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FLOOD INUNDATION MODELLING USING MILHY

Final Technical Report

by

L.Baird and M.G.Anderson

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User Manual
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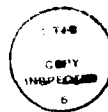
Professor M.G.Anderson

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VOLUME 1

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Dr. Ed Link and Mr. John Collins provided the initial supporting framework for this project at Waterways Experiment Station. Their support is gratefully acknowledged,



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Preface

This report details work undertaken under contract DAJA-45-C-0053 to September 1990. We report the further development of the ungauged flood forecasting model MILHY. Specifically, new routines are introduced to allow the discrete routing of floodplain and main channel flow and to incorporate the exchange of momentum between main channel and floodplain. The program (MILHY3) is applied to a large watershed, the River Fulda, West Germany, where both MILHY3 developments and MILHY2 developments (DAJA-37-81-C-0221) are assessed.

In addition, a two-dimensional hydrodynamic model, RMA-2V, has been applied to a 30km reach of the River Fulda, and 11km reach of the River Culm, England. Results from the River Fulda application showed that it is feasible to consider linking RMA-2V with MILHY3 to generate detailed inundation modelling within a hydrologic catchment simulator. The River Culm application has explored some of the limitations and problems of large-scale applications of RMA-2V, and the importance of topographic resolution is highlighted. Results from these two applications allow environments to be identified, where RMA-2V may be successfully applied at the large-scale, and environments where further developmental work is required.

In Volume 2 of this report, the MILHY3 code is presented with a user manual and source code details.

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Chapter 1
Introduction

1.1 Background

This study relates to the further development of an operational model for ungauged flood forecasting. The model used as the starting basis for the project was MILHY2; a model delivered to Waterways Experiment Station in 1986, under contract DAJA-45-C-0029.

The history of MILHY development as an ungauged forecasting model and research scheme is as follows:

- MILHY : model for ungauged flow forecasting using Curve Number (CN) scheme to generate runoff, 1982
- MILHY1 : adaptation of MILHY under contracts DAJA-37-82-C-0092 and DAJA-37-81-C-0221 by Dr M G Anderson to replace CN scheme by a physically-based runoff generation method (finite difference), 1984
- MILHY2 : development and validation of MILHY1 on small subcatchment scale ($\leq 1\text{km}^2$) watersheds by Dr M G Anderson and Dr S Howes, under contract DAJA-45-83-C-0029. Code delivered to WES in 1986
- MILHY3 : further development of MILHY2 to upgrade the prediction of the hydrograph under floodplain inundation conditions. Subject of contract here presented by Dr M G Anderson and Dr L Baird, 1990

Upon initiation of the current contract MILHY2 represented a fully working scheme (see Figure 1.1) and was subjected to limited validation, as shown in Table 1.1 and Figures 1.2 and 1.3. The main conclusions of contract DAJA-45-83-C-0029 relating to the development of MILHY2 were that:

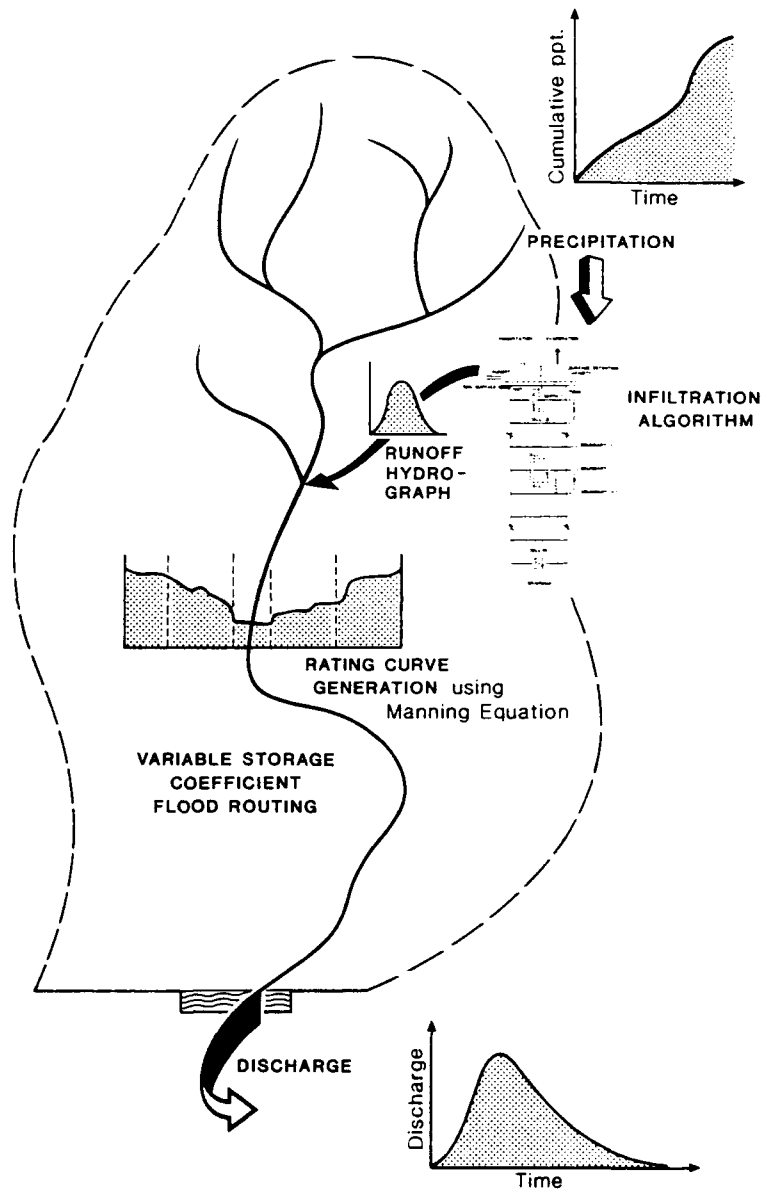


Figure 1.1
Conceptual Structure of MILHY2 Scheme

Table 1.1
Comparison of catchment characteristics which are
required by the unit hydrograph procedure

	Area (km ²)	Difference in elevation (m)	Length of main channel (km)
W-2 North Danville Vermont	0.6	79.3	1.2
W-1 Treyvor, Iowa	0.3	27.4	1.1
W-2 Treyvor, Iowa	0.3	21.3	0.9
W-4 Treyvor, Iowa	0.6	30.5	0.6

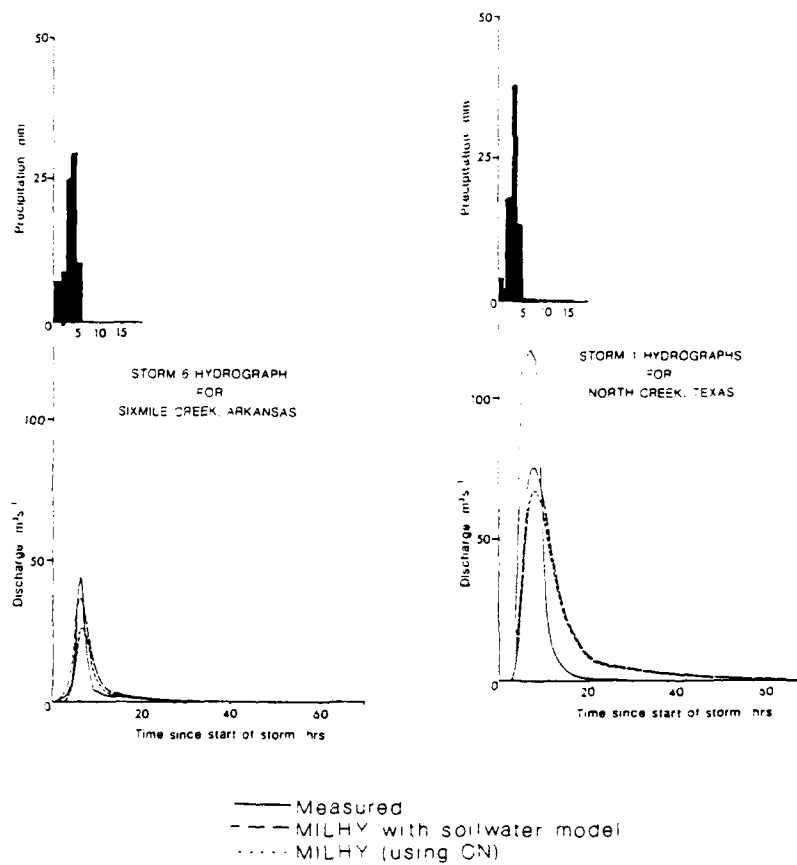


Figure 1.2
Comparison of observed and MILHY, MILHY2 simulations for
 a) Sixmile Creek, Arkansas
 b) North Creek, Texas

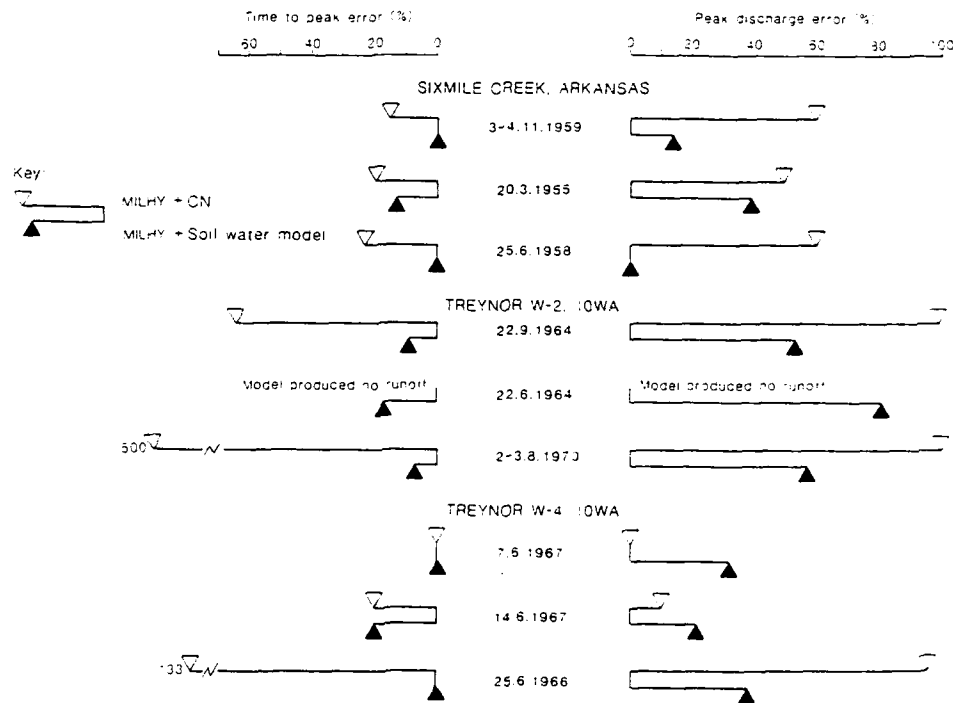


Figure 1.3
Comparison of MILHY and MILHY2 simulations for Sixmile
Creek, Arkansas, Treynor W-2, Iowa, and Treynor W4, Iowa

Chapter 1

- (i) the correlation between predicted and measured peak discharge using MILHY2 was high ($r = 0.91$)
- (ii) the time to peak discharge estimation was good using MILHY2 ($r = 0.97$)
- (iii) a comparison of MILHY and MILHY2 for 32 experimental frames showed strong evidence of the overall improvement achieved by MILHY2

There was, therefore, a strong basis for pursuing the further development of MILHY2. In particular, it is important to investigate the development of MILHY2 for larger subcatchment scales than previously undertaken, where channel processes become more significant.

1.2 Objectives and Scope

The overall objective of this contract is to improve the predictive capabilities of MILHY2 whilst retaining its parsimonious data requirements, portability and simplicity of application. This objective has been achieved through the investigation of the following specific objectives:

- 1) to develop the modular structure of MILHY to allow not only flexibility in catchment representation but also choice in the process resolution of the solution.
- 2) to develop the channel routing components of MILHY2 to improve prediction of out-of-bank flood events
- 3) to investigate and implement several alternative validation schemes for the revised MILHY scheme, using both field data and hydrodynamic simulation of the River Fulda catchment in West Germany

Chapter 1

- 4) to explore the suitability of a finite-element hydrodynamic model (RMA-2V) for large scale floodplain modelling,
- 5) to investigate the linking of MILHY with a hydrodynamic model (RMA-2V) to provide enhanced inundation modelling within a catchment scheme.

Chapter 2

Research Design

2.1 Introduction

The overall research design of the MILHY project as developed by Dr Anderson's group at Bristol University over the last seven years is shown in Figure 2.1. The initial decision regarding MILHY1 related to utilisation of finite difference methods for runoff generation. Subsequent research identification suggested the need to examine modular program structures and out-of-bank channel modelling, and it is these developments that are major constituents of MILHY3. However, more general issues are raised here in the context of the interaction of hydraulic and hydrologic schemes and their respective suitability for ungauged inundation modelling.

2.2 Modular Program Structures

Any model design is essentially a two-dimensional matrix of components. This is illustrated in general terms in Figures 2.2 and 2.3. Decisions have to be made in two principal areas: (1) submodel inclusion, and (2) resolution of the selected submodel. It is the latter decision area that has proved the focus of the current research. In particular, we have sought to examine the effects of changes in the channel routing submodel, in the context of examining and implementing alternative models for out-of-bank flow conditions. In addition, a somewhat lesser effort has been expended in an examination of precipitation. Figure 2.3 shows the scope of the submodel resolution development. An important consideration here relates to achieving a submodel resolution that is considered, broadly at least, to be consistent between all submodels. Earlier research in the MILHY program concentrated on improving the resolution of the infiltration submodel (see Figure 2.1). An important consideration in this project has therefore been to assess the relative performance of the infiltration algorithm in comparison

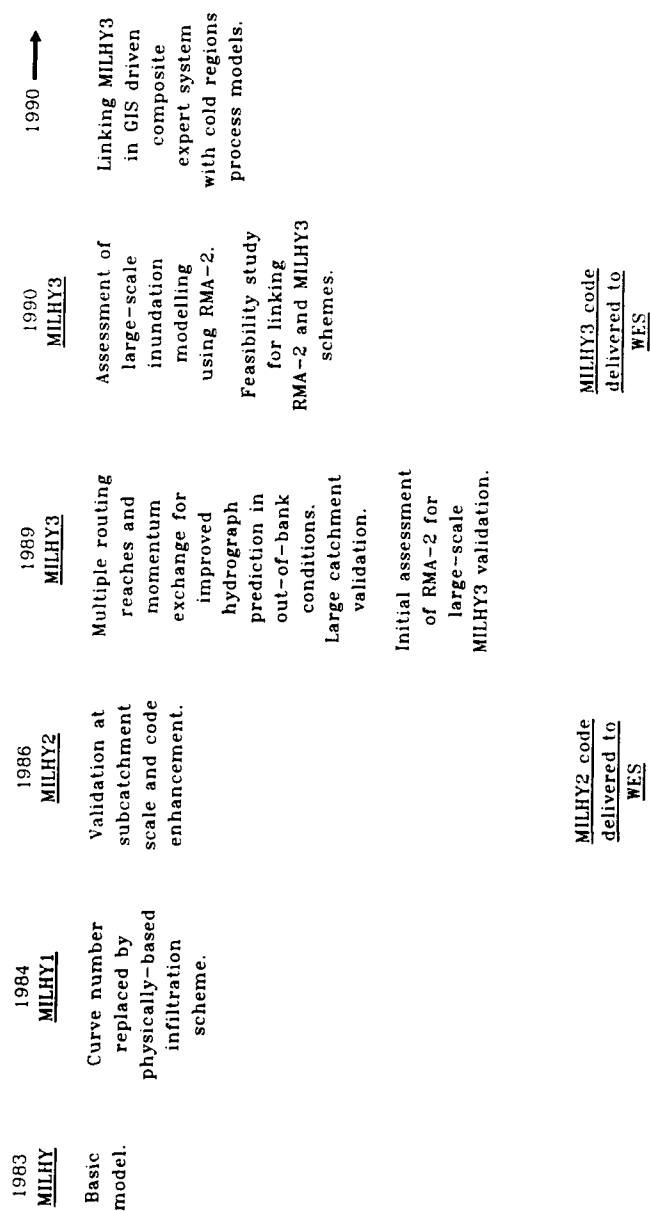


Figure 2.1
MILHY Project Research Programme at the University of Bristol

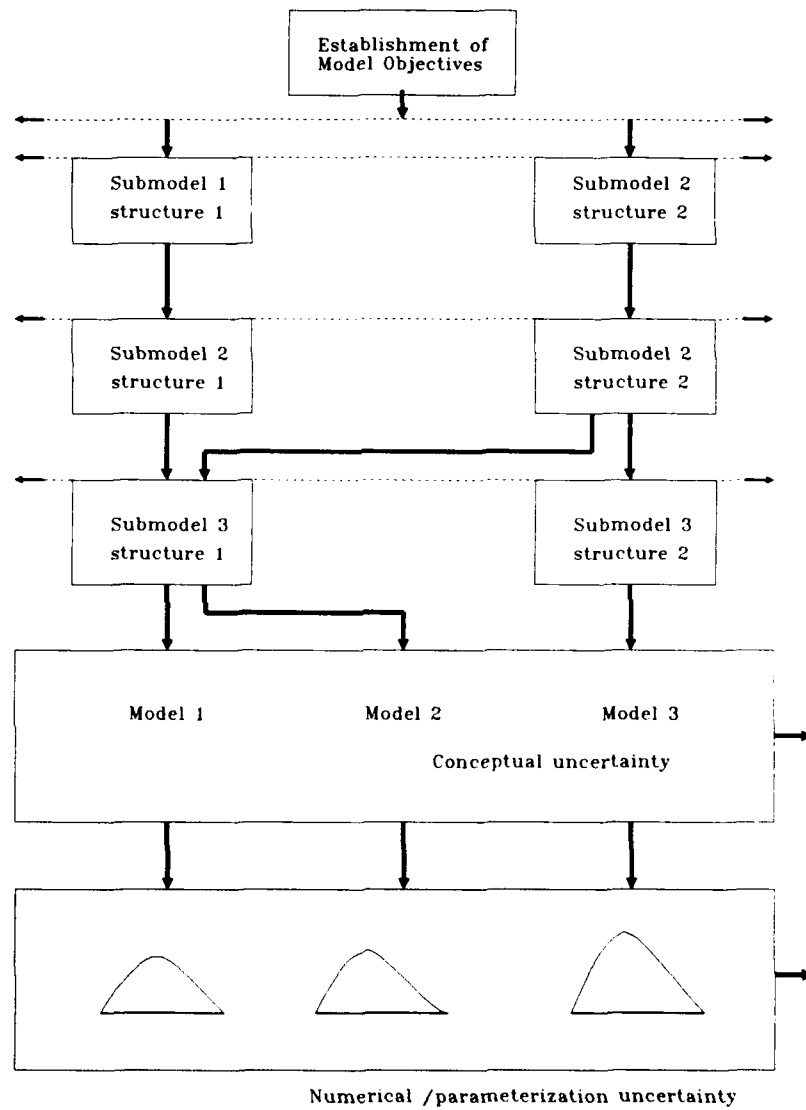
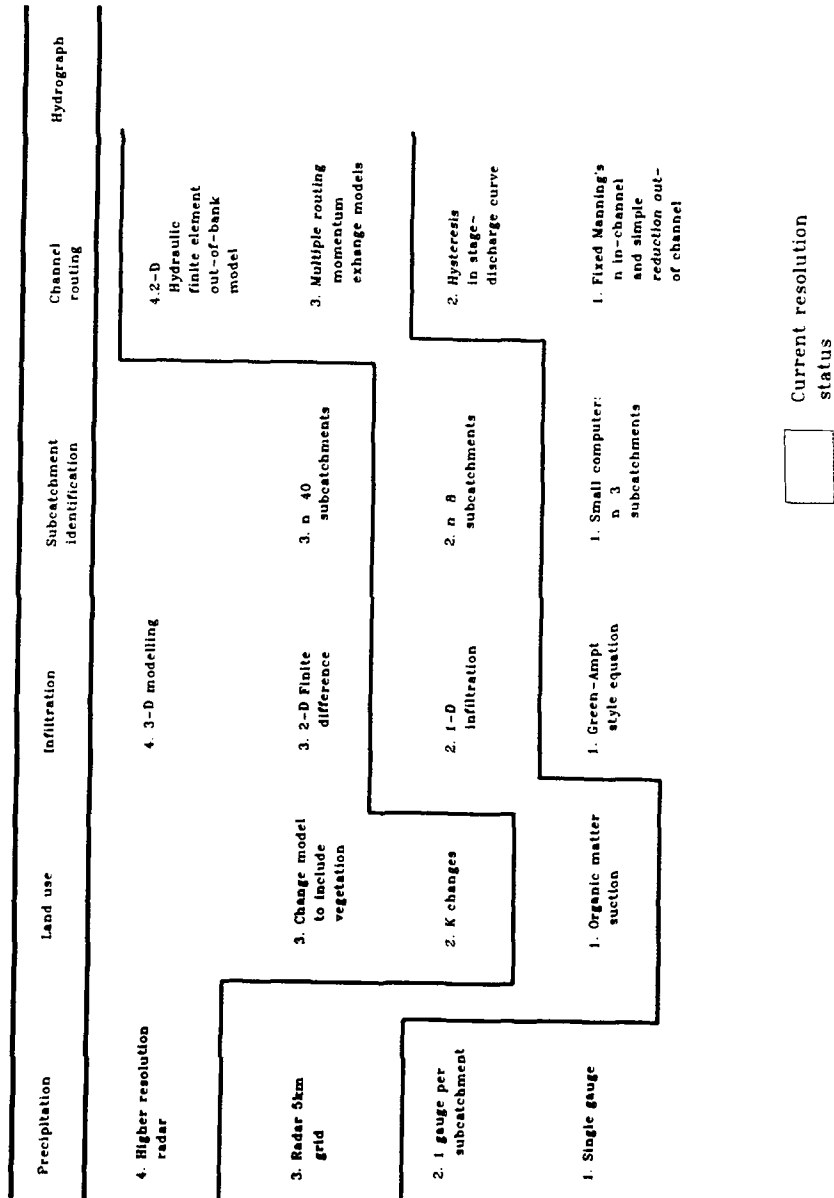


Figure 2.2
Composite Modelling Structure Incorporating Variability in Model Structure
 (after Anderson and Sambles, 1988)

Decision 1: submodel inclusion



Decision 2: submodel resolution

Figure 2.3
Composite Logic Structure for MILHY3

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with improved resolution in the channel routing.

Submodel resolution cannot of course be divorced from the resolution of user supplied information. Varying, perhaps unavoidable, resolution in the user supplied information may be considered, potentially at least, to have significant ramifications for submodel and overall model performance. Thus user supplied data resolution cannot, and should not, be divorced from model formulation, design and validation.

2.3 Out-of-bank Routing

Out-of-bank flows have highlighted in this research project an area which allows the investigation of several important issues. Firstly, as noted above, it allows comparison of modelling resolution in different process submodels within the MILHY scheme. Secondly, it extends the range of applications to which MILHY may be profitably applied and upgrades the out-of-bank channel modelling beyond any currently available alternative scheme. Finally, it allows investigation of the narrowing distinction between hydrologic catchment schemes and hydraulic reach models such as RMA-2V.

Table 2.1 summarizes the current handling of out-of-bank conditions in a range of hydrologic ungauged catchment models. The table shows that the resolution of the out-of-bank simulation is generally low and that channel modelling has been generally ignored in preference for more sophisticated modelling of the runoff generation processes.

2.4 Hydrologic and Hydraulic Schemes

It is proposed in this report that the traditional distinction between hydrologic models and engineering hydraulic schemes is no longer useful to the modeller, and that all modelling strategies

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must be considered within a suite of schemes. The hydrodynamic model RMA-2V is evaluated against the interests in modular program structures and out-of-bank routing noted above. Packages such as RMA-2V are taken to represent the state-of-the art in river reach modelling, and yet the utility of the schemes in both a catchment and ungauged context remains unevaluated. In this report the potential of RMA-2V is assessed for, firstly, extending "ground-truth" field data sets against which the MILHY3 scheme may be evaluated, and, secondly, as a modular component of a future MILHY scheme. This necessitates application of RMA-2V to reach lengths significantly longer than previous applications and with limited topographic data sets.

In the next chapter of this report, the physical behaviour of out-of-bank flows is assessed; this assessment determines the structure of the development and validation of the MILHY3 scheme. In Chapters 4 and 5, components of the MILHY3 model are developed and assessed. The model is then compiled and validated in Chapters 6, 7 and 8. Chapter 6 describes the establishment of the River Fulda test catchment, West Germany, whilst Chapter 7 uses the data set to test the performance of the new modules developed in Chapters 4 and 5. In Chapter 8, RMA-2V is applied to a reach of the River Fulda, to test its suitability for validating the MILHY3 scheme. Chapter 9 continues the assessment of the RMA-2V scheme for large-scale floodplain applications with a test reach of the River Culm, Devon, UK. In this chapter, the accuracy of the velocity vector and spatial inundation distributions of the RMA-2V model are assessed, using a more operational perspective of application. Chapter 10 concludes this volume, with operational guidelines for both MILHY3 and RMA-2V models. Future developments of the scheme are discussed in general.

Table 2.1
Hydrograph Simulators for Ungauged Catchments

Author	Model details	
	Parameter estimation	Routing/ two-stage capability
<u>Calibrated Models</u>		
NERC (1975)	Empirical regional formula	-
HEC-1 (1981)	Observed or regional catchment coefficients empirical soil parameters	Muskingham or Puls Separate Manning's n
USDAHL Nicks et al. (1980)	Maps and regional groundwater coefficients	Main channel
Stanford Watershed Model Ross (1970)	Optimized parameters for catchment	CSRX, FSRX Parameter for in-bank and out-of-bank routing
4 Parameter Water Yield Model Jarboe and Haan (1974)	Regression analysis	Main channel length, slope
<u>Physical Based Parameters</u>		
HEC-1 (1981)	Observed or regional parameters	Kinematic wave routing Fixed geometry
Engman and Rogowski (1974)	Observed soil and overland flow parameters	-
HYSIM Manley (1978)	Observed or regional groundwater parameters	
WATSIM Aston et al. (1980)	Observed and calibrated overland parameters	No channel routing

Table 2.1 (cont.)

TOPMODEL Beven (1977)	Observed or optimized parameters	CHA and CHB channel velocity
SPUR Renard <u>et al.</u> (1984)	All parameters derived from	No channel routing

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Chapter 3

Identification of Key Processes in
Downstream Conveyance in Two-Stage Channels

The need to improve the modelling of two-stage channels in ungauged catchments has been identified in Chapters 1 and 2. The objectives of undertaking an investigation into the modelling of two stage channels are:

- i) to investigate the importance of the resolution of modules in a composite modelling structure from the perspective of a model user
- ii) to investigate the relationship between module resolution and the effects of scale, in order to generate guidelines for the user
- iii) to incorporate hydraulic technique into a hydrologic modelling strategy
- iv) to potentially improve the predictive accuracy of MILHY2 in out-of-bank conditions
- v) to initiate the potential for the development of flood-plain inundation modelling in future research projects

The next step in the research strategy is to identify the key processes that control the behaviour of flow in two-stage channels and then consider how these may be incorporated in an ungauged model. In the next section, therefore, a review of the behaviour of flow in two-stage channels is reported and then a

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rationale for selecting certain processes that could profitably be modelled is developed.

3.1 Difficulties in Modelling Two-Stage Channels

As defined earlier, two-stage channels consist of a main channel and adjoining floodplain or berm which is subject to inundation. Water on the floodplains may be either stationary, when the floodplains act as stores of water, or flowing when the floodplain acts as a channel conveying water downstream.

3.1.1 The complexity of physical processes

A river is a complex three-dimensional system such that the inclusion of the floodplain system is not simply a matter of extending the cross-sectional area of the main channel. As the floodplains may act as both stores of water or flowing channels, it is inappropriate to simply extend the rating curve relationship from in-bank conditions.

Bhowmick and Demissie (1982) have shown the carrying capacity of a two-stage channel is not directly proportional to cross-sectional area. Figure 3.1 shows the relationship between the area ratio (floodplain area/total cross-sectional area) and the discharge ratio (floodplain discharge/total cross-sectional area) for a theoretical two-stage channel. The figure also shows a theoretical line of proportionality which identifies the relationship we would expect. If, for example, the floodplain area was half of that of the total area (area ratio = 0.5), then we might expect the floodplain to contribute half the discharge (discharge ratio = 0.5). However, the actual relationship shows that when the area ratio is 0.5, the discharge ratio is

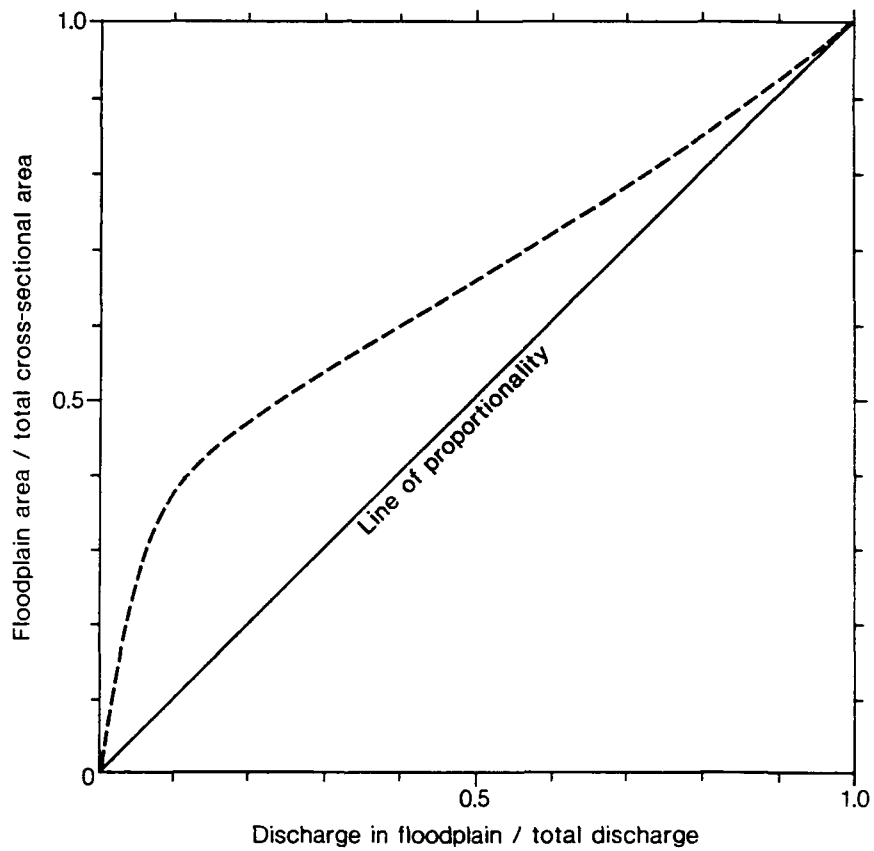


Figure 3.1
Relationship Between Floodplain/Total Area Ratio and Floodplain/Total
Discharge Ratio
(after Bhowmick and Demissie, 1982)

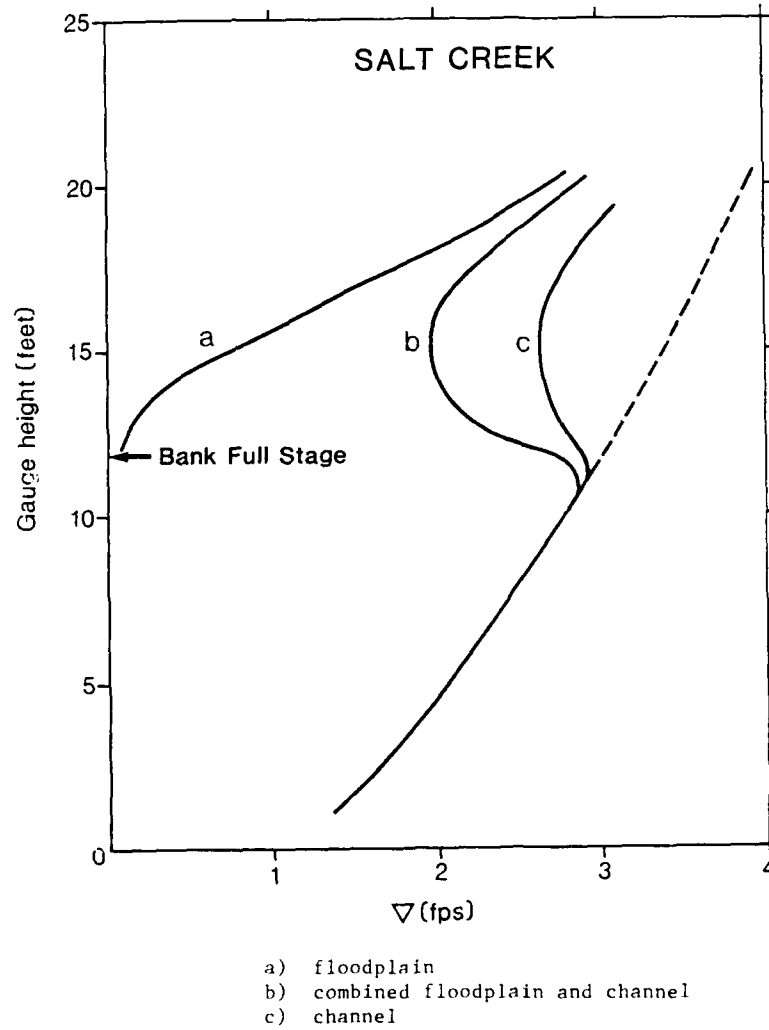
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actually 0.25. When the area ratio is less than 0.5, the discrepancy between the actual and theoretical is particularly large, so that as the area ratio increases the two lines converge. The figure shows that the two-stage channel system should not, therefore, be treated as a single channel system.

Bhowmick and Demissie (1982) found discontinuities in the velocity fields of the main channel and shear at the boundary between the channel and floodplain. Figure 3.2 shows the stage/velocity relationship in Salt Creek, USA, and illustrates the discontinuity of velocity in the two-stage channel. Both the floodplain and main channel show an overall increase in flow velocities as the depth of flow increases. This can be attributed to the reduction in the impact of boundary friction as depth increases. If the two-stage channel is treated as a single system, shown as the compound channel line on Figure 3.2, then the velocity decreases as out-of-bank conditions occur. The separate lines for the floodplain and main channel velocities show that the behaviour of the compound flow cannot be simply explained by the additional wetted perimeter and boundary friction of the floodplain. The velocity of the main channel flow also drops when out-of-bank conditions occur. Figure 3.2 shows that the main channel velocity reaches a local minimum when the floodplain inundation is approximately 35% of the main channel stage. As the floodplain inundation depth increases, then the velocity of the floodplain flow increases and the three lines of Figure 3.2 converge to a common velocity.

Research by Rajaratnum and Ahmadi (1979) confirmed these velocity patterns and showed discontinuity in the pattern of bed shear in the main channel and floodplain flow segments in a series of flume experiments. Together these results suggest

Figure 3.2
Relationship Between Stage and Velocity in a
Two-Stage Channel
(after Bhowmick and Demissie, 1982)



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that there is transverse mass transfer between the fast moving main channel flow to the slower moving floodplain flows. This would effectively retard flows in the main channel and accelerate flows in the floodplain. This momentum transfer may be envisaged to occur through the action of turbulent shear stresses, first recognised and photographed by Sellin (1964).

From Bhowmick and Demissie's (1982) research, it is possible to conclude, therefore, that the floodplain and main channel flows interact and the nature of this interaction varies with the depth of floodplain inundation.

So far, however, only the cross-sectional geometry of the two-stage channel has been considered. The plan geometry, however, is also important in the prediction of a flood hydrograph. Along a meandering channel the cross-sectional geometry of the two-stage channel will vary as the channel oscillates from one side of the floodplain to the other. This generates different longitudinal path lengths and downstream slopes for the floodplain and main channel. The downstream path length of the main channel is longer than its less sinuous floodplain and consequently the channel's slope is smaller. Fread (1976) suggested that such differences exacerbate the distinct pathways of the floodplain and main channel flows.

However, Toebe and Sooky (1967) showed in a series of flume experiments, that the momentum transfer between the floodplain and main channel is exacerbated where floodplain flows are not parallel to the main channel. This increase in the effects of momentum exchange would reduce the effects of the separate flood paths by accelerating floodplain flows and retarding main channel flows. As in most ungauged catchments the channel system meanders, it is important for the accurate prediction of the discharge hydrograph that the conflicting

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processes identified by Fread (1976) and Toebes and Sooky (1967) be resolved.

Chang (1983) showed that in a meandering river the energy expenditure is much greater than in a straight channel. The increase in energy expenditure can be associated with the generation of secondary currents usually at meander bends. Chang also showed that when the depth of flow is high, or when boundary frictions are small, the energy losses associated with the secondary current system can be greater than the energy losses associated with the main longitudinal flow. Chang's work suggests, therefore, that in the floodplain environment, where flow depths are relatively small and roughnesses high, even if a secondary current system were to develop it would be of little significance.

The literature shows that the processes active in a two-stage channel occur in the orthogonal and longitudinal dimension. Orthogonally the flows of the floodplain and main channel interact through turbulent stresses and these stresses are exacerbated when the floodplain and channel flows are not parallel. Longitudinally, floodplain flows tend to be less sinuous and therefore their downstream path length is shorter. The effects of this path length difference is increased by the steeper longitudinal slope of the floodplain flows. In both dimensions, the impact of boundary shear stresses on the wetted perimeter must also be considered.

In trying to rationally identify the most important of these processes which could realistically be modelled, one alternative would be to undertake a sensitivity analysis of an existing model which incorporated as many of the processes active in a two-stage channel as possible. The advantage would be that the time taken to identify the most important processes

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could be reduced and therefore more time would be available for the validation of the ungauged model. The disadvantage would be that the identification would be reliant on the success of the model used. However, it has been noted already in this report that the skills available to model the active processes in a catchment have been sufficiently developed to initiate a second stage of research, that is the linking of the most appropriate skills with particular applications. The originality of this report lies not in the development of process models from first principles, but the development of composite modelling structures from the perspective of the model user.

The potential of using existing models of two-stage channels for the identification of dominant processes is therefore accepted. Section 3.1.2 as a consequence investigates various modelling alternatives available for two-stage channels.

3.1.2 Modelling alternatives for two-stage channels

The handling of the channel and the potential for two-stage channel modelling in catchment models has already been summarized in Chapter 2, table 2.1. The review concluded that the modelling of two-stage channels in these catchment models was of a resolution no greater than the handling of the channel in MILHY2. In this section, alternative models specifically for channel flows are investigated.

Models of two-stage channels have come from the fields of both hydrology and hydraulics. Hydrologic approaches are limited to one and (quasi) two dimensional approaches, whilst hydraulic approaches include one, two and even prototype three dimensional models.

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One dimensional and quasi two dimensional approaches usually take one of three alternative approaches to the problem of the two-stage channel. These approaches are to:

- 1) Treat the channel/floodplain cross-section as a single system and average the boundary roughness and velocity differences between the flow segments, or
- 2) Treat the floodplain as an area for storing water only, or
- 3) Divide the cross-section into homogeneous segments of flow but do not consider the momentum transfers between these segments.

These one-dimensional approaches may be hydrologic or hydraulic. Hydrologic approaches tend to be incorporated as part of catchment models, whilst hydraulic channel models have been widely developed. Hydraulic techniques approximate the St. Venant equations of flow using either a kinematic or diffusion wave scheme.

Physically based two-dimensional models use the Reynolds equations to model the transfer of momentum between the segments of flow. These usually utilise either finite element or finite difference techniques and compute the fluxes between segments of flow using one of the three following techniques to quantify the fluxes:

- 1) Compute the force to provide equilibrium in each segment of flow (apparent boundary shear force).
- 2) Compute the effective friction factors for each segment.

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- 3) Compute the turbulence between a shear layer and the velocity profile (turbulence model).

There are no hydrologic models that incorporate the processes in the two-stage channel, although some empirical equations have been developed. Knight and Demetriou (1983) developed an empirical expression for the relationship between the shear stresses produced by the momentum exchange and the cross-sectional geometry, utilizing data from a series of flume experiments. This relationship has not been incorporated into a channel routing model.

3.2 Selection of a Two-Stage Conveyance Model

The objective of this section is to select from the model types identified in section 3.1.3, a model that could be applied to identify the most significant processes in the prediction of flow in two-stage channels. From section 3.1, it would seem that a successful two-stage conveyance model must incorporate:

- 1) Plan and cross-sectional geometries.
- 2) Momentum transfer between the floodplain and main channel flow segments.
- 3) Boundary friction differences between the floodplain and the main channel.

The most appropriate model for this application, however,

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must exhibit as many as possible of the following criteria:

- 1) the model should be physically-based or ideally not require calibration
- 2) the model should be validated
- 3) the model should be easy to understand and apply
- 4) the model should not require large amounts of data
- 5) the physically-based processes should be clearly defined and behave as independently as possible
- 6) from the model it must be possible to compute the discharge

State-of-the-art two-stage models incorporate complex physically-based finite element or finite difference schemes. The well documented schemes that exist, for example RMA-2V and EMBER, require extensive system knowledge by the operator, and demand large amounts of data. In addition, such models have not been applied to the scale of reach under investigation here (i.e. greater than 10km in length). The sophistication of these schemes, therefore, made them unsuitable for this type of application. A simpler approach was required.

Analysis of the physically-based one-dimensional schemes showed that there are no models that incorporate all three of the essential elements identified from the literature in section 3.1. However, an analytical scheme by Ervine and Ellis (1987) was identified as incorporating two of the essential physical

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requirements given. Further, it met most of the modelling requirements.

The Ervine and Ellis scheme incorporates the effects of plan and cross-sectional geometry and the effects of boundary friction in a meandering two-stage channel. It does not, however, attempt to incorporate the effects of momentum exchange between the segments of flow. However, this is an area which has been and still is being intensively investigated by, for example, Knight and Hamed (1984), Myers (1987) and Prinos et al. (1985). As there are a great number of papers on strategies for incorporating momentum exchange, it was felt that it would be reasonable to include the exchange of momentum as an important process per se, and utilise the Ervine and Ellis scheme to investigate other active processes.

3.2.1 The Ervine and Ellis model

Ervine and Ellis' model allows a meandering plan geometry to be modelled by dividing flow into three segments, shown on Figures 3.3a and b, which can be defined as:

- 1) Main channel flow
- 2) Floodplain flow contained within the meander belt of the main channel
- 3) Floodplain flow outside of the meander belt

For each segment, the energy loss is computed and hence the mean velocity for each segment and the discharge total are also calculated. Ervine and Ellis (1987) firstly identified the main sources of the energy loss in each flow segment and

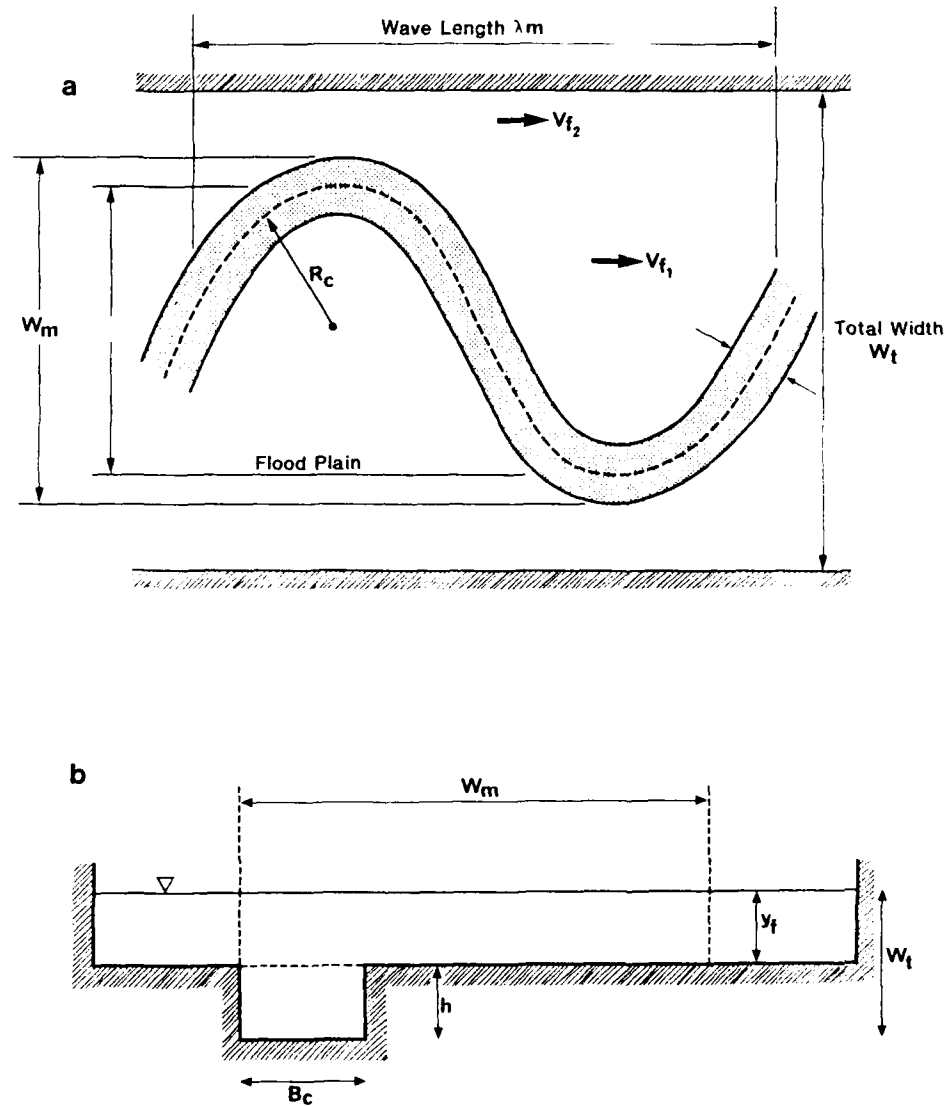


Figure 3.3
Definition of Two-Stage Channel Geometry for
Ervine and Ellis Model, 1987
 a) plan geometry definition
 b) cross-sectional geometry
 definition

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then brought together a series of geometric and frictional relationships to describe them.

Main Channel Energy Losses

Ervine and Ellis considered there to be four possible sources of energy loss in the main channel:

- 1) Frictional losses at the boundaries of flow
- 2) Transverse currents (secondary currents) at meander bends
- 3) Turbulent shear stress (momentum transfer to the floodplain)
- 4) Pool/riffle sequences causing head losses at low flows

They chose to omit the turbulent shear stresses and pool/riffle losses in their computations. Shear stresses were omitted because three-dimensional interpretations of established techniques (e.g. Knight and Demetriou, 1983), are still under investigation by Willets (see Ervine and Ellis, 1987). Pool/riffle losses are considered less important in times of overbank flow, when bed form effects are usually flooded out.

Floodplain Energy Losses

Two sources of loss were identified by Ervine and Ellis:

- 1) Frictional losses at the boundaries.

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- 2) Expansion and contraction losses, shown in Figure 3.4, where flow orthogonal to the main channel suddenly expands as it drops into the channel, and contracts as it re-enters the floodplain region.

3.2.2 Quantifying the Energy LossesMain Channel Energy Losses1. Friction

Head losses due to friction are computed over a meander wavelength ($r\lambda_m$) as:

$$h_f = \frac{f_c}{4} \cdot \frac{r\lambda_m}{R} \cdot \frac{V_c^2}{2g} \quad 3.1$$

where f_c is the Colebrook-White friction factor, given by:

$$\frac{1}{\sqrt{f_c}} = -2 \log \left(\frac{K_c}{14.8R} + \frac{2.51}{R_e \cdot (f_c)^{1/2}} \right) \quad 3.2$$

2. Transverse Currents

Head losses due to secondary currents at meander bends are computed using a simplified method developed by Chang (1983). Chang used a mean transverse current velocity because over a meander amplitude, the velocity varies from a maximum at the apex to a theoretical zero at the cross-over thalweg. Chang ignored the effects of superelevation, where the centrifugal forces cause the water level on the outside of the bend to be higher than those of the inside. Yen (1967) showed that in two-stage flow, superelevation effects are suppressed by the

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head of water above the main channel.

Head loss due to transverse currents computed over a meander wavelength is given by:

$$h_1 = \frac{2.86 (f_c)^{1/2} + 2.07 f_c \cdot \left(\frac{R}{R_c} \right)^2}{0.565 + (f_c)^{1/2}} \cdot \frac{r \lambda_m}{y_c} \cdot \frac{v_c^2}{2g} \quad 3.3$$

Floodplain within the Meander Belt Width

1. Friction

As in the main channel, the total frictional head loss along a meander wavelength is described by:

$$h_f = \frac{f_{fl} \cdot l}{4 y_f} + \frac{v_{fl}^2}{2g} \cdot (W_m \cdot \lambda_m - r \lambda_m \cdot B_c) \quad 3.4$$

where the last term is the wetted area.

2. Expansion Losses

$$\text{Assuming: } y_c \approx y_f + h \quad 3.5$$

the head loss due to expansion of floodplain flow into the main channel over a meander wavelength is given by:

$$h_e = \frac{r \lambda_m}{y_c} \cdot l - \frac{y_f^2}{2g} \cdot \frac{v_{fl}^2}{2g} \cdot \sin^2 \bar{\sigma} \quad 3.6$$

where $\bar{\sigma}$ is the mean average angle of the floodplain flow to the main channel over a meander length.

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3. Contraction Losses

The total head loss due to the contraction of flow as it rejoins the floodplain segment from the main channel (illustrated in Figure 3.4), is given by:

$$h_c = C_L \frac{v_{fl}^2}{2g} \cdot \sin^2 \bar{\sigma} \cdot (r\lambda_m) \quad 3.7$$

where C_L is a loss coefficient, generated by Yen and Yen (1984); and is a function of:

- i) the density, specific weight and kinematic viscosity of the flow,
- ii) meander wave length and amplitude, the mean angle of incidence of floodplain flow in the main channel, the valley width, valley slope, floodplain roughness, and width and depth of the main channel, and
- iii) discharge and slope.

Yen and Yen (1984), using data collected from flume experiments computed the total loss coefficient after flow had been subjected to expansion and contraction. Then assuming:

$$C = C_E + C_L \quad 3.8$$

where C = total loss coefficient

C_E = loss coefficient due to expansion

C_L = loss coefficient due to contraction

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and

$$C_E = \left(1 - \frac{y_f}{y_c}\right)^2 \quad 3.9$$

the contraction loss coefficient was computed. This is shown in Table 3.1

Yen and Yen (1984) considered that the coefficients shown in Table 3.1 should be treated as upper limits because the channel sidewalls in their flume experiments were vertical rather than a more realistic trapezoidal shape.

Floodplain Flow outside the Meander Belt1. Friction

In the floodplain segment outside the meander belt, flow is considered to be uniform, so that the friction slope is given by:

$$S_o = \frac{f_{f2} \cdot 1}{4} \cdot \frac{V_{f2}^2}{y_f \cdot 2g} \quad 3.10$$

Combining all the head loss equations, Ervine and Ellis (1987) obtained:

1) for the main channel

$$\frac{f_c \cdot r\lambda_m \cdot V_c^2}{4 \cdot R \cdot 2g} + \frac{2.86(f_c)^{1/2} + 2.07f_c}{0.565 + (f_c)^{1/2}} \cdot \left(\frac{R}{R_c}\right)^2 \cdot \frac{r\lambda_m \cdot V_c^2}{R \cdot 2g} = S_o \lambda_m \quad 3.11$$

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Table 3.1
Contraction Loss Coefficients
(after Yen and Yen, 1984)

y_f / y_c	C_L
0.0	0.5
0.1	0.48
0.2	0.45
0.3	0.41
0.4	0.36
0.5	0.29
0.6	0.21
0.7	0.13
0.8	0.07
0.9	0.01
1.0	0.0

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2) for the floodplain inside the meander belt

$$\frac{f_{f1}}{4} \cdot \frac{1}{y_f} \cdot \frac{V_{f1}^2}{2g} \cdot (W_m Y_m - \lambda_m B_c r) +$$

$$r \lambda_m \cdot \frac{V_{f1}}{2g} \cdot \sin^2 \bar{\sigma} \cdot \left(\left(1 - \frac{y_f}{y_c} \right)^2 + C_L \right) = S_o \lambda_m W_m \quad 3.12$$

3) for the floodplain outside the meander belt

$$\frac{f_{f2}}{4} \cdot \frac{1}{y_f} \cdot \frac{V_{f2}^2}{2g} = S_o \quad 3.13$$

4) total discharge

$$Q = V_c (B_c h) + V_{f1} \cdot y_f \cdot W_m + V_{f2} \cdot y_f (W_t - W_m) \quad 3.14$$

3.3 Sensitivity Analysis of the Ervine and Ellis Scheme

The objective of undertaking a sensitivity analysis of the Ervine and Ellis model was to identify the physical processes controlling the velocity and discharge predictions. Once identified, the most appropriate method of incorporating these processes into the MILHY2 scheme can be investigated. A sensitivity analysis of the scheme has not previously been undertaken and this investigation therefore presents the opportunity to assess the scheme.

3.3.1 Sensitivity analysis design

Analysis of equations 3.1 to 3.10 and 3.14 identified five groups of physically based parameters which control

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the processes identified and modelled by Ervine and Ellis.
These five groups are:

- 1) slope
- 2) plan geometry - channel width, floodplain width,
meander belt width and radius of curvature
- 3) depth of flow - channel and floodplain segments
- 4) sinuosity - sinuosity and angle of inclination of
floodplain flow to the main channel
- 5) friction - for consistency with MILHY the sensitivity
of the model to Manning's n was used, utilizing the
conversion from Colebrook-White f given below:

$$f = \frac{8gn^2}{R^{1/3}} \quad 3.15$$

Each of these five groups was investigated individually by varying each one by a systematic, 30% and 5% reduction and 5% and 30% increase in parameter values.

As the objective of this analysis was to identify processes for further investigation, it was decided to apply the model to a single reach.

The structure of analysis, varying each of the five parameter groups four times, generated 30 simulations, each with the geometry varying in some way. It was felt that to provide statistically meaningful variations, a much larger number of simulations would be required, and this would not necessarily

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improve the identification of the most important processes in the scheme.

The model was applied to a reach for which the parameter values were generated from a reach on the River Fulda, West Germany. A reach on the River Fulda was selected as the catchment was being established for the validation of the completed MILHY3 model. For the analysis of the Ervine and Ellis scheme, the exact geometry of the reach is not significant. In Chapters 4 and 5, a cross-section and reach from the River Fulda catchment will be similarly utilised. A description of the River Fulda catchment and the reason behind its selection is reported in Chapter 6. The initial parameter values are shown in Table 3.2 where the velocity predictions for each flow segment and discharge total generated by the Ervine and Ellis scheme are also given. Observed stage/discharge relationships from the River Fulda show that the scheme gives realistic results, the discharge predictions being less than 10% out.

3.3.2 Results

The results from the sensitivity analysis are tabulated in Tables 3.3 to 3.7, and show the percentage deviation from the computed values tabulated in Table 3.2. Below is an analysis of the velocity predictions by considering each of the sources of head loss identified by Ervine and Ellis in turn:

(a) Frictional losses are modelled in all three flow segments and Tables 3.3, 3.4 and 3.5 show that variation in the frictional parameter values cause the largest variation in the predicted velocity of the five parameter groups. However, in the main channel the Darcy-Weisbach friction factor is also linked to the modelling of the transverse (secondary) circulation. From the first term in equation 3.3, it can be

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Table 3.2
Parameter Specification for a Hypothetical Reach

	<u>SI units</u>
Bed slope	0.0007
Sinuosity	1.3
Hydraulic radius	2.5
Radius of curvature	125.0
Width of meander belt	175.0
Total floodplain width	300.0
Channel width	30.0
Friction channel (f_c)	0.071
Friction floodplain 1 (f_{f1})	0.356
Friction floodplain 2 (f_{f2})	0.356
Channel depth	3.5
Floodplain depth	0.5
Angle of floodplain flow to channel (radians)	0.785
Contraction loss coefficient	0.47

Results

Main channel, velocity	1.205
Floodplain 1, velocity	0.360
Floodplain 2, velocity	0.278
Discharge	157.2

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seen that as the friction factor decreases, head losses from the transverse currents decrease, and when the friction factor increases the head losses are increased. Therefore, the velocity variations shown in Table 3.3 incorporate both friction head losses and transverse circulation losses.

(b) The transverse circulation in the main channel can be attributed to the friction (as noted above) and the ratio of the hydraulic radius to the radius of curvature. This ratio is included in the geometry variation reported in Table 3.3, which shows that the velocity predictions are not sensitive to geometric variation in the channel. As noted above, however, the model is sensitive to the frictional aspects of the transverse circulation.

