The Standing Technical Committee on Sewers and Water Mains
WORKING PARTY ON THE HYDRAULIC DESIGN OF STORM SEWERS

Design and analysis of urban storm drainage

THE WALLINGFORD PROCEDURE

Volume 1

Principles, methods and practice

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Design and analysis of urban storm drainage

THE WALLINGFORD PROCEDURE

Principles, methods and practice (Volume 1)

Describes the background and development of the Wallingford Procedure for the design and analysis of urban storm drainage for use in the UK. Four methods, incorporated within an economic framework are used to design sewer dimensions, depths and gradients and to simulate the behaviour of a system under surcharge and with surface flooding.
ISBN 0 901090 29 1.

Program user's guide (Volume 2)

Gives detailed instructions on the data to be collected for a sewer system and how to prepare the data for the computer programs implementing the procedure for the UK.
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Maps (Volume 3)

Contained in a folder, four large scale maps of the UK giving meteorological and soil data required to implement the computer programs. To a scale of 1:1 million, the maps can be used to obtain the most accurate values of various parameters for a given location. The soil map (in colour) is the first of its type to include Scotland and Northern Ireland.
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Presents a hand calculation for a modified version of the Rational Method for use in the UK. This volume is suitable for those designing or analysing small sewer systems or for planning urban drainage schemes.
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Programmer's manual (Volume 5)

Supplied only with the main-frame computer programs which are available from Hydraulics Research Limited, Wallingford, Oxon OX10 8BA.

MicroWASSP User's Guide (Volume 6)

Gives instructions on the use of the micro-computer version of the computer programs implementing the Procedure.

WASSPOS User's Guide (Volume 7)

The parallel volume to Volume 2 for use of the computer programs in applications outside the UK.

Course Notes
Contains extensive notes on lectures given in the course run by Hydraulics Research for users of the Procedure. The notes include details not described in Volume 1.

Foreword

In 1974 the Department of the Environment set up the Working Party on the Hydraulic Design of Storm Sewers to review all aspects of the hydraulic design of storm drainage systems, to coordinate and promote research, and produce a manual of good practice. In view of the importance of the subject, the Department made available considerable funds for a significant programme of research at the Hydraulics Research Station and at the Institute of Hydrology. The Meteorological Office was also involved through a sub-contract let by the Hydraulics Research Station. During this work the main scientific and engineering bases of the meteorological, hydrological and hydraulic aspects of storm drainage were reviewed and, where appropriate, improved methods for the design and analysis of storm sewer systems were prepared. This work was conducted under the general guidance of the Working Party.

Following the reorganisation of the water industry, a new structure of committees was agreed by the Department of the Environment and the National Water Council. The Standing Technical Committee on Sewers and Water Mains then assumed responsibility for all work on sewers. It was agreed that from 1975 the Working Party should be brought under the umbrella of the Standing Technical Committee and report to it. In view of the importance of the work, the progress that had been made and the resources which had been allocated, the Standing Technical Committee saw every reason for the work to be pursued to a conclusion.

The research has now been concluded and the Final Report of the Working Party is published in five volumes under the general title *Design and analysis of urban storm drainage – the Wallingford Procedure.*

The Standing Technical Committee has approved this report for use by all concerned with the more efficient and cost-effective drainage of our urban areas and believes that utilisation of the computer-based methods of design incorporated in the Wallingford Procedure will advance the principles for which the committee stands. The committee will monitor the use of these new methods during their early years of application and will always be ready to consider suggestions for improvement.

I commend this report and trust that it will find extensive use within the water industry in the United Kingdom.

E.C. Reed
Chairman
Standing Technical Committee on Sewers and Water Mains.

Contents of volume 1

Preface

THE PUBLICATION of this report describing the Wallingford Procedure marks the end of another stage in the long and fascinating development of methods for the design and analysis of storm drainage systems.

Every major new advance depends on the work which preceded it and this research project is no exception. At the outset the Working Party wishes to acknowledge the great advance which was made by the TRRL in the early 1960s and the benefits which use of the TRRL method has brought over the last two decades.

No one wishes to introduce change for change's sake, but members of the Working Party feel that they can reasonably claim that they acted in response to demand from the water and public health engineering profession for improvement. This was most clearly expressed at a research colloquium organised by CIRIA in 1973 under the title 'Rainfall, runoff and surface water drainage of urban catchments'.

Much applied research in civil engineering proceeds on the assumption that, since professional practice tends to be conservative, the greater knowledge gained from research will usually mean that savings can subsequently be made in the capital or operating costs of the works concerned. It is pertinent to point out that, although this is generally so, there may be cases where a truer appreciation of the real performance of a system leads to a realisation that the margin of safety was not as great as had previously been supposed.

In the case of storm drainage the new Wallingford Procedure properly used will enable the designer to have a much better feel for his system and those points in it which are under most 'stress' and at which surcharging (or surface flooding) may occur. As a result, it is possible that some sewers may need to be made larger than had previously been thought necessary, but the overall result should be more cost-effective.

The Working Party wishes to emphasise that the new procedure will be as relevant to the improvement of existing systems as to the design of entirely new systems. Rehabilitation will be one of the key concepts in the water industry in the United Kingdom for years to come and the Wallingford Procedure will play a vital part in the engineer's continuing endeavour to alleviate existing surcharge problems and to improve the level of service using existing assets.

The research work which has now culminated in the issue of the Wallingford Procedure was undertaken principally at the Hydraulics Research Station, the Institute of Hydrology and the Meteorological Office, under the general guidance of the Working Party. The scientific and engineering principles incorporated in the new procedure were the responsibility of research teams led by C.K. Folland at the Meteorological Office (rainfall inputs), Dr M.J. Lowing at the Institute of Hydrology (overland flow), Dr R.K. Price at the Hydraulics Research Station (sewer flow hydraulics) and P.J. Colyer at the Hydraulics Research Station (costs and economic models). Dr R.K. Price was also responsible for combining the various contributions into a single set of computer programs.

A question which exercised the Working Party (along with all those who seek to model natural phenomena) was the extent to which extra complexity was justified in the Wallingford Procedure. Although the real world displays almost infinite variability and subtlety, which make description and definition very difficult, the procedure seeks to provide a decision making framework matched to both this real world and the need for practical cost-effective design. Some aspects of the theories on which the new procedure is based may appear complex (though this should not give rise to undue difficulty in their application) but the Working Party believes that the extra work required (for example by an increased data requirement) will be more than offset by the extra convenience of the enhanced facilities which are offered by the Wallingford Procedure.

Preparation of this report was guided by the basic principle that those who publish computer-based design methods have an obligation to describe the bases of those methods clearly. The description should also be sufficiently full for intending engineering users to check that the methods do what they need on sound theoretical bases with acceptable accuracy over the required range of inputs. The Working Party trusts that this final report meets this requirement and that the designer will be able to satisfy himself on these matters and so assume professional responsibility for the results produced by the procedure.

A suite of programs of the range encompassed by the Wallingford Procedure inevitably contains errors when first written and the Working Party organised trials of the programs by organisations represented on the Working Party (internal) and by organisations with no previous connection with it (external). The tests done by the trialists revealed a number of most useful and important corrections, and these have been taken into account in the programs which have finally been issued. It is very difficult (and indeed uneconomic) to carry computer program validation to the point where the programmer can be certain that all errors have been eliminated, but on the basis of the internal and external trials and the tests made by Dr Price and his colleagues it is believed that the programs will yield reliable answers when used by experienced designers.

The preparation of final reports on research programmes of this magnitude is always a major task. The first drafts of parts 1, 2 and 3 of volume 1 were prepared by the researchers chiefly involved, while part 4 was provided by practising engineering members of the Working Party. However the onerous task of editing the final text into a coherent whole fell to P.J. Colyer to whom the members of the Working Party wish to express their sincere appreciation. Dr R.K. Price wrote the Program User's Guide (volume 2) and edited it in line with comments from the Working Party and the trialists. The maps (volume 3) were prepared in conjunction with the Soil Survey of England and Wales and the Meteorological Office. Mr Colyer was the author of volume 4 on the Modified Rational Method, while Dr Price wrote the Programmer's Manual (volume 5).

The Working Party considered the text of the whole report at a number of full day meetings and is satisfied that it adequately presents the various concepts embodied in the Wallingford Procedure and describes how it may be used. The Working Party therefore approved the report for submission by the Standing Technical Committee on Sewers and Water Mains to the National Water Council and Department of the Environment for publication and use within the water industry of the UK.

Membership of the Working Party was drawn from practising engineers, research groups and Government departments; the practising design interests were representative of water authorities, local government and consulting firms. Although membership has varied during the life of the Working Party, this balance of interests has been broadly retained. A full list of all those who served on the Working Party from its inception is given in Appendix 3. Membership of the Working Party at the time the report was finally prepared is given below.

The organisations employing members of the Working Party have been most generous in their provision of time; the whole water profession is indebted to them. The Working Party greatly appreciated the interest shown and the time spent by the organisations and their staffs who undertook to test the programs in the internal and external trials. Their names are given in the acknowledgements.

Members of the Working Party are grateful for the support and encouragement they have consistently received from the Standing Technical Committee on Sewers and

Water Mains and in particular its Sub-Committee 1 on Hydraulic Design and Planning.

The work of T.W.G. Hucker of the Department of the Environment, the first chairman of the Working Party who skilfully guided its deliberations in the period from March 1974 to November 1976, must be acknowledged.

No working party can function efficiently without a good secretary and it is a pleasure for me to record the sterling service rendered to the Working Party by Peter Colyer from March 1974 to December 1979 and by John Forty from January 1980 onwards.

Finally and on a personal note I wish to express my own appreciation of the work and support of each member of the Working Party and for the way in which each member has contributed to the work of the whole project. Cooperation of this kind in a joint endeavour to improve the cost-effectiveness of its techniques is one of the hallmarks of an active, responsible and worthwhile profession. I trust our work and the new Wallingford Procedure will be viewed in the context of a continuing process of professional advance.

D.E. Wright
Chairman
Working Party on the Hydraulic Design of Storm Sewers

Membership

Membership of the working party at the time the final report was prepared was as follows:

Dr D.E. Wright (Chairman)	Sir William Halcrow and Partners and Halcrow-Balfour Ltd.
D.H. Garside (Vice-Chairman)	Central Lancashire Development Corporation
J. Bonsall	Water Directorate, Department of the Environment
H.M.G. Cockbain	Welsh Water Authority
W.R. Ferguson	Lothian Regional Council
D. Fiddes	Water Research Centre
C.K. Folland*	Meteorological Office
Dr M.J. Hall	Sir William Halcrow and Partners
E.W. Jones	Ministry of Agriculture, Fisheries and Food
Dr M.J. Lowing	Institute of Hydrology
D.B. Males	National Water Council
A.J. Price	John Taylor and Sons
Dr R.K. Price	Hydraulics Research Station
N.G. Semple	Scottish Development Department
P.J. Colyer (Editor)	Hydraulics Research Station
E.J. Forty (Secretary)	Hydraulics Research Station

*Mr B. May replaced Mr Folland in February 1981.

Summary

THE REPORT describes and explains the Wallingford procedure for the design and analysis of urban storm drainage networks. The procedure incorporates a hierarchy of methods and is available as a suite of computer programs and accompanying documentation. The present volume indicates the results of the research on which the procedure is based and justifies the approaches adopted.

Urban storm runoff processes are analysed in sections dealing with rainfall, surface runoff and pipe flow. The influences of ancillary sewer structures, water quality and cost and economic factors are also considered. The range of methods made available for design and analysis is described, along with the data requirements and guidance in the choice of an appropriate method for a particular situation.

The programs and program user manuals are available separately.

Part 1

Introduction

CHAPTER 1

Guide to the report

1.1 Brief guide to contents

THE PURPOSE of this report is to describe the Wallingford procedure for the design and analysis of urban drainage systems and to explain how it is to be used. The report is in five volumes.

Volume 1 is itself divided into four parts. Part 1 describes various aspects of storm drainage design in the United Kingdom and includes a brief summary of the recommended procedure.

Part 2 sets out the principles of the four main technical fields involved (storm rainfall, surface runoff, flow through pipes and ancillary sewer structures and costs/economics). It provides the explanation and justification of the calculation methods used in the recommended procedure. References are given to more detailed reports.

The options available in the procedure are described in Part 3 in sufficient detail to give the user an understanding of the modelling processes.

Part 4 describes several ways in which the procedure may be used to improve the design of urban storm drainage, particularly through the information it provides on previously neglected hydraulic, hydrological or economic problems.

Volume 2 (available separately) is a program user's guide which provides the information required by those operating the programs. Volume 3, also available separately, is a folder of 1:1 000 000 maps of meteorological and soil parameters for the United Kingdom.

Volume 4 provides a separate description of the Modified Rational Method for designers without access to a computer and for whose work a simple hand calculation method will be adequate.

Volume 5 is a programmer's manual which will be required by those implementing any internal changes to the computer programs.

1.2 Method of use

The professional engineer should not regard computer models as 'black boxes' but should attempt to understand the calculation procedures included. This report provides both instructions in the use of the programs and the reasoning behind them. Therefore it is not expected that the report will be read straight through because the information in the various chapters meets a variety of requirements. Table 1.1 is provided to direct readers to the chapters appropriate to their interest.

Definitions of the technical terms used are given in the extensive glossary in Appendix 1. Symbols are defined when first used in the text, and also in Appendix 2.

Table 1.1. Guidance in the use of the report

Requirement	Sections and chapters to be read
Summary of the recommended procedure	4.2 to 4.5
Summary of the work of the Working Party on the Hydraulic Design of Storm Sewers	Appendix 3
The social, historical and engineering context of the research activity	2 to 3
Understanding the physical and mathematical basis of the procedure	5 to 12
Description of alternative methods:	
Summary	4.2 to 4.5
Details	13
Data requirements	14
Selection of an appropriate method	5 to 12 to obtain familiarity with concepts used 15 17 to 18
Application of a selected method	
Wallingford Modified Rational Method	6.1 to 6.5, 7.4 and 7.10, 8.1, 8.2, 8.6 to 8.8 for background 13.1 for details of the method Volume 2 (Program user's guide) Volume 3 (Maps) Volume 4 (Hand calculation)
Wallingford Hydrograph Method	6 to 12 for background 13.2 for details of the method Volume 2 (Program user's guide) Volume 3 (Maps)
Wallingford Optimising Method	6.1 to 6.5, 7.4 and 7.10, 8.1, 8.2, 8.6 to 8.8 for background 12.4, 13.3 for details of the method Volume 2 (Program user's guide) Volume 3 (Maps)
Wallingford Simulation Method	6 to 12 for background 13.4 for details of the method Volume 2 (Program user's guide) Volume 3 (Maps)
Aspects of economic evaluation	2 12 13.4 16
Changes to programs	6 to 12 Volume 5 (programmer's manual)
Sources of information and advice	Appendix 4

CHAPTER 2

Storm drainage in the United Kingdom

2.1 Engineering aspects

THE BASIC function of storm drainage is to remove excess rainwater from the vicinities of buildings, roads, pavements and other impermeable surfaces associated with urban development and to convey it safely and economically to suitable watercourses. Various methods of providing storm drainage have evolved as the nature and pace of urban development have changed, but generally drainage systems consist of networks of underground pipes contributing to streams and rivers which flow through or near urban areas. Many such streams have become culverted and now form an integral part of an underground drainage system.

Storm drainage networks may be broadly classified as either 'combined' or 'separate'. Combined systems are built to convey in a single pipe both storm runoff and foul sewage; since the flows generated by rainstorms are typically many times larger than the foul sewage flows from the same area, and because it is normally impractical to carry such large flows to a sewage treatment works, most combined systems include storm water overflows, which spill a proportion of the flow to a watercourse during storm conditions. Separate sewer systems consist of two pipe networks, one for foul sewage and the other for storm runoff. The storm runoff is generally discharged without treatment to convenient watercourses.

Various combinations of these two systems are also found. For example, the 'partially separate' system, in which roads and front roofs drain to a separate storm system while rear roofs and yards drain to a combined system, is common in extensive areas of terraced housing built during periods of industrial expansion. Other catchments include separate systems where recent new development has taken place, but these may re-unite as a combined system in areas not yet redeveloped.

The procedure described in this report is applicable to any urban drainage system in which the storm runoff plays a dominant role. It is therefore applicable to both separate and combined sewer systems, and to pipe systems and natural or formerly natural channels which receive all their flow from the urban area. Separate consideration would have to be given to any flood flow which derived from rural areas beyond the urban catchment. Certain limitations in the application of the procedure to the performance of combined systems are described in section 4.3 below.

The examination of urban storm runoff is also important to the management of the larger river basin to which the drainage system contributes. The process of urbanisation has had a noticeable effect on both the flow regime and the water quality of rivers which include in their catchments a significant proportion of urban area. The procedure described in this report is intended primarily for the determination of flows within and arising from an urban catchment, but this may be an important step in the hydrological analysis of partly urbanised catchments, which have been the object of a separate investigation[35].

2.2 Public health and social aspects

The engineering function of storm drainage described in the previous section must be expanded to take account of the social and environmental benefits of drainage. The wider purposes of urban storm drainage may therefore be defined as:

(a) to maintain public health;
(b) to provide protection from the physical damage and economic losses caused by flooding;
(c) to create an urban environment acceptable to the community;
(d) to make land available for development.

Recent improvements in river water quality in the United Kingdom have drawn attention to pollution resulting from the discharge of urban stormwater runoff from both separate and combined sewer systems. The quality of these intermittent discharges may have an important effect on the attainment of river quality objectives. In the future the responsible authorities will be required to specify consent conditions for storm discharges and this, as well as the identification of river quality objectives, will increase the need for specific information on the quality of storm runoff.

The processes which determine the runoff of pollutants from urban surfaces are not yet thoroughly understood, and models for predicting pollution in urban drainage systems are not included in the procedure. The problem of storm runoff quality is discussed further in section 3.3, where available data illustrating the possible scale of the problem are tabulated.

An examination of worldwide practice shows that the provision of drainage, and the capacity and efficiency of the drainage system, are closely related to a society's standard of living. Storm drainage may be seen as one of a great number of desirable public services (water supply, foul sewerage, electricity, education, health, transport etc) among which priorities and levels of expenditure have to be determined.

The close connection between engineering decisions and social requirements is immediately apparent. The engineer has to determine the required flow capacity of a storm drainage system, but this is also a measure of its performance in meeting the objectives set out above. Therefore storm drainage is concerned with economic choices, since a range of schemes could be designed to meet a corresponding range of performance criteria.

The procedure developed by the Working Party includes a method by which the main tangible costs involved in the provision of storm drainage (including damage costs avoided) may be evaluated. Such a method cannot be regarded as a universal approach to be applied uncritically; it involves a considerable degree of judgement and local knowledge. However, it provides a means by which the engineer, or those whom he advises, may arrive at an acceptable combination of system performance, social benefit and costs.

2.3 Administrative aspects

The administrative and legal aspects are complex but in general the statutory authorities responsible for the provision of surface water drainage in England and Wales are the ten water authorities, local authorities acting as agents to the water authorities and highway authorities. In Scotland the responsibility is held by the regional and islands councils and in Northern Ireland by the Department of the Environment.

These bodies may be responsible not only for the design, financing and maintenance of drainage services but the water authorities in England and Wales also have a statutory duty to evaluate future needs and to prepare future investment programmes. Similar programmes are developed by the authorities in Scotland and Northern Ireland. The legislation affecting sewers and land drainage is described in the latest revision of the British Standards Institution's Code of Practice 2005[11].

In addition to public sewerage development, a large volume of sewer construction is carried out by private developers.

Drainage design or analysis may take place initially for planning purposes to evaluate comparative costs of alternative developments, and subsequently for detailed engineering purposes. The Working Party has recognised that flexible calculation methods, incorporating the option to evaluate costs and alternative solutions, are therefore required.

2.4 Expenditure on storm sewers

The purpose of this section is to illustrate the magnitude of the current expenditure on storm drainage in the United Kingdom, and thus to place in its context the recent research activity and the need for improved methods of analysis. The expenditure information is drawn from the annual reports of the National Water Council [68] and of the water authorities; it therefore represents public sector expenditure only.

The total capital expenditure of the water authorities in England and Wales in 1979-80 (at outturn prices) was about £605million, of which £171million (28 per cent) was devoted to sewerage (excluding a further £127million (21 per cent) attributed to sewage treatment and disposal). Revenue expenditure in the same year on sewerage (excluding interest charges and depreciation) was about £101million, of which about £44million was attributable to repair and maintenance.

Thus about £272million (171 + 101) was spent on sewerage by the water authorities in 1979-80. This figure may be expected to change from year to year, but indicates the scale of national expenditure.

An analysis carried out for the Working Party [28] in one authority suggested that about 60 per cent of all sewerage costs were attributable to the storm runoff element. On this basis about £160million was spent on storm drainage by the water authorities in 1979-80. Adding for Scotland and Northern Ireland in proportion to population gives a total annual public expenditure on storm drainage in the United Kingdom of about £180 million at 1979-80 prices.

While the pattern of urban development is likely to change, the need for storm drainage is bound to continue. New urban developments will be built and attention is also being given to replacement of ageing sewers. The design and analysis procedure described in this report should assist drainage engineers to prepare proposals which make more efficient use of the nation's resources.

CHAPTER 3

Present design practice and its limitations

THE JUSTIFICATION for introducing new methods of stormwater drainage design and analysis lies principally in the limitations of existing procedures. Such limitations may be concerned with the fidelity with which natural processes are represented, or with the ease of application. The wide availability of high speed digital computers has removed some of the latter type of limitation, so that interest may be centred on the representation of natural processes. In order to provide an appreciation of the limitations in current methods, this chapter presents a brief review of present design practice and its historical development. The process of 'design' described here refers equally to works on undeveloped sites and to modifications to existing systems.

3.1 The design storm

Urban development causes changes in the runoff process, so design discharges for new systems cannot be determined from an analysis of flow records but must be estimated from a knowledge of rainfall and the physical characteristics of the urban catchment draining to the system. Design methods therefore consist essentially of procedures for transforming a design storm into a rate of flow. The design storm may be an average rate of rainfall corresponding to a given storm duration and specified return period, which is read from a statistical summary referred to as a rainfall intensity-duration-frequency relationship, or a storm profile, which describes the variation of rainfall intensity with time throughout the duration of the event.

Until the beginning of the twentieth century, drainage systems were designed on the basis of an average rainfall intensity which was assumed to be independent of duration. However, with the collection of information on heavy falls of rain in short periods by the British Rainfall Organisation, and their publication of statistical summaries in 1888 and 1908, the inverse relationship between average rainfall intensity and duration became well-established by observation. Lloyd-Davies [57] was among the first in Britain to attempt to quantify this relationship for drainage design purposes. His analysis of five years of records from the Edgbaston Observatory resulted in a rainfall intensity-duration relationship of the general form

$$I = x_1/(D + x_2) \qquad \text{......3.1}$$

where I is the average rainfall intensity within the duration D, and x_1, x_2 are constants. Equation 3.1 subsequently became known as the 'Birmingham curve'. Several of the more progressive municipal engineers of the day followed the lead given by Lloyd-Davies and produced equations similar to the Birmingham curve from their local rainfall records. The proliferation of these relationships, each differing marginally in the values of the constants, x_1 and x_2, led the then Ministry of Health to convene a committee for the purposes of recommending a standard working curve for the design of drainage systems. In its 1930 report [65] the Committee proposed the use of two expressions of the general form of equation 3.1 applying to durations between five and 20 minutes and between 20 and 100 minutes respectively, the latter being the original Birmingham curve. These equations became universally known as the 'Ministry of Health formulae'. Unfortunately, the frequency of occurrence of the average rainfall intensities of different durations given by the formulae was only expressed in the most general of terms. A more precise definition was left until 1948, when Norris [70] showed that the Ministry of Health formulae corresponded to a once-a-year event.

With the publication of the work of Bilham [9] in 1936, more comprehensive information became available on the frequency of different rainfall depths within specified durations. Using the first complete decade of data from 12 autographic (continuously-recording) raingauges located mainly in the Midlands and south east of England, Bilham derived the following rainfall depth-duration-frequency relationship:

$$N = c_b D(r_b + 0.1)^{-3.55} \qquad \text{......3.2}$$

where N is the number of occasions in 10 years on which a rainfall depth r_b inches is recorded within a duration D hours or less during one observer-day, and c_b is a constant. Although intended to apply to durations between five and 120 minutes equation 3.2 has been extrapolated to longer durations in the absence of other information.

With the accumulation of many good records of short-duration rainfall, a reappraisal of Bilham's work was carried out in 1964 by Holland [38]. The latter study showed that, despite the paucity of the original data base, equation 3.2 had stood the test of time well. The recommendation made by Holland merely included a small adjustment to the value of the constant c_b and an alternative equation for rainfall intensities exceeding 1.25ins/hr(32mm/hr):

$$N = r_b \exp(1-0.8I)(r_b + 0.1)^{-3.55} \qquad \text{......3.3}$$

where $I = r_b/D$. As in the original Bilham work, no allowance was made for variations in the relationship with geographical location.

The publication of the Ministry of Health formulae in 1930 also marked the beginning of a renewed interest in methods of estimating the discharge hydrographs in drainage systems. However, a prerequisite for the calculation of a discharge hydrograph is a knowledge of the temporal variations of rainfall intensity. Since few autographic raingauges were operating at that time, the majority of studies employed synthetic storm profiles based upon the Ministry of Health formulae. Perhaps the most widely known profiles of this type were those suggested by Ormsby [71] which peaked at one-half and one-third of their duration, and within which the average rainfall intensities corresponding to different elapsed times centred on the peak were those given by the Ministry of Health formulae. Other contemporary studies, such as Judson[49], used arbitrary, and somewhat unrealistic, rearrangements of such rainfall intensities. As with the first rainfall intensity-duration relationships, the frequency that could be attributed to these synthetic storm profiles was never specifically stated.

During the 1960s, further work on the spatial and temporal variability of storm rainfall was carried out by the Meteorological Office using a closely-spaced network of autographic raingauges at Cardington in Bedfordshire. An analysis of a considerable number of recorded storm profiles by Holland [39] showed that the maximum rainfall intensity occurred before the mid-point of the storm duration, and that the rise to the maximum rainfall rate was steeper than the subsequent recession. Using these profiles and the original Bilham equation, storm profiles corresponding to different return periods were derived and published in the first edition of Road Note 35 [91].

The rainfall depth-duration-frequency relationship given by the Holland modification to the Bilham equation and the storm profiles given in the first edition of Road Note 35 have only recently been superseded by the recommendations contained in the Flood Studies Report [69]. These recommendations were based upon a substantial data set which included
(i) 600 daily raingauges having an average of 60 years of record;
(ii) a further 6000 daily raingauges operating during the decade 1961-1970;
(iii) short-duration rainfall data from 200 sites;
(iv) similar data from dense networks of autographic raingauges at Cardington (referred to above) and at Winchcombe in Gloucestershire; and
(v) data on rainfall within very short durations obtained from a small number of Jardi rainfall recorders.

These data were employed to assess the geographical variations of rainfall depth-duration-frequency relationships, the first occasion on which such information has been made widely available to designers of drainage systems.

Volume II of the Flood Studies Report also provides information on the variation of storm rainfalls over areas of different sizes, and on the construction of storm profiles corresponding to different percentile peakedness for both winter and summer events. Peakedness was defined by the ratio of the maximum to the mean intensity; percentile peakedness is the percentage of storm events with a peakedness less than or equal to that of a given profile.

The second edition of Road Note 35 [91], which appeared shortly after the publication of the Flood Studies Report, contained two sets of advice on the choice of design rainfalls for storm drainage systems. In order to apply the more accurate approach, the designer was encouraged to approach the Meteorological Office for design storms based upon the 50 per cent summer profile. An approximate method of constructing a design storm profile, which included an allowance for geographical variation, was also provided. This method was based upon a condensed version of the full Flood Studies Report procedure, and the differences to be expected between the profiles produced by the two approaches have since been discussed by Folland [30]. Since the full procedure is included in chapter 6 of this report together with a recommended shape of profile, the use of the profiles tabulated in Road Note 35 (1976) should only be used for preliminary designs. The methods outlined in chapter 6 for the construction of intensity-duration-frequency relationships also follow Flood Studies Report procedures.

The effects on storm discharges of the movement and development of rainstorms have also been examined in recent research [51]. A pilot study by the Meteorological Office [87] suggested that the majority of storm rainfall patterns moved at speeds much greater than the speed of propagation of flows through a sewer system, and that storm movement could probably be ignored in urban drainage applications. This provisional conclusion will need to be reviewed as further research results become available.

3.2 The rainfall-runoff relationship

The flood estimation procedures applied in stormwater drainage design may be broadly classified into those which provide the peak rate of flow only, and those which also give the runoff hydrograph. These two aspects of the transformation of a design storm into runoff provide a convenient framework within which to review the procedures that are prevalent in current design practice. In discussing either type of approach, attention must be paid to the proportion of the total volume of rainfall which appears as runoff (and, where appropriate, the distribution of the runoff volume in time in the form of a discharge hydrograph) and to the relationship between the frequency of the design rainfall and the frequency of the resultant peak rate of flow.

3.2.1 Maximum discharge methods

Perhaps the most widely known of the simple flood estimation procedures used in the design of drainage systems is the Rational Method. British literature credits Lloyd-Davies [57] with the introduction of this approach in 1906, whereas American sources refer to an earlier paper by Kuichling [56] in 1889. However, the principles of the Rational Method were clearly expounded by Mulvaney [67] in 1850.

The Rational Method is based upon the premise that every drainage area has a time of concentration, which may be defined as the time taken for flow from the most remote point in the catchment to reach the point under design. The peak discharge Q_p is then assumed to occur when the whole of the catchment contributes to the flow, ie at an interval equal to the time of concentration after the rainfall begins. The magnitude of this peak is taken to be proportional to the volume of effective (runoff-producing) rainfall during the time of concentration:

$$Q_p = C\,iA \qquad \text{......3.4}$$

where A is the catchment area upstream of the design point; i is the average rate of rainfall during the time of concentration; C is a coefficient. An appropriate adjustment is applied to allow for the units of measurement.

From several viewpoints, the Rational Method is deceptively simple. In a survey of storm drainage design practices within the State of Wisconsin, Ardis *et al*[5] found in 1969 that only six out of 23 design offices were using the technique correctly. The principal errors were in the choice of the average rainfall intensity, which was not allowed to decrease with increase in the time of concentration, and in the selection of appropriate runoff coefficients.

It has been shown [101] that, on the assumption of a constant flow velocity, the coefficient C may be defined as both the ratio of the peak rate of runoff to the average rate of rainfall during the time of concentration, and the ratio of the total volume of runoff to the total volume of rainfall. The tabulated values of the so-called 'impermeability factors' for different types of development, presented in many engineering handbooks, are treated as volumetric ratios. For catchment areas of mixed land use, weighted average coefficients are often employed. Alternatively, the ratio of the paved to the total area has frequently been adopted as a representative coefficient for urban drainage design, particularly in British practice. Introduced by Lloyd-Davies, the latter approximation was verified experimentally by Meek [64] and later reaffirmed as a reasonable design assumption by Watkins [95].

Since the product CA is assumed to be constant for any given catchment area, the statistical properties of the average rainfall intensity are transferred without modification to the peak rate of runoff. This equality was used by Schaake *et al* [85] to test the validity of the Rational Method on different types of urban area for which both rainfall and runoff records were available. In many cases, those authors found that the estimated peak flow rates were equalled or exceeded at the same frequency as that of the causative average rainfall rates, thereby lending considerable support to the Rational Method as a simple design tool.

Despite these encouraging results the Rational Method is known to give erroneous results under certain design conditions. In particular, for catchments in which the contributing area does not increase uniformly with time, a large peak rate of runoff may be computed for storm durations less than the time of concentration. This difficulty may be avoided by employing a diagram showing the variation of the contributing area of catchment with time from the beginning of the storm.

The time-area diagram formed the basis for two distinct groups of methods, the first of which, known as the Tangent Methods, may be regarded as extended versions of the Rational Method, capable of estimating peak rates of flow only. As their name implies, the Tangent Methods involve a geometrical construction in which the time of concentration associated with the largest peak rate of flow is identified by drawing a line tangential to the time-area diagram. As Watkins [95] demonstrated, Tangent Methods may produce estimates of peak rates of flow that are greater than or equal to, but never smaller than, the Rational Method estimates. This bias in the direction of increasing design discharges has resulted in the Tangent Methods being abandoned in favour of more flexible alternative procedures.

The second group of approaches involving the use of the time-area diagram may for convenience be referred to as the Typical Storm Methods. The latter differ from the Tangent Methods in producing a complete runoff hydrograph rather than simply an estimate of the peak flow rate, and are therefore considered in more detail in the following section.

3.2.2 Hydrograph methods

The development of techniques for estimating the runoff hydrographs from completely sewered catchment areas began with the Typical Storm Methods. These consisted of the combination of an incremental rainfall profile and an incremental time-area diagram. Given a storm profile in which the average rainfall intensities within successive time increments are i_1, i_2, i_3, the successive ordinates of the discharge hydrograph may be written as:

$$
\begin{aligned}
q_1 &= Ci_1A_1 \\
q_2 &= Ci_1A_2 + Ci_2A_1 \\
q_3 &= Ci_1A_3 + Ci_2A_2 + Ci_3A_1 \qquad 3.5
\end{aligned}
$$

where C is the coefficient of the catchment area, and A_1, etc, are successive increments on the time-area diagram.

Individual Typical Storm Methods differed primarily in the variation of the storm profile. In the absence of records from autographic raingauges, synthetic storm profiles constructed from rainfall intensity-duration relationships were employed by many authors. However, as discussed in section 3.1, the frequency of these design storms was defined only in the broadest of terms.

In Britain, the development of the Typical Storm Methods culminated in what is now referred to as the Transport and Road Research Laboratory (TRRL) Hydrograph Method [91]. This method is a two-stage procedure, the first part of which is a straightforward typical storm type of calculation in which runoff is assumed to occur only from the paved areas within the catchment. When this approach was applied to observed rainfall-runoff events, the computed runoff hydrographs consistently reached their peak too early and overestimated the maximum discharge. This lack of agreement was attributed to the neglect of the reservoir storage within the pipe system. In practice, as the rate of runoff varies, depths of flow within the drainage system alter and the volume of water retained within the sewers also changes. This effect was simulated by routing the time-area hydrograph through a hypothetical reservoir having the same storage-discharge relationship as the pipe network down to the point of interest. For existing drainage systems, this relationship could be constructed from the recession curve of an observed hydrograph. For drainage systems under design, the storage-discharge curve was derived by assuming that at any time the proportional depth of flow would be the same throughout the pipe network. This assumption could also be applied when examining existing systems for which no flow records were available, but was shown to be invalid for many older types of development with exceptionally large upstream sewers. Subsequently, the redesign of existing systems was found to be a more frequent requirement than the design of entirely new systems, and the TRRL Hydrograph Method was modified so that the constant proportional depth assumption was only applied to individual pipe lengths.

For the past 15 years, guidance on the choice of design procedures for storm drainage systems has been available in Road Note 35 [91]. This publication recommended that the use of the Rational Method should be confined to the design of smaller schemes in which the diameter of the outfall did not exceed 600mm, and that the TRRL Hydrograph Method should be used for larger schemes and cases in which an outfall hydrograph was required. The modification to the original TRRL Hydrograph Method relating to the constant proportional depth assumption was introduced in 1965, but was not incorporated into Road Note 35 until the second edition.

A survey carried out in 1975 [102] showed that by then the methods described in Road Note 35 were used for 96 per cent of all design work in Britain. Furthermore, an analysis by Colyer [15] showed that the TRRL Hydrograph Method appeared to be more reliable in simulating observed storm events than several other computer based design procedures developed subsequently in other countries. However, a major distinction may be drawn between the TRRL Hydrograph Method and its more recent competitors. The TRRL method uses the power of the computer to avoid laborious calculations, but the structure of the method is closely related to earlier methods whose evolution may be traced over a period of more than 40 years. Subsequent computer based methods have shown a change towards more elaborate and physically-plausible representations of the rainfall-runoff relationship. A particular feature of the new generation of urban drainage design methods has been the separate consideration of the above-ground and below-ground components of the rainfall-runoff process. The former involves the determination of the hydrograph of surface runoff at each stormwater inlet or road gulley, and the latter is concerned with the routing of the hydrographs through the pipe system. The Storm Water Management Model [40], the University of Cincinnati Urban Runoff Model [73] and the Chicago Hydrograph Method [90] provide ready examples of this type of approach.

These developments have led many designers to question whether the simplifications inherent in the TRRL Hydrograph Method may not lead to errors in the estimation of flows in certain circumstances. The features of the method which have attracted criticism have included:

(i) the simulation of the overland flow phase of runoff by means of a time of entry;

(ii) the assumption of 100 per cent runoff volume from paved and zero runoff from pervious areas;

(iii) the method of allowance for storage in the pipe system; and

(iv) the assumption that the storm profile of a selected return period produces a peak rate of flow with the same return period.

Recent research has re-examined these simplifications, and the improved solutions are presented in chapters 7, 8 and 9.

The TRRL method is also limited in its simulation of surcharged flows, surface flooding and antecedent catchment conditions. These constraints have made the method difficult to apply for economic studies, the checking of existing systems for deficiencies and investigations of system performance for events more severe than the design storm. A major objective of recent research has therefore been the development of new procedures capable of fulfilling these requirements.

3.3 Storm runoff quality

Growing interest has been shown in recent years in the quality of urban storm runoff. Some field data have been collected and experimental models do exist [81], but it is not yet possible to recommend a predictive procedure for the determination of the quality of storm runoff. A particular difficulty in the calculations is to predict the mass of pollutant available for transport at the start of a given rainfall event. This difficulty has been the source of considerable recent criticism [98] of the complex models available in North America. Another limitation in the available models is the assumption that the pollution due to heavy metals and biochemical oxygen demand (BOD) is directly proportional to the suspended solids load.

That separate urban stormwater runoff is polluted was originally demonstrated in the UK by Wilkinson [100] and this early conclusion has been confirmed by several more recent and comprehensive studies [24, 29, 61, 92]. Two further studies have examined the quality of runoff from motorways [37, 78]. The available water quality data from published studies of many storm events have been summarised in Table 3.1 which is provided only to illustrate the possible scale of the problem. The water quality of combined storm sewage has received less attention [22, 33, 36].

Table 3.1. A summary of water quality in separate stormwater runoff from urban catchments and motorways (Average concentrations in mg/l during storms)

Location	Oxhey	Stevenage	Nottingham	Hendon	Aston Expressway A38M
Suspended solids	194	112	21	581	1178
Total solids		364	418	978	
Chloride		49	142		2176
Soluble organic carbon		9.7			
Biochemical oxygen demand	7		6.8		32.3
Chemical oxygen demand			39.1	265	
Nitrate-nitrogen		1.7	7.8		
Ammoniacal-nitrogen		0.28	0.96		
Copper		0.03			0.70
Lead		0.21			2.40
Zinc		0.27			3.60
Reference No.	(100)	(61)	(29)	(24)	(37)

Blanks indicate that the item was not measured.

CHAPTER 4

Summary of the Wallingford Procedure

4.1 Sophistication in design

GOOD ENGINEERING practice is based on a correct appreciation of physical processes. Increased knowledge of the urban runoff process, in conjunction with advances in computational skills, have made it possible to produce computer programs to take account of factors only partially considered in previous methods. These factors include surface storage and attenuation, flood wave movement in pipes, flows in surcharged pipes, and the selection of design conditions based on an assessment of costs and benefits.

However, it is recognised that the sophistication (and cost) of design must be related to the total construction cost of the scheme involved. Technical excellence cannot be pursued for its own sake without regard to the potential value of the system under design and the improvements or savings which might be gained by more detailed design or additional effort. It is arguable that for large expensive storm drainage schemes it is worthwhile to apply a greater effort and to use more sophisticated techniques than could be justified for small, relatively inexpensive schemes: a small percentage change in the cost of a large scheme may involve a greater sum of money than a larger percentage change in the cost of a small scheme. On the other hand the total regional or national expenditure on the large number of small schemes may be more than on the smaller number of large schemes. This suggests that, from the wider point of view, efficient design is economically justifiable for all sizes of scheme.

4.2 Range of methods available in the Wallingford procedure

It is recognised that because of the range in size of storm drainage schemes and the consequent range in permissible design costs, designers need several calculation techniques. A method may be chosen which is appropriate to the size and cost of the scheme under design and the accuracy required. However, design costs and accuracy are not the only criteria which influence the choice of method: if information is required about one aspect of the performance of a system, it may be necessary to use a more sophisticated technique specifically to obtain that information.

The procedure therefore contains the following four methods:

Wallingford Modified Rational Method – A simple method, based on the Rational formula but modified in accordance with the results from recent research. The method gives a value of peak discharge only; no information is obtained on runoff volume or hydrograph shape. This method may be used without a computer but a programmed version is also available. The computer version includes the facility to represent storm overflows. The Modified Rational Method is recommended for initial designs and for use on homogeneous catchments up to 150ha in total area.

Wallingford Hydrograph Method – A computer-based hydrograph method incorporating separate models of the surface runoff and pipe-flow phases. This method will be appropriate to the majority of applications, for both the analysis of existing systems and the design of parts or the whole of new systems. Storm overflows, on-line and off-line tanks and pumping stations may be represented.

Wallingford Optimising Method – A computer-based method incorporating the optimised design of pipe depth and gradient as well as diameter. The design is optimised for minimum construction cost.

Wallingford Simulation Method – A computer-based method to examine the performance of an existing system or proposed design when operating under surcharged conditions. Storm overflows, on-line and off-line tanks, tailwater levels and pumping stations may be represented.

The construction cost program which forms an essential part of the Optimising Method may also be used in conjunction with the other methods.

The methods are described in detail in chapter 13.

Many improvements have been incorporated in these methods, but they cannot be regarded as perfect. Further improvements will be made as experience is gained in their use and as the results of further research become available.

4.3 Scope and limitations

The procedure is concerned with the hydraulic design and analysis of pipe networks (including relevant meteorological and surface flow aspects) and is therefore applicable to:
- both storm and combined sewer systems (flow from separate foul systems contributing to a combined system may be represented as an inflow at the head of a branch);
- the design of diameters and gradients of new pipe systems;
- the examination of the performance of existing systems;
- the behaviour of structures in the system in their hydraulic effect on the passage of flow;
- cost and economic considerations relevant to the selection of criteria for engineering design.

The procedure is not directly concerned with:
- the plan layout of sewer systems;
- the relative merits of combined or separate systems;
- the calculation of foul sewage flows;
- the selection and detailed hydraulic design of storm overflows and other ancillary structures;
- the quality of urban storm water;
- economic criteria and investment decisions beyond those concerned with the selection of design criteria;
- the calculation of flow from any rural area which may contribute to an urban drainage network.

However, the procedure can be used to examine the hydraulic and cost consequences of alternative decisions relating to these latter aspects.

Several parts of the procedure are based on conditions and engineering practices within the UK and should not therefore be used in any overseas applications. These limitations are set out in chapter 18.

