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Pumped catchments - Guide for hydrology and hydraulics

Project: SC090006

Flood and Coastal Erosion Risk Management Research and Development Programme

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Miranda Kavanagh

Director of Evidence

Executive summary

This guide is meant for practitioners in the operating authorities responsible for flood risk management in catchments where the drainage relies upon artificial pumping, particularly Internal Drainage Boards (IDBs). The guide is primarily aimed at IDB technical staff and engineers, including their consultants, but it may also help other interested readers such as IDB members, other professionals and the public to develop a basic understanding of technical issues related to pumped catchments. The guide focuses primarily on England and Wales, but many of the issues and methods are generic. The guide was developed for use on pumped catchments, but much of the text also applies to flat low-lying catchments. The guide does not cover highland or urban catchments, but it does give guidance on dealing with highland or urban elements of pumped catchments.

The guide offers technical support for flood risk assessment in pumped catchments. It concentrates on methods for hydrological assessment, hydraulic analysis and pump capacity and operation. It is intended to offer good practice guidance to inform sound decision-making. It describes the methods available for each element of flood risk assessment, and clarifies how generic methods (such as *the Flood Estimation Handbook* FEH) can be applied to pumped catchments. For some tasks several methods are available, from basic to advanced. The guide describes these „tiered methods‟ and helps users decide which tier to use in which situation. This depends on aspects such as the availability of data, tools and resources and the physical situation. The choice of method also depends on the reason why the flood risk assessment is being carried out, or in other words, the management context.

This guide is an update of the existing guidance in this field, incorporating developments in methods, tools and approaches since 1994. In particular, the guide clarifies the use of the Flood Studies Report (FSR) rainfall-runoff method for pumped catchments and provides an improved approach for combining this hydrological input with hydraulic models.

While developing this guide, it became apparent that some elements would benefit from being tried out in practice by the IDBs. It would be useful if users‟ experiences could feed back into the document. All users are encouraged to contact the Environment Agency or the Association of Drainage Authorities (ADA) with their feedback.

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1 Introduction

1.1 Background

In low-lying areas of the UK, land drainage pumps are a core component of the overall flood risk system, and as such an understanding of the critical components and their functioning is essential to understanding and managing flood risk. Closely linked with this is the management context, which has traditionally involved the Environment Agency and the Internal Drainage Boards, but following the 2007 flooding, the Pitt Review, the Flood Risk Regulations (2009) and the Flood and Water Management Act (2010), local authorities also have important responsibilities. Much work was undertaken in the mid-1980s for the former Ministry of Agriculture, Fisheries and Food (MAFF) to develop best practice guidance for pumped catchments, which was published through the report *The Hydraulics and Hydrology of Pumped Drainage Systems - An Engineering Guide* (Samuels, 1994). The *Engineering Guide* has been used successfully since then, although its application has varied considerably.

Since 1994 there have been significant changes in methods but also in management driven by the change in policy from providing flood protection to that of flood risk management as set out in 2004 in the Department for Environment, Food and Rural Affairs (Defra) policy document *Making Space for Water*. This guide provides a technical update of the 1994 *Engineering Guide*, and places the technical management of pumped catchments into its current context.

1.2 Intended users and usage

This guide is meant for practitioners: the operating authorities that manage pumped catchments, particularly Internal Drainage Boards (IDBs). The intended users are primarily IDB technical staff and engineers, including their consultants, but other interested readers such as IDB members, other professionals and the public may also find the guide useful for developing a basic understanding of technical issues related to pumped catchments. The guide focuses primarily on England and Wales, but many of the issues and methods are generic.

The guide offers technical support for flood risk assessment in pumped catchments. It is not intended as a mandatory enforced manual but as good practice guidance. It describes the methods available for each element of flood risk assessment, and clarifies how generic methods (such as the *Flood Estimation Handbook* FEH) can be applied to pumped catchments. For some tasks various methods are available, from basic to advanced. The guide describes these „tiered methods" and helps users decide which tier to use in which situation. This depends on aspects such as the availability of data, tools and resources and the physical situation. The choice of method also depends on the reason why the flood risk assessment is being carried out, or in other words, the management context.

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1.3 Layout of the guide

The management of flood risks can be viewed as series of activities, each of which may involve supporting technical analysis and assessment. Thus, this guide is set up in a matrix structure, see Figure 1.1.

Chapter 2 describes the management context, shown on the left hand side of the matrix in Figure 1.1. It does not go into the detail of these topics, but refers to specific literature and guidance with a focus on the application for pumped catchments.

The principal content of this guide is in the remaining chapters, which discuss the technical methods for flood risk assessment, distinguishing the topics listed at the top of the matrix. Chapter 3 starts by setting out some general elements of good practice, and Chapters 4, 5 and 6 cover hydrology, hydraulics and pump capacity and operations respectively. Where relevant, the text refers between the chapters to explain how the management context can determine or influence the technical methods to be used.

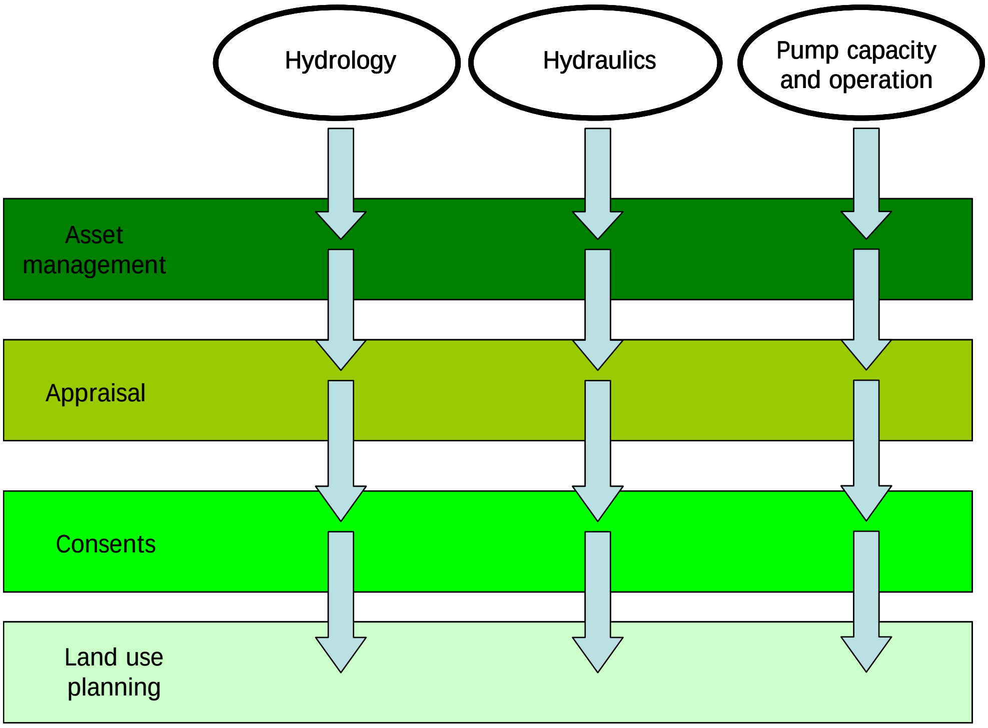


Figure 1.1: Matrix structure of the guide

2 Context for management of pumped catchments

2.1 Legal and policy framework

This section focuses on the roles and responsibilities of Internal Drainage Boards (IDBs) in flood risk management.

The Land Drainage Act of 1991 gives IDBs the duty to supervise all land drainage in their area and powers to do works for that purpose. The Floods and Water Management Act of 2010 identifies IDBs as risk management authorities. This gives them duties to act consistently with the local and national flood risk management strategies; to cooperate with other flood risk management authorities and provide information; and to contribute to sustainable development. IDBs remain responsible for flood risk management of the ordinary watercourses in their areas. The 2010 Act makes it possible for local authorities to delegate some of their functions to IDBs. In addition, Paragraph 36 of the Flood Risk Regulations (2009) gives the lead local flood authority a power to require information from an IDB relating to the lead local flood authority"s responsibilities under the 2009 Regulations.

2.2 Asset management

Pumped catchments consist of physical infrastructure assets which are critical to achieving their owners" business objectives and service delivery. This means that the concepts of asset management are applicable to pumped catchments. Publicly Available Specification 55 (PAS 55) is the best reference for generic concepts and guidance for asset management. Figure 2.1 shows some of the main concepts.

The policy and strategy of pumped catchment management is set by the IDB, who balances the different interests of their members, local communities, land use and available resources, within the IDB"s legal framework. Under the Flood and Water Management Act, this includes a duty to act consistently with the local and national flood risk management strategies.

Key practical elements of asset management are inspection, continuous assessment of system performance and planning of interventions within the resources available. The programmes for inspection and maintenance are set on a practical basis, using local judgement and experience to assess risks and prioritise activities accordingly.

The impact of maintenance, interventions and events is fed back to the IDB and this continuously informs the development of their management strategies.

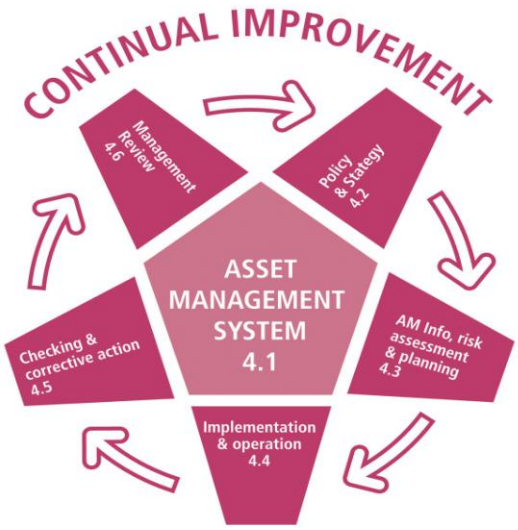


Figure 2.1 Elements of asset management systems (from BSI, 2004)

2.3 Project appraisal

The Environment Agency‟s Flood and Coastal Erosion Risk Management (FCERM) appraisal guidance (Environment Agency, 2010b) provides the general framework, which also applies to pumped catchments and their operators. Its Section 2 sets out a number of general principles, which are quoted here:

- Adopt a risk-based approach: take into account both the probability (likelihood) and the consequences (positive and negative impacts); see Section 3.1 of this guide for further background.
- Adopt a proportionate approach: the effort put into appraisals should be proportional to the amount of information needed to choose a preferred option, depending on the project‟s size and complexity; see Section 3.2 of this guide for further background.
- Work within the hierarchy of FCERM decision-making: appraisal should be undertaken within the hierarchy of policy and strategic directions, including shoreline management plans (SMP), catchment flood management plans (CFMP) and strategies (where they exist).
- Work with others throughout the appraisal process: this can concern individuals, a group of individuals, communities, organisations or political entities. It is critical to good appraisal and must be done from the start. This provides opportunities to establish common understanding and ownership of the problem, develop partnership working and meet multiple objectives.
- Integrate environmental assessment: this underpins sustainable solutions that take account of our natural and built environment and the intrinsic, social and economic benefits they afford.

The Appraisal Guidance makes a number of specific references to pumped catchments, in particular:

- dealing with pump reliability in determining benefits (see Section 6.5 of this guide);

- definition of „do nothing" and „sustain standard of service" in relation to pump operation;
- setting project boundaries where a pumped drainage system forms part of a river system and contains a main pumping station supported by smaller stations.

2.4 Land use planning and consents

The role of drainage authorities in land use planning is described in Planning Policy Statement 25: Development and Flood Risk. Local planning authorities have to consult the Environment Agency, and where relevant other drainage authorities, in a number of situations. This particularly concerns policies in their local development documents on flood risk management and in relation to areas at risk of flooding. This consultation is also needed on applications for development in flood risk areas.

The role of drainage authorities is to respond to these consultations and the associated strategic flood risk assessments (SFRA, accompanying local development documents) and flood risk assessments (FRA, accompanying development proposals). Drainage authorities must assess the impact of developments on flood risk within the system, and assess whether this is acceptable. The IDB"s systems will often be able to provide sustainable drainage on a catchment basis, including future development where needed.

On a smaller scale, drainage authorities have a role in consenting activities that influence run-off. In practice, they typically do this by changing the system locally to accommodate any increase in run-off, and charging a levy to cover associated costs.

These assessments of the impact of external changes require a review of the hydrology of the catchment (for example, changes in urban extent which influence run-off), although in practice this often does not require full hydrological analysis, as explained in Text box 4.1.

3 Good practice approach

3.1 General principles

The *Fluvial Design Guide* (Environment Agency, 2010a) sets out eight general principles for good design. At a broad level, these principles are also valid for the design and management of pumped catchments, which is why they are repeated in this guide in Text box 3.1. Section 1.4 of the *Fluvial Design Guide* provides further information on these principles.

Examples of these principles for catchments that rely on pumped drainage include:

- Adopting a **systems approach** by considering the rural and urban run-off components, interaction with tidal outfall conditions, the sizing and maintenance of channels, capacity and operation of pumps.
- Considering **performance-based risks** by assessing the potential for failure of pumps, blockage of culverts and intakes, failure of major flood banks, instability of batter or bank side slopes and impaired capacity of channels through seasonal vegetation.
- Considering the **full range of loading conditions** in the assessment of the management and operation of the system; the cost of operation is likely to be dominated by flows much less than the design standard but the consequences of above standard conditions need to be assessed to understand and mitigate residual risks. A practical example is also that different parts of a system could require different design standards, for example 1:10 per year for agricultural land and 1:100 per year for urban areas.

3.2 Tiered approaches

For most topics of analysis, there is a range of available methods, from simple rules of thumb to advanced mathematical modelling. The choice between these different tiers depends on a number of considerations. The general guiding principle is to use simple methods where possible, and complex detailed methods where needed.