(c) Sinuosity changes generate significant variability in the channel velocity results (see Table 3.3). From equation 3.11, it can be seen that the sinuosity term is used to calculate channel length in both the frictional head losses and transverse circulation computation. For the main channel, therefore, the model can be interpreted as being sensitive to channel length.

On the floodplain within the meander width belt (area 1), Table 3.4 shows the velocity predictions are not sensitive to sinuosity variations. From equation 3.12 it can be seen that sinuosity is utilized to compute the flow path length and the angle of incidence of floodplain flow to main channel flow, which itself is used in the computation of the expansion and contraction head losses. From Table 3.4, it would seem reasonable to conclude that, because of the linear flow path of the floodplain flow, the velocity predictions are not affected by the length of the path, and it is not necessary, therefore, to include the angle of incidence of floodplain flow in the

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Table 3.3
Channel Velocity Results
 (% deviation from origin velocity)

% Change in variable	Decrease 30%	Decrease 5%	Increase 5%	Increase 30%
Slope	-19	-2	+3	+13
Friction	+50	+5	-4	-24
Geometry	-5	-0.5	+1	+3
Sinuosity	+20	+3	-11	-12

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Table 3.4
Floodplain 1 Velocity Results
(% deviation from origin results)

% Change in variable	Decrease 30%	Decrease 5%	Increase 5%	Increase 30%
Slope	-19	-2	+2	+13
Friction	+23	+5	-4	-27
Geometry	-4	-0.5	+1	+2
Sinuosity	+1	0.0	0.0	-1
Contraction coefficient	0.0	0.0	0.0	0.0

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Table 3.5
Floodplain 2 Velocity Results
(% deviation from origin results)

% Change variable	Decrease	Decrease	Increase	Increase
	30%	5%	5%	30%
Slope	-19	-2	+3	+13
Friction	+25	+5	-5	-28

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modelling of expansion and contraction head losses. Table 3.4 also shows that the exact value of the Yen contraction loss coefficient need not be of concern to the modeller.

(d) The effects of slope variations on velocity predictions were only significant where variations were large ($\pm 30\%$), as can be seen in Tables 3.3, 3.4 and 3.5. For the purposes of this sensitivity analysis the frictional slope (S_o) was assumed to be parallel to the bed slope, S . Hence uniform flow conditions were assumed. The effects of the different longitudinal slopes of the main channel and floodplain were not directly included in the Ervine and Ellis scheme.

(e) The impact of variation in the depth of flow on the floodplain velocity results is shown in Table 3.6. Equation 3.11 shows that the main channel depth is incorporated in the velocity computation as hydraulic radius, and the analysis of intermediate computations in the analysis shows it is the frictional head loss computation to which velocity results are sensitive.

On the floodplain within the meander width belt, equation 3.12 shows it is the ratio of the floodplain to channel depth, that is utilized to compute the expansion and contraction head losses. However, the velocity predictions for floodplain area 2 are identical to those in floodplain area 1, and as the head loss in area 2 is entirely attributable to frictional losses (see equation 3.13), it would seem that the velocity variations in area 1 are due to the same frictional effects and not due to expansion and contraction losses.

Tables 3.6 and 3.7 illustrate the effects of variability in the five parameter groups on the discharge predictions

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Table 3.6
Flow Depth Effects on Velocity and Discharge
 (% deviation from origin results)

Depth		Channel Velocity	Floodplain Velocity		Discharge
y_f	y_c		Area 1	Area 2	
0.33	2.31	-12	-33	-23	-29
0.475	3.325	-2	-20	-20	-19
0.525	3.675	+4	-15	-16	-16
0.665	4.635	+15	-5	-5	-4
0.33	3.5	-	-18	-19	-14
0.475	3.5	-	-2	-2	-2
0.525	3.5	-	+2	+3	+2
0.665	3.5	-	+15	+15	+16

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Table 3.7
Discharge Results
(% deviation from origin results)

% Change in variable	Decrease 30%	Decrease 5%	Increase 5%	Increase 30%
Slope	-19	-2	+3	+13
Channel friction	+35	+4	-3	-17
Floodplain friction	+15	+2	-1	-9
Geometry	-28	-4	+4	+27
Sinuosity	+14	+2	-8	-9

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computed using equation 3.14. Geometry is the only group to create additional influence on the discharge predictions, over those already identified in the velocity results reported above. The geometry variables effectively weight the velocity results for each flow segments based on their cross-sectional area, to give the total discharge.

3.3.3 Conclusions

From the analysis of the results above, it is possible to make a number of conclusions:

- 1) The Ervine and Ellis scheme is highly sensitive to the Darcy-Weisbach friction factor.
- 2) The model is sensitive to the depth of inundation (incorporated in the computation of frictional head losses) in all flow areas.
- 3) The sinuosity of the main channel is important in determining the length of the flow path and hence time to peak in a hydrograph.
- 4) The incorporation of head losses due to expansion and contraction of floodplain flow as it crosses the the main channel is best achieved through the friction head loss computation.

3.4 Implication for the Development of MILHY2

The results of the analysis of the Ervine and Ellis scheme isolate friction as being the single most important factor in the prediction of discharge in two-stage channels. Friction is

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identified, therefore, as being the key to improving the channel routing model in MILHY2. The analysis showed that the handling of frictional head losses can successfully incorporate both boundary roughness effects and effects of transverse currents in the meandering channels. The second area worthy of investigation is the impact of the relatively longer, sinuous path length of the main channel over the floodplain path length.

Three key areas that need further investigation have, therefore, been identified. These are:

- i) Improvement of the handling of friction to incorporate boundary roughness and transverse circulation within the main channel.
- ii) Incorporation of turbulent shear stresses between the main channel and floodplain flow segments.
- iii) The adoption of different path lengths for main channel and floodplain flow areas thereby incorporating sinuosity.

3.4.1 Present frictional capability of MILHY2

Friction is incorporated in the MILHY2 utilizing the Manning's n coefficient. In selecting the most appropriate n value, Chow (1959) identified ten factors which should be considered. These are:

- i) surface roughness
- ii) vegetation
- iii) channel irregularity
- iv) channel alignment
- v) silting and scouring

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- vi) obstructions
- vii) size and shape of channel
- viii) stage and discharge
- ix) seasonal change
- x) suspended material and bed load

The list here shows that the Manning's n coefficient incorporates the frictional effects identified in the analysis above as being important in the analysis of two-stage flow. However, it is impossible to select one Manning's n value that can represent the frictional conditions at all times during the passing of the floodwave through a two-stage channel.

At present, the only additional complexity to the Manning's n handling of friction incorporated in MILHY2, is an algorithm (equation 3.16) that reduces the coefficient value with increasing stage:

$$n' = n - 0.0025R \quad 3.16$$

If the dominant process active in the channel is boundary roughness then this algorithm will improve the prediction of the carrying capacity of the cross-section. In the main channel as stage increases, the cross-sectional area of flow generally increases more rapidly than the wetted perimeter, thus reducing the retarding effects of boundary friction, (SCS, 1954). On the floodplain too, Manning's n may decrease as the depth of inundation increases and the frictional effects of vegetation become less significant. Table 3.8, taken from Chow (1959), illustrates this frictional decline with increasing stage for pasture and meadows.

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Table 3.8
Manning's n values for pasture and meadow floodplains
from Chow (1959)

Depth of inundation feet	Manning's n value	
	Pasture	Meadow
< 1	0.05	0.10
1-2	0.05	0.08
2-3	0.04	0.07
3-4	0.04	0.06
> 4	0.04	0.05

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Petryk and Bosmajian (1975) showed, however, that this is an over-simplification of the frictional effects of vegetation. They accept that when the vegetation is totally submerged the boundary frictional effects will decrease with increasing stage. However, when the inundation depth is below the top of the vegetation, there is a complex relationship between vegetation density and Manning's n . Petryk and Bosmajian (1975) used equation 3.17 to calculate the change in n with depth.

$$n = n_b \sqrt{1 + \frac{C_d \sum A_i}{2gAL} \left(\frac{1}{n_b} \right)^2 R^{4/3}} \quad 3.17$$

where n_b = Manning's n value with vegetation effects
 C_d = vegetation drag coefficient dependent on vegetation type
 L = length of reach
 A_i = projected area of i^{th} plant
 A = cross-sectional area of flow

Petryk and Bosmajian's (1975) scheme would not be suitable for the ungauged catchment because of the spatially detailed data it would require. However, the scheme does expose the inability of a single Manning's coefficient value to be selected for all stages if the effects of vegetation are considered.

The Manning's n reduction algorithm, equation 3.16, utilized in MILHY2 similarly only considers the effects of boundary friction in developing a relationship between n and stage. In the two-stage channel the boundary friction effects may not be the dominant frictional effect. Pasche and Rouve (1985) suggest that the eddies generated at the main channel/floodplain interface have a much greater frictional effect than boundary friction.

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Investigation of the present frictional capability of MILHY2 has exposed the inadequacy of the routine to incorporate the frictional effects of the dominant processes of two-stage flow. In addition, application of MILHY2 to two-stage flow conditions exposed a situation where the hydraulic radius became so great that the frictional algorithm (equation 3.16) generated negative discharge predictions. An example of this phenomenon is given in Table 3.9. In two-stage flow conditions, therefore, this frictional algorithm has been removed from the computation of the channel capacity.

Having exposed the inability of the present frictional capability of MILHY2 to incorporate the processes identified as being important in two-stage flow, the next stage in the research project is to develop a strategy for the incorporation of two-stage flow processes.

3.4.2 Strategy for the development of HYMO2

The analysis undertaken in this chapter has identified two areas of investigation that could improve the predictive capabilities of MILHY2. These are:

1. Incorporation of Turbulence

Two significant sources of turbulence have been identified by the analysis in this chapter. These are:

- i) apparent shear stresses between the floodplain and main channel flow segments, and
- ii) transverse circulation stresses generated in meanders of the main channel (also known as secondary currents)

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Table 3.9
Rating Curve Computation for a Wide
Cross-sectional Geometry

Water Surface elevation m	Flow Area m^2	Flow Rate $m^3s^{-1} (x10^3)$
1.0	8.0	0
2.2	27.8	0.03
3.3	53.3	0.09
4.4	95.7	0.2
5.4	381.5	0.3
6.5	783.8	1.8
7.6	1200.4	7.3
8.7	1679.6	1009
9.8	1206.3	-1.5
10.9	2841.2	-9.3

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The relative importance of these two sources and the interaction between them is not clear from the literature, although it does seem to be dependent on the depth of the flow on the floodplain. However, the analysis of the Ervine and Ellis (1987) scheme suggested that the effects of transverse currents at meander bends could be incorporated in the modelling of boundary friction, whilst Chang (1983) suggests that the effects of transverse currents are suppressed in out-of-bank conditions. This report concentrates, therefore, on the shear stresses between the main channel and the floodplains.

2. Incorporation of Multiple Routing Pathways

Both the analysis of the Ervine and Ellis scheme and work done by Fread (1976) suggest that the different path lengths of the main channel and floodplain flows will affect the timing of the floodwave travelling downstream. At present MILHY2 models a single pathway for the floodplain and channel flows using a mean reach length and travel timetable. A priority, therefore, is to investigate alternative methods of incorporating multiple routing pathways of flow through the reach.

Having identified the process areas that can be most profitably investigated, it is important that a strategy for the implementation of the investigation is developed. The overall objectives of this report, outlined in Chapter 1, include the validation of any new modules.

The importance of a well structured validation or model evaluation procedure cannot be under-stated. Miller et al. (1976) pointed out that model evaluation procedures have usually been postponed until the stage has been reached when observed and simulated output can be compared. The danger of leaving evaluation to this late stage is that a large amount of time and

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resources have been invested in model development and the friction in any radical change in methodology might be too great.

Sargent (1982) generated a three-stage model evaluation programme illustrated in Figure 3.5, incorporating mathematical validation, model verification and operational validation. Sargent's scheme shows the importance of model evaluation to all stages of development; from the conceptual development and mathematical description of the system to the computer coding and performance of the completed model.

This report attempts to incorporate Sargent's evaluation scheme in the development of a composite modelling strategy utilising MILHY2. The new modules that will be developed in Chapter 4 and 5 increase the resolution of the channel routing component, will be evaluated individually before the new programme, MILHY3, is put together. This individual model evaluation will include critical assessment of the mathematical techniques selected and the use of observed and hypothetical data sets to judge the performance of the module.

MILHY3 will be subject to further evaluation investigations described and developed in Chapter 6 and analysed in Chapters 7 and 8.

An approach and an area for this research project have now been defined. The approach of the project is to develop a model suitable for flood forecasting in ungauged catchments from the perspective of the potential user, but utilizing a composite modelling structure. The research area that has been identified is the introduction of a two-stage behaviour.

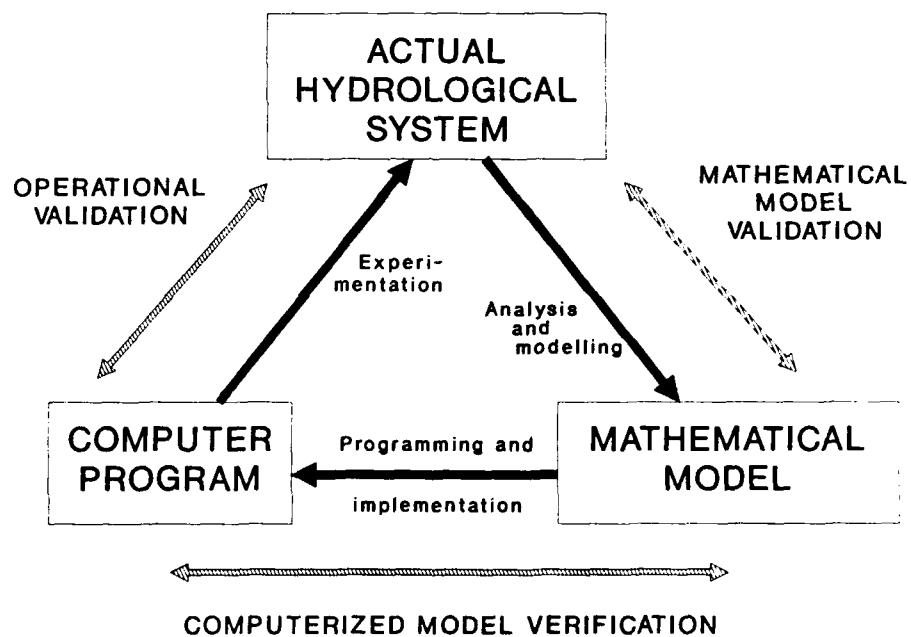


Figure 3.5
Three Stage Model Evaluation Programme
(after Sargent, 1982)

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The development of a composite modelling structure has highlighted the importance of module resolution in the establishment of operational guidelines for the potential user of the MILHY3 scheme. The objective of the rest of this report is to explore the relationship between module resolution and model predictive accuracy and performance.

3.5 Summary

This chapter has discussed the behaviour of flow in two-stage channels and alternative methods of identifying the dominant processes in the system. A model developed by Ervine and Ellis (1987) was identified as incorporating the cross-sectional and plane geometry of two-stage channels. This model was then subjected to a sensitivity analysis to help identify the most important factors influencing flow prediction in two-stage channels.

Analysis of the results from the Ervine and Ellis simulations showed that friction is the most important process controlling cross-sectional flood prediction. The components of the friction were then identified as being boundary friction effects and turbulent eddies. The turbulent eddies are generated from the interaction of channel and floodplain flows and at meander bends. The modelling of the turbulent interaction of channel and flood flows was selected as being the most profitable as it is an area ignored by other models. An investigation into the turbulence generated at the floodplain/channel interface is reported in Chapter 4.

The analysis of the Ervine and Ellis scheme also identified the importance of the differing path lengths of channel and floodplain in the prediction of the timing of the flood hydrograph. The incorporation of such multiple routing pathways is investigated in Chapter 5.

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The importance of a structured model evaluation strategy has been stressed and Sargent's (1982) three-stage programme of development will be followed. This programme includes the individual evaluation of the turbulent exchange and multiple routing modules, prior to the evaluation of the new MILHY3 model.

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Incorporation of Momentum Transfer between
Floodplain and Channel Segments

The transfer of momentum between the main channel and floodplain flow segments was identified in section 3.4 as a process which it was felt would, if incorporated into MILHY2 downstream routing scheme, make a significant improvement to the overall predictive capability of MILHY2. The objective of the work reported in this chapter is, therefore, to investigate, implement and validate a method of incorporating momentum transfer between flow segments whilst retaining MILHY2's parsimonious data requirements. In the first section, therefore, the process of momentum transfer is investigated.

4.1 The Hydraulics of Momentum Transfer

In two-stage channels, the irregular cross-sectional geometry of the deep channel, and its associated shallow floodplains, generate higher velocities in the main channel than those in the floodplain flow segments. This is due to the relatively greater depth of flow and smaller wetted perimeter of the main channel in comparison with the floodplain. Figure 4.1 illustrates the velocity isovels (lines of equal velocity) for a two-stage flume experiment conducted by Knight *et al.* (1983). The velocity isovels are dimensionless parameters because the observed values are divided by the mean velocity for the cross-section, where $V=Q/A$. Figure 4.1 shows that maximum main channel velocities are at least 25% greater than the average velocity, whilst floodplain velocities are as

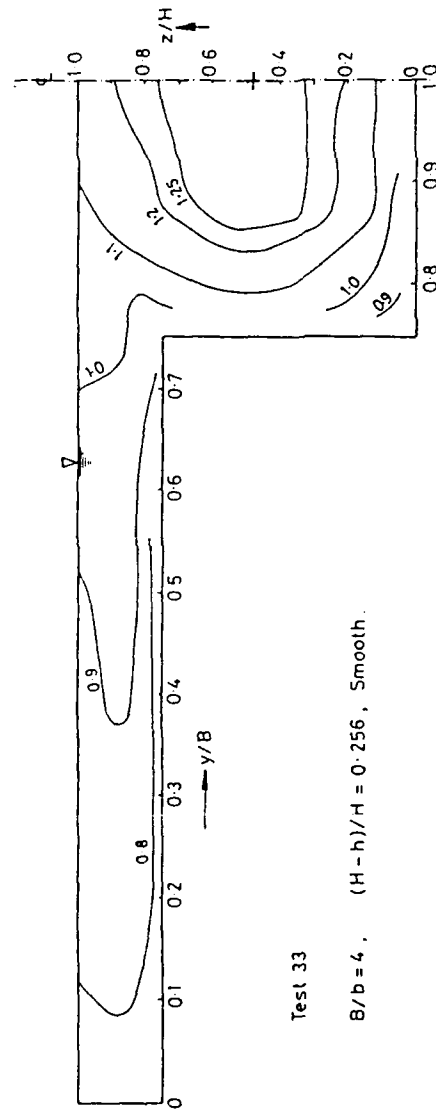


Figure 4.1
Velocity Isovels Generated from a Two-Stage Channel
Experiment Conducted by Knight et al. (1984)

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low as 70% of the average. Figure 4.1 also illustrates the distribution of the velocity isovels, with maximum channel velocities occurring in the centre of the channel away from the influence of the floodplain. In contrast, the maximum velocities in the floodplain occur close to the main channel, and velocities decrease with increasing distance from the channel. The difference in the flow velocities between the main channel and floodplain cause a transfer of longitudinal momentum generally from the main channel to the floodplain.

There are four physical mechanisms by which linear momentum can be transported perpendicular to the direction of flow. Wright and Carstens (1970) ranked these processes on a scale of one to four in the order of their effectiveness of transporting momentum, the first being the most effective. Their ranking is:

- i) transverse circulation stresses (secondary currents)
- ii) eddies generated in the mixing zones of stream tubes of differing velocities
- iii) eddies generated by flow along a boundary
- iv) molecular motion

Figures 4.2a and 4.2b illustrate the first three processes, that is, secondary currents, the eddies generated in the mixing zones and eddies generated by flow along a boundary. Figure 4.2a shows that the eddies generated by flow along a boundary are contained within the main channel and that the eddies generated by mixing zones are positioned at the main channel/floodplain interface. Figure 4.2b shows the secondary

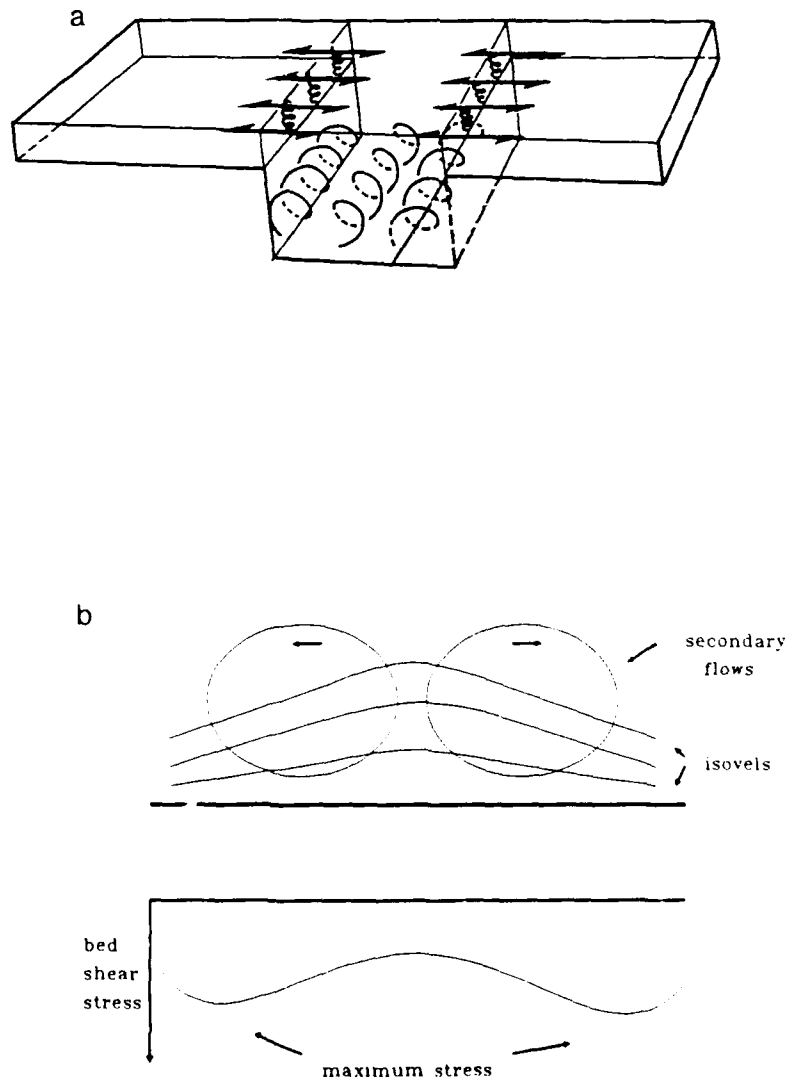


Figure 4.2
Mechanisms by which linear momentum may be transported
perpendicular to the direction of flow

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currents generated in a meandering main channel. The centrifugal forces in a meander bend cause the flow to be drawn to the outer bank of the main channel. This generates a cell orthogonal to the longitudinal flow; a second cell near the outer bank may also develop. The direction of the circulation of these secondary cells varies according to the relative position of the cells within a meander wavelength. Keller and Melhorn (1973) show that the cells diverge at the surface in riffle sections and converge at the surface in pool section. In addition, Hey and Thorne (1975) have shown that at the apex of the meander bend the cells may be asymmetric, with the strong centrifugal forces suppressing the cell nearest the outer bank.

As suggested earlier in section 3.4, it is not known which of these processes is dominant in two-stage channels. The analysis of the Ervine and Ellis (1987) scheme identified the first two of these processes as being important in the two-stage channel but did not account for the other two processes. In theory it would seem that the eddies generated by the mixing of differing velocity tubes must be greater in a two-stage channel than in a single channel system because the cross-sectional velocity gradient is greater. It would also seem logical that the secondary current eddy system would be suppressed by the head of water in over-bank flow in much the same way as the effects of superelevation are suppressed (Yen, 1967). The importance of boundary friction eddies and molecular transport transfer is also not clear. However, in view of the fact that Wright and Carstens (1970) ranked these two processes as being less efficient at the transfer of momentum than the first two processes, they can for the moment be considered as insignificant in momentum transfer in two-stage channels.

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It is accepted in this report, therefore, that the dominant control over the transfer of momentum between the main channel and the floodplain is the mixing eddies caused by the velocity gradient. If the validation procedure highlights the importance of momentum transfer in the improvement of MILHY2's flood prediction accuracy, then the importance of the other three processes of momentum transfer would need to be reconsidered.

A great deal of research on the transfer of momentum in two-stage channels has been carried out in the last twenty-five years, starting with Sellin (1964) and Zheleznyakov (1965). Sellin (1964) was the first to identify turbulence at the interface between main channel and floodplain by photographing the vortices generated by turbulence in a flume-based study. Zheleznyakov found in both flume (1965) and field experiments (1971) that the momentum transfer mechanism decreased the overall rate of discharge for floodplain depths just over bankfull.

Radojkovic (1976) identified the dependence of the shear stress on the velocity profile in two-stage channels, while flume studies by Knight (1989) have shown that the momentum transfer distorts the shear stress profiles on the bed of the main channel and floodplain. Most noticeable was the increase in the shear stresses on the floodplains near the junction with the main channel.

The distribution of the shear stress in two-stage channels is significant as the pattern provides a means of visualising and interpreting the behaviour of the momentum exchange process. Figure 4.3 illustrates a typical shear stress distribution measured in a flume-based experiment conducted by Knight and Lai (1985). By analysing this distribution, an interface can be

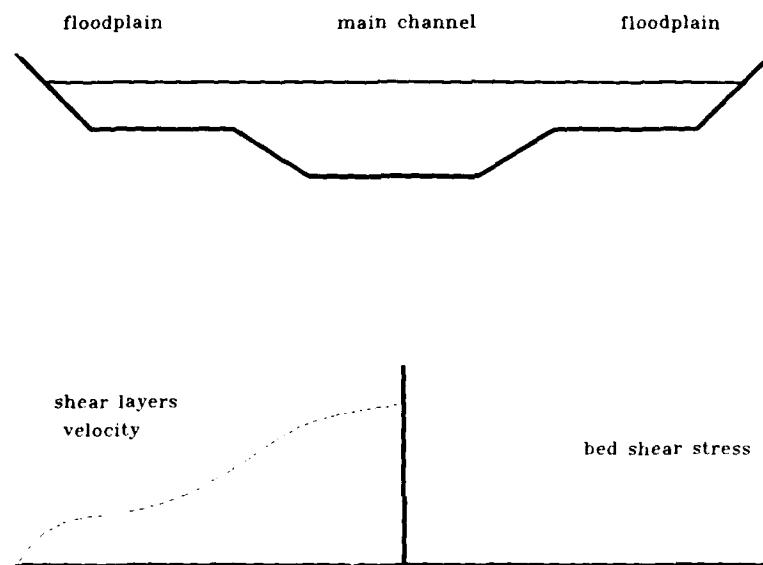


Figure 4.3
Distribution of Shear Stresses in a Two-Stage Channel
(after Knight and Lai, 1985)

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imagined acting along the velocity gradient between the main channel and floodplain. The position of this interface is marked by the concentration of the shear stresses, as shown in Figure 4.3.

The concept of an interface has been utilised in many recent investigations in two-stage channels. In the United Kingdom, the Science and Engineering Research Council is funding a large flume-based project in four universities, based on the concept of such an imaginary interface (see Knight *et al.* 1984). Further, Holden and James (1989) have attempted to quantify the rate of momentum transfer utilising the shear stress distribution on this imaginary interface.

The shear stresses acting on this imaginary interface are generally known as the 'apparent shear stresses', to distinguish them from the shear stresses that act on the physical boundary (bank or berm) between the main channel and the floodplain. In the next section, the alternative concepts available for modelling momentum transfer utilising the concept of the imaginary interface and apparent shear stresses are investigated.

4.2 Modelling of Momentum Transfer

4.2.1 A theoretical approach

If a geometrically regular two-stage channel experiencing uniform flow is analysed, then the total retarding shear force acting on the wetted perimeter is equal to the gravitational force acting downstream. The gravitation component is given by:

$$F_g = w A_t S_o \quad 4.1$$

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where w = weight of water per unit length of channel

A_t = total cross-sectional area

S_o = bed slope

The boundary shear force per unit length is given by:

$$F_b = T_c P_c + T_f P_f \quad 4.2$$

where T_c and T_f = average boundary shear stresses for the channel and floodplain respectively

and P_c and P_f = wetted perimeters of the channel and floodplain

Equations 4.1 and 4.2 must balance for the two-stage channel cross-sections but they must also balance for the individual floodplain and channel flow segments. However, if the flow segments are considered individually, then part of the boundary shear force is provided by the apparent shear stress force acting on the boundary between the flow segments. Thus in the case of the main channel the total retarding force per unit length is given by:

$$F_{sc} = T_c P_c + T_{ai} P_{ai} = F_{bc} + 2F_a \quad 4.3$$

where T_{ai} = apparent shear stress acting upon the assumed interface i

P_{ai} = length of assumed interface i

F_{bc} = main channel solid boundary shear force

Figure 4.4 illustrates these forces for a theoretical example where the apparent shear stresses are assumed to be acting on a vertical planar boundary where the channel and floodplain meet. Rewriting equations 4.1 and 4.2 for the channel segment only and combining with equation 4.3 gives:

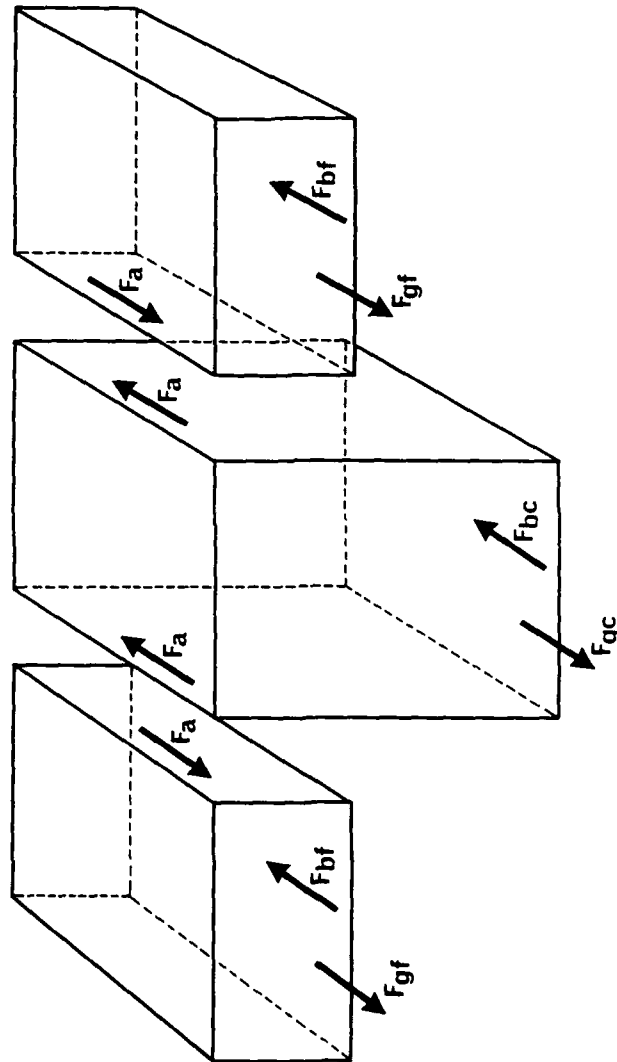


Figure 4.4
Theoretical Shear Stresses Acting on the Boundaries and Assumed
Vertical Interfaces of a Two-Stage Channel
 (after Wormleaton et al., 1982)

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$$T_{ai} = \frac{1}{P_{ai}} (wA_{ci}S_o - T_c P_c) \quad 4.4$$

In any application A_{ci} , P_c and S_o are known from the geometry of the cross-section and a length for P_{ai} can be assumed. In the flume the average boundary shear stresses, T_c , may be measured and so T_{ai} can be estimated using equation 4.4. The discharge for each flow segment can be computed from the corrected retarding forces.

However, in an ungauged catchment, it is unlikely that boundary shear stress data would be available, or that the length of the apparent shear stress boundary could be estimated. It is not suggested, therefore, that this type of analysis be incorporated into MILHY2 but investigation of the application of this method in flume experiments does provide a useful insight into the relationship between the cross-sectional geometry, apparent shear stress interfaces and accuracy of the discharge prediction.

4.2.2 Flume experiments investigation of apparent shear stresses

Flume-based research programmes provide, at present, the only means of collecting data on the distribution boundary shear stresses which will enable the understanding and later modelling of the active processes in two-stage channels. Field data of two-stage flood events can be both very difficult and sometimes dangerous to collect. The variable nature of flood events means that flows are never steady enough to allow even reasonable measurement of the velocity fields, while accurate field measurements of boundary shear stresses are almost impossible. Reliance on flume-based investigations has, therefore, led

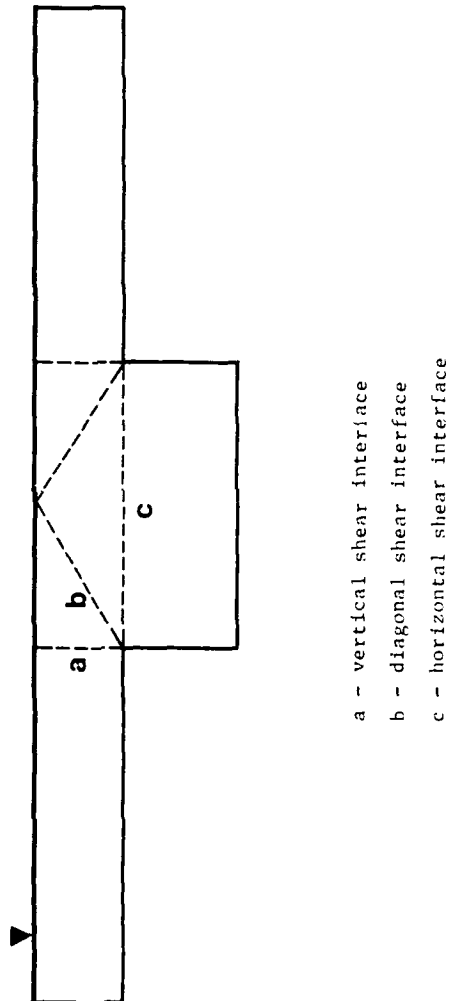
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to an extensive programme of modelling a variety of geometrical and roughness environments.

In these flume experiments, the principal objective of the investigations is to develop a relationship between the stage and discharge in the main channel and floodplain flow segments. The investigations attempt to achieve this objective by solving equation 4.4 using observed flume data to compute the apparent shear stresses from the solid boundary shear stresses. The boundary shear stresses are computed using the Prandtl-Von Karmen velocity law, utilizing observed velocity data, or using Patel's (1965) relationship between head difference and boundary shear stresses. As noted earlier, however, and seen in equation 4.4, the value of the computed apparent shear stresses is dependent on the length of the assumed interface over which the apparent shear theoretically acts. Figure 4.4 illustrates an assumed interface in a vertical plane, a method which has been utilized by Chow (1959) and Wright and Carstens (1970). Figure 4.5 shows the vertical plane and diagonal interfaces used by Wormleaton et al. (1980), and Yen and Overton (1973), and the horizontal interfaces utilized by Deuller et al. (1967).

Wormleaton et al. (1982) carried out a comparative investigation of the apparent shear stresses computed over the three types of planar interface; vertical, diagonal and horizontal. Their results were reported as an apparent shear stress ratio, that is the ratio of the apparent shear stress to the average shear stress including the assumed interface. As the apparent shear stress tends to zero, the ratio will tend to zero, implying no shear on the interface. The results of Wormleaton et al. (1982) are shown in Figures 4.6a, 4.6b and 4.6c for the vertical, diagonal and horizontal interfaces respectively, where the apparent shear stress ratio and an

Figure 4.5
Vertical, Diagonal and Horizontal Assumed Interfaces
Between Floodplain and Main Channel Flows



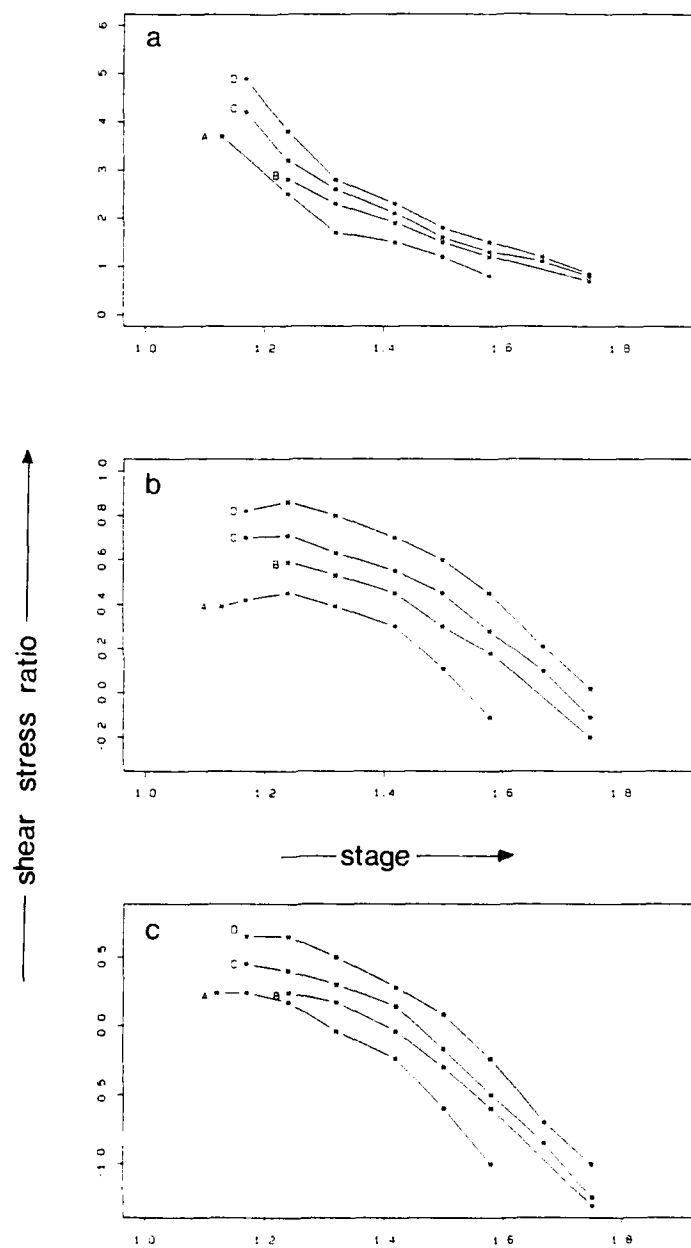


Figure 4.6
Stage/Apparent Shear Stress Ratio Relationships for Vertical, Diagonal
and Horizontal Assumed Interfaces
 (after Wormleaton et al., 1982)

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inundation ratio are compared. The inundation ratio is defined as the depth of flow on the floodplain divided by the depth of the main channel. The series A, B, C and D illustrate the effects of increasing the floodplain Manning's n roughness coefficient from 0.011 for series A through 0.014 (B), 0.017 (C), to 0.021 for series D.

Analysis of Figures 4.6a, 4.6b and 4.6c shows that the apparent shear stress declines with increasing depth of flow on the floodplain in all three planar interfaces. The order of magnitude difference between the apparent shear stresses computed for the vertical interfaces and those computed on the diagonal and horizontal interfaces should be noted. This shows that the vertical interface is much nearer to the turbulent eddies photographed by Sellin (1964). Analysis of the boundary stress distributions showed that the negative apparent shear stress ratios computed for the diagonal and horizontal interfaces at higher floodplain inundation depths indicate a transfer of momentum from the zone of flow above the main channel to the within-bank main channel zone.

Wormleaton et al. (1982) apparently aware of the criticism that all their shear stress values were computed using a single cross-sectional width, developed a relationship by regression analysis between geometric and velocity parameters for the apparent shear stresses. This could then be compared with data collected by other authors, often for very different applications, and so utilize data from a wide variety of cross-sectional geometries. Wormleaton et al. (1982) give a final regression equation for the stresses on a vertical interface as:

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$$T_{av} = 13.84 (\Delta V)^{0.882} \cdot \frac{y_t}{d_c}^{-3.123} \cdot \frac{B_f}{B_c}^{-0.727} \quad 4.5$$

where ΔV is the velocity difference between the floodplain and main channel flow segments, computed from the Manning equation. Utilizing the data from 34 experimental frames, the coefficient of determination was 0.983. Data collected by Myers (1978), Crory and Elksawy (1980) and Ghosh and Jena (1971) were found to conform closely with the relationship given in equation 4.5.

Yen and Overton (1973) tackled the problem from an alternative perspective by using the measured boundary shear stress profiles to position an interface along which no shear would take place. The cross-section could then be divided up using these no-shear boundaries and the discharge computed easily, as it would be directly related to the segment's cross-sectional area. Yen and Overton (1973) attempted to relate the angle of a zero shear interface, pivoting around the main channel/floodplain intercept (see Figure 4.7) to observed discharge values. If this angle could then be related to cross-sectional geometric parameters, this method could be applied simply to a wide variety of problems.

Yen and Overton's (1973) results showed that the angle of inclination of the zero shear stress plane varied with both the ratio of floodplain to main channel width, and the ratio of floodplain inundation to main channel depth. With a range of width ratios between 2.2 and 5.4, the angle of inclination varied by as much as 20° , with the angle increasing as the width ratio decreased. The angle of inclination varied with a depth ratio range of 0.2 to 1.8 by 60° , the angle increasing linearly (between 15° and 50°) until a depth ratio of about 1.0, after

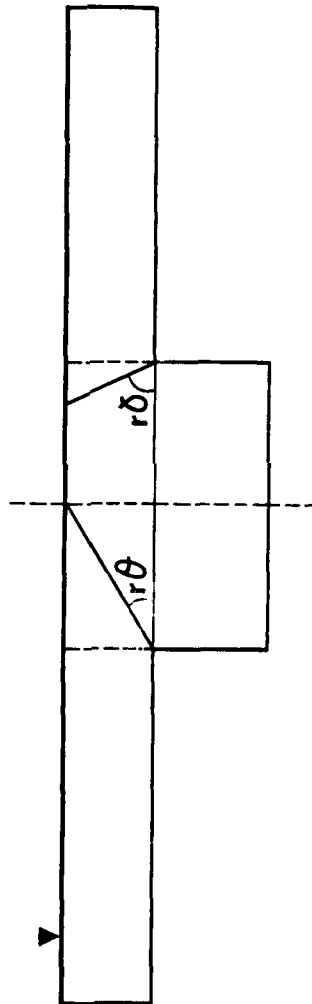


Figure 4.7
Angle of Assumed Interface Inclination
(after Yen and Overton, 1973)

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which the relationship became exponential. The angle of inclination of zero shear stress for a particular cross-section does not vary, therefore, when the depth ratio is above 2.

The results of Wormleaton *et al.* (1982) reported in Figure 4.6 agree with those of Yen and Overton (1973) and show that when the ratio of the floodplain inundation to main channel depth is approximately 2 or above, the two-stage channel may be considered as a single system. Below this ratio, the distribution of the turbulent shear stresses has been shown to be complex, where no one single position of the apparent shear stress interface or stress ratio can be adequately applied to describe the boundary shear stresses over a variety of cross-sectional geometries.

4.2.3 Implications of flume-based experiments for the prediction of the discharge capacity of two-stage channels

It was noted earlier in Section 4.1, that the main reason that the relationship between cross-sectional area and discharge does not hold for two-stage channels is the transfer of momentum between the main channel and the floodplain. The flume-based experiments reported in Section 4.2.2 attempt to quantify these momentum transfers by balancing the gravitational and retarding forces by the introduction of an apparent shear stress over a dividing interface between segments of flow. However, in order to compute the discharge capacity, there is a need to develop a relationship between easily measured geometric parameters and the stage/discharge rating curve. There are several alternatives that could be used in this respect:-

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- 1) Empirical relationships, developed for flume experiments, designed to predict the percentages of flow in each cross-sectional segment. These are developed from regression analysis of the computed apparent shear stresses on assumed interfaces. Examples include the relationships developed by Wormleaton et al. (1982) and by Knight and Demetriou (1983).
- 2) Division of the cross-section using the zero-shear interfaces, suggested by Yen and Overton (1973).
- 3) Division of the cross-section using shear interfaces and making some assumption about the amount of momentum transfer across these interfaces

Each of these alternatives are now considered. The first proposition to use empirically developed relationships seems attractive, in that it would be simple to apply. However, the relationships have been developed using data collected in flume experiments which have limited cross-sectional geometries. Table 4.1 shows the geometric parameters of the major flume investigations that have published this type of data. Comparison of the floodplain to main channel widths shows a maximum ratio of 3 where, in many catchments, flood inundation maps illustrate a width ratio of up to 50. Similarly, the maximum Manning's n roughness coefficient applied to the floodplain is 0.022, whilst Chow (1959) suggests a typical grazed pasture to have a Manning's value of 0.03. To generate empirical relationships applicable to the sorts of two-stage channels, typical in Europe, therefore, there is a need for further flume experiments with much wider and rougher floodplains. Until this is achieved, it would

Table 4.1
Cross-sectional geometric and roughness parameters for flume-based investigations
into the transfer of momentum between the main channel and floodplain

Author	Channel width m	Floodplain width m	Channel depth m	Floodplain inundation range m	channel Manning's 'n'	floodplain Manning's 'n'
Wormleaton <u>et al.</u> (1982)	0.29	0.46	0.12	0.02-0.09	0.011	0.011-0.021
Myers (1984)	0.16	0.18, 0.30	0.08	0.01-0.07		perspex
Crory & Elksawy (1980)	0.10, 0.15	0.36	0.10			
Ghosh & Jena (1971)	0.20	0.15	0.10			
Knight & Demetriou (1983)	0.15	0.00-0.30	0.08	0.01-0.03		perspex
Rajaratnam & Ahmadi (1981)	0.51	0.71	0.10	0.01-0.06	0.013	0.013
Prinos <u>et al.</u> (1985)	0.51	0.38	0.10	0.05-0.12	0.011	0.011-0.022
Noutsopoulos & Hadjipanos (1983)	0.51	0.26-0.43	0.08	0.09-0.15	0.011	0.011-0.021
Smith (1978)	0.27	0.47	0.08	0.05-0.08	0.013	0.013
Asano <u>et al.</u> (1985)	0.9-2.4	3.0	0.03-0.12	0.01-0.21	0.009-0.01	0.013

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be inadvisable to extrapolate the existing relationships to geometries and roughnesses outside those reported in Table 4.1

The second alternative given is to divide the cross-section using zero-shear interfaces, as suggested by Yen and Overton (1973). As there is no momentum transfer across the zero-shear interfaces, the Manning equation will hold for each cross-sectional flow segment. Although Yen and Overton computed the angle of incidence of the interfaces for width ratios up to 5, the sensitivity of this angle to floodplain roughness means the results cannot be reliably applied. It is also rather more difficult to compute the area of flow in a cross-section using Yen and Overton's method than a vertical, diagonal or horizontal interface method.

However, zero-shear interface has been applied widely for a number of years. One of the most frequently used techniques was developed by Lotter (1933) and computes the capacity of a cross-section by dividing the section using vertical interfaces. These interfaces are designated as being zero shear faces and, therefore, the length of the interface is not included in the computation of the channel capacity. As Yen and Overton (1973) have shown, however, such interfaces are not vertical, but vary from the inclined towards the horizontal as the depth of flow increases. Zero-shear interfaces can be applied, therefore, for vertical, diagonal and horizontal inclinations by ignoring the assumed interface in the wetted perimeter computation and taking into consideration the solid boundaries only.

The third suggested means of computing the discharge capacity of the cross-section involves dividing the cross-section using the shear interfaces, making an assumption

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about the amount of momentum transfer across these interfaces. In a similar way to the zero-shear interfaces, shear interfaces have been applied in a great number of environments, using vertical, diagonal and horizontal inclinations. The assumption here is that the apparent shear stress is equal to the average shear stress (apparent shear stress ratio = 1, see Figure 4.6), so that the interface can be included as part of the wetted perimeter in the discharge capacity computation.

Wormleaton et al. (1982) computed the discharge for the zero-shear and shear interfaces for all three inclinations over a variety of floodplain roughnesses up to $n = 0.021$. As expected, their results showed that for all interface inclinations the computed discharge values converged to, or were smaller than, the observed values, when the floodplain/channel depth ratio increased to 2. However, the accuracy of the discharge prediction using these six techniques was considered only with variation in the depth ratio and floodplain roughness; the width ratios were not considered.

The implication of the flume-based experiments to the computation of discharge in two-stage channels, is that no single technique of incorporating turbulent exchange between the main channel and floodplain is appropriate for all geometric and roughness environments. The flume experiments need to be extended before a set of operational rules on the suitability of zero-shear or shear interfaces and their angle of inclination can be developed.

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4.3 Incorporation of Momentum Transfer into MILHY2

Analysis of the flume-based experiments, in section 4.2, has shown that there is no single method of incorporating momentum transfer between flow segments that is appropriate for cross-sectional geometries and roughnesses. For this reason, and because of the lack of comparative work on wide and rough floodplains, it seems appropriate to incorporate a number of different methods into MILHY2 and test the accuracy of the discharge predictions against observed field data.

4.3.1 Selection of methods for incorporation into MILHY2

Four methods of dividing the cross-section to incorporate momentum transfer were selected from the alternatives identified above. These four were selected primarily because of the ease with which they could be incorporated into the MILHY2 code and because they are the four techniques utilised by Knight and Hamed (1984). By using the same techniques as Knight and Hamed, a comparison between field data collected for this project and flume experiments would be possible. It is accepted, therefore, that the four techniques selected offer no theoretical advantages over the other methods of dividing the cross-section identified earlier. The four techniques selected are:

- 1) Vertical subdivision, with zero shear interfaces
- 2) Vertical subdivision, with an apparent shear stress ratio = 1
- 3) Diagonal subdivision, with zero shear interfaces
- 4) Diagonal subdivision, with an apparent shear stress ratio = 1

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At present, method 2, that is vertical subdivision with an apparent shear stress ratio equal to 1, is incorporated into MILHY2. By application of these four techniques it should be possible to test the sensitivity of the generated rating-curve to the interface inclination and apparent shear stress ratio. This sensitivity could then be compared to the impact on the rating curve of the variation in the cross-sectional geometry and roughness parameters. If the analysis showed that the rating-curve is sensitive to the computational method then further methods including horizontally inclined interfaces and Yen and Overton's (1973) angle of inclination could be incorporated and tested.

4.3.2 Incorporation of the four methods into MILHY2

The four methods, identified above, of incorporating momentum transfer between the main channel and floodplain, are the same four methods utilized by Knight and Hamed (1984). Knight and Hamed tested the accuracy of the four identified techniques in predicting discharge by comparing the predicted results with those collected from flume experiments conducted by Knight and Demetriou (1983), reported in Table 4.1. For consistency, and to ensure that the correct cross-sectional definitions were being applied to MILHY2 for each of the four methods, the equations of definition reported in Knight and Hamed's (1984) paper were incorporated into MILHY2. These equations are given in Table 4.2, whilst Figure 4.8 defines the cross-sectional geometry variables used. The equations in Table 4.2 show that the wetted perimeter of the interface is included in the main channel computation in methods 2 and 4, where the apparent shear stress ratio is 1, and the interface

Table 4.2
Alternative geometric definitions to incorporate segment interaction
 (after Knight and Hamed, 1984)

Method	Flood Plain		Main Channel	
	Area	Wetted Perimeter	Area	Wetted Perimeter
1	$(H-h)(B-b)$	$B-b + H-h$	$2 bH$	$2b + 2h$
2	$(H-h)(B-b)$	$B-b + 2(H-h)$	$2bH$	$2b + 2H$
3	$(H-h)(B-b/2)$	$B-b + H-h$	$b(H+h)$	$2b + 2h$
4	$(H-h)(B-b/2)$	$B-b + H-h$	$b(H+h)$	$2b + 2h + 2((H-h)^2 + b^2)^{1/2}$

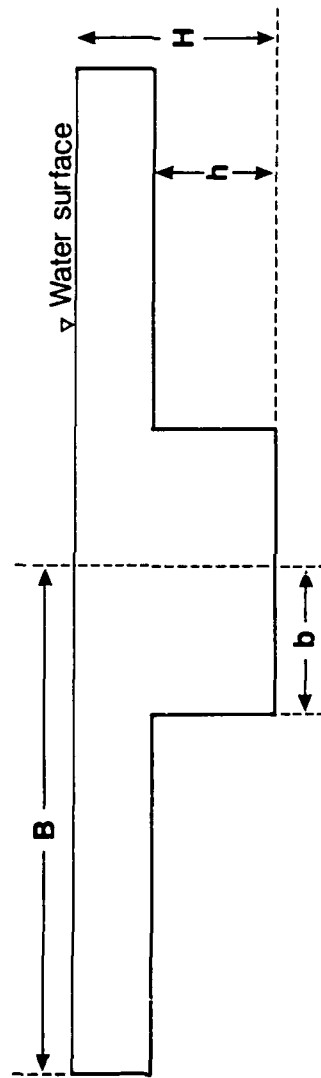


Figure 4.8
Definition of Cross-Sectional Geometric Parameters
Utilised by Knight and Hamed (1984)

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is excluded in methods 1 and 3 where zero shear is assumed. These four methods were incorporated into the rating curve generation routine (subroutine CMPRC) of MILHY2 for stage elevations above bankfull.

4.4 Sensitivity of the Rating Curve to Interface Inclination

There are a number of objectives in undertaking a sensitivity analysis of the rating curve to the interface computation method. These are:

- 1) to establish whether any one method improves the accuracy of the predicted rating curve in comparison with observed field rating curves for a field cross-section;
- 2) to establish whether there is a significant difference in the predicted rating curve generated by each of the four methods for wide floodplains with greater boundary roughnesses than those reported in Table 4.1;
- 3) to compare the difference in the computed rating curve attributable to the interface inclination method, with the difference due to variability in the cross-sectional geometry and roughness parameters.

To answer these three questions, it was necessary to apply the four interface inclination methods to both field cross-sections, to achieve objective one, and hypothetical reaches to achieve objectives two and three. Whilst the field cross-sections are similar to the theoretical cross-

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sections in that they have broadly rectangular main channels and flat wide floodplains (see Figure 4.9) application of field cross-sections provided the only comparison to an observed rating curve possible. Objectives two and three can be achieved by comparison of the predicted rating-curves generated by the four computation methods utilizing to hypothetical cross-sections.

4.4.1 Application of the four interface inclination methods

The cross-section at Bad Hersfeld on the River Fulda, West Germany, was selected in order to compare the accuracy of the four computation methods against a field rating curve. The rating curve at Bad Hersfeld was extended to out-of-bank conditions using data from gauged extreme events for floodplain inundation depths of up to 3.2 metres. This depth corresponds approximately to the 1 in 100 year event. At Bad Hersfeld, the floodplains are symmetrical about the main channel with a floodplain to main channel width ratio (B/b) of 10. The bankfull depth (h) is 4.1 metres, whilst the floodplains on either side of the main channel are pasture. The four interface inclination methods were applied for three sets of geometric and roughness environments and the discharge at stage increments of 0.5 metres computed. The rating curve was also computed for the first two cases with the cross-section being treated as a single system, that is with no interfaces to divide the cross-section into segments. The rating curves produced from these applications are given in Tables 4.3, 4.4 and 4.5.

A theoretical cross-section was established to achieve objectives two and three noted above, with a rectangular main channel and a floodplain rise from the channel to the valley



Figure 4.9
Channel Cross-Section at Mecklar, 10km Downstream of
Bad Hersfeld on the River Fulda, West Germany

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side of 0.1 metres. The floodplain to main channel width ratios considered were 10 and 20. As noted earlier, flume experiments by numerous authors have investigated smaller width ratios. Wormleaton *et al.* (1982) reported that discharge predictions from all the interface inclination methods, that is vertical, diagonal and horizontal, converged to a common solution as the floodplain inundation depth to main channel depth ratio (H/h) approached 2. To check this, discharge predictions calculated for depth ratios of up to 2.2 were computed at 0.5 metre stage increments. As well as the four interface inclination methods, the rating curves were computed treating the cross-section as a single flow segment. Where friction or slope parameters varied between the main channel and floodplain segments, a mean average was applied to the single segment case. This was true for both the hypothetical and Bad Hersfeld cross-sections. The hypothetical cross-section results are given in Tables 4.6, 4.7, 4.8, 4.9 and 4.10.

4.4.2 Results of the sensitivity analysis

The results of the sensitivity analysis of the computed rating curve to the interface inclination and variation in geometric parameters, are tabulated in Tables 4.3, 4.4, 4.5, 4.6, 4.7, 4.8, 4.9 and 4.10. Tables 4.3, 4.4 and 4.5 show the results computed for the Bad Hersfeld cross-section and also record the percentage error of each of the interface inclination methods against an observed rating curve. Tables 4.6, 4.7, 4.8, 4.9 and 4.10 show the results for a hypothetical cross-section, and the percentage error in these tables indicates the deviation from the MILHY2 solution as no observed rating curve was available.

