal (Canada) the methods that take into account the spacial variability give better results than models that do not, at least for daily rainfall depths. Witter (1984) investigated the heterogeneity of the rainfall in the Netherlands. He concluded that the spacial correlation of hourly precipitation depths could not be estimated satisfactorily in the Netherlands. Bastin et al. (1984) demonstrated a successful application of the determination of daily areal rainfall using Kriging in the Semois basin with approximately 17 gauges.

The unweighed mean is an improper method, because the number of gauges in the subcatchment areas is small, if not zero. Therefore the areal precipitation determined by means of that method is inaccurate, if it is determined at all.

The Thiessen method is more accurate, because also gauges nearby the subcatchment area are involved. The set of weights can automatically and rapidly be determined. No statistical information is obtained.

The isohyetal method is not quick in operational use or requires a meteorological computer model. It is therefore inappropriate.

In Trend Surface Analysis, a function type has to be chosen that could represent the precipitation depth function. If the Meuse catchment area is modelled with only one function type, the function type should be intricate and with a great number of degrees of freedom. If the subcatchment areas are modelled separately, the function type may be simpler. However,

<table>
<thead>
<tr>
<th>station</th>
<th>method</th>
<th>unweighed mean</th>
<th>Thiessen Polygons</th>
<th>Finite elements</th>
<th>Kriging</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rochefort</td>
<td>0.250</td>
<td>0.340</td>
<td>0.346</td>
<td>0.229</td>
<td></td>
</tr>
<tr>
<td>Dourbes</td>
<td>0</td>
<td>0</td>
<td>0.054</td>
<td>0.026</td>
<td></td>
</tr>
<tr>
<td>Nadrin</td>
<td>0</td>
<td>0.019</td>
<td>0.027</td>
<td>0.028</td>
<td></td>
</tr>
<tr>
<td>Carlsbourg</td>
<td>0.250</td>
<td>0.094</td>
<td>0.107</td>
<td>0.105</td>
<td></td>
</tr>
<tr>
<td>Ernage</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.017</td>
<td></td>
</tr>
<tr>
<td>St-Hubert</td>
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<td>0.226</td>
<td>0.181</td>
<td>0.182</td>
<td></td>
</tr>
<tr>
<td>Bierset</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.018</td>
<td></td>
</tr>
<tr>
<td>Spa(Aerodromme)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.020</td>
<td></td>
</tr>
<tr>
<td>Florennes</td>
<td>0</td>
<td>0</td>
<td>0.034</td>
<td>0.033</td>
<td></td>
</tr>
<tr>
<td>Gosselies</td>
<td>0</td>
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<td>0</td>
<td>0.015</td>
<td></td>
</tr>
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<td></td>
</tr>
<tr>
<td>Cerfontaine</td>
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<td>0</td>
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</tr>
<tr>
<td>Daverdisse</td>
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<td>0.223</td>
<td></td>
</tr>
<tr>
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<td>0</td>
<td>0.034</td>
<td>0.044</td>
<td></td>
</tr>
<tr>
<td>Bertogne</td>
<td>0</td>
<td>0</td>
<td>0.019</td>
<td>0.026</td>
<td></td>
</tr>
<tr>
<td>Boussu-lez-Walcourt</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.009</td>
<td></td>
</tr>
</tbody>
</table>

168
TABLE 10.11 PERFORMANCE OF CALCULATION METHODS FOR THE DETERMINATION OF AREAL PRECIPITATION IN THE MEUSE CATCHMENT AREA

<table>
<thead>
<tr>
<th>Method</th>
<th>Unweighted Mean</th>
<th>Thiessen Polygons</th>
<th>Isohyets</th>
<th>Trend Surface Analysis</th>
<th>Finite Elements</th>
<th>Kriging</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quick in operational use</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Robust</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>0</td>
<td>+</td>
</tr>
<tr>
<td>Accurate</td>
<td>-</td>
<td>0</td>
<td>+</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Objective</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>+</td>
</tr>
<tr>
<td>Automatic adaptation to another set of stations</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>+</td>
</tr>
<tr>
<td>Statistical information</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

+ = good; 0 = neutral; - = bad

scores are presented on a relative scale

It is not clear in advance to what extent gauges outside the subcatchment area should be included in the function determination. Special attention should be paid to preventing negative precipitation values in the area studied. An advantage of the method is the fact that reliable statistical information can be obtained, if the required assumptions are valid.

The finite element method has a subjective element because of choosing the triangle network. The method might seem to be similar to the Thiessen Polygon method, however it is advantageous for very sparse networks. It is caused by the fact that more stations are involved in the precipitation determination and moreover the influence of the random nugget effect on the calculated area precipitation will be smaller than using Thiessen’s method.

The Kriging method minimises the variance of the residuals when determining the set of weights. As the intrinsic hypothesis is not fulfilled, the correctness of the values found might be questioned. Sets of weights change when the station set or the time step is changed.

The computation of a new set of weights takes more time than when using the Thiessen Polygon method.

Radar may become a useful tool in determining the distributed precipitation and in short-term forecasting. As in the catchment considered no high quality radar information is available the method is impossible, at least for this study.

As an example the weights for the Lesse subcatchment area are calculated for the methods which use a fixed set of weights (table 10.10).

In Table 10.11 a survey of the performance of the methods discussed in the various criteria is given. The radar has been omitted from the table, as results are dependent on the exact way of determining the precipitation. It is recognised that the scores are subject to personal judgement.

The quickness and robustness are restrictive for an operational forecasting model. The accuracy is important, whereas objectiveness, the automatic adaptation to another set of stations and the possibility for obtaining statistical information are desirable.

The table shows that the more sophisticated methods add little to the determination of the areal precipitation. Some of them have the advantage of giving statistical information. That advantage is achieved after making suppositions that may be doubtful. Therefore, it is chosen to abstain from statistical information. Then the Thiessen Polygons method provides the overall best alternative.

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The Thiessen Polygon methods will be used for the estimation of the areal precipitation. The sets of weights for the subcatchment areas can be found in table 10.12. The accuracy of the method depends on the number of stations involved. That is an important consideration when determining the operational network.

**TABLE 10.12 THIessen WEIGHTS FOR THE SUBCATCHMENT AREAS**

<table>
<thead>
<tr>
<th>subcatchment</th>
<th>Lesse</th>
<th>Sambre</th>
<th>Mehaigne</th>
<th>Vesdre</th>
<th>Amblève</th>
<th>Ourthe</th>
</tr>
</thead>
<tbody>
<tr>
<td>station</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rochefort</td>
<td>0.340</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.016</td>
</tr>
<tr>
<td>Dourbes</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Nadrin</td>
<td>0.019</td>
<td>0</td>
<td>0</td>
<td>0.143</td>
<td>0</td>
<td>0.344</td>
</tr>
<tr>
<td>Carlsbourg</td>
<td>0.094</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Ernage</td>
<td>0</td>
<td>0.096</td>
<td>0.214</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>St-Hubert</td>
<td>0.226</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.078</td>
</tr>
<tr>
<td>Bierset</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Spa(Aerodrome)</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.857</td>
<td>1.000</td>
<td>0.016</td>
</tr>
<tr>
<td>Florennes</td>
<td>0</td>
<td>0.191</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Gosselies</td>
<td>0</td>
<td>0.078</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Acoosse</td>
<td>0</td>
<td>0</td>
<td>0.786</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Cerfontaine</td>
<td>0</td>
<td>0.217</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Daverdisse</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Somme-Leuze</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.344</td>
</tr>
<tr>
<td>Bertogne</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.202</td>
</tr>
<tr>
<td>Boussu-lez-Walcourt</td>
<td>0</td>
<td>0.418</td>
<td>0</td>
<td>0</td>
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<td>0</td>
</tr>
</tbody>
</table>

**10.3.3 THE EFFECTIVE PRECIPITATION**

Only a part of the precipitation contributes to the direct runoff. That part, called the effective precipitation, is predominantly determined by the present and past precipitation. This section describes a suitable model for the determination of the effective precipitation for the tributaries discharging into the Belgian Meuse.

The section starts with the introduction of some definitions. Then some existing models will be shown, and their strong and weak points. During an analysis a system of three equations will be derived. The equations themselves and the determination of the parameters are described in the last part of the section.

**Definitions**

In this study it has been attempted to use the definitions of hydrologic terms as used in Hooghart (ed., 1986). The definitions used there for precipitation and its derived terms are not practical for the theory presented below and are also inconsistent. Therefore, alternative definitions are introduced:
- The precipitation intensity is the volume of water coming down from the atmosphere per unit of horizontal surface per unit of time. Hence its dimension is \([LT^{-1}]\), its symbol will be \(p\). It includes the intercepted water.
The effective precipitation intensity is that part of the precipitation intensity that contributes to the direct runoff. Its symbol is \( P_{\text{ef}} \), its dimension [LT\(^{-1}\)].

The precipitation or precipitation depth is the precipitation intensity function integrated with respect to time. Its symbol is \( P \), its dimension [L].

The loss intensity is the difference between the precipitation intensity and the effective precipitation intensity. Its symbol is \( f \), its dimension [LT\(^{-1}\)].

By definition:

\[
p(t) = f(t) + P_{\text{ef}}(t) \tag{10.58}
\]

where:

\[
\begin{align*}
p(t) & = \text{precipitation intensity} \\
f(t) & = \text{loss intensity} \\
P_{\text{ef}}(t) & = \text{effective precipitation intensity}
\end{align*}
\]

In this section the term rainfall will not be used: it is replaced by the term precipitation. It also applies to the existing models which have been developed for rainfall only.

**Existing models**

**The \( \phi \)-index**

A very simple model is the \( \phi \)-index (used for example by Williams et al., 1983 and Weeks et al., 1987):

\[
f(t) = \min(\phi, p(t)) \tag{10.59}
\]

where:

\[
\phi = \text{positive constant} \quad [\text{LT}^{-1}]
\]

The model does not recognise the dependence of the loss intensity on the preceding precipitation intensity, nor the spatial variability.

**The ratio model**

As an alternative the ratio between effective precipitation intensity and precipitation intensity can be considered to be constant:

\[
P_{\text{ef}}(t) = \psi p(t) \tag{10.60}
\]

where:

\[
\psi = \text{factor} \quad [-]
\]

It leads to:

\[
f(t) = (1 - \psi) p(t) \tag{10.61}
\]

The model will be called the ratio model. The strong point of the model is that it recognises that there is a direct runoff before the total loss intensity available is consumed if the precipi-
tation intensity is small. From a theoretical point of view, however, the fact that there is no upper limit to the loss intensity is a weak point, although very great intensities are rare. Like the previous model, the model does not account for the influence of the preceding precipitation intensities. The incorporation of the spatial variability in the model will be discussed later on in this section.

It should be noted that the $\psi$-factor can be made dependent on the month and the accumulated precipitation depth (Plate et al., 1988).

The $\phi$-index and the ratio model can be combined, by determining the effective precipitation intensity by subtracting $\phi$ from the precipitation intensity prior to applying a proportional loss (Bates et al., 1988).

Klatt et al. (1983) use a time dependent $\psi$-factor.

The Horton infiltration model
A model that does incorporate the preceding precipitation is the well-known Horton infiltration model:

$$f(t) = f_{\min} + (f_d - f_{\min}) \exp\left(-\frac{t-t_d}{k}\right)$$  \hspace{1cm} (10.62)

where:

\begin{align*}
  f_{\min} & = \text{minimum loss intensity} \quad \text{[LT$^{-1}$]} \\
f_d & = \text{loss intensity at } t_d \quad \text{[LT$^{-1}$]} \\
t_d & = \text{time at which the precipitation begins} \quad \text{[T]} \\
k & = \text{decay constant} \quad \text{[T]}
\end{align*}

If the precipitation intensity is less than the loss intensity calculated, Horton's equation is no longer valid. The equation can be adapted to that situation; two adjustments are necessary.

The first adjustment is that the loss intensity should be limited to the precipitation intensity. For that purpose a new term is introduced, the potential loss intensity. The potential loss intensity is defined as the maximum loss intensity that may occur at a moment when precipitation is abundant. The potential loss intensity is a variable dependent on the moisture conditions and is therefore dependent on time. Equation (10.62) is changed into the system:

$$f(t) = \min\{p(t) f_{\text{pot}}(t)\}$$ \hspace{1cm} (10.63)

$$f_{\text{pot}}(t) = f_{\text{min}} + (f_d - f_{\text{min}}) \exp\left(-\frac{t-t_d}{k}\right)$$ \hspace{1cm} (10.64)

where:

\begin{align*}
f_{\text{pot}}(t) & = \text{potential loss intensity} \quad \text{[LT$^{-1}$]} \\
f_{\text{min}} & = \text{minimum potential loss intensity} \quad \text{[LT$^{-1}$]} \\
f_d & = \text{potential loss intensity at } t_d \quad \text{[LT$^{-1}$]}
\end{align*}
Because the left hand term of (10.64) is a potential loss intensity, the loss intensity functions in the right hand term of (10.64) are also changed into potential loss intensity functions.

A second adjustment is necessary because the reduction of the potential loss intensity will be delayed if the infiltration intensity is less than when the precipitation is abundant. Several ways can be followed to model the adjustment. One way to deal with the problem is to assume that the loss intensity is dependent only on the total depth of infiltrated water. At the time \( t \) the amount is equal to:

\[
F(t) = \int_{t_d}^{t} f(\tau) \, d\tau
\]

(10.65)

where:

\[
F(t) = \text{total depth of infiltrated water} \quad [\text{L}]
\]

The corresponding \( t' \), for which the same loss depth is obtained if the precipitation intensity is greater than the \( f \) calculated using equation (10.64), can be found by:

\[
F_1(t') = \int_{t_d}^{t'} f_{\min} + (f_d - f_{\min}) \exp\left(-\frac{\tau - t_d}{k}\right) \, d\tau
\]

(10.66)

where:

\[
t' = \text{time at which the same loss depth is obtained as at } t \text{ if the precipitation intensity is greater than the } f \text{ calculated using equation (10.64)} \quad [\text{T}]
\]

\[
F_1(t') = \text{loss depth if the precipitation intensity is greater than the } f \text{ calculated using equation (10.64)} \quad [\text{L}]
\]

If depths \( F(t) \) and \( F_1(t') \) are equal then:

\[
\int_{t_d}^{t} f(\tau) \, d\tau = \int_{t_d}^{t'} f_{\min} + (f_d - f_{\min}) \exp\left(-\frac{\tau - t_d}{k}\right) \, d\tau
\]

(10.67)

from which \( t' \) can be derived numerically. Then at a moment \( t \) the loss intensity is equal to:

\[
f(t) = \min\left(p(t) f_{\min} + (f_d - f_{\min}) \exp\left(-\frac{t' - t_d}{k}\right)\right)
\]

(10.68)

By means of equations (10.67) and (10.68) the loss model is completely defined. The principle of the adaptation has been formulated in different ways among others by Verma (1982) and Green (1986).

Although a certain dependence on earlier precipitation is present, it cannot be considered completely satisfying, because recovery of the loss capacity is not included. The subdivision into rainy periods and therefore the determination of new values of \( f_d \) and \( t_d \) is not clear for frontal zone precipitation, because periods of heavy and light rain alternate. Also
in the Horton model no spatial variability is incorporated.

To overcome the first drawback Foroud et al. (1981) propose a constant rate of increase of the potential loss intensity if no rain is apparent.

**Snyder model**

Snyder (1971a,b) developed a loss function which he called a watershed retention function. The basis is the Horton infiltration function, here written in a differential equation. Because it is meant as an areal process, tildes are added:

\[
\frac{d\tilde{f}_{pot}(t)}{dt} = - \frac{(\tilde{f}_{pot}(t) - \tilde{f}_{min})}{k_s(t)}
\]  
(10.69)

where:

\( \tilde{f}_{pot}(t) \) = areal mean potential loss intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

\( \tilde{f}_{min}(t) \) = minimum value of the areal mean potential loss intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

\( k_s(t) \) = decay constant for the Snyder function \hspace{1cm} \text{[T]}

Also for the Snyder function the loss intensity should be less than the precipitation intensity:

\[
\tilde{f}(t) = \min(\tilde{\beta}(t)\tilde{f}_{pot}(t))
\]  
(10.70)

where:

\( \tilde{f}(t) \) = areal mean loss intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

\( \tilde{\beta}(t) \) = areal mean precipitation intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

The function \( \tilde{f}_{pot} \) is continuous in time. In the Snyder function the fact is special that the decay constant \( k_s \) is dependent on the precipitation intensity as well as on some other variables. The sign of \( k_s \) may change dependent on the rainfall intensity, and therefore the value of \( \tilde{f}_{pot} \) is able to decrease and increase in time. Therefore \( k_s \) is dependent on time.

The previous theory is interesting, as Snyder developed a function that is based on the Horton infiltration function, but has incorporated the recovery of the potential loss. In the author’s view the expression of \( k_s \) is not easy to interpret physically:

\[
k_s(t) = \frac{1}{a_s + b_s\tilde{f}_{pot}(t)} \left( \frac{\tilde{\beta}(t) + \tilde{f}_{max} - \tilde{f}_{min}}{\tilde{\beta}(t) + \tilde{f}_{max} - \tilde{f}_{pot}(t)} \left( \frac{\tilde{\beta}(t) + \tilde{f}_{min}}{\tilde{\beta}(t) - \tilde{f}_{min}} \right) \right)
\]  
(10.71)

where:

\( \tilde{f}_{max} \) = maximum value of the mean areal potential loss intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

\( \tilde{f}_{min} \) = minimum value of the mean areal potential loss intensity \hspace{1cm} \text{[LT\(^{-1}\)]}

\( a_s \) = constant \hspace{1cm} \text{[T\(^{-1}\)]}

\( b_s \) = constant \hspace{1cm} \text{[L\(^{-1}\)]}
Snyder states that other functions may be as appropriate. Because \( k_s \) is dependent on time, as is \( f_{pot} \), the original Horton infiltration function is only obtained from the Snyder function for a very specific course of \( \beta(t) \) in time.

Snyder (1971b) argues that the equation is a macro-scale approach, but does not make clear why. The macro-scale approach can be doubted, because all the water above a certain threshold value will discharge directly (equation (10.70)). If any spatial variability is present, threshold values will vary over the catchment and therefore one threshold value cannot be determined for the whole catchment area considered.

The Stanford Watershed Model
The \( \phi \)-index model, the Horton model and the Snyder model assume that all the precipitation intensity above a certain threshold intensity contributes to the direct runoff. If there is any spatial variability in the parameters determining the loss, the assumption is incorrect if the area considered is treated lumped.

A very simple way of introducing spatial variability is the ratio model. Strictly speaking, it may stated that for \( \psi \times 100\% \) of the area all the precipitation contributes to the direct runoff, and that \( (1 - \psi) \times 100\% \) of the area does not discharge directly.

A greater resemblance to reality is given by the infiltration function of the Stanford Watershed Model, where the potential loss intensity is dependent on place. The relation between the potential loss intensity and the area is linear, and passes through the origin (fig. 10.5). It is the simplest possible potential loss intensity function that incorporates the spatial variability, except from a constant.

If \( x_A \) denotes a surface unit of the total area \( A \) (fig. 10.5), the potential loss intensity at \( x_A \) is equal to:

\[
f_{pot x_A}(t) = \frac{x_A f_m(t)}{A}
\]  

(10.72)

where:

\( x_A \) = unit area in the catchment area (fig. 10.5) [L^2]

\( A \) = catchment area [L^2]

\( f_m(t) \) = maximum point infiltration loss intensity [LT^{-1}]

\( f_{pot x_A}(t) \) = potential loss intensity for unit area \( x_A \) [LT^{-1}]

An important unit area for the calculation is \( x_0(t) \), where the precipitation intensity is equal to the threshold value \( f_m(t) \):
Figure 10.5 The potential loss intensity as a function of place

\[ x_0(t) = \frac{\bar{p}(t) A}{f_m(t)} \]  \hspace{1cm} (10.73)

If the precipitation intensity \( \bar{p}(t) \) is greater than or equal to \( f_m(t) \) then the mean areal loss intensity is:

\[ \bar{f}(t) = \frac{1}{A} \int_{0}^{A} f_{pot}(x_A, t) \, dx_A \]
\[ = \frac{1}{A} \int_{0}^{A} x_A f_m(t) \, dx_A \]
\[ = \frac{1}{2} f_m(t) \] \hspace{1cm} (10.74)

If the precipitation intensity \( \bar{p}(t) \) is less than \( f_m(t) \):
\[ \tilde{f}(t) = \frac{1}{A} \int_0^A \min \left( \frac{x}{A}, f_{\text{por}}(x, \lambda) \right) dx \]

\[ = \frac{1}{A} \left( \int_0^{x_0(t)} \frac{x}{A} f_m(t) dx + \int_{x_0(t)}^A \tilde{p}(t) dx \right) \]

\[ = \frac{1}{A} \left( \int_0^{x_0(t)} \frac{1}{2} x^2 f_m(t) dx + \int_{x_0(t)}^A \tilde{p}(t) x dx \right) \]

\[ = \frac{1}{A} \left( \frac{1}{2} x_0^2(t) \frac{f_m(t)}{A} + \tilde{p}(t)(A - x_0(t)) \right) \]

\[ = \frac{1}{A} \left( \frac{1}{2} \tilde{p}^2(t) A \frac{f_m(t)}{A^2} + \tilde{p}(t) \left( A - \frac{\tilde{p}(t)}{f_m(t)} A \right) \right) \]

\[ = \frac{1}{2} \frac{\tilde{p}^2(t)}{f_m(t)} + \left( f_m(t) - \tilde{p}(t) \right) \frac{\tilde{p}(t)}{f_m(t)} \]

Combining (10.74) and (10.75) yields:

\[ \tilde{f}(t) = \min \left( \frac{1}{2} f_m(t), \frac{1}{2} \tilde{p}^2(t) + \left( f_m(t) - \tilde{p}(t) \right) \frac{\tilde{p}(t)}{f_m(t)} \right) \]

(10.76)

The Stanford Watershed Model shows a more gradual transition of the ratio of the effective precipitation (fig. 10.6).

**Figure 10.6** Loss intensity as a function of precipitation intensity, for the \( \Phi \)-index, the ratio model, the Horton model, the Snyder model and the Stanford Watershed model
The incorporation of the spatial variability makes the Stanford Watershed Model noteworthy. For the computation of the loss more steps are involved, which results in a complex model. For reasons of brevity, the rest of the model is not described here.

Somewhat artificial in the Stanford Watershed Model is the fact that the space-potential infiltration relationship is chosen to be linear, and the origin is a fixed point in the relation.

**Analysis**

For the tributaries considered, real-time input data of a sufficiently high frequency (i.e. once per three hours or more often) are limited to discharge and precipitation data. That fact limits the data available for the determination of the effective precipitation to those kinds of data. As the relation between discharge and effective precipitation is not very simple, it is chosen to start the analysis with the calculation of the effective precipitation using precipitation data only. The information given through measured discharges is not disregarded, but it will be used to improve the discharge forecasts (§ 10.3.6).

A water particle may change its way of discharge in time. An intercepted particle may later fall on the earth's surface and percolate, etcetera. Such changes are most interesting for a micro-scale research. On a catchment basis, such changes are usually neglected. Here such changes will not be incorporated in the model either. Therefore the way of discharge of a water particle is thought to be determined at the moment of its touching the earth's surface or its vegetation.

In principle seasonal influences have to be incorporated in the determination of the effective precipitation. Due to the low number of calibration events, the fact that most floods occur in winter and that the seasonal influence hardly changes during a flood, those influences can hardly be identified. It has been decided to treat the seasonal influences in a different way (§ 10.3.5).

The evapotranspiration is influenced by the day-night rhythm. As evapotranspiration is small in winter, and all the calibration events are in that season, calibration of the parameter does not promise to be successful. Therefore the influence of the day-night rhythm is not modelled explicitly.

As the loss is considered to be only dependent on the precipitation at the moment and in the past, and infiltration is the main cause of loss, only one state variable will be used to indicate the current moisture situation. That state variable is the potential loss intensity. The relation between the loss intensity, the potential loss intensity and the precipitation is described in equation (10.63). The course of the potential loss intensity is continuous in time. If the precipitation intensity is constant in time, the potential loss intensity will tend to an equilibrium value, here called the *equilibrium potential loss intensity*. The equilibrium potential loss intensity is therefore considered to be dependent on the precipitation intensity only.

In fig. 10.7 an example of the course of the potential loss intensity is given, based on heuristic relations. Only for elucidating purposes the precipitation intensity is subdivided into blocks of equal values.
For a proper loss model with only one state variable three functions are necessary:

1. The loss intensity as a function of the potential loss intensity and the precipitation intensity. In the function the effect of spatial variability of the process should also be incorporated.

2. The change of the potential loss intensity as a function of the potential loss intensity and the equilibrium potential loss intensity.

3. The equilibrium potential loss intensity as a function of precipitation intensity.

The three functions form the core of the rest of this section.

It should be noted that the three functions may be reduced, for example by substituting the third function by the second. Because of clarity, that reduction has not been made here.

The three functions can also be written down for the models discussed. For example for the Horton model ((10.63) and (10.64)) the equations are:

1. \[ f(t) = \min(p(t), f_{\text{pot}}(t)) \]  \hspace{1cm} (10.77)

2. \[ f_{\text{pot}}(t) = f_{\text{max}}(t) + (f_{\text{pot}}(t_d) - f_{\text{max}}(t)) \exp\left(-\frac{t-t_d}{k}\right) \]  \hspace{1cm} (10.78)

3. \[ f_{\text{max}}(t) = f_{\text{min}} \]  \hspace{1cm} (10.79)

where:

\[ f_{\text{max}} \] = equilibrium potential loss intensity \hspace{1cm} [LT^{-1}]
For the Horton model the tripartition may appear somewhat artificial, as the third equation is very simple. However, just that third equation makes the Horton model unsuitable for the calculation of the loss intensity during floods, as no increase of the potential loss intensity is possible. Because the infiltration model of Horton is often used, a method of calculation is sought, that is associated with the Horton model, but does not have that disadvantage.

In the following text all three functions will be discussed, in the order of increasing complexity. It means that the discussion will be done in reverse order as presented above. As all three functions are mutually related, it is inevitable to refer forward in the text.

1. The equilibrium potential loss intensity
The equilibrium loss intensity is defined as that potential loss intensity that ultimately will be reached if the precipitation intensity is constant. It is therefore a function of the precipitation intensity only.

The equilibrium potential loss intensity will have a certain fixed maximum value, as the potential loss intensity and the loss intensity are limited due to the limited values of evapotranspiration and infiltration. As evapotranspiration and infiltration also have a certain minimum value, the equilibrium potential loss intensity will also have a certain minimum value.

The maximum value of the equilibrium potential loss intensity is obtained if the precipitation intensity is equal to zero, its minimum if the precipitation intensity tends to infinity. The course of the equilibrium potential loss intensity as a function of precipitation intensity however is bounded by other restrictions as well. Suppose that the expression (10.77) is chosen as the function for the calculation of the loss intensity. Then two considerations are important:

1. If the precipitation intensity is constant during an infinite length of time, the potential loss intensity will be equal to the equilibrium potential loss intensity. From a hydrologic point of view, the loss intensity will be greater (or at least stay equal), as the constant precipitation intensity is greater. If such an expression as (10.77) is valid, the consideration will restrict the kinds of suitable expressions for the equilibrium potential loss intensity.

2. Suppose the precipitation intensity is constant up to \( t_0 \) and from \( t_0 \) onward it is \( p, p \) being greater or smaller than the precipitation intensity before \( t_0 \). Then, again for hydrologic reasons, the loss intensity directly after \( t_0 \) will be greater (or stay equal) as the constant precipitation intensity before \( t_0 \) was smaller, because of the moisture situation of the catchment at \( t_0 \).

Both considerations reduce the number of suitable expressions. Later on reference will be made to the two considerations.

Because of the principle of parsimony, only functions with a minimum number of parameters are investigated. The following expressions are included:
\[ f_s(p(t)) = \begin{cases} f_{\text{max}} & \text{for } p(t) = 0 \\ f_{\text{min}} & \text{for } p(t) > 0 \end{cases} \quad (10.80) \]

b. \[ f_s(p(t)) = f_{\text{min}} + (f_{\text{max}} - f_{\text{min}}) \exp \left( -\frac{p(t)}{p_{\text{ref}}} \right) \quad (10.81) \]

c. \[ f_s(p(t)) = \begin{cases} f_{\text{max}} - \Omega p(t) & \text{for } 0 \leq p(t) \leq \frac{f_{\text{max}} - f_{\text{min}}}{\Omega} \\ f_{\text{min}} & \text{for } p(t) > \frac{f_{\text{max}} - f_{\text{min}}}{\Omega} \end{cases} \quad (10.82) \]

where:

- \( f_{\text{max}} \) = maximum potential loss intensity [LT^{-1}]
- \( f_{\text{min}} \) = minimum potential loss intensity [LT^{-1}]
- \( p_{\text{ref}} \) = reference value of the precipitation intensity (constant) [LT^{-1}]
- \( \Omega \) = reduction parameter (constant) [-]

All four constants depend only on the catchment. In fig. 10.8 the three functions proposed are shown graphically.

The three functions are considered to be representative of all the suitable functions with a small number of parameters.

The function in equation (10.80) is physically not realistic, as the smallest amount of precipitation has a decisive influence on the equilibrium potential loss intensity value, and therefore on the loss intensity. Therefore equation (10.80) is rejected as a possible function.

The equilibrium potential loss intensity function as proposed in equation (10.81) is continuous, and therefore does not have the disadvantage of (10.80). The minimum value of the equilibrium loss intensity can only be approached, which is in contradiction to the statement in the second consideration, if (10.77) is used for the calculation of the loss intensity.

The function proposed in (10.82) is also continuous. If the statement in the second consideration is valid and (10.77) is used for the calculation of the loss intensity, it can be derived that \( \Omega \geq (f_{\text{max}} - f_{\text{min}})/f_{\text{min}} \).

It is relevant to analyse what precipitation intensities (sequences) determine the values of the parameters and how. Periods with no rain, and they also occur during floods, determine \( f_{\text{max}} \), \( f_{\text{min}} \) is predominantly determined by high precipitation intensities.

\( p_{\text{ref}} \) in (10.81) is incorporated on behalf of small precipitation intensities. However, due to the nature of the expression, the value of \( p_{\text{ref}} \) is also determined by high precipitation intensities.
Figure 10.8 The equilibrium potential loss intensity as a function of the precipitation intensity, according to the three functions proposed

Only small precipitation intensities will influence the value of $\Omega$ in equation (10.82), because for high precipitation intensities the second expression is valid, in which $\Omega$ is not incorporated. Therefore $\Omega$ cannot accurately be determined. $\Omega$ will therefore be replaced by $(f_{\text{max}} - f_{\text{min}})/f_{\text{min}}$, thus reducing the number of unknowns by one. The proper expression reads:

\[
\begin{align*}
  f_{\text{c}}(p(t)) &= f_{\text{max}} - \frac{f_{\text{max}} - f_{\text{min}}}{f_{\text{min}}} p(t) \quad \text{for} \quad 0 \leq p(t) < f_{\text{min}} \\
  f_{\text{c}}(p(t)) &= f_{\text{min}} \quad \text{for} \quad p(t) > f_{\text{min}}
\end{align*}
\]

(10.83)

Calibration of the equilibrium potential loss intensity function can only take place if the other functions of the precipitation-runoff process are known. In the tables presented in this section, the model ultimately used has been applied, except for the functions that are being investigated (§ 10.3.5).

The calibration results are presented for the Amblève basin. The variances of the model parameters in table 10.13 have been determined with the SPSS-X-routines NLR and CNLR. The residual variances of the model stand for the complete model, so inclusive of the errors caused by the determination of the mean areal rainfall and the effective precipitation-runoff relation. The model has been determined according to the criteria stated in § 10.3.5. For reasons of completeness the parameters of (10.80) have also been calculated.

It was concluded that (10.81) is not so appropriate as (10.83), because: a. the high
TABLE 10.13 CALCULATED PARAMETERS FOR THE EQUILIBRIUM POTENTIAL LOSS INTENSITY FUNCTIONS FOR THE AMBLÈVE SUBCATCHMENT

<table>
<thead>
<tr>
<th>eq.</th>
<th>$f_{\min}$ [mm/h]</th>
<th>variance [mm²/h²]</th>
<th>$f_{\max}$ [mm/h]</th>
<th>variance [mm²/h²]</th>
<th>$P_{\text{ref}}$ [mm/h]</th>
<th>variance [mm²/h²]</th>
<th>$s_0^2$ model [m⁰/s²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(10.80)</td>
<td>0.541</td>
<td>$3.16 \times 10^{-3}$</td>
<td>6</td>
<td></td>
<td></td>
<td>0.01</td>
<td>29.1</td>
</tr>
<tr>
<td>(10.81)</td>
<td>0.450</td>
<td>$1.03 \times 10^{-2}$</td>
<td>3.28</td>
<td>1.46</td>
<td></td>
<td></td>
<td>29.1</td>
</tr>
<tr>
<td>(10.83)</td>
<td>0.466</td>
<td>$1.60 \times 10^{-3}$</td>
<td>3.19</td>
<td>$1.99 \times 10^{-1}$</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Precipitation intensities have an undesired influence on the value of $P_{\text{ref}}$ and b. the expression (10.81) contradicts the second consideration. Therefore the function of equation (10.83) will be applied.

2. The change of the potential loss intensity in time

If the precipitation intensity is constant in time, the potential loss intensity will tend to a constant value. That fact has a resemblance to the Horton infiltration function, and therefore the choice of the same kind of function is reasonable:

$$f_{\text{pot}}(t) = f_{\infty} + (f_{\text{pot}}(t_0) - f_{\infty}(t)) \exp\left(\frac{t - t_0}{k}\right)$$  \hspace{1cm} (10.84)

The value of the potential loss intensity will tend to $f_{\infty}$ if the precipitation intensity is constant.

As the potential loss intensity is related to the moisture content, it must be continuous in time. To simulate the time-dependence of the process, $f_\infty$ has to be dependent on time; and it is, because it depends on the current precipitation intensity.

There are two important objections to (10.84) as the expression for the potential loss intensity:

1. In reality the recovery of the retention capacity will essentially proceed more slowly than the reduction of the retention capacity due to heavy rainfall: In a short period of time the moisture conditions in an area may evolve from dry to completely wet, the period needed to evolve to the dry situation is much longer. That objection also applies to the Snyder equation (10.69).

Here a simple solution of the problem was chosen, through the introduction of a situation dependent decay constant $k$. If the equilibrium potential loss intensity is greater than or equal to the potential loss intensity, $k$ in (10.84) is replaced by $k_A$, otherwise by $k_B$: 

183
\[
\begin{align*}
\tilde{f}_p(t) &= \tilde{f}_p(t_0) + (\tilde{f}_p(t_0) - \tilde{f}_p(t)) \exp \left( -\frac{t-t_0}{k_A} \right) \quad \text{if} \quad \tilde{f}_p(t) > \tilde{f}_p(t_0) \\
\tilde{f}_p(t) &= \tilde{f}_p(t_0) + (\tilde{f}_p(t_0) - \tilde{f}_p(t)) \exp \left( -\frac{t-t_0}{k_B} \right) \quad \text{if} \quad \tilde{f}_p(t) < \tilde{f}_p(t_0)
\end{align*}
\]

(10.85)

where:
\[ k_A = \text{decay constant for an increasing potential loss intensity} \]  
\[ k_B = \text{decay constant for a decreasing potential loss intensity} \]  

2. The following situation using one decay constant cannot be excluded: Suppose up to time \( t_0 \) there are two identical cases. Then, for case A precipitation falls with a constant intensity \( p \) up to \( t_2 \). In case B there is no precipitation between \( t_0 \) and \( t_1 \), but a constant intensity \( p \) between \( t_1 \) and \( t_2 \) (fig. 10.9). From a physical point of view, the loss depth in case B should be less than or equal to the loss depth in case A, as the infiltration capacity of the ground is not used during the time interval between \( t_0 \) and \( t_1 \). If one decay constant is used, that boundary condition is not fulfilled in all the circumstances. If two decay constants are used, the boundary condition is fulfilled if the relation between the two constants satisfies the following expression:

\[
k_A > \frac{f_{\max} + f_{\min}}{f_{\min}} k_B
\]

(10.86)

The complete derivation of the expression is omitted here. For the derivation the loss depths for both cases are compared. It can be derived that if the loss depth of case B is less than or equal to that of case A, \( t_1 - t_0 \) is infinitesimally small and \( t_2 - t_1 \) is infinitely great, then the boundary condition is fulfilled for all the possible combinations of \( t_0, t_1 \) and \( t_2 \). Furthermore it can be shown that if the boundary condition is satisfied if the potential loss intensity at \( t_0 \) is equal to \( f_{\min} \), the boundary condition is satisfied for any potential loss intensity at \( t_0 \). If the two situations are combined, (10.86) is derived in a few steps. (It should be noted that for the loss intensity the expression (10.77) has been used.)

As an example the calculations made for the Amblève subcatchment are shown. The differences when using one decay constant versus two are shown in table 10.14, although the first function should be rejected for theoretical reasons. The same conditions as mentioned in the determination of the equilibrium potential loss intensity equation are in effect.

The parameters \( k_A \) and \( k_B \) in (10.85) are strongly correlated, which results in high values of their variances. Therefore it is investigated whether the model can also be determined successfully with only one independent decay constant. That investigation is made by changing the condition (10.86) into:
Figure 10.9 The loss depth of case B should be less than or equal to the loss depth of case A

\[ k_A = \frac{f_{\text{max}} + f_{\text{min}}}{f_{\text{min}}} k_B \]  \hspace{1cm} (10.87)

Then the expression (10.85) is adjusted to:

\[
\begin{align*}
\begin{cases}
  f_{\text{pot}}(t) = f_{\text{a}}(t) + (f_{\text{pot}}(t_0) - f_{\text{a}}(t)) \exp\left(-\frac{t-t_0}{k_B} \frac{f_{\text{min}}}{f_{\text{max}} - f_{\text{min}}} \right) & \text{for } f_{\text{a}}(t) > f_{\text{pot}}(t) \\
  f_{\text{pot}}(t) = f_{\text{a}}(t) + (f_{\text{pot}}(t_0) - f_{\text{a}}(t)) \exp\left(-\frac{t-t_0}{k_B} \right) & \text{for } f_{\text{a}}(t) < f_{\text{pot}}(t)
\end{cases}
\end{align*}
\]  \hspace{1cm} (10.88)

Using (10.88) the results of the calibration of the discharges of the Amblève subcatchment are also shown in Table 10.14.

Because the last choice has only one parameter, is physically acceptable and has a low value of the mean square error of the model, that alternative has been chosen.

Note.
In common hydrologic practice precipitation is measured in depths which occurred during a certain time step. If it is assumed that during the considered time step the precipitation intensity is constant and the total period of the time involved is the limit to infinity, it can be proven that the potential loss intensity is a weighted mean of previous values of the equilibrium potential loss intensity:
TABLE 10.14 CALCULATED DECAY CONSTANTS FOR THE POTENTIAL LOSS INTENSITY FUNCTION FOR THE AMBLÈVE SUBCATCHMENT

<table>
<thead>
<tr>
<th>eq.</th>
<th>( k ) [h]</th>
<th>variance ([h^2])</th>
<th>( k_A ) [h]</th>
<th>variance ([h^2])</th>
<th>( k_B ) [h]</th>
<th>variance ([h^2])</th>
<th>( \alpha_0 ) [m^6/s^2]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(10.84)</td>
<td>12.1</td>
<td>5.77</td>
<td>30.3</td>
<td>99.4</td>
<td>5.96</td>
<td>2.18</td>
<td>79.6</td>
</tr>
<tr>
<td>(10.85)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>79.4</td>
</tr>
<tr>
<td>(10.88)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>79.5</td>
</tr>
</tbody>
</table>

\[ f_{pol}(t_j) = \sum_{i=1}^{\infty} \alpha_i f_{\Delta}(t_j, t_{p, i}) \quad (10.89) \]

where:
- \( \alpha_i \) = a constant, dependent on the time discretisation and the values of \( k_A \) and \( k_B \) before \( t_j \) [-]
- \( t_{p, i} \) = moment at which a new value of the precipitation intensity has become valid [T]

Furthermore:

\[ \sum_{i=1}^{\infty} \alpha_i = 1 \quad (10.90) \]

The proofs of (10.89) and (10.90) can be found in appendix 3.

3. The calculation of the loss intensity

For a point process the following equation for the calculation of the loss intensity is conventional:

\[ f(t) = \min(p(t) f_{pol}(t)) \quad (10.91) \]

In combination with (10.58):

\[ p_{eff}(t) = \max(0, p(t) - f_{pol}(t)) \quad (10.92) \]

For the areal process several functions have been investigated. In the end, no function proved to be significantly better than a combination of (10.60) and (10.92):

\[ \bar{p}_{eff}(t) = \psi \max(0, \bar{p}(t) - f_{pol}(t)) \quad (10.93) \]

where:
\[ \hat{p}_{\text{eff}}(t) = \text{mean areal effective precipitation intensity} \quad [\text{LT}^{-1}] \]

and so:

\[ \hat{f}(t) = \hat{p}(t) - \psi \max(0, f_{p_{\text{opt}}}(t) - \hat{p}(t)) \quad (10.94) \]

Several observations can be made:

- If \( \psi = 1 \), the same equation for the loss function of Horton (10.77), the \( \phi \)-index and the Snyder function is obtained.
- For the equation only one parameter has to be determined: the factor \( \psi \).

One of the functions investigated for the calculation of the loss intensity was the following, containing 3 parameters:

\[ p_{\text{eff}}(t) = \psi \max(0, p(t) - f_{p_{\text{opt}}}(t)) + a_1 \max(0, p(t) - f_{p_{\text{opt}}}(t)) - a_2 \quad (10.95) \]

where \( a_1 \) and \( a_2 \) are constants. Because of the coherence between \( \psi \) and \( a_1 \), the accuracy of the estimated values will be relatively poor.

In table 10.15 the results of the models are shown for the Amblève subcatchment. The same conditions as mentioned in the determination of the equilibrium potential loss intensity equation are valid.

<table>
<thead>
<tr>
<th>( s_0 )</th>
<th>( a_1 )</th>
<th>( a_2 )</th>
<th>( \psi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \text{model} )</td>
<td>( \text{variance} )</td>
<td>( \text{variance} )</td>
<td>( \text{variance} )</td>
</tr>
<tr>
<td>( \text{eq.} )</td>
<td>[-]</td>
<td>[-]</td>
<td>[mm/h]</td>
</tr>
<tr>
<td>(10.93)</td>
<td>0.676</td>
<td>2.50 ( 10^{-3} )</td>
<td>0.141</td>
</tr>
<tr>
<td>(10.95)</td>
<td>0.576</td>
<td>2.26 ( 10^{-1} )</td>
<td>2.46 ( 10^{1} )</td>
</tr>
</tbody>
</table>

The calculation with discrete time steps

In the preceding text, expressions for the equilibrium potential loss intensity (10.82), the potential loss intensity (10.88) and the areal loss intensity (10.94) have been determined. The conversion into a system with discrete time steps, which is necessary due to the nature of the available data, will be described below. The discretisation will be made for equidistant time-series only.

The principle of the intensities will be maintained. To obtain the precipitation intensities, the discrete values of precipitation depths will be divided by the time interval in which the depth has occurred. Then the discretised precipitation intensity is defined as:
\[ \tilde{\beta}(t) = \frac{1}{\Delta t} \int_{t_{i}}^{\bar{t}_{i}} \beta(t) dt \]  

(10.96)

Other discretised intensities are defined in a similar way. For reasons of brevity, no extra index has been added to demonstrate the discretised character of the expression.

For the calculation of the loss a first order approach is used, in order to minimise calculation time. It yields the system:

\[ \tilde{\beta}_{\text{eff}}(t) = \psi \max(0, \bar{\beta}(t) - f_{\text{pot}}(t)) \]  

(10.97)

\[ \begin{align*}
    f_{\tilde{\omega}}(t) &= f_{\text{max}} - \Omega \bar{\beta}(t) \quad \text{for} \quad \bar{\beta}(t) \leq \frac{f_{\text{max}} - f_{\text{min}}}{\Omega} \\
    f_{\tilde{\omega}}(t) &= f_{\text{min}} \quad \text{for} \quad \bar{\beta}(t) > \frac{f_{\text{max}} - f_{\text{min}}}{\Omega}
\end{align*} \]  

(10.98)

\[ \begin{align*}
    f_{\text{pot}}(t_{i+1}) &= f_{\tilde{\omega}}(t_{i}) + (f_{\text{pot}}(t_{i}) - f_{\tilde{\omega}}(t_{i})) \exp \left( -\Delta t \frac{f_{\text{min}}}{k_{B} f_{\text{max}} + f_{\text{min}}} \right) \quad \text{for} \quad f_{\tilde{\omega}}(t_{i}) \geq f_{\text{pot}}(t_{i}) \\
    f_{\text{pot}}(t_{i+1}) &= f_{\tilde{\omega}}(t_{i}) + (f_{\text{pot}}(t_{i}) - f_{\tilde{\omega}}(t_{i})) \exp \left( \frac{-\Delta t}{k_{B}} \right) \quad \text{for} \quad f_{\tilde{\omega}}(t_{i}) < f_{\text{pot}}(t_{i})
\end{align*} \]  

(10.99)

The expression (10.97) makes it possible to calculate the areal loss intensity (based on (10.93)), the expression (10.98) to calculate the equilibrium potential loss intensity (based on (10.82), where \( p \) is replaced by \( \bar{\beta} \)) and the expression (10.99) to calculate the potential loss intensity (based on (10.88)). \( f_{\text{pot}} \) is bounded by \( f_{\text{min}} \) and \( f_{\text{max}} \).