More advanced methods are usually needed when the analysis needs to provide more detailed answers, for which simpler methods use assumptions. On the other hand, a word of warning is needed against the use of advanced methods where there is a lack of reliable information and data.

Sections 4.1 and 5.2 provide specific guidance on the choice of method for hydrology and hydraulic analysis.

Text box 3.1: The eight principles of fluvial design (from *Fluvial Design Guide*)

1. Fluvial design must be **sustainable.** It must aim to work with natural processes and meet the needs of the present without compromising the ability of future generations to meet their own needs. Consequently, all fluvial design work must aim to:

- avoid negative impacts to the river system and users of it;
- be efficient in its use of resources;
- maximise opportunities for win-win scenarios.

2. Fluvial design must consider all stages in the **lifecycle** of the intervention – not only its primary role during its operational life, but also the construction stage at the beginning, its operational and maintenance requirements, and the decommissioning stage at the end.

3. Fluvial design must include engagement with all **stakeholders** from the early stages of a project. This allows early identification of project opportunities and risks. It also helps to ensure that nothing is overlooked, reduces the risk of conflicts arising, and promotes „ownership" of the project, which may be important once the scheme is in operation.

4. Fluvial design must adopt a **systems approach**. It has to look at the complete river system insofar is it can be affected by, or may have an impact on, the proposed interventions. This includes potential interaction with surface drainage systems.

5. Fluvial design must be **performance-based**. It has to take account of the mechanisms that can cause failure of the assets to perform as intended. This is relevant for defence assets and their function to defend against flooding, but also for watercourses and their function to convey water. It is also relevant for other functions such as facilitating navigation or improving aquatic habitat.

6. Fluvial design must consider the **full range of loading conditions** that the asset is likely to meet in its design life. Traditionally the practice has been to adopt a design condition such as the one per cent annual probability flood and to focus exclusively on this. Such an approach is no longer acceptable and the designer must examine both lower flow conditions (which are much more likely to occur) and extreme floods beyond the design event, in order to reduce the risk of catastrophic failure and other adverse impacts.

7. Fluvial design must be **flexible** and **adaptable.** We cannot accurately predict the future, particularly in terms of global climatic change. Designs should therefore be flexible and adaptable so that changes can be made readily at a later date, if necessary, rather than fully designing now in an attempt to meet an uncertain future requirement.

8. Fluvial design must take account of the inherent **uncertainty** associated with natural events and our understanding of them. Designs should be **robust** and **resilient**, so that they provide the required level of service now and in the future.

4 Hydrology

4.1 Introduction

This chapter discusses methods to determine the flood flow for a chosen probability of exceedence in pumped catchments, and in flat low-lying catchments in general. This could be a design flood, or any other probability of exceedence for which the user needs to determine flood flows. This chapter replaces Section 2.2 of the MAFF *Engineering Guide* (Samuels, 1994).

The three main methods in current use are:

- use of gauged records;
- use of the Flood Studies Report (FSR) rainfall-runoff method as presented in the *Flood Estimation Handbook*, adapted to flat low-lying catchments („tailored FSR method‟);
- rule of thumb.

Where long records are available for the particular pumping station, gauged data will often provide the most reliable estimate of flood risk. Some back-adjustments may be required where the drain system or the installed capacity has changed during the period of record. In other cases, the tailored FSR method is recommended. A rule of thumb should be adopted only where detailed analysis of one or more pumped systems support its use locally, and then only for a first approximation, when very limited accuracy is acceptable.

More advanced methods may be possible in principle. However, these would require much more data than normally available (such as long-term sub-daily rainfall) and are therefore usually not applicable in practice. Other rainfall-runoff models might be considered where these have been shown (by calibration and validation) to represent low-lying catchments of similar character and climate. It should be recalled that *event-based* estimation of the *T*-year flood (such as the FSR rainfall-runoff method) requires more than just a rainfall-runoff model: assumptions are needed about the *design inputs* (to the rainfall-runoff model) to yield the desired *T*-year extreme runoff. If an alternative method is chosen that is based on *continuous simulation*, this limitation is not relevant. However, unless physically strongly based, such models may be better at simulating typical flows than representing extreme flows.

The following sections outline some considerations for the use of each of the three methods, with detailed discussion of the tailored FSR method.

Text box 4.1: Dealing with small changes to the catchment
When determining the impact of small changes to the catchment (as part of consenting or land use planning, see Chapter 2), there is often no need to carry out a full hydrological analysis. The manager of the system will use his local knowledge to estimate the impact and assess how the local system can accommodate this.

However, a number of small additions may end up as a problem to the whole catchment. In this case, it is recommended that the user develops a “catchment strategy”, determined by the methods in this guide, that sets out permissible development and how the catchment can adapt. This approach should also take into account the gradual changes caused by climate change.

4.2 Use of gauged records

If gauged records are available of the rainfall and run-off in a particular catchment, then these can be used to estimate the run-off. This involves the analysis of a long-term record of pumped quantities, if possible supported by water level data. The analysis requires basic statistical analysis, interpolating or extrapolating to the required probability of exceedence, similar to estimating the flood frequency for rivers by an analysis of a long series of gauged flood flows.

However, in practice the availability of gauging data and the length of gauged records is often limited. In addition, simple statistical analysis of rainfall and run-off will not provide an accurate result because it neglects the potential variability of storm characteristics and initial system state.

Another way to use gauged records is to analyse rainfall and run-off data for a number of flood events to calibrate a tailored FSR rainfall-runoff model. This practice is strongly recommended, as discussed in Section 4.3.2 and 4.3.6.

In practice the available data are often limited to anecdotal observations, which then become the only method possible for calibrating a tailored FSR rainfall-runoff model.

4.3 FSR method tailored to pumped catchments

4.3.1 Introduction

Volume 4 of the *Flood Estimation Handbook* provides complete and detailed guidance on using the *Flood Studies Report* (FSR) rainfall-runoff method to estimate flood flow.

It has been known for some time that this method is not fully applicable to pumped catchments due to their artificial nature. This guide advocates retaining broad use of the FSR rainfall-runoff method, adapted where needed to make it applicable to pumped catchments. Although the method was not developed for low-lying catchments, it is helpful that its so-called *Unit Hydrograph Losses* model has a simple structure that is relatively well understood. It also helps that the one method has been extensively applied in the UK.

Partial alternative approaches have been disseminated and used over recent years, but these have never been collated and published. This section aims to fill this gap by providing step-by-step guidance, with clear reference to the sections in FEH Volume 4.

The overall approach from FEH is valid for pumped catchments, but some elements should not be used, and other elements should only be used with caveats and conditions. Figure 4.1 below is a combination of Figures 1.1 and 3.3 from FEH Volume 4 and gives an overview of all the steps required to estimate the flood hydrograph for a given frequency. The colours of the boxes indicate how to find guidance for each step:

- Green: Use FEH.
- Orange: Use FEH but with caveats.
- Red: Do not use FEH.

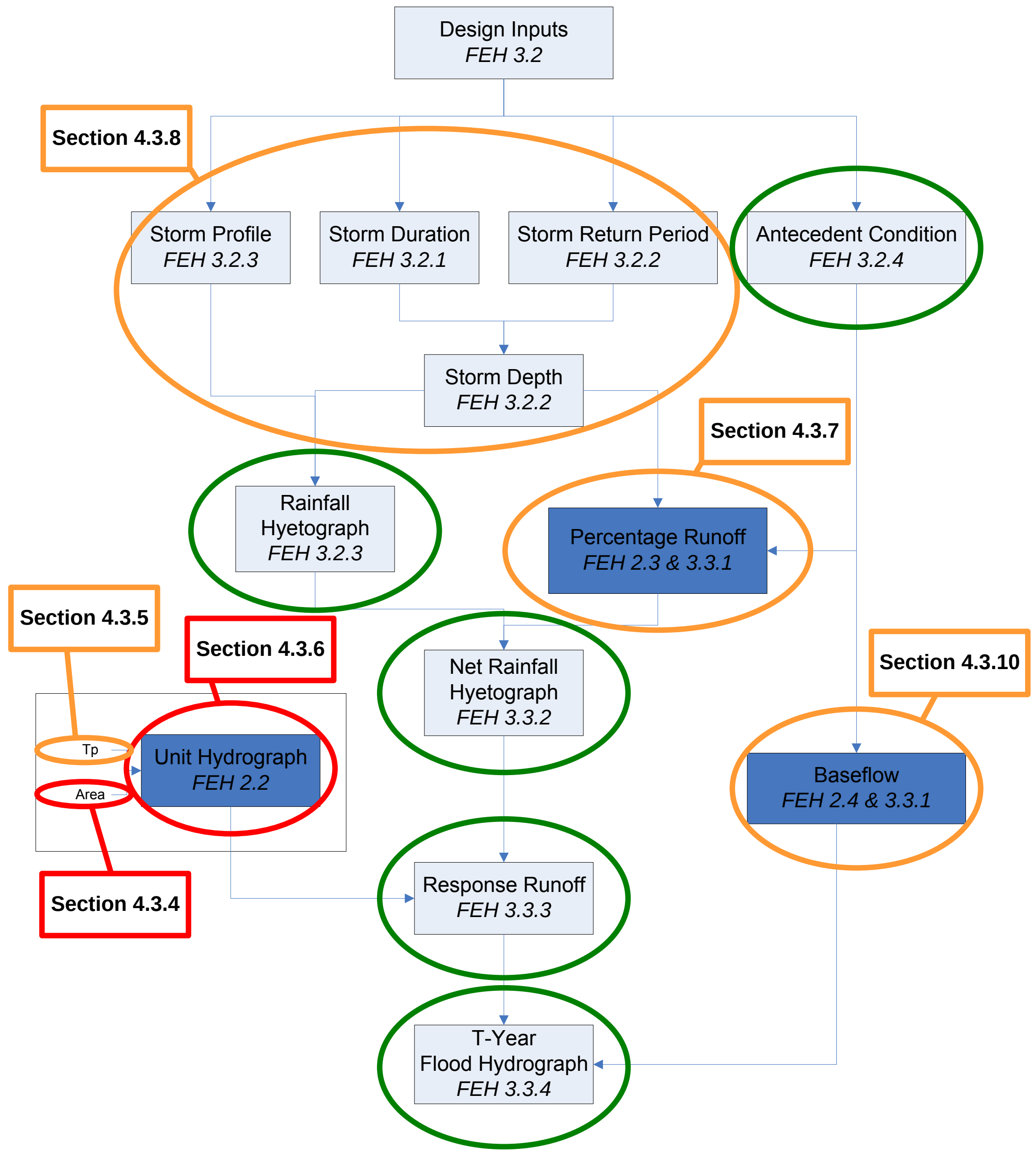


Figure 4.1 Process of determining flood run-off – use of FEH and this guide

4.3.2 General approach and principles

Section 1.1 of FEH Volume 4 gives an overview of the FSR rainfall-runoff method. In essence, the method converts a rainfall input to a flow output using a deterministic model of catchment response. This model is the Unit Hydrograph and Losses model, which has three parameters:

- Catchment response to rainfall (unit hydrograph time-to-peak Tp).
- Proportion of rainfall which directly contributes to flow in the river (percentage run-off).
- Quantity of flow in watercourse prior to the event (baseflow).

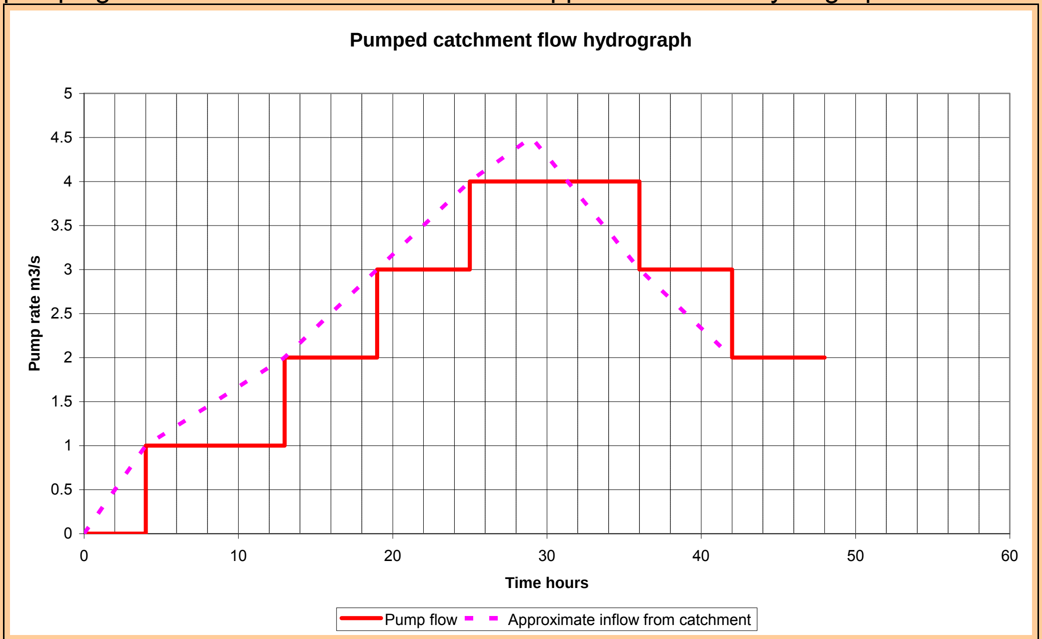
See Figure 4.1, where these three components of the methods are shown by the darker colour of their boxes. In FEH Volume 4, Chapter 2 gives guidance on how to develop the Unit Hydrograph and Losses model for a given catchment, while Chapter 3 describes how to complete and use this model to determine a flood hydrograph for a given rainfall event.