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Table 4.3 contains the observed discharge values and the computed values from MILHY2 and interface methods 1 to 4. Table 4.3 confirms that MILHY2 incorporates inclination method 2 and in further tables, therefore, both are not shown. Manning's n values of 0.035 for the main channel and floodplain were selected for the first simulation reported in Table 4.3. This value corresponds to the tabulated values suggested in Chow (1959). The channel and floodplain slopes were set at 0.0006, computed from the field rating curves from Bad Hersfeld and Rotenburg, the next gauging station downstream.

Results from the Bad Hersfeld Station

Table 4.3 shows the discharge predictions from the four interface methods computed using the parameter values reported above. The mean average error of the discharge predictions over the observed figures was computed for each method over a range of inundation depths.

Table 4.3 shows that all methods at all inundation depths overpredicted the carrying capacity of the cross-section. The average error shows that method 2, (vertical interfaces with an apparent shear stress ratio = 1), the method utilized by MILHY2, gave the worst prediction. The best overall prediction was given by the single segment method. This, however, was not so surprising as both the boundary roughness and slope variables were constant across the section.

Table 4.3 also shows that there was no consistent difference in the predictive performance between the methods incorporating the shear face, that is methods 2 and 4, and the zero shear, methods 1 and 3. It is also important to note that the percentage error increases with depth in all methods except method 3.

Table 4.3
Comparison of the predictive accuracy of interface inclination methods
1-4 for Bad Hersfeld, River Fulda

Height above bankfull m	Observed $\frac{3}{m} \frac{-1}{s}$	MILHY2		Method 1		Method 2		Method 3		Method 4		Single Segment	
		Q	Error	Q	Error	Q	Error	Q	Error	Q	Error	Q	Error
		$\frac{3}{m} \frac{-1}{s}$	%	$\frac{3}{m} \frac{-1}{s}$	%	$\frac{3}{m} \frac{-1}{s}$	%	$\frac{3}{m} \frac{-1}{s}$	%	$\frac{3}{m} \frac{-1}{s}$	%	$\frac{3}{m} \frac{-1}{s}$	%
0.29	86	134	+55	118	+37	134	+55	163	+89	124	+44	67	-22
0.78	134	232	+73	212	+58	232	+73	240	+79	197	+47	152	+14
1.27	205	411	+101	286	+88	411	+101	400	+95	352	+72	336	+64
1.76	321	650	+103	619	+93	650	+102	617	+92	563	+76	572	+78
2.25	490	950	+94	913	+86	949	+94	891	+82	833	+70	866	+77
2.74	675	1311	+94	1268	+88	1310	+94	1224	+81	1160	+72	1219	+81
3.22	830	1734	+109	1684	+103	1732	+109	1617	+95	1547	+86	1631	+97
Mean error		+90		+79		+90		+88		+67		+55	

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B/b = 10
h = 4.1m
n_{ch} = 0.035
n_{fp} = 0.035
S_{ch} = 0.0006
S_{fp} = 0.0006

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Comparison of Tables 4.3 and 4.4 shows how increasing the floodplain boundary roughness can more than halve the error of the predictions for all four methods. The difference in mean average errors between computation methods is, however, the same as those in Table 4.3. This suggests that the carrying capacity computation is more sensitive to the boundary roughness value than the form of the main channel/floodplain interface.

Table 4.4 also shows that the percentage error does not increase with increasing floodplain inundation depth, as suggested in Table 4.3. In Table 4.4 the percentage error values indicate that all four computation methods are converging to the observed discharge as the inundation depth increases and approaches the main channel depth. This suggests that a floodplain roughness value of 0.07 corresponds more closely to the field conditions than the initial value used of 0.035. The logic behind this argument lies in the acceptance that as the floodplain inundation depth increases to the main channel depth, the two-stage channel behaves as a single system and therefore all the computation methods should converge on a common solution. If the solutions do not converge this suggests that the initial boundary conditions are not realistic.

Table 4.3 shows the effects of incorporating meandering in the channel by reducing the slope value used to 0.0001 from 0.0006. This value is calculated from the ratio of the main channel routing length to the valley length between Bad Hersfeld and Rotenburg on the River Fulda. Comparison of Tables 4.3 and 4.5 shows that reducing the slope of the main channel improves the predictions of the carrying capacity in all computation methods.

The results from the Bad Hersfeld (see Tables 4.3 and 4.4)

Table 4.4
Comparison of the predictive accuracy of interface inclination methods
1-4 for Bad Hersfeld, River Fulda, with variation in the floodplain roughness

Height above bankfull m	Observed $m^3 s^{-1}$	Method 1		Method 2		Method 3		Method 4		Single Segment	
		Q	Error $m^3 s^{-1}$ %	Q	Error $m^3 s^{-1}$ %	Q	Error $m^3 s^{-1}$ %	Q	Error $m^3 s^{-1}$ %	Q	Error $m^3 s^{-1}$ %
0.29	86	113	+31	129	+50	155	+80	116	+35	47	-46
0.78	134	176	+32	197	+47	202	+51	158	+18	101	-24
1.27	205	281	+37	306	+49	291	+42	243	+18	224	+9
1.76	321	416	+30	447	+39	408	+27	355	+11	381	+19
2.25	490	583	+19	620	+27	555	+13	497	+1	577	+18
2.74	675	782	+16	825	+22	732	+8	668	-1	812	+20
3.22	830	1013	+22	1062	+28	939	+13	869	+5	1087	+31
Mean error		+27		+37		+34		+12		+4	

B/b = 10 $n_{ch} = 0.035$ $n_s = 0.0525$ $S_{fp} = 0.0006$
h 4.1m $n_{fp} = 0.07$ $S_{ch} = 0.0006$

Table 4.5
Comparison of the predictive accuracy of interface inclination methods
1-4 for Bad Hersfeld, River Fulda, with variation in the downstream main channel slope

Height above bankfull m	Observed $m s^{-1}$	Method 1		Method 2		Method 3		Method 4		Single Segment	
		Q	Error $m s^{-1}$ %	Q	Error $m s^{-1}$ %	Q	Error $m s^{-1}$ %	Q	Error $m s^{-1}$ %	Q	Error $m s^{-1}$ %
0.29	86	53	-38	60	-30	76	-12	39	-56	51	-41
0.78	134	129	-4	137	+2	143	+7	61	-55	116	-13
1.27	205	282	+38	292	+42	293	+43	126	-39	257	+25
1.76	321	493	+54	505	+57	498	+55	273	-15	437	+27
2.25	490	763	+56	777	+59	761	+55	477	-3	661	+35
2.74	675	1092	+62	1108	+64	1082	+60	737	+9	931	+38
3.22	830	1481	+78	1500	+81	1462	+76	1056	+27	1246	+50
Mean error			+35		+39		+41		-18		+17

$B/b = 10$
 $n_{fp} = 0.035$
 $h = 4.1m$
 $S_{ch} = 0.0001$
 $n_{ch} = 0.035$
 $S_{fp} = 0.0006$

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simulations suggest that the computation utilised in MILHY2 give the poorest prediction of the carrying capacity of the cross-section. Tables 4.4 and 4.5 show that the prediction can be much improved by the more accurate selection of parameter values than by altering the computation technique.

Results from a Hypothetical Cross-section

Tables 4.6, 4.7, 4.8, 4.9 and 4.10 give the predictions of the carrying capacity for a hypothetical cross-section, comparing computation methods 1 to 4 and the discharge prediction computed by treating the section as a single flow segment. The percentage error values reported are computed from the MILHY2 predictions, which utilise method 2 (vertical interface, apparent shear stress ratio = 1). The percentage error values allow comparison of the relative sensitivity of the discharge predictions to variation in the computation method and parameters. The absolute accuracy of the techniques cannot be computed as this is a hypothetical application.

Analysis of Tables 4.6, 4.7, 4.8, 4.9 and 4.10 shows that method 1 (vertical interface, zero shear) produces very close approximations to the predictions produced from the MILHY2 computation for all the boundary roughness and geometry environments. In all cases, methods 3 and 4 (diagonal interface, zero shear and shear ratio = 1, respectively), rank second and third consistently in their closeness to the MILHY2 predictions. Methods 1, 3 and 4 under-predict the carrying capacity in comparison to the MILHY2 predictions in all five experimental frames.

Comparison of Table 4.6, where the floodplain/channel width ratio is 10, with Table 4.10, width ratio = 20,

Table 4.6
Comparison of the predictive accuracy of interface inclination methods 1-4
for a hypothetical reach

Height above bankfull m	MILHY2 3^{-1} m s $^{-1}$	Method 1		Method 3		Method 4		Single Segment	
		Q	Error 3^{-1} m s $^{-1}$ %	Q	Error 3^{-1} m s $^{-1}$ %	Q	Error 3^{-1} m s $^{-1}$ %	Q	Error 3^{-1} m s $^{-1}$ %
0.56	61	58	-5	42	-31	34	-44	33	-45
1.01	153	150	-2	127	-17	118	-23	131	-14
1.56	295	291	-1	261	-12	250	-15	275	-7
1.90	477	472	-1	433	-9	421	-12	460	-4
2.35	695	689	-1	641	-8	636	-10	679	-2
2.80	946	939	-1	880	-7	863	-9	931	-2
			-2		-14		-19		-12

B/b = 10 $n_{fp} = 0.035$
h = 2.4m S = 0.0005
 $n_{ch} = 0.035$

Table 4.7

Comparison of the predictive accuracy of interface inclination methods 1-4
for a hypothetical reach, with variation in the floodplain roughness

Height above bankfull m	MILHY2 3^{-1} $m s^{-1}$	Method 1		Method 3		Method 4		Single Segment	
		Q	Error 3^{-1} $m s^{-1}$ %	Q	Error 3^{-1} $m s^{-1}$ %	Q	Error 3^{-1} $m s^{-1}$ %	Q	Error 3^{-1} $m s^{-1}$ %
0.56	53	51	-4	34	-36	26	-51	68	+28
1.01	107	103	-2	110	-26	70	-35	275	+157
1.56	185	181	-2	241	-20	137	-26	580	+214
1.90	284	279	-2	410	-16	225	-21	969	+241
2.35	401	396	-1	614	-14	330	-18	1432	+257
2.80	536	529	-1	850	-13	451	-16	1963	+266
Mean error			-2		-21		-28		+194

$B/b = 10$ $n_{fp} = 0.07$
 $h = 2.4m$ $n_s = 0.0525$
 $n_{ch} = 0.035$ $S = 0.0005$

Table 4.8
 Comparison of the predictive accuracy of interface inclination methods 1-4
 for a hypothetical reach, with variation in the downstream main channel slope

Height above bankfull	MILHY2 $\text{m}^3 \text{s}^{-1}$	Method 1		Method 3		Method 4		Single Segment	
		Q	Error $\text{m}^3 \text{s}^{-1}$ %	Q	Error $\text{m}^3 \text{s}^{-1}$ %	Q	Error $\text{m}^3 \text{s}^{-1}$ %	Q	Error $\text{m}^3 \text{s}^{-1}$ %
0.56	35	34	-4	28	-21	24	-32	25	-30
1.01	121	119	-2	110	-9	106	-12	101	-17
1.56	254	252	-1	241	-5	236	-7	213	-16
1.90	427	425	-1	410	-4	404	-5	356	-17
2.35	635	633	0	614	-3	608	-4	526	-17
2.80	876	873	0	850	-3	843	-4	721	-18
Mean error			-1		-8		-11		-19

$B/b = 10$
 $h = 2.4\text{m}$
 $n_{ch} = 0.035$
 $n_{fp} = 0.035$
 $S_{ch} = 0.0001$
 $S_{fp} = 0.0005$
 $S_c = 0.0003$

Table 4.9
Comparison of the predictive accuracy of interface inclination methods 1-4 for a
hypothetical reach with variation in the downstream main channel and floodplain slope

Height above bankfull m	MILHY2 3^{-1} m s ⁻¹	Method 1		Method 3		Method 4		Single Segment	
		Q	Error %	Q	Error %	Q	Error %	Q	Error %
		3^{-1} m s ⁻¹	%	3^{-1} m s ⁻¹	%	3^{-1} m s ⁻¹	%	3^{-1} m s ⁻¹	%
0.56	192	184	-4	132	-31	107	-44	103	-46
1.01	485	475	-2	403	-17	374	-23	413	-15
1.56	933	920	-1	824	-12	790	-15	870	-7
1.90	1508	1493	-1	1370	-9	1330	-12	1453	-4
2.35	2197	2179	-1	2026	-8	1980	-10	2148	-2
2.80	2990	2968	-1	2783	-7	2731	-9	2945	-2
Mean error			-2		-14		-19		-13

$B/b = 10$
 $n_{fp} = 0.035$
 $h = 2.4m$
 $n_{ch} = 0.035$
 $S = 0.0005$

Table 4.10
Comparison of the predictive accuracy of interface inclination methods 1-4
for a hypothetical reach with variation in the floodplain/main channel width ratio

Height above bankfull m	MILHY2 m s^{-1}	Method 1		Method 3		Method 4		Single Segment	
		Q	Error m s^{-1} %	Q	Error m s^{-1} %	Q	Error m s^{-1} %	Q	Error m s^{-1} %
0.56	77	74	-4	58	-25	50	-35	48	-38
1.01	254	251	-1	228	-10	219	-14	231	-9
1.56	531	527	-1	497	-6	486	-9	511	-4
1.90	891	886	-1	847	-5	834	-6	873	-2
2.35	1324	1318	0	1270	-4	1256	-5	1308	-1
2.80	1825	1818	0	1759	-4	1743	-5	1811	-1
Mean error			-1		-9		-12		-4

$B/b = 20$
 $n_{fp} = 0.035$
 $h = 2.4\text{m}$
 $S = 0.0005$
 $n_{ch} = 0.035$

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shows that this increase has made little impact on the comparative accuracy of the computation methods. There has been no radical change in the difference in the mean average errors between the four computation methods.

Comparison of the Hypothetical and Bad Hersfeld Applications

Analysis of the two sets of results has shown that the method incorporated into MILHY2, that is method 2, generates the greatest carrying capacity of the cross-section in both the Bad Hersfeld and hypothetical sections. The Bad Hersfeld section results suggest that method 2 generates the poorest prediction of the four methods, which all over-predict the carrying capacity. This suggests that all four methods do not introduce enough friction over the assumed interfaces between the floodplain and main channel to mimic the retarding effects of momentum exchange. Method 4 assumes a diagonal interface and an apparent shear stress ratio equal to one, and introduces the greatest additional boundary friction of the methods, hence producing the lowest prediction of carrying capacity (see Tables 4.3 to 4.10). This suggests that in the field apparent shear stress ratios on diagonal interfaces may be greater than 1, rather than less than 1, as Wormleaton *et al.* (1982) found (see Figure 4.6b). Alternatively, these results suggest that the true position of the interface is between the vertical and the diagonal, as the apparent shear stress ratios on the vertical interface are very much greater than 1 (see Figure 4.6a). Apparent shear stress ratios of greater than 1 could be incorporated into the MILHY2 scheme by increasing the length of the wetted perimeter of the apparent interface in the main channel computation until the stress ratio was reduced to one.

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4.4.3 Conclusions

From the analysis of the results above, it is possible to make several conclusions:

- 1) The three methods utilized to incorporate turbulent exchange between the main channel and floodplain, predict more accurately the carrying capacity of a cross-section than the technique used in MILHY2.
- 2) All four methods (section 4.3.1) over-predicted the carrying capacity because they failed to introduce enough additional boundary friction to mimic the effects of turbulent exchange. Method 4 introduced the most additional friction and gave the best predictions.
- 3) Increasing the boundary roughness, Manning's n , for the floodplain was more effective at reducing the over-prediction of the carrying capacity of the section than increasing the wetted perimeter of the interface, or assuming an apparent shear stress ratio of one.

4.5 Implications of the Incorporation of Momentum Exchange

The results of the introduction of the incorporation of momentum exchange into MILHY2 show that the predictive accuracy of the model could be improved. The results also suggested that method 4 (diagonal interface, apparent shear ratio = 1) gave the best prediction. This is in contrast to the results of Knight and Hamed (1984), who found that method 3 (diagonal interface, zero shear) gave the best prediction. One explanation for this is that the field data used in this application were for more turbulent conditions than the flume experiments conducted

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by Knight and Hamed (1984). This turbulence may have been generated from the rougher boundary friction conditions in the field. Another explanation for the different results from this chapter and Knight and Hamed, is that this analysis looked at the effects of the four methods on the rating curve, that is at a point of the reach. Knight and Hamed investigated the accuracy of the four methods at the end of a reach after the water has been routed. Knight and Hamed, therefore, compared the accuracy of the methods with a three-dimensional flume result, whilst this analysis only considered the predictive accuracy with reference to two dimensions. It is important, therefore, that the significance of the incorporation of the four methods along a reach should be considered as part of the MILHY3 evaluation programme.

The results of the evaluation of the routine also showed that increasing the Manning's n coefficient was just as successful at reducing the over-prediction of the channel capacity as the introduction of momentum exchange. This suggests that boundary friction effects are more significant on the prediction of the rating curve than the introduction of momentum exchange. However, the incorporation of momentum exchange may still be considered to be significant as it allows the more accurate selection of the Manning's n coefficient based on the effects of boundary friction. If evaluation of the MILHY3 model further highlights the importance of boundary friction, then it may be appropriate to consider replacement of the Manning's n coefficient with a more sophisticated measure of boundary friction.

The success of the incorporation of momentum exchange supports the approach taken of evaluating hydraulic concepts and techniques for incorporation into hydrologic models. The

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concepts of the interface and apparent shear stresses have been shown to be portable.

The evaluation of the momentum exchange routine explored in this chapter is seen to be part of the first two stages of Sargent's (1982) model evaluation programme. The investigation of hydraulic concepts and techniques is considered to be part of the mathematical evaluation, whilst the evaluation of the four methods is part of the computerised model verification. The success of the introduction of momentum exchange in improving the predictive accuracy of MILHY2 means that it is considered suitable for inclusion in the further evaluation of the module as a catchment model. The four methods are therefore evaluated further in the analysis of the MILHY3 catchment model and are reported in Chapter 7.

4.6 Summary

This chapter investigates the transfer of momentum between the main channel and the floodplain. It has explored various methods of incorporating the transfer of momentum into the MILHY3 composite scheme, applied and evaluated four techniques. The results and conclusions of this chapter can be summarized into several points. These are:-

- 1) It is accepted in this chapter that the dominant process controlling the transfer of momentum is the velocity gradient across the floodplain/main channel cross-section. This transfer can be visualised using an imaginary interface and the apparent shear stresses acting on this interface. These concepts have been developed by hydraulic engineers.

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2) The concepts of an interface and apparent shear stresses have not previously been applied to hydrological modelling. Attempts at empirical formulations using results from flume experiments are inappropriate for hydrological applications, as the flume reaches utilised cross-sectional geometries that were too narrow and floodplain roughnesses too smooth.

3) Two interface inclinations and two apparent shear stress ratios were selected for evaluation. These four techniques were successfully incorporated into the MILHY3 scheme. Evaluation of the four techniques on a hypothetical and field rating curve showed that method four produced the greatest effect by introducing the greatest amount of momentum exchange. In addition, the rating curve produced by method 4 was the closest to the observed field cross-section.

4) The results of this evaluation were compared with the results of Knight and Hamed (1984), who applied the same four techniques. The results of the Knight and Hamed (1984) experiments had shown that method 3 produced the rating curve closest to the observed curve measured in the flume. It is suggested that the success of method 3, which incorporated less momentum exchange than method 4, was due to the narrow and smooth floodplain used in the flume experiments and the possible effects of the third downstream dimension.

5) The results showed that the effects of increasing the Manning's n coefficient were greater than the introduction of the momentum exchange routine. It is suggested, therefore, that boundary roughness may be more significant on the prediction of channel flow than momentum exchange.

6) It is considered essential that the momentum exchange routine is evaluated as part of a catchment model if the

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importance of the effects of the longitudinal dimension are to be considered, and if the relative importance of the effects of boundary friction are to be established.

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Incorporation of Multiple Routing Reaches

Analysis of the behaviour of flow in two-stage channels undertaken in Chapter 3, identified three key processes that need further investigation. These were the handling of boundary friction and transverse circulations, the incorporation of turbulent shear stresses between the main channel and the floodplain, and the incorporation of the different path lengths for the main channel and floodplain. The incorporation of turbulent shear stresses has been investigated and reported in Chapter 4. This chapter, therefore, concentrates on the handling of the different path lengths of the main channel and floodplain flow segments. The relative importance of the effects of boundary friction are discussed in both Chapter 4 and this chapter; the effects of transverse circulations are not investigated.

The sensitivity analysis of the Ervine and Ellis (1987) scheme reported in Chapter 3, found that the sinuosity of the main channel was important in determining the length of the downstream or longitudinal flow path. In two-stage channels, the downstream reach length in the relatively sinuous main channel may be up to 30% longer than the straighter floodplain flows. The concept of multiple routing paths has been identified as a useful tool in visualising these different pathways.

In this chapter, the downstream behaviour of two-stage flow is investigated and alternative methods of incorporating the concept of multiple routing pathways considered. A method

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incorporating multiple routing reaches will then be selected and an initial sensitivity analysis undertaken in order to validate the new module.

5.1 The Behaviour of Downstream Two-Stage Channel Flow

Water on the floodplain may return to the channel either by overland flow or by throughflow. Water that is ponded by the topographic pattern of the floodplain must return via throughflow but flowing water may return directly to the channel, or may route down slope for some distance before rejoining the channel. In bankfull conditions floodplain flows may cross and recross the sinuous main channel beneath them, with only relatively small amounts of momentum exchange taking place. It is these high flow conditions when the main channel is full that this chapter is primarily concerned with. The effects of throughflow of ponded water and the direct return of floodplain flow to the main channel are not considered. The effects of the more direct return of floodplain flow in comparison with the larger flood events will have a nominal effect on the nature of the predicted hydrograph. The importance of throughflow of floodplain water on the hydrograph is not known but it is considered that the effects would only be significant in the predictions of a continuous simulator. This is because it is likely that the seepage from the floodplain would only have an impact on the hydrograph long after the main floodwave had passed.

In two-stage channels there is a tendency for floodplain flow to "short-circuit" the generally more sinuous route of the main channel, taking a more direct route downstream (Fread, 1976). One possible explanation for the phenomenon of less sinuous floodplain flows is that the infrequency of out-of-bank

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events do not provide the opportunity for the development of a secondary flow system on the floodplain. A secondary flow system is thought to be a necessary precursor for the development of a meander system (Richards, 1982). Einstein and Shen (1964) suggested that the secondary flow system is itself initiated by shear, possibly along a rough bank.

The reason for the generally less sinuous pathway of the floodplain flow in comparison with the main channel flow is no nearer explanation than is agreement on the initiation of meanders. Whatever the explanation, the shorter path length of the floodplain is exacerbated by the steeper gradient of the floodplain in comparison with the main channel. This increases velocities on the floodplains and potentially generates faster travel times for floodwaves passing downstream on the floodplain than in the main channel.

The accelerating effects of the path length and slope on floodplain flows are diminished, however, by the effects of boundary friction. If floodplain flow depths are small, then the hydraulic radius will also be small and hence velocities will be reduced. Floodplain boundary roughnesses also tend to be higher than those in the main channel because of vegetation and obstructions such as hedges.

As noted earlier in Chapter 3, the retarding effects of boundary roughness tend to decline as the hydraulic radius or stage increase. This is particularly true for the broadly rectangular main channels such as those found in the River Fulda catchment. On the floodplains, however, the situation is complicated by vegetation and man-made structures. Petryk and Bosmajian (1975) showed that the boundary friction of vegetation is related to the drag and cross-sectional area of particular plant species. This drag may be increased when

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the floodplain inundation depth increases because debris may become trapped in hedges and fences causing an increase in boundary roughness. Klassen and Zwaard (1974) showed that the spacing of hedges and trees is critical in determining the debris build up and hence in computing the friction of floodplains.

The correct selection of the Manning's n coefficient for the floodplain flow segments for MILHY2, must consider not only the general land use but also the spacing and height of any hedges or fences and how the pattern varies with the inundation stage.

Overall, velocities on the floodplain tend to be lower than those in the main channel. This is because of the relatively greater frictions on the floodplain, whilst the travel time of the floodwave tends to be faster on the floodplain because of the shorter reach length. The faster floodwave travel times on the floodplain generate complexity in the prediction of two-stage flows. It has been assumed in the previous chapters that the primary direction of momentum transfer has been from the main channel to the floodplain. This effectively caused the floodplain flows to act as a drag on the main channel flows whilst the floodplain flows are accelerated by the main channel flows. In a sinuous reach, however, where the floodwave in the floodplain may be travelling downstream at a much greater rate than the floodwave in the channel, the transfer of momentum may be from the floodplain to the channel.

The concept of multiple routing reaches incorporates the effects of large flood events in two-stage channels, where the floodplain flows have a separate and direct route downstream crossing and recrossing the bankfull main channel. The

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importance of the routing path in two-stage channels is summarised by Fread (1976), who noted:

"The characteristics of the floodwave are influenced predominantly by the one-dimensional motion of the floodwave along the longitudinal axes of the river and the floodplain."

In the next section alternative methods of incorporating multiple routing reaches into the composite structure of MILHY2 are considered.

5.2 Modelling Alternatives

The objectives of incorporating the concept of multiple routing reaches into MILHY2 are:-

- 1) to improve the representation of the downstream routing of flow in two-stage channels by incorporating the effects of the sinuosity of the main channel and the short-circuiting of floodplain flows;
- 2) to improve the selection of the appropriate Manning's n coefficient for two-stage flows. By removing considerations of the sinuosity of floodplain flows, the selection of the correct n value should be simplified as it would then only incorporate the effects of boundary friction.

The aim of this chapter is therefore to develop a one-dimensional or quasi-two-dimensional technique that can be incorporated into MILHY2. Such a technique should therefore have:

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- little additional data requirements
- low computer demands
- be capable of validation

These restrictions leave several alternative approaches available. These are:-

- 1) To develop a stage/reach length relationship. This approach was suggested by Perkins (1970), when he incorporated a routine to increase reach length linearly from the main channel thalweg distance at bankfull to the shortest reach length dictated by the floodplain slope, at the maximum stage.
- 2) To develop an empirical adjustment to the roughness coefficients of the floodplain and main channel. This approach was suggested by Tingsanchali and Ackermann (1976), where the Manning's n value was weighted by the ratio of reach lengths between the actual floodplain distance and the schematized straight floodplain and main channel, such that:

$$n^* = n_f \frac{L_f^{3/2}}{L_{mc}} \quad 5.1$$

where n^* - adjusted Manning's n
 n_f - Manning's n floodplain
 L_f - reach length of floodplain
 L_{mc} - reach length of the main channel

- 3) Replace the Variable Storage Coefficient Routing routine in MILHY2, with a St. Venant technique utilizing a weighted four-point implicit difference solution

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modified by Fread (1976) to incorporate the differing path lengths of floodplain and main channel flows.

- 4) Separate floodplain and main channel flows and route, using the existing routines in MILHY2, assuming no exchange of flow along the reach.

The simplest solution to apply is approach four, where the inflow hydrograph is apportioned to the cross-sectional segments using the rating curves developed for the upstream cross-section. Each segment of the cross-section is then routed individually downstream using travel time tables developed for each cross-sectional segment. Conceptually this solution may seem rather simplistic and it does have several disadvantages. These are:-

- i) flow has to be apportioned to floodplain or channel at the top of the reach, and these proportions are fixed throughout the reach. This assumes that the cross-sectional geometry is fairly constant downstream;
- ii) there is no exchange of momentum between the main channel and floodplain along the reach;
- iii) floodplain flows on either bank cannot cross the main channel flows.

However, the other three possible alternatives also exhibit some of these and other disadvantages. None of the other three methods incorporate the momentum exchange between the main channel and the floodplain in either direction. Perkin's (1970) method assumes that there is a gradual transfer in the routing reach length from the main channel to the floodplain. This conceptually seems attractive because when floodplain flows are relatively small, the frictional effects of the

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sinuous main channel may be great; and as floodplain flow depths increase, the effects of the main channel are flooded out.

The approach suggested by Tinsanchali and Ackermann (1976), of modifying the Manning's n coefficient to incorporate the effects of the differing routing length, would not improve the ease of selection of the coefficient (objective 2 identified above). It would also not improve the physical representation of the pathways as it is essentially a calibration procedure.

Fread's (1976) method incorporated into the FLDWAV package (Fread, 1985) separates floodplain and main channel flow. In a similar way to the simple procedure proposed by MILHY2. The advantage of Fread's model, however, is that the mass balance between the floodplain and main channel flows are computed in each cell.

Despite the disadvantages of approach 4, it is an approach which seems to be a logical first step in tackling the problem of floodplain flows "short-circuiting" the main channel. Exchange of momentum between the main channel and floodplain has been incorporated at the valley-sections (see Chapter 4), and it is felt important at this stage to compare the sensitivity of the outflow hydrograph to the effects of variability in the momentum exchange routines, or the multiple routine of floodplain and channel flows. If, as Fread (1976) suggests, the downstream short-circuiting effects were identified as being significant, then it would be appropriate to investigate Perkin's approach as a simple alternative, or to attempt a more radical replacement of the routing subroutine with Fread's (1976) St. Venant solution.

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5.3 Application of Multiple Routing Reaches

Testing the impact of multiple routing on the accuracy of the predicted outflow hydrograph is an essential part of the model evaluation procedure identified by Sargent (1982). The relative importance of the technique compared to other modifications, and the sensitivity of the whole scheme to parameter variability, is investigated in Chapter 7. In this section, therefore, MILHY2 is applied using only the routing routines with an observed or generated hydrograph being input at the upstream end of the reach. The results of the analysis are notated such that the label MILHY2 refers to the model without the incorporation of the separate or multiple routing reaches. The notation multiple routing or m. routing implies the MILHY2 model with the multiple routing routine.

Multiple routing reaches were applied to a theoretical reach with rectangular cross-sectional geometry assumed to be constant downstream and an observed reach from the River Fulda, between Bad Hersfeld and Rotenburg. A variety of inflow hydrographs were applied to the theoretical reach, in order to investigate the impact of the depth of inundation on the travel time of the floodplain and the effects on the outflow hydrograph. The River Fulda, however, provided field data against which various roughness and routing lengths could be tested but with a limited number of observed flood events. A 1 in 10 year event was available and enough flood frequency data were available to generate a 1 in 100 year event, assuming a similar hydrograph shape to the 1 in 10 year event.

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5.3.1 Application to the Bad Hersfeld - Rotenburg reach

The results from the application of multiple routing reaches to the Bad Hersfeld to Rotenburg reach are found in Tables 5.1 and 5.2, and Figures 5.3, 5.4 and 5.5. As reported earlier in Section 4.4.1, the cross-sectional geometry at Bad Hersfeld is broadly rectangular with the floodplains being symmetrical about the main channel. The reach from Bad Hersfeld to Rotenburg is approximately 24km in length with a sinuous main channel; this can be seen in Figure 5.1. At Rotenburg, the bankfull depth is 4.8m as compared to 4.1m at Bad Hersfeld, with a bankfull discharge of $180\text{m}^3\text{s}^{-1}$. The valley section is asymmetrical at Rotenburg, with the left hand floodplain being approximately 300m wide whilst the right hand floodplain rises steeply. The bankfull width at Rotenburg is approximately 50m as compared with 30m at Bad Hersfeld.

When multiple routing is invoked, the observed hydrograph at Bad Hersfeld is apportioned to floodplain and main channel segments according to the rating curve developed from the Bad Hersfeld cross-section. The travel timetable is then developed for each cross-sectional segment using the smaller of either the upstream or downstream rating curve. The maximum floodplain values reported in Tables 5.1 and 5.2 are computed at the downstream station, that is Rotenburg.

The 1 in 10 year observed inflow at Bad Hersfeld and observed outflow at Rotenburg is shown in Figure 5.2. This figure shows that the travel time of the 1 in 10 year event between the two stations is approximately nine hours. The inflow hydrograph at Bad Hersfeld has been scaled up, in line with the flood frequency data available to provide the 1 in 100 year event, and consequently the 1 in 100 year event has the same form as the 1 in 10 year event. At Bad Hersfeld the 1 in

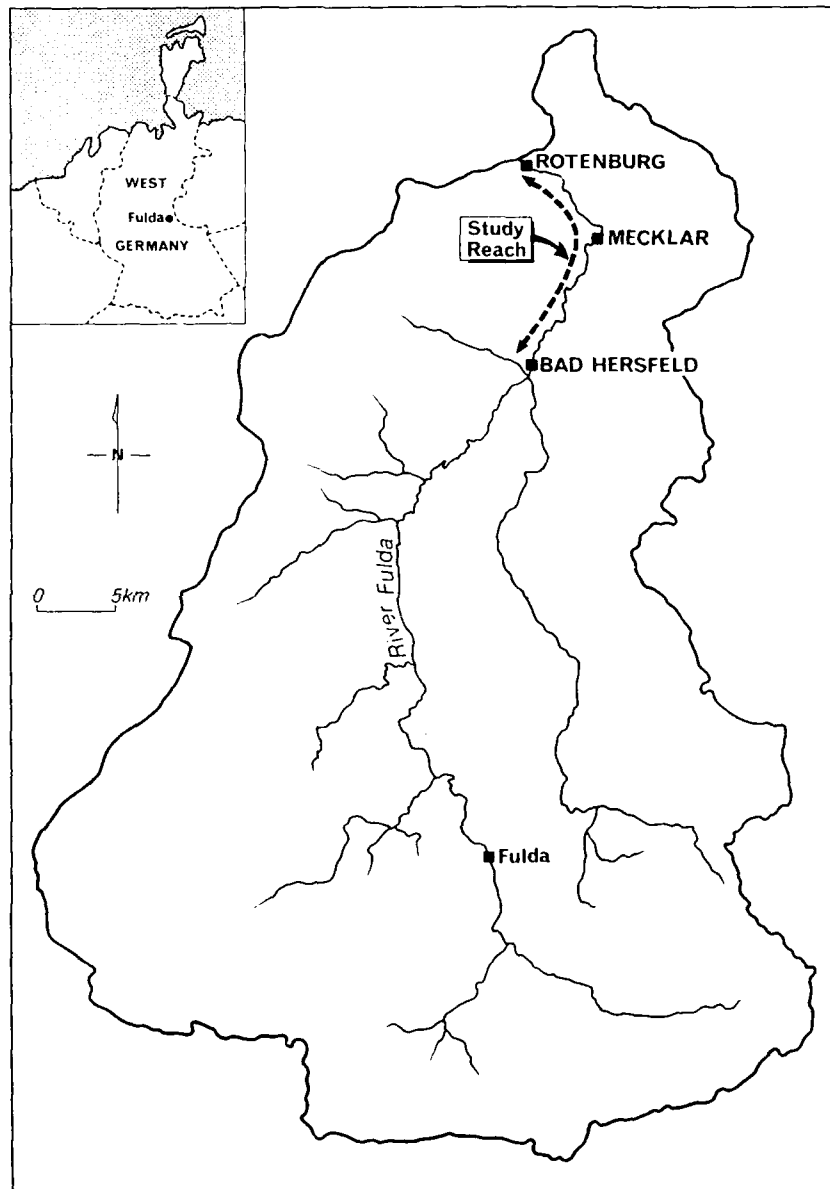


Figure 5.1
Location of the Bad Hersfeld-Rotenburg Reach
of the River Fulda, West Germany

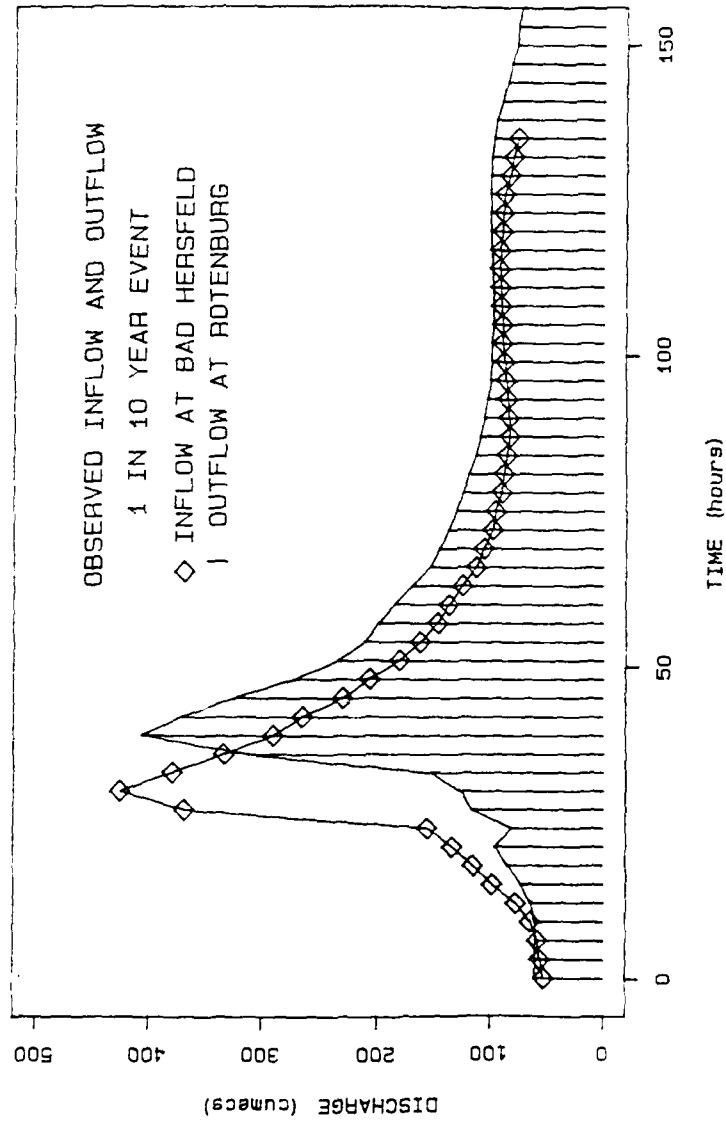


Figure 5.2
Observed Inflow and Outflow Hydrographs at Bad Hersfeld and
Rotenburg for the 1 in 10 Year Event

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100 year event corresponds to an increase in the floodplain inundation depth of approximately 1m over the 1 in 10 year event. At Bad Hersfeld the 1 in 100 year event corresponds to an increase in the floodplain inundation depth of approximately 1m over the 1 in 10 year event.

Figure 5.3 compares the observed outflow hydrograph at Rotenburg with the outflow hydrograph simulated by MILHY2 and the multiple routing technique. The greatest difference in the three hydrographs occurs in the overbank section of the hydrographs; the bankfull discharge is marked on the figure. The corresponding time to peak, peak discharge and maximum inundation depths of these three hydrographs are recorded in Table 5.1. Both Figure 5.3 and Table 5.1 show that the single routing technique used in MILHY2 effectively smooths the inflow hydrograph to too great an extent. This reduces the peak discharge and inundation depth. The multiple routing technique reduces the attenuation of the floodwave and thereby halves the HYM02 errors in both the peak discharge and the inundation depth. Table 5.1 also shows that the multiple routing technique produced a time to peak of 40 hours, two hours later than the observed peak. However, as the observed inflow and outflow hydrographs were digitised at three hour intervals, errors of less than three can be ignored.

As noted earlier in this section, the main objective of incorporating multiple routing reaches was to simulate the effects of the short-circuiting of floodplain flow, reducing the floodplain reach length. In the next simulation reported in Table 5.1, therefore, the reach length of the floodplain segments was reduced by 5%. This produced only very small variations in the outflow hydrograph in comparison with the multiple routing hydrograph shown in Figure 5.3.

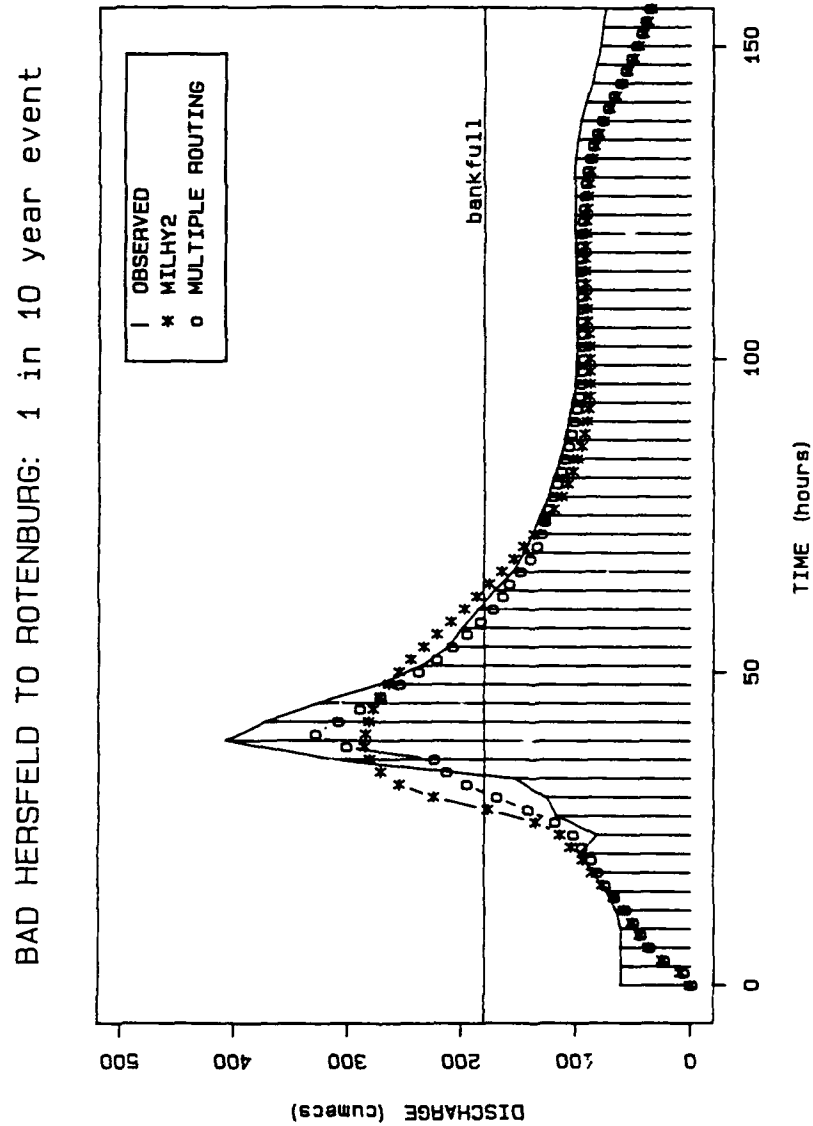


Figure 5.3
 Comparison of MILHY2 and Multiple Routing Reach
 Simulated and Observed Hydrographs for the 1 in 10
 Year Event at Rotenburg

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Table 5.1
Characteristics of Observed and Simulated Hydrographs
at Rotenburg, for the 1 in 10 Year Event

	Time to peak hours	Peak discharge $\text{m}^3 \text{s}^{-1}$	Maximum floodplain inundation m
Observed	38	407	0.33
MILHY2	38	285	0.09
Multiple routing	40	330	0.17
Multiple routing floodplain length ↓5%	40	333	0.18
Multiple routing floodplain length ↓30%	40	352	0.21
Multiple routing floodplain n ↓30%	38	355	0.22

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Analysis of the flood inundation maps available for the River Fulda indicated, however, that the floodplain reach length may be up to 30% shorter than the main channel. The hydrograph produced by reducing the floodplain reach length by 30% is shown on Figure 5.4. Comparison of Figures 5.3 and 5.4 shows that reducing the floodplain length by 30% makes a significant improvement in the accuracy of the prediction.

Figure 5.4 shows, though, that a similar effect can be achieved by reducing the Manning's n roughness coefficient by 30%. Chow (1959) showed that the effects of sinuosity of a channel can alter the n coefficient by up to 30%. As noted earlier, however, one of the objectives of this investigation is to incorporate processes operating in two-stage channels and reduce reliance of empirical coefficients.

Table 5.2 and Figure 5.5 show the simulation results for the 1 in 100 year event on the River Fulda reach. In contrast to the 1 in 10 year event, the MILHY2 prediction gives higher peak discharge results than the multiple routing reach. The percentage error between the MILHY2 and multiple routing technique is, though, much smaller in the 1 in 100 year storm being approximately 4%; whilst the 1 in 10 year event difference was 11%. This suggests that as the floodplain inundation depth increases, the cross-section behaves as a single system causing the solutions to converge.

BAD HERSFELD TO ROTENBURG: 1 in 10 year event

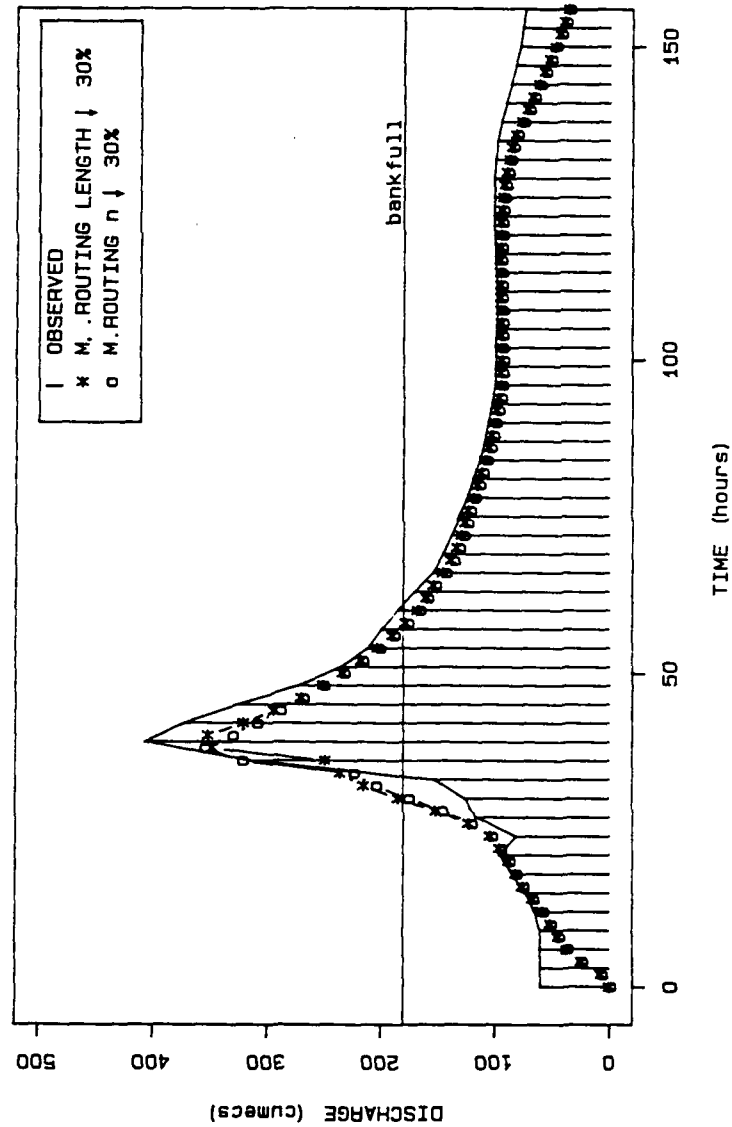


Figure 5.4
Comparison of Observed and Multiple Routing Reach
Simulated Hydrographs with Variations in Reach Length
and Boundary Roughness for the 1 in 10 Year Event

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Table 5.2
Characteristics of the Observed and Simulated Hydrographs
at Rotenburg, for the 1 in 100 Year Event

	Time to peak hours	Peak discharge $M^3 s^{-1}$	Maximum floodplain inundation m
Observed	38	744	0.90
MILHY2	38	665	0.78
Multiple routing	40	634	0.73
Multiple routing floodplain length ↓30%	36	684	0.81
Multiple routing floodplain n ↓30%	36	668	0.78

BAD HERSFELD TO ROTENBURG: 1 in 100 year event

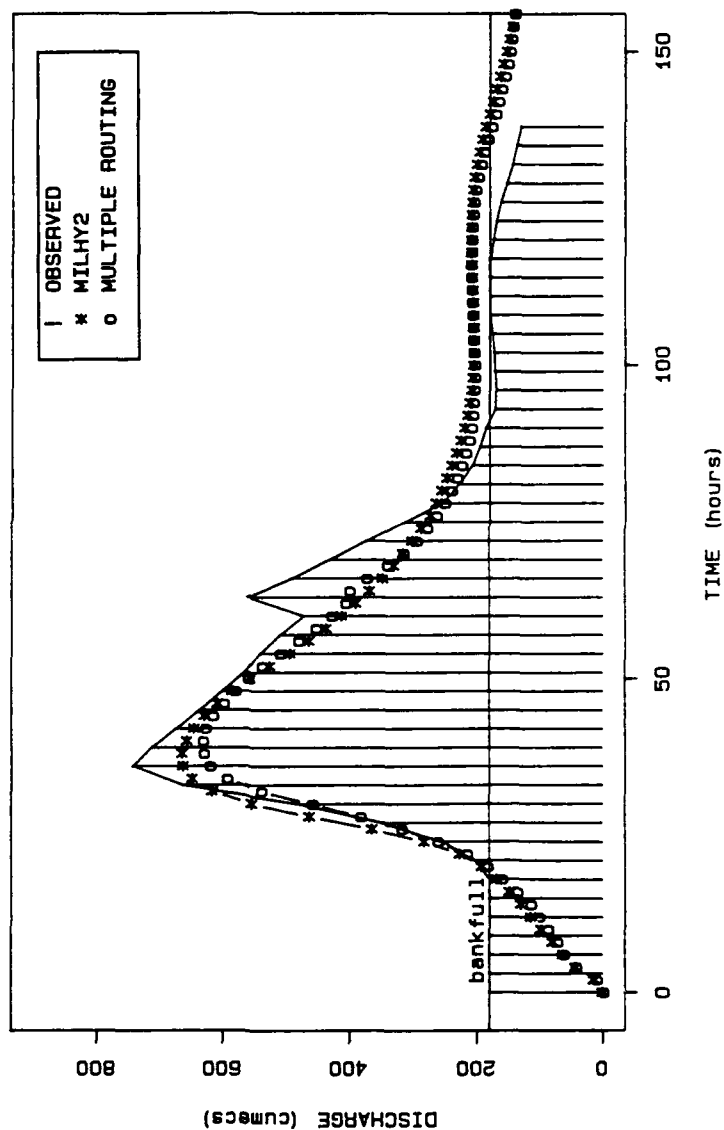


Figure 5.5
Comparison of MILHY2 and Multiple Routing Reach
Simulated and Observed Hydrographs for the 1 in 100
Year Event at Rotenburg

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5.3.2 Application to a hypothetical reach

The aim of investigating the impact of multiple routing on a hypothetical reach was to examine the relative impact of the floodplain inundation depth on the outflow hydrograph. A hypothetical reach was set up with symmetrical rectangular cross-sections at upstream and downstream stations with floodplain/main channel width ratios of 10. The main channel was a constant depth of 2.4m, so that the main channel capacity remained constant downstream. This meant that the proportion of flow on the floodplain was correct throughout the reach and, therefore, the analysis could concentrate solely on the effects of the floodplain inundation depth.

Table 5.3 and Figures 5.6 and 5.7 summarize the results from this investigation into the impact of floodplain inundation depth on the outflow hydrograph. In all these simulations, the floodplain and channel reach length were held constant at 20km. The seven inflow hydrographs were generated by scaling the 1 in 10 year observed hydrograph from Bad Hersfeld (see Figure 5.2).

Analysis of Table 5.3 shows that the predictions from the two techniques converge as the floodplain/main channel depth ratio increases to 0.8. When the depth ratio is less than 0.4 the MILHY2 model gave greater peak discharge predictions. As depth ratios increased up to 0.5, the multiple routing routine generated greater peak discharge predictions. The maximum error between the two techniques occurs when the depth ratio is approximately 0.3.

When floodplain inundation depths are small, that is with a depth ratio of 0.4, the MILHY2 generated larger peak discharges because the separate floodplain flows of the multiple routing routine are retarded by the effects of

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Table 5.3
Hydrograph Characteristics
Hypothetical Reach Application

Storm	Multiple		Time to peak hours	Peak discharge $m^3 s^{-1}$	Maximum floodplain inundation m
1	1	MILHY2	36	309.8	1.16
		M. Routing	36	317.4	1.18
2	0.1	MILHY2	36	34.8	-
		M. Routing	36	32.8	-
3	0.2	MILHY2	42	57.2	0.09
		M. Routing	36	55.0	0.07
4	0.5	MILHY2	40	139.5	0.61
		M. Routing	38	126.9	0.56
5	1.5	MILHY2	36	536.0	1.68
		M. Routing	36	540.8	1.74
6	2	MILHY2	34	682.0	1.98
		M. Routing	34	685.0	1.99
7	3	MILHY2	34	1061.0	2.65
		M. Routing	34	1062.0	2.65

HYPOTHETICAL REACH: STORM1

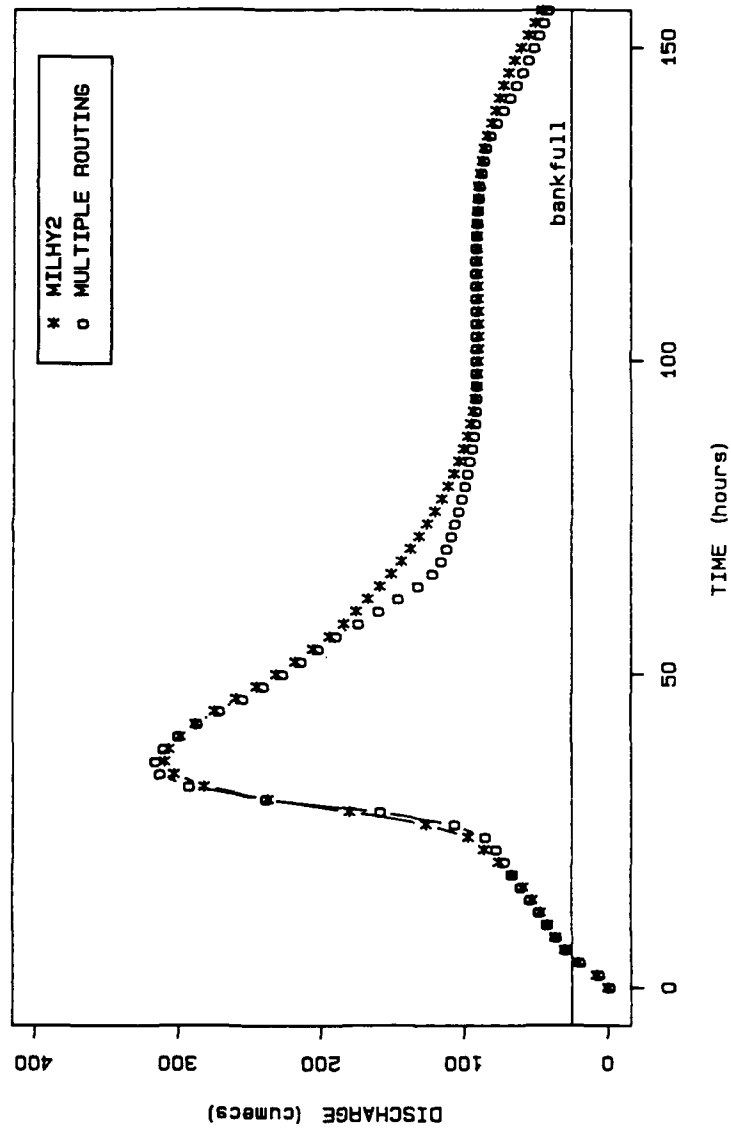


Figure 5.6
Comparison of MILHY2 and Multiple Routing Simulations
of Storm 1 in a Hypothetical Reach

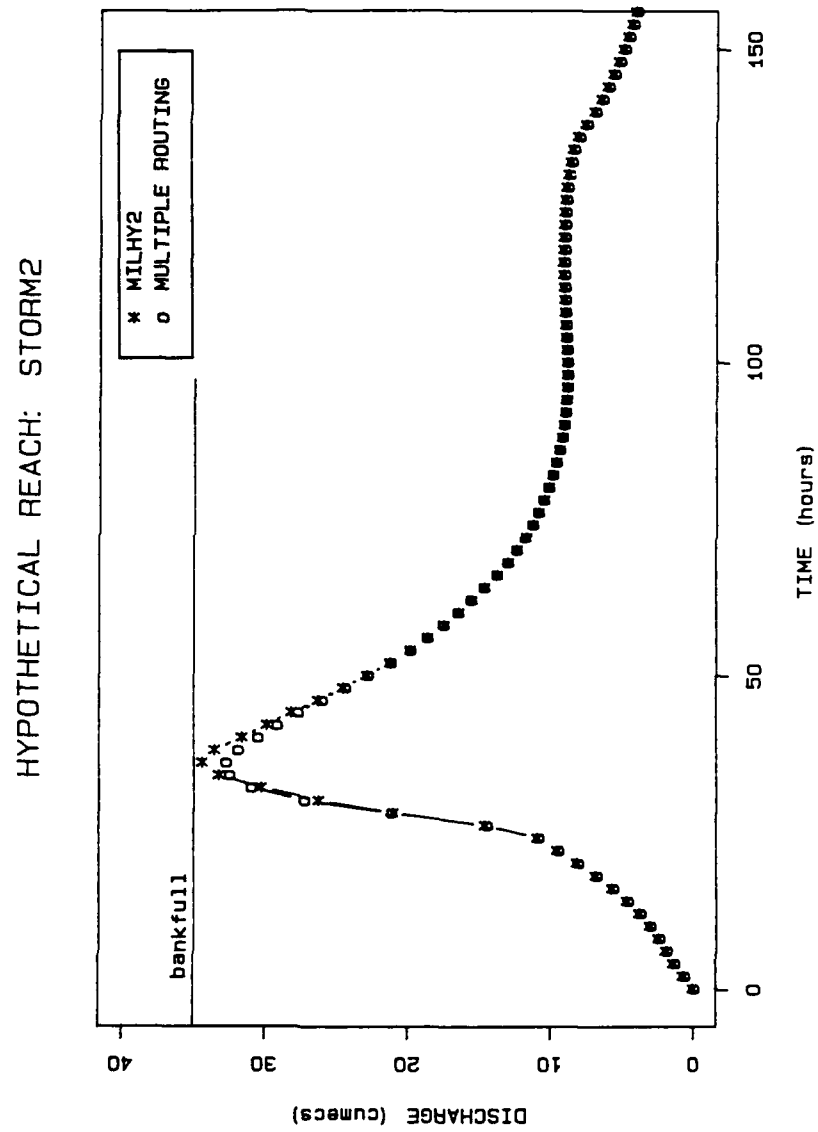


Figure 5.7
Comparison of MILHY2 and Multiple Routing Simulations
of Storm 2 in a Hypothetical Reach

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boundary friction. However, as Table 5.3 shows, the percentage error between the two methods is small when the depth ratio is very small and increases as the depth ratio increases up to 0.4. When the depth ratio is very small, that is less than 0.1, the majority of the flow is carried in the main channel and in the multiple routing routine the wetted perimeter of the main channel is small. The flow in the floodplain segments in the multiple routing routine in very low flow conditions will have very large travel times but they only contribute a small percentage of the total discharge predictions. In MILHY2 simulations with very small floodplain inundation depths, the wetted perimeter calculations include the floodplain boundaries as well and, therefore, flows are retarded by the effects of boundary friction.

