The Hydraulics and Hydrology of Pumped Drainage Systems

An Engineering Guide

P G Samuels

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HR Wallingford

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Contract

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Summary

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Between 1978 and 1989, the Ministry of Agriculture Fisheries & Food commissioned a programme of research into the hydraulics and hydrology of pumped drainage systems at HR Wallingford and at the Institute of Hydrology, Wallingford.

This report draws together findings from that research programme which are of direct use in the design, maintenance and operation of pumped drainage systems.

The report covers

- design flood estimation
- channel capacity
- simulation modelling
- computer control of pump operation.

This report was prepared with the assistance of the Institute of Hydrology, the technical committee of the Association of Drainage Authorities and McMillan Computing.

Further enquiries on the topics in this report should be addressed to the Rivers Group at HR Wallingford or the Engineering Hydrology Division of the Institute of Hydrology.

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1 Introduction

1.1 Background

The MAFF sponsored research into the hydrological and hydraulic performance of lowland drainage systems started in 1978 and continued until March 1989. During this period the economic and political factors applied to direct agricultural production changed significantly. In the mid 1970s there was incentive to increase arable crop yields with substantial grants available for drainage and other works. By the late 1980s concern had moved to over-production within the European Community and the environmental impact of flood control and drainage activities; there had also been redirection of MAFF grant aid preferentially to urban flood defence and coastal problems.

These changes in priorities were reflected in the issues studied in the research programme. Early in the research programme, work was undertaken on the economics of pumped drainage and the optimization of new or renovated drainage systems. Later, the research programme addressed the problems of cost effective management and control of water in pumped drainage systems. Thus, the emphasis changed from capital projects to activities funded by revenue.

Even so, the need to provide effective land drainage has not diminished. Many areas of the country, particularly the fen lands of East Anglia, depend crucially upon artificial drainage. Hence, although the factors which motivated the research may no longer apply, the research findings themselves on the performance of lowland drainage systems should be of continuing benefit to drainage engineers.

The research programme drew extensively on information collected at several locations around the UK, with Newborough Fen in the North Level IDB (NLIDB) area featuring most prominently. In parallel with the research, a computer controlled telemetry and water management system was installed at NLIDB which embodied some of the research findings. The system has become known as AFCOPS (Automatic Flood Control Of Pumping Stations) and it proved a valuable practical test bed for the MAFF research. This report covers some of the experience gained in this practical implementation.

1.2 The Research Summary Report

The Research Summary Report (Samuels, 1993) complements this current engineering guide. Both reports have been written with the sponsorship of MAFF as final documentation of the R&D commissioned at HR Wallingford and the Institute of Hydrology (IH). The Research Summary Report contains two main chapters describing the research undertaken by the two organisations. The intention of that report is to provide a reasonably full account of work which was done including some cases of lines of research which were later discontinued. The Research Summary Report contains an appendix giving abstracts of all the papers, reports and other substantial documents which were produced during the research programme. Reference copies of all these documents have been collected and compiled into a single document which is held by HR Wallingford, IH, MAFF and the Technical Committee of the Association of Drainage Authorities (ADA). Copies have also been lodged with the libraries of the Institution of Civil Engineers and the Institution of Water and Environmental Management.

Appendix 1 of this Engineering Guide contains the summary and conclusions from the Research Summary Report and Appendix 2 gives a full list of the papers, reports and other documents produced during the research programme. Some sections of this Engineering Guide are abstracted directly from the Research Summary Report.

1.3 Disclaimer of Liability

This Engineering Guide has been produced using the results generated from research commissioned by the Ministry of Agriculture Fisheries and Food (MAFF) at HR Wallingford Ltd and the Institute of Hydrology which is a component institution of the Natural Environment Research Council (NERC). Whilst this report has been produced using all due skill and care, and in accordance with accepted professional standards, HR Wallingford, MAFF and NERC shall not be liable for the use to which this report is put, nor for the results therefrom, nor for any loss or damage, including consequential losses, arising out of such use.

2 *Designing for flood capacity and routine operation*

2.1 Introduction

Although this chapter concentrates on design for flood conditions, one of the recommendations from the Research Summary Report (Samuels, 1993) is that design of pumping stations and drainage channels should not be divorced from operational considerations. Hence the design should also be tested against the routine performance criteria to ensure cost-effective operation.

Designing for flood capacity involves the following choices amongst others:

- selection of drain dimensions;
- selection of design runoff rates;
- selection of pump capacities;
- selection of pump operating levels.

Here it has been assumed that the locations of the pumping station and the general layout of the drainage network are fixed by historic conditions and practice. The choices are, of course, interdependent but they are discussed separately below.

The Research Summary Report also recommends that effort is put into the collection of information on factors which affect the performance of the existing drainage system should a renovation be contemplated. This should include where possible:

- the boundaries of the catchment to the pumping station;
- potential linkages to neighbouring catchments in times of high flow;
- rainfall and pumping history for any significant storms;
- water level variation in the drainage network under storm conditions;
- current cost of pumping.

This information will allow the design parameter estimates to be adjusted for local conditions and the reduction in running costs to be calculated.

The procedure for the design of lowland drainage systems is covered in Chapter 6 of the IWEM Water Practice Manual no.7 (IWEM, 1987). The IWEM manual embodies knowledge and experienced gained from the MAFF sponsored research and this current Engineering Guide should be used in conjunction with the IWEM manual.

2.2 Design flood runoff

In assessing the design flood runoff it is important to:

- delineate the catchment draining to the pumping station; and
- account separately for runoff from urban areas and peripheral highlands.

The runoff characteristics for lowland areas, which are often artificially drained, differ from those of natural rivers. Lowland areas may have a denser network of farm ditches and other drains than would be typical of higher land; the arable land may have an underdrainage network installed and the water table may be influenced by the operation of land drainage pumps. Likewise the runoff characteristics of urban areas will differ from natural upland and lowland areas. Urban runoff is characterised by short times of concentration and very flashy response to rainfall. The IWEM manual suggests (p189) how the boundaries of the lowland catchment should be assessed taking account of local ground levels, public roads and layout of watercourses. Runoff from the upland and urban areas should be assessed from the appropriate design and analysis methods such as the Flood Studies Report (FSR) (NERC, 1975) with its associated supplementary reports (FSSR) and the Wallingford Procedure (NWC/DoE, 1981).

The design runoff from the lowland catchment can be estimated in several ways:

- from an historic, severe flood;
- from a specified rate in say, mm per day; or
- from rainfall-runoff modelling

A further alternative which has been used is to base the design of the whole system by analogy with another system that functions satisfactorily, although, Reed (1993) cautions against undue reliance of this option. The problem with the first two alternatives listed above is that, although they are simple, no objective measure of the standard of protection (return period) is available. This will be important if cost-benefit analysis is undertaken. Hence the recommended procedure is to use the unit-hydrograph/losses rainfall-runoff technique. Although the general form of the method follows that described in the Flood Studies Report the procedure must be modified to account for the distinct nature of lowland catchments.

The following parts of the FSR procedure **must not** be used for lowland catchments.

- the no-data equation for the mean annual flood;
- the no-data equation for the time to peak, T_p, of the unit hydrograph
- the triangular shape for the design unit-hydrograph; and
- the short-cut convolution procedure, given in FSSR9.

The standard FSR procedures are to be used for:

- selection of the design storm duration;
- selection of the storm return period and precipitation;
- selection of percentage runoff (using FSSR16);
- calibration of the net runoff and unit-hydrograph convolution; and
- assessment of base flow.

The calculation procedure should follow steps (9) to (19) of Section 6.8 of FSR Vol I starting on page 458 as modified by FSSR16. In the absence of local data, T_p should be set to 24 hours. It may be possible to compute a local value of T_p from an analysis of historic rainfall and pumping from the drainage system. If a T_p of 24 hours is used then the design storm duration is related to the average annual rainfall as given in step (1) of the suggested procedure on p182 of the IWEM Water Practice Manual no.7.

A trapezoidal unit hydrograph should be used as in Figure 2.1. The unit hydrograph shown is for 10mm-6hr block of rain (ie 10mm of net rainfall spread evenly over 6 hours). This basic data interval of 6 hours is appropriate for T_p of 24 hours. As an alternative to hand calculation of the runoff hydrograph, the computer package Micro FSR may be used, entering the appropriate value for the data interval T and the unit hydrograph ordinates to give the trapezoidal shape. An example calculation for Anderby pumping station in Lincolnshire is given in Appendix 3.

2.3 Return period, probability and risk

The return period of an event (occurrence or exceedance of water level, depth, discharge, velocity or any other hydrological parameter) is a convenient label to characterise the frequency of occurrence of that event. Return period is conventionally quoted in years. If the analysis is in terms of annual maxima rather than the occurrence of level or flow exceeding a certain amount (peaks over threshold) then, there is no direct meaning to a return period of less than two years. For large values (say 5 years or more) the return period becomes the average frequency of occurrence of the event in a very long sequence. This of course begs the question of whether the processes which force the event (climate, river morphology, human development) are statistically stationary. Non-stationarity of the hydrological processes may influence the statistical assessment in two ways. First of all the underlying statistical model (for example an extreme value distribution) may assume that the data are homogeneous. This may not be the case if, say, the catchment has undergone significant urban development in the period of a flow record. The second influence is illustrated by the effects of long term trends in changing the annual probability of occurrence of a specific flow or level during the design lifetime of the drainage or flood defence works. In this case the assessment of the appropriate design standard should perhaps be based upon the risk of failure during the design life. For the drainage of lowland areas probably the most important long term trend is that of rising sea level which will influence the protection given by tidal defences and the operation of drainage pumps into tidal water courses. There is a danger in the use of the notion of return period for non-specialists. It can be misinterpreted as implying some form of regularity of occurrence; that is a five year level occurs every five years or if it has occurred this year it will not do so next year or if it has not done so for four years it will do so next year. Floods are random.

Return period is usually defined in terms of the annual probability of exceedance, P, by

$$T = 1.0 / P$$

Return periods of less than two years should be estimated as a described in FSSR2.

When analysing a sequence of observations, the return period associated with a given level depends upon several factors:

- the representative nature of the sample used in the estimation (ie are there "too many" wet or dry years?);
- the number of observations (ie record length);
- the statistic analysed (annual peak value, peak over threshold etc);
- the statistical model (or probability *distribution*) used for the random process (eg extreme value (EV) type 1 distribution - the Gumbel plot);
- the method of fitting the distribution to the observed probability values

These factors are considered in setting up standard hydrological analysis software such as HYFAP produced by IH.

A related concept is that of the risk of occurrence of a given event in a particular period. This is given by the risk equation

$$R = [1-(1 - 1/T)^N] \times 100\%$$

Where T is the return period of the event (in years) and N is the number of years over which the percentage risk R is assessed. Thus, the risk of occurrence of a 50 year event in a period of 50 years is 64%. The event with a 50% risk over a period of 50 years has a 73 year return period.

2.4 Channel sizing

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 runoff and the desired design water level at the pumping station. Assuming that the discharge is effectively steady along the main drain, then a standard step backwater method may be used to determine the water levels along the drain see p157 and p186 of the IWEM Water Practice Manual no.7. This procedure is valid because the generally slow response of lowland catchments to rainfall ensures that the flow around the peak is reasonably steady. The channel dimensions should be adjusted so that any design constraints on water level or velocity are met.

The backwater analysis will involve the calculation of flow resistance which determines the conveyance (or transportation) of the drainage channels. The Research Summary Report and Chapter 6 of the IWEM Water Practice Manual no.7 recommend that the Colebrook-White resistance law should be used with a roughness size, k_s, of 1.0m. Engineers may, however, wish to use the Manning equation because of its more simple form for hand calculation. In Appendix 4 it is shown that the resistance law used in the full flow simulations at Newborough Fen is approximately equivalent to 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.