Throughout the FEH, its authors provide recommendations and advice about practical and sensible application of the methods. Two of these are repeated here as being particularly relevant for flood estimation for pumped catchments:

- The FSR rainfall-runoff method should not be used as a black box. The method was not developed originally for pumped catchments, which means it is especially important to use site-specific surveys, local judgement and specialist input where needed rather than following generic models.
- For this same reason, it is important to base model parameters on real flood events. This guide strongly recommends the analysis of events to derive parameters and validate models.

Text box 4.2: Use of pumping records to estimate the flow hydrograph

Many lowland pumping stations have a relatively long history. How the drain systems have behaved in the more extreme events that they have experienced is likely to be the best knowledge available. It would be good practice, if not done already, for IDBs and their consultants to identify about five occasions when the station has been most stretched or after which significant residential, commercial or agricultural losses have occurred. The list of dates involved might not be great, with particular events appearing in many lists. The relevance would be to look at the rainfall durations (ideally, the profiles also) of those events and to compare them with the design storm assumptions made in flood risk estimation. The graphic below (for a fictitious case) illustrates how pumping records could be translated to an approximate flow hydrograph.



4.3.3 Elements for which standard FSR/FEH should be used

Figure 4.1 shows that the approach from FEH Volume 4 should be used for the following elements:

- Antecedent condition: see FEH Section 3.2.3.
- Rainfall hyetograph and net rainfall hyetograph: see FEH Section 3.2.3.

- Response run-off: see FEH Section 3.3.3.
- T-year flood hydrograph: see FEH Section 3.3.4.

4.3.4 Catchment description

Catchment area (AREA)

An accurate assessment of the catchment area is essential for all hydrological studies. The area is an important factor in the development of the unit hydrograph as discussed in FEH Chapter 2.

The FEH CD-ROM provides this information, based on contour information and spot heights mapped at 1:50,000 scale. However, these data are neither recommended for, nor suitable for, use on low-lying catchments. The problem resides in the artificiality of low-lying pumped catchments.

The consolidated guidance is that, for low-lying catchments, the catchment boundary has to be determined based on drainage plans at (for example) 1:10,000 scale and knowledge of how the particular drainage system operates. This could be confirmed by site inspection and surveys.

Other descriptors

It is sometimes possible to approximate the true catchment area by summing/subtracting areas given by the FEH CD-ROM. It is then possible to arrive at catchment values of hydrological parameters such as (for example) *SAAR*, *SPRHOST* and *URBEXT* (as defined in FEH) by area-weighted calculation. Appendix A shows an example of these calculations.

In some applications, descriptors are sought for the false catchment that *intervenes* between one site on a river system and another. This is the area that does drain to the downstream site but does not drain to the upstream site. FEH Volume 5 Section 7.2 indicates methods that can help with this. Area-weighting suffices for many of the descriptors, including: *SAAR*, *SPRHOST* and *URBEXT*. However, such back-calculation does not work for the mean drainage-path length (*DPLBAR*) nor, on low-lying catchments, is it ideal for estimating mean drainage-path slope (*DPSBAR*).

Catchment descriptors will normally only be needed for highland and possibly urbanised sub-catchments. This is because the catchment descriptors method for determining time-to-peak (Tp) is not suitable for flat or low-lying rural parts of the catchment (see Section 4.3.5).

Urban extent (URBEXT)

Management of pumped catchments is concerned with run-off from, and flood risk to, property, in addition to the management of water levels for agriculture and the environment. The characterisation of urban development is therefore important.

A feature of assessing the degree of urbanisation from mapped data (paper or digital) is that assessments are scale-dependent. It is therefore important to assess urbanisation in a fashion consistent with the calculation method being used.

If the user becomes aware that maps fail to mark particular developments, a pragmatic approach is to adjust values – such as the FEH descriptor of urban extent (*URBEXT*) – based on how the specific map-format is known to deal with other developments of similar character.

The use of sub-catchments is discussed separately in Section 4.3.9.

4.3.5 Time-to-peak (Tp)

The time-to-peak is discussed in Section 2.2 of FEH Volume 4. Some of the proposed approaches are fully applicable, but there are some caveats as follows.

As for many of the other hydrological parameters, the general guidance for pumped catchments is to estimate the time-to-peak from gauged data if possible, as described in FEH Section 2.2.2.

FEH Section 2.2.3 describes the approach to develop Tp from catchment lag. This approach is applicable in principle to pumped catchments as well.

FEH Section 2.2.4 describes the approach to determine Tp from catchment descriptors. This approach requires parameters on the slope of the catchment, which means that it is not suitable for use on flat catchments. It is suitable for any sub-catchments that do have a natural slope; see Section 4.3.9 on how to select and use sub-catchments.

Finally, FEH Section 2.2.5 describes how Tp can be determined on the basis of nearby similar catchments. This concept is also applicable for pumped catchments, although the calculation has to be based on the catchment lag method instead of the catchment descriptors approach.

4.3.6 Unit Hydrograph

General approach: trapezoidal unit hydrograph

The unit hydrograph described in FEH is not suitable for pumped catchments. The MAFF *Engineering Guide* (Samuels, 1994) suggests using a trapezoidal unit hydrograph instead, as introduced originally in the Institution of Water and Environmental Management (IWEM)"s *Water Practice Manual* Volume 7 (IWEM 1988). This Guide confirms that the trapezoidal unit hydrograph should be used as described in the MAFF *Engineering Guide*. In addition though, this Guide aims to raise awareness that the trapezoidal (flat-topped) form partly reflects the influence of storage implicit within the drain system and its role in attenuating the flood discharge. As a result, use of the trapezoidal unit hydrograph for sub-catchment response combined with a hydraulic model that also explicitly includes this channel storage could cause underestimation of flood levels through over-representation of the attenuation. See Text box 4.3.

The trapezoidal (flat-topped) form of the unit hydrograph (UH) reflects the influence of the drain system on catchment response to rainfall. Because of the flatness of the catchments, run-off reaches the inlet pond at the pumping station only when a pump-run has stimulated a hydraulic gradient in the drain system. As a result, the main drain behaves in part as reservoir and in part as conveyor. The general effect of storage on a flood is that its peak is systematically reduced and delayed. As a result, the overall system becomes particularly sensitive to heavy rainfall of long duration. These effects are likely to be of greatest influence in modest floods rather than those approaching the design standards where the peak run-off is sustained for several hours.

The IWEM manual implies setting the time-to-peak Tp to 24 hours. In line with the MAFF *Engineering Guide*, this guide recommends using a locally derived value for Tp, following the approach in Section 2.2 of FEH Volume 4.

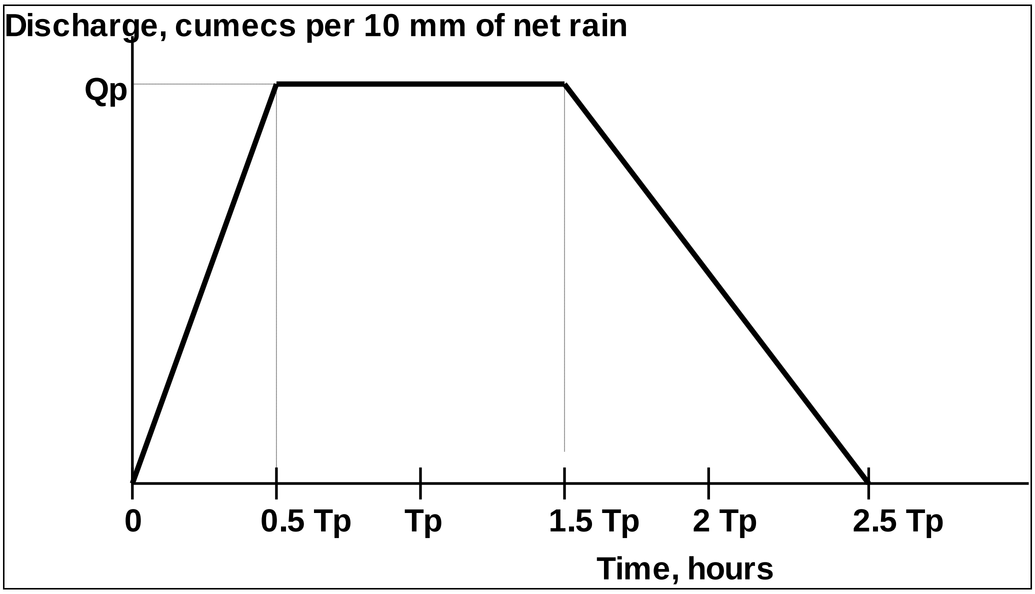


Figure 4.2: Trapezoidal unit hydrograph for use on flat low-lying catchments

The calculation procedure should follow the guidance in Section 2.2 of FEH Volume 4. This includes the estimation of a locally based value for the time-to-peak Tp, using analysis of rainfall and pumping station data for historical flood events (see Section 4.3.3 for the FEH approaches that can and cannot be used). In the absence of local data, Tp could be set to 24 hours, as suggested in IWEM (1988). The unit hydrograph shown is for a 10-mm six-hour block of rain (10 mm of net rainfall spread evenly over six hours). This basic data interval of six hours is appropriate for Tp of 24 hours. Appendix B shows the example calculation for Anderby pumping station in Lincolnshire that was also included in the MAFF *Engineering Guide* (Samuels, 1994).

The peak of the trapezoidal UH is adjusted to maintain the constraint that it represents the response to one cm (10 mm) of net rainfall. The relevant equation is:

$$Qp = 1.5873\, AREA/Tp$$

where Qp is peak response ($m^3 s^{-1}$), $AREA$ is in km^2 and Tp is unit hydrograph time-to-peak (hours).

Text box 4.3: Double-counting of storage

The trapezoidal form of the unit hydrograph partly reflects the process of attenuation through storage in the drain system. If this is combined with a hydraulic model that includes storage, this effect is wrongly counted twice, which leads to underestimation of flood risk.

The size of uncertainty introduced depends on the degree of influence of the pumping station at the upstream model boundaries. The degree of uncertainty can be limited by ensuring that the model is based on and validated for actual rainfall-pumping station behaviour in historical flood events. The trapezoidal unit hydrograph should be used to represent the low-lying catchment response including passage through the main drain system. It should not be used as a model boundary condition at the point of entry to the main-drain system.

Section 5.4.3 provides an iterative approach for dealing with this issue, which enables hydraulic modelling with appropriate hydrological inputs for pumped catchments and resolves the double-counting issue.

Unit hydrograph for urban sub-catchments

Highland and urbanised areas behave sufficiently differently to the low-lying area to require subdivision of the catchment (see Section 4.3.9). In such cases, it is appropriate to apply a conventional representation when synthesising run-off from the highland and urbanised sub-catchment. This conventional representation could be a triangular unit hydrograph as discussed in Section 2.2 of FEH Volume 4, or Revitalised FEH if the sub-catchment meets the application criteria described in FEH Supplementary Report No. 1.

4.3.7 Percentage run-off

FEH states that estimation of the percentage run-off is probably the most important and the most uncertain part of flood estimation. This is even more pertinent for pumped catchments, because in drainage systems that offer much storage, and with pumping stations designed to operate on demand rather than continuously, the volume of run-off tends to be of greater concern than the peak intensity of run-off.

Percentage run-off reflects many factors, including land gradients and land use. Urbanisation has a particular impact where it displaces naturally permeable surfaces. However, the dominant influence is the hydrological behaviour of soils, their management and the underlying geology and topography.

The general recommendation is to make broad use of the methods presented in FEH Volume 4 (Sections 2.3 and 3.3.1). Rainfall and pumping station data should be analysed for flood events and Standard Percentage Run-off (*SPR)* estimated by back-calculation. This requires an estimate of soil moisture deficit (SMD); Met Office systems such as MORECS or MOSES (Hough, 2003) can provide semi-standard values. The period over which pumped quantities and incident rainfall are compared should be chosen to reflect the main period of flood-producing rainfall and so that start and end profiles (of the water level in the main drain) are broadly similar.

Where analysis reveals a markedly different *SPR* value to that inferred from soil mapping (*SPRHOST*), it may be helpful to seek more detailed soil maps or to consult a soil surveyor.

There should normally be sufficient data to carry out a flood event analysis to determine SPR (pumping information from the drainage board, rainfall information from Met Office or Environment Agency). This analysis will often strongly improve any estimates based on soil mapping alone, which are typically of low accuracy.

4.3.8 Rainfall depth – duration – frequency

The FEH CD-ROM supports the use of the FEH rainfall depth-duration-frequency (DDF) procedure as described in FEH Volume 4, Section 3.2. This procedure is fully applicable for pumped catchments: the particular characteristics of pumped catchments are no obstacle for use of this element of FEH because the procedure is based on point locations, generally related to rainfall depths measured at particular rain gauges.

The main caveat concerns the determination of the design storm duration. FEH Volume 4 Section 3.2.1 provides an equation for this. This guide recommends using this equation as a first approximation, but then to follow the process described in FEH Section 9.2.2 to determine the duration that is critical for the purpose of the analysis.

The FEH rainfall DDF procedure can be applied at any one-km grid-point. Useful rainfall DDF estimates can be obtained by choosing the grid-point nearest to the true centroid of the pumped catchment. See Section 3.4 of FEH Volume 2 for guidance on determining catchment rainfall values.

The FEH rainfall DDF procedure has recently been revised (Stewart *et al.*, 2010) and it will be some time before the FEH CD-ROM is updated to support use of the revised method. Although targeted at extending applicability of the FEH procedure to estimate design rainfall depths at very long return periods, the method has changed sufficiently to alter estimates at shorter return periods also. Nevertheless, the original FEH procedure (supported by all existing versions of the FEH CD-ROM) remains applicable.