The equations are shown in the order of calculation. The steps in the calculation of the effective precipitation are shown graphically in fig. 10.10.

The method presented in this section satisfies the demands of a calculation of the effective precipitation. It uses precipitation data only, it is physically acceptable, there are few parameters to be determined, it incorporates the spatial variability in some way, and it is associated with existing methods.
10.3.4 THE DIRECT RUNOFF

Between the moment when a raindrop hits the ground and the moment it leaves the subcatchment a certain period of time passes. The length of time is dependent on: the place where the raindrop hits the ground, the slope, the vegetation, the depth of water which contributes to the direct runoff, etcetera. However, the most important factor is place. For reasons of simplicity the time-dependent influences of the other factors are usually neglected. Then for each point in the subcatchment considered a travel time to the outlet station can be determined. If furthermore the precipitation is considered to be uniform (with distance), the conditions for using the unit hydrograph are fulfilled. The conditions are, mathematically stated, as follows:

1. The effective precipitation-direct runoff system should be linear, meaning that the system should have:
   a. the property of proportionality, and
   b. the property of superposition.
2. The system is time-invariant.

Conditions implicitly used are:
3. The system is causal (which means that an output value at \( t \) is only determined by input values at and before \( t \)).
4. The input and output are always non-negative.
5. The integrated sums of the input and the output in time are equal for any input.

It should be remarked that in rainfall-runoff modelling the term direct runoff is more conventional than the term direct discharge and is therefore used, although the latter term is more associated with the other sections. Consistently the term rainfall is replaced by precipitation.

The most essential function in the unit hydrograph theory is the instantaneous unit hydrograph
(IUH), which is defined as the hydrograph that would be obtained if a precipitation depth of unity falls during an infinitesimal unit of time. The direct runoff is expressed in terms of specific direct runoff (i.e. [LT\(^{-1}\)]). If the precipitation occurs when \( t = 0 \) then:

\[
q_d(t) = P_0 IUH(t)
\]  
(10.100)

where:
\[
\begin{align*}
P_0 & = \text{unit effective precipitation depth, uniformly distributed over the subcatchment area} \quad [L] \\
IUH(t) & = \text{instantaneous unit hydrograph} \quad [T^{-1}] \\
q_d(t) & = \text{specific direct runoff} \quad [LT^{-1}]
\end{align*}
\]

The direct runoff is obtained if the specific direct runoff is multiplied by the subcatchment area \( A \):

\[
Q_d(t) = A \ q_d(t)
\]  
(10.101)

where:
\[
\begin{align*}
A & = \text{subcatchment area} \quad [L^2] \\
Q_d(t) & = \text{direct runoff} \quad [L^3T^{-1}]
\end{align*}
\]

Due to the properties of the linear time-invariant system the direct runoff caused by \( p_d(t) \) can be calculated by the convolution integral:

\[
q_d(t) = \int_0^t p_d(t-\tau) \ IUH(\tau) \ d\tau
\]  
(10.102)

In the equation it is assumed \( p_d(t) \) is uniformly distributed over the subcatchment area.

The shape of the IUH as a function of time is dependent on catchment properties, such as size, shape, orography, vegetation type, the geomorphologic system, etcetera. Unfortunately, the impact of those properties on the unit hydrograph is hardly known. A method more commonly used to determine the IUH is based on the analysis of the relationship between the effective precipitation and discharge data known. One important supposition has to be made: The course of the IUH is smooth in time. Then the IUH can be represented by a few values, for example one value per time step \( \Delta t \). If it is known or assumed that the time length of the IUH is less than or equal to \( n \Delta t \) then the IUH may be represented by \( n + 1 \) values:

\[
IUH(0), IUH(\Delta t), IUH(2\Delta t), \ldots, IUH(n\Delta t)
\]  
(10.103)

For natural catchments the \( n + 1 \) values show some coherence. For the calibration of a flood forecasting model the choice for \( n \) is limited. If the value \( n \) is too small, the specific shape of the IUH may not be represented well. On the other hand, if \( n \) is too great, the calculating time needed will be (too) long and the risk of modelling a great deal of noise will increase. This may lead to negative values in the IUH (cf. for example Khan, 1989).
Instantaneous unit hydrograph functions are an effective solution for the problem. Those functions have incorporated a hydrologically suitable function: only the maximum value, a shape parameter and/or the moment of peak discharge have to be determined. Once the values of the parameters are known, the IUH is determined completely. Well-known is the Nash-cascade, which is defined as:

\[
IUH(t) = \frac{1}{\Gamma(n_N)} \left( \frac{t}{k_N} \right)^{n_N-1} e^{-\frac{t}{k_N}} \quad \text{for } t \geq 0 \tag{10.104}
\]

\[
IUH(t) = 0 \quad \text{for } t < 0
\]

where:
\[
\begin{align*}
n_N & = \text{constant in the Nash-cascade} \quad [-] \\
k_N & = \text{decay constant in the Nash-cascade} \quad [T] \\
\Gamma(\cdot) & = \text{gamma function} \quad [-]
\end{align*}
\]

The \( N \) subscripts in \( k_N \) and \( n_N \) (the \( N \) stands for Nash) are added to prevent confusion with other \( k \) and \( n \) in this chapter.

For any effective precipitation intensity the specific direct runoff can be calculated by:

\[
q_d(t) = \int_0^t \frac{1}{\Gamma(n_N)} \left( \frac{\tau}{k_N} \right)^{n_N-1} e^{-\frac{\tau}{k_N}} d\tau \quad \tag{10.105}
\]

Only two parameters need be estimated if a Nash-cascade is used: \( n_N \) and \( k_N \).

The determination of the effective precipitation-direct runoff relation should have as few parameters as possible. The Nash-cascade is a method to reduce the number of parameters for the effective precipitation-direct runoff relation to only two. As the method has proven to be successful in normally shaped basins, the method has been applied in this thesis.

Both precipitation data and discharge data are in a discrete form. Therefore equation (10.105) has to be reformulated before it can be used for the calibration of the Meuse tributaries. The derivation of the discrete function is made using (10.101). The effective precipitation intensity uniformly distributed over the area has been replaced by the areal effective precipitation intensity. The middle of the time step considered is regarded to be representative for the IUH during the time step:

\[
Q_d^i(t_j) = A \sum_{i=0}^{i_{\max}} \bar{p}_{d^i}(t_{j-1}) IUH((i+\frac{1}{2})\Delta t) \Delta t
\]

\[
= A \sum_{i=0}^{i_{\max}} \bar{p}_{d^i}(t_{j-1}) \frac{1}{\Gamma(n_N)} \left( \frac{(i+\frac{1}{2})\Delta t}{k_N} \right)^{n_N-1} e^{-\frac{(i+\frac{1}{2})\Delta t}{k_N}} \Delta t
\]

where:
\[ Q_d(t_j) = \text{direct runoff at } t_j \text{ calculated} \]  
\[ \Delta t = \text{time step} \]  
\[ i_{\text{max}} = \text{number of time steps to be considered in the equation} \]  
\[ p_{\text{eff}}(t_{j-1}) = \text{the areal effective precipitation intensity between } t_{j-t-1} \text{ and } t_{j-t} \]  

Equation (10.106) has been used in the model to derive the effective precipitation-direct runoff relation, for all the subcatchments discharging in the Belgian Meuse considered. The way of calibrating \( k_n \) and \( n_n \) and the results can be found in § 10.3.5.

10.3.5 THE CALIBRATION

The precipitation-direct runoff function in the module of the Belgian tributaries is composed of three functions, which are mutually related: the determination of the areal precipitation (§ 10.3.2), the determination of the effective precipitation (§ 10.3.3) and the determination of the effective precipitation-runoff relation (§ 10.3.4). The time series data available for calibration are restricted to two types of data: precipitation and discharge. That means that a separate calibration of the parameters in each of the three functions will be difficult, if not impossible. Besides, influences of baseflow, season, etcetera are not included in the three functions. In this section the calibration procedure will be dealt with, and it will be shown how problems were tackled. It includes practical problems such as an incomplete real-time data set and the limited time available for calculation.

The calibrated function will only need one measured discharge data per subcatchment, and can therefore also be applied to situations where the discharge data set available is small. If however a complete discharge data set is available, better forecasts can be obtained by means of a modification of the forecasts on the basis of discharges measured before. The modification will be elucidated in § 10.3.6.

The package SPSS-X offers a routine for the determination of coefficients for a function to be chosen by the user. The routine, called NLR (Non-Linear Regression), needs input data, output data, the function containing the parameters which have to be determined and a first estimation of the values of the parameters.

For a Belgian tributary the output data consist of a selection of the discharge data available. Only discharge data of periods that may contribute to the accuracy of the flood forecasting model have been used. The periods were determined using precipitation intensities, snow cover heights and, of course, the discharges themselves. The determination was made subjectively and shown in table 10.9. Every output data is considered to be the output of a so-called case. Thus, there are as many cases as there are discharge output data.

The functions for which the parameters should be determined can be given in a fortran-like manner. For this application the hydrologic functions mentioned are combined.

The input data consist of calculated areal precipitation data and of measured discharge data. The areal precipitation data have been determined before composing the input file for SPSS-X, thus reducing the number of input data. In SPSS-X it is necessary for every output data to give all the input data which determine the output data. Only data before the moment of the corresponding data measured are used, as the investigated system is a causal one.

Because for all the output data all the input data that might be used should be listed, the data file used for the SPSS-X-routine is relatively great.

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Practical considerations

- The program NLR was used on a mini-computer (VAX). That combination limited the amount of data incorporated as follows. First, the input was limited to approximately 100 input data per output data. If all the available precipitation data are used, one value per hour, only four days can be covered by the input data. Besides, also discharges should be incorporated in the input. Therefore the time step of the precipitation data used has been increased. For the calibration a time step of 3 hours was used.

- The time needed for the calculation is another serious obstacle for a fast progress of the calibration phase. If all the output data are to be used, the calculation may last many days. Not all the cases have been used, but only one per 3 hours.

The routine NLR minimises a user-defined function of the residuals. As a standard it is the least squares function, but other functions may be used as well.

The problem of parameter optimisation is well known in hydrology, but until now it has not been satisfactorily solved. Several investigators have tried to find the best criterion for the determination of the model parameters (Williams et al., 1983, Wang et al., 1986). The conclusions do not agree with each other. Wang et al. (1986) conclude that the least squares method is appropriate for discrete linear hydrologic input-output models. Other investigators propose other functions and techniques. For example, Bates et al. (1988) investigated the maximum a posteriori procedure, Chapman (1986) analysed entropy and Kuczera (1988) described a way to develop more uncorrelated and normal distributed errors, by defining the error as a result of an ARMA-process with disturbances.

Because there is no convincing reason to reject the simplest criterion, i.e. the least squares method, it has been used. It should be noted that for the calibration of the low flow forecasting model (cf. chapter 13) another criterion has been used.

In the sections 10.3.2 to 10.3.4 the materials for the precipitation-runoff module have been dealt with. Three functions have been determined:

The first function determines the areal precipitation (§ 10.3.2). A model for the areal precipitation has been adopted that is based on precipitation data only. The function does not contain parameters that have to be calibrated.

The second function determines the effective precipitation as a function of the areal precipitation (§ 10.3.3). The model ultimately chosen contains three expressions and has 4 parameters that have to be calibrated: The maximum potential loss intensity, the minimum potential loss intensity, the decay constant $k_b$ and the factor $\psi$.

The third function shows the relation between the effective precipitation and the direct runoff (§ 10.3.4). It has two parameters, $k_N$ and $n_N$.

The output of the combined functions is the calculated direct runoff. The calculated discharge is found through the addition of the baseflow:

$$Q_{r;i}^* = Q_{d;r;i}^* + Q_{b;r;i}^* \quad (10.107)$$

where:

- $Q_{r;i}^*$ = calculated discharge for event $r$ at time serial number $i$ [LT$^3$]
- $Q_{d;r;i}^*$ = calculated direct runoff for event $r$ at time serial number $i$ [LT$^3$]
- $Q_{b;r;i}^*$ = calculated baseflow for event $r$ at time serial number $i$ [LT$^3$]

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in which the event \( r \) stands for the sequence number of the flood event, and \( i \) for the serial number of the time of a discharge within the flood event.

One of the subjects not incorporated in the model so far is the baseflow. Commonly the baseflow is considered to be equal to the minimum discharge value in the recent past. Presented in a discrete notation:

\[
Q^*_{b, r; t} = \min(Q_{r; i}, Q_{r; i-1}, Q_{r; i-2}, \ldots, Q_{r; i_d; t})
\]

(10.108)

where:

\[
Q_{r; i} = \text{measured discharge for event } r \text{ at time serial number } i \quad \text{[LT}^1]\n\]

\[
i_s(r; i) = \text{first moment for which the discharge should be incorporated in the calculation of the baseflow for event } r \text{ at time serial number } i \quad [-]
\]

\( i_s \) can be chosen as dependent on \( r \) only (a fixed moment) or only on \( i \) (a fixed time length), or on a combination which may be dependent on the discharge as well.

The greatest advantage of the model is its simplicity. In real-time \( i_s \) may simply be chosen as the first moment for which a complete series of discharge data is regularly available. If however a non-interrupted record is available (for example for longer than a year) choices have to be made. The choice is not obvious for the Belgian tributaries, as a flood is regularly preceded by a rainy period that may have lasted for several weeks. Then the discharge may have an increasing character in time, and the baseflow as well. Then the choice of \( i_s \) and therefore the baseflow will be very subjective, which is considered to be a disadvantage. (The application of an AR-filter on measured discharges in order to determine the baseflow, as described by Hasebe et al. (1989), is impossible here, because of the relatively constantly high direct runoff.)

Another disadvantage of (10.108) is the fact that the discharge calculated (according to (10.107)) cannot be less than the smallest discharge observed.

It is important that during calibration and in real-time the same kind of baseflow determination is used.

In the first instance the following expression was minimised to obtain the parameters required:

\[
\Phi = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} [Q^*_{d, r; i} + Q^*_{b, r; i} - Q_{r; i}]^2
\]

(10.109)

where:

\[
\Phi = \text{the value to be minimised} \quad \text{[L}^6\text{T}^2]\n\]

\[
m(r) = \text{number of discharges observed included in the calibration for event } r \quad [-]\n\]

\[
r_{\text{max}} = \text{number of events included in the calibration} \quad [-]\n\]

The calculations made using the expression gave disappointing results, although several expressions for the baseflow were used and cases with snowmelt and with summer vegetation were excluded. The core of the problem was that too small or too great a discharge for a whole event was forecast (fig. 10.11). The residuals were of a great magnitude. It was
concluded that the criterion presented in expression (10.109) is not appropriate, because among other things the assumption of non-correlated errors is not complied with. This assumption is essential in regression analysis. Therefore another way of calibrating was looked for.

An important data that will be available in real-time and that has not been used yet, is the discharge. Discharges are known up to a certain moment. That certain moment will be called the moment of observation. In the criterion which will be presented, the discharge at the moment of observation will be incorporated.

Before the new criterion is introduced the influences of elements in the hydrologic circle that are not incorporated in the calculation of the direct runoff will be analysed.

If the three functions calibrated are used in combination, in principle the direct runoff can be calculated. However, apart from the baseflow several influences are not included explicitly in the calculation procedure of the runoff:

a. the influence of snowmelt
b. the influence of the baseflow
c. the influence of reservoirs
d. the influence of current meteorological conditions as temperature, sunshine, wind, etcetera, and
e. the influence of the season in relation to vegetation cover, moisture demand, etcetera.

The influences are of a wide variety and for a proper modelling they require a lot of data. However it can be made plausible that the effect of the influences is small or relatively constant:

a. The mean areal snowmelt is one of the phenomena in hydrology that are difficult to determine quantitatively. Several causes may be mentioned: The corresponding water depth of a snow layer is difficult to determine. A simple snow level measurement is insufficient, because the density of the snow can vary between 0.06 and 0.5 (Bodeux, 1971). However, those measurements are often the only ones that are available. For the Meuse catchment snow depths are the only information about snow in real-time (and unfortunately only once a day and in few places).

The snowmelt is influenced by external energy fluxes, such as the air temperature, the wind speed, the way the land is used, the sunshine, the degree of cloudlessness, the temperature of the bottom of the clouds, the temperature of the vegetation over and under the layer of snow, the initial temperature and water equivalent depth of the layer of snow, the rainfall intensity and the rain temperature, and the humidity. Snowmelt models tend to be physically based (Berris, 1987, Bauwens, 1987) and are therefore data demanding to a high degree. Besides, most models are point simulation models.

The determination of the mean areal snowmelt is difficult because of the insufficient amount of data and the spatial variability of the process. However, the variability in time is small as compared to that of rainfall, because that component of the snowmelt process is predominantly influenced by the type of weather. Therefore the contribution of snowmelt to the runoff is relatively constant.

During a flood the snow cover will regularly vanish completely. Due to the processes described before, and the spatial variability of the snow depth before the melt, the vanishing will only cause a gradual decrease in the snowmelt runoff.

b. The baseflow reservoirs will be supplied by rainfall, and therefore the baseflow will increase (cf. chapter 13). However, it is difficult to determine quantitatively the baseflow during a period with direct runoff (Nathan et al., 1990, Chapman, 1991),
because all kinds of (natural) reservoirs are filled, and the discharge and the exchange between reservoirs may be significant. However, the change of the magnitude of the baseflow is much lower than that of the direct runoff. Therefore assuming a constant baseflow during the lead time cannot create serious errors.

It should be noted that in the future the hydrograph separation in a surface runoff and a subsurface runoff will probably be improved, with the help of geochemical and/or isotopic tracers (Wels et al., 1991).

c. Man-made reservoirs in the area covered by the Belgian tributaries only have a small contributing area (cf. chapter 6). Therefore the influence on the natural discharge is small. In appendix 1 it is analysed if and how the reservoirs influence the discharge. There it has been made plausible that if the reservoir operation is known both of the calibration period and in real-time, the best forecast results will be obtained. The best results can also be obtained if the operation is not known, but estimated correctly. However, if the estimations are wrong, the errors may be relatively great. The relatively great errors are avoided if no specific operation is explicitly incorporated (so if the influence of the reservoirs is ignored). Because little, if any, information about reservoir operation is available, that is the choice which has been made.

d. The most important meteorological influence on flood runoff is precipitation. Other influences, such as temperature, air moisture, wind and sunshine, are of relatively minor importance.

e. The influence of the season is important, because the interception, evapotranspiration and soil moisture demand as well as meteorological parameters influence the rainfall-runoff relation. But during the lead time, the influence of the season can be considered to be constant. Unfortunately, it does not result in a relatively constant change of the discharge. Especially the maximum potential loss intensity may be strongly dependent

![Diagram](image-url)

*Figure 10.11 Example of a calibrated series of the Ourthe subcatchment if criterion (10.109) is used*
on the season. However, if a flood occurs the catchment will be wet and then the influence is smoothed. Besides, all the calibration events have taken place in winter conditions, so season dependent parameters can hardly be determined. Therefore the influence is considered to be constant in terms of discharge in the runoff during the lead time.

In the preceding text it has been made plausible that the influences mentioned may be considered to be constant during the lead time. If that hypothesis is true, the influences can be incorporated in the baseflow. A considerable reduction of the number of parameters to be calibrated can be obtained by calibrating the change of the discharge instead of the discharge itself. Then the criterion reads:

$$\Phi = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} \left( \left( Q_{d,r,i}^* + Q_{b,r,i}^* - Q_{r,i} \right) - \left( Q_{d,r,i+h'}^* + Q_{b,r,i+h'}^* - Q_{r,i+h} \right) \right)^2$$ \hspace{1cm} (10.110)\]

where:

\[ h' \quad \text{parameter for the lead time} \quad [-] \]

If \( h' \) is multiplied by \( \Delta t \), the lead time \( h \) is obtained.

Equation (10.110) can be reduced, because the baseflow is assumed to be constant during the lead time:

$$\Phi = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} \left( Q_{d,r,i}^* - Q_{r,i} - Q_{d,r,i+h}^* + Q_{r,i+h} \right)^2$$ \hspace{1cm} (10.111)\]

Now only the direct runoff has to be calibrated.

The parameter \( h' \) is chosen in agreement with a lead time of 6 hours. That is the average lead time for which a forecast is necessary. In real-time use the parameters will not be modified if \( h' \) is changed. It should be noted that the precipitation falling between the moment of observation and \( h' \Delta t \) later should be forecast precipitations, but during the calibration they have been replaced by measured precipitations.

The parameters calibrated are used to calculate the direct runoff. If the lead time is \( h \), the forecast for the lead time is calculated by:

$$Q^*(t+h) = Q_d^*(t+h) - Q_d^*(t) + Q(t)$$ \hspace{1cm} (10.112)\]

where:

\[ h \quad \text{lead time} \quad [T] \]

\[ Q(t) \quad \text{measured discharge at } t \quad [L^3T^{-1}] \]

\[ Q_d(t) \quad \text{calculated direct runoff at } t \quad [L^3T^{-1}] \]

\[ Q^*(t+h) \quad \text{calculated discharge at } t+h \text{ (forecast if } h \geq 0) \quad [L^3T^{-1}] \]

Here the continuous way of notation is used, to emphasise the general character of the
Figure 10.12 Example of a calibrated series of the Amblève subcatchment if the criterion (10.111) is used

This way of modelling has several advantages and disadvantages with respect to the calibration of a simulation model. The advantages are:
- Only few parameters have to be determined.
- Only discharge and precipitation data are required.
- The results of the calibration are more accurate.

The disadvantages are:
- No information will become available about the course of the baseflow, the interception, the snowmelt, etcetera in time.
- If the same number of parameters should be estimated, the time needed for calculation would be approximately doubled, because in one case two discharges have to be calculated, respectively $Q_{d1}$ and $Q_{d1-h}$ instead of only $Q_{d1}$. However, the disadvantage is more than neutralized by the smaller number of parameters, because no parameters for baseflow, etcetera have to be determined.
- At least one discharge is necessary in real-time. Because that data is only used as a constant it can, if unavoidable, be guessed. It will lead to an almost constant error of the discharge forecast at Borgharen-Dorp. That kind of errors will be dealt with in § 11.3.

Because the advantages are important, and the disadvantages are of minor importance or are neutralised, equation (10.111) was used to calibrate the parameters. The calibration results are shown in the tables 10.16 and 10.17. In the latter table also a survey is given of the mean square error of the forecasts of the verification period.

Some of the parameters have a significant coherence with some other parameters. That
<table>
<thead>
<tr>
<th>subcatchment</th>
<th>( f_{\text{min}} ) [mm/h]</th>
<th>variance ([\text{mm}^2/\text{h}^2])</th>
<th>( f_{\text{max}} ) [mm/h]</th>
<th>variance ([\text{mm}^2/\text{h}^2])</th>
<th>( k_B ) [h]</th>
<th>variance ([\text{h}^2])</th>
<th>( \psi ) [-]</th>
<th>variance [-]</th>
<th>( k_N ) [h]</th>
<th>variance ([\text{h}^2])</th>
<th>( n_N ) [-]</th>
<th>variance [-]</th>
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<tbody>
<tr>
<td>Lesse</td>
<td>0.076</td>
<td>(8.07 \times 10^4)</td>
<td>0.87</td>
<td>0.221</td>
<td>18.2</td>
<td>119</td>
<td>0.342</td>
<td>(7.84 \times 10^4)</td>
<td>8.62</td>
<td>0.329</td>
<td>3.77</td>
<td>(2.69 \times 10^2)</td>
</tr>
<tr>
<td>Sambre</td>
<td>0.008</td>
<td>(2.96 \times 10^4)</td>
<td>6</td>
<td>-</td>
<td>8.64</td>
<td>7.45</td>
<td>0.249</td>
<td>(2.12 \times 10^3)</td>
<td>11.1</td>
<td>4.93</td>
<td>2.54</td>
<td>(8.35 \times 10^2)</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>0.159</td>
<td>(1.65 \times 10^3)</td>
<td>2.08</td>
<td>2.06</td>
<td>5.18</td>
<td>11.2</td>
<td>0.484</td>
<td>(7.40 \times 10^3)</td>
<td>26.8</td>
<td>18.1</td>
<td>1.79</td>
<td>(9.41 \times 10^3)</td>
</tr>
<tr>
<td>Ourthe</td>
<td>0.212</td>
<td>(1.49 \times 10^4)</td>
<td>6</td>
<td>-</td>
<td>2.99</td>
<td>0.301</td>
<td>0.608</td>
<td>(1.37 \times 10^3)</td>
<td>15.9</td>
<td>1.11</td>
<td>2.64</td>
<td>(8.95 \times 10^3)</td>
</tr>
<tr>
<td>Amblève</td>
<td>0.466</td>
<td>(1.60 \times 10^3)</td>
<td>3.19</td>
<td>0.199</td>
<td>4.84</td>
<td>0.952</td>
<td>0.676</td>
<td>(2.50 \times 10^3)</td>
<td>7.66</td>
<td>0.360</td>
<td>2.94</td>
<td>(2.13 \times 10^2)</td>
</tr>
<tr>
<td>Vesdre</td>
<td>0.276</td>
<td>(6.50 \times 10^4)</td>
<td>4.95</td>
<td>203</td>
<td>2.02</td>
<td>0.707</td>
<td>0.437</td>
<td>(1.09 \times 10^3)</td>
<td>7.09</td>
<td>0.432</td>
<td>2.63</td>
<td>(2.11 \times 10^2)</td>
</tr>
</tbody>
</table>
applies to $k_N$ and $n_N$, because their combination determines the maximum value of the instantaneous unit hydrograph. Another combination with a significant coherence is $k_R$ and $f_{\text{max}}$. The combination of the two determines the rate of the increase of the infiltration capacity. Because predominantly wet periods are used, the maximum loss intensity is more or less a parameter of an academic nature. For some subcatchments the maximum loss intensity tended to rise to unrealistically high values which will never be reached, because $k_R$ also increased to very high values. In those subcatchments the value of $f_{\text{max}}$ has been set to 6 mm/h. The calibration results for the Lesse are fairly good. The parameter $\Psi$ is relatively small. That is more or less compensated for by the low value of $f_{\text{min}}$. The relatively good results are obtained due to the fact that the reaction time between precipitation and discharge is relatively long, and therefore the changes in the measured discharge are relatively small.

The worst results of the calibration gives the Sambre. That is caused by the fluctuations of the discharge, which amounts up to 30 m³/s per 3 hours. Therefore the accuracy of the parameters found is relatively poor.

The accuracy of the Mehaigne forecasts is relatively good (mean square error of 1 m³/s²), although the accuracy of the parameters is relatively low.

The Ourthe subcatchment was one of the subcatchments with a strong correlation between $f_{\text{max}}$ and $k_R$, resulting in unrealistic values. Therefore $f_{\text{max}}$ has been set at 6 mm/h. The mean square error of the model is small.

The Amblève model produces a forecast with a relatively high variance. Probably the size of the subcatchment and the small reaction time are the cause. The value of $\Psi$ is the highest of the tributaries under investigation.

The smallest reaction time is found for the Vesdre. The value of $f_{\text{max}}$ is high, denoting
TABLE 10.17 MEAN SQUARE ERRORS CALCULATED OF THE MODEL FOR A LEAD TIME OF 6 h

<table>
<thead>
<tr>
<th>Subcatchment</th>
<th>$\hat{\sigma}^2_0$ (calibration period) [m$^4$/s$^2$]</th>
<th>$\hat{\sigma}^2_0$ (verification period) [m$^4$/s$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lesse</td>
<td>21.2</td>
<td>24.2</td>
</tr>
<tr>
<td>Sambre</td>
<td>227.9</td>
<td>1.7</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Ourthe</td>
<td>25.7</td>
<td></td>
</tr>
<tr>
<td>Amblève</td>
<td>79.5</td>
<td>67.3</td>
</tr>
<tr>
<td>Vesdre</td>
<td>39.7</td>
<td>59.4</td>
</tr>
</tbody>
</table>

A significant correlation with $k_8$. The consequence is great variances of those parameters.

The results of the verification periods are comparable to the results of the calibration period. Differences in accuracy can be explained by the limited number of flood events involved.

An example of a discharge measured versus a calculated one can be found in fig. 10.12. The model has also been used to forecast the discharge with a 3, 9 and 12 h lead time. A simulation can be found in fig. 10.13. Snowmelt played an important role in the first part of the period shown.

10.3.6 MODIFICATION OF THE FORECAST

The model for the Belgian tributaries described in the preceding sections requires precipitation data in real-time for each subcatchment up to the last moment for which forecasts are made and one discharge data at the moment of observation. In the future the precipitation data may be replaced by precipitation forecasts.

If data transmittance functions ideally, every hour a discharge data is available for each subcatchment. Then not all the information available is used for the calculation of the discharge forecast. In this section a model is described that modifies the forecast using the real-time discharge information available.

In hydrology complex updating procedures are used frequently. Serban et al. (1991) distinguish several updating procedures: updating of input variables, state variables, output variables and model parameters. They consider input variable updating (think of precipitation and snowmelt) as difficult, and establish the fact that in most cases input variable updating is done using trial and error procedures. State variable updating is often made by the Kalman filter. Serban et al. (1991) state that that kind of updating requires a lot of computer memory and time. Troch et al. (1991) think that for stochastic models a simpler procedure may suffice. For output updating usually procedures like ARMA are used. Until now the updating of model parameters has been unusual (Serban et al., 1991).

Those complex models are considered to be unsuitable for the Meuse, because of the various input sources of precipitation (chapter 12) and the chance of incidently missing data and the calculation time required. Therefore a simpler model has been chosen.

As the output the model described in § 10.3.5 gives among other things a series:
\[ Q_{d, i}, Q_{d, i+1}, \ldots, Q_{d, b}, \ldots, Q_{d, b+h'}, Q_{d, b+h'} \]  

(10.113)

where:

\[ Q_{d, i} = \text{calculated direct runoff for } i \Delta t \]  

\[ i_c = \text{time serial number corresponding to the first moment for which a calculation has been made} \]  

\[ i_0 = \text{time serial number corresponding to the moment of observation} \]  

For reasons of brevity, the index \( r \) denoting the event has been omitted.

The discharge data up to \( i_0 \) are known:

\[ Q_{i_f, Q_{i+1}, \ldots, Q_{b}} \]  

(10.114)

where:

\[ i_f = \text{serial number corresponding to the first moment for which a discharge is known} \]  

For the moment the limited length of the series presented in (10.113) and (10.114) will be neglected, as will the situation of incidental missing discharge data. Chapter 12 deals with that kind of missing information.

Corresponding to § 10.3.5 the analysis has been made on the differences between discharges measured and direct runoff values calculated. The differences will be indicated by the symbol \( Y \). Hence:

\[ Y_i = Q_i - Q_{d, i} \]  

(10.115)

where:

\[ Y_i = \text{difference between discharge measured and direct runoff calculated at } i \Delta t \]  

\[ [L^3 T^{-1}] \]

It should be noted that the mean of \( Y \) will be significantly greater than zero, because of the baseflow.

Because a time series with equidistant time steps is available, and the physical interpretation of the series \( Y \) is not completely clear, an ARIMA-analysis has been performed. An example of a series of \( Y \) is shown in fig. 10.14.

Equation (10.112) also contains aspects of an ARIMA forecast. That can be shown as follows.

Equation (10.112) is rewritten into:
\[ Q_{i+h'} = Q_{i+h'}^* - Q_{i}^* + Q_i + e_{i+h'} \]  \hspace{1cm} (10.116)

where:

\[ e_{i+h'} = Q_{i+h'} - Q_{i+h'}^* \]  \hspace{1cm} (10.117)

It is assumed that the mean of \( e \) is equal to zero, and is distributed normally. Combining (10.115) and (10.116) yields:

\[ Y_{i+h'} = Y_i + e_{i+h'} \]  \hspace{1cm} (10.118)

which has the character of an ARIMA(0,1,0) model with a time-step of \( h' \Delta t \). However, other ARIMA-models than the one shown in (10.118) may be more appropriate. In practice, the number of ARIMA-models to be used is limited, because:

- If moving average terms are included in the model, in real-time it is necessary to calculate the forecast at all the discrete moments involved up to the moment of observation. For a proper forecast a great number of available discharge data and a lot of calculating time are required. Therefore MA-terms should preferably be avoided.

- \( Y \) merely serves as a correction for the omittance of the baseflow. If only autoregressive terms are incorporated in the model, the sum of the coefficients should be equal to 1 in order to prevent a systematic deviation.

First the attention is concentrated on a forecast one step ahead \((h' = 1)\). Several models have
been investigated. The determination of the model was made with the Box-Jenkins method. The calculations were made using the SPSS-X-routine ARIMA. The result for the Ambîève subcatchment is shown in table 10.18.

For a proper model choice the possibility of missing data should also be considered. If a data is missing, it should be replaced by an interpolation of known data, or a different model should be used. The first alternative may lead to great errors if the coefficients involved are of a great magnitude. The second alternative will lead to a set of ARIMA-models, and therefore to a complex modification model.

TABLE 10.18 PARAMETER ESTIMATION FOR A
ONE STEP FORECAST FOR THE AMBLÊVE
SUBCATCHMENT FOR THE CALIBRATION PERIOD

<table>
<thead>
<tr>
<th>ARIMA model</th>
<th>$s^2$ [m$^3$/s$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1,0,0)</td>
<td>31.7</td>
</tr>
<tr>
<td>(1,1,0)</td>
<td>29.0</td>
</tr>
<tr>
<td>(2,0,0)</td>
<td>28.6</td>
</tr>
<tr>
<td>(3,0,0)</td>
<td>28.7</td>
</tr>
<tr>
<td>(2,1,0)</td>
<td>29.1</td>
</tr>
<tr>
<td>(3,1,0)</td>
<td>29.1</td>
</tr>
<tr>
<td>(0,0,1)</td>
<td>284.9</td>
</tr>
<tr>
<td>(0,1,1)</td>
<td>29.2</td>
</tr>
<tr>
<td>(1,1,1)</td>
<td>29.1</td>
</tr>
</tbody>
</table>

It can be seen that a simple ARI(1,1) model will give almost the same accuracy as the other models. For that reason and the above remarks it has been decided to choose an ARI(1,1) model. If the coefficient found is indicated as $\alpha$, then the model will be:

$$Y_{i+1} = (1+\alpha)Y_i - \alpha Y_{i-1} + e_{i+1}$$  \hspace{1cm} (10.119)

The discharge forecast can be calculated replacing $e_{i+1}$ by its mean (zero):

$$\hat{Q}_{i,1} = Q_{d,i-1} + (1+\alpha)Y_i - \alpha Y_{i-1}$$

$$\hspace{1cm} = Q_{d,i-1} + (1+\alpha)(Q_i - Q_{d,i}) - \alpha(Q_{i-1} - Q_{d,i-1})$$  \hspace{1cm} (10.120)

where:

\[\hat{Q}_{i,1}\] discharge forecast made at $t_i$ with lead time 1, so for $t_{i+1}$ [L$^3$T$^{-1}$]

Equation (10.120) was derived for a forecast one step ahead. In practice a forecast for more time steps ahead is also required. Because the ARI(1,1) was successful for the one step ahead forecast the investigation has been restricted to similar types of models. The models investigated are of the type:

$$Y_{i+1} = (1+\alpha_{m,h})Y_i - \alpha_{m,h}Y_{i-m} + e_{i+1}$$  \hspace{1cm} (10.121)

where:
### TABLE 10.19 PARAMETERS CALCULATED AND THEIR VARIANCE FOR THE LESSE SUBCATCHMENT

<table>
<thead>
<tr>
<th>( h' )</th>
<th>( m )</th>
<th>( \alpha_{m, h'} )</th>
<th>( \sigma_0^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta t = 3h )</td>
<td></td>
<td>[( m^2/s^2 )]</td>
<td>model</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>0.780</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.340</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.198</td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.135</td>
<td>4.7</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>1.200</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.504</td>
<td>15.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.290</td>
<td>17.5</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.194</td>
<td>18.6</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>1.352</td>
<td>30.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.558</td>
<td>34.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.313</td>
<td>36.7</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.205</td>
<td>38.1</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>1.377</td>
<td>50.0</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.551</td>
<td>55.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.301</td>
<td>58.2</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.196</td>
<td>59.4</td>
</tr>
</tbody>
</table>

\( m \) = integer constant
\( \alpha_{m, h'} \) = coefficient

The corresponding equation for the modification of the discharge calculated yields:

\[
\hat{Q}_{i, h'} = Q_{d, i, h'} + (1 + \alpha_{m, h})Y_i - \alpha_{m, h}Y_{i-m}
\]

\[
= Q_{d, i, h'} + (1 + \alpha_{m, h})(Q_i - Q_{d, i}) - \alpha_{m, h}(Q_{i-m} - Q_{d, i-m})
\]  \( (10.122) \)

The model has been investigated for several lead times and values of \( m \). The variances calculated for the Lesse are shown in table 10.19. In the table the values of \( \alpha_{m, h'} \) are also given. The Lesse has been taken as an example because the parameters found show a certain resemblance to the parameters which would have been found if the theory of § 10.2 had been applied.

Calculations show that the minimum variance is obtained if the value of \( m \) is equal to 1 time step. It applies to all the subcatchments. Therefore \( m \) will be 1 (3 hours) in the modification function. Results of the calibration are summarised in table 10.20.

It might be questioned whether the three hour time step is the optimal value, as no other time steps near three hours have been investigated. Especially a smaller time step seems to promise a further reduction of the mean square error. However, such influences as a local
### TABLE 10.20 CALCULATED VALUES FOR $\alpha$

<table>
<thead>
<tr>
<th>subcatchment</th>
<th>$h'$ [-]</th>
<th>$\alpha_{1,h'}$ [-]</th>
<th>$s_0^2$ model [m$^2$/s$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>($\Delta t = 3$ h)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lesse</td>
<td>1</td>
<td>0.78</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.20</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.35</td>
<td>30.1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.38</td>
<td>51.0</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>1</td>
<td>0.54</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.67</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.60</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.34</td>
<td>2.5</td>
</tr>
<tr>
<td>Vesdre</td>
<td>1</td>
<td>0.43</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.41</td>
<td>37.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.26</td>
<td>63.3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.10</td>
<td>85.0</td>
</tr>
<tr>
<td>Amblève</td>
<td>1</td>
<td>0.28</td>
<td>28.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.33</td>
<td>75.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.34</td>
<td>126.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.32</td>
<td>179.4</td>
</tr>
<tr>
<td>Ourthe</td>
<td>1</td>
<td>0.61</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.92</td>
<td>18.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.17</td>
<td>36.9</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.43</td>
<td>57.8</td>
</tr>
</tbody>
</table>

Shower and random errors in the discharge determination will not make the optimum value to tend to zero hours. For that reason and because of the limited time available for this investigation the value of three hours has been maintained.

It is remarkable that for some subcatchments the value of $\alpha$ increases with increasing $h'$, and for other subcatchments it decreases. It depends on the autocorrelation function of the $Y$ series. For the Sambre, with its strongly fluctuating discharge values, no successful modification model could be determined.

For the subcatchments for which verification periods have been defined, the verification was executed. Results can be found in table 10.21. Because the verification periods are relatively short, the variance found is significantly dependent on the greatest residual. The results obtained are considered to be acceptable.
TABLE 10.21 RESULTS OF THE VERIFICATION

<table>
<thead>
<tr>
<th>subcatchment</th>
<th>$h'$ [-] $(\Delta t = 3h)$</th>
<th>$\sigma_0^2$ model $[m^2/s^2]$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>calibration</td>
<td>verification</td>
</tr>
<tr>
<td>Lesse</td>
<td>1</td>
<td>2.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>30.1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>51.0</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>1</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>2.5</td>
</tr>
<tr>
<td>Vesdre</td>
<td>1</td>
<td>11.1</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>37.0</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>63.3</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>85.0</td>
</tr>
<tr>
<td>Amblève</td>
<td>1</td>
<td>28.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>75.3</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>126.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>179.4</td>
</tr>
</tbody>
</table>

10.3.7 PRECIPITATION FORECASTS

Many discharge forecasting models are based on precipitation data as a part of the inputs. Precipitation forecasts are only necessary if the precipitation occurring after the moment of observation may influence the discharge significantly. Precipitation forecasts are of great importance in the forecasts of flash floods (Hall, 1981), especially if the reaction time between the moment of precipitation and the runoff is less than the lead time required. That situation occurs in particular when convective rains cause floods. Extensive research has been made to forecast the convective rainfall, with the help of mathematical models and radar pictures.

The precipitation forecast on behalf of the discharge forecast of Borgharen-Dorp is not so important as for some other rivers, because:
- floods in the Meuse are caused by frontal zones or a series of convective showers. One single shower is very unlikely to cause a flood in the Meuse.
- the wave travel time between the gauge in a Belgian tributary and Borgharen-Dorp is at least 5 hours, which holds for the Vesdre.

However, the influence of precipitation occurring after the moment of observation must not be neglected. It is illustrated in fig. 10.15, where the direct runoff for the Vesdre subcatchment is shown including and excluding precipitation data after the moment of observation. In the latter case it is assumed that no precipitation occurs after the moment of observation. It should be noticed that a moment of observation has been selected when the differences between the discharges are relatively great.
Although many hydrologists have tried to forecast precipitation, the subject is rather a meteorological specialty than a hydrologic one. The KNMI (Koninklijk Nederlands Meteorologisch Instituut) supplies Rijkswaterstaat with precipitation forecasts. The forecasts are based on meteorological forecasting models in combination with the meteorologists’ experience.

The precipitation forecasts are supplied by telephone. In principle forecasts are available 24 hours per day. However, a new precipitation forecast can only be made if the Institute has new data or meteorological forecasts at its disposal. In practice, the number of precipitation forecasts is limited to 4 per day.

Precipitation forecasting is one of the more complex subjects in meteorological forecasting. Precipitation is more or less the balance term of weather simulation models. Therefore the precipitation is difficult to forecast quantitatively. Also the place is hard to determine. The Meuse catchment is small in relation to the uncertainty of the place of the heavy precipitation zone. The forecast of precipitation intensities looks relatively accurate if considered on a logarithmic scale (0.1, 1, 10 mm, etcetera). On a normal scale (0, 5, 10 mm, etcetera) the errors are significant, especially for greater intensities.

Precipitation forecasts are given by the KNMI in terms of North and South, etcetera,
and per 6 or 12 hours. Also an indication of the accuracy of the forecasts is given by the
KNMI. Unfortunately, the greatest accuracy is always obtained if the forecast amount is equal
to zero.

As the forecasts are influenced by human interpretation and precipitation forecasting
models are modified through the years, a quantitative determination of precipitation
forecasting accuracy is difficult. An analysis of the accuracy is therefore omitted here.

The forecasts may be implemented in the model. More information can be found in
chapter 12.

10.3.8 WAVE TRAVEL TIMES AND UNGAUAGED AREAS

This section deals with two small subjects that have not yet been dealt with in earlier sections.
They include the travel time between the station and the mouth of the river, and the ungauged
areas.

The flood wave between the station and the mouth of the tributary for the Belgian tributaries
The flood wave that passes a station and the mouth of a river is different with respect to flood
forecasting in two ways. The shape has been attenuated and some time has passed since the
passage of the station.

Appendix 2 shows that the change of the shape of the wave in the section considered
may be neglected for the tributaries considered.

The wave travel times have been determined using among other things the data in
chapter 6. The travel times found are listed in table 10.22.

The ungauged areas discharging into the Belgian Meuse
Contrary to the French (§ 11.2) and Dutch (§ 11.4) reaches, the contributions of the Belgian
unmeasured areas are not implicitly incorporated in the model, due to the fact that calibration
of the model for the Belgian Meuse (§ 11.3) on the basis of measured discharges could not
take place. Therefore the unmeasured discharges have to be estimated. The ungauged areas
are thought to behave like nearby measured areas which have the same hydrologic properties.
For all the areas to which an ungauged area is added, a factor $\zeta$ is determined giving the
ratio of the incorporated ungauged area to the gauged area.

The travel time from the output station to the Meuse is set to zero for the ungauged
subcatchments, as the ungauged areas are situated adjacent to the river. It leads to:

$$Q_p^*(t) = \dot{Q}(t - T_m) + \zeta \dot{Q}(t) \quad (10.123)$$

where:

$Q_p^*(t)$ = calculated discharge which is one of the inputs of the module for the
Belgian Meuse [L$^3$T$^{-1}$]

$T_m$ = wave travel time between the measuring station and the mouth of the
tributary [T]

$\zeta$ = factor [-]

Note: For moments before the time of observation the forecast discharges in the right hand
term should be replaced by values measured or interpolated.
TABLE 10.22 VALUES FOR THE FACTOR $\zeta$ AND THE WAVE TRAVEL TIME $T_m$

<table>
<thead>
<tr>
<th>subcatchment</th>
<th>$\zeta$ [-]</th>
<th>$T_m$ [h]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lesse</td>
<td>0.45</td>
<td>1</td>
</tr>
<tr>
<td>Sambre</td>
<td>0.21</td>
<td>2</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>1.70</td>
<td>1</td>
</tr>
<tr>
<td>Vesdre</td>
<td>0.33</td>
<td>0</td>
</tr>
<tr>
<td>Amblesve</td>
<td>0.33</td>
<td>2</td>
</tr>
<tr>
<td>Ourthe</td>
<td>0.33</td>
<td>2</td>
</tr>
</tbody>
</table>

The values of $\zeta$ should preferably be significantly smaller than one, in order to prevent a local intensive shower from having a much greater impact on the calculation than it should. Only for the Mehaigne subcatchment the $\zeta$ factor is not significantly less than 1, but its discharges are usually small. Table 10.22 gives the values obtained for $\zeta$.