4.4 Principal new features

The recommended procedure departs in several ways from previous approaches. The major new features are:

(i) A range of calculation methods is provided, from which an approach suitable to each particular problem may be selected.

(ii) The results of the Meteorological Office's most recent studies of UK rainfall are included within the procedure, giving the user direct access to locally applicable rainfall data.

(iii) The surface flow and pipe flow phases of urban runoff are treated separately.

(iv) Existing overloaded systems may be analysed accurately, using a program which calculates the performance of pipe systems in a surcharged state.

(v) Designs are based on the return period of the flow rather than of the rainfall. A research exercise has examined the relationship between rainfall frequency, runoff frequency and other rainfall and catchment variables. Values are provided which permit systems to be designed for a discharge rather than a rainfall of a specified return period.

(vi) The design return period may be selected on the basis of the performance of alternative designs rather than as an arbitrary value specified at the start of calculations. Since the purpose of storm drainage is to reduce the risk of surface flooding to acceptable levels, designers should not be concerned solely with pipe-full flows in sewers. They should consider the occurrence of more severe events which sometimes cause various degrees of surcharging and surface flooding. The likely consequences of rainfalls less frequent than those used in design can be evaluated by using the program which calculates the performance of a surcharged system. It is recommended that testing a proposed system and associated alternatives before construction should be an essential part of any design exercise.

(vii) Cost calculations may be included. Storm drainage has to provide a public service within the constraint of total resources available. Therefore designs should ideally be based on an optimisation of construction costs and the service provided. A routine to calculate the costs of the resources used in construction is therefore included as an option to permit the comparison of the cost of alternative designs, leading to a more logical choice of final design.

(viii) Pipe gradients may be designed within the computer program, rather than specified by the engineer. An optimised design to achieve minimum construction costs may then be produced.

4.5 Testing of the procedure

In the course of its development the procedure described in this report has been tested in several ways.

At the Hydraulics Research Station the programs have been tested for their reliability and appropriateness to typical urban drainage problems. The accuracy of the programs in simulating observed rainfall-runoff events has also been examined.

The limited quantity of reliable rainfall-runoff data from adequately surveyed sewered catchments restricted the simulation tests to 142 events on three catchments. It is recognised that performance on three catchments might not be representative of accuracy in general use. Furthermore, the tests led to various improvements in the methods under development and could be misinterpreted if regarded as representative of the final versions of the new methods. The results of the simulation tests are available in the Hydraulics Research Station research report [77]. The overall findings of the tests were as follows:

(i) All the methods examined (both previously available methods and those included in the Wallingford procedure) showed a large scatter in their accuracy in predicting observed events. The standard deviation in the calculated peak discharges or runoff volumes was typically 30 per cent of the observed values. This scatter was due partly to errors in the data and partly to limitations of modelling the minute details of the rainfall-runoff process. Considerable caution should therefore be used when interpreting the results from any urban runoff model of a simulation of a single observed event.

(ii) The Wallingford Hydrograph Method was considerably more accurate than the TRRL method in simulating runoff volume, and marginally more accurate in simulating peak discharge.

(iii) The Wallingford Modified Rational Method was considerably more accurate than the Rational (Lloyd-Davies) Method in simulating peak discharge.

(iv) The Wallingford Modified Rational Method was no less accurate than the Wallingford Hydrograph Method in simulating peak discharge. This result led to the recommendation that the Modified Rational Method could be used for the determination of peak discharges from catchments up to 150ha in total area with reasonably uniform slope and distribution of impervious area.

The tests also showed that in some circumstances different peak discharges were obtained from the Wallingford Hydrograph Method and the Modified Rational Method. For the sake of consistency, networks which are to be examined using the Simulation Method should be designed by the Hydrograph Method, since these incorporate many of the same principles.

During 1979 and 1980 three organisations represented on the Working Party on the Hydraulic Design of Storm Sewers applied the programs to some of the drainage projects handled by their offices, and valuable suggestions for modifications to the procedure resulted. During 1980, a further eight design offices representative of water authorities, local authorities, and consultants made use of the final draft programs and reports. These eight organisations had no previous contact with the Working Party or with the research activities; their independent evaluation led to valuable improvements which were incorporated in the programs and accompanying reports.

PART 2

The basis of the Wallingford Procedure

CHAPTER 5

Introduction to Part 2

THE DESIGN and analysis procedure described in this report has been produced as a result of cooperation between research projects at several organisations. The Meteorological Office was responsible for the rainfall aspects, the Institute of Hydrology for the surface runoff phase (between rainfall and pipe entry) and the Hydraulics Research Station for the pipe flow phase. Other material was supplied by members of the Working Party on the Hydraulic Design of Storm Sewers and its drafting groups. The Hydraulics Research Station was also responsible for combining all contributions into a single set of programs and reports.

Chapters 6 to 12 provide the background to and scientific justification of the various facilities which have been made available in the procedure. This information is provided to give the user confidence in the basis of the programs, and also to discourage application of the programs outside the limits for which they were intended.

The provision of scientific explanation does not mean that the user himself has to apply all the concepts and equations; the majority are incorporated within the computer programs which will perform the calculations on the basis of the data supplied.

The material is divided into the following themes:

chapter 6 Storm rainfall
7 Surface runoff
8 Pipe flow
9 Selection of design storm and antecedent conditions
10 Runoff from sewered sub-areas
11 Ancillary structures in sewerage systems
12 Costs and economics

The material in these chapters represents different degrees of advance of scientific knowledge, and therefore different levels of discussion and justification are appropriate. In those aspects where the procedure continues to use a widely accepted principle less justification is required.

Chapter 6 relies extensively on the rainfall studies included in the Flood Studies Report [69]. Most of the information is not new, but the chapter brings together for the first time the information of special relevance to storm drainage design. One new feature is that the user may calculate his own design rainfall data, either by using the programs included in the procedure or by a slightly more approximate manual method.

Chapter 7, describing the surface phase of runoff, is based on the results of entirely new research into the behaviour of rainfall on urban surfaces. Many of the concepts will be new to storm drainage designers and the resulting surface runoff modelling represents a major change for storm drainage design. The level of detail in this chapter reflects the relative novelty of the subject matter.

The hydraulics of pipe flow have been relatively well understood for some time and parts of chapter 8 therefore contain only a summary of existing knowledge and design practice which has been maintained in the procedure. More detailed consideration is given to the new features in this area: the use of the Muskingum-Cunge technique for the routing of flood waves in pipes, and a new method to simulate the behaviour of surcharged flow in pipe networks.

Chapter 9 describes the basis for determining the design values of rainfall and antecedent conditions so that a discharge of a specified return period may be determined.

Chapter 10 describes a simplified method of calculating the runoff hydrograph from a sewered area for which sufficient data are not available for the application of the more detailed programs. The simplified method makes use of the findings of recent research into the relative roles of surface flow and pipe flow, but on the basis of a much reduced set of data.

The ancillary structures which can be included within the programs are described in chapter 11.

Cost calculations and aspects of economic evaluation which can be examined using the procedure are considered in chapter 12. Wider principles of the economic assessment of storm drainage are discussed in Part 4 (chapter 16).

CHAPTER 6

Storm rainfall

6.1 Rainfall research results in the Flood Studies Report

THE MAJOR RECENT advance in our knowledge of UK storm rainfall is embodied in the Flood Studies Report [69], see section 3.1 above. Volume II and Volume V (Maps) of the report contain the rainfall information, though Volume I should be consulted for the details of the statistical basis of the work. The statistical methods used are modern developments of the statistics of extremes.

The rainfall intensities and profiles recommended in the Flood Studies Report may be obtained in one of three ways:

(i) The Meteorological Office will provide constant intensities and storm profiles for any specified range of duration and return periods. These values are for a particular National Grid reference provided by the user.

(ii) The procedure used by the Meteorological Office is included in the computer programs prepared for storm sewer design. The user may therefore generate his own site-specific rainfall data, based on a small number of data values read from maps. The only approximation involved is the slight inaccuracy of reading the required values from small scale maps.

(iii) A manual method of calculating intensities and profiles is provided for users without access to a computer. This method involves a greater number of approximations, but should be adequate for those using the manual methods of design and analysis.

The computer programs will also accept any specified series of rainfall intensity values for any specified storm duration and time interval. This facility will be used when examining the behaviour of a storm drainage system under an observed rainfall event. An intensity-duration relationship can also be determined for any input hyetograph; this facility is provided for preliminary simulation of observed events using the Modified Rational Method.

The following sections of chapter 6 describe the major rainfall parameters which affect the choice of the correct storm intensities or profilies:
6.2 Design storm duration
6.3 Design return period
6.4 Total depth of rainfall
6.5 Areal reduction factor
6.6 Shape of the storm profile
6.7 Smoothing the point rainfall profile

6.2 Design storm duration

A design storm is a sequence of rainfall intensities of a defined total duration. Real recorded storms could be adapted to produce such a design storm but in most systematic design methods it is usual to specify some simple characteristics of the storm which lead to a calculated sequence of rainfall intensities.

The duration required for the design storm cannot be defined exactly. When using a uniform intensity method such as the Rational Method it has been customary to use a duration equal to the time of concentration, and the continuation of this practice is recommended. Studies with hydrograph methods, using a variable intensity design storm (see section 3.1) have shown that the storm duration which produces the maximum discharge is usually larger than the time of concentration. It has been suggested [19] that the duration may be dependent on other characteristics of the rainfall regime, with somewhat longer durations being required in the north and west of the UK.

It is apparent that the design storm duration will vary within a drainage system, as the time of concentration increases proceeding downstream. The recommended hydrograph procedure therefore uses a series of rainfall durations and the largest discharge calculated at each point in the system is taken as the design discharge. The peak discharge is not very sensitive to the storm duration: a doubling of duration causes a change of less than 10 per cent in peak discharge. Therefore a fairly coarse series of rainfall durations may be used: values of 15, 30, 60 and 120 minutes are recommended. For most storm drainage systems, calculations will not need to proceed beyond a duration of 120 minutes.

The above paragraphs refer to the design storm duration for the determination of peak discharge. For the determination of runoff volumes (for example, for the design of storage tanks) the same principles apply but the durations will be longer. Some recent work [19] has suggested that durations about 30 per cent greater than those giving the peak discharge are required.

6.3 Design return period

The selection of the design return period is an economic rather than a meteorological decision. Longer return periods will lead to systems with greater capacities, providing a higher standard of drainage at higher cost. Many factors influence the selection of the best balance between expenditure and the service provided, and these issues are explored in chapter 16.

The drainage engineer is concerned primarily with the return period of flows rather than of rainfalls. The rainfall is of course the dominant cause of the flow in the storm drainage system, but the antecedent condition (wet or dry) of the catchment has an important secondary influence. It can be shown that the frequency distributions of rainfalls and peak discharges are bound to differ. However, it is very convenient in design to assume equal return periods of rainfall and runoff. So long as it is realised that the T-year rainfall does not necessarily *cause* a T-year discharge, the use of equal return periods is acceptable if the other recommendations on storm duration (section 6.2), storm profile (section 6.6) and antecedent conditions (section 9.4) are followed. This subject is covered in more detail in chapter 9.

The return period used in the rainfall input should therefore be the same as that of the required discharge.

6.4 Total depth of rainfall

A uniform intensity design method, such as the Rational Method, requires an average rate of rainfall of given return period over a series of durations. A hydrograph design method requires a rainfall hyetograph (values of intensity varying with time). Both these approaches require a total depth of rainfall of given return period occurring in a given period of time.

Two methods for obtaining the values are provided: a computer method incorporated within the programs and a manual method. Both are based on the following sequence of calculations (the convention used is that MT-D represents the depth of rainfall in mm occurring in a duration D with a return period T years; D is in hours unless otherwise stated):

(i) Determine values of M5-60 min and the ratio r ($= \dfrac{\text{M5-60 min}}{\text{M5-2 days}} = \dfrac{\text{M5-60 min}}{\text{M5-48}} \times 1.06$)

(ii) Determine values of M5-D for various durations D

(iii) Determine values of MT-D for various return periods T

The difference between the computer and manual methods lies in the way in which these calculations are performed and the precision of the result.

The methods described below involve the use of two maps covering the United Kingdom together with graphs and tables and is adopted directly from the Flood Studies Report. The maps are provided at a scale of 1:1000 000 in a separate folder; smaller scale maps are also included within this report. The M5-2 day map published in the Flood Studies Report has here been replaced by a map of M5-60 min as being more appropriate to urban drainage.

All durations in the range five minutes to 48 hours and return periods in the range one year (M1) to 100 years (M100) are included. Note that the rainfalls corresponding to M5 are estimated with the best accuracy. Those corresponding to higher and lower return periods, eg M100 and M1, are less well estimated but the degree of uncertainty will not usually be appreciable, especially in M1.

The computer method

This method is incorporated within the package of computer programs. The method proceeds as follows:

(i) Values of M5-60 min and r are obtained from Figs. 6.1 and 6.2 or the larger scale maps available separately. M5-60 min should be estimated to the nearest mm (or half mm in areas of weak gradient); r should be estimated to $\pm$ 0.01.

(ii) M5-D is given by[50]:

$$\ln M5\text{-}D = \ln D + \ln\left(1.06\,\frac{M5\text{-}60\text{ min}}{48r}\right) + \left[\ln \frac{721}{1 + 15D}\,\ln\left(\frac{48r}{1.06}\right)/\ln\left(\frac{721}{16}\right)\right] \qquad \ldots\ldots 6.1$$

Note that M5-60 min refers to the 1 hour rainfall starting in any minute as opposed to the 'clock hour' rainfall where rainfall starts at a definite time; it is the former which is appropriate to storm sewer design.

(iii) The general equation linking rainfall depths of different return periods for a specific duration to those of return period 5 years is independent of duration D for return periods greater than 5 years:

$$\ln \frac{MT\text{-}D}{M5\text{-}D} = C_r(\ln(T) - 1.5) \qquad \ldots\ldots 6.2$$

where C_r is a constant varying with geographical location and with the value of M5-D itself. C_r can be expressed as a quadratic in M5-D:

$$C_r = J_0 + J_1 M5\text{-}D + J_2\,(M5\text{-}D)^2 \qquad \ldots\ldots 6.3$$

The values of J_0, J_1 and J_2 used to calculate C_r are given in Table 6.1. These values are stored within the programs; the user has to specify only the appropriate location index, also tabulated in Table 6.1.

For return periods less than five years, no general formula such as equation 6.2 has been obtained, so the empirical proportions set out in the appropriate columns of Tables 6.2 and 6.3 are used. The values for M5 are slightly greater than unity in order to convert values based on annual maximum series to partial duration series (which describe the return periods of all events above a given threshold). For the same reason some other values in Tables 6.2 and 6.3 differ slightly from those published in the Flood Studies Report.

Manual method

(i) Values of M5-60 min and r are again obtained from Figs 6.1 and 6.2 or the larger scale maps as described above for the computer method.

(ii) M5-D is obtained from the relationship:

M5-D = Z1(M5-60 min) 6.4

The factor Z1 is read from Figures 6.3a or 6.3b for increments of r of 0.03 from r = 0.12 to r = 0.45 and for durations between five minutes and 48 hours. Z1 should be read from the graphs to an accuracy of about 0.01.

(iii) MT-D is obtained from the relationship:

MT-D = Z2(M5-D) 6.5

The factor Z2 is read from Table 6.2 for England and Wales and Table 6.3 for Scotland and Northern Ireland.

The following example illustrates the use of the manual method to provide the two hour once in twenty years (M20-2) rainfall for a location in Ashford, Kent:

(M5-60 min) from Figure 6.1 = 19mm
r from Figure 6.2 = 0.37
Z1 from Figure 6.3b = 1.19
∴(M5-2) = 1.19 x 19 = 22.6mm
Z2 from Table 6.2 for M20 = 1.44
∴(M20-2) = 1.44 x 22.6 = 32.5mm

The manual method described above provides rainfall depths of any duration and return period for a specified location. For use in manual calculations using average rainfall intensities, these rainfall depths may be converted to average intensities by dividing by the appropriate durations.

6.5 Areal reduction factor

In the storm sewer design procedure the design rainfall is assumed to be the same at all parts of the catchment and storm movement or development is ignored. However, it is possible to take some account of the areal variation of rainfall over larger catchments by the use of an areal reduction factor.

The average rainfall, of a given return period, over an area is less than the corresponding point rainfall of the same return period, at least for return periods of greater than several months. Several methods of defining the areal reduction factor are available but the one that is most relevant to flood design work is the ratio of design areal to point rainfall, each of the same return period. The Flood Studies Report values of areal reduction factor were obtained for a given area by comparing the average of many annual maximum point rainfalls of a given duration (which may have arisen from different storms) with annual maximum areal rainfalls of the same duration (each of which arose from one storm). These ratios were then averaged over many years and the entire exercise repeated for different areas. The values quoted in the Report are based on a number of regions in England. The method of calculation corresponds to a return period of two to three years. Recent work [7, 31] suggests that the areal reduction factor is only very weakly dependent on return period for return periods greater than one year. For commoner return periods a stronger relationship is indicated but such (short) return periods are beyond the scope of this report.

The areal reduction factor has the most noticeable effect on rainfalls of short durations and on large areas. Because of its dependence on catchment area the factor should, in theory, increase as calculations proceed downstream through a drainage area. In most urban drainage catchments the areal reduction factor will be greater than 0.9, and the variation will not be large.

For these reasons the areal reduction factor is calculated within the programs using total catchment area.

The areal reduction factor, ARF, is calculated from

$$ARF = 1 - f_1 D^{-f_2} \quad \text{....6.6}$$

where f_1 and f_2 are functions of the drainage area. Table 6.4 gives the values for areas less than $100km^2$. Further details can be found in reference 50.

For manual application two methods of calculation are available. Either equation 6.6 or Figure 6.4 may be used to give the areal reduction factor, which may be regarded as uniform throughout the UK, for durations of five minutes to 48 hours and areas of $1km^2$ to $100km^2$. Values read from Figure 6.4 may differ very slightly from values derived from equation 6.6.

This completes the calculation of rainfall values for use in methods using average intensities. The calculation of rainfall hyetographs for use in hydrograph methods continues in sections 6.6 and 6.7.

6.6 Shape of the storm profile

Symmetrical rainfall profiles, with the maximum rainfall intensity at the centre of the storm, are recommended for design purposes by the Flood Studies Report. The ratio of maximum to mean intensity defines the 'peakedness' of the profile.

Volume II of the Flood Studies Report studied the relative frequency distribution of peakedness in summer and winter months separately. It was found that summer storms were more peaked on average than winter ones. The distribution of peakedness was defined in terms of 'percentile peakedness', ie the percentage of storms with a peakedness less than or equal to that of a given profile. Percentile peakedness is an important parameter, substantial variations of which can cause significant variations in maximum flow. As a result of the research exercise described in chapter 9, the 50 per cent summer profile is recommended for storm sewer design.

The design methods recommended in this report have been developed for use with the 50 per cent summer profile, and the use of other profiles is not recommended.

Table 6.5 gives the shape of the 50 per cent summer profile expressed as a percentage of the mean rainfall intensity and cumulative depth against percentage storm duration. The mean intensity of the total storm profile $\bar{i}$ is given by:

$$\bar{i} = \frac{MT\text{-}D}{D} \quad \text{....6.7}$$

where D is the duration in hours.

The mean intensity over a specific duration t to t + 1 is given by:

$$i_{t,\,t+1} = \frac{P'_{t+1} - P'_t}{D'_{t+1} - D'_t} \times \bar{i} \quad \text{....6.8}$$

where P' is the percentage cumulative depth and D' is the percentage duration as shown in Table 6.5.

Continuing the example given at the end of section 6.4, the intensity in the 38th minute of a 120 minute 50 per cent summer profile at Ashford, Kent may be calculated as follows:

The 38th minute is bounded by t = 37 and t + 1 = 38.

Thefore $D'_{37} = \frac{37}{120} \times 100 = 30.83$ per cent and $D'_{38} = \frac{38}{120} \times 100 = 31.67$ per cent

By interpolation from Table 6.5 the appropriate values of P'_{37} and P'_{38} are 13.47 and 14.07 respectively. The total rainfall in the storm was 32.5mm so the average intensity is 16.25mm/hr.

$$\text{Therefore } i_{37,\ 38} = \frac{14.07 - 13.47}{31.67 - 30.83} \times 16.25 = 11.6 \text{ mm/hr}$$

6.7 Smoothing the point rainfall profile

The storm profile obtained so far corresponds to a single point whereas the programs require a catchment-average profile. The process of averaging rainfall over a catchment leads to a smoother profile than the corresponding point profiles. This is a separate process from the ARF adjustment for *total* rainfall depths between point and areal values.

The smoothing effect will vary from one event to another and would ideally be modelled differently in the two cases of design (using the 50 per cent point profile) and simulation (using a recorded rainfall sequence). However, the effect is not large and the smoothing process may be represented in all cases by a simple moving average process, or filter, dependent only on catchment area.

The filter may be written:

$$p'_t = \mu p_{t-1} + (1-2\mu)p_t + \mu p_{t+1} \qquad \ldots.6.9$$

where p'_t is the filtered rainfall ordinate at time t
p_t is the original rainfall ordinate at time t
μ is a parameter determined from

$$\mu = 0.1615\ AT^{0.405} \times \frac{60}{\Delta t} \qquad \ldots.6.10$$

where AT is catchment area (km^2)
and Δt is the data interval (sec).

This smoothing process will extend the storm duration by one time interval at each end. It is suggested that the smoothing process should be carried out on both design rainfall profiles and on observed hyetographs. The process is included in the computer programs but it should be noted that research on profile smoothing is still at an early stage and may be altered later.

Table 6.1 Values of constants to calculate C_r

Geographical location	Range of M5-D	Location index	J_0	J_1	J_2
England and Wales	0-10		1699×10^{-4}	2800×10^{-6}	114000×10^{-9}
	10-30		1644	5831	−134300
	30-75	1	2644	−1621	3150
	75-150		2718	−1947	6187
	>150		1454	−194	114
Scotland and N Ireland	0-13		1648	8330	−304700
	13-25		2349	−771	−17250
	25-50	2	2502	−2109	12130
	50-150		2274	−1208	3220
	>150		1460	−202	120

Table 6.2 Relationship between rainfall of return period T (MT) and M5 – England and Wales (Ratio Z2)

M5 Rainfall mm	M1	M2	M3	M4	M5	M10	M20	M50	M100
5	0.62	0.79	0.89	0.97	1.02	1.19	1.36	1.56	1.79
10	0.61	0.79	0.90	0.97	1.03	1.22	1.41	1.65	1.91
15	0.62	0.80	0.90	0.97	1.03	1.24	1.44	1.70	1.99
20	0.64	0.81	0.90	0.97	1.03	1.24	1.45	1.73	2.03
25	0.66	0.82	0.91	0.97	1.03	1.24	1.44	1.72	2.01
30	0.68	0.83	0.91	0.97	1.03	1.22	1.42	1.70	1.97
40	0.70	0.84	0.92	0.97	1.02	1.19	1.38	1.64	1.89
50	0.72	0.85	0.93	0.98	1.02	1.17	1.34	1.58	1.81
75	0.76	0.87	0.93	0.98	1.02	1.14	1.28	1.47	1.64
100	0.78	0.88	0.94	0.98	1.02	1.13	1.25	1.40	1.54
150	0.78	0.88	0.94	0.98	1.01	1.12	1.21	1.33	1.45
200	0.78	0.88	0.94	0.98	1.01	1.11	1.19	1.30	1.40

Table 6.3 Relationship between rainfall of return period T (MT) and M5 - Scotland and Northern Ireland (Ratio Z2)

M5 Rainfall mm	M1	M2	M3	M4	M5	M10	M20	M50	M100
5	0.67	0.82	0.91	0.98	1.02	1.17	1.35	1.62	1.86
10	0.68	0.82	0.91	0.98	1.03	1.19	1.39	1.69	1.97
15	0.69	0.83	0.91	0.97	1.03	1.20	1.39	1.70	1.98
20	0.70	0.84	0.92	0.97	1.02	1.19	1.38	1.66	1.93
25	0.71	0.84	0.92	0.98	1.02	1.18	1.37	1.64	1.89
30	0.72	0.85	0.92	0.98	1.02	1.18	1.36	1.61	1.85
40	0.74	0.86	0.93	0.98	1.02	1.17	1.34	1.56	1.77
50	0.75	0.87	0.93	0.98	1.02	1.16	1.30	1.52	1.72
75	0.77	0.88	0.94	0.98	1.02	1.14	1.27	1.45	1.62
100	0.78	0.88	0.94	0.98	1.02	1.13	1.24	1.40	1.54
150	0.79	0.89	0.94	0.98	1.02	1.11	1.20	1.33	1.45
200	0.80	0.89	0.95	0.99	1.01	1.10	1.18	1.30	1.40

Table 6.4 Areal reduction constants for use in equation 6.6

Size of area, AT (Km^2)	f_1	f_2
AT<20	$0.0394\ AT^{0.354}$	$0.40\text{-}0.0208\ \ln(4.6 - \ln AT)$
20≤AT<100	$0.0394\ AT^{0.354}$	$0.40\text{-}0.00382\ (4.6 - \ln AT)^2$

Table 6.5 Shape of the 50 percentile summer storm profile

% Duration	% of Mean Intensity	% of cumulative depth	% Duration	% of Mean Intensity	% of cumulative depth
1	32	0.32	26	54	10.52
2	33	0.65	27	56	11.08
3	33	0.98	28	58	11.66
4	34	1.32	29	61	12.27
5	34	1.66	30	64	12.91
6	35	2.01	31	68	13.59
7	35	2.36	32	72	14.31
8	36	2.72	33	78	15.09
9	36	3.08	34	84	15.93
10	37	3.45	35	91	16.84
11	37	3.82	36	99	17.83
12	38	4.20	37	110	18.93
13	38	4.58	38	123	20.16
14	39	4.97	39	136	21.52
15	40	5.37	40	152	23.04
16	41	5.78	41	170	24.74
17	42	6.20	42	188	26.62
18	43	6.63	43	208	28.70
19	44	7.07	44	228	30.98
20	45	7.52	45	250	33.48
21	46	7.98	46	274	36.22
22	48	8.46	47	300	39.22
23	49	8.95	48	328	42.50
24	51	9.46	49	358	46.08
25	52	9.98	50	392	50.00

The profile is symmetrical about its mid-point (50 per cent duration)

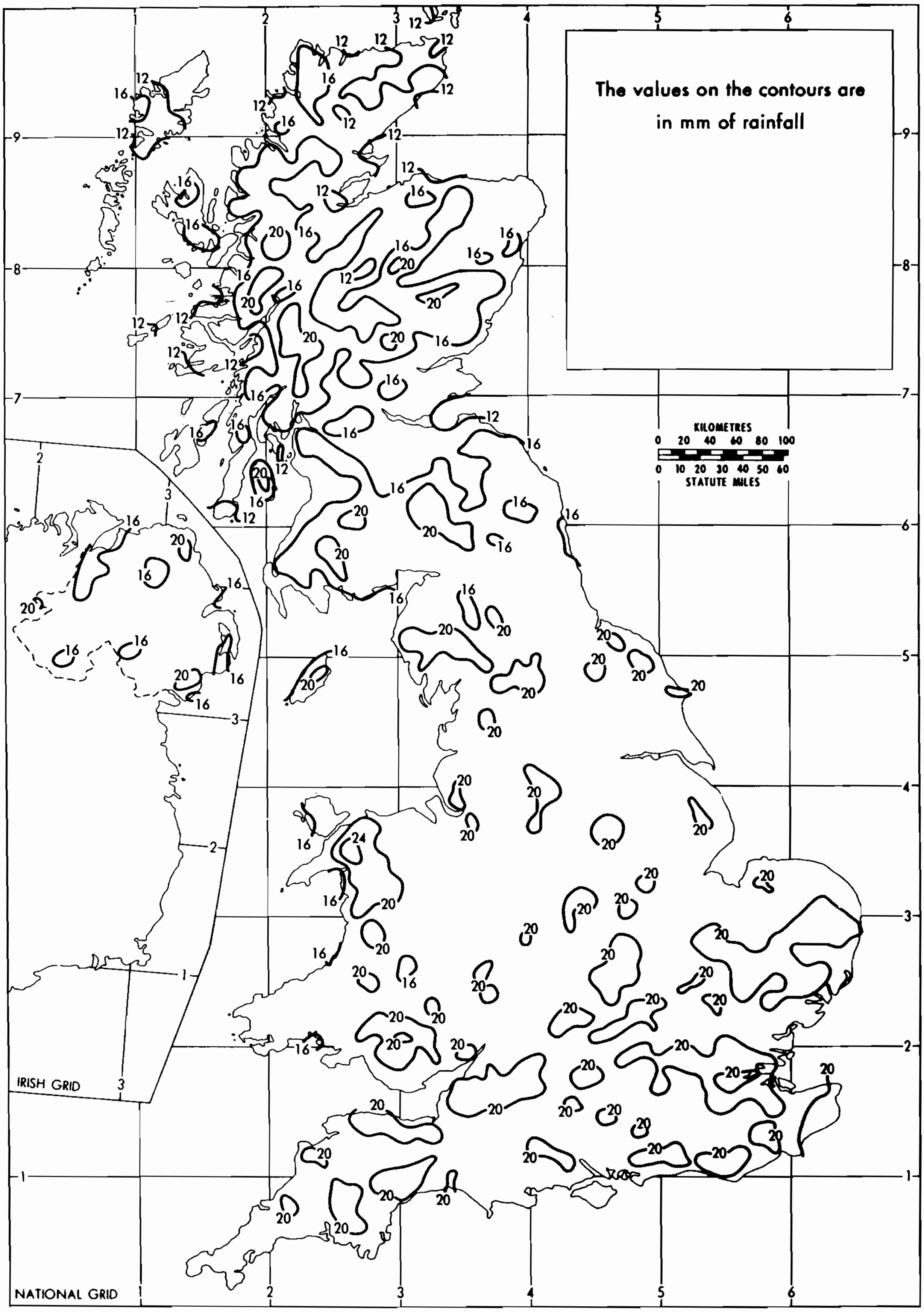

Figure 6.1. Rainfall depths of five year return period and 60 minutes duration (M5-60 min)

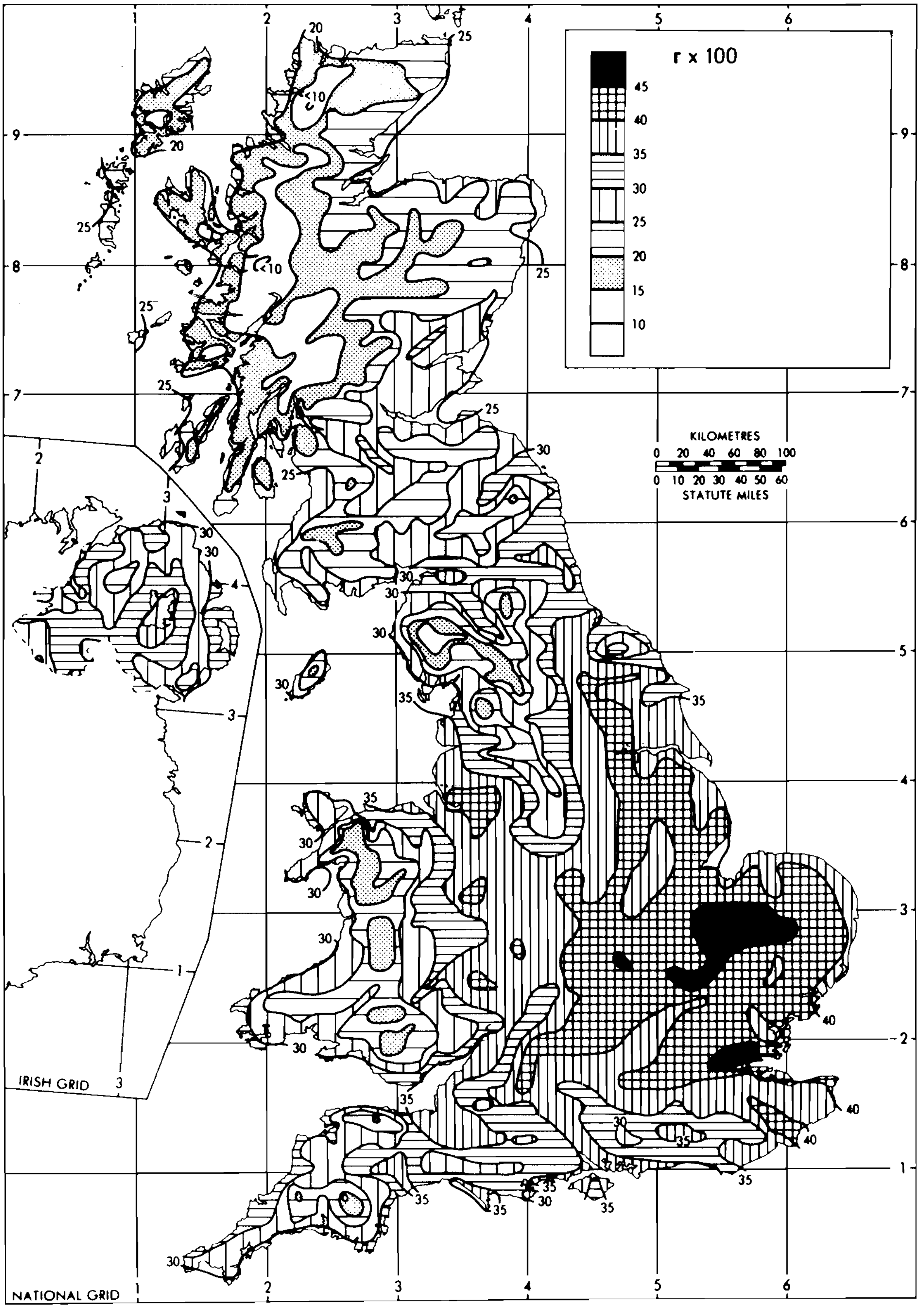

Figure 6.2. Ratio of sixty minute to two day rainfalls of five year return period (r)

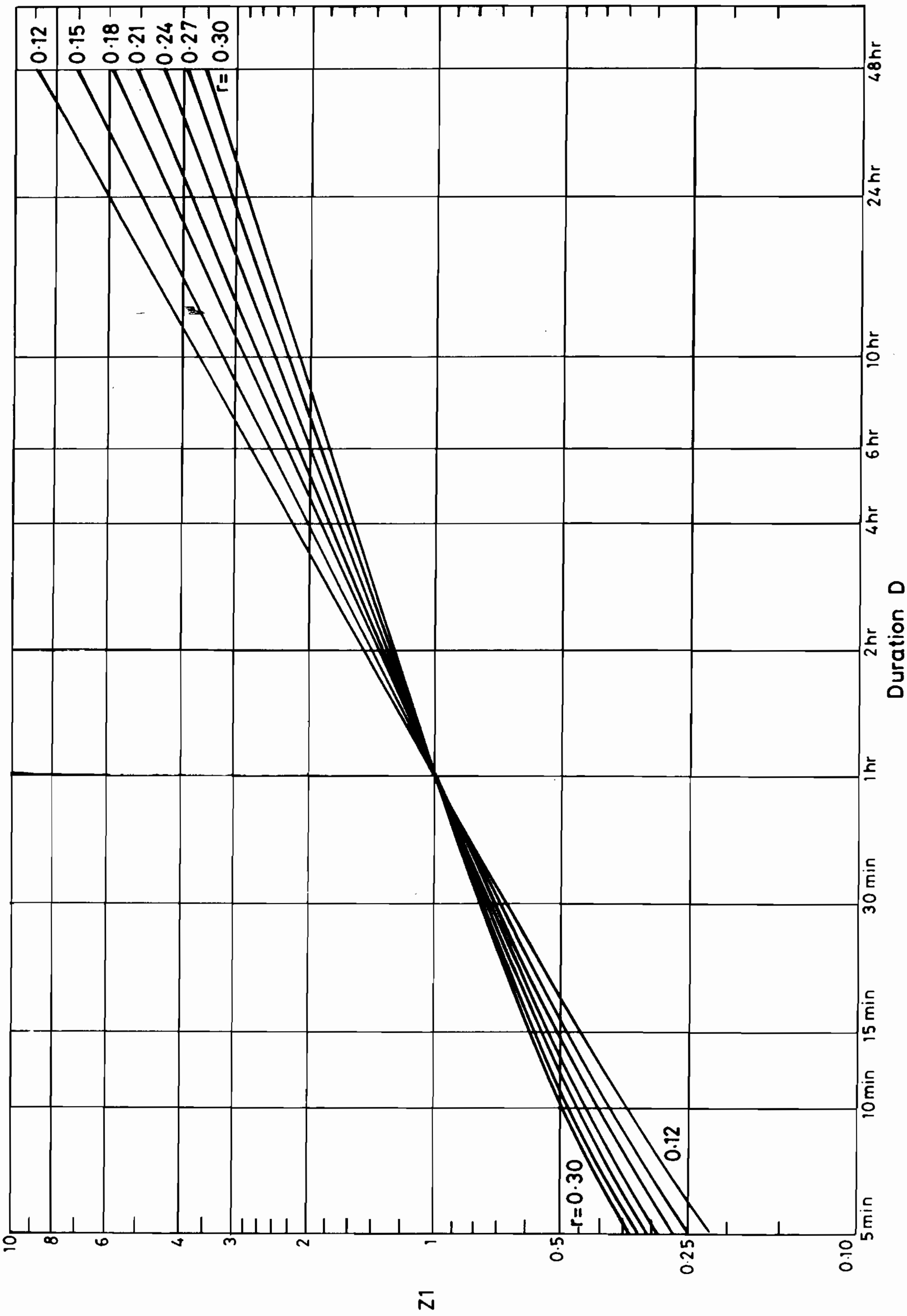

Figure 6.3a. Relation between Z1 and D for different values of r. (0.12≤r≤0.30)

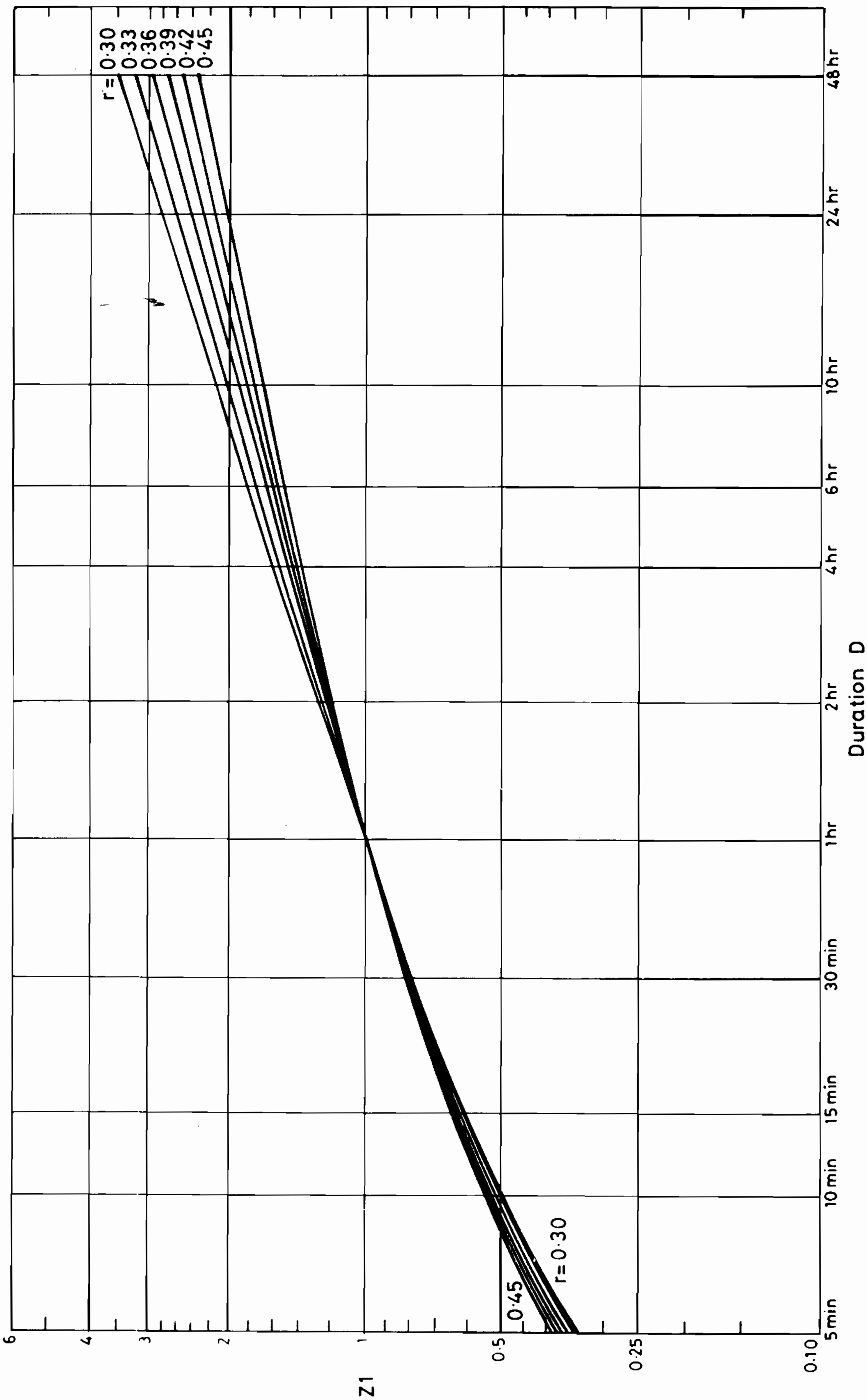

Figure 6.3b. Relation between Z1 and D for different values of r. (0.30≤r≤0.45)

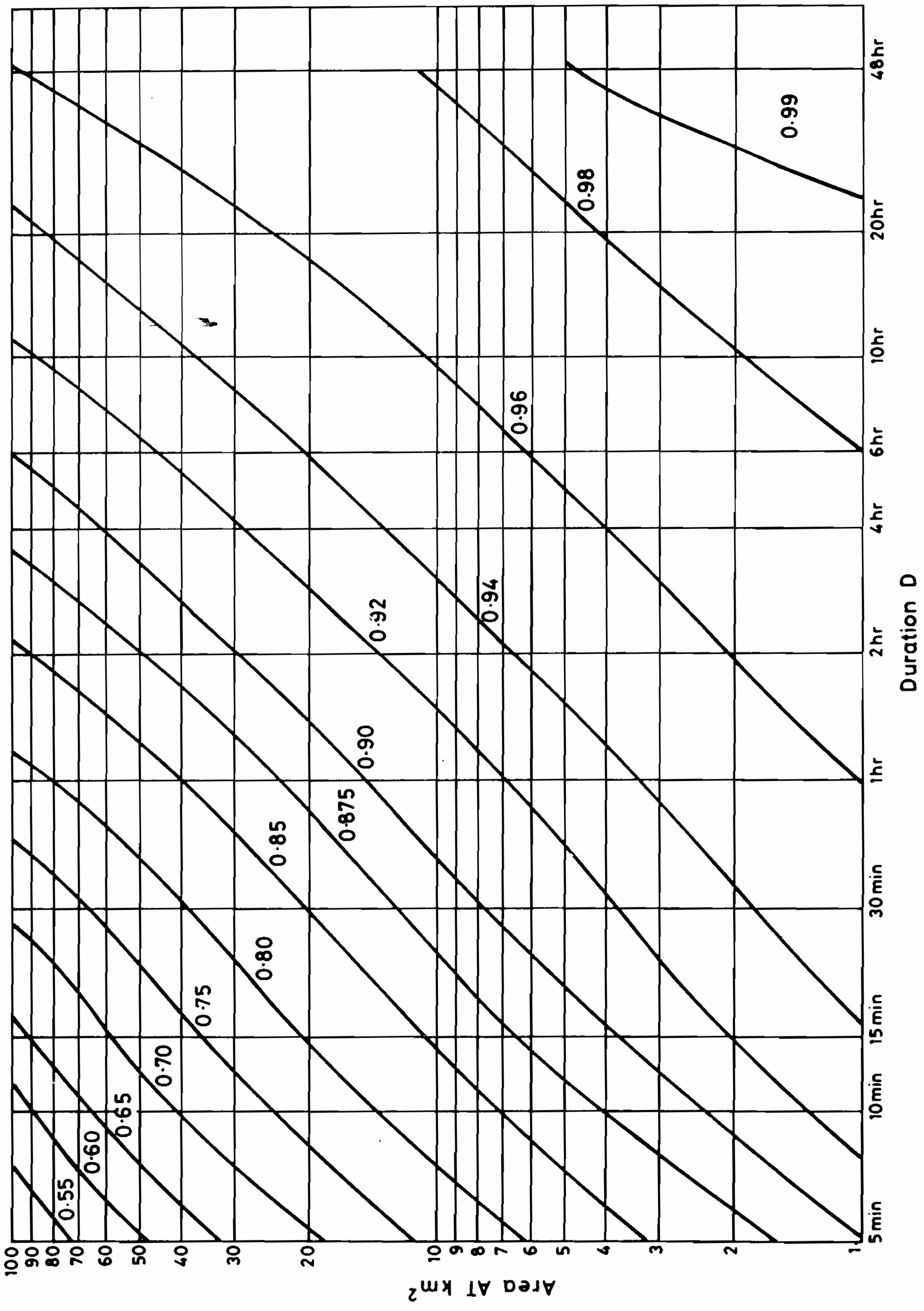

Figure 6.4. Areal reduction factor ARF related to Area AT and Duration D

CHAPTER 7

Surface runoff

7.1 Introduction

CHAPTER 7 is concerned with the way in which rain falling on an urban catchment is changed into a runoff hydrograph at the points of entry to the sewer system. This is termed the 'surface runoff phase' in urban runoff modelling but it is only recently that it has been clearly separated from the underground phase of pipe flow routing.