The resulting T-year hydrograph is illustrated in Figure 4.3 (calculated and visualised with ISIS), which shows how the flat-topped unit hydrograph produces a flood hydrograph that also has a relatively broad and flat top.

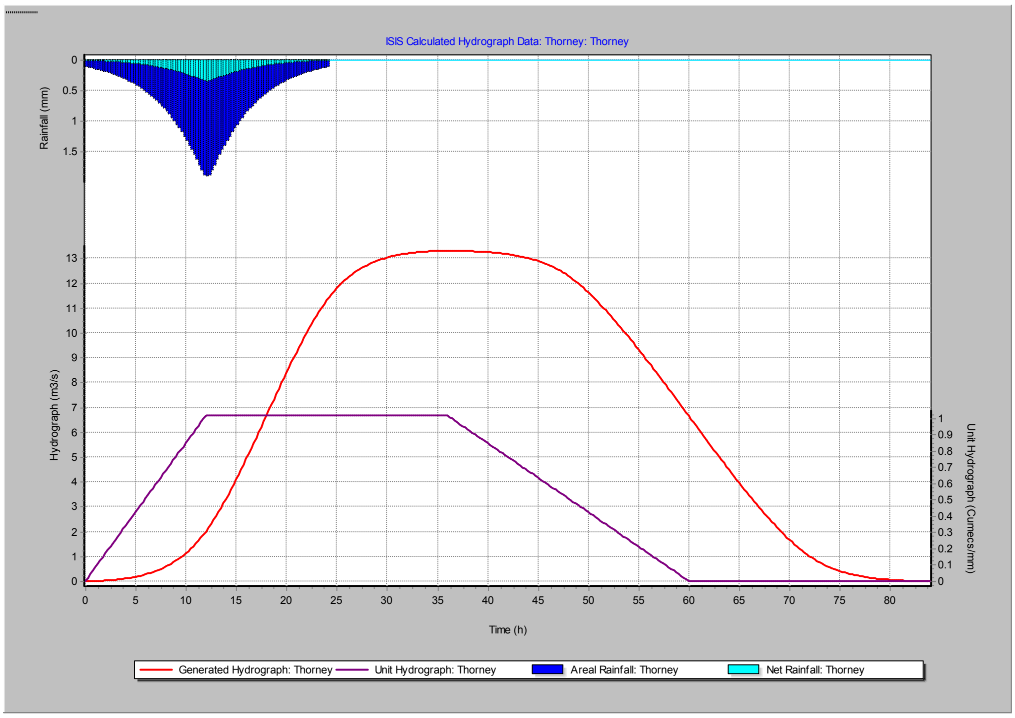


Figure 4.3: Illustration of T-year flood hydrograph based on trapezoidal UH

4.3.9 Sub-catchments

Selecting sub-catchments

There can be good reasons to subdivide catchments for hydrological analysis to represent important internal differences. The sub-catchments should be hydrologically meaningful units: they should reflect catchment properties and structure. Valid reasons for subdivision are to distinguish areas:

- contributing highland water;
- which are urbanised;
- of disparate soils/geology or land use;
- upstream/downstream of a booster pumping station.

It may also be useful to subdivide the catchment to reflect special features of the drainage system, such as

- systems with a pronounced dual-drain structure;

- where run-off from some areas reaches the pumping station by a siphon under the arterial drain.

In principle, the approach should be to estimate inflows to the drain for hydrologically meaningful units and then, if required, to break down (or "apportion") these, *pro rata* by area, for entry into the hydraulic model. There are several disadvantages to excessive subdivision of catchments.

First, the hydrologist is required to supply design inflow hydrographs from areas that are *false catchments*. These are areas defined only by subtraction, such as areas that drain to a downstream site of interest but not to particular upstream sites of interest. It is incorrect to apply rainfall-runoff models calibrated on real catchments to false catchments. See Section 4.3.4 for more details.

Second, the breakdown into smaller sub-catchments requires rainfall-runoff methods to be applied to ever smaller areas. Generalised models are calibrated on such *gauged* catchments as are available. For low-lying catchments, the methods described in this guide are largely based on catchments in the 10 – 100 km^2 range. Undue subdivision means that the use of the methods is extrapolated beyond the extent for which they were developed, reducing their accuracy.

Third, subdivision will typically lead to a hydraulic model that covers the channel network in more detail, into the smaller tributaries and channels. The hydrological rainfall-runoff methods are calibrated on flood data that represent *all parts* of the catchment, including the river channel and floodplain. If the hydraulic model extends further into the minor tributaries, this exacerbates the extent to which there is double-accounting for the attenuating effect of channel and floodplain storage. This reinforces the message that the relationship between hydrological and hydraulic modelling needs to be assessed carefully. This is discussed further in Text box 4.3, in Section 4.3.6.

Using sub-catchments

The general principles for the appropriate use of sub-catchments in pumped catchment are derived from good practice in river flood-risk estimation, reservoir flood safety and urban run-off studies. See also FEH Volume 4, Section 9.2. The basic principles can be summarised as follows:

- Hydrological assessment should generally be based on a catchment-wide design rainfall.
- Sub-catchment floods should not be combined unless they derive from a common design storm.
- Systems should be tested across a wide range of design storm durations, to ensure that flood risk assessments are based on the rainfall duration most likely to cause a severe flood.
- In principle, each significant point on the watercourse (or each significant point in the drain system) should be assessed separately.
- In special cases – for example, downstream of significant storage, barrier or constriction – the possibility of a flood arising from a storm on the catchment *intervening* between that point and the subject site should be checked. If this scenario presents a risk comparable to that presented by a catchment-wide storm, the assumption of one scenario (or the other) is likely to lead to underestimation of flood risk.
- There is a need to be careful when using a symmetrical design rainfall for durations longer than about 48 hours. It may be more realistic to assume that

rainfalls over such long durations are two-peaked. For example, it is known that in Eastern England, a proportion of large two-day and four-day maximum rainfalls derives from chance recurrence of short-duration storms on neighbouring days. The recommended approach is to adopt the design rainfall depths as determined by the FEH procedure, but distribute them in time using the temporal profile of one or more notable long-duration rainfall events that were experienced locally or regionally (which may or may not be double-peaked). The results should then be compared with those for the synthetic symmetrical rainfall profile to test the sensitivity.

4.3.10 Baseflow

Baseflow is discussed in section 2.4 and 3.3.1 of FEH Volume 4. As for some of the other parameters in this section, the general guidance for pumped catchments is to estimate the baseflow term from gauged data if possible (FEH section 2.4.2), or use information from a nearby similar catchment (FEH section 2.4.4). The formula for baseflow presented in FEH section 2.4.3 is not valid for low-lying catchments.

Baseflow in lowland areas is different from that in upland (sloping) rivers. Lowlands are at the bottom of the system, so they gather baseflow rather than pass it on downstream; they are also prone to baseflow from artesian pressure. Baseflow could be expected to be more important on flatter or slower-responding catchments, although there is no research to quantify this statement. Some pumped systems have relatively fixed imports from (or exports to) neighbouring highland or pumped catchments, e.g. through piped connection or by seepage. Adjusting the baseflow term provides a useful way of representing these in assessments, at least nominally.

4.4 Rule of thumb

IDB engineers in many areas have been using the long-established rule of thumb that the design discharge for a flat low-lying system is 1.4 l/s/ha. Old literature (MAFF training materials; no publication details available) indicates that this rule was intended to determine the one in 10 years (10 per cent annual exceedence probability) flood flow. This was then used as the starting point for the hydraulic design of the channels (see Section 5.4.1).

The IWEM Manual (page 174) describes how this rule has evolved over time as standards and units have changed. It was originally formulated as the run-off depth that the pumps should be capable of removing in a day. This amount gradually increased from a quarter inch when pumps were first introduced, to a half inch from the 1950s and 60s. This is equivalent to 20.7 cusecs/1,000 acres, which equals 1.45 l/s/ha. At some point, these values were rounded off to 20 cusecs/1,000 acres, or 1.4 l/s/ha.

The old MAFF training materials indicate that values in use ranged from 15 to 25 cusecs/1,000 acres (which equates to 1.05-1.75 l/s/ha). The training materials warn against using the rule on hilly catchments and emphasise that its accuracy is limited.

Any use of this method in current practice should be limited to first approximations only. The user should apply sensitivity analysis for a range of at least plus and minus 25 per cent and be aware that the resulting value is intended as the 10 per cent annual exceedence probability flood flow.

It is possible to use the Flood Studies Report‟s regional growth curves to create a rough estimate of the flood flow at other frequencies: apply the appropriate factor to reduce the 10 per cent annual exceedence probability flood into the Mean Annual Flood, then apply a growth factor to scale up to the desired exceedence probability.

4.5 Additional matters

Water balance

It can be helpful to use pumping station data to attempt a partial long-term water balance of the catchment, as follows:

rainfall – evapo-transpiration

=

stored water (ground and watercourses)+ volume pumped.

Storage can be eliminated from the equation by choosing a long time period for the analysis between two dry situations. Evapo-transpiration rates can be obtained from the Meteorological Office Rainfall and Evaporation Calculation System (MORECS), available from the UK Met Office.

Because of the relatively low rainfall depths and high rates of potential/actual evapo-transpiration in (for example) Eastern England, such a water balance will be difficult to achieve. However, it is good practice to at least estimate the total quantity pumped (for example, each water-year) and to express this as an equivalent catchment run-off depth (in mm). Unexpectedly high or low values will need to be explored.

It can also be useful to develop an event-based water balance to estimate the flood water that will need to be stored outside the drains in a particular event. This looks as follows:

Floodplain storage

=

Rainfall – pump capacity – storage in watercourses

Used as a broad-scale assessment, this can give a useful first indication of the flood extent in a particular event, in the absence of better information or in advance of more detailed analysis.

Groundwater seepage

Some low-lying catchments receive additional water by groundwater seepage. Although such quantities can be difficult to estimate, the attempt to do so provides a useful reminder of the phenomena. For example, it is suspected that groundwater seepage contributed appreciably to flood run-off in some pumped catchments close to the Lincolnshire Wolds in summer 2007.

5 Hydraulic analysis

5.1 Introduction

Hydraulic analysis can provide information about water levels and flow velocities, primarily in the channels but in some cases also in the fields and floodplain. This information can be needed for pumped catchments for the following purposes:

- determining design channel size;
- identifying and improving local constraints;
- optimising channel management (dredging and vegetation management);
- estimating flood probability and depth to determine flood risk;
- optimising operation of the pumps (see Chapter 6).

These purposes all relate to elements of the management context as described in Chapter 2: the assessments can be needed for asset management, for project appraisal or for assessing the impacts of third party changes to the catchment. Text box 5.1 introduces some of the key concepts for calculating and managing channels.

This Section 5 replaces Sections 2.3, 2.4, 2.6 and 2.7 of the MAFF *Engineering Guide* (Samuels, 1994).

Text box 5.1: Conveyance and afflux

Conveyance is a quantitative measure of the discharge capacity of a watercourse. It depends on channel roughness, channel shape (section and plan form) and cross-sectional area. Conveyance describes the relationship between the gradient and the discharge: a higher conveyance leads to a higher discharge under a given hydraulic gradient. For further details and equations, see *Conveyance User Manual* (Defra/Environment Agency, 2004).

Afflux is an increase in water level that can occur upstream of a structure at high flows. More formally, afflux can be defined as the maximum difference in upstream water level, for a specific flow, between conditions with the structure in place and those arising if the structure were to be removed. It is caused by energy losses at high flows through bridges and culverts, and it is made worse by blockage. See Section 5.2.5.

5.2 Choice of method

Similar to hydrology, a range of methods is available for hydraulic analysis, from simple to advanced. The choice of method depends primarily on the question that needs to be answered, and is influenced by availability of data and resources. The methods are:

- backwater calculations (without computer modelling);
- steady-state modelling or dynamic modelling;
- one-dimensional or two-dimensional modelling;

This section also provides advice on the need to calculate afflux.

5.2.1 Is hydraulic analysis needed at all?

Hydraulic analysis is needed if there is a need for information about water levels and flow velocities, primarily in the channels but in some cases also in the fields and floodplain. There is, however, no need for hydraulic analysis to determine pumping capacity for a pumped catchment. As discussed in Chapter 4, it is better to use the tailored FSR rainfall-runoff method only. Combining this with hydraulic modelling can only confuse the calculation of the pumping capacity, because the combined model will have to be calibrated against the trapezoidal unit hydrograph from the tailored FSR rainfall-runoff method anyway (see Section 5.4.3).

5.2.2 Backwater calculations or simulation modelling?

Standard steady-state backwater calculations can be used to determine water levels and flow velocities in the channels. The dimensions of the main drain leading to the pumping station may be calculated from the maximum flow rate assessed in the analysis of flood run-off (see Chapter 4) and the desired design water level at the pumping station (see Section 6.3). The method is described in Knight *et al.* (2010). In design, the channel dimensions should be adjusted so that any design constraints on water level or velocity are met. A typical design criterion for the depth-averaged flow velocity is 0.3 m/s.

Backwater calculations can also be a practical tool to calculate the impact of small changes to the system, such as additional run-off from a small-scale development or a local change in channel conveyance. This can often be more practical than the use of simulation models.

It is not practical to carry out backwater calculations for complex systems. In addition, they can only be used for steady-state situations, see Section 5.2.3.

An even simpler approach could be to use pipe capacity curves; however, this is not recommended because these curves assume that the hydraulic gradient is the same as the laid gradient, which is not true for watercourses.

5.2.3 Steady state or dynamic?

Steady-state methods can be used in many cases because the generally slow response of lowland catchments to rainfall ensures that the flow around the peak is reasonably steady, as illustrated by the trapezoidal unit hydrograph in Figure 4.2 and the flood hydrograph in Figure 4.3.