10.4 THE DUTCH TRIBUTARIES

The tributaries discharging into the Dutch Meuse upstream from Lith are in most cases of minor importance for flood forecasting, except for two tributaries: The Roer and the Niers. Most important of the two is the Roer. The tributary distinguishes itself from the other main tributaries of the Meuse in several ways. The two most important are:
1. The place of its mouth is situated relatively far downstream and
2. the discharges during floods are significantly influenced by the reservoirs and the flatness of the downstream part of the subcatchment area.

Because the Niers is less important than the Roer, attention will be focussed on the Roer. At the end of the section the modelling of the Niers will be dealt with.

The Roer

The discharge of the Roer significantly affects both the total discharge of the Meuse and the water level. As a rule of thumb it may be said that for the Meuse just downstream from Roermond, an increase of the discharge of 4 m$^3$/s will cause a rise of the water level of 1 cm in flood conditions.

The discharge of the Roer should preferably be modelled so that the error of the model will cause a minor error compared to the errors of the water level measurements in the river Meuse (5-10 cm). In the model, preferably as few parameters as possible should be used (principle of parsimony). The lead time should be 24 hours.

To restrict the number of data, only discharge information was used. Information of the stations De-Drie-Bogen (the Dutch station), Jülich and Zerkall was available (fig. 6.11).

Because both the discharges of the Meuse at Borgharen-Dorp and the Roer at De-Drie-Bogen are dependent on the precipitation in the Ardennes, and a discharge forecast of 24 h of Borgharen-Dorp is available, also discharges and discharge forecasts of Borgharen-Dorp are used during the analysis.
The Roer does not react so fast as many other tributaries (cf. chapter 6). Therefore, the time step could be set at 12 hours.

The availability of discharges of many stations led to the choice of a Multiple Linear Regression model. The calculations were made on a mini-computer using the regression routine of SPSS-X and using the procedure stepwise. It is probably the most commonly used procedure in regression (Norusis, 1983). The method repeatedly enters and removes variables in the regression equation. What variables should be entered in or removed from the equation is based on partial correlation coefficients. The process continues until a certain fit has been achieved. During the Roer modelling variables were removed only if the model performance did not change significantly. During the calibration, the default criteria have been used. 60 discharges of the Roer measured at De-Drie-Bogen from 1980 to 1985 were used for calibration.

The types of data which should be included in the real-time forecast, are not clear in advance.

Discharges at Borgharen-Dorp (BhD) are available at both 8.00 h and 20.00 h. It is questionable whether discharge forecasts of the Meuse should also be incorporated in real-time, regarding the significance of the errors (choice 1).

Discharges at De-Drie-Bogen (DDB) are available, however not automatically. The discharge at 8.00 h can be obtained during the routine gathering taking place every morning. The effect of the use of the discharge at 20.00 h is questionable (choice 2).

That leads to four possible ways of forecasting: using or not using discharge forecasts at Borgharen-Dorp (choice 1) in combination with using only 8.00 h or both 8.00 h discharges and 20.00 h discharges at De-Drie-Bogen (choice 2). In the first instance the BhD forecasts are replaced by measured data, because the forecasts for Borgharen-Dorp are not available at the moment.

The exercise has been made for 12 h, 24 h and 36 h forecasts for De-Drie-Bogen. As a measure for the model performance, the mean square error has been used. The principle of parsimony was included in the model selection procedure. The results of the calibration are shown in table 10.23.

The contents of table 10.23 will be explained using an example. If the discharges at De-Drie-Bogen are available at 8.00 h, and forecasts of the discharges at Borgharen-Dorp are used, equation 6 should be used for a 24 h forecast. For the real-time situation the equation reads:

\[ Q^{DDB}(t, 24) = 0.94 Q^{DDB}(t) + 0.041 Q^{BAD}(t, 24) - 0.031 Q^{BAD}(t) - 4.51 \quad (10.124) \]

where:

- \( Q^{DDB}(t,h) \) = forecast discharge for De-Drie-Bogen, made at time \( t \) with a lead time \( h \) [m\(^3\)/s]
- \( Q^{BAD}(t,h) \) = forecast discharge for Borgharen-Dorp, made at time \( t \) with a lead time \( h \), using the model described in the chapters 9 to 12 [m\(^3\)/s]
- \( Q^j(t) \) = measured discharge in place \( j \), at time \( t \) [m\(^3\)/s]

The model would be the same if discharges at 20.00 h at De-Drie-Bogen are available.
<table>
<thead>
<tr>
<th>eq</th>
<th>lead time</th>
<th>$Q_{DDB}^{available}$ at</th>
<th>forecast $Q_{BAD}$</th>
<th>model</th>
<th>$Q_{DDB}^{*}$ [-]</th>
<th>$Q_{BAD}^{*}$ [-]</th>
<th>$\beta_{j}$ *</th>
<th>$s_0^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>8 h, 20 h</td>
<td>yes</td>
<td>II</td>
<td>+1.34</td>
<td>-0.53</td>
<td>+0.20</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>8 h</td>
<td>yes</td>
<td>II</td>
<td>+1.00</td>
<td>-</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>8 h, 20 h</td>
<td>no</td>
<td>III</td>
<td>+1.53</td>
<td>-0.60</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>4</td>
<td>12</td>
<td>8 h</td>
<td>no</td>
<td>III</td>
<td>+1.14</td>
<td>-0.23</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>24</td>
<td>8 h, 20 h</td>
<td>yes</td>
<td>II</td>
<td>+0.94</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>6</td>
<td>24</td>
<td>8 h</td>
<td>yes</td>
<td>II</td>
<td>+0.94</td>
<td>-</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>24</td>
<td>8 h, 20 h</td>
<td>no</td>
<td>III</td>
<td>+1.34</td>
<td>-0.55</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>8</td>
<td>24</td>
<td>8 h</td>
<td>no</td>
<td>III</td>
<td>+1.06</td>
<td>-0.28</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>36</td>
<td>8 h, 20 h</td>
<td>yes</td>
<td>II</td>
<td>+0.91</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>10</td>
<td>36</td>
<td>8 h</td>
<td>yes</td>
<td>II</td>
<td>+0.91</td>
<td>-</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>36</td>
<td>8 h, 20 h</td>
<td>no</td>
<td>III</td>
<td>+1.51</td>
<td>-0.79</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>12</td>
<td>36</td>
<td>8 h</td>
<td>no</td>
<td>III</td>
<td>+0.71</td>
<td>-</td>
<td>x</td>
<td>-</td>
</tr>
</tbody>
</table>

$x$ = available but not used
- = not available

Remark: all the discharges further back in time and discharges of Jülich and Zerkall were available. None of them was used.
Calculations have shown that it is not worthwhile using the discharge at Jülich and Zerkall. They only add little to the model performance (a lowering of only 0.25 m³/s of s₀). That is remarkable, because the major part of the discharge at De-Drie-Bogen has also passed Jülich. In the author’s view, too short a travel time between Jülich and De-Drie-Bogen in combination with a certain noise might be the cause.

One of the suppositions made is the normal distribution of the residuals. The residuals found do not obey the supposition completely: there are too many extreme values. The origin of the outliers is thought to be caused by local effects in the precipitation intensity. The effect of transforming measured values by a logarithmic or arctan transform is considered to be useless, as the extreme values are not determinable by means of the input data. The results are considered to be acceptable.

Table 10.23 gives the parameters for four types of equations, one of which should be chosen. The first choice to be made is whether discharge forecasts at Borgharen-Dorp should be incorporated in the model. For the decision the equations resulting from table 10.23 are rewritten in another form:

\[ Q^{DDB}(t,h'\Delta t) = \sum_{i=0}^{n} \beta_i^A Q^{DDB}(t-i \Delta t) + \beta_i^B Q^{BAD}(t-v_i^B \Delta t) + \beta_i^C Q^{BAD}(t+v_i^C \Delta t) + \beta_i^* \]  

(10.125)

where \( \beta^* \) is a constant, all the other \( \beta \) are coefficients, and all the \( v \) are shifts in time. \( j \) is an indication number. \( \Delta t \) is equal to 12 hours. The constants \( v \) are positive.

The investigation starts with the introduction of three models:

\[ Q^{DDB}(t,h'\Delta t) = \sum_{i=0}^{n} \beta_{i1}^A Q^{DDB}(t-i \Delta t) + \beta_{i1}^B Q^{BAD}(t-v_{i1}^B \Delta t) + \beta_{i1}^C Q^{BAD}(t+v_{i1}^C \Delta t) + \beta_{i1}^* \]  

(10.126)

Model I makes use of discharges measured at De-Drie-Bogen and discharges measured and forecast at Borgharen-Dorp. The model can be used in real-time, its coefficients, however, cannot be calculated exactly, because not sufficient forecast data of Borgharen-Dorp are available.

\[ Q^{DDB}(t,h'\Delta t) = \sum_{i=0}^{n} \beta_{i2}^A Q^{DDB}(t-i \Delta t) + \beta_{i2}^B Q^{BAD}(t-v_{i2}^B \Delta t) + \beta_{i2}^C Q^{BAD}(t+v_{i2}^C \Delta t) + \beta_{i2}^* \]  

(10.127)

Model II makes use of discharges measured at both De-Drie-Bogen and Borgharen-Dorp. The model is calibrated (table 10.23), but cannot be used in real-time, because at the moment of prediction discharges of Borgharen-Dorp in the future are unknown.

\[ Q^{DDB}(t,h'\Delta t) = \sum_{i=0}^{n} \beta_{i21}^A Q^{DDB}(t-i \Delta t) + \beta_{i21}^B Q^{BAD}(t-v_{i21}^B \Delta t) + \beta_{i21}^* \]  

(10.128)

Model III makes use of discharges measured at De-Drie-Bogen and discharges measured at Borgharen-Dorp up to time \( t \). The model is calibrated (table 10.23) and can be used in real-
time, because no values of Borgharen-Dorp in the future are necessary.

The coefficients for model I are assumed to be the same as for model II.

For operational use only the models I and III can be used, as the discharge measured at Borgharen-Dorp is not known in the future. The decision whether model I or III should be chosen is dependent on the smallest mean of the squares of the residuals. If the forecast at Borgharen-Dorp is excellent, clearly model I should be chosen, as the sum approaches that of model II. However, if the forecast at Borgharen-Dorp is poor, the sum of the square of the residuals of the De-Drie-Bogen forecast will be much greater than that of model II, and probably higher than that of model III.

A critical value for the mean of the squares of the residuals of the Borgharen-Dorp forecast is defined, \( s^2_{cr} \). If the mean square residual of the discharge forecast at Borgharen-Dorp is equal to \( s^2_{cr} \), then the performances of models I and III are equal with regard to the used criterion.

For a linear relation between \( y \) and \( x_i \):

\[
y = \sum_i a_i x_i
\]

(10.129)

where \( a_i \) are constants, it can be derived if the correlation \( \rho \) of \( x_j \) and \( x_k \) is equal to 0 (\( j \neq k \)) that:

\[
s^2_y = \sum_i a_i^2 s^2_{x_i}
\]

(10.130)

and if \( \rho \) is equal to 1 that:

\[
s_y = \sum_i a_i s_{x_i}
\]

(10.131)

Equations (10.130) and (10.131) will be used during the analysis.

For the analysis model I is rewritten into:

\[
Q^{DBB}(t, h', \Delta t) = \sum_{i=0}^{n_1} \beta_i^A Q^{DBB}(t-i, \Delta t) + \beta_i^B Q^{DBB}(t-v_i^B, \Delta t) + \beta_i^C Q^{DBB}(t+v_i^C, \Delta t) + \beta_i^C (Q^{DBB}(t+v_i^C, \Delta t) - Q^{DBB}(t+v_i^C, \Delta t))
\]

(10.132)

Further rewriting gives for model I:
\[ Q^{DDB}(t, h' \Delta t) - Q^{DDB}(t+h' \Delta t) = \sum_{i=0}^{n_1} \beta_i^A Q^{DDB}(t-i \Delta t) + \beta_i^B Q^{BAD}(t-v_i \Delta t) + \beta_i^C Q^{BAD}(t+v_i \Delta t) - \beta_i^C Q^{BAD}(t+v_i \Delta t) \]

(10.133)

Indicating the number of the model and rewriting gives:

\[ Q_{model}^{DDB}(t, h' \Delta t) - Q^{DDB}(t+h' \Delta t) = Q_{model}^{DDB}(t, h' \Delta t) - Q^{DDB}(t+h' \Delta t) + \beta_i^C (Q^{BAD}(t+v_i \Delta t) - Q^{BAD}(t+v_i \Delta t)) \]

(10.134)

which can be simplified into:

\[ e^{DDB}_{model}(t,h' \Delta t) = e^{DDB}_{model II}(t,h' \Delta t) + \beta_i^C e^{BAD}(t, v_i \Delta t) \]

(10.135)

where:

\[ e^{I}_{model}(t,h) = Q^{I}(t+h) - Q^{I}_{model}(t,h) \]

(10.136)

By definition:

\[ S_{crit}^{BAD} = s^2(e^{BAD}(t, v_i \Delta t)) \]

(10.137)

for the value \( v_i \), which appears in the equation for the required lead time.

Now, if the correlation of model II and the forecast of Borgharen-Dorp is 0, then (using equation (10.130)):

\[ s^2(e^{DDB}_{model}(t,h' \Delta t)) = s^2(e^{DDB}_{model II}(t,h' \Delta t)) + (\beta_i^C)^2 s^2(e^{BAD}(t, v_i \Delta t)) \]

(10.138)

If:

\[ s^2(e^{DDB}_{model}(t,h' \Delta t)) = s^2(e^{DDB}_{model II}(t,h' \Delta t)) \]

(10.139)

the value of \( S_{crit}^{BAD} \) is found. Eliminating \( s^2(e^{DDB}_{model I}) \) from equations (10.138) and (10.139) gives, using a simpler notation:

\[ S_{crit}^{BAD} = \frac{s^2 e^{DDB}_{model III} - s^2 e^{DDB}_{model II}}{(\beta_i^C)^2} \]

(10.140)

If the residuals of model II and the forecast of Borgharen-Dorp have a correlation coefficient equal to 1, then (using equation (10.131)):

\[ s(e^{DDB}_{model}(t,h' \Delta t)) = s(e^{DDB}_{model II}(t,h' \Delta t)) + \beta_i^C s(e^{BAD}(t, v_i \Delta t)) \]

(10.141)

In an analogous way it can be derived that:
\[ S_{\text{BAD}}^{\text{crit}} = \frac{S_{\text{model III}} - S_{\text{model II}}}{\beta_C^i} \]  

(10.142)

The values for \( S_{\text{BAD}}^{\text{crit}} \) are calculated for all the cases using equations (10.140) and (10.142) and can be found in table 10.24. The table shows that even a relatively inaccurate forecast of the discharge at Borgharen-Dorp contributes to the reduction of the mean square residual of the forecast at De-Drie-Bogen. Therefore, that forecast will be incorporated in the model.

It concludes the investigation for the first choice.

<table>
<thead>
<tr>
<th>lead time</th>
<th>( Q_{DDB} ) available at</th>
<th>used equations (table 10.23)</th>
<th>( S_{\text{BAD}}^{\text{crit}} ) [m³/s]</th>
<th>( \rho = 0 )</th>
<th>( \rho = 1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>8 h, 20 h</td>
<td>1, 3</td>
<td>210</td>
<td>77</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>8 h</td>
<td>2, 4</td>
<td>202</td>
<td>84</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>8 h, 20 h</td>
<td>5, 7</td>
<td>203</td>
<td>78</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td>8 h</td>
<td>6, 8</td>
<td>210</td>
<td>82</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>8 h, 20 h</td>
<td>9, 11</td>
<td>282</td>
<td>103</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>8 h</td>
<td>10, 12</td>
<td>327</td>
<td>133</td>
<td></td>
</tr>
</tbody>
</table>

Beside the choice of the incorporation of the forecasts at Borgharen-Dorp, the second choice, the frequency of data gathering at De-Drie-Bogen, has still to be made. Two frequencies are considered, once per day and twice per day. The equations to be used in both situations are gathered in table 10.25.

It may be noted that for a 12 h respectively 24 h forecast at 20.00 h, with measurements of De-Drie-Bogen only available at 8.00 h, a 24 h respectively 36 h forecast equation has to be used. The omission of a measurement at 20.00 h almost leads to nearly a doubling of the mean square residual, which is a significant change in the accuracy of the total forecast of the Meuse river. Therefore, the discharges at De-Drie-Bogen should be collected twice a day.

The wave travel time in the 20 km long section between De-Drie-Bogen and the mouth of the river is approximately 3 hours. The value was obtained by using physiographical constants and some simple hydraulic equations. Any change of discharge during the passage of the reach is neglected, among other things because of the relatively flat shape of the wave (cf. appendix 2).
TABLE 10.25 EQUATIONS TO BE USED FOR THE ROER IN VARIOUS SITUATIONS

<table>
<thead>
<tr>
<th>$Q^{DDB}$ available at</th>
<th>forecast at</th>
<th>used equation (table 10.23)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>12 h forecast</td>
</tr>
<tr>
<td>8 h, 20 h</td>
<td>8 h</td>
<td>1</td>
</tr>
<tr>
<td>8 h, 20 h</td>
<td>20 h</td>
<td>1</td>
</tr>
<tr>
<td>8 h</td>
<td>8 h</td>
<td>2</td>
</tr>
<tr>
<td>8 h</td>
<td>20 h</td>
<td>6'$</td>
</tr>
</tbody>
</table>

* equation put 12 h back in time

In conclusion, the models to be used for the Roer are the following:

$$\dot{Q}^{Roer}(t, 3) = Q^{DDB}(t)$$

$$\dot{Q}^{Roer}(t, 15) = 1.34 Q^{DDB}(t) - 0.53 Q^{DDB}(t - 12) + 0.20 Q^{DDB}(t - 24) + 0.020 \dot{Q}^{BkD}(t, 12) - 0.017 Q^{BkD}(t - 12) - 1.96$$

$$\dot{Q}^{Roer}(t, 27) = 0.94 Q^{DDB}(t) + 0.041 \dot{Q}^{BkD}(t, 24) - 0.031 Q^{BkD}(t) - 4.51$$

where:

$$\dot{Q}^{Roer}(t,h) = \text{discharge forecast at the mouth of the Roer made at } t \text{ with a lead time of } h$$

$$h = \text{lead time}$$

For other moments, the discharge desired can be obtained by interpolating known discharges and/or estimated discharges.

**The Niers**

Because the discharge of the Niers is very small compared to that of the Meuse (chapter 6), and the subcatchments of Niers and Roer are adjacent, it has been decided to approach the discharge of the Niers using that of the Roer. It was done by linear regression. After the calibration with 50 discharges the following equation was found:

$$Q^{Goch} = 0.176 Q^{DDB} + 7.84$$

where:

$$s_0^2 = 16 \text{ m}^2/\text{s}^2$$
\[ Q^{Goch}(t) = \text{calculated discharge at the Niers station of Goch} \] [m³/s]

If necessary the discharge of the Roer is replaced by a forecast. The wave travel time between the station and the mouth is considered to be 4 hours.
Chapter 11. Flood forecasting: The river

11.1 A SUBDIVISION INTO THREE PARTS

The character of the Meuse changes when the river passes a border. It applies not only to the physical character of the river, but also to the kind of available information and the desired forecasts. Therefore it is logical in principle to treat the French, Belgian and Dutch parts separately.

In contrast to the rainfall-runoff process the propagation of flood waves can be calculated accurately with the help of hydrodynamic flow routing models. That type of models is based on the St. Venant equations, which have been derived analytically. An advantage of hydrodynamic flow routing models is the possibility to investigate the effect of changes in the channel system. A disadvantage is the necessity to have the disposal of a lot of geometrical and frictional data.

The demands for the French Meuse are relatively small: The accuracy should be of the same order as that of longer term forecasts of the tributaries of the Ardennes. The lead time may be relatively short. Therefore the research effort should not be concentrated on that subject. Because the available calibration data are restricted to discharge data, a hydrodynamic flow routing model cannot be applied. Experience with multiple linear regression (MLR) in the Rijkswaterstaat department for the river Rhine led to the choice of that type of model.

Compared to the French Meuse the Belgian Meuse is much more important for the flood-forecast of Borgharen. Here the form of the bed of the river as well as the weirs have changed during recent years. However, large quantities of geometrical data of the ultimate bed form are available. Discharges have been calculated by the authorities at weirs with the help of the heights of the gates and by discharges passing the locks and the hydro-electric power stations. Those values are relatively inaccurate, and therefore they were not used in this study.

Because of the weir influences, the changing geometry of the river, the absence of high quality discharge data, the availability of geometrical data and the importance of an accurate forecast it was decided to apply a hydrodynamic flow routing model.

More ideal is the situation for the Dutch Meuse. The river geometry can be considered to be constant in time in view of flood forecast modelling. A lot of data were available, both in a geometric sense as in the form of water levels and discharges. In the past good results had been achieved with a flood-routing model for the Dutch Meuse. There was no need to change the type of model, but a new calibration was desirable because the old one tended to be out-dated. Therefore a new schematisation for the flood-routing model was developed.

From the preceding text it is clear that the subdivision into three parts according to the borders is a logical choice. The three parts of river modelling will be elucidated in the next three sections. Special attention will be given to weir operation and bank storage. The subjects will be dealt with in the sections where they are most important: the Belgian and Dutch Meuse respectively.

11.2 THE FRENCH MEUSE

In view of the development of a flood forecasting model for the Netherlands the French reach of the Meuse is different from the Belgian and Dutch reaches. The differences are threefold:
a. a relatively great data set of discharges is available, as several stations along the river are covered,
b. no profile data are available, and
c. the travel time to the Dutch reach of the river is relatively long.
The last aspect is important. The accuracy of the forecast at Borgharen is predominantly
determined by the weakest link in the forecasting model. For a 24 hour lead time the weakest
link is the precipitation forecast for the Ardennes tributaries. It is therefore ineffective to
concentrate much attention on the French Meuse, in other words: a relatively simple but
reasonably accurate model will suffice. For those reasons a multiple linear regression model
has been chosen. Rijkswaterstaat has some positive experiences with those kinds of models for
the flow forecasting of the Rhine at Lobith (on the German-Dutch border) (De Ronde, 1986,
Promes, 1987).

The discharge data were available for the stations:
a. Chooz (Meuse),
b. Haulmé (Semois),
c. Montcy-Notre-Dame (Meuse, near Charleville-Mézières),
d. Stenay (Meuse),
e. Carignan (Chiers),
f. Chauvency-le-Château (Chiers, upstream from Carignan), and
g. Treignes (Viroin).

The data are available in a non-equidistant time series and for several floods, except for
Treignes, for which hourly data are available. For the non-equidistant series the time is
indicated by a day number in the year and in thousandths of days. Especially in leap-years it
may cause confusion. The French data were compiled at the request of Rijkswaterstaat.
Unfortunately, for several floods and various stations mean daily discharge data obtained from
official publications do not agree with the non-equidistant data (Mugie, 1990). From a
scientific point of view the reasons of the differences should be found before starting
modeling. As the differences appear to be of various types, an intensive research will take a
lot of time, especially because foreign information is required. Because the goal of the
investigation is not the solution of that problem, but a real-time flood forecasting model that
has to function within a limited period of time, and because of the relatively small importance
of the French Meuse, a simple and pragmatic solution was chosen. The solution contains two
aspects:
1. The river upstream from Montcy-Notre-Dame has not been modelled, because wave
trace travel time is relatively long (cf. chapter 7), and
2. For some stations the hourly series has been replaced by one or two days,
corresponding to official publications of mean daily discharges.

The last aspect is rigorous. However, the time shift is unavoidable, as an analysis and
calculations with the multiple linear regression model with the original series did not result in
the determination of a wave travel time.

In agreement with the French tributaries a time step of 4 hours was used (§ 10.2). Due to the
small number of data available, all the events have been used for calibration. The calculations
have been executed on a mini-computer using the SPSS-X routine Regression. It leads to the
model:
### TABLE 11.1 COEFFICIENTS FOR THE FORECAST OF THE DISCHARGE AT CHOOZ FOR A LEAD TIME OF 4 HOURS

<table>
<thead>
<tr>
<th>x</th>
<th>$\beta_{0,4}^x$</th>
<th>$\beta_{4,4}^x$</th>
<th>$\beta_{8,4}^x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch</td>
<td>1.599</td>
<td>-0.778</td>
<td>0.106</td>
</tr>
<tr>
<td>Tr</td>
<td>0.659</td>
<td>-0.617</td>
<td></td>
</tr>
<tr>
<td>Ha</td>
<td>0.313</td>
<td>-0.214</td>
<td></td>
</tr>
<tr>
<td>MD</td>
<td>0.541</td>
<td>-0.458</td>
<td></td>
</tr>
</tbody>
</table>

$\beta_4^x = -2.87 \text{ m}^3/\text{s}$

$s^2 = 38 \text{ m}^6/\text{s}^2$

### TABLE 11.2 COEFFICIENTS FOR THE FORECAST OF THE DISCHARGE AT CHOOZ FOR A LEAD TIME OF 8 HOURS

<table>
<thead>
<tr>
<th>x</th>
<th>$\beta_{0,8}^x$</th>
<th>$\beta_{4,8}^x$</th>
<th>$\beta_{8,8}^x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch</td>
<td>1.774</td>
<td>-1.121</td>
<td>0.201</td>
</tr>
<tr>
<td>Tr</td>
<td>1.481</td>
<td>-1.480</td>
<td></td>
</tr>
<tr>
<td>Ha</td>
<td>1.028</td>
<td>-0.825</td>
<td></td>
</tr>
<tr>
<td>MD</td>
<td>1.779</td>
<td>-1.617</td>
<td></td>
</tr>
</tbody>
</table>

$\beta_8^x = -2.69 \text{ m}^3/\text{s}$

$s^2 = 151 \text{ m}^6/\text{s}^2$

### TABLE 11.3 COEFFICIENTS FOR THE FORECAST OF THE DISCHARGE AT CHOOZ FOR A LEAD TIME OF 12 HOURS

<table>
<thead>
<tr>
<th>x</th>
<th>$\beta_{0,12}^x$</th>
<th>$\beta_{4,12}^x$</th>
<th>$\beta_{8,12}^x$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ch</td>
<td>1.673</td>
<td>-1.134</td>
<td>0.274</td>
</tr>
<tr>
<td>Tr</td>
<td>2.103</td>
<td>-2.221</td>
<td></td>
</tr>
<tr>
<td>Ha</td>
<td>1.909</td>
<td>-1.651</td>
<td></td>
</tr>
<tr>
<td>MD</td>
<td>3.561</td>
<td>-3.365</td>
<td></td>
</tr>
</tbody>
</table>

$\beta_{12}^x = 5.25 \text{ m}^3/\text{s}$

$s^2 = 372 \text{ m}^6/\text{s}^2$
TABLE 11.4 COEFFICIENTS FOR THE FORECAST OF THE DISCHARGE AT CHOOZ FOR A LEAD TIME OF 16 HOURS

<table>
<thead>
<tr>
<th></th>
<th>$\beta_{0,16}$</th>
<th>$\beta_{4,16}$</th>
<th>$\beta_{8,16}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Ch$ (Chooz)</td>
<td>0.794</td>
<td>-0.095</td>
<td>-0.177</td>
</tr>
<tr>
<td>$Tr$ (Treignes)</td>
<td>3.242</td>
<td>-3.108</td>
<td></td>
</tr>
<tr>
<td>$Ha$ (Haulmé)</td>
<td>2.295</td>
<td>-1.682</td>
<td></td>
</tr>
<tr>
<td>$MD$ (Montcy-Notre-Dame)</td>
<td>4.219</td>
<td>-3.703</td>
<td></td>
</tr>
</tbody>
</table>

$\beta_{16} = 3.42$ m$^3$/s
$s^2 = 1005$ m$^2$/s$^2$

\[
\dot{Q}_{i,h'}^x = \sum_j \beta_{j,k'}^x Q_{i-j}^x + \sum_j \beta_{j,k'}^{Tr} Q_{i-j}^{Tr} + \sum_j \beta_{j,k'}^{Ha} Q_{i-j}^{Ha} + \sum_j \beta_{j,k'}^{MD} Q_{i-j}^{MD} + \beta_{k'}^x
\]  \hspace{1cm} (11.1)

where:

$\dot{Q}_{i,h'}^x$ = discharge forecast in place $x$ at $i\Delta t$ for lead time $h'\Delta t$ [L$^3$T$^{-1}$]

$Q_{i}^x$ = measured discharge in place $x$ at $i\Delta t$ [L$^3$T$^{-1}$]

$\beta_{j,k'}^x$ = coefficient for a lead time $h'\Delta t$ applied to $x$ and at $j\Delta t$ before the moment of observation [-]

$\beta_{k'}^x$ = constant for a lead time $h'\Delta t$ [L$^3$T$^{-1}$]

The meaning of the abbreviations of the station names can be found in table 11.1.

Because there is some uncertainty about the moments of the discharge data, and because a flood wave lasts several days, it has been decided to enter only two discharges of each tributary in the regression equation. It is expected that for each station both coefficients have approximately the same value, except for the sign. Then only the change of the discharge is incorporated in the model. For Chooz more coefficients are accepted. After a careful model selection, where attention has been paid to the accuracy obtained, the nature of the available data and the principle of parsimony, models for lead times of 4, 8, 12 and 16 hours were selected. The accuracy of the models is considered to be acceptable. As the wave travel time between Chooz and Borgharen is approximately 16 hours and the lead time required is 24 hours, those forecasts will suffice. A summary is given in tables 11.1 to 11.4. As an example the equation for Chooz for a lead time of 4 h is given:

\[
\dot{Q}_{i,4}^{Ch} = 1.599 Q_{i-4}^{Ch} - 0.778 Q_{i-4}^{Ch} + 0.106 Q_{i-4}^{Ch} + 0.659 Q_{i-4}^{Tr} - 0.617 Q_{i-4}^{Tr} + 0.313 Q_{i-4}^{Ha} - 0.214 Q_{i-4}^{Ha} + 0.541 Q_{i-4}^{MD} - 0.458 Q_{i-4}^{MD} - 2.87
\]  \hspace{1cm} (11.2)

An example of the discharge calculated is given in fig. 11.1.

Because of the uncertainty of the data and the fact that they are not always available in real-time, a simple extrapolation model for the forecast of the discharge at Chooz has also
been derived. That simple model has been described in § 10.2. If the model described in this section causes trouble in the real-time application, the model described in § 10.2 may be shifted to. It should be noted that the accuracy of the model described in this section is superior to that of § 10.2.

11.3 THE BELGIAN MEUSE

11.3.1 HYDRODYNAMIC FLOW ROUTING MODELS

In this section the hydrodynamic flow routing models that will be used for the modelling of the Belgian and Dutch reach of the Meuse are elucidated. More specific information on the modelling of the separate reaches is given in § 11.3.2 to 11.4.5.

At present a 1-dimensional model or a 2-dimensional (2H) model may be chosen. At present computations made with a 2-dimensional model are only made for steady-state situations (Ogink et al., 1986). The long calculation time for non-steady states makes a 2-dimensional model inappropriate for real-time forecasting purposes. Therefore in these sections only 1-dimensional models will be dealt with.

The basis for a hydrodynamic flow routing model is formed by the hydrodynamical equations (St. Venant equations). For the Meuse, several influences may be neglected, as the
wind, the Coriolis acceleration and the change in density. The remaining equations read (Anonymous, 1981):

\[ \frac{\partial Q}{\partial x} + B_e \frac{\partial h}{\partial t} = 0 \]  (11.3)

\[ \frac{1}{gA_f} \frac{\partial Q}{\partial t} + \frac{1}{gA_f} \frac{\partial Q u}{\partial x} + \frac{\partial h}{\partial x} + \frac{Q |Q|}{C^2A_f^2R} = 0 \]  (11.4)

where:

- \( Q \) = discharge [L^3T^{-1}]
- \( x \) = distance [L]
- \( B_e \) = storage width [L]
- \( t \) = time [T]
- \( g \) = gravity acceleration [LT^{-2}]
- \( A_f \) = conveying cross-sectional area [L^2]
- \( h \) = water level (with respect to a horizontal reference level) [L]
- \( u \) = current velocity [LT^{-1}]
- \( C \) = Chézy friction coefficient [L^{0.6}T^{-1}]
- \( R \) = hydraulic radius [L]

Equations (11.3) and (11.4) are respectively called the laws of conservation of mass, and of momentum.

The models used within the Rijkswaterstaat organisation (IMPLIC, ZWENDL, DUFLOW) use the Preissmann implicit scheme for the discretised equations. It is a four-point scheme (fig. 11.2). Water levels and discharges are calculated in the nodes, which are connected by sections.

\[ \begin{array}{c}
\text{Figure 11.2 The Preissmann scheme}
\end{array} \]
That gives the equation set (Anonymous, 1981): 

\[ \frac{Q_{i+1} - Q_{i}}{\Delta x} + B_s \frac{j_{i+1} - j_i}{\Delta t} + \frac{h_{i+1} - h_i}{\Delta t} = 0 \]  

(11.5)

\[ \frac{Q_{i+1} - Q_{i+\frac{1}{2}}}{g A_{f,i} \Delta t} + 2 \frac{Q_{i+\frac{1}{2}} (Q_{i+1} - Q_{i})}{g (A_{f,i}^2) \Delta x} - \frac{Q_{i+\frac{1}{2}} Q_{i+1} (A_{f,i+1} - A_{f,i})}{g (A_{f,i}^3) \Delta x} + \frac{h_{i+1} - h_i}{\Delta x} + \frac{j_{i+1} - j_i}{(C^2 A_f^2 R_f)^{1/2}} = 0 \]  

(11.6)

where:

\[ h_{i,t} = \theta h_{i+1} + (1 - \theta) h_i \]  

(11.7)

\[ h_{i+\frac{1}{2}} = \frac{1}{2} h_i + \frac{1}{2} h_{i+1} \]  

(11.8)

and:

\[ \theta = \text{implicity factor} \]  

\[ \Delta t = \text{time step} \]  

\[ \Delta x = \text{distance between two adjacent nodes} \]

The subscript \( i \) denotes the time serial number, the superscript \( j \) the place sequence. For \( B_s, Q \) and \( C^2 A_f^2 R_f \) and for sub-indexes expressions similar to (11.7) and (11.8) are used.

It should be noticed that steps of time and place in theory need not be equidistant, but the programmes IMPLIC, ZWENDL and DUFLOW can treat only equidistant time steps.

The following data are required:

1. Schematisation.
   a. the scheme of nodes and sections
   b. for each section: the basis levels at both adjacent nodes and the length.
   c. for each section, at several levels above the basis levels: the storage width, the conveying cross-sectional area (at both nodes), the friction coefficient of Chézy and the hydraulic radius.

In most cases the basis levels will be chosen to be equal to the bottom levels. Instead of a Chézy coefficient also a Manning coefficient might be used, but then one coefficient for all the levels must be stuck to in ZWENDL.

Several simplifications in the schematisation are possible, for example by using default values for the hydraulic radius (which then are calculated by using conveying cross-sectional area values).

2. General calculation data.
   It concerns the moments of the start and the end of the calculation, the value of the implicity factor, the switches for respectively the Froude term, iterations, connection with water quality models, etcetera. The nodes for which output is wanted are also
3. Boundary conditions.
   a. At boundaries where the schematisation ends but in reality the water system
does not, so-called boundary conditions should be given. In most cases a time
series containing water level or discharge data will be specified. At several
boundaries a water level-discharge relation is sufficient.
   b. Operation instruction for the weirs.
4. Initial conditions.
   At both ends of each section a water level and a discharge should be specified. Also
the initial levels of the gates should be given.
If desired the results of another computation can be used as a boundary condition and/or
initial condition.

Values of parameters at other water levels than given in 1b and 1c are calculated by linear
interpolation. It looks as if only one interpretation is left (and, therefore, no details are given
in the manuals), but several interpretations are possible. As an example the default
interpolation calculation of the hydraulic radius is elucidated. The default value is calculated
by dividing the conveying cross-sectional area by the perimeter. IMPLIC calculates the
hydraulic radius at both adjacent water levels using the data indicated, and interpolates
afterwards. DUFLOW works the other way around: first it calculates the conveying cross-
sectional area and the perimeter at the desired level, and then it divides them. In this study it
has been tried to avoid that type of interpolation problems, by stating every parameter
explicitly.

For both the Dutch and Belgian reaches, the nodes are located at:
1. gauging stations
2. upstream and downstream from weirs
3. inflows from and outflows into canals and river mouths
4. places where the profile change is discontinuous
5. other places to reduce the length of a section, or to keep the number of sections
   between two weirs over a certain minimum.

11.3.2 MODELLING THE RIVER BED AND THE BOUNDARY CONDITIONS

At present the Belgian reach of the Meuse is being modified. Profile data are available to
Rijkswaterstaat only in a graphical form, and relating to the future situation. Because the
model under development will be used for a period of many years and the modification
process is ending now (early nineties), the schematisation will be based on the future
situation.

In the future, the Belgian Meuse will discharge almost only via the minor bed, even
during most floods (cf. § 5.6).

For the Meuse Namuroise (Namur Meuse, the Belgian Meuse upstream from Andenne, a
suburb of and downstream from Andenne) rough cross section sketches were available for
every hundred metres. They were not available near weirs. The cross sections were
trapezoidal. Those simplified profiles facilitate the development of the schematisation, because
the data at a few levels will suffice (§ 11.3.1, 1c). River reaches with islands are modelled
with two parallel sections.
For the Meuse Liègeoise (Liege Meuse, the Belgian Meuse downstream from Andenelle) only average data of the cross section profiles were available. They have been used to compose the schematisation. For the Dutch reach of the river upstream from Borgharen-Dorp, partly the Lukovic-Barbic (1989) schematisation has been used, and partly a schematisation based on the method described in § 11.4.2.

**Boundary conditions**
The following boundary conditions have been incorporated:
- the discharge at Chooz (measurements or forecasts using one of the methods described in § 10.2 and § 11.2),
- the discharges at the mouths of the rivers Lesse, Sambre, Mehaigne, Ourthe, Ambliève and Vesdre (cf. § 10.3), including a contribution of the residual areas (§ 10.3.8) and
- a stage-discharge relation approximately 500 m downstream from Borgharen-Dorp.
(The station Borgharen-Dorp could not be used as a boundary, because at that location a part of the discharge flows via the Bosscheveld flood way.)
Other flows have not been incorporated explicitly. Their influences are small or relatively constant. As those situations are the same in calibration as in operational use, these flows may be considered to be incorporated implicitly. Most important in magnitude is the Albertkanaal (§ 3.2).
The total number of sections is 179, the number of nodes 161.

Unlike in the Dutch and in the French reach of the Meuse several Belgian weirs function during floods (§ 7.3). The weirs influence the water level artificially, and may influence the shape and the velocity of the wave. Whether the influences are significant will be investigated in section 11.3.3.

11.3.3 ANALYTICAL INVESTIGATION OF THE INFLUENCES OF WEIRS ON FLOOD WAVE ATTENUATION

Weirs control the water levels in the headwater reaches and the discharges passing the weirs, so that the natural discharging properties of a river are changed. The consequences for the propagation of flood waves will be investigated in this section. The analysis will have a general character. The conclusions will only have consequences for the modelling of the Belgian Meuse, and not for the French and Dutch parts. Therefore the subject is dealt with here.

The consequences of the presence of weirs are especially perceptible in stretches where the water levels in the Meuse are influenced from weir to weir. It is especially the case in the Netherlands and Belgium. In general in France the influences of weirs on the water level are perceptible over a relatively short distance. During floods, only the weirs between Namur and the Dutch border are in operation.

In general it may be said that as long as the weirs are in operation the river stays in its minor bed. In what follows it is assumed that the flow width is independent of the water level and is equal to the storage width.

Here 4 situations are investigated (fig. 11.3):
a. without weirs (e.g. the Grensmaas),
b. with weirs with a constant headwater level (e.g. most Dutch headwater reaches),

c. with weirs with a constant level at the most upstream point in the headwater reach (it has been investigated for the reach from the French border to Namur), and

d. a situation between (b) and (c) (it is the situation for most weirs in the stretch between Namur and the Dutch border).

The law of conservation of mass (per unit of width) reads:

\[ \frac{\partial q}{\partial t} + \frac{\partial q}{\partial x} = 0 \]  \hspace{1cm} (11.9)

where:

- \( q \) = discharge per unit of width \hspace{2cm} [L^2T^{-1}]
- \( a \) = water depth \hspace{2cm} [L]

For the situation without weirs (situation a) it applies that \( q \) is only a function of \( a \), so that \( \partial q/\partial x = dq/da \) \( \partial a/\partial x \), and consequently \( \partial a/\partial t + dq/da \) \( \partial a/\partial x = 0 \). The celerity is:

\[ c = \frac{dq}{da} \]  \hspace{1cm} (11.10)

where:

- \( c \) = celerity \hspace{2cm} [LT^{-1}]

With Chézy it follows:

\[ c = \frac{3}{2} u \]  \hspace{1cm} (11.11)

Normative for the celerity is the change of the volume of the water stored. In situations with functioning weirs (situations (b), (c) and (d)) the celerity is dependent on the place in the headwater reach, because the change of the volume stored is dependent on the place. A mean celerity in the headwater reach is defined:

\[ \overline{c} = \frac{L_{hr}}{T} \]  \hspace{1cm} (11.12)

where:

- \( \overline{c} \) = mean celerity in the headwater reach \hspace{2cm} [LT^{-1}]
- \( L_{hr} \) = length of the headwater reach \hspace{2cm} [L]
- \( T \) = wave travel time through the headwater reach \hspace{2cm} [T]

\( T \) is the time that is needed in a headwater reach to change the water volume \( V \) belonging to a discharge \( q \) into the water volume \( V + dV \) belonging to \( q + dq \), if the incoming discharge is changed from \( q \) into \( q + dq \). In an equation:
Figure 11.3 Schematised reproduction of four kinds of weir operation
a. without weirs
b. with weirs and a constant headwater level at the downstream end
c. with weirs and a constant headwater level at the upstream end
d. a situation between b and c.

Meaning of the symbols:

\( a_n \) \quad normal depth belonging to \( q \)

\( a_n + da_n \) \quad normal depth belonging to \( q + dq \)

\( a_n \) \quad water depth belonging to \( q \)

\( a_n + da_n \) \quad water depth belonging to \( q + dq \)
\[ Tdq = dV \]  \hspace{1cm} (11.13)

where:
\[ V = \text{water volume (per unit of width) in a headwater reach} \] [L^2]

For the mean celerity in the headwater reach it follows:
\[ \frac{-L_{hr}dq}{T} = \frac{L_{hr}}{dV} \]  \hspace{1cm} (11.14)

Here the situation without weirs is a special case, because \( da \) is independent of place, so \( dV = L_{hr} da \). Then it can be derived:
\[ \frac{-c}{c} = \frac{L_{hr}dq}{dV} = \frac{L_{hr} dq}{L_{hr} da} = \frac{dq}{da} \]  \hspace{1cm} (11.15)

which is equal to the celerity derived in (11.10).

Now the mean celerities in the headwater reaches of the four situations are compared. The situation without weirs (a) will be the starting point.

Assume that the discharge at the edge of the incoming flow is enlarged from \( q \) to \( q + dq \). The normal depth belonging to it \( a_n \) changes into \( a_n + da_n \). Then the change of the volume stored amounts to \( L_{hr} da_n \). That volume is called \( dV_0 \).

In the situation of a constant headwater level at the downstream end of a headwater reach (b) the change of the volume will be smaller than in the situation without weirs and will roughly be equal to \( 0.5 dV_0 \) in the situation that has been drawn. It results in halving the wave travel time \( T \) and doubling the mean celerity \( c \).

If the headwater level had been higher than drawn in fig. 11.3b, the change in volume would become smaller, because the stretch in which the water level rise is small is increased. Consequently the mean celerity \( c \) would become greater. In the situation of a very high headwater level the change in volume would be very small, and therefore the mean celerity \( c \) very great (eq. (11.14)). In practice the celerity is limited, because among other things the headwater level is limited in height. Besides, the incoming water should be distributed over the headwater reach in a particular way (compare the lines \( a \) and \( a + da \) in fig. 11.3b).

If in situation (b) the headwater level had been lower than drawn, the change in volume would become greater and consequently the mean celerity \( c \) would become smaller.

In case of a constant headwater level (c) the volume of the water in a weir reach decreases when the discharge increases. So \( dV \) will be negative and consequently the wave travel time and the celerity, too. If the weir operation is executed in an optimum way the discharging of an extra volume of water should already have started before the flood wave reaches the headwater reach.

In situation (d) the water volume stored hardly changes. A very short wave travel time will be possible, as a result the celerity may be very great.
Also in the situations (c) and (d) the celerities are limited in real practice.

If the moment of letting pass the increased discharge at the downstream end is later than the wave travel time $T$ after the increase of the discharge at the upstream end, the volume of water stored in the headwater reach is too large as compared to the volume belonging to the new equilibrium situation. The superfluous water should as yet be discharged. It applies to all three situations (b), (c) and (d).

The mean celerity is also limited by the slow reaction of the weir. But if the weirs are caused to respond to discharges further upstream, the wave travel time may be reduced considerably. In theory even a negative wave travel time may be achieved, which means that the flood wave sooner passes the downstream edge of the reach studied than the upstream edge.