The Lloyd-Davies method [57] (a special case of the more flexible Rational Method), which was introduced in 1906, implicitly assumes that the contributing area of the catchment increases linearly with time (ie if, in steady continuous rain, runoff from all of the potential contributing area of a catchment is reaching the outlet after the time of concentration, t_c, then half the area is contributing after $t_c/2$, a quarter after $t_c/4$ and so on). Later methods, of which the TRRL method [91] is the most recent and most widely used in Britain, have made use of the time-area diagram which allows the true distribution of impervious area to be taken into account. The base length of the time-area diagram (or time of concentration) embraces delay times arising from both flow in the pipes and surface flow. The delay due to the surface processes is usually referred to as the time of entry.

There is general agreement that flow hydrographs predicted by Rational or time-area methods require to be attenuated to reflect the effects of reservoir-type storage in the system as a whole. In the TRRL method a storage allowance is made but it is based solely on the geometry of the pipe system; it is now thought that some of the attenuation should in fact be attributed to surface storage.

Another argument for separate modelling of the surface runoff phase rests on the possible oversimplification in the common assumption, for design purposes, of 100 per cent runoff from directly connected impervious areas and zero runoff from pervious areas.

The case for a separate and more realistic treatment of the surface runoff process has been gathering momentum for some years but it is only in the last few years that data have been collected for this specific purpose. There is more to be learnt but it is felt that the time is right to introduce the new concepts not only because they are more firmly based on hydrological observations and on the application of engineering science but also because savings in construction costs may be possible. In some situations the recognition of surface storage potential could lead to fewer inlets, longer drainage paths and a shortening of the pipe system. The procedure provides a tool with which to examine these aspects of drainage design.

Most of this chapter argues the case for a more detailed calculation of the surface phase of urban runoff than is employed in the 'runoff coefficient' and 'time of entry' concepts. The more detailed surface runoff modelling which is described is incorporated in the hydrograph methods included in the Wallingford procedure. However, it is recognised that in some situations a simple calculation method is required, and for this purpose a modified version of the Rational Method has been retained: alternative values for the runoff coefficient and the time of entry, which go some way towards improving the method, are recommended in section 7.10.

7.2 Hydrological processes

Consider a storm starting to fall on a dry urban area. The first few tenths of a millimetre are absorbed by all the different surfaces. Then, on the least pervious surfaces, droplets coalesce and small pools form quickly.

On steep slopes, downhill movement begins immediately: in flat areas, puddles form. As the storm continues, overland flow is generated on an increasing proportion of the catchment. Finally, all the surface depressions are full and most of the impervious area is contributing some incident rainfall to the overland flow process. Most urban surfaces, however nominally impervious, allow some degree of infiltration or hold water by surface tension and so do not contribute 100 per cent runoff. A proportion of the impervious area will contribute even less because its runoff is immediately passed to pervious surfaces either inadvertently or by design.

A similar sequence is followed on the pervious parts of the catchment but usually at a much reduced rate since infiltration is the dominant process. Runoff generation is dependent on the type of soil, the rainfall intensities, surface slopes, and vegetation. A steeply sloping, close cropped grass verge on a clay soil can become a runoff contributing area before a flat and poorly maintained (ie potholed) car parking area.

Clearly, if the catchment is wet at the start of the storm with some of the surface depressions already full and with saturated soils, the proportion of the rainfall which forms runoff is likely to be significantly increased.

As the generated runoff moves downhill over the assorted surfaces, its depth and velocity change with distance and time depending on the surface slope and roughness. These changes can be described mathematically in the idealised case of sheet flow over plane surfaces but the required continuity and momentum equations are too complex for analytical solution. Numerical solutions are possible but time consuming.

The processes discussed above – infiltration, filling and overflow from depression storages and overland flow – are those of most significance in the surface phase of urban rainfall-runoff modelling. Other processes, such as interception (by trees for example) and evaporation, have a negligible effect in the time scale of a storm event on urban areas, although evaporation is clearly important in determining how quickly a catchment dries out between events.

This brief account of the major processes is intended to show that a model based on the detailed reproduction of observed phenomena would be too complex for simulation purposes (ie to reproduce observed events on an existing catchment) let alone design (imaginary events on a future catchment). This is partly because the physical laws are complicated and require an unlikely knowledge of initial conditions but mainly because the individual microtopography of sub-catchments draining to inlets may be beyond description and is certainly beyond prediction.

Although it would be unrealistic for a model to be based explicitly on the individual physical processes, their effects can be represented in a simplified way consistent with the accuracy required by, and the data available to, design users. This is done by conceiving the various processes as equivalent storage elements (simple channels or reservoirs) each with prescribed capacities and/or storage versus outflow relationships. There may also be parameters defining the branching of input between different storages (eg some runoff from grassed areas passing on to impervious areas and vice versa). Clearly, this conceptual approach to modelling could, like the deterministic approach, become much more complicated than is warranted by the problem and some degree of simplification has to be accepted.

Whatever the level of conceptual representation which is attempted by the modeller, the main requirement is for data which can be used to calibrate the model, ie to determine the parameters which define the size and behaviour of the linked storage elements.

7.3 Available data

Several experimental investigations [25], [84], [93], have been conducted in the USA and Europe, but this discussion is restricted to work carried out in the UK.

Runoff data collected before, at, or very soon after entry to the pipe system come from three sources:

(i) measurements made with small weirs or flumes at the end of short pipe lengths. The Road Research Laboratory's data collection programme in the 1950s [95] included one such area (Oxhey Road) and other work has been done more recently on motorways [89] and at Southampton University [53];

(ii) measurements made in road gulleys by the Institute of Hydrology from 1976 to 1979 [60];

(iii) a programme of research conducted on the Imperial College laboratory catchment in 1977 [48].

A full account of these data and the analyses applied to them in the development of the new procedures appears elsewhere [54]. The data enable a direct comparison to be made between reality and the assumptions of the Lloyd-Davies and TRRL design methods, namely that there is 100 per cent runoff from impervious areas and that surface storage is adequately represented by a time of entry. Figure 7.1 gives two examples of recorded hydrographs, the rainfall that caused them and the hydrograph which would be predicted from both those assumptions. In each case, significantly less runoff is observed than predicted and a greater attenuation is produced. Many such hydrographs have been analysed [60] and two main conclusions were reached:

1. There is a wide variation in losses due to infiltration and depression storage. The assumption of 100 per cent runoff from impervious surfaces and zero runoff from pervious surfaces is a simplification leading, in general, to over-estimation of runoff for a given rainfall input. This is true even for large events comparable with the magnitude of a design storm.

2. Surface storage is inadequately modelled by a time of entry. In particular, the previously-used value of two minutes underestimates observed effects of attenuation.

There is, in short, more storage available above the ground than has been allowed for in past design practice.

Data on flow measurement further down sewer systems cannot be used for separate study of surface storage. However, they can be used to study catchment average values for losses. Most of the suitable data in this category were collected by TRRL [95] but three catchments have been gauged more recently as part of a DoE-sponsored programme begun in 1971 and continued, in part, under the supervision of the Hydraulics Research Station. Study of these data reinforced the first conclusion above and enabled runoff percentages to be related to catchment characteristics.

Having suggested that the concepts of the Rational/TRRL model of surface runoff are inadequate representations of actual observed events, the following sections consider how much more of the physical 'truth' can be accommodated without incurring penalties of time and cost associated with excessive catchment data requirements.

7.4 Prediction of percentage runoff

The following factors may be expected to affect the runoff volume (RUNVOL, in mm over the total catchment area) (also given is the notation used in the subsequent discussion):

rainfall depth (mm)	P
rainfall duration (hours)	D
total catchment area (ha)	AREAC
percentage of catchment area covered by impervious surfaces intended to drain to the storm sewer (percentage)	PIMP
antecedent wetness condition (mm)	UCWI
catchment slope (percentage)	SLOPEC
soil type (-)	SOIL

One other variable, for which data are not generally available, is the permeability of the nominally 'impervious' surface. Water can pass through asphalt and even concrete at rates which depend on the particle sizes of the constituent mix and on the presence of cracks. Therefore the infiltration rate is dependent not only on materials and methods of construction, but also on operating conditions such as maintenance and loading. There is evidence that this infiltration can have an important effect on percentage runoff values but the difficulties of quantifying it make it unsuitable for inclusion in a design procedure. The data used in estimating percentage runoff were obtained on a range of catchments which can be expected to include the average effect of this type of infiltration loss.

The soil index SOIL is based on the Flood Studies Report [69], and may be obtained from the revised soil map [46] or from the 1:1000 000 version covering the whole of the United Kingdom; this version is available in Volume 3 of this report. The index takes one of the values 0.15, 0.3, 0.4, 0.45 or 0.5 according to the five soil types 1 to 5 respectively. Where a catchment contains more than one soil type, an average value weighted by area should be used.

The urban catchment wetness index UCWI is a modified form of the catchment wetness index used in the Flood Studies Report. In the urban form, more weight was given to the short term antecedent conditions (rainfall over the preceding five days) as against the longer term antecedent conditions (soil moisture deficit). In design use UCWI takes a recommended value which varies with geographical location and is intended to ensure that the required return period of the peak discharge is achieved (see chapter 9). The method of calculating UCWI for the simulation of observed rainstorms is described in section 7.9.

The relationship between the variables listed above was studied in a multiple regression analysis. The aim of multiple regression is to find the best relationship between the variable to be predicted (runoff volume) and the other variables. A regression analysis is a statistical tool with clear rules for its use and it is necessary to be cautious in applying the resulting prediction equations. For example, the runoff volume must not be allowed to exceed 100 per cent of the rainfall on the total area.

Several different forms of equation and many combinations of variables were examined. A full account of the investigation, which used 510 observed events from 17 catchments, is given elsewhere [54] but the main conclusion was that runoff volume was best predicted in the following form from just five of the seven variables mentioned above:

$$\mathrm{RUNVOL} = P \times \mathrm{AREAC}(a_1.\mathrm{PIMP} + a_2.\mathrm{SOIL} + a_3.\mathrm{UCWI} + a_4) \qquad \text{......7.1}$$

where a_1, a_2, a_3 and a_4 are constants derived from the regression analysis.

The percentage runoff (PR) from the total catchment area (not from the impervious area alone) may be defined as:

$$\mathrm{PR} = \frac{\mathrm{RUNVOL}}{P \times \mathrm{AREAC}} \times 100 \qquad \text{......7.2}$$

The recommended prediction equation for PR is:

$$\mathrm{PR} = 0.829\,\mathrm{PIMP} + 25.0\,\mathrm{SOIL} + 0.078\,\mathrm{UCWI} - 20.7 \qquad \text{......7.3}$$

The coefficient of multiple correlation for equation 7.3 is 0.76: this means that 0.76^2 or 57 per cent of the variance of PR (ie the scatter about the mean value of the 510 observations) is explained by the equation. The standard error of estimate is 10.3: this means that two-thirds of the observations lie within ± 10.3 of the value predicted by the equation. The ranges of values included in the data set were: PIMP, 20 to 70; SOIL, 0.15 to 0.45; UCWI, 0 to 300.

Regression equations can give incorrect predictions when applied outside the range of data available for their calibration. For example, the minimum values of SOIL and UCWI are 0.15 and 0 respectively. With these values, and for values of PIMP less than 20 per cent, negative values of PR would be predicted. To avoid such occurrences, a limiting condition is used: if equation 7.3 predicts PR to be less than 40 per cent of PIMP then PR is made equal to 40 per cent of PIMP. No mater how dry a catchment might be therefore, the model is not allowed to calculate a total runoff less than 40 per cent of the rain falling on the impervious surfaces. In practice, this limiting condition is very unlikely to be invoked.

A further limitation in the use of regression equations arises from the possibility that the 'independent variables' may be, to some extent, correlated. For this reason it might be unwise to use equation 7.3 to predict the relative contributions made to the percentage runoff by the three variables on the right hand side.

It is recognised, however, that engineers will wish to predict within one catchment the effect on PR of changing the value of UCWI or, more practically, PIMP. This would be necessary, for example, in order to examine the effect on design pipe sizes of putting roof runoff into pipes (full allowance for roof areas included in PIMP evaluation), soakaways (no allowance), or onto grassed areas (some allowance). If equation 7.3 were to be used for this purpose, the changing values of PR could sometimes appear anomalous. Consider an extreme example of this apparent anomaly, a catchment on light soils (SOIL = 0.15) with a UCWI design value of 60. A change from 50 per cent to 25 per cent of impervious surface causes a reduction in predicted percentage runoff from 29 per cent to 10 per cent. Whether this seems reasonable or not depends on the type of surface and the way it is connected to the sewer system before and after the change in its extent. If all impervious surfaces really are directly connected to the sewer system then it must be considered unreasonable in that the expected identical reduction following a further change from 25 per cent to 0 per cent of impervious area cannot be achieved. However, in many practical situations, there are opportunities for water to pass from impervious surfaces onto surrounding pervious areas and these opportunities multiply as the proportion of impervious surfaces decreases. It is quite plausible therefore that a decrease in impervious area will sometimes cause a more than proportional decrease in runoff. It follows that the reliability of the result depends partly on details of design and construction.

The following alternative form of regression equation avoided this particular anomaly:

$$PR = a_5 PIMP + a_6(100 - PIMP).SOIL.UCWI \qquad ......7.4$$

An analysis of the same data described above for the derivation of equation 7.3 showed that the optimum values of a_5 and a_6 were 0.662 and 0.00219 respectively.

In equation 7.4 soil type and antecedent conditions affect only the contribution from unpaved parts of the catchment. This concept has been found useful in overseas application of the TRRL method [32]. However, neither equation 7.4 nor any of the other alternatives was as good a fit to the available data as equation 7.3 and it was decided to accept a possible anomaly in order to achieve the best statistical prediction. If, in a particular study such as that of roof drainage disconnection, a user considers the anomaly unacceptable, it is possible to use the simpler estimate of PR (average value 0.75 PIMP) as recommended for use with the Modified Rational Method (section 7.10).

Equation 7.3 can be compared with the previous assumption implicit in the Lloyd-Davies and TRRL methods (section 7.3) that PR is equal to PIMP. It is clear that such an equivalence is denied by the available data. Furthermore, neither the magnitude (P) nor the intensity (P/D) of the rainfall event were significant factors in the regression. Some of the events included in the regression were larger than design rainfalls but there was no trend for PR to be bigger in these events.

Having determined the volume of runoff arising from a given rainfall and from the catchment as a whole, the next step is to specify its distribution between the several sub-catchments and types of surface area.

7.5 Distribution of runoff volume within the catchment

It might be thought that equation 7.3 above could be used directly to determine runoff volumes from each separate sub-catchment. But the problem of extrapolating beyond the range of the data would then be even more marked. Some planned sub-catchments might be wholly pervious and some wholly impervious whereas none of the catchments contributing to the regression study was in either category.

The solution adopted within the procedure is to distribute the total runoff first between the three main surface types (paved surfaces, pitched roofs, pervious areas) and then combine as appropriate for each sub-catchment in turn.

If the runoff volume were considered to derive from impervious surfaces alone, the average value of percentage runoff would be about 70 per cent. The percentage runoff predicted by equation 7.3 is therefore compared with that which would result from 70 per cent runoff from the impervious surfaces. If it is less, the runoff is assumed to be confined to the impervious surfaces (paved ground surfaces and roofs):

$$PR_{pav} = PR_{roof} = \frac{PR \times 100}{PIMP} \qquad \text{......7.5}$$

$$PR_{perv} = 0 \qquad \text{......7.6}$$

where the subscripts $_{pav}$, $_{roof}$ and $_{perv}$ refer to the paved, roof and pervious areas respectively. If it is more, the excess is assumed to be generated equally on all surfaces:

$$PR_{pav} = PR_{roof} = 70 + (PR - \frac{70 \times PIMP}{100}) \qquad \text{......7.7}$$

$$PR_{perv} = PR - \frac{70 \times PIMP}{100} \qquad \text{......7.8}$$

This distribution of the total runoff volume between the three surface types is of only secondary importance. The peak discharge calculated by the surface runoff model is relatively insensitive to the selection of 70 per cent as the value above which some runoff from pervious surfaces is presumed to occur.

It is necessary also to determine the distribution of this volume with time in order to calculate the runoff hydrograph. This distribution is calculated in two stages: first by determining the net rainfall profile and second by calculating the attenuating effects of surface storage.

7.6 Calculation of the net rainfall profile

The value determined for PR will always indicate that some of the rain falling on the catchment will not enter the storm drainage system. These rainfall 'losses' are assumed to occur in two ways: first an initial loss to depression storage and then a continuing loss by infiltration during the storm.

A regression study [26] of data from sub-catchments containing a mixture of paved and pervious areas showed that:

$$DEPSTOG_{perv} = DEPSTOG_{pav} = 0.71 \times SLOPE^{-0.48} \qquad \text{......7.9}$$

where DEPSTOG is the average depth of depression storage in mm and SLOPE is the average overland slope of the sub-catchment (per cent).

To avoid the need to measure the overland slope of every sub-catchment, it is necessary only to define SLOPE in one of three broad categories. The computer programs require only the slope index defined as follows:

Description	Index	Range	Value used
Mild	1	Less than 2 per cent	1.25 per cent
Medium	2	Between 2 and 3.5 per cent	2.75 per cent
Steep	3	Greater than 3.5 per cent	4 per cent

For pitched roofs, a value of $DEPSTOG_{roof}$=0.4mm is recommended.

These are hardly significant quantities in the context of a design storm but, as will be shown in section 7.8, they require no extra catchment data or work by the user and so justify inclusion in the computer model. The user has the option to specify that the depression storage is already filled at the start of a storm.

After subtracting depression storage from the beginning of a design storm, the remaining loss is assumed to be distributed proportionally throughout the storm. In the model, this is represented in the form of a reduced contributing area:

$$AR_{pav} = \frac{PR_{pav}}{100} \times \frac{P}{P\text{-}DEPSTOG_{pav}} \times AREA_{pav} \qquad ......7.10$$

where AR_{pav} and $AREA_{pav}$ are the contributing and actual areas respectively of paved ground surface within a sub-catchment.

Similar relationships hold for AR_{perv} and AR_{roof}.

Large areas of flat roof should be treated as a paved ground surface.

7.7 Prediction of surface storage

The theoretical equations of flow are too complex for application to a design problem, but they can be simplified to produce a conceptual alternative. The 'kinematic wave' is one such approximation. A kinematic wave passes downstream with change of shape but without attenuation. It has been shown[83] that overland flow resulting from uniform rainfall and concentrating to an inlet point is particularly well modelled by the kinematic wave equation, which reduces to an equation relating velocity (V) and depth (h), such as the familiar Chézy formula applicable to wide, shallow flow:

$$V \alpha h^{1/2} \qquad ......7.11$$

Together with the continuity equation, which states that in any reach or section of the flow path, input minus output equals change of storage, an equation of the form of 7.11 provides a method of flood wave routing. But it may still be too complicated for a design method. In the Rational and TRRL methods, the time of entry concept was used to represent the surface flow routing. It can be demonstrated [54] that this is equivalent to the use of a set of linear channels in parallel and that such a model requires that, contrary to all empirical findings represented in formulas such as 7.11, velocity should be assumed to be independent of depth. A better concept is that of the non-linear reservoir, a development of 7.11 which preserves the dependence of velocity on depth.

If the flow (q) is assumed to be taking place in a wide rectangular channel of constant width and gradient, both the cross sectional area and storage volume (S) vary directly with depth. Hence

$q \alpha h^{3/2}$ and $S \alpha h$

Therefore $q \alpha S^{3/2}$

or $S = k_r q^{2/3}$ 7.12

Equation 7.12 is an equation of a non-linear reservoir. There is still the equation of continuity to be considered, but, as the model is now reduced to a reservoir representation, the problem is simply that of reservoir routing, which is fast and easy for numerical solution by computer.

The storage constant, k_r, controls the degree of attenuation of the hydrograph and, from the description of processes in section 7.2, it might be expected to be related mainly to area and slope. The best relationship obtained from a multiple regression study [54] on 28 sub-catchments (in England, Sweden and the Netherlands) comprising paved and pervious ground surface was:

$$k_r = 0.051 \times SLOPE^{-0.23} PAPG^{0.23} \qquad ......7.13$$

PAPG is the paved area ($AREA_{pav}$) divided by the number of gulleys. The multiple correlation coefficient of equation 7.13 is 0.67 and the standard error of estimate is −24 per cent to + 32 per cent.

The value of PAPG is calculated within the program if the number of gulleys per sub-catchment (NGULLS) is specified. Alternatively the paved area per gulley may be defined as one of three size ranges by specifying the appropriate index in the program data:

Description	Index	Range	Value used
Small	−1	Less than 200m^2	125m^2
Medium	−2	Between 200 and 400m^2	300m^2
Large	−3	Greater than 400m^2	600m^2

If no value is specified for either PAPG or NGULLS, the program will use the largest standard area per gulley consistent with the size of the paved area. If the distribution of gulleys within a sub-catchment is irregular, a value of PAPG appropriate to the major flow contributing area should be specified rather than NGULLS.

It can be argued that the runoff from pervious areas is slower than from paved areas and should therefore be modelled separately. However, the k_r value predicted from equation 7.13 is based on data collected on sub-catchments in which the effects of the two types of surface could not be separated. A single reservoir model therefore seemed adequate. The k_r value should be applied to the effective contributing area (AR_{pav} + AR_{perv}) of the two surfaces together. For pitched roofs, however, a smaller storage constant would seem appropriate. Few data are available but a k_r value of 0.04 is recommended.

Although the reservoir routing algorithm is fast and simple, many sub-catchments may be involved in a design exercise. The procedure is therefore simplified by calculating initially the runoff hydrographs for a standard set of nine sub-catchments, represented by three values each of SLOPE and PAPG (see Figure 7.2). On a real catchment, every sub-catchment is represented by one of these nine, thus reducing the amount of subsequent computation and removing the need for the slope and paved area per gulley to be specified exactly for each sub-catchment.

In the next section, the various steps in the surface runoff model are put together to illustrate the sequence of calculations in the computer program.

7.8 Applying the surface runoff model

Figure 7.3 summarises the several steps which are followed in the application of the model to a rainfall event.

1. The rainfall hyetograph. The choice of an event for use in design is discussed in sections 6.2 and 6.3 and the basic information provided in sections 6.4 to 6.7.

2. For each of nine standard sub-catchments (Figure 7.2), depression storage (if available) is deducted from the beginning of the rainfall, and the remaining rainfall is routed through a non-linear reservoir. The process is repeated for pitched roofs. The resulting set of 10 standard hydrographs (with ordinates of mm/hr) is stored in the computer.

3. A catchment-wide estimate of percentage runoff (PR) is calculated from equation 7.3. The UCWI value to be used in the design application of equation 7.3 is obtained from Figures 9.7 and 9.8; the value to be used in simulating an observed rainstorm is calculated from rainfall and soil moisture deficit data as described in section 7.9. Values of percentage runoff applying to pervious, paved and roof areas are calculated from PR and the rules referred to in section 7.5.

4. The inlet hydrograph for a given sub-catchment is calculated by the following steps:

(a) Effective areas of each surface type are calculated by multiplying the actual areas by the appropriate percentage runoff value with allowance for depression storage.

(b) The appropriate standard hydrograph for paved/pervious areas is taken from Step 2 and converted to ordinates of discharge by multiplying by the sum of the contributing areas of pervious and paved surfaces.

(c) The roof hydrograph (if required) is taken from Step 2 and multiplied by the contributing area of pitched roofs in the sub-catchment. (Large flat roofs are treated as paved ground surface.)

(d) The hydrographs calculated in (b) and (c) are added together.

Most of this procedure is entirely automatic within the programs, the user being involved only to the extent of determining a soil index and catchment wetness index for the catchment as a whole, stating the percentage of each sub-catchment occupied by roofs and paved surfaces, and specifying one of the standard types of slope and paved area per gulley. Precise instructions for the application of the procedure in the design of new systems or the analysis of existing ones are given in the program user's guide (Volume 2 of this report).

The above modelling procedure also underlies the simplified method for calculating the runoff hydrograph from a sewered sub-area (see chapter 10). For that application, however, the data requirements are considerably less.

7.9 Calculation of Urban Catchment Wetness Index (UCWI)

The Urban Catchment Wetness Index (UCWI), required for the percentage runoff equation 7.3, is calculated in one of two ways depending on whether recorded or design rainfall is being used. For a design rainfall event, or in the absence of the information required to calculate UCWI, the recommended value of UCWI is read from the relationship with standard average annual rainfall described in chapter 9.

For the simulation of observed rainstorms

$$UCWI = 125 + 8API5 - SMD \qquad \ldots .7.14$$

where API5 is the five day antecedent precipitation index and SMD is the soil moisture deficit.

API5 is calculated using the following procedure. First determine the rainfall depths (in mm) for the five days prior to the event being used in the simulation. The API5 value at 0900 on the day of the event is then defined by:

$$API5_9 = \Sigma P_{-n}\, C_p^{\,n-0.5} \text{ for } n = 1 \text{ to } 5 \qquad \ldots .7.15$$

P_{-n} is the rainfall on the n^{th} day before the event and $C_p = 0.5$. Finally the API5 at the time of the event is given by:

$$API5 = API5_9 C_p^{\,(t'-9)/24} + P_{t'-9}\, C_p^{\,(t'-9)/48} \qquad \ldots .7.16$$

where t' is the time (in hours) of the beginning of the event and $P_{t'-9}$ is the rainfall depth between time t' and 0900. An event starting between 0000 and 0900 hours has to be regarded as starting at an equivalent time related to the previous calendar day.

The soil moisture deficit SMD is calculated from the equation

$$SMD = SMD_9 - P_{t'-9} \qquad \ldots .7.17$$

where SMD_9 is the soil moisture deficit at 0900 on the day of the event. SMD_9 can be calculated as a weighted average of the values at the nearest SMD stations. These values can be obtained for the relevant day from the Meteorological Office.

7.10 Surface runoff assumptions for use with the Modified Rational Method

The previous sections of this chapter have described a method of modelling the surface runoff process in a physically realistic way which takes account of observations of the surface runoff process. In some situations, however, a simpler calculation method is required. The Modified Rational Method is included in the Wallingford procedure for this purpose. It may be applied either manually or by use of a computer program.

The method gives the peak discharge Q_p from the equation:

$$Q_p = CiA \qquad \ldots .7.18$$

where C is coefficient
i is the average rainfall intensity during the time of concentration
and A is the catchment area.

The main values which the user has to select in applying the Modified Rational Method are the coefficient C and the time of entry, t_e.

Under the strict assumptions of uniform rainfall intensity and a linear time-area diagram, the coefficient can be shown to represent only the reduction in total volume between rainfall and runoff. For a variable rainfall intensity and a non-linear time-area diagram, the Rational coefficient must be considered to include the effects of these factors, which will tend towards an increase in discharge. Thus the Rational coefficient can be written:

$$C = C_v C_R \qquad \ldots .7.19$$

where C_v is the volumetric runoff coefficient and C_R is a routing coefficient.

The volumetric runoff coefficient C_v is defined as the proportion of the rainfall on the catchment which appears as surface runoff in the storm drainage system.

The recommended value is affected by whether the whole catchment (impervious and pervious areas) is considered, or the impervious areas alone. If the whole catchment is considered, as in the computer version of the method (see Section 13.1.2) then:

$$C_v = \frac{PR}{100} \qquad \ldots .7.20$$

where PR is given by equation 7.3.

In this case, the areas used in the calculation must be the total catchment area (impervious and pervious) upstream of the point under consideration.

If impervious areas alone are considered, as may be preferred in a hand calculation (see section 13.1.1 and Volume 4) then:

$$C_v = \frac{PR}{PIMP} \qquad \ldots .7.21$$

where PR is given by equation 7.3

and PIMP is the percentage of the catchment covered by impervious surfaces intended to drain to the storm sewer.

The data used in the development of the percentage runoff equation suggest that the overall average value of C_v (when defined by equation 7.21) is about 0.75, ranging from about 0.6 on catchments with pervious soils to about 0.9 on catchments with heavy soils.

The value of the routing coefficient C_R should, theoretically, vary with the shape of the time-area diagram and the peakedness of the rainfall profile. Using typical time-area diagrams and a range of rainfall profiles of varying peakedness, values of C_R between 1 and 2 were obtained. However the relationship with peakedness was not substantiated by observed rainfall-runoff data. For the present, a value for C_R of 1.3 is recommended for simulation and design. This value is included in the computer programs.

The time of concentration t_c is defined by:

$$t_c = t_e + t_f \qquad \ldots .7.22$$

where t_e is the time of entry

and t_f is the time of flow through the pipe system to the point under consideration, based on the pipe-full velocity which is a good approximation over a wide range of proportional depths.

The time of entry may therefore be regarded as representing the time of flow over the ground surface. It also has the effect of reducing the calculated discharge, since an increase in the time of entry and, consequently, in the time of concentration reduces the corresponding rainfall intensity. Recent research [52] using data from several catchments in England, Sweden and the Netherlands gave the following equation for the time of entry:

$$t_e = 7.44\ \mathrm{LENGTH}^{0.133}\mathrm{SLOPE}^{-0.274} \qquad \ldots .7.23$$

where LENGTH is the sub-catchment overland flow length (m) and SLOPE is the sub-catchment slope (per cent).

The correlation coefficient of equation 7.23 is 0.64. The equation gives typical values of t_e in the range 8 to 12 minutes.

However, the data set used in the exercise was biased towards small events equivalent to a return period of a few weeks or months. Equation 7.23 would not be appropriate for typical design return periods of a few years. The attenuation produced by the non-linear reservoir model described in sections 7.7 and 7.8 and by the simpler time of entry were therefore compared. This analysis led to the following optimum values for the time of entry:

Design return period	Time of entry (minutes)
5 years	3-6
2 years	4-7
1 year	4-8
1 month	5-10

In all cases, the larger values are applicable to larger, flatter sub-catchments as shown in Figure 7.2, and vice versa.

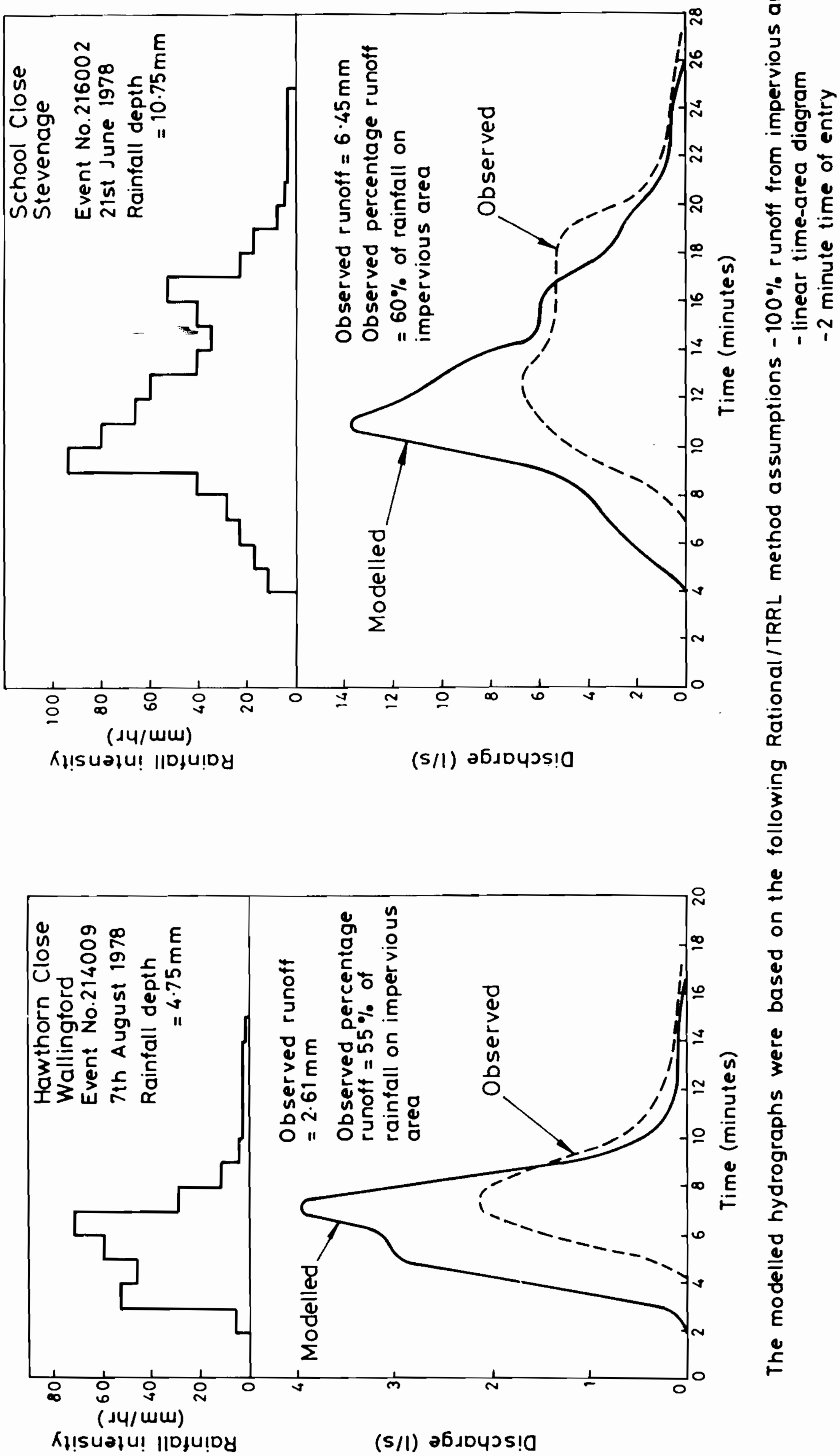

Figure 7.1. Comparisons of observed and modelled surface runoff hydrographs

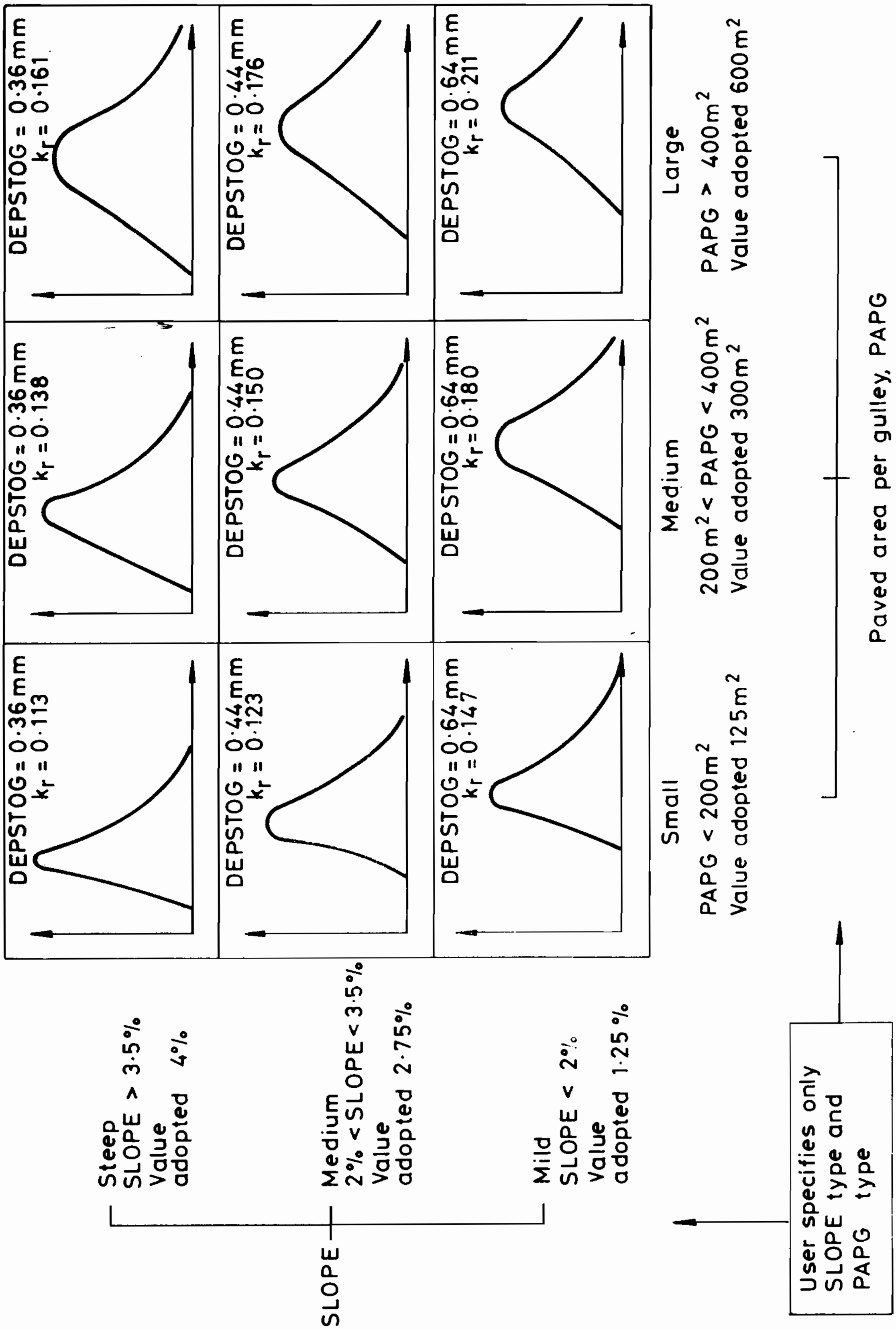

Figure 7.2. 3 × 3 matrix of standard runoff hydrographs

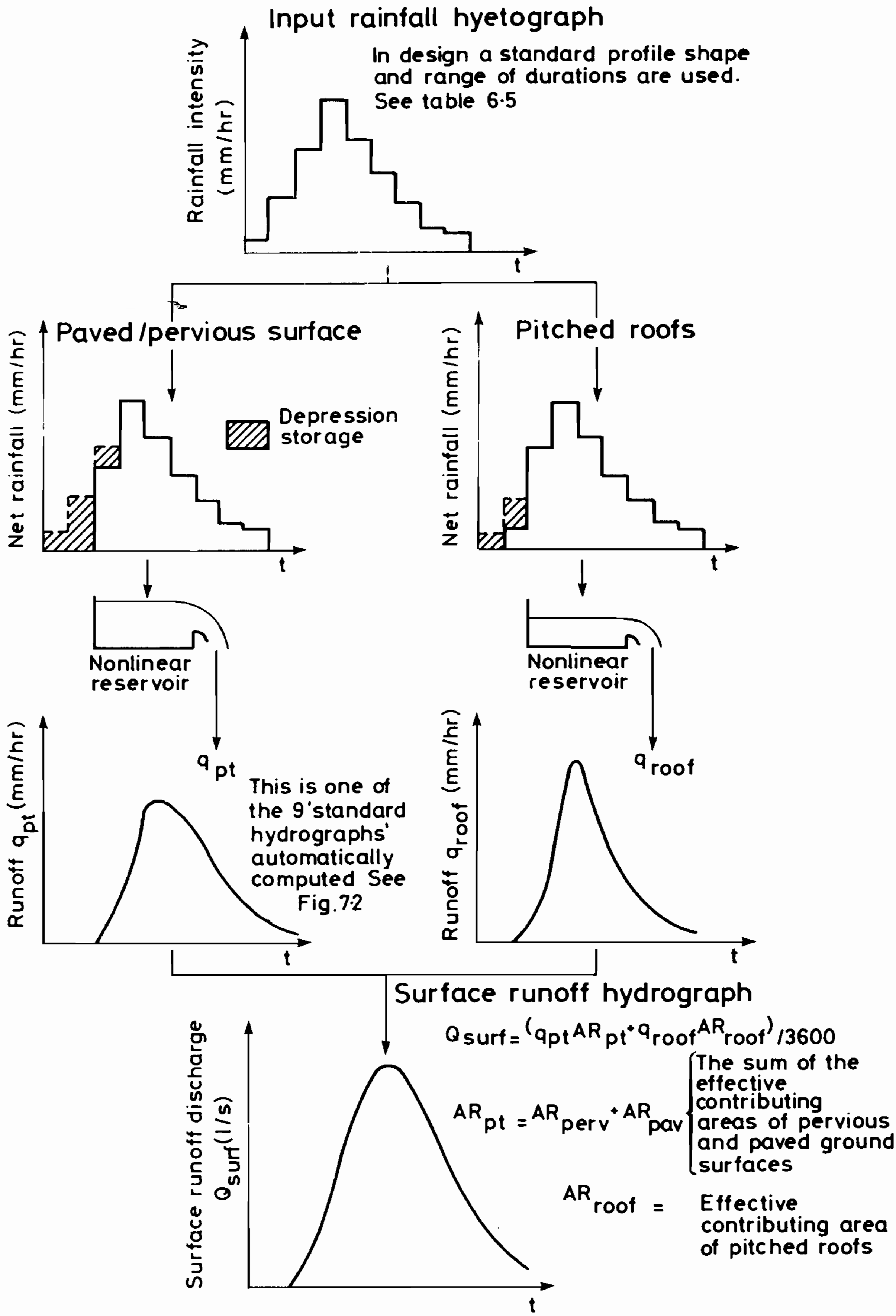

Figure 7.3. Conversion of rainfall into runoff

CHAPTER 8

Pipe flow

8.1 Design criteria

IN THE past storm sewers have normally been designed to convey without surcharge the flows generated by rainstorms of a selected frequency. It has been assumed that flows and rainfall intensities will display the same frequency function (or 'growth curve').