There are, however, some constraints to the use of steady-state modelling. A steady-state calculation will not produce any information on the timing of pump runs or on the attenuation of run-off through storage within the drainage channels; these aspects require dynamic modelling.

The assumption of steady channel flow may not be valid for cases where the inflow to the pumping station has a significant component from peripheral uplands or from urban areas. The response of these catchments to rainfall will be quite different from that of the lowland area, which means that the resulting hydrograph in the channels will be different from the hydrograph with a flat top with a long period of steady flow that is typical for lowland catchments. See Section 4.3.6 for the hydrograph of urban catchments and Section 4.3.9 for the selection and use of sub-catchments. In such cases an unsteady flow simulation model should be used. This will account for the attenuation of urban run-off by the storage in the drainage channels and the relative timings of the urban, upland and lowland catchment run-off.

Full dynamic modelling is necessary to track the performance of a drainage system under historical or hypothetical inflow conditions. The reasons for this are as follows.

- The length of the pump backwater is different for drawdown and ponded water surface profiles below and above the normal depth line. This is due to the non-linearity in the flow resistance formulae. Steady-state modelling does not capture this distinction.
- Only under the design flood conditions with pumps running at full capacity will the flow conditions be relatively steady, although even then there will be a rise and fall of water levels in the drains (due to time-varying inflow and cutting in and out of the pumps). Use of steady-state modelling for this situation provides a conservative approximation of flood risk.

5.2.4 One-dimensional or two-dimensional?

Normally, the main benefit of two-dimensional hydraulic modelling is the more detailed representation of floodplain flow and the possibility to determine flow patterns and velocities. For flat low-lying areas, however, the floodplain is often so flat that flow velocities are negligible, and a Digital Terrain Model GIS analysis or the use of LiDAR-based stage storage curves linked to one-dimensional models could suffice to determine flood areas and depths. If the land around the drains is not flat, the water may not be able to get to the lowest-lying area; in such cases, it can be worthwhile investing in the better accuracy of two-dimensional modelling.

5.2.5 Is there a need to calculate afflux?

There is no need to calculate afflux when determining the design pumping capacity, using the tailored FSR approach. However, if there is a need for information about water levels and flow velocities, and hydraulic modelling is needed, afflux may be relevant.

Afflux has a local impact on flood levels, but this is normally only significant at catchment level if there are critical features at risk near the constricting structure. Afflux is important for analysis of the impact of specific structures on flood risk, or if there are erosion or sedimentation issues around a specific structure. A culvert, particularly if partially or fully blocked, will throttle flow downstream and raise water levels upstream providing a temporary increase in flood storage and thus attenuation.

Afflux is included in all relevant one-dimensional hydraulic models and the Afflux Estimation System (AES) is available as an integrated package with the Conveyance Estimation System (CES). There is also a separate spreadsheet application called the Afflux Advisor, which can make a relatively quick calculation for simple structures in a uniform channel.

5.3 Determining hydraulic roughness

General approach

All types of hydraulic analysis require the calculation of flow resistance (also known as hydraulic roughness) which determines the conveyance (or transportation) of the drainage channels. There are various formulae in use, but most engineers use the Manning equation. MAFF-funded research in the 1980s on the Newborough Fen catchment produced the following relation for Manning's n at shallow depths:

$$n = 0.0389\, k_s^{0.67}\, R^{-0.50}$$

where R is the hydraulic radius of the cross-section and k_s is the roughness size according to the Colebrook-White resistance law.

The research on the Newborough Fen catchment found a value of 1.0 m for k_s in design conditions, leading to a recommendation to use a value for Manning"s n of 0.04 when R is greater than 1.0 m. For smaller values of R, $0.04/\sqrt{R}$ is recommended. Knight *et al* (2010) provides further guidance on determining roughness values.

For a more precise analysis, and for other values of k_s, the Conveyance Estimation System"s Roughness Advisor can be used to calculate roughness for a wide range of bed materials and vegetation types, including composite channels and the impact of vegetation growth and seasonality. Note that the unit roughness n_l produced by the Roughness Advisor needs to be translated if the user requires Manning"s n or k_s, as explained in the Conveyance Manual (Defra/Environment Agency, 2004).

The hydraulic effects of weed

The principal effects of vegetation are:

- to reduce the effective cross-section area;
- to increase the effective wetted perimeter;
- to trap sediment and so reduce the section area.

When analysing the effects of vegetation, these processes are normally compounded into a change in the effective value of Manning's n with the hydraulic properties based on the dimensions of the clear cross-section without vegetation. The research reported in the MAFF *Engineering Guide* (Samuels, 1994) indicated that Manning"s n values of up to 0.3 have been observed in the most severe cases. However, if the physical obstruction due to weed is significant, it is preferable to reduce the channel dimensions as well as changing the roughness.

The CES Roughness Advisor has a useful facility to determine the expected seasonal variation of roughness as a result of vegetation growth (Defra/Environment Agency, 2004). Section 5.4.2 explains how this can be used in channel management.

5.4 Practical use of hydraulic analysis

5.4.1 Channel sizing

Hydraulic analysis plays an important role in determining the size and shape of the channels. Together with the hydraulic roughness, the shape and size of the channel determines the discharge capacity (or conveyance) of the channels. From a hydraulic point of view, the design criteria typically relate to:

- Maximum flow velocity – this needs to be limited to prevent significant sediment transport and erosion in design conditions. A typical design value for the maximum flow velocity (average over the cross section) is 0.3 m/s. Note that flow velocity can also have an impact on habitats, which may pose different design requirements.

- Minimum water levels at the pumping station – the discharge capacity of the channels needs to ensure that in flood situations, the flood water is able to reach the pumping station. Insufficient channel discharge capacity can cause a situation where the water level at the pumping station reaches the minimum for pumping, while levels further upstream are so high that they cause flooding.
- Minimum freeboard throughout the system – to prevent flooding of fields and floodplain (increasingly including urban areas) in design situations (with a return period that may vary with land use) and to maintain agricultural freeboard for normal operating situations, both in winter and in summer.

Other design considerations for channel size and shape are operational (access for maintenance), environmental (suitability for habitats), economic (reduction of productive land and costs of excavation) and spatial constraints such as the presence of structures or roads. These are not discussed further in this guide.

5.4.2 Channel management

Channel management includes dredging, clearing obstacles and vegetation management. The type and timing of each of these measures have to be chosen carefully, aiming to achieve the right balance between economic, environmental and social interests. Hydraulic modelling can be a useful tool to support local engineering judgement.

In pumped catchments, with their typical low flows and negligible sediment transport, dredging is carried out with relatively long time intervals (for example 10 years). Siltation typically happens very gradually and therefore dredging can be planned on a reactive basis. Hydraulic analysis can be used to assess the sensitivity of the system to realistic siltation levels; a steady-state 1D model is likely to suffice for this.

The presence of obstacles (for example, due to fly tipping) is relatively rare in rural areas but can be significant in urban systems. As for dredging, hydraulic analysis can be used to assess the potential significance for systems where fly tipping is an issue.

The considerations can be very different for vegetation management. Section 5.3 discusses the influence of vegetation on the hydraulic roughness. In practice, channel vegetation in summer can lead to the situation described in Section 5.4.1 where the channel discharge capacity is reduced so far that it causes flooding in the upstream parts of the system while the water level at the pumps is below the minimum level for pumping. This situation occurred, for example, in the summer 2007 floods in north Lincolnshire. As an indication, vegetation can cause water level increases up to more than 0.2 m per km (or „one foot per mile").

Vegetation management is often based on practical considerations and expert judgement. The key decisions are the timing of the cuts in relation to the vegetation growth curves and the seasonal likelihood of flooding (both of which are difficult to predict), and whether to cut once or twice per growing season. The operational capacity of the organisation is an important constraint, as is the impact of vegetation management on habitats – it is important to take account of aspects such as birds" breeding seasons. The recent *River Sediments and Habitats* project has produced guidance and case studies for the generation and assessment of channel maintenance options (Environment Agency, 2011).

It is possible to use hydraulic modelling to plan and optimise maintenance strategies, for example by using vegetation growth curves as included in the Conveyance Estimation System"s Roughness Advisor. This can be used to test the sensitivity of flood risk to maintenance regimes by modelling a range of vegetation management options. Such an analysis can be used to quantify the benefits of increased maintenance (for example moving from one to two cuts per year) and compare these benefits to the additional costs.

5.4.3 Assessing flood risk

Hydraulic analysis is needed to determine the flood extents and water levels that are required to estimate flood risk. This can then be used to determine the benefits of improvement works, feeding into project appraisal. Section 2.3 describes the general principles of appraisal and mentions specific issues for pumped catchments, but refers to the FCERM appraisal guidance for details (Environment Agency, 2010b). Appraisal requires the calculation of flood extents, water levels (using the approaches described in this guide) and associated damages for a number of event probabilities of exceedence. Section 7.4.2 of the appraisal guidance suggests using at least three (preferably five) events, and determining the choice of events carefully, ensuring that significant flooding thresholds are captured.

For properties at risk, the analysis needs to determine whether or not the threshold is flooded at a particular return period. In addition, the depth of flooding has an impact on the damage calculation (although in practice this is sometimes simplified). For agricultural land, the flood extent is the key parameter. In more advanced analysis, it would also be possible to take account of the duration of flooding. The *Multi-Coloured Manual* (FHRC 2010) provides detailed guidance on the calculation of damages, including recommendations for the application of freeboard.

As indicated in Text box 4.3 there is a risk of double-counting the storage effect of the drain system and of the floodplain if the tailored FSR approach is used to provide hydrological input into hydraulic modelling.

The recommended approach uses an iterative procedure where the flow hydrograph at the pumping station, developed on the basis of the tailored FSR approach (or preferably, based on gauged data) is used to calibrate the combined hydrological and hydraulic model. This is based on the idea that the trapezoidal UH (at the heart of the tailored FSR approach) was developed through analysis of complete (10 to 100 km^2) low-lying catchments, and therefore should be assumed to be correct at the pump; however, it is not necessarily accurate for generating the response of small sub-catchments and calculating their contribution to the drains.

Step one is to follow the procedure described in Section 4.3 to determine the design flow hydrograph at the pumping station, or alternatively base this on gauged data where available. This flow hydrograph is seen as the most accurate representation of reality; the aim of the procedure is to calibrate the combined hydrological and hydraulic model to represent this flow hydrograph.

The trapezoidal UH developed as part of Step 1 is then used to provide a first approximation of the hydrology input for the hydraulic model. For a uniform catchment, this needs to be done pro-rata, as a function of the area that drains to each inflow node of the drain system.

Running the hydraulic model with this input will then simulate channel flow but also (if the user needs this info) the flow out of the channel back into the fields (the user can choose to use a „no glass wall" approach). In this first iteration, the resulting hydrograph at the pump will then be lower than the 'true' flow hydrograph from Step 1, that is, too low.

The next step then is to change the hydrological inputs (again based on pro-rata approach, so making the same changes to all inflow nodes) until the resulting flow to the pump matches the 'true' flow hydrograph for the whole system.

There are various ways to change the inflow hydrograph for this calibration. The simplest approach is to change the time-to-peak Tp. A more involved, but possibly more accurate approach would be to change the shape of the unit hydrograph, shortening the flat-topped part. If the timing of the flow hydrograph is acceptable but the total volume needs adjusting, the percentage run-off could be changed. Whatever the chosen approach, the user must ensure that the unity of the UH is maintained: the area of the hydrograph must remain equal to one by making the required changes to the other parameters (Tp, unit hydrograph shape or Qp).

Text box 5.2: Level of detail of hydraulic modelling

As highlighted in Text box 4.3, combining hydraulic modelling with the tailored FSR approach for hydrology can lead to double-counting of storage and therefore underestimation of flood risk. The approach described in Section 5.4.3 ensures that the modelled flow hydrograph at the pumping station is appropriate (the approach prevents double-counting at that point). However, on a more local level within the catchment the reliability of the calculated water levels and flows can still be affected. This effect is larger as the level of detail of the hydraulic model increases: a more detailed hydraulic model requires more detailed hydrological inputs and further subdivision of the catchment. Even if this is done pro-rata, the reliability of the local hydrological inputs reduces, and therefore also the reliability of the local results of the hydraulic model.

6 Pump capacity and operation

6.1 Introduction

The installed pump capacity should be capable of discharging the flood run-off that is expected to reach the pumping station in design conditions, including the effect of storage (as determined from the procedures in Chapter 4). Following this, the dimensions of the main drains should be set so that the discharge capacity is sufficient to pass the design discharge to the pumping station with an adequate freeboard, as discussed in Chapter 5. The next step is to determine the best means of providing the required total pumping capacity.

The key considerations in determining pump capacity and operations are the reliability of the system to perform as required, optimisation of running costs and ease of maintenance. Over the last 20 years, the development of automatic weedscreen cleaners and telemetry have strongly improved reliability and increased pumping capacity.

This guide does not provide detailed guidance on choosing types and makes of pumps and associated whole-life considerations.

This section 6 replaces sections 2.5 and 2.8 of the MAFF Engineering Guide (Samuels, 1994).

6.2 Number and type of pumps

It is common practice to distribute the total capacity between several pumps for several reasons including:

- to reduce the maximum demand for electricity under normal operating conditions which will reduce running costs and carbon footprint;
- to allow for routine maintenance of the pumping plant;
- to allow for the wear and tear of operation to be distributed around the installation by rotating the 'duty' pump;
- to give flexibility of operation.