**Conclusion**

By means of weirs the wave travel times of the flood waves can be influenced.

In the Netherlands and in France that effect will only count with small flood waves, because already with a relatively small discharge the weirs are opened completely. Therefore the influence of weir operation on the flood waves can be ignored.

Through the changes in the Belgian Meuse an ever larger stretch of the river has been influenced by weirs during floods through the years. The influences of weirs may be significant.

For that reason, some numerical analyses with respect to weir operation have been made for the Belgian Meuse, which are presented in section 11.3.4.

**11.3.4 NUMERICAL ANALYSIS**

The program ZWENDL offers an option to simulate the behaviour of weirs. At the moment of investigation the following simulation options can be applied:

1. an adaptation of the level of the gate in relation with the difference between the level or discharge calculated and desired at some point (the change in height is fixed),

2. the level of the gate given as a time series (which has to be specified in advance), or

3. a time independent relation between the water level or discharge at some point and the level of the gate.

The level of the gate may be seen as: a. height of the gate with a flow between the sill and the gate, or b. width of the opening. At the moment of investigation the option c. height of the gate with the flow over the gate was not available in ZWENDL. In the program all the gates of a weir have to be schematised to one gate.

Option 2 is inadequate for forecasting purposes. Option 3 requires an almost unique relation between discharge and the level of the gate. Investigations showed that that situation cannot be achieved for changing discharges; calculations showed errors of several metres in the calculation of the water level. Therefore option 1 has been selected. However, the first option has two disadvantages: first, the change of the desired water level upstream from the weir as a function of the discharge (§ 7.3) cannot be simulated, and secondly, the adaptation of the gate may proceed too fast (causing unstable calculations) or too slow (causing serious errors in the water level calculated, and causing unstable calculations afterwards). A small time step in combination with a small value of the adaptation of the gate level may neutralise the effect.
The ZWEN DL program can only treat ten weirs with a non-constant gate level. For this investigation the weirs with the greatest striking and lifting discharges were selected. The other weirs are supposed to be opened completely.

Unfortunately the discharges measured are inaccurate and therefore they cannot serve as a basis for calibration and/or verification. However, a sensitivity analysis can be made. Two kinds of influences are analysed by means of an experiment:
1. The influence of the presence of functioning weirs on the wave, and
2. The influence of a water level lowering.

Before the sensitivity analysis can start, friction values of the river sections have to be determined. Because the only information about water levels and discharges is restricted to the discharges when the weirs are opened completely, those data have been used to determine friction values. In most sections the friction values found were common values, but in some sections the friction coefficient of Chézy got values of approximately 30 m³/s.

Due to the absence of calibration data, the sensitivity analysis has to be executed as an academic experiment. Several discharge scenarios have been composed. A survey can be found in table 11.5. It should be noted that discharges before 0 h and after 24 h are the same as at 0 h respectively 24 h, and that at other times they can be obtained by linear interpolation.

**Experiment 1. The influence of the presence of functioning weirs on the flood wave**
For three discharge scenarios the influence of functioning weirs on the wave travel time has been analysed. The experiment shows that the difference in wave travel time is significant. The difference is greater if the discharge is smaller (compare the scenarios 1, 2 and 3 in table 11.6 or fig. 11.4). The influence of the weirs and of the discharges mentioned in the scenarios on the change of the maximum discharges is not significant, because the minimal change in the level of a gate causes a change in the discharge of approximately 10 m³/s. Therefore it has been concluded that the weirs do not significantly influence the wave attenuation in the experiment. (It should be noted that a steeper wave might lead to a more pronounced influence of weirs and discharge on the attenuation. Because a significantly different way is followed then, which is more academic, the result cannot be considered as representative of the river and real-world flood waves.)

The same type of wave applied to the tributaries gives values for the wave travel times between the mouths and Borgharen (table 11.7).

**Experiment 2. The influence of water level lowering**
For Belgian weirs the headwater level is lowered if the discharge exceeds a critical value. Water level lowerings of 10 cm are common. For the weirs between Namur and the Dutch border commonly two more lowerings of 10 cm are to be made for discharges greater than 1000 m³/s. The discharges at which level lowering takes place are listed in tables (not included in this thesis). Numerical experiments have been executed in which a stationary situation is disturbed by a instantaneous lowering of the headwater level of 10 cm. The wave that was caused by the level lowering was insignificant. Also an instantaneous lowering of the headwater level of 50 cm at Andenne-Seilles did not result in a significant wave. One of the causes of the absence of such a wave is that the headwater level desired may instantaneously be lowered 50 cm, but the real water level slowly decreases, because the gates need time to adapt to the new situation.
### TABLE 11.5 A SURVEY OF THE DISCHARGE SCENARIOS

<table>
<thead>
<tr>
<th>Scenario 1</th>
<th>discharge [m$^3$/s] at</th>
<th>Scenario 2</th>
<th>discharge [m$^3$/s] at</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse (Chooz)</td>
<td>400</td>
<td>450</td>
<td>460</td>
</tr>
<tr>
<td>Lesse (mouth)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Sambre (mouth)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Ourthe (mouth)</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Scenario 3</th>
<th>discharge [m$^3$/s] at</th>
<th>Scenario 4</th>
<th>discharge [m$^3$/s] at</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse (Chooz)</td>
<td>1200</td>
<td>1250</td>
<td>1260</td>
</tr>
<tr>
<td>Lesse (mouth)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Sambre (mouth)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Ourthe (mouth)</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Scenario 5</th>
<th>discharge [m$^3$/s] at</th>
<th>Scenario 6</th>
<th>discharge [m$^3$/s] at</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse (Chooz)</td>
<td>800</td>
<td>800</td>
<td>800</td>
</tr>
<tr>
<td>Lesse (mouth)</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Sambre (mouth)</td>
<td>200</td>
<td>250</td>
<td>260</td>
</tr>
<tr>
<td>Ourthe (mouth)</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
</tbody>
</table>
The influences found do not agree with current discharge measurements at Borgharen, as the discharge at Borgharen fluctuated significantly, even during floods. The following explanations can be found:

- The weir operation is still the responsibility of the weir keeper. He has contacts with his colleagues (cf. § 7.3). The operation will probably be coordinated, thus accumulating and possibly enforcing influences on the discharge. The unknown course of the discharge in the nearby future will possibly play a role.

- Because the weirs are unmanned at night and the gate levels may not be influenced between 06.00 and 08.00 a.m., the operation strategy is not constant in time. Actions that anticipate the unmanned period may also influence the discharge.

Because the operation of the weirs is dependent on many influences, among which human ones, many of which are unknown in real-time, and because the ZWENDL program is very

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**Figure 11.4 The influence of functioning weirs and discharge on the wave travel time**

**Table 11.6 Influence of Weirs and Discharges on the Wave Travel Time between Chooz and Borgharen**

<table>
<thead>
<tr>
<th>Discharge scenario</th>
<th>Flow at Borgharen at $t = 0\ h$ [m$^3$/s]</th>
<th>Wave travel time (Chooz-Borgharen) with functioning weirs [h]</th>
<th>Wave travel time (Chooz-Borgharen) with functioning weirs completely opened [h]</th>
<th>Maximum discharge at Borgharen with functioning weirs [m$^3$/s]</th>
<th>Maximum discharge at Borgharen weirs completely opened [m$^3$/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1200</td>
<td>15</td>
<td>20</td>
<td>1262</td>
<td>1250</td>
</tr>
<tr>
<td>2</td>
<td>1600</td>
<td>16</td>
<td>19.5</td>
<td>1653</td>
<td>1648</td>
</tr>
<tr>
<td>3</td>
<td>2000</td>
<td>17</td>
<td>17</td>
<td>2050</td>
<td>2040</td>
</tr>
</tbody>
</table>
### TABLE 11.7 WAVE TRAVEL TIME BETWEEN MOUTHS OF RIVERS AND BORGHAREN

<table>
<thead>
<tr>
<th>river</th>
<th>discharge scenario</th>
<th>wave travel time to Borgharen with functioning weirs [h]</th>
<th>maximum discharge at Borgharen with functioning weirs [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse (Chooz)</td>
<td>2</td>
<td>16</td>
<td>1653</td>
</tr>
<tr>
<td>Lesse (mouth)</td>
<td>4</td>
<td>12</td>
<td>1652</td>
</tr>
<tr>
<td>Sambre (mouth)</td>
<td>5</td>
<td>7</td>
<td>1655</td>
</tr>
<tr>
<td>Ourthe (mouth)</td>
<td>6</td>
<td>5</td>
<td>1657</td>
</tr>
</tbody>
</table>

limited with respect to the simulation of weirs, it has been decided not to try to simulate the exact behaviour of the weirs, but to be satisfied with the developed schematisation. It has consequences for the discharge forecast at Borgharen-Dorp (§ 11.3.5).

**Sensitivity analysis of the friction coefficient on the flood wave**

Because the friction values have to be based on weak arguments a sensitivity analysis has been performed. Two alternative schematisations have been developed, one with all the friction coefficients 5 m³/s higher than the original set, and one with all the coefficients 5 m³/s lower than the original set. The wave, described before in scenario 2, was used as an input. The results are given in table 11.8. It is shown that the influence on the wave travel time is significant. The influence of the friction coefficient on the maximum discharge is small. Because the shape of the wave is relatively flat, and the results could not be verified, an intensive analysis of the friction coefficient has not been executed.

### TABLE 11.8 INFLUENCE OF THE FRICTION COEFFICIENT ON THE WAVE TRAVEL TIME AND THE MAXIMUM DISCHARGE

<table>
<thead>
<tr>
<th>friction coefficients (with respect to the original schematisation)</th>
<th>discharge scenario</th>
<th>Wave travel time (Chooz-Borgharen) with functioning weirs [h]</th>
<th>Maximum discharge with functioning weirs [m³/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 m³/s higher (smoother) original</td>
<td>2</td>
<td>14</td>
<td>1655</td>
</tr>
<tr>
<td>5 m³/s lower (rouglier) original</td>
<td>2</td>
<td>16</td>
<td>1653</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>18</td>
<td>1648</td>
</tr>
</tbody>
</table>
Conclusions
The weirs cause a reduction of the wave travel time. The reduction decreases if the discharge increases. An increase of the peak discharge due to the influence of weir operation could not be confirmed by means of the ZWENDL program. However, as the wave travel time between Chooz and Borgharen reduces, the chance of coinciding peak discharges from Chooz and the Ourthe increases. A quantitative analysis of the magnitude of the change is beyond the scope of this study.

The weir construction and its operation are too complex to model them exactly with ZWENDL. A forecast of the weir operation requires a lot of information.

11.3.5 THE MODIFICATION OF THE DISCHARGE FORECASTS AT BORGHAREN

The ZWENDL program calculates discharges at Borgharen-Dorp. The discharges will differ from the discharges measured, among other things because of:
- weir operation (§ 7.3 and § 11.3.3),
- systematic differences between Belgian and Dutch discharge measurements (Example: In 1988 the difference between the peak discharge at Lixhe and Borgharen amounted to 161 m³/s, being 2180 m³/s and 1919 m³/s respectively. The difference is a multiple of the net inflow into the stretch in between.),
- errors in the approximation of the discharge of the ungauged areas (§ 10.3.8), and
- model and schematisation errors, as inflows from and outflows to canals.

The causes of the errors are known, and therefore their properties. The fluctuations caused by the operation of weirs has a period of several hours, the differences in Belgian and Dutch measurements can be considered to be a function of the discharge only, the error in the approximation of the discharge of the ungauged areas as well as model and schematisation errors will be small compared to the first two causes.

It is necessary to have a model that modifies the calculated discharge of the ZWENDL program into a good forecast. Therefore an adaptation model has been developed. Unfortunately a calibration period is missing. It means that a correction model must be based on the limited knowledge that is available (and that has been presented in this thesis).

The most important errors are periodic of character or relatively constant within a short reach of time. Therefore the following correction model will be appropriate:

\[ \dot{Q}_{i,h'} = Q^*_{Z,i-h'} + G_i \]  

(11.16)

where:

\[ \dot{Q}_{i,h'} \] = discharge forecast for \((i+h')\Delta t\) made at \(i\Delta t\) \([\text{L}^3\text{T}^{-1}]\)

\[ Q^*_{Z,i-h'} \] = discharge calculated by ZWENDL for \((i+h')\Delta t\) \([\text{L}^3\text{T}^{-1}]\)

\[ G_i \] = discharge correction for forecasts made at \(i\Delta t\) \([\text{L}^3\text{T}^{-1}]\)

\(G_i\) is calculated by:

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\[
G_i = \frac{\sum_{j=0}^{m} Q_{i-j} - Q_{i-1}^*}{m+1}
\]

where:

\( Q_{i-j} \) = discharge measured at \((i-j)\, \Delta t\) \( [L^3T^{-1}] \)

\( m \) = positive integer constant \([-]\)

After an inspection of the character of the fluctuations and the desire to minimise \( m \) in order to be as actual as possible, \( m \) is chosen to be 8, corresponding to 8 hours.

Equation (11.16) will lead to a difference between the discharge 'forecast' with a lead time of zero and the discharge measured at the moment of observation. The effect has two disadvantages:

a. The user of the model will observe a difference between the discharge measured and the forecast value already at the moment of observation. The conclusion that the total forecast is of a poor quality is easily made. To prevent that conclusion a proper instruction of the forecaster is necessary.

b. Up to the moment of observation measured discharges will serve as the input for the Dutch reach of the river. After the moment of observation the discharge forecasts will be used. Then the discharge may change significantly at the moment of observation. Therefore the change can generate unstable calculations for the Dutch reach of the river. To avoid it, the discharge input file for Borgharen is adapted for the first 4 hours after the moment of observation: during that period the input is determined by a linear interpolation of the discharge measured at the moment of observation and the discharge forecast (equation (11.16)) for a lead time of 4 hours. The value of 4 hours is chosen as an acceptable minimum.

In real-time the modified discharge forecast for Borgharen (11.16) is the forecast that is presented by the model.

11.4 THE DUTCH MEUSE

11.4.1 INTRODUCTION

For several decades it has been tried to forecast the maximum water levels and the travel times of a flood in the Dutch reach of the Meuse. The start of the calculation was always the maximum water level at Borgharen.

Most popular were the stage relation curves which gave a graphical relation between the maximum water level at Borgharen and the maximum water levels at other stations. The graphical relation was subjectively drawn on the basis of earlier floods. The method is fast, simple and only needs data of the maximum water level at Borgharen. One of the drawbacks was that the form of the wave and the discharges of the tributaries were not dealt with.

At the time hydrodynamic flow models became available a schematisation for the Dutch part of the Meuse was developed using the IMPLIC-model. That was around 1978. At the time the problem was the lack of calibration floods: the last had been in 1970. Since that time some parallel sections have been added. In order to make the schematisation also usable for water quality calculations for normal discharges, canals and smaller tributaries were
added. In essence the schematisation used in 1989 was the same as in 1978.

In the eighties the friction coefficients were re-calibrated to obtain good results. Also the effect of storage of water in the banks was investigated. It has led to an artificial widening of the storage width. Twelve years after the development of the original schematisation it was felt desirable to make a new start with recent measurements of the river and its flood plains. But there was another good reason for making a new schematisation. Due to a lack of computation capacity, the original schematisation was restricted to a limited number of nodes and sections. Therefore no distinction could be made between the minor bed and the flood plains. A section contained the minor bed as well as the flood plains. That led to extraordinary values for the friction coefficients. Also it was supposed implicitly that the flowing water in the flood plains would have the same velocity as in the minor bed. Nowadays the artificial combination need not be made any more, as computation limits are wider.

The aim of modelling the Dutch part was to make an accurate and reliable flood routing model that is able to forecast the peak water level at measuring stations with an accuracy of the same order as the measuring error, once the discharge at Borgharen is known. It should be noted that nowadays the gauges, which were the common measuring instruments during the calibration and verification periods and have a measuring error of approximately 10 cm during floods, are replaced by stations with stilling wells. Their measuring error is in the order of 2.5 cm (Van der Made, 1980).

At the moment of the present study the IMPLIC-model was out of date. The successor of that model in Rijkswaterstaat is ZWENDL. However, ZWENDL was only available on the mainframe in a user-unfriendly form. Therefore, most calculations were done using the personal computer model DUFLOW. The model can also be seen as a successor of IMPLIC. After the calibration the schematisation was transferred to ZWENDL. A general description of the models DUFLOW and ZWENDL has been given in § 11.3.1.

Because of the different types of flow it has been decided to separate the minor bed and the flood plains. Four alternatives are considered (fig. 11.5):

1. Two parallel sections, no parallel nodes.
2. Three parallel sections, no parallel nodes. In this type of schematisation a distinction is made between the left and the right bank.
3. Two parallel sections with parallel nodes. This type of schematisation creates the possibility of a realistic modelling of the summer dikes which separate the minor bed and the flood plains, and are present over a long part of the river.
4. Three parallel sections with parallel nodes.

The alternative 1 has the smallest number of nodes and sections, alternative 4 the greatest. Despite the computation capacity available at present, the number of nodes and sections is still limited. Besides, it should be kept in mind that in real-time the computation time should be as short as possible. That has been the reason to abandon alternative 4.

The connection of the minor bed with the flood plains by sections (alternative 3) has also been abandoned. The connecting sections would be very short, and in the sections the water velocity would be great. It might cause unstable computations, which must be avoided, especially for real-time models. Besides, for the important and higher water levels the importance of the summer dikes decreases. Another reason for abandoning alternative 4 is the large number of extra nodes and sections: approximately 50% more sections and approximately 100% more nodes compared to alternative 1.

Most important in the schematisation will be the division of the major bed into a minor bed and a flood plain. An extra division between the left bank and the right bank adds
only little to the accuracy, especially in alternative 2 where all the water flows along the same nodes. Therefore also alternative 2 has been abandoned.

The schematisation has been executed according to alternative 1, because the schematisation makes the desired distinction between minor bed and flood plains, gives the greatest possible guarantee for a stable computation and requires the smallest number of nodes and sections as well as the shortest computation time.

### 11.4.2 MODELLING THE MINOR BED, THE FLOOD PLAINS AND THE BOUNDARY CONDITIONS

The modelling started for a 29 km long reach downstream from Borgharen-Dorp. The research section has been used to develop a proper way of modelling. Later on, the same technique was used to develop the schematisation of the rest of the Dutch reach of the Meuse. The positioning of nodes and sections is done according to alternative 1 in § 11.3.1.

First the minor bed has been modelled. Recent measurements of the bottom level were available. The data contained bottom level measurements every 5 m in a profile perpendicular to the flow direction of the river. For every 100 m such a profile was available. However, it was not obvious how these measurements should be converted into a section where, for example, the bottom level can be given only at two points. That type of question appeared several times. In more general terms: How should a lot of data be represented by a few data without losing the hydraulic properties?

In order to solve that question several simple algorithms have been developed, which depend more on good bookkeeping than on scientific model development. Because many exceptions have to be dealt with (data in irregular places, different lengths of sections, locally low bottom levels) the expressions used look complex. A description of the modelling technique used can be found in Bos et al. (1989). Here only the problem of the bottom level will be elucidated.

The lowest point in a cross section is the bottom level and in principle also the basis level. Due to the characteristics of the river, the bottom level has a stochastic component (fig. 11.6). Therefore the bottom level is less appropriate to serve as a basis for the remaining parameters: its gradient may differ significantly from that of the water level. Therefore the basis level has been determined by means of other data than the bottom level.
Figure 11.6 The lowest level in each cross section. The level is not representative of the gradient of the bottom level.
The level at which the width is 60 m is seen as a more suitable basis for the remaining parameters. 60 m is about half the width of the river at the water surface under normal conditions. That level will be called the 60 m width level. Furthermore to reduce the stochastic influence the mean values of those levels 500 m upstream and downstream from a node are calculated. The value of 500 m has been chosen in order to have a certain minimum length, and can be considered to be representative of the node. If the distance between two nodes was less than 500 m, the distance has been reduced in order to prevent one measured bottom level from serving in the calculation of the basis level of two nodes. The schematised basis level is situated 3 m below the 60 m width level. The cross-sectional areas for the lowest levels may be equal to zero, because the basis level is situated below the bottom level. For very low water levels it leads to unstable computations. Therefore the storage widths and the conveying cross-sectional areas have been enlarged, corresponding to a conveying width of approximately 1 m (fig. 11.7). Those artificial areas will be called gutters. (It should be noted that the schematisation has been developed for floods in particular, but it should also be a basis for a schematisation for all flow conditions. Between the moment of the development of the schematisation and the writing of this thesis, the schematisation was used for that purpose, indeed (Barneveld et al., 1991).) The gutters are not included in the calculation of the hydraulic radius (fig. 11.7).

The dimensions of the weirs have been included in the model. They are considered to be completely opened.

The data of the flood plains were available in another way than those of the minor bed. Maps containing the topography of the flood plains and inundation maps served as a basis for the determination of the flood plain schematisation. For approximately 8 water levels (it depends on the section considered) the storage widths were determined. From those widths the cross sections were calculated. The cross sections had to be modified before they could be entered in the model as flow cross sections, because, in contrast to the minor bed, the volume of stored water does not contribute completely to the flow (Photo 11.1). Research with a two-

![Figure 11.7 The gutters in the minor bed and the flood plains. Only the area surrounded by a heavy line was incorporated in the calculation of the hydraulic radius of the minor bed](#)
dimensional (2-H) model WAQUA showed that in some areas the water in the flood plains would flow from nearly a stand-still to fast with an increasing water level (Bastings, 1989). The transition takes place gradually. However, the model ZWENDL accepts only two alternatives for a certain area: a flow under the same conditions as the other areas in the flood plain or being stagnant. With the help of the WAQUA calculations for each section the part of the cross section which contributes to flood plain flow was determined.

The model accepts only situations in which there is some water flowing through each cross section. It means that also with low flows some water must be present in the flood plain section. That is far beyond reality, the flood plains are only inundated during floods. The solution found is providing the flood plain sections with gutters (fig. 11.7). The gutters have their bottoms at the basis levels of the minor bed. The perimeter has been calculated excluding the gutters, just as for the minor bed.

The following boundary conditions have been applied:
- The upstream boundary is the discharge at Borgharen-Dorp. However, as the flow at that point is divided into the main stream and a stream via the Bosscheveld weir, the input has been moved to the bifurcation point, 600 m upstream from the Borgharen-Dorp gauge. The wave travel time between those points is neglected.
- The discharge at the mouth of Roer and Niers.
- A constant water level several kilometres downstream from the weir of Lith. It should be noticed that in the first instance a stage-discharge curve was used at the weir of Lith, but that the neglect of the hysteresis effect in the calculation caused significant errors.

Other flows have not been incorporated explicitly. Their influences are small, relatively constant or are a factor of other flows that are incorporated explicitly. As those situations are the same in the calibration as in operational use, those flows may be considered to be incorporated implicitly. The most important flows, in magnitude, are:
- Julianakanaal (16 m³/s)
- Lateraal Kanaal (8 m³/s)
- Kanaal Wessel-Nederweert (1 m³/s)
- Small rivers (sum approximately 10-20 m³/s)
- Rain and evapotranspiration (directly on/from the river water).

The total number of sections is 226, the number of nodes 136. In the numbers the groundwater units as described in § 11.4.4 are included. The calibration started with two floods: those of February 1984 and of November 1984.

Before the calibration could really start, several stability problems had to be solved.
1. Great changes of the conveying cross-sectional area between both ends of a flood plain section had to be smoothed.
2. Great changes in the conveying cross-sectional area, and in the storage width as a function of water level had to be smoothed. That problem often occurred in the flood plain sections, at the top end of the gutter. The problem was solved by smoothing the transition from the gutter to the flood plain.
3. It seems that weirs may initiate unstable computations due to the different type of equation involved. The stability of the calculation was influenced in a positive way by coupling the node upstream and the node downstream from a weir in another way than had been done for the rest of the nodes: The minor bed and the flood plains were disconnected.
After those modifications the calculations using uncalibrated schematisation were stable. The results of the calibration will be described after the analysis of the influence of groundwater flow on flood levels.

11.4.3 THE INFLUENCE OF THE BANK STORAGE ON THE PEAK FLOW

The flood-peak attenuation plays an important part in many rivers. The influences of the cross section, the gradient, the shape of the wave etc. on the attenuation are commonly well-known. In general much less is known about the contribution of the groundwater. The causes of it are: the fact that many data needed such as the transmissivity and the entrance resistance are hardly known; that a different discipline is required (geo-hydrology instead of hydrodynamics); and the fact that the influence of the groundwater is not clearly demonstrable in the discharges.

From a theoretical investigation Hunt (1990) found that the bank storage effect may be of a significant influence. It is confirmed by Breukel et al. (1990), who for the river Rhine found a mean bank storage flow rate of 1 m³/km/s during the rising limb of the flood of 1988 at Urmitz and Bonn. The flow rate has been determined from groundwater level data.
In this section it is investigated whether the bank storage effect is significant for floods in the Meuse.

The influence of the bank storage on the flood-peak attenuation is greater as the transmissivity is greater, the changes of the water level are greater, the entrance resistance is smaller and the surface area on which the exchange between surface water and groundwater takes place is larger. In the Meuse catchment great influences may be found especially in the Grensmaas between Borgharen and Maaseik. In that stretch the transmissivity of the aquifer reaches values up to 5,000 m²/day. As there are no weirs in the stretch the differences of the water level in time are greatest. For that reason the subject is dealt with in this section (§ 11.4).

Near the Grensmaas the groundwater level proves to be very sensitive to changes of the water level of the river, which is clear if the water levels of the river at Borgharen and the water levels in the observation well 61F-174 are compared. The observation well is located between Borgharen and Itteren at approximately 250 m from the (minor bed) bank of the Meuse. The thickness of the covering layer is approximately 2 m. In fig. 11.8 the course of the water level of the river, the groundwater level and the precipitation are shown. (Note: from a random test it appears that the water levels of the river do not completely correspond to the levels mentioned in Anonymous: Jaarboeken der Waterhoogten.) From the figure it can be read that the groundwater level follows the water levels in the Meuse well and that the precipitation in the area considered plays a less important part (Van der Heijde et al., 1980).

From a simple consideration of the continuity (using the measured groundwater levels and the distance from the observation well to the river) it can be deduced that the contribution of the bank storage due to flood-peak attenuation might be of importance.

If an area is inundated the groundwater will be replenished much faster than if the inflow comes wholly from sideward infiltration. Therefore a distinction is made between the sideward (mainly horizontal) infiltration from the minor bed and the vertical infiltration from the flood plains, caused by flooding. The terms horizontal and vertical are applied here because they reflect the kinds of mechanisms involved best.

Apart from the influence of the river level, the groundwater level at Borgharen is also influenced by the precipitation. However, the contribution of the precipitation to the groundwater will result in a rise of the water level of no more than a few centimetres. Therefore, the influence of the precipitation has been neglected in the following analysis.

The constant component of the inflow has been left aside. The same applies to the extractions from the groundwater, which in comparison with the flood discharges are negligible. In the analysis it will be supposed that at the beginning of the period considered the groundwater levels are exactly equal to the water level in the Meuse. It is permitted, because here only an investigation into the order of magnitude of the contribution of the bank storage during flood waves is concerned.

The calculation was done for a profile near Borgharen.

In order to investigate the time dependent phenomenon a simple numerical simulation has been executed.

Opposite:

Figure 11.8 The course of the precipitation near Maastricht and the water level and the piezometric level at Borgharen. Source: Van der Heijde et al. (1980)
The starting point is one single section perpendicular to the river axis (fig. 11.9). The transmissivity and the storage coefficient are supposed to be constant. The entrance resistance has been schematized in a simple way by means of an additional width (the shaded area in fig. 11.9). The water that is stored in the "imaginary" width does not count in the calculation of the inflow and the outflow.

The equations used read:

\[ \frac{\partial q}{\partial x} + \mu \frac{\partial H}{\partial t} = 0 \]  
\[ q = -kD \frac{\partial H}{\partial x} \]  

where:

\[ q \quad \text{flow per unit of width} \quad \left[ \text{L}^2 \text{T}^{-1} \right] \]
\[ H \quad \text{water level with respect to the horizontal impermeable base} \quad \left[ \text{L} \right] \]
\[ kD \quad \text{transmissivity of the subsoil} \quad \left[ \text{L}^2 \text{T}^{-1} \right] \]
\[ \mu \quad \text{storage coefficient} \quad [-] \]

Combining (11.18) and (11.19) yields:

\[ \frac{\partial H}{\partial t} = \frac{kD}{\mu} \frac{\partial^2 H}{\partial x^2} \]  

For \( x = 0 \) it applies that \( H \) is equal to the water level of the river. The numerical calculation is done in accordance with the explicit calculation scheme as:

\[ \frac{H_{i+1}^j - H_i^j}{\Delta t} = \frac{kD}{\mu \Delta x^2} \left( H_{i+1}^{j-1} - 2H_i^j + H_{i-1}^{j-1} \right) \]  

Here the subscript indicates the time and the superscript indicates the place.

In the simulation \( kD \) has been chosen to be 5,000 \( \text{m}^2/\text{day} \) and \( \mu \) to be 0.3. The time step used in the calculation amounts to 1 hour, however, in the figures a time step of 24 hours has been used. The grid distance \( \Delta x \) is 50 m. The calculation scheme complies with the stability criterion, reading:

\[ 2 \frac{kD}{\mu} \frac{\Delta t}{\Delta x^2} < 1 \]  

Here that value is equal to 0.56.

An investigation has shown that the entrance resistance amounts to approximately 1 day. Here the imaginary width has been taken equal to \( 5 \Delta x = 250 \text{ m} \), a value that simulates the entrance resistance well.

As an initial condition the groundwater level has been chosen equal to the water level of the river. Because of this supposition it follows that no conclusions must be drawn about
the total quantities of incoming and outgoing groundwater, but only about the quantities of water due to flood waves.

To a great extent the volume of water that disappears from the river as groundwater, is influenced by the extent of the inundations. That is, because in the case of infiltration caused by flooding, both the surface and the gradient are much greater than in the case of sideward infiltration only. In the case of an inundation the groundwater levels in the inundated area are raised to the water level of the river.

The incoming and outgoing flows are determined by means of the continuity equation:

\[ q_i = \frac{2\mu}{\Delta t} \left( \sum_{j=1}^{n} \Delta x (H_j^i - H_j^{i-1}) + \frac{1}{2} \Delta x (H_0^i - H_0^{i-1}) \right) \]  (11.23)

In the equation \( H \) is limited by the local land surface. The factor 2 in the equation is due to the fact that the inflow and the outflow come both from the left bank and from the right. The schematisation has been extended up to 3,750 m from the minor bed bank. At that distance hardly any changes of the water level occur owing to flood waves, according to the calculation.

For the first few months of 1978 a simulation of the groundwater flow has been made,
starting from the water levels at Borgharen-Dorp. The results of the calculation compare rather well with the values in fig. 11.8. It should be noted that in the simulation it concerns an observation well 250 m from the river near Borgharen, whereas the observation well 61F.174 is a few hundred metres further downstream (mean water level of the river approximately 1 m lower). The sudden drop of the groundwater level at the beginning of 1978 cannot well be explained by means of the model.

The greatest groundwater flow is directed inland and occurs some time before the peak, when the water level of the river is still rising fast and the flood plains are being inundated. As soon as a part of the land is inundated the bank storage flow and the
groundwater level are strongly changed as compared with the situation before the inundation.

Apart from the period of 1978 mentioned, calculations have been made for the floods of February and November 1984 (figs. 11.11 and 11.12). The February flood is characterised by the water level rising irregularly. With the wave a considerable part of the flood plains was inundated. It resulted in a great increase of the volume of water stored in the freatic aquifer.

During the November flood of 1984 the rise of the water level was very fast. It resulted in great gradients of the groundwater level and because of that in a sideward infiltration into the ground. If the results are compared with an antecedent wave of the
February flood, it is seen that the inflow differs more than 1 m³/s/km, whereas the maximum groundwater level calculated is significantly lower.

On account of a different previous history of the water levels the infiltration may differ by approximately 1 to 2 m³/s/km. For the Grensmaas it produces a difference of approximately 50 m³/s, which corresponds to a difference of the water level of 0.1 m near Roermond. Whether the difference will increase further downstream cannot be told beforehand, because downstream the changes of the water level are slower, and therefore the infiltration flows will be less there than in the Grensmaas.
It should be noted that a calculation of the contribution of groundwater for the Borgharen Maaseik stretch turned out to be unsuccessful, at least if it is the intention to explain the difference of the discharges at Borgharen and Maaseik also on account of the groundwater.

To a great extent the point of time at which the inflow into the ground is greatest depends on the changes of the river level and the size of the inundations.

If only the sideward infiltration is considered (no inundations), the greatest intake of water will take place when the gradient of the piezometric level of the groundwater is at its maximum. In general it occurs some time before the maximum water level of the river is reached; just before reaching the peak level of the river the rise of the river water level is only small in time, and near the peak it is even zero. Therefore, at the moment of the peak of the river water level, the infiltration decreases in size. Consequently the moment of the maximum sideward infiltration must occur before the peak.

If on account of the flooding the bank storage is of special importance, the bank storage is largely connected with the surface of the inundated area. Analogous to the foregoing argumentation it may be stated that as a result the maximum value of the infiltration occurs in general before the peak.

In other places along the Meuse, too, flood-peak attenuation takes place because of the infiltration into the groundwater.

In the Meuse Lorraine the flood-peak attenuation is only influenced by the bank storage effect to a limited extent, as the groundwater level in the valley before the peak is not far below the land surface.

In the Meuse Ardennaise in France and in Belgium south of Namur the valley is much narrower and the changes of the water level during floods are greater. Owing to the narrow valley the groundwater levels nearly run in phase with the water levels in the river. Therefore the infiltration will only be small near the peak. After the renewal of the Belgian weirs the differences of the water levels during flood waves will decrease, and so will the resulting bank storage effect.

As has been argued before, the intake of water will be greatest in the case of the Grensmaas. More downstream it will be smaller, because of the smaller transmissivity (approximately 2,500 m²/day) and the less smoother peak.

For reasons of completeness it should be remarked that the Dutch hydrodynamic flow model of 1986 takes into account the bank storage by means of an enlarged storage width of the river. It means that the real storage widths have been enlarged in order to be able to simulate the bank storage. The great advantage of the choice is that the structure of the model need only be altered in some places. However, the disadvantages are that the contribution of the groundwater has been simulated for an idealised situation and that the water flows off equally fast as it has come. With constant water levels no decrease of the discharge occurs in the model. However, at the peak the decrease of the discharge in accordance with the simulation performed in the section before, may amount to the order of 5 m³/s/km, from which it follows that the schematisation used in IMPLIC is rather rough.

Conclusions
1. The volume of water to be received by the ground is influenced by a high or a low water level in the period before the flood wave. For the Grensmaas it may cause differences in the inflow to the groundwater of up to over 1 m³/s/km. From it arise
deviations in the water level of the river near Maasbracht of a maximum of the order of one decimetre. That is why it is desirable to take that aspect into account when making forecasts. For the other parts of the Meuse the differences are much smaller. Therefore bank storage modelling may be limited to the Grensmaas.

2. The floodrouting model of 1986 takes the groundwater into account by means of an increased storage width. With the modelling certain deviations are created, for example a flow to the groundwater is lacking if the water levels of the river remain constant. In the model of 1986 the deviation is partially compensated for by the adaptation of the friction, so that continuity and momentum are mixed in an incorrect way.

3. The intake of groundwater is also influenced by the high or low water levels in the period before the flood wave. Through the construction of weirs and river modification the differences of the water levels in a large part of the Meuse have become smaller. Owing to it the storage in the groundwater has decreased, so that the frequency distribution of peak discharges and peaks of the water level has been changed. In general it may be said that since the construction of the first weirs the volume of water of floods that is stored in the form of bank storage has decreased.

11.4.4 MODELLING GROUNDWATER FLOW USING A HYDRODYNAMIC FLOW MODEL

Many hydrodynamic flow models have extensive possibilities to calculate the water motion through a canal system. However, in most models the interaction between river water and groundwater cannot be simulated, although interaction nearly always takes place. Ignoring the interaction might be acceptable in many cases. In § 11.4.3 it has been shown that for some parts of the river the interaction is significant. In this section it is investigated if and how a hydrodynamic flow model can be used to simulate the interaction. Before starting it should be stated that even a rough estimation of the groundwater flow is considered to be acceptable, as long as that rough estimation is better than an artificially increased storage width.

The direction of the groundwater flow is almost perpendicular to the flow direction of the river. The groundwater model will therefore contain sections directed perpendicularly to the flow direction of the river. Groundwater flow parallel to the river flow is ignored. As the number of sections in ZWENDL is limited, only a minimum number of sections will be used for the simulation of groundwater flow. Two sections are considered to be a minimum, one to represent Darcy's flow equation and one to represent storage (fig. 1.13).

It is noted that the word section will always be used in the terms of surface water and hydrodynamic flow models, and never in the groundwater meaning of geo-hydrologic profile. Variables with an accent breve are related to the simulation model.

For reasons of simplicity the storage in the flow section and the flow in the storage section will be neglected.

The continuity equation for the flow section reads:

$$\frac{\partial \bar{q}_f}{\partial x} = 0$$  \hspace{1cm} (11.24)

where:
Figure 11.13 A simple schematisation to model the interaction between river water and groundwater

\[ Q_f = \text{flow in the flow section} \quad [L^3T^{-1}] \]

\[ x = \text{distance along the axis of the groundwater flow sections} \quad [L] \]

The continuity equation for the storage section reads:

\[ Q_s = W_s \frac{dh_s}{dt} \quad (11.25) \]

where:

\[ h_s = \text{water level in the storage section} \quad [L] \]

\[ Q_s = \text{flow into the storage section} \quad [L^3T^{-1}] \]

\[ W_s = \text{storage area of the storage section} \quad [L^2] \]

\( W_s \) may be a function of the water level \( h_s \) (fig. 11.13).

Because the storage section and the flow section are adjacent, \( Q_f \) in equation (11.24) and \( Q_s \) in equation (11.25) are equal in value:

\[ Q_f = Q_s \quad (11.26) \]

The groundwater flow is determined by the gradient of the piezometric level and the friction (represented by the reciprocal value of the conductivity). It is therefore a logical choice to try to simplify the equation of momentum to an equation with only those terms. Then for the flow section the equation of conservation of momentum reads:
\[ \frac{\partial \tilde{h}_f}{\partial x} = -\frac{\tilde{Q}_f |\tilde{Q}_f|}{C_f^2 \tilde{A}_f^2 \tilde{R}_f} \]  

(11.27)

where:
\( \tilde{h}_f \) = water level in the flow section [L]
\( \tilde{C}_f \) = Chézy coefficient of the flow section [L^{0.5}T^{-1}]
\( \tilde{A}_f \) = conveying cross-sectional area of the flow section [L^2]
\( \tilde{R}_f \) = hydraulic radius of the flow section [L]

In the first instance it is assumed that \( C_f^2 \tilde{A}_f^2 \tilde{R}_f \tilde{L}_f \) and \( \tilde{W}_s \) are not a function of the water level. Then it can be deduced from (11.27):

\[ \frac{\tilde{h}_s - h_r}{\tilde{L}_f} = -\frac{\tilde{Q}_f |\tilde{Q}_f|}{C_f^2 \tilde{A}_f^2 \tilde{R}_f} \]  

(11.28)

where:
\( \tilde{L}_f \) = length of the flow section [L]
\( h_r \) = water level in the river [L]

For the groundwater flow in the flow section it gives:

a. if \( h_r \geq \tilde{h}_s \):

\[ \tilde{Q}_f = \sqrt{\frac{h_r - \tilde{h}_s}{\tilde{L}_f}} C_f^2 \tilde{A}_f^2 \tilde{R}_f \]  

(11.29)

b. if \( h_r \leq \tilde{h}_s \):

\[ \tilde{Q}_f = -\sqrt{\frac{\tilde{h}_s - h_r}{\tilde{L}_f}} C_f^2 \tilde{A}_f^2 \tilde{R}_f \]  

(11.30)

If \( h_r \) is constant and equal to \( h_R \) the course of \( \tilde{h}_s \) in time can be calculated easily. For \( h_R > \tilde{h}_s \):

\[ \tilde{W}_s \frac{d \tilde{h}_s}{dr} = \sqrt{\frac{h_R - \tilde{h}_s}{\tilde{L}_f}} C_f^2 \tilde{A}_f^2 \tilde{R}_f \]  

(11.31)

Then:

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\[ \frac{d\tilde{h}_s}{\sqrt{h_R - \tilde{h}_s}} = \sqrt{\frac{C_f^2 A_f^2 \tilde{R}_f}{L_f \tilde{W}_s}} \, dt \]  

(11.32)

Integration gives:

\[ -2 \sqrt{h_R - \tilde{h}_s} = \sqrt{\frac{C_f^2 A_f^2 \tilde{R}_f}{L_f \tilde{W}_s}} (t - t_0) \]  

(11.33)

which results in:

\[ \tilde{h}_s = h_R - \frac{C_f^2 A_f^2 \tilde{R}_f}{4 L_f \tilde{W}_s} (t - t_0)^2 \]  

(11.34)

where \( t_0 \) is an integration constant \([T]\).

If the river level \( h_r \) is constant except at \( t = t_a \) where it rises from \( h_R \) to \( h_R + \Delta H \) (\( \Delta H \geq 0 \)), \( t_0 \) can be calculated using the situation immediately after the rise. It leads to:

\[ \tilde{h}_s = h_R + \Delta H - \frac{C_f^2 A_f^2 \tilde{R}_f}{4 L_f \tilde{W}_s} (t - t_0)^2 \]  

(11.35)

and:

\[ \tilde{Q}_f = -\frac{C_f^2 A_f^2 \tilde{R}_f}{2 L_f \tilde{W}_s} (t - t_0) \]  

(11.36)

where:

\[ t_0 = t_a + \frac{2 \tilde{W}_s}{C_f A_f \sqrt{\tilde{R}_f}} \sqrt{\frac{\Delta H L_f}{\tilde{R}_f}} \]  

(11.37)

in which:

\[ t_a = \text{moment of the water level rise of the river} \]  

\([T]\)

It should be observed that the flow will decrease linearly in time, until \( \tilde{h}_s = h_R \). Then the flow will be and remain zero (fig. 11.14). The relation between \( \tilde{Q}_f \) and \( \Delta H \) is non-linear.
Figure 11.14 The course of the water level and of the discharge in the storage section in time en 11.4.12)
If $\Delta H \leq 0$ then:

$$h_s = h_R + \Delta H + \frac{C^2 A_f^2 \bar{R}_f}{4 L_f \bar{W}_s^2} (t - t_0)^2$$

(11.38)

and:

$$\bar{Q}_f = \frac{C^2 A_f^2 \bar{R}_f}{2 L_f \bar{W}_s} (t - t_0)$$

(11.39)

where:

$$t_0 = t_a + \frac{2 \bar{W}_s}{C A_f} \sqrt{-\frac{\Delta H L_f}{\bar{R}_f}}$$

(11.40)

It should be noted that (11.35), (11.36), (11.38) and (11.39) are only valid for:

$$t_a < t \leq t_0$$

(11.41)

For the groundwater flow in case of a sudden rise of the water level of the river Edelman found the relation for the infiltration flow:

$$Q_G = \begin{cases} 0 & \text{for } t \leq t_a \\
E \Delta H \sqrt{\frac{k D \mu}{\pi (t-t_a)}} & \text{for } t > t_a \end{cases}$$

(11.42)

where:

$Q_G$ = Infiltration flow according to Edelman [$L^3T^{-1}$]

$E$ = length of the river section considered [L]

$k D$ = transmissivity of the subsoil [$L^2T^{-1}$]

$\mu$ = storage coefficient [-]

It may be observed that immediately after the level rise the flow approximates infinity. If time approximates infinity, the flow will tend to zero. A second observation is the fact that the relation between $Q_G$ and $\Delta H$ is linear.

In the preceding text differences have been shown between the simulation and the Edelman model. Because of the differences, the values proper for the simulation model cannot be determined uniquely, neither if those of the Edelman model are known. The method presented below may be considered as an example of the determination of the hydraulic constants of the simulation model.

The geo-hydrologic constants $E$, $k D$, and $\mu$ should be known. Furthermore a representative level rise $\Delta H_{ref}$ and a representative time $T_{ref}$ should be determined. $\Delta H_{ref}$ is a
rise that is representative of the level rises in the river section considered. \( T_{\text{ref}} \) is the time for which both the inflow \( \dot{Q}_f [\text{L}^3\text{T}^{-1}] \) and the volume of water having flowed from the river into the bank \([\text{L}^3] \) should be calculated correctly. Now all the independent parameters are determined as there are only two degrees of freedom (respectively \( \dot{W}_s \) and \( \frac{C^2_f A_f^2 \tilde{R}}{L_f} \)).

Combining (11.36) and (11.42) for \( t = t_a + T_{\text{ref}} \) and combining the integrated functions of those expressions gives, after some intermediate steps:

\[
\dot{W}_s = \frac{9}{8} E \sqrt{\frac{k D \mu T_{\text{ref}}}{\pi}} 
\]

\[
\frac{C^2_f A_f^2 \tilde{R}}{L_f} = \frac{9 \Delta H_{\text{ref}} E^2 k D \mu}{\pi T_{\text{ref}}} 
\]

where:

\( \Delta H_{\text{ref}} \) = rise that is representative of level rises in the river section considered \([\text{L}] \)

\( T_{\text{ref}} \) = time at which both a. the flow and b. the volume of water which has flowed from the river into the bank \([\text{L}^3] \) are calculated correctly in comparison with the Edelman model for an instantaneous level rise \( \Delta H_{\text{ref}} \) \([\text{T}] \)

It should be remarked that \( \tilde{C}_f, \tilde{A}_f, \tilde{R}_f \) and \( \tilde{L}_f \) in \( \frac{C^2_f A_f^2 \tilde{R}}{L_f} \) should be chosen in such a way that only the first two terms of the equation of conservation of momentum can be neglected indeed (compare (11.4) with (11.27)).