As the depth ratio increases beyond 0.1 but remains smaller than 0.4, the *percentage of discharge* on the floodplain is large enough to be a significant part of the hydrograph. However, the inundation depths on the floodplain are not great enough to overcome the effects of boundary friction and, therefore, the MILHY2 simulation generates much larger peak discharge predictions.

When inundation depths are larger, that is greater than 0.4, the multiple routing routine generates peak discharge predictions greater than the MILHY2 prediction. This is because the inundation depth of the floodplain is large enough to overcome the effects of boundary friction and the travel time of the floodplain flows at this stage is small. The predicted outflow hydrograph of the multiple routing routine is then less attenuated and so a larger peak discharge is produced. However, the error between the two methods is small, and is not greater than 2%.

Table 5.3 also compares the effects of reducing the

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length of the floodplain routing length on the peak discharge prediction. The table shows that when the depth ratio exceeds 0.1, the reduced multiple routing length generates larger peak discharge predictions than the MILHY2 simulation. Table 5.3 shows, therefore, that the effect of reducing the length of the floodplain routing reach is to decrease the inundation depth from 0.3 to 0.1. At this depth, the multiple routing routine lowers the attenuation of the floodwave below that of the MILHY2 simulation.

5.3.3 Conclusions

1. The maximum impact of the multiple routing technique occurs when the floodplain inundation depths are small.
2. At these small inundation depths (depth ratios = 0.3), when the utilization of multiple routing significantly improves the prediction of the peak discharge, errors are halved.
3. Reducing the floodplain routing length by 30% reduced the travel time of the peak discharge and decreased the attenuation of the floodwave.
4. Reducing the floodplain Manning's n coefficient by 30% reduces the attenuation of the floodwave to a similar degree as 30% reduction in the routing length of the floodplain flow segment.

5.4 Implications for the Improvement of MILHY2

The concept of multiple routing routines has been approached hydrologically and hydraulically. Tingsanchali and Ackerman (1976) use a hydrologic empirical adjustment of the

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Manning's n coefficient (equation 5.1), which is incorporated into a hydrologic linear adjustment of the routing length with increasing stage, this is incorporated into a hydraulic routing model. Perkins (1970) utilizes a similarly hydrologic linear adjustment of the routing length with increasing stage; this is incorporated into a hydraulic routing model. Fread's (1976) approach is the only hydraulic based routine; this routine is incorporated into a full hydraulic reach model. The approach utilized in this chapter is the simple separation of floodplain and channel flows and discrete routing of them downstream. This approach has not been used in either hydrologic or hydraulic routines. Conceptually, this approach has advantages over empirical adjustments and still provides the potential for the incorporation of a more sophisticated adjustment of stream flow length with a stage such as the scheme proposed by Perkins (1970). Importantly, this is also the first application of the concept of multiple routing with an hydrologic or storage flood routing routine.

The results of the analysis of the multiple routing routine suggest that the predictive accuracy of MILHY3 could be improved by the introduction of the module. This is particularly true for flow conditions where floodplain inundation depths are great enough for the floodplain water to be flowing rather than stored and where floodplain flows contribute a significant proportion of the total discharge from the reach. The introduction of the multiple routing concept, therefore, has been shown to be important even when the hydrological flood routing techniques are used.

The results also showed that decreasing the floodplain routing length by 30% reduced the attenuation of the floodwave and that a similar effect could be achieved by reducing the floodplain Manning's n coefficient by 30%. This shows that Chow's (1959) suggestion that the sinuosity of the channel

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accounts for 30% of the Manning's n coefficient value, is correct. It also highlights the importance of boundary friction. Comparison of Figures 5.3 and 5.4 shows, however, that the introduction of the multiple routing routine and adjustment in the Manning's n coefficient have a similar effect on the shape of the predicted hydrograph. It is concluded, therefore, that the multiple pathways of floodplain and main channel flows and boundary friction effects are of equal importance.

The positive results of this chapter support the further evaluation of the multiple routing routine. This analysis has shown that a simple technique can improve the predictive performance of the model and incorporate the basic behaviour of the two-stage channel. Further evaluation of the multiple routing routine as part of a catchment model is considered in Chapters 7 and 8.

5.5 Summary

This chapter has investigated methods of incorporating the multiple pathways taken by floodplain and main channel flows. A simple routine has been selected which apportions flow to floodplain and channel segments and routes them downstream using separately derived travel timetables. This approach allows the adjustment of the lengths of the reaches, thus incorporating the effects of the sinuosity of the main channel on the travel time of the hydrograph.

Application of the routine to a hypothetical and a field reach show that the routine improves the prediction of the hydrograph, by the more accurate simulation of the attenuation of the hydrograph. The results of this application show that

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the introduction of the multiple routing routine has just as great an effect on the hydrograph as variation in the Manning's n coefficient. The multiple routing routine, therefore, is included in the final evaluation and validation of MILHY3.

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Chapter 6

Validation of MILHY3I - A Strategy for Model Validation and Evaluation

The third objective of this report, outlined in Chapter 1, is to validate the new model, MILHY3, which incorporates the new turbulent exchange and multiple routing modules. The validation of MILHY3 needs to answer the following questions:

- 1) Do the mathematical algorithms introduced represent the processes we are trying to model?
- 2) Are the mathematical algorithms robust?
- 3) Is the accuracy of the predicted outflow hydrograph a significant improvement over earlier versions of MILHY?
- 4) Is the resolution of each new module appropriate for ungauged applications?
- 5) Can a set of operational rules be developed for MILHY3?

The strategy for the validation programme can be divided into three parts. Part I, reported in this chapter, provides an introduction to the basis of model validation and evaluation. It will also include the identification and establishment of a data set for the validation procedure. In Part II of the programme, reported in Chapter 7, a hydrological validation will be

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undertaken, investigating the sensitivity of the composite modelling structure. In Part III, a hydraulic analysis is undertaken to investigate the utility of using hydraulic models to provide "ground-truth" records against which MILHY3 may be evaluated. This analysis is reported in Chapter 8.

As noted earlier in Chapter 4 the validation of MILHY3 will follow the model evaluation programme developed by Sargent (1982). Sargent's programme provides one of the few structured approaches available for model evaluation and consists of three parts; mathematical validation, computerised model verification and operational validation.

Mathematical Validation

The first of these stages is the mathematical validation of the model. Howes and Anderson (1988) noted that the objective of this section of the model evaluation programme is to:

"establish that the assumptions made about the real system by the model are reasonable and that the model adequately reflects the essential features and behaviour of the real system which are relevant to their application in mind."

However there is no deterministic method of testing the assumptions and representativeness of a model; instead the procedure must be rather subjective. In any modelling exercise it is important that the modeller has a clear understanding of the processes active in the physical system to be modelled so that the implications of the assumptions necessary to model the system are fully appreciated. In this report, the importance of appreciating the potential applications and uses of the

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model in the selection of a modelling strategy, has also been stressed. This 'user's perspective' of the modelling strategy determines to a greater degree the selection of the modelling techniques than the ability and availability of techniques or solutions to modelling problems.

Computerised Model Verification

This second stage of model evaluation attempts to check the transfer of the mathematical techniques, that approximate the physical environment, into a computer code. The verification procedure needs to ensure, therefore, that several aspects of the program are checked.

- 1) that the program is internally valid, (Hermann, 1967). This ensures that if all the program input data and controls are kept constant then the output from the model remains constant;
- 2) that there is conservation of mass. That is that the volume of flow entering the simulation is matched by the output from the simulation;
- 3) that the model behaves under a range of input conditions and the limits of the physical conditions that a model can handle are specified.

One of the most effective tools available for computerised model verification is a sensitivity analysis. A sensitivity analysis measures the change in one variable when one or more parameters are varied. Other available techniques utilise hand calculations to check the validity of simple routines.

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Operational Validation

The third stage of the model evaluation programme involves measuring the accuracy of the model to predict the behaviour of the natural environment, and developing a set of rules or guidelines for the operation of the model.

Alternative strategies of undertaking an operational validation programme include the use of goodness-of-fit and error estimates, both utilised by Howes (1986). These have been used primarily with data sets of field data, however the availability of field data is not always guaranteed. The availability of field data is a particular problem for extreme events which by their very nature occur infrequently and are difficult to measure.

It is proposed in this report that the model in question can be tested against other modelling strategies utilising the goodness-of-fit and error estimates. The technique is potentially useful in assessing the accuracy of models in extreme conditions when field data is unavailable or the record is not long enough. Assessing one model's performance against another could potentially replace some hardware flume scale models.

6.1 Present Model Evaluation Status

The importance of a systematic model evaluation programme was appreciated at the initiation of this report. The programme therefore began with the selection of the most appropriate modelling solutions to the incorporation of the effects of momentum exchange and multiple routing in two-stage channels in ungauged catchments. The evaluation procedure continued with an initial analysis of the module's performance. All three

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stages of Sargent's evaluation programme have therefore already been initiated.

The validation programme has so far utilised a data set from the River Fulda, West Germany and hypothetical data sets based on rectangular channel geometries. It is accepted therefore that the results of these simulations are by no means exhaustive. However, given the limited availability of data sets for extreme events and the time available for this project, it is not the aim of this report to undertake an extensive validation procedure. This does not under-emphasize the importance of the validation procedure, rather it was felt that a more pressing objective would be to investigate alternative techniques of validation. The validation procedure will therefore concentrate on a limited number of simulations and maximize the interpretations possible from these simulations.

The results of the validation procedure so far, therefore, have been summarised below.

6.1.1 Mathematical validation

The selection of the momentum exchange and multiple routing techniques involved an investigation of the alternative techniques available. The philosophy forwarded in this report is that there is a need for models to be developed from the perspective of the potential user rather than from the state-of-the-art conceptual or technical progresses in hydrology. This report therefore proposes that the most important part of Howes and Anderson's (1988) statement on mathematical evaluation, noted above, is that "the model adequately reflects the essential features and behaviour of the real system which are relevant to their application in mind".

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In the selection of the most appropriate techniques for the modelling of the momentum exchange and multiple routing, the primary determining factor was the limitations posed by the ungauged catchment perspective. These limitations were:

- 1) the data requirements of the new algorithms should be small; in particular, field work should not be required;
- 2) any additional demands made of the user in the establishment of the data sets should not require detailed hydrological knowledge of the physical processes or computer expertise.
- 3) The computer demands, in terms of CPU and operating space, of the new routines should allow the application of the model to the IBM-PC level.

Both the momentum exchange and multiple routing routines developed in Chapters 4 and 5 meet these limitations. It is accepted, however, that the solutions proposed are not the only feasible alternatives to meet these limitations. However, the techniques selected from the shortlist of techniques identified in Chapters 4 and 5 were, it was felt, the simplest techniques to implement that still incorporated the essential features of the behaviour of the system in question. If the increased resolution of the modelling of two-stage channels should prove significant in improving the predictive accuracy of the catchment model, then some of the other technical solutions should be reconsidered.

The mathematical validation of the momentum exchange

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and multiple routing modules (sections 4.3 and 5.2) show, therefore, that the modules 'adequately represent' the real system and that the assumptions made are reasonable for a first assessment of the relative importance of two-stage modelling.

6.1.2 Computerised model verification

The simulations reported in Chapters 4 and 5 of the performance of the momentum exchange and multiple routing modules, serve to illustrate that the modules are internally valid (Hermann, 1967) and the algorithms are mathematically robust. The new model version, MILHY3, given the same model control parameters and data set, will simulate an identical set of results to the earlier MILHY2. This confirms that the coding of the new routines has not altered the continuity of the program.

The results reported in Chapters 4 and 5 also appear to be logical. When more turbulent shear stresses are introduced into the momentum exchange routines, for example, the predicted capacity of the cross-section is reduced. Similarly, when the length of the floodplain routing length is reduced, the travel time of the floodplain is reduced.

The behaviour of the modules under 1 in 10 year and 1 in 100 year events has been tested. The behaviour of much smaller events in hypothetical reaches has also been tested. Tables 4.3, 4.4 and 4.5 explore the application of the four momentum exchange routines to the Bad Hersfeld station on the River Fulda. Tables 4.6, 4.7, 4.8, 4.9 and 4.10 explore the impact of the same four techniques on a hypothetical reach. These tables show that the routines are stable for a variety of geometrical conditions and

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boundary roughness values. Table 4.3 also confirms the correct coding of the routines by comparing MILHY2 solutions with momentum exchange method 2. Both of these techniques utilize the same interface inclination method and have an apparent shear stress ratio of 1. Table 4.3 confirms, therefore, that identical computation methods produce identical results.

Tables 5.1 and 5.2 report the results of the application of the multiple routing routine to the Bad Hersfeld-Rotenburg reach on the River Fulda. Table 5.3 reports the results of the application of the routine to a hypothetical reach. These tables show that the multiple routing routine is stable for a variety of storm events and that there is continuity in the conservation of mass in the routine.

Application of the momentum exchange and multiple routing routines confirms, therefore, that the codings remain stable under a range of conditions and that the routines seem to operate logically.

6.1.3 Operational validation

The initial simulations reported in Chapters 4 and 5 suggest that both the momentum exchange and multiple routine modules make a significant improvement in the predictive accuracy of the hydrograph. The improvement in the predictive accuracy generated by the two models supports the advancement of the two modules into the next stage of the validation procedure.

This initial analysis, therefore, has confirmed the mathematical validity of the two modules and verified their computer coding. The analysis has also suggested that the predictive performance of the modules is an

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improvement over the previous techniques. However, there are still several important questions in the evaluation of the new model that must be answered. These outstanding questions are:

- 1) Is the outflow hydrograph more sensitive to variability in the physically-based parameters, or to the process submodels utilised?
- 2) What is the relative impact of the submodels introduced in MILHY3 in comparison with the infiltration algorithm introduced in MILHY2?
- 3) What is the impact on the outflow hydrograph of the conflicting effects of the new submodels?
- 4) What are the effects of the scale of the catchment on the three questions posed above?

These questions show the need to investigate both the verification of the computer modelling and the validation of the operational performance of MILHY3.

6.2 Design of a Model Evaluation Strategy

The analysis of the initial evaluation of the new modules has shown the need for further computerized model verification and operational validation.

The computerized model verification procedure must investigate the relative impact of the new modules when they are part of a catchment simulation model. It must also investigate the interaction between the new modules. Perhaps most importantly, the verification procedure needs to investigate the effects of variability in the model

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structure. The composite structure of MILHY3 offers differing module resolution solutions for both the simulation of the runoff excess (Curve Number and Infiltration Algorithm) and the simulation of channel routing. The composite structure therefore generates the need for a sensitivity analysis that will investigate not only the sensitivity of the outflow hydrograph to variability in the parameters but also the sensitivity of the predicted outflow to variation in the structure of the model.

A well structured sensitivity analysis could provide answers to all the questions posed above. An operational validation is still required, however, to establish the accuracy of the new routines.

The undertaking of a sensitivity analysis and an operational validation requires the availability of a data set. As noted earlier, however, the provision of data sets for extreme events is not necessarily easy. A sensitivity analysis can utilise relatively small amounts of field data and still achieve an acceptable level of accuracy. The number of data sets required for an operational validation, however, is much greater. One of the objectives of this report is therefore to investigate the utility of using other models to validate the operational performance of a new model.

The evaluation of the MILHY3 model will therefore be split into two parts; the sensitivity analysis and the operational analysis. Both of these analyses will utilise the same data sets collected from the River Fulda. The problem of data availability is a common one in model evaluation and it is felt that an investigation into alternative techniques and the maximizing of the utility of the data available would be profitable. The implement-

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ation and results of the sensitivity analysis is reported in Chapter 7 and the operational validations are reported in Chapter 8. In the the rest of this chapter, the Fulda data set is established.

6.3 Selection of a Field Catchment

There are three sources that generate prerequisites on the selection of a suitable data set for the validation and evaluation of MILHY3. These three sources are:

- 1) the outstanding questions to be answered in the evaluation and validation of MILHY3. These outstanding questions are listed in section 6.1;
- 2) the capabilities of MILHY2, in particular the boundary conditions for which the model has been validated, see section 2.2;
- 3) data characteristics or additional information that make the model evaluation simpler or more efficient.

These three sources place differing prerequisites on the selection of a suitable field catchment. The first and the second of the groups of prerequisites, noted above, must be met; the third group is not essential.

6.3.1 Prerequisites of a study catchment

1. Outstanding questions

- a) The catchment should be large so that it can be divided into a number of smaller subcatchments. Each of these subcatchments should have a gauging station for which

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the cross-sectional geometry is known and for which rating curves and hydrographs are available. A multi-subcatchment environment will allow the establishment of a suite of catchment scales. These requirements will allow the investigation of the effects of the inclusion of the infiltration algorithm in a multi-subcatchment environment and allow the relative impact of the new modules to be assessed.

- b) The catchment must be subject to floodplain inundation so that the new modules may be utilized.

2. MILHY2 limitations

All of these limitations are generated from the operational conditions set out by Williams and Hann (1973) and Howes (1986). These limitations represent the environmental conditions for which the process modules have been incorporated and for which the model has been validated.

- a) The catchment must be in a temperate region with a minimum of forested area.
- b) The catchment must not exceed a maximum area of 2500km².
- c) There should be a minimum of man-made interferences in the catchment, such as urban areas or land drainage schemes.
- d) A minimum data set should consist of:
 - topographic maps
 - soils classificatory maps
 - upstream and downstream valley cross-sections, rating curves and hydrographs

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- precipitation data corresponding with the observed hydrograph events.

3. Evaluation convenience

- a) Selection of a relatively simple and regular cross-sectional geometry would allow the importance of the two new modules to be assessed in the most unambiguous of environments. A simple cross-sectional geometry would make comparison of the four multiple routing routines easier. Regularity of the cross-sectional geometry of the floodplain environment between gauging stations would provide the environment for which the multiple routing routine would make the greatest improvement in the predictive accuracy of the model.
- b) Additional data such as a flood frequency analysis and a greater resolution in the rainfall, soils, or topography parameters, would allow more accurate comparison between observed and simulated results.

6.3.2 The River Fulda Catchment

These prerequisites limited prospective study catchments to the rural regions of Western Europe and areas of the U.S.A. From a short list of regions meeting the prerequisites, the River Fulda catchment in West Germany, (see Figure 6.1) was selected, primarily because of the efficiency and rapid response to requests for data from the relevant water authorities and meteorological offices. As well as the prerequisite data, the local authorities* in the River Fulda catchment were able to provide:

- i) an outline of the extent of floodplain inundation for a storm event in 1946, corresponding to the 1 in

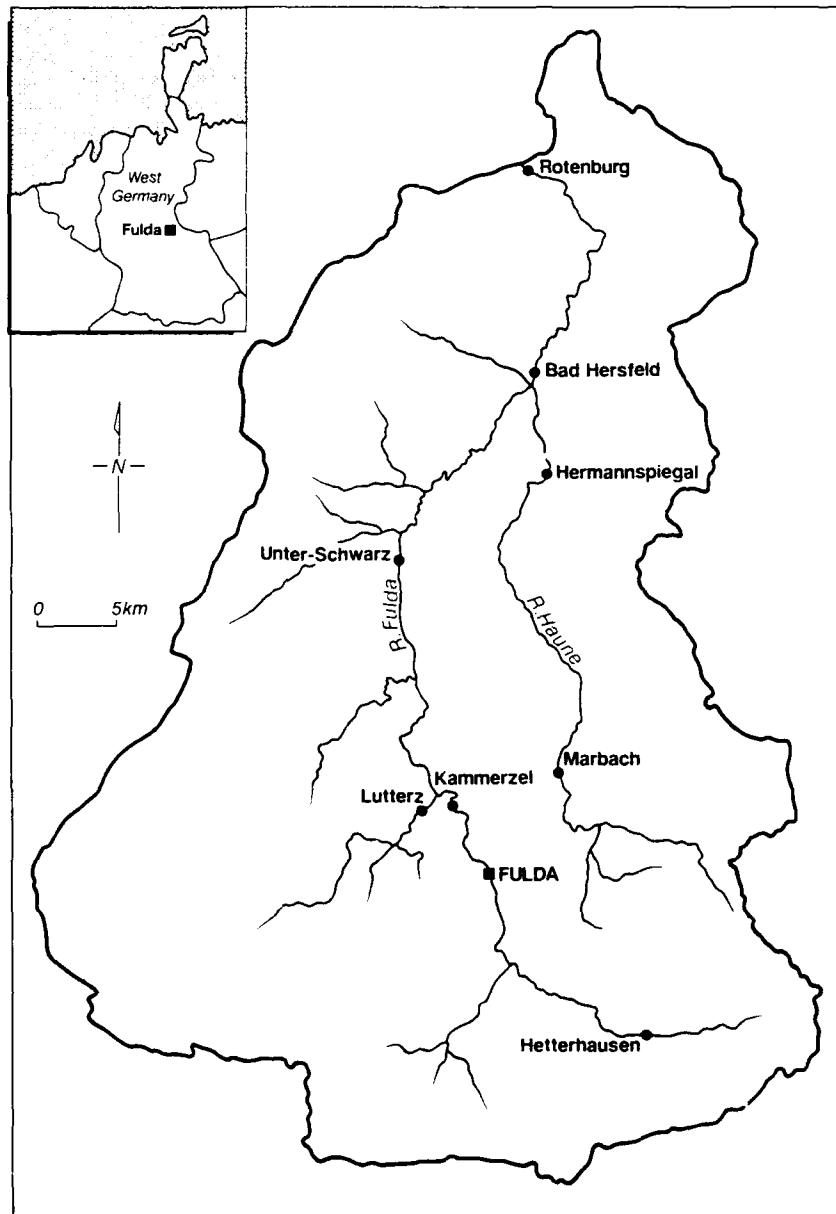


Figure 6.1
River Fulda Catchment and Gauging Stations

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200 year event.

- ii) daily precipitation values for approximately 45 rain gauge stations (see Figure 6.2)
- iii) continuous rainfall data for two stations, Bad Hersfeld and Kunzell-Dietershausen
- lv) for one storm, the water-equivalent of snow, daily minimum and maximum, temperature, relative humidities and cloud cover
- v) long-profiles of two of the reaches, between Bad Hersfeld and Rotenburg, on the River Fulda, and between Marbach and Hermannspiegel, on the River Haune (a tributary of the River Fulda).

The provision of further meteorological data provides the potential for the use of the River Fulda catchment in the simulation of snow melt events. Although snow melt events are not considered in this report, the potential for the simulation of such events is available within the MILHY project (Pangburn, 1987). This, therefore, provided further impetus for the selection of the River Fulda catchment.

* The help and cooperation of the following authorities in the provision of the data is acknowledged: Water Authority, Wasserwirtschaftsamt, Fulda, for the provision of the hydrological data, and the Meteorological Office, Deutscher Wetterdienst Zentralamt, Offenbach, Frankfurt, for the meteorological records, collected during three visits to the catchment in the period November 1986 to June 1988. The soils classificatory maps were supplied by the Environmental Laboratory, Waterways Experiment Station, US Corp Engineers, Vicksburg, Mississippi.

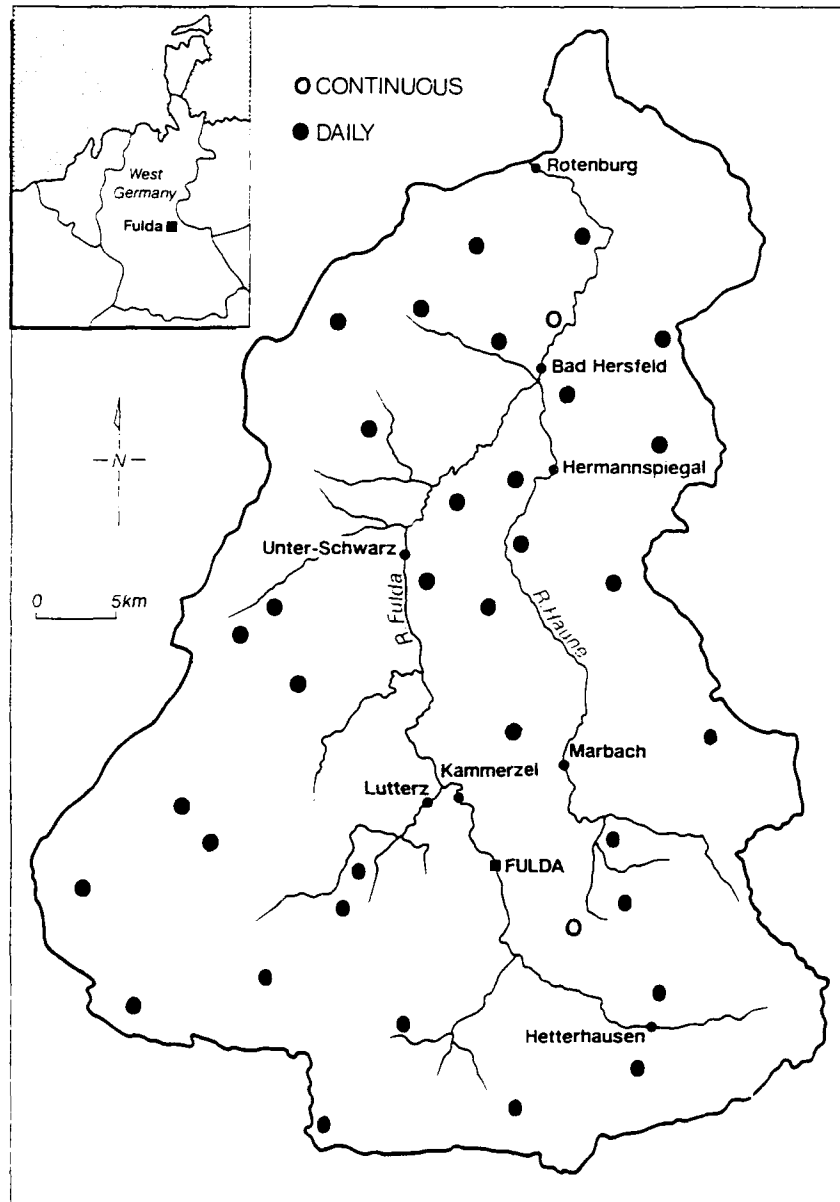


Figure 6.2
Raingauge Network for the River Fulda Catchment

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6.4 Establishment of the River Fulda Catchment

The River Fulda catchment to Rotenburg consists of a drainage area of approximately 2523km^2 , drained by the River Fulda and its tributaries. The main tributary is on the River Haune, which joins the Fulda at Bad Hersfeld; in addition, the River Luder joins the Fulda at Lutterz. There are eight river gauging stations in the catchment, marked on Figure 6.1, for which six storm events have been collected. The positions of the gauging stations have enabled the division of the catchment into nine subcatchments, depicted in Figure 6.3.

During the visits to the catchment, drawings were made and photographs taken that enabled the technical channel cross-sections to be extended across the floodplains. Estimates were also made during these visits of the Manning's n roughness values of the channels and floodplains throughout the catchment. Figures 6.4, 6.5 and 6.6 are photographs taken at Hetterhausen, Unter-Schwarz, and Rotenburg, and show the topography and land-uses typical throughout the catchment. In the photographs it can be seen that:

- i) in the upper reaches the channel is tree-lined
- ii) the floodplains are extensive and relatively flat
- iii) the floodplains throughout the catchment are vegetated by short grass
- iv) there are few obstructions on the floodplains, there are few fences, and the small villages tend to have been built clear of the areas subject to flooding

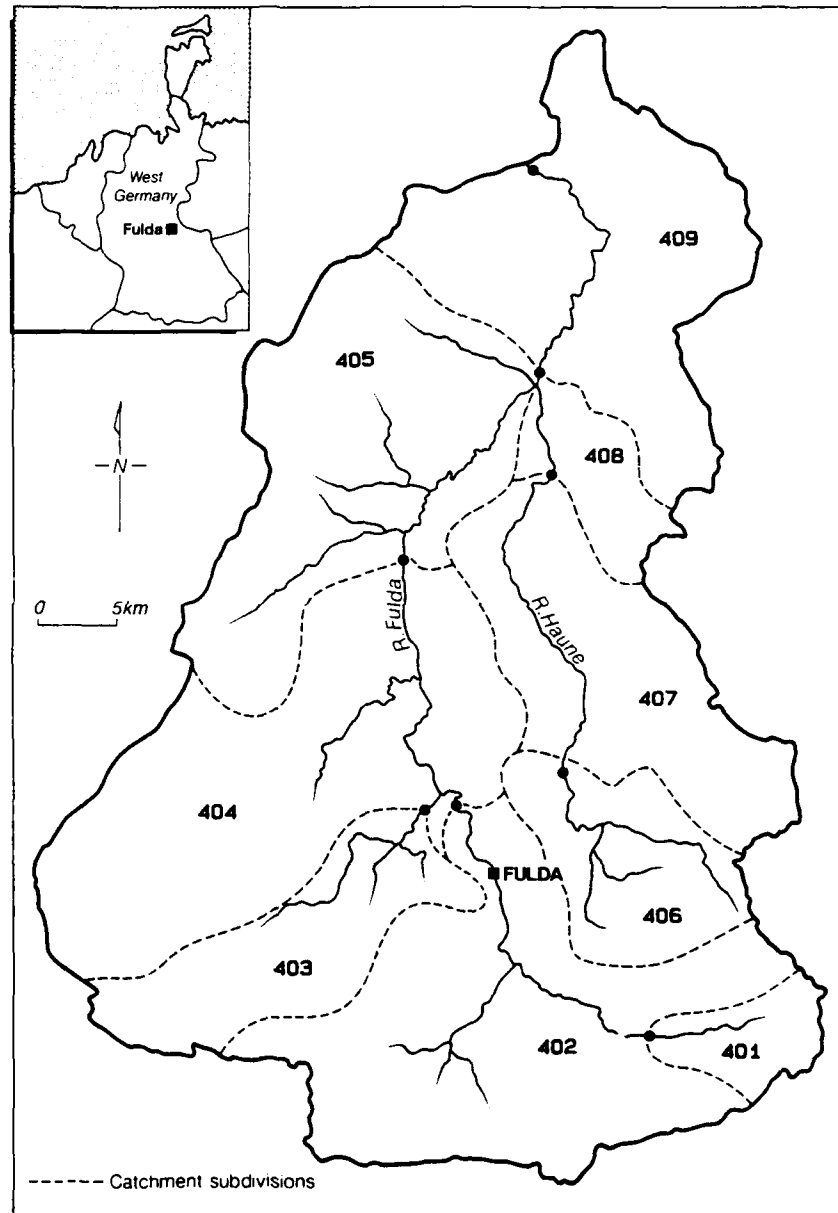


Figure 6.3
Catchment Subdivision of the River Fulda Catchment



Figure 6.4
Cross-Section at Hetterhausen, River Fulda

Figure 6.5
Cross-Section at Unter-Schwarz, River Fulda



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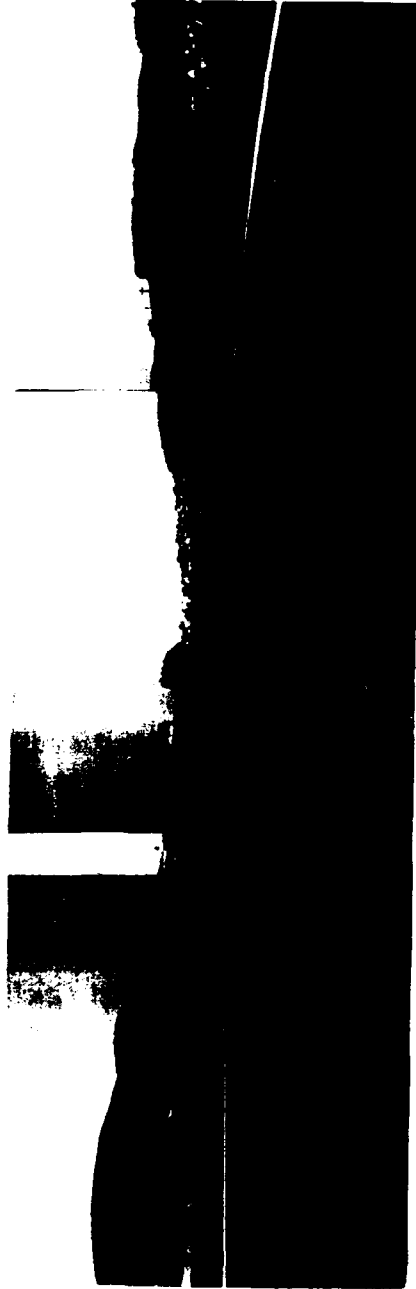


Figure 6.6
Cross-Section at Rotenburg, River Fold

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v) the channel is broadly rectangular in cross-section

vi) the channel is sinuous

Tables 6.1 and 6.2 collate some of the topographic dimensions of the subcatchments and the channel geometries at the gauging stations.

Six storm events were identified as being discrete events, that is where the hydrographs rose and fell back to baseflow conditions within a single seven day record. For each of these events, the daily rainfall totals for the three preceding weeks were collected in order to compute antecedent conditions.

In order to compute the rainfall in each of the nine subcatchments, the Theisson polygon technique was used to weight the daily rainfall total from each of the 45 rain gauges shown in Figure 6.2. Polygons of the area associated with a particular raingauge were drawn as if the catchment had no relief.

Table 6.3 shows the percentage occurrence of each of the major soil groups in each of the nine subcatchments. A certain amount of interpolation and generalisation occurred during the computation of this table, as the pixel definition of the soils classificatory maps was 1 pixel = 100 metres. The use of a graphics tablet attached to an IBM-AT, however, considerably speeded the computation of both the raingauge polygons and soil group areas.

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Table 6.1
Parameter Values for Subcatchments in
the River Fulda Catchment

Subcatchment	Area km ²	Max. elev m	Min. elev m	Main channel length km
401	56	838	365	14
402	506	550	232	36
403	182	700	232	25
404	469	775	216	27
405	394	416	193	33
406	148	700	265	24
407	274	610	209	34
408	90	518	193	9
409	403	391	179	24
Total	2523	838	179	227

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Table 6.2
Parameter Values for Gauging Stations
in the Fulda Catchment

Station	Bankful depth m	Bankfull width m	Bankfull capacity $\text{m}^3 \text{s}^{-1}$
Hetterhausen	2.3	17.0	26
Kammerzell	2.0	20.1	33
Lutterz	3.2	18.0	18
Unter-Schwarz	3.0	18.0	50
Marbach	2.3	8.0	10
Hermannspiegel	2.5	16.5	22
Bad Hersfeld	4.1	30.3	76
Rotenburg	4.8	50.0	179

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Table 6.3
Soil Group Classification for the Sub-Catchments in
the River Fulda Catchment

Subcatchment	USCS Soil Classification System				
	percentage occurrence				
	SC/SM	ML	CH	CL	G
401	54.6	11.6	11.3	10.5	12.1
402	45.6	10.3	5.2	27.7	11.4
403	25.0	2.9	4.0	59.9	7.9
404	36.6	2.7	15.2	33.0	12.0
405	65.8	4.1	4.7	8.2	17.3
406	50.1	13.4	9.8	21.4	5.4
407	46.4	8.4	25.2	15.5	4.6
408	41.0	0.0	15.2	34.7	9.2
409	86.5	0.0	4.8	0.0	8.6

SC Clayey sands or clayey gravelly sands
 SM Silty sand or silty gravelly sand
 ML Silts, sandy silts, gravelly silts
 CH Fat clays
 CL Lean clays, sandy clays, or gravelly clays
 G Gravels

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6.4.1 Establishment of the data sets

Two data sets are required by MILHY3: 'data1' contains the program commands, hydrological commands and associated data, whilst 'data2' contains only data for the infiltration algorithm. The rules for setting up these data sets and examples of them are given in Volume 2.

Data1

Figure 6.3 illustrates the division of the River Fulda catchment into nine catchments. In each of these subcatchments, a runoff hydrograph must be developed, for all except the headwater subcatchments, and this must be added to the flow routed through the subcatchment. In each routing reach, two cross-sections are developed, one at either end of the reach. In subcatchment 404, where the River Luder joins the River Fulda at its inflow, the Kammerzell cross-section is used.

The Curve Number routine for the generation of the runoff hydrograph using the SCS method were identified from tables in the Student Handbook on streamflow forecasting by James and Stinson (1981).

Data2

Each soils group in each subcatchment was represented by a single column, giving a total of 42 columns, see Table 6.3. The runoff generated by these columns was weighted by the percentage area of each subcatchment that a soil group occupied. For the six storm events identified during the establishment of the River Fulda catchment, a common theme was a period preceding each event of small low intensity showers. This enabled the fairly safe assumption that the soils were saturated three days prior

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to the beginning of each of the six identified events. The infiltration generation routines were then used to simulate the three days prior to the storm, in order to generate the antecedent conditions.

Each of the 42 soil columns was split into 3 layers, typical of well-developed soils, and a total of 10 cells were specified in each column. The soils hydrological parameters were calculated using the empirical charts and regression equations developed by Brakensiek and Rawls (1983), and reported in Anderson and Howes (1986).

A test simulation using an observed 1 in 10 year event to check the establishment of both the data sets, showed that the infiltration algorithm was failing to generate sufficient runoff in comparison with the observed hydrographs from the gauging stations. Figure 6.7 illustrates the generated hydrograph at the Hetterhausen station and shows that the simulated discharge is significantly smaller than the observed discharge. Investigation of the rainfall distribution procedure showed that the observed runoff volume was feasible for the rainfall distribution and therefore the error was associated with the soils data.

As noted earlier, the soils at the start of the simulation is assumed to be saturated and then the simulation is allowed to run for three days prior to the commencement of the storm event to allow antecedent conditions to be generated. Analysis of the results from the infiltration algorithm suggested that the hydraulic conductivity of the soils were too high. This was because during the storm event, despite being the 1 in 10 year event, the rainfall failed to saturate the soil and the rainfall was lost as drainage from the bottom of the soil columns. This suggests that either the hydraulic

HETTERHAUSEN STORM1: 1 in 10 year event

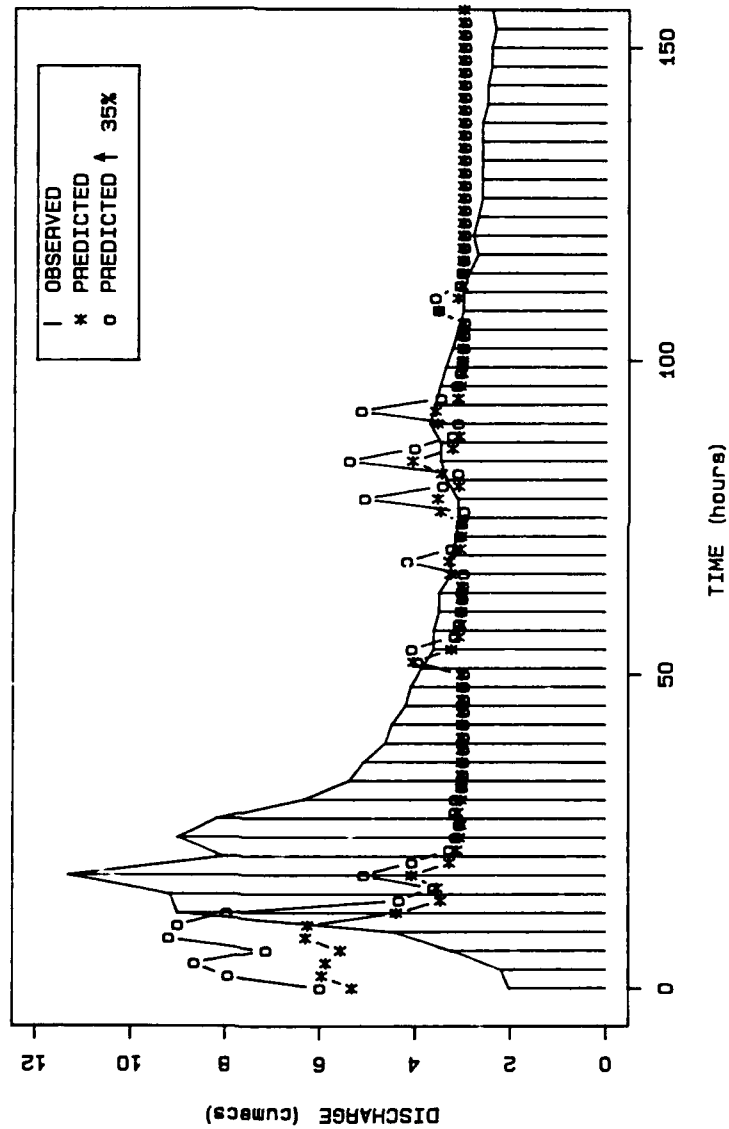


Figure 6.7
Comparison of Observed and Simulated Hydrographs with
Varying Percentage Clay Contributions, for the 1 in
10 Year Event at Hetterhausen

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conductivities were too high, or that the dominant process in the catchment was throughflow rather than overland flow. The review of the simulation of infiltration in MILHY3, reported in section 2.2.3, shows that the model is capable of simulating the movement of water soil in the vertical dimension only. It was important, therefore, to establish whether such an error in the generation of the runoff hydrograph could feasibly be attributed to the selection of too high a hydraulic conductivity. If a throughflow dominated catchment was shown to be the only possible explanation, then it is important to identify the limitation of the MILHY3 scheme and to possibly identify operational guidelines to allow the more approximate application of MILHY3 in such circumstances.

There are two possible sources of error that could generate too low a hydraulic conductivity. Either the incorrect hydraulic conductivity from the Brakensiek and Rawls (1983) charts was selected for each or some of the soil groups, or the soil groups in the high hydraulic conductivity groups were too highly represented.

A second simulation used the lowest hydraulic conductivity feasible for each soil group using the Brakensiek and Rawls (1983) charts. This simulation still failed to generate sufficient runoff and therefore this source of error was excluded.

6.4.2. Soils classificatory errors

There are several feasible sources of error in the generation of the proportion of a subcatchment that a soils group occupies. These include:

- 1) Resolution. This includes the resolution of the soil

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surveyor's report, and the interpretive work carried out in the establishment of the data set. The resolution of the original survey will depend upon the purpose to which the map is aimed. Beckett and Webster (1971) point out that there is little practical purpose in having a resolution size less than the minimum land-use management area, usually a field.

2) Purity. This is the percentage of each group that is occupied by that group. Beckett and Webster (1971) identify the level of purity of many of the soil survey organisations aim to achieve. This includes the USDA purity level of 80-90%, and the US Bureau of Reclamation purity level of 75%.

Analysis of the results from the infiltration algorithm simulations showed that it was only the clay groups (CH and CL in Table 6.3) that had a low enough saturated hydraulic conductivity to generate overland flow, given the intensity of the 1 in 10 year event. It is possible, therefore, that the purity of the high conductivity groups was particularly low, or that the resolution of these groups ignored lower conductivity areas. Could the percentage occurrence of the clay groups be legitimately increased in line with the purity level suggested by Beckett and Webster (1971)? If this proved successful, then the purity level could potentially provide an important operational guideline.

Closer analysis of the soils classificatory maps showed that the distribution of the clay groups were heavily biased to the floodplain area where runoff could be expected to join the channel flow, due to high water tables and the effect of topography. Under-estimating these groups, therefore, would have a large effect on the predicted hydrograph. These results suggest that in event

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simulations such as MILHY3 that do not simulate the throughflow contribution to the hydrograph, it is important to accurately estimate the percentage contribution of the soil groups with low hydraulic conductivities. This is particularly true for low intensity rainfall events, where the resolution of high conductivity groups may be reduced.

Trials showed that a 35% increase in the occurrence of the clay groups was required before the required volume of runoff was generated, see Figure 6.7. As the 35% increase is somewhat larger than the purity levels noted by Beckett and Webster (1971), who gave a maximum error of around 20-25%, this suggested that the minimum misclassification error was approximately 10-15%. Estimation of the classificatory map and the distribution of the clay groups suggests such an error is feasible.

In the simulations reported in the rest of this report, therefore, the soils classificatory groups have been adjusted in line with the error sources identified above. It is accepted that this is a partial calibration of the model, which could not occur in a truly ungauged application. However, given the concentration of this report on the channel routing components of MILHY and on the development of a composite modelling structure, and given the problems of collecting another data set to validate the assumptions relating to the sources of the classificatory errors, it seems appropriate to accept this limitation of the data set and pursue the rest of the evaluation analysis.

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Validation of MILHY3II - Hydrological Sensitivity Analysis of MILHY3

The sensitivity analysis of MILHY3 aims to complete the computerised verification section of Sargent's (1982) model evaluation programme. Howes and Anderson (1988) identified three areas that a well structured sensitivity analysis should investigate. These three areas are:

- 1) The analysis should demonstrate that the model behaves realistically when the model inputs and parameter values are varied
- 2) The analysis should show that the model is sufficiently sensitive to represent the actual variation in the system
- 3) The analysis should identify the model parameters or inputs to which the model is most sensitive

The design of the sensitivity analysis will incorporate these three areas into the specific objectives of the analysis of MILHY3.

7.1 Design of the Sensitivity Analysis

The main objectives of the sensitivity analysis are to investigate:

- 1) the sensitivity of the hydrograph to variation in MILHY3's parameters;

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- 2) the sensitivity of the hydrograph to the submodels used in its derivation;
- 3) the interaction of the effects of the momentum exchange and multiple-routing submodels;
- 4) the relative impact of the inclusion of infiltration algorithm if sub-catchments are utilized; and
- 5) the effects of scale.

First it is necessary to identify the parameters and submodels that make up MILHY3's composite structure. Table 7.1 lists all the variables in three groups; spatial variables, temporal variables and physically-based parameters. The spatial and temporal variables describe the resolution of information in each submodel or process area, whilst the physically-based parameters describe the initial conditions and geometry of the catchment.

Figure 7.1 shows the composite structure of MILHY3 and the process modules available. The figure shows that there are five stages in the development of the outflow hydrograph. These five stages are: the spatial variability of the precipitation, the runoff generation technique, generation of the rating curve, designation of the routing pathways, and finally the routing of the hydrograph. The figure also shows that at the first four stages of the hydrograph development the user is now given a choice in the resolution of module required. The original modules prior to the developmental work in this report are shown on the left-hand side of the diagram, whilst the new, higher resolution modules are on the right-hand side of the diagram. The user may now choose modules at either resolution levels at each of the hydrograph development stages. One of the aims of this

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Table 7.1
Variables Utilised in MILHY 3

Spatial Variability

Number of subcatchments
Number of raingauges
Number of soil columns
Number of soil cells
Thickness of soil cells
Number of valley sections in reach
Number of segments in cross-section
Rating curve increment

Temporal Variability

Rainfall time increment
Infiltration simulation iteration period
Routing time interval

Physically-based parameters

Initial soil moisture content
Saturated soil moisture content
Suction moisture curve
Saturated hydraulic conductivity
Surface detention capacity
Maximum evaporation
Watershed area
Watershed elevation
Main channel length
Cross-sectional geometry
Slope, channel and floodplain
Routing length
Manning's 'n', channel and floodplain
Reservoir outflow storage
Soil, crop, conservation and gradient factors
Precipitation, storm duration and intensity

MILHY3 COMPOSITE MODEL STRUCTURE

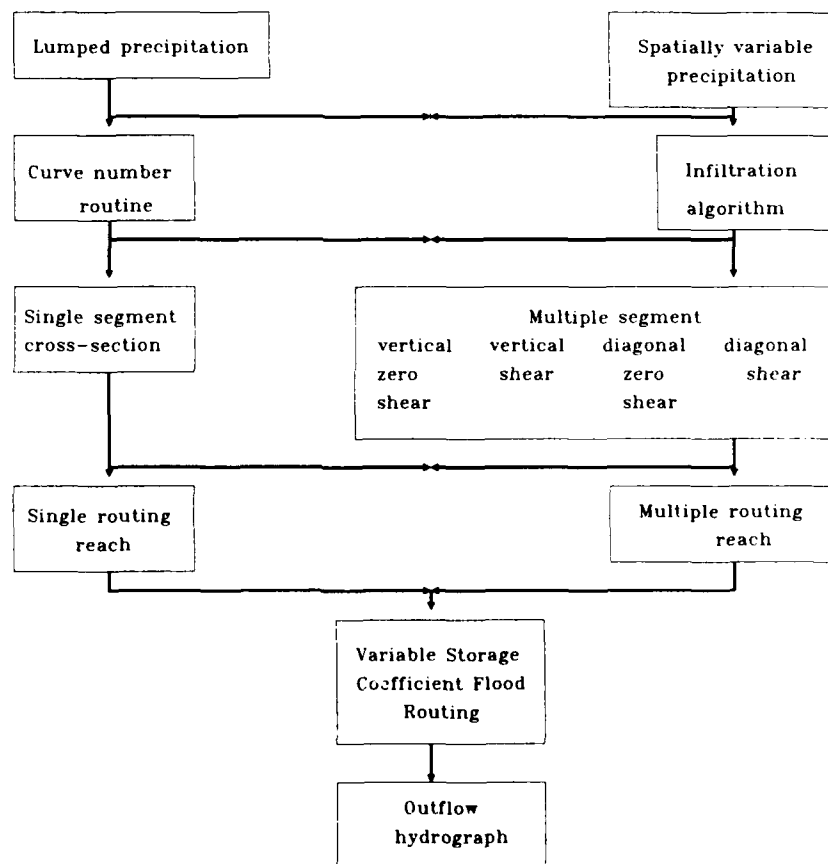


Figure 7.1
MILHY3
Composite Model Structure

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report is to develop a series of guidelines to help the user select the most appropriate module resolution level for a particular application.

To avoid repetition of the analysis of sensitivity of any of the parameters identified in Table 7.1, a review was made of all previous evaluation programmes undertaken on all versions of MILHY and MILHY2. A summary of this review is reported in Table 7.2, which shows that previous evaluation programmes have concentrated on the parameters and controls of runoff excess generation. Most importantly, the evaluation of the incorporation of the infiltration algorithm has taken place only in single catchments. This is significant in the evaluation of the success of the composite structure of MILHY3 and in particular the problem of module resolution. An important part of this analysis will be to investigate the performance of the high resolution infiltration algorithm in multiple subcatchments.

Comparison of Figure 7.1 showing the variable structure of MILHY3 comprising of 12 process modules, and Table 7.1 detailing 34 parameter and control variables for these modules, shows that there are a total of 46 parameters in the MILHY3 model. Taking into consideration the parameters that have already been investigated and that are summarised in Table 7.2, there are still 32 parameters to investigate.

If these 32 parameters were to be varied in a statistically meaningful way, that is considering twenty storm events, twenty cross-sectional geometries, twenty routing reach lengths, twenty boundary roughness ratios, and utilising all the possible module combinations, then several thousand simulations would be required. This would take 5 years of central processing time on the SUN

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Table 7.2
Existing Sensitivity Analyses of MILHY2

<u>Author</u>	<u>Parameter or Variable</u>
Williams (1975)	Routing length Routing time interval
Anderson (1982)	Detention capacity
Anderson and Howes (1984)	Suction moisture curve Saturated moisture content Saturated hydraulic conductivity Initial moisture content
Anderson and Howes (1986)	Watershed area Watershed elevation Main channel length Storm intensity Cell size Infiltration simulation increment
<u>Initial results</u>	
Chapter 4	Number of segments in cross- section Rating curve stage increment Cross-sectional geometry Manning's n coefficient Slope

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workstation. It is important, therefore, to select carefully a series of simulations that will enable the sensitivity of MILHY3 to be ascertained within a fair degree of certainty.

Interest has been directed in this project to the modelling of the frictional effects of over-bank flow or flow in two-stage channels. The sensitivity analysis will reflect this concentration and therefore focus primarily in this area. The analysis will investigate the structure of MILHY3 in terms of its module components and explore variability of the physically-based parameters in the downstream conveyance submodels. These parameters will include the slope of the channel and floodplain, the routing length, and Manning's 'n' values for the channel and floodplain.

Having identified a manageable group of variables for investigation, the next stage is to develop a strategy for investigating the sensitivity of these variables.

7.1.1 Alternative methods of undertaking a sensitivity analysis

McCuen (1973a and b) defines sensitivity as:

"the rate of change in one factor with respect to change in another factor"

McCuen points out that it is the failure of researchers to appreciate the generality of this definition that has limited the application of the sensitivity analysis tool to the final verification of models. Several authors have identified the utility of the sensitivity analysis in all stages of the development of a model (McCuen, 1973a and b, 1976; Miller et al. 1976;

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Hornberger and Spear, 1981). This is why an initial analysis was incorporated in the identification of the most important processes in two-stage channels (Chapter 3), and in the development on the submodels to model these processes (Chapters 4 and 5).

Jones (1982) identified two approaches to undertaking a sensitivity analysis: one involves a deterministic methodology, the other a stochastic methodology. A deterministic methodology involves making small changes in the input parameters and investigating changes these changes effect on the models output. A stochastic approach involves selecting the input parameter values from a probability distribution according to, either the accuracy of the input values or the error bands to which the model internally operates. Because the probability distributions can contain all the physically realistic data input values, a stochastic analysis can usually encompass a wider range of input data values than a deterministic analysis. In a deterministic analysis the operator must usually either select input values systematically, or use intuitive knowledge of the behaviour of input parameters.

Work by Anderson and Howes (1984 and 1986) has concentrated on a stochastic analysis of the soil input parameters of the infiltration algorithm. The difficulties of measuring these parameters in the field mean that a relatively large error distribution can be associated with the observed field values. A stochastic analysis was an ideal method of incorporating these error distributions in an analysis of the effects of parameter variability on the model output.

In the sensitivity analysis reported here, the objective is to investigate the behaviour of the model

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output with respect to both the model structure and variability in certain parameters. As the model structure cannot be described by a probability distribution, this necessitates a deterministic analysis.

There are two methods of computing the sensitivity of a model to a parameter, known as the sensitivity function, in a deterministic approach. These are:

1) Differentiation: the model described as a function is parametrically differentiated with respect to each parameter. The mathematics of this approach have yet to be made portable enough to enable this approach to be widely used.

2) Factor Perturbation: each parameter is incremented and the changes in the output quantified. This method was used by Smith (1976) and as noted earlier has extensive computing time requirements.