Although in the past it has been usual to divide the required capacity between three or four identical pumps, the adoption of several pumps of differing capacities or of one or more variable speed pumps will allow the pumping rate to be more easily equated to the channel conveyance, which will help to optimise pump operation.

6.3 Setting pump operation rules

The water levels at which the pumps are turned on or off are usually sensed automatically in the drain. It can be helpful if real-time control of the pumping station is also informed by water levels in a remote part of the main drain, as well as by water levels inside and outside the pumping station weedscreen.

The switching levels should be chosen to maximise the extent of the pump backwater, with the switch-on level as high as possible given other constraints. The switching level for each pump or combination of pumps must be set to give an absolute minimum depth equivalent to the normal depth for that installed pump capacity. If this is not done, the effective conveyance capacity of the drains will be insufficient for the pumps to operate continuously. For example, the levels may be set too low in an attempt to provide more flood storage, but the effect is to reduce the effective conveyance of the channel which might otherwise have sufficient capacity to serve the pumps

The choice of switching levels has to be based on the structure of the energy tariffs. The cost of electricity varies according to the time of day, day of the week and the month of the year. Running costs can be reduced by choosing different operating levels for each tariff period, to avoid unnecessary pumping in periods of high energy cost. Further savings could be made by setting different rules for starting an extra pump when this would incur a fresh “maximum demand” charge for the power supply, especially in periods approaching a change-point in the charging tariff. This will require special operational strategies.

The length of the pump backwater, L, is given approximately by the formula (Samuels, 1989)

$L = 0.7 D/S$

where D is the design depth of the flow and S is the water surface gradient. An overestimate for a severely drawn down (M2) profile and an underestimate for a (M1) profile are shown in Figure 6.1.

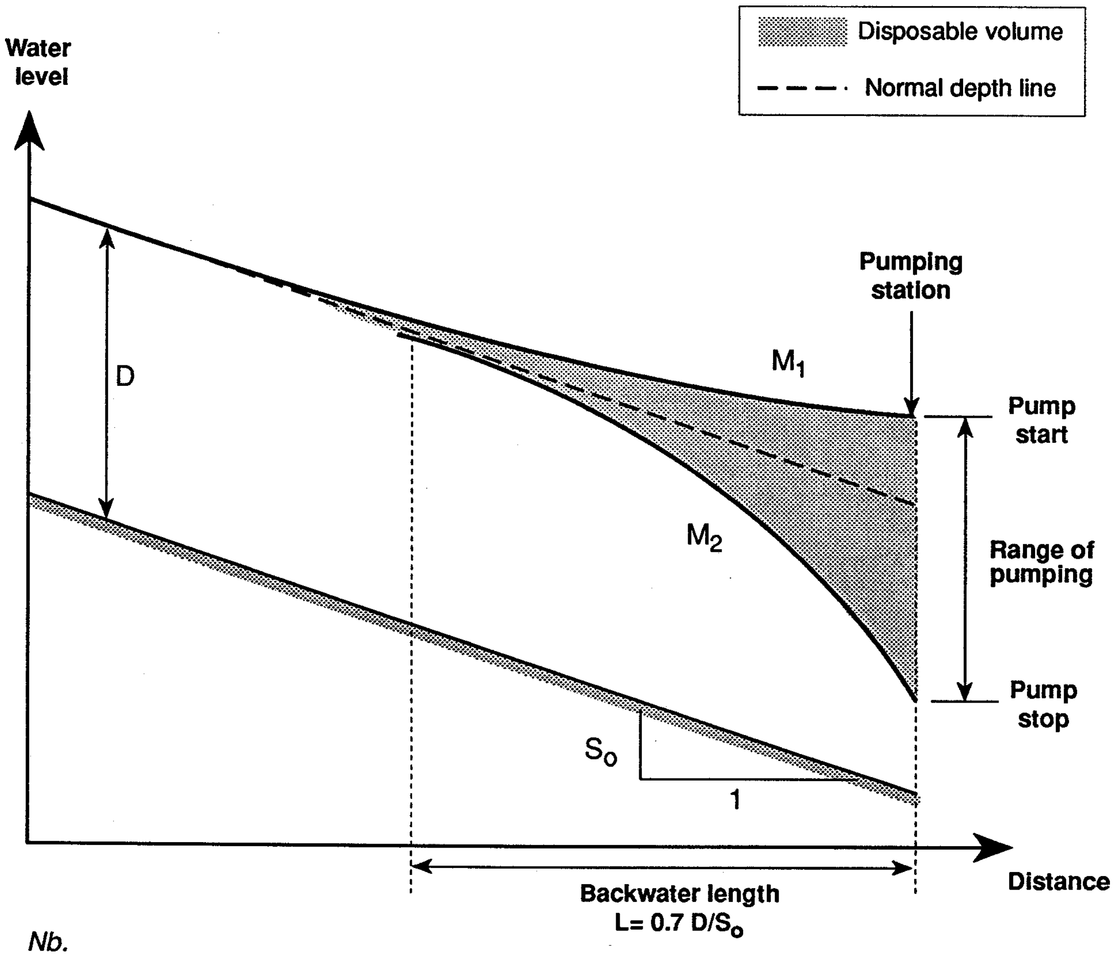


Figure 6.1: Pump backwater and disposable volume

The disposable volume accessible during a pump run lies between the pump-on and pump-off water surface profiles; see Section 6.3 of the IWEM Water Practice Manual no.7. Figure 6.1 illustrates the drain profile in a flood condition when a reasonable water surface gradient is established. For much of the time, the inflow into the drainage system is much smaller than the capacity of a single pump running continuously. Under these conditions, the water surface profile in the main drain may become nearly horizontal in the periods between pump runs. The normal depth line on Figure 6.1 represents the water surface profile for steady uniform flow in the main drain. This will occur if the inflow exactly equals the capacity of any pumps running and if this situation continues for a sufficiently long time for a steady state to be achieved.

Artificial lowland drainage channels differ in their design from the characteristics of a natural river. The 'bankfull' discharge in an artificial channel may be the 50-year flow whereas in a natural river, bankfull conditions occur at around the mean annual flood. Also, the flow velocities are much lower than are typical. Both these factors lead to channel dimensions which are much larger than in a natural river, giving a much enhanced volume of available storage below the bank top level. It is this storage which can be exploited to reduce the cost of pumping under routine operational conditions. Although the channel size is set for flood performance, an added benefit is that designs for large return periods tend to create significant online storage for normal flow conditions.

Reed (1993) introduces the idea to express the storage in a system in terms of pump-hours; this can be a convenient measure for the analysis of system capacity. At Newborough Fen, the storage available between the minimum and maximum desirable water surface profiles for summer conditions was about 17.3 pump-hours, indicating considerable scope for phasing pumping with cheap energy tariffs at this site. At Postland in the North Level IDB area, the storage available for manipulation is less, being about 7.8 pump-hours in summer and 5.5 in winter, indicating a smaller degree of flexibility at this site. Reed (1993) gives an expression for the storage within the drainage system which can be used in conjunction with computer control of the pumping station. Although this may be based on design or surveyed drain geometry, it was found helpful to calibrate the equations by observations from the catchment. The calibration correction will include the effect of:

- deficiencies in the simple storage equation;
- storage not modelled by the equation from side drains;
- differences between current pump capacity and the capacity when installed.

In most drainage systems, the storage available is not significant compared with the volume of the design flood, see Text box 6.1. Consequently around the peak of the design flood, the flow in the channels becomes steady and the pumps run (nearly) continuously. There is no possibility of offsetting the cost of a reduced pumping capacity against introducing more storage, since the cost of the land required would be prohibitive. There is, however, considerable scope for optimising the day-to-day running costs of the system for normal flows as opposed to the design flood condition.

Text box 6.1: Over-reliance on storage rather than conveyance

There is scope for over-reliance on storage rather than conveyance. Ageing pumping stations were abandoned in Bideford and were insufficiently renewed in Hull. At Bideford, flooded on 25 June 1993, the storage provided was large but the altered strategy failed to note that outlet arrangements made the system sensitive to rainfalls of very long duration. At Hull, flooded on 25 June 2007, it was thought that tunnel storage of 103,963 m^3 could compensate for a reduction in the total pumped discharge capacity to the estuary. When expressed as a depth over the entire catchment, the new storage represented just 1.7 mm of run-off. This example demonstrates that it is important to find the right combination of storage and conveyance, and that it is worthwhile assessing the role of each element as part of the whole system.

6.4 Other influences on pump operation

During the MAFF-sponsored research programme in the 1980s, the effects on pumping of two other variables were investigated, wind and tide. The effect of wind is to alter the water surface slope along a straight drain aligned approximately with the wind direction. The effect depends upon the length of drain, the direction of the wind and the square of the wind speed (amongst other factors). In lowland drainage systems wind speeds of less than about Force 6 are unlikely to cause any serious operational problems. Marshall and Beran (1985) describe strong wind on the Newborough Fen channels that was found to change water levels by about 0.1 m. Such changes in levels may be sufficiently large to trigger or halt a pump run at an automatic station.

The effect of tide (and any other water level variation in the receiving watercourse at a pumping station) is to modify the effective head on the pump. Each design of pump will have its own operating characteristics, relating discharge and energy consumption to the operating head. Pumping stations discharging into tidal watercourses will suffer an increase in pumping cost at periods of high tide. Since the timing and height of the tide at coastal locations can be readily predicted from astronomical effects, it should be possible to include these in determining a pumping strategy that avoids operation at time of high water. At Boy Grift in Lincolnshire, the cost of pumping at the times of highest tide was found to be 55 per cent greater than at low water (Marshall, 1993), though at certain times of the year differential electricity costs showed an even greater variation. Marshall discusses the potential for altering pumping patterns to reduce the need to pump at high tide for this site.

Some pumping stations, for example at York on the Foss and at Boroughbridge on the Tutt, are designed to over-pump a watercourse in times when its discharge is “locked” by a larger river. These are often mildly graded rivers, and this type of operation will warrant special study.

6.5 Reliability of pumping stations

The operation of the pumping station during floods is critical for the performance of the pumped catchment system. Potential causes of failure are inadequate maintenance, failure of automatic operation (remote sensing of water levels, controls), power failure, drowning of the pumps, reduced access when needed and human error.

There has not been much research into the relative importance of each source of failure. In theory, it is possible to assign a probability to each source of failure and optimise management regimes accordingly. In practice, pumping stations are managed according to procedures that have developed over time based on local knowledge and experience, and informed by sharing of this knowledge among practitioners. When needed for project appraisal, pump reliability will have to be estimated based on local knowledge and expert judgement.

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List of abbreviations

ADA	Association of Drainage Authorities
AES	Afflux Estimation System
CES	Conveyance Estimation System
CFMP	Catchment Flood Management Plan
DDF	Depth-duration-frequency
FCERM	Flood and Coastal Erosion Risk Management
FEH	Flood Estimation Handbook
FSR	Flood Studies Report
IDB	Internal Drainage Board
IWEM	Institution of Water and Environmental Management
MAFF	Ministry of Agriculture, Fisheries and Food
SMD	Soil Moisture Deficit
SMP	Shoreline Management Plan
UH	Unit Hydrograph

Glossary

Afflux:	The rise in water level above the normal surface of water in a channel that is caused by a partial obstruction, such as a bridge or culvert. The afflux may also be described as the maximum change in water level that would occur, at a particular flow, if the structure were to be removed.
Arterial drainage:	Primary land drainage channels designed to achieve rapid transfer of water to rivers.
Asset:	In flood defence, any man-made or natural feature – such as a raised defence, retaining structure, channel, pumping station or culvert – that performs a flood defence or land drainage function. Includes components owned by the Environment Agency or another body, whether or not flood defence is the primary function or is incidental to some other purpose, and components which may be detrimental to flood defence objectives.
Asset management:	Systematic and coordinated activities through which an organisation manages its assets and asset systems for the purpose of achieving its strategic aims This includes the performance of the assets and the associated risks and expenditures throughout their lifecycles, and carries an implication that the management is undertaken in an optimal and sustainable manner.
Attenuation:	The reduction in the peak discharge of a flood which may occur as the flood passes downriver, including the effects of any flood storage ponds and reservoirs.
Baseflow:	That part of the flow in a watercourse that emerges from groundwater storage.
Catchment:	The land (and its area) which drains (normally naturally) to a given point on a river, drainage system or other body of water.
Catchment flood management plan:	A large-scale strategic planning framework setting out policies for the integrated management of fluvial flood risks to people and the developed and natural environment in a sustainable manner.
Conveyance:	For a channel, function of the flow area, shape and roughness of a channel, which can be used as a constant in a formula relating discharge capacity to channel gradient.
Culvert:	Covered channel or large pipe to convey water below ground level, for instance under a road, railway or urban area, or beneath a building or other structure.
Design flood:	Magnitude of the flood adopted for the design of the whole or part of a flood defence system, usually defined in relation to the severity of the flood in terms of its return period.
Discharge:	The volume of water that passes through a channel cross-section

in unit time, normally expressed at cubic metres per second (m^3/s) in fluvial design (often more simply referred to as „flow").

Dredging: Underwater excavation, usually including removal of the excavated material.

Erosion: Process by which particles are removed by the action of wind, flowing water or waves (opposite is accretion).

Failure mechanism: One of any number of ways in which an asset may fail to meet a particular performance requirement, target or threshold.

Flood risk management: The activity of understanding the probability and consequences of flooding, and seeking to modify these factors to manage flood risk to people, property and the environment in line with agreed policy objectives.

Floodplain: Area of land bordering a river which is partly or wholly covered with water during floods.

Fluvial: Relating to a river.

Freeboard: The height of the top of a bank, floodwall or other flood defence structure, above the design water level (normally the water level that would occur disregarding any effects from wave action).

Groundwater: Water contained in the interstices of soil and rock, above and below the water table.