For the design condition of k_s = 1.0m this becomes approximately

$$n = \frac{0.04}{\sqrt{R}}$$

Hence it is recommended that when R is greater than 1.0m, n is set to 0.04 in the design case and to $0.04/\sqrt{R}$ for smaller values of R. For other values of k_s an equivalent formula for n must be deduced following the analysis outlined in Appendix 4. These recommended values of n and k_s were found to be representative of the Newborough Fen drainage system studied in the course of the MAFF research. If too low a value of the roughness coefficient is used in the assessment of channel capacity then it will result in under-design. Likewise, inadequate maintenance will also compromise the channel capacity. In such cases, the design flood flow will not be conveyed to the pumping station within the constraints of water level or velocity set for the design. In extreme circumstances the freeboard on the drains may be insufficient to allow the full installed pump capacity to be utilised for any reasonable length of time and the water levels in the channels which are remote from the pump will remain too high. A further cause of inadequate operation of automatic pumps is setting the levels too low at which the pumps switch on and off. This may be done 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.

2.5 Pump capacity and operation

The installed pump capacity should be capable of discharging the design flood runoff (as determined from the procedures in Section 2.2). Following this the dimensions of the main drains should be set so that, for the required water surface profile, the conveyance is sufficient to pass the design discharge to the pumping station as discussed in Section 2.4. Next, the best means of providing the overall total pumping capacity must be considered. 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;
- 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 nominating the 'duty' pump on rotation; and
- 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 with obvious advantages.

The water levels at which the pumps are turned on or off are usually sensed automatically in the drain. These 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. The choice of switching levels cannot be isolated from the nature of the energy tariff. The cost of electricity varies according to the time of day, day of the week and the month of the year. Thus, different operating levels should be chosen to avoid unnecessary pumping in periods of high energy cost.

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. This is an over-estimate for a severely drawn down (M_2) profile and an under-estimate for a (M_1) profile, see Figure 2.2. 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. It should be noted that Figure 2.2 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 2.2 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 this situation prevails 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 potentially a much enhanced volume of storage available 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 produce significant on-line storage for normal flow conditions.

Reed (1993) introduces a convenient measure of storage in terms of pump-hours. 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 when compared with the volume of the design flood. 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.

2.6 Simulation modelling

2.6.1 Introduction

Having advocated steady flow analysis for the design of channels in a lowland catchment the limitations of this simplified method must be clearly stated. The method will not produce any information on the timing of pump runs or on the attenuation of runoff through storage within the drainage channels. The effects of the peak discharge distribution in the channel network can, however, be included in the analysis. The most severe restriction will be 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. In such cases an unsteady flow simulation model should be used to assist the design of the drainage channel dimensions. An unsteady flow model will account for the attenuation of urban runoff by the storage in the drainage channels and the relative timings of the urban, upland and lowland catchment runoff.

2.6.2 The rôle of full flow simulation

The analysis of the pump backwater and pump cycle time given in the Research Summary Report is only approximate, in that the dynamic effects are ignored. In order to track the performance of a drainage system under historic or hypothetical inflow conditions full dynamic modelling is necessary. The reasons for this are as follows.

- The length of the pump backwater is different for drawdown and ponded (M_2 and M_1 water surface profiles) below and above the normal depth line. This is due to the non-linearity in the flow resistance formulae (Samuels, 1989).

- Only under the design flood condition with pumps running at full capacity, will the flow conditions be steady.

- The conveyance of the drainage channels (and hence the length of pump backwater and the pump cycle time) depend upon the drain roughness which can be influenced significantly by weed growth (see Section 2.7).

- Water balance studies do not give specific information on the water surface profiles within the drainage system.

Information on these factors at the design stage of a new or renovated pumping station and drainage channels can only be generated from a dynamic simulation model such as that used in the research programme. In selecting a model for design purposes it is crucial that this has an appropriate treatment of flow resistance. Appendix 5 describes the application of the HR model to the Sutton and Mepal drainage system.

2.6.3 The rôle of storage models

Having argued that a full dynamic model is needed to simulate the actual performance of the whole drainage system, the use of a simple storage method for flow forecasting and control requires some justification. This comes from the different character of the problem being addressed.

The objective of operational control strategies such as OCOPO (Reed, 1993) is to phase pump use with times of cheap energy costs whilst maintaining satisfactory land drainage conditions. The control strategy does not require the water level at some site remote from the pumping station to be maintained within a certain close band, or velocity in the drain to remain below a threshold. If such parameters are required to be controlled then a means of predicting them would be necessary; this would entail full dynamic modelling. However, the objective set is more simple and a model of the system performance, which is consistent with that more limited objective, can be used. If during the pump operations, the water levels fluctuate broadly within the same range throughout most of the year, then the effective storage within the system can be represented as linear between a few observed water levels. This embodies the assumption of similarity between the water surface profiles in different pump operating cycles. The calibration of the storage (or *stock model* according to Reed (1993)) will include the effects of small distributaries and unmeasured storage and is described by Reed (1993). At Newborough Fen two calibrations of the stock model for horizontal water levels were undertaken. The first was based on design drawings supplemented by some measurements of the drain geometry. In the second calibration the stock was estimated from observations of water levels before and after pump runs in periods of small but steady inflow which gave effectively a horizontal water surface in the drain before and after a pump run. This field calibration demonstrated the need to adjust the stock model calibration from the survey by about 15%. For the application of OCOPO in the North Level IDB area, the stock model was based on observations at two points in the drainage system. An alternative in the absence of observation would be to tune a storage model using the results of a full dynamic simulation containing the whole channel network. This, however, has not been attempted during the course of the research programme. The ultimate test of the suitability of the simple storage model as a concept for automatic control of pumping must lie in the fact that, after calibration at Newborough Fen, it has delivered realistic advice.

2.7 The hydraulic effects of weed

In separate research funded by MAFF, HR Wallingford has been examining the hydraulic resistance of vegetation mainly in natural rivers. Although the species of weed may be different from that typical in artificial drainage channels, the gross effects of the weed on channel conveyance are likely to be similar.

The principal effects of vegetation are:

(1) to reduce the effective cross-section area

(2) to increase the effective wetted perimeter
(3) 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. On this basis, the field measurements in a natural river gave observed values of Manning's n of up to 0.3 in the most severe cases for a cross section where the resistance coefficient was 0.034 in the weed-free state. The results are presented by Whitehead (1992), but the equations presented in that report for estimating the effective value of Manning's n should not be used as definitive for lowland drainage channels. Larsen, Fries and Vestergaard (1990) present field observations from a stream in Denmark where the flow was artificially controlled. These conditions may be more representative of those in lowland drainage systems; again, Manning's n values of 0.3 were found in some conditions.

Although definitive design information for the roughness of vegetation is still needed, the message of these field observations is clear. Substantial vegetation within a drainage channel will severely impair its conveyance. This will prejudice the standard of flood defence offered by the drainage system and will also influence the operation of the pumps by restricting the pump backwater. This will reduce the pump cycle time and may cause 'hunting' (Slade, 1985). The engineer should monitor the condition of the drainage channels throughout the year and take effective action to control weed when appropriate by cutting, mulching, biological control or herbicides. The decision on when to remove weed is primarily a matter of experience.

2.8 Other influences on pump operation

During the MAFF sponsored research programming, the effect 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 the effects of a strong wind on the Newborough Fen channels which was to change water levels by about 0.1m. Such changes in level 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 water-course at a pumping station) is to modify the effective head on the pump. Each design of pump will have its own operating characteristic relating discharge and energy consumption to the operating head. Pumping stations discharging into tidal water-courses 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 is 55% greater than at low water (Marshall, 1993) though at certain times of the year differential electricity costs show an even greater variation. Marshall discusses the potential for altering pumping patterns to reduce the need to pump at high tide for this site.

3 Computer controlled operation

3.1 Introduction

In the later stages of the MAFF R&D project, attention turned to the automatic computer control of land drainage pumps. The research ideas were tested at the Newborough Fen Catchment in the North Level IDB area where telemetry had been installed in the early 1980s. The telemetry provides information on water levels, rainfall, and pump states over the catchment and this is fed to an engineers' console for manual control of the pumping operations. The telemetry and control system is called AFCOPS (Automatic Flood Control of Pumping Stations) and the original installation has been augmented by the OCOPO forecasting and control algorithm developed at the Institute of Hydrology (Reed, 1993). The remainder of this chapter describes the technical background to the computer control of pumping and the experience gained of its implementation at North Level IDB. Appendix 6 contains a further description of the hardware, software and software support for the NLIDB installation.

3.2 Hydrology, hydraulics and control algorithms

Providing real time control advice for the drainage system poses different challenges from those encountered in the design of the pumping station and the main drainage channels. In the design case the inflow to the drainage system is calculated from a hypothetical design storm, probably with a single peak (see Appendix 3) and the runoff generated assuming an average antecedent catchment condition. These assumptions are not valid for the control problem. Fortunately, the catchment response times are reasonably long compared with the review intervals implicit in the electricity tariffs. This means that rainfall forecasting has not proved necessary in the implementation of the computer control at North Level IDB. Nevertheless, it is important to use the recorded rainfall patterns and account for the actual state of catchment wetness when producing runoff forecasts.

The hydrological response of the Newborough catchment is represented by a relatively simple rainfall-runoff model. This links rain falling on the 32.5 km^2 catchment to the resultant runoff rate. The catchment rainfall is estimated by readings from a tipping bucket rain-gauge which is sited just outside the catchment and registers each 0.5 mm of rainfall that accumulates. For the type of rainfall-runoff model used it is convenient to express the rainfall rate and runoff rate in common units of mm per hour.

The rainfall-runoff model used at Newborough Fen is a nonlinear storage model, and is described by Reed (1993). It is based upon the net rainfall, lagged by a fixed time interval since all rainfall entering the storage is first delayed. The term "net rainfall" means net of any losses. The "losses" includes any process by which rainfall is prevented from running off. For fenland catchments, the most obvious losses are infiltration and surface detention (eg on vegetation or in puddles). The "routing" behaviour of the nonlinear storage model represents the effects of the net rainfall being temporarily detained by vegetation, reaching the ground, passing through the upper layers of the soil, then to a field drain or minor watercourse, before finally arriving at the main drain.

The low gradient and predominantly rural nature of fenland catchments give rise to a relatively slow response to rainfall. Thus there is no hydrological requirement to consider rainfall data at a very fine time interval; research at Newborough Fen suggested that a data interval of 3 hours would suffice. However, implementation of the rainfall-runoff model to simulate and forecast runoff in real time is complicated by other factors, notably the electricity tariff and the need to consider operational periods of uneven length. Use of a half hour data interval allows a match to be obtained with the operational periods applicable at Newborough, and is therefore adopted as the basic data interval in the real-time implementation.

The telemetry system maintains, and updates, a record of half-hourly rainfall depths over the last 24 hours. Because of the need to allow time for telemetered data to be gathered and processed, and the pump decision software executed, the periods used in assessment are a quarter of an hour in advance of the pump operating periods. Thus, for example, the decision on the number of pumps to be used in the operating period beginning at 07.30 is based on telemetered observations up to 07.15.

The volume of water stored within the drainage channels is represented by a simplified model of the drain geometry, calibrated for the catchment (see Sections 2.5 and 2.6.3 and Reed (1993)). The control algorithm computes the pumping required from the actual volume of water in the drainage channels (as represented by the stock model), an assessment of the inflow, the target conditions and the electricity tariff structure. There may be a difference the pumped volumes requested by the algorithm and that which actually occurs for several reasons:

- the channel capacity may be insufficient for the pump to run for the requested duration;
- the engineer may intervene; or
- the pump may fail.

Hence the control algorithm is based upon the water stored in the drain calculated from the telemetred water levels and not from an accumulation of recommended pump running hours.

The control algorithm can be fine tuned to take account of the nature of cost penalties incurred by running the pumps. For example adjustments can be included which deter pumping:

- between 1600 and 1900 GMT during December and January;
- on Thursday and Friday if electricity is cheaper on Saturday and Sunday;
- towards the end of the month for second or third pumps if the maximum demand charge is calculated on calendar month basis.

Alternatively pumping may be encouraged if, after the next break (or break but one) in the tariff structure, the energy cost becomes punitively high.