In fig. 11.15 the courses of \( Q_E \) and \( \dot{Q}_f \) are given for some instant level rises. The following constants have been used:

\[
\begin{align*}
k D & = 5000 \text{ m}^3/\text{day} \\
E & = 1000 \text{ m} \\
\mu & = 0.3 \\
\Delta H_{\text{ref}} & = 5 \text{ m} \\
T_{\text{ref}} & = 20 \text{ days}
\end{align*}
\]

It leads to:

\[
\dot{W}_s = 1.1 \times 10^5 \text{ m}^3
\]

\[
\frac{C^2_f A_f^2 \tilde{R}}{L_f} = 0.14 \text{ m}^3/\text{s}^2
\]

It should be noted that from fig. 11.15 the values of \( \Delta H_{\text{ref}} \) and \( T_{\text{ref}} \) cannot be reconstructed. From the figure and the preceding text the following conclusions can be drawn:

- The process using a hydrodynamic flow model is finite, in contrast to the Edelman model. The flow determined by means of the simulation model decreases linearly in time, in contrast to the Edelman model.

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Figure 11.15 The course of the outflow of river water into the banks as a function of time for both models and several instantaneous level rises

**Ed** Edelman model

**HFM** Hydrodynamic Flow Model

- The flow just after the level rise is modelled too small, when modelling with a hydrodynamic flow model.
- The flow for great values of $t$ is modelled too small, when modelling with a hydrodynamic flow model.
- The flows for intermediate values of $t$ may be too great or too small when using a hydrodynamic flow model.

Therefore it is theoretically incorrect to model a groundwater flow using a two section system of a hydrodynamic flow model.

**Extension**

In reality the level rise will be gradual, and therefore the results of both models are smoothed and so are the differences. That is illustrated by an example. The river level will start rising linearly in time from $h_R$ at $t_1$ to $h_R + \Delta H$ at $t_2$, and then fall linearly in time to $h_R$ at $t_3$. As no analytic solution was found for the simulation with the hydrodynamic flow model, a numerical solution was obtained. For the Edelman model an analytical solution was obtained by superposition of the equations that Edelman found for a water level increasing linearly. The latter reads:
\[ Q = 0 \quad \text{for } t \leq t_1 \] (11.45)

\[ Q = E \nu \sqrt{\frac{4kD \mu (t-t_1)}{\pi}} \quad \text{for } t > t_1 \]

where:

\( \nu \) = rate of the level rise \[\text{[LT}^{-1}]\]
\( t_1 \) = moment of the start of the water level rise in the river \[\text{[T]}\]

(11.45) applied to the example yields the system:

\[ Q = 0 \quad \text{for } t \leq t_1 \]

\[ Q = E \frac{\Delta H}{t_2 - t_1} \sqrt{\frac{4kD \mu (t-t_1)}{\pi}} \quad \text{for } t_1 < t \leq t_2 \]

\[ Q = E \left( \frac{\Delta H}{t_2 - t_1} \sqrt{\frac{4kD \mu (t-t_1)}{\pi}} - \frac{\Delta H}{t_3 - t_2} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} - \frac{\Delta H}{t_2 - t_1} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} + \frac{\Delta H}{t_3 - t_2} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} \right) \quad \text{for } t_2 < t \leq t_3 \]

\[ Q = E \left( \frac{\Delta H}{t_2 - t_1} \sqrt{\frac{4kD \mu (t-t_1)}{\pi}} - \frac{\Delta H}{t_3 - t_2} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} + \frac{\Delta H}{t_2 - t_1} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} - \frac{\Delta H}{t_3 - t_2} \sqrt{\frac{4kD \mu (t-t_2)}{\pi}} \right) \quad \text{for } t > t_3 \] (11.46)

where:

\( t \) = time \[\text{[T]}\]
\( t_1 \) = moment of the start of the water level rise in the river \[\text{[T]}\]
\( t_2 \) = moment of the peak of the water level of the river \[\text{[T]}\]
\( t_3 \) = moment of the end of the water level fall in the river \[\text{[T]}\]
\( \Delta H \) = water level rise of the river \[\text{[L]}\]

Calculations have been executed for the same constants as used in fig. 11.15, and:

\( t_1 = 0 \text{ days} \)
\( t_2 = 5 \text{ days} \)
\( t_3 = 10 \text{ days} \)

The results are shown graphically in figure 11.16. The differences between the two models are smoothed, as expected. The differences in ratio between level rise and flows are maintained. In the simulation model the flow is unequal to zero during a limited period. After the moment the original river level is obtained, the flows in the simulation model decrease linearly in time.

It can be concluded that the results of the way of modelling presented are acceptable for the calculation of the sideward infiltration if both the level rise and the length of time involved
Figure 11.16 The course of the flow as a function of time for both models and several river water level functions

Ed  Edelman model
HFM  Hydrodynamic Flow Model

are limited, although the way of modelling is theoretically incorrect.

More important than the sideward infiltration (fig. 11.17) is the infiltration caused by flooding (fig. 11.18). If during a level rise the flood plains are inundated, the infiltration will be significantly higher than when having the same level rise without flooding. The greater infiltration is caused by the facts that:
Figure 11.17 Sideward infiltration

a. the length over which infiltration takes place is greater, and
b. The area near the river that can be saturated is greater.

In terms of hydrodynamic flow modelling, the greater length should be represented by an increase of \( \frac{\partial^2 x^2 \partial y^2}{k_f L_f} \) with a rising water level, and the greater area saturated with an increase of \( \tilde{W}_s \) with a rising water level. As the cause of the enlarged infiltration is the storage width of the river, the ratios of \( A_f \) respectively \( \tilde{W}_s \) to the river storage width \( B_s \) are chosen in such a way that they are constant:

\[
\frac{\tilde{A}_f(h)}{B_s(h)} = \frac{\tilde{W}_s(h)}{B_s(h)} = \text{constant} \tag{11.47}
\]

(11.25) is modified into:

\[
\tilde{Q}_s = \tilde{W}_{s0} \frac{B_s(h_s)}{B_s} \frac{d\tilde{h}_s}{dt} \tag{11.48}
\]

where:

- \( \tilde{W}_{s0} \) = storage area of the storage section at a level where \( B_s(h) \) is equal to the mean storage width of the river \([L^2]\)
- \( B_s \) = mean storage width of the river \([L]\)
- \( B_s(h_s) \) = storage width of the river as a function of the water level \([L]\)

and (11.28) into:
Figure 11.18 Flooding causes an important increase in the infiltration area

\[ \frac{\bar{h}_t - h_r}{L_f} = - \frac{\bar{Q}_f |\bar{Q}_f|}{C_f A_f R_f \left( B_s(\bar{h}_r') \right)^2} \]  

(11.49)

where:

\[ \hat{A}_{r0} = \text{conveying cross-sectional flow of the flow section at a level where} \]

\[ B_s(\bar{h}_r') \text{ is equal to the mean storage width of the river} \]  

\[ [L^2] \]

\[ \bar{h}_r' = \text{water level in the flow section that determines the storage width of the flow section} \]  

\[ [L] \]

Several observations can be made.

1. If the gradient of the flood plain ground level \( dB/\delta h_c \) is small, the maximum flow after a sudden level rise may appear a certain time after the moment of the rise. For example: if \( dB/\delta h_c = 200 \), the river width before the level rise is 100 m, and the level rise \( \Delta H > 1 \) m, then the maximum inflow will appear some time after the level rise (fig. 11.19). In the example it is assumed that \( \bar{h}_r' = (\bar{h}_r + h_r)/2 \), \( \bar{W}_{50} = 1.1 \times 10^5 \text{ m}^2 \) and \( C_f A_f R_f L_f = 0.14 \text{ m}^3/\text{s}^2 \).

2. By introducing \( B_s(h) \) into the equations, the time at which the flow is decreased to zero is not the same for a level rise and a level fall. In general, the former is less than the latter. The cause is the fact that especially at the end of the period of flow, the profile of the flow section will be significantly smaller for a level fall than for a level rise. That is according to reality, where most of the time the profile available for groundwater flow near the river is greater for a level rise than for a level fall. If the difference obtained is not satisfactory, the difference may be modified by choosing different friction coefficient values of the flow section for different flow directions.
The most important parts of the simulation model have been investigated. However, for a groundwater simulation in a hydrodynamic flow model many more parameters have to be determined. In the next few lines it is indicated how a groundwater simulation model can be developed. These lines may be of interest only for those who want to use the model.

a. Determine the transmissivity and the storage coefficient of the subsoil.
b. Determine the length of the river section where the infiltration takes place.
c. Choose a representative value for the storage width $\overline{B}_s$ (for example half the flood plain width), a representative level rise $\Delta H_{\text{ref}}$ and a representative time $T_{\text{ref}}$.
d. Choose reasonable values for $\overline{L}_f$, $\overline{R}_f$ and $\overline{C}_f$ for the flow section in order to prevent unstable computations. Values which are common in a river section are appropriate.
e. Calculate $\tilde{W}_{s0}$ and $\tilde{A}_{\text{ho}}$ assuming $B_s = \overline{B}_s$ and using adapted forms of expressions (11.43) and (11.44):

$$\tilde{W}_{s0} = \frac{9}{8} \frac{kD\mu T_{\text{ref}}}{\pi}$$  \hspace{1cm} (11.50)

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure11_19.png}
\caption{The infiltration flow as a function of the level rise (cf. text) }
\end{figure}
Figure 11.20 An example of the calculation of groundwater flow using a hydrodynamic flow model

\[ \dot{A}_{p0} = \sqrt{\frac{9 \dot{L}_f \Delta H_{ref} E^2 k D \mu}{\bar{C}_f^2 \bar{R}_f \pi T_{ref}}} \]  

(11.51)

f. Compose the flow section (in § 11.3.1 all the required parameters are listed), using the determined values of \( \dot{L}_f, \bar{R}_f, \bar{C}_f, \dot{A}_{p0} \) and the value of the storage width of the river \( B_s \). A small storage width of the flow section should be chosen, to obey equation (11.24).

g. Compose the storage section. The length of the storage section is chosen in such a way that it is the root of \( \dot{W}_{so} \). The complete profile is assumed to be flowing. The friction coefficient of the storage section is greater than common for rivers, the hydraulic radius of the storage section may have its default value.

h. Calibrate the model by adapting various parameters using measured groundwater levels.

i. Check the correctness i.e. whether indeed the terms neglected in the equation of conservation of momentum have an insignificant influence. That can be done by substituting the calculated values in the discretised equation.

For the Meuse in the Netherlands 4 groundwater units have been included, each containing one flow section and one storage section. Three units are situated along the stretch of the Grensmaas, one near Roermond. After calibrating the units the conclusions were:

1. A reasonably stable schematisation can be obtained that meets the conditions for which the neglected terms of the equation of conservation of momentum have an insignificant
Figure 11.21 An example of minor bed and flood plain flow in two parallel sections

influence.

2. Flows up to 20 m³/s and possibly also greater flows can be modelled using the theory described in the preceding text. Data on the transmissivity and the storage coefficient should be known or otherwise be assumed.

3. As only data of a few flood waves were available and flow measurements were scarce the advantage of using a groundwater model could not be proved. A similar effect on the water levels can be obtained by choosing higher friction coefficient values for higher water levels.

4. A more accurate groundwater simulation can be obtained if:
   a. accurate measurements of groundwater levels and correct data of the transmissivity and the storage coefficient are available. Calculating the flow into the subsoil by subtracting two measured river discharges will not be sufficiently accurate, and
   b. a groundwater flow simulation model based on the groundwater flow equations is incorporated in the hydrodynamic flow model.

An example of calculated groundwater flows is given in fig. 11.20.

11.4.5 CALIBRATION AND VERIFICATION

The floods of February 1984 and November 1984 were used for calibration. The procedure was one of the commonly used trial and error type (Williams et al., 1988). The friction coefficient was the parameter that was adapted. The friction coefficient is a function of the hydraulic radius. Calibrating was done according to:
where $k_M$ and $n_M$ are constants, independent of the water level. The water levels and wave travel times were relatively insensitive to changes of the friction coefficient of the flood plain. Therefore calibration has been concentrated on the friction coefficient of the minor bed.

Unstable computations occurred when the starting levels were not appropriate.

The calibration took place from Borgharen to Lith, and from a great number of sections combined to a few. Particular attention was paid to the peak level and peak moment of the studied waves.

It was found that the wave travel time could hardly be influenced by the friction coefficient.

During the calibration process indistinctness arose about the exact locations of the used gauges. It was not possible to reconstruct all the locations exactly.

Results of the calibration and verification are presented in table 11.9. The moment of the measured peak cannot be determined accurately. A peak level expressed in centimetres, may last up to 12 hours and more. Therefore the difference in time between values measured and calculated cannot be calculated in units of one hour. That is the reason why the errors are given in time steps of 6 hours. The wave travel time is considered to be accurate.

An example of minor bed and flood plain flow in two parallel sections is given in fig. 11.21. An example of water levels measured and calculated is given in fig. 11.22. In the figure the influence of the omittance of weirs is shown clearly at the beginning and the end of

![Graph showing water level changes over time]

*Figure 11.22 An example of the calculation of the water level at Heel-boven*
## TABLE 11.9 RESIDUALS BETWEEN MEASURED AND CALCULATED PEAK LEVELS AND MOMENTS

<table>
<thead>
<tr>
<th>node</th>
<th>station</th>
<th>kmr</th>
<th>Calibration</th>
<th>Verification</th>
<th></th>
<th></th>
<th></th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>(e_h) [cm]</td>
<td>(e_t) [h]</td>
<td>(e_h) [cm]</td>
<td>(e_t) [h]</td>
<td>(e_h) [cm]</td>
<td>(e_t) [h]</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>29.1</td>
<td>Elsloo</td>
<td>29.330</td>
<td>-2</td>
<td>0</td>
<td>+6</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>+7</td>
<td>0</td>
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<tr>
<td>37.1</td>
<td>Grevenbicht</td>
<td>44.945</td>
<td>-7</td>
<td>-6</td>
<td>+4</td>
<td>0</td>
<td>-7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>44.1</td>
<td>Stevensweert</td>
<td>61.680</td>
<td>-7</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>48.1</td>
<td>Heel-boven</td>
<td>67.340N</td>
<td>+2</td>
<td>0</td>
<td>+5</td>
<td>0</td>
<td>+9</td>
<td>0</td>
<td>+6</td>
<td>-6</td>
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<td></td>
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<td>58.1</td>
<td>Heel-beneden</td>
<td>85.075</td>
<td>+1</td>
<td>0</td>
<td>+7</td>
<td>0</td>
<td></td>
<td></td>
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<tr>
<td>62.1</td>
<td>Belfeld-boven</td>
<td>100.725</td>
<td>-5</td>
<td>+6</td>
<td>-14</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>67.1</td>
<td>Venlo</td>
<td>108.175</td>
<td>-1</td>
<td>-6</td>
<td>+5</td>
<td>+6</td>
<td>+5</td>
<td>0</td>
<td>-1</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>74.1</td>
<td>Well-Dorp</td>
<td>132.100</td>
<td>-8</td>
<td>0</td>
<td>+6</td>
<td>0</td>
<td>+10</td>
<td>+6</td>
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</tr>
<tr>
<td>79.1</td>
<td>Sambeek-boven</td>
<td>146.500</td>
<td>-10</td>
<td>-6</td>
<td>+9</td>
<td>+6</td>
<td>+9</td>
<td>+6</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>91.1</td>
<td>Grave-boven</td>
<td>175.645⁴</td>
<td>+8</td>
<td>-12</td>
<td>+6</td>
<td>0</td>
<td>+1</td>
<td>+12</td>
<td>-21</td>
<td>0</td>
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<td></td>
</tr>
<tr>
<td>106.1</td>
<td>Lith-Dorp</td>
<td>202.370</td>
<td>+5</td>
<td>0</td>
<td>-1</td>
<td>+6</td>
<td>-8</td>
<td>+6</td>
<td>-8</td>
<td>-6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

kmr: number of kilometres  
\(e_h\): measured water level - calculated water level  
\(e_t\): measured peak moment - calculated peak moment (rounded to the nearest multiple of 6 h)

The flood wave, where the calculated water level is lower than the headwater level.

The computed water levels are also considered to be satisfying, as the aim, errors less than 10 cm, is achieved in most cases. Compared to earlier studies especially the wave travel time was improved. It is probably caused by the introduction of minor bed and flood plain sections.

### Conclusions

- By dividing the bed into a minor bed and a flood plain a schematisation of the Dutch part of the river Meuse is obtained that is more like reality than earlier schematisations.
- The flood plain sections need gutters to allow the calculation of normal flow situations. By making the gutters small the influence of the gutters on the river flow was negligible.
- The influence of groundwater storage can be modelled using a hydrodynamic flow model. However the results are not likely to be very accurate.

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³ In the mean time the station of Stevensweert has been moved to kmr 61.570

⁴ In the mean time the station of Grave-boven has been moved to kmr 174.900

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The calibration shows that the results are more accurate than those of the schematisation used before. Especially the wave travel time has been improved.
Chapter 12. Flood forecasting: The FLOFOM model

12.1 INTRODUCTION

In several respects a real-time flood forecasting model is different from a model that is only used for an analysis. A real-time flood forecasting model should satisfy the following criteria:

1. The applicability of the model in real-time should be ensured in a wide variety of real-world circumstances. An excuse as a defect gauge or troublesome data transmittance for having no forecast is not likely to be accepted. It means that missing data, incorrect data, etcetera must not cause the impossibility of making a forecast. The risk of failing due to missing data can be reduced by defining alternative models needing a reduced amount of data or by having different data sources. Strongly incorrect data can be detected, small errors in the data should be neutralised.

2. The model should be fast.

3. The forecasts should be reliable, even if trouble of various kinds is apparent.

4. The model should be user-friendly. It is a calamity model and therefore the moment of use is only known a few hours before. The last use of the model may be one year ago, and the user is not necessarily a modelling specialist. Therefore the user should not be bothered by determining starting up periods, choosing the proper model for the available data, etcetera.

In the chapters 9 to 11 the modelling in equations has been presented. In some places attention was paid to the applicability of the real-time model. In this chapter the real-time aspects will be elucidated in more detail.

In §12.2 the adjustments of the modules described in chapters 10 and 11 to the modules used in the model will be described, as well as how missing data are dealt with. In §12.3 the construction of the real-time model, called FLOFOM (FLOod FOREcasting for the river Meuse), is elucidated. Finally in §12.4 the first experiences in real-time are described. All the sections will be concise, because the description of all the technical details would take a multiple of the number of pages used now.

A survey of data flows and information flows is given in figures 12.1 and 12.2.

12.2 MISSING DATA AND MODEL ADJUSTMENTS

12.2.1 GENERAL REMARKS

The risk of missing data is relatively great, because:

a. the model functions in real-time,

b. floods are a calamity which puts all measurement installations to the test, and

c. many data links, some of which international, have to be used.

During the floods of 1988 and 1991 data from many sources were unexpectedly not available. For example:

- Precipitation data coming from the KNMI were not available at the end of the 1988 flood. The computer in the international communication centre at Reading (United Kingdom) was out of order due to leakage caused by heavy rainfall.

- The acoustic flow meter at Maastricht-St.-Pieter was out of order during both floods. The causes of the malfunctionings are unknown.
Figure 12.1 Simplified data flow chart
- The float in several Dutch water level stations could not rise to the proper level during the flood of 1991, resulting in incorrectly measured water levels. The cause was a lamp placed over the rod on which the float has been fixed. The lamp should produce heat to prevent incorrect measurements due to frost.

- Discharge and precipitation data from the Belgian Ministère Wallonne de l'Équipement et des Transports were not available during the flood of 1991 because of capacity problems, due to the flood in the Meuse.

It should be noted that most causes of the missing data have direct relations to the characteristics of a flood. (For the floods mentioned not all the arrangements were made which are required for the FLOFOM model, otherwise the list might have been essentially longer.)

The FLOFOM-model will run as a calamity model. It means that the length of the available time series is limited. It causes the need of starting up periods, especially for the modules containing hydrodynamic flow models.

The data gathering, the way of dealing with missing data, the starting periods and other practical problems will now be elucidated module by module.

12.2.2 THE FRENCH TRIBUTARIES

Data of the module of the French tributaries are partly obtainable by means of a telephone with an automatic answering device (Semois at Haulmé), partly by way of Belgian measuring stations (Viroin at Treignes, Semois at Membre), and partly directly by way of the central office of the Service de la Navigation (Meuse at Montcy-Notre-Dame). It should be noted that for the Semois two sources are available in slightly different places (fig. 6.2). The Semois stations must not be used interchangeably, because of the possibility of systematic differences in the discharge determination. Preferably the Belgian station is chosen.

The module has been developed in such a way that an irregular data stream can be handled. If no data of one or more of those stations are available, the module of the French Meuse is changed by a simpler module.

The output of the module of the French tributaries consists of:

a. the observed discharges,

b. interpolated values between the observed discharges, and

c. a forecast discharge obtained by the model explained in § 10.2. (Only measurements and no estimations are used. The time interval between the measurements used should be at least 3 hours.)

It means that complete sets of hourly discharge series are available. It should be noted that if the data supply is in no way lacking, only measured (and interpolated) discharges of the French tributaries will be used in the module of the French Meuse (which can be derived from § 11.2).

12.2.3 THE FRENCH MEUSE

The module of the French Meuse needs discharge data of the French tributaries (§ 12.2.2) and of Chooz. The discharge data of Chooz are obtained by way of a telephone with an automatic answering device, which gives water levels, and a stage-discharge curve.

The discharge data of Chooz are transformed into hourly discharges by linear interpolation.

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If discharges of the French tributaries are lacking in a serious way, an alternative model may be used (§ 10.2). The discharge data at Chooz are indispensable, however. If the discharge is not known it has to be estimated, if necessary with the help of the services in charge.

It should be noted that the Belgian and French stage-discharge curves at the station of Chooz may be different, therefore Belgian and French data must not be used after each other. The output consists of the discharge at Chooz and in more detail:

a. the observed discharges,
b. interpolated values between the observed discharges,
c. a forecast discharge obtained by the model explained in § 11.2 up to 16 hours after the discharge observed last, and
d. a constant discharge equal to the last forecast between 16 and 28 hours after the discharge observed last. If for a longer period discharges are required, estimations have to be made. The constant discharge must not be longer than 12 hours, because otherwise the constant discharge might influence the discharge forecast at Borgharen.

The data are modified before they are entered in the hydrodynamic flow model of the Belgian Meuse (§ 12.2.5).

It should be noted that in future the access to the French data will probably be more sophisticated, because of the introduction of an automatic system in that part of the catchment (Anonymous, 1988).
12.2.4 THE BELGIAN TRIBUTARIES

Precipitation data
The module for the Belgian tributaries needs precipitation data from at least 4 days before the moment of observation (definition cf. page 195) up to the last moment a forecast is developed for. Therefore both measured and forecast precipitation data are necessary.

Two ways of data transmission are foreseen for the measured data:
1. from the Ministère Wallonne de l’Équipement et des Transports in Belgium. They furnish hourly subcatchment precipitation data. Until now (1991), the hourly data have not reached the information centre in real-time.
2. from the KNMI. They furnish point precipitation data every 6 hours. The data are available from the international network.

Because of the more detailed information, the former type is preferred. But if that information is not available, the areal precipitation data are determined by the latter and by using the Thiessen polygons method (§ 10.2). Incidentally missing values are automatically replaced by the mean precipitation value of the three nearest stations. It means that at least three precipitation values have to be known. The stations incorporated in the Thiessen polygons can be changed within the program.

Forecast precipitation values can be obtained by consulting the meteorological institute (KNMI). In the case of insufficient measurement data the institute may also give estimated data of the fallen precipitation before the moment of observation. Estimated precipitations and precipitation forecasts are entered in the model by a separate submenu, in which a subdivision into regions has been made. The KNMI is manned night and day.

It is necessary to have some (estimated) precipitation input, because the model is not able to handle a situation without precipitation data.

Discharge data
The most important data in FLOFOM are the discharges (fig. 12.1). A complete hourly series is necessary for a proper functioning of the hydrodynamic flow model for the Belgian Meuse. Currently (1991), the data are not available on a formal basis. Therefore also forecasts without the availability of a complete discharge series can be developed (§ 10.3). Then the forecast will be less accurate.

In case of the availability the output consists of measured discharges up to the moment of observation of:

a. the observed discharges,
b. interpolated values between the observed discharges, and
c. a forecast discharge obtained by the model explained in § 10.3.6 after the moment of the discharge observed last.

If discharge data are not present, the model needs an estimated discharge data at the moment of observation. Chapter 6 of this thesis or contacting Belgian colleagues by telephone may help. If the estimated discharge has a significant error (say 20 %) the effect on the discharge forecast at Borgharen is almost neutralised by the correction model (§ 11.3). The output of the Belgian tributaries will consist in calculated direct runoff values increased by the difference between estimated discharge and calculated direct runoff at the moment of observation.

The data are modified before they are entered in the hydrodynamic flow model of the Belgian Meuse (§ 10.3.8, § 12.2.5).
12.2.5 THE BELGIAN MEUSE

For the Belgian Meuse discharge data at Chooz and at the mouths of the Belgian tributaries are required.

Due to the character of the model used, namely the hydrodynamic flow model ZWENDL, some adaptations are necessary. As an input the hydrodynamic flow model requires among others things (§ 11.3 and 11.4):
- discharge data of all the input nodes from the beginning to the end of the calculation period. Only at the downstream end a stage-discharge relation is sufficient.
- values of water levels, discharges and gate levels at the beginning of the calculation (initial conditions).

To prevent unstable calculations, at the beginning of the calculation period the boundary conditions such as discharges and water levels have to correspond to the initial conditions. The initial conditions are an average situation at the beginning of a flood period. It means that it is impossible immediately to start with the measured discharges as an input, but a starting period is unavoidable. The starting period consists in a linear course of the discharge between the initial value and the moment 2 days before the moment of observation. The length of the starting period is dependent on the discharge at Borgharen 2 days before the moment of observation, because the adaptation period of the weirs is dependent on the discharge.

The output of the model is a calculated discharge at Borgharen. The discharge is adapted to neutralise several influences (§ 11.3.4).

12.2.6 THE DUTCH TRIBUTARIES

The station De-Drie-Bogen is equipped with a telephone with an automatic answering device, able to give the actual water level. The discharge is obtained with the help of a stage-discharge curve.

The discharge data of the Roer are indispensable for the flood forecast. If the discharge at De-Drie-Bogen is not available, it has to be estimated with the help of § 6.8. To develop discharge forecasts for the Dutch tributaries also the measured discharge and discharge forecasts at Borgharen-Dorp are necessary. If unfortunately they are absent, they have to be estimated. The output with respect to the Roer consists of:
- the observed discharges,
- interpolated values between the observed discharges, and
- a forecast discharge obtained by the model explained in § 10.10 up to 24 hours after the discharge observed last.

It should be remarked that those moments are delayed due to the wave travel time between the station and the river mouth.

For the Niers the output consists in a calculated discharge from the beginning to the end of the calculation (§ 10.4).

The data are modified before they are entered in the hydrodynamic flow model of the Dutch Meuse (§ 10.4, § 12.2.7).

12.2.7 THE DUTCH MEUSE

For the Dutch Meuse discharge data at Borgharen and at the mouths of the Roer and the Niers are required.
Like the Belgian Meuse, the Dutch Meuse has been modelled with a hydrodynamic flow model. Therefore it is subject to the same needs and restrictions as the Belgian Meuse. However there are some complicating factors:

1. From the moment of observation on, no measurements are available, and so the forecasts will be used as an input. However, at Borgharen the transition from the measured discharge to the forecast one does not have to be continuous (§ 11.3.4). The discontinuity may cause unstable calculations. Therefore the discharge at Borgharen is supposed to have a linear course in time between the moment of observation to 4 hours after that moment. At the latter time the forecast is used.

2. For the Dutch Meuse, forecasts with a lead time of several days are required, once the peak has passed Borgharen. The hydrodynamic flow model is able to make that forecast, if for all the boundaries a discharge series for the complete calculation period is available. For Borgharen, the Roer and the Niers only 24-h forecasts are available. The discharge forecast a long time after the peak will not influence the maximum water level significantly. Therefore it is allowed to estimate the discharge in that period relatively roughly. Here a slowly decreasing discharge is chosen after the last forecast discharge, because a forecast for the complete reach of the Dutch Meuse will only be developed if the peak passes Borgharen within the next 24 hours. If necessary, the estimated values may be changed by hand.

The starting period could be limited to two days, because no weirs function in the schematisation.

12.2.8 ESTIMATED VALUES

However much effort has been made to handle each module as well as possible, it cannot be prevented that erroneous discharges form the input of a module. The cause may be incorrect data, typing errors, program errors and model errors. Therefore it has been made possible to replace each discharge by an estimated value. That can be done in the 'calculations' menu of the program. The option increases the real-time applicability of the model.

12.3 THE CONSTRUCTION OF THE FLOFOM MODEL

The model construction has been based on the System Development Method II (Turner et al., 1989). The method subdivides the construction of the system into 5 phases.

1. The definition study. In this phase the system requirements are specified, as well as the starting-points for the system development. A plan containing the effects for the organisation, in terms of needed equipment, time, staff, the ways of reporting, the ways of implementation, the required training of the future users, a cost/benefits analysis, etcetera, is one of the products of the first phase.

2. The system design. The system requirements are refined, and the basis of the system is developed to a level where subsystems can be developed.

3. The detailed system design. The system requirements and the design of subsystems is refined to a level where the implementation of the specific components of the subsystems can take place.

4. The implementation. The programmes as well as the manual procedures are created and tested.

5. The installation. All the activities which are necessary to make the system work
operationally take place in the last phase. During the system design phase a considerable effort was made to develop the so-called prototype. With the help of the future users the structure of the menus in FLOFOM have been designed. For that purpose a computer program similar to the future system has been used, in which, however, only the screens function correctly. It enables the future users to express their wishes very explicitly. Changes in their wishes can be realised fast, because at this stage there are few consequences for the program.

The model is composed of a number of menus. Central is the main menu, at which the user arrives after he has got access to the system. The main menu gives five choices (for the sake of clearness, all Dutch words have been translated into English here):
- input
- calculations
- results
- model-info
- quit

When one of those choices is made, a pull-down menu shows on the monitor.

In the input menu all the data can be entered. It has been composed in accordance with the way the data are collected in the office. It means that for example all data measured at Belgian stations are to be entered by means of one sub-menu, although some of the discharges are used in the module of the French tributaries (Membre, Treignes). Precipitations can be entered by means of three sub-menues: one for point measurements, one for areal data and one for forecasts. The model is linked with the multi-functional presentation station (MFP) at the information centre. If data are available on the MFP they can be entered using a floppy disc or a data line. The same applies to the point precipitation information. In case of technical trouble in the linked computers, all the data can be entered by means of the keyboard.

The calculation menu gives two options: Borgharen-Dorp and Dutch Meuse. If one of those options is chosen, the proper modules will run. Data are evaluated on their completeness, alternative models are selected and run if necessary, etcetera. If important data are missing, the user is asked to enter them by means of the keyboard. Because of the long calculating time of the hydrodynamic flow model, the model will only start after touching the proper key. That enables the user to investigate intermediate results and to detect errors in the calculation at an early stage.

The calculation results can be shown with the use of the result menu. Precipitations, discharges and water levels can be shown both graphically and numerically. Values measured and calculated are displayed. Also results of earlier runs can be shown, dependent on the requests of the user.

The model contains many data which are fixed until recalibration or any changes in the physiography of the (measuring) system. A number of them can be looked at or changed using the choice model-info. For example, stations incorporated in the Thiessen Polygons Method can be changed with the help of the menu. It also applies to the constants in regression equations, however not to the type of the equations themselves. The hydrodynamic flow schematisation can only be modified through an ordinary editor. Changes in the parameters can only be executed by the system manager and not by normal users.

The option end will quit the program.
The model runs on a 80386 25 MHz personal computer with co-processor. The calculations will last for about 20 minutes for the Borgharen forecast and 15 minutes for the Dutch Meuse forecast. The computer has been installed at the Inland Water Information and Warning Centre in Lelystad.

12.4 FIRST RESULTS

Just after the model was calibrated, but before the construction of the user-friendly FLOFOM-model, a flood with an exceeding frequency of once per five years occurred (maximum discharge approximately 1800 m³/s). It was the first time the model could work in real-time conditions. Exceptionally, the flood mainly originated from the French part of the basin, causing a lot of damage in the region of Charleville-Mézières. The top of the wave was flat compared to earlier recorded waves.

The information received from France was according to expectations. For Montcy-Notre-Dame no agreements of data transmittance were made; those data were absent. The Belgian discharge data were available 2 or 3 times per day; hourly precipitation data were totally absent. Discharge data at De-Drie-Bogen were only available on weekdays (at the moment of the flood the telephone with an automatic answering device had still to be installed). The telephones with automatic answering devices functioned satisfactorily. Water level data at Dutch stations were available, although later the question arose about the quality of the values (§ 12.2.1). Precipitation data from the international network and precipitation forecasts were available as expected. The time between the precipitation measurements and
the moment the measurements were available in the office was about 2 hours.

For the Chooz discharge forecasts the model described in § 10.2 was used, as only data for the Semois and Chooz were available. For the Belgian Meuse the modification described in § 10.3.6 could not be applied due to the missing data. For the Roer (De-Drie-Bogen) the discharge had to be estimated.

In fig. 12.3 the measured discharge and the discharge forecasts are shown for Borgharen. The drop in the discharge measured on 7 January 1991 has probably been caused by a wrongly measured water level. The forecasts are considered to be accurate, because the errors were about 10 cm. Downstream from Borgharen errors in the forecasts up to 18 cm occurred, which was much compared to the aim of 10 cm. A study was initiated to find the causes of those errors (Heijens, 1991). Several causes were found:

a. Measurement stations were moved between the calibration period and the January 1991 flood.

b. For some new measuring stations the local situation is extremely unfavourable with respect to floods. The measuring stations are placed just beneath the locks in the shipping canal. For many hundreds of metres the shipping canal is situated parallel to the river, separated by a low dam, which is inundated at discharges between 900 and 1700 m³/s. The measured water level may differ up to more than 10 cm from the river water level in that place. Only if the local situation is known very well an accurate forecast for the measuring station can be made. However, the modelling of such a situation in a one-dimensional hydrodynamic flow model is very complicated.

c. Some measurements were incorrect due to the wrong position of a lamp (placed there to prevent the installation from being frozen).

d. Sometimes the places in which the calibration measurements were made are not known for sure.

e. For some places the gauge used was changed during a particular flood during the calibration period.

Although the schematisation had been altered in some places, the general conclusion was that if better forecasts for the Dutch Meuse were required, first of all the measurements had to be improved with respect to correctness and representativeness of the river water level. The measuring stations should not be moved for a longer period, and their places should be known exactly.

In general it was concluded that the model functions satisfactorily. The model has shown to be flexible with respect to the availability of input data.
Chapter 13. Low flow forecasting

13.1 INTRODUCTION

Discharges of rivers which do not have a great storage capacity such as glaciers, lakes or marshes, may fall to low values during dry periods. That may lead to problems for various users of the river and its water (shipping, agriculture, public water supply, aquatic flora and fauna, etcetera). In regulated rivers, such as the Meuse, the water distribution and use can be influenced by man. To take proper measures, a forecast of the discharge at the border of the territory is required. In this chapter the development of a low flow model will be described. As the border station Maastricht-St.-Pieter has been chosen, which is upstream from the first bifurcation in the Netherlands.

The total natural river flow consists of three components: surface flow, interflow and baseflow. The magnitude of the components changes with time, depending on the weather and the hydrologic properties of the catchment.

The baseflow is thought to come from the reservoirs reacting most slowly, such as groundwater storage. The reservoirs will be called baseflow reservoirs. In sufficiently long dry periods, the discharge consists of baseflow only. An analysis of the baseflow depletion is important in modelling low flows. Different kinds of depletion curves and their suitability for the Meuse catchment are dealt with in § 13.2.

Also in a relatively dry period some rainfall may occur. That may lead to an increase of the baseflow. The influence of rainfall on the baseflow is analysed in § 13.3. In order to obtain a longer model lead time the chances of future rainfall are analysed, resulting in a stochastic rainfall model. That model is incorporated in the total low flow forecasting model, as described in § 13.4.

In § 13.5 the model elements are combined, thus forming the complete low flow forecasting model. A description of the model as well as a simulation of the 1978 baseflow can be found in this section.

13.2 DEPLETION CURVES

One of the most important properties that baseflow depletion is thought to have is a unique and invertible discharge-storage relation:

\[ Q_b = f(S) \quad \text{(13.1)} \]

where:

- \( S \) = storage of water in baseflow reservoirs \([\text{L}^3]\)
- \( Q_b \) = baseflow \([\text{L}^3\text{T}^{-1}]\)
- \( f \) = an invertible function \([\text{L}^3\text{T}^{-1}]\)

Both \( S \) and \( Q_b \) are functions of time.
Together with the continuity equation (no reservoir recharge):

\[ Q_b = -\frac{dS}{dt} \]  \hspace{1cm} (13.2)

where:

\[ t \] = time \hspace{1cm} [T]

sufficient equations are available to calculate the depletion curve.

For hydrologically suitable models for natural catchments it is necessary that:

1. \( Q_b(t) \geq 0 \) for \( \forall t \geq t_0 \) \hspace{1cm} (13.3)

2. \( \frac{dQ_b(t)}{dt} \leq 0 \) for \( \forall t \geq t_0 \) \hspace{1cm} (13.4)

3. \( \frac{d^2Q_b(t)}{dt^2} \geq 0 \) for \( \forall t \geq t_0 \) \hspace{1cm} (13.5)

for a certain \( t_0 \).

Condition 3 can be seen as: the reduction of the discharge in time must reduce (or at least stay equal) in time.

The linear discharge-storage relation is commonly used:

\[ Q_b = \frac{S}{k_1} \] \hspace{1cm} (13.6)

where:

\[ k_1 \] = decay constant \hspace{1cm} [T]

In combination with (13.2) it leads to:

\[ Q_b = Q_0 \exp\left(-\frac{t-t_0}{k_1}\right) \hspace{1cm} \text{(model 1)} \hspace{1cm} (13.7) \]

where \( Q_0 \) [L²T⁻¹] and \( t_0 \) [T] are constants. This depletion curve has been used for example by Bako et al. (1988).

For a period without rain \( Q_0 \) and \( t_0 \) are completely dependent on each other in principle, so one of them can be omitted. In this section a suitable and fixed value for \( Q_0 \) has been chosen, thus restricting \( (t-t_0) \) to small positive values.

If the model coefficients are known, only one baseflow data is necessary to obtain the course of the baseflow in time. The course can be found by substituting \( t = t_t \) and \( Q_b = Q_t \) in

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equation (13.7), where \( Q_i \) is the baseflow known at \( t_i \). It should be observed that it is not necessary to have an analytical expression for the function \( f \) in (13.1).

As an example several series of measured discharges of the Lesse have been plotted, together with the calculated depletion curve (fig. 13.1). At the first glance the curve may look appropriate as a depletion function. However, the same data plotted on semi-logarithmic paper show that better functions may be applied (fig. 13.2). Therefore a second and third model for the depletion curve are proposed.

The second model is an extension of model 1:

\[
Q_b = Q_0 \exp \left( \frac{(t-t_0)^{v}}{k_2} \right) \tag{model 2} \tag{13.8}
\]

where \( v \) and \( k_2 \) are constants. \( k_2 \) is positive.

Several observations can be made:

- If \( v = 1 \) model 1 is obtained.
- Contrary to model 1 \( Q_0 \) and \( t_0 \) are not completely dependent (for \( v \neq 1 \)).
- To obey the second condition \( v \) has to be zero or positive.
- For non-integer values of \( v \) the term \((t-t_0)\) has to be positive or integer to prevent a complex expression.
- If \( v > 0 \) and \( t - t_0 > 0 \) the maximum of the exponential function is 1 (for \( t = t_0 \)), so the maximum value of the baseflow is \( Q_0 \).

To prevent a low value of \( Q_0 \) (which may cause trouble if using the equation in real-time) and to limit the number of unknowns, \( Q_0 \) has a fixed and chosen value during the calibration. The value is greater than the greatest baseflow observed. An example of this model is given in fig. 13.3.

The third model is derived starting from an extension of (13.6):

\[
Q_b = \frac{S^{n_3}}{k_3} \tag{13.9}
\]

where \( n_3 \) and \( k_3 \) are positive constants.

Combining it with (13.2) yields for \( n_3 \neq 1 \) (Youngs, 1985):

\[
Q_b = \frac{Q_0}{(1 + v(t-t_0))^w} \tag{model 3} \tag{13.10}
\]

where:

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Substituting (13.10) in (13.9) and rewriting it gives an expression for $S$.

Equation (13.9) was derived for the drawdown of the groundwater table in uniform soils drained by parallel drains.

For the same reasons as for the second model, $Q_0$ is chosen in advance. An example is shown in fig. 13.4.

The models to be chosen from will be restricted to the three models mentioned. It should be noted that for karstic regions essentially different depletion curves may be found (Soullos, 1991). In the case of the low flow modelling of the Meuse, however, the three models are sufficient, as will be shown.
Figure 13.2 The same data as in fig. 13.1 plotted on semi-logarithmical paper

The uniqueness of (13.1) only holds if the catchment is homogeneous with respect to:

a. the discharge-storage relation, and
b. the baseflow-reservoir recharge.

It can be shown as follows: Subdivide the catchment into $N$ equal areas. Then, using (13.1) and (13.2):

$$Q_b = \sum_{i=1}^{N} Q_{b}^{A_i} = \sum_{i=1}^{N} f_{A_i}(S_{A_i})$$  \hspace{1cm} (13.13)

$$Q_{b}^{A_i} = -\frac{dS_{A_i}}{dt} \quad \text{for} \quad i=1,\ldots,N$$  \hspace{1cm} (13.14)

where:

- $S_{A_i}$ = storage of water in baseflow reservoirs in area $A_i$ [L$^3$]
- $Q_{b}^{A_i}$ = baseflow of area $A_i$ [L$^3$T$^{-1}$]
- $f_{A_i}$ = an invertible function for area $A_i$ [L$^3$T$^{-1}$]
- $N$ = number of sub-areas in the catchment [-]

It is obvious that:
\[ Q = 100 \exp(-0.498 \ t^{0.498}) \]

\[ S = \sum_{i=1}^{N} S_{A_i} \quad (13.15) \]

The two conditions mentioned earlier will now be elucidated separately.

a. If not all the discharge-storage relations \( f_{A_i} \) are the same, the various storages \( S_{A_i} \) will be mutually different after some time. Now it will be supposed that the total storage decreases from \( S(t_1) \) to \( S(t_2) \), and that at \( t_2 \) all the areas receive the same recharge, which increases the total storage to the same quantity as that at \( t_1 \). Then the storages \( S_{A_i} \) will differ, and therefore the resulting baseflow \( Q_b \) will be different from that at \( t_1 \) (in almost any case). That is contrary to the assumption of uniqueness of the discharge-storage relation of the complete catchment (invertibility of the function \( f \) in (13.1)).

b. If the baseflow reservoir recharge is different for the various areas, then different \( S_i \) will arise. Then it is possible to have different distributions of \( S_i \) belonging to the same \( Q_b \), although all the \( f_i \) are the same. That is contrary to the uniqueness.

In the Meuse catchment the conditions of homogeneity with respect to the discharge-storage relation and the baseflow-reservoir storage are not fulfilled. The depletion coefficients differ per subcatchment (this section), and rainfall, a measure for the recharge, is not homogeneous.
Figure 13.4 The same data as in fig. 13.1 and the calibrated depletion curve using model 3, plotted on semi-logarithmical paper

over the catchment (§ 13.3). Therefore it is not correct to model the catchment with only one discharge-storage relation. To overcome the problem, a subdivision into subcatchments has been made. That choice is also useful with respect to changes in catchment behaviour (for example caused by man): only the parameters of the subcatchments involved need be adapted or recalibrated.

Using the system description and regarding the available data, a subdivision into ten subcatchments has been made (left hand two columns of table 13.1). The remaining area is allocated to the subcatchments later.

Mean daily discharges of dry spells in the period 1971-1981 have been used for calibration. The model selection and the coefficient determination was carried out as follows. Periods of low flows have been selected. Then the data have been reduced to periods with no (significant) rainfall and only a baseflow present (fig. 13.5). The coefficients of each subcatchment have been calculated for all three models.

Commonly used parameter determination methods such as the correlation method and the matching strip method (Nathan et al., 1990) compute the decay constant by means of a tangent. Those methods cannot be applied here, because in the models 2 and 3 more than 1 parameter have to be determined and model selection has to be executed.