Head: Hydraulic head, either in terms of the water level or the energy level, depending on the context.

Hydraulic gradient: The gradient of the energy line (preferred term is „energy gradient").

Hydraulic roughness: A measure of the resistance to flow in a channel, representing the irregularity of the bed and banks, vegetation growth, and other factors that act to impede flow (see also Manning"s „n").

Hydrograph: Graph that shows the variation with time of water level or discharge in a river, channel or other water body.

Maintenance: Work that sustains the desired condition and intended performance of an asset.

Manning"s „n": A coefficient used in hydraulic calculations to represent the resistance to water flow that is presented by the roughness, irregularities and vegetation growth on the channel bed and banks (see also „hydraulic roughness").

Mean annual flood: The arithmetic mean of the series (AMAX) that comprises the maximum flood flows in each water year, defined as QBAR in the *Flood Estimation Handbook*.

Performance: The creation or achievement of something that can be valued

	against some stated initial aim or objective, and also the degree to which a process succeeds when evaluated against some stated aim or objective.
Reliability:	The probability that a flood defence asset will not fail during a given period of time.
Residual risk:	The risk that remains after risk management and mitigation.
Return period:	Average interval of time between events that equal or exceed a particular magnitude.
Run-off:	Overland flow produced by rainfall.
Sediment:	Material ranging from clay to gravel (or even larger) that is transported in flowing water and that settles or tends to settle in areas where the flow slows down.
Sedimentation:	The deposition of sediment in the bed of a channel or within a hydraulic structure.
Stakeholder:	An individual or group with an interest in, or having an influence over, the success of a proposed project or other course of action.
Standard of service:	The performance of an asset at a specific point in time.
Watercourse:	Defined natural or man-made channel for the conveyance of drainage flows and floods by gravity.
Weed screen:	A screen comprising closely-spaced bars placed upstream of a hydraulic structure to prevent waterborne debris from progressing downstream where it might otherwise cause a problem (for example by blocking a culvert or damaging pumps).

Appendix A
Example: determining catchment descriptors

The FEH CD-ROM does not represent the 32.5 km^2 Newborough Fen catchment correctly. However, the sum of two catchments (shown bounded in green and red in Figure A1) provides a reasonable approximation for the purpose of evaluating catchment values of *SAAR* and *SPRHOST*. The latter is the estimate of standard percentage run-off based on HOST soil mapping.

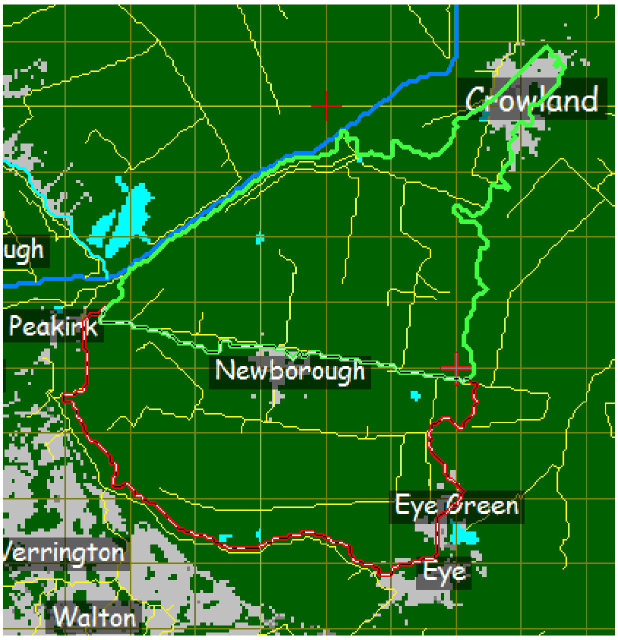

Figure A1: Newborough Fen "catchments" according to FEH CD-ROM v3

Because the two areas in this example are almost identical in size, the descriptors derived for the whole catchment (final column of Table A1) are close to the arithmetic mean of those for the Southern and Northern sectors. However, in general it is crucial to use *area-weighting* when adding/subtracting areas to form the true catchment (as in the Newborough example) or when subtracting true catchments to estimate descriptors for an intervening area.

Table A1: Coping with defective digital catchment-boundaries

Descriptor	Unit	Southern sector	Northern sector	Sum	*AREA*-weighted average
Grid reference	*m*, *m*	523250 305750	521250 309600		
AREA	km^2	15.82	15.87	31.69	
SAAR	*mm*	548	550		549
SPRHOST	*%*	25.0	23.5		24.2
$URBEXT_{1990}$	-	0.0167	0.0226		0.0197
ALTBAR	*mAOD*	8	7		

The fractional urban extent (*URBEXT*) thus estimated is less trustworthy. The catchment boundary will need to be checked, especially in Crowland and Eye Green.

Appendix B: Example: calculating a design flood hydrograph

This appendix first appeared in the MAFF *Engineering Guide* of 1994 (Samuels, 1994) but has been updated for use with current methods and data.

The following calculations use the unit-hydrograph losses model of the Flood Studies Report (FSR) and Volume 4 of the *Flood Estimation Handbook* with modifications as described in Section 4.3 of the main text.

The example is based on a 36.7 km^2 pumped catchment located in east Lincolnshire. Run-off drains under gravity to a pumping station at Anderby Creek (TF 5455 7600). The installation was built in 1946 and consists of two centrifugal pumps, each driven by a 10 RHC diesel engine. The original combined capacity of the two pumps was 4.59 m^3/s when operating at a design gauge head of 3.65 m. Following pumping, run-off drains under gravity the remaining 700 metres to the coast. The pumping station is manned during periods of operation and was due for renewal and automation when this example was produced for the 1994 *Engineering Guide*.

The calculations are laid out as a sequence of numbered steps. Some of the calculations require information from table and figures from the FEH Volume 4; the reference to these is given as FEH V4 Figure X for Figure X in Volume 4 of the FEH and so on.

Step	Commentary	Output
1	The recommended design storm duration is obtained from FEH V4 equation 3.1: D = Tp (1 + (SAAR/1000) where for the six-hour unit hydrograph the time to peak, Tp(6) is assumed to be 24 hours (see Section 4.3.5 of this guide) and annual average rainfall SAAR = 650 mm (Figure B.2). Therefore D = 39.6 hours. For lowland catchments a basic data interval of six hours simplifies the calculations and it is convenient to take D to the nearest odd integer multiple of T.	D = 42 hours
2	The return period for the design run-off must be selected. The storm return period (SRP) is obtained from FEH V4 Table 3.1.	Select 10 years SRP = 17 years
3	The rainfall, P mm, for the storm is calculated follows: The point rainfall with a return period of 17 years for a duration of 42 hours is abstracted from the rainfall depth-duration-frequency data presented on the FEH CD-ROM. This point rainfall estimate is then reduced to a catchment average estimate by applying an areal reduction factor obtained from FEH V4 Figure 3.4, for the area of 36.7 km^2 and duration of 17 hours Hence, rainfall for the storm return period, P in D hours over the catchment = ARF x M17	M17 = 72 mm ARF = 97% P = 70 mm

Step	Commentary	Output
4	The antecedent catchment condition is expressed by the design catchment wetness index, CWI and read from FEH V4 Figure 3.7.	CWI = 95
5	The standard percentage run-off SPR is calculated with the methods in FEH V4 section 2.3. For this case, local information is available that provides the value to be used: The dynamic percentage run-off, DPR_{cwi} representing the increase in percentage run-off with catchment wetness is given by DPR_{cwi}, see FEH V4 equation 2.14: = 0.25 (CWI - 125) = 0.25 (95 - 125) = -7.5 The dynamic percentage run-off, DPR_{rain} representing the increase in percentage run-off from large rainfall events is given by FEH V4 equation 2.15: $DPR_{rain} = 0.45\ (P-40)^{0.7}$ for P>40 mm or $DPR_{rain} = 0$ for P<40 mm Therefore DPR_{rain} $= 0.45\ (70-40)^{0.7}$ = 4.87 The percentage run-off appropriate to the design event is then calculated with FEH V4 equation 2.13: $PR_{rural} = SPR + DPR_{cwi} + DPR_{rain}$ = 42 - 7.5 + 4.87 = 39.37% As there is no urban area in the catchment (ie URBEXT=0) $PR = PR_{rural}$	SPR = 42% DPR_{CWI} = -7.5% DPR_{rain} = 4.87% PR = 39.37%
	The net rain for application to the synthetic unit hydrograph $= PR_{total}\ ^{x}\ P$ = (39.37 x 70)/100 = 27.56 mm	net rain = 27.6 mm
6	The net rainfall is now distributed over the duration D of the storm according to the 75% winter profile of FEH V4 Figure 3.5. The basic data interval T chosen in Step 1 = 6 hours therefore each time interval represents 14.3% of the storm duration. The distribution of rainfall is as tabulated below. Figure B.4 shows the rainfall distribution hyetograph for the design storm.	

Duration	(%)	14.3	42.9	71.5	100
Rain	(%)	34	74	91	100
Incremental Rain	(%)	34	40	17	9
Incremental Rain	(mm)	9.4	11.0	4.7	2.5

Time Interval	(hr)	6	12	18	24	30	36	42
Net Rainfall	(mm)	1.2	2.4	5.5	9.4	5.5	2.3	1.2

Step	Commentary	Output
7	The synthetic unit hydrograph recommended for lowland drainage catchments is trapezoidal in shape and is illustrated in Figure B.4. The peak flow Q_p of the unit hydrograph is given by the following: Q_P= (1.5837AREA)/T_P Area = 36.7 km^2 T_p = 24 hours	Q_P = 2.43 m^3/s per 10 mm rainfall
8	Convolution of the unit hydrograph with the net rainfall pattern may be best carried in tabular form, see Table B.1. The six-hourly ordinates of the unit hydrograph are divided by 10 (the unit hydrograph is for 10 mm of rain); these figures are set down in column 2. Rainfall periods 1-7 (six-hour intervals) are set out along the headings of columns 3-9 together with the net rainfall for the period in mm. The unit-hydrograph ordinates (column 2) are multiplied by the net rainfall for period 1 and the product is set down in column 3 opposite. The process is repeated for each rainfall period, only each successive period is displaced one period (starts one six-hour period lower) because it represents the response to a later element of net rainfall. The row sums give the response run-off hydrograph.	
9	The average non-separated flow (ANSF) per km^2 is calculated using FEH V4 equation 2.19: ANSF = (33 (CWI - 125) + 3.0 SAAR + 5.5) x 10^{-5} = (33 (95 - 125) + (3.0 650) + 5.5) x 10^{-5} = 0.0097 m^3/s per km^2 Baseflow= 0.0097 x 36.7 = 0.354 m^3/s Hence peak flow for a flood with a 10-year return period is 6.74 m^3/s and run-off/km^2 at peak flow = 6.74/36.7 = 0.184 m^3/s per km^2 Figure B.5 shows the calculated run-off hydrograph which has a period of about 12 hours of steady flow from 38 to 50 hours after the start of the storm.	ANSF = 0.0097 m^3/s per km^2 BASEFLOW = 0.35 m^3/s

Table B.1 Anderby Catchment Unit Hydrograph convolution

TIME (hours)	6hr UH m3/s per mm rain	RAINFALL PERIODS 1) 6hrs 1.24	2) 6hrs 2.34	3) 6hrs 5.51	4) 6hrs 9.37	5) 6hr 5.51	6) 6hrs 2.34	7) 6hrs 1.24	Runoff Hydrograph m^3/s	Base flow m^3/s	Total Hydrograph m^3/s
0	0	0							0.00	0.35	0.35
6	0.12	0.15	0.00						0.15	0.35	0.50
12	0.24	0.30	0.28	0.00					0.58	0.35	0.93
18	0.24	0.30	0.56	0.66	0.00				1.52	0.35	1.87
24	0.24	0.30	0.56	1.32	1.12	0.00			3.31	0.35	3.66
30	0.24	0.30	0.56	1.32	2.25	0.66	0.00		5.09	0.35	5.44
36	0.24	0.30	0.56	1.32	2.25	1.32	0.28	0.00	6.04	0.35	6.39
42	0.18	0.22	0.56	1.32	2.25	1.32	0.56	0.15	6.39	0.35	6.74
48	0.12	0.15	0.42	1.32	2.25	1.32	0.56	0.30	6.33	0.35	6.68
54	0.06	0.07	0.28	0.99	2.25	1.32	0.56	0.30	5.78	0.35	6.13
60	0	0	0.14	0.66	1.69	1.32	0.56	0.30	4.67	0.35	5.02
66			0.00	0.33	1.12	0.99	0.56	0.30	3.31	0.35	3.66
72				0.00	0.56	0.66	0.42	0.30	1.94	0.35	2.29
78					0.00	0.33	0.28	0.22	0.84	0.35	1.19
84						0.00	0.14	0.15	0.29	0.35	0.64
90							0.00	0.07	0.07	0.35	0.42
96								0.00	0.00	0.35	0.35