Two improvements to this design philosophy are now possible. First, designs may be related to the conditions with which the engineer is really concerned, the *flows* rather than the rainfall of a selected return period. A research exercise has been completed, as a result of which the engineer may now select appropriate values of design variables to design against runoff frequency rather than rainfall frequency. The results of this research exercise are given in chapter 9.

Second, it is now possible to calculate the performance of storm drainage systems under surcharged flow conditions as well as free surface flow conditions. When storm sewers are designed to convey without surcharge discharges up to and including those of the selected frequency, it is implicitly accepted that under more severe conditions some degree of 'failure' will occur. This feature is common to many other aspects of engineering design especially those providing protection against natural phenomena (flood levels, wind loadings, spillway capacities, etc), but it is not always realised that a certain frequency of failure of storm sewers is inevitable, and is implicitly accepted in the design calculations. 'Failure' in the case of storm sewers means that the pipe system in the ground is unable to convey all the discharge produced by particularly intense rainfall, with the possible consequence of surface flooding. Failure does not necessarily imply structural failure; the pipe system may continue to perform its normal function after the severe event has passed.

Surcharging may be regarded as undesirable in certain circumstances, for example if there is a danger of flooding in basements or if the structural condition of the sewer pipes is not sufficiently sound. However, it is not usually possible to eliminate surcharging as it will occur whenever runoff is greater than that used in designing the pipe system. Programs to calculate the behaviour of a system under surcharge have therefore been included in the procedure for two main reasons:

(a) To study the behaviour of existing systems in order to assess whether a limited degree of surcharging would provide sufficient extra capacity to defer or eliminate the need for new sewers.

(b) To study the performance of existing or proposed systems under more severe rainfall conditions than those assumed for design.

It is recommended that the basic criterion for the design of new storm sewer systems should continue to be the occurrence of pipe-full flow of a specified frequency. However, when existing or proposed systems are analysed they should be examined by simulation, including the possibility of surcharged flow, to assess their hydraulic performance under conditions more extreme than those used in the design.

8.2 Flow equations

Two equations are in general use to calculate steady flow in pipes and open channels:

(a) The Colebrook-White equation

$$V = -\sqrt{32gRs}\,\log_{10}\left[\frac{k_s}{14\,800\,R} + \frac{1.255\nu}{R\sqrt{32gRs}}\right] \qquad \ldots.8.1$$

where k_s = equivalent sand roughness (mm)
R = hydraulic radius (m)
V = velocity (m/s)
s = hydraulic gradient
g = acceleration due to gravity (m/s^2)
ν = kinematic viscosity (m^2/s)

This equation has a very sound theoretical basis, being founded on the work of Prandtl and von Karman on turbulence, and it fits the data that have been obtained from experiments on a large variety of commercial pipes, tested over a wide range of conditions. The equivalent sand roughness k_s is not the actual size of roughness of the boundary, but is the diameter of sand that, if spread uniformly over the boundary, would give the same resistance as the actual roughness when the flow is rough turbulent. The equivalent sand roughness corresponds very approximately to the d_{90} size of the actual roughness, ie 90 per cent of the roughness projections are smaller than this size. Although the Colebrook-White equation is not easy to solve directly design charts [44] and tables [43] are available, showing the relationships between the major variables.

(b) The Manning velocity equation

$$V = \frac{1}{n}R^{2/3}s^{1/2} \qquad \ldots.8.2$$

The Crimp and Bruges equation, $V = 83.45\ R^{2/3}s^{1/2}$ is a particular case of the Manning equation with $n = 0.012$.

The Manning equation does have limitations and these outweigh the advantages of simplicity[1,45]. It is a good approximation to the Colebrook-White equation in the rough-turbulent region where $7<R/k_s<130$ and where $Vk_s>0.001m^2/s$. (In these inequalities, k_s is expressed in metres.)

The limits of application of the Manning equation increase as the pipe roughness increases. For example, for combined sewers with a k_s value of 1.5mm, the equation could be applied to diameters up to 800mm if the velocity exceeded 0.8m/s; or for combined sewers with a k_s value of 3.0mm, the equation is applicable to diameters up to 1500mm if the velocity exceeds 0.3m/s. However, for separate storm sewers in which the k_s value is normally 0.6mm or less, the Manning equation should not be used since it over-estimates the full capacity of the pipe.

It is recommended that the Colebrook-White equation should be used to determine the hydraulic behaviour of storm sewers, as it applies over the whole range of turbulent flow – (smooth, transitional and rough turbulent) – for any pipe size and surface. It is this equation which is the basis of the recommended procedure for the design and analysis of storm sewers.

8.3 Routing of free surface flows in sewers

Time dependent flow in a sewer can be represented by a discharge hydrograph. A comparison of hydrographs for the same event at successive locations along the sewer reveals that:

(i) the hydrograph progresses along the sewer at a speed which is usually greater than the speed of the storm water;
(ii) the shape of the hydrograph is modified; and
(iii) the peak discharge will reduce downstream provided there is no lateral inflow.

Variations of the wave speed with discharge modify the shape of the discharge hydrograph as does any lateral inflow. The attenuation for a particular event depends on the storage which is available in the sewer and on the shape of the hydrograph. Between junctions the attenuation of the peak discharge is most noticeable for pipes with a small slope and for sharply peaked hydrographs.

A number of models could be used to compute the propagation of a discharge hydrograph along a storm water sewer, such as the time off-set model which simply translates the hydrograph without any change in shape, or a storage routing model such as the Muskingum method which includes an allowance for a change in hydrograph shape but ignores backwater effects, or a model based on a numerical solution of the equations for gradually varying flow in open channels. The last type of model includes backwater effects at the expense of greater computing time. Because most sewers in Britain are comparatively steep, backwater effects are not usually important; therefore the version of the Muskingum method proposed by Cunge [21] is adopted in the procedure. This method gives an accuracy for peak flows of within 2 per cent when simulating a typical discharge hydrograph along a pipe 240m long, 1m in diameter at a gradient of 1/200 [8]. For smaller pipe lengths and diameters and steeper gradients the accuracy improves. The method is also computationally as fast as the time off-set method. The method cannot solve the flow conditions along pipes with negative gradient.

The basic equation for the method is the storage equation:

$$\frac{dS}{dt} = Q_{in} - Q_{out} \qquad \ldots .8.3$$

where S is the storage in the pipe with inflow discharge Q_{in} and outflow discharge Q_{out} and t is time. The Muskingum method relates the storage to a linear combination of Q_{in} and Q_{out}:

$$S = K_F[\varepsilon Q_{in} + (1 - \varepsilon)Q_{out}] \qquad \ldots .8.4$$

where K_F and ε are the storage and proportionality coefficients respectively. Cunge has shown that by a judicious choice of K_F and ε in terms of the geometric characteristics of the pipe the two equations can be solved in finite difference form to give an accurate description of the translation and attenuation of the discharge hydrograph. In the procedure

$$K_F = \frac{L}{\omega} \qquad \ldots .8.5$$

and

$$\varepsilon = \tfrac{1}{2}\left[1 - \frac{Q_n}{BL\omega s_p}\right] \qquad \ldots .8.6$$

where Q_n, B and ω are the discharge at normal depth, surface width and kinematic wave speed along a pipe of length L and gradient s_p at a fixed proportional depth h/d, where d is the diameter of the pipe. Q_n is defined by the velocity equation 8.1 and ω is given by:

$$\omega = \frac{1}{B}\frac{dQ}{dh} \qquad \ldots .8.7$$

The finite difference scheme for equations 8.3 and 8.4 gives Q_{out} explicity in terms of Q_{in}.

8.4 Flow routing in surcharged pipes

A sewer flowing under pressure is said to be surcharged and this occurs when the incoming flow is greater than the just-full pipe capacity or if a downstream tailwater level imposes a backwater effect. When the hydraulic gradient exceeds the pipe gradient the discharge under surcharge is greater than the just-full capacity; this increase in capacity can be considerable especially for pipes at low gradients. Figure 8.1 shows the maximum increased carrying capacity of a sewer as a result of surcharging without flooding for a 100 metre length of sewer with a free outfall discharge and a surcharge head of one metre in the upstream manhole. This figure applies only to a 100m length of pipe-line; over a larger catchment area the effect of surcharging on capacity will be less than that indicated in the figure.

Raising the tailwater levels can cause a sewer to run full when it would otherwise run only part full. Under such circumstances the hydraulic gradient will be flatter than the pipe gradient and the discharge under surcharge will be less than the just-full capacity.

The head loss, Δh, along a surcharged pipe has two components; the loss due to friction in the pipe and the losses at the upstream and downstream manholes. Assuming that the manhole losses are proportional to the velocity head in the pipe:

$$\Delta h = \left(\frac{L\lambda}{d} + k_m\right)\frac{V^2}{2g} + \frac{1}{g}\frac{dV}{dt} \qquad \ldots .8.8$$

where V is the velocity of flow in the pipe with length L and diameter d, k_m is the head loss coefficient for manholes and λ is the Darcy-Weisbach friction coefficient ($= 8gRs/V^2$). Recommended values for k_m are given in section 8.5 below. Time dependent storage, S, in a manhole, including any water stored on the surface above the manhole, is described by the equation:

$$\frac{dS}{dt} = Q_{in} - Q_{out} \qquad \ldots .8.9$$

where Q_{in} is the total discharge into the manhole both from the pipes upstream and from the direct surface runoff from that sub-catchment. Because S is a function of the water level, h, in the manhole and Δh is the difference between the levels in the upstream and downstream manholes for a pipe, equations 8.8 and 8.9 describe completely the time dependent flow in a surcharged group of pipes. The models do not simulate the transfer of water over the ground surface from one sub-catchment to another. The user is given the option either to store the water on the sub-catchment associated with each manhole or to remove the excess water from the system completely.

The transition from free surface to surcharged flow in a pipe depends on the water level in the downstream manhole; see Figure 8.2. The following rules are used in the programs to change from free surface to surcharged flow.

If the downstream level is less than or equal to the downstream soffit level the pipe becomes surcharged when the average discharge along the pipe is greater than the just-full capacity at the pipe gradient (Fig 8.2a). Alternatively if the downstream water level is greater than the downstream soffit level surcharging occurs when the head difference between the upstream soffit level and the level in the downstream manhole is insufficient to convey the discharge through the pipe (Figure 8.2b).

The reverse transition from pressurised to free-surface flow depends on the level in the upstream manhole. If this level is less than the upstream soffit level free surface flow is resumed (Fig 8.2c). Alternatively, if the downstream manhole is not surcharged free surface flow is resumed when the water level in the upstream manhole is less than that level required to convey the just-full discharge through the pipe, assuming that the water level in the downstream manhole is at soffit level (Fig 8.2d).

8.5 Localised head losses

Local turbulence at manholes, bends and pipe junctions causes a loss of head which increases the hydraulic head upstream. As the discharge in the pipe approaches or exceeds its pipe-full capacity, local head losses become more important. These losses can be calculated from the equation:

$$\text{Head loss} = \frac{k_m V^2}{2g} \qquad \ldots .8.10$$

The head loss coefficients (k_m) for full bore flow through manholes are as follows [3,4]:

Description	k_m	Index
manhole - straight through	0.15	1
manhole - with 30° bend	0.50	2
manhole - with 60° bend	0.90	3

The above values are for bends incorporated within the manhole. Only the index value need be specified in the program data.

If a manhole incorporates a junction losses can be higher and depend on the relative magnitude of the flows in the branches and the geometry of the junction. Data are available that can be used as guidance in estimating head losses at a junction manhole [41]. The values of k_m above should be increased to allow for these losses. Index values of 4(k_m = 1.0) and 5(k_m = 1.5) are available for this purpose.

For convenience in operating the computer programs, losses at a bend without a manhole can be adequately represented by the addition of a 'notional' manhole.

8.6 Roughness values of sewers

Pipe roughness values (k_s) for different materials, based on recommendations by the Hydraulics Research Station, are given in Table 8.1. The range of k_s values reflects the permitted tolerances in manufacture as laid down in British Standards and also the quality of the pipe-laying. These data have been obtained from a number of experiments on pipes commonly used in drainage work and take into account various factors influencing roughness, especially the following four conditions:

(a) *Sliming* – In designing systems which contain foul sewage the effect of sliming on roughness must be taken into account because the roughness value of a new pipe changes soon after sewage starts to flow. The extent to which it is increased by sliming depends on the relation between the sewage discharge and the pipe-full capacity. Sliming will occur over the whole of the perimeter below the water level that corresponds to the maximum daily flow. The slime growth will be heaviest in the region of the maximum water level. Over the lower part of the perimeter, the surface will still be slimed, but to a lesser extent than at the waterline; above the maximum waterline the pipe surface will be fairly clean. Although in foul sewers sliming will have a significant effect on roughness [76], in a combined system the maximum daily flow of foul sewage will not usually be a significant part of pipe capacity and will probably not exceed 20 per cent full flow (corresponding to a maximum proportional depth of 0.3). However, values of k_s appropriate to heavily slimed sewers have been included in Table 8.1 for application where the foul sewage half-fills the pipe.

(b) *Ageing* – Very little information is available on the effect of ageing on the surface roughness of stormwater sewers. Experiments carried out on a stormwater overflow pipe gave roughness values that were of the same order as the new pipe value [2].

(c) *Sediment* – When sediment is present on the invert of the sewer the roughness increases quite significantly, but it is difficult to relate the roughness to the nature and time-history of the sediment deposits. It is very probable that most stormwater sewers contain some sediment deposits, even if only held in temporary storage in the pipe whilst passing through the sewerage system: hence some account of this should be taken. The only data available suggest that the roughness value can range from 30 to 300mm, depending on the configuration of the deposit and on the flow conditions. The higher roughness value is more appropriate when the sewer is flowing part full and the Froude number is 0.3 – 0.5, when considerable energy is lost as a result of the generation of surface disturbances (although in such cases, the use of an equivalent roughness value cannot really be justified).

The selection of an appropriate value of k_s in sewers affected by sediment deposits is difficult but has an important effect on the calculated flow capacity. The value should be based wherever possible on inspection of pipe lengths. The hydraulic effects of various values of k_s can be tested by the programs.

Effective maintenance can prevent sediment deposits from having any permanent effect on the capacity of the storm water system. Measures can also be taken, especially on construction sites, to reduce the entry of sediment to the sewer system.

(d) *Eccentricity* – Any eccentricity at joints will introduce an energy loss in addition to that produced by the roughness of the pipe surface. The use of O-ring joints has made it easier to obtain good alignment and when this system is employed, it should not be necessary to make any additional allowance for losses due to eccentricity. The use of sleeved joints for clayware pipes does permit some misalignment; British Standards BS65 and BS540 [12] specify 6mm as the permitted eccentricity. The range of values recommended in Table 8.1 for clayware pipes with sleeved joints is for eccentricities of between 2 and 8mm respectively. For existing clayware pipes with eccentricities of more than 8mm, the roughness value can be calculated from [99]:

$$k'_s = k_s + \frac{1.1e^2}{Lj} \qquad \ldots .8.11$$

where k_s = roughness value without joint eccentricity (approx 0.025mm for 900mm pipe lengths);
e = joint eccentricity (mm);
Lj = pipe length between joints (mm).

8.7 Recommended velocities in sewers

It has been customary to impose maximum and minimum design velocities in sewers. The maximum velocity was 3.66m/s (12.5ft/s) because it was thought that serious erosion of the sewer would be produced by the abrasive action of sediment. The minimum velocity was 0.76m/s (2.5ft/s) and the aim was to prevent the deposition of solids.

Recent research has shown that the fear of severe erosion is unjustified and this has led to the removal of the limit on maximum velocity [94]. This means that in areas of steeply sloping ground, some economies in the construction cost of sewerage schemes might be possible. Higher velocities allow smaller pipes to be used; sewers can be laid at the prevailing ground slope rather than the slope dictated by the need to restrict the velocity, thus eliminating the need for expensive backdrop manholes. However, higher velocities lead to problems not normally encountered in the design of sewerage schemes and these factors must be taken into account [42].

Energy losses at bends and junctions are more significant. Standing waves can form when sewers are running part-full and bulking of flow will occur as a result of air entrainment. The surface of the sewer could be affected by cavitation erosion or by corrosion from the release of hydrogen sulphide. Energy dissipation measures will need to be considered, particularly where sewers are discharging into a water course. Thought needs to be given to safety provisions. These phenomena do not all assume importance at a common threshold velocity: some become apparent at lower velocities than others. As a general guide, they should be considered whenever the flow in the sewer flowing half-full is supercritical. This occurs when the velocity is greater than $0.63\sqrt{gd}$ where g is the acceleration due to gravity and d is the diameter. In metric units and with d in metres this may be closely approximated by $2\sqrt{d}$.

The question of the minimum velocity required to prevent deposition of the solids has not yet been satisfactorily answered. Although sewers have been designed on the basis of a minimum velocity of 0.76m/s occurring each day, this criterion has not been invariably successful in preventing permanent deposits. However, it is not yet possible to recommend any alternative design criterion.

Sediment increases the energy loss in the system and, if permanent, reduces the effective cross-sectional area of the sewer. In consequence the hydraulic gradient is increased. For a pipe designed to flow full at the design flow, any increase in the hydraulic gradient will cause the pipe to be surcharged. Flooding may occur when the manholes are not sufficiently deep for the necessary hydraulic gradient to be generated and under such circumstances the sewer may eventually become blocked with sediment because it can never develop the sediment transporting capacity to match the amount of sediment brought into the system.

In a separate stormwater system there may be periods when there is no flow at all. Sediment deposits will dry out and have a chance to consolidate. In a combined system there will always be some flow in the sewer which should be designed so that the self-cleansing velocity occurs for a significant period every day. The velocity in an egg-shaped sewer varies very little over a wide range of depth, which helps to keep the sediment in motion for as long a period as possible.

8.8 Infiltration and inflow

Infiltration is ground water entering a sewerage system through pipe joints, broken pipes and manholes. Inflow is here defined as surface runoff from areas not originally intended to drain to the system and has a more rapid response to rainfall than infiltration.

Inflow and infiltration have a number of effects on combined systems. The risk of flooding is increased, storm water overflows operate more frequently, and the capital and running costs of pumping stations and treatment works are greater. In both old and new combined systems infiltration and inflow may be many times the normal sewage flow.

The effect of infiltration and inflow on separate storm water systems is less obvious. Although the rate of infiltration and inflow does not significantly affect the peak flows the total volume of infiltration may have to be taken into account when designing storage ponds.

When existing systems are to be connected to a new system under design the magnitude of infiltration should be assessed, by inspection or by measurement, and allowed for in the design as a base flow; after assessing inflow it may be necessary to increase the assumed impermeable area.

Table 8.1 Roughness values based on recommendations by the Hydraulics Research Station

Classification (assumed clean and new unless otherwise stated)	Values of k_s (mm) Suggested Design value	Likely Range		
SEPARATE SURFACE WATER SEWERS				
Asbestos cement	0.03	0.015 - 0.03		
Concrete:				
Precast concrete pipes with 'O' ring joints	0.15	0.06 - 0.6		
Clayware (Glazed or unglazed pipes):				
with sleeve joints and 'O' ring seals	0.06	0.03 - 0.15		
with spigot and socket joints and 'O' ring seals	0.06	-		
Pitch fibre	0.03	-		
Glass fibre	0.06	-		
UPVC:				
with chemically cemented joints	0.03	-		
with spigot and socket joints, 'O' ring seals at 6 to 9m intervals	0.06	-		
Brickwork:				
glazed	1.5	-		
well pointed	3.0	-		
old brickwork	15.0	-		
COMBINED SEWERS				
Pipe full roughness on sewers slimed to about half depth and flowing at velocities between 0.5 and 1.0m/s:		Good*	Normal*	Poor*
Concrete		0.6	1.5	3.0
Asbestos cement		0.6	1.5	3.0
Clayware		0.6	1.5	3.0
UPVC		0.3	0.6	1.5
Pipe full roughness on sewers slimed to about half depth and flowing at velocities greater than 1.0m/s:				
Concrete		0.3	0.6	1.5
Asbestos cement		0.3	0.6	1.5
Clayware		0.15	0.3	0.6
UPVC		0.06	0.15	0.3
SEWERS PARTIALLY BLOCKED BY SEDIMENT	Likely range 30-300 (upper value when Froude number of order 0.3-0.5)			

*The recommended values for slimed sewers are based on a series of experiments on 225mm diameter pipes. The roughness of the pipes varied continuously throughout the course of the experiments. The 'normal' value is the standard roughness value used in compiling the HRS tables[43] that is nearest to the median value of the experimental measurements. The 'good' and 'poor' values are standard roughness values below and above the standard value for the normal condition. Roughly three-quarters of the experimental values fell within the 'good', 'normal' and 'poor' categories.

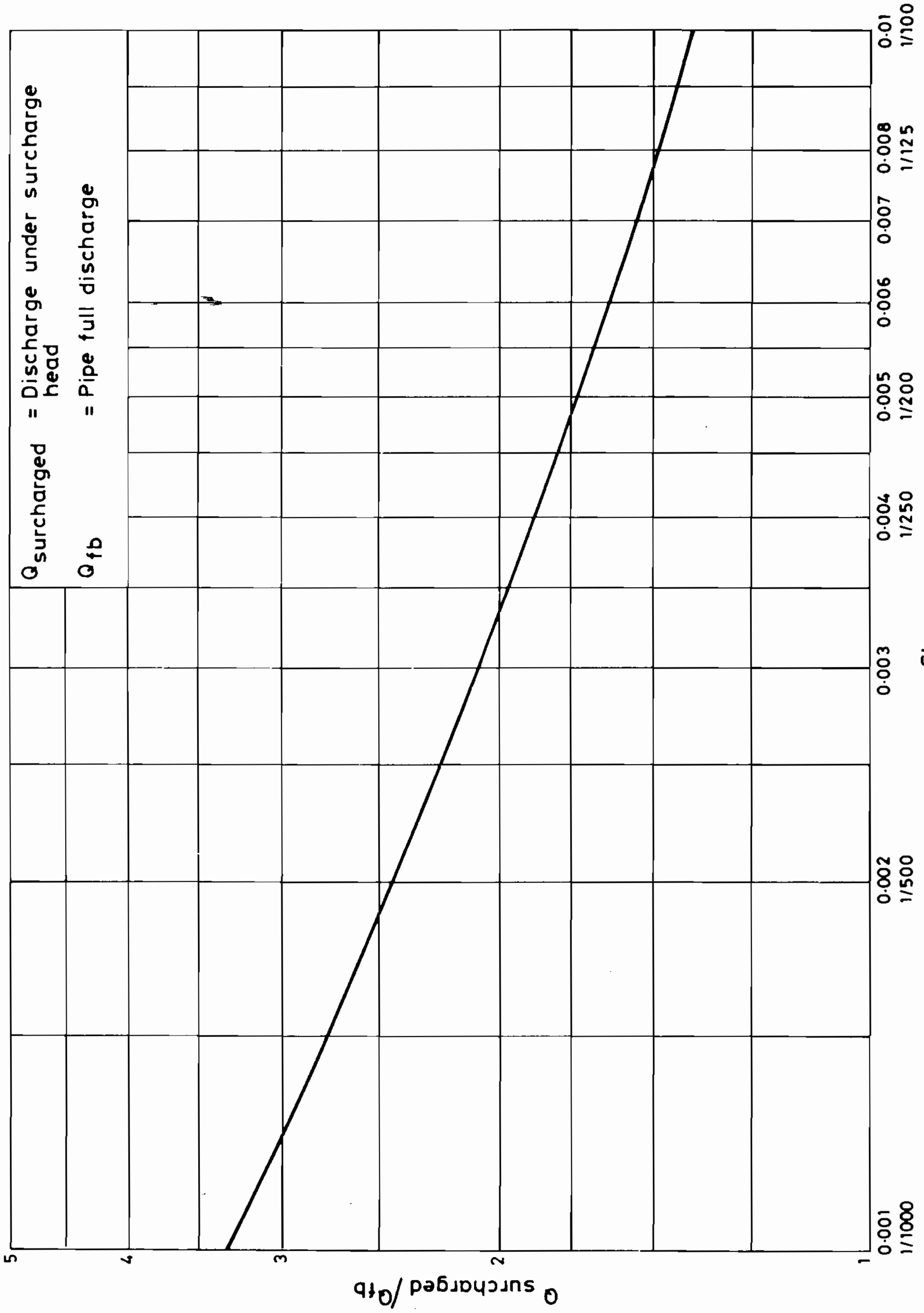

Figure 8.1. Increase in capacity of pipes at different slopes, produced by 1m of surcharge along a 100m length of pipe

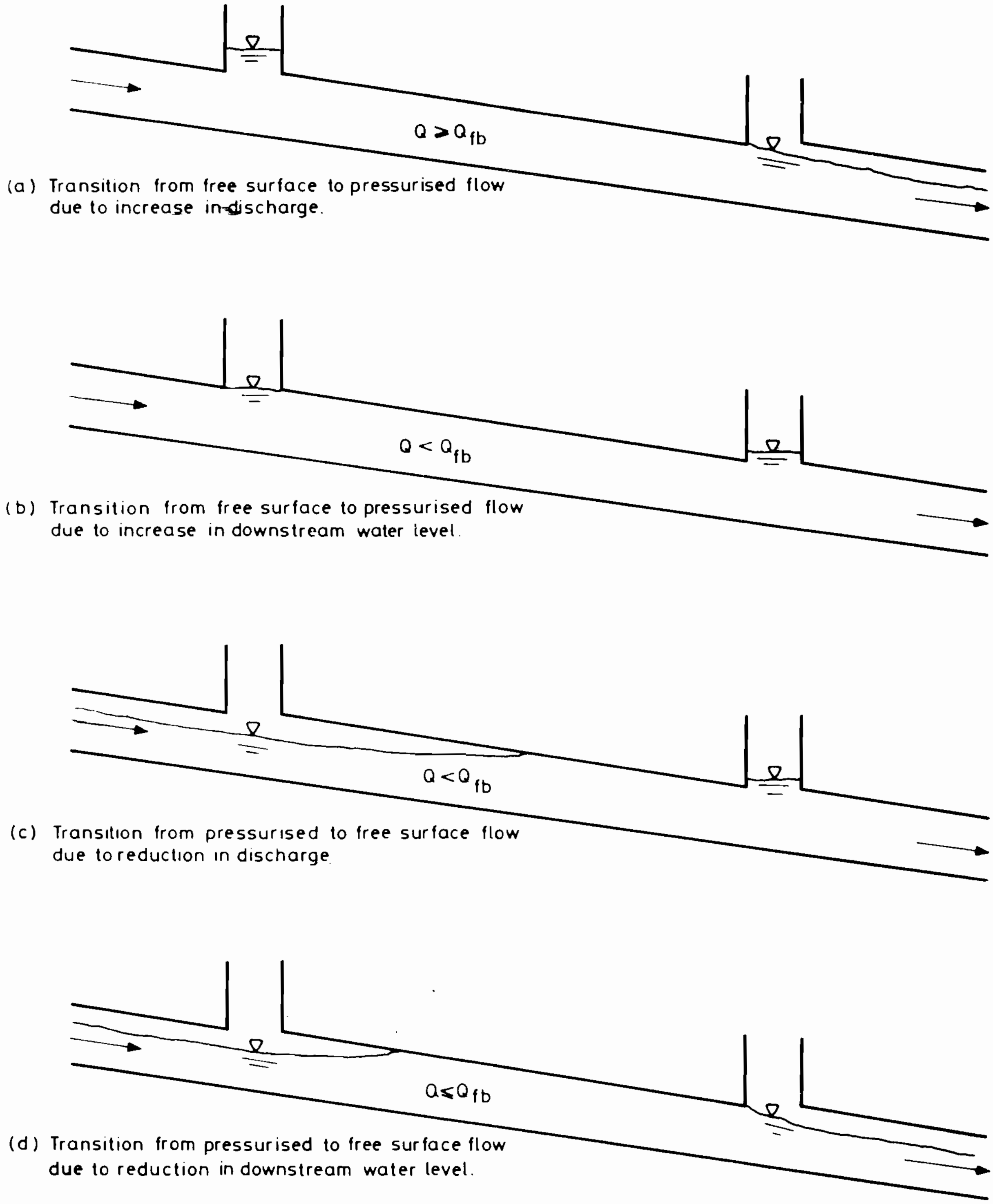

Figure 8.2. Transitions between free surface and pressurised flow in a pipe

Selection of design storm and antecedent conditions

9.1 Introduction

CHAPTER 6 describes the procedure by which rainfall of a specified return period and duration can be determined for any location in the UK. It also shows how the rainfall total can be distributed in time within the duration according to the selected profile. The following design storm values are recommended in chapter 6:

(i) The storm duration should be that which gives the maximum discharge (section 6.2)
(ii) The return period of rainfall should equal that of the required discharge (6.3)
(iii) The profile should be the 50 per cent summer (section 6.6), which is modified within the programs by application of a smoothing filter (section 6.7).

Chapter 7 demonstrates the effect of the antecedent condition of the catchment, as expressed by the variable UCWI, on percentage runoff but it does not include a design value for UCWI.

This chapter provides the UCWI design value (see section 9.4) and outlines the research behind its choice and that of the above design storm.

The use of any isolated-event rainfall-runoff model to estimate a design hydrograph with peak flow of specified return period should involve the selection of a suitable combination of design storm and antecedent conditions. In general, this choice has not been supported by the level of research effort which has been directed to the improvement of the rainfall-runoff model itself. In the Rational and TRRL methods, the antecedent condition of the catchment is not considered. This can lead to large errors in the sense that the actual frequency of the design discharge is not as intended. (Fortunately for designers, it is usually rarer.) To reduce these errors, a research study was undertaken at the Institute of Hydrology [55, 72]. The steps in the research can be summarised as follows:

(a) To obtain the best possible estimate of the flow frequency curve (the graph of discharge against the frequency of its exceedance) for a number of different catchments.

(b) Using the Wallingford Simulation Method to examine the sensitivity of the output (discharge) to the four input variables (rainfall return period, storm duration, profile peakedness and UCWI) and to choose design values for three of them.

(c) To develop a method for choosing a design value of the fourth variable such that the frequency curves of the modelled discharges were a good fit to the previously obtained flood-frequency curves.

9.2 The derivation of flood frequency curves

There are no long flow records from sewered catchments in the UK suitable for flood frequency analysis. To establish a flood frequency curve it was therefore necessary to generate synthetic flow records by applying the Simulation Method to a long observed rainfall record. Two real catchments were used: St Marks Road, Derby, a 10 ha catchment (53 per cent impervious, 87 pipes, clay soil) subject to frequent surcharging; and Shephall, Stevenage, a 142 ha catchment (24 per cent impervious, 796 pipes, relatively pervious soil) subject to only minor surcharging. Three further catchments entitled SW, ST1 and ST2 were also used. Each had the same pipe layout as Stevenage. SW was assumed to be located in the south-west of England. ST1 and ST2 were of increased imperviousness and milder slope, and consequently had larger pipe diameters; ST1 was assumed to be located in south-east England, and ST2 in south-west England.

Two rainfall series were obtained from the Meteorological Office and used in obtaining the flood frequency curves. One was a 98 year series (representative of SE England) which was applied to Derby, Stevenage and ST1; the other a 34 year series (representative of SW England) which was applied to SW and ST2. Significant events were abstracted from each sequence together with an index of antecedent condition (UCWI), giving 318 events for the south-east England series and 119 events for the south-west England series. These data were input to the Simulation Method and the 98 and 34 maximum discharges obtained (for the SE England and SW England series respectively) were plotted as flood frequency curves. Such curves were derived for the outfall and for nine other points in each catchment, making a total of 50 flood-frequency curves. The use of real catchments allowed a useful check on the fidelity with which the model represented catchment behaviour: first on the fit of the model to observed events, and second on the fit to the limited flood frequency data which were available.

It is worth noting, for those familiar with the Flood Studies Report [69], that in a similar exercise on natural catchments, the synthetic flood frequency curves were produced by a different technique (by sampling from the known frequency distributions of the input variables). From here on, however, the methodology is similar.

9.3 The sensitivity analyses

Figures 9.1 to 9.4 illustrate the sensitivity of calculated peak discharge to storm return period (hence rainfall depth), storm profile, storm duration, and antecedent catchment wetness (UCWI). Of necessity, only a small number of curves (relating to different catchments or pipes therein) are shown. Also, each figure relates to only one combination of the other three variables which were held constant while the illustrated variable was changed.

Figure 9.1 shows the effect of changing only the return period of the rainfall. At Stevenage, pipe 1.41, for example, a five-year (30 minute, 50 per cent summer profile) rainfall produces a peak discharge 12 per cent higher than that from a three-year rainfall. This figure provides a useful reminder of the fact that storm return period is just another variable. It has been pointed out in section 6.3 that T-year rainfalls do not necessarily *cause* T-year floods. However, whilst recognising that T-year floods can arise from many different combinations of the input variables, it is still *convenient* to use the T-year rainfall in the design case. (This approach differs from that of the Flood Studies Report [69] concerning design hydrographs on natural catchments, where the optimum solution demanded a non-linear relationship between the return period of the storm and that of the flood.)

Figure 9.2 shows the effect of storm profile on calculated peak discharge. For example, in pipe 1.33 of catchment ST2 the 75 per cent summer profile produces a peak 23 per cent greater than that of the 50 per cent summer profile. Because the model is fairly sensitive to the profile, it was decided to make it one of the prescribed variables. It was found that, to achieve sensible values of the required UCWI (see section 9.4), a 50 per cent summer profile should be used.

Figure 9.3 shows the variation of peak discharge with storm duration. This is a variable which is in fact taken into account in the way in which the computer program works in its design mode. As it works its way downstream, the model is considering successively larger catchments, and therefore designing against successively longer storms. To simplify the process, the model examines a fixed set of durations for each pipe length and sizes the pipe to accommodate the largest flow produced. As Figure 9.3 shows, the peak flow, near to its maximum value, is not very sensitive to the duration and only a few durations need be considered.

Having thus determined design values for three of the four variables, it remains to define a value for UCWI. Figure 9.4 shows how the peak flow varies with UCWI. It is interesting to see the marked difference between the minimal effect at Derby and the more significant effect at Stevenage (5 and 25 per cent respectively in going from UCWI = 50 to UCWI = 100). This can best be explained in terms of equation 7.3 which can be re-written as:

$$PR = SPR + 0.078\ UCWI \qquad \text{......9.1}$$

where SPR is the fixed component of percentage runoff (based on the impermeable area and the soil factor) and is constant for a given catchment. When SPR is high, as at Derby, the effect on PR of changing UCWI is small. When SPR is low, as at Stevenage, the effect is large. The effect on peak flow will be essentially the same as that on PR unless surcharging occurs when it will be reduced. This is a secondary explanation of the small effect at Derby where surcharging is frequent.

9.4 Design value of UCWI

The sensitivity of peak discharge to UCWI was examined for each catchment for a range of storm return periods from 1.6 to 20 years. Figure 9.5 shows a typical result. The plotted points are the optimum UCWI values to match the flood-frequency curve previously obtained: for the pipe illustrated, a value of UCWI = 80 gives a good fit over a range of return periods from 1.5 to 10 years. Similar plots for other sub-catchments in the Stevenage catchment showed UCWI = 80 was a good fit for the whole of Stevenage. However, for the equivalent catchment SW where the flood-frequency curve was obtained from the SW England rainfall sequence, a value of 140 was better.

Figure 9.6 shows a plot of the optimisation of UCWI for each major catchment. Note that for return periods greater than 10 years the optimum UCWI value is slightly reduced (see Figure 9.5) so use of the Figure 9.6 values would lead to an overestimate of the T-year flood at these higher return periods. This could be 5 per cent when $T > 20$.

The optimum value of UCWI may reasonably be expected to vary with climatic regime, so a study of median observed (summer months) UCWI values was made for 28 stations throughout the UK. Median UCWI was found to be well correlated with standard average annual rainfall (SAAR). Moreover, the least error values of UCWI for the SE England and SW England rainfall series (65 and 115 respectively, see Figure 9.6) fit very closely to the fitted curve shown as Figure 9.7. Consequently, it is recommended that the design value of UCWI be taken from this graph. A map of SAAR at a scale of 1:1000 000 is available in a separate folder, and at a reduced scale as Figure 9.8. Figure 9.9 shows the fit obtained for the Stevenage catchment (using the design value of 65 for UCWI) to the synthetic flood frequency curve derived previously.

9.5 Conclusions

Although this analysis has been limited by the quantity and quality of the available data it has provided a logical approach to the choice of design conditions. In addition to the three listed in the Introduction above (section 9.1), we can now add:

(iv) The design value of UCWI varies with the annual average rainfall and should be taken from Figure 9.7.

This particular set of the four design variables produces a good estimate of the complete synthetic flood-frequency curve which is itself an acceptable approximation of the true curve. To achieve design against a T-year flow, therefore, the design recommendations should be carefully followed.