Therefore computations have been made on a mini-computer, using the SPSS-X-routine Non-Linear Regression. The curves have been applied as well as possible, causing the smallest possible errors between measured and calculated baseflow. However, the exact
<table>
<thead>
<tr>
<th>River</th>
<th>Gauging station</th>
<th>Model</th>
<th>$Q_0$ [m$^3$/s]</th>
<th>coefficients</th>
<th>$\nu$</th>
<th>$\omega$</th>
<th>$Q_e$ [m$^3$/s]</th>
<th>$\zeta$</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse</td>
<td>Stenay</td>
<td>3</td>
<td>100</td>
<td>$k_1$ [day]</td>
<td>$k_2$ [-]</td>
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<td>2.180</td>
<td>0</td>
<td>0.9961</td>
</tr>
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<td>Carignan</td>
<td>2</td>
<td>100</td>
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<td>0.295</td>
<td></td>
<td></td>
<td>0.9420</td>
<td>0.9979</td>
</tr>
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<td>Membre</td>
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<td>75</td>
<td>4.23</td>
<td>0.675</td>
<td></td>
<td></td>
<td>0.0920</td>
<td>0.9957</td>
</tr>
<tr>
<td>Viroin</td>
<td>Treignes</td>
<td>3</td>
<td>10</td>
<td>0.055</td>
<td>1.658</td>
<td></td>
<td></td>
<td>0.9614</td>
<td>0.9975</td>
</tr>
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<td>Gendron</td>
<td>2</td>
<td>100</td>
<td>2.00</td>
<td>0.498</td>
<td></td>
<td></td>
<td>0.9170</td>
<td>0.9860</td>
</tr>
<tr>
<td>Sambre</td>
<td>Namur</td>
<td>2</td>
<td>100</td>
<td>5.10</td>
<td>0.623</td>
<td></td>
<td></td>
<td>0</td>
<td>0.9957</td>
</tr>
<tr>
<td>Meharga</td>
<td>Moha</td>
<td>1</td>
<td>10</td>
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<td></td>
<td></td>
<td></td>
<td>1.6029</td>
<td>0.9884</td>
</tr>
<tr>
<td>Ourthe</td>
<td>Tabreux</td>
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<td>3.083</td>
<td></td>
<td></td>
<td>0.1929</td>
<td>0.9986</td>
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<td>Martinrive</td>
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<td>50</td>
<td>18.18</td>
<td>1.091</td>
<td></td>
<td>2.10</td>
<td>0</td>
<td>0.9596</td>
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<tr>
<td>Vesdre</td>
<td>Chaudfontaine</td>
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<td>0.86</td>
<td>0.150</td>
<td></td>
<td></td>
<td>0</td>
<td>0.9047</td>
</tr>
</tbody>
</table>

* unit dependent on $\nu$
Figure 13.5 Examples of baseflow hydrographs for the Lesse

criterion to be used was not clear beforehand.

This is because the ordinary least squares criterion is found unsuitable to determine the coefficients, as an error of 1 m³/s during low flows is much more important than an error of 1 m³/s having a relatively great baseflow. A log-transformation of the discharges (cf. for example Bako et al., 1988) will improve the result. However, when having very small discharges the measuring error and other disturbances will be relatively great (due to vegetation for example), which makes the very small discharges less suitable for calibration. After considering several criterion expressions, a square root transformation has been chosen, as it suits the difficulties mentioned and is relatively simple:

$$y_{r,i} = \sqrt{Q_{r,i}}$$

(13.16)

where:

- $Q_{r,i}$ measured discharge for period $r$ at time serial number $i$ [L³T⁻¹]
- $y_{r,i}$ square root of $Q_{r,i}$ [L¹⁵T⁻⁹]

The minimisation criterion yields:
\[ \Phi = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} (y_{r,i}^* - y_{r,i})^2 \]  
\( = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} \left( \sqrt{Q_{r,i}^*} - \sqrt{Q_{r,i}} \right)^2 \)  
\( \text{(13.17)} \)

where:

- \( \Phi \) = the value to be minimised  
- \( Q_{r,i}^* \) = calculated discharge for period \( r \) at time serial number \( i \)  
- \( y_{r,i} \) = square root of \( Q_{r,i}^* \)  
- \( m(r) \) = number of depletion discharges observed during period \( r \)  
- \( r_{\text{max}} \) = number of periods included in the calibration process

For model 1 it leads to:

\[ \Phi = \sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} \left( \sqrt{Q_0 \exp \left( \frac{t_{r,i} - t_0}{k_1} \right)} - \sqrt{Q_{r,i}} \right)^2 \]  
\( \text{(13.18)} \)

where:

- \( t_{r,i} \) = time for time serial number \( i \) in period \( r \)  
- \( t_0 \) = \( t_0 \) for period \( r \)

\( Q_0 \) is chosen, \( k_1 \) and \( t_0 \), \( r = 1, \ldots, r_{\text{max}} \) are determined. It is unavoidable to incorporate \( t_0 \), \( r \) in the calculation, because the moment in every period at which the discharge would have been equal to \( Q_0 \) is not known. Of all the parameters only the value of the decay constant \( k_1 \) is used to obtain the shape of the depletion curve in real-time. In real-time one baseflow value is needed to determine the \( t_0 \) of the actual baseflow course.

For the models 2 and 3 similar expressions can be written.

The performance of the models can be determined by several methods. Here the coefficient of determination is used, which reads for (13.17):

\[ R^2 = 1 - \frac{\sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} (y_{r,i}^* - y_{r,i})^2}{\sum_{r=1}^{r_{\text{max}}} \sum_{i=1}^{m(r)} (y_{r,i} - M_y)^2} \]  
\( \text{(13.19)} \)

where:
\[
M_y = \frac{\sum_{r=1}^{n} \sum_{t} y_{r,t}}{\sum_{r=1}^{n} m(r)}
\] (13.20)

Only the numerator of (13.19) is dependent on the model. The model achieves the best results when \( R^2 = 1 \).

For all the 10 subcatchments the coefficients for all three models have been calculated, and then the best model for each subcatchment has been selected.

Besides the coefficient of determination also a visual inspection of the results has been incorporated in the selection procedure. A choice on a visual basis for a certain subcatchment was made when the coefficients of determination of the two best models were almost the same.

For some subcatchments the results were questionable, as it appeared that a minimum discharge was maintained when using reservoirs. In those cases the models were altered by incorporating a constant \( Q_c \), for example for model 2:

\[
Q_b = Q_c + Q_0 \exp\left(-\frac{(t-t_0)}{k_2}\right)
\] (13.21)

That leads to slightly better results.

The Sambre has been modelled with the use of approximation 7, and by adding the minimum constraint \( Q_b = 5 \text{ m}^3/\text{s} \) (§ 6.6):

\[
Q_b = \max \left[ 5 \text{ m}^3/\text{s}, Q_0 \exp\left(-\frac{(t-t_0)}{k_2}\right) \right]
\] (13.22)

Ungauged areas are thought to behave similarly to nearby measured areas which have the same hydrologic properties. For example: the area west of the Chiers has approximately the same hydrologic properties as the Chiers subcatchment, rather than the Meuse upstream from Stenay. Therefore the whole area is assigned to the Chiers. However, if the coefficient of determination for a subcatchment is low, or the discharges of the catchment have been estimated, no ungauged areas have been assigned to those catchments. For all the areas which have an ungauged area added, the model results will be multiplied by a factor \( (1 + \zeta) \):

\[
Q_{b,u}(t) = (1 + \zeta) Q_b(t)
\] (13.23)

with:

\[
\zeta = \frac{A_u}{A}
\] (13.24)

where:

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Figure 13.6 The depletion curves for the selected subcatchments

\[ Q_{b,s}(t) = \text{baseflow of the subcatchment including the surrounding ungauged areas} \]
\[ Q_b(t) = \text{baseflow of the subcatchment} \]
\[ \zeta = \text{factor} \]
\[ A = \text{subcatchment area} \]
\[ A_s = \text{ungauged area which is assigned to the gauged area considered} \]

The selected models and the coefficients used can be found in Table 13.1. The results are presented graphically in Fig. 13.6.

As no direct comparison of the depletion curves can be made with the use of that table, also the coefficients of the first model are included in this report (Table 13.2). It should be noted that the decay constant \( k_1 \) for the Vesdre is relatively great. Probably the reservoirs influenced the discharges. The idea is strengthened by the small value of the coefficient of determination.

The equations indicated in Table 13.1 were entered in the model. In the total forecast 10 m³/s are subtracted for withdrawal by canals, etcetera (cf. chapter 3). The model will be explained further in § 13.5, here it is remarked that the input of the model requires baseflow discharges at various stations in the catchment.

For the Meuse upstream from Liège several investigations after the baseflow have been executed. VandeWiele et al. (VandeWiele, 1989, VandeWiele et al., 1989a,b) developed a continuous flow simulation model for the Meuse at Liège on a weekly basis. They found that the decay constant \( k_1 \) for the baseflow was equal to 46 days. The value is high compared
to the other values of \( k_1 \) in table 13.2. Three explanations for the difference can be given:

a. The baseflow at Maastricht-St.-Pieter is for the greater part determined by the baseflows of the Chiers and of the Meuse at Stenay. Those subcatchments have relatively high values of \( k_1 \).

b. VandeWiele used discharge series of Borgharen for the period 1951-1983, which were transposed to Liège (and corrected for the difference of discharge between those stations) (VandeWiele, 1989b). An error in the correction of the difference will be relatively constant in time. As no constant discharge is modelled in the study of VandeWiele, an erroneous value for the extractions may influence the value of the decay constant \( k_1 \) significantly.

c. During the period 1951-1983 the catchment changed with respect to reservoirs and reservoir operation, probably causing an increase of the lower discharges. In the study presented in this thesis only low flows in the second half of the 1971-1981 period have been used.

Van der Made (De Wildt, 1983b) found a decay constant \( k_1 \) equal to 70 days for Liège without any baseflow reservoir recharge, and 105 days as an average course of the baseflow during the summer/autumn season with an occasional rainy period. The same kind of explanation may be given for the high values of \( k_1 \) compared to those in table 13.2.

<table>
<thead>
<tr>
<th>River</th>
<th>Gauging station</th>
<th>( k_1 ) [day]</th>
<th>( R^2 ) [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse</td>
<td>Stenay</td>
<td>27.0</td>
<td>0.9953</td>
</tr>
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<td>Chiers</td>
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<td>Semois</td>
<td>Membre</td>
<td>13.3</td>
<td>0.9878</td>
</tr>
<tr>
<td>Viroin</td>
<td>Treignes</td>
<td>29.4</td>
<td>0.9930</td>
</tr>
<tr>
<td>Lesses</td>
<td>Gendron</td>
<td>12.8</td>
<td>0.9844</td>
</tr>
<tr>
<td>Sambre</td>
<td>Namur</td>
<td>25.6</td>
<td>0.9951</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>Moha</td>
<td>23.3</td>
<td>0.9884</td>
</tr>
<tr>
<td>Ourthe</td>
<td>Tabreux</td>
<td>17.2</td>
<td>0.9964</td>
</tr>
<tr>
<td>Amblève</td>
<td>Martinrive</td>
<td>17.9</td>
<td>0.9506</td>
</tr>
<tr>
<td>Vesdre</td>
<td>Chaudfontaine</td>
<td>71.4</td>
<td>0.8712</td>
</tr>
</tbody>
</table>

13.3 THE INFLUENCE OF RAINFALL ON THE BASEFLOW

Like any natural water reservoir also those of the baseflow are ultimately filled by rainfall. The temperate climate in which the Meuse catchment is situated causes a replenishment of the baseflow reservoirs, which may even occur occasionally in summer. Therefore the influence
of rain on the baseflow is analysed. To obtain a statistical distribution of the baseflow a stochastic rainfall model has been developed.

The purposes of the development of the rainfall model are threefold:
1. The baseflow data form only a part of the discharge data, as surface runoff and interflow may even occur in predominantly dry periods. However, a forecast of the baseflow can only be made if data of the baseflow are known. With the rainfall model, a baseflow forecast can be generated, using baseflow data back in time and adding the influence of the current wet period on the calculated baseflow.
2. If a rainfall forecast is available, the baseflow after the rainfall event can be estimated.
3. A longer term forecast can be developed with the use of the stochastic properties of the process of rainfall.

The influence of rainfall on baseflow reservoir storage is dependent on many time-dependent and time-independent parameters, such as evapotranspiration, surface runoff, current reservoir storage, soil moisture content, permeability, etcetera. Many models that simulate the process have been built for continuous use and they have a great data demand. For example, the Sacramento model developed for the Meuse (Vermaas, 1985) needs data of a period of one month as a starting period.

As the interest is focussed on baseflow, and data requirements should be kept to a minimum, the development of a continuous flow model is too much of a good thing. Therefore, and also because of the limited amount of data available in real-time, the input data are restricted to discharge and rainfall.

The basis for the analysis is the continuity equation:

$$\frac{dS}{dt} = -Q_b + A \phi(t)$$

(13.25)

where:
- $\phi(t) = \text{areal rainfall intensity at time } t$ [LT$^{-1}$]
- $A = \text{area of the subcatchment}$ [L$^2$]
- $g = \text{function to determine}$ [LT$^{-1}$]

In order to investigate the influence of rainfall, the hydrograph of a dry period interrupted by a period of rainfall is analysed.

The notation introduced in fig. 13.7 will be used in the next few lines. The curves 1 depict the discharge respectively reservoir storage without rain, the curves 2 include the period of rain. The wet period starts at $t_1$ and ends at $t_2$. At $t_3$ the direct runoff has come to an end.

Before $t_1$ the discharge decreases, following the depletion curve to $Q_1$ at $t_1$. The baseflow reservoir storage decreases to $S_1$ at $t_1$. If no rainfall occurred, curve 1 would also be valid after $t_1$. Then the discharge and the baseflow reservoir storage at $t_2$ would be equal to $Q_2$, respectively $S_2$, and at $t_3$ $Q_3$, respectively $S_3$. Due to the rain the actual discharge at $t_2$ is equal to $Q_2$. At $t_3$ the discharge is equal to the baseflow and is $Q_{32}$, the baseflow reservoir storage at that moment is $S_{32}$.

For reasons of simplicity, the baseflow reservoir recharge between $t_2$ and $t_3$ will be thought to have occurred between $t_1$ and $t_2$. Then the data at $t_3$ can be used to calculate the
values at \( t_2 \): baseflow and baseflow reservoir storage are equal to \( Q_{22} \) respectively \( S_{22} \).

The course of baseflow and baseflow reservoir storage between \( t_1 \) and \( t_2 \) cannot be determined. From fig. 13.7 it is concluded that, due to the rain, the baseflow reservoir storage at \( t = t_2 \) is equal to \( S_{22} \) instead of \( S_{21} \).

\( S_{22} - S_{21} \) may be calculated by way of the following steps, using model 2 as an example:

1. Determine \( t_0 \) for the first baseflow hydrograph, and call that \( t_0 \) further \( t_{01} \). Rewriting
(13.8) gives:

\[ Q_1 = Q_0 \exp \left( -\frac{(t_1 - t_{01})^a}{k_2} \right) \]  

(13.26)

Further rewriting yields \( t_{01} \):

\[ t_{01} = t_1 - \left( k_2 \ln \frac{Q_0}{Q_1} \right)^{\frac{1}{a}} \]  

(13.27)

2. Determine \( t_{02} \), the \( t_0 \) for the second hydrograph. Similar to step 1 it is found:

\[ t_{02} = t_3 - \left( k_2 \ln \frac{Q_0}{Q_{32}} \right)^{\frac{1}{a}} \]  

(13.28)

3. As \( t_{01} \) and \( t_{02} \) are known, \( S_{22} - S_{21} \) can be calculated easily:

\[
S_{22} - S_{21} = \int_{t_2}^{t_{01}} Q_0 \exp \left( -\frac{(t-t_{01})^a}{k_2} \right) dt - \int_{t_2}^{t_{02}} Q_0 \exp \left( -\frac{(t-t_{02})^a}{k_2} \right) dt \\
= Q_0 \int_{t_2}^{t_{02}} \exp \left( -\frac{t^a}{k_2} \right) dt 
\]  

(13.29)

For the other models similar expressions are obtained.

The number of available calibration periods was limited, because a baseflow should be determinable before and after the period of rain. The number of suitable periods for each subcatchment varied between 5 and 8. The number of realisations is so small that an accurate calculation of baseflow reservoir recharge in time was impossible, although several attempts have been made. No method proved to be superior to a simple method where only a constant fraction of the rainfall volume recharges the baseflow reservoir:

\[
S_{22} = S_{21} + A \Psi \int_{t_1}^{t_2} p(t) dt 
\]  

(13.30)

where \( \Psi \) is a dimensionless coefficient between 0 and 1. For reasons of simplicity \( \Psi \) is considered to be constant for a subcatchment.

\( \Psi \) can be interpreted as the ratio between the increase of the volume of the baseflow reservoir at the end of the considered period of rain and the total volume of rainfall in units of volume. Hence:
Figure 13.8 Simulation of baseflow reservoir storage for the Lesse tributary

\[
\psi = \frac{S_{22} - S_{21}}{\int_{t_1}^t \rho(t) \, dt}
\]  

(13.31)

For practical reasons \(\psi\) was computed in two steps. First the part of the rainfall that contributes to the discharge has been calculated. That part is called \(\psi_1\). It is not equal to \(\psi\), because \(\psi\) relates to the increase of the baseflow reservoir storage at the end of the wet period only. Therefore \(\psi_1\) has to be multiplied by a factor \(\psi_2\) in order to obtain \(\psi\), where \(\psi_2\) is the ratio of the increase of the volume of baseflow reservoir storage that is still present at \(t_2\) and the total increase in volume that will discharge.

\(\psi_1\) is calculated as the mean of \(\psi_{1r}\), where \(r\) denotes the serial number of the calibration event. \(\psi_{1r}\) can be written as:
TABLE 13.3 COEFFICIENTS $\psi_1$, $\psi_2$ AND $\psi$ FOR THE SUBCATCHMENTS

<table>
<thead>
<tr>
<th>River</th>
<th>Gauging station</th>
<th>$\psi_1$</th>
<th>$\psi_2$</th>
<th>$\psi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meuse</td>
<td>Stenay</td>
<td>0.11</td>
<td>0.85</td>
<td>0.09</td>
</tr>
<tr>
<td>Chiers</td>
<td>Carignan</td>
<td>0.15</td>
<td>0.84</td>
<td>0.13</td>
</tr>
<tr>
<td>Semois</td>
<td>Membre</td>
<td>0.10</td>
<td>0.69</td>
<td>0.07</td>
</tr>
<tr>
<td>Viroin</td>
<td>Treignes</td>
<td>0.11</td>
<td>0.70</td>
<td>0.08</td>
</tr>
<tr>
<td>Lessse</td>
<td>Gendron</td>
<td>0.13</td>
<td>0.73</td>
<td>0.10</td>
</tr>
<tr>
<td>Sambre</td>
<td>Namur</td>
<td>0.13</td>
<td>0.53</td>
<td>0.07</td>
</tr>
<tr>
<td>Mehaigne</td>
<td>Moha</td>
<td>0.13</td>
<td>0.53</td>
<td>0.07</td>
</tr>
<tr>
<td>Ourtre</td>
<td>Tareaux</td>
<td>0.11</td>
<td>0.70</td>
<td>0.07</td>
</tr>
<tr>
<td>Amblève</td>
<td>Martinrive</td>
<td>0.22</td>
<td>0.68</td>
<td>0.15</td>
</tr>
<tr>
<td>Vesdre</td>
<td>Chaudfontaine</td>
<td>0.24</td>
<td>0.71</td>
<td>0.17</td>
</tr>
</tbody>
</table>

\[
\psi_1 = \frac{\int Q(t) \, dt + S_{32, r} - S_{1, r}}{t_2 - t_1} \quad (13.32)
\]

where:

\[
Q(t) = \text{(total) discharge} \quad [L^3T^{-1}]
\]

The subscript $r$ denotes the calibration period considered.

$\psi_2$ is calculated as the mean of $\psi_{2, r}$. $\psi_{2, r}$ can be written as:

\[
\psi_{2, r} = \frac{S_{22, r} - S_{21, r}}{t_2 - t_1} \quad (13.33)
\]

For Belgium, mean daily areal rainfall data of the IRM have been used (Anonymous, Lames d'eau menuelles précipités sur les bassins hydrographiques belges/Maandelijkse gebiedsneerslagen voor de Belgische stroombekkens). For the subcatchments in France, the Chiers and the Meuse upstream from Stenay, Thiessen Polygons with the use of 4 respectively 10 stations have been used for the determination of the mean daily areal rainfall. As the depletion curve of the Sambre is relatively inaccurate, for the Sambre the same coefficients $\psi_1$ and $\psi_2$ have been adapted as for the Mehaigne. Table 13.3 gives the
calculated coefficients. Only the coefficients $\psi$ are used in the model.

It should be noted that for the operational use the wet period is subdivided into as many periods as there are wet days. So there are as many combinations of $t_1$ and $t_2$ as there are wet days. That will lead to a slightly different value of the baseflow, as $t_2$ is calibrated to be at the end of the wet period, instead of each wet day. For reasons of simplicity, this variation will be neglected.

An example of a simulation of the baseflow reservoir storage is given in fig. 13.8.

13.4 STOCHASTIC RAINFALL PREDICTION

Rainfall is a process that is correlated both in space and in time. Both the spatial and the time component are significant in the Meuse catchment, as will be demonstrated in this section. However, the development of a multiple probability density function, containing all the correlations, was considered to be too complex. Hence, a rainfall simulation model was developed.

It is important to note that the stochastic rainfall prediction model is not a purpose on its own, but only a supplement of the baseflow forecast. It allows some simplifications to be made.

The analysis is made with the use of rainfall and discharge data of the period for which areal rainfall data were published (1969-1978). According to the needs of Rijkswaterstaat model forecasts will only be generated from 1 May to 31 October.

First the spatial component of the process has been analysed. Computing the spatial correlation structure based on daily measurements is not promising ($\S$ 10.3.2). Therefore a longer time basis has been chosen: the relationship between monthly rainfall values was determined. However, a calibration period of 10 years gives only 10 monthly realisations of monthly rainfall depths per subcatchment. The value of 10 is considered to be too small for having a representative population, therefore monthly rainfall depths of 3 successive months will be considered to belong to the same population. That gives 30 realisations per period.

Months with predominantly the same kind of rainfalls have been joined, resulting in the combinations of April-May-June (spring situation), July-August-September (summer showers) October-November-December (frontal zones) respectively, which is the same subdivision as used by Bastin et al. (1984) for an investigation of spatial variograms of rainfall intensity in the Semois basin. The optimum linear relation between monthly rainfall depths of two catchments has been determined. The calculated coefficients of determination for the period of April-May-June are shown in table 13.4, the other coefficients in Korsten (1989). Only logical combinations have been calculated (which means starting from the Chiers going in a northern direction, as can be seen later).

Table 13.4 clearly shows a certain influence of distance. Values over 0.9 only occur for adjacent subcatchments.

A computer subroutine generating random numbers considering all the stochastic properties of monthly rainfall would be ideal, but was not available. Therefore a simpler method has been developed.

First a subcatchment is chosen that has a relatively great baseflow and is reasonably in the centre of the catchment of the Meuse.

For the central subcatchment $A_1$ the statistical properties of the monthly rainfall
<table>
<thead>
<tr>
<th>Tributary</th>
<th>Chiers</th>
<th>Meuse (Stenay)</th>
<th>Semois</th>
<th>Viroin</th>
<th>Lesse</th>
<th>Sambre</th>
<th>Mehaigne</th>
<th>Ourthe</th>
<th>Amblève</th>
<th>Vesdre</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chiers</td>
<td>1</td>
<td>0.78</td>
<td>0.83</td>
<td>0.65</td>
<td>0.77</td>
<td>0.58</td>
<td>0.44</td>
<td>0.76</td>
<td>0.57</td>
<td>0.52</td>
</tr>
<tr>
<td>Meuse (Stenay)</td>
<td></td>
<td>1</td>
<td>0.61</td>
<td>0.54</td>
<td>0.54</td>
<td>0.51</td>
<td>0.36</td>
<td>0.58</td>
<td>0.44</td>
<td>0.38</td>
</tr>
<tr>
<td>Semois</td>
<td></td>
<td></td>
<td>0.76</td>
<td>0.92</td>
<td>0.69</td>
<td>0.43</td>
<td>0.91</td>
<td>0.79</td>
<td>0.74</td>
<td></td>
</tr>
<tr>
<td>Viroin</td>
<td></td>
<td></td>
<td></td>
<td>0.80</td>
<td>0.93</td>
<td>0.70</td>
<td>0.78</td>
<td>0.68</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>Lesse</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.72</td>
<td>0.54</td>
<td>0.95</td>
<td>0.80</td>
<td>0.77</td>
<td></td>
</tr>
<tr>
<td>Sambre</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.73</td>
<td>0.73</td>
<td>0.68</td>
<td>0.75</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mehaigne</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.54</td>
<td>0.50</td>
<td>0.56</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ourthe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.89</td>
<td>0.83</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Amblève</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>0.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vesdre</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
depths, in particular the mean and the standard deviation, are estimated. After assuming a normal distribution for the error, values of monthly rainfall depths can be generated.

The corresponding value of a nearby subcatchment $A_2$ is found by using the linear regression equation:

$$
\bar{P}_{\text{month } A_2} = \beta_0 + \beta_1 \bar{P}_{\text{month } A_1} + e_s
$$

where:

$$
\bar{P}_{\text{month } A_x} = \text{monthly areal rainfall depth for area } A_x
$$

and $e_s \sim N(0,1)$. The regression coefficients $\beta_0$ and $\beta_1$ and the estimated standard deviation $s$ are already available, as they have been calculated in order to obtain the coefficients of determination. In a similar way monthly rainfall values of other subcatchments have been generated.

The connection between the subcatchments has a kind of tree-structure. As an example the tree-structure of April-May-June is shown in fig. 13.9. The Chiers is the central catchment. The monthly areal rainfall depth of the Ourthe is determined by using the equation (13.34) with the Chiers for $A_1$ and the Ourthe for $A_2$. The monthly areal rainfall of the Amblève is determined by using (13.34) with the Ourthe for $A_1$ and the Amblève for $A_2$. Of course, the constants $\beta_0$, $\beta_1$ and $s$ should be adapted to that combination of subcatchments. The direct relation between monthly rainfall of Chiers and Amblève is ignored. The tree-structure is obtained by interpreting the matrix of the coefficients of determination, and looking for high values of the coefficient of determination. For the other 3 month periods other matrices and tree-structures are obtained.

Autocorrelation of the rainfall depth in time is neglected. Hence, the simulated values of the monthly areal rainfall of the central subcatchment Chiers are independent in time.

The simulation of monthly rainfall depths is the most important part of the stochastic baseflow determination. However, the distribution of the monthly rainfall depths over the months is also important with respect to the baseflow.

Five distribution algorithms for the central subcatchment have been proposed:

a. equal amounts on every day of the month
b. three equal amounts distributed randomly on 3 different days of the month
c. four equal amounts distributed randomly on 4 different days of the month
d. four equal amounts on the 3rd, 11th, 19th and 28th day of the month
e. three amounts in the proportion of $x:y:z$ distributed randomly on 3 different days of the month.

The algorithms were tested on the Lesse and Chiers subcatchments. Measured and calculated discharges according to the five methods have been compared for a few periods. For reasons of simplicity, the random choices were replaced by the dates on which actually the greatest depths of rainfall they occurred. The ratio of $x:y:z$ has also been chosen according to the values as occurred in reality. Besides the rainfall data, also the baseflow at the beginning of the month has been used as an input. (It should be noted that for the Lesse subcatchment also data after 1978 were available and have been used here.)

The simulation results for the Chiers and Lesse subcatchments can be found in table 13.5. The root of the mean square residual $s_0$ is also given in the table.

<table>
<thead>
<tr>
<th>subcatchment</th>
<th>period</th>
<th>measured baseflow [m³/s]</th>
<th>simulated baseflow using method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>a [m³/s]</td>
</tr>
<tr>
<td>Chiers</td>
<td>August 1971</td>
<td>7.56</td>
<td>7.50</td>
</tr>
<tr>
<td>Chiers</td>
<td>June 1977</td>
<td>8.61</td>
<td>8.62</td>
</tr>
<tr>
<td>Chiers</td>
<td>July 1977</td>
<td>10.15</td>
<td>9.92</td>
</tr>
<tr>
<td>Lesse</td>
<td>August 1974</td>
<td>2.49</td>
<td>2.91</td>
</tr>
<tr>
<td>Lesse</td>
<td>August 1981</td>
<td>5.55</td>
<td>6.06</td>
</tr>
<tr>
<td>$s_0$</td>
<td></td>
<td></td>
<td>0.575</td>
</tr>
</tbody>
</table>

The methods a and d are the simplest, as no extra data generation is necessary. Method e is the most complex, because besides a distribution over the month also the ratio of $x:y:z$ has to be determined.

Simulation results show that the methods a and d give a relatively poor performance, probably because the stochastic process of rainfall within the month has been neglected. The method e gives the best simulation results. But with respect to earlier simplifications the extra determination of the ratio seems to be not worthwhile. The methods b and c seem to be appropriate. As the performance of method c is slightly better than that of method b, method c has been chosen.

It is assumed that the conclusion is also valid in the other subcatchments.

Summarising: The rainfall distribution simulation within a month in the central subcatchment takes place by randomly choosing four different days, and assigning to them each a quarter of the generated monthly depth of rainfall.
The distribution over the time of the dates of rainfall in the central catchment and those in other catchments are correlated. To simulate that part of the process, the tree-structure of fig. 13.9 is used (for the months of April-May-June). The days for the central catchment are chosen randomly. With a chance $\xi$, a chosen day is maintained in the adjacent subcatchment (according to the tree-structure) and with a chance $(1 - \xi)$ another day is determined randomly. Also here the direct correlation between non-adjacent subcatchments is neglected.

The chance $\xi$ is determined by comparing the days with depths of rainfall for different subcatchments. It was found that with a chance of 0.75, the day belonging to the four days with the highest depths of rainfall, corresponds with one of the four periods in another nearby catchment with its highest depths of rainfall. Therefore, $\xi$ was given the value 0.75. It must be admitted that the method is only a rough estimation, but it suits the purpose reasonably.

13.5 THE MODEL

In the preceding sections the components of the baseflow forecasting model have been described. They showed a lot of ways of obtaining a forecast, dependent on the availability of rainfall forecasts, the availability of baseflow data at the moment of forecasting, the kind of forecast (dry weather forecast/stochastic forecast), etcetera. In this section ways of calculation and data input will be elucidated.

The kind of computation may change at several moments. The points of time which are used in the model are:

\[ t_b \] = day on which baseflow in subcatchment $j$ is known [day]

\[ t_s \] = day on which the forecast is made [day]

\[ t_f \] = last day for which rainfall forecasts are available [day]

\[ t_e \] = last day for which baseflow forecasts are requested [day]

In fig. 13.10 the time axis is drawn. It should be noted that the sequence of dates must not be changed, but coincident points of time are allowed.

Figure 13.10 The time axis in the baseflow forecasting model

In fig. 13.11 a simplified computing scheme is shown, using the following notation:

\[ Q_s(t) \] = baseflow in subcatchment $j$ on day $t$ [m$^3$/s]

\[ P(i,t) \] = depth of rainfall in gauge $i$ on day $t$ [mm]

\[ P_A(j,t) \] = areal rainfall depth for subcatchment $j$ on day $t$ [mm]
START

give \( t_s \) and \( t_e \)

**MENU A**

- baseflows known at \( t_s \) (\( t'_{j} = t_s \) for all \( j \)) .................................................. 1
- baseflows known at a date before \( t_s \) (\( t'_{j} < t_s \) for some \( j \))
  - for the intermediate period are known
  - raingauges measurements (per day) .................................................. 2
  - areal rainfalls per subcatchment (per day) ........................................... 3

**INPUT DATA**

if \( A = 1 \)  \( \) give \( Q_j(t_s) \) for each subcatchment \( j \)
if \( A = 2 \) or \( A = 3 \)  \( \) give \( t'_j \) and \( Q_j(t'_j) \) for each subcatchment \( j \)
if \( A = 2 \)  \( \) give \( P(t) \) for each gauge for all \( \min_{j} t'_j \leq t < t_s \)
if \( A = 3 \)  \( \) give \( P_{A,j}(t) \) for each subcatchment for all \( t'_j \leq t < t_s \)

**MENU B**

- no rainfall forecast available (\( t_f = t_s \)) .................................................. 1
- dry weather flow ................................................................................. 1
- rainfall simulation .................................................................................. 2
- rainfall forecast available (\( t_f > t_s \))
  - areal rainfall forecast per subcatchment (per day)
    - dry weather flow after \( t_f \) ............................................................... 3
    - rainfall simulation after \( t_f \) ......................................................... 4
  - areal rainfall forecast for whole catchment (per day)
    - dry weather flow after \( t_f \) ............................................................... 5
    - rainfall simulation after \( t_f \) ......................................................... 6

**INPUT DATA**

if \( B = 3 \) or \( B = 4 \) or \( B = 5 \) or \( B = 6 \)  \( \) give \( t_f \)
if \( B = 3 \) or \( B = 4 \)  \( \) give \( \hat{P}_{A,j}(t) \) for each subcatchment \( j \) for all \( t_s \leq t < t_f \)
if \( B = 5 \) or \( B = 6 \)  \( \) give \( \hat{P}_{w}(t) \) for all \( t_s \leq t < t_f \)
if \( B = 2 \) or \( B = 4 \) or \( B = 6 \)  \( \) give number of simulations

**CALCULATIONS** using given rainfall for \( \min_{j} t'_j \leq t < t_f \)

if \( B = 2 \) or \( B = 4 \) or \( B = 6 \) using rainfall simulations for \( t_f \leq t \leq t_s \)
if \( B = 1 \) or \( B = 3 \) or \( B = 5 \) using depletion curves only for \( t_f \leq t \leq t_s \)

OUTPUT -------- END
\[ P_{A}(t) = \text{areal rainfall depth for the Meuse catchment on day } t \] [mm]
\[ \min_j t^j_b = \text{minimum value of } t^j_b \text{ for all } j \] [day]

Parameters provided with a circumflex accent are forecasts.

The input/output procedures work interactively.

The output may be obtained graphically and/or numerically. Forecast discharges are given on the day of forecast and the next 7 days, as well as on the first days of the next four weeks and the first day of every month and finally at \( t_e \).

**TABLE 13.6 CALCULATION OF THE BASEFLOW OF THE MEUSE (500 SIMULATIONS)**

| date      | base flow [m³/s] | chance of not exceeding
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>no precipitation</td>
<td>5%</td>
</tr>
<tr>
<td>1 May</td>
<td>162</td>
<td>162</td>
</tr>
<tr>
<td>2 May</td>
<td>152</td>
<td>152</td>
</tr>
<tr>
<td>3 May</td>
<td>144</td>
<td>144</td>
</tr>
<tr>
<td>4 May</td>
<td>137</td>
<td>137</td>
</tr>
<tr>
<td>5 May</td>
<td>130</td>
<td>130</td>
</tr>
<tr>
<td>6 May</td>
<td>125</td>
<td>125</td>
</tr>
<tr>
<td>7 May</td>
<td>119</td>
<td>119</td>
</tr>
<tr>
<td>8 May</td>
<td>114</td>
<td>114</td>
</tr>
<tr>
<td>15 May</td>
<td>87</td>
<td>88</td>
</tr>
<tr>
<td>22 May</td>
<td>68</td>
<td>74</td>
</tr>
<tr>
<td>29 May</td>
<td>55</td>
<td>63</td>
</tr>
<tr>
<td>1 June</td>
<td>51</td>
<td>59</td>
</tr>
<tr>
<td>1 July</td>
<td>27</td>
<td>40</td>
</tr>
<tr>
<td>1 August</td>
<td>18</td>
<td>34</td>
</tr>
<tr>
<td>1 September</td>
<td>13</td>
<td>30</td>
</tr>
<tr>
<td>1 October</td>
<td>10</td>
<td>28</td>
</tr>
<tr>
<td>31 October</td>
<td>8</td>
<td>20</td>
</tr>
</tbody>
</table>

*Opposite:*
*Figure 13.11 Simplified flow scheme for baseflow forecasting model*
The water withdrawal upstream from Maastricht-St.-Pieter was estimated to be 10 m³/s. Travel times have been neglected.

The simulation forecasts are given in 4 columns, specifying a dry-weather forecast, and three columns giving the baseflow with a 5%, 50% and 95% chance of non-exceeding. An example is given in table 13.6 and fig. 13.12.

**Model performance**

The model performance is usually tested by comparing computed results with measurements. However that is not possible in a satisfactory way for a stochastic model in combination with a small number of realisations.

Parts of the model might be tested. Here only a comparison between measured discharges and calculated baseflow is made for the months of July and August 1978, using measured rainfalls during those months and discharges of July 1st (fig.13.13). It seems that even for a period as long as 2 months containing a period of surface flow and interflow, the calculated baseflow approximates the measured discharge.

Between the moment of putting into use the model and that of writing this thesis, some experience was gathered. The discharge at Maastricht-St.-Pieter was significantly influenced by weirs, reservoirs, hydraulic power plants, and the withdrawal of water (Chapters 3, 6 and 7). Moreover periods with low flows are used for the execution of repairs. In 1989 in several reaches in the Belgian Meuse all the water was removed to carry out some repairs. Those facts cause a great change in discharge during consecutive days, thus making forecasts with a

---

*Figure 13.12 An example of the graphical model output*
lead time of one day useless. However, it does show that a model state updating by using measured discharges at Maastricht-St.-Pieter is inappropriate.

The model gives a forecast that should be seen as representative of the natural baseflow. It gives a decisive idea of the mean value of the baseflow during several days.

Data gathering gave some problems. Water levels appeared to be too low to get any information from a stage-discharge curve. At a moment the sum of the discharges at Stenay, Membre, Treignes and Carignan equalled that of Chooz, which caused doubts about the accuracy of the data.

The model also seems to be useful if during a period of rain information about the baseflow after the period of rain is desired.

Until now, the model has not been operational during a period of very small discharges, because of the absence of those discharges.

This chapter described a baseflow forecasting model for the Meuse at Maastricht-St.-Pieter. It has options for using rainfall forecasts and rainfall simulation. The output is presented stochastically. Results promise to be valuable if correct measurements are available and the current situation of human interference is known.
Appendix 1. The influence of the reservoir operation on the discharge and discharge modelling

If the operation of reservoirs is unknown the risk is run of making mistakes in rainfall-runoff modelling. That applies both to the calibration phase and to the real-time use. In the following paragraphs an attempt will be made to arrive at a quantitative evaluation of such a mistake.

The starting point is the situation that the precipitations and the discharges are known, but not the reservoir operation pursued. In the catchment of the Meuse the measuring stations of the discharge are at great distances from the reservoirs. On that account only a small part of the catchment drains via the reservoir. The area that does not drain via the reservoir is called a and the area that does is called b.

For the sake of clarity a simple discharge model has been chosen for this analysis. It is assumed that the reservoir either retains all the water that is supplied or lets it all pass. The precipitation is assumed to be constant with respect to place. The catchment is supposed to be uniform as to its hydrologic properties. The aspect of time is left aside. The baseflow is neglected. On account of those suppositions the following analysis is not justified for the Roer and the Rurtalsperre.

If the dam lets pass all the water the discharge is:

\[ Q = (K_a + K_b) \bar{p} \]  
(A1.1)

where:

- \( Q \) = discharge \quad [L^3T^{-1}]
- \( \bar{p} \) = areal precipitation intensity \quad [LT^{-1}]
- \( K_a \) = runoff parameter for area a \quad [L^2]
- \( K_b \) = runoff parameter for area b \quad [L^2]

The runoff parameters may be considered to be composed of an area multiplied by a runoff coefficient. The parameters \( K_a \) and \( K_b \) are linearly dependent on the areas a and b. From the suppositions it follows:

\[ \frac{K_a}{A_a} = \frac{K_b}{A_b} \]  
(A1.2)

where:

- \( A_a \) = area of area a \quad [L^2]
- \( A_b \) = area of area b \quad [L^2]

The total area of the catchment considered is \( A_a + A_b \). It is obvious that both \( A_a \) and \( A_b \) are greater than zero.

If the dam lets no water pass (A1.1) is modified to:
\[ Q = K_a \hat{p} \]  

\textit{Q1.3}

In the calibration the parameters \( K_a \) and \( K_b \) are determined as well as possible, which leads to \( K_a' \) and \( K_b' \).

Analogously to (A1.2) it applies:

\[ \frac{K_a'}{A_a} = \frac{K_b'}{A_b} \]  

\textit{Q1.4}

\( K_a' \) = calculated runoff parameter for area a \[ \text{[L}^2\text{]} \]

\( K_b' \) = calculated runoff parameter for area b \[ \text{[L}^2\text{]} \]

It is possible that it is supposed that the reservoir lets pass the water (it is thought that \( K_a + K_b \) is calculated), but that in reality the reservoir has retained the water (\( K_a \) was calculated), so that the error has been made that \( K_a' + K_b' \) is supposed to be equal to \( K_a \).

In a more general way: When calibrating it may be supposed that the dam has or has not let pass any water (\( K_a' + K_b' \) or \( K_a' \) respectively has been calculated). In reality the reservoir may or may not have passed the water (\( K_a + K_b \) or \( K_a \) respectively has been calculated).

All taken together it produces twice 2, so 4 possibilities for calibration (left half of table A1.1).

For the real-time phase a similar argumentation can be followed. The discharge is forecast, starting from \( \dot{Q} = K_a' \hat{p} \) or \( \dot{Q} = (K_a' + K_b') \hat{p} \). The discharge occurring is \( Q = K_a \hat{p} \) or \( Q = (K_a + K_b) \). Here, too, there are twice 2, so 4 possibilities (right half of table A1.1).

In all there are 4 times 4, so 16 possibilities to calibrate and to forecast a discharge, which have been presented schematically in table A1.1.

**TABLE A1.1 COMBINATIONS OF CALCULATED AND REAL RUNOFF PARAMETERS**

<table>
<thead>
<tr>
<th>Calibration phase</th>
<th>Operational phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reality</td>
<td>Model</td>
</tr>
<tr>
<td>( K_a )</td>
<td>( K_a' )</td>
</tr>
<tr>
<td>( K_a )</td>
<td>( K_a' + K_b' )</td>
</tr>
<tr>
<td>( K_a + K_b )</td>
<td>( K_a' )</td>
</tr>
<tr>
<td>( K_a + K_b )</td>
<td>( K_a' + K_b' )</td>
</tr>
</tbody>
</table>
Now for all the 16 possibilities the deviations in relation to the discharge occurring can be determined. The calculation will be illustrated by means of an example.

Suppose that in the calibration it is wrongly thought that the discharge has been retained by the reservoir. \( K'_a \) is calculated with \( Q = K'_a \bar{p} \). In reality \( K_a + K_b \) is calculated, so that \( K'_a = K_a + K_b \). Further suppose that in the operational phase it is assumed that the water is not retained and that this assumption is right. Then \( \dot{Q} = (K'_a + K'_b) \bar{p} \). In reality \( Q = (K_a + K_b) \bar{p} \) will occur.

In this example the relation between the discharge that has been forecast \( \dot{Q} \) and the discharge that has occurred \( Q \) is:

\[
\frac{\dot{Q}}{Q} = \frac{(K'_a + K'_b) \bar{p}}{(K_a + K_b) \bar{p}} = \frac{K'_a + A_b K'_b}{A_a} \frac{A_a + A_b}{A_b} = \frac{A_a + A_b}{A_b} \frac{K'_a}{K_a}
\]

\[
\left( \frac{A_a + A_b}{A_a} \right) \frac{K'_a}{K_a} = \left( \frac{A_a + A_b}{A_a} \right) \frac{K'_a}{K_b}
\]

\[
\frac{A_a + A_b}{A_a} \frac{K'_a}{K_a} = \frac{A_a + A_b}{A_a} \frac{K'_a}{K_b}
\]

It appears that also in other combinations the ratio is a function of \( (A_a + A_b)/A_a \).

Now the parameter \( \eta \) is introduced, being the ratio between the area that drains via the reservoir and the total area of the catchment. Hence:

\[
\eta = \frac{A_b}{A_a + A_b} = 1 - \frac{A_a}{A_a + A_b}
\]

(A1.6)

where:

\[
\eta = \text{ratio between the area that drains via the reservoir and the total area of the catchment}
\]

For all the 16 possibilities the discharge relations have been given in table A1.2. The ratios as functions of \( \eta \) have been presented in a diagram in fig. A1.1.

It may be clear that if the assumptions made in the calibration and operational phases are correct, the discharge relation is equal to 1 (the four corners in the table). But if the assumption made in the calibration phase is not altered in the operational phase, and moreover the operation of the reservoir is not altered as well, the forecast will also be correct (the diagonal in the table). In the other cases deviations will occur.

The greatest deviations arise if in the operational phase an operation is assumed different from the one in the calibration phase and that this train of thought is the opposite of reality. Then ratios of \((1 - \eta)^2\) and \((1 - \eta)^{-2}\) will arise.

The greatest mistakes can be avoided if in the calibration and in the operational phase the same assumptions are used. Then the occurring ratios are only 1, \((1 - \eta)\) and \((1 - \eta)^{-1}\). In fact in that case it is not necessary to make any assumption for the operation of the reservoir at all.
From table A1.2 it can be deduced that great errors can be made in catchments in which most of the water is discharged via the reservoir ($\eta$ is almost 1, consequently $A_p > A_d$). But especially there the operation of the reservoir can be determined with the greatest certainty. In those places the discharges measured will strongly have to be stuck to, or the operation pursued has to be known.