The algorithm reviews regularly the net runoff rate from changes in water stored in the drainage network, the volume pumped and the recorded rainfall. This allows the runoff estimates to include recent changes in the catchment condition rather than depending upon some artificial reference state as used in the design analysis.

3.3 Telemetry

Clearly some means must be provided to transmit data from outstations, such as pumping stations, to a central controller. Drainage Boards have usually seen telemetry - monitoring at a distance - as being the way to provide this. A typical system of telemetry provides remote sensors and transmits data back to a host. However, experience has shown that the use of standard telemetry is not well suited to the special needs of land drainage. Two problems arise; firstly, many telemetry systems provide large numbers of readings at each remote site. Secondly, an IDB needs to perform complex and specialist calculations on the data.

In fact, for each poll of a rain or water level sensor the values of this one parameter need only to be read. For pumping and gate stations, more data are needed. For example, sensors have been used to check pump rotation and syphon valve positions.

A telemetry system can be expensive to install, and may not present information in the best way for a drainage engineer. Certainly standard telemetry will not provide any automatic control of pumping. The solution adopted at North Level IDB is a central computer that runs special software and communicates with transmitters at the outstations. There are a number of communications products available which are much cheaper than full telemetry. The latest stations installed at North Level use low cost radio modems, with all control built into the software of the central computer. The present AFCOPS system consists of ten pumping stations, eighteen remote water level stations and two rainfall stations. These latter stations are located in the eastern and western part of the Board's area respectively.

3.4 Experience gained at NLIDB on the practical side of AFCOPS

The system has allowed the North Level IDB, at any time of the day or night, to check water levels and verify the state of all pumping stations using either the central control station based at the Thorney office or the mobile engineer's station in a vehicle or at a Duty Officer's home. The two stations give an easily understood display screen and print-out respectively. Both of the control units also allow the starting and stopping of pumps at any of the pumping stations and the opening and closing of a gravity outfall sluice. Audible alarms, when a fault occurs or when high and low water levels exist, are transmitted to the central control unit during office hours and outside of these hours to the Duty Officer. These alarms continue until acknowledged.

Some of the pumping stations are in remote locations and a mains failure alarm has enabled the Duty Officer to notify the local Electricity Board of such a failure prior to that Board being aware of the fault. A further example is that, when a differential of 300mm in water levels exists either side of the weed-screen, an alarm is transmitted and usually indicates a weed-screen blockage. This can be dealt with promptly and avoids the intermittent operation of the pumps. Over the several years of operation a single person has been able to monitor water levels and the condition of all of the pumping stations from a dedicated unit rather than incurring high expenses in employing several people to check visually the water levels and operate the pumps as required.

Moreover the continuous updating and appraisal of water levels at the pumping stations and the upstream remote water level stations, allows the engineer to decide whether pumping may be inhibited in order to enable optimum use of the "off-peak" electricity tariffs. Whilst the electricity tariffs have changed over the past years, experience has shown that savings can be made in electricity costs without adversely affecting the Board's land drainage responsibilities.

As the system allows for the continual monitoring of water levels and pumping station conditions and provides immediate operation of pumps, the Board, in recent years, has agreed to raise water levels during the summer months above the normal maximum water levels without any due concern. These higher water levels provide greater depths of water in drains and are certainly more environmentally acceptable.

All of the information collected through the automatic and intermediate polling of all of the stations is archived and the information can be retrieved to verify previous water levels and pumping station states if subject to any complaints or queries. The software also allows for the analysis of all of the information available, in report and graphical form. In report form, the water levels in the separate catchments are recorded together with the times and number of pumps operated within the period. The separate electricity tariffs have been incorporated within the software and the report identifies the period of time that the pumps have operated within each charge tariff. This information can be used to verify electricity accounts and has previously been utilized to calculate an electricity account when failure of the Electricity Board's time clock occurred.

The graphical format gives a combined visual display, over any period of time, of the pumping station water level, upstream water level/s, rainfall amounts and the number and duration of pump operations. This display again helps in the decision making of whether to delay pumping or not and aids the engineer in understanding and appreciating how the individual pumping catchments react to rainfall events. The analysis of the reports and graphs should enable the drainage engineer to design the optimum efficient and cost effective pumping station and associated improved drainage channels.

3.5 Management information

One benefit of AFCOPS is that details of rainfall, water levels and pumping are available from the Personal Computer. As information is received from an outstation, it is copied to a disc file for the outstation and programs have been provided for analysing these data.

Typically, a report is requested monthly which provides tabular data of pumping for a drain; Figure 3.1 shows a sample of a report for the Newborough drain during a storm in 1988. A line is printed for every time at which data were received by the central controller. Data may be recorded water level or a change in the number of running pumps. Whenever a pump starts or stops, the total pump hours since the last change is computed and shown on the report. This figure for pump hours is analysed across the electricity tariffs. For example at 1.27 on 25 January, two pumps had been running for 62 minutes, giving a total of 2.07 pumping hours. Of this, 0.17 pumping hours (2 pumps for 5 minutes) had taken place at the higher charge which was in effect before half past midnight. At the end of the report, the pump hours are multiplied by the nominal pump rating and the electricity charge to work out the cost of pumping.

It is also possible to plot a graph of rainfall, levels and pumping against time. The graph is divided into three sections: a rainfall hyetograph, plots of water levels and pump operation. Figure 3.2 shows a sample graph plotted for the same period as the report. The dates (25th and 26th) are shown below the y axis, with each graticule representing an hour. The hyetograph and level curves - a sharper curve at the pumping station and more gradual one upstream can be seen clearly, as can the pump operation. The graphs are especially useful for Board meetings, as they clearly demonstrate the performance of the drain system.

3.6 Cost savings

North Level IDB has calculated a saving of £215,000 during the first ten years of operation. Much of this has come from a reduction in staffing by three since the scheme was installed. It is sobering to compare the monitoring and control using the AFCOPS scheme with the logistics of manual checks on the water levels and operation of the pumps. It can take best part of a day to drive round a sizeable catchment. By contrast the engineer can request a report of all 30 stations from the office. This takes around ten minutes to generate, during which time the engineer is free to do other tasks. Pumps may be switched from the central console, vehicle stations or from the Duty Officer's house.

Energy savings are potentially large. During the storm of January 1988 the AFCOPS software was being tested, and whilst it recommended pumping, it did not actually control the pumps. It was found that the software recommended running all pumps during the night periods, whereas the existing method used less than half the available night rate. It is rare for North Level IDB to pump outside the night period. An analysis of another large Board has found that little more than half the pumping was done at the cheapest rate, even during the dry year of 1991/92. Marshall (1993) found that, at the Boy Grift pumping station in Lincolnshire, just under 30% of pumping was achieved at the cheapest electricity tariff rate (see Table 1 of Marshall's paper). Savings will clearly vary from Board to Board, but a 30% reduction in electricity charges may be possible under a typical tariff.

4 Conclusions

The MAFF sponsored research and practical implementation of computer control by North Level IDB have led to improved understanding of the behaviour of lowland pumped drainage systems. Important conclusions of the studies which form the basis of this engineering guide are as follows:

(1) The design of new or renovated pumping stations and drainage channels should not be divorced from operational considerations.

(2) Design inflows for flood conditions should be estimated by the Flood Studies Report procedures as modified in Section 2.2 of this engineering guide.

(3) The conveyance capacity (or transportation) of the drainage channels should be calculated from the Colebrook-White equation (see Appendix 3) with a roughness size k_s of 1.0m.

(4) Manning's equation for conveyance calculations may be used for the larger channels close to a pumping station, with a constant value of n but with n varying with depth for shallow channels. A design value for Manning's n of 0.04 is recommended for R > 1.0m and $n = 0.04/\sqrt{R}$ for smaller values of R.

(5) In the absence of computer control the switching levels at which pump operation commences and ends should be set to maximise the disposable volume in the pump backwater. This can be achieved by setting the switch-on level for each pump at a level equal to or higher than the normal depth in the channel for that pump discharge.

(6) The pump switching levels may be adjusted to encourage pump operation in periods of cheap electricity tariff.

(7) Excessive weed growth can severely limit the conveyance capacity of drainage channels. Hence weed control is essential for water management in the drainage network.

(8) Although, initially high in cost, telemetry and computer control of pump operation are practical with current technology and can provide significant benefits in terms of:

- reduced labour costs
- reduced energy costs
- availability of information on system failures; and
- management information.

Further conclusions from the research as presented in the Research Summary Report are contained in Appendix 1.

5 Acknowledgements

It is clear from the authorship of the publications listed in Appendix 2 that many people contributed to the research programme. The preparation of this report has been facilitated by the cooperation of Dr Duncan Reed and David Marshall of the Institute of Hydrology. Rod Hunt of North Level IDB and John McMillan of McMillan Computing Services contributed draft text on their experience with the implementation of AFCOPS at North Level IDB. Representatives of the ADA technical committee reviewed this report at the draft stage and David Sisson contributed the material for Appendix 3. The Author is also grateful for the encouragement of David Noble, Secretary of ADA, to produce this engineering guide and to colleagues at HR Wallingford for their comments on the draft report.