Given the mathematical difficulties in utilising the differential solution for the derivation of the sensitivity function, the factor perturbation technique was selected. The prospect of initialising a large number of simulations necessary for the factor perturbation approach, however, provided the impetus for an investigation into other feasible alternatives. One interesting alternative would be the utilisation of optimization techniques, previously generally only utilised for the calibration of hydrologic models.

Although optimization techniques are well-established, it seems that they have not been used as an alternative to the sometimes tedious development of factor perturbation matrices. It was hoped that, if successful, the intermediate values of the optimization scheme would

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provide a good indication of the sensitivity of the outflow hydrograph, as the scheme searched to find the 'best-fit' between an observed and a computed hydrograph. This would remove the necessity to initialize a great number of data sets and manually search through the results sets. A post-processor could search through the iterations of the optimization scheme and find the parameter values that caused the greatest or smallest impact on the computed outflow hydrograph. Although there would not be any direct control over the parameter values selected by the optimization scheme, in a factor perturbation analysis the selection of values is usually subjective and therefore could just as easily overlook significant points. However, it must be noted that this investigation into the utility of optimization is rather exploratory. The feasibility of using optimization schemes as an alternative to traditional factor perturbation techniques will depend upon:

- 1) interpretative techniques required to interrogate the results of the simulations
- 2) the initialization time period set-up of the scheme
- 3) computer demands
 - i) CPU
 - ii) disk storage

As the potential benefits of the optimization approach seemed large, it was decided to divide the sensitivity analysis into two parts, the factor perturbation approach and the optimization approach.

Figure 7.1 illustrates the composite structure of MILHY3 and shows there are 12 modules which generate a

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possible 40 module combinations. As this is a finite number and of manageable proportions it seemed appropriate to utilize the established factor perturbation technique to investigate the importance of variability in the structure of MILHY3. The investigation of this source of variability is significant if the implications of module resolution are to be discussed, and it was important to guarantee results from the analysis.

By contrast, the variability of the physically-based parameters identified earlier in this section provided for a possible infinite number of combinations and permutations. The scope for the potential results from an optimization approach are much larger therefore.

The sensitivity analysis will firstly investigate the importance of variability in the structure of MILHY3 using traditional factor perturbation techniques and secondly investigate the potential utility of optimization techniques using the variability in parameters.

The River Fulda catchment has been introduced in Chapter 6 and six storms have been established. However, only two of these events are out-of-bank events. This once again highlights the problems of obtaining consistent data sets for extreme events and stresses the importance of alternative methods of synthetically extending these records using other models (see Chapter 8).

In this analysis, therefore, two storms are utilised. Storm 1 is the 1 in 10 year event used in the initial analyses in Chapters 4 and 5 (the outflow at Rotenburg of this event is shown in Figure 5.3). The second storm has a return period of 1 in 1.5 years and is known as Storm 3 to distinguish it from the synthetic 1 in 100 year event used in Chapters 4 and 5. For both storms, observed

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hydrographs are available for all of the intermediate stations identified in Figure 6.1, except one. The exception is the observed hydrograph for the 1 in 1.5 year event at Rotenburg. For this reason, the simulations of both events are halted at Bad Hersfeld, upstream of Rotenburg. This still gives a catchment of 2100 km², with eight subcatchments.

7.2 Traditional Factor Perturbation

The objective of this analysis is to investigate the sensitivity of the outflow hydrograph to variability in the structure of MILHY3. The variable or composite structure of MILHY3 is summarised in Figure 7.1.

The first stage of the development of the outflow hydrograph is the specification of the precipitation input into the catchment. MILHY3 offers two modules with varying spatial resolution for the specification of the precipitation pattern. The lower resolution module, used in previous versions of MILHY, lumps the precipitation into subcatchments, so the distribution for each subcatchment is specified by a single hyetograph. If there is only one raingauge in the catchment then the user may choose to use the same hyetograph for all the subcatchments.

The alternative higher resolution module, introduced in the development of MILHY3, incorporates the spatial and temporal effects of frontal rainstorms tracking across the catchment. In the next section, the importance of variability in the precipitation distribution in the MILHY3 scheme is assessed.

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7.2.1 Spatially variable precipitation

The importance of the effects of variability in the spatial and temporal distribution of precipitation has been accepted by many authors, for example; Huff (1967), Dawdy and Bergmann (1969) and Beven and Hornberger (1981). Authors have investigated the effects of the resolution of raingauge networks, (Wilson et al., 1978), the effects of the resolution of the catchment, (Huff, 1968), and the effects of catchment shape, (Tabios et al., 1986). All authors have agreed, however, that it is difficult to quantify the significance of spatial and temporal variability on the prediction of flood hydrographs, given the complexity of storm events and the variability of catchment geometrical characteristics.

Modelling approaches to incorporate this variability range from manual distributions using Thiessen polygons (Nguyen and Berndtsson, 1986), stochastic distributions (Tabios et al., 1986), to geomorphic distributions (Corradini and Singh, 1985). Engdahl and Collins (1985) conclude, however, that such models preclude realistic predictions of hydrographs from ungauged catchments where calibration data is not available, or for catchments where extrapolation of gauge data would be unacceptable. In such circumstances, Engdahl and Collins recommend the use of weather radar systems, such systems are explored by Cluckie et al., (1982).

Given the importance of spatial variability in the precipitation distribution on the predictive performance of hydrological models, it was essential to assess the effects of such variability on the performance of MILHY3. The relative importance of such variability in comparison with other sources of variability in differing process modules, must also be assessed. Following Engdahl and

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Collins (1985) recommendations, therefore, a grid network was established to distribute rainfall throughout the subcatchments during the generation of the runoff hydrograph in the MILHY3 model. The resolution of the grid utilised was 5km, this being the resolution of weather radar data available for the UK, (see Collinge and Kirkby, 1987).

As the purpose of the investigation was to rank the importance of the effects of variability in rainfall distribution in comparison with other processes, the experimental frames utilised were restricted to a single subcatchment. It is accepted, therefore, that such an investigation does not allow an exploration of the absolute importance of such variability, nor does the investigation allow conclusions to be drawn on the techniques available for incorporation. However, the literature suggests that there are many models readily available for the distribution of rainfall that could be included in MILHY3's composite structure, if this initial analysis showed the distribution to be significant in the MILHY3 scheme.

The subcatchment utilised in this investigation is catchment 406, which has an area of 145 km^2 . The subcatchment is on Figure 7.2 with the 5 km^2 grid network superimposed. This subcatchment was further divided into seven smaller catchments, ranging in area from 14 km^2 to 31 km^2 , shown in Figure 7.2. An experimental design of 16 frames was established and executed. The analysis included two storms; a 13 mm storm in a 24 hour interval, typical of a frontal storm system, and a 25 mm storm in 6 hour interval, typical of a convective cell system. Two storm velocities were utilised to compute the temporal distributions. These were; 5 kmhr^{-1} and 10 kmhr^{-1} , typical respectively of frontal and convective cell systems.

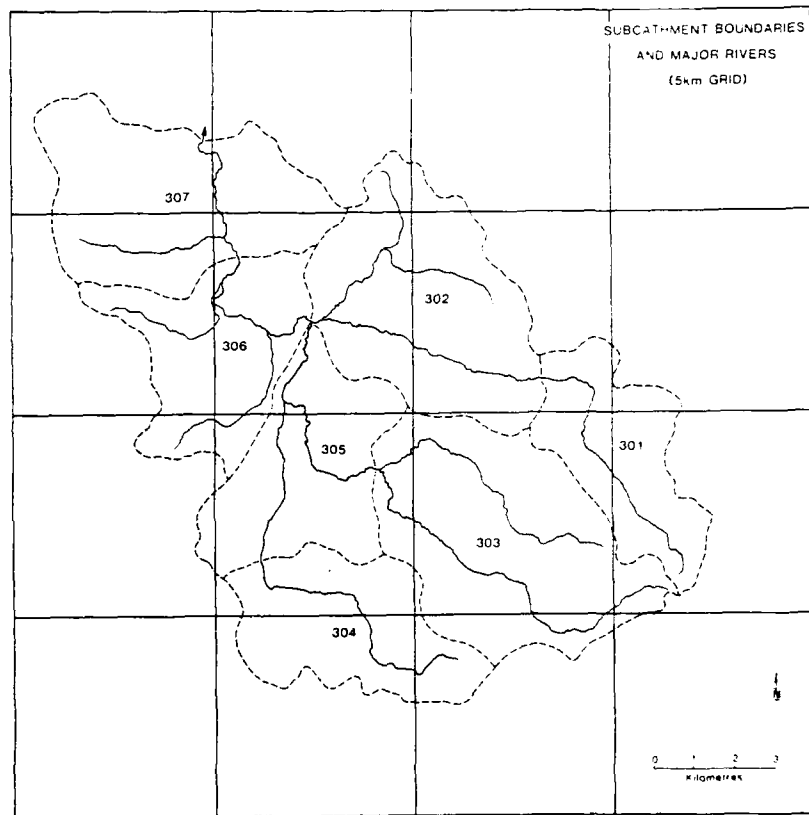


Figure 7.2
Marbach Subcatchment 406
Division into Further Subcatchment and 5km² Grid

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Four directions of storm movement were also considered, north, south, east and west. Rainfall distributions were computed assuming that rainfall was homogeneously distributed throughout a 5 km^2 grid square. Rainfall falling in grid squares in several catchments was distributed between the catchments depending on the percentage area on each catchment in the square.

Table 7.3 contains the simulated peak discharge results for the 13 mm in 24 hours storm event for the seven smaller subcatchments and for subcatchment 406. Table 7.4 contains similar results from the 25 mm in 6 hour event. Table 7.3 shows that the runoff hydrograph generated for subcatchment 406 and the seven smaller subcatchments varies very little for the eight experimental frames simulating the 13 mm in 24 hour storm event. By contrast, in Table 7.4 certain differences in the prediction of the peak discharge of the 25 mm in 6 hour event are apparent. The largest difference is to be found in the 10 km hr^{-1} northerly moving storm compared with the 5 km hr^{-1} southerly moving storm. In this case the peak discharges are respectively $123 \text{ m}^3 \text{ s}^{-1}$ and $114.2 \text{ m}^3 \text{ s}^{-1}$. These results suggest that a catchment area of 145 km^2 may be regarded as the minimum for which 5 km^2 grid square precipitation data is required for low intensity storms. For higher intensity storms at 145 km^2 then differences of perhaps not greater than 5% in the predicted peak discharge parameter are attributable to variability in the precipitation distribution.

This analysis has shown, therefore, that the runoff hydrograph is sensitive to variability in the precipitation distribution when subcatchment areas are greater than 145 km^2 for all storm events and for smaller areas for intense storm events. However, the simulations from the validation of the multiple routing routine,

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Table 7.3
Peak discharge prediction of
13 mm in 24 hour storm event

Catchment	Peak discharge ($\text{m}^3 \text{s}^{-1}$)							
	5 km hr^{-1}				10 km hr^{-1}			
	n	s	e	w	n	s	e	w
301	1.11	1.13	1.13	1.12	1.14	1.25	1.13	1.13
302	1.96	1.92	1.92	1.94	1.95	1.94	1.93	1.95
303	2.29	2.33	2.31	2.28	2.31	2.32	2.30	2.31
304	1.33	1.34	1.33	1.33	1.35	1.35	1.34	1.33
305	1.37	1.36	1.38	1.38	1.37	1.37	1.38	1.38
306	1.20	1.22	1.21	1.22	1.21	1.21	1.21	1.31
307	1.90	1.90	1.90	1.89	1.89	1.90	1.91	1.90
406	10.99	10.90	10.93	10.79	11.04	10.97	10.99	11.14

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Table 7.4
Peak discharge prediction of
25 mm in 6 hour storm event

Catchment	Peak discharge ($\text{m}^3 \text{s}^{-1}$)							
	5kmhr^{-1}				10kmh^{-1}			
	n	s	e	w	n	s	e	w
301	14.85	14.51	14.22	14.66	14.81	24.61	14.71	14.71
302	22.72	22.66	22.30	22.36	22.72	22.76	22.81	20.94
303	29.56	29.63	29.41	29.62	29.64	29.72	29.77	29.73
304	15.86	15.79	16.08	16.00	15.99	16.11	16.08	16.15
305	14.97	14.97	15.30	15.36	15.18	15.24	15.36	15.18
306	11.42	11.42	11.35	11.37	11.58	11.55	11.58	11.52
307	19.49	19.50	19.57	19.58	20.02	20.05	20.07	20.26
406	120.8	114.2	114.4	117.3	123.0	118.8	120.9	120.0

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reported in Tables 5.1 and 5.2, suggest variability in the predicted hydrograph of around 30%. Given the concentration of this report on the two-stage channel aspects of ungauged flood forecasting and the availability of models for the distribution of rainfall data, the utility of the 5 km² grid distribution is not explored further in this report. In the remaining simulations of this report, the lower resolution rainfall module is utilised.

7.2.2 Baseflow conditions

The precipitation data are used by the Curve Number routine in MILHY and the infiltration algorithm in MILHY2, to generate the Hortonian runoff excess. To this baseflow must be added. For the purpose of this investigation the baseflow levels have been taken from the observed hydrographs. As the thrust of this analysis is the importance of module resolution in the MILHY3 scheme the use of observed baseflow conditions is felt to be acceptable.

It is acknowledged that this would not necessarily be possible in an ungauged catchment and there is a need for the development of a module generating baseflow conditions for inclusion in the MILHY3 scheme. Such a module could utilise either catchment characteristics or channel geometry characteristics that are already available in the MILHY3 scheme.

The notation used in this analysis refers to module combinations that have been available in previous versions of the MILHY program. The term MILHY therefore refers to a module combination that utilises the curve number generation of the runoff excess, whilst MILHY2 uses the infiltration algorithm generation. Both MILHY and MILHY2

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however, do not utilise the modules developed in this report, although they do incorporate the second method of incorporating momentum exchange, namely the use of vertical interfaces with an apparent shear stress ratio =1. The term MILHY3 is used to represent a module combination that utilises the highest resolution in all stages of the development of the outflow hydrograph. MILHY3 therefore uses the infiltration algorithm and momentum exchange and often the multiple routing routine. The exact combination of modules used in the MILHY3 simulations is specified in each case using the IT and MR notation. IT varies from 1 to 4 to indicate the method of incorporating the momentum transfer, whilst MR indicates that the multiple routing routine is either invoked (MR=1) or not invoked (MR=0). This notation was utilised in Chapter 5.

To summarise:

MILHY - Curve Number routine, IT=2, MR=0
 MILHY2 - Infiltration Algorithm, IT=2, MR=0
 MILHY3 - Infiltration Algorithm, IT=1/2/3/4, MR=0/1

IT=1 Vertical interface, apparent shear stress ratio=0
 IT=2 Vertical interface, apparent shear stress ratio=1
 IT=3 Diagonal interface, apparent shear stress ratio=0
 IT=4 Diagonal interface, apparent shear stress ratio=1

MR=0 Single routing reaches
 MR=1 Multiple routing reaches

7.2.3 Storm 1: 1 in 10 year event

Figure 7.3 illustrates the precipitation pattern at Fulda and the observed hydrograph at Bad Hersfeld for the 1 in 10 year event. The temporal distribution of

STORM1: 1 in 10 year event

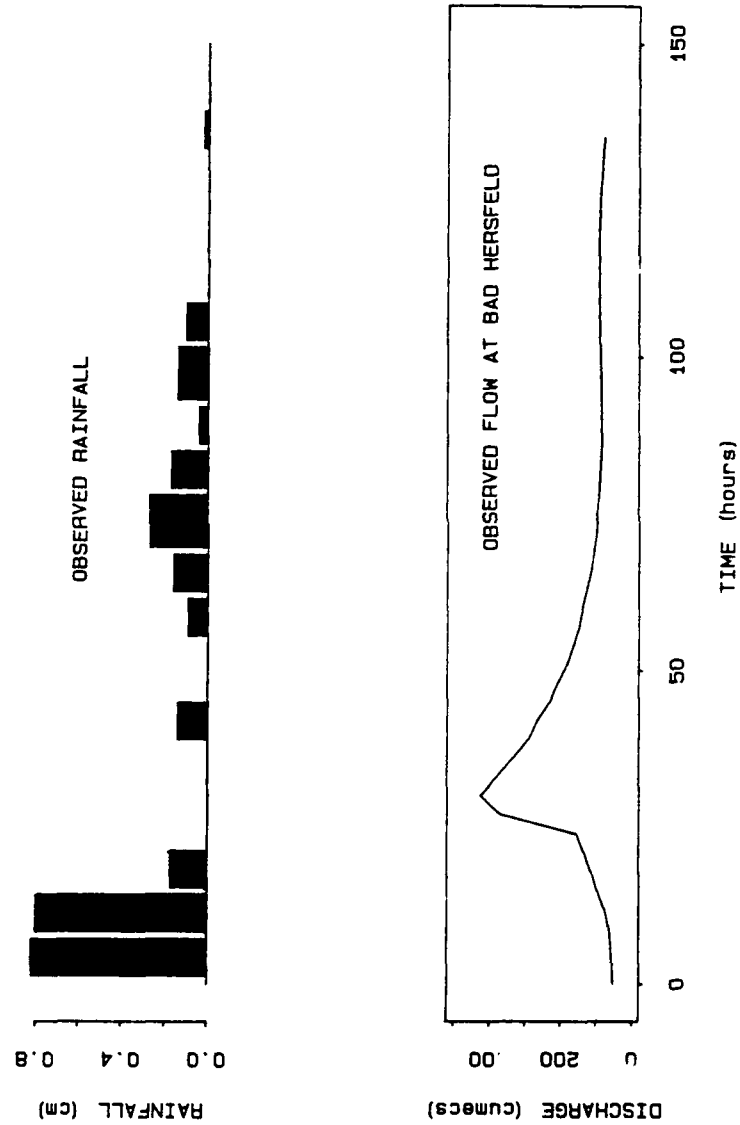


Figure 7.3
Observed Hyetograph for the River Fulda Catchment
and Observed Hydrograph at Bad Hersfeld for the
1 in 10 Year Event

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precipitation for all the subcatchments is based on this hyetograph. The magnitude of the rainfall in each subcatchment is determined from the daily precipitation records of the stations shown in Figure 6.2. The rainfall is assumed to be lumped and the spatial distribution of the rainfall over the catchment for the duration of the storm is not considered. The specification of the rainfall therefore assumes the lower resolution offered in the composite structure of MILHY3 shown in Figure 7.1.

The minimum cumulative rainfall total for this storm event occurs in subcatchment 408 where 45mm fell; the maximum total occurs in subcatchment 403 where 75mm fell. The daily precipitation values indicate, therefore, a strong spatial element in the distribution pattern of this storm.

The observed hydrograph shown in Figure 7.3 illustrates that the discharge peak occurred approximately 24 hours after the rainfall peak. The time to peak of the observed hydrograph is 30 hours from the start of the storm event and the peak discharge is $426 \text{ m}^3 \text{ s}^{-1}$.

The simulated outflow hydrographs at Bad Hersfeld using Curve Number and Infiltration algorithms for all the momentum exchange and multiple routing combinations are shown in Tables 7.5 and 7.6 respectively. These tables summarize the characteristics of the predicted hydrographs, noting the peak discharge, time to peak and equivalent runoff depths.

Figure 7.4 illustrates the impact of the computation method on the predicted hydrograph at Bad Hersfeld. MILHY utilizes the Curve Number routine, whilst MILHY2 uses the Infiltration algorithm. Both MILHY and MILHY2 use

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Table 7.5
Storm 1 : 1 in 10 year event

Predicted Outflow at Bad Hersfeld Utilising the
Curve Number Routine

Computation method		Peak discharge $\text{m}^3 \text{s}^{-1}$	Time to peak hours	Runoff depth m
IT=1	MR=0	265	17.5	0.03
IT=2	MR=0	272	17.5	0.03
IT=3	MR=0	265	17.5	0.03
IT=4	MR=0	249	17.5	0.03
IT=1	MR=1	312	16.5	0.03
IT=2	MR=1	328	16.5	0.03
IT=3	MR=1	323	16.5	0.03
IT=4	MR=1	281	17.0	0.03

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Table 7.6
Storm 1 : 1 in 10 year event

Predicted Outflow at Bad Hersfeld Utilising the
Infiltration Algorithm

Computation method		Peak discharge $\text{m}^3 \text{s}^{-1}$	Time to peak hours	Runoff depth m
IT=1	MR=0	312	18.5	0.03
IT=2	MR=0	321	18.0	0.03
IT=3	MR=0	310	18.0	0.03
IT=4	MR=0	290	18.0	0.03
IT=1	MR=1	364	16.5	0.04
IT=2	MR=1	383	16.5	0.04
IT=3	MR=1	372	16.5	0.04
IT=4	MR=1	332	17.5	0.04

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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momentum exchange method 2, that is a vertical interface with an apparent shear stress ratio =1, and do not incorporate multiple routing. MILHY3 uses the infiltration algorithm and in this case momentum exchange method 3 (diagonal interface, zero shear), and the multiple routing routine is invoked.

The initial trial simulations and consequent changes in the specification of the soils classificatory data during the establishment of the Fulda data set, reported in Chapter 6, ensure that the runoff depths reported are comparable with the observed data for all the subcatchments. The Curve Number routine, for example, predicted a runoff depth of 0.02m for subcatchment 407, whilst the Infiltration algorithm generated 0.03m of runoff. The observed runoff at Hermannspiegel at the downstream extremity of the catchment showed the measured runoff to be 0.028m.

The predicted peak discharge is used as a measure of the accuracy of the prediction in this analysis because of the simple shape of the hydrograph and because of the importance of the peak discharge in determining the depth and extent of the floodplain inundation. The initial analysis of the behaviour of two-stage channels reported in Chapters 3, 4 and 5 shows that the depth of inundation is significant in determining whether the floodplain acts as a store of water when stages are low, a separate channel, or if the two-stages act as a single flow unit when inundation depths are large.

Comparison of the Curve Number and Infiltration algorithm predictions of peak discharge at Bad Hersfeld and Hermannspiegel (Figures 7.4 and 7.5) shows that at both stations the infiltration algorithm produces higher peak discharges. This behaviour has been noted previously

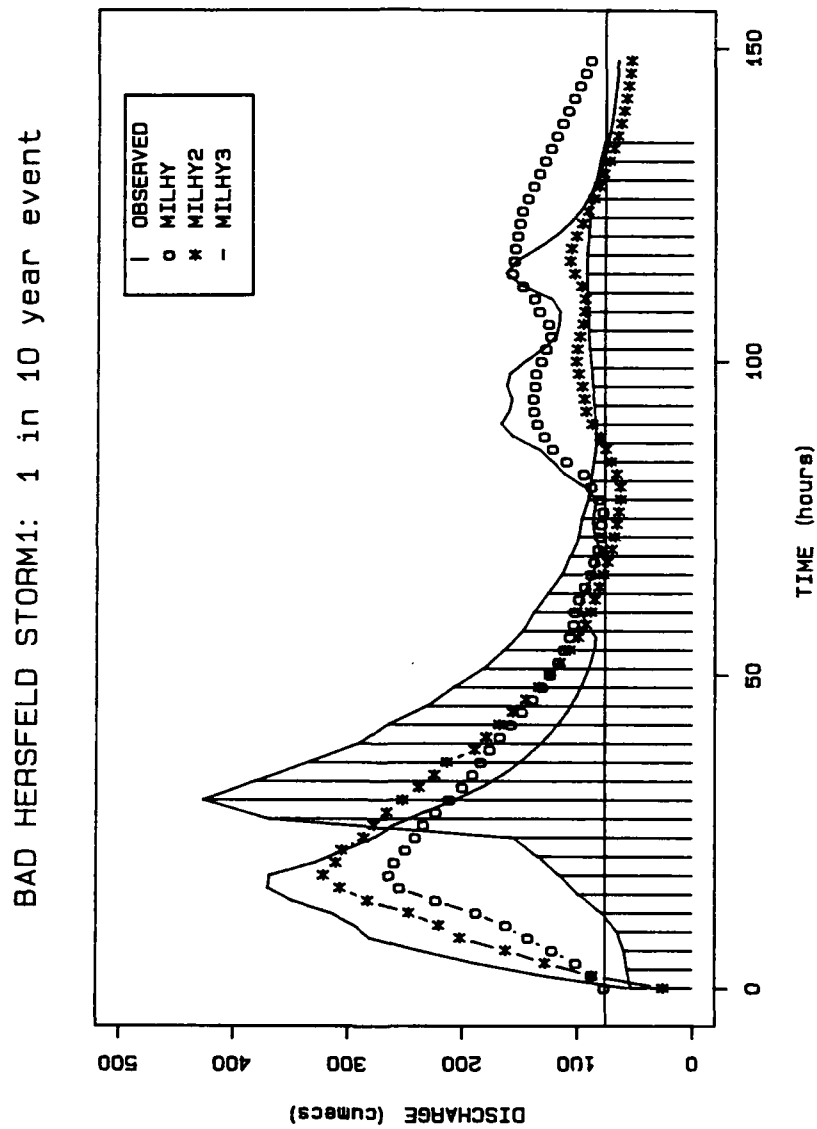


Figure 7.4
 Comparison of Observed and Simulated Hydrographs from
 MILHY, MILHY2 and MILHY3 Models at Bad Hersfeld for
 the 1 in 10 Year Event

HERMANNSPIEGAL STORM1: 1 in 10 year event

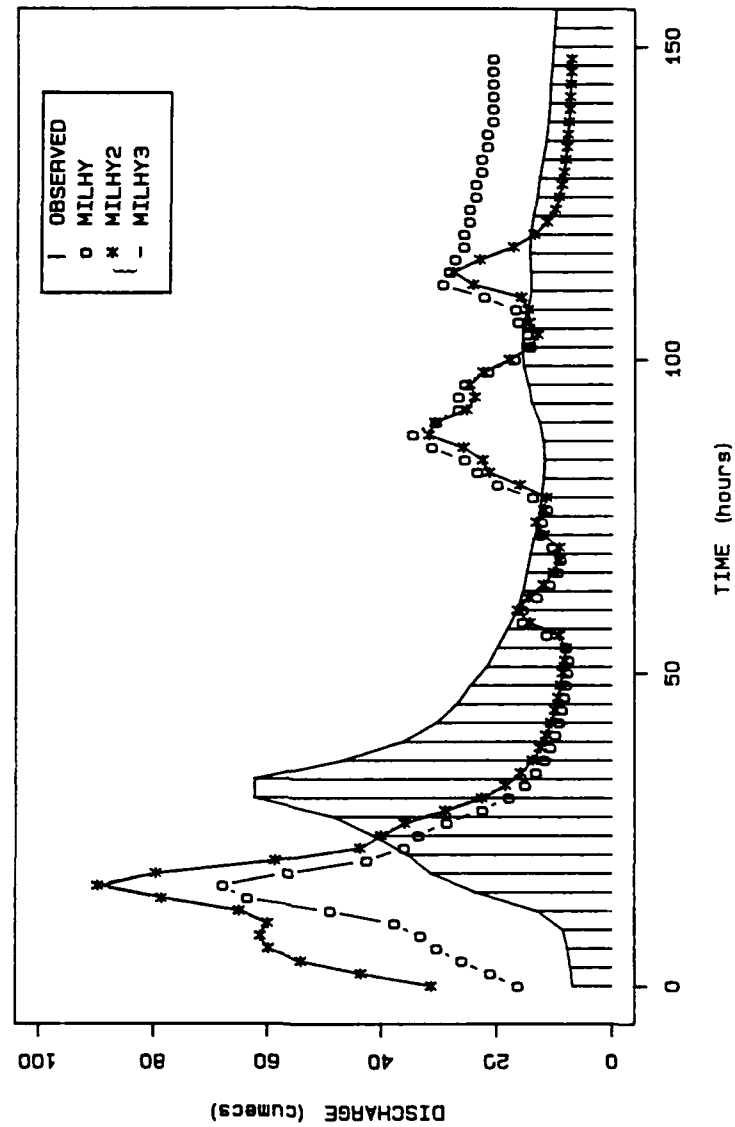


Figure 7.5
 Comparison of Observed and Simulated Hydrographs from
 MILHY, MILHY2 and MILHY3 Models at Hermannspiegel for
 the 1 in 10 Year Event

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by Anderson (1982) and Anderson and Howes (1984), in single subcatchments, where this behaviour occurred during high and low intensity storms. In this analysis it is worth noting that this behaviour is still visible after the hydrographs have been routed and hence attenuated through up to four subcatchments. This re-emphasises earlier work by Anderson and Howes (1984, 1986) that illustrated the importance of the shape of the runoff hydrograph in determining the outflow hydrograph.

Analysis of Tables 7.5 to 7.6 shows that the peak discharge is the parameter most sensitive to the downstream computation method. The impact of the momentum exchange and multiple routing routines seems to be related to the depth of flow on the floodplain. The outflow hydrograph at Hermannspiegel, shown in Figure 7.5, is routed from Marbach, located upstream on the River Haune. The distribution of the inflow hydrograph at Marbach as part of the multiple routing routine means that only three coordinates at the peak of the hydrograph are assigned to the floodplains. The impact of the momentum exchange and multiple routing routines on the predicted outflow hydrograph at Hermannspiegel is therefore minimal.

At Bad Hersfeld, however, the impact of the new modules is more pronounced. Tables 7.5 and 7.6 show that the momentum exchange methods that assume zero shear stress, (methods 1 and 3) produce very similar results, whilst the apparent shear stress ratio=1 (methods 2 and 4) produce smaller peak discharge predictions. These results show that the modules are behaving realistically, as the methods that introduce the greatest degree of turbulence, (2 and 4), produce the greatest changes in the predicted outflow hydrograph.

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Table 7.7
Storm 1 : 1 in 10 year event

Predicted Outflow at Hermannspiegel Utilising the
Curve Number Routine

Computation method		Peak discharge $\text{m}^3 \text{s}^{-1}$	Time to peak hours	Runoff depth m
IT=1	MR=0	70	15.5	0.02
IT=2	MR=0	70	15.0	0.02
IT=3	MR=0	70	15.0	0.02
IT=4	MR=0	69	15.0	0.02
IT=1	MR=1	70	15.0	0.02
IT=2	MR=1	71	15.0	0.02
IT=3	MR=1	69	15.0	0.02
IT=4	MR=1	68	15.0	0.02

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Table 7.8
Storm 1 : 1 in 10 year event

Predicted Outflow at Hermannspiegel Utilising the
Infiltration Algorithm

Computation method		Peak discharge $\text{m}^3 \text{s}^{-1}$	Time to peak hours	Runoff depth m
IT=1	MR=0	89	16.0	0.03
IT=2	MR=0	90	16.0	0.03
IT=3	MR=0	90	16.0	0.03
IT=4	MR=0	89	16.0	0.03
IT=1	MR=1	90	16.0	0.03
IT=2	MR=1	90	16.0	0.03
IT=3	MR=1	89	16.0	0.03
IT=4	MR=1	87	16.0	0.03

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Comparison of the results from Bad Hersfeld and Hermannspiegel show that relative difference between the momentum exchange methods remains relatively constant whatever the magnitude of the absolute discharge. From these results and the early results reported in Chapter 4 it is possible to conclude that the discrepancy generated between the various momentum exchange techniques is a function of the depth ratio between the floodplain depth and the main channel depth and not the absolute floodplain inundation depth.

Tables 7.4, 7.5, 7.6, 7.7 and 7.8 show that all the modules incorrectly predict the time to peak of the hydrograph, all simulating the peak too early. As the error is similar in both the Curve Number and Infiltration algorithm approaches, this suggests that the source of the error is the precipitation data.

The hyetograph shown in Figure 7.3 was developed from data of the rainfall totals at three time intervals during the day. These data were then distributed throughout the eight hour intervals assuming a minimum intensity of 1mm/hour. The accuracy of the rainfall data can therefore only be guaranteed at the eight hour intervals. The estimates of the time to peak, therefore, should only be judged to the nearest eight hours.

Despite the accuracy bounds posed by the precipitation data, the predicted time to peak are undoubtedly too small for all the module combinations. One feasible interpretation of these results is that the resolution of the subcatchments is too large. The runoff excess is convolved with the unit hydrograph to generate the outflow from a catchment; however, this is only added to channel outflow from the upstream catchment at the downstream extremity of the subcatchment in question.

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This means that the time taken for the runoff to reach the channel is not directly considered.

In the case of the River Fulda catchment and this particular storm event, the analysis in Chapter 6 showed that only the clay soil groups generated runoff and these groups were situated close to the channels. The runoff hydrograph would very rapidly have an effect on the channel flows. However, the subcatchments used in the River Fulda are relatively large; Table 6.1 shows that the largest is 506 km^2 and the smallest is 56 km^2 . The time taken for rainfall falling on the upstream extremity of a subcatchment to be converted into runoff and then routed to the downstream extremity of the catchment could in reality be measurable. The solution to this problem would be to reduce the size of the subcatchments considerably. However, this is not possible in the Fulda catchment as the data set is not detailed enough in this regard.

The importance of the size of the subcatchment in the accuracy of the predicted runoff hydrograph has not been fully investigated by the previous MILHY evaluation programmes. All the test catchments used in the evaluation of the infiltration algorithm were relatively small. The largest subcatchment utilised was the North Creek catchment, Texas, which has an area of 61.6 km^2 ; and many of the subcatchments were less than 1 km^2 in size (Howes, 1986).

This analysis suggests that the subcatchment size utilised in the River Fulda simulation was too large to accurately simulate the time to peak of the runoff hydrograph. The relationship between the accuracy of this characteristic and subcatchment size is not clear and is worthy of further investigation.

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The impact of the momentum exchange and multiple routing routines is investigated in section 7.2.5 where the results of the two storms are compared.

7.2.4 Storm 3: 1 in 1.5 year event

Figure 7.6 shows the hyetograph for subcatchment 402 and the observed outflow hydrograph at Bad Hersfeld for the 1 in 1.5 year event. The hyetographs for each of the subcatchments were derived for this storm from the hourly rainfall data available for a raingauge situated in Bad Hersfeld. The spatial variability of the rainfall was generated from the daily precipitation records available for the 45 rain gauges identified in Figure 6.2. The minimum subcatchment rainfall total occurred in subcatchment 403 where a total of 58mm of rainfall fell, whilst the maximum rainfall of 71mm fell in subcatchment 406.

In comparison with the 1 in 10 year event, the rainfall total of the 1 in 1.5 year event is very similar. The difference between the two events is that in the 1 in 10 year event most of the precipitation fell in one discrete event, whilst the 1 in 1.5 year event is characterised by a double peak in the rainfall and consequent discharge hydrographs.

In both peaks of the 1 in 1.5 year event, the observed hydrograph shown in Figure 7.6 shows that the discharge peak occurs approximately 30 hours after the peak of the rainfall event. This is a similar response time to the 1 in 10 year event.

A summary of the hydrograph characteristics of the simulations using the Curve Number and Infiltration

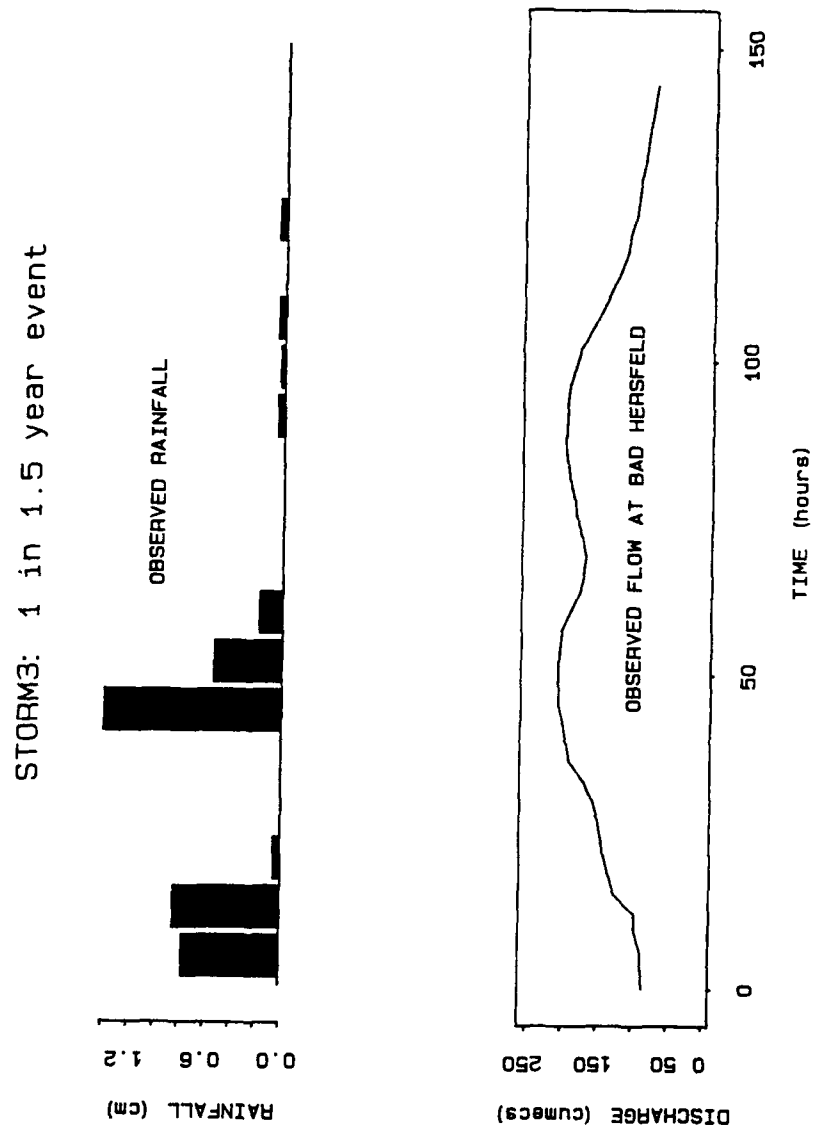


Figure 7.6
Observed Hyetograph for the River Fulda Catchment
and Observed Hydrograph at Bad Hersfeld for the
1 in 1.5 Year Event

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algorithm routines are recorded in Tables 7.9 and 7.10 for the Bad Hersfeld station and Tables 7.11 and 7.12 for Hermannspiegel station. Figures 7.7 and 7.8 show the MILHY, MILHY2 and MILHY3 simulated hydrographs for Bad Hersfeld and Hermannspiegel respectively.

Comparing the Curve Number and Infiltration algorithm simulations at both the Bad Hersfeld and Hermannspiegel stations shows that the error in the prediction between the two techniques is much smaller for the first discharge peak of the hydrograph than for the second. The maximum error between the techniques for the first peak at Bad Hersfeld is approximately 4%, whilst the maximum error in the second peak is 35%. Figure 7.7 illustrates this error and shows that it is the Curve Number module (MILHY) that generates the error by over-predicting the size of the second peak of the hydrograph. This behaviour is confirmed in the Hermannspiegel simulations where the Curve Number second peak prediction is even more pronounced, having only been smoothed by one routing routine.

These simulations expose the superior predictive capability of the infiltration algorithm over the Curve Number. The improved performance of the infiltration algorithm in this storm is due to the capability of the routine to model the infiltration of the rainfall between the two peaks when the rainfall intensity was too low for runoff to occur. Modelling of the infiltration process generated conditions below saturation, so that when the next rainfall peak began the first part of the rainfall was utilised to bring the soil back to saturation before runoff occurred again. The Curve Number routine does not model the process of infiltration and assumes a constant

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Table 7.9
Storm 3 : 1 in 1.5 year event

Predicted Outflow at Bad Hersfeld Utilising the
Curve Number Routine

Computation method		Peak discharge		Time to peak		Runoff depth
		$\text{m}^3 \text{s}^{-1}$		hours		m
		1	2	1	2	
IT=1	MR=0	238	355	19.5	55.5	0.04
IT=2	MR=0	242	359	19.5	55.0	0.04
IT=3	MR=0	235	361	19.5	55.0	0.04
IT=4	MR=0	224	353	19.5	55.0	0.04
IT=1	MR=1	262	364	19.0	54.5	0.04
IT=2	MR=1	273	383	19.0	54.0	0.04
IT=3	MR=1	273	377	19.0	54.5	0.04
IT=4	MR=1	224	352	19.5	55.5	0.04

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Table 7.10
Storm 3 : 1 in 1.5 year event

Predicted Outflow at Bad Hersfeld Utilising the
Infiltration Algorithm

Computation		Peak discharge		Time to peak		Runoff depth
method		$\text{m}^3 \text{s}^{-1}$		hours		m
		1	2	1	2	
IT=1	MR=0	236	238	16.0	55.0	0.04
IT=2	MR=0	241	232	16.0	54.5	0.04
IT=3	MR=0	238	235	16.0	54.5	0.04
IT=4	MR=0	229	235	16.0	54.5	0.04
IT=1	MR=1	240	255	16.0	54.0	0.04
IT=2	MR=1	249	269	16.0	54.0	0.04
IT=3	MR=1	253	266	16.0	54.5	0.04
IT=4	MR=1	226	234	16.0	54.0	0.04

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Table 7.11
Storm 3 : 1 in 1.5 year event

Predicted Outflow at Hermannspiegel Utilising the
Curve Number Routine

Computation method		Peak discharge		Time to peak		Runoff depth
		$\text{m}^3 \text{ s}^{-1}$		hours		m
		1	2	1	2	
IT=1	MR=0	81	138	18.0	52.0	0.05
IT=2	MR=0	81	139	18.0	52.0	0.05
IT=3	MR=0	81	138	18.0	52.0	0.05
IT=4	MR=0	81	135	18.0	52.5	0.05
IT=1	MR=1	81	138	18.0	52.0	0.05
IT=2	MR=1	81	140	18.0	52.0	0.05
IT=3	MR=1	81	132	18.0	52.0	0.05
IT=4	MR=1	81	134	18.0	52.0	0.05

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

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Table 7.12
Storm 3 : 1 in 1.5 year event

Predicted Outflow at Hermannspiegel Utilising the
Infiltration Algorithm

Computation method		Peak discharge		Time to peak		Runoff depth
		$\text{m}^3 \text{ s}^{-1}$		hours		m
		1	2	1	2	
IT=1	MR=0	67	88	13.0	52.0	0.05
IT=2	MR=0	67	88	13.0	52.0	0.05
IT=3	MR=0	67	89	13.0	51.0	0.05
IT=4	MR=0	66	88	13.0	52.0	0.05
IT=1	MR=1	66	88	13.0	52.0	0.05
IT=2	MR=1	66	89	13.0	52.0	0.05
IT=3	MR=1	67	87	13.0	52.0	0.05
IT=4	MR=1	66	84	13.0	52.0	0.05

IT = momentum exchange routine

IT=1 vertical interface, zero shear

IT=2 vertical interface, apparent shear stress ratio = 1

IT=3 diagonal interface, zero shear

IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine

MR=0 routine not invoked

MR=1 routine invoked

BAD HERSFELD STORM3: 1 in 1.5 year event

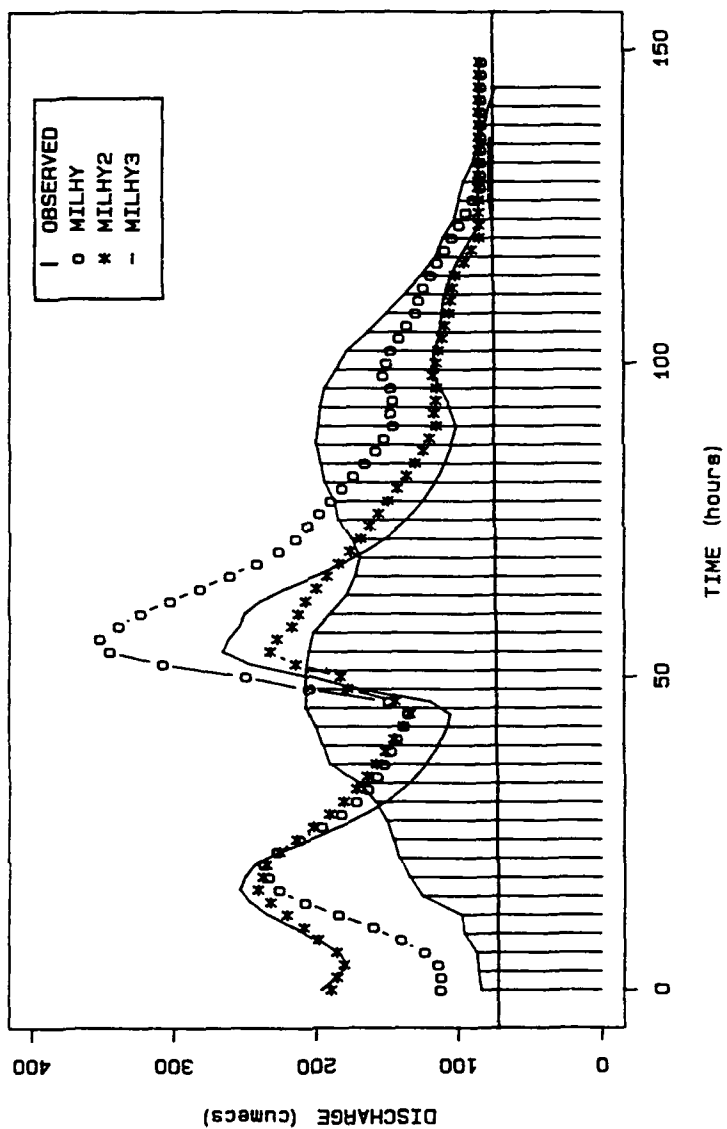


Figure 7.7
Comparison of Observed and Simulated Hydrographs from
MILHY, MILHY2 and MILHY3 Models at Bad Hersfeld for
the 1 in 1.5 Year Event

HERMANNSPIEGAL STORM3: 1 in 1.5 year event

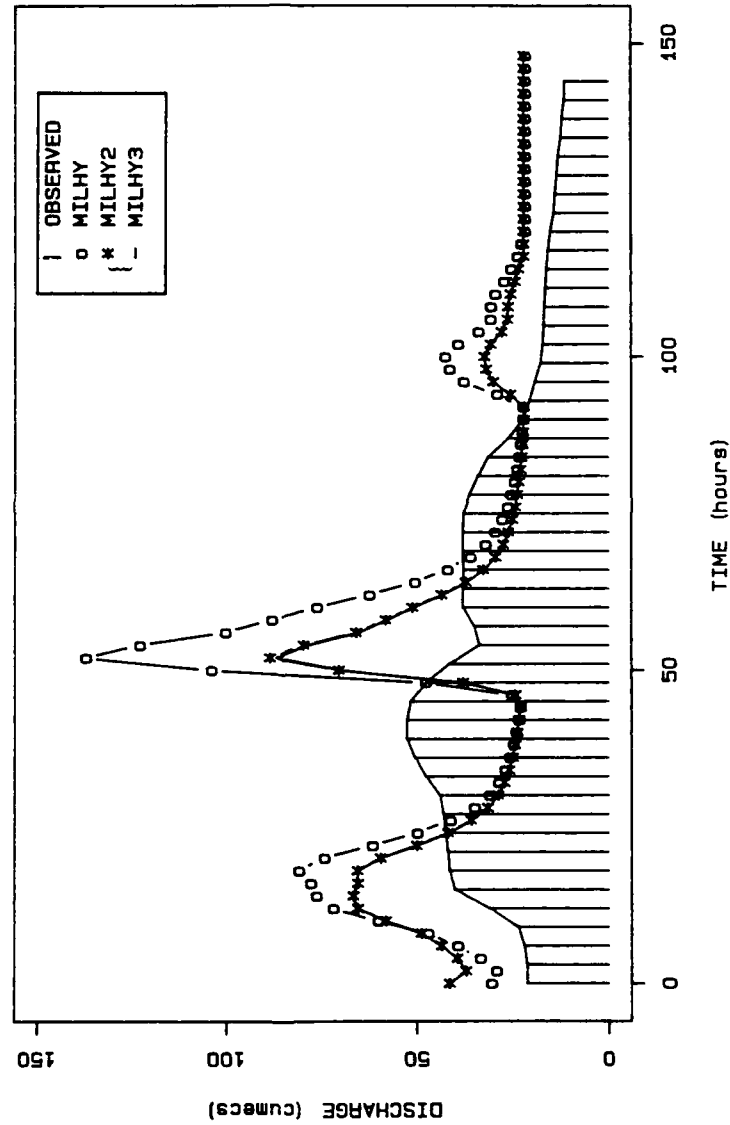


Figure 7.8
 Comparison of Observed and Simulated Hydrographs from
 MILHY, MILHY2 and MILHY3 Models at Hermannspiegel for
 the 1 in 1.5 Year Event

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amount of the rainfall is stored throughout the storm. Consequently the Curve Number routine generated too much runoff in the second peak.

Figures 7.7 and 7.8 confirm the conclusions drawn from the 1 in 10 year event, that the subcatchment areas are too large for the accurate prediction of the time to peak discharge of the hydrograph. The figures show that the observed time to peak of the first peak is approximately 50 hours from the start time of the simulation. The simulated time to peak for the Curve Number and Infiltration algorithm predict the time to peak being approximately 16 and 19 hours respectively. Compared to the error between the observed and predicted time to peak, the discrepancy between the two techniques is insignificant.

The effects of the momentum exchange and the multiple routing routines is explored in the next section, where the results from the two storms are compared.

7.2.5 Comparison of the two storm events

Comparison of the simulation results from the 1 in 10 year and 1 in 1.5 year events provides a means of investigating the impact of the inundation stage on the relative importance of the momentum exchange and multiple routing routines.

The Tables 7.5, 7.6, 7.7, 7.8, 7.9, 7.10, 7.11 and 7.12 show that the impact of the multiple routing routine on the prediction of the peak discharge is greater than the impact of the momentum exchange techniques. This is particularly true at the Bad Hersfeld station, where a much greater proportion of the flow is routed on the

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floodplain. At Hermannspiegel only a small proportion of the flow is carried on the floodplain.

The pattern of predictions made by the four momentum exchange routines confirms the analysis made in Chapter 4, which showed the method employed in MILHY2, that is IT=2, gives the highest estimate of the channel capacity. Method 4, with a diagonal interface and an apparent shear stress ratio = 1, introduces the most turbulence into the cross-section and as a result predicts the lowest channel capacity of the four methods.

In this analysis the impact of the differences in the cross-sectional conveyance capacity are converted to discrepancies in the peak discharge predictions. Method 4, which produces the lowest channel capacity estimate, therefore assigns the greatest proportion of the flow into the floodplain segments. At these relatively low inundation depths on the floodplain, the effects of boundary friction are large and hence the attenuation of the floodwave is increased and the peak discharge estimate reduced.

Figures 7.9 and 7.10 illustrate the impact of the momentum exchange and multiple routing routines on the predicted outflow at the Bad Hersfeld station for both of the storms. By comparing these figures with Figures 7.4 and 7.7, the effects of the two routines applied separately and together can be assessed. The momentum exchange method utilised throughout this analysis is method 3, which uses a diagonal interface and zero shear. This method was selected because it was the technique recommended by Knight and Hamed (1984), and because it incorporates a different interface inclination and apparent shear stress ratio to the method incorporated in the MILHY2 model.

MULTIPLE ROUTING

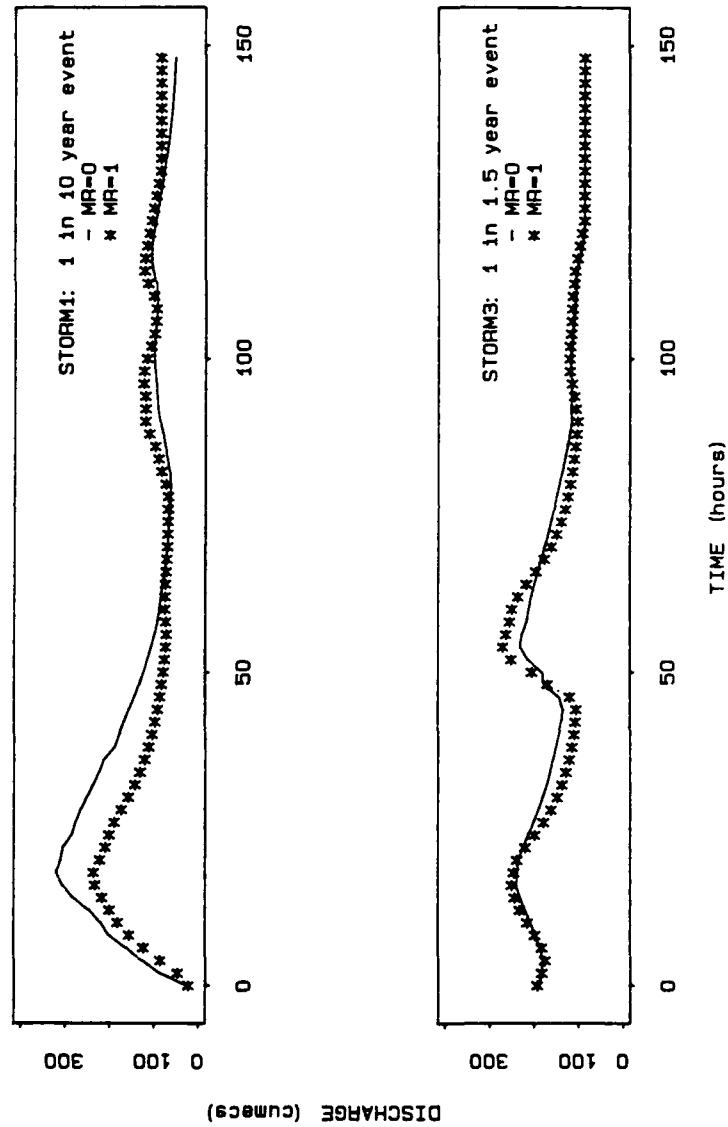


Figure 7.9
 Comparison of the Impact of the Introduction of
 Multiple Routing for the 1 in 10 and 1 in 1.5 Year Events

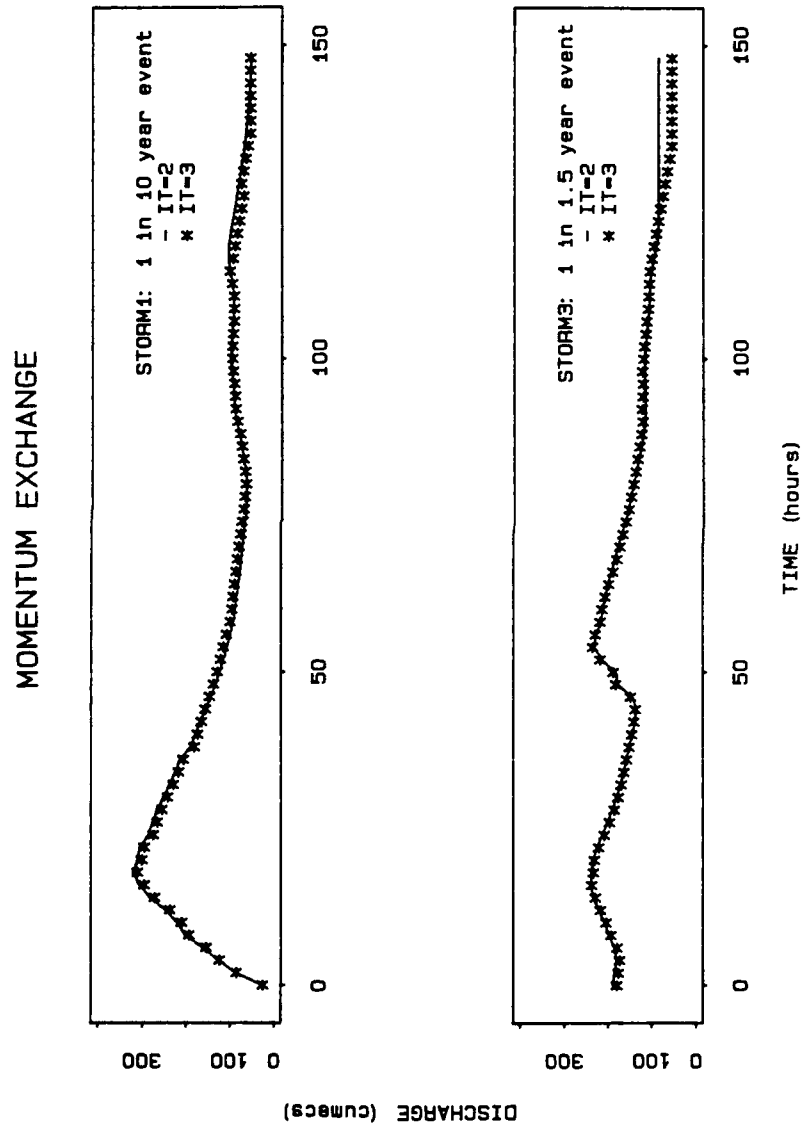


Figure 7.10
Comparison of the Impact of the Introduction of
 Momentum Exchange for the 1 in 10 and 1 in 1.5 Year Events

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Figure 7.10 shows that the momentum exchange method 3 has no noticeable impact on the predicted hydrographs of either storm event when applied without the multiple routing routine. Figure 7.9 shows, however, that the multiple routing routine does have a significant effect on the hydrographs and that the routine's impact varies between the two storm events. In Storm 1 the main peak of the hydrograph is reduced whilst the minor peaks on the recession limb are accentuated. In Storm 3, by contrast, the recession from the first peak is steepened and the second peak is significantly accentuated.