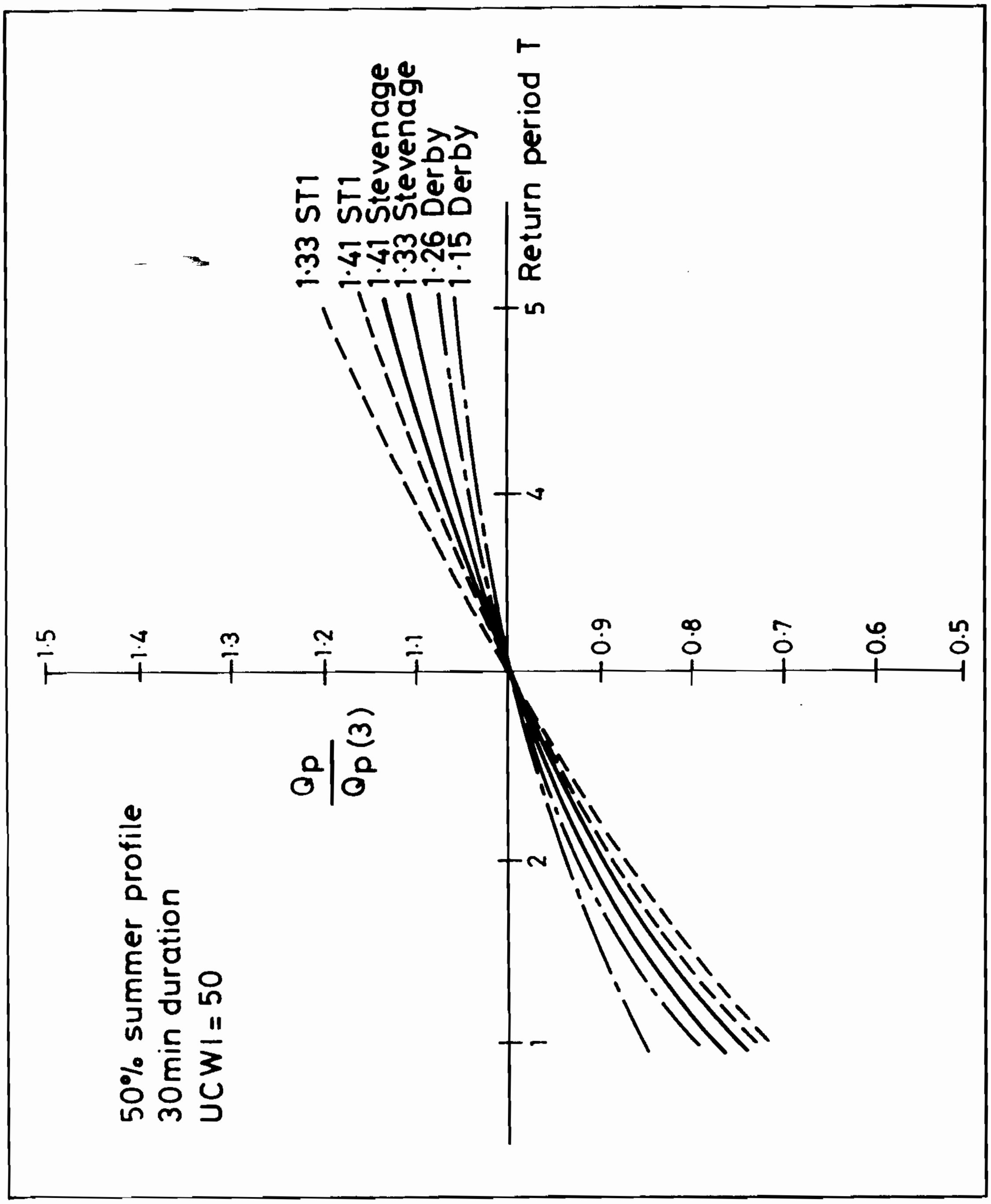

Figure 9.1. Sensitivity to storm return period

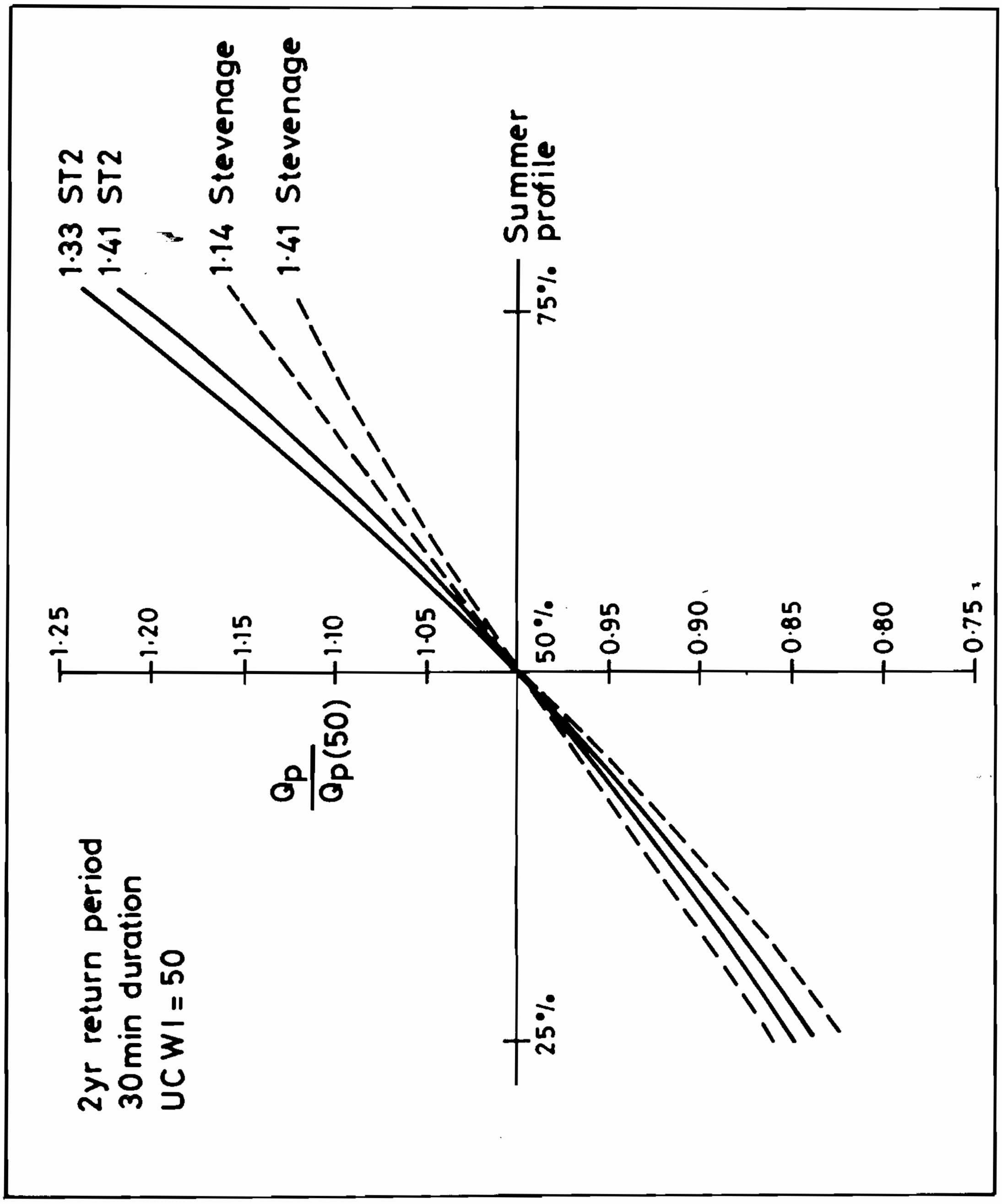

Figure 9.2. Sensitivity to storm profile

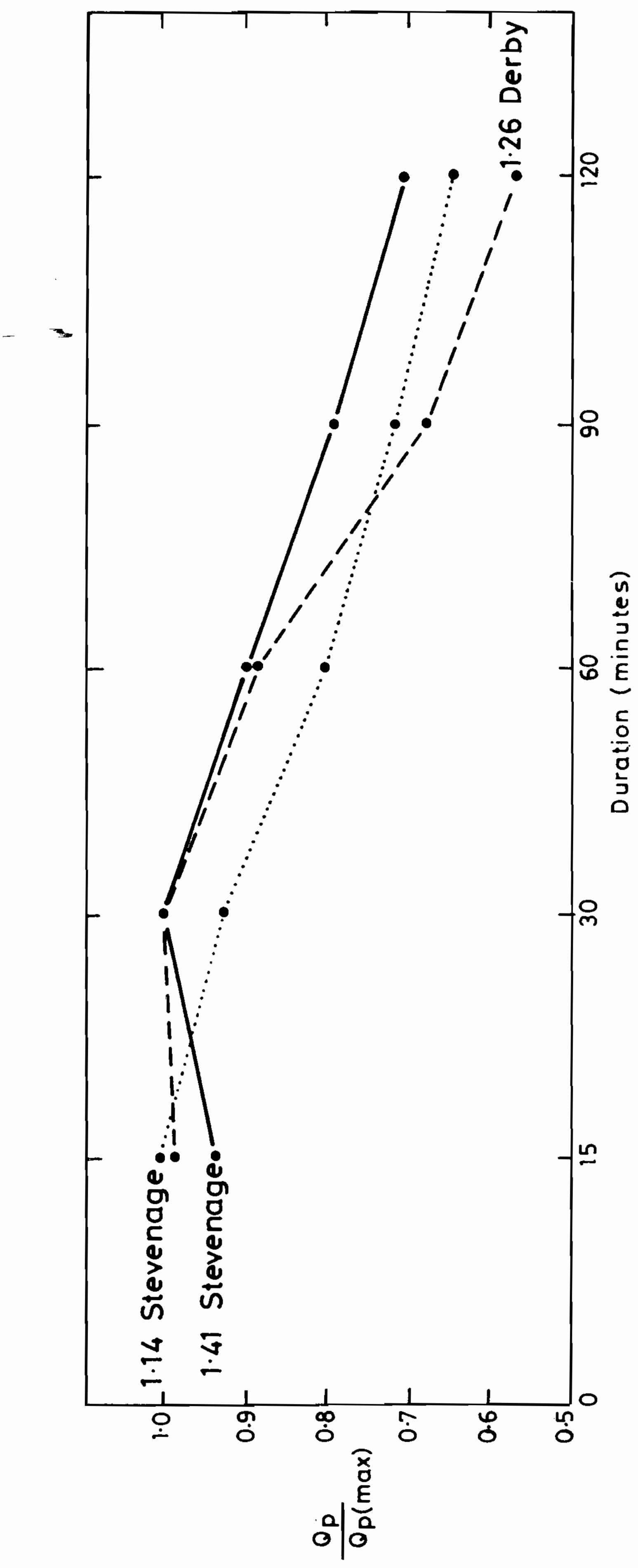

Figure 9.3. Sensitivity to storm duration

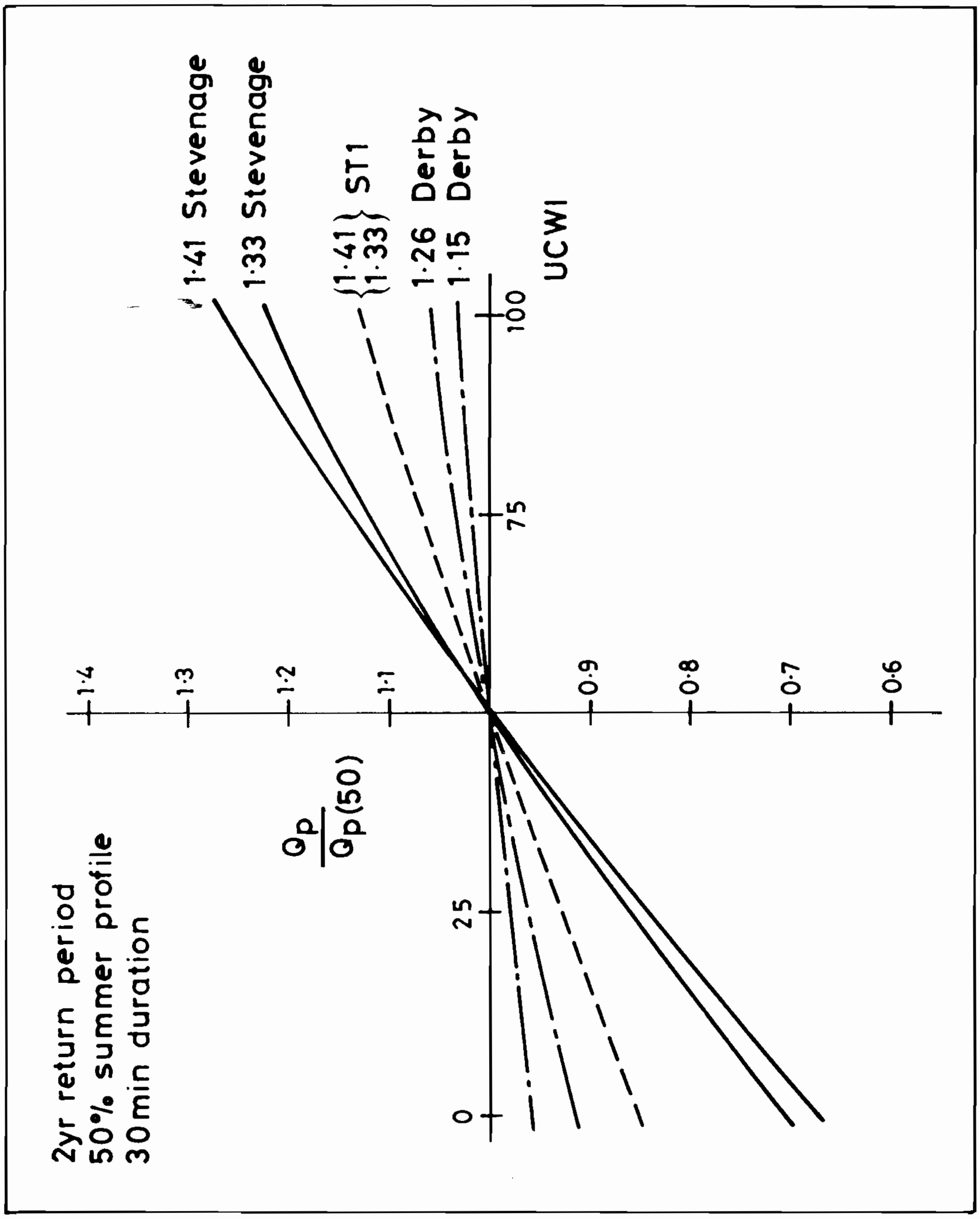

Figure 9.4. Sensitivity to catchment wetness

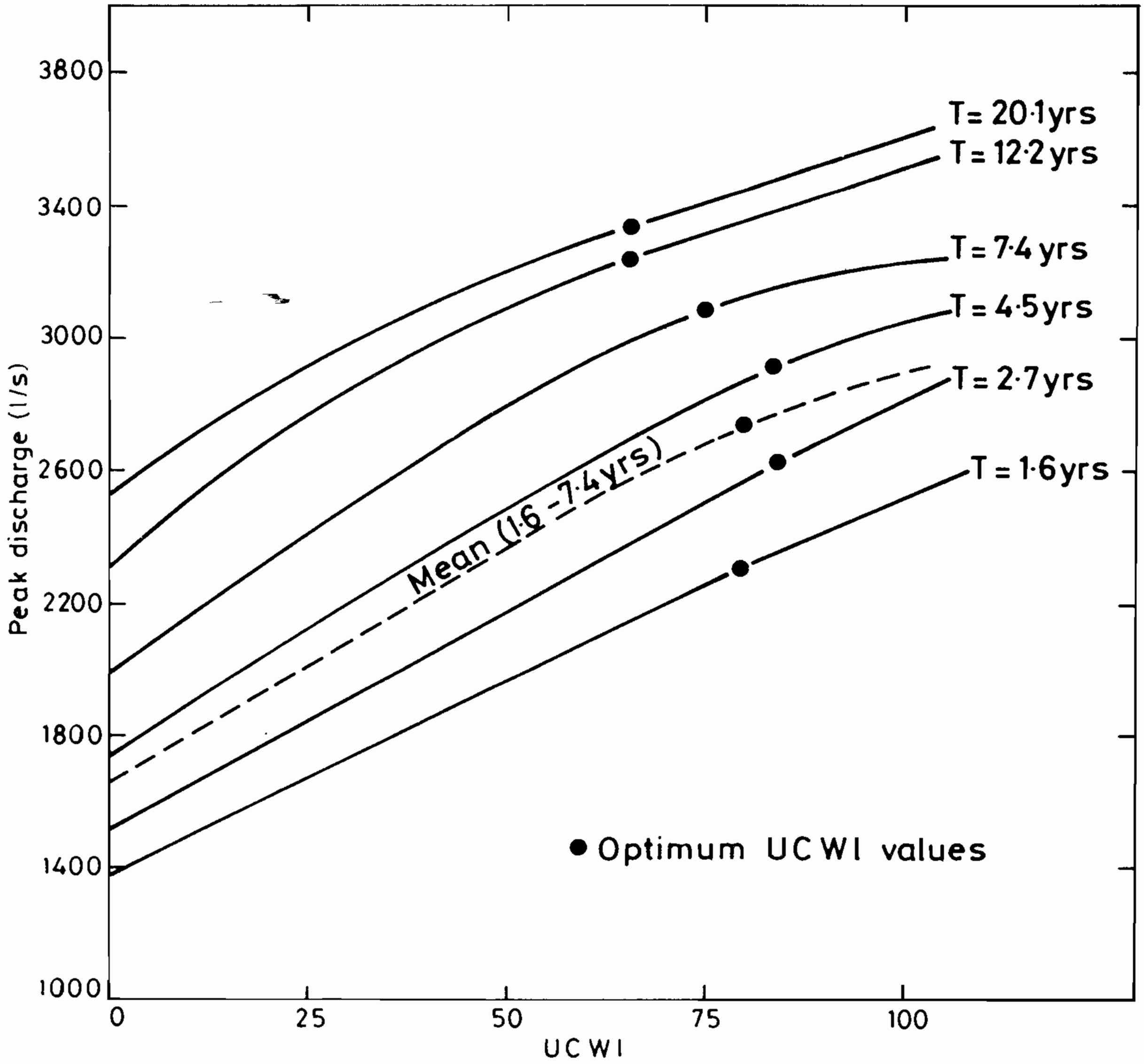

Figure 9.5. Variation of peak discharge with return period and UCWI for pipe 1.41 at Stevenage

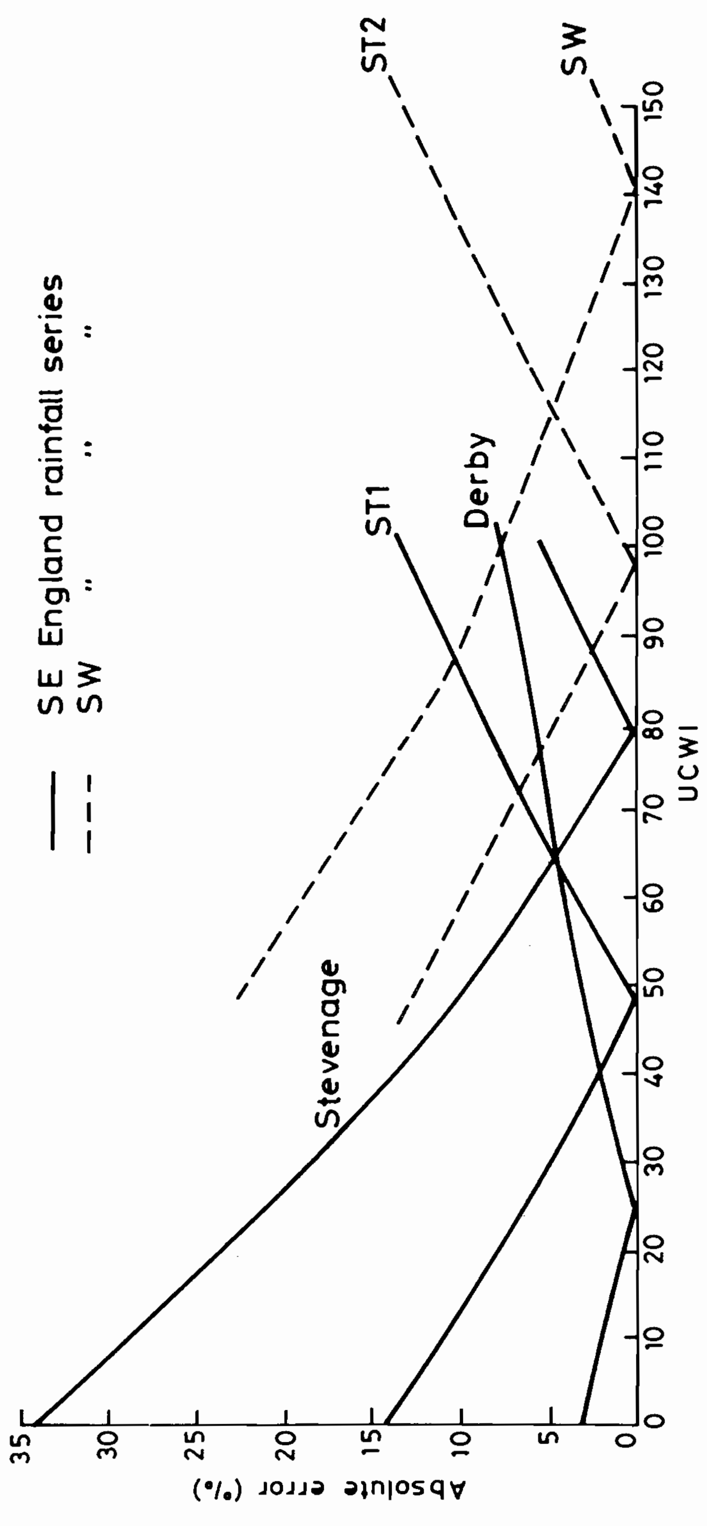

Figure 9.6. Optimisation of UCWI for all catchments

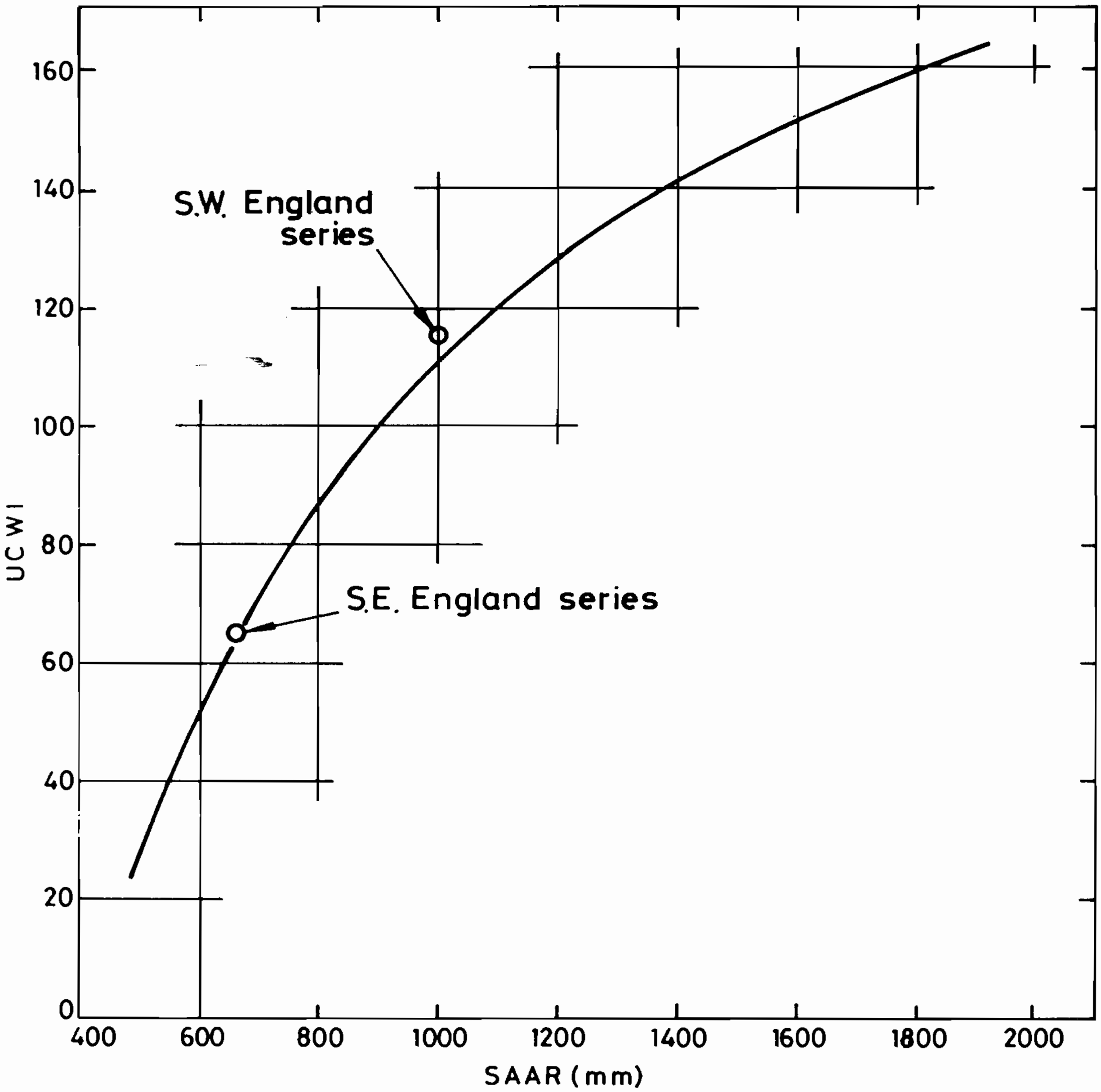

Figure 9.7. Relationship between UCWI and SAAR

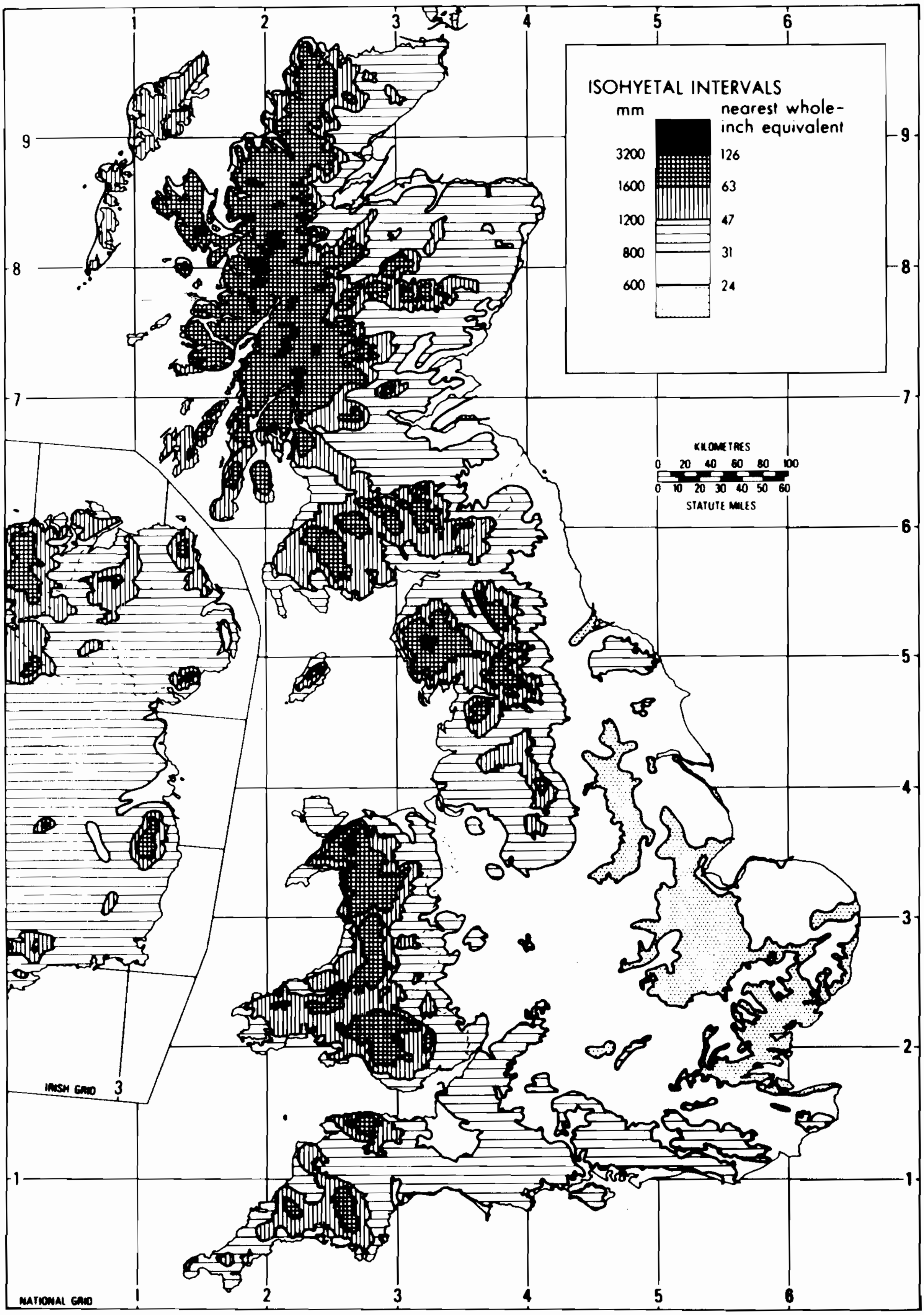

Figure 9.8. Average annual rainfall (1941-1970)

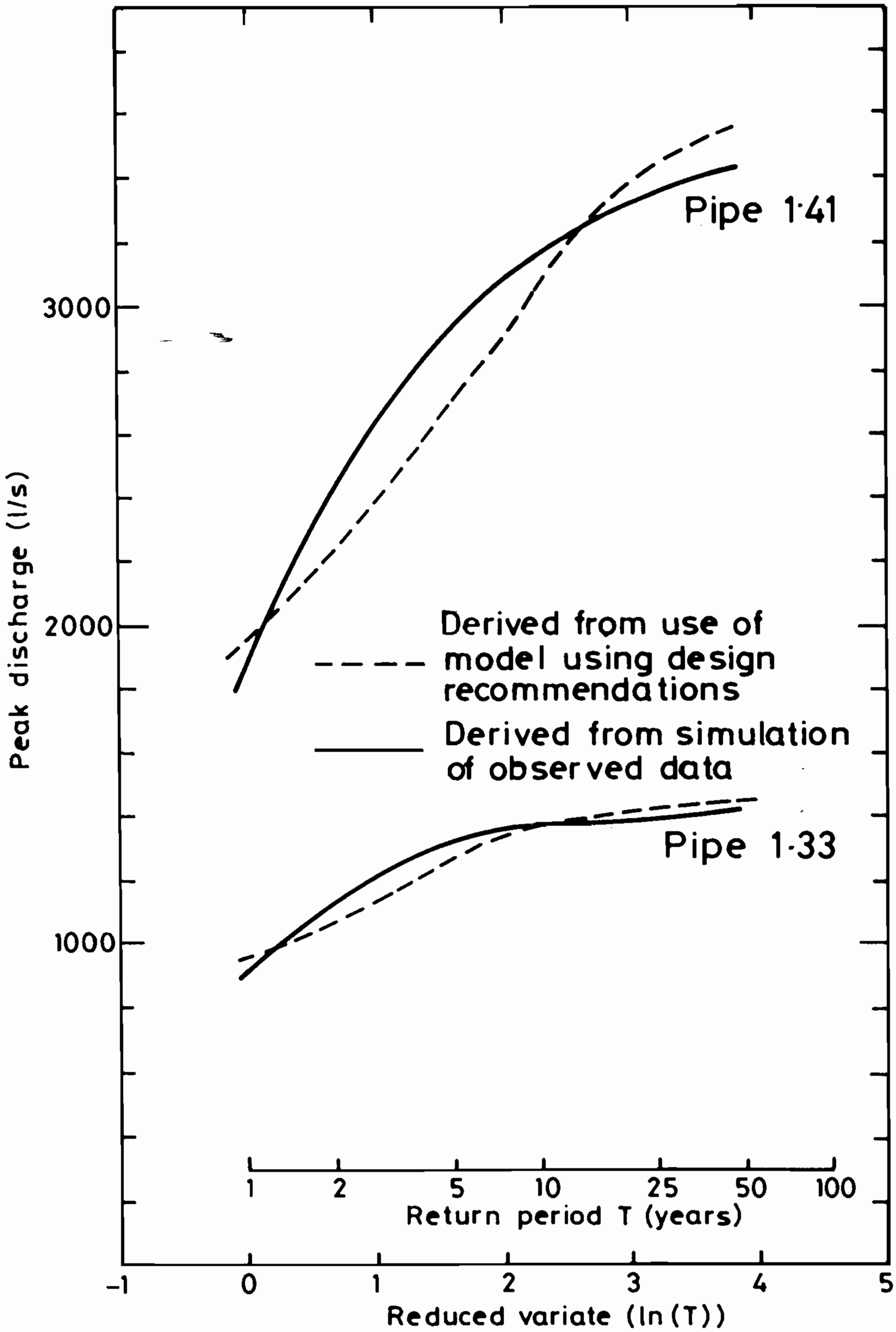

Figure 9.9. Comparison of flood frequency distributions at Stevenage

CHAPTER 10

Runoff from sewered sub-areas

IN SOME circumstances sufficient data may not be available to permit the modelling of the complete above-ground and below-ground runoff process for each contributing area and pipe length. Alternatively, the data may be available but unacceptably large handling or computing costs may be incurred by detailed calculations for a large catchment. For these circumstances a simplified sub-area model has been prepared [79]. It may be used alone, or to calculate the runoff hydrograph from part or parts of a larger catchment which is otherwise modelled by the more detailed procedure.

The sub-area model makes use of the models developed to calculate the surface runoff and pipe flow, described in sections 7.4 to 7.8 and 8.2 to 8.3. The method of application is, however, simplified, which permits a reduction in the data required.

In the sewered sub-area model the surface runoff phase is modelled by the procedure described in section 7.8, but as a single calculation rather than separately for each contributing pipe length. The resulting runoff hydrograph is divided into a number of equal parts (see Figure 10.1) and distributed equally to the same number of lengths of an 'equivalent pipe' through the sub-area. The equivalent pipe consists of a tapered system of pipes in series which approximates to the complex reality of a branched network. The number of divisions is dependent on the time of flow along the equivalent pipe, and is calculated within the program.

All the lengths of equivalent pipe have the same length and slope, and the diameter is calculated from a specified outfall diameter and a degree of tapering assumed within the program. Routing through the equivalent pipe is by a fixed parameter Muskingum-Cunge method (see section 8.3). The only data requirements for the use of the model are the total length of the major pipe run in the sub-area, the average pipe slope, and the diameter and slope of the outfall pipe. These data are easily determined for an existing network; for a proposed development the engineer must determine the probable length and slope of the main pipe, and calculate the outfall diameter using the Modified Rational Method. Sensitivity analyses have indicated that a 10 per cent error in estimating any one of these values will result in a maximum error in peak discharge of 3 per cent.

The model was tested and verified by comparing its output with calculations by the full modelling procedure, for a range of catchments and rainfall-runoff events. On one catchment the sewer system of 109 pipes was represented satisfactorily in the simplified model by just four equivalent pipes. The sewered sub-area model may be used on sub-areas up to 60 ha in area with little loss of accuracy.

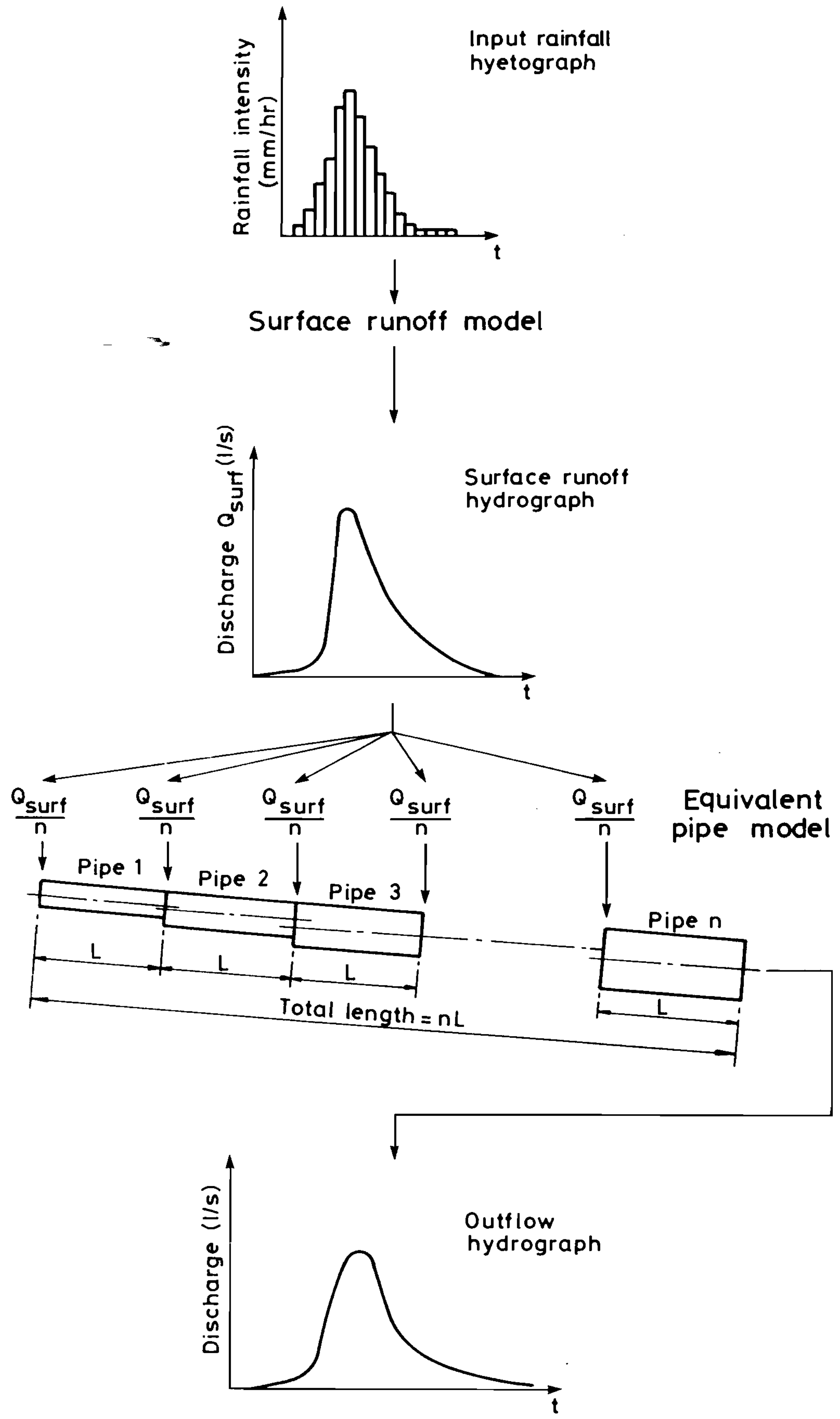

Figure 10.1. Schematic representation of sub-area model

CHAPTER 11

Ancillary structures in sewerage systems

IN DESIGNING sewerage systems it is often necessary to incorporate ancillary structures to overcome physical or economic constraints, by releasing the storm sewage into a natural watercourse, attenuating peak discharges or storing water for subsequent release. This chapter describes how these facilities are incorporated in the Wallingford procedure. The procedure does not include the detailed hydraulic design of these structures, but the effects of alternative designs on the flow conditions in the network may be examined. In this way the use of the procedure will be a valuable step towards the final design of sewer ancillary structures.

11.1 Storm overflows

Sewers carrying surface water may contain diversion or overflow devices to limit the peak flow downstream. The detailed design of such devices in a combined sewer is aimed at ensuring that the maximum amount of polluting material is passed to treatment to minimise pollution loads in receiving streams. A Ministry of Housing and Local Government report [66], dealing with storm overflows and the disposal of storm sewage, considered a number of structures including low side weirs, stilling ponds, vortex overflows and storage chambers with high side weirs. Opportunities for the modelling of storm overflows are discussed further in section 17.5.

The procedure incorporates in the Hydrograph Method and the Simulation Method two alternative models for the analysis of storm overflows. One considers only the proportion of flow diverted, the other takes account of the storage within the overflow chamber. The first type is also applicable to a bifurcation in the pipe system, but is not appropriate under surcharged flow conditions. A storm overflow without an overflow chamber is modelled by specifying a discharge setting, Q_s, for the overflow and either the maximum discharge, Q_{max}, permitted downstream of the overflow or the constant proportion, k_o, of the excess discharge $(Q_{in}\text{-}Q_s)$ which is permitted to continue downstream. Q_{in} is the flow arriving at the overflow. The discharge, Q_{over}, diverted from the overflow is given by:

$$Q_{over} = 0 \qquad \text{if } Q_{in} \leqslant Q_s$$
$$Q_{over} = (1 - k_o)\ (Q_{in} - Q_s) \qquad \text{if } Q_{in} > Q_s \qquad \text{......11.1}$$

The overflow discharge is either fed into the head of another branch of the system or excluded from further consideration.

The modelling of a storm overflow incorporating an overflow chamber is described below (section 11.2).

The effect of storm overflows is taken into account in the Modified Rational Method by calculating new values of the contributing area for both the continuation pipe downstream of the overflow and the pipe receiving the overflow discharge. These values are determined according to the proportion of flow diverted at the overflow. The catchment area, A_{over}, contributing to the diverted discharge is:

$$A_{over} = (1 - k_o)(A_u - 0.36\frac{Q_s}{Ci}) \qquad ......11.2$$

where A_u is the area upstream of the overflow, C is a dimensionless coefficient (see section 7.10) and i is the rainfall intensity in any calculation at or downstream of the overflow.

The corresponding area for the discharge continuing downstream of the overflow is $A_u - A_{over}$.

If $\frac{CiA_u}{0.36}$ is less than Q_s then $A_{over} = 0$.

11.2 Detention tanks

Detention tanks are structures designed to hold back excess storm sewage which is only released when the flow in the downstream pipe is less than its just-full capacity. A particular application of detention tanks [86] is in conjunction with storm overflows. In moderate storms there may be no overflow from the detention tank to the watercourse but during severe storms some overflow will take place. Models of both on-line tanks and off-line tanks are incorporated in the procedure: on-line tanks consist of enlarged flow sections within the pipe network, whereas off-line tanks are physically separated from the flow through the pipe system. Further aspects of the use of detention tanks are discussed in section 17.4.

On-line tank/storm overflow chamber

The on-line tank model assumes that the tank or chamber stills the flow sufficiently to permit the assumption of a uniform head over the weir. This assumption is not appropriate for many storm overflow configurations, especially side weirs, for which the model described in 11.1 should be used.

Assuming that the plan area, A_{st}, of the tank is independent of water level, y, the storage equation for the tank is

$$A_{st}\frac{dy}{dt} = Q_{in} - Q_{out} \qquad ......11.3$$

where t is time

$$\text{and } Q_{out} = Q_{over} + Q_{orfc} \qquad ......11.4$$

Q_{over} is the overflow discharge diverted out of the tank, and may be fed into the head of another branch of the system or ignored. It is defined by

$$Q_{over} = C_w L_w \sqrt{g} H_w^{3/2} \qquad ......11.5$$

where C_w is the discharge coefficient for the overflow weir
L_w is the weir length
and H_w is the head above the weir crest.

Q_{orfc} is the flow which continues along the main pipe from the tank or chamber, and may be calculated from:

$$Q_{orfc} = C_o A_o \sqrt{gH_o} \qquad ......11.6$$

where C_o is the discharge coefficient for the orifice(s)
A_o is the cross sectional area of the orifice(s)
and H_o is the head difference between the water levels in the tank and the downstream pipe.

Off-line tank

The off-line tank is regarded as a structure which receives water from an overflow. The formulation of the off-line tank model is the same as the on-line tank model (equation 11.3), but in this case Q_{in} is the flow entering the tank from the upstream overflow and

$$Q_{out} = Q_{over} + Q_{ret} \qquad \text{......11.7}$$

Q_{over} is the overflow discharge out of the tank, which may be fed into the head of another branch of the system or ignored. Q_{ret} is the flow which returns to the pipe from the off-line tank: it may be determined from an orifice flow equation (11.6) or specified as a constant pumped rate.

11.3 Pumping stations

The high cost of constructing and operating pumping stations to increase the hydraulic head is only justified when drainage downstream cannot be achieved by gravity. Pumping modifies the runoff hydrograph which is transferred to the head of a downstream system. The procedure incorporates a pumping station model which requires data on wet well levels and geometry and pump characteristics.

Two equations describe the flow through the pump wet well and the pumps:

$$A_{st}\frac{dy}{dt} = Q_{in} - Q_{pump} \qquad \text{......11.8}$$

$$Q_{pump} = Q_{des}\left\{1 + \frac{y - h_w}{H}\right\}^{1/2} \qquad \text{......11.9}$$

where A_{st} is the plan area of the wet well
y is the water level
Q_{in} is the inflow discharge
Q_{pump} is the pumped discharge
Q_{des} and H are the design discharge and head respectively of the pump(s)
and h_w is the mean of the switch on and switch off levels in the wet well.

In order to obtain the time-varying flow through a pump, A_{st}, Q_{des}, H and the switch on and switch off levels must be specified. Alternatively, a constant value of Q_{pump} may be specified directly. These data are required for each pump included in the pumping station.

11.4 Inverted syphons

Occasionally, the design of the sewer system may be complicated by obstructions such as existing buried services, waterways or roads in cutting. Inverted syphons which remain full even when there is no flow are a means of carrying the sewer under the obstructions. The hydraulic performance of a syphon is similar to that of a normal sewer at a continuous gradient running full. If open channel flow occurs in the adjacent parts of the pipe system the syphon is neglected apart from the change in invert levels across it. If the adjacent parts of the network are surcharged the syphon is regarded as an equivalent length of surcharged pipe with additional head losses at the upstream end.

11.5 Outlet structures

A drainage system should normally be designed with a free flowing outfall. However, there may be occasions when the outfall is below the level of a river, pond or the sea. In these conditions the surcharged behaviour of the final lengths of the sewer system may be examined given the time-varying level of the receiving waters.

CHAPTER 12

Costs and economics

THE ECONOMIC evaluation of storm drainage consists of the examination of the costs and the benefits to society of the provision of storm drainage in particular locations, and the way in which those costs and benefits are related to the design standard of the scheme constructed.

This chapter concentrates on the facilities available within the procedure. Wider aspects of economic evaluation and the way in which the procedure can be used in economic assessments are considered in Part 4 of this report (chapter 16).

12.1 Facilities available in the procedure

The procedure includes two optional features which enable the designer to take account of cost and economic factors when considering alternative designs:

(a) Calculation of the cost of the resources used in construction (materials, plant and labour). It must be emphasised that the cost of resources used does not represent the total cost of a scheme; the difference between resource cost and total cost is explained in section 12.2, and the resource cost model is described in detail in section 12.3.

(b) Selection of pipe gradients to give minimum construction cost (section 12.4). An optimising method allows the pipe depths, gradients and diameters to be varied within specified limits to yield a hydraulically satisfactory design at minimum construction cost, based on the cost of the resources used.

These features may be used in a variety of ways, and therefore provide a flexible means of evaluating storm drainage schemes. Some of the ways the options may be treated are:

(i) Both options may be ignored. A scheme may then be designed to gradients specified by the engineer and for a fixed return period of input values. This may well be the best option for small schemes in which little gain would be achieved by a more detailed exercise.

(ii) Pipe gradients may be optimised for minimum construction cost, but costs are not considered outside the optimising process and the performance of alternative designs is not considered.

(iii) Construction costs, for either optimised or non-optimised alternative designs, are calculated and evaluated in conjunction with the calculated performance of the alternative designs. This is termed a 'performance-cost' approach; it is introduced in section 12.5 and considered again in section 16.6.

(iv) An evaluation of the frequency of flooding and consequent damage costs may be used in a calculation of the benefits of alternative schemes, and compared with construction costs. This benefit-cost approach is described in section 16.3 and in the Annex to chapter 16.

An important factor affecting the cost of storm sewer construction is whether the scheme under design is completely new or whether it includes existing sewers which may be retained. A thorough examination of existing sewers will be necessary to determine whether they should be retained in the new design. This examination will include inspection, estimation of structural condition and future life and calculation of hydraulic capacity. Engineering skill and judgement will be required in this process. It is possible to use the modelling procedure to test the cost and economic effects of either retaining or replacing parts of the existing sewer system.