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Figures

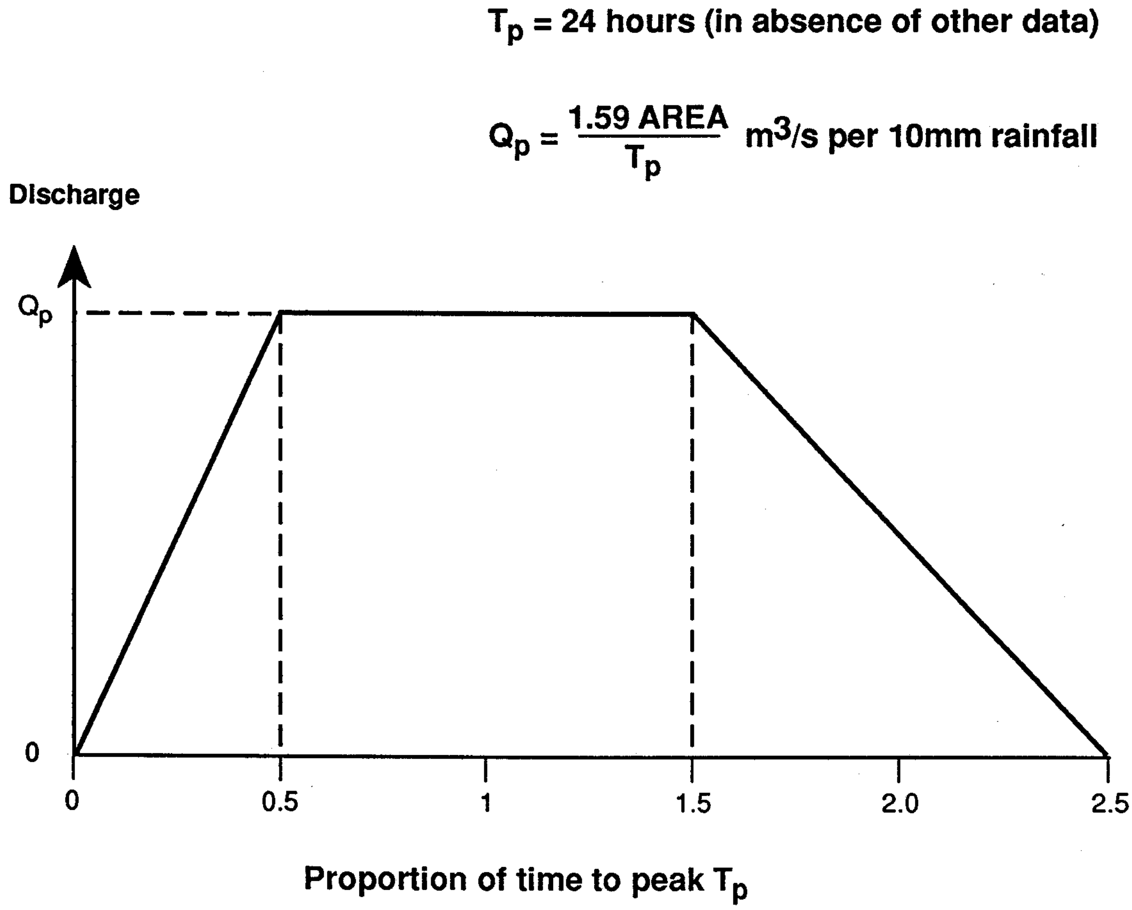

Figure 2.1 Recommended unit hydrograph for lowland catchments

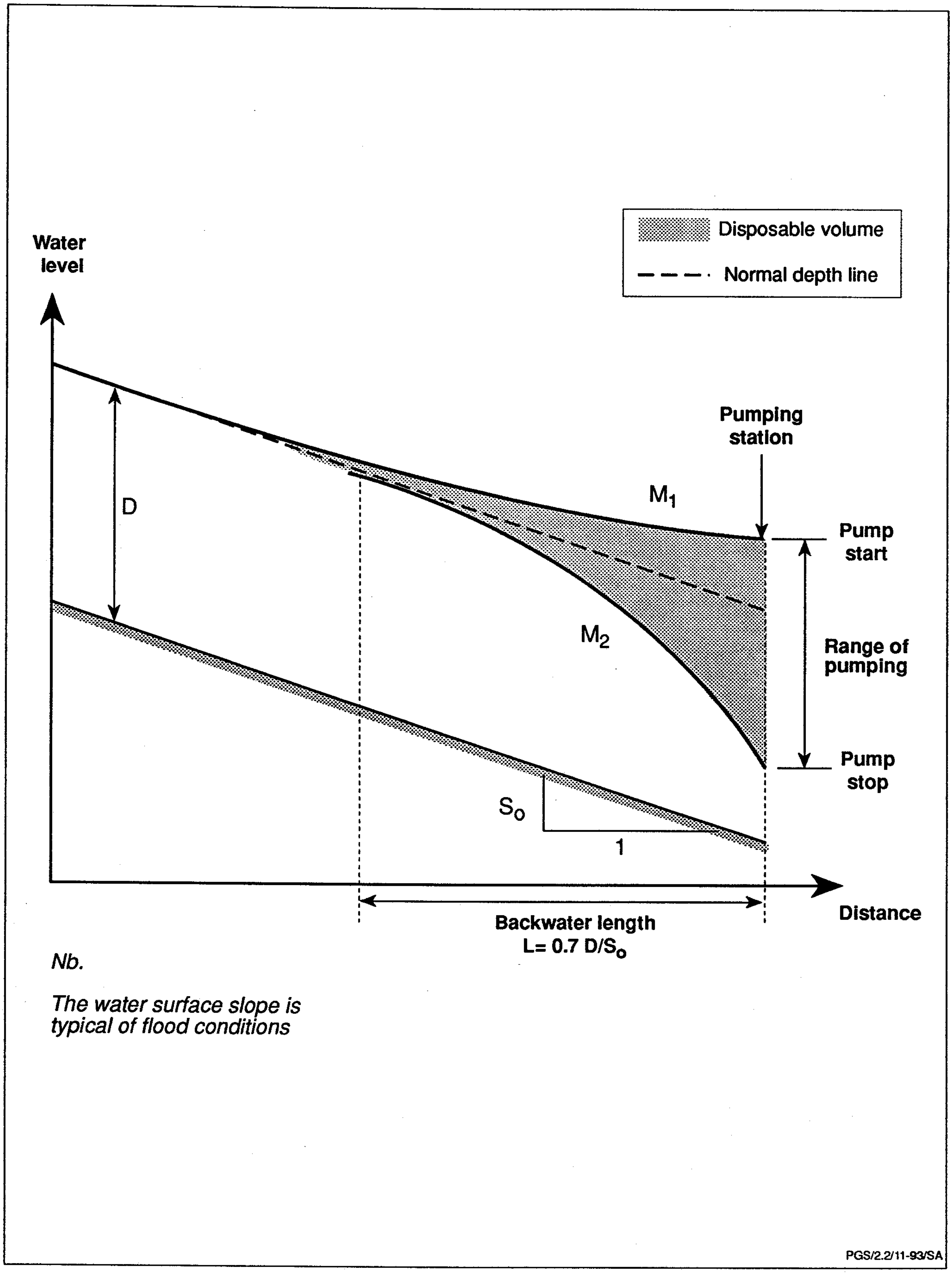

Figure 2.2 The pump backwater and disposable volume

History at Newborough Page 1

Time	Level	Remote Levels	Pumps	Total pumping charge 1	charge 2	charge 3	charge 4
11:45			0	14.00	12.27	0.00	0.00
24.01.88							
11:45	-1.160	-0.440	0				
11:47			2				
13:33			1	0.00	3.53	0.00	0.00
14:10			2	0.00	0.62	0.00	0.00
15:22			1	0.00	2.40	0.00	0.00
15:38			2	0.00	0.27	0.00	0.00
15:45			0	0.00	0.23	0.00	0.00
15:45	-1.200	-0.470	0				
15:47			2				
18:20			1	0.00	5.10	0.00	0.00
18:45			0	0.00	0.42	0.00	0.00
18:45	-1.310	-0.520	0				
18:47			1				
19:45			0	0.00	0.97	0.00	0.00
19:45	-1.210	-0.540	0				
19:47			1				
20:12			2	0.00	0.42	0.00	0.00
21:34			1	0.00	2.73	0.00	0.00
25.01.88							
0:25			2	0.00	2.85	0.00	0.00
1:27			1	1.90	0.17	0.00	0.00
3:45	-1.250	-0.640	2				
5:27			2	4.00	0.00	0.00	0.00
6:17			1	1.67	0.00	0.00	0.00
7:15			0	0.97	0.00	0.00	0.00
7:15	-1.290	-0.620	0				
7:17			1				
7:30			0	0.22	0.00	0.00	0.00
8:14			1				
11:45			0	0.00	3.52	0.00	0.00
11:45	-1.080	-0.560	0				
11:47			1				
15:45			0	0.00	3.97	0.00	0.00
15:45	-0.990	-0.550	0				
15:47			1				
15:49			0	0.00	0.03	0.00	0.00
18:45	-0.730	-0.530	0				
19:45	-0.670	-0.500	0				
20:18			1				
20:20			3	0.00	0.03	0.00	0.00
22:37			2	0.00	6.85	0.00	0.00
Total pump hours				22.75	46.37	0.00	0.00
Unit charge				2.14	4.25	7.10	38.00
Cost @194 KVA				94.45	382.29	0.00	0.00

Figure 3.1 Sample tabular output from AFCOPS

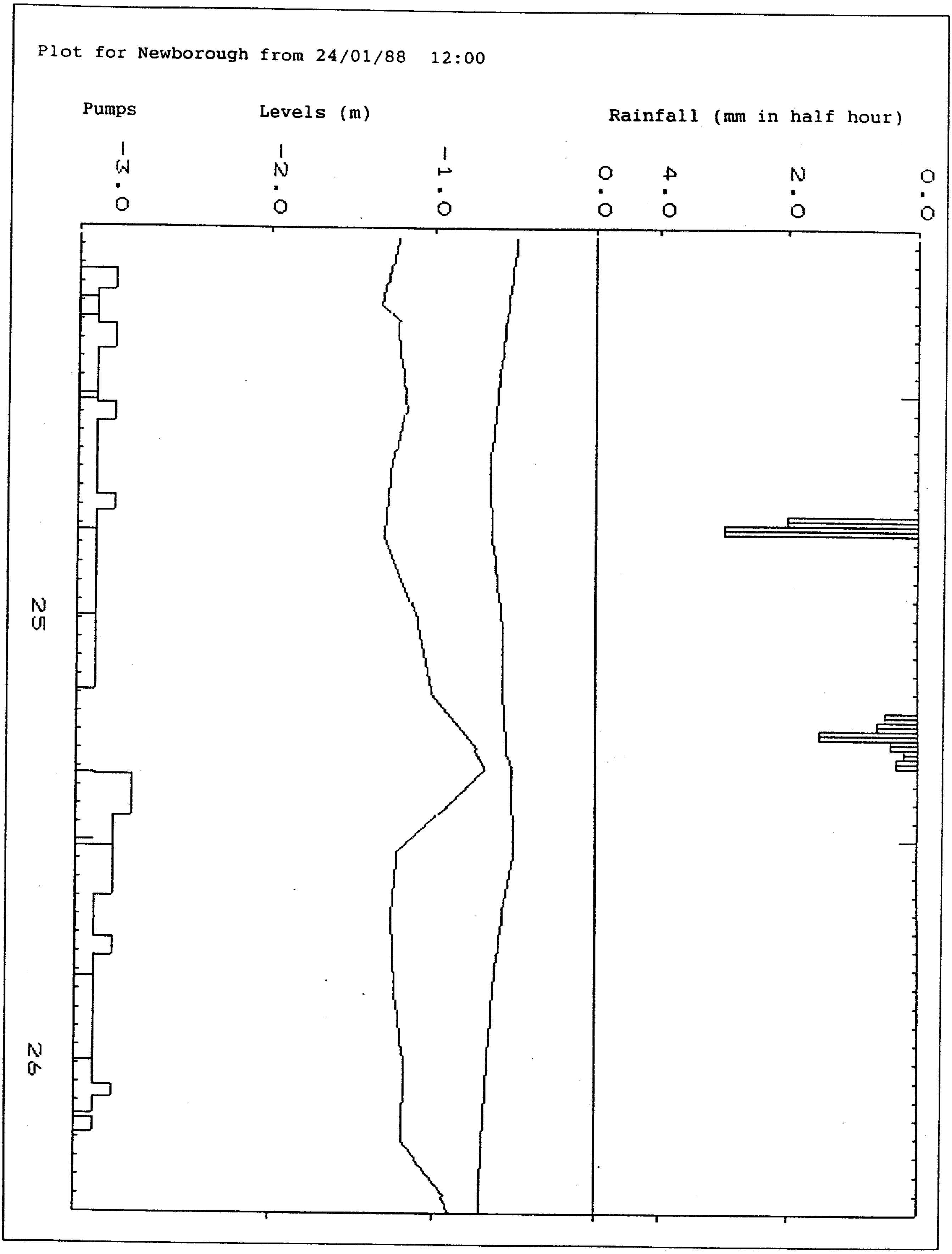

Figure 3.2 Sample graphical output from AFCOPS

Appendices

Appendix 1

Conclusions and Recommendations from
the Research Summary
Report SR 330

Appendix 1 Conclusions and Recommendations from the Research Summary Report SR 330

The principal conclusions of the research were as follows:

(1) The actual standard of installed pumping capacity in lowland drainage systems varies around the UK, with return periods varying from about 1 in 2 years to 1 in 70 years. The full capacity of many land drainage pumping stations is rarely utilized. The scope of research did not include an assessment of the appropriate design standard of protection. Such standards are influenced by flood defence policy, public expectation, environmental impact and the costs, benefits and financing of any proposed scheme.

(2) Should any improvement or restoration be considered to a drainage system, then a data collection exercise should be started at the earliest opportunity to provide information on rainfall, pumping and water levels to aid the understanding of the current behaviour of the catchment.

(3) The design inflows to pumped drainage systems can be calculated within the framework of the Flood Studies Report (FSR) unit hydrograph losses model. The validity of the standard percentage runoff estimate was confirmed and a new trapezoidal design unit hydrograph is recommended.

(4) Some aspects of the FSR method are not appropriate to lowland drainage systems; in particular the standard no-data equations should not be used to estimate the mean annual flood discharge nor the time to peak of a unit hydrograph.

(5) In the absence of local data, a time to peak of 24 hours should be used in unit hydrograph synthesis of the design flood.

(6) Operational control of land drainage pumps is possible using telemetered information on rainfall, water level and pump running times, coupled with flow forecasting. Automation of control can lead to reduction in operational costs and improve the quality of information available on the system management; these and other benefits must be considered alongside the cost of the telemetry system.

(7) Simple storage based models may be calibrated for use in operational computer control of pumping operations.