In Storm 1 it is important to appreciate that the floodplain flow only occurs from the station at Unter-Schwartz to Bad Hersfeld on the River Fulda, a channel length of 33 km. The inflow hydrographs at Unter-Schwarz for the simulation utilizing the multiple routing routine are identical to that without the routine, and the inflow from the River Haune is channel flow only and therefore can be ignored from the analysis. Comparison, therefore, concentrates on the travel times for the Unter-Schwarz to Bad Hersfeld reach for the two simulations.

At the main peak of the hydrograph of around $350\text{m}^3\text{s}^{-1}$, in the application without the multiple routing, the time taken for the peak to travel the length of the reach is 13 hours. In the multiple routing simulation, 45% of this peak is apportioned to the floodplain where the travel time is 19 hours. The remaining 55% is assigned to the main channel where the travel time is only 5 hours. This difference in travel times means that the peak of the hydrograph is flattened out in the multiple routing simulations, as there are effectively two peaks coinciding from the floodplain and main channel.

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In the minor peaks, the effect of the division of the floodplain and channel flows is rather different. Looking at two points on the inflow hydrograph, the travel time without multiple routing is 11.4 hours, whereas with multiple routing the floodplain travel time is 55 hours, and the main channel travel time is 6 hours. However, as only 4% of the flow is assigned to the floodplain, when the multiple routing routine is invoked the flow arrives earlier and the peaks are less attenuated.

The results from the simulation of Storm 1 suggest that the effects of the multiple routing routine are determined by the percentage of flow that is assigned to the floodplain and channel segments.

Storm 3 confirms this conclusion, because when 15% of the flow is assigned to the floodplain, the multiple routing routine prediction is more attenuated than the single routine prediction, due to the longer travel time of the floodplain flow segment. Where the floodplain flows account for 10% or less of the total discharge the multiple routing prediction is less attenuated than the single routing technique, due to the travel time of the channel flow segment.

The predicted hydrographs at Bad Hersfeld for the application of both the momentum exchange and multiple routing routines (MILHY3) are shown in Figures 7.4 and 7.7, for Storms 1 and 3 respectively. Comparison with Figures 7.9 and 7.10 shows that the hydrograph for Storm 1 is significantly different from those produced by the application of the two routines separately. The joint application of the routines in Storm 3 hydrograph matches the predicted hydrograph from using the multiple routing routine alone. It would seem that the effects of applying both routines varies according to the storm.

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In Storm 1, the MILHY3 prediction seems particularly strange as, when compared to the MILHY2 solution, the MILHY3 model predicts the main peak as being earlier and the attenuation is less. This contrasts with the results shown in Figure 7.9, where the multiple routing routine increases the attenuation of the peak. Analysis of the rating curves and travel times generated by MILHY3 and those from the simulation shown in Figure 7.9 showed that it was the travel times that control the attenuation of the hydrograph. When the routines are applied together, the travel times are reduced and more of the flow is assigned to the floodplain. The momentum exchange routine generates small changes in the rating curve and in the travel time. However, these changes have no impact when the momentum exchange routine is applied without the multiple routing routine, see Figure 7.10. When applied with the multiple routing routine, these small changes become significant. For example, at the fifth hour of the simulation, for $IT=2$ and $MR=1$, 18% ($39\text{m}^3\text{s}^{-1}$) of the total discharge is assigned to the left floodplain. This water has a travel time of 70 hours. In contrast, when $IT=3$, $MR=1$ (MILHY3), 26% ($58\text{m}^3\text{s}^{-1}$) of the discharge is assigned to the left floodplain and the travel time is 60 hours.

In Storm 3, however, there are no noticeable differences in the hydrographs produced by the single application of the multiple routing routine and the joint application of the multiple routing routine and momentum exchange routines. Although there are differences in the travel times between these two techniques, these differences do not become significant as a much greater proportion of the hydrograph is out-of-bank.

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7.2.6 Conclusions

The analysis of the factor perturbation investigation reported above has identified several important points for the application of MILHY3. These points are:

- 1) Catchment areas of 145 km^2 may be regarded as the minimum for which 5 km^2 grid square precipitation data has an impact on the outflow hydrograph for low intensity storms. For higher intensity storms, then the application of 5 km^2 resolution data generates a maximum variability in the outflow hydrograph of no greater than 5%.
- 2) The analysis has shown that there is a need to investigate the relationship between the subcatchment area utilised and the accuracy of the time to peak predictions. The area of subcatchments used in this analysis have proved to be too large.
- 3) The analysis has shown that the infiltration algorithm predicts the complex two-peaked hydrograph of Storm 3 to a much greater degree of accuracy than the Curve Number routine.
- 4) With a simpler hydrograph shape, such as Storm 1, the analysis has shown that the improvement in the accuracy of the hydrograph generated by the utilisation of the infiltration algorithm over the Curve Number routine is much smaller (5-10%). Given the extra time taken to establish the data set and the CPU demands of the infiltration algorithm, the user may consider the infiltration algorithm is not necessary for such an application.
- 5) The impact of the multiple routing routine on the predicted hydrograph is generally much greater than the

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impact of the momentum exchange routines when they are applied separately. The impact of each of these routines is dependent on the proportions of the total flow contained in the floodplain and main channel segments.

6) The impact of the momentum exchange and multiple routing routines when applied together is different from their impact when applied separately.

7) When the multiple routing routine is applied separately, the attenuation of the floodwave is increased if the floodplain flows account for 15% or more of the total flow. When floodplain flows account for 10% or less of the total flow, the attenuation of the floodwave is reduced. This is due to the larger travel times of the floodplain and smaller travel times of the channel in comparison with the single channel method of computation.

8) When applied separately, the momentum exchange routine makes no significant impact on the predicted hydrograph, despite the small changes the four routines make on the calculated conveyance capacity of the channel and the travel time table.

9) When the momentum exchange routine and multiple routing routine are applied together, the small changes generated by the momentum exchange become significant if 15% or more of the flow is assigned to the floodplain.

10) For cases where the floodplain flow accounts for 15% or more of the total discharge, then the joint application of the momentum exchange and multiple routing routine improves the accuracy of the predicted hydrograph.

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7.3 The Optimization Approach

There are two objectives for undertaking a sensitivity analysis utilising optimization techniques. These are:

- 1) to investigate the sensitivity of the various module combinations of MILHY3 to variability in the parameters, by analysing the intermediate iterative solutions as the optimization scheme converges to a minimum, and
- 2) to explore the utility of optimization techniques as part of a structured sensitivity analysis specifically for areas where field data is limited.

As noted earlier, the technique of optimising the fit of parameters in hydrologic models using a sensitivity analysis for the purposes of calibration, is well established. Applications have included Armstrong et al. (1980) and Ibbitt and O'Donnell (1971). McCuen (1973a) identifies a range of techniques mostly based on the work of Cauchy (1847), who developed the method of converging the solution utilizing the rate of descent or gradient of an objective function of the models output in response to parameter input variability.

A range of optimization techniques for minimising and maximising a function is available in the NAG (Numerical Algorithms Group) library. Libraries such as these are widely available on high and low level main frame and mini computers. Depending on the level of sophistication required, and the availability of the derivatives of the function, an appropriate routine can be selected. A simple routine was selected for this exploratory investigation (e04jaf) which allows the user to select the

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upper and lower boundaries of each variable and does not require derivatives. The routine works by developing a surface of values for a function (F) that describes the difference between a computed value and an observed value. The routine then searches for a minimum in this surface by selecting parameter values within specified boundaries.

A prerequisite, therefore, of this approach is that a function can be computed that adequately describes, in this case, the difference between an observed and computed hydrograph. The 'least squares' approach was identified as being a function already computed by MILHY3, in subroutine 'ERROR', and provided a simple test of the fit of the observed hydrograph. Figure 7.11 describes how the MILHY3 model, the function and the optimization routine, e04jaf, fit together schematically. In terms of the computer coding, MILHY3 is treated as a function called by e04jaf, which is itself called by a short front program which sets up the boundary conditions. Once the routine is running, it is difficult to interrupt as all the commands are issued by the library routine, e04jaf.

As this investigation was exploratory in nature and because of the concentration of the analysis on the downstream conveyance subroutine, the infiltration algorithm and Curve Number routines were not included in the optimization scheme. The demands of the processor due to the iterative nature of the optimization scheme, and the storage of the results files were foreseen as potential problem areas.

Setting up the optimization scheme, shown in Figure 7.11, proved a reasonably straight forward task, complicated only by the intermittent nature of the 'read' statement in MILHY3. The 'read' statements were rewritten so that all the commands and data were read in the front

OPTIMIZATION SCHEME

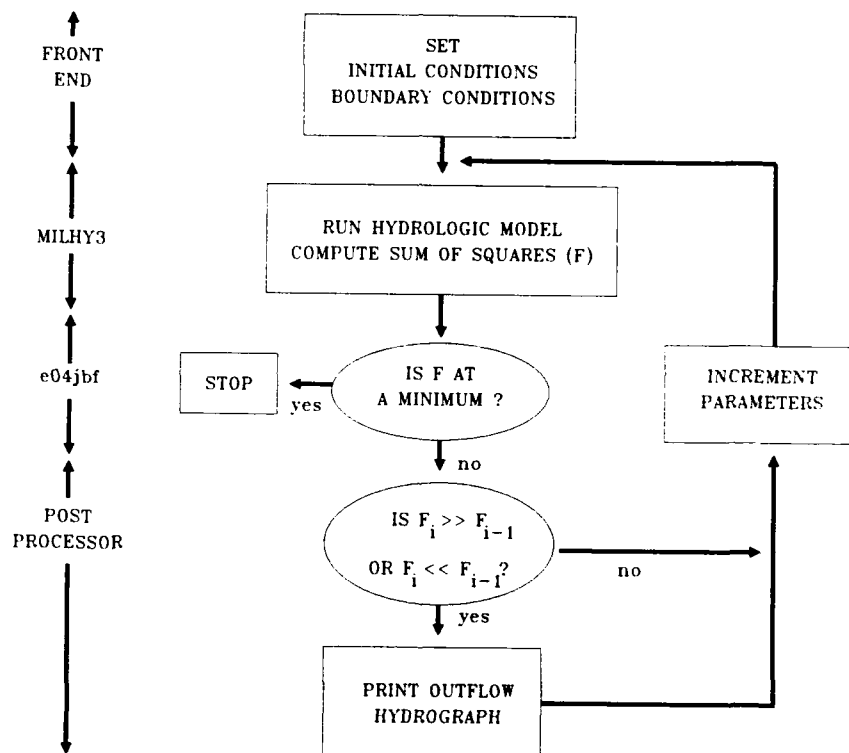


Figure 7.11
Optimization Scheme

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and routine. All 'write' statements were edited out, bar the warnings and failure statements, and the printing of the outflow hydrograph. The ability of MILHY3 to tolerate any set-up structure in the 'datal' data set was retained, to allow the use of the multiple routing routine which is invoked using additional commands in the data set.

MILHY3 then had to be fronted so that it appeared as a function to the optimization routine. This necessitated the addition of several COMMON BLOCKS to ensure all the data was correctly passed from the initialization (front-end) routine. Lastly, all the parameters had to be defined as being double-precision to enable them to be correctly incremented by e04jaf.

To test that the optimization was working properly and reaching a minimum, for one particular application, three simulations were undertaken. In each of these simulations the initial parameter values were changed to check that the scheme was stopping at an absolute minimum. Each of the three simulations started from either the upper or lower boundary limits or mid-point between these limits. The function values at which these three simulations stopped at, however, were not the same.

Analysis of the iterative solutions of the three simulations showed that each of the simulations became lodged in local minima close to the initial conditions. The local minima and the hydrograph predictions resulting from the three simulations, were widely different. A closer examination of the parameter values as each of the simulations converged on its local minimum showed that the parameter values were only changing from iteration to iteration by approximately 1×10^{-4} . The resolution of such parameter changes is too small to generate any change in the predicted hydrograph and hence there was no change in

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the objective function and the solution converged. Three problems associated with the resolution of the parameter variability were therefore identified. These are:

- 1) that the simulation failed to converge on an absolute minimum
- 2) the parameter variability increments cannot be resolved with the accuracy of the data available, in ungauged catchment applications
- 3) the parameter variability increment caused no interpretable changes in the predicted hydrograph.

The solution to these problems was to replace the optimization scheme, e04jaf, with a scheme that allows the user to select the size of the increment steps. Such a scheme, e04jbf, also allows the maximum number of iterations to be specified and allows an estimate of the likely value of the objective function at its minimum to be specified. These additional features should reduce the number of iterations required and therefore reduce the time taken by the simulation. However, as the routine e04jbf is more complex it would take the novice user longer to establish. Further, the CPU demands of the scheme are greater.

Having established the logic of the optimization scheme for the e04jaf, the alterations for the e04jbf scheme were minimal as the overall scheme, shown in Figure 7.11, remained unchanged. Trial simulations of e04jbf showed that an absolute minimum was reached from the simulations initiated at the upper and lower boundary conditions. These results proved promising enough to encourage application to the River Fulda data set.

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7.3.1 Application of optimization to the River Fulda

As this investigation is rather exploratory in nature, a single reach and storm were selected to investigate the utility of optimization techniques. As noted earlier, the optimization scheme is being tested on only the downstream conveyance components of MILHY3; that is the rating curve, travel time and routing modules. An observed hydrograph is input at the top of the reach and the observed outflow at the downstream end of the reach is compared with the simulated hydrograph. The additional runoff generated between the upstream and downstream gauging stations is not simulated.

The reach selected is between the stations Bad Hersfeld and Rotenburg on the River Fulda, and the storm event is the 1 in 10 year event shown in Figure 5.3.

Two methods of computing the objective function (F), were selected to investigate whether the value of the function (F) had any influence on the parameter values selected. The function (F) describes the difference between the observed and predicted outflow hydrographs. These two techniques are:

- 1) ordinary sum of squares

$$OF2 = \sum_{i=1}^n (q_{m_i} - q_{m_c})^2$$

7.1

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- 2) absolute error divided by variance

$$OF7 = \frac{\sum_{i=1}^n (q_{m_i} - q_{m_c})^2}{\sum_{i=1}^n (q_{m_i} - \bar{q}_m)^2} \quad 7.2$$

where q_m - observed peak discharge
 q_c - computed peak discharge
 $\bar{q}_m$ - mean peak discharge

Both of these methods of analysis are incorporated in the subroutine 'ERROR', which is described in Volume II and remains unchanged in MILHY3.

Five variables were identified for this investigation; these are:

- 1) Floodplain Manning's n
- 2) Main channel Manning's n
- 3) Floodplain slope
- 4) Main channel slope
- 5) Floodplain routing reach length

These five parameters were selected as they represent the most important physical parameters identified by the analysis of the Ervine and Ellis (1987) scheme in controlling the discharge and velocity predictions in two-stage channels, (see Chapter 3).

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The upper and lower boundary limits for the five parameters identified are given in Table 7.13, which also includes the initial values for the values at the start of each simulation and specifies the increment intervals. The maximum number of iterations was specified as 500, although the trial simulations showed that a minimum at the parameter resolution feasible for field investigations was reached after 400 iterations. The specifications for the acceptance of the absolute minimum in the optimization scheme were rather more stringent than necessary for the purposes of a sensitivity analysis. All of these values were selected based on experience in application of the MILHY3 scheme during the development of the program and the evaluation programme. It is accepted that an investigation of these limits is necessary before the sensitivity of MILHY3 can be determined with any degree of confidence. However, the subjective selection of the boundary conditions is sufficient for this exploratory investigation.

As the computer demands of this approach were foreseen as being a major limitation of the scheme, a record was kept of the CPU demands and the size of the output files. Output at each iteration was limited to the parameter values and the function (F) value, the post-processor added the computed hydrograph approximately every tenth iteration. The CPU demands for 500 iterations for the single reach varied from 400 to 800 seconds, depending on the module combination utilised. Trial simulations for the 2500 km² Fulda catchment, utilising the Curve Number routine for the generation of the runoff hydrograph took 6000 seconds of CPU. As every iteration using the Infiltration Algorithm was taking approximately 9 hours (324000 seconds) an optimization trial using the infiltration algorithm was not attempted.

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Table 7.13
Initial Conditions, Boundary Conditions and Variable
Increments for the Optimization Scheme

	Initial Conditions	Boundary Limits		Variable Increments
		Upper	Lower	
Floodplain Manning's n	0.05	0.16	0.025	0.01
Channel Manning's n	0.035	0.1	0.025	0.01
Floodplain Slope	0.0006	0.001	0.0001	0.0001
Channel Slope	0.0006	0.001	0.0001	0.0001
Floodplain routing reach length(m)	21951	23750	16860	1525

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The size of the output files proved to be more of a limitation. Sizes varied from 12 kbytes to 2.9 megabytes, the latter being too large to edit. Clearly the post-processor needs to be more selective in the iteration results it saves. Because of the size of the results files, the results presented here are only a small selection of the most interesting data produced by the analysis.

The representation of the results has been structured in order to answer three questions. These are:

- 1) to which of the five parameters is the hydrograph most sensitive?
- 2) does the method of computation affect this sensitivity?
- 3) do the parameters interact to increase or decrease the sensitivity of the hydrograph?

In much of the analysis presented, the relative error and absolute errors are used to compare the differences between the computational methods. The relative error is dimensionless and is defined as:

$$RE = \frac{x_c - x_i}{x_i} \quad 7.3$$

where x_c - computed value of x
 x_i - value of x under initial conditions

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7.3.2 Sensitivity to parameter variability

Tables 7.14, 7.15, 7.16, 7.17 and 7.18 show the absolute error (AE) and relative error's (RE) for the peak discharge, time to peak and sum of squares (OF2) for five of the possible module combinations. The errors are computed for one increment step above and below the initial conditions. These tables contain data from the the first iterations of the optimization simulations, shown schematically in Figure 7.12.

Analysis of these tables shows the asymmetrical sensitivity of the three indicators, peak discharge, time to peak and sum of squares, around the initial conditions. For example, Table 7.14 shows that the sensitivity of the peak discharge to variation in the floodplain Manning's n value, is markedly different for values greater than the initial conditions than for values less than the initial conditions. This asymmetrical effect is particularly noticeable for the variation in Manning's n , both in the floodplain and channel (see Tables 7.14 and 7.15), suggesting that the sensitivity of the hydrograph to variation in ' n ' is not linear.

Tables 7.14 to 7.18 show the sensitivity of the hydrograph to a one increment step in the mid-point between the upper and lower boundary limits for all five variables. Table 7.19 shows an example of the relative errors generated from a one increment step in each of the parameters at the boundary limits, and compares these with the mid-limit values. This particular example compares the relative errors in the peak discharge for one module combination and is typical of the tables derived for other combination methods.

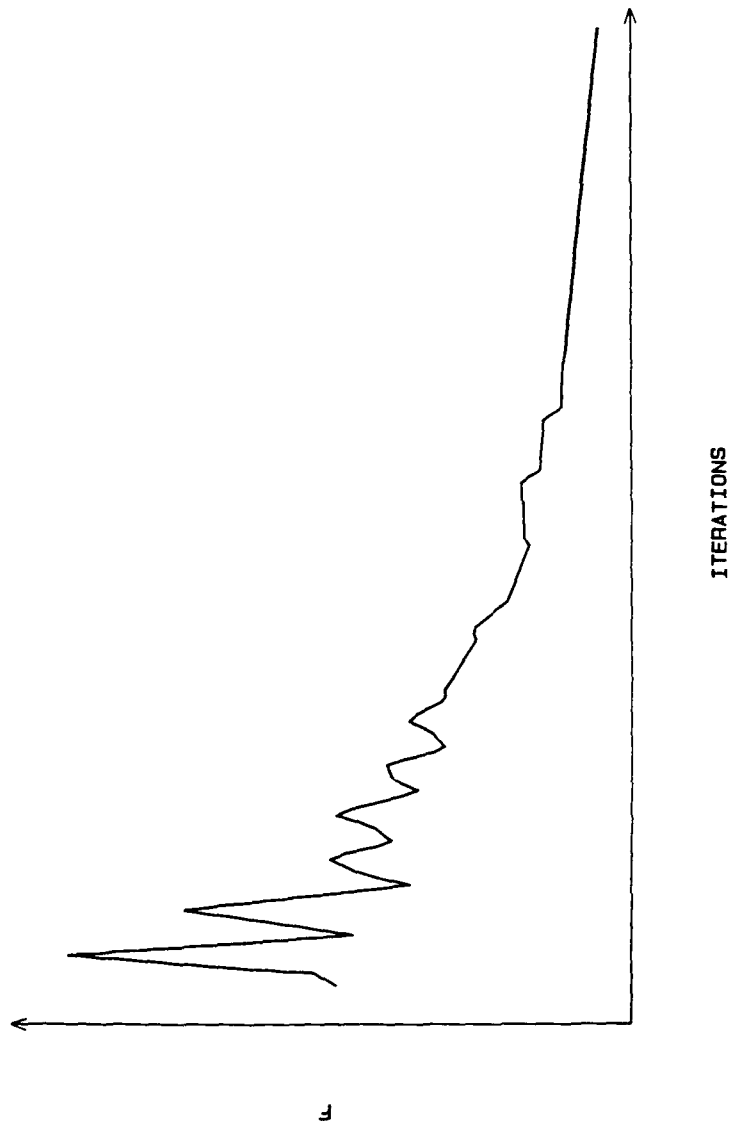


Figure 7.12
Conceptual Convergence of Function F to a Minimum During
Application of the Optimization Scheme

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Table 7.14
 Errors from one increment step variation in
floodplain Manning's n

Increment	Computation method	Peak discharge		Time to peak		Sum of squares	
		AE m ³ s ⁻¹	RE	AE hours	RE	AE OF2	RE
+1	IT=2 MR=0	-125	0.01	0	0.00	62941	0.00
	IT=3 MR=0	-115	0.13	0	0.08	62951	0.31
	IT=1 MR=1	-107	0.01	+9	0.00	124796	0.12
	IT=2 MR=1	-127	0.10	+9	0.07	105431	0.12
	IT=3 MR=1	-88	0.03	0	0.00	86893	0.08
0	IT=2 MR=0	-121		0		63177	
	IT=3 MR=0	-72		-3		91568	
	IT=1 MR=1	-110		+9		111038	
	IT=2 MR=1	-95		+6		94106	
	IT=3 MR=1	-79		-3		94782	
-1	IT=2 MR=0	-123	0.01	0	0.00	68801	0.09
	IT=3 MR=0	-71	0.00	-3	0.00	92675	0.01
	IT=1 MR=1	-71	0.13	+6	0.06	92118	0.17
	IT=2 MR=1	-102	0.02	+3	0.07	70169	0.25
	IT=3 MR=1	-70	0.03	-3	0.00	106950	0.13

IT = momentum exchange routine
 IT=1 vertical interface, zero shear
 IT=2 vertical interface, apparent shear stress ratio = 1
 IT=3 diagonal interface, zero shear
 IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine
 MR=0 routine not invoked
 MR=1 routine invoked

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Table 7.15
Errors from one increment step variation in
channel Manning's n

Increment	Computation method	Peak discharge		Time to peak		Sum of squares	
		$\frac{AE}{m \cdot s^{-1}}$	RE	AE hours	RE	AE OF2	RE
+1	IT=2 MR=0	-138	0.06	0	0.00	68270	0.08
	IT=3 MR=0	-91	0.06	-3	0.00	74276	0.19
	IT=1 MR=1	-143	0.11	+9	0.00	85134	0.23
	IT=2 MR=1	-103	0.03	+9	0.07	107817	0.15
	IT=3 MR=1	-82	0.01	-3	0.00	91518	0.03
0	IT=2 MR=0	-121		0		63177	
	IT=3 MR=0	-72		-3		91568	
	IT=1 MR=1	-110		+9		111038	
	IT=2 MR=1	-95		+6		94106	
	IT=3 MR=1	-79		-3		94782	
-1	IT=2 MR=0	-105	0.06	0	0.00	68694	0.09
	IT=3 MR=0	-96	0.07	-3	0.00	73639	0.19
	IT=1 MR=1	-96	0.05	+3	0.13	70847	0.36
	IT=2 MR=1	-58	0.12	+3	0.07	68635	0.27
	IT=3 MR=1	-73	0.02	-3	0.00	107051	0.13

IT = momentum exchange routine
 IT=1 vertical interface, zero shear
 IT=2 vertical interface, apparent shear stress ratio = 1
 IT=3 diagonal interface, zero shear
 IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine
 MR=0 routine not invoked
 MR=1 routine invoked

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Table 7.16
Errors from one increment step variation in
floodplain slope

Increment	Computation method	Peak discharge		Time to peak		Sum of squares	
		AE m ³ s ⁻¹	RE	AE hours	RE	AE OF2	RE
+1	IT=2 MR=0	-120	0.00	0	0.00	63421	0.00
	IT=3 MR=0	-72	0.00	-3	0.00	92115	0.00
	IT=1 MR=1	-105	0.02	+9	0.00	109250	0.02
	IT=2 MR=1	-89	0.02	+6	0.00	92953	0.01
	IT=3 MR=1	-75	0.01	-3	0.00	98774	0.04
0	IT=2 MR=0	-121		0		63177	
	IT=3 MR=0	-72		-3		91568	
	IT=1 MR=1	-110		+9		111038	
	IT=2 MR=1	-95		+6		94106	
	IT=3 MR=1	-79		-3		94782	
-1	IT=2 MR=0	-123	0.01	0	0.00	62997	0.00
	IT=3 MR=0	-112	0.12	0	0.08	63756	0.30
	IT=1 MR=1	-99	0.04	+9	0.00	124144	0.12
	IT=2 MR=1	-110	0.05	+3	0.07	104471	0.11
	IT=3 MR=1	-83	0.01	-3	0.00	90447	0.05

IT = momentum exchange routine
 IT=1 vertical interface, zero shear
 IT=2 vertical interface, apparent shear stress ratio = 1
 IT=3 diagonal interface, zero shear
 IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine
 MR=0 routine not invoked
 MR=1 routine invoked

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Table 7.17
Errors from one increment step variation in
channel slope

Increment	Computation method	Peak discharge		Time to peak		Sum of squares	
		AE m s ⁻¹	RE	AE hours	RE	AE OF2	RE
+1	IT=2 MR=0	-117	0.01	0	0.00	63290	0.00
	IT=3 MR=0	-107	0.10	0	0.08	66138	0.28
	IT=1 MR=1	-112	0.01	+9	0.00	107855	0.03
	IT=2 MR=1	-100	0.02	+6	0.00	90519	0.04
	IT=3 MR=1	-78	0.00	-3	0.00	96431	0.02
0	IT=2 MR=0	-121		0		63177	
	IT=3 MR=0	-72		-3		91568	
	IT=1 MR=1	-110		+9		111038	
	IT=2 MR=1	-95		+6		94106	
	IT=3 MR=1	-79		-3		94782	
-1	IT=2 MR=0	-126	0.02	0	0.00	63223	0.00
	IT=3 MR=0	-78	0.02	-3	0.00	84485	0.08
	IT=1 MR=1	-103	0.02	+9	0.00	110171	0.01
	IT=2 MR=1	-121	0.08	+9	0.07	108635	0.15
	IT=3 MR=1	-80	0.00	-3	0.00	93286	0.02

IT = momentum exchange routine
 IT=1 vertical interface, zero shear
 IT=2 vertical interface, apparent shear stress ratio = 1
 IT=3 diagonal interface, zero shear
 IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine
 MR=0 routine not invoked
 MR=1 routine invoked

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Table 7.18
Errors from one increment step variation in
floodplain routing reach length

Increment	Computation method	Peak discharge		Time to peak		Sum of squares	
		AE m ³ s ⁻¹	RE	AE hours	RE	AE OF2	RE
+1	IT=1 MR=1	-114	0.01	+9	0.00	114505	0.03
	IT=2 MR=1	-100	0.02	+6	0.00	96189	0.02
	IT=3 MR=1	-82	0.01	-3	0.00	91338	0.04
0	IT=1 MR=1	-110		+9		111038	
	IT=2 MR=1	-95		+6		94106	
	IT=3 MR=1	-79		-3		94782	
+1	IT=1 MR=1	-106	0.01	+9	0.00	106985	0.04
	IT=2 MR=1	-111	0.05	+9	0.07	91810	0.02
	IT=3 MR=1	-769	0.01	-3	0.00	98741	0.04

IT = momentum exchange routine
 IT=1 vertical interface, zero shear
 IT=2 vertical interface, apparent shear stress ratio = 1
 IT=3 diagonal interface, zero shear
 IT=4 diagonal interface, apparent shear stress ratio = 1

MR = multiple routing routine
 MR=0 routine not invoked
 MR=1 routine invoked

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Table 7.19
Relative error in peak discharge from one increment
variation at the boundaries and mid-way

Variable	Relative Errors		
	Upper Boundary	Mid-way	Lower Boundary
Floodplain Manning's n	0.053	0.10	0.01
Channel Manning's n	0.003	0.025	0
Floodplain Slope	0.083	0.019	0.058
Channel Slope	0.083	0.014	0.03
Routing reach length	0.029	0.014	0.011

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Table 7.19 shows that at both the upper and lower boundaries the outflow hydrograph is most sensitive to slope. In the mid-ranges, the hydrograph is most sensitive to Manning's n . For MILHY3 applications with significant inundation, it is important to define the Manning's n values as accurately as possible, especially on the floodplain. Table 7.19 also shows that the outflow hydrograph is not sensitive to relatively small changes in the floodplain routing length, except when the slopes are steep, that is approximately 1×10^{-3} or greater.

7.3.3 Sensitivity variations associated with the computational method

The computational method of the optimization scheme incorporates two sources of variation:

- 1) the structure of MILHY3, specifically in this case which of the momentum exchange routines has been used and whether the multiple routing routine has been invoked;
- 2) the factor (F) used to quantify the difference between the observed and computed hydrographs

Section 7.2 investigated the sensitivity of the outflow hydrograph to variations in MILHY3's structure for two storms in the River Fulda catchment. Here the relative impact of variability in the model structure in comparison with the effects of variability in the physical parameters can be assessed. In addition the impact of the module combination on the sensitivity of the hydrograph to parameter variability is investigated. Tables 7.14 to 7.18 compare the relative errors for three measures of hydrograph fit, for a range of model structures.

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Section 7.3.2 identified Manning's n as being the variable to which the predicted hydrograph is most sensitive. Tables 7.14 and 7.15 show, however, that the relative errors between momentum exchange and multiple routing routine combinations are as great, if not greater in some circumstances, than errors generated from Manning's n variability. This suggests, therefore, that the momentum exchange and multiple routing routines are significant processes in comparison with the effects of boundary friction.

If the sensitivity of the model to variation in the five physically-based parameters were not affected by the model structure, then it could be expected that the relative errors for all the module combinations would be the same. The fact that there are variations suggests that certain module combinations increase the sensitivity of the model to parameter change. This problem is particularly noticeable in the sensitivity to variation in floodplain Manning's n and in the channel Manning's n with the introduction of the multiple routing routine.

The variation in the relative error between the momentum exchange techniques can be attributed to the difference in the predicted conveyance capacity of the two-stage channel computed by the four alternative techniques. The four techniques utilise the Manning equation to compute the conveyance capacity and the discrepancy between the techniques is attributable to the different methods the four techniques utilise to compute the hydraulic radius of the two-stage channel.

For example, momentum exchange method 3 (diagonal interface with a zero shear interface) reduces the peak discharge prediction when the Manning's n coefficient is increased by one increment step in Table 7.14. The

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discharge prediction is reduced to a greater extent than momentum exchange method 2. This is because more of the flow is assigned to the floodplain segment in method 3 than in method 2, as the predicted conveyance capacity of the channel is smaller in method 3 than in method 2. This makes method 3 more sensitive to increases in the Manning's n coefficient. An increase in the Manning's n coefficient does not have the opposite effect, however, and increase the peak discharge prediction of method 3 to any large degree. As the analysis in Chapter 4 has shown, an increase in the volume of water on the floodplain can increase the attenuation of the floodwave, as there is an effective double peak from the channel and the floodplain flow segments.

The results from Chapter 4 show that momentum exchange method 4 (diagonal interface with an assumed apparent shear ratio = 1) has the lowest conveyance capacity and therefore method 4 has the greatest proportion of water on the floodplain. In contrast, momentum exchange method 2 (vertical interface with an apparent shear stress ratio = 1) has the highest conveyance capacity and, therefore, the lowest proportion of flow on the floodplain. Method 1 (vertical interface with zero shear) and method 3 (diagonal interface with zero shear) produce conveyance estimates between the two extremes. It is logical, therefore, to expect the increase in Manning's n coefficient in the floodplain to affect method 3 and 4 most, whilst the increase in channel Manning's n to affect methods 1 and 2 most.

However, this situation is complicated by the addition of the multiple routing routine which, depending on the depth of flow on the floodplain, can increase or decrease the attenuation of the floodwave. So whilst method 2 is most sensitive to the increase in the

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Manning's n in the channel, with most of the water being assigned to the channel, the multiple routing techniques route this water more rapidly down the reach than the single routing technique. The peak discharge of the multiple routing technique is therefore less reduced than the single routing method, for momentum method 2.

The results confirm that the effects of the momentum exchange and multiple routing routine on the hydrograph are often opposing. In terms of the sensitivity of the modules to variation in the five physically-based parameters, these results show that variation in the Manning's n coefficient is most sensitive to change in the modules. The effects of Manning's n sensitivity of the momentum exchange routines, whether it is in the channel or on the floodplain, is reduced by the addition of the multiple routing routine.

The second question raised in this section is: does the function (F) utilized to describe the fit of the predicted, affect the utility of using optimization techniques as part of a sensitivity analysis? Analysis of the results showed that the exact function value did not influence the routine's selection of parameter values; only the relative function value between simulations was used. The solutions from both functions converged on minima for which the five parameter values were very close. It is accepted, however, that this may not be the case if a radically different function were applied.

7.3.4 Conclusions

The optimization results have shown that the relationship between the sensitivity to the five physically-based techniques and the modules selected is an extremely complex one, dependent on the depth and

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proportion of the flow that is contained in the channel and floodplain segments. The results from the one storm and reach do, however, suggest several conclusions:

- 1) An optimization scheme may well provide a viable alternative to traditional factor perturbation sensitivity analysis, provided that:-
 - i) the interpretative approaches required to analyse the results are further developed
 - ii) the CPU demands can be met
 - iii) a satisfactory function can be found to describe the fit of the hydrograph
- 2) The sensitivity of MILHY3 in two-stage applications is dominated by:
 - i) slope when slopes are $> 1 \times 10^{-2}$
 - ii) floodplain Manning's when slopes are $> 1 \times 10^{-4}$
- 3) The short-circuiting of floodplain flows is only significant if the floodplain slopes are $< 1 \times 10^{-3}$ and the floodplain routing length is at least 10% shorter than the main channel routing length.
- 4) The momentum exchange and multiple routing routines generate similar relative errors in the predicted hydrograph as variation in the Manning's coefficient.
- 5) The sensitivity of the hydrograph to variation in the Manning's n coefficient is reduced if the momentum exchange and multiple routing routines are utilised.

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7.4 Summary

This chapter has explored the sensitivity of MILHY3 to variability in certain physical parameters and to variability in the process modules used to construct the MILHY3 scheme. In particular, the interaction between the momentum exchange and multiple routing routines has been explored, and the relative impact of these modules in comparison to the introduction of the infiltration algorithm has been assessed. The effects of scale and resolution of the precipitation distribution and catchment subdivision has also been investigated.

The sensitivity of MILHY3 has been investigated using two techniques; a traditional factor perturbation analysis, reported in section 7.2, investigated the effects of variability in the physical parameters, and a new approach utilizing optimization techniques, reported in section 7.3, investigates the composite structure variability. The conclusion of these investigations are found in subsections 7.2.6 and 7.2.4. The main conclusions from these two analysis are:

- 1) For slopes less than 1×10^{-2} and greater than 1×10^{-4} , the outflow hydrograph is most sensitive to variability in the Manning's n coefficient. However, this sensitivity is reduced by the joint application of the momentum exchange and multiple routing routines. In addition, the joint application of these two routines simplifies the selection of the Manning's coefficient by removing the effects of turbulent exchange and sinuosity, thus the coefficient now only represents the effects of boundary friction.
- 2) Variation in the predicted hydrograph generated from variation in the momentum exchange and multiple routing routines has been shown to be of a similar magnitude to

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the variation generated by variability in the Manning's n coefficient. This suggests that the inclusion of the effects of momentum exchange and multiple routing pathways is as significant as the effects of boundary friction.

3) The momentum exchange routines do not have a significant effect on the outflow hydrograph when applied without the multiple routing routine. The influence of the joint application of the multiple routing and momentum exchange routines is on the attenuation of the outflow hydrograph. The exact nature of the influence of the routines on the attenuation of the floodwave is determined by the proportion of the discharge carried on the floodplain and in the main channel flow segments.

4) Comparison of the relative importance of the infiltration algorithm, momentum exchange, and multiple routing routines in the River Fulda catchment, utilizing large subcatchments, has shown that the infiltration algorithm generates the largest improvement in the prediction of the outflow hydrograph. However, for simple storm events the user may consider that the improved performance generated by the inclusion of the infiltration algorithm does not justify the additional computational demands.

5) The application of the optimization technique has been shown to be a viable alternative to traditional factor perturbation sensitivity analysis. There is a need, however, for the interpretative techniques required to interrogate the results, to be further developed.

Chapter 8

Validation of MILHY3III - Hydraulic Validation of MILHY3

The objective of this chapter is to investigate the utility of applying state-of-the-art models in order to validate developments made in simpler models such as MILHY3. The need for such an investigation arises from the difficulties of establishing a field data set large enough to allow a meaningful evaluation of MILHY3's performance, and for a set of operational guidelines for MILHY3 to be developed.

The main limitation of the evaluation of MILHY3 using the River Fulda data set is the limited number of observed out-of-bank events. This is a common problem as extreme events are not only difficult to measure but also occur infrequently. The collection of further data sets from other catchments that meet the other requirements of the model evaluation programme would not necessarily negate this problem of a limited number of events.

The objective of this chapter is therefore to utilise a state-of-the-art model to extend the number of out-of-bank events for the River Fulda catchment. Ideally, such a scheme would operate by taking certain "design" rainfall events and developing the outflow hydrographs at each of the eight gauging stations. These hydrographs could then be treated as the "observed" or ground-truth against which the performance of MILHY3 could be judged.

The only feasible alternative to using a state-of-the-art model to extend the record of extreme events would be to utilise the flood frequency data. Pilgrim and Doran (1987) noted that:

"The extent to which a flood frequency analysis can be extrapolated before an alternative rainfall-based method becomes preferable depends, among other factors, on the relative accuracy of the latter method."

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The "other factors" include the characteristics of the available sample of recorded flows for the development of the flood frequency analysis. In catchments with limited data, regression of flood parameters may be used, or parameters transferred from other catchments, or regional rules developed. In reality, in order to generate a complete hydrograph from the flood frequency analysis, rather than just the usual peak discharge, the frequency data are used to calibrate a hydrograph produced by either a unit hydrograph procedure or a runoff routing method.

As noted earlier, the collection of data for extreme events is difficult for nearly all catchments, and therefore the derivation of empirical coefficients for a flood frequency analysis would not easily be achieved. The determination of the shape of the hydrograph in flood frequency analysis is also unsatisfactory for the purpose of evaluating the performance of MILHY3. In MILHY3 a unit hydrograph procedure is already utilised for the development of the runoff hydrograph, which is then routed downstream using the modules developed in this report. To extend the record of storm events utilising a similar technique to that used in MILHY3 would not provide any indication of the predictive accuracy of MILHY3. In order to extend the record of extreme events, the level of confidence in the predictive accuracy of the techniques used needs to be high.

The alternative available for the derivation of the hydrograph shape in flood frequency analysis is to utilise runoff routing. Runoff routing gives a more realistic account of the stores in the catchment than the unit hydrograph approach. However, both the unit hydrograph and the runoff routine technique generate the runoff hydrograph for a subcatchment. No account is made of the routing of these flows downstream. Application of a runoff routing routine would therefore only allow validation of the runoff generation modules in MILHY3.

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Analysis of the flood frequency analysis approach has shown that in order to extend the record of storm events and to generate a complete hydrograph, there is no alternative but to employ another model. Traditionally, flood frequency analysis has employed relatively simple hydrologic techniques, namely the unit hydrograph or runoff routing. The level of resolution in the process areas of these models is similar to or lower than the modules that make up MILHY3. To extend the record of extreme events using the traditional flood frequency analysis and then evaluate MILHY3 using this record, would thus be comparing like with like. Although this may be a useful exercise, and would help determine the efficiency of the MILHY3 model, it is not the objective of this evaluation programme. A different modelling approach to MILHY3 is required for the evaluation programme, a model to which the level of confidence in the predictive accuracy of the hydrograph is high.

A series of prerequisites for a model that would meet the demands and limitations specified for the extension of the record of extreme events, can now be drawn up. Such a list would consist of:-

- 1) the model's strategy in conceptualizing the physical processes needs to be different to MILHY3's conceptual strategy
- 2) the model needs to be evaluated against field data and have an acceptable predictive accuracy
- 3) the data set demands of the model, including any calibration requirement, must be met by the data set for the River Fulda catchment
- 4) the model should be well-documented and readily available

The conceptual strategy of MILHY3 is basically semi-lumped. Variations within the catchment are considered on the subcatchment level. An alternative strategy would therefore be distributed to

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some degree; this would include catchment models such as the SHE and VSAS2. However, review of these distributed physically-based models shows that such models have a doubtful level of predictive accuracy and require large data sets. In addition, such models tend not to incorporate the cross-sectional and plan geometrical effects of two-stage flows. Using a distributed catchment model, therefore, to extend the record of extreme events would seem to be inappropriate.

An alternative to using catchment models would be to consider using models that simulate certain processes in the catchment. For example, the development of the runoff hydrograph in MILHY3, and the associated problems in specification of the subcatchment size, could be investigated using a data set developed from a model incorporating the routing of the overland excess and throughflow. More relevant to the main thrust of this report would be the evaluation of the two-stage flow capability using a data set developed from a model specifically for channel and floodplain flows.

With the advent of computer technology, models of river channels have been developed rapidly and are widely used, (Cunge, et al., 1980). All of these models can be classified as being hydraulic in approach, in contrast to the simpler hydrologic approaches of models such as MILHY3. A hydraulic approach to modelling involves solving the equations of the conservation of momentum in addition to the conservation of mass. In a hydrologic approach only the conservation of mass is solved and a simple storage equation replaces the conservation of momentum.

In the next section hydraulic approaches to two-stage flows are investigated, with the aim of identifying a model that could be used to extend the record of extreme events in order to complete the operational validation of MILHY3.

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8.1 Identification of a Hydraulic Model of Two-stage Flow

8.1.1 Hydraulic modelling alternatives

Hydraulic approaches to the modelling of channel flows have traditionally been divided into classes depending on the number of spatial dimensions the model incorporates. In one-dimensional models the flow is averaged across a section perpendicular to the main direction of flow, and the St. Venant equations of open channel flow are used. One-dimensional models are widely used, especially in long reaches. However, they do not incorporate the exchanges that take place in a two-stage channel. Two-dimensional models divide the floodplain into a number of cells. The exchange of flow between adjacent cells is computed using the difference in the water elevation between the two cells and the condition of the common boundary. Three-dimensional approaches, where the velocity depth profiles and the river bed boundary are allowed to vary, are not usually utilised in geomorphological investigations in temperate regions, as the time-scales of bed elevation changes in such climatic regions are large. The complexity of three-dimensional modelling approaches means that the number of applications has been limited, and often to single channel systems, see for example Tominga *et al.* (1989), despite the potential of such schemes.

The prerequisites in the selection of an appropriate model, for the extension of the number of extreme events, require that the model has been evaluated; this excludes three-dimensional models which are in the early stages of development and may currently only be considered to be research tools. Evaluated models that are well documented adopt one or two-dimensional approaches. The resolution of one-dimensional models is usually greater than that of MILHY3 in the handling of the routing component of the downstream conveyance. However, the handling of the complex geometry of a two-stage channel system is usually of a lower resolution than the new modules of MILHY3. This leaves a two-dimensional approach as being the most

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appropriate for the objectives of this chapter.

8.1.2 Two-dimensional models of two-stage flow

In contrast to one-dimensional models where the St. Venant solution to the equations of flow are widely accepted, there is no generally accepted single solution to the equations of flow in two-dimensions. Solutions vary widely in complexity, from simple steady state solutions to complex forms that include the energy dissipation and plan geometry variations with stage.

Samuels (1985) classifies these models into two groups, based on the type of solution to the equations of flow. These are:

- i) cell type - flow is computed from cell to cell on the floodplain according to certain laws
- ii) differential type equations - flow is described by a set of coupled partial differential equations derived from physical parameters

Cell type models are based on grids determined from the topography of the river reach, usually following the outline of the main channel and extremity of the floodplain, for example Lesleighter (1983). Lesleighter's model, shown conceptually in Figure 8.1, neglects the effect of inertia and the convection terms in the solution of the conservation of momentum, and uses a Manning type relationship to compute the flow between adjacent cells. These models work well when the stresses between the computational cells are greater than the boundary stresses on the bed on the floodplain and main channel; such conditions could occur where slopes or flow velocities are small. Generally, flow is controlled in cell type models by equations such as:

$$Q_{ij} = K_1(h_i - h_j)^{0.5} \quad 8.1$$

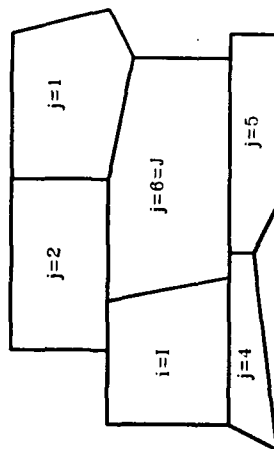
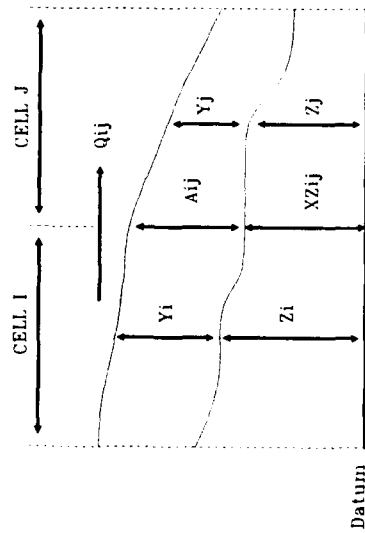


Figure 8.1
A Two-Dimensional Cell Type Model
 (after Lesleighter, 1983)

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where i and j consecutive cells
 K_l conveyance function at link l between
 cells i and j
 H water surface elevation
 Q_{ij} discharge flux between cells i and j

The limitation of cell type models is that the conveyance function K is dependent on the local direction of flow, so that K_l may change if the floodplain inundation changes dramatically. Cell type models, therefore, work best in fairly stable inundation conditions.

Differential type solutions to two-dimensional modelling utilize one of three methods. These are:

- i) the method of characteristics
- ii) finite difference methods
- iii) finite element methods

In the method of characteristics, the partial differential equations are transformed into ordinary differential equations. An example was developed by Schmitz et al. (1983), who utilised a rectangular grid capable of greater resolution in required areas. Schmitz et al. (1983) acknowledge, however, that computer requirements of their scheme are large, and that an approximation of the streamlines required prior to computation may be a limitation.

Finite difference schemes utilise regular grid systems to solve the equations of flow, although the most complex schemes now allow for the curving or stretching of cell boundaries to improve the physical representation of the channel and floodplain, see for example Banks and Falconer (1989). Vreugdenhil and Wijnbenga (1982) compared the results of a finite difference scheme using a 30m grid resolution, with the results from a flume-based experiment. The predictive

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accuracy of the finite difference model compared favourably with the results from the flume experiments. Milityeev and Shkolnikov (1981) and Zielke and Urban (1981) agree, however, that the main problem with finite difference schemes is the resolution of the cell size in order to represent the complex plan geometry of the channel and floodplain. Reducing the size of the computational cell so that the channel and floodplain flow segments are resolved separately, increases the overall number of cells required to describe the reach and thereby increases the computation time.

Finite element models utilize a flexible computational grid that more accurately portrays the geometry of the two-stage channel. A variety of element shapes have been used to build up the computational grid, including triangles, rectangles and quadrilaterals, examples of which include Su et al. (1980), and Zielke and Urban (1981). The finite element method uses the computational grid elements to approximate the solution of equations of flow using some type of continuous function. The computational difficulties of finite element models are generally greater than their finite difference counterparts because of approximation techniques required to solve the continuous function.

All three methods of differential type two-dimensional analysis involve complex mathematical analysis. The relative advantages offered by each technique depend upon the degree of complexity in the geometry of the application. In the case of two-stage models a method that offers a flexible grid element system would be most advantageous. This excludes the method of characteristics and the finite-difference approach and leaves the finite element approach.

Comparison of the cell type model and finite element model, which both offer a flexible grid network, shows that the finite-element systems have been more widely developed and evaluated than their cell type counterparts. In the next section, two-

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dimensional finite element models suitable for two-stage flow and for the extension of the record of extreme events are, therefore, investigated.

8.1.3 Two-dimensional finite-element models

Two-dimensional finite-element models have been developed over the past fifteen years, primarily for the simulation of flows in estuaries, or for complex flow environments around bridges or weirs. Examples of models developed for these environments include Tseng (1975) and King and Norton (1978), who simulate flows around a bridge, and Herrling (1978) and Holtz and Nitsche (1980), who simulate estuarine flow conditions.

More recently these models have been adapted for flow in two-stage channels. Zielke and Urban (1981) adapted the Holtz and Nitsche (1980) estuary model for flow in the floodplain system, and Su *et al.* (1980) developed a model simulating the flows at the confluence of two channels.

In all of these applications finite-element models have been utilised as alternatives to the establishment of flume-based experiments, and the scale of interest has been small. An exception to this has been a study of McAnnally *et al.* (1984a, 1984b), who undertook a large scale (tens of miles) simulation of an estuary and utilised a hybrid approach using both finite-element and flume-based simulations.

The limitation of finite-element models in the context of this investigation is, therefore, that all previous river reach applications have been on a small scale, whilst the river reach lengths in the River Fulda catchment range from 9 to 36 km. It is not clear whether finite element models would provide the level of predictive accuracy required at this scale to provide the ground-truth hydrographs for the evaluation of MILHY3. Specifically, it is

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not known if the size of the element in the computational grid may be increased sufficiently to maintain a manageable computational problem, or if the stability of the solution would be maintained. If the water elevation drop were too great from one element to the next, the simulation may, for example, become unstable. In addition, would the resolution of data available for a typical reach in the River Fulda catchment be great enough?

An investigation into the utility of finite-element methods as part of the MILHY3 evaluation programme has therefore raised further general conceptual questions. These are:

- 1) can a finite-element model suitable for two-stage flow, that has been evaluated and documented on small scale applications, be identified?
- 2) could this model be applied to larger reach lengths without increasing the number of computational cells beyond a manageable level and without requiring additional data?
- 3) what would the predictive accuracy of such a large-scale application of a finite-element model be?

If a finite-element scheme could be successfully applied to extend the record of an extreme event and meet all the criteria above, could this not be incorporated as a high resolution module in the MILHY3 scheme? Although this was not the objective of this investigation, the issue of the development of a composite modelling strategy and the inclusion of hydraulic techniques in hydrologic modelling, have been identified as some of the most pertinent issues in flood forecasting. The feasibility of including a finite-element module in MILHY3 should therefore be considered.

The first task, however, is to identify a finite-element model that incorporates two-stage flows, has been evaluated, and is

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well-documented. The number of finite-element models that meet all these criteria is small, as many models are still in the developmental stage. Two schemes have been evaluated here: RMA-2V developed for the US Corp of Engineers (King and Norton, 1978), and FLOUT developed at Hydraulics Research, Wallingford, (Samuels, 1983 a and b). Both of these packages are commercially available and neither seems to offer any conceptual advantage over the other.

The RMA-2V package was selected as the backup and support was offered by the US Corps of Engineers. All simulations of the RMA-2V model were undertaken at the Hydrologic Engineering Centre, Davis, California, on a Harris 100 super microcomputer with the help and cooperation of Dr D.M. Gee. It should be noted that the simulations undertaken using RMA-V2 in this chapter are the first applications of any hydrodynamic finite-element model at a large scale in a floodplain environment. These simulations are also the first applications of RMA-2V using only the limited data available in ungauged catchments. In addition, RMA-2V has not been previously used to develop ground-truth hydrographs for the evaluation of other models. In the next section, the RMA-2V model is briefly described.

8.2 RMA-2V A Two-Dimensional Finite-Element Model

RMA-2V is a finite element model for the solution of the two-dimensional depth averaged shallow water flow equation. It can simulate both steady and unsteady flow conditions by using the Reynolds form of the Navier-Stokes equations of flow. Velocity is used as the flow variable and it is computed at every node in each element of the finite-element network. Three energy transfer computations are included in the Reynolds solution. These are:

- 1) the effects of boundary friction
- 2) the effects of surface wind friction
- 3) the effects of momentum exchange

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The finite-element network enables boundary roughness and topographic variations to be realistically simulated. The computational cell network is usually constructed based on series of lines running the length of the reach parallel to the contours. The resolution of the cells is determined by the operator, so that a greater number of cells can be utilized to give more detail in areas required.

The RMA-2V package, written in FORTRAN 77, was originally developed by Norton and King (Norton *et al.* 1973), under contract to the US Corps of Engineers. The model has since undergone many further developments and the version now being operated is version 4.1. This version allows the incorporation of one- and two-dimensional cells, thereby reducing the computational demands of the model. RMA-2V is also incorporated into the TABS-2 package (Thomas and McAnnally, 1984), which also offers a sediment budget routine and a water quality package.

RMA-2V consists of network generation and checking module (RMA1), the hydrodynamic package (RMA2), a contour plotting package (CONTUR), and a velocity vector package (VECTOR). A geometry file is generated by the RMA1 module, which once verified by the operator passes into the hydrodynamic package. The CONTUR and VECTOR packages allow the interpretation of solution files from the RMA1 and RMA2 modules. The CONTUR package draws the network and the topographic features incorporated in the cell network allowing the operator to check the representation of the topography in the reach. The VECTOR package allows the operator to plot the velocity vectors over the whole reach at any computational time increment.

8.2.1 Establishing the mesh: RMA1

The geometrical network is specified by the operator and consists of a series of quadrilateral and triangular cells that represent the river reach. Each of the cells and corner nodes of

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the cells must be numbered and a connection table developed that describes the number of the nodes in each cell, starting at any node but always in an anti-clockwise direction. In addition, the x, y and z coordinates of each of the nodes must be specified. The mid-node coordinates are added automatically by the RMA1 package interpolating linearly between the coordinates of the corner nodes.

A boundary roughness value must be specified for each of the cells but to reduce the number of data that must be entered into RMA1 the cells are classified into a number of groups, each of which has the same boundary roughness. The number of boundary roughness groups allowed is unlimited but usually no more than 5 are required. If one- and two-dimension solutions are to be utilised, it is important to note in the application of RMA-2V to two-stage channels that the boundary roughness values in the one-dimensional data set will be different from the two-dimensional data set. This is because in the one-dimensional data set, the composite roughness of both the main channel and floodplain flow segments must be included.

In addition to the boundary roughness values that must be specified for each cell, an eddy viscosity coefficient must also be specified. The eddy viscosity coefficient incorporates the effects of momentum exchange generated by the velocity gradient across the section. This momentum exchange is approximated by multiplying the velocity in each direction by the eddy coefficient in that direction. For the most accurate results, therefore, the eddy coefficient should be entered for each side of the element and the direction of the element to the dominant direction of flow also noted. The RMA2 code can then distinguish between eddy coefficient values parallel to the main flow direction and perpendicular to the main direction of flow.

It can be difficult to establish a value of the eddy coefficient, which has dimensions of lb-sec/ft^2 , for a particular element side. It is important to remember that the value of the

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coefficient depends on the momentum of the fluid and the distance over which the momentum acts, divided by the velocity of the flow and the surface area of the element. This means that as an element's size increases the eddy coefficient increases, or when the velocity of the flow increases the coefficient increases.

Up to ten eddy exchange coefficient values may be specified in the RMA1 data set, and the classification of each element side in one of these ten groups should be specified. Element sides with short lengths and parallel to the dominant flow direction should have the lowest coefficient values, whilst long element sides perpendicular to the dominant flow direction should have higher coefficient values.