A flow chart illustrating the decisions involved in using the cost and economic features of the procedure is shown in Figure 12.1. In practice, the user will return, perhaps several times, to earlier parts of the process to test different approaches, but these options have been omitted for simplicity.

It should be noted that many factors influence the efficient design and operation of storm sewers, and it is not possible to include all these in the economic evaluation procedure. It is therefore unlikely that a design optimised on the basis of hydraulic performances and costs would remain optimal if all factors (engineering, social, economic and financial) could be taken into account. In particular, the design is based on certain assumptions about the future, which may not be correct: for example, catchment development may not occur as expected, or rainfall frequencies may vary, and these changes will affect the flow frequency in the system. Subsidence may affect the gradients of some pipelines; maintenance requirements may increase due to unexpected sediment loads or structural failures. However in many catchments the possibility of some of these occurrences can be assessed (eg future catchment development, future mining subsidence and its effect on gradients) and the programs can be used to consider their effects on the design. The difficulties involved in economic evaluation are discussed in greater detail in chapter 16.

12.2 Alternative approaches to cost modelling

Two alternative approaches to cost modelling have been developed in recent research. These approaches are complementary, and each is necessary in different parts of the cost evaluation process.

The first approach was based upon site studies of the resources and operations involved in sewer construction. This approach was used by Farrar [27] and Bramwell [10]; it has the advantage that it is based on separate consideration of the various elements which affect construction costs such as trench depth, pipe diameter, ground and site conditions and the efficiency of plant and labour. A model of this type is essential for optimisation applications in which one or more of these elements has to be varied independently of the others. The disadvantage of the resource cost approach is that the results will differ considerably from final costs, because of such factors as the extent of works ancillary to the actual pipeline, the contractor's other commitments, and his estimate of the state of the market. Therefore the results should not be used as an estimate of total construction costs unless an adjustment is made to take account of these additional factors.

The resource cost model developed by Farrar [27] was selected for use in the optimising procedure because it was a simpler approach requiring less information, and because it permitted significant factors affecting construction costs to be varied independently. The model was based on observations at a number of sewerage contracts, in which the materials, plant and labour elements employed to construct flexibly jointed rigid pipelines were recorded or timed.

The model includes a complete set of the necessary cost data, but the user may specify alternative values if available. The only data required by the model, in addition to the values already required for the hydraulic calculations, are indices describing site conditions and ground conditions. These indices are shown in Table 12.1. The definition of site conditions is required to establish the need for excavation and reinstatement of carriageway, and the feasibility of using trenches with battered sides. The definition of ground conditions is required to establish the rate of excavation and the need for trench sheeting. Costs incorporated within the program may be updated by specifying an appropriate weighted Baxter index. The resource cost model is described in detail in section 12.3.

The second approach to cost modelling examined successful tender costs to give a better indication of total cost to the client, as a function of the more significant features of each contract. This approach was used by the Water Research Centre [97] and is the most suitable for broad consideration of national or regional planning. The tender cost model cannot be used for the optimisation of individual elements, because they are treated in a much broader fashion than in the resource cost model.

The WRC study examined successful tender documents for 80 sewerage schemes carried out by local authorities, water authorities and consultants. Cost details were related to various categories and parameters. Multiple regression techniques were used to determine the total cost as a function of the most significant variables. The final form of the model gives total cost as a function of sewer length, mean diameter weighted for length, mean depth and an 'over-under factor'. This factor consisted of a subjective assessment of the difficulty of a particular contract, based on problems including the range of depth and diameter, surface, site and ground conditions, and water table level. The best means of updating the costs were also studied, and relevant cost indices are recommended.

The exact form of the equation and instructions in its use and limitations are given in the WRC report [97]. The 80 per cent confidence limits have been calculated as 56 per cent to 178 per cent; these values indicate that although the model is valuable for general planning and initial appraisal purposes, it should not be used when an accurate assessment of the costs of a particular scheme is required. The equation has not been included in the present procedure, in order to guard against its application in circumstances to which it is not appropriate.

An exercise has been carried out in which 26 schemes have been costed by both the resource cost and tender cost models, and compared with the actual tender cost. The results are shown in Table 12.2. In very general terms, the resource cost model accounts for about half of the likely tender cost, but the wide scatter in the results, indicated by the standard deviation, provides a warning against applying this finding to particular schemes.

12.3 The resource cost model of sewerage construction costs

The construction cost model incorporated in the procedure is based on field studies by Farrar[27], and may be described as a 'resource cost' model. It takes account of the resources of plant, labour and materials used in sewerage construction.

It is assumed that the utilisation of plant and labour can be taken as 50 per cent and 75 per cent of the time on site respectively, and the costs are increased accordingly to allow for non-productive time. The model includes an allowance for overheads and ancillary plant, but no allowance is made for administrative or other preliminary costs (eg flow diversions) incidental to pipeline construction, nor for special features such as extensive de-watering, the construction of backdrops or inverted syphons, and the provision of concrete bedding or haunching. The relationship between the resource costs calculated by the model and final costs has been discussed in section 12.2.

The model considers the most common operations undertaken during the construction of sewer networks. The costing is broken down into two separate aspects for each operation:

(a) Cost of labour and plant

(b) Cost to supply materials

The construction methods described below were used for costing purposes only; they do not necessarily represent a recommended method of construction.

12.3.1 Breaking surface

The cost of breaking out the surface is calculated if Surface type 1 or 2 (road or pavement, see Table 12.1) is specified. The minimum trench width is set at 0.75m, but if the diameter exceeds 300mm then the width is defined by the equation:

$$W = 0.6 + 1.3\,d \qquad \text{....12.1}$$

where d = nominal pipe diameter in metres. It is assumed that the outside diameter of the pipe can be defined as 1.3 d.

12.3.2 Excavation

If Surface type 3 (open fields) is specified in the data it is assumed that a trench with battered sides, side slope 45 degrees, is excavated. The width of the trench up to the pipe soffit is taken as W, as defined above. For other surface types, a vertically-sided trench of width W is assumed.

The depth to formation level, D_e is expressed as a function of the pipe diameter and average depth by the relationship:

$$D_e = D_a + 0.1 + 0.15\,d \qquad \text{....12.2}$$

where D_a = average depth to pipe invert in metres. Hence the volume of excavation can be calculated. The rate and cost of excavation are then calculated depending upon the ground classification, the depth of excavation, the need to place open or closed sheeting and the use of tracked or wheeled excavators.

If the depth of cover is less than 0.9m there is an increased risk that other services will have to be diverted or avoided. Therefore alternative methods of design or construction may be more appropriate. Similarly if the depth of cover exceeds 5m tunnelling may be more appropriate. In these conditions the total resource cost excluding manhole construction is multiplied by a factor of 1.5 and a warning is given that alternative construction techniques should be considered.

12.3.3 Sheeting

Open or closed sheet piling is only included if the depth of excavation exceeds 1.5m or if unstable ground (Ground type 4) is specified in the data. When open fields (Surface type 4) are specified in the data, sheeting is only included for trenches deeper than 3m.

12.3.4 Bedding

It is assumed that class B granular material will be used and that the pipe will be founded on 0.1m of material and surrounded to a depth of half the pipe diameter, ie a bedding factor (F_b) of 1.9 [96]. The volume of bedding material can therefore be defined by the relationship:

$$\text{Volume per m length} = (0.1 + 0.65\,d)\,W - 0.66\,d^2 \qquad \text{....12.3}$$

12.3.5 Pipe

The costs to supply and lay rigid pipes are calculated assuming that the rate of laying the pipes is not affected by the length of each pipe unit or the type of joint. But, depending on the type of surface, the average depth and pipe diameter, an allowance is made for the differing rates of laying the pipes according to whether manual methods, excavators or cranes are used.

If the depth of cover exceeds 3m and the pipe diameter exceeds 0.3m the cost of the pipe is multiplied by a factor of 1.1 to allow for the increased pipe strength required. This factor is not applied if the 50 per cent increase described in section 12.3.2 is appropriate.

12.3.6 Backfill

The cost to backfill is calculated on the basis that the excavated material is suitable and that the costs of transporting the material on the site are small. It is assumed that dumper trucks and power rammers will be used. Additional cost would be incurred if the excavated material was unsuitable for backfilling or if it had to be stockpiled away from site.

12.3.7 Surfacing

The cost of resurfacing with tarmacadam is included if pavement (Surface type 2) is specified in the data. In addition the cost to supply, lay and compact a concrete sub-base is calculated if road (Surface type 1) is specified in the data.

12.3.8 Remove surplus

The model includes typical costs for the removal of surplus excavated material. In practice, the cost of this operation varies considerably according to the distance the spoil has to be transported. Thus it may be necessary to increase the final estimate if the distance is large or if transport costs are disproportionately high.

12.3.9 Craneage

It is assumed that cranes are not used for pipes with diameters less than 750mm. However, if the pipe diameter exceeds 750mm or 1200mm the cost of 6 or 15 tonne cranes respectively is included. The cost of operation of a crane is dependent on the duration of the work, and this duration is a function of the type of surface (ie ease of operation), the rate of excavation and the depth of excavation.

12.3.10 Manholes

The cost to supply and construct precast concrete manholes is calculated on the assumption that there will be one manhole per pipe run considered. This costing does not include any allowance for backdrops or other ancillary works.

The diameter of the manhole is set at 1.05m for all pipes with diameters less than 380mm. If the nominal pipe diameter specified in the data exceeds 380mm the manhole diameter is defined as:

$$\text{Manhole diameter} = d + 0.762 \qquad \text{....12.4}$$

The cost to supply and construct the manhole is obtained from data relating the cost to depth of formation for various manhole diameters [59].

12.3.11 Revision of costing data

The model is so designed that the user can easily revise the cost data to take account of inflation or to include improved data. This revision can be undertaken in two ways.

(a) Where sufficient data are available the datum costs can be altered in the model itself by revising the values set in the data statements. Care must be exercised to ensure all costs are to a common date.

(b) The calculated costs may be updated to any month by specifying an inflation factor. Since the construction cost model is based on the cost of the resources employed, an inflation index similarly related to the cost of resources is required. The Baxter series [82] is a readily available set of such indices. The inflation factor required by the model (BAXTER) is a weighted average of Baxter indices 1 (labour), 2 (plant), 3 (aggregates) and 5 (cement) as follows:

$$\text{BAXTER} = 0.45\ \text{BAXTER 1} + 0.30\ \text{BAXTER 2} + 0.15\ \text{BAXTER 3} + 0.10\ \text{BAXTER 5} \qquad \text{....12.5}$$

This weighting was based on the distribution of major costs in several sewerage contracts.

If a value of BAXTER is not specified, or is specified as zero, the cost estimates are related to January 1979.

12.4 Depth-gradient-diameter optimisation

The Wallingford Optimising Method is included in the procedure to provide optimised designs of pipe depth and gradient as well as pipe diameter. The objective of the optimisation is to minimise the construction cost of a sewer system (including the cost of pipes and other materials, trenching, laying, resurfacing, etc) within specified limits. The limits include minimum depths of cover, maximum depth to invert, pipe diameters adequate to convey the calculated discharge without surcharge, and a minimum pipe-full flow velocity to prevent deposition of sediment.

The design discharges used in the optimisation are calculated by the Modified Rational Method (see section 13.1). The optimisation technique used is the Discrete Differential Dynamic Programming method (DDDP) described elsewhere [63, 105].

The method first assumes for each branch a set of soffit elevations forming a corridor: see Fig 12.2. The longitudinal profile closest to the ground is sought within the specified constraints on pipe cover and full-bore velocity and then adopted as the improved profile to form a new corridor. The width of each new corridor is reduced and the process continues until the width of the corridor is less than a specified value.

These calculations are performed for each branch in turn beginning with the branch with the smallest number of pipes. The process is then repeated starting with the branch with the largest number of pipes and at this stage the minimum cost solution is sought within all the specified constraints. The method may require several runs, with the user making adjustments to certain constraints, in order to obtain a viable solution.

Pumping of stormwater is avoided whenever possible and the Optimising Method is intended for the normal situation of continuous flow under gravity. In networks where a pumping station is unavoidable, depths, gradients and diameters may be optimised separately on each side of the pumping station, but joint optimisation of the pipe network and the pumping station parameters is not included. The effects of a pumping station on the passage of flows through a system may be modelled using the non-optimising Hydrograph Method described in sections 13.2 and 11.3. The approximate cost of a pumping station may be determined from information given in the report of the WRC study [97] of the costs of water supply and sewage disposal.

The optimising method has been tested by redesigning the sewer system in a catchment in Derby. It was found [80] that over a range of return periods, the cost of the optimised design was 11 to 15 per cent cheaper than designs based on the non-optimising Hydrograph Method.

12.5 Performance-cost evaluation

The design procedure includes a program which calculates the hydraulic behaviour of a system operating under surcharge. This program may be used to examine the behaviour of an existing or proposed system under rainfall conditions rarer than those used in the design. By comparing the cost and performance of alternative systems, a measure of cost-effectiveness is obtained. This is described as a 'performance-cost' approach.

Construction costs of alternative designs are obtained by one of the methods described in section 12.2. For convenience, costs are calculated within the procedure by the resource cost model used for the depth-gradient-diameter optimising method, but it must be remembered that the costs given by the resource cost model do not represent the total costs of a scheme.

The performance of a pipe system under rainfall conditions rarer than those used in design may be examined in one of three ways, listed below in order of increasing sophistication:

(a) By examining the frequency with which the surcharged flow in the system reaches ground level or a selected depth below ground level. The effects of flooding on the surface are not considered.

(b) By extending the calculations to allow for surface flooding, but giving attention only to the calculated volumes of flooding and not attempting to determine flood depths. In this way a comparison can be made between the costs of alternative systems and their performance in terms of flood volumes and their frequency of occurrence.

(c) By proceeding to calculate depths of flooding and to use these to determine the damage cost associated with various frequencies and alternative design standards. Since the cost of damage averted by constructing to a higher standard is properly regarded as a benefit, this is a type of benefit-cost evaluation.

The opportunities now available for the application of these methods of economic evaluation to the provision of storm drainage are examined in chapter 16.

TABLE 12.1 Special data requirements for the resource cost model

	Index no	Description
Site condition	1	Road
	2	Pavement
	3	Unpaved urban (eg gardens, verges)
	4	Open field
Ground condition	1	Rural (less than 1 buried service per 10m)
	2	Suburban (1 buried service per 2 to 10m)
	3	Dense urban (more than 1 buried service per 2m)
	4	Unstable. Requiring continuous support but not any specialised geotechnical process.

TABLE 12.2 Comparison of resource cost and tender cost models

	$\frac{\text{Actual tender cost}}{\text{Calculated tender cost}}$	$\frac{\text{Tender cost of resources}}{\text{Calculated resource cost}}$	$\frac{\text{Actual tender cost}}{\text{Calculated resource cost}}$
Mean value for 26 schemes examined	0.99	1.17	1.99
Standard deviation	0.27	0.33	0.70

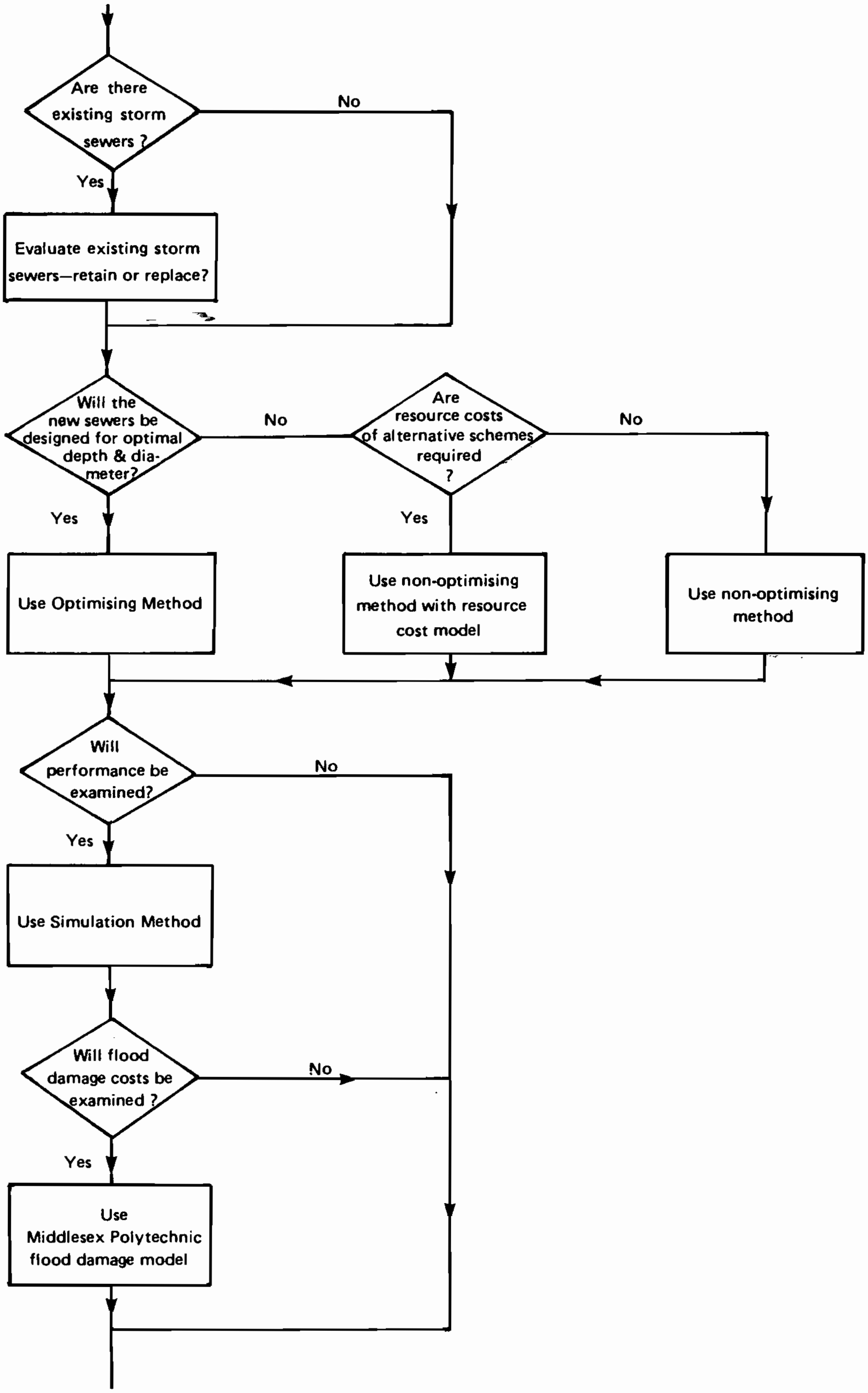

Figure 12.1. Decisions required in the use of the cost and damage models

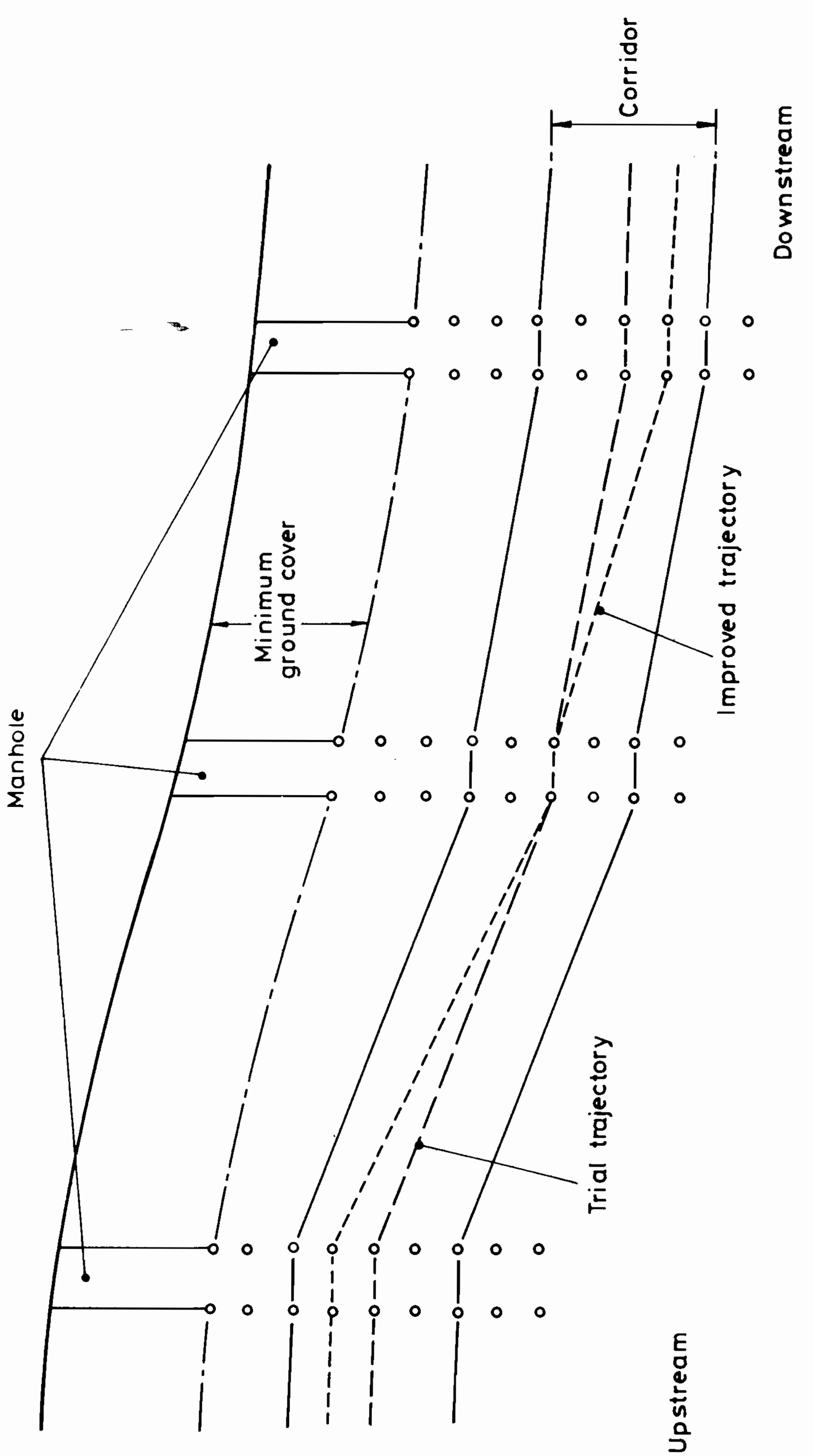

Figure 12.2. Sketch of trajectories and corridors (after Yen et al (105))

PART 3

Application of the procedure

CHAPTER 13

The recommended methods

THE WALLINGFORD PROCEDURE contains the following four methods, which may be used for analysis or design as indicated:

Modified Rational Method (for analysis and design)
Hydrograph Method (for analysis and design)
Optimising Method (for design only)
Simulation Method (for analysis only)

These methods are described below. Several of the calculation routines described in this report are used in more than one method; for example the methods for determining rainfall input described in chapter 6 are appropriate to all four methods, and the surface runoff modelling described in section 7.8 is used in both the Hydrograph Method and the Simulation Method. The way in which the detailed parts of the calculation procedure (or 'models') are linked together into the four methods is illustrated in Table 13.1.

Most of the engineering and scientific bases of the methods have been explained in chapters 6 to 12. This chapter therefore gives a more general statement of the calculation steps included in each of the four methods. The data requirements for the methods are summarised and compared in chapter 14, and chapter 15 gives guidance in the selection of an appropriate method for a particular application.

13.1 Modified Rational Method

The Rational (or Lloyd-Davies) method is in widespread use and provides a convenient and easily understood tool for design. Studies have shown [15, 55] the variable accuracy of the method, and recent research has suggested ways in which it could be improved. A Modified Rational Method has therefore been included in the procedure.

The method may be used either to size diameters of pipes for a specified return period of flow in a storm sewer system of given layout and gradients, or to estimate peak discharges in an existing system for given rainfall and catchment conditions. A computer-based version of the method, incorporating storm overflows, is included in the procedure, and the method can also be applied by hand calculation. The Hydrograph Method should be used if the system includes storage tanks or pumping stations, or if a flow hydrograph is required.

The Modified Rational Method gives a peak discharge from the equation:

$$Q_p = 2.78\, C_v C_R i A \qquad ......13.1$$

where Q_p is the peak discharge in l/s
C_v is the volumetric runoff coefficient

C_R is a dimensionless routing coefficient
i is a rainfall intensity in mm/hr
and A is the catchment area in ha.

The steps involved in the hand and computer versions are now described. A self-contained summary of the hand calculation is also available as Volume 4 of this report.

13.1.1 Hand calculation

1. Rainfall intensity-duration-frequency data for the location under examination are obtained, either from the Meteorological Office or by using the manual method described in sections 6.2 to 6.5.

2. The catchment network is divided into pipe lengths and the contributing area (either total area or impermeable area) associated with each pipe length is determined.

3. The percentage runoff PR is calculated from equation 7.3.

4. The coefficients C_v and C_R are determined. If total sub-catchment areas are being used,

$$C_v = \frac{PR}{100} \qquad ......13.2$$

If impermeable areas alone are considered,

$$C_v = \frac{PR}{PIMP} \qquad ......13.3$$

where PIMP is as defined in section 7.3.

A value of 1.3 is recommended for C_R (section 7.10).

Steps 5 to 10 are then repeated for each pipe in the network. If the pipe is being designed an initial diameter equal to the diameter of the largest in-coming pipe at the upstream manhole is assumed.

5. The cumulative catchment area is determined.

6. The pipe-full velocity is obtained from tables [43] and the time of flow along the pipe is determined.

7. The time of concentration is taken as the cumulative time of flow plus a time of entry (see section 7.10 for recommended values). The time of flow through long lengths of carrier pipe (ie pipes with no contributing sub-catchment) should not be included in the calculation of the time of flow for that pipe, but should be for pipes downstream.

8. A rainfall intensity corresponding to the time of concentration and the required return period is read from the rainfall data. When simulating an observed storm the required intensity is obtained from the observed hyetograph.

9. The peak discharge is calculated from equation 13.1.

10. If the pipe is being designed, the smallest available diameter which will convey Q_p is determined from design tables [43]. If this diameter is equal to the diameter originally assumed, the diameter given by the design tables is accepted. Otherwise, steps 5 to 10 are repeated using an alternative diameter.

11. Downstream of a junction the cumulative time of flow should be determined along the branch with the longest time of concentration. In exceptional circumstances this may lead to a lower calculated discharge than that from a major branch entering the upstream junction. In these circumstances the design discharge should not be reduced below the largest value entering the junction. This check may have to be repeated for several subsequent pipe lengths downstream.

13.1.2 Computer version with storm overflows

This version operates in a very similar way to the hand calculation and may be described in the following steps. Circular, egg-shaped or rectangular pipes or trapezoidal open channels may be used. Upstream of storm overflows the method follows steps similar to the manual method described above.

1. Rainfall intensity-duration-frequency data are derived within the program from a rainfall hyetograph (for simulating an observed event) or from the rainfall variables M5-60min, r, location index and return period. The same design rainfall data may be obtained from the Meteorological Office or calculated by the manual method described in sections 6.2 to 6.5.

2. Pipe network and contributing area data, and the values required to determine the percentage runoff, are supplied to the program. Steps 3 to 5 are then repeated for each pipe in the network.

3. The programme calculates the cumulative total and impermeable areas and the percentage runoff.

4. The program estimates a size for the current pipe, and calculates the time of flow, a cumulative time of concentration, a corresponding rainfall intensity and the peak discharge Q_p.

5. In design mode the program determines the smallest pipe size of specified shape to convey the peak discharge Q_p. If this size is greater than the initial assumption, steps 4 and 5 are repeated.

6. Downstream of a junction the times of concentration along the incoming branches are compared and the largest value is retained.

The following additional features are included for the representation of storm overflows.

7. If there is an overflow upstream of the current pipe, the program calculates the peak discharge from the total area upstream of the overflow using the percentage runoff and rainfall intensity determined for the current pipe.

8. The effective area contributing to the discharge continuing downstream of the overflow is calculated as a proportion of the total area upstream of the overflow (see section 11.1). This proportion is used to define the ratio of the continuing to the incoming discharge.

9. Revised values of the cumulative total and impermeable areas to the current pipe are then determined, together with the percentage runoff.

10. Steps 7 to 9 are repeated until there is only a marginal change in the percentage runoff.

11. The peak discharge, Q_p, for the current pipe is then calculated. In design mode an appropriate pipe size is selected.

12. In design mode, the program repeats steps 7 to 11 using the new time of concentration until there is no change in the design diameter.

13. Steps 7 to 12 are repeated for each pipe downstream of a storm overflow.

14. If the pipe receives the diverted discharge from an overflow the appropriate design discharge for that pipe is selected from two alternatives. The first assumes that the pipe is downstream of the overflow and the second regards the pipe as the top pipe in a branch. The larger of the two discharges is selected.

15. Other overflows upstream of the current pipe are dealt with in the same manner and are included in the calculations for the current pipe.

13.2 Hydrograph Method

This method is used either to size diameters of pipes for a specified return period of flow in a storm sewer system of given layout and gradients, or to calculate the discharge hydrograph throughout an existing system for given rainfall and catchment conditions. The method should be used in preference to the Modified Rational Method when storage tanks and pumping stations are present and when a discharge hydrograph is required. If there is extensive surcharging in a system it should be analysed with the Simulation Method.

As may be seen from Figure 13.1, the method is constructed from several of the models which have been described earlier. Further details may therefore be found in the relevant parts of earlier chapters. All the ancillary models described in chapter 11 (except surcharged outlets) and the construction cost model described in section 12.3 may be used in conjunction with the Hydrograph Method. Circular, egg-shaped or rectangular pipes or trapezoidal open channels may be included. The sewered sub-area model (chapter 10) may also be used; the sewered sub-area is treated as a single pipe length.

The basic steps followed by the Hydrograph Method are described below.

1. Design rainfall profiles are calculated within the program for specified values of duration, return period, location index and rainfall parameters M5-60min and r. The same rainfall profiles may alternatively be obtained from the Meteorological Office or calculated by the user according to sections 6.2 to 6.7. Summer profiles of 50 percentile peakedness should always be used. Profiles of 15, 30, 60 and 120 minutes are normally required for each return period examined. Any observed hyetograph may also be used. For either design or observed rainfalls the filter described in section 6.7 is applied.

2. Pipe network and contributing area data, and the values required to determine the percentage runoff, are supplied to the program.

3. The percentage runoff from the whole catchment is calculated and separate percentage runoff factors are deduced for the three surface types (see sections 7.4 and 7.5).

4. Each rainfall hyetograph is converted into the ten standard runoff hydrographs using the surface runoff model for the three characteristic slopes and the three characteristic paved areas, and for pitched roofs (see Figure 7.2). In each case the corresponding depression storage (if available) is subtracted from the rainfall before runoff begins.

5. The surface runoff hydrograph contributing directly to the pipe is calculated from the standard runoff hydrographs, the contributing areas and the percentage runoff factors (see sections 7.6 and 7.7).

6. The runoff hydrograph is added to the inflow hydrographs to the upstream manhole.

7. In design mode, the program determines the smallest available pipe size of specified shape to convey the peak discharge of the combined hydrograph at the upstream manhole.

8. The combined hydrograph is then routed through the pipe using the Muskingum-Cunge model for free-surface flow (see section 8.3). If the discharge exceeds the pipe-full capacity the routing continues as if the flow had a free surface.

9. A sewer ancillary structure may be modelled after the hydrograph for a particular pipe has been calculated. The program does not design such structures.

10. Steps 5 to 9 are repeated for each pipe length.

11. In design mode, steps 5 to 10 are repeated for another rainfall event with the same return period but with a larger duration. The largest discharge from all durations examined is taken as the design discharge at that pipe, but the hydrographs for all durations are retained for the continuation of the calculations downstream.

13.3 Optimising Method

This method is used to design pipe diameters and depths (and therefore gradients) for a minimum construction cost. The method should only be used if the pipe could be laid at a wide range of possible depths. Limitations imposed by other underground services and special ground conditions sometimes leave little or no choice in the selection of depths; in these circumstances gradients may be specified by the user and the Modified Rational Method or the Hydrograph Method should be used to design the pipe diameters. Only circular pipes may be designed with the Optimising Method.

Design discharges throughout the system are calculated from the Modified Rational Method. The longitudinal profile is then optimised (within constraints on depth and velocity) separately for each branch of the system to achieve the minimum construction cost along that branch. For this reason a true minimum cost is not obtained, but the method is designed to give a final cost as near to the minimum as possible. For a successful application the method may require several runs with the user making the necessary adjustments to certain constraints to obtain an acceptable solution.

The method consists of the following steps:

1. The rainfall and catchment data are supplied as for the Modified Rational Method (see section 13.1.2), except that pipe gradients are not required in the Optimising Method and additional constraints must be specified.

2. The program numbers the manholes in the system as defined by the order of the pipes, and for each manhole stores the number of the next manhole downstream. The reference number of the first pipe and the number of pipes in each branch are also recorded.

Steps 3 to 5 are then repeated for each branch in turn, beginning with the first branch with the fewest number of pipes.

3. Initial gradients and manhole depths are specified for each pipe along the branch taking care that the constraints on the minimum depth of cover are satisfied.

4. The sewer profile nearest to the ground surface is found using the Discrete Differential Dynamic Programming technique [63, 105]. Each pipe diameter is sized using the discharge calculated by the Modified Rational Method from the percentage runoff, total area and time of concentration to the downstream manhole. The constraints of minimum depth of cover at each manhole and minimum pipe velocity are respected, but no account is taken at this stage of the constraint on maximum depth nor of the cost of the pipes.

5. The minimum depths of cover at the manholes along the branch sewer are fixed as the difference between the ground and downstream soffit levels of the pipes.

6. The program then reconsiders each branch, but beginning with the branch containing the largest number of pipes. The minimum cost sewer along a branch is found such that the constraints on both minimum and maximum depths of cover and minimum velocity are satisfied.

13.4 Simulation method

This method is used to simulate flow throughout an existing sewer system for given rainfall and catchment conditions. Surcharging and surface flooding are included. Circular, egg-shaped and rectangular pipes may be included. Sewered sub-areas (chapter 10) and ancillary structures (chapter 11) may also be represented, but there are limitations to the way these additional features are modelled under surcharged conditions.

The construction cost model (section 12.3) may also be used in conjunction with the Simulation Method.

The Simulation Method should be used in preference to the Hydrograph Method in analysis mode when surcharging is extensive or surface flooding occurs.

The method uses the same rainfall and surface runoff models described above under the Hydrograph Method, but the pipe flow calculations differ in several important respects:

(a) In the Simulation Method the same flow equations are used but instantaneous discharges are calculated throughout a sewer system at a given time increment as opposed to complete discharge hydrographs being routed sequentially from one pipe to another. This permits the solution to allow for the interactions within groups of surcharged pipes.

(b) The Simulation Method allows for the storage of flood water both within manholes and on the ground surface above a manhole. The flooded area is assumed to be a specified percentage of the area contributing to the pipe length, and the flood volume is stored uniformly over this area. The relationship between the calculated flood volume and the severity of flooding in the particular area must be assessed by the local engineer. The user may specify either that the flood water returns to the same pipe after the flood event, or that it is lost from the system. The program does not allow for minor variations of level on urban areas, nor for the above-ground movement of flood water.

The Simulation Method is the only method in the procedure which can be used to make an assessment of the economic benefits of flood alleviation. As discussed above, the model calculates volumes and depths of surface flooding, although the depths may need to be adjusted by the engineer to take account of his knowledge of local surface topography. The time-sequence of flooding for specified rainfall events is also calculated. With this information the engineer can estimate the depth and duration of flooding of various return periods and can compare the relative performance and construction costs of alternative drainage systems.

The technique developed by the Flood Hazard Research Project at Middlesex Polytechnic[74,75] may be used to calculate the physical cost of flooding of various frequencies, and thus obtain a present value of the benefits of flood alleviation. Because of the human judgement needed in each local situation to convert volumes of flooding into depths of floodwater, the flood damage model has not been included in the package of programs. In most storm drainage design, the engineer will wish to ensure that, although the sewer system may surcharge, the surcharge heads will not rise above ground level for all reasonable return periods.

The Simulation Method may be summarised in the following steps:

1. Rainfall and catchment data are supplied to the program as for the Hydrograph Method (section 13.2), plus additional items relevant to surface flooding.

2. The program numbers the manholes in the system as defined by the order of the pipes and overflows, and for each manhole stores the number of the next manhole downstream. The program will re-order the manholes if necessary, to take account of the diverted discharge from a storm overflow or storage tank.

3. The percentage runoff from the whole catchment is calculated and separate percentage runoff factors are deduced for the three surface types (see sections 7.4 and 7.5).

4. The rainfall hyetograph is converted into the ten standard runoff hydrographs using the surface runoff model for the three characteristic slopes and the three characteristic paved areas, and for pitched roofs (see Fig. 7.2). In each case the corresponding depression storage (if available) is subtracted from the rainfall before runoff begins.

Steps 5 to 8 are then repeated for each pipe and each time step.

5. The instantaneous runoff discharges from the contributing areas to each pipe are introduced to the relevant upstream manholes.

6. Beginning at the first manhole a check is made on the status of the flow in the downstream pipe. The pipe is defined to be surcharged if the discharge at the previous time step exceeds the full bore discharge or a back-surcharge is generated by the water level in the downstream manhole. If the pipe is not surcharged the discharge at the downstream end of the pipe is calculated using the Muskingum-Cunge model for free-surface flow. Otherwise the pipe is regarded as the member of a surcharged group of pipes (which may include only the pipe itself) and is therefore stored until every pipe in the group is known.

7. When the program detects the last pipe in a surcharged group it calculates the

volumes stored in each surcharged manhole at the end of the time increment. New levels in the manholes and therefore discharges in the pipes are deduced from the volumes.

8. Having determined the flows in a group of surcharged pipes the program continues in the same manner checking and solving for the flow in the remaining pipes.

Table 13.1 Methods and models

Models / Methods	Rainfall models	Overland flow models	Pipe flow models	Sewer ancillaries models	Construction cost model	Flood alleviation benefit model
Modified Rational Method	Intensity - duration - frequency relationship	percentage runoff model + time of entry	Pipe full velocity	Storm overflow		—
Hydrograph Method	Rainfall profiles	Complete surface runoff model	Muskingum-Cunge	Storm overflow Storage tank Pumping station	TRRL resource cost model	—
		Sewered sub-area model may be used for selected sub-areas				
Optimising Method	As for Modified Rational Method	As for Modified Rational Method	Pipe full velocity	—		—
Simulation Method	As for Hydrograph Method	Complete surface runoff model	Muskingum-Cunge and surcharged flow	As for Hydrograph Method plus Tailwater level		Middlesex Polytechnic Flood Hazard Research Project Model (not included in programs)
		Sewered sub-area model (without surcharging) may be used for selected sub-areas				

CHAPTER 14

Data requirements

14.1 Data requirements for the four methods

DATA REQUIREMENTS for all four methods in the procedure are described in detail in Volume 2. A summary is given in Table 14.1. It will be noted that many data requirements are common to several methods, although the requirements naturally tend to increase for the more sophisticated methods.

Several of the items are optional; if specific data are not available default values are either specified or already incorporated in the programs. For several items global values (ie uniform values for the whole catchment) may be specified if local information for each contributing area or pipe length is not available.

It should also be noted that some data items are able to perform more than one function; for example if manhole cover levels at the upstream end of each pipe are supplied for the application of the Simulation Method, these levels may be used to calculate the surface slope of each contributing area; in this case it is not necessary to specify the surface slope type of each contributing area.

14.2 Required accuracy of data

The accuracy with which the programs are able to represent observed sewer networks or forecast design conditions will depend partly on the inherent reliability of the model procedures, and partly on the accuracy of the data supplied. The development of the models has concentrated on making the simulation of flow over the ground surface and in the sewer system as accurate and reliable as possible while preserving the efficiency of the computer programs and restricting the data collection and preparation to a minimum. The effect of uncertainties in the input data representing rainfall, contributing areas, the pipe network, pipe roughness, the location and operation of storm overflows, etc needs to be recognised and reduced where possible. (The engineer also needs to be aware that deviations from the design during construction may also lead to a loss of performance, but the control of construction procedures lies outside the scope of this report.)

The effect of variations in data input on peak discharges calculated by the Simulation Method were examined in sensitivity tests. The significant variables affecting the calculation of peak discharge are storm return period; percentage runoff; total area and its division between paved and roof areas; surface slope and gulley indicators; the length, slope and roughness of the pipes; and the manhole headloss coefficient. The tests led to the broad conclusions given in Table 14.2.

The selection of the appropriate storm return period is of prime importance in deciding on a final design (see chapter 16). Of the catchment and system data, percentage runoff and percentage paved and roof areas are the most important variables. Percentage runoff is calculated from a regression equation within the programs (see section 7.4) and the accurate determination of impermeable areas will improve the accuracy of this calculation. Impermeable areas are usually estimated to the nearest 10 per cent or better; input errors of 10 per cent cause an error in peak discharge of about 10 per cent. Pipe length can generally be measured to within 2 per cent and the corresponding effect on peak discharge is therefore significantly less than 10 per cent. Errors in specifying the surface slope, number of gulleys or area per gulley, and the headloss coefficient at manholes lead to comparatively smaller percentage errors in peak discharge.

The pipe roughness height can be regarded as a relatively unimportant variable in clean pipe networks carrying surface water only; in these circumstances the pipe roughness can normally be estimated to within a factor of two, and the effect on peak discharge will be negligible (see Table 14.2). In systems suffering from pipe sliming or siltation the possible range of roughness values is much greater (see Table 8.1) and the effect on peak discharge will be greater than that shown in Table 14.2.

The main conclusion is that improvements in the accuracy of the predictions of peak discharge of a given return period depend primarily on the effort put into collecting accurate data for the impermeable contributing areas. In existing systems with pipe sliming or sediment deposits, inspection of the condition of the network will assist in the selection of appropriate pipe roughness values. In these circumstances, flow measurements within the network under analysis may also be considered.

14.3 Field checking of sewer system data

The Wallingford procedure will frequently be used to analyse the performance of existing sewer networks. The records describing these systems are often incomplete, especially for the older combined systems. A particular problem is the presence of unrecorded storm overflows. If such features are not included in the sewer system data, the flow calculations will inevitably be erroneous; this could lead to incorrect designs or, if some site observations are available, to an unjustified undermining of confidence in the accuracy of the programs.

A technique of catchment-wide flow measurement can be used to detect the presence of overflows or other features which have a significant effect on the runoff process. Applications by the Water Research Centre [34] have demonstrated the feasibility of in situ flow surveys and the consequent improvement of computer simulations.