(8) The design of pumping stations and drainage channels should not be divorced from operational considerations.

(9) The efficient operation of the pumping station for normal flow conditions should be considered when selecting the number and size of pumps to achieve the design flood capacity.

(10) The principal rôle of storage within the drainage system is to allow economic phasing of pumping during normal flow conditions rather than as a means of reducing the maximum flows in the design flood condition.

(11) A key parameter in the design of a pumped drainage system is the conveyance (or transportation) of the open channels. Traditional methods of estimating conveyance can lead to under-design of channel capacity.

(12) A roughness size of 1.0m should be used in the full Colebrook-White resistance law for the design of drainage channels. Alternatively a roughness value of 0.040 may be used with Manning's equation for $R > 1.0m$ and $n = 0.04/\sqrt{R}$ for smaller values of hydraulic radius, R.

(13) Weed growth within drainage channels can severely impair their conveyance. Quantitative estimates of the roughness associated with particular weed conditions, however, are uncertain for drainage channels.

(14) Efficient operation of land drainage pumps requires the backwater influence of the pumps to reach as far inland from the pumping station as possible.

(15) A full dynamic model is necessary to simulate the water surface profiles and discharges in detail within a drainage network and optimise system design for both flood or normal flows.

(16) Correct simulation of the full range of flow conditions, from dry season to flood flows, in a computational model requires careful choice of the conveyance formulae and numerical procedures used within the model.

(17) Channel sizing for the design flood condition may be undertaken using traditional backwater analysis.

(18) Future hydraulic modelling of drainage channels should allow for a general cross-section geometry rather than being restricted to a trapezoidal section shape. Non trapezoidal, compound cross-section shapes may be preferable to satisfy environmental criteria.

(19) In the study of optimum drain geometries, the costs associated with land acquisition dominated the choices made. The optimal solutions obtained minimised the drain widths subject to other constraints being satisfied.

(20) An economic appraisal of land drainage schemes can be made using the methods developed by the Flood Hazard Research Centre of Middlesex University.

(21) In some cases, still-water set-up due to strong winds may adversely influence the operation of land drainage systems. However, wind speeds of less than force 6 are unlikely to be of much significance.

Appendix 2

References to research publications

Complete sets of these publications are held by

HR Wallingford
Institute of Hydrology
Institution of Civil Engineers
Institution of Water and Environmental Management
Ministry of Agriculture Fisheries and Food
Technical Committee of the Association of Drainage Authorities

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(22) Slade J E (1987), *Dog-in-a-Doublet pumped drainage catchment Peterborough,* Un-numbered report, HR Wallingford, (March 1987)

Appendix 3

Example calculation of a design flood hydrograph

Appendix 3 Example calculation of a design flood hydrograph

The following calculations use the unit-hydrograph losses model of the Flood Studies Report (FSR) with modifications from the supplementary report FSSR 16 and the hydrological studies undertaken in the MAFF R&D programme 1 (see Section 2.2 of the main text).

The example is based on a 36.7km^2 pumped catchment located in east Lincolnshire. Runoff 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 cumecs when operating at a design gauge head of 3.65m. Following pumping, runoff drains under gravity the remaining 700 metres to the coast. The pumping station is manned during periods of operation and is due for renewal and automation.

The calculations are laid out as a sequence of numbered steps. Several of these are identical to the form of calculation given in Section 6.8.2 of Volume I of the FSR, starting on p482. For ease of cross reference, the FSR step number is given in square brackets []. Some of the calculations require information from table and figures from the FSR, the reference to these is given as Figure I.6.44 for Figure 6.44 in Volume I of the FSR etc.

Step	[FSR step]	Commentary	Output
1	[9]	The recommended design storm duration is obtained from FSR equation I.6.46 D = (1.0 + (SAAR/1000) Tp where for the 6-hour unit hydrograph the time to peak, Tp(6) = 24 hours (see Section 2.2 of the main text) and Annual Average Rainfall SAAR = 650mm (Figure A3.2) Therefore D = 39.6 hours. For lowland catchments a basic data interval of 6 hours simplifies the calculations and it is convenient to take D to the nearest odd integer multiple of T.	D = 42 hours
2	[10]	The return period for the design runoff must be selected. The storm return period (SRP) is obtained for Figure I.6.61.	Select 10 years SRP = 17 years

Step	[FSR step]	Commentary	Output
3	[11]	The rainfall, P mm, for the storm is calculated follows: The ratio r is found from Volume IV Figure II.3.5(S)	r = 39%
	[3]	The ratio for the five year 42hr to the five year 2 Day rainfall M5-42h/M5-2D is interpolated from Volume I Table 6.21 (p460) or Volume II Table 3.10 as M5-42h/M5-2D = 89 + [(106-89)/(48-24)] (42-24)	102%
	[3]	M5-2D rainfall is found from Volume IV Figure II.3.2(S) (see Figure A3.2)	M5-2D = 48mm
		M5-42hours = 1.02x48 = 49mm	M5-42hrs = 49mm
		From Table II.2.7, the growth factor MT/M5 is assessed for the storm return period	M17/M5 = 1.28
		Storm return period rainfall, M17 = 1.28 x 49 = 63mm	M17 = 63mm
		This point rainfall estimate is then reduced to a catchment average estimate by applying an areal reduction factor obtained from Figure II.5.1 - Area = 36.7km^2	ARF = 97%
		Hence, rainfall for the storm return period, P in D hours over the catchment = ARF x M17	P = 61mm
4	[12]	The antecedent catchment condition is expressed by the design Catchment Wetness Index, CWI and read from Figure I.6.44	CWI = 95

Step	[FSR step]	Commentary	Output
5	[14] [Using FSSR 16]	The Standard Percentage Run off, SPR is expressed in terms of the proportions of the soil types S1, S2, S3 etc. and is given by SPR = 10S1 + 30S2 + 37S3 + 47S4 + 53S5 The relative proportions of the catchment occupied by the various classes S1 to S5 are determined from Vol IV Figure I.4.18 (S), (see also Figure A3.3)	S2 = 0 S5 = 0 S1 = 0.24 S3 = 0.40 S4 = 0.36
		Therefore SPR = (10 x 0.24) + (37 x 0.4) + (47 x 0.36) = 34.12%	SPR = 34.12%
		The Dynamic Percentage Run-off, DPR_{CWI} representing the increase in percentage run-off with catchment wetness is given by DPR_{CWI} = 0.25 (CWI - 125) = 0.25 (95 - 125) = -7.5	DPR_{CWI} = -7.5%
		The Dynamic Percentage Run-off, DPR_{rain} representing the increase in percentage run-off from large rainfall events is given by $DPR_{rain} = 0.45(P-40)^{0.7}$ for P>40mm or $DPR_{rain} = 0$ for P<40mm	
		Therefore DPR_{rain} $= 0.45\ (61-40)^{0.7}$ = 3.79	DPR_{rain} = 3.79%
		The percentage run-off appropriate to the design event is then calculated as $PR_{rural} = SPR + DPR_{CWI} + DPR_{rain}$ = 34.12 - 7.5 + 3.79 = 30.41%	PR = 30.41%
		As there is no urban area in the catchment (ie URBAN=0) $PR_{total} = PR_{rural}$	

Step	[FSR step]	Commentary	Output
5 Cont		The net rain for application to the synthetic unit hydrograph $= PR_{total} \times P$ $= (30.41 \times 61)/100$ $= 18.55mm$	net rain = 18.5mm
6	[16]	The net rainfall is now distributed over the duration D of the storm according to the 75% Winter Profile of Table II.6.3. 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 A3.4 show the rainfall distribution hyetograph for the design storm.	
7	[17]	The synthetic unit hydrograph recommended for lowland drainage catchments is trapezoidal in shape and is illustrated in Figure A3.5. The peak flow Q_p of the unit hydrograph is given by the following: $Q_p = (1.59 Area)/T_p$ cumecs/10mm Area = 36.7km^2 T_p = 24hours (see Section 2.2 of main text)	$Q_p = 2.43$ cumecs/10mm

Duration	(%)	14.3	42.9	71.5	100
Rain	(%)	34	74	91	100
Incremental Rain	(%)	34	40	17	9
Incremental Rain	(mm)	6.3	7.4	3.15	1.7

Time Interval	(hr)	6	12	18	24	30	36	42
Net Rainfall	(mm)	0.85	1.58	3.7	6.3	3.7	1.58	0.85

Step	[FSR step]	Commentary	Output
8	[18]	Convolution of the unit-hydrograph with the net rainfall pattern may be best carried in tabular form, see Table A3.1. The six hourly ordinates of the unit-hydrograph are divided by 10 (the unit hydrograph is for 10mm of rain) these figures are set down in column 2. Rainfall periods 1-7 (ie 6 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 (ie starts on period lower) because it represents response to a later element of net rainfall. The row sums give the response run-off hydrograph.	
9	[19] [of FSSR 16]	The average non-separated flow ANSF per km^2 is calculated using ANSF = (33 (CWI - 125) + 3.0 SAAR + 5.5) x 10 E - 5 = (33 (95 - 125) + (3.0 650) + 5.5) x 10 E - 5 = 0.0097 cumecs/km^2	ANSF = 0.0097 cumecs/km^2
		Baseflow= 0.0097 x 36.7 = 0.354 cumecs	BASEFLOW = 0.35 cumecs
		Hence peak flow for a flood with a 10 year return period is 4.65 cumecs and run-off / km^2 at peak flow = 4.65/36.7 = 0.127 cumecs/km^2	
		Figure A3.6 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.	

Table 3.1 Anderby Catchment - Unit Hydrograph Convolution

TIME (hours)	6hr UH m^3/s per mm rain	RAINFALL PERIODS							Runoff Hydrograph m^3/s	Base flow m^3/s	Total Hydrograph m^3/s
		1) 6hrs 0.85mm	2) 6hrs 3.70mm	3) 6hrs 3.70mm	4) 6hrs 6.30mm	5) 6hr 3.70mm	6) 6hrs 1.58mm	7) 6hrs 0.85mm			
0	0	0							0	0.35	0.35
6	0.12	0.10	0						0.10	0.35	0.45
12	0.24	0.20	0.19	0					0.39	0.35	0.74
18	0.24	0.20	0.38	0.44	0				1.02	0.35	1.37
24	0.24	0.20	0.38	0.89	0.76	0			2.23	0.35	2.58
30	0.24	0.20	0.38	0.89	1.51	0.44	0		0.42	0.35	3.77
36	0.24	0.20	0.38	0.89	1.51	0.89	0.19	0	4.06	0.35	4.41
42	0.18	0.15	0.38	0.89	1.51	0.89	0.38	0.10	4.30	0.35	4.65
48	0.12	0.10	0.29	0.89	1.51	0.89	0.38	0.20	4.26	0.35	4.61
54	0.06	0.05	0.19	0.67	1.51	0.89	0.38	0.20	3.89	0.35	4.24
60	0	0	0.10	0.44	1.13	0.89	0.38	0.20	3.14	0.35	3.49
66			0	0.22	0.76	0.67	0.38	0.20	2.23	0.35	2.24
72				0	0.38	0.44	0.29	0.20	1.31	0.35	1.66
78					0	0.22	0.19	0.15	0.56	0.35	0.91
84						0	0.10	0.10	0.20	0.35	0.55
90							0	0.05	0.05	0.35	0.40
96								0	0	0.35	0.35