Vreugdenhil (1973) suggests the following relationship for the approximation of the eddy coefficient value but notes that the relationship only gives coefficient values to the nearest order of magnitude. Vreugdenhil's relationship is:

$$E = - \frac{6h}{C} gV^2 \quad 8.2$$

where h - water depth
 C - Chezy coefficient
 V - velocity

If too small values of the eddy exchange coefficient are selected by the operator, then the solution of RMA-2V may become unstable. This is because small values of the coefficient allow the direction of the velocity vectors to vary widely. It is often necessary, therefore, to increase the values of the coefficient during the initialisation of the scheme.

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The maximum of nodes allowed in the package is 2100, whilst the maximum number of cells is 900. These limits can be easily altered in the source code of RMA-2V, although this consequently increases the size of the buffers required for the operation of the package.

At the upstream and downstream extremities of the reach, boundary conditions must be specified. Usually a stage and discharge condition are entered at the upstream boundary and a rating curve is specified at the downstream boundary. The rating curve takes the form of a single power function typically:

$$Q = A_1 + A_2(\text{ELEV} - \text{EO})^C \quad 8.3$$

where A_1 , A_2 and C are coefficients.

8.2.2 Establishing the criteria for wetting and drying of elements

One of the main features of RMA-2V is that as the inundation stage rises and falls, elements of the mesh enter and leave the solution. This is known as the wet/dry capability. This is important not only in improving the efficiency of the solution, but also in increasing the physical representation of the modelling of the floodplain inundation. As the inundation levels fall, the cells with the highest elevation leave the solution, exposing knolls in the floodplain. The velocity vector plots show this distribution and show how the vectors converge around such knolls. This level of topographic resolution would not be possible with a hydrologic approach.

The wet/dry capability of RMA-2V has recently been upgraded in the latest version, version 4.1. In previous versions, cells left

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the solution when the highest node in a cell fell below the stage elevation, plus a specified drying criterion. Cells then re-entered the solution when the highest node was below the stage elevation, plus a specified wetting criterion. These wetting and drying criteria are specified by the operator but the wetting criterion must always be of a greater depth than the drying criterion. This introduces a hysteric effect, shown in Figure 8.2, where the stage elevation must be lower to cause a cell to go dry than the elevation required for a cell to be inundated. However, the error involved in this inaccuracy, especially if the difference between the specified wet and dry criteria was small, are not significant. The reason for keeping as many cells as possible in the solution is that it improves the continuity of the computation and hence improves the stability of the solution.

In version 4.1 of RMA-2V, the hysteric wet/dry behaviour has been replaced by a pseudo-porosity or marsh element routine, (King and Roig, 1988). Developed to improve the predictive accuracy of RMA-2V in marsh environments, the stability of the solution is improved as flow through cells is gradually increased and decreased. When cells drop out of the solution, the discharge flowing through the cell is very small and, therefore, the impact on the overall solution is small. The introduction of marsh elements into RMA-2V has not previously been validated. It is one of the objectives of this investigation, therefore, to compare the results from simulations utilising the wet/dry criteria and the marsh elements.

The behaviour of these marsh elements has been likened to the effects of macroporosity. An effective depth parameter, h_o , has been introduced which is a function of the porosity of the cell. The effective porosity is defined such that:

$$h_o = \int_0^h K \cdot dh \quad 8.4$$

where h_o - effective depth
 K - porosity
 h - depth

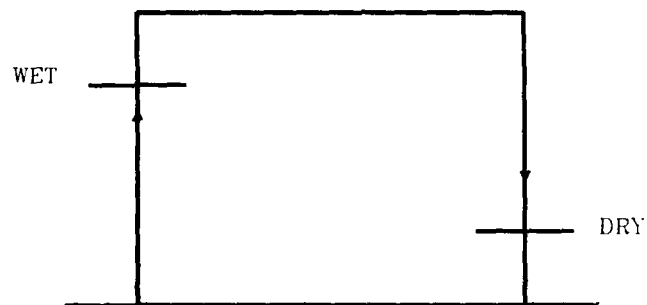


Figure 8.2
Hysteretic Effect of the Wet/Dry Capability
of RMA-2V, Version 3

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When an element is fully inundated all nodes are inundated at a stage greater than the effective depth. As the stage falls, the porosity weights the stage, generating the effective depth, thus reducing the flow through the cell. The porosity varies at present on a linear scale over the range of stage elevations, and this is shown in Figure 8.3a. An S-shaped relationship would be physically more realistic and such a relationship should be incorporated in the next version of RMA-2V.

Figure 8.3 compares the wet/dry criteria with the marsh element method of computation. The wet/dry criteria are illustrated on the left-hand side of the diagram, whilst the marsh elements are shown on the righthand side. The figure shows that as the inundation depth falls, the cell is removed from the solution as soon as one node falls below the drying criterion. The bottom figure on the left-hand side shows the relationship this infers between the conceptual porosity and stage elevation.

On the right-hand side of Figure 8.3, the diagram illustrates how the porosity of the cell varies across the cell with the variation of the depth of inundation. Segment 'a' has the lowest depth of inundation and thus has the lowest porosity. This porosity then weights the stage to give the effective depth which is considerably lower than the actual depth; this reduces the discharge prediction from segment 'a'. By contrast, in segment 'c', the depth of inundation is greater, the consequent porosity is larger and, therefore, the velocity prediction is reduced only slightly.

When the depth of inundation is greater than 1m the porosity of the cell is set at 1.0, so that the effective depth is the same as the actual depth. The 1m rule has been set as an appropriate value for floodplains, although it can be altered with reasonable ease.

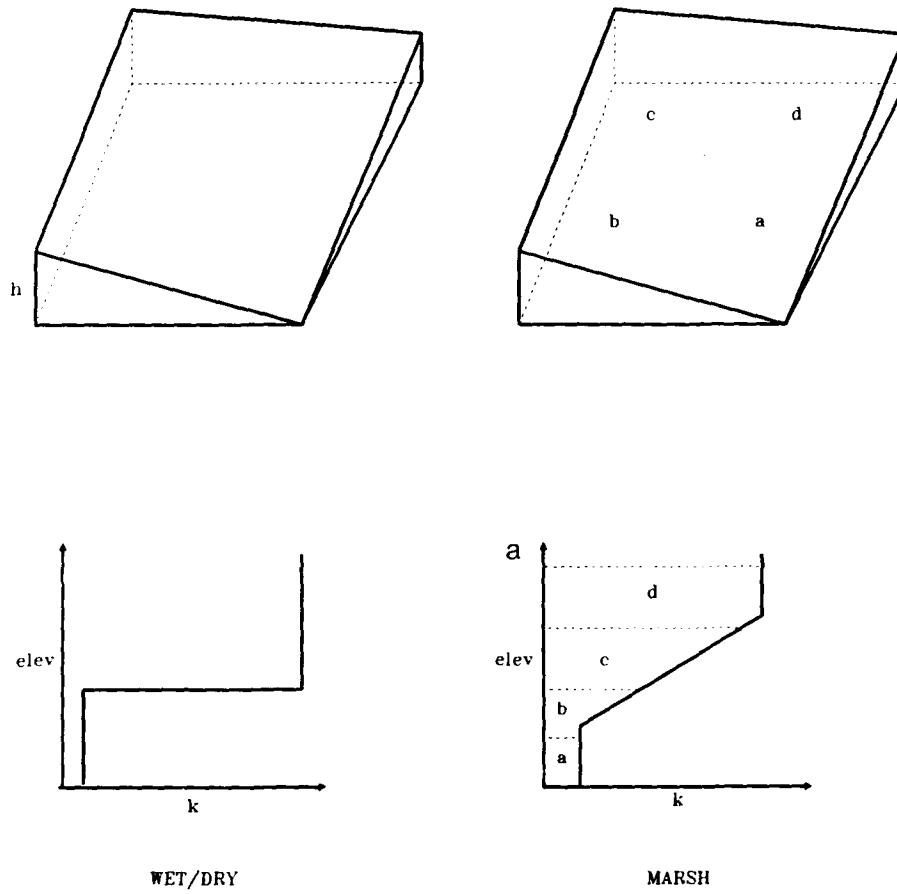


Figure 8.3
Comparison of the Wet/Dry Capability of Version 3 and Marsh
Element Capability of Version 4 of RMA-2V

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8.3 Application of RMA-2V

The objectives of undertaking an application of the RMA-2V model are to explore several areas.

- 1) To establish whether a two-dimensional model of two-stage flow can be applied to large reach lengths, that is reach lengths of approximately 20km, and whether the simulation produces meaningful hydrograph solutions. Further, is the resolution of data available for the River Fulda catchment sufficiently great for such an application?
- 2) To establish whether the predictive accuracy of RMA-2V is acceptable in comparison with the field data available, and to determine if the model can be used to extend the record of extreme events.
- 3) To investigate the impact of the introduction of the marsh elements in RMA-2V by comparing the results from simulations utilising versions 3 and 4.
- 4) To examine whether RMA-2V could be incorporated as a module in the composite modelling structure of MILHY3.

To undertake these objectives, a reach of the River Fulda between the gauging stations at Bad Hersfeld and Rotenburg was selected. This reach has been used in the analysis reported in Chapters 5 and 7, and was selected for this investigation as the inundation of the floodplain is extensive and defined on the available flood inundation maps.

The reach between Bad Hersfeld and Rotenburg is 24km (15 miles) in length, with a slope of 0.0008, that is a drop of 15m over the reach length. The floodplain is typically 1km wide and has a very shallow orthogonal slope to the main channel of around 0.0001.

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A typical scene along this reach is given in Figure 6.6, which shows that the floodplains are bounded by steep hills which are often forested.

Field estimates of the Manning's n coefficients of the floodplain and channel along the reach length can be derived from the photographs taken during visits to the River Fulda catchment, using the tabulated values given in Chow (1959). Roughness values were assessed as being 0.045 on the floodplain and 0.035 in the main channel. However, in a few sections of the reach, the floodplain roughness was considerably higher than 0.045, and for these sections the Manning's n coefficient was estimated as being 0.07.

The cross-sectional geometry and rating curves are available for the gauging stations at Bad Hersfeld and Rotenburg. At Bad Hersfeld the channel is approximately 4m deep and 30m wide, whilst at Rotenburg the channel is 5.5m deep and 50m wide.

8.3.1 System schematization

RMA-2V utilises a finite element network composed of both triangular and quadrilateral elements. Ground elevations are defined at the corners of the elements and assumed to vary linearly between corner nodes. In this investigation, the channel is represented by a strip, two elements wide (Figure 8.4), producing a triangular cross-section. Overbank areas were represented by much larger elements. The triangular elements were most frequently used in the description of the floodplain on the inner bank of the major meanders in the main channel. The lateral extent of the network was determined by a bluff line, beyond which none of the simulated flood event would extend.

Two boundary roughness classes have been specified, one for the channel cell elements and one for the floodplain elements. The resulting finite element network is composed of 860 elements and



Figure 8.4
Elemental mesh for application of RMA-2V to the
River Fulda, West Germany

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2660 nodes. The ratio of the maximum element size to the smallest element size is approximately 200 to 1.

The rating curve relationship at the downstream station, Rotenburg, was developed using the single power relationship specified above. The exact form of the best-fit relationship is:

$$Q = 0.012H^{4.85} \quad 8.5$$

This relationship was developed using the flood frequency data for out-of-bank events. Figure 8.5 illustrates this single-power relationship and compares the curve with the data from the flood frequency analysis. The figure shows the inadequacy of the single power function to describe the rating relationship, even though only the out-of-bank events have been used. A trial using in-bank and out-of-bank events showed that the single power relationship could not satisfactorily describe the rating relationship.

8.3.2 Storm events

As this investigation is the first application of RMA-2V at this scale, two storm events were selected. These are the 1 in 10 year event (Storm 1), and the 1 in 100 year event (Storm 2), both used in the other parts of the evaluation program of MILHY3 reported in Chapters, 4, 5 and 7.

The objectives of this investigation have been identified above. The primary objective is to investigate the utility of RMA-2V for extending the record of extreme events, and thus validating MILHY3 by investigating the stability and accuracy of the two model versions. The secondary objective is to investigate whether it would be feasible to incorporate RMA-2V as another module in the composite structure of MILHY3.

These two objectives require different simulations of Storm 1

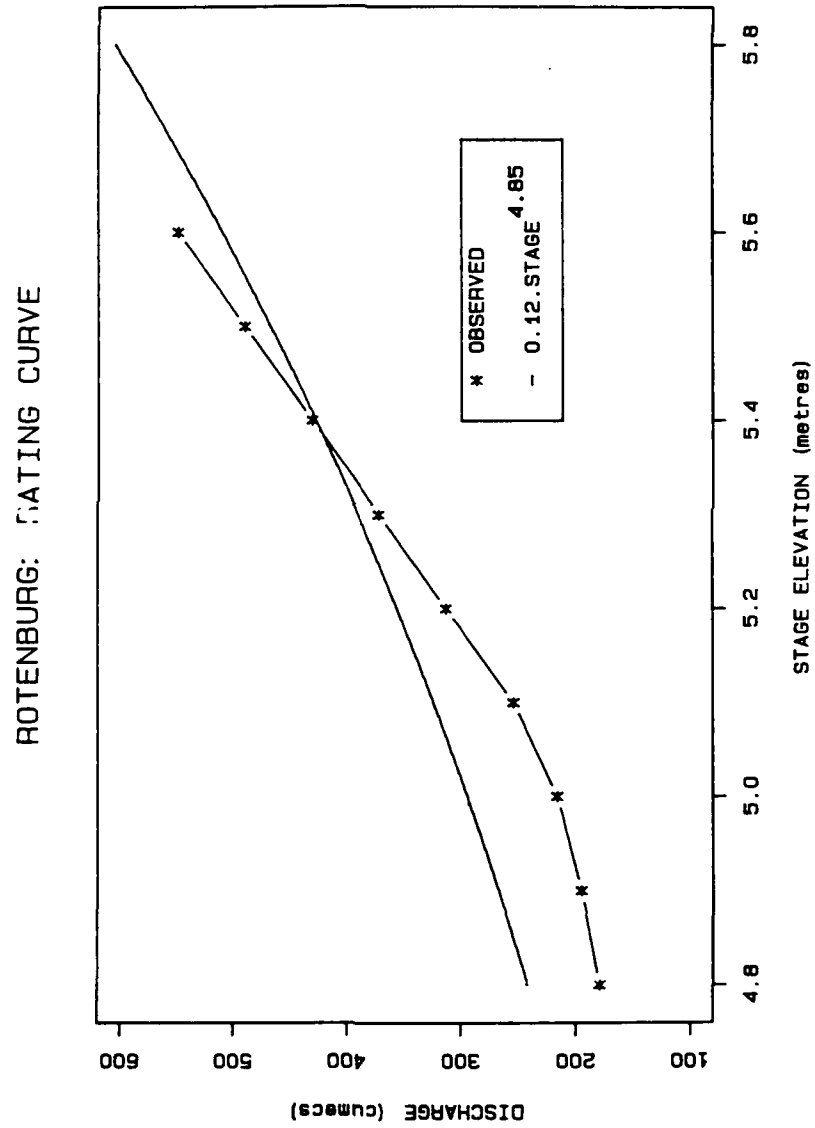


Figure 8.5
Comparison of observed and generated rating curve relationships
 for Rotenburg

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using RMA-2V. In the first objective, the first aim is to show that RMA-2V is capable of simulating storm events at this scale of reach, and that the predictive accuracy in comparison with observed storm events is acceptable. This requires, therefore, the comparison of a simulated hydrograph with an observed hydrograph at the downstream station, using the observed upstream hydrograph as the input.

The secondary objective (to investigate the incorporation of RMA-2V as part of the MILHY3 scheme) requires a comparison of the relative performances of the MILHY3 downstream conveyance models with RMA-2V. This comparison can be partly achieved using the simulation type identified in the primary objective, but also requires the simulation of the storm events from Bad Hersfeld to Rotenburg using RMA-2V, using the output from the MILHY3 simulation to Bad Hersfeld as the input. This will enable the relative advantage of RMA-2V in a catchment simulation to be judged.

These two investigations are reported in section 8.4 and 8.5.

8.3.3 Initialisation of the River Fulda simulations

As the application of RMA-2V to a reach on the River Fulda was to be the first application of the model at this scale, a series of trials were undertaken. The purpose of these trials was to identify any conceptual or operational difficulties, and to initialise the scheme ready for the simulation of the storm events. The first task in the initiation of the simulations is to generate the baseflow conditions in the reach. This is achieved by undertaking a drawdown simulation where the reach is inundated with water to such a depth that there is no water slope from the upstream to downstream extremities of the reach, rather like a reservoir. This water is then allowed to flow out of the reach until steady conditions are reached.

In addition to the generation of the baseflow conditions,

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this initialisation allows the grid network to be checked for gaps in the elements. With approximately 15,000 three and four digit numbers to be entered into the computer to generate the River Fulda mesh, the potential for missing a cell and creating a hole through which the water could drain was great. With logical cell and node numbering, such gaps are relatively easily found and the data set corrected.

More of a problem was the generation of the baseflow conditions by the lowering of the downstream elevation of the 'reservoir dam'. As the length of the reach is much larger than any previous application, the consequent overall drop in the channel bed elevation, (some 15m), is also large. To force the generation of baseflow conditions, the incremental stage drop in the drawdown test had to be reduced to 0.2m steps. The generation of baseflow conditions, therefore, took much longer than anticipated. It is also important to note that once the baseflow conditions have been generated, then no alterations to the specifications of the grid network must be made.

The initialisation also showed that the time increment of the inflow hydrograph need only be 0.5 hours. Trials with a 0.25hr increment did not improve the accuracy of the outflow hydrograph and, therefore, it was not utilised. Throughout the analysis, to reduce the computational requirements of the simulations, only the rise of the hydrograph and the first part of the decline of the hydrograph were simulated. This accounted for just over 20 hours of the hydrograph and 40 computational steps. As the peak of the hydrograph has been the primary area of interest in this report, and because of the exploratory nature of these applications, the limitations of simulating only the first 20 hours of the hydrograph were accepted.

The computational requirements for simulating some 40 time increments in the relatively large grid network for unsteady flow

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conditions were considerable. Each simulation took several hours on the Harris 100 super microcomputer. It was important, therefore, to select the simulations required with care.

8.4 RMA-2V Simulations Using Observed Inflow Hydrographs

The first aim of this investigation is to establish the stability of the RMA-2V solutions for a variety of initial conditions. Having established the grid network of the reach, which is determined by the geometry of the reach, and having established the baseflow conditions, three variables can be identified as controlling the behaviour of the reach under different flow conditions. These three variables are:

- 1) boundary roughness - specified by Manning's n coefficients
- 2) eddy coefficient - which incorporates the effects of turbulent exchange
- 3) wet/dry criteria - version 3 only

The boundary roughness and wet/dry criteria are investigated in this analysis. However, the sensitivity of the solution to variation in the eddy exchange coefficient has not been incorporated. A single eddy exchange coefficient of 50 which has been utilised for all the element sides, has been used in all the simulations. It is accepted that it is very important to investigate the effects of variation in the eddy coefficient exchange. However, the time available for this investigation was limited. The value of 50 is the adjusted figure after the stability of the solution has been ensured. The investigation of the effects of variation in the eddy exchange coefficient has been identified,

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therefore, as a priority area for any further work on the utility of RMA-2V in large-scale applications.

In the next two sub-sections, the effects of variation in the two remaining variables, that is the boundary friction variable and the handling of the wet/dry criteria, are investigated.

8.4.1 The effects of variability in boundary friction

As the turbulent exchange of momentum has been incorporated in the two-dimensional solutions of RMA-2V, it is recommended in King and Norton (1978) that the Manning's n coefficients utilized to incorporate the effects of boundary friction should be lower than the n values recommended in texts such as Chow (1959). To investigate this recommendation in the context of a long reach application, and to establish the sensitivity of the solutions to the friction variables, the first set of simulations of RMA-2V used a variety of the friction values.

Figure 8.6 illustrates the simulation of Storm 1, the 1 in 10 year event, using version 4 of RMA-2V that incorporates the marsh elements. The figure illustrates the effect of increasing the Manning's n coefficient of the floodplain elements from 0.045 to 0.07, and compares these two simulated outflow stage hydrographs with the observed hydrograph at the downstream station, Rotenburg. All other variables are kept constant, the Manning's n coefficient of the main channel being 0.035 and the eddy exchange coefficient being 50.

Figure 8.6 shows that the adoption of the Manning's n coefficient of 0.045 generates a downstream hydrograph that arrives too soon in comparison with the observed hydrograph. This suggests that the reach is transmitting the hydrograph too efficiently. Increasing the Manning's n coefficient to 0.07 reduced the efficiency of the reach and the match between the observed and

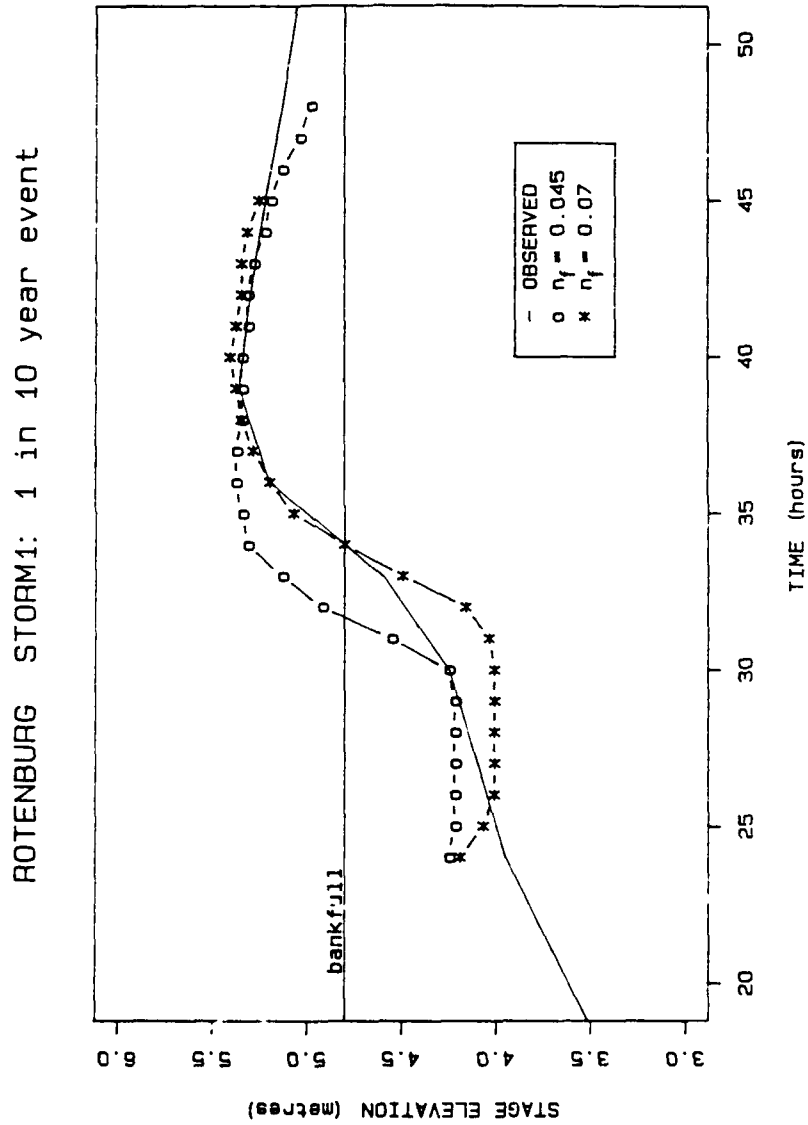


Figure 8.6
Comparison of observed and RMA-2V, version 4, simulated hydrographs
with variations in the floodplain Manning's n , of the 1 in 10 year
event at Rotenburg

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simulated hydrographs is much improved.

The effects of increasing the floodplain Manning's n coefficient in Storm 2, the 1 in 100 year event are shown in Figure 8.7. The 1 in 100 year event has been generated from the 1 in 10 year event using the flood frequency data available, and it is interesting to note, therefore, the much more rapid travel time of the peak stage in the 1 in 100 year event in comparison with the 1 in 10 year event. This faster travel time can be attributed to the greater inundation depths on the floodplain and consequently the diminished friction retarding effects of the boundary roughness.

This reduced effect of the boundary roughness in Storm 2 is confirmed in the behaviour of the simulated hydrograph, when the Manning's n coefficient is increased to 0.07. Although, as in Storm 1, the travel time of the peak is increased, in comparison with Storm 1 the relative increase in the travel time attributable to the increase in boundary roughness is small.

The effect of increasing both the main channel and floodplain roughness coefficients in version 3 of RMA-2V can be seen in Figure 8.8. Version 3 incorporates the wet/dry criteria where elements leave or enter the solution depending on the depth of inundation. Figure 8.8 shows that as the Manning's n coefficient is increased, the stability of the solution is reduced. As this behaviour was not experienced in Version 4 of RMA-2V, this instability must be due to the handling of the wet/dry criteria rather than directly attributable to the boundary friction effects.

8.4.2 Variation in the handling of the wet/dry criteria

The previous sub-section illustrated that the stability of the two versions of RMA-2V is different when the Manning's n roughness coefficient is varied. Version 4, which incorporated the marsh elements, seems more stable than Version 3, which is based on the

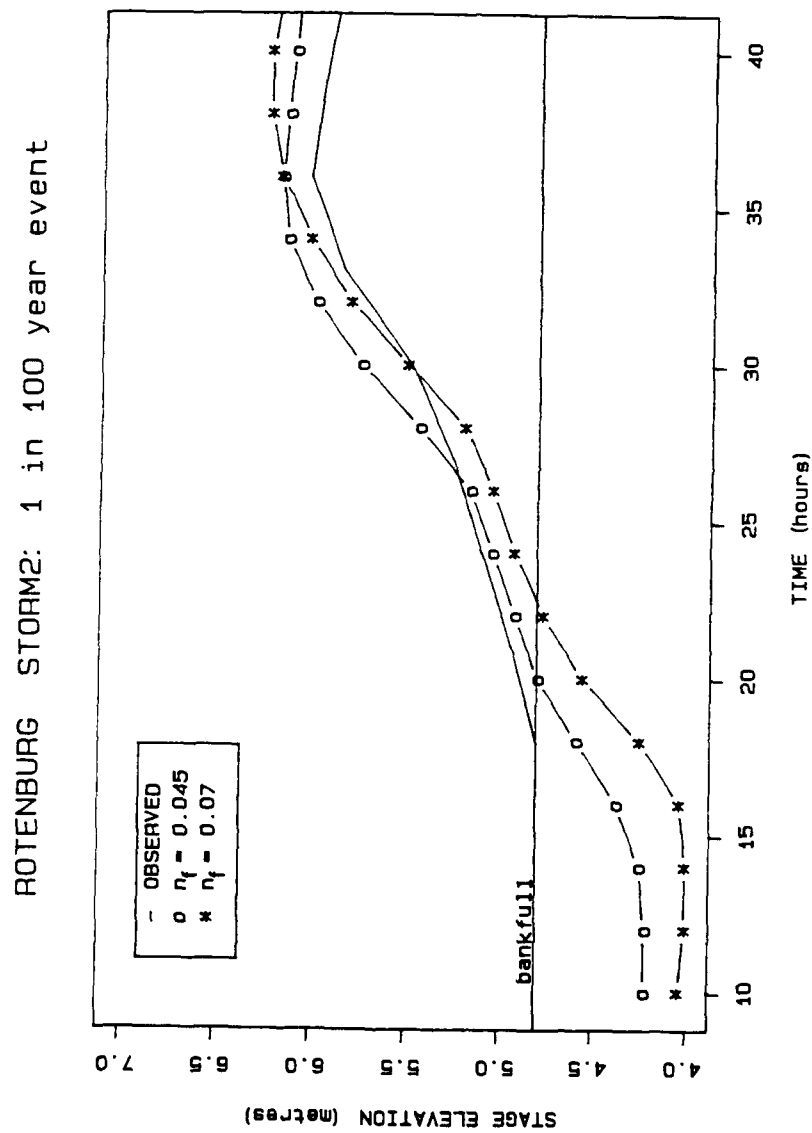


Figure 8.7
 Comparison of observed and RMA-2V, version 4, simulated hydrographs
 with variations in the floodplain Manning's n , of the 1 in 100 year
 event at Rotenburg

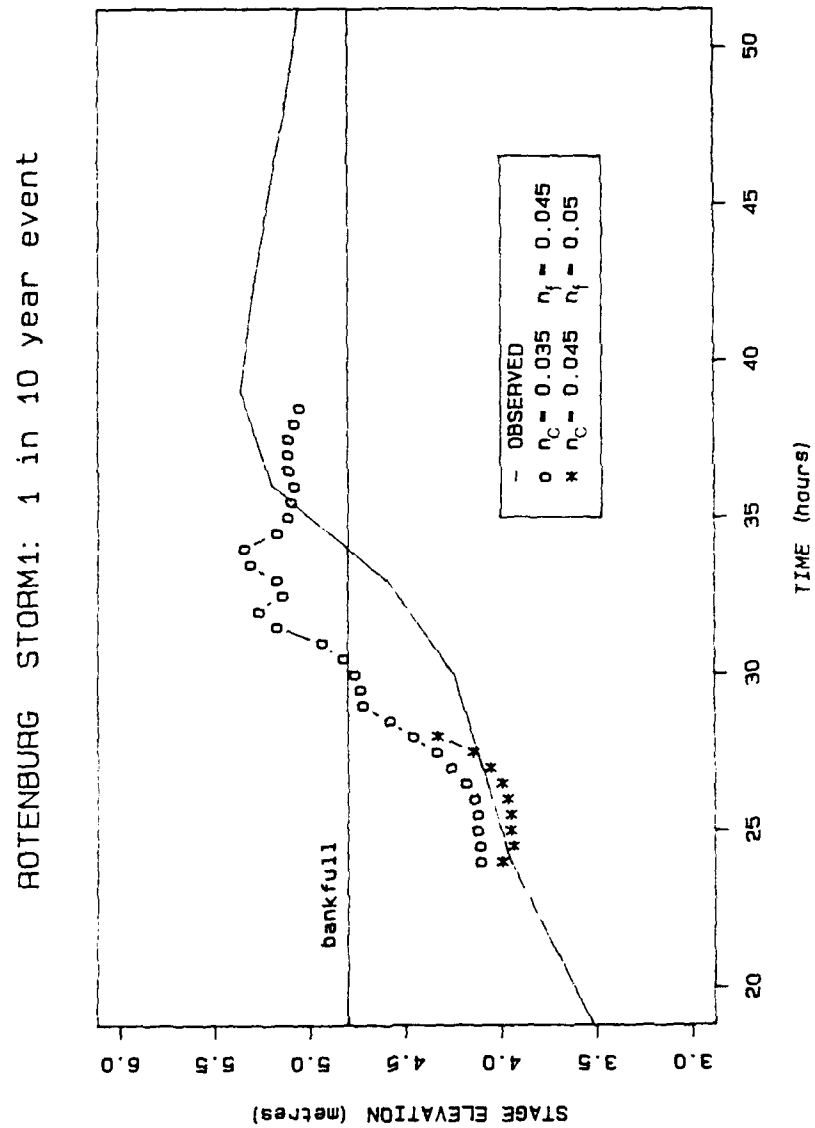


Figure 8.8
Comparison of observed and RMA-2V, version 3, simulated hydrographs
with variation in the floodplain and main channel Manning's n, of the
1 in 10 year event at Rotenburg

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wet/dry elements.

Analysis of the corresponding velocity vector plots for Figure 8.8 showed that most of the flow in the simulation of Storm 1 using Version 3 is concentrated in the main channel, irrespective of the Manning's coefficient values. It is probable, therefore, that the failure of the simulations is due to a channel element falling out of the solution as an element became dry.

One way of testing this hypothesis would be to investigate the effects of varying the wet/dry criteria. Figure 8.9 shows the effects of four different wet/dry combinations on the simulation of Storm 1 using Version 3 of RMA-2V. The effect of decreasing the wet and dry criteria is to extend the stage elevations for which an element is included in a particular solution. This should ensure that all the channel elements remain in the solution and, therefore, that the length of the simulation is increased. In addition, reducing the wet/dry criteria should increase the number of floodplain elements that are in the solution, so that the spatial extent of the floodplain inundation is increased. The data available for the 1 in 10 year event indicate that the 1 in 10 year event generates a large inundated area and that nearly all the elemental areas, except the boundary elements, should be inundated.

Figure 8.9 shows that as the wet/dry criteria are decreased from 1.0/0.5 to 0.6/0.1, the stability of the hydrograph solution is increased and the length of the simulation is increased. Further decreases to 0.4/0.05 and 0.2/0.05, however, reduce the stability and hence length of the solution. This decrease in stability of the solution as the wet/dry criteria are reduced further, is theoretically unsound. However, the coding of Version 3 of RMA-2V prevents the criteria tending to zero. This limitation has only become apparent in this application because of the very low slope across the elemental areas. In previous applications, the criteria have been set to exceed the slope of the individual elements. These

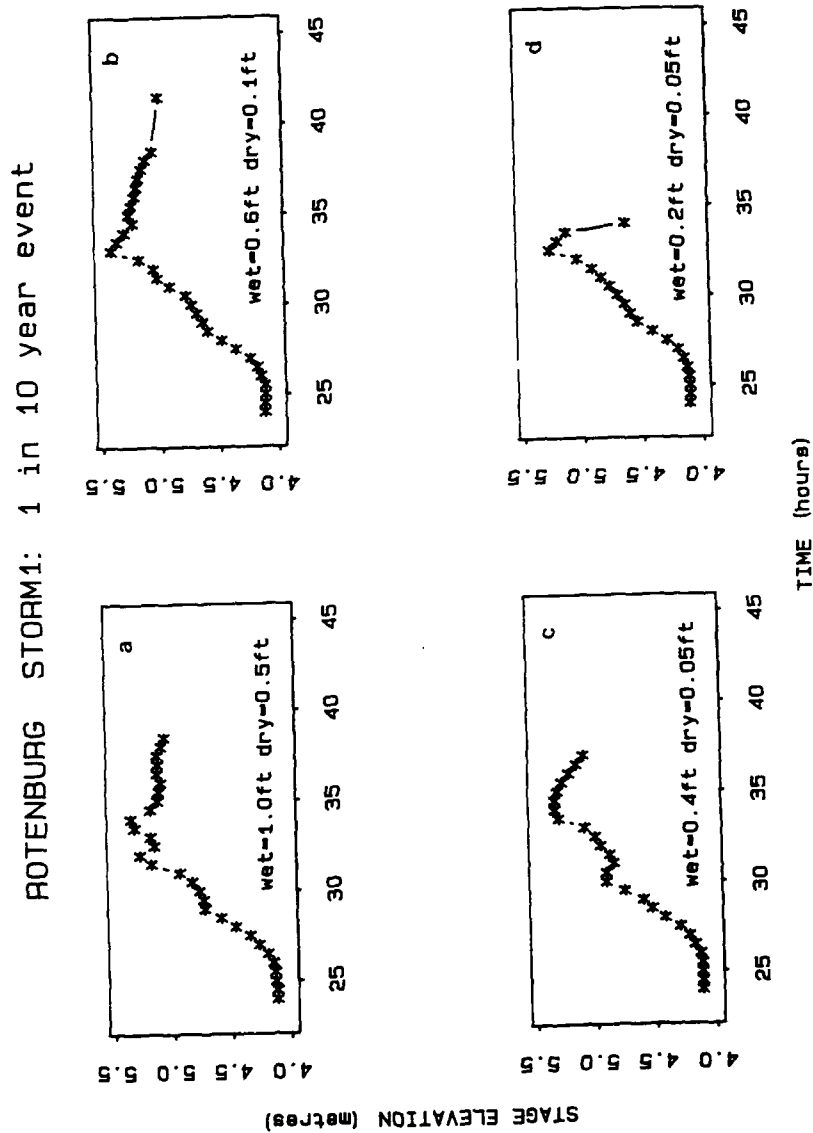


Figure 8.9
Comparison of the stability of solutions of RMA-2V, version 3, for four wet/dry parameter combinations, of the 1 in 10 year event at Rotenburg

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slopes, however, are of a greater magnitude than the slopes of the elements in this application, particularly in the direction orthogonal to the main channel.

The stability of the solution is, though, improved by the lowering of the wet/dry criteria before this limitation is reached. The effects of lowering the criteria to 0.6/0.1 improves the stability of the solution, and the extent of the floodplain inundation should also be increased. Figures 8.10 and 8.11 are the velocity vector plots for the peak discharge conditions of the initial wet/dry criteria, that is 1.0/0.5, and the improved stability solution with wet/dry criteria of 0.6/0.1. Comparison of these two vector plots, however, shows that the area of inundation is more or less identical in the two solutions. In both vector plots, the area of inundation is too small for the size of the 1 in 10 year event.

The failure of the manipulation of the wet/dry criteria to have any impact on the area of inundation suggests that too much of the flow is being carried in the main channel element rather than on the floodplains. The main channel is represented by a triangular cross-section consisting of two elements. This simple representation was made to reduce the number of elements used in the schematization of the reach to improve the performance of RMA-2V. The premise of this simplification was that the main area of interest in this investigation was the floodplain. These results suggest, however, that the representation of the channel was insufficient to ensure that the bankfull channel capacity was accurately modelled. In further applications of this type, therefore, it is recommended that the cross-sectional geometry be more accurately given.

Comparison of the simulation utilising the two versions of RMA-2V of Storm 1 with identical Manning's n coefficients is shown in Figure 8.12. The comparison of the wet/dry solution (Version 3)



Figure 8.10
Velocity vector plot for the peak discharge condition simulated
utilising RMA-2V, version 3, with wet/dry criteria of 1.0/0.5



Figure 8.11
Velocity vector plot for the peak discharge condition simulated
utilising RMA-2V, version 3, with wet/dry criteria of 0.6/0.1

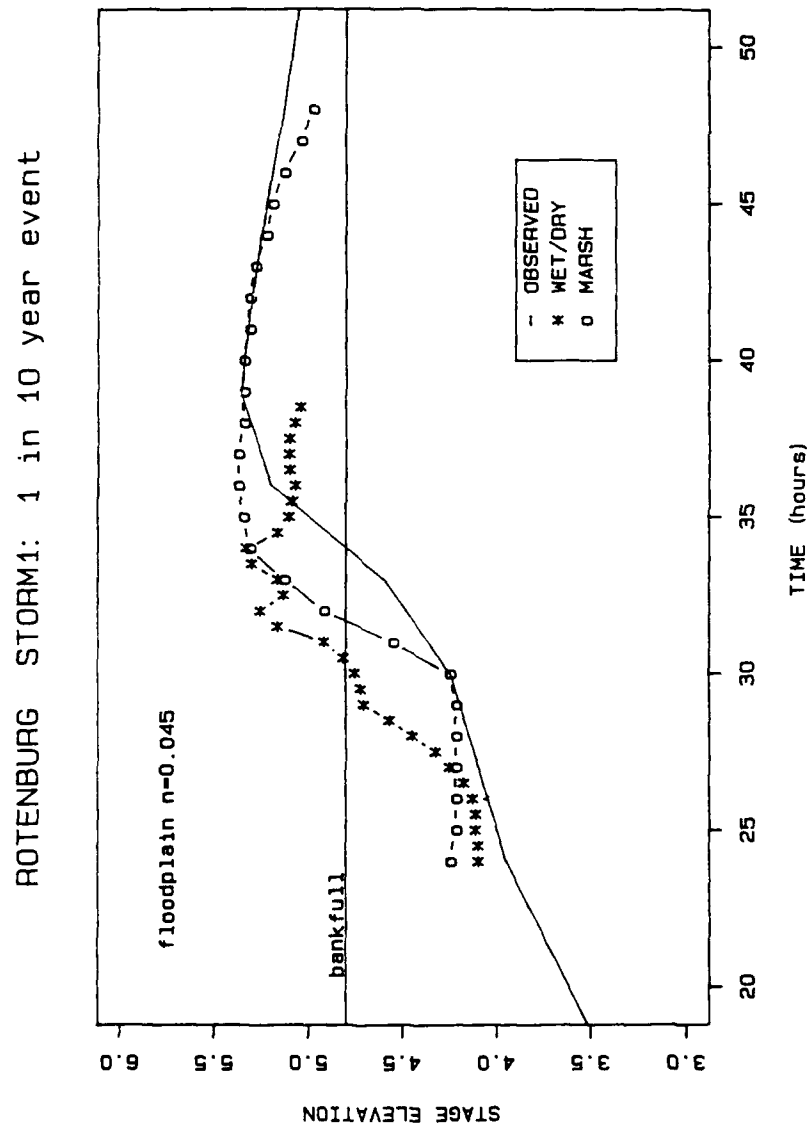


Figure 8,12
 Comparison of observed and simulated hydrographs generated by
 RMA-2V, versions 3 and 4, of the 1 in 10 year event

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and the marsh element solution (Version 4), shows that the wet/dry solution is not as smooth as the solution of Version 4. This is because in Version 3 elements are entering and leaving the solution, which generates rapid increases and decreases in the flow area and velocity distribution across the floodplain. The smoother solution of Version 4, and the superior fit of Version 4 to the observed hydrograph, show that the more gradual reduction or increase in flow through the elements incorporated in Version 4, is more physically realistic or, at least, the predictive accuracy of the version is improved.

The stability of the marsh elements version when subjected to variation in the Manning's n coefficients and the superior fit to the observed hydrographs suggest that the development of the marsh elements has made a significant improvement in the predictive behaviour of RMA-V2. It is proposed, therefore, that Version 4 be adopted for further validation.

8.4.3 Conclusions

The application of RMA-2V to the River Fulda reach using observed inflow hydrographs and comparing the simulated outflow hydrographs at Rotenburg, has shown that RMA-2V can be used on reaches of this length. It has also shown that a topographic map and upstream/downstream cross-sectional data provide adequate data for the derivation of a stable solution.

Comparison of Versions 3 and 4 of RMA-2V have shown that Version 4 is superior. The solutions are stable to variations in the specifications of the boundary roughness specifications, and the shape of the hydrograph using Version 4 fits the observed hydrograph to a much greater degree.

However, neither version of RMA-2V produced reasonable estimates of the inundated area of the floodplain. This has been

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attributed to the poor representation of the main channel. Overall, the results are promising for the utility of RMA-2V as a tool for testing simpler hydrologic models. Most important is the generation of a stable solution of RMA-2V for a relatively large reach with limited data.

8.5 Inclusion of RMA-2V as a Module of MILHY3

The results of the analysis reported above, show that it is feasible to incorporate RMA-2V as a module of MILHY3. The scale of the reach has not proved to be a problem, nor has the amount and quality of the data set. However, these initial results have shown that the time taken to initialise the network is extensive, and the level of expertise required is much higher than for the simpler MILHY3.

Before the inclusion of RMA-2V can be recommended, it is necessary that a set of circumstances be identified where the advantages of using RMA-2V would outweigh the disadvantages of using RMA-2V. It is important to compare the performance of MILHY3 and RMA-2V, and to identify the possible advantages of either scheme.

Figure 8.13 compares the performance of MILHY, MILHY3 and RMA-2V against the observed outflow hydrograph at Rotenburg. All schemes have only modelled the reach between Bad Hersfeld and Rotenburg, and all have used the observed hydrograph at Bad Hersfeld as the inflow hydrograph. The MILHY3 model has utilised both the momentum exchange and multiple routing routines and is the version that best-fits the observed hydrograph identified in Chapter 7. The RMA-2V model utilised is Version 4.

Figure 8.13 shows that the MILHY prediction is the least accurate of the three models, and that RMA-2V provides the closest fit to the observed hydrograph. Not surprisingly, the difference

BAD HERSFELD - ROTENBURG STORM1: 1 in 10 year event

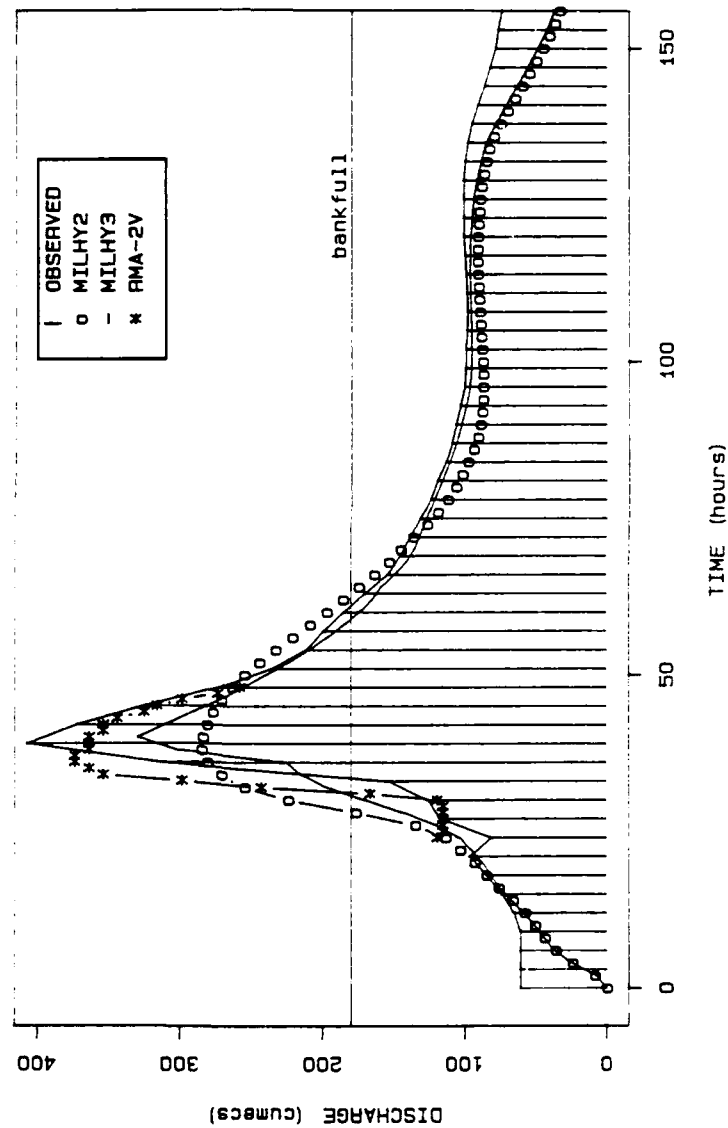


Figure 8.13
Comparison of observed and simulated hydrographs generated by
MILHY2, MILHY3 and RMA-2V, of the Bad-Hersfeld-Rotenburg reach
for the 1 in 10 year event

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between the MILHY, MILHY3 models is smaller than the difference between the two MILHY models and RMA-2V. However, the difference between the three models is less than the error between any of the three simulations and the observed event. This figure suggests, therefore, that for the resolution of data available, the improved accuracy of the predicted hydrograph using RMA-2V does not automatically outweigh the disadvantages of the initiation of RMA-2V. If a highly accurate model of just the downstream conveyance of a reach is required, then RMA-2V does provide a more accurate hydrograph than MILHY3, and undoubtedly RMA-2V's accuracy could be increased with a higher data resolution.

In the context of the ungauged catchment, however, where it is unlikely that more data would be available, the use of RMA-2V as an alternative to MILHY3 in the prediction of the hydrograph in a single reach is not so attractive. However, where data are required on the extent of the floodplain inundation, or the velocity of the inundated flow, then RMA-2V undoubtedly is a superior model. The accuracy of the prediction of the inundated area and the velocity of the flow in inundated areas, has not been evaluated in this analysis. It is suggested that a data set should be established to enable this evaluation at this scale of application as a matter of priority.

Figure 8.14 compares the performance of RMA-2V in the prediction of the outflow hydrograph as a catchment model. RMA-2V has been used to replace the MILHY3 scheme on the last section of the reach between Bad Hersfeld and Rotenburg. The rest of the catchment has been modelled using MILHY3; the inflow at Bad Hersfeld for the RMA-2V scheme is generated using MILHY3. Figure 8.14 compares the performance of the replacement of this last reach with RMA-2V, with the prediction using MILHY and MILHY3 for the entire catchment.

Comparison of the MILHY models and the MILHY3+RMA-2V model

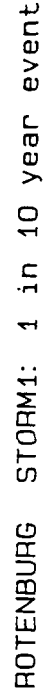


Figure 8.14
Comparison of observed and simulated hydrographs generated by MILHY,
MILHY3 and MILHY3 + RMA-2V, of the River Fulda catchment, for
the 1 in 10 year event

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shows that the difference between the MILHY models is much greater than the difference generated by the introduction of RMA-2V in the last section of the reach. In the prediction of the response of the entire catchment, the introduction of the infiltration algorithm has a greater effect than the more accurate modelling of the last reach.

These results suggest that the inclusion of RMA-2V as a module into MILHY3 is not recommended if the operator intends to use the model for the prediction of an outflow hydrograph. If the operator requires information on the nature of the floodplain inundation, then the inclusion of RMA-2V for the area prone to inundation may prove worthwhile. It should be noted again, however, that whilst the accuracy of the predicted hydrograph produced by RMA-2V has been evaluated in this analysis, and shown to be more accurate than MILHY3, the predictive accuracy of the extent of floodplain inundation and velocity vectors remains unevaluated.

8.6 Conclusions

The objective of this chapter was to investigate whether a state-of-the-art model could be used to extend the record of extreme event hydrographs, so that simpler models, such as MILHY3, can be evaluated. Analysis of the sophisticated models available showed that these models are still essentially research models and have not been evaluated themselves. As a result of this, it was accepted that the investigation should concentrate on the downstream conveyance modules of MILHY3, and attempt to identify a sophisticated model for conveyance.

A two-dimensional approach was identified as one which incorporated the cross-sectional geometrical effects of two-stage flow, and yet still offered a range of evaluated models. The finite-element solution of the equations of flow in two-dimensions was identified as being the method capable of incorporating the plan geometry of the system.

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However, analysis of all the two-dimensional models available showed that none had been applied to reach lengths of the scale required for the evaluation of MILHY3. Most schemes were limited to scales of around 2 km, whereas reach lengths on the River Fulda are typically 20km. It was not known whether such hydraulic models could be utilised at these larger scales, if their solutions would be stable, or if the concepts on which they are based could be scaled up.

The RMA-2V model was selected to investigate the suitability of hydraulic models in general for application to these larger scale reaches. If successful, they could not only be used to evaluate simpler models but also to replace them.

The results of the application of RMA-2V to the River Fulda are very promising. The latest version of the model that incorporates marsh elements is stable for a variety of boundary roughness conditions, and the predictive accuracy of the model seems better than the predictive accuracy of MILHY3. In addition, the application of RMA-2V provides the potential for more detailed modelling of the floodplain, as the inundated extent and velocity vectors are computed.

Before RMA-2V is ready for application to the larger scale catchments, however, it is important that certain areas not included in this analysis should be investigated and evaluated as a matter of priority. These areas include:

- 1) the sensitivity of the RMA-2V solution to variation in the eddy exchange coefficient.
- 2) the evaluation of the predicted area of floodplain inundation against a field data set
- 3) the evaluation of the accuracy of the predicted velocity vectors against a field data set

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Once these areas have been evaluated, then RMA-2V could be used to generate hydrographs from the flood frequency data to validate simpler hydrologic models for extreme events. The investigation of the above areas should also enable the set of circumstances where the inclusion of RMA-2V in a catchment model such as MILHY3 would be profitable. The results reported above suggest that including RMA-2V in a catchment model would be profitable where a higher level of predictive accuracy is required in the simulated hydrograph, or where details of the floodplain inundation would be useful.

The success of the RMA-2V application to a large-scale catchment shows that the separation of hydraulic and hydrologic modelling on the grounds of scale alone is not a valid one. RMA-2V, a hydraulic model, operates successfully on a scale usually reserved for hydrologic approaches. In addition, the results suggest that RMA-2V could be successfully incorporated into a composite modelling strategy. The combining of hydrologic and hydraulic approaches in a single modelling structure provides a powerful and flexible tool for the prediction of floods in ungauged catchments.

Chapter 9

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Floodplain Inundation Modelling Utilising RMA-2V

The results shown in Chapter 8 of this report indicate a promising future for the linking of the RMA-2V and MILHY3 schemes. Initial results from the River Fulda show that large-scale floodplain modelling using RMA-2V is possible, even with parsimonious data sets. The analysis carried in Chapter 8, however, concentrated on the predictive accuracy of the outflow hydrograph at the Rotenburg station on the River Fulda. The objective of this chapter is to continue to explore the utility of RMA-2V but specifically to investigate the two-dimensional components of the model. The application of RMA-2V to shallow floodplain environments is investigated from a more "operational" perspective, with questions on resolution and topographic representation explored more explicitly than in Chapter 8.

For this investigation a new study reach, the River Culm in Devon, England, has been established. The River Culm offers a more detailed inundation data set than available for the River Fulda and, in addition, provides a very much more topographically complex terrain to simulate (see section 9.1). The problems associated with establishing the River Culm elemental mesh are discussed in section 9.2, and the results of simulations of the scheme are presented in section 9.3. In section 9.4, the implications of these results for ungauged flood forecasting are discussed and section 9.5 provides guidelines and recommendations for future applications.

9.1 River Culm

The River Culm is a tributary of the River Exe, joining the

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Exe approximately 3km north of Exeter (see Figure 9.1). It has a catchment of 275 km² and in its lower reaches has a well-developed floodplain averaging 500m in width. The main channel is gravel-bedded and approximately 12m wide. The banks are 1m high and consist of fine alluvial material. Overbank flooding is relatively frequent during the winter months when usually ten overbank events a year occur. During these events, a substantial area (5.5 km²) is inundated, with typical inundation depths being 40cm for the mean annual event and 70cm for a 50 year event. Land use on the floodplain is almost exclusively permanent pasture (see Figure 9.2) which is used for summer grazing, hay and silage production.

This study has focused on the 11km stretch of the River Culm between the flow gauging stations at Woodmill (upstream) and Rewe (downstream), see Figure 9.1. Between these gauging stations the tributary, the River Weaver, joins the River Culm but flows from the Weaver especially during flood events on the Culm are small enough to be ignored for the purposes of this investigation.

Along this stretch of the River Culm the channel is restricted in two places by paper mills at Hele and Silverton. Figure 9.3 shows the main channel constricted through the mill at Hele. Both Figures 9.2 and 9.3 were taken during a visit to the reach in October 1989. The overall slope between the Woodmill and Rewe gauging stations is 0.002, although this varies considerably within the reach.

In comparison with the River Fulda, Bad Hersfeld to Rotenburg reach, the identified stretch of the River Culm is a highly complex environment. Inundation is very spatially variable with areas of storage and areas of fast moving floodplain flows. Figure 9.4 illustrates this variability through a radiological investigation carried out by Walling *et al.* (1986).

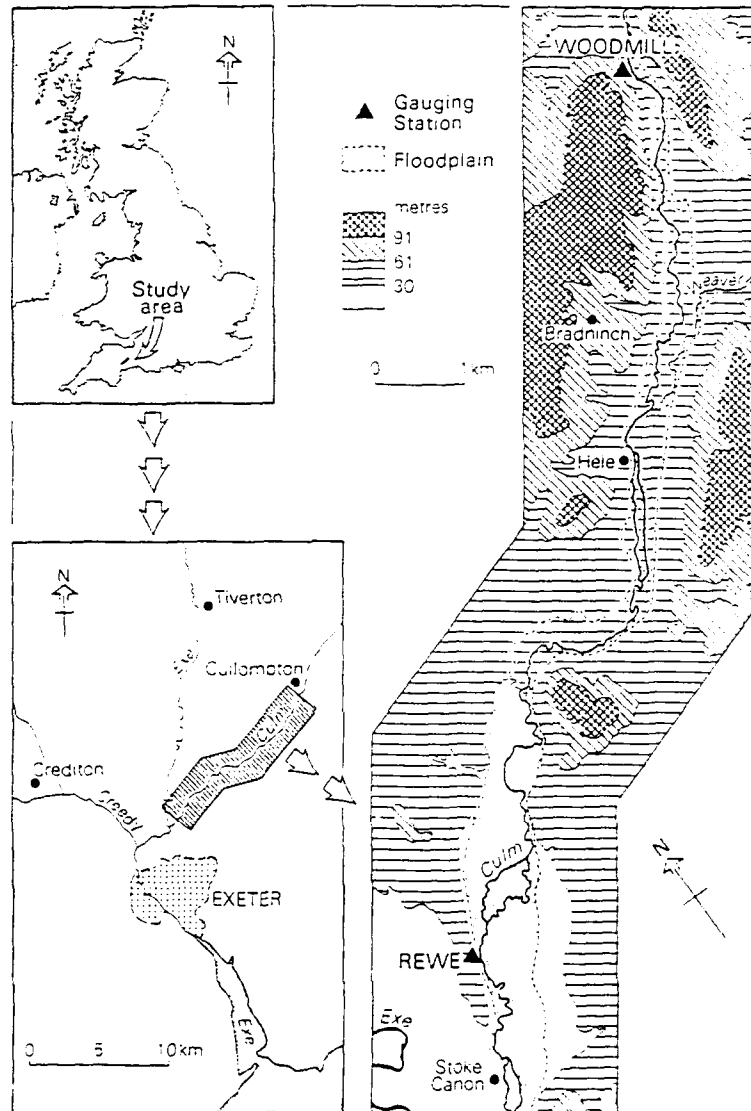


Figure 9.1
Location of River Culm reach, Devon, U.K.