The discharge relations mentioned in table A1.2 are unlikely to occur in reality because mostly a mixed operation is pursued in relation to the release of water, so that the extremes are not realised. Consequently in reality the ratios will be nearer to 1.

When during a flood the outflow of the reservoir is greater than the inflow (it may occur when a reservoir is filled to the edge) the errors may become greater than the values given here.

The discharge relations mentioned have been brought about by means of a very simple model, in which in real-time the discharges measured up to the moment of the forecast have not been used. If those data were used the influences of the factors $K_a$ and $K_b$ in the discharge forecast will decrease and so will the error.
TABLE A1.2 RATIOS BETWEEN DISCHARGES FORECAST AND DISCHARGES REALLY OCCURRING

<table>
<thead>
<tr>
<th>OPERATIONAL PHASE</th>
<th>reality</th>
<th>( K_a )</th>
<th>( K_a' )</th>
<th>( K_a + K_b )</th>
<th>( K_a' + K_b' )</th>
<th>( K_a' )</th>
<th>( K_a' + K_b' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>reality</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>model</td>
<td>( K_a' )</td>
<td>1</td>
<td>1 - ( \eta )</td>
<td>( (1 - \eta)^{-1} )</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( K_a )</td>
<td></td>
<td>( K_a' )</td>
<td>( (1 - \eta)^{-1} )</td>
<td>1</td>
<td>( (1 - \eta)^{-1} )</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>( K_a + K_b )</td>
<td>( K_a' )</td>
<td>( 1 - \eta )</td>
<td>( (1 - \eta)^2 )</td>
<td>1</td>
<td>1 - ( \eta )</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>( K_a' + K_b' )</td>
<td>( K_a' )</td>
<td>1</td>
<td>( 1 - \eta )</td>
<td>( (1 - \eta)^{-1} )</td>
<td>1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Perhaps unnecessarily it is noted once more that only the error caused by the incorrect taking into account of the reservoir operation has been investigated.

**Example**

In the Meuse catchment above Borgharen the only important reservoirs for the reduction of floods are the reservoirs in the Vesdre and the Gileppe. About a quarter of the Vesdre catchment is drained via these reservoirs. That means that \( \eta \) is equal to 0.25. In such a case the discharge ratios vary from 0.56 to 1.78 (fig. A1.1). With great flood waves the Vesdre at Chaudfontaine will have a brief peak discharge of approximately 200 m\(^3\)/s, even if the baseflow is in the order of 20 m\(^3\)/s.

From the areas known it may be deduced that a maximum of 50 m\(^3\)/s is discharged via the reservoirs. According to the preceding theory the discharge forecast was to be between 112 m\(^3\)/s and 356 m\(^3\)/s (calculated using \( (1 - \eta)^2 \) and \( (1 - \eta)^{-2} \) respectively). The range in the discharge, however, will be smaller than 244 m\(^3\)/s because the part of the error caused in the calibration phase is fixed.

**Conclusions**

Reservoirs that as an additional objective have the reduction of flood discharges need not reduce the discharge during all flood waves. Therefore without the information on the reservoir operation the rainfall-runoff system is not to be determined univocally. With an estimate of the operation the forecasts can be improved, but with an incorrect estimate the error made strongly increases as compared to the situation that an estimate is not made.

If the operation is not known and changes continually, the accuracy of the model will be relatively small. In such cases the discharges measured will have to get a greater importance than with a reservoir operation that is known. That especially applies if the reservoir is close to the measuring station.

In the Meuse catchment above Borgharen the Vesdre reservoirs are of particular importance. If neither the reservoir operation nor the discharges are known in real-time the deviations caused by those reservoirs may amount to dozens of cubic metres per second.
Appendix 2. Flood-peak attenuation in tributaries

In most tributaries of the Meuse it appears that in general a flood wave has a pointed shape, i.e. during a flood the discharge increases in a short time to a high value and it decreases rapidly after the peak discharge. The peak of the flood wave can be attenuated by phenomena such as inundation of flood plains and friction. When calibrating a discharge forecasting model of a subcatchment the most downstream measuring station of discharges available is made use of.

Mostly a stretch of some kilometres is still to be found between the measuring station used and the main river. It is generally assumed that flood waves are not attenuated there, and therefore the calculation of the wave travel time is sufficient (cf. for example Lodder, 1982, Van Breusegem et al., 1987). In some tributaries, however, the distance between the measuring station and the mouth is considerable, as e.g. in the Chiers and the Ourthe. In this appendix it will be investigated if the supposition is correct that between the measuring station of discharges and the mouth the flood-peak attenuation is to be neglected.

The derivation of the equations is done according to the diffusion analogy. The starting-point is the system of St. Venant equations. Expressed per unit of width the system is (derived from (11.3) and (11.4)):

\[
\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial a}{\partial x} + g \frac{\partial z}{\partial x} + g \frac{u^2}{C^2a} = 0 \tag{A2.1}
\]

\[
\frac{\partial a}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{A2.2}
\]

where:

\begin{align*}
\text{u} & = \text{current velocity} \quad [\text{LT}^1] \\
\text{a} & = \text{water depth} \quad [\text{L}] \\
\text{g} & = \text{gravity acceleration} \quad [\text{LT}^{-1}] \\
\text{C} & = \text{Chézy friction coefficient} \quad [\text{LT}^{-1}] \\
\text{q} & = \text{discharge per unit of width} \quad [\text{LT}^1] \\
\text{t} & = \text{time} \quad [\text{T}] \\
\text{x} & = \text{distance} \quad [\text{L}] \\
\text{z} & = \text{bottom level (with respect to a horizontal reference level)} \quad [\text{L}]
\end{align*}

The effect of the inertia (\(du/dt\)) is of minor importance, so that the sum of the first two terms of (A2.1) may considered to be equal to zero, through which it is reduced to:

\[
\frac{\partial a}{\partial x} + \frac{\partial z}{\partial x} + \frac{u^2}{C^2a} = 0 \tag{A2.3}
\]

From differentiating (A2.3) and (A2.2) to \(\partial t\) and \(\partial x\) respectively and using \(q = ua\) the system follows:

313
\[
\frac{\partial^2 a}{\partial x \partial t} = -\frac{2q}{C^2 a^3} \frac{\partial q}{\partial t} + \frac{3q^2}{C^2 a^4} \frac{\partial a}{\partial t} - \frac{3q^2}{C^2 a^3} \frac{\partial q}{\partial x}
\]
(A2.4)

\[
\frac{\partial^2 a}{\partial x \partial t} = -\frac{\partial^2 q}{\partial x^2}
\]
(A2.5)

Combining (A2.4) and (A2.5) yields the following differential equation for \( q \):

\[
\frac{\partial q}{\partial t} - C^2 a^3 \frac{\partial^2 q}{2q \partial x^2} + \frac{3}{2} u \frac{\partial q}{\partial x} = 0
\]
(A2.6)

That equation can also be written as:

\[
\frac{\partial q}{\partial t} - K \frac{\partial^2 q}{\partial x^2} + c \frac{\partial q}{\partial x} = 0
\]
(A2.7)

where:

\[
K = \frac{C^2 a^3}{2q}
\]
(A2.8)

\[
c = \frac{3}{2} u
\]
(A2.9)

The dimension of \( K \) is \([L^2 T^{-1}]\), that of \( c \), the celerity, \([LT^{-1}]\).

Analogously it applies for \( a \), with neglecting of \( K \frac{\partial^2 a}{\partial x^2} \):

\[
\frac{\partial a}{\partial t} - K \frac{\partial^2 a}{\partial x^2} + c \frac{\partial a}{\partial x} = 0
\]
(A2.10)

With a moving coordinate system \( (y = x - ct) \) (A2.7) and (A2.10) are simplified to:

\[
\frac{\partial q}{\partial t} - K \frac{\partial^2 q}{\partial y^2} = 0
\]
(A2.11)

\[
\frac{\partial a}{\partial t} - K \frac{\partial^2 a}{\partial y^2} = 0
\]
(A2.12)

If \( K \) and \( c \) and consequently \( u \) are constant there exists an analytical solution. Assume that a volume of water \( P \) is instantaneously discharged at \( t = 0 \), then it follows:
\[
a(y,t) = \frac{P/B}{2\sqrt{\pi K t}} \exp\left(-\frac{y^2}{4Kt}\right) + a_b
\]

where:
\[
\begin{align*}
a_b &= \text{water depth belonging to the baseflow} \\
B &= \text{river width}
\end{align*}
\]

The factor \(1/(2\sqrt{\pi})\) in the equation results from the fact that the integral of the rises of the water level (over the distance) must be equal to \(P/B\).

At any point of time the water level is symmetrical with respect to the distance in relation to the point \(y = 0\). If the term before the exponential function decreases fast in time the flood-peak attenuation is likely to be great.

For \(q(y,t)\) and \(Q(y,t)\) are found respectively:

\[
q(y,t) = u \frac{P/B}{2\sqrt{\pi K t}} \exp\left(-\frac{y^2}{4Kt}\right) + q_b
\]

\[
Q(y,t) = u \frac{P}{2\sqrt{\pi K t}} \exp\left(-\frac{y^2}{4Kt}\right) + Q_b
\]

where:
\[
\begin{align*}
q_b &= \text{baseflow per unit of width} \\
Q_b &= \text{baseflow} \\
Q &= \text{discharge}
\end{align*}
\]

In the derivation of the latter equation \(Q = qB\) has been used.

The parameters \(K\), \(c\) and \(u\) are not constant, but they are functions of the water depth and the discharge. Yet they may be considered constant, because only a short stretch is considered, because only the course of the peak discharge is investigated and because here it is only investigated what the relevance of the flood-peak attenuation is.

The preceding equations were adopted after the diffusion analogy. More information on the application of the diffusion analogy on flood propagation in trapezoidal canals can be found in Gonwa et al. (1986).

As only the attenuation of the peak discharge is analysed only \(y = 0\) is interesting, so that (A2.15) is simplified into:
\[ Q(t) = u \frac{P}{2\sqrt{nKt}} + Q_b \]  \hspace{1cm} (A2.16)

Now assume that the peak enters the stretch considered at \( t_1 \) and leaves it again at \( t_1 + T \). It is necessary that \( T < t_1 \), because otherwise the supposition that \( K \) and \( c \) are to be considered constant is not satisfied sufficiently.

\( t_1 \) is an imaginary period between the moment of the release of the water and the moment at which the maximum discharge has become equal to \( Q_m \). \( Q_m \) may be filled in as desired and then \( t_1 \) can be calculated. With the aid of the length of the stretch \( L \) now \( T \) can also be calculated, making use of:

\[ T = \frac{L}{c} \]  \hspace{1cm} (A2.17)

where:

\[
\begin{align*}
T &= \text{wave travel time} \quad [T] \\
L &= \text{length of the stretch considered} \quad [L]
\end{align*}
\]

The shape of the wave entering the stretch considered is determined with the aid of (A2.15).

For reasons of simplicity the same values are taken for the parameters \( P, K, u \) and \( Q_b \) between the moments 0 and \( t_1 \) as between \( t_1 \) and \( t_1 + T \). From (A2.15) it follows that the simplification has no particular consequences for the shape of the entering wave.

Now the relative flood-peak attenuation (\textit{r.f.a.}) is introduced, being defined as:

\[ r.f.a. = \frac{Q(t_1) - Q(t_1 + T)}{Q_m} \]  \hspace{1cm} (A2.18)

where:

\[
\begin{align*}
r.f.a. &= \text{relative flood-peak attenuation} \quad [-] \\
Q_m &= \text{peak discharge at } t_1 \quad [L^3T^{-1}] \\
t_1 &= \text{moment at which the discharge is at its peak at the beginning of the stretch} \quad [T]
\end{align*}
\]

\( Q_m \) is equal to \( Q(t_1) \), but in the continuation of the analysis the two forms will be maintained for reasons of clarity in the next few expressions.

For calculating the flood-peak attenuation it is necessary to express \( u, K, t_1 \) and \( T \) in known values, viz. \( P, Q_m, i_b, B \) and \( C \). The water depth \( a \) is unknown and is not asked for either.

Eliminating \( a \) from \( Q_m = Bua \) and \( Q_m = BCa^{3/2}i_b^{1/2} \) (Chézy) produces for \( u \):
\[ u = \frac{\frac{2}{C^3 Q_m i_b^3}}{\frac{1}{B^3}} \]  

(A.2.19)

where:

\( i_b \) = bottom gradient

\( K \) is determined by substituting Chézy in the definition (A.2.8):

\[ K = \frac{C^2 a^3 B}{2Q_m} = \frac{Q_m}{2Bi_b} \]  

(A.2.20)

By rewriting (A.2.16) and substituting \( u \) and \( K \) an expression for \( t_1 \) arises:

\[ t_1 = \frac{P^2 u^2}{4\pi K(Q_m - Q_b)^2} = \frac{P^2 C^3 i_b^3 B^3}{2\pi Q_m (Q_m - Q_b)^2} \]  

(A.2.21)

The wave travel time amounts to:

\[ T = \frac{L}{c} = \frac{2L}{3u} = \frac{\frac{1}{2C^3 Q_m i_b^3 B^3}}{3} \]  

(A.2.22)

The relative flood-peak attenuation is (with (A.2.16) and (A.2.18)):

\[ r_{f.a.} = \left( \frac{u - \frac{P}{2\sqrt{\pi KT_1} - Q_b}}{2\sqrt{\pi KT_1} + Q_b} \right) \left( \frac{u - \frac{P}{2\sqrt{\pi KT_1} + T} - Q_b}{2\sqrt{\pi KT_1} + T} \right) \]  

(A.2.23)

Substituting \( u \), \( K \) and \( t_1 \) in the first term of the right hand member reduces the equation to:
\[ r.f.a. = \frac{Q_m - Q_b}{Q_m} \left( \sqrt{t_1 + T} - \sqrt{t_1} \right) \]  

(A2.24)

\[ \sqrt{t_1 + T} \text{ can be approximated using Taylor's series:} \]

\[ \sqrt{t_1 + T} = \sqrt{t_1} + \frac{1}{2} T t_1^{-\frac{1}{2}} + O(T^2) \]  

(A2.25)

Substituting (A2.25) in the second term of (A2.24) and assuming \( T < t_1 \) yields:

\[ \frac{\sqrt{t_1 + T} - \sqrt{t_1}}{\sqrt{t_1 + T}} = \frac{\frac{T}{2\sqrt{t_1}} + O(T^2)}{\sqrt{\frac{1}{2} + \frac{T}{2} + O(T^2)}} = \frac{\frac{1}{2} T + O(T^2)}{t_1 + \frac{1}{2} T + O(T^2)} = \frac{\frac{1}{2} T}{t_1 + \frac{1}{2} T} + O(T^2) = \frac{T}{2t_1} \]  

(A2.26)

By substituting the result in (A2.24) it is finally obtained:

\[ r.f.a. = \frac{2\pi L(Q_m - Q_b)^3}{3Q_m i_b^2 p^2 C^2} \]  

(A2.27)

The following conclusions may be drawn:
- Through some simplifications the r.f.a. is linearly related to the length of the stretch.
- The r.f.a. increases if the gradient decreases.
- The r.f.a. increases if the friction coefficient \( C \) decreases.
- If the peak discharge increases, but the volume of the flood wave remains equal, the r.f.a. will be greater.
- The r.f.a. increases as the baseflow decreases.
- In (A2.27) the width \( B \) is lacking. However, it does not imply that the width does not play a part. An increase of the width will result in a decrease of the mean depth of the water and consequently also in a change of the size of the hydraulic radius, so that the \( C \)-value is influenced.
- The formula applies if \( T < t_1 \), so if

\[ \frac{T}{t_1} = \frac{4\pi L(Q_m - Q_b)^2}{3i_b^2 p^2 C^2} < 1 \]  

(A2.28)

If the flood-peak attenuation can be neglected the condition (A2.28) will mostly be complied with, too.
In the derivation it has been assumed that the complete major bed contributes to the flow. In the case of a great flood plain, the equation should be used with caution.

The result is applied to the Ourthe between Comblain-au-Pont and Angleur. It is the stretch where the Ourthe and Amblève have united, but where the Vesdre has not joined yet. The stretch has been chosen because of the importance of the tributaries and the great length of the stretch (approximately 30 km). The mouth of the Ourthe in the Meuse is still 2 km downstream from the confluence of the Ourthe and the Vesdre.

For reasons of simplicity most parameters are supposed to be constant. Unfortunately not for all the parameters exact values are available, but the following system produces a good approximation of the reality:

- length of the stretch of the river: \( L = 30 \text{ km} \)
- friction coefficient: \( C = 40 \sqrt{\text{m/s}} \)
- bottom gradient: \( i_b = 10^{-3} \)

The values of \( P, Q_m, \) and \( Q_b \) have been chosen in such a way that they correspond well to the flood waves of July 1980 and February 1984. The values are:

- volume flood wave: \( P = 10^8 \text{ m}^3 \)
- peak discharge at Comblain-au-Pont: \( Q_m = 500 \text{ m}^3/\text{s} \)
- baseflow: \( Q_b = 100 \text{ m}^3/\text{s} \)

The relative flood-peak attenuation amounts to \( 5.02 \times 10^{-4} \) (with (A2.27)), which means that the peak discharge between Comblain-au-Pont and Angleur decreases by 0.25 m³/s only, which will be less than the attenuation caused by the small flood plains. Those numbers are very small, so that it may be argued that the attenuation is to be neglected.

Condition (A2.28) has been satisfied: the left hand member has the value of \( 1.25 \times 10^{-3} \).

A more detailed investigation has been made, in which the width \( B \) has been assumed to be equal to 30 m. Then with equations (A2.19) to (A2.22) inclusive it follows:

- \( u = 3.0 \text{ m/s} \)
- \( K = 8.3 \times 10^4 \text{ m}^2/\text{s} \)
- \( t_1 = 5.3 \times 10^4 \text{ s (≈ 62 days approximately)} \)
- \( T = 6.7 \times 10^3 \text{ s (≈ 1.9 hours approximately)} \)

It is true that \( T < t_1 \).

The shape of the wave can be calculated with equation (A2.15). Then it appears that at \( t_1 \) the wave calculated is still strongly symmetrical, which for this analysis is not problematic. 24 hours before \( t_1 \) the discharge was 272.9 m³/s, 24 hours after \( t_1 \) it was 273.3 m³/s.

The value for the relative flood-peak attenuation is so small that it may be concluded that for flood waves in the Ourthe between Comblain-au-Pont and Angleur the flood-peak attenuation can be neglected.

With (A2.27) the relative flood-peak attenuation can also be calculated for other tributaries and other waves. For a more general analysis (A2.27) is split into three terms:
Figure A2.1 The graphical interpretation of \( \tau \) (code B1.1)

\[
\begin{align*}
\text{r.f.a.} &= \frac{Q_m - Q_b}{Q_m} \quad \frac{(Q_m - Q_b)^2}{P^2} \quad \frac{2\pi L}{3i_0^2 C^2} \\
\text{I} & \quad \text{II} & \quad \text{III}
\end{align*}
\]

The term I is always smaller than 1.

The term II indicates the shape of the wave. In order to be able to interpret the parameter better, the parameter \( \tau \) is defined according to:

\[
\tau = 2 \frac{P}{Q_m - Q_b} \tag{A2.30}
\]

\( \tau \) is the time basis which a flood wave (with a maximum discharge \( Q_m \), a baseflow \( Q_b \) and a discharge volume \( P \)) would take if the hydrograph of the wave had the shape of a isosceles triangle with the basis at \( Q_b \) and the apical angle at a height of \( Q_m \) (fig. A2.1).

For several flood waves in various tributaries the value of \( \tau \) has been determined. It appears that \( \tau \) is in the same order of magnitude as the period of time of the observed flood wave.

If the wave becomes more pointed \( \tau \) becomes smaller, consequently the r.f.a. becomes greater. \( \tau \) will be smallest for showers of a short duration. However, with a shower of a short duration the term I will be small because the volume of the additional discharge is limited. The product of the terms I and II will be greatest with short and heavy showers.

The size of the term III is fixed for a definite river with a definite measuring station. For the Meuse tributaries the values of \( L \) and \( C \) are always of the same order of magnitude.
However, the value of $i_s$, the bottom gradient, may vary greatly. As the term occurs as a quadrature the bottom gradient is of great importance for the magnitude of the term III.

First the tributaries with a gradient greater than $10^4$ will come up. Here the Semois, Virion, Lesse and Ourthe as well as the small tributaries Houille, Hermeton, Moliène, Bocq and Meaigne are concerned. Flood waves have been looked for with a value of $\tau$ as small as possible. The values that have been found are greater than $6 \times 10^4$ s (= 16 hours approximately). However, the smallest values of $\tau$ corresponds to discharges that are so small that a flood in the Meuse is very improbable. With greater discharges (when Borgharen has a discharge of more than 1,000 m$^3$/s) the values of $\tau$ are greater than $8 \times 10^4$ s (= approximately 22 hours). The lengths of the stretches are always 10 km or less (with the exception of the stretch between Comblain-au-Pont and Angleur). It is reasonable to suppose that the peak discharge is three times as great as the baseflow and that $C$ is equal to $40 \sqrt{m/s}$. Substituting these values into (A2.29) produces as the upper limit for the relative flood-peak attenuation:

$$r.f.a. = \frac{Q_m - Q_s}{Q_m} \frac{(Q_m - Q_s)^2}{P^2} \frac{2\pi L}{3i_s^2 C^2} = \frac{2}{3} \frac{2^2}{(8 \times 10^4)^2} \frac{2\pi 10^4}{3 \times 10^{-3} \times 40^2} = 5.45 \times 10^{-3} = 0.5\%$$

This upper limit is so small that it may be concluded that the flood-peak attenuation in the tributaries is to be neglected. It appears that the condition (A2.28) is also complied with.

From that argumentation the Chiers, the Sambre, the Roer and the Niers are excepted.

In the Chiers the gradient in the stretch below Carignan amounts to approximately $5 \times 10^4$ and moreover, the length of the stretch is considerable. Data from the Carignan station are not used for the flood forecasting model, considering the great wave travel time up to Borgharen, so that the problem of flood-peak attenuation there is not important for the flood forecasting model.

The Sambre has been excluded because the flow is strongly influenced by the weirs and the locks.

For the Roer and the Niers the gradient is $7 \times 10^4$ and $2 \times 10^4$ respectively. As there are also small gradients over a large stretch further upstream, the waves are likely to have a flatter shape than those in the Ardennes tributaries. Therefore it is acceptable not to include the flood-peak attenuation between the measuring station and the Meuse in the calculation.

It may be concluded that for none of the tributaries of the Meuse it is necessary to calculate explicitly the flood-peak attenuation between the downstream measuring station and the Meuse for developing the flood forecasting model.

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Appendix 3. The potential loss intensity as a function of previous values of the equilibrium potential loss intensity

For discrete time steps, it can be proven that:

\[ f_{pot}(t_i) = \sum_{i=1}^{\infty} \alpha_i f_{\infty}(t_{p, i}) \]  \hspace{1cm} (A3.1)

where:
- \( f_{pot} \) = potential loss intensity \hspace{1cm} \text{[LT}^{-1}\text{]}
- \( f_{\infty} \) = equilibrium potential loss intensity \hspace{1cm} \text{[LT}^{-1}\text{]}
- \( \alpha_i \) = a constant, dependent on the time discretisation and the values of \( k_A \) and \( k_B \) \hspace{1cm} [-]
- \( t_{p, i} \) = moment at which a new value of the precipitation intensity has become valid \hspace{1cm} \text{[T]}

Furthermore:

\[ \sum_{i=1}^{\infty} \alpha_i = 1 \]  \hspace{1cm} (A3.2)

Equations (A3.1) and (A3.2) refer to (10.89) and (10.90) in the main text. It should be emphasised that the series \( t_{p, i} \) need not be equidistant. For reasons of brevity, the subscript \( p \) in \( t_{p, i} \) will be omitted in the rest of the appendix.

**Proof**

Let:

\[ \Delta t_i = t_i - t_{i-1} \]  \hspace{1cm} (A3.3)

for all \( i \).

Because of the continuity of the potential loss intensity:

\[ f_{pot}(t_i) = \lim_{s \rightarrow \Delta t_i} f_{pot}(t_{i-1} + s) \]  \hspace{1cm} (A3.4)

Substituting and using the fact that \( f_{\infty} \) only changes at the moments at which a new value of the precipitation intensity has become valid:

\[ f_{pot}(t_i) = f_{\infty}(t_{i-1}) + (f_{pot}(t_{i-1}) - f_{\infty}(t_{i-1})) \exp \left( -\frac{\Delta t_i}{k_{i-1}} \right) \]  \hspace{1cm} (A3.5)

where:
\[ k_{j-1} = k_A \quad \text{if } f_{w}(t_{j-1}) > f_{pot}(t_{j-1}) \]
\[ k_{j-1} = k_B \quad \text{if } f_{w}(t_{j-1}) < f_{pot}(t_{j-1}) \]  

(A.3.6)

Using the same kind of equation for \( t_{j-1} \) and \( t_{j-2} \) gives after rewriting:

\[
f_{pot}(t_j) = f_{w}(t_{j-1}) \left(1 - \exp\left(-\frac{\Delta t_j}{k_{j-1}}\right)\right) + f_{w}(t_{j-2}) \left(1 - \exp\left(-\frac{\Delta t_{j-2}}{k_{j-2}}\right)\right) +
\]
\[
\left[f_{w}(t_{j-3}) \left(1 - \exp\left(-\frac{\Delta t_{j-3}}{k_{j-3}}\right)\right) + f_{pot}(t_{j-3}) \exp\left(-\frac{\Delta t_{j-3}}{k_{j-3}}\right) \right] \exp\left(-\frac{\Delta t_j}{k_{j-1}}\right)\]

(A.3.7)

In the expression three time steps are involved. If \( n+1 \) time steps are involved:

\[
f_{pot}(t_j) = f_{w}(t_{j-1}) \left(1 - \exp\left(-\frac{\Delta t_j}{k_{j-1}}\right)\right) + \sum_{i=1}^{n} f_{w}(t_{j-i}) \left[1 - \exp\left(-\frac{\Delta t_{j-i}}{k_{j-i}}\right) \prod_{m=0}^{i-1} \exp\left(-\frac{\Delta t_{j-m}}{k_{j-m}}\right)\right]
\]

(A.3.8)

\[
+ f_{pot}(t_{j-n}) \prod_{i=0}^{n} \exp\left(-\frac{\Delta t_{j-i}}{k_{j-i}}\right)
\]

As all the exponential functions in the equation above have a value between 0 and 1 the influence of a value of \( f_w \) will decrease with time. The influence of a certain equilibrium potential loss intensity on the current potential loss intensity is greater if the time step over which the equilibrium potential loss intensity lasted is longer.

If the length of the involved time is the limit to infinity, so:

\[
\sum_{i=0}^{n} \Delta t_{j-i} \rightarrow \infty
\]

(A.3.9)

the last term of equation (A.3.8) is equal to zero. \( f_{pot}(t_j) \) is then a linear combination of \( f_{w}(t_i) \), \( i = j-1, j-2, \ldots \):

\[
f_{pot}(t_j) = \sum_{i=0}^{n} \alpha_i f_{w}(t_{j-i})
\]

(A.3.10)

where:

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\[ \alpha_i = \left\{ \begin{array}{ll}
1 - \exp\left(-\frac{\Delta t_{j-1}}{k_{j-1}}\right) & \text{for } i = 0 \\
1 - \exp\left(-\frac{\Delta t_{j-i}}{k_{j-i+1}}\right) \prod_{m=0}^{i-1} \exp\left(-\frac{\Delta t_{j-m}}{k_{j-m+1}}\right) & \text{for } i \geq 1
\end{array} \right. \] (A3.11)

That finishes the first part of the proof.

The second part is proven by writing out the first two terms of (A3.8). Many terms will neutralise one another, which results in a simple expression:

\[
\sum_{i=0}^{n} \alpha_i = \left(1 - \exp\left(-\frac{\Delta t_{j}}{k_{j-1}}\right)\right) + \sum_{i=1}^{n} \left[1 - \exp\left(-\frac{\Delta t_{j-i}}{k_{j-i+1}}\right) \prod_{m=0}^{i-1} \exp\left(-\frac{\Delta t_{j-m}}{k_{j-m+1}}\right)\right]
\]

\[
= 1 - \exp\left(-\frac{\Delta t_{j}}{k_{j-1}}\right) + \exp\left(-\frac{\Delta t_{j}}{k_{j-1}} - \frac{\Delta t_{j-1}}{k_{j-2}}\right) + \exp\left(-\frac{\Delta t_{j}}{k_{j-1}} - \frac{\Delta t_{j-1}}{k_{j-2}} - \frac{\Delta t_{j-2}}{k_{j-3}}\right)
\]

\[\text{+ ...,} \] (A3.12)

\[= 1 - \exp\left(-\frac{\Delta t_{j}}{k_{j-1}} - \frac{\Delta t_{j-1}}{k_{j-2}} - \frac{\Delta t_{j-2}}{k_{j-3}} - \cdots \frac{\Delta t_{j-n+1}}{k_{j-n}}\right)\]

Because of (A3.9) and because all \( k \) are positive, expression (A3.12) reduces to 1, so that the second part has been proven.
References


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List of symbols

If a symbol contains more indexes than given below, the meaning of the symbol can be found by combining the meanings of the indexes given under the main symbol.

\[ A \quad = \quad \text{catchment area/subcatchment area (chapter 10, 13, appendix 1)} \quad [L^2] \]
\[ A_{\text{region}} \quad = \quad \text{area of the named region} \]
\[ A_j \quad = \quad \text{area in the subcatchment for which the nearest station is } j \]
\[ A_u \quad = \quad \text{ungauged area which is assigned to the considered gauged area} \]
\[ A \quad = \quad \text{cross-sectional area (chapter 11, appendix 2)} \quad [L^2] \]
\[ A_f \quad = \quad \text{conveying cross-sectional area} \]
\[ A_i \quad = \quad \text{cross-sectional area in place } j \Delta x \text{ at time } i \Delta t \]
\[ A_f^\prime \quad = \quad \text{conveying cross-sectional area of the flow section} \]
\[ A_f^{\prime 0} \quad = \quad \text{conveying cross-sectional flow area at a level where } B_s(h') \text{ is equal to the mean storage width of the river} \]
\[ AF \quad = \quad \text{assessment factor} \quad [-] \]
\[ B \quad = \quad \text{river width} \quad [L] \]
\[ B_s \quad = \quad \text{storage width} \]
\[ B_j \quad = \quad \text{width in place } j \Delta x \]
\[ B \quad = \quad \text{mean width} \]
\[ B(h_s) \quad = \quad \text{width of the river as a function of water level} \]
\[ B_s \quad = \quad \text{storage width of the storage section} \]
\[ C \quad = \quad \text{Chézy friction coefficient} \quad [L^{1/2} T^{-1}] \]
\[ \tilde{C}_f \quad = \quad \text{Chézy coefficient of the flow section} \]
\[ D \quad = \quad \text{diagonal matrix containing the eigen values} \]
\[ D_i \quad = \quad \text{diameter of the considered drop} \quad [L] \]
\[ E \quad = \quad \text{the length of the river section considered} \quad [L] \]
\[ F \quad = \quad \text{depth of infiltrated water} \quad [L] \]
\[ F(t) \quad = \quad \text{total depth of infiltrated water at time } t \]
\[ F_i(t') \quad = \quad \text{loss depth if the precipitation intensity is greater than the calculated } f \text{ using equation (10.64)} \]
\[ G \quad = \quad \text{antenna gain (chapter 10)} \quad [-] \]
\[ G_i \quad = \quad \text{discharge correction made at } i \Delta t \text{(chapter 11)} \quad [L^3 T^{-1}] \]
\[ H \quad = \quad \text{pulse length (in the air) (chapter 10)} \quad [L] \]
\[ H \quad = \quad \text{water level with respect to the horizontal impermeable basis (chapter 11)} \quad [L] \]
\[ H_i^j \quad = \quad \text{water level in place } j \Delta x \text{ and at time } i \Delta t \]
\[ IUH \quad = \quad \text{instantaneous unit hydrograph} \quad [T^{-1}] \]
\( K \) = parameter in the calculation of the wave propagation \([\text{L}^2\text{T}^{-1}]\)

\( |K|^2 \) = constant in the radar equation \([-\text{]}\)

\( K_{\text{area}} \) = runoff parameter of the named area \([\text{L}^2]\)

\( K'_{\text{area}} \) = calculated runoff parameter of the named area

\( L \) = length \([\text{L}]\)

\( L_{\text{hr}} \) = length of the headwater reach

\( L_f \) = length of the flow section

\( M_y \) = mean value of \( y \) \([\text{L}^{\text{My}}\text{T}^{-\text{My}}]\)

\( N \) = number of precipitation stations (chapter 10) \([-\text{]}\)

\( N' \) = number of precipitation stations in the subcatchment area considered

\( N'' \) = number of nearby gauges that should be considered in the equation

\( N \) = number of sub-areas in the catchment (chapter 13) \([-\text{]}\)

\( N(d) \) = number of paired points with a mutual distance \( d \) \([-\text{]}\)

\( P \) = precipitation depth (chapter 10); rainfall depth (chapter 13) \([\text{L}]\)

\( \hat{P} \) = estimated precipitation depth

\( \hat{P} \) = estimated areal precipitation depth

\( \hat{P}_{A_j}(t) \) = areal rainfall depth for area \( j \) on day \( t \)

\( \hat{P}_{M}(t) \) = areal rainfall depth for Meuse catchment on day \( t \)

\( P_j \) = precipitation depth in place \( j \)

\( \hat{P}_{\text{month} A_j} \) = monthly areal rainfall depth for area \( A_j \)

\( \hat{P}_0 \) = unit effective precipitation depth, uniformly distributed over the subcatchment area

\( P(i,t) \) = rainfall depth in gauge \( i \) on day \( t \)

\( P \) = volume of a flood wave (appendix 2) \([\text{L}^3]\)

\( \overline{P}_r \) = received power \([\text{ML}^2\text{T}^{-3}]\)

\( P_r \) = emitted power \([\text{ML}^2\text{T}^{-3}]\)

\( Q \) = discharge, runoff \([\text{L}^3\text{T}^{-1}]\)

\( \dot{Q} \) = discharge forecast

\( Q^* \) = calculated discharge

\( Q_e \) = infiltration flow according to Edelman

\( Q_{vc} \) = discharge originating from the area between Verviers and Chaudfontaine

\( Q^*_Z \) = discharge calculated by ZWENDL

\( Q_b \) = baseflow

\( Q_{bA_i} \) = baseflow of area \( A_i \)

\( Q_{bu} \) = baseflow of the subcatchment including the surrounding
\( Q_c \) = constant discharge
\( Q_d \) = direct runoff
\( Q_f \) = flow in the flow section
\( Q_i \) = discharge at time \( i \Delta t \)
\( Q_{i,h'} \) = discharge forecast made at \( i \Delta t \) with a lead time of \( h' \Delta t \)
\( Q_y \) = discharge as presented in fig. 13.7
\( Q_m \) = maximum discharge at \( t_1 \)
\( Q_n \) = natural discharge
\( Q_{model,i} \) = discharge forecast according to model \( i \)
\( Q_{r;i} \) = discharge in period \( r \) at time serial number \( i \)
\( Q_s \) = flow into the storage section
\( Q_r^* \) = calculated discharge which is one of the inputs of the module for the Belgian Meuse
\( Q_0 \) = fixed value of the discharge
\( Q_{BhD} \) = discharge forecast for Borgharen-Dorp, using the model described in the chapters 9 to 12
\( Q^j \) = discharge in place \( j \) respectively \( j \Delta x \)
\( Q_{river} \) = discharge forecast at the mouth of the river mentioned
\( Q(t) \) = discharge at \( t \)
\( Q(t,h) \) = discharge forecast made at time \( t \) with lead time \( h \)

\( R \) = hydraulic radius (chapter 11) \([L]\)
\( \bar{R}_f \) = hydraulic radius of the flow section

\( R^2 \) = coefficient of determination (chapter 13) \([-\]

\( S \) = storage of water in baseflow reservoirs \([L^3]\)
\( S_{A_i} \) = storage of water in baseflow reservoirs in area \( A_i \)
\( S_{ij} \) = storage of water in baseflow reservoirs as presented in fig. 13.7
\( S_{ij,r} \) = \( S_{ij} \) for period \( r \)

\( T \) = wave travel time \([T]\)
\( T_m \) = wave travel time between the measuring station and the mouth of the tributary \([T]\)
\( T_{ref} \) = time at which both a. the flow and b. the volume of water which has flown from the river into the bank are calculated correctly in comparison with the Edelman model for an instantaneous level rise \([T]\)

\( \Delta H_{ref} \)

\( V \) = water volume (per unit of width) in a headwater reach \([L^2]\)
\( \bar{W}_s \) = storage area of the storage section \([L^2]\)
\( \bar{W}_{s0} \) = storage area at a level where \( B_s(\bar{h}_s) \) is equal to the mean storage width of the river

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\( X \) = radar reflection \([L^2]\)

\( Y_i \) = difference between the measured discharge and the calculated direct runoff at \( i \Delta t \) \([L^3T^{-1}]\)

\( Z \) = intrinsic random function

\( \hat{Z} \) = estimated intrinsic random function

\( a \) = discharge withdrawn from the natural course and not restored to the Vesdre above Verviers (chapter 6) \([L^3T^{-1}]\)

\( a \) = water depth (chapter 11, appendix 2) \([L]\)

\( a_b \) = water depth belonging to the baseflow

\( a_n \) = normal depth

\( a_R \) = constant in the calculation of the precipitation intensity when radar is used

\( a_s \) = constant in the Snyder decay constant function \([T^{-1}]\)

\( a_i \) = constant in the calculation of the effective precipitation intensity

\( b \) = ratio of the natural discharge at Verviers and that at Chaudfontaine \([-]\)

\( b_R \) = constant in the calculation of the precipitation intensity when radar is used \([-]\)

\( b_s \) = constant in the Snyder decay constant function \([L^{-1}]\)

\( b_j \) = coefficient in the Thiessen polygons method \([-]\)

\( \bar{b}_i \) = \[
\begin{pmatrix}
  e_i \\
  0
\end{pmatrix}
\]

\( c \) = ratio of the discharge that is returned between Verviers and Chaudfontaine and the total discharge withdrawn (chapter 6) \([-]\)

\( c \) = celerity (chapter 11, appendix 2) \([LT^{-1}]\)

\( \bar{c} \) = mean celerity in the headwater reach

\( d \) = distance \([L]\)

\( d_{ji} \) = distance between place \( j \) and place \( i \)

\( d_{ro} \) = distance radar-object

\( e \) = residual

\( e_b \) = residual: measured water level minus calculated water level \([L]\)

\( e_i \) = residual: measured discharge minus calculated discharge at \( i \Delta t \) \([L^3T^{-1}]\)

\( e_{i,j}^{(t,h)} \) = residual: difference at station \( j \) at time \( t+h \) between measured discharge and the discharge forecast according to model \( i \) made at \( t \) with a lead time of \( h \) \([L^3T^{-1}]\)

\( e_t \) = residual: measured peak moment minus calculated peak moment \([T]\)

\( f \) = loss intensity (chapter 10) \([LT^{-1}]\)

\( \bar{f} \) = areal mean loss intensity

\( f_d \) = potential loss intensity at \( t_d \)
\( f_d \) = loss intensity at \( t_d \) \\
\( f_m \) = maximum point loss intensity \\
\( f_{\text{max}} \) = maximum potential loss intensity \\
\( f_{\text{min}} \) = minimum loss intensity \\
\( f_{\text{min}}' \) = minimum potential loss intensity \\
\( f_{\text{pot}} \) = potential loss intensity \\
\( f_{\text{pot}} x_A \) = potential loss intensity for unit area \( x_A \) \\
\( f_m \) = equilibrium potential loss intensity \\
\( f(t) \) = loss intensity at time \( t \) \\
\( f \) = an invertible function (chapter 13) [L^3T^{-1}] \\
\( f_{A_i} \) = an invertible function for area \( A_i \) \\
\( g \) = gravity acceleration (chapter 11) [LT^{-2}] \\
\( g \) = function to determine the effective precipitation (chapter 13) [L^2T^{-1}] \\
\( g_{ij} \) = certain function in Trend Surface Analysis [L] \\
\( h \) = lead time (chapter 10) [T] \\
\( h \) = water level (with respect to a horizontal reference level) (chapter 11, appendix 2) [L] \\
\( h_r \) = water level in the river before the rise \\
\( h_f \) = water level in the flow section \\
\( h_f' \) = water level in the flow section that determines the storage width \( B_s \) \\
\( h_i' \) = water level in place \( j \Delta x \) and at time \( i \Delta t \) \\
\( h_r \) = water level in the river \\
\( h_s \) = water level in the storage section \\
\( h' \) = parameter for the lead time (when multiplied by \( \Delta t \) equal to the lead time \( h \)) [-] \\
\( i \) = general serial number [-] \\
\( i_c \) = time serial number corresponding to the first moment for which a calculation is made \\
\( i_f \) = serial number corresponding to the first moment for which a discharge is known \\
\( i_m \) = number of functions \( g \) to be incorporated in the equation \\
\( i_{\text{max}} \) = number of time steps to be considered in the equation \\
\( i_s(r;i) \) = first moment for which the discharge should be incorporated in the calculation of the baseflow for period \( r \) at time serial number \( i \) \\
\( i_0 \) = time serial number corresponding to the moment of observation [-] \\
\( i_b \) = bottom gradient [-] \\
\( j \) = general serial number [-]
\[ k = \text{decay constant} \]  
\[ k_A = \text{decay constant for an increasing potential loss intensity} \]  
\[ k_B = \text{decay constant for a decreasing potential loss intensity} \]  
\[ k_N = \text{decay constant in the Nash cascade} \]  
\[ k_{t_i} = \text{decay constant at } t_i \]  
\[ k_S = \text{decay constant for the Snyder function} \]  
\[ k_i = \text{decay constant in depletion curve model 1} \]  
\[ KD = \text{transmissivity of the sub-soil} \]  
\[ k_M = \text{constant in the calculation of the friction coefficient} \]  
\[ k_2 = \text{constant in depletion curve model 2} \]  
\[ k_3 = \text{constant in depletion curve model 3} \]  
\[ m = \text{positive integer constant} \]  
\[ m(r) = \text{number of observed discharges included in the calibration for period } r \]  
\[ n = \text{positive integer constant} \]  
\[ n_M = \text{constant in the calculation of the friction coefficient} \]  
\[ n_N = \text{constant in Nash cascade} \]  
\[ n_i = \text{number of drops with a diameter } D_i \text{ per unit of content} \]  
\[ n_3 = \text{constant in depletion curve model 3} \]  
\[ p = \text{precipitation intensity (chapter 10, appendix 2); rainfall intensity (chapter 13)} \]  
\[ \bar{p} = \text{areal precipitation/rainfall intensity} \]  
\[ p_{\text{eff}} = \text{effective precipitation intensity} \]  
\[ p_{\text{ref}} = \text{reference value of the precipitation intensity (constant)} \]  
\[ p(t) = \text{precipitation intensity at time } t \]  
\[ \bar{p}(t_i) = \text{areal precipitation intensity between } t_{i-1} \text{ and } t_i \]  
\[ p_i = \text{constant in (10.18)} \]  
\[ q = \text{flow per unit of width} \]  
\[ q_j = \text{flow per unit of width in place } j \Delta x \text{ and at time } i \Delta t \]  
\[ q_b = \text{baseflow per unit of width} \]  
\[ q_d = \text{specific direct runoff (chapter 10)} \]  
\[ r = \text{serial number of the considered event} \]  
\[ r_{\text{max}} = \text{number of events included in the calibration} \]  
\[ r.f.a. = \text{relative flood-peak attenuation} \]  
\[ s^2 = \text{variance} \]  
\[ s_{\text{crit}}^{2\text{BD}} = \text{critical value of the mean of the squares of the residuals of the Borgharen-Dorp forecast (page 214)} \]  
\[ s_{m,n}^2 = \text{variance of the forecast error if } Q_{i-m} \text{ is forecast using } Q_i \text{ and } Q_{i-m} \]
\( s_{model}^2 \) = simplified notation for \( s^2(\epsilon_{model i}(t,h)) \) 
\( S_x^2 \) = variance of \( x \) 
\( S_0^2 \) = mean square error 

\( t \) = time \[T\] 
\( t' \) = time at which the same loss depth is obtained as at \( t \) if the precipitation intensity is greater than the calculated \( f \) using equation (10.64) 
\( t_a \) = moment of river water level rise 
\( t_b' \) = day on which the baseflow in subcatchment \( j \) is known 
\( t_d \) = time at which the precipitation begins 
\( t_e \) = last day for which baseflow forecasts are requested for 
\( t_f \) = last day for which rainfall forecasts are available 
\( t_i \) = time for serial number \( i \) 
\( t_{p,i} \) = moment at which a new value of the precipitation intensity has become valid 
\( t_{r,i} \) = time for serial number \( i \) in period \( r \) 
\( t_s \) = day on which the forecast is made 
\( t_0, t_1, t_2, t_3 \) = certain fixed moments (cf. text) 
\( t_{01} \) = \( t_0 \) for the first baseflow hydrograph 
\( t_{02} \) = \( t_0 \) for the second baseflow hydrograph 
\( t_{0,r} \) = \( t_0 \) for period \( r \)

\( u \) = current velocity \[LT^{-1}\] 
\( v \) = constant in depletion curve model 3 \[T\] 
\( v_i \) = eigen vector number \( i \) 
\( w \) = constant in depletion curve model 3 [-] 

\( w_i \) = \[
\begin{pmatrix}
Q_i \\
Q_{i-1}
\end{pmatrix}
\]

\( x \) = distance \[L\] 
\( x_A \) = unit area in the catchment area in the Stanford Watershed Model \[L^2\] 
\( x_j \) = Cartesian \( x \)-ordinate of point \( j \) \[L\] 
\( x_i \) = point in the area 
\( x_0 \) = unit area where the precipitation intensity is equal to the threshold value \( f_m(t) \) \[L^2\] 
\( \bar{x} \) = distance along the axis of the groundwater flow sections \[L\] 
\( y_j \) = Cartesian \( y \)-ordinate of point \( j \) [-] 
\( y_{r,i} \) = square root of \( Q_{r,i} \) \[L^{1/2}T^{-1/2}\] 
\( y_{r,i}^* \) = square root of \( Q_{r,i}^* \) 
\( z \) = realisation of the random intrinsic function \( Z \) (chapter 10)
\( \zeta \) = best linear unbiased predictor of \( z \)
\( z(x_i) \) = value of \( z \) in place \( x_i \)
\( z(x_{i,t}) \) = value of \( z \) in place \( x_i \) at time \( t \)

\( z \) = bottom level (appendix 2) [L]
\( \Gamma(\cdot) \) = gamma function [-]
\( \Delta H \) = water level rise of the river [L]
\( \Delta H_{\text{ref}} \) = rise that is representative of level rises in the considered river section

\( \Delta t \) = time step [T]
\( \Delta t_i \) = \( t_i - t_{i-1} \)
\( \Delta x \) = distance between two adjacent nodes [L]
\( \Theta_1 \) = horizontal beam width [-]
\( \Theta_2 \) = vertical beam width [-]

\( \Lambda = \begin{pmatrix} 1 + \alpha & -\alpha \\ 1 & 0 \end{pmatrix} \) [-]
\( \Xi \) = basis of eigen vectors (matrix) [L^5T^2]
\( \Phi \) = the value to be minimised
\( \Omega \) = reduction parameter (constant) [-]
\( \alpha \) = \( \alpha_{1,i} \) [-]
\( \alpha_i \) = a constant, dependent on the time discretisation and the values of \( k_A \) and \( k_B \) before \( t_n \) [-]
\( \alpha_{m,n} \) = coefficient in the equation, if \( Q_{i,n} \) is forecast using \( Q_i \) and \( Q_{i-m} \) [-]
\( \hat{\alpha}(t) \) = correction coefficient in eq. (10.53)
\( \beta \) = coefficient; may contain various indexes [-]
\( \beta^* \) = constant, may contain superscripts and superscripts [L^3T^1]
\( \gamma(d) \) = semi-variogram
\( \gamma_i(d) \) = semi-variogram at time \( t \)
\( \gamma_i^*(d) \) = modified form of the semi-variogram at time \( t \)

\( \varepsilon \) = residual [L^3T^1]
\( \zeta \) = factor in the calculation of the discharge of ungauged areas [-]
\( \eta \) = ratio between the area that drains via the reservoir and the total area of the catchment [-]
\( \theta \) = implicit factor [-]
\( \kappa \) = constant in a proposed function of the semi-variogram (eq. (10.53)) [L^1]
\( \lambda \) = wave length [L]
\( \lambda_i \) = eigen value number \( i \) [-]
\( \mu \) = Lagrange multiplier (chapter 10)
\( \mu \) = storage coefficient (chapter 11)
\( \nu \) = coefficient denoting a shift in time; may contain various indexes (chapter 10) [-]
\( \nu \) = rate of the river water level rise (chapter 11) [LT^1]
\( \rho \) = correlation coefficient [-]
\( \sigma_{m,n}^2 \) = variance of the forecast error if \( Q_{i,n} \) is forecast using \( Q_i \) and \( Q_{i-m} \) [LT^2]
\( \tau \) = time which a flood wave with a volume \( P \) would take if the hydrograph had the shape of an isosceles triangle with the basis at \( Q_b \) and the maximum at \( Q_m \) (appendix 2) [-]
\( v \) = constant in depletion curve model 2 [-]
\( \phi \) = constant in the \( \phi \)-index model [LT^{-1}]
\( \psi \) = factor in the determination of the effective precipitation [-]
\( \psi_1 \) = first part of the factor that determines the effective precipitation
\( \psi_2 \) = second part of the factor that determines the effective precipitation
\( \psi_{i,r} \) = \( \psi_i \) for calibration event \( r \)
\( \omega \) = positive constant in the Reciprocal Distance Method [-]

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Summary

The river Meuse has a lot of functions for man with respect to shipping, water supply, supply of cooling water, etcetera. At the same time the Meuse may be harmful, in case of floods, droughts, ice drift, calamities through disposals, etcetera. In many cases a flow forecast is desirable, because then the necessary measures can be taken in time. In this thesis the development of two important types of flow forecasting models are described: flood forecasting and low flow forecasting.