Table 14.1 Data requirements

Method	Rainfall	Urban Surfaces	Pipe Network	Sewer ancillaries	Costs
Modified Rational Method (for design and simulation)	Intensity - duration - frequency relationship or M5-60 min, r, location index and return periods or observed hyetograph Areal reduction factor (optional and for hand calculation only).	Total area of each contributing area Percentage of impermeable area in each contributing area or Total catchment area Impermeable area in each contributing area Time of entry (G) *Percentage runoff PR	Diameter of each pipe (or 2 dimensions of non-circular pipes) (simulation only) Gradient or Upstream and downstream invert or soffit levels of each pipe Length of each pipe Branch and pipe numbers Network layout Dry weather flow to each pipe (G) k_s of each pipe (G) Global value of minimum diameter (optional and for design only) Global value of minimum velocity (optional and for design only)	*Storm overflow* Discharge setting Proportion of excess discharge passing downstream	Can be included as an additional calculation Requirements as for Optimising Method plus cover level and soffit level at each manhole
Hydrograph Method (for design and simulation)	Rainfall profiles or M5-60 min, r, location index, durations and return periods or observed hyetograph	Total area of each contributing area Percentage of roof and paved surface in each contributing area or Total area of each contributing area Percentage of impermeable area in each contributing area or Total catchment area Impermeable area in each contributing area Surface slope type for each contributing area (D) Number of gulleys per contributing area (D) or Index of paved area per gulley *Percentage runoff PR	As for Modified Rational Method plus Number of manholes in pipe length (D)	*Storm overflow* Discharge setting Proportion of excess discharge passing downstream or maximum downstream discharge or modelled as an on-line tank *On-line tank* Plan area Level of base Level, length and discharge coefficient of weir Area and discharge coefficient of orifices *Off-line tank* As for on-line tank plus return pumping rate (optional) *Pumping station* Constant discharge, or plan area of wet well Design discharge, head and switch-on -off levels of each pump	As for Modified Rational Method

Table 14.1 Data requirements (continued)

Method	Rainfall	Urban Surfaces	Pipe network	Sewer ancillaries	Costs
Optimising Method (for design only)	As for Modified Rational Method	As for Modified Rational Method	As for Modified Rational Method plus Maximum and minimum permitted depths at upstream manhole (G) Cover level at upstream manhole Minimum velocity (G) Soffit level decision indicators (D) But excluding Gradient of each pipe Diameter of each pipe	Ancillaries not included	Ground index for each pipe (D) Surface index for each pipe (D) Current Baxter indices 1-3 and 5 (optional)
Simulation Method (for simulation only)	As for Hydrograph Method	As for Hydrograph Method, plus Percentage of each contributing area subject to flooding But optionally excluding Surface slope type for each contributing area	As for Hydrograph Method, plus Cover level at upstream manhole Head loss index for upstream manhole (D)	*Storm overflow* As on-line tank *Other ancillaries* as for Hydrograph Method *Outfall flap* level hydrograph at submerged outfall	As for Modified Rational Method

D indicates that a default value is provided within the program

G indicates that a single global value may be specified for the whole catchment

***Percentage runoff may be applied in one of three ways:**

(a) For design, calculated within the program from the soil index (SOIL) and the urban catchment wetness index (UCWI). The user obtains UCWI from the relationship with the standard average annual rainfall (SAAR) (Fig. 9.7).

(b) For simulation, calculated within the program from the soil index (SOIL) and the urban catchment wetness index (UCWI). The user calculates UCWI from the soil moisture deficit (SMD) and the antecedent rainfall index (API5) (see section 7.9).

(c) For either design or simulation, supplied by the user on the basis of alternative information.

The sewered sub-area model may be used in the Hydrograph Method and Simulation Method to represent any pipe length or lengths. In this case the data listed above under Pipe network are replaced by: Total length of major pipe run; average pipe slope; slope and diameter of outfall pipe; k_s (G); dry weather flow.

Table 14.2 Sensitivity of peak discharge to data values

Variable	Base value	Revised value or change in value	Percentage change in peak discharge
Storm return period	2 year	1 year	-20
		5 year	20
		10 year	50
Percentage runoff	45	40-50	±10
Percentage paved and roof areas	0 to 100	nearest 10 per cent nearest 20 per cent	10
Surface slope	mainly mild	all mild all medium all steep	1
Number of gulleys per pipe	1 to 10	1 gulley per pipe	1
Pipe length	3m to 120m	± 10 per cent	10
Roughness height	0.3mm	0.6mm 0.15mm	0.1
Headloss coefficient	0.2	± 10 per cent	2

CHAPTER 15

Selection of an appropriate method

OF THE FOUR methods in the procedure, three are suitable for the design of new systems and three for the analysis of existing systems or proposed designs. The Simulation Method is specifically intended for the analysis of systems under surcharged flow conditions.

	Design	Analysis
Modified Rational Method	✓	✓
Hydrograph Method	✓	✓
Optimising Method	✓	–
Simulation Method	–	✓

For both design and analysis, moving down the table above represents an increase in the detail of the calculation and the availability of additional facilities. These additional benefits are achieved at the expense of some additional data requirements (see Table 14.1) and computer running time. The user must determine the balance between the improved results and the additional costs of data preparation and computer running for the more complex methods. The following general guidance can be provided.

15.1 Comparability of methods

Different calculations are performed in each of the methods, therefore some differences in the results should be expected. The following similarities and differences will occur.

The Modified Rational Method is used for the flow calculations in the Optimising Method; calculated discharges should therefore be similar in these two methods, although the gradient optimising in the latter method will affect times of concentrations and consequently different design flows will be obtained. The magnitude of this difference will reflect the degree of change achieved by the optimised design as compared with the fixed gradients used in the Modified Rational Method.

The surface runoff calculations used in the Hydrograph Method are also included in the Simulation Method. The pipe routing method is also similar *until surcharging occurs.* Therefore similar results should be obtained in non-surcharged conditions. After the onset of surcharging, the Hydrograph Method continues to route the hydrograph as though free-surface flows still prevailed. The correct solution of surcharged flow conditions in the Simulation Method will usually mean that peak discharges are lower than those given by the Hydrograph Method, because of the throttling effect of surcharging. Additional storage will occur in the pipe system upstream, and this volume will be released as the flow recedes. Flow conditions downstream of a surcharged length will therefore appear very different in the Hydrograph Method and the Simulation Method. The latter is of course the more realistic representation.

Because of the compatability of these two methods in non-surcharged conditions, it is recommended that proposed systems which are to be analysed by the Simulation Method should be designed by the Hydrograph Method.

The Modified Rational and Hydrograph Methods have both been developed to give an optimal representation of available data *taken as a whole;* this does not mean that they will necessarily give similar results for individual storm events or in all circumstances. The Hydrograph Method, by modelling more of the physical reality of local catchment conditions, may well show a greater variation of results than the simpler Modified Rational Method.

Tests have shown that the Modified Rational Method is no less accurate than the Hydrograph Method in determining peak discharges (see section 4.5). However, for the reasons set out above it is recommended that its use should be restricted to catchments up to 150 ha in area with reasonably uniform slope and distribution of impermeable area.

The above considerations are also relevant to any comparison between the methods in the Wallingford procedure and other methods previously available. The results obtained from any method will be related to the processes represented and the modelling techniques used. At present there is insufficient evidence to say whether the methods in the Wallingford procedure will give flows which differ in a consistent way from those given by any other methods.

15.2 The design of new systems

The Modified Rational, Hydrograph or Optimising Methods may be used. (See Figure 15.1). The Optimising Method is specifically intended for the optimal design of pipe depth, diameter and gradient for minimum construction costs of a network. It has been reported [80] that cost savings of 10 to 15 per cent may be achieved by the use of this technique; however it may be inappropriate in situations where the longitudinal profile of the sewer is strongly determined by other underground installations. The gradient of each pipe does not have to be specified by the user, and this may represent a significant saving in data preparation. On the other hand, the maximum and minimum permitted depth and the cover level at manholes are required.

If the optimal design of pipe depth, diameter and gradient is not required, the choice lies between the Modified Rational and Hydrograph Methods. The Modified Rational Method is based on calculation of peak discharge only; if a hydrograph is required, the Hydrograph Method must be used. The additional data required by the Hydrograph Method are shown in Table 14.1.

Storm overflows may be incorporated in both the Modified Rational and Hydrograph Methods; storage tanks and pumping stations are only included in the Hydrograph Method.

Several methods may of course be used at different stages of a network design; for example the Modified Rational Method to give an immediate appreciation of catchment conditions and probable flows, the Optimising Method to design pipe gradients, and the Hydrograph Method for final design.

15.3 The analysis of existing systems or proposed designs

The Modified Rational, Hydrograph or Simulation Methods may be used to analyse an existing system (see Figure 15.1). The relative data requirements of these three methods may again be seen in Table 14.1. The Modified Rational Method has the limitation that it calculates only the peak discharge and not the complete hydrographs. The Simulation Method allows for the interaction of surcharging throughout a network, and therefore should always be used when surcharging is extensive and when surface flooding occurs.

Again, several methods may be used in the process of analysing an existing catchment: the Modified Rational Method to give an indication of probable flows, the Hydrograph Method to identify surcharged areas within the catchment and the Simulation Method to study the surcharged areas in detail. On a large catchment the iterative use of several methods in this way will probably be the most efficient way of analysing the network.

The purpose in simulating a proposed system designed by one of the design methods in the procedure is to examine the effects of surcharging under storm conditions rarer than the design event. Therefore the Simulation Method should always be used for this application.

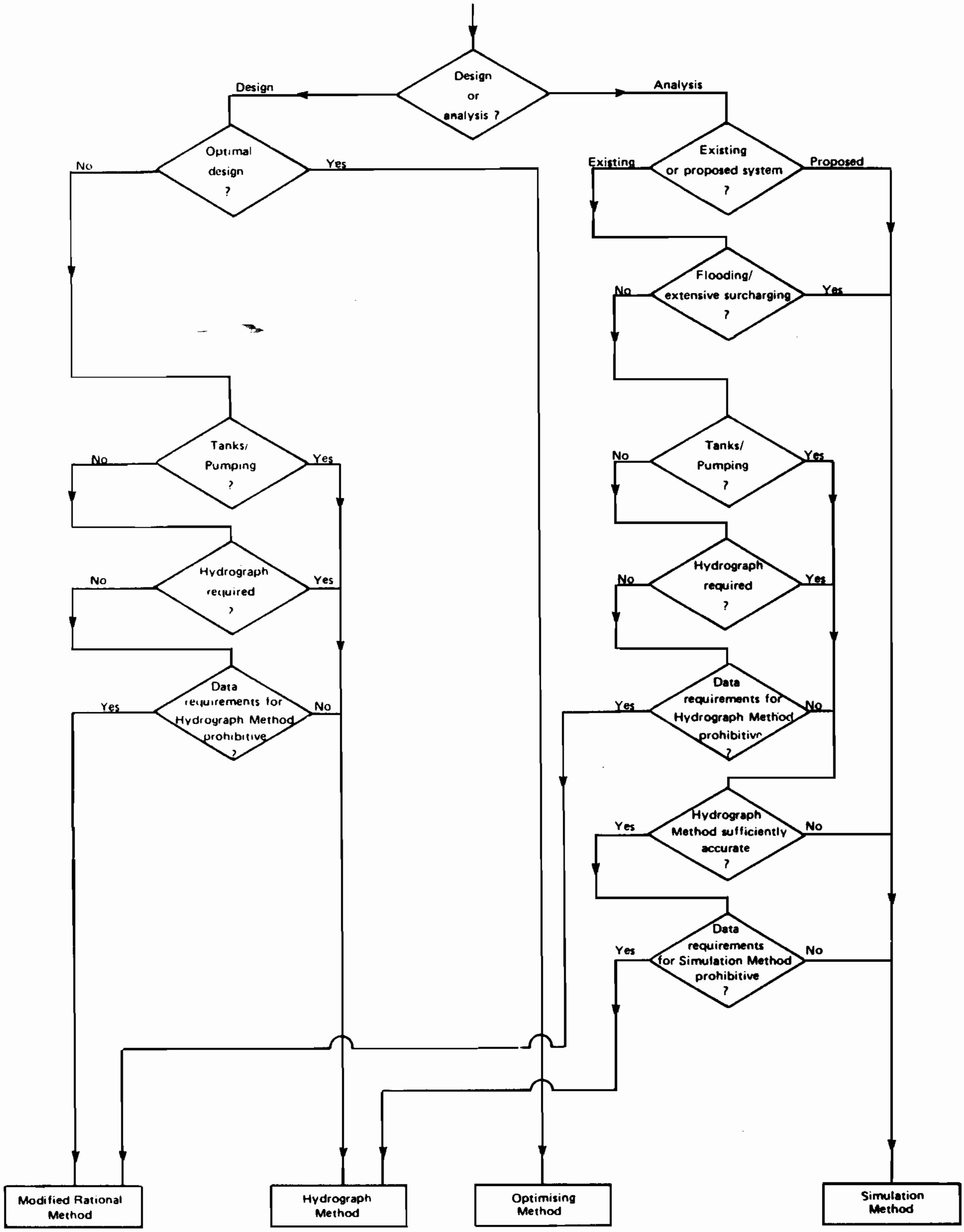

Figure 15.1. Selection of a method

PART 4

Design philosophy: A forward look

The previous parts of this report concentrate on the basis and development of the new design procedure and how it is used. This part is devoted to a discussion of possible changes in engineering philosophy and practice which may be implemented with the new procedure. In particular, part 4 considers the economic appraisal of storm drainage systems (chapter 16) and new approaches to the engineering design of drainage schemes which will alleviate existing or anticipated problems (chapter 17). The essence of this difference is that parts 1-3 of this volume answer the question 'What is the new procedure and how is it to be used?', while part 4 offers some answers to the question 'What approach should the designer adopt in the future?'.

CHAPTER 16

Economic appraisal and storm drainage

16.1 Introduction

IN THIS chapter the place of economic evaluation in storm drainage management is first discussed (section 16.2). The opportunities for applying economic evaluation in an ideal situation are then described (16.3), and problems militating against the ideal application are considered (16.4). It is concluded that, at the present time, a cost-benefit approach to the provision of storm drainage cannot be recommended for general application. However it does appear feasible to adopt a limited form of economic evaluation which has been termed a 'performance-cost' approach. After consideration of levels of service (16.5), the performance-cost approach is described in section 16.6. This approach includes the evaluation of alternative levels of service, by the design engineer or by the community or its representatives. Alternatively, the designer may revert to a minimum cost (or cost-effective) approach for an arbitrarily selected design return period (16.7).

The chapter ends with some discussion of investment timing in storm drainage (16.8) and energy conservation (16.9). An annex to the chapter gives a more detailed description of a full cost-benefit procedure, and provides some support for traditionally used design return periods.

16.2 The place of economic evaluation in storm drainage management

Economic evaluation as applied to civil engineering projects compares the consumption of resources (labour, money, materials and plant) with the production of goods or services thus created. Its objective is to provide the engineer with a means of judging the relative economic merits of schemes, so as to ensure that the available resources are allocated to maximum effect. Money values for costs and, where possible, for benefits are used only as convenient and readily appreciated yardsticks of the relative worth of different types of resources consumed and benefits generated by a project. Discounting techniques are used to take account of the different time patterns of expenditure and income of different schemes. A useful introduction to engineering economics has been published by the Institution of Civil Engineers [47].

The economic implications of design return period for rainfall or sewer flow have not always been emphasised sufficiently by the storm drainage engineer. The design of storm drainage will be improved if the processes of decision making are made more logical and objective, so that the real issue – the cost of providing a given level of service and the benefit thereby derived – is discussed openly in an accepted rationale. The Working Party considers that engineering economics provides an essential framework for these discussions, even though it cannot always yield a complete solution.

Traditionally storm sewer design has been based on the concept of the pipe-full flow produced by a rainfall event of a specified frequency – usually a one, two or five year return period for most urban situations. Seldom has any attempt been made to determine the capacity of the system to accommodate rarer and hence more intense rainfalls by taking surcharge into account. Hence the actual return period at which a sewerage system fails hydraulically by ejecting storm sewage from gulleys and manholes or by refusing to accept more surface water is usually unknown. It follows that there is little experience available to indicate what is the actual factor of safety against this form of hydraulic failure. Use of the Simulation Method will help to quantify this factor of safety and may show that previous practice has been over-cautious, or under-cautious.

It may be expected that a report of this nature should set out guidelines for a level of service that is based on a stated flood return period. The Working Party believes, however, that this would be unduly simplistic in view of the large number of parameters that have to be considered in any complete design, in which the choice of return period may be wholly or only partly dependent on cost-benefit considerations.

It is recognised that for design purposes the most convenient way of defining the hydraulic performance of any storm sewerage system is by reference to the return period of pipe-full flow (see section 8.1). A supplementary check on the performance of the system during rainfall events of greater intensity and lesser frequency and/or on the acceptability of occasional surface flooding can usefully be made.

Two aspects of the new procedure described in this report are particularly relevant to the engineer at the present time. The first is the ability to simulate the hydraulic conditions within sewerage networks under a wide range of differing conditions. This facility will be of considerable assistance to the sewerage system manager who will be better able to understand how the existing systems perform and how they should be operated and maintained to achieve the desired levels of performance. With a clearer understanding of the performance of existing systems the engineer should also be able to predict how extensions or alterations in different parts of a system may provide for the future drainage needs of the community. A second facility enables the designer to select a combination of pipe depth, gradient and diameter which represents the minimum capital cost solution. For the design criterion of pipe-full flow, the designer can examine the hydraulic and cost consequences of varying such parameters as return period, minimum velocity, minimum pipe depth, and so on.

For many years storm drainage systems have been designed using only simple techniques to compare alternative designs and it could be argued that this approach needs no improvement. However, the Working Party believes that the choice of design criteria and the level of protection to be provided require a more sophisticated economic evaluation, even though the available techniques have imperfections and their application to storm drainage is difficult. The process of economic evaluation imposes a framework of disciplined thinking both upon engineers and upon decision makers that is helpful in reaching fairer and more rational decisions about investment in this area of public enterprise. Such an approach is not new, as this type of evaluation is already implicit in the choice of a rainfall return period for design; for example, areas likely to suffer greater damage have been provided with systems designed for rarer rainfalls. However, the selection of return periods has been to some extent arbitrary and the efficiency of the resulting designs has been uncertain. The intention of the procedures proposed below is to place these choices on a more objective basis.

The Working Party believes that one of the hallmarks of the professional engineer is that he recommends projects only after careful appraisal of alternative solutions. While social, political and environmental factors must be considered, the engineer's overriding professional objective will be to ascertain the costs of providing different levels of service and to prepare reliable estimates of benefits. Sounder decisions can then be made with a better appreciation of the various consequences of the technically feasible alternatives. In the context of storm drainage, economic evaluation is concerned with the assessment of the cost and benefits to a community of a new or enhanced sewerage system designed to provide a particular level of performance. As outlined in the following sections, economic evaluation can assist the choice of the desired level of performance and will thus influence the capacity of the scheme actually constructed.

16.3 The application of economic evaluation: the ideal case

Before proceeding further it will be helpful to outline two distinct design approaches:

1. A scheme may be designed to perform a prescribed function or achieve an arbitrary

level of service at minimum total annual cost (ie the cost-effective approach). The benefits are assumed to be constant but unquantified in this type of approach.

2. A scheme may be designed with such capacity that, in addition to satisfying agreed technical criteria, the optimum difference between the benefits provided and the costs incurred is achieved. Approaches of this nature depend upon the acceptability of a rationale for judging costs and performance either in monetary units or against some other scale of values.

16.3.1 The cost-effective approach

In the past the cost-effective approach has been generally adopted in the field of storm drainage. Typical examples are schemes designed to cater for specified rainfall intensities or flows, or to a specified return period, at the lowest possible cost. If the designer wishes, the techniques provided in the Optimising Method may be used to identify a minimum cost scheme.

The designer may then go further and compare this cost with the minimum cost of other schemes which are designed to cater for runoff of different return periods or are of differing configurations. In this way it is possible to assess the difference in costs associated with meeting various levels of performance (a 'performance-cost' approach). Each alternative scheme considered on this basis will be 'cost-effective' in engineering content within the design philosophy. Two approaches on these lines are described in sections 16.6 and 16.7 below.

16.3.2 The cost-benefit approach

Extending the procedure to the cost-benefit approach calls for the inclusion of benefits in the analysis. In this approach the overall benefits of schemes of increasing capacity are examined and compared with their relative costs.

Rainfalls which cause flows exceeding the design capacity of a proposed scheme will lead through full-pipe flow to surcharging in manholes, and finally to street and property flooding, causing damage and distress. The assessment of future benefits from storm drainage rests on the proposition that averted anticipated damage represents a benefit. Damage which is caused by intense rainfalls which exceed the actual capacity of the system is termed 'residual' damage.

The relationship between rainfall frequency and surface flooding can be examined using the Simulation Method. It should be noted however that although surcharged lengths of sewer can be identified for various rainfall events, the probability of surface flooding can only be inferred from the probability with which the storm flows in excess of the capacity of the system are ejected or rejected at gulleys and manholes etc. The surface topography will determine the extent and significance of any flooding.

The reason for this limitation in the method arises from the discontinuity in the performance of the mathematical model when a quantity of storm sewage is ejected from the system.

A range of storm severities, each with an associated probability will, when flooding occurs, cause a range of damage costs. Given sufficient data it is possible to construct a curve relating damage costs to the probability of occurrence. The approach is illustrated diagrammatically in Figures 16.1 and 16.2. (A fuller exposition, including justification of the curves adopted, is given in the annex to this chapter and illustrated in Figure 16.4).

Examination of an alternative scheme with greater capacity (and higher capital cost) will show that the curve of the probability of residual damage costs is shifted downwards because the residual damages have been reduced or eliminated.

Any reduction in anticipated damage is properly regarded as the benefit obtained as a result of an increase in capital investment. The difference in area between the two curves of probability against damage costs therefore represents the level of benefit. A number of such alternatives can be examined in order to determine the optimum drainage development. Other things being equal, the preferred alternative will be that which shows the maximum net benefit (or, if costs always exceed benefits, that which shows the minimum net cost).

Flood damage and other associated costs fall into two categories: those which are tangible (or quantifiable) and those which are intangible (or unquantifiable). Tangible costs are usually described as direct (eg physical damage to property) and indirect (eg loss of business revenue).

Intangible costs include factors like ill-health through worry about a repetition of the event, and, as such, cannot be included in any numerical analysis. This means that inevitably there will be some element of subjectivity in deciding the total benefits arising from the reduction or elimination of flood risks. Problems imposed on economic evaluation by the intangible benefits are considered further in section 16.4.

The calculation of damage costs will be a new exercise to many engineers, but recent research has provided considerable assistance in this matter. The Flood Hazard Research Project at Middlesex Polytechnic has studied the costs of damage arising from the flooding of property of various kinds to different depths and has recently published a manual [74] containing the research results. The damage cost model was originally developed as an aid in the evaluation of river improvement works but it is equally applicable to flooding from storm water within an urban catchment. It is now possible to make a first estimate of the cost of damage to a given class of property for various depths of inundation. In cases of street flooding, most of the costs are incurred through loss of business and the costs of traffic diversion and delay. The method of use of the damage cost model is described in the annex to this chapter.

Sewerage authorities have tended to improve their drainage systems as soon as shortcomings in these systems have been observed. As a result the damage costs arising purely from deficiencies in an existing system are unlikely to produce net benefit-cost ratios greater than unity.

16.4 Problems in the application of economic evaluation

The discussion above represents an idealised approach to the application of the principles of economic evaluation. During the course of its investigation the Working Party found that there are factors which currently militate against a full application of the principles outlined above. These problems may be summarised under the following headings:

1. The importance of intangible benefits.
2. The difficulty of defining flood depths.
3. The enhancement of land values.

16.4.1 The importance of intangible benefits

The Working Party has found that there is a tendency on the part of the relevant authorities, both in the United Kingdom and overseas, to approve expenditure on storm drainage that may be much greater than the financial costs of damage it is intended to prevent. The Working Party has evidence of at least one case in which more was spent on a new sewer to prevent the flooding of a group of houses than the market value of the houses so protected. Such action is probably due in part to an understandable tradition on the part of drainage engineers who view any surface flooding by storm water as a 'failure' of their design and in part to the emotional reaction of the public at the appearance of any storm water in the streets. From information available in the UK, it seems unlikely that flooding caused solely by inadequacies in storm drainage systems contributes any significant risk to human life, compared with flooding caused by high river floods or abnormally high tides. If the opinion of the Working Party is correctly based, the inescapable conclusion is that many storm drainage systems may be oversized when viewed solely on an economic assessment of the actual incidence of flood damage. The implication of this is that intangible benefits and 'political' factors play an important role in determining the size and capacity of storm drainage systems.

16.4.2 The difficulty of defining flood depths

A further difficulty makes a general adoption of benefit assessment problematical at present. This is that there are no methods currently available to calculate the exact area and depth of flooding caused by a surcharging sewer. The Simulation Method in the Wallingford procedure calculates a volume of surface flooding and, from an area specified by the user, determines an average flood depth. From work instigated by the

Working Party with the cooperation of Wessex Water Authority and Bristol City Council it is believed that an engineer with detailed local knowledge can make a fair estimate of the areas and depressions in which excess storm water will collect, and so assess the costs of any damage which may ensue. This suggests that a cost-benefit approach to the selection of design parameters could, in principle, be introduced for appropriate cases but that this approach would only involve first approximations. Furthermore, such an analysis could be costly to carry out. Nevertheless the approach may be useful in avoiding the unwarranted allocation of capital to protect a few properties at substantial cost.

16.4.3 The enhancement of land values

The Working Party recognises that land values are enhanced when utility services including sewerage are provided. It is however concluded that the increment in value attributable to storm sewerage cannot be separated from the other infrastructure elements; furthermore, some double-counting could be incurred by the consideration of both land values and sewerage construction costs. For these reasons engineers cannot be expected to allow for any increase in land values.

Because of the above problems, the Working Party cannot recommend the detailed cost-benefit approach for general adoption at this time. However, a limited form of economic evaluation which has been termed a 'performance-cost' approach does appear to be feasible, and is outlined in section 16.6 after a discussion of levels of service in section 16.5. The simpler cost-effective approach is described in section 16.7. Where complex problems do exist and the cost-benefit approach is seen as the appropriate method of analysis, it is recommended that an economist with experience in this field should be appointed to the design team.

16.5 Levels of service

Before explaining the basis of the 'performance-cost' and 'cost-effective' approaches, both of which involve the application of the concept of a 'level of service', this concept itself is discussed.

At the time of preparing this report there appear to be no nationally accepted levels of service for urban storm sewerage within the United Kingdom and little consensus on how such a level of service should be defined. In an extensive survey [102] of UK practice in 1974, the Working Party found that design rainfall return periods varied from one to 100 years. The most frequently used values were one year (in 65 per cent of replies which quoted return periods), five years (20 per cent) and two years (17 per cent). The percentages total more than 100 since many replies gave more than one value.

Any definition of acceptable levels of service will almost certainly involve an understanding of and a willingness to accept the statistical nature of risk when providing for recurring natural events. This may be widely misunderstood by those who are unaccustomed to probability analysis and who may seek assurances in absolute terms. It must also be recognised that the exposure to risk will be related to the topography of individual sites.

Most statutory authorities must seek to establish a broad measure of equitable treatment for their consumers, whether they are dealing with existing problems or providing new facilities. In considering storm drainage systems it is relevant to examine the nature of the level of service and the consequences of failure to meet these levels for the types of property involved. Table 16.1 sets out these relationships in increasing order of the seriousness of failure of the system to perform (ie of disbenefit).

It must be noted that receiving water courses are akin to a 'type of property' enjoyed in common and regard must be given to the consequences of failing to meet an appropriate level of service for discharges of storm water.

As noted in section 8.1, failure in the level of service is not necessarily related to failure of the physical structure of the pipe network or its ancillaries. However, structural failure may well reduce the capacity of a pipeline. The physical and structural condition of the pipe-barrel, the joints, the surrounding and supporting earth, the degree of siltation, etc, are all criteria that will affect the hydraulic performance of the system but the establishment of standards of sewer maintenance lies outside the scope of this report. Flooding is sometimes the result of pipe blockage (from many possible causes) but while maintenance will be in the mind of the system designer it is not usually a consideration in the economics or computation of pipe sizes in relation to flood frequencies. An exception to this approach may occur in areas of very flat gradients where an allowance for siltation, based on experience, may be introduced. Where necessary the designer may have to specify the level of maintenance required to keep the system in a serviceable condition and retaining its design capacities.

It will rarely be possible to increase the capacity of the system beyond that determined by its original physical parameters, and it is the initial selection of this capacity at the design stage that imposes the ultimate level of service available throughout the lifetime of the system. The capacity of an existing rough pipeline can in some circumstances be increased by the insertion of a smooth lining.

Factors influencing decisions on the size of sewers are summarised in Table 16.2, from which it will be appreciated that such decisions must involve judgements of a social and financial nature that are beyond purely technical considerations. For this reason the Working Party considers it realistic for designers and those ultimately responsible for decision-making on behalf of the community to take account of a notional concept of an 'equitable level of service'.

This 'equitable level of service' is conceived as that provided for the general good at a cost which reflects all the circumstances (topography, age of property, residual capacity in existing systems, etc) but which does not otherwise discriminate unfairly in the overall utility provided for different groups, classes or individuals. The collective benefit, including the protection of public health, provided to the community by sewerage systems is very large. In these circumstances small differences in the levels of service provided by different systems, or by different parts of the same system, are accepted as normal by the consumers.

The acceptable 'level of service' may vary from area to area and, probably, with time. It will be one of the most difficult tasks of the drainage engineer in future to engage the attention of the community in order to explain to them the consequences of different proposals and to obtain a view that will enable the professionals to construct to levels of service which command general support. This will mean assessing the relative importance of the items listed in Tables 16.1 and 16.2.

16.6 The choice of return period by a performance-cost approach

In section 16.4 it was concluded that a detailed cost-benefit approach to the provision of storm drainage could not be recommended for general use at present. However, it is believed that the techniques of economic analysis lead to better value for money and should be accepted as part of the design and management of storm sewerage.

The performance-cost approach involves the calculation of the hydraulic performance and associated construction costs for systems designed to a range of alternative return periods. The methods described in this report allow these calculations to be performed with an ease which has not previously existed, and, with experience, it should be possible to analyse more alternatives without incurring large design costs. The drainage engineer can then explain the outcome of these investigations to the representatives of the community of the area concerned, so that a decision can be taken on the level of service (as explained in section 16.5) for which the community is prepared to pay. Initially there may be difficulty explaining that the proposals entail some risk of flooding, but it is considered worthwhile to persevere with this approach. The general public is used to assessing the risk of different types of event even though it may not attempt to quantify them in terms of exceedance probabilities.

Where performance can be assessed only in terms of return period it is reasonable to look at what is the normal extra cost (percentage) to be paid for extra capacity and to select a larger or smaller capacity depending on what is the preferred capacity in relation to anticipated needs. Engineering approximations can be made in order to produce a relationship such as that shown in Table 16.3 and Figure 16.3. It must be emphasised that the relationship is only approximate and engineers should develop their own data for a particular case being examined.

Although such a 'performance-cost' approach does not attempt a full economic analysis, it represents a real advance on present design practice which has tended to adopt a single design criterion selected arbitrarily.

16.7 A cost-effective approach to design for a specified return period

If it should prove impossible to adopt the approach described in section 16.6 above, whereby a range of schemes meeting various levels of service is analysed and costed and the community given the opportunity to influence the design standard (or level of service), it is suggested that the straightforward 'cost-effective' approach should be adopted. This takes its starting point as a pre-determined level of service, usually the probability of occurrence of pipe-full flow. The designer will normally choose this on the basis of past practice and his knowlege of the character of the area to be drained.

Alternative designs, which may be produced by the Optimising Method, can be evaluated using the construction cost and performance data for each alternative. Value judgements may still be required in comparing the advantages and disadvantages of different physical configurations in relation to their respective costs. The random effect of pipes being sized upwards to the nearest manufactured size available produces distortions which may also need to be assessed when considering particular cases.

To assist the selection of design return periods, a decision matrix of the type shown in Table 16.4 should be prepared. The table clarifies the catchment conditions which have to be considered when selecting design return periods. The actual values of return period will have to be supplied by the drainage authoritiy on the basis of local experience and practice.

Clearly the use of such a matrix presupposes a willingness on the part of sewerage authorities that levels of service should be established for all appropriate situations. Designers should be guided by the considerations shown in Table 16.4 in deciding how far economic appraisal should be pursued in individual schemes.

16.8 The timing of investment

While it may be essential to provide capacity in the design of a sewerage system for effectively draining the area under consideration, provision for future development and indeed the rate of that development will have an effect on the most economical design.

When it is expected that a catchment will be developed further the drainage engineer will have to decide whether to design a system of the capacity to match the flood flow for the anticipated eventual development. If such a system is constructed its capacity will at first correspond to a flow of considerably higher return period. Putting this another way, the risk of failure (however defined) will be low initially and will gradually increase as the years go by. The engineer will have to forecast, in conjunction with the planning authority, the rate at which each development will proceed. Clearly there will be a high degree of uncertainty about the actual rate at which the apparent level of service will deteriorate. The continued expansion of the development will usually be modified by a degree of redevelopment which, in extreme cases, will include the total replacement of the existing engineering infrastructure. This type of situation can be described in terms of changes to the risk of failure, although it may be more convenient for some purposes for the engineer to think in terms of a 'spare' hydraulic capacity.

Associated with the determination of actual risks of failure at any stage in the development of the catchment are the following points:

(a) The concealed and random additional capacity that arises in the selection of pipe sizes from those that are commercially available.

(b) The influence of a design philosophy advocating that diameters shall not decrease along the direction of flow in the system.

It is recommended that a discounted cash flow evaluation should be carried out before including surplus capacity within the system where this capacity will only be required in the period beyond a planning horizon of twenty years. This probably accords with the views of many that it is unreasonable deliberately to contemplate further works within ten to fifteen years of the inception of any scheme.

One particular advantage of the Simulation Method is that it will enable a more accurate forecast to be prepared for the timing of investment and will enable the risks involved to be better appreciated. The steps involved could be as follows:

1. Design the new system according to the current parameters laid down for return period or level of service allowing for any inputs from outside the study area on the same design basis.

2. Test the liability of the existing system to flooding using the Simulation Method. Identify problem areas within the system each with its capacity and related return period.

3. Assess the damage costs for alternative designs.

4. Using discounting techniques, compare damage costs with potential savings resulting from deferred investment.

5. If the timing of development upstream of the proposed works is uncertain, the assessment can be repeated to show the effect of the unused capacity on the benefits and financing of the project.

During design an assumption will have been made about the life of the system, given reasonable maintenance. The accuracy of such assumptions will improve as additional information on sewer lives is accumulated. Decisions on repair, renewal or replacement are made on the basis of the actual condition of the system at any particular point in time. The designer has to assess:

(a) The future growth in storm flows.

(b) The capital cost of different sewer construction strategies to meet growth (all to specified return periods).

(c) The cost of future maintenance commensurate with the sewer construction strategies considered in (b).

(d) The cost of flood damages associated with each strategy.

The total costs of each strategy can then be compared and a decision made on the appropriate form of construction. Note that in this analysis the designer is considering the future and he needs to relate anticipated maintenance expenditure to the remaining life of an underground asset.

16.9 Energy conservation

Problems of energy conservation are relatively new in the appraisal of engineering projects other than as part of the assessment of running costs. In the past the designer usually contented himself with the consideration of the relative merits of gravity or pumped schemes but now it is probably desirable to consider the overall energy consumed in pumping, in treatment, in transport and in manufacturing the materials and in constructing the works.

The initial costs of these items will reflect the cost of the energy component, but a separate assessment should be made of the effect on the economics of an increase in energy cost greater than the general level of inflation (ie reflecting the increasing scarcity of petroleum based products). This would involve considering factors such as:

(a) The number and rating of pumping stations.

(b) The effects of balancing on pumping costs.

(c) Storm water overflow policy and the requirements regarding the pollution of receiving waters.

(d) The maintenance and operation of pumping stations, pipe systems, flood meadows, storage tanks etc.

The Working Party believes that the significance of energy conservation will be of greater importance in the future and designers should be encouraged to include energy assessments in the appraisal of projects in order to improve and standardise the assessment techniques as well as improving the level of conservation.

Annex: Evaluation of damage costs

(1) Graphical technique

In section 16.3 a general explanation was given of the relationship between storm severity and consequential flood damage. Figure 16.1 illustrated the concepts involved. The purpose of this Annex is to explain further the detailed implications.

A more detailed version of Figure 16.1 has been developed in Figure 16.4. The following notes explain the curves used in the four parts of the figure.

Curve A is based on columns (1) to (3) of Table 16.3, where the basis of its derivation is explained. The curve trends asymptotically towards the horizontal axis, but for practical purposes a cut-off at a very low probability may be applied.

Curve B has been constructed from the following assumptions:

(a) Any flow above the design flow for a system is ejected to form a flood flow.

(b) Flood depth may be derived from flood flow by an overland flow relationship

$$h \alpha q^{2/3} \qquad \text{......16.1}$$

where h is depth and q is discharge (see section 7.7).

The figures plotted in curve B were therefore obtained from:

$$\text{Flood depth } \alpha \left(\frac{Q_{d(T)}}{Q_{d(1)}} - \frac{Q_d}{Q_{d(1)}} \right)^{2/3} \qquad \text{......16.2}$$

where Q_d is the design flow of any specified return period.

Assumption (a) neglects the effects of

(i) pipeline capacity greater than the design flow, due to the use of the next larger available pipe size; and

(ii) pipeline capacity modified by the effects of surcharge heads along the pipeline.

In any particular catchment these effects could be significant.

Because of assumption (a) the curves in graph B intercept the horizontal axis vertically above the equivalent points in graph A. This implies that the occurrence of any flow marginally greater than the design flow will initiate flooding. In practice the curves in B would probably be displaced to the right by a distance depending on local catchment geometry.

Assumption (b) regards the flood water as flowing freely over the land surface. In practice, ponding on the surface may cause a different relationship between excess discharge and flood depth.

For these reasons, graph B can only be used for general illustrative purposes.

Graph C is a representative shape based on extensive data [74].

Graph D is constructed by linking the plotted data on annual probability, flood depth and damage cost, as shown by the example line.

When the damage costs and their relative probability of occurrence are combined in Graph D, the area under each curve gives the expected annual damage cost for the aggregate of all storms of varying probabilities in a system of that particular capacity. It can be seen that the high damage costs have low probabilities and the high frequency events have low cost effects. The moderately frequent events with moderate damage costs appear to have the greatest influence on the areas and so upon the total damage cost.

This information can be further developed to show the relationship between the total costs (sewer cost and damage cost) incurred by systems of different capacities. This has been done in Figure 16.5.

The generalised shapes of the damage cost and sewer cost curves are broadly valid but their scale relationship will not normally be known. The following notes explain the assumptions made in constructing Figure 16.5.

(a) The sewer cost curve was derived from column 4 of Table 16.3, and is expressed in annual cost terms.

(b) The annual damage cost curves were derived from Figure 16.4(D). For comparative purposes a range of four damage cost curves (representing alternative assumptions about the level of flood damage cost) has been shown. Experience suggests that annual damage costs caused by excess storm runoff are low in relation to the annual (amortised) cost of the provision of storm sewers. If sewers cost about £1000 per property, the annual amortisation based on a rate of five per cent per annum will be about £50 per property per annum. However, the annual damage cost incurred in sewer catchments designed against a one year rainfall is (from common knowledge) much less than this figure of £50 per property per annum. Therefore, the damage cost curves on Figure 16.5 must normally be well below the sewer cost curve. This suggests that the damage cost curves 1 and 2 in Figure 16.5 are more realistic (in their relation to the sewer cost curve) than curves 3 or 4.

The optimum design will be that for which the total of the damage cost and sewer cost is a minimum. Hence the cost totals are plotted to show a range of minima depending on the proportion of flood damage cost to sewer cost.

Certain general conclusions can be drawn from this approach. Leaving aside the unquantifiable factors (the intangibles described in Table 16.2), Figure 16.5 shows that for the lowest assumed annual damage cost (curve 1) the optimum design return period is less than one year. Moving to higher damage cost curves (2, 3 and 4) shifts the minimum total cost towards the left, demonstrating that a larger sewer is justified.

The next logical step is to consider whether a storm drainage system should ever be designed so that the capacity is less than that necessary to contain flows of a one year return period. A decision that it should never be so designed, which may seem reasonable, is equivalent to insisting that the lowest sewer-plus-damage cost should never occur at a return period of less than one year for a new system of any normal cost.

Considering next the economics of designing to slightly larger return periods (say two to five years) it can be seen that the minimum total sewer-plus-damage cost curves are flat in this part of the diagram. This means that the economic minima are poorly defined in this area, that is, that the optimum design is not sensitive to return period over this range. It might then be held that if once in one year is a normally acceptable rainfall design return period to give pipe-full flow, only identifiably higher than normal flood damage consequences or risks will justify the extra capital cost of providing systems with a capacity larger than this.

It must be stressed that there are difficulties in drawing firm conclusions from the broad generalisations that have to be made using representative data. Data derived from specific schemes might not be suitable for determining general trends because of local catchment effects and because of the use of standard manufactured pipe sizes; this can cause distortions when trends are being considered. An attempt has been made in this analysis to produce generalised information but designers must satisfy themselves on this subject using any data they may have or can obtain. Nonetheless there seems evidence from economic theory to suggest, as shown above, that the current practice of using return periods of one to five years for pipe-full design in routine cases has some justification.

(2) The flood damage model

The flood damage model prepared by the Flood Hazard Research Project at Middlesex Polytechnic is not included within the package of programs, but some information is given here to enable the user to consider its appropriateness to particular storm drainage problems. Further information is given in references 74 and 75.

The model comprises two components. First, there is depth/damage data for various types of properties (a few of which are given in Table 16.5) and second there is a procedure for totalling the damages and converting these into a discounted annual benefit figure.

The data on flood damages were compiled for five main types of property:

Residential dwellings
Professional office premises
Retail trading and related premises
Agricultural buildings
Manufacturing and extractive industries

In the case of the first three, data on damage costs were obtained by first ascertaining the likely average characteristics of typical shops, offices and residential properties, mainly from national census information, large retail companies, market research organisations, insurance companies and members of the Chartered Institute of Surveyors. Having obtained this information on average characteristics, each typical property was assessed for potential flood damage for two durations and fifteen depths by loss adjusters experienced with assessing flood damage in these types of properties. This assessment was made for each of 52 inventory items and for 15 building fabric components. In the cases of agricultural buildings and the manufacturing and extractive industries, average damage costs were derived from site surveys of flood-prone locations backed up with surveys of premises flooded in the past. The industrial data should be used with caution as they are derived from a limited geographical area and may not be nationally applicable as are the other data.

The computer model to calculate discounted annual average benefits relies on a detailed land use survey of the flood-prone areas to identify the types of property present to which the standard depth/damage data can be allocated, the heights of each of the ground floors of these properties (or the height at which damage would begin, whichever is the lower) together with their grid reference, and the return periods of floods of different depths and/or extents. In addition, the disruption of traffic caused by the residual flooding should be costed. This is likely to be difficult but may be a large component of the benefit of designing to a higher standard. Both the costs of delay and the marginal costs of traffic diversions should be established following the methods detailed elsewhere [58, 74].