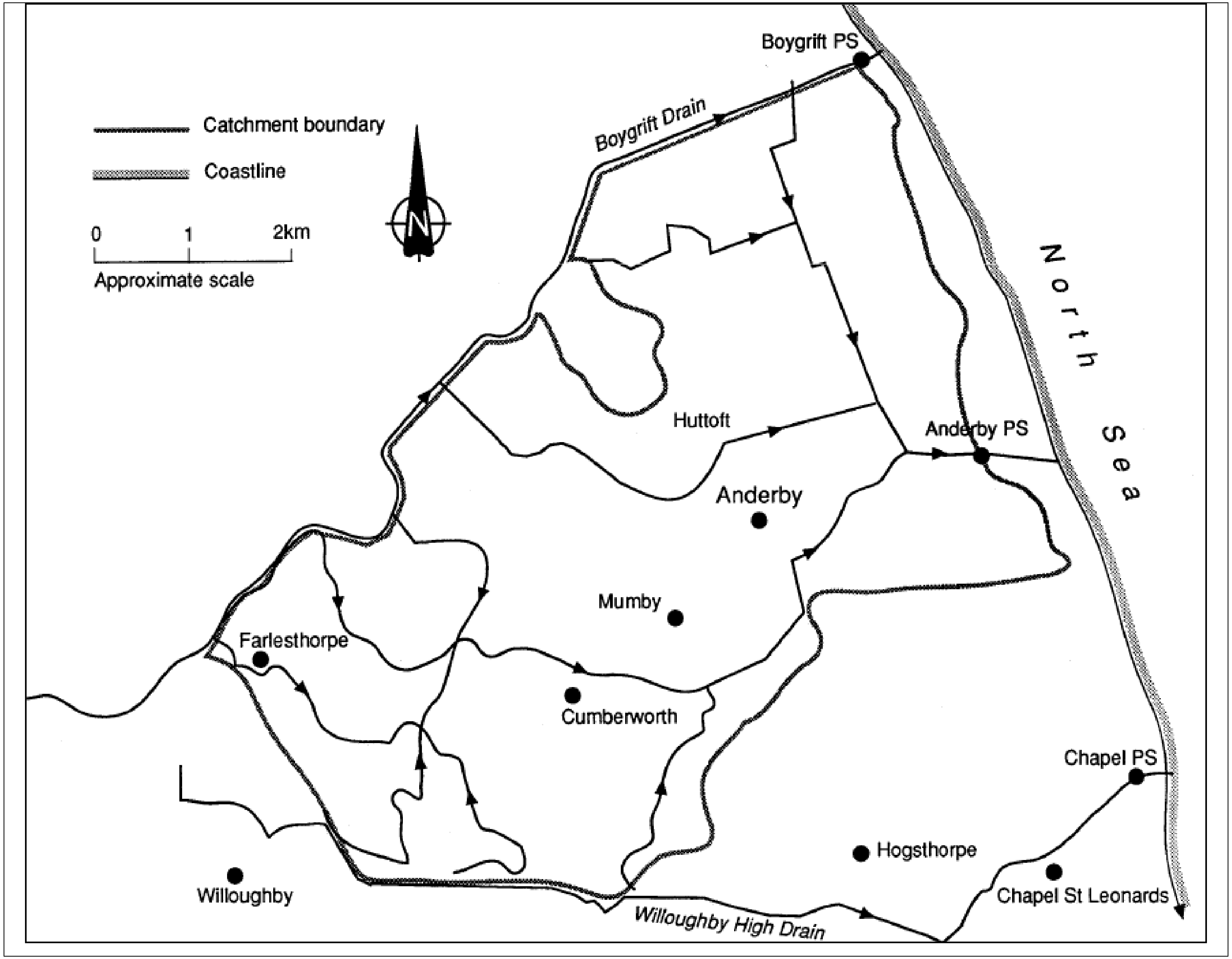

Figure B.1: Anderby catchment plan

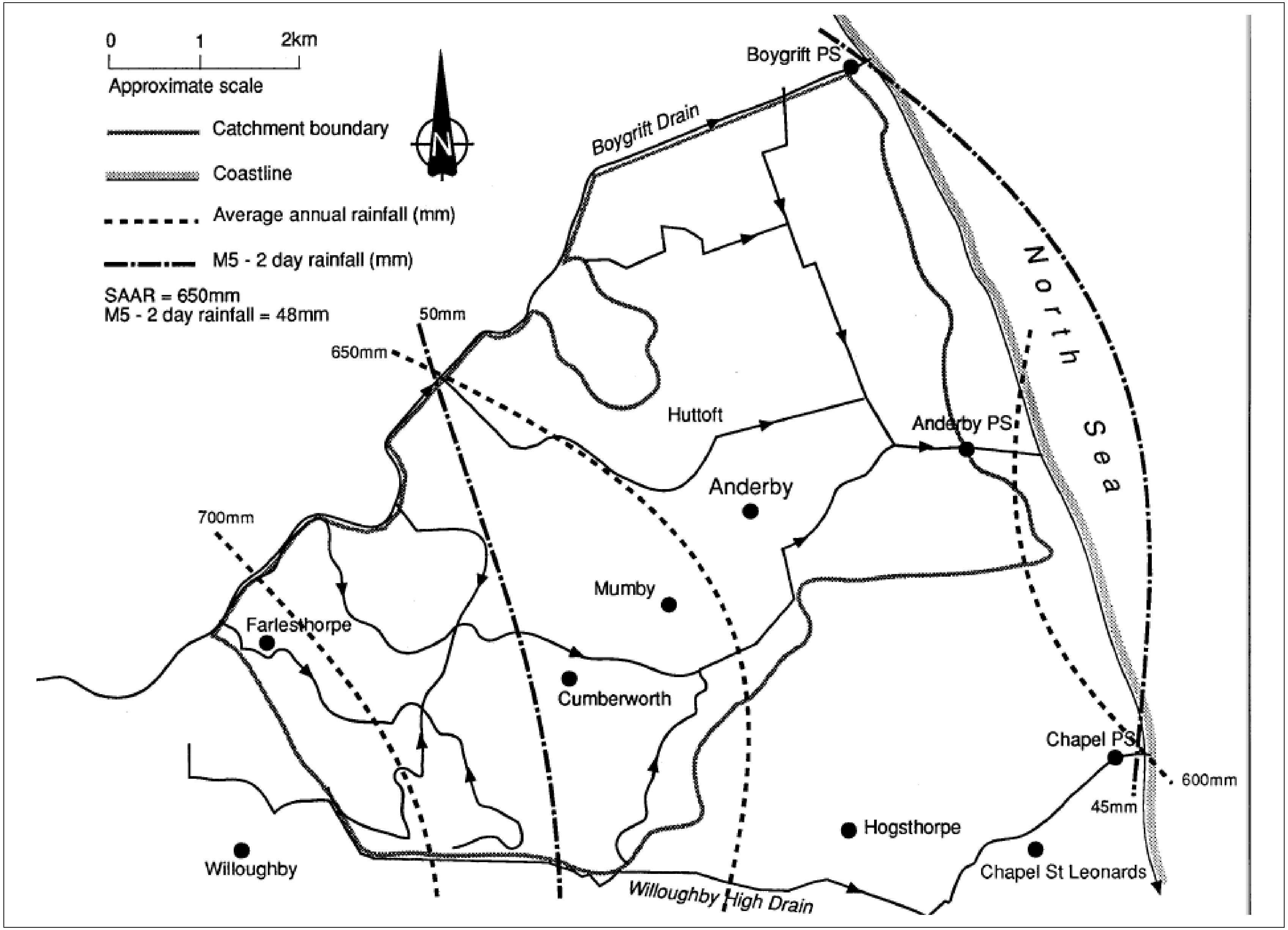

Figure B.2: Anderby catchment and M5-2 day rainfall

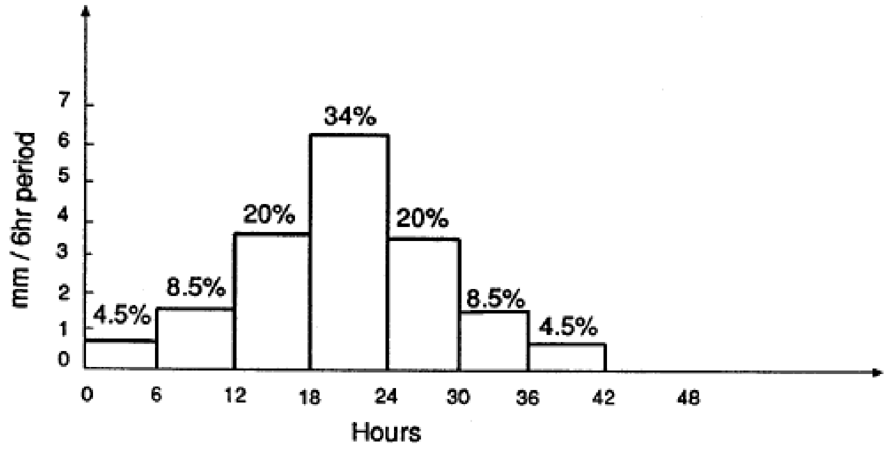

Figure B.3: Rain hyetograph for the design storm

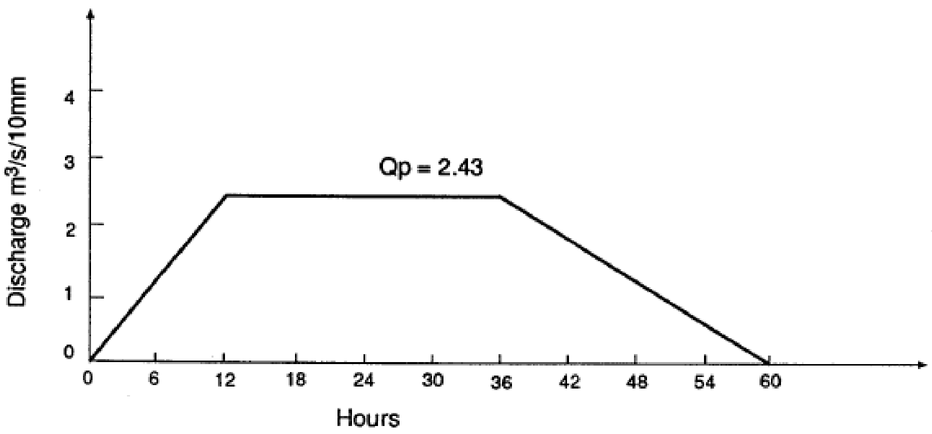

Figure B.4: Synthetic 10 mm – six-hour unit hydrograph

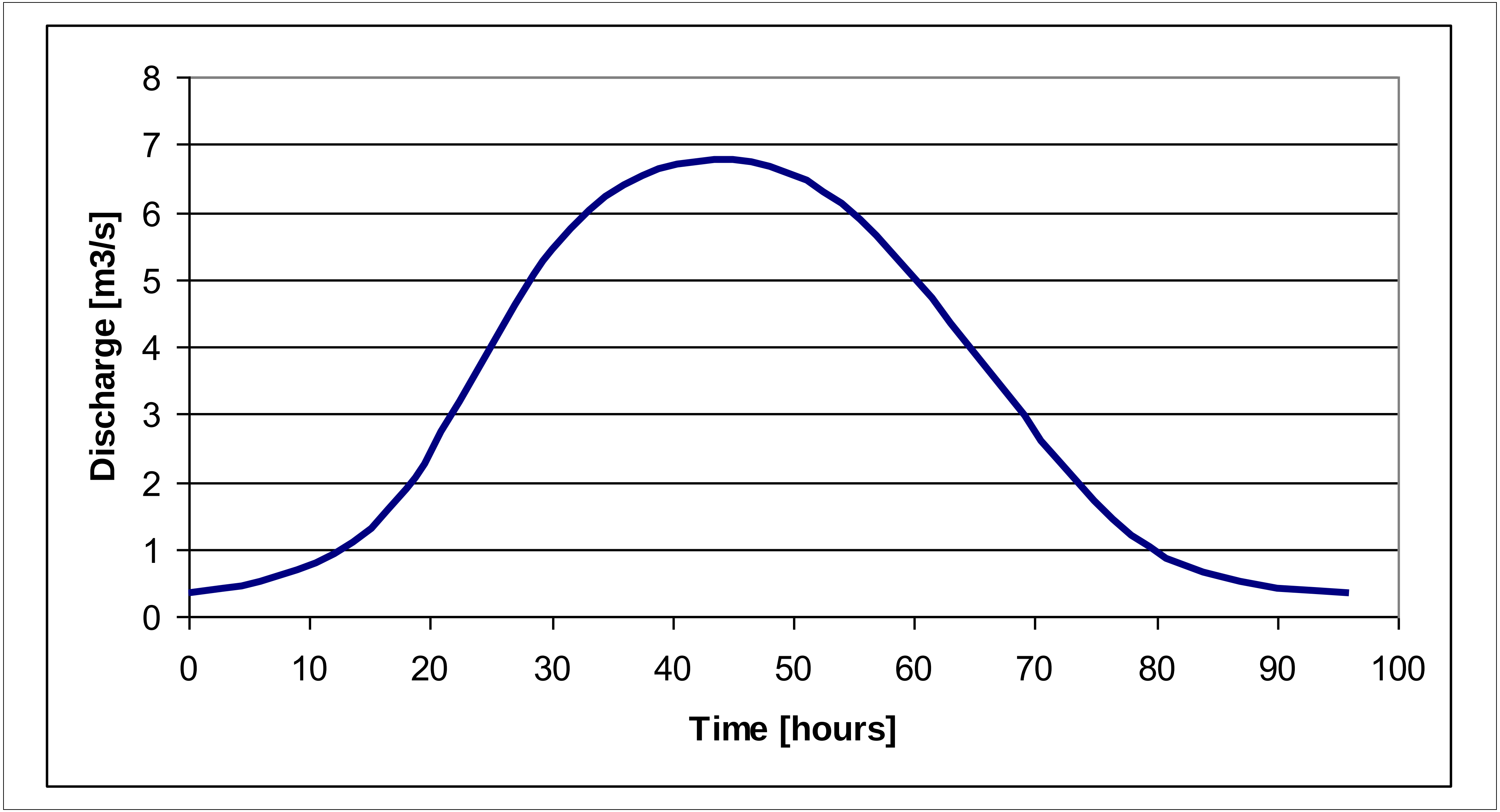

Figure B.5: The design hydrograph

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