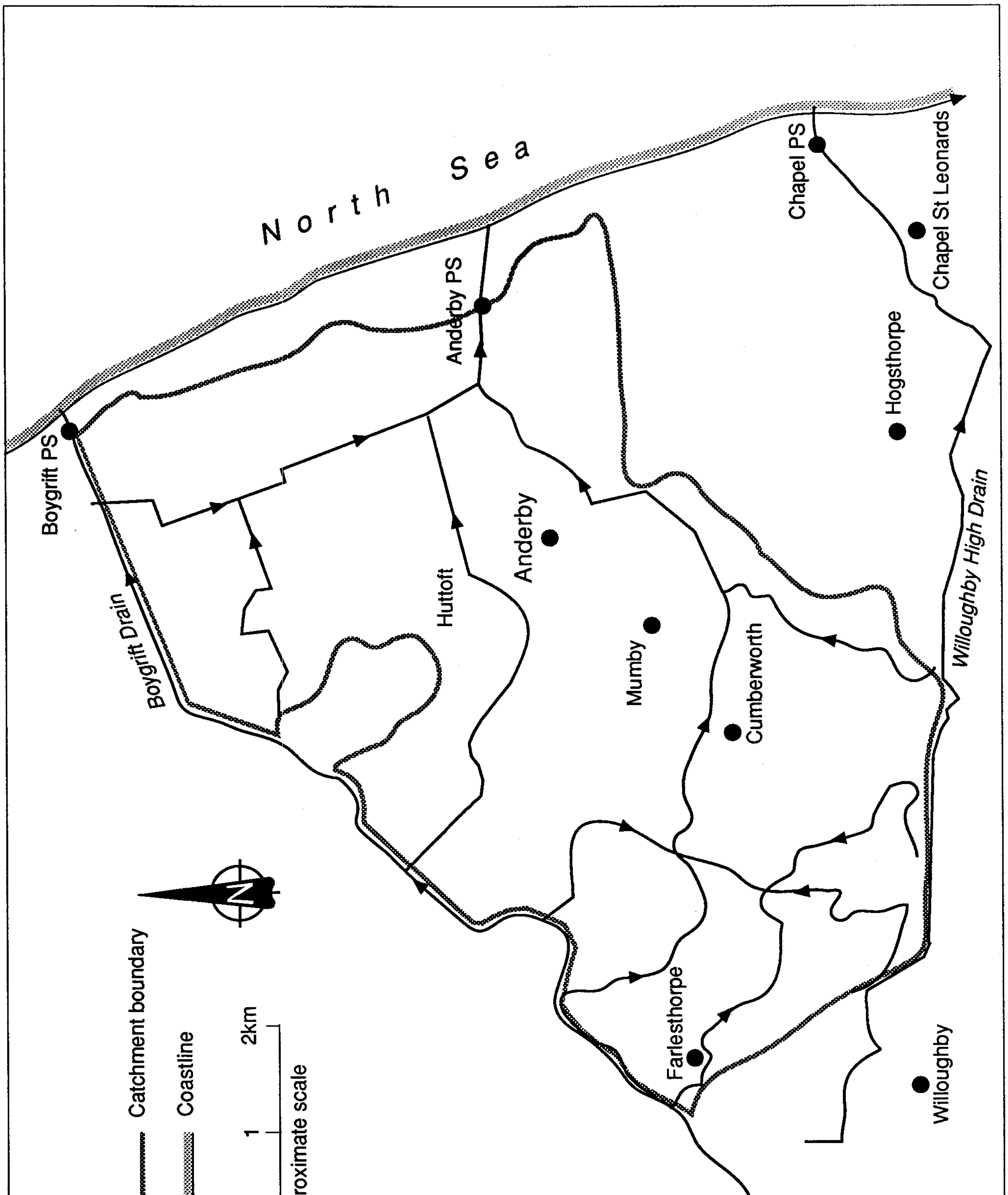

Figure A3.1 Anderby Catchment Plan

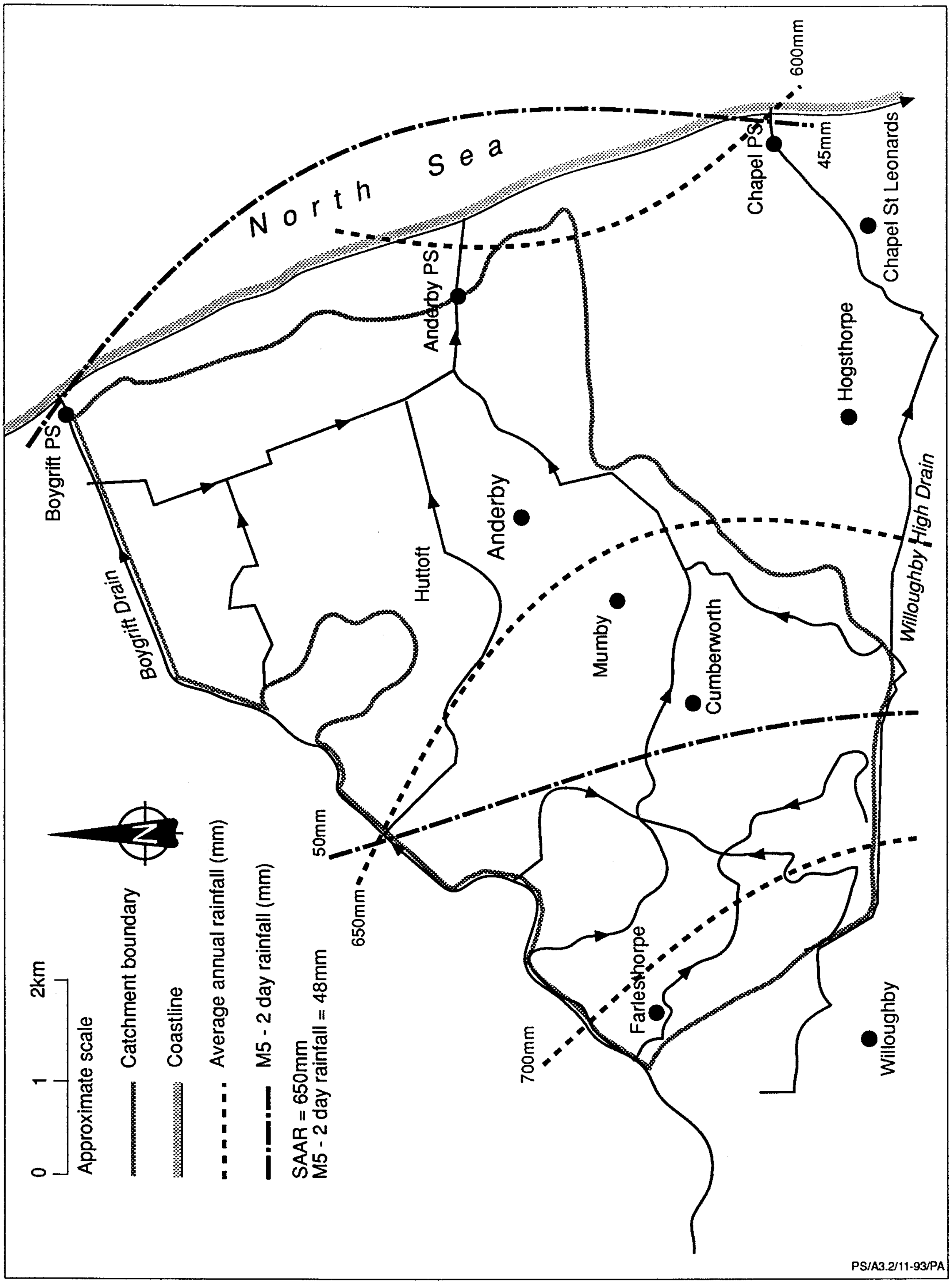

Figure A3.2 Anderby Catchment and M5-2 day rainfall

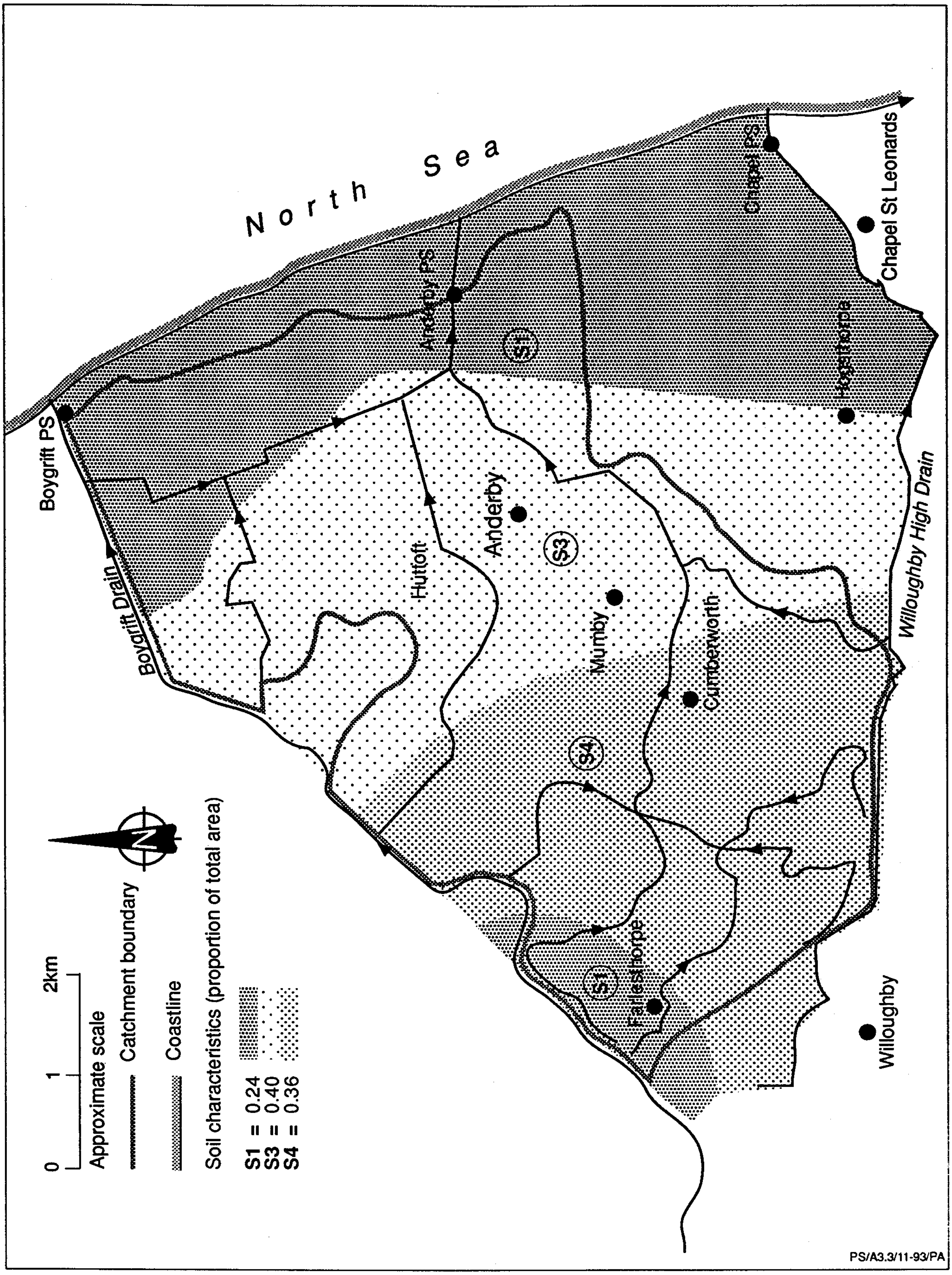

Figure A3.3 Anderby Soil Characteristics

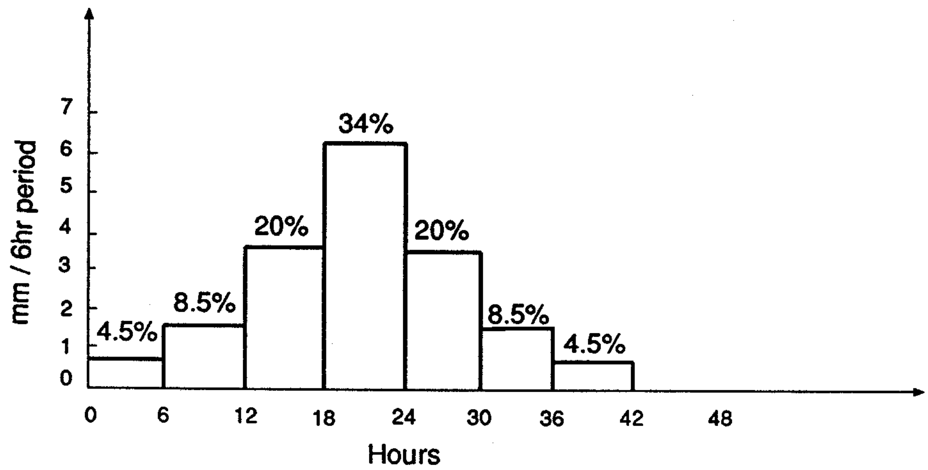

Figure A3.4 Rainfall hyetograph for the design storm

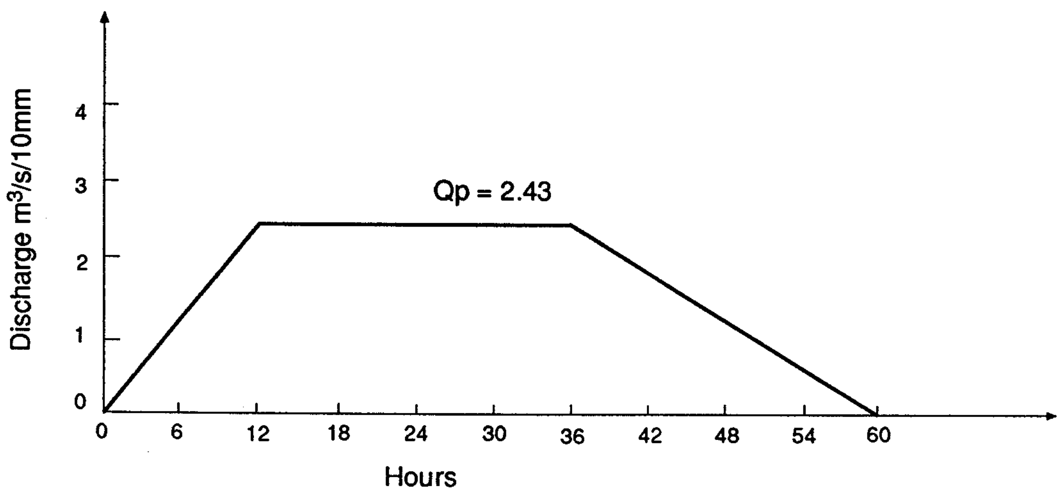

Figure A3.5 Synthetic 10mm - 6 hour unit hydrograph

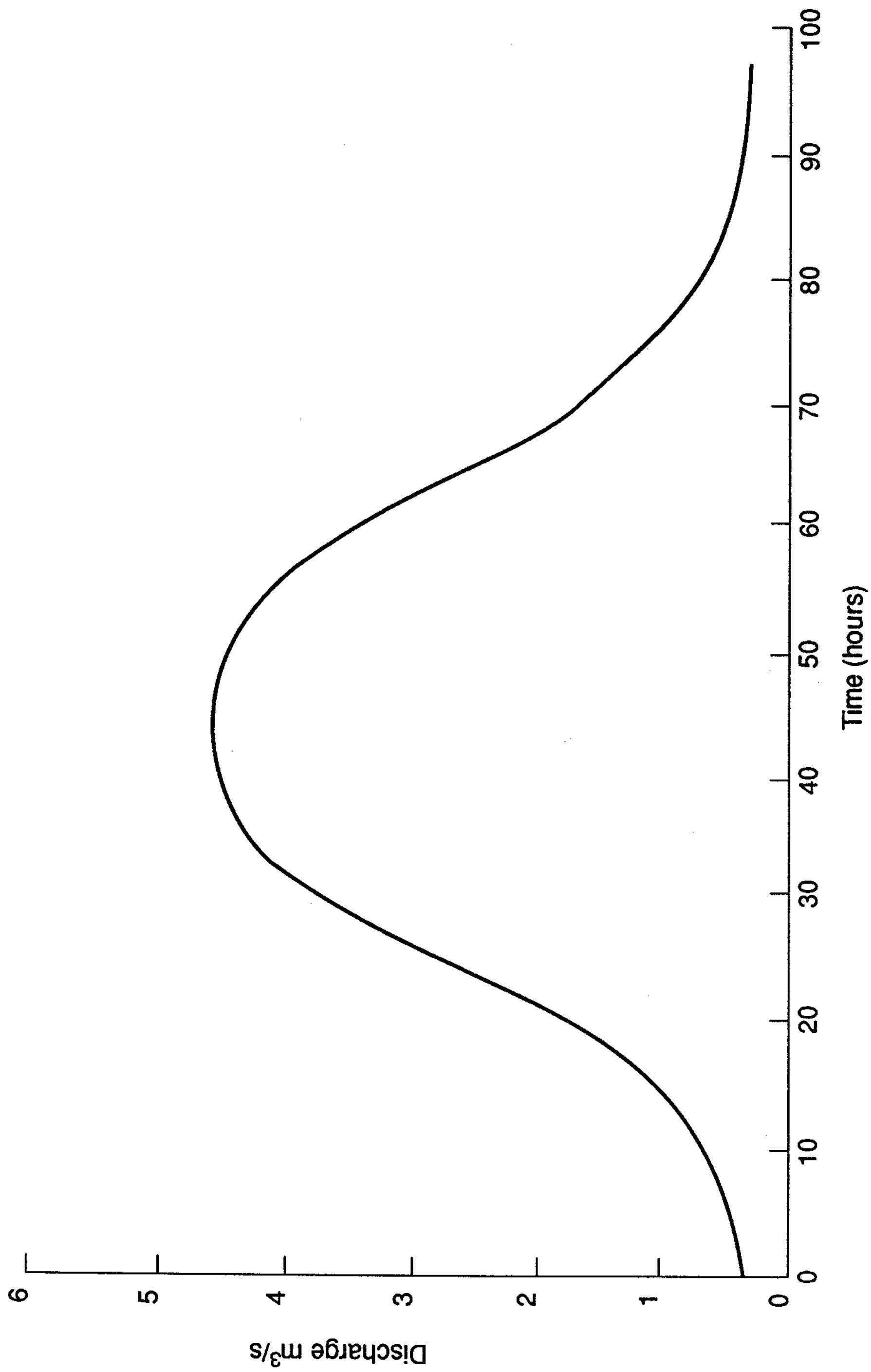

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Figure A3.6 The design hydrograph

Appendix 4

Conveyance formulae

Appendix 4 Conveyance formulae

The friction slope S_f in a channel is given by the formula

$$S_f = \frac{Q|Q|}{K^2} \qquad (1)$$

where Q (m^3/s) is the flow rate (discharge) and K (m^3/s) is the conveyance. Good estimates of conveyance are essential for predicting the variation of water level within a drainage network. Historically there have been many attempts to quantify conveyance in terms of channel geometry, flow velocity, slope, texture of the boundary etc. The most commonly used equation for channels, by UK engineers, is the Manning equation

$$K = \frac{AR^{2/3}}{n} \qquad (2)$$

where A is the flow area, R = A/P is the hydraulic radius and n is the Manning roughness coefficient. Manning's n has been related to typical descriptions of boundary texture, see for example French (1986) and much of the analysis of the effects of vegetation is performed in terms of changes in "n" (eg Whitehead, 1992).

However, there is ample evidence that Manning's n is not a constant for a channel but depends upon the flow depth even for a uniform boundary texture.

Colebrook and White developed the formula which bears their names based upon experimental observations of artificially roughened pipes and a semi-empirical description of the velocity profile within a cross-section. The Colebrook-White equation for turbulent flow is

$$K = (8gR)^{1/2} A/\sqrt{f} \qquad (3)$$

$$\frac{1}{\sqrt{f}} = -2.0 \log_{10}\left[\frac{k_s}{14.83R} + \frac{2.52}{Re\sqrt{f}}\right] \qquad (4)$$

where f is the Darcy friction factor and Re is the Reynolds number

$$Re = \frac{4QR}{A\nu} \qquad (5)$$

Here $\nu \approx 1.1 \times 10^{-6}$ m^2/s is the kinematic viscosity of water.

The coefficients 2.0, 14.83 and 2.52 in equation (4) are valid for pipe flow and may require modification for open channels. In equation (4), when the term $k_s/14.83R$ dominates, the flow is called "rough" turbulent and when the term $2.52/(Re\sqrt{f})$ dominates, the flow is "smooth" turbulent. Equation (4) should be used for Reynolds number above about 2000 to 3000. Below this a transition to laminar flow, with its higher resistance takes place. A deficiency of this resistance equation for open channel flow is that it has not been proven experimentally at small vales of relative roughness (R/k_s).

Resistance equation is that it has not been proven experimentally for small vaues of relative roughness (R/k_s).

For a typical drainage channel under flood conditions we may have

$$\frac{Q}{A} \approx 0.3\,m/s$$

$$R \approx 1.5\,m$$

giving Re = 4 x 10^5. Hence the flow is turbulent in this case. However, in nearly dry, shallow conditions

$$\frac{Q}{A} \approx 0.05\,m/s$$

$$R \approx 0.1\,m$$

which gives Re = 4.5 x10^3, close to the limit of laminar flow. Hence for simulating the full range of flow conditions in drainage channels the full Colebrook-White equation was chosen in the HR research studies.