Figure 9.2
Channel and Floodplain Cross-Section 2km Upstream
of Rewe

Figure 9.3
Hele Mill Race, River Culm



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Figure 9.4 shows the Caesium 137 activity of sediment cores taken at various locations throughout the reach. The wide variation in the C^{137} activity of these cores illustrates the spatial and temporal variability of floodplain deposition since nuclear tests carried out in the late 1950s. Caesium 137 released during these nuclear tests which peaked in 1964, was rapidly absorbed in the upper soil horizons. Subsequent movement of C^{137} within the landscape is therefore associated with the erosion transport and deposition of sediment particles (Walling *et al.*, 1986). Enhanced levels of C^{137} above a background level can, therefore, be associated with deposition of sediment eroded from elsewhere in the catchment.

The radiological investigation conducted by Professor Des Walling and his team at the University of Exeter, UK, has generated a large volume of inundation data. Access to this data set has proved an invaluable part of this investigation, and the help of Professor Walling is acknowledged.

9.2 River Culm Elemental Mesh

Figure 9.5 shows the completed mesh for the River Culm between Woodmill and Rewe. This mesh has been generated from the 1:2500 series of topographic maps of the area, maximum flood extent maps from the National River Authority, air photographs of flood events and ground surveys, and drawings taken during visits to the catchment. Generally, the data available enabled a more topographically detailed mesh than the River Fulda mesh (see Figure 8.4), and this is highlighted in the smaller elemental areas utilized. River Fulda elemental areas averaged 200m by 200m, whilst River Culm elements are generally 100m by 100m.

Following the conclusions of the River Fulda applications, section 8.4.2, the River Culm mesh incorporates a more accurate channel cross-sectional geometry. Summarized in Figure 9.6, the

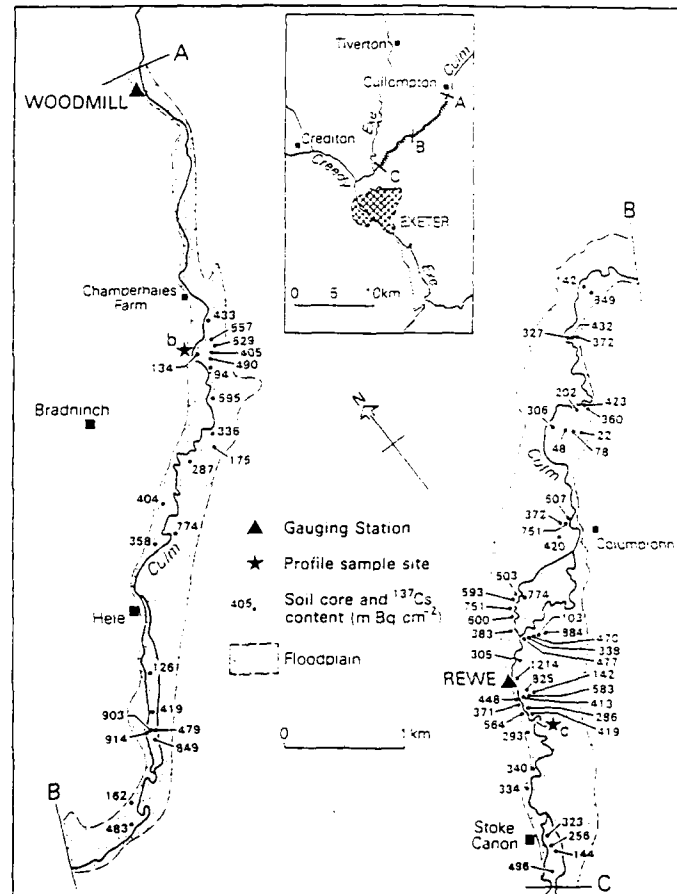


Figure 9.4
Caesium 137 distribution from experimental data by
Walling et al. (1986)

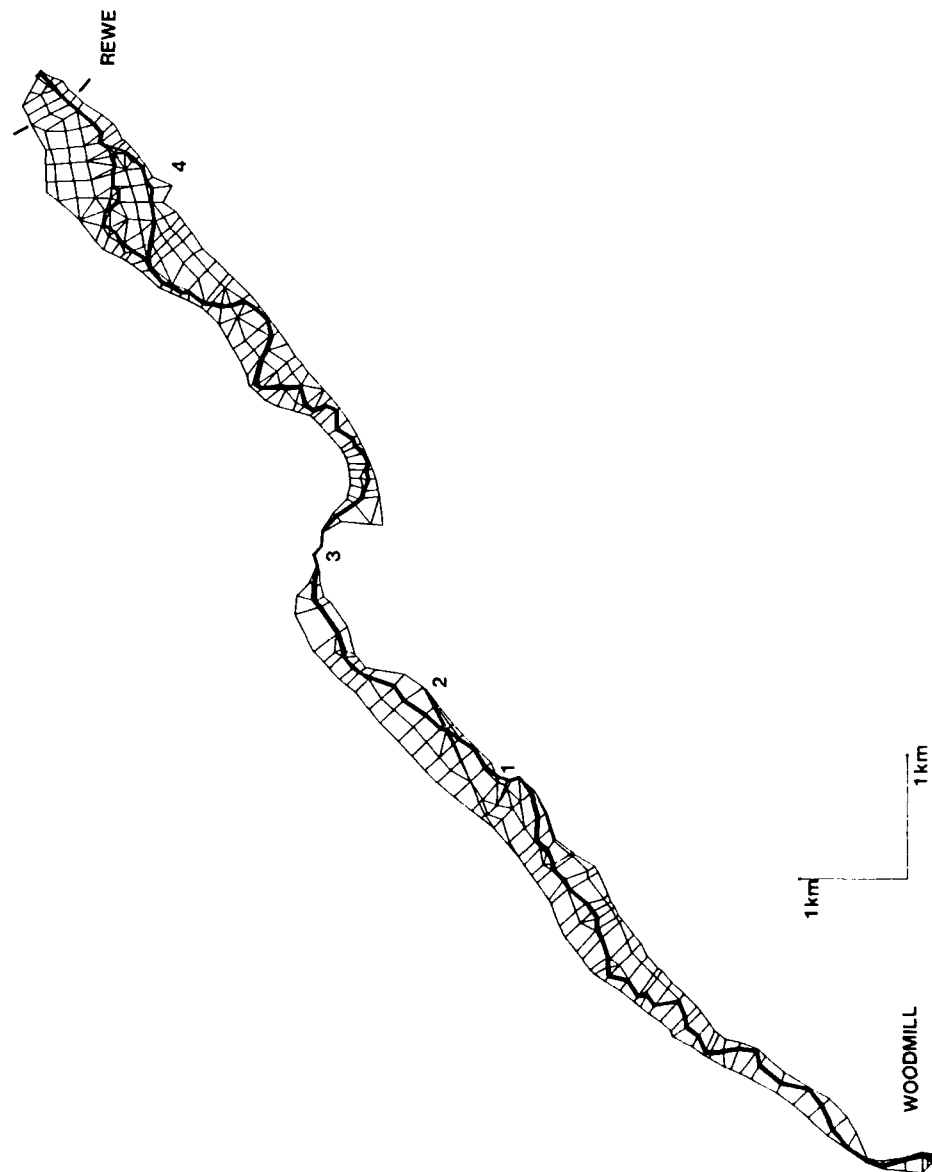


Figure 9.5
River Culm elemental mesh for application of RMA-2V

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River Fulda channel can be seen to be represented by a two element strip generating a triangular cross-section. This proved to be too efficient a shape. The River Culm mesh, therefore, utilizes a trapezoidal cross-section with a three element strip representing the channel. In addition, this three element strip is flanked by another element either side of the channel to represent bankside vegetation.

Ten boundary roughness and hence ten element groups have been identified. These are:

	Number of elements
1 Channel bed	164
2 Channel bed, vegetated	6
3 Channel bank sides, grass	269
4 Channel bank sides, trees	1
5 Channel bank sides, scrub	3
6 Bankfull, trees	27
7 Bankfull, scrub	6
8 Bankfull, grass	227
9 Floodplain, grass	374
10 Floodplain, woodland/carr	12

Approximately 1090 elements have been utilized, the distribution of which is indicated above.

Several specific features of the River Culm reach were also incorporated, identified on Figure 9.5. These are:

- 1 Hele mill
- 2 Railway embankment
- 3 Silverton mill
- 4 Channel bifurcation

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The two paper mills at Hele and Silverton are accurately represented as channelized channels, with low boundary friction and high velocities. Silverton mid flows are contained entirely within the race and no floodplain elements are required. The railway embankment creates a barrier across the floodplain which, since raising of the line in 1989, has not been breached. This feature, therefore, creates a large pond of low velocity flow and the funnelling of flow through a bridge under the embankment. The bifurcation near the downstream gauge creates a large complex floodplain island, part of which may be seen in Figure 9.2. Flows are equally divided between the two parts of the channel, and the bifurcation is approximately 1.5km in length.

9.2.1 Mesh development

Figure 9.5 represents the final version of the elemental mesh developed for the River Culm; several intermediate versions have been discarded. Application of RMA-2V to the River Culm generated some new and novel problems, particularly in the resolution of topographic representation. Taking each of the developmental stages in turn, the first mesh version incorporated all the topographic details available from the data set. Having ensured there were no errors in the generated mesh, a test run was undertaken which proved to be very unstable. Interrogation of the results files enabled several problematic areas of the mesh to be identified. These were characterized in general by one or more of the following problems:

- 1) areas where large and small elemental areas are adjacent
- 2) areas of steep lateral or longitudinal slope
- 3) sharp changes in the direction of flow
- 4) all four of the specific features (mills, etc.) identified earlier

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Each of these problems created a numerical "shock" to the solution, which either were overcome with poor continuity solution or perturbed until finally the solution failed. To overcome these problems, a degree of "smoothing" of the mesh was required. This created some fundamental questioning of the application of finite-element schemes to large scale reaches. In particular, can enough topographic resolution be incorporated to provide a meaningful solution and to meet the numerical stability of the program?

It is important here to distinguish between topographic resolution and mesh resolution. Mesh resolution may be increased without gaining topographic resolution by interpolating intermediate data. Decreasing topographic resolution requires smoothing physical features of the system, although mesh resolution may increase or decrease.

The first problem tackled was in areas where very large elements were adjacent to very small elements. This situation almost always occurs at the floodplain channel interface. This required widening of the channel side elements (see Figure 9.6) and, by necessity, to maintain channel volume narrowing of the channel bed elements throughout the reach. This improved the stability of the solution without reducing topographic resolution. Such adjustments were, however, not enough to overcome the fatal perturbations generated elsewhere in the mesh.

The second set of modifications undertaken were to the lateral slopes of the floodplain and channel elements. Figure 9.6 shows the conceptual cross-sectional structure adopted for the River Culm, and shows that the bankside elements have been assumed to be horizontal. This geometrical combination not only provides a lack of continuity in the lateral slopes, but also means that a very small variation in stage elevation will cause these elements to be either totally inundated or totally dry. Given the success of the marsh element

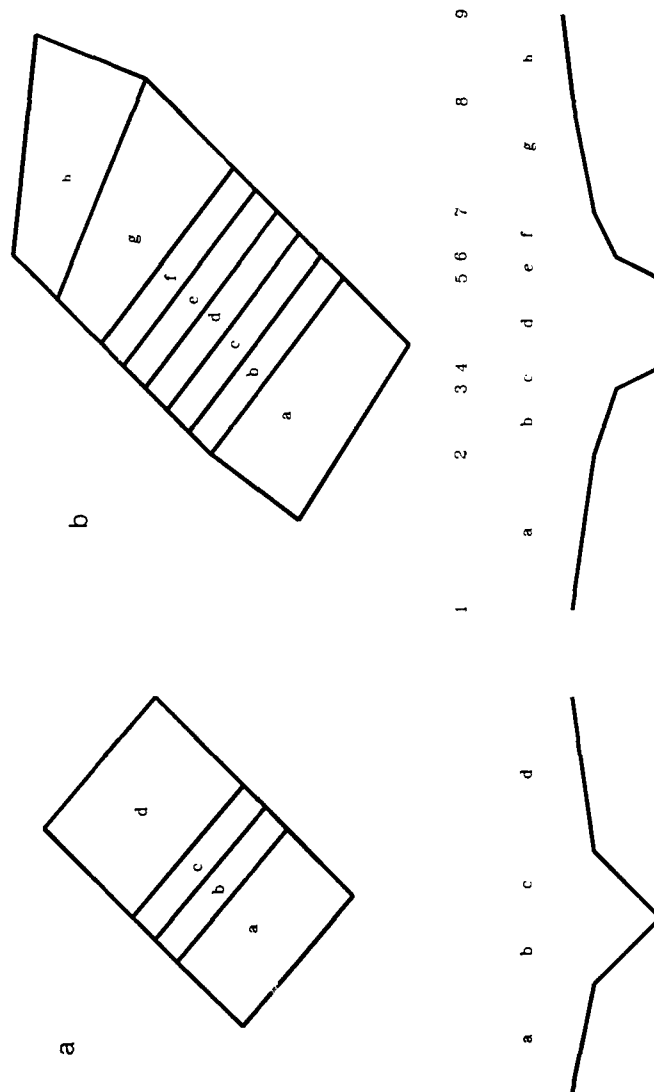


Figure 9.6
Conceptual Mesh Cross-Sections for
 a) River Fulda
 b) River Culm

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version of RMA-2V, discussed in section 8.2.2, in gradually phasing elements in and out of the solution, it was decided to introduce a slight slope to the bankside elements. In other areas of the mesh the lateral slopes within elements were decreased such that no element contained an elevation drop of greater than 3m (10ft.). This change necessitated some loss of topographic resolution. Once again the stability and continuity of the solution were improved, but "shocks" to the system still occurred.

The third series of modifications undertaken are those seen most clearly when comparing a segment of the two versions on Figure 9.7, and involving modifying the sharpness of angle in the direction of flow. Most of these modifications were to reduce the sinuosity of the main channel. Close examination of Figure 9.7 shows that this straightening has been achieved through the removal of small meander bends and the straightening of larger bends. No channel meander is now more acute than 110° . These changes also meant several smaller elements, situated within meander bends, were no longer required. These changes made throughout the reach made a noticeable improvement in the efficiency of the program. The front width was reduced from 260 to 210 and stability was much improved. Comparison of Figures 9.7a and 9.7b also shows that the downstream extremity of reach in version 3 has been stretched. This was to allow the positioning of the downstream gauge away from the effects of the outflow control structure.

The final modifications to the mesh involved adjustment of the longitudinal slope and coincided with the release of version 4.2c of RMA-3V. The modifications involved distributing the relatively steep longitudinal slope of a short section of the reach, close to the upstream gauge, over a longer distance. The area concerned had an elevation drop of approximately 0.75m every 150m; this was followed by a section with 0.003m drop per 150m. These two slopes were then averaged over the two reach segments. These changes, together with the modifications to the wetting and drying



Figure 9.7
Evolution of mesh for the bifurcation on the River Culm
a) field data
b) smoothed

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sequence in the new release of RMA-2V, proved successful and a stable initialisation solution was achieved.

This series of modifications indicates fairly clearly that there is a limit to the topographic resolution for RMA-2V applications at this scale. Attempts to improve the stability of the solution by increasing the mesh resolution of topographically complex areas failed. Figure 9.8 shows an extreme example of this, where a complex S bend just upstream of Silverton Mill was being simulated. Figure 9.8a illustrates the first mesh established, whilst figure 9.8b shows an increased resolution of mesh initialised when the first mesh solution failed. Comparison of Figures 9.8a and 9.8b also illustrates the widening of the channel side elements and narrowing of the channel bed elements. Figure 9.8c shows the final solution where the bend has been totally removed. Although this is the most extreme example on the River Culm application, it does serve to show the problem of topographic and mesh resolution.

From the establishment and initialisation of the River Culm mesh, it is possible to suggest several guidelines for future applications of RMA-2V at this scale. These are:

1. Channel platform - an upper limit of 110° has been found to apply to the representation of meanders. This criteria then sets constraints on the minimum length of channel element.
2. Channel long profile - an upper limit for slopes appears to be 0.3m drop per 150m.
3. Lateral slopes - an upper limit here appears to be 3m in any one element.
4. Floodplain topographic representation - resolution appears limited to features 10 - 100m in size, rather than 1-10m.

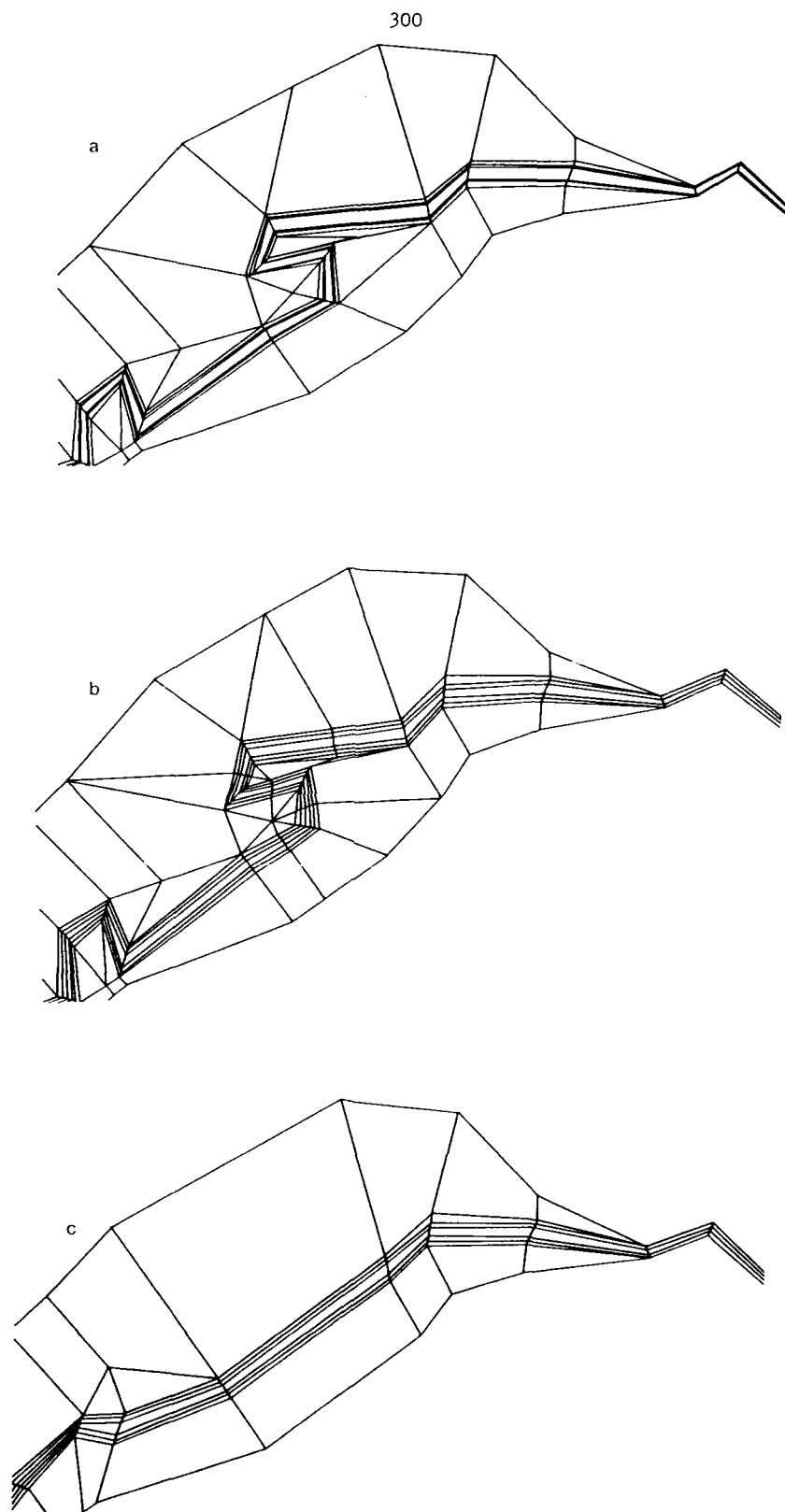


Figure 9.8
Evolution of mesh for a double bend on the River Culm
a) field data, b) increased mesh resolution, c) smoothed

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9.3 Application of the River Culm

The overall objective of applying RMA-2V to the River Culm reach is to continue evaluating the performance of RMA-2V at this scale. In particular, the River Culm has allowed the evaluation in a more complex topographic terrain than previously undertaken, and the more exhaustive data set will allow rigorous evaluation of the simulated inundation distributions as well as predicted outflow hydrographs. The establishment and initialisation of the mesh has already proved useful in exploring topographic and mesh resolution issues and these issues will be further discussed later in this chapter. The remaining sections of the chapter explore the sensitivity of RMA-2V and more practical issues of reach calibration and the interpretation of results. To calibrate the reach some indication of the sensitivity of the reach to parameter variability is required, and it is to this objective that the rest of this section is dedicated.

9.3.1 Sensitivity of the River Culm Application

The results from the River Fulda application suggested that applications of RMA-2V are most sensitive to changes in the Manning's 'n' coefficient. However, other parameters identified in Chapter 8 must not be ignored, and include:-

- turbulent eddy coefficients
- marsh element parameters
- rating curve relationship
- mesh resolution
- time step resolution

As the objective of this analysis is not to undertake an exhaustive and full sensitivity analysis, but rather to explore sensitivity for the purposes of calibration, each of the above groups of parameters is explored somewhat more subjectively.

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Firstly, Manning's n having been identified by the River Fulda application. A series of steady-state applications were undertaken where individual or groups of Manning's n elements were varied, and the impact of this sensitivity assessed on the specific flow (depth $\times$ velocity) across a continuity line close to the downstream gauging station. The continuity line includes all nodes from the boundary of one floodplain across the channel and through the opposite floodplain, and the position of this line is shown on Figure 9.5. Figure 9.6b identifies the position of the nodes specified in the analysis.

Tables 9.1 and 9.2 collate the results of varying the Manning's n coefficients under two flow regimes. Table 9.1 shows the results for bankfull conditions (24m elevation, $30 \text{ m}^3 \text{ s}^{-1}$ discharge), whilst Table 9.2 shows the results for the peak of the 1 in 10 year event, where floodplain inundation depths are 80cm and discharge is approximately $120 \text{ m}^3 \text{ s}^{-1}$. Both tables make comparisons with original Manning's ' n ' values specified by Chow (1959) during visits to the reach. The original Manning's n values for each element type were specified as:

Element type	Manning's n
1 Channel clear of vegetation	0.03
2 Channel, vegetated	0.035
3 Channel bank sides, grass	0.04
4 Channel bank sides, trees	0.042
5 Channel bank sides, scrub	0.04
6 Bankfull, trees	0.05
7 Bankfull, scrub	0.05
8 Bankfull, grass	0.045
9 Floodplain, grass	0.045
10 Floodplain, woodland/carr	0.055

Table 9.1

Specific flow results with variations in Manning's n for bankfull conditions

Node	Original	Specific flow ($\text{ft}^2 \text{ s}^{-1}$)						
		$n_9 = 0.1$	$n_8 = 0.1$	$n_3 = 0.1$	$n_1 = n_2 = 0.1$	$n_3 = n_4 = n_5 = 0.1$	$n_6 = n_7 = n_8 = 0.1$	$n_9 = n_{10} = 0.1$
1	0.30	0.278	0.360	0.342	0.361	0.382	0.360	0.277
2	1.215	1.051	1.188	1.395	1.346	1.395	1.188	1.049
3	4.033	4.824	3.089	3.425	3.787	3.425	3.089	4.824
4	17.252	19.735	14.403	13.173	13.589	13.174	14.403	19.735
5	18.202	21.086	15.591	13.382	14.139	13.382	15.590	21.086
6	4.248	5.190	3.406	3.289	3.961	3.289	3.406	5.190
7	1.156	0.955	1.210	1.333	1.289	1.333	1.210	0.955
8	0.179	0.174	0.226	0.239	0.227	0.239	0.226	0.173
9	0.097	0.103	0.126	0.133	0.125	0.133	0.126	0.103

Table 9.2
Specific flow results with variations in Manning's n for 1 in 10 year peak conditions

Node	Original	Specific flow ($\text{ft}^2 \text{s}^{-1}$)		
		$n_3 = 0.1$	$n_8 = 0.1$	$n_9 = 10$
1	2.551	2.708	2.700	2.478
2	4.940	4.931	4.419	4.702
3	9.122	7.389	6.877	11.429
4	24.336	18.451	20.019	29.759
5	24.888	18.314	20.942	30.778
6	9.104	7.022	7.157	11.450
7	4.735	4.807	4.526	4.245
8	1.856	2.003	2.012	1.875
9	1.304	1.421	1.431	1.414

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In each of the simulations documented in Tables 9.1 and 9.2, one or more of the element type Manning's n values were increased to 0.1. The simulations documented show not surprisingly that changing the n value in the most frequently occurring element groups has the most effect. Table 9.2, therefore, reports only the three most important element groups. Both Table 9.1 and 9.2 show that the solution behaves logically, with increasing n in a particular element type causing smaller specific flows in that group, and larger specific flows in other groups. Table 9.1 shows that variation in element type 3 (grassed channel sides) causes the greatest impact, both in the channel and on the floodplain. Channel element specific flow is decreased by an average 78%, whilst floodplain flows are increased by 123%. Comparison of Tables 9.1 and 9.2 show that the 1 in 10 year specific flows are not as sensitive to Manning's n variations as the bankfull results. Nevertheless, raising the n value of element type 3 is still the most effective way of reducing channel flows. These results illustrate that fairly radical adjustments in the Manning's n coefficients can generate changes in the specific flow of up to 25%. Manning's ' n ' is therefore a powerful tool in calibrating the reach and, in particular, is useful for apportioning the correct volumes of flow to the floodplain and main channel.

It is important to consider the physical realism of calibrating utilising the Manning's n coefficient. The variations shown in Tables 9.1 and 9.2 involved raising the coefficient to 0.1, a value corresponding to dense brush or a heavy stand of timber, according to Chow's (1959) tables. Such adjustments cannot, therefore, be justified physically. Instead the Manning's n values used in the RMA-2V scheme should be considered as belonging to a separate roughness scheme.

9.3.2 Hydrograph Simulation of the River Culm

The next stage in the calibration procedure was to attempt a

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dynamic simulation and compare predicted and observed hydrographs. Figure 9.9 shows the observed inflow discharge hydrograph and observed outflow stage hydrograph for an event commencing in January 1984. The figure shows, in a double peaked hydrograph, with the first peak having a recurrence interval of approximately 1 year, and the second peak 10 years. It is important to note the very attenuated nature of both the peaks, and to consider that the bankfull discharge at the downstream gauge is $30 \text{ m}^3 \text{ s}^{-1}$.

Figure 9.10 illustrates a first pass at simulating this event, using parameter values estimated in the field. Figure 9.10 shows that a simulated stage hydrograph barely attenuates either of the flood waves, and consequently predicted stage elevations are too high. If this stage hydrograph is converted to a discharge hydrograph, the discrepancy between the predicted and observed is exacerbated due to the power function rating relation.

There are two important issues raised by this hydrograph. The first and more significant for this scale of application is that the "smoothing" or loss of topographic resolution in the mesh may have created a system that is too efficient. Storage areas on the floodplain and meander bends have both been lost to meet the stability criteria of the model. The second issue is that of field data accuracy, the error band associated with field data, and its significance for RMA-2V simulation. RMA-2 requires a downstream boundary condition, usually a rating relationship. The reliability of out-of-bank rating relations, however, is relatively small. If the observed outflow stage hydrograph is considered, for example, is the attenuation of the inflow hydrograph a product of the topography of the reach, or is there a problem with the catchment of floodplain flows by the downstream stage recorder. A degree of attenuation can be introduced into the RMA-2V prediction by increasing the Manning's n roughness coefficients. Figure 9.10 shows the results of increasing the floodplain roughness parameter and the impact of increasing all the roughness value for all element types. Although

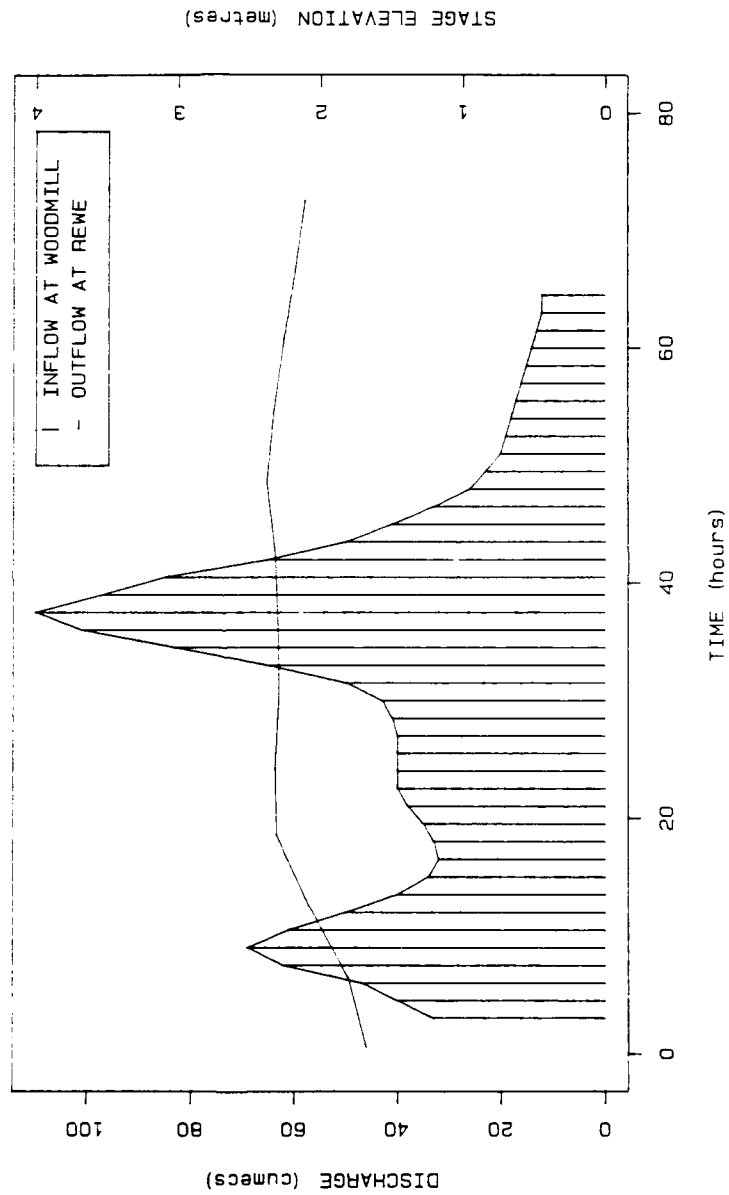


Figure 9.9
Comparison of observed inflow and outflow hydrographs
at Rewe for a 1 in 1 + 1 in 10 year event

REWE STORM1: 1 in 1 year + 1 in 10 year event

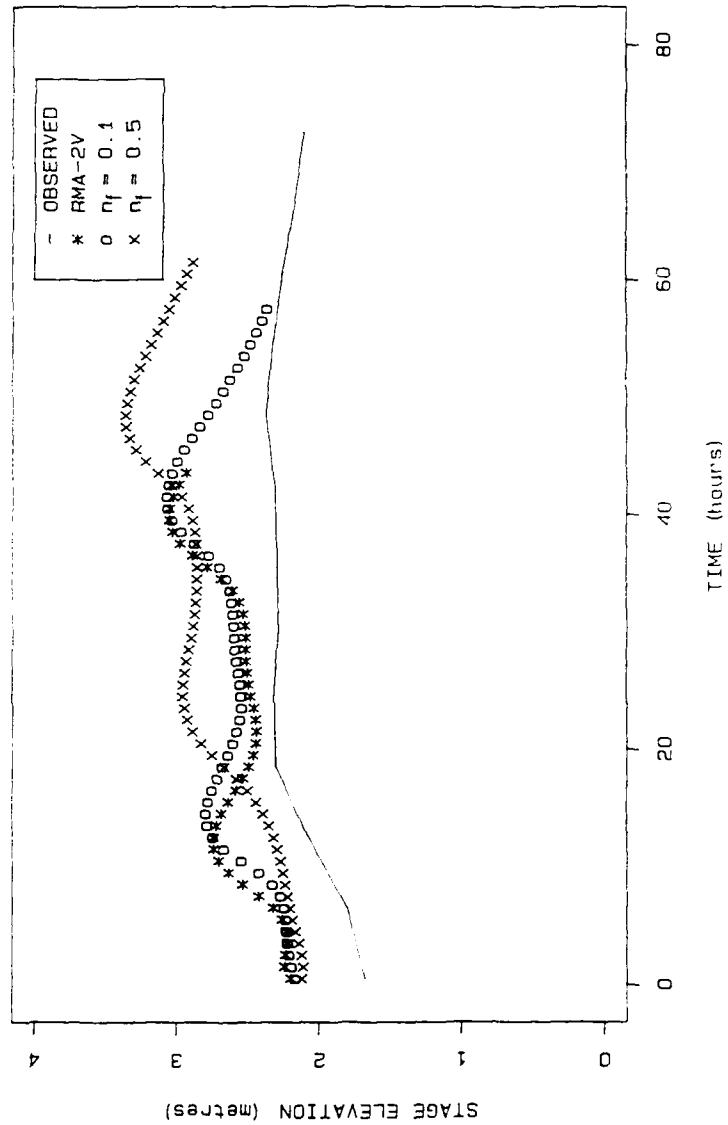


Figure 9.10
Comparison of RMA-2V predictions with varying floodplain roughnesses
at Rewe for a 1 in 1 + 1 in 10 year event

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the attenuation of the two flood waves is certainly increased, the discrepancy in stage predictions is not improved. This suggests that there may be a problem with both the topographic resolution and field data accuracy.

9.4 Implications for Ungauged Flood Forecasting

The initial results from the application of RMA-2V to the River Fulda reach highlight the issue of topographic representation. Generation of stable initial conditions for the reach necessitated a "smoothing" of the mesh and hence a loss of topographic resolution. During dynamic simulation, however, a lack of field data accuracy was identified with problems with the downstream boundary conditions.

From the ungauged perspective, the River Culm application has shown that it is relatively easy to provide enough topographic resolution to create stable initial conditions in a complex floodplain environment at this large scale. The important issue is, however, if the smoothing of the mesh generates a loss of attenuation in the system, can this be overcome by parameter calibration without contravening the objective of improved spatial and temporal inundation prediction. This is an issue which requires further investigation.

9.5 Conclusions

Application of RMA-2V to the River Culm has explored the issue of topographic resolution and a series of guidelines for generation at this scale have been generated. Although the initial objective was to investigate the accuracy of the spatial and temporal prediction of inundation, more fundamental issues have been raised.

Several important limitations of the topographic capabilities of RMA-2V have been identified but, clearly, further work is required.

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Conclusions

The aim of this report has been to investigate the issues affecting flood forecasting in ungauged catchments. An ungauged catchment has been defined as one for which the only available data set consists of precipitation data, a topographic map, and a soil classificatory map. In particular, no historical streamflow data are assumed to be available.

The application of a model to an ungauged catchment is often seen as the ultimate test of the ability of the model to accurately predict the behaviour of that catchment. Despite this challenge, flood forecasting in ungauged catchments has remained a relatively uninvestigated part of hydrological modelling.

It is proposed in this report that the lack of interest in ungauged flood forecasting is a side-effect of the general philosophy that has been driving hydrological modelling for the past twenty-five years. This philosophy is based upon the assumption that the predictive performance of hydrological models will improve when the physical representation of the processes in the catchment are improved. Developments in models have therefore been conceptual with increasing spatial or temporal resolution being incorporated in the modelling of the processes. The SHE model (see, for example, Bathurst, 1986), epitomises this philosophy as it incorporates the most highly developed distributed physically-based routines presently available.

The assumption that increasing the physical representation of the processes in the catchment will increase the predictive performance of the model, is not borne out in the studies comparing different model types undertaken, for example, by Loague and Freeze

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(1985). These comparative studies have shown that even when the state-of-the-art models are supplied with the extensive data sets and the experienced operators they require, their predictive accuracy is not significantly better than their simpler counterparts. Further, in the ungauged application these state-of-the-art models are inappropriate.

It is accepted that this physically-based conceptual philosophy must continue if our understanding of the catchment processes is to improve, and that its role as a research tool is vitally important. However, it is proposed that until the predictive performance of state-of-the-art models can be improved, perhaps with the incorporation of further physical representation of the processes, then another approach to catchment modelling is required.

This report shows there is a need to develop models specifically from the perspective of the potential application and operator. The application of hydrological models to ungauged catchments provides an ideal opportunity to investigate modelling from this perspective. In addition, the review of models suitable for application to ungauged catchments has shown that many have been evaluated at a late stage in the model development programme and consequently are conceptually unsuitable for ungauged applications. This report, therefore, has developed a model specifically for ungauged catchment applications and has attempted to consider the needs of the potential operator of the model during its development.

This report has identified four issues that are considered to be important to forecasting in ungauged catchments and may be of relevance to hydrological modelling in other areas. These issues are:

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- 1) the impact of model complexity on model performance
- 2) how composite modelling structures could develop model diversity whilst retaining model portability
- 3) how techniques utilised in hydraulics could be applied to improve hydrologic modelling
- 4) the development of validation strategies that are thorough and flexible

These four issues have been investigated through the selection, development and validation of an ungauged flood forecasting model, namely MILHY3. MILHY3 has been developed specifically for application to ungauged catchments and in addition it has been developed using relatively simple techniques, making it easy to apply for the inexperienced operator. The objective of this report has been to investigate the four issues identified above through the development and validation of MILHY3.

The rest of this chapter is divided into two sections. The first summarises the results of the development of MILHY3 and sets out the specifications for the composite structure proposed. In the second section, the potential for the further development of MILHY3 is identified and the implications of the results for the other areas of hydrology are discussed.

10.1 Specifications of MILHY3

The objective of this report has been to identify and develop and validate a model suitable for application to ungauged catchments based on a composite modelling structure. A review of MILHY3 showed that the model has a flexible structure and has been developed specifically for application to ungauged catchments.

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10.1.1 Model Complexity versus Model Performance

This report concludes that there is a complex relationship between model complexity and model performance.

The introduction of a composite modelling structure consisting of process modules at varying levels of resolution and complexity has allowed the exploration of this relationship. Results from the comparison of the predictive performance of the infiltration algorithm and the curve number routine in large catchments, a scale previously unexplored, are reported in Chapter 7. The results of the investigation showed that the predictive performance of the infiltration algorithm was superior to the performance of the simpler Curve Number routine. However, in the larger River Fulda catchment, the analysis showed that this improvement was only significant if the precipitation event had a complex temporal distribution. In Storm 1, the 1 in 10 year event, the precipitation pattern was simple with the majority of the rainfall falling at the beginning of the event. The improvement in the predictive accuracy of the infiltration algorithm over the Curve Number routine was therefore small, and not considered to be significant. In contrast, the precipitation pattern of Storm 3, the 1 in 1.5 year event, was complex with rainfall falling in two peaks, with a period of very low intensity rainfall between the two peaks. The improvement in the predicted hydrograph achieved by the infiltration algorithm was still evident and significant at the outflow of the reach at the Bad Hersfeld station.

The time taken to initiate and simulate a storm event using the infiltration algorithm for a catchment the size of the River Fulda basin, which is approximately 2500 km^2 , is significantly greater than the time taken by the Curve Number. In the case of Storm 1, for example, the infiltration algorithm version of MILHY3 took approximately nine hours of CPU, whilst the curve number routine took only about one hour of CPU on the SUN workstation.

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The results of the sensitivity analysis suggest that for large catchments the infiltration algorithm need only be invoked for complex storm events.

These results show that there is not a simple relationship between model complexity and model performance. The predictive performance of the infiltration algorithm is better than the performance of the simpler Curve Number routine but this difference in performance is only significant in large catchments for storm events with a complex rainfall distribution. Given the extra time taken to set up the data set required by the infiltration algorithm and the larger computational demands of the algorithm, the operator of the model must consider the size of the subcatchments being utilised, the complexity of the storm event and the accuracy required in the outflow hydrograph.

10.1.2 The Role of a Composite Modelling Structure

This report concludes that the utilization of a composite modelling structure allows model diversity to be developed whilst model portability is maintained.

It is proposed that a composite modelling structure could potentially close the split between model developers and operators by providing a flexible model structure into which new components or modules could be slotted. The perspective of the composite model would be biased towards the model operators such that a series of guidelines could be developed to help the operator select the most appropriate module combination for a particular application. It is envisaged that such a composite structure would incorporate modules simulating the same processes at different levels of spatial or temporal resolution or complexity.

The results of this report have shown that a composite modelling structure is a viable option. The results summarised

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above of the relative performance of the two runoff generation routines has shown that simpler modules have an important role and should not be discarded when more complex techniques become available. The results of the evaluation of the new momentum exchange and multiple routing routines, reported in Chapter 7, also illustrate that techniques drawn from hydraulics can be incorporated in a hydrologic composite structure.

The flexibility of a composite structure has been illustrated in the evaluation programme reported in Chapter 7. The model has also been developed in such a way as to maintain the portability of the scheme. The new modules required no additional data and the extra computational requirements are minimal in comparison with the computational requirements of the infiltration algorithm. The potential for the further development of the composite structure and the implications this would make on the portability of the scheme are discussed in Section 10.2.

10.1.3 Hydraulic versus Hydrologic Modelling

This report concludes that the concepts and techniques developed in hydraulics are appropriate for inclusion in hydrologic catchment models, and that the traditional distinction between hydraulics and hydrology, based on scale, is no longer valid.

The need to investigate methods of incorporating the cross-sectional and plan geometrical effects of two-stage flows has been identified. Review of current modelling capabilities has shown that this is an area that has not previously been incorporated in hydrological catchment models. Investigations into the behaviour of two-stage flows have almost exclusively been carried out in a hydraulic approach. This report has aimed to investigate these hydraulic approaches and see if they are appropriate for applications to hydrologic models.

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The behaviour of flow in two-stage flow was investigated and the review of the literature highlighted the importance of the effects of momentum exchange between the main channel and floodplain flow segments. The momentum exchange was shown to have significant effects on the velocity of flow in the two flow components and to affect the conveyance capacity of the whole cross-section. The implications of this exchange on the prediction of discharge from the reach were, however, not clear. In addition, the relative importance of momentum exchange in comparison with the effects of boundary friction and the downstream effects of two-stage flow were not assessed. The results of the sensitivity analysis of the Ervine and Ellis scheme isolated the effects of boundary friction and the importance of the difference between the length in the downstream routing pathways of the floodplain and more sinuous main channel flows.

The development of a momentum exchange module was investigated in Chapter 4, by firstly exploring the process of momentum exchange. Flume investigations showed that the most useful concept is to consider the exchange of momentum to be taking place across an interface between the main channel and floodplain. The extent of the exchange can then be computed by calculating the apparent shear stresses that would occur if the interface were a real boundary. This concept of an interface and the use of apparent shear stresses is widely used in hydraulic analysis to compare computed and observed discharge and velocity values, but this approach has not been incorporated into any hydrological models.

In hydraulic analyses a measured shear stress distribution is used to estimate the position of the interface and the stresses on the interface. In a hydrologic approach this is not possible and some estimate must be made of both the position of the interface and the stresses upon it. Analysis of these assumptions in comparison with flume studies has been attempted by many authors, for example Wormleaton et al. (1982), and Knight and Demetriou (1983), but no

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one clear technique has been identified. In addition, the flume investigations have concentrated on river reaches where the floodplain width is relatively small and the boundary roughnesses of the floodplain are small (see Table 4.1).

In order to investigate a variety of floodplain widths and boundary roughnesses, four interface position and stress combinations were incorporated into the momentum exchange module. These four techniques, tabulated in Table 4.2, include vertical and diagonal interfaces with an apparent shear stress ratio of 1 and 0. The analysis of the effects of the four techniques on the prediction of the rating curve showed that the techniques which introduced the most exchange reduced the capacity of the cross-section to the greatest degree. The longer the interface or higher the apparent shear stress ratio, the greater the amount of momentum exchange incorporated and hence the more turbulent friction incorporated.

The development and incorporation of a multiple routing module was investigated in Chapter 5. This module incorporated the effects of the short-circuiting of floodplain flows around the sinuous main channel flows. Analysis of the alternative methods utilised shows that the only techniques that had been previously used were either (1) the use of empirical adjustments, or (2) the replacement of the scheme with a hydraulically-based St. Venant scheme developed by Fread (1976).

The results of the inclusion of multiple routing reaches showed that the impact of the routine was greatest when the depth of water on the floodplain was small, that is when the depth ratio (floodplain depth : main channel depth) was less than 0.4 but more than 0.1. At this range of inundation depths, the multiple routing routine significantly reduces the error in the prediction of the peak discharge. In comparison with MILHY2, the error in the prediction of peak discharge is halved. The results also showed that reducing the length of the floodplain routing length to mimic

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the effects of the "short-circuiting", was only significant when the floodplain length was approximately 30% shorter than the main channel length. In these circumstances, the attenuation of the floodwave is decreased and a higher peak discharge is predicted.

The momentum exchange and multiple routing routines have been successful, therefore, in incorporating their respective effects of turbulent exchange and the short-circuiting of flow. Both modules have been shown to improve the predictive accuracy of the model and in addition, by removing the effects of the two processes from the selection of the Manning's n coefficient, have made the selection of the most appropriate value a simpler task.

The success of these two modules in incorporating the effects of two different processes has shown the portability of the concepts of traditionally hydraulic approaches to hydrologic modelling. The limitations posed by the ungauged nature of the catchment modelling have been shown not to restrict this portability.

Application of hydraulic techniques to a hydrologic model are considered in Chapters 8 and 9, where the inclusion of RMA-2V as a module of MILHY3 is considered. RMA-2V has been shown to be capable of application not only to the scale of reach required for a hydrologic approach but also to meet the limitation of an ungauged hydrologic approach. The operational guidelines for application of RMA-2V to such large scale reaches are summarized in Table 9.3.

The analysis of the relative performance of RMA-2V as a module in a catchment model is reported in section 8.5. This section concludes that the impact on the accuracy of the outflow hydrograph of the inclusion of RMA-2V in the lower reaches of the River Fulda is less than the impact of the inclusion of the infiltration algorithm. The inclusion of RMA-2V is therefore only recommended if more detailed information is required on the extent or behaviour of the floodplain inundation.

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10.1.4 Model Evaluation and Validation Strategies

This report proposes that optimization techniques provide a viable alternative to traditional factor perturbation analysis as part of a model validation strategy. It is also proposed that state-of-the-art process models could be used to provide "ground-truth" conditions for the evaluation of simpler models, and this evaluation technique is potentially most useful in the prediction of extreme events.

The introduction of a composite structure generates variability in the structure of the model in addition to the existing parameter variability within the process modules. A sensitivity analysis must assess, therefore, not only the impact of changes in the module's parameters but also the impact of different module combinations, and the interaction between these two sorts of variability. Given the large number of variables involved in the composite structure, alternative methods of undertaking a sensitivity analysis were examined. The potential for using optimization techniques as part of a sensitivity analysis was investigated. Optimization techniques have not been utilized in this way before and consequently the application of the technique was rather exploratory. The results from the investigation were promising, provided that the interpretive skills necessary to understand the results files can be established. The potential for the further development of the optimization technique is discussed in section 10.2.

The aim of the optimization technique, in addition to investigating the utility of the technique, was to investigate the sensitivity of the predicted hydrograph to variation in five key parameters: Manning's n coefficient for the floodplain and the main channel, the longitudinal slope for the floodplain and main channel, and the floodplain routing reach length. The results showed that when the longitudinal slopes are greater than 1×10^{-2} , then the

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slope is the dominant parameter controlling the shape of the simulated hydrograph. When the slopes are less steep then the Manning's n coefficient is the dominant parameter.

The optimization results also showed that when both the momentum exchange and multiple routing routines are utilised, then the variation in the predicted peak discharge is as great as variation generated by variation in the Manning's n coefficient. This suggests that the incorporation of the momentum exchange and multiple routing routines is significant in comparison with the effect of boundary friction. This result, therefore, supports the inclusion of the momentum exchange and multiple routing routines in the MILHY3 model.

Analysis of the momentum exchange and multiple routing routines showed that the relationship between them is an extremely complex one which depends on the exact nature of the storm event. Despite the effects of the momentum exchange routine on the predicted rating curve, the results reported in Chapter 7 show that the routine has no impact on the predicted hydrograph when applied without the multiple routing routine. When the momentum exchange and multiple routing routine were applied, they had an impact which was visible on the predicted hydrograph and this impact was different to the impact of the application of just the multiple routing routine. The effect of the momentum exchange routine is to make changes in the rating curve of a reach. These changes only make significant differences to the hydrograph if the multiple routing routine is also applied.

The impact of the multiple routing routing, as reported in Chapter 5, is dependent on the proportion of the total flow that is contained on the floodplain. If the floodplain inundation depth is very small, then the travel time of the floodplain flow will be large, for example a hundred hours. More significant in low floodplain inundation depths is the travel time of the main channel

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which is reduced in comparison with the composite travel time where the average time for the floodplain and main channel is used. This means that in low floodplain inundation conditions the attenuation of the floodwave is reduced in comparison to the composite approach. Where floodplain inundation depths are larger and floodplain flows contribute a significant proportion of the total discharge, a proportion of around 15%, then the attenuation of the floodwave is increased as there are effectively two floodwaves, one from the main channel and one from the floodplain. When the proportion of flow in the floodplain exceeds 15%, then the joint application of the momentum exchange and multiple routing routines improves the accuracy of the predicted hydrograph. It is suggested, therefore, that the potential operator of the MILHY3 scheme uses both the momentum exchange and multiple routing routines for out-of-bank events.

A third part of the model validation and evaluation programme, is reported in Chapter 8, and investigates the feasibility of using RMA-2V to extend the record of "observed" extreme events, in order to further validate the performance of MILHY3. The application of RMA-2V to the River Fulda catchment is the first application of a hydrodynamic finite-element model to this scale of problem.

Previous finite-element applications to two-stage river reaches have been limited to scales of around 2km. This is also the first application attempted with a limited data set. Information in this instance is limited to a topographic map and upstream and downstream cross-sections and rating curves; in particular, no intermediate reach data or velocity vector data are available. This is also one of the first applications of version 4 of RMA-2V, which incorporates the new marsh elements with pseudo-porosity effect. The marsh elements allow elements to enter and leave the computation smoothly and improve the stability and accuracy of the predicted extent of inundation.

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The results of the application of RMA-2V to the River Fulda showed that the model could be applied to river reaches of a scale around 20 km, and that there seemed to be no inherent conceptual problems in such an application.

In Chapter 9 this theme is pursued from a more practical viewpoint with application of RMA-2V to the River Culm. The more exhaustive data set available for the River Culm and the more complex terrain environment of the reach, enabled a series of guidelines and limitations to be established for applications of RMA-2V to this type and scale of problem. Importantly, the River Culm highlighted the issue of the effect of topographic resolution on computational stability.

The results from the River Fulda and River Culm applications were promising in that reasonable predictions of both the discharge hydrograph and inundation distributions can be made with very limited topographic data. The results showed that RMA-2V can be used to extend "observed" data sets, and that incorporation of RMA-2V into a hydrological catchment model suite is viable and worthy of further investigation.

The guidelines generated for the application of MILHY3 have been summarized in Table 10.1. These guidelines represent the results of work undertaken in this report and, therefore, are appropriate for applications to large catchments, that is 1000 - 2500 km² (385 - 1000 sq.mi.).

10.2 Further Development of MILHY3 and Future Research Needs

This section considers the further development of MILHY3 and the composite modelling structure. The implications of the specifications already achieved, and the potential implications of further developments on other areas of hydrological modelling, will

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Table 10.1
Guidelines for the Application of MILHY3
to Large Catchments

Catchment Subdivision

Subcatchments size $< 60 \text{ km}^2$

Rainfall

Frontal type storms - consider use of radar data for
subcatchment $> 145 \text{ km}^2$.

For convective storms - consider use of radar data.

Soils Classification

Concentrate on the accurate representation of soils groups
with low hydraulic conductivities.

Runoff Generation

For complex precipitation patterns use the infiltration algorithm,
for simpler rainfall distributions consider advantages of
curve number routine.

Channel Geometry

For out-of-bank events - use both the momentum exchange and
multiple routing routines.

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be briefly discussed.

Figure 10.2 shows the components of MILHY3 that have developed and validated in this paper. Throughout this report, alternatives have been discarded for a variety of reasons. Certain processes, such as the secondary current system, were not included in the final specifications of MILHY3 because other processes were considered to be of more significance. Alternative methods of modelling processes were excluded on the grounds that they were difficult to incorporate into the MILHY scheme. This was true for the modelling of multiple routing routines, where the simplest approach was incorporated so that the relative importance of the process could be considered.

If the out-of-bank flood forecasting of MILHY3 is to be improved, then the results of this report suggest several steps that could be taken to achieve this. These steps include:

- 1) The upgrading of the handling of the multiple routing routine. The results of this report have shown that this routine has a significant effect on improving the predictive accuracy of the outflow hydrograph. Several alternatives have been suggested in Chapter 5.
- 2) Linking of RMA-2V as a module in the MILHY3 model suite. The results of applications of RMA-2V reported in Chapters 8 and 9, suggest that RMA-2V can provide more accurate discharge and inundation predictions in ungauged environments. The operational guidelines developed suggest this may be achieved in a limited time frame.

If the performance of MILHY3 is to improve for all types of applications, both in-bank and out-of-bank, then this report has identified several other steps that could be taken. These include:

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- 1) An investigation into the relationships between the resolution of subcatchment area required for the accurate prediction of the time to peak discharge. Potentially, this may require the introduction of the modelling of the overland routing of the runoff to the channel.
- 2) Investigation of the packages available to incorporate the relationship between the resolution of rainfall required for particular storm characteristics and subcatchment sizes.
- 3) Assessment of the importance of a secondary current module for sinuous channels for in-bank events.
- 4) More accurate representation of the effects of boundary friction, to include the stage/roughness relationships and an assessment of the spatial resolution of boundary roughness classification required.

The validation strategy developed in this report has proposed the utilization of optimization techniques and the application of RMA-2V for the extension of the record of "observed" events. New validation techniques need to be considered if the composite modelling structure is to be thoroughly evaluated and the gap between model development and operation to be bridged.

The optimization techniques could be used by any model that allows the comparison of a simulated and observed hydrograph or similar output. The volume of output from the application of the optimization technique in this report and the interpretive skills required, however, suggest that the optimization technique would not be appropriate if a larger number of parameters needed to be assessed. Optimization techniques would not be suitable, therefore, for assessing the sensitivity of physically-based distributed models which have a very large number of parameters. The optimization technique was used in this report to assess the sensitivity of the

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outflow hydrograph to variability in five parameters. Given the volume of output from this analysis, an upper limit of the testing of ten parameters sensitivities would seem an appropriate guideline. For simple models, or as in this case as part of an analysis incorporating many different approaches, then optimization is a viable and promising alternative.

The application of RMA-2V to the River Fulda and River Culm was also a new approach to the validation of MILHY3. The aim of the approach was to utilise the RMA-2V to extend the record of storm events, providing a ground-truth against which MILHY3 could be validated. The need to extend the record of extreme events stems from the inherent difficulties of collecting data for events which are either impossible to measure or occur too infrequently.

The results of the application of RMA-2V are promising enough for it to be suggested that this approach could be utilised in other processes of hydrology where extreme events are simulated. The approach relies on the availability of a state-of-the-art model that has been validated and has a good predictive performance. This precludes, therefore, catchment models which are still basically research tools. The approach is, therefore, most suitable for the validation of certain process modules within catchment models for which state-of-the-art models exist. This could include, for example, the validation of catchment stability models against engineering stability models, or the validation of catchment throughflow modules on detailed hillslope models.

The potential of a composite modelling structure for the bridging of the gap between model developers and operators has been stressed for ungauged flood forecasting. The utility of incorporating concepts and techniques from the field of hydraulics has also been highlighted. The potential flexibility of the composite modelling structure, however, does not stop with hydraulics. Other engineering disciplines such as soil mechanics

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for the modelling of river bank stability could also be potentially incorporated.

This report has shown that a composite modelling structure can support a multi-disciplinary approach. In addition, the application of concepts and techniques from the field of hydraulics has shown that the incorporation of engineering techniques is feasible and appropriate. With the application of an expert system, a composite modelling structure encompassing the disciplines of hydrology and engineering may help to bridge the gap between model developers and model operators.

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