The development of flow forecasting models for the Meuse is relatively complicated, because:

a. the amount of available data is limited;
b. the catchment is heterogeneous with respect to the importance of aspects of the hydrologic cycle, but also with respect to the equipment, the discharge determination, and the available data, inclusive of the time-series data;
c. the discharge is heavily influenced by man; and
d. the time between rainfall and runoff is relatively small for the greater part of the catchment.

Due to the international character of the catchment, insufficient knowledge of the hydrologic processes was available at the beginning of this study. Therefore a systems analysis was performed before starting model development.

The Meuse has a length of 874 km from the source in France to the mouth in the Hollands Diep, the catchment area amounts to 33,000 km². From the hydrologic point of view the Meuse can be subdivided into three parts.

The upper course, called Meuse Lorraine, flows from the source in the utmost south of the catchment to the mouth of the Chiers. The number of tributaries is low, the river bottom slope is relatively small for an upper course and the valley, which is inundated during floods, is wide. Those properties cause the Meuse Lorraine slowly to react to precipitation. Its peak discharge during floods is relatively small for a catchment of this size. In periods with small amounts of rainfall the discharge is relatively high, due to a considerable volume of water stored in the catchment.

The middle course, called Meuse Ardennaise, flows from the mouth of the Chiers to the Belgian-Dutch border. The river slope is relatively steep for a middle course. The number of tributaries is high, and they are situated in hilly areas with a low infiltration capacity. It causes a fast runoff of the water, which results in high flood peak discharges and low flows during rainless periods. This part of the river differs from its upper and lower course in having weirs functioning during floods, and by the absence of large flood plains.

The lower course flows from the Belgian-Dutch border to the river mouth. In a downstream direction the slope of the river decreases to low values. The number of important tributaries is small. Downstream from Boxmeer dikes are present, downstream from Lith there are tidal influences. The most downstream part used to be most sensitive to floodings, because of ice drift and the connections with the river Waal, a branch of the Rhine.

Nowadays due to a temperature rise of the water due to cooling by power stations and river improvement projects, the most sensitive area in the Netherlands has changed to the south, where there are no dikes to protect villages from the water.

Man has influenced the flow regime of the river for centuries. A continuous change of the runoff characteristics is caused by changes in the use of land; the application of irrigation,
sprinkler-irrigation and drainage; the construction of canals, weirs, hydro-electric power
stations and reservoirs; modifications in the course and the cross-sections; getting gravel and
sand; extractions and disposals. Changes in the flow regime seem to have quickened in the
course of time. The most important aspects with respect to flow forecasting are the changes in
the cross-sections and the weir operation, as well as changes in extractions and disposals.

The systems analysis showed that, in respect of flood forecasting, the most important
hydrologic processes vary from area to area. Therefore it has been decided to develop a
specific type of model, called module, for each hydrologic unit. When the subdivision into
modules was made, also the kind, quality and quantity of data were taken into consideration.
The result has been a subdivision after the kind of process (rainfall-runoff relation versus
wave propagation in the river) and after the country.

In the choice of the type of the modules the availability of data in real-time has played a
role from the start.

The most important tributaries, with respect to flood forecasting in the Netherlands, are
situated in Belgium. Consequently, the module for the Belgian tributaries has gained a great
deal of attention. The module consists of six rainfall-runoff models, which have been
developed for the six most important tributaries. The cores of the models are direct runoff
models, based on precipitation and precipitation forecasts only. In order to obtain the required
total discharges, at least one discharge measurement or an estimation of it is necessary. The
direct runoff model consists of four parts.

a. The areal precipitation values are determined from point measurements. After evaluating
   several methods, the ordinary Thiessen polygons method showed to be the optimum
   method.

b. From the areal precipitations the effective precipitations are determined. The effective
   precipitation is that part of the precipitation that contributes to the direct runoff. In this
   part of the direct runoff model one state variable is used, being the potential loss
   intensity. The course of the potential loss intensity in time is dependent on the
   equilibrium potential loss intensity. The equilibrium potential loss intensity is only a
   function of the precipitation intensity at the moment considered. The effective
   precipitation is a function of the precipitation and the potential loss intensity.

c. The series of effective precipitations is transformed with the use of a Nash-cascade into
   a series of direct runoffs.

d. The discharge forecasts of the Belgian tributaries are compiled from the direct runoffs
   calculated and the discharges measured. This part of the model has deliberately been
   kept simple, thus allowing also for forecasts when measured discharges are partly or
   completely absent.

Optimisation of the model parameters has been performed by means of regression analysis.
The results appear to be accurate with respect to the ultimate aim: a discharge forecast for the
station near the Dutch border (station of Borgharen-Dorp).

The greater tributaries debouching into the French and the Dutch Meuse react more
slowly and/or have a long travel time to the Dutch border. For those reasons models will
suffice which are simpler and require fewer data. For both types of tributaries a regression
model only based on discharges has been developed. It is striking that for the Roer the
incorporation of the discharge of the Meuse station of Borgharen-Dorp, and its forecast, leads
to a reduction of the mean square error of the forecast.

In general the wave propagation through the main river is much easier to determine than the
runoff of tributaries, because in the former case only the wave travel time and the attenuation are important. However, problems arise if important parts of the discharge inputs are unknown. That applies to the French and the Belgian Meuse. The determination of the wave propagation for the French Meuse is complicated by the fact that no information on cross-sections was available. For the Belgian Meuse the situation is complicated by the facts that the river is currently being modified and that only information on cross-sections of the future situations has been available.

For the French Meuse a multiple linear regression model only based on discharges has been developed.

For Belgium a hydrodynamic flow model was used. The model could only partly be calibrated, as discharges and water levels were hardly known, and the schematisation developed does not exactly correspond to the river in reality. In the model the weirs are in operation, just as in reality.

The Dutch Meuse is scarcely subject to modifications in respect of flood forecasting modelling. A hydrodynamic flow model schematisation could be developed on the basis of many cross-sectional data. The minor bed and the flood plains were modelled by parallel branches, resulting in a more accurate schematisation than a single branch schematisation. The sections of the flood plain branch had to be supplied with artificial gutters to prevent an unstable computation of the wave propagation in the case of a minor bed flow only. An investigation after the bank storage along the course of the Grensmaas showed the importance of this phenomenon. As a result special groundwater flow simulation sections were added, which have been constructed with only the use of the options of a normal hydrodynamic flow model. Calibration of the schematisation has been performed using measured water levels. It appears that often the exact locations of the measuring stations are unknown, that stations are moved and that the water level at the stations is influenced by dams. It was therefore concluded that for an improved flood forecasting of the Dutch Meuse the measurements rather than the model have to be improved.

The real-time flood forecasting model consists of more than just the sum of the mathematical equations. Aspects as user-friendliness, reliability, capability of handling data from different sources, capability to produce a forecast if data are missing, and a limited calculation time are essential. Therefore a personal computer program has been developed in which the model equations form the basis. The program is called FLOFOM, an abbreviation of FLOod FOREcasting for the Meuse.

As an input FLOFOM requires data of several kinds. For an optimum use they are: discharge data of 12 stations that are situated in France, Belgium and the Netherlands; and point precipitation data (or areal precipitation data) as well as precipitation forecasts for the Belgian part of the catchment. The observation frequency of the discharges and the precipitations is preferably once per hour. The model is able to handle data with a lower observation frequency successfully, and also water level measurements in combination with a stage-discharge curve. The input menus are adapted to the way the data become available.

The output consists of forecasts of the water level and the discharge of Borgharen-Dorp up to 24 hours ahead. As soon as the top is less than 24 hours away from Borgharen-Dorp, forecasts can also be generated for the top water levels of 11 other stations along the trace of the Dutch Meuse. The accuracy of the forecasts is in the order of 10 to 15 cm. The number of stations can adapted. The output is available both in a numerical and graphical form. FLOFOM is also able to produce information about intermediate results such as calculated areal precipitations and discharge forecasts of tributaries.
In January 1991 the model was used in real-time for the first time. The results were considered to be satisfying, especially the Borgharen-Dorp forecast. It is remarkable, because many data were absent and some other data showed errors.

For the low flow forecasting model another approach has been chosen than for the flood forecasting model, as the processes involved in recession and depletion flows are of a different character from those during a flood. The Meuse basin upstream from Maastricht has been subdivided into 10 subcatchments. The subdivision was unavoidable, because:

a. the catchment of the Meuse is heterogeneous with respect to depletion, and
b. the discharges at Maastricht are heavily influenced by the operation of weirs and hydroelectric power stations, thus causing strong discharge fluctuations, even in the daily discharge.

An advantage of the subdivision is the fact that for the individual subcatchments the numbers and lengths of the periods that can be used for the calibration of the depletion curves are greater than for the complete catchment. The heterogeneous character of the basin is proven by the depletion curves found: not only by the means of the decay constants of the depletion curves but the curve types also differ from one subcatchment to another. It has been shown that the southern part of the catchment is the most important with respect to the discharges during low flows.

It appears that the baseflow decreases to very low values if no rain occurs at all. However, the chance of no rain is very small. Therefore the influence of rain on the discharge has been determined, and at the same time a rainfall simulation model has been developed. The rainfall simulation model determines daily rainfall depths for each subcatchment on the basis of the stochastic properties of the rainfall in the Meuse basin.

The low flow forecasting model requires in real-time the baseflows of the 10 tributaries at the moment of the making of the forecast or before. In the latter case daily rainfall values of approximately 15 stations have to be entered about the period between the measuring of the baseflow and the moment of making the forecast. The rainfall simulation can be replaced by a rainfall forecasts of some or all the tributaries.

Experiences with the model in real-time show that the model is able to give a reliable forecast for the mean discharge of a number of days, with deviations of up to the order of 5 m³/s. The fluctuations in the discharge caused by human interferance are too great for a good forecast of the daily discharge. Also temporary works, such as weir repairs, may cause significant deviations between the measured and the calculated baseflow.
Conclusions and recommendations

Conclusions

1. The analysis of the hydrologic processes in the Meuse catchment, preceding the modelling, proved to be necessary, in particular with respect to the collection and verification of the data, the choice of appropriate models and avoiding mistakes.

2. The Meuse basin is heterogeneous with respect to the importance of aspects of the hydrologic cycle. For example the most important contributions to floods come from the Ardennes tributaries, whereas the main parts of the low flows stem from the southern part of the catchment.

3. The Meuse basin is heterogeneous with respect to the measuring equipment, the discharge determination and the available data, inclusive of the time-series data. The main cause is the international character of the basin.

4. A lot of hydrologic information is not available in a written form or can only be found with the water management service in charge.

5. The changes in the flow regime of the Meuse caused by man seem to have quickened in the course of time. The need of international cooperation has therefore been increased.

6. The risk of missing data is relatively great for the flood forecasting model for the Meuse, because:
   a. the model functions in real-time,
   b. floods are a calamity which puts all the measurement installations to the test, and
   c. many data links, some of which international, have to be used.
   The missing data may include all types of time series data.

7. Time series data with an observation frequency of once-per-day are much easier to obtain than those with a higher frequency.

8. Time series data collected in a routine are easier to obtain than data which are only collected during particular events.

9. The measured water levels along the course of the Dutch Meuse cannot be used in an optimum way for flood forecasting purposes, due to movements of the stations, irrepressiveness of the measured level in comparison to the river level, indistinctness about the places used for the measurements and a questionable quality of the real-time measurements.

10. International cooperation at the modellers' level is indispensable. This applies both during the modelling process and thereafter.

11. The subdivision of the flood forecasting model after the countries and the processes proved to be a good way of modelling the particular properties of the Meuse system.

12. The study has led to a user-friendly and case suited flood forecasting model that is flexible with respect to the input data.

13. For the Belgian part of the Meuse basin it was not possible to prove that a more sophisticated precipitation distribution method would give more accurate results than the Thiessen Polygons.

14. Before radar pictures can be used effectively in operational flood forecasting for the river Meuse many obstacles have to be overcome.

15. If the risk of the absence of measured discharge data is great, the use of a hydrologic simulation model with a stochastic correction model is more suitable for rainfall-runoff forecasting than a purely stochastic model based on discharges.
16. With the so-called potential loss intensity a state variable is introduced that enables the calculation of the precipitation loss, is able to recover, is simple in structure and links up with existing methods.

17. The influence of weirs on the wave travel time may be significant. The influence on the wave attenuation could however not be proven with the use of the hydrodynamic flow model of ZWENGL.

18. A river schematisation in a one-dimensional hydrodynamic flow model is more like reality if two parallel branches are used, one for the minor bed and one for the flood plains (this changes the 1-dimensional model into a quasi 1+1-dimensional model). Disadvantages of the parallel branch model are an increase of computation time and possibly the need of minor adaptations in flood plain sections to prevent unstable calculations.

19. Bank storage may be of significant influence on the flood wave attenuation.

20. The incorporation of the groundwater flow in one-dimensional hydrodynamic flow models can be done by using standard sections. However, the results will not be very accurate.

21. Low-flow forecasts for the Meuse can be produced with reasonable accuracy if correct measurements are available and the current situation of human interference is known.

22. In many cases the depletion curve of a river or tributary may be approached more accurately by other functions than the negative exponential function, at least for the Meuse basin.

23. The magnitude of the baseflow of the Meuse is strongly dependent on the rainfall that has fallen during the last few months. Therefore long-term forecasts based on no-rainfall scenarios always lead to extremely low baseflows.

Recommendations from the conclusion

1. Before a hydrologic forecasting model is developed, the hydrologic process in the catchment should be analysed, as well as the way of data collection. 1

2. An agreement should be reached about the magnitude of the discharges at the border by the services involved. The study made by Teuber et al. (1986) may serve as an example. 2

3. The way of data storage should be dependent among other things on the way of data storage of other services in the catchment, thus facilitating cooperation between the services. 3

4. The cooperation between services should be improved: exchanging investigation results, information of changes in the physiography of the system and hydrologic data; instituting a joint database; compiling a monograph of the basin and joint research may contribute to a more successful river management. Here the expression 'cooperation between services' is used instead of the more usual term 'international cooperation', because also the cooperation between domestic services may be improved. 2,4,5, 10,21

5. Efforts should be made to maintain/increase the availability of the time-series data required for the model. Also the availability of current stage-discharge curves and information about changes in the system are important. 6

6. When a real-time model is developed, the effects of missing data should be incorporated from the start. 6

7. Before modelling is started the real-time availability of data should be ensured. 6
Only data from those stations should be chosen which are and will be well maintained.

8. The use of time series data with an observation frequency higher than once a day should be avoided, if the model performance is not affected seriously.

9. The amount (of types) of time series data required in real-time should be minimised, as well as the number of services involved.

10. Data which are not collected in a routine should preferably not be used in a real-time calamity model.

11. Data storage should be done conscientiously. It applies in particular to data which are not collected routinely. An indication of the quality and the exact location of the measuring should be added to the data.

12. In the determination of the geographical position of measuring stations the representativeness of the measured parameter during floods should be incorporated.

13. The quality of real-time measured data should be improved. The validation of data months after the measurement is too late for using a real-time forecast among other things.

14. If weirs are in operation during floods, the effect on the wave propagation should be investigated when developing a flood forecasting model.

15. If flood plains are important in the wave propagation and the modelling is done by means of a hydrodynamic flow model, the modelling of the minor bed and the flood plains in separate branches should be considered.

16. Groundwater flow models should be incorporated in hydrodynamic flow models.

17. For the determination of the depletion curves other functions than just the negative exponential functions should be investigated.

18. Every summer/autumn season the authorities should be ready to expect a low-flow situation for the Meuse within a few months, regardless of the current discharge.
Samenvatting

De Maas vervult voor de mens een aantal nuttige functies, met betrekking tot de scheepvaart, de watervoorziening, koelwater, enzovoort. Tegelijkertijd kan de Maas schade veroorzaken, zoals in het geval van hoogwaters, lage afvoeren, ijsgang, calamiteiten ontstaan door lozingen, enzovoort. In veel gevallen is een voorspelling van de afvoer gewenst, omdat dan de eventueel benodigde maatregelen op tijd genomen kunnen worden. In deze dissertatie wordt de ontwikkeling van twee typen afvoervoorspellingsmodellen beschreven: hoogwatervoorspelling en laagwatervoorspelling.

De ontwikkeling van afvoervoorspellingsmodellen voor de Maas is relatief gecompliceerd, omdat:

a. de hoeveelheid beschikbare gegevens beperkt is;

b. het stroomgebied heterogeen is met betrekking tot de belangrijke aspecten van de hydrologische kringloop, maar ook met betrekking tot de meetuitrusting, de afvoerbepaling en de hoeveelheid beschikbare gegevens, inclusief de tijdreeksgegevens;

c. de afvoer sterk door de mens wordt beïnvloed; en

d. de tijd tussen neerslag en afvoer relatief kort is, althans voor het grootste deel van het stroomgebied.

Door het internationale karakter van de rivier was er aan het begin van het onderzoek onvoldoende kennis van de hydrologische processen beschikbaar. Daarom is voorafgaande aan de modelontwikkeling een systeem-analyse uitgevoerd.

De Maas heeft een lengte van 874 km van de bron in Frankrijk tot de monding in het Hollands Diep. Het stroomgebied heeft een oppervlak van 33000 km². Hydrologisch gezien kan de Maas in drie stukken opgedeeld worden.

De bovenloop, Meuse Lorraine genaamd, loopt vanaf de bron in het uiterste zuiden van het stroomgebied tot de monding van de Chiers. Het aantal zijrivieren is gering, het bodemverhang van de rivier relatief klein voor een bovenloop en de riviervallei, die tijdens hoogwatergolven onder water lopet, is breed. Door deze eigenschappen reageert de Meuse Lorraine langzaam op de neerslag. De topafvoer bij hoogwatergolven is relatief klein voor een stroomgebied van deze omvang. In perioden van weinig neerslag is de afvoer relatief groot, omdat een aanzienlijke hoeveelheid water in het stroomgebied geborgen kan worden.

De middenloop, Meuse Ardennaise genaamd, omvat het gedeelte tussen de monding van de Chiers en de Belgisch-Nederlandse grens. Het bodemverhang is relatief groot voor een middenloop. Het aantal zijrivieren is groot, en veel zijrivieren zijn gelegen in heuvelachtige gebieden met een kleine infiltratiecapaciteit. Hierdoor wordt het water snel afgevoerd, wat resulteert in grote piekgolven en lage afvoeren gedurende droge perioden. Dit gedeelte van de rivier verschilt van de boven- en de benedenloop door het feit dat tijdens hoogwaters stuwenu nog in bedrijf zijn en de grootte van de geïnundeerde gebieden gering is.

De benedenloop is het gedeelte tussen de Belgisch-Nederlandse grens en de monding. Het bodemverhang van de rivier neemt af in stroomafwaartse richting tot lage waarden. Het aantal belangrijke zijrivieren is gering. Stroomafwaarts van Boxmeer zijn er dijken, stroomafwaarts van Lith ook getijinvloeden. In Nederland was vroeger het meest stroomafwaartse deel van de rivier het gevoeligst voor overstromingen, wegens ijsgang en de verbindingen met de Waal. Tegenwoordig is door een temperatuurstijging van het water door de aanwezigheid van energiecentrales en door rivierverbeteringswerken het meest gevoelige deel in het zuiden van het land te vinden, waar geen dijken aanwezig zijn om dorpen tegen
het water te beschermen.

Het afvoerregime van de Maas wordt al sinds eeuwen beïnvloed door de mens. Een continue verandering van de afvoerkarakteristieken wordt veroorzaakt door veranderingen in landgebruik; de toepassing van irrigatie, beregening en drainage; de bouw van kanalen, stuwen, waterkrachtcentrales en stuwmuren; veranderingen in de loop en de dwarsprofielen, de winning van grond en zand; lozingen en onttrekkingen. De veranderingen in het afvoerregime lijken in de loop van de tijd steeds sneller te gaan. De meest belangrijke aspecten met betrekking tot de afvoervoorspelling zijn de veranderingen in de dwarsprofielen en het stuwebeheer, alsmede veranderingen in de onttrekkingen en lozingen.

De systeemanalyse wees uit dat de, voor de hoogwatervoorzorg, belangrijkste processen veranderen van gebied tot gebied. Daarom is besloten om voor elke hydrologische eenheid een specifiek model, module genaamd, te bouwen. Bij de onderscheiding van het model in modulen werden ook soort, hoeveelheid en kwaliteit van de gegevens in de beschouwing betrokken. Het resultaat was een onderscheiding naar proces (neerslag-afvoerrelatie versus golfvoortplanting in de rivier) en naar land.

Bij de modelkeuze heeft de beschikbaarheid van gegevens in real-time vanaf het begin een rol gespeeld.

De meest belangrijke zijrivieren zijn, in het licht van een hoogwatervoorzorg in Nederland, gelegen in België. Daarom heeft de module van de Belgische zijrivieren veel aandacht gekregen. Deze module bestaat uit zes neerslag-afvoermodellen, die zijn ontwikkeld voor de zes meest belangrijke zijrivieren. De kern van deze modellen wordt gevormd door een model dat de directe afvoer bepaalt als functie van de neerslag en neerslagvoorspellingen. Om de gevraagde totale afvoer te kunnen berekenen is tenminste één meting, of een schatting daarvan, noodzakelijk. Het model dat de directe afvoer bepaalt bestaat uit vier delen.

a. De gebiedsneerslag wordt bepaald als functie van puntneerslagen. Na het vergelijken van verschillende methoden bleek de Thiessen Polygonen methode de optimale te zijn.

b. Met behulp van de gebiedsneerslagen worden de effectieve neerslagen bepaald. De effectieve neerslag is dat deel van de neerslag dat bijdraagt tot de directe afvoer. In dit gedeelte van het directe-afvoermodel wordt één toestandssvariabele gebruikt, namelijk de potentiële verliesintensiteit. Het verloop van de potentiële verliesintensiteit is afhankelijk van de evenwichtswaarde van de potentiële verliesintensiteit. De evenwichtswaarde van de potentiële verliesintensiteit is alleen een functie van de neerslagintensiteit op het beschouwde moment. De effectieve neerslag is een functie van de neerslag en de potentiële verliesintensiteit.

c. De reeks effectieve neerslagen wordt met behulp van een Nash-cascade omgezet in een reeks directe afvoeren.

d. De afvoervoorspellingen van de Belgische zijrivieren worden samengesteld uit de berekende directe afvoeren en de gemeten afvoeren. Dit gedeelte van het model is met opzet eenvoudig gehouden, waardoor ook voorspellingen gegenereerd kunnen worden als gemeten afvoeren geheel of gedeeltelijk afwezig zijn.

Optimalisatie van de modelparameters heeft plaatsgevonden met behulp van de regressie-analyse. De resultaten blijken nauwkeurig te zijn in het licht van het uiteindelijke doel: een nauwkeurige afvoervoorspelling van het station nabij de Nederlandse grens (Borgharen-Dorp).

De grotere zijrivieren die in de Franse en Nederlandse Maas uitmonden reageren langzamer of/en hebben een grote loop tijd tot de Nederlandse grens. Voor deze zijrivieren volstaat een eenvoudiger en minder gegevens vergend model. Voor beide types zijrivieren zijn regressiemodellen opgesteld die alleen op afvoeren gebaseerd zijn. Opvallend is dat de
prestatie van het model van de Roer significant verbeterd wordt door de afvoermetingen en -voorspellingen van het aan de Maas gelegen meetpunt Borgharen-Dorp in de berekening te betrekken.

Over het algemeen is de golfvoortplanting door de hoofdtak van de rivier veel eenvoudiger te bepalen dan de afvoer van de zijrivieren, omdat in het eerste geval alleen de looptijd en de afvlakking van belang zijn. Er ontstaan echter problemen wanneer belangrijke delen van de instroom onbekend zijn. Dat is het geval voor de Franse en Belgische Maas. De vaststelling van de golfvoortplanting voor de Franse Maas wordt nog complexer door het feit dat geen informatie over de dwarsprofielen aanwezig is. Voor de Belgische Maas is het modelleren lastig door het feit dat de rivier momenteel wordt verbeterd en dat informatie over dwarsprofielen slechts aanwezig is voor wat betreft de toekomstige toestand.

Voor de Franse Maas is een meervoudig lineair regressiemodel ontwikkeld dat uitsluitend met behulp van afvoeren is bepaald.

Voor de Belgische Maas is een hydrodynamisch afvoermodel gebruikt. Het model kon slechts gedeeltelijk gecalibreerd worden, omdat afvoeren en waterstanden slechts gedeeltelijk bekend waren, en de ontwikkelde schematisatie niet geheel overeenkomt met de rivier in werkelijkheid. De stuwen zijn in het model in bedrijf, net zoals in de werkelijkheid.

De Nederlandse Maas is slechts in geringe mate onderhevig aan veranderingen, gezien in het licht van de ontwikkeling van een hoogwatervoorzieningsmodel. Een schematisatie voor een hydrodynamisch afvoermodel kon ontwikkeld worden. Hierbij kon over een grote hoeveelheid informatie over dwarsprofielen beschikt worden. Het zomerbed en het winterbed zijn beschreven met behulp van evenwijdige takken, wat resulteerde in een meer nauwkeurige schematisatie dan met behulp van slechts één tak. De vakken die deel uitmaken van de tak van het winterbed moesten voorzien worden van kunstmatige gootjes, teneinde een instabiele berekening van de golfvoortplanting te voorkomen in het geval dat het water alleen door het zomerbed stroomt. Een onderzoek naar de grondwaterberging langs de Grensmaas duidde het belang aan van het fenomeen grondwaterberging. Daarom zijn aan het model speciale vakken toegevoegd, die de stroming van het grondwater simuleren. Deze zijn geconstrueerd met behulp van de mogelijkheden die een normaal hydrodynamisch afvoermodel biedt. Calibratie van de schematisatie heeft plaatsgevonden met behulp van de gemeten waterhoogten. Het is gebleken dat de exacte locatie van de meetstations vaak onbekend is, dat stations verplaats zijn en dat de waterhoogten bij de stations significant worden beïnvloed door dammen. Daaruit is de conclusie getrokken dat voor een verbeterde hoogwatervoorzetting voor de Nederlandse Maas eerder de metingen dan het model dienen te worden verbeterd.

Het real-time hoogwatervoorzieningsmodel bestaat uit meer dan alleen de som van de mathematische vergelijkingen. Aspecten als gebruiksvriendelijkheid, betrouwbaarheid, de mogelijkheid om gegevens van andere bronnen te gebruiken, de mogelijkheid om een voorspelling te maken als gegevens ontbreken en een beperkte rekentijd zijn essentieel. Daarom is een programma ontwikkeld voor de personal computer, waarin de modelvergelijkingen de basis vormen. Het programma heet FLOOM, wat een afkorting is van FLOod FOrecasting for the Meuse (hoogwater-voorspelling voor de Maas).

Als invoer heeft FLOOM gegevens van verschillende soorten nodig. Voor een optimaal gebruik zijn dat: afvoergegevens van 12 stations, die gelegen zijn in Frankrijk, België en Nederland; en puntneerslagen (of gebiedsneerslagen) alsmede neerslagvoorspellingen voor het Belgische deel van het stroomgebied. De waarnemingsfrequentie van afvoeren en neerslagen bedraagt bij voorkeur eens per uur. Het model is in staat om ook succesvol om te gaan met gegevens met een lagere
waarnemingsfrequentie, alsmede met waterstandsmetingen in combinatie met een afvoerkromme in plaats van afvoeren. De menu’s voor de gegevensinvoer zijn aangepast aan de wijze waarop de gegevens beschikbaar komen.

De uitvoer bestaat uit voorspellingen van de waterstand en afvoer van Borgharen-Dorp tot 24 uur vooruit. Zodra de top minder dan 24 uur van Borgharen-Dorp verwijderd is, kunnen ook voorspellingen worden gemaakt van de topwaterstanden voor 11 andere stations langs de Nederlandse Maas. De nauwkeurigheid van de voorspellingen is in orde van grootte 10 à 15 cm. Het aantal stations kan naar believen worden aangepast. De uitvoer is beschikbaar in zowel numerieke als grafische vorm. FLOFOM kan ook informatie verschaffen over tussensresultaten als berekende gebiedsneerslagen en afvoervoorspellingen van zijrivieren.


Voor het laagwatervoorspellingsmodel is voor een andere opzet gekozen dan voor het hoogwatervoorspellingsmodel, omdat de van belang zijnde processen bij recessie- en uitputtingsskrommen van een ander karakter zijn dan die bij een hoogwater. Het stroomgebied van de Maas stroomopwaarts van Maastricht is ondervdeeld in 10 deelstroomgebieden. De onderverdeling was onvermijdelijk, omdat:
- het stroomgebied van de Maas heterogene is met betrekking tot uitputting, en
- de afvoer bij Maastricht sterk beïnvloed wordt door het beheer van stuwen en waterkrachtcentrales, waardoor sterke afvoerschommelingen ontstaan, zelfs in de dagafvoer.

Een voordeel van de onderverdeling is het feit dat voor de deelstroomgebieden het aantal en de lengte van de perioden die gebruikt kunnen worden voor de calibratie van de uitputtingsskrommen groter is dan voor het stroomgebied als geheel. Het heterogene karakter van het stroomgebied kan worden aangetoond met de gevonden uitputtingsskrommen: niet alleen de nepereringsstijl van de uitputtingsskromme maar ook het type uitputtingsskromme kan verschillen per deelstroomgebied. Aangetoond kan worden dat gedurende laagwaters het zuidelijk deel van het stroomgebied het meest belangrijk is voor de afvoer.

Het blijkt dat de basisafvoer tot zeer lage waarden kan dalen als er zich totaal geen regen voordoet. De kans op geen regen is echter zeer klein. Daarom is de invloed van regen op de afvoer onderzocht, en is tegelijkertijd een simulatiemodel voor de regen ontwikkeld. Het simulatiemodel voor de regen bepaalt de dagelijkse neerslaghoogten voor elk deelstroomgebied op basis van de stochastische eigenschappen van de regenval in het stroomgebied van de Maas.

Het laagwatervoorspellingsmodel benodigt in real-time de basisafvoeren van de 10 deelstroomgebieden op het moment van het maken van de voorspelling of daarvoor. In het laatste geval moeten dagneerslagen van ca. 15 stations worden ingevoerd van de periode tussen de momenten van het meten van de basisafvoer en het maken van de voorspelling. De simulatie van de regenval kan worden vervangen door een voorspelling van de regen voor enkele of alle zijrivieren.

Ervaringen met het model in real-time leren dat het model een betrouwbare voorspelling van de gemiddelde afvoer bij Maastricht over een aantal dagen kan genereren, met afwijkingen tot in orde van grootte van 5 m³/s. De door de mens veroorzaakte afvoerfluctuaties zijn echter te groot voor een goede voorspelling van de dagafvoer. Ook werken van tijdelijke aard, zoals reparaties aan stuwen, kunnen een grote afwijkings veroorzaken tussen gemeten en berekende basisafvoer.
Conclusies en aanbevelingen

Conclusies

1. De analyse van de hydrologische processen in het Maasgebied, die voorafging aan het modelleren, bleek noodzakelijk te zijn, in het bijzonder met betrekking tot het verzamelen en verfijnen van de gegevens, de keuze van geschikte modellen en het vermijden van fouten.

2. Het stroomgebied van de Maas is heterogeen met betrekking tot de belangrijke aspecten van de hydrologische kringloop. Bijvoorbeeld de belangrijkste bijdragen aan hoogwaters zijn van de Ardennen afkomstig, terwijl het belangrijkste deel van de laagwaterafvoer uit het zuiden van het stroomgebied komt.

3. De Maas is heterogeen met betrekking tot de meetinstallaties, de afvoerbepalingen en de beschikbare gegevens, inclusief de tijdreeksen. Dit hangt in het bijzonder samen met het internationale karakter van het stroomgebied.

4. Een grote hoeveelheid hydrologische informatie is niet in geschreven vorm beschikbaar of kan alleen met behulp van de verantwoordelijke waterbeheerder gevonden worden.

5. De veranderingen in het afvoerregime van de Maas onder invloed van de mens lijken in de tijd steeds sneller te verlopen. De behoefte aan internationale samenwerking neemt daarom toe.

6. De kans op het ontbreken van gegevens is voor het hoogwatervoorspellingsmodel voor de Maas relatief groot, omdat:
   a. het model in real-time werkt,
   b. hoogwaters een calamiteit zijn waardoor alle meetinstallaties op de proef worden gesteld, en
   c. veel verbindingen, waarvan sommige internationaal, nodig zijn voor de gegevenstransmissie.

7. Tijdreeksgegevens met een waarnemingsfrequentie van éénmaal per dag zijn eenvoudiger te verkrijgen dan die met een hogere frequentie.

8. Tijdreeksgegevens die routinematig worden verzameld zijn eenvoudiger te verkrijgen dan gegevens die alleen gedurende bepaalde gebeurtenissen worden verzameld.

9. De gemeten waterhoogten langs het traject van de Nederlandse Maas kunnen niet optimaal gebruikt worden bij de hoogwatervoorspelling, ten gevolge van verplaatsingen van de meetstations, de niet-representativiteit van de gemeten waterhoogten met betrekking tot de waterhoogten in de rivier, onduidelijkheid omtrent de ligging van de meetinstallaties en een twijfelachtige kwaliteit van de real-time gemeten waterhoogten.

10. Internationale samenwerking op het niveau van de ontwikkelaar van het model is onmisbaar. Dit geldt gedurende het modelleren, maar ook erna.

11. De verdeling van het hoogwatervoorspellingsmodel naar landen en naar processen bleek een goede manier te zijn om de bijzondere eigenschappen van het stroomgebied van de Maas goed te modelleren.

12. Het onderzoek heeft geleid tot een gebruikersvriendelijk en specifiek op de Maas toegespitst hoogwatervoorspellingsmodel dat flexibel is met betrekking tot de invoer- en gegevens.

13. Voor het Belgische deel van het stroomgebied was het niet mogelijk aan te tonen dat met een meer verfijnd neerslagverdelingsmodel betere resultaten zouden worden.
verkregen dan met de Thiessen Polygonen.
14. Voordat radarbeelden doeltreffend gebruikt kunnen worden bij de operationele hoogwatervoorspellingen voor de Maas dienen nog veel hindernissen genomen te worden.
15. Als de kans op het ontbreken van gegevens in real-time groot is, is voor de neerslag-afvoervoorspelling het gebruik van een hydrologisch simuliemodel met een stochastisch correctiemodel meer geschikt dan een geheel stochastisch model.
16. Met de zogenaamde potentiële-verliesintensiteit is een toestandsvariabele geïntroduceerd waarmee het neerslagverlies berekend kan worden, die zich kan herstellen, die een eenvoudige structuur heeft en die een aansluiting heeft met bestaande methoden.
17. De invloed van stuwen op de looptijd van golven kan significant zijn. De invloed op de golfafvlakking kon met het hydrodynamisch afvoermodel ZWENDEL echter niet bewezen worden.
18. Een rivierschematisatie in een 1-dimensionaal hydrodynamisch afvoermodel is meer overeenkomstig de werkelijkheid indien twee parallelle takken worden gebruikt, één voor het zomerbed en één voor het winterbed (dit verandert het 1-dimensionale model in een quasi 1"-dimensionaal model). Nadelen van het gebruik van parallelle takken zijn de toename van de rekentijd en eventueel de noodzaak van kleine aanpassingen in de winterbedvakken, teneinde instabiele berekeningen te voorkomen.
19. De grondwaterberging kan een significante invloed hebben op de afvlakking van hoogwatergolven.
21. Laagwatervoorspellingen voor de Maas kunnen met een behoorlijke nauwkeurigheid gemaakt worden indien goede waarnemingen beschikbaar zijn en de invloed van de op dat moment uitgevoerde kunstmatige ingrepen bekend is.
22. In veel gevallen kan de uitputtingskromme van een (zij)rivier beter met een andere functie dan de negatief exponentiële benaderd worden, zeker voor het stroomgebied van de Maas.
23. De grootte van de basisafvoer van de Maas is sterk afhankelijk van de regen die gedurende de paar laatste maanden is gevallen. Daarom leiden lange-termijn voorspellingen gebaseerd op een-neerslag scenario's altijd tot extreem lage basisafvoeren.

Aanbevelingen

uit conclusie
1. Voordat een hydrologisch voorspellingsmodel ontwikkeld wordt dient het hydrologisch proces in het stroomgebied en de wijze van gegevensverzameling geanalyseerd te worden.
2. Er dient bij de betrokken waterbeheerders overeenstemming te komen over de grootte van de afvoer bij de grens. De studie van Teuber et al. (1986) kan als voorbeeld dienen.
3. De wijze van gegevensopslag dient ondermeer aan te sluiten op de wijze van gegevensopslag van andere waterbeheerders in het stroomgebied, waardoor de samenwerking tussen de waterbeheerders eenvoudiger wordt.
4. De samenwerking tussen de waterbeheerders dient verbeterd te worden. Het uitwisselen van: onderzoeksresultaten, informatie met betrekking tot

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veranderingen in de fysiografie van het systeem en de hydrologische gegevens, het opzetten van een gezamenlijke gegevensbank, het samenstellen van een monografie van het stroomgebied en gezamenlijk onderzoek kunnen bijdragen tot een meer succesvol rivierbeheer. De term 'samenwerking tussen waterbeheerders' is hier gebruikt in plaats van de gebruikelijke term 'internationale samenwerking', omdat ook de samenwerking binnen de landsgrenzen verbeterd kan worden.

5. Er dienen inspanningen gedaan te worden om de beschikbaarheid van tijdreeksgegevens die nodig zijn voor het model te handhaven/verbeteren. Ook de beschikbaarheid van geldige afvoerkrommen en informatie over de veranderingen in het systeem zijn van belang.

6. Wanneer een real-time model wordt ontwikkeld, dienen de effecten van ontbrekende gegevens vanaf het begin van de studie hierbij betrokken te worden.

7. Voordat met het modelleren wordt begonnen dient de beschikbaarheid van real-time gegevens bekend te zijn. Alleen gegevens van stations die goed zijn en zullen worden onderhouden zouden gebruikt moeten worden.

8. Het gebruik van gegevens met een waarnemingsfrequentie van groter dan eens per dag dient vermeden te worden, als de prestatie van het model hierdoor niet significant wordt aangetast.

9. De hoeveelheid tijdreeksgegevens(soorten) die nodig zijn in real-time dient geminimaliseerd te worden, alsmede het aantal betrokken instanties.

10. Gegevens die niet routinematig verzameld worden zouden bij voorkeur niet in een real-time calamiteitenmodel gebruikt moeten worden.

11. De gegevensverzameling dient met zorg te geschieden. Dit geldt vooral voor gegevens die niet routinematig verzameld worden. Een aanduiding van de kwaliteit en de exacte locatie van de meting dient aan de gegevens te worden toegevoegd.

12. Bij de bepaling van de geografische plaats van meetstations dient ook de representativiteit van de betrokken parameters tijdens hoogwaters betrokken te worden.

13. De kwaliteit van de gegevens dient verbeterd te worden. De validatie van gegevens maanden na de meting is te laat om nog van nut te zijn voor onder meer een real-time voorspelling.

14. Indien de stuwern tijdens hoogwaters nog in bedrijf zijn, dient bij het ontwikkelen van een hoogwatervoorspellingsmodel de invloed van stuwern op de golfvoortplanting in het onderzoek betrokken te worden.

15. Indien het winterbed belangrijk is bij de golfvoortplanting en het modelleren geschiedt met een hydrodynamisch afvoermodel, dient het modelleren van zomer- en winterbed in aparte vakken overwogen te worden.


17. Voor de bepaling van uitputtingskrommen dienen ook andere functies dan de negatief exponentiële onderzocht te worden.

18. De beheerders dienen gedurende het zomer/berfst-seizoen voorbereid te zijn op het optreden van een laagwatersituatie op de Maas binnen enkele maanden, dit onafhankelijk van de momentane afvoer.
Curriculum Vitae

Herbert Berger was born in Enschede on 6 March 1960. He obtained the Atheneum-B certificate at the Jacobus College in Enschede in 1978. After that he started studying Civil Engineering at the Delft University of Technology. He obtained the degree of civil engineer in 1986 after completing his study in the group of hydraulic engineering and hydrology. The doctoral thesis dealt with the behaviour of the interface between fresh and salt groundwater in semi-confined aquifers in a non-steady situation. It was written under the supervision of prof.dr.ir. J.C. van Dam.

After his study he started his career with the Rijkswaterstaat, the Institute for Inland Water Management and Waste Water Treatment and built a flow forecasting model for the Meuse. Although the work took place in the Rijkswaterstaat office in Lelystad, he was formally employed by the Delft University of Technology, in view of the promotion.

At present he works as a secretary/coordinate of the working community of integrated water management in the Delft University of Technology.