It can be shown that the Manning equation is an approximation to the Colebrook-White equation under rough turbulent conditions with the conversion:-

$$n = 0.038\,k_s^{1/6} \tag{6}$$

$$\text{for} \quad 7 < \frac{R}{k_s} < 150$$

Outside this range of R/k_s the effective value of Manning's n increases, (see pp 189-190 of the IWEM water practice manual No.7). This forms the basis of the recommendation that a design value of Manning's n of about 0.04 is used for the larger channels in the drainage network.

The simulation modelling carried out in the MAFF sponsored research programme indicated that a typical value of k_s in lowland drainage channels is 1.0m if equation (4) is used.

At large Reynolds numbers $R_e > 10^4$ there is still the difficulty in using equation (4) for small values of relative roughness. In this case for rough turbulence a power law approximation to the logarithm term was used which ensured that the friction factor and its derivative with respect to hydraulic radius were continuous at the transition between the log-law and the power-law. For a transition at R/k_s = 0.3, the following equation applies:

$$\frac{1}{\sqrt{f}} = 2.90\left(\frac{R}{k_s}\right)^{0.67} \tag{7}$$

In modelling the peripheral channels of the Newborough drainage system the frictional resistance was represented by equation (7) for most of the time. Equation (7) is equivalent to setting

$$n = 0.0389\ k_s^{0.67}\ R^{-0.5} \qquad (8)$$

at the point of transition between the log-law and the power law.

A useful engineering approximation is

$$n = 0.04/\sqrt{R} \qquad (9)$$

for the design value of $k_s = 1.0$m.

Comparing equations (6) and (9) it is plausible to use the combination

$n = 0.04$ for $R > 1.0$m

$n = 0.04 / \sqrt{R}$ for $R < 1.0$m

as the design recommendation for Manning's n in place of $k_s = 1.0$m for rough turbulent conditions in the Colebrook-White equation. The conveyances predicted with this approximation to Manning's n are within 20% of those obtained from the rough turbulent approximation to the full Colebrook-White equation for flow depths of practical interest in drainage channels.

Reference

French R H (1986) Open Channel Hydraulics, McGraw-Hill International.

Appendix 5

HR summary sheet 145. Sutton and Mepal Catchment pumped drainage system

SUTTON AND MEPAL CATCHMENT PUMPED DRAINAGE SYSTEM

In common with most parts of the Fens in East Anglia the Sutton and Mepal catchment lies well below high tide level and is thus prone to flooding. It is the responsibility of the Sutton and Mepal Inland Drainage Board to protect the local rural community from flooding by providing adequate drainage. For over 100 years excess runoff in the Sutton and Mepal drainage network has been directed by the Board into the Great Ouse river system using pumps at a station near Mepal. Unfortunately these pumps need refurbishing; they no longer control water levels adequately in several stretches along the network. As a consequence of this the Middle Level Commissioners, who are advising the Board on the redesign of the drainage system, propose that another pump station should be built at Sutton West for use in conjunction with the station at Mepal. They commissioned Hydraulics Research Limited to check the hydraulic details of the proposals and to recommend an optimum geometry and operating procedure[1].

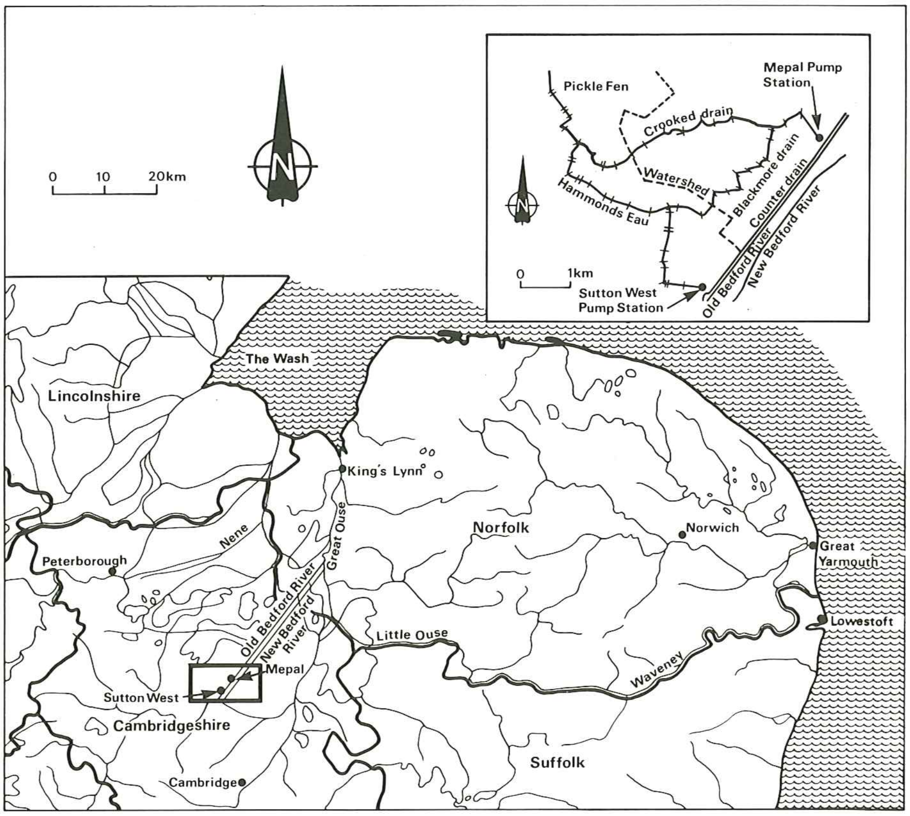
Hydraulics Research used computational models to simulate both the layout of the network and the behaviour of water in the channels. Two major problems to overcome were 1) a lack of adequate hydrological and hydraulic data for the Sutton and Mepal catchment for calibrating the models and 2) the uncertainty of appropriate resistance flow coefficients for an artificial drainage system. Hydraulics Research found answers to both by drawing from research they had done in collaboration with the Institute of Hydrology for the Ministry of Agriculture, Fisheries and Food.

The proposals

The Middle Level Commissioners calculated the basic design conditions for the Sutton and Mepal catchment. They used a design criterion of a 1.2m freeboard at Pickle Fen and a design run-off of $0.107m^3/s$ per Km^2, which is equivalent to a return period of 12 years. This requires a maximum water level of −2m ODN at Pickle Fen.

The catchment has three main drainage channels: Crooked Drain, Blockmore Drain and Hammonds Eau and is divided in two by a low ridge that runs north west to south east. The pumping station for the catchment is at Mepal and lies in the north eastern corner of the north eastern half of the catchment. Re-designing the drainage system around this pump station would lead to many problems; major construction work would be necessary at the existing road crossings and through the intervening high land to make adequate channel improvements, and the pumps at the pump station would need to be improved to cope with the Commissioners' design criterion.

The proposed scheme comprises installing a new pump station at Sutton West and in effect dividing the area into two separate drainage catchments along the ridge; the south western half will then be serviced by the Sutton West station and the north eastern half will continue to be serviced by the



Location of the Sutton and Mepal catchment, East Anglia, England.

Hydraulics Research
Wallingford

Mepal station. The scheme will reduce the load on the north eastern half of the district thus helping to prolong the life of the existing pumps and eliminate the need for extensive improvements to the existing channels in that area for the time being.

The new pump station at Sutton West will have three electric pumps with automatic level control, each rated at 1.04m^3/s. The channel improvement work will produce a new main drain in the south western half of the catchment. This will follow the line of existing minor channels via Tubb's Farm and Bedingham's Drove to Hammonds Eau.

Setting up and calibrating the model

To examine the effects of such a scheme Hydraulics Research used land drainage computational models which were first programmed to describe the geometry and hydraulic structures (culverts and bridges) within the scheme and then fed with hydrological data to relate rainfall, run-off and return periods for a specified design condition.

1. **Geometry**

   The geometry of the channel system in the Sutton and Mepal drainage catchment is extremely varied; particularly noticeable are the ways in which the bed widths and batter slopes vary in a non-systematic way with local reversals in the bed slope. Each modelled channel in the Sutton and Mepal catchment was represented in the models by trapezoidal sections. The model simulated the changes in the geometry and also several large jumps in the discharge (which occur for example at the junction of modelled drains, or at the confluence with an unmodelled large tributary).

2. **Hydraulic structures**

   Details of all bridges and culverts were included in the model. There are 17 bridges at present in the catchment; most are single span structures with the exception of four main road bridges. Hydraulics Research calculated afflux generated by each bridge using Yarnell's equation and surcharge flow using a culvert-type equation.

Mepal Pumping Station, Black Bridge, Blockmore.

Model tests

After the model had been successfully set up and its sensitivity tested for existing conditions at the site, HR engineers adjusted it to predict likely changes of water level in the system for a variety of designs.

The test programme comprised four main stages. Each stage examined a different combination of bed slope, bed width and level of drains, and alternative sites and dimensions of culverts and bridges with several distributions of inflow. Throughout the programme tests were done on the effect of using different policies of operating the pumps on the water levels at various positions in the drainage network.

Results

The results were presented as inflow and outflow hydrographs, stage hydrographs (at important positions in the network) and water level profiles.

During stage-1 of the test programme the water level upstream of the Hammonds Eau and Crooked Drain confluence was unacceptably high. This level could not be reduced by altering the switching levels for the three pumps at the Sutton West pump station. The water level at Pickle Fen Drain was also too high to maintain the design freeboard.

The fourth stage of the tests combined results from earlier stages. The main drain was made slightly deeper throughout in comparison to the preliminary design to produce a gradient of 1:7170 and the switching levels were set as follows:

Pump 1 on 97.4m AMLD off 96.9m AMLD
Pump 2 on 97.5m AMLD off 97.1m AMLD
Pump 3 on 97.7m AMLD off 97.2m AMLD

This combination of geometry and pump operating procedure reduced the water level in Pickle Fen Drain to an acceptable depth (98.05m AMLD) and was recommended to Middle Level Commissioners as the basis for the Sutton and Mepal proposed redesign scheme.

Reference

1. Hydraulics Research Limited. Sutton and Mepal re-organisation scheme. Report No EX 1411, February 1986.

DDB 6/89

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RIVERWAY APPLICATION
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Appendix 6

The computer implementation
of AFCOPS at North Level IDB

Appendix 6 The computer implementation of AFCOPS at North Level IDB

1. Introduction

The North Level IDB installed telemetry in the early 1980s with the data being captured by dedicated hardware. In 1985 the Engineer to the Board contracted McMillan Computing Services through HR Wallingford to upgrade the original system. The upgrade consisted of:

- revised data capture from the outstation;
- provision of management information (see Section 3.5 of the main text); and
- integration of the control algorithm OCOPO developed at IH into the software.

The remainder of this Appendix gives details of the hardware, software and software support for the upgraded system. Although the name AFCOPS was coined for the original system it is used now formally to describe the revised system installed at North Level including the OCOPO control software.

2. Hardware

The original North Level scheme used a custom built central controller. The pump control rules were implemented on a Personal Computer - originally a Tandon PCX using an 8088 processor running at 5 MHz. This Tandon was still in use at the start of 1993 and was controlling 10 pumping stations, 18 level stations and 2 rain stations. The machine had 512Kbytes of main store and a 10 Mbyte disc drive. Apart from breakdowns and power cuts, it ran continuously from mid 1985 to early 1993, although all the mechanical parts had to be replaced. There have never been any serious problems in performance with this machine. By modern standards its specification is modest. A modern PC containing a 80386 processor running at 25 MHz will operate between 20 and 40 times faster. Modern hard discs tend to be at least 40Mbyte in capacity. It is clear that the demands of computer control for a drainage system do not pose performance problems for any modern PC.

3. Software

The original central station software was written in assembler. This was almost certainly because that was no alternative. The pump control algorithms were developed at IH in Fortran, this being the most appropriate computer language for the development phase.

When it came to interface the control rules with the central station, a software house was contracted to do the work. They adopted the C language as the best for the practical implementation of the whole system. Although this did entail translating the IH ICOPO control software from Fortran into C, it did allow the system to be compiled and tested as a whole. The only exception was a small routine written in assembler to interface to the custom built controller. This was necessary because it needed to access the PC hardware, a feature not easily done in high level languages such as Fortran or C. The C language has proved robust, flexible and suitable for the complex interfacing needed. It has allowed a much better user-interface to be provided than would have been possible with Fortran, and has been found to be suitable for modelling electricity tariffs. Since the software was developed, C++ has emerged as a serious programming language which contains new features relevant to modelling and any future development of AFCOPS.

Over the years the software has been refined with more functions being handled by the PC rather than the original controller. Now, there is no need for a complex telemetry interface, and radio modems communicate with outstations. A lot of processing is specific to a type of outstation or to a communications provider. To accommodate this, an "object orientated" approach has been adopted which allows all such processing to be separated from the main process and C++ is particularly suitable for this approach.

The system now provides a complete "control panel" for the engineer. It is possible to view, poll or control any station in the network. The main functions of the software are to:

- receive incoming messages from the outstations and hold them pending processing;
- provide control menus;
- display outstation data on the screen;
- poll outstations at preset intervals;
- generate and transmit alarms;
- compute pumping required;
- send commands to switch on pumps;
- provide tabular analysis of pumping; and
- print graphs of water levels and pumping.

Figure A6.1 shows a typical screen display

The software promotes more uses of itself. Before it was installed, water levels could only be checked a few times a day. There was a small chance that the level would be at its highest when the engineer arrived to check it. With the AFCOPS software in place, the engineers can find water levels at any time, and also know the maximum level that occurred during any rainfall event. Screen displays may be modified easily to present more useful information.

4. Support

Support of the system has not proved to be a problem. It is vital that the suppliers of each part of the system provide maintenance cover for it. Software in particular requires high quality support. At North Level, an agreement provides telephone support from the system integrator with occasional site visits. It is likely that an "on-line" support facility will soon be implemented. However, it is essential that the authors of the software are kept available for support, and the small companies involved have been well placed to do this. A detailed trace of all messages is kept and, using this, it is simple to isolate problems. Many failures have not been caused by the AFCOPS equipment. Queries arising from software operation have been traced to power cuts at outstations, vandalised radio aerials or even blocked weed-screens. The trace data has always enabled these to be identified within hours and which equipment has caused the problem.

```
28/05/93     0:00          800 Newborough

Max. Design Water Level -1.22 m

Inside  w/s -0.93
Outside w/s -0.97
Downstream  +0.00

On    -1.20 -1.10 -0.91
Off   -1.76 -1.39 -1.32

Pump states:       H* H* H
Rotation:          -  -  -
Siphon valves      -  -  -

Water High  -  W/S Block -  Sensor    -
Mains Fail  -  Water Low -  Pump Fail -

Time Switch X  Override  -  Remote   X
AC Amps 0
Previous level was  0.00 at 00:00
```

Figure A6.1 Typical screen display