The proportion of traffic disruption costs to total costs can vary widely. In a case study of river flooding in Pulborough cutting the A29 trunk road the traffic disruption and delay costs were £13,192 out of a total of £90,816 (14.5 per cent). In another study in Bristol involving storm sewer surcharge cutting the main A38 trunk road the figures were £28,000 out of a total of £2,810,199 (0.9 per cent).

The computer model is flexible so that the data inputs can be manipulated to suit specific requirements. The effects of different flood durations and warning times on damage costs may be examined, the discount rate may be varied, and costs may be updated to allow for inflation and other price changes.

Table 16.1 Consequences of failure of storm drainage

Type of property	Nature of risk	Consequences of not meeting level of service
Highways	Surface flooding	Increasing loss of amenity
		Splashing of pedestrians
		Delays to traffic
		Traffic accidents
		Structural collapse of road surface and supporting structures together with the secondary effects of further delays and damage to vehicles and property
Domestic properties	Flooding with storm sewage or surface water	Increasing loss of amenity
		Difficulty of access
		Damage to floor coverings
		Damage to furniture
		Damage to decorations
		Structural damage
		Risks to health
		Risks to life
Commercial and industrial premises	Flooding with storm sewage or surface water	Inconvenience to users
		Difficulty with staff and vehicle movement
		Direct damage to stored goods and materials
		Damage to plant and machinery
		Consequential losses to workers/consumers through loss of production or stock
		Serious structural damage
		Risks to health
		Risks to life

Table 16.2 Factors influencing the size of surface water sewers

(1) Factors tending to make sewers smaller	(2) Factors tending to make sewers larger
(a) Financial savings from not having additional protection against flooding	(a) Financial savings from having less flooding and less flood damage
(b) Consideration that the extra capacity and hence the marginal utility of a larger sewer may be used only occasionally	(b) Reduced risk of distress, inconvenience or disease to people who may be affected by flooding
(c) Statutory requirement to provide effectual drainage but only at 'reasonable cost'	(c) Less political embarrassment
(d) The professional responsibility not to be over-cautious but to have regard to normal professional practice	(d) Less professional engineering embarrassment
(e) On trunk sewers, the avoidance of over-sizing to allow for future development which may not take place	(e) On trunk sewers, allowance for future development where future augmentation of sewer capacity may be difficult or costly
(f) A short estimated life for the system	(f) Long projected 'in-service' life for the system

Note: Only item (a) may be evaluated in money terms. The other items have to be classed as intangibles.

Table 16.3 Approximate effect of design return period on storm sewer costs

(1) Return period (years)	(2) Annual exceedance probability	(3) $\frac{\text{Design flow}}{\text{1-year flow}}$ (Q_T/Q_1)	(4) $\frac{\text{Sewer cost}}{\text{Cost for 1 year return period}}$ $(Q_T/Q_1)^{3/8}$
1	0.63	1.00	1.00
2	0.39	1.27	1.09
5	0.18	1.56	1.18
10	0.095	1.92	1.28
20	0.05	2.27	1.36
50	0.02	2.70	1.45
100	0.01	3.17	1.54

Notes:

Column 2 was obtained from the formula

Annual exceedance probability = 1 - exp (-1/T)

where T is the return period in years

Column 3 was derived by assuming that the increase of storm sewer flow with return period is the same as the increase of rainfall with return period. The figures used were typical figures for 1-hour rainfalls in England and Wales (see Table 6.2).

Column 4 was derived from the approximations:

cost α diameter

and diameter α (design flow) $^{3/8}$ (Manning formula).

Table 16.4 Decision matrix for selection of design return periods

	Design return period for stated level of risk		
	Normal	**Intermediate**	**High**
Flow condition	Normally sloping terrain Normal surface flood escape routes No foul sewage present Normal property value Normal road use	Intermediate cases	Terrain liable to ponding Few surface flood escape routes Foul sewage present High property value High road use
Pipe-full (where the next condition is manhole-full not basement flooding)			
Manhole-full (Surface flooding starts by discharge or rejection of flow)			
Property flooding starts (Sewer system and surface channels full)			

Note: Appropriate return periods to be filled in by the drainage authority

Table 16.5 Potential flood damage to dwellings, shops, retail services and industrial premises (£1977 prices)

	Dwellings								
	All	**Detached**				**Semi-detached**			
Depth of flooding (m)	**All**	**Pre-1918**	**1918-1938**	**1939-1968**	**Post 1968**	**Pre-1918**	**1918-1938**	**1939-1968**	**Post 1968**
0.3	908	1710	1070	1590	1230	821	798	821	825
0.2	629	1240	735	1070	854	575	545	554	551
0.1	338	569	387	574	485	335	305	303	275
0.05	212	453	283	311	149	240	215	212	137
≤0.0	104	133	147	155	13	142	113	113	36
		Terrace				**Bungalow**			
0.3		772	841	207*	930	1110	1490	1550	1750
0.2		572	611	135*	608	753	1050	1080	1230
0.1		365	369	106*	282	387	475	495	605
0.05		283	287	74*	139	213	281	218	234
≤0.0		186	175	36*	98	110	116	55	54

*Town house

Table 16.5 (continued)

Depth of flooding (m)		Flat			
		Pre-1918	1918-1938	1939-1968	Post 1968
0.3		1000	1040	642	1080
0.2		672	678	491	704
0.1		308	312	313	326
0.05		135	165	191	143
≤0.0		39	43	109	32

Other land uses

	Retail shops**					
	All	1	2	3	4	5
0.3	35.5	27.6	34.9	41.1	72.2	51.8
0.2	21.2	17.3	22.4	18.1	51.4	44.1
0.1	6.5	4.5	10.5	9.8	5.0	21.6
0.05	1.8	1.4	2.8	1.8	1.1	8.1

1 Food shops
2 Clothing and footwear
3 Household goods
4 Other non-food goods
5 General stores

	Retail Services/offices**						
	1	2	3	4	5	6	7
0.3	24.9	12.3	25.0	22.9	32.6	35.9	32.1
0.2	24.9	12.3	12.5	14.5	18.2	35.9	15.3
0.1	14.9	4.7	12.5	11.6	15.3	15.1	15.3
0.05	14.0	1.7	2.2	4.1	10.9	1.9	12.4

1 Public houses
2 Cafes and restaurants
3 Other
4 All retail services sampled
5 Professional office (all)
6 Banks
7 Commercial offices

	Industry related services**			Manufacturing industry**		
	1	2	3	1	2	3
1.5	50.2	35.7	21.2	52.4	39.5	26.6
0.3	11.6	8.3	4.9	20.5	14.1	7.6

1 Upper confidence limit
2 Mean
3 Lower confidence limit

**Damage per m^2 of floor space

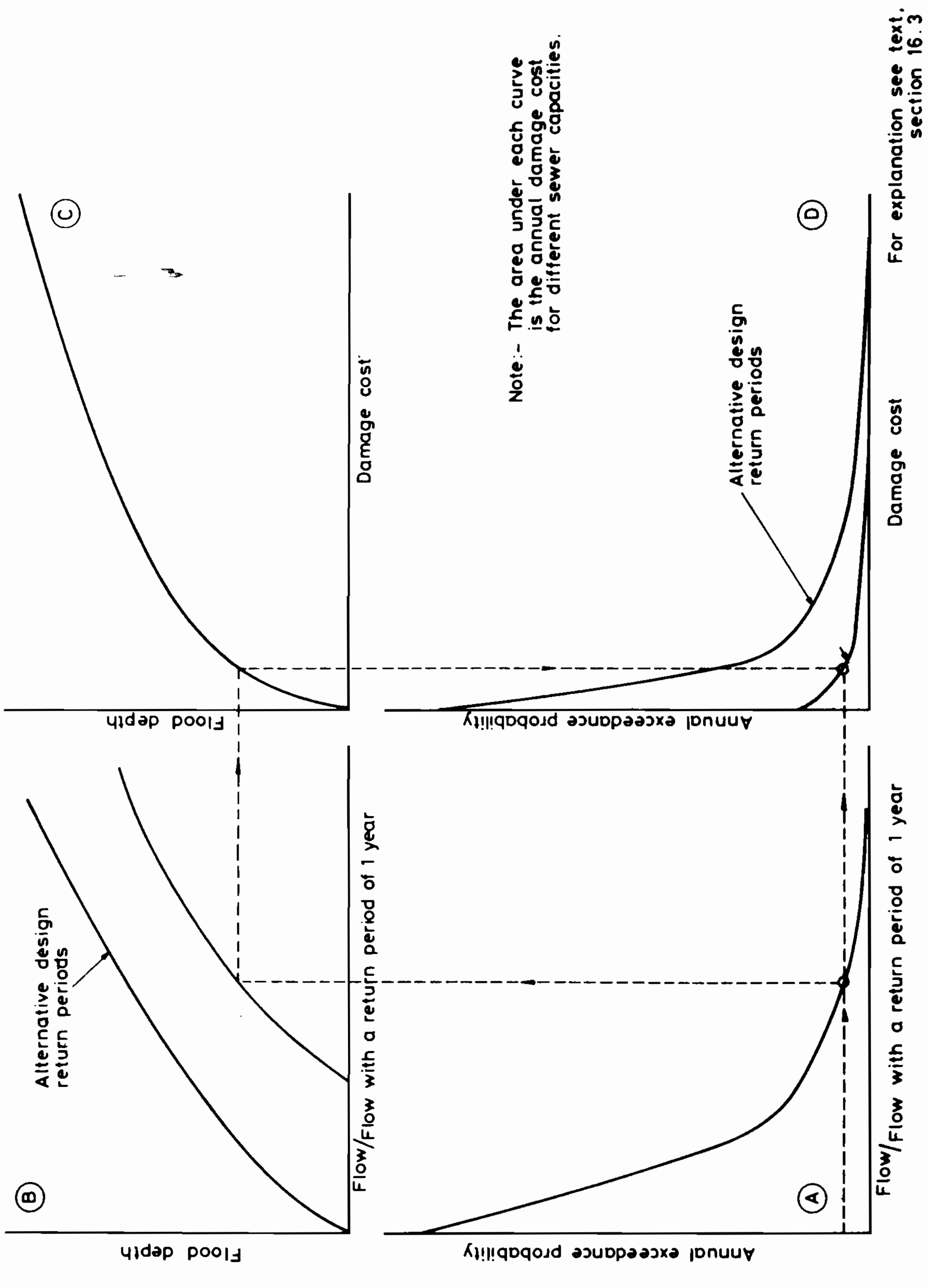

Figure 16.1. Evaluation of annual damage cost for a range of sewer capacities – diagrammatic example

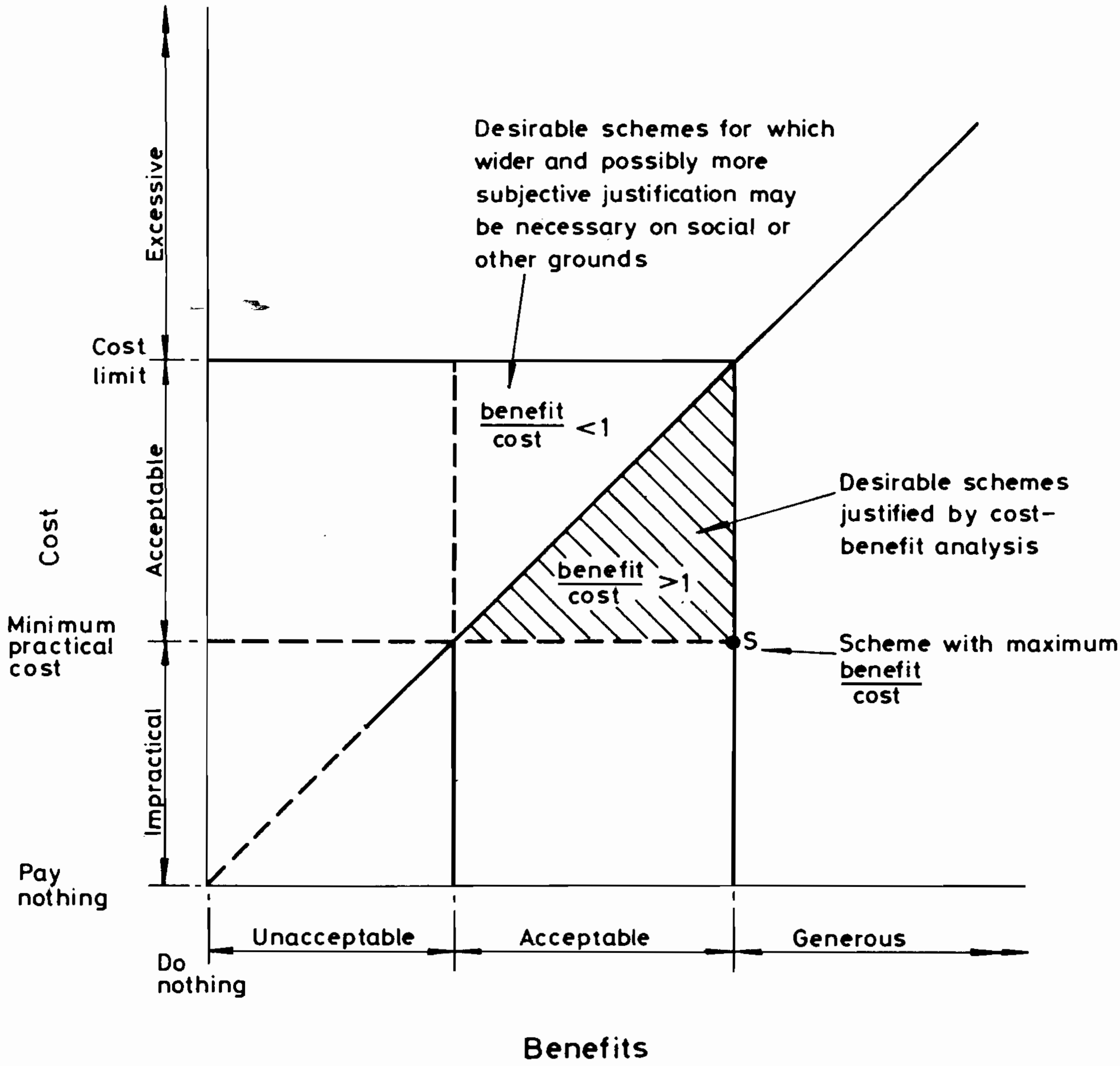

Figure 16.2. Selection of schemes by cost-benefit analysis

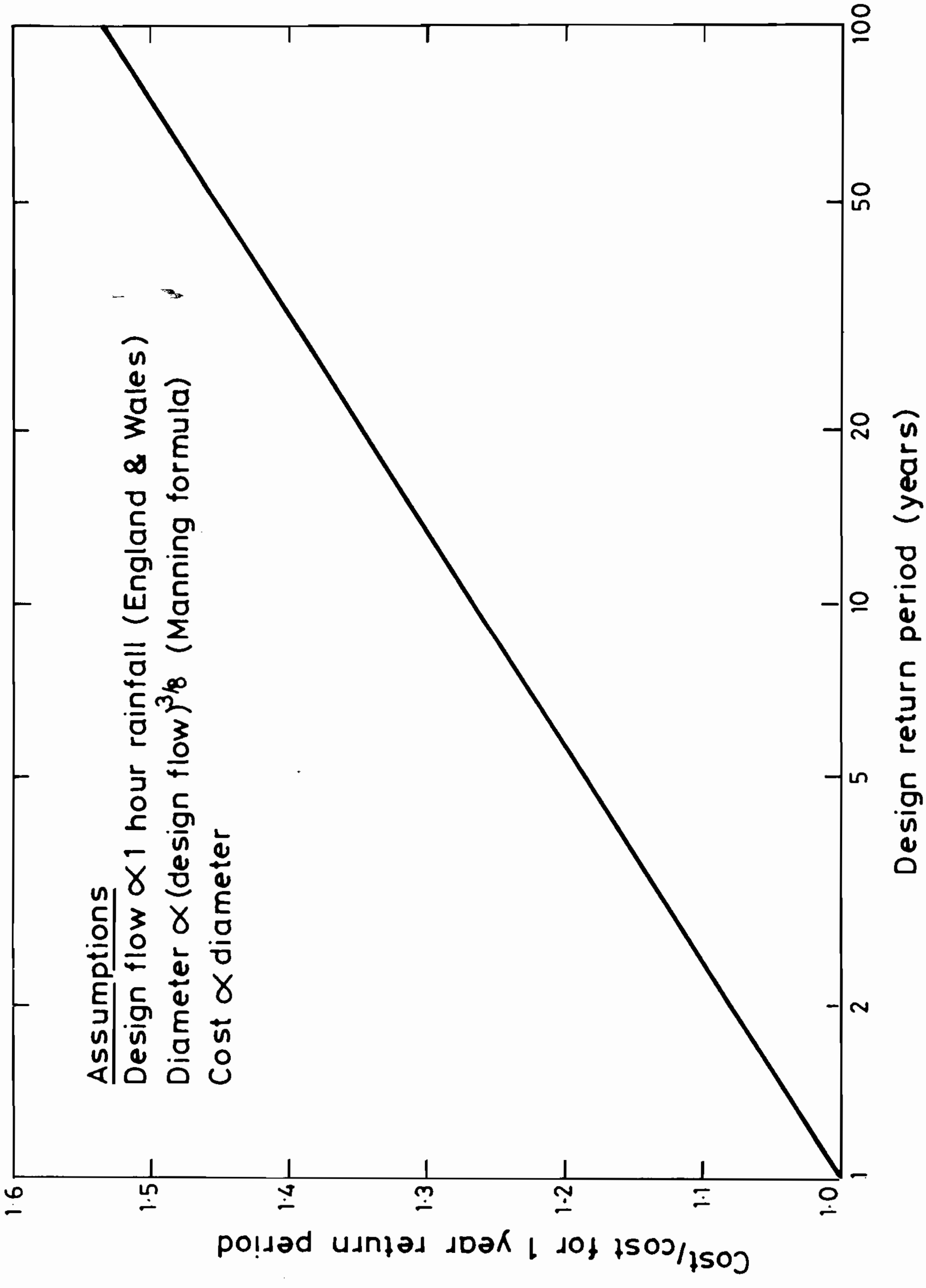

Figure 16.3. Approximate effect of design return period on storm sewer costs

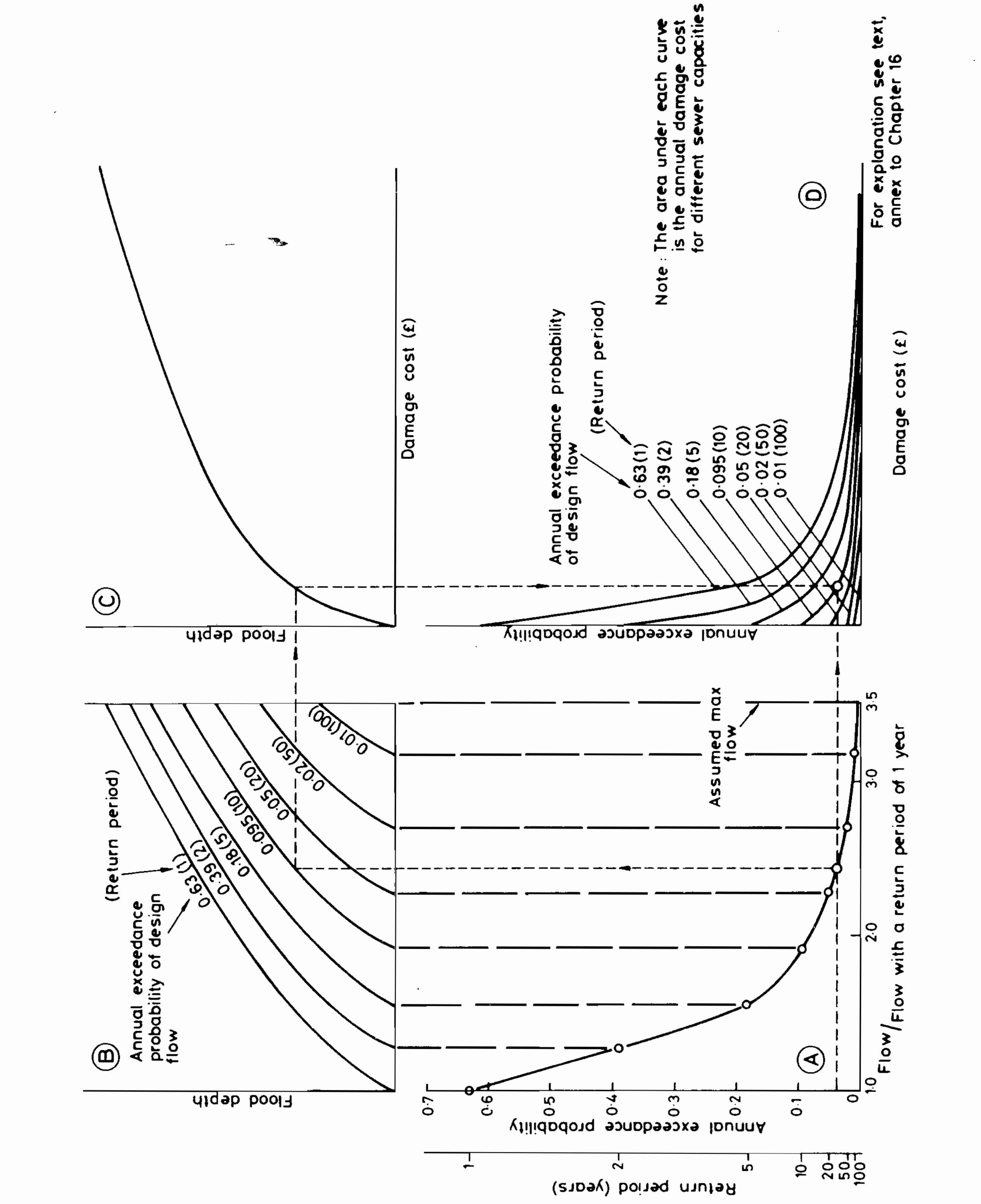

Figure 16.4. Evaluation of annual damage cost for a range of sewer capacities – detailed example

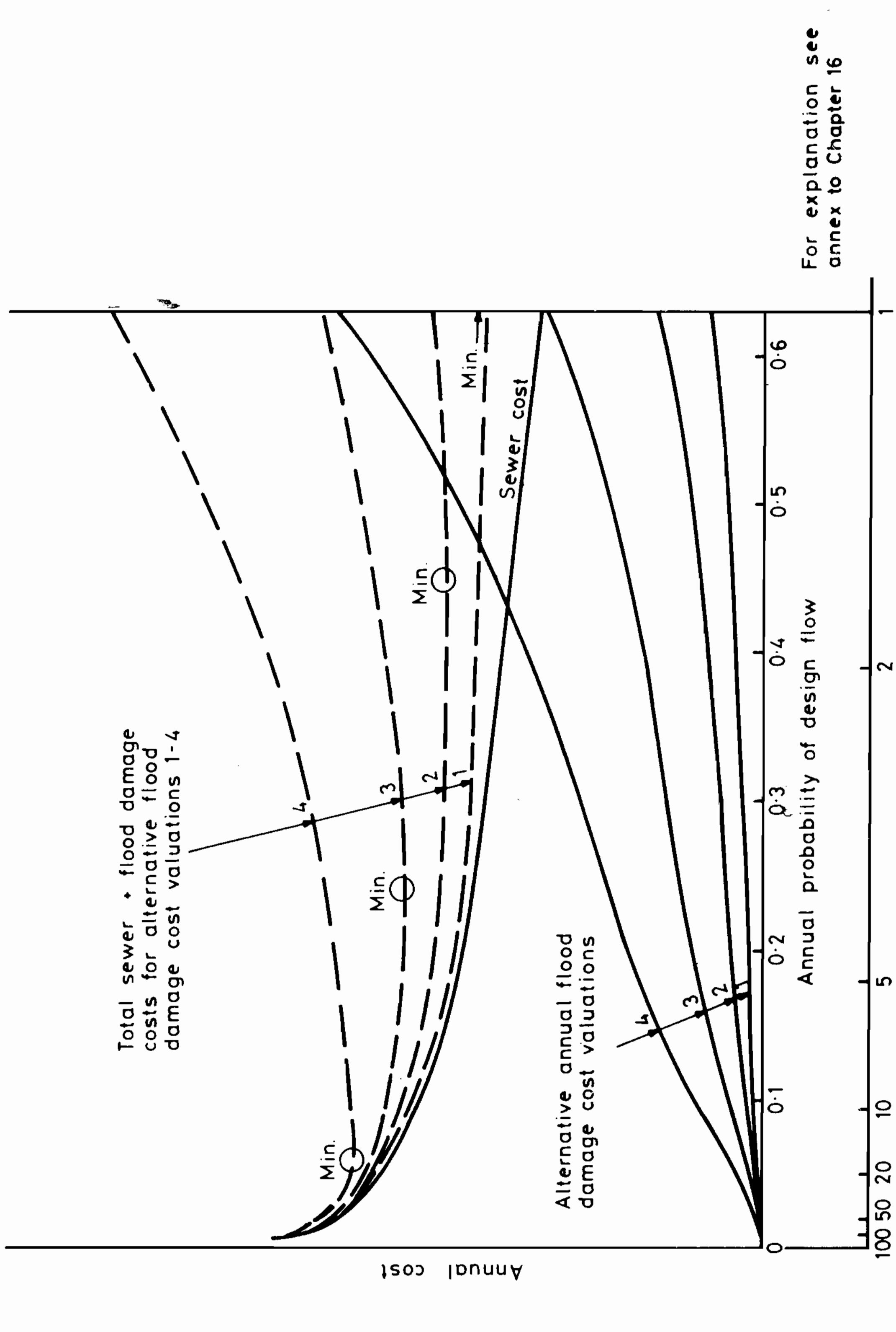

Figure 16.5. Identification of optimum design condition

CHAPTER 17

Opportunities for improvements to sewerage practice

17.1 Introduction

ANY FRESH APPROACH to the hydraulic analysis of sewer systems as innovative as the procedure described in this report will take some time to be fully assimilated by practising engineers. It will be a developing process involving feed back from users to the research team at the Hydraulics Research Station; this has already taken place during the period when the procedure was tested by selecting design groups within the water industry.

This chapter gives general guidance on the options open to designers, many of which have not been available in the past. Based on limited initial experience, it advises on when they should be considered. The chapter does not seek to cover every facility available in the programs, nor is it strictly limited to those facilities. As experience of the use of the new procedure grows, it will be possible to extend the recommendations of this chapter.

Before proceeding to a detailed consideration of particular aspects of the new procedure it is helpful to consider again how the methods differ in range and capability from those they are intended to replace (see also chapter 3).

Earlier chapters of the report describe the developments of the hydrological and hydraulic concepts contained in both the Rational and TRRL Hydrograph design methods. The methods in the Wallingford procedure are intended to approximate more closely to reality and therefore to give more accurate design. The most significant change introduced is the ability to check system performance under flow conditions greater than pipe full.

Over the next decade engineers will be transferring their attention more and more from the design of new sewer systems to the rehabilitation of existing systems, so the ability to undertake the hydraulic analysis of installed systems will be of paramount importance.

If the TRRL method programs were operated in the simulation mode, sensible answers were obtained only for flows of less than pipe-full capacity. If greater flows occurred a surcharge 'flag' was raised and in order to proceed the program artificially increased the pipe diameter to a fictitious value which would pass the flow without surcharging. Now, using the Simulation Method, surcharged flow conditions can be simulated. This enables designers to check the performance of existing systems suspected of being inadequate, using storms with varied return periods, or to check a proposed network under storms of greater magnitude than the 'design storm'.

This chapter also deals with adjustments to the design process not covered elsewhere. These include minor adjustments for cost or performance reasons and consideration of the longitudinal profile to take account of maintenance costs and operational efficiency.

17.2 Simulation of flows

17.2.1 Existing sewerage systems

The ability to simulate flows in existing, overloaded sewerage systems is one of the main advantages of the new procedure over previously available methods. However, in common with any modelling exercise, uncertainties in the available data will affect the reliability of the results. Attention must be given to the following aspects of the catchment data in order to obtain the best use of the surcharging program:

(a) the sewer layout
(b) the size of contributing paved areas
(c) the operating characteristics of storm overflows
(d) the possible presence of unrecorded storm overflows
(e) the possibility of partial blockages or constrictions.

All of these can be checked in the field, but the cost can be prohibitive, particularly if the analysis is required only to redesign a small part of the system such as the outfall sewer. If it is considered that any of the above data problems could be affecting the calculations, then it is desirable to check that the calculated hydraulic conditions are sufficiently close to those of the prototype before testing the effect of possible system alterations. For this purpose it will be necessary to obtain on-site flow measurements.

A simple check is to simulate the outfall discharge resulting from an observed storm and to compare the recorded and predicted flows. However, the results are often difficult to interpret, particularly if storm overflows are present. A more extensive alternative is to monitor flows at several points within the system during a rainstorm and again compare model and prototype performance. Where significant differences occur, the system can be further examined for engineering explanations. It should be emphasised that the objective is to check the reliability of the input data, not to modify any part of the model.

Demonstration surveys by the Water Research Centre [34] have shown that this type of data verification is sometimes necessary (especially in older systems) and that it can be achieved with small, relatively frequently occurring storms (approximately one month recurrence interval).

17.2.2 Proposed sewerage systems

When the design of a new sewer or sewerage network has been completed using one of the methods described in this report, the designer may then use the surcharging program to investigate the behaviour of the system during storms with return periods longer than the design return period. By repeating the exercise with storms of increasing rarity the designer will be able to estimate the return period of the storm which could cause surface flooding. This may indicate the advisability of economic appraisal using one of the methods described in chapter 16. It may also indicate that (if no flooding occurs even with a very rare storm) the original design could be reconsidered in order to effect savings, or alternatively that modest adjustments could give extended protection.

17.3 Reduction of flows entering storm sewers

17.3.1 Detention of storm water

The traditional philosophy of drainage was that storm water should be removed as rapidly as possible from developed areas and discharged to a suitable watercourse. In recent years this view has been increasingly questioned for both environmental and financial reasons.

Artificial drainage interferes with natural hydrological conditions. Natural recharge of the ground water is restricted and extra storm water discharges disturb the natural regime of streams. The usual consequence is that streams in urbanised areas display lower base flows and higher peak discharges and runoff volumes than in their natural condition.

Channel scouring may also occur. These effects could be minimised by detaining more of the rainfall volume at or near the point where it falls on the catchment.

Financial considerations also favour a reduction in the extent or size of piped drainage networks. Reduced inflows require smaller pipes and hence lower costs are incurred.

These reasons apply not only within areas of new development but also where new development is to be connected to an existing system with insufficient capacity. A reduction in the flows generated by the new development may allow an otherwise unacceptable development to proceed. In addition, flooding in an existing system may be relieved by reducing inflows from subcatchments further upstream.

17.3.2 Surface storage

One method of reducing inflows is to make greater use of surface storage by controlling the run-off through road and other gulleys. This may be achieved by increasing their spacing or reducing the size of their outlet. This practice may cause local ponding during short high intensity storms and should therefore be restricted to areas such as car parks and residential roads with slow traffic. Care should be taken to ensure that ponding does not threaten any properties. Surface ponding is often regarded as undesirable because of its effect on the movement of pedestrians or vehicles, but it should be remembered that this restriction will be short-lived and that the storm itself may well cause as much difficulty as the accumulation of water on the surface. The ponded water will, however, remain on the surface for a short period after the storm has ended.

17.3.3 Roof drainage

Other contributing areas such as flat roofs should also be considered as possible areas on which rainwater can be temporarily detained and discharges consequently reduced. The water depth produced by short high intensity storms is only a few millimetres even if no outflow takes place.

Roof water can be excluded from the piped drainage system by the use of soakaways. These can only be used, however, in areas with a permeable subsoil and a low water table and where local regulations permit their use. They should not be used in heavy clay or where a rise in the water table could affect the foundation of buildings. In some areas partial soakaways, with high level connections to the drainage network, may be feasible. An alternative to soakaways which may have wider application is the discharge of roof water onto grass. It should be noted in this connection the heaviest storms generally occur after a dry period in summer when the extra runoff from the roof may well be of benefit to the ground.

17.3.4 Permeable paved surfaces

Following consideration of the hazard from spray caused by fast-moving vehicles, road engineers have for some time been considering the use of permeable road surfaces. These consist of a coarse-graded surface layer containing at least 20 per cent voids laid on an impermeable underlayer. Several examples in the United States and elsewhere have been reported [62], [88] and trial lengths have also been constructed in the United Kingdom [13], [14]. Reduced spray and lower accident rates have been reported.

Such surfaces have additional benefits for the drainage engineer. The rate of discharge particularly during high intensity storms, is reduced because of the detention of water in the voids. There is also some evidence of reduced pollution in surface water discharges from these surfaces.

Disadvantages of permeable road surfaces have also been reported. Construction control has to be more stringent than for normal road surfaces and costs are higher. Some of the experimental lengths have deteriorated after four or five years, especially in areas of additional stress such as junctions and corners. Voids in the surfaces tend to fill after a few years, causing poorer performance (though not a complete loss) of both spray reduction and flow attentuation. On kerbed roads the permeable surface is drained by small longitudinal gutters which could be hazardous to cyclists.

The most feasible application of permeable road surfaces appears to be on high speed rural roads without kerbs. Clearly the drainage engineer will gain little benefit from this application. There may, however, be alternative applications, such as large car parks, where the benefits to drainage would alone be sufficient to justify this type of construction.

An alternative to the permeable surface with impermeable underlayer is the fully permeable surface allowing free percolation to the water table. Design of the sub-base has to take account of the infiltration capacity of the subsoil. Examples for car park construction have been reported[6, 23].

17.4 Attenuation of flow within the sewerage system

Storage capacity added to a system attenuates the flood wave, that is it reduces the peak flow and extends the duration of lower flows. This option may be used to relieve an overloaded system, or to enable additional development to be connected without increasing pipe sizes, or to permit a reduction in the diameter of pipe lengths under design. Various ways to achieve these results can be examined using the facilities of the Hydrograph and Simulation Methods and some of these are indicated in this section.

Storage tanks can be provided to increase the below-ground storage either on the line of the sewer or off-line, with arrangements made to discharge the contents back into the sewer as the flow falls. Examples associated with combined sewer storm overflows are described in a report by the Scottish Development Department [86]. A length of oversized pipe fulfils a similar role as an on-line tank if it has a controlled outlet and any of these alternatives can be included in the analysis. The effectiveness of alternative storage arrangements, including the case of no additional storage, can be compared. Recent studies of the optimum sizing of tanks have been reported[17, 19].

The designer must always remember that the overall behaviour of the system (including the receiving watercourse) has to be considered as well as the relief of local problems. Therefore it is necessary to check that a proposed detention tank does not delay the peak discharge from one part of the system so that it coincides with the peak from another part and thus makes the downstream condition worse.

Detention tanks can retain polluting solids and so improve the quality of the storm water. Tanks should be designed to be self-cleansing if possible, or arrangements made for their periodic cleansing.

It must not be overlooked that the storage capacity of manholes will in surcharge situations have a significant effect on attenuating flows and this can be investigated using the Simulation Method.

17.5 Reducing the impact of stormwater discharges on the receiving watercourse

No design of a surface water sewerage system is complete without consideration being given to the watercourse into which it discharges. The watercourse must of course have sufficient capacity to handle the peak discharge. The effect of the quality of the discharge on the receiving watercourse must also be considered.

17.5.1 Pollution in storm water

Section 3.3 has pointed out that only limited information is available on the quality of urban storm runoff. The design engineer should, however, be aware of the potential pollution load carried by a surface water sewerage system. A summary of published studies of many storm events is given in Table 3.1; extensions to the information may be expected as other studies are completed.

The authority responsible for the watercourse in which it is proposed to construct the new outfall will take account of the likely pollution load, the dilution available and the appropriate quality standards when setting the conditions for their consent to the new discharge.

One of the primary sources of polluting matter is the stagnant water which has been stored in gulley pots, which creates a foul flush of water at the start of the storm. This polluting matter could be reduced by reducing the number of gulley pots, or eliminated if gulley pots were empty at the start of a storm. This could be achieved by the use of gulley pots incorporating an outlet from the bottom of the pot to the sewer by means of a slow rate filter. An additional benefit of such a design would be the availability at the start of runoff of an additional storage volume, which would introduce further slight attenuation of the runoff hydrograph from the subcatchment. Modified gulley pots might require additional maintenance and odour problems might prevent their use on combined sewer systems.

Low flow connections from storm to foul sewers would reduce the pollution load reaching the watercourse. Such devices would eliminate pollution from foul sewers wrongly connected to the stormwater system. In conjunction with storage tanks, they would also retain and pass to treatment the foul flush often observed in stormwater systems. In designing such connections it would be necessary to ensure that the rate of storm flow to the foul sewer was carefully controlled and that no reverse flow of foul sewage to the storm system could occur.

The potential danger of accidental pollution by spillage will be related to traffic density and type of industry within the catchment. Where appropriate the provision of oil interceptors or temporary surface storage should be considered.

The impact of the new flow regime on the receiving stream should also be considered, in order to limit flooding, erosion of the bed or banks, or excessive deposition of solids.

17.5.2 Modification of storm overflows

The design and setting of storm water overflows should have due regard to the recommendations of the Technical Committee on Storm Overflows and the Disposal of Storm Sewage [66] and the Working Party on Storm Sewage (Scotland) [86], including the following:

(a) Overflow designs incorporating storage and settling are more effective in reducing the gross solids load discharged to the watercourse.

(b) The calculation of the overflow setting should take account of any planned future development in the catchment up to a limiting date.

(c) Aesthetic problems may be reduced or eliminated by screening overflows.

Overflows cannot be designed directly by the methods described in this report, but the behaviour of overflows with specified parameters can be represented and the effects of different designs or settings on the remainder of the system can be determined.

A storm overflow without an overflow chamber can be represented by specifying the overflow discharge setting and either the maximum discharge permitted downstream or the proportion of the discharge above the overflow setting which passes downstream. A storm overflow incorporating an overflow chamber may be represented as an on-line tank. The methods of calculation are described in detail in sections 11.1 and 11.2.

The reliability of the modelling of overflows depends on the accuracy with which the overflow geometry or performance can be specified. Many modern storm overflows achieve an accurate division between the flow spilled and passed to treatment. The overflow discharge setting can be calculated adequately and the flow to treatment does not significantly increase above this figure. In this category may be included the syphon, vortex, high side weir, and stilling pond type overflows. All of these have a throttle device on the outlet which controls the overflow setting.

A second category of overflows is more difficult to represent. This includes low side weir overflows and pipe overflows set in a manhole wall. The overflow discharge is dependent on the hydraulic conditions in the main pipeline which, as throttles are generally not present, can be uncertain. Such overflows are not now constructed, but may be encountered during studies of the hydraulic behaviour of existing systems. If serious doubt exists regarding the storm overflow performance, in situ flow measurements are required to provide the necessary data for the programs.

Some overflows may not be amenable to analysis. An example is the leaping weir, in which the overflow is a continuation of the inflow pipe with the onward flow falling through a gap in the sewer to a lower level pipe. At low flow all the discharge falls to the lower level but at high flows a proportion of the flow leaps the gap to the overflow. The amount overflowing depends on the approach velocity and the position of a plate laid as a leading edge to the outfall sewer. The leaping weir also has the unusual characteristic that the flow passing to treatment tends to reduce as the inflow increases. Flow measurement is the only way to calibrate such a weir.

Where existing overflows operate too frequently and pollution of the receiving watercourses is evident there are several possibilities available for reducing the frequency of spill and the pollutant mass passed to the watercourse.

If the downstream capacity of the sewer is sufficient an overflow can be modified by raising its weir level so as to delay the start of overflow and contain a greater proportion of the first flush. If the system downstream cannot accept higher flows, the use of on-line or off-line detention tanks already referred to in section 17.4 and described in the Scottish Development Department's report [86] can be considered as an alternative to reconstructing the overloaded sections of the downstream network. The effect of such tanks on the downstream network can be calculated from the programs.

Where some reconstruction of the downstream network is being considered the possibility of replacing several old overflows with a modern one serving a larger area can be investigated, though the effect on the receiving watercourse of the larger discharge from such an overflow must not be overlooked. The volumes of water likely to be passed to the watercourse can be obtained as an output from the programs enabling an assessment to be made of the dilution which will be provided.

17.6 Combined and separate surface water sewerage

The methods described in this report calculate the surface water discharge from a drained area and the size of sewerage system necessary to cater for it. This sewerage system may be a separate system dealing only with the surface water or a combined system receiving the foul flow as well as the surface water.

The type of system to be provided in any area will be determined by the local water authority who should be consulted. Even so the design engineer is recommended to consider the advantages and disadvantages of both systems as described in the report of the Working Party on Storm Sewage (Scotland) [86].

It should be noted that some authorities are suspending their policy of the gradual separation of combined systems. Instead use is being made of detention tanks to improve the efficiency of storm overflows on combined sewers as described in section 17.5.2.

17.7 Maintenance of the sewerage system

This report is concerned with the hydraulic design of storm sewers and not the structural design which has the most effect on the maintenance costs and reliability of the system. The most economical design will be the one which gives the lowest present value when capital and maintenance costs are discounted. Publications such as the British Standards Institution's Code of Practice 2005 [11] are concerned with structural design. There are, however, some aspects of hydraulic design where choices are available that affect maintenance costs and operational efficiency and these are now considered collectively.

17.7.1 Flow velocities

Minimum and maximum flow velocities are discussed in section 8.7. Flow velocities must be considered and checked by the engineer. There may sometimes be a case for increasing capital cost if this can be justified by a saving on maintenance costs or by extra reliability of the system. For example, where the generally prevailing pipe gradient flattens sharply with a substantial reduction in velocity there is a greater likelihood of silting. Some modification of gradient at extra capital cost to achieve a smoother profile may be justified. Local catchment and sewage characteristics affect the nature and volume of silt carried into sewers and experience of the performance of existing sewers in a similar situation will therefore help in any such evaluation. From an operational viewpoint a smooth pipe profile is desirable but because of capital costs and physical restraints this cannot always be achieved.

17.7.2 Minimum and non-decreasing pipe diameters

Water authorities generally set a minimum diameter for sewers they are willing to adopt as public sewers. The value should be obtained from the relevant water authority if the sewer is being offered for adoption. Authorities may also require that diameters should not decrease in a downstream direction even though the availability of a steeper gradient might make this theoretically possible.

Where the design is not subject to these constraints, the smallest pipe size compatible with the anticipated maximum flow and the gradient should be selected. The use of over-large pipes gives reduced velocities and increases the risk of the deposition of sediment and subsequent blockage.

17.8 Longitudinal profile

In the Optimising Method the longitudinal profile is designed for minimum construction cost. In the Modified Rational and Hydrograph Methods the user has to specify design gradients and the profile chosen has important cost consequences. Engineers should be aware of the factors which influence construction costs so that when using these latter methods they may design the profile in an informed manner approximating to the ideal solution. The process used in the Optimising Method is summarised in section 12.4. The main factors to be considered are generally well known but deserve to be repeated.

The sewer should be shallow for reasons of economy in capital cost and recommendations are given in the British Standards Institution's Code of Practice 2005 [11]. On the other hand the sewer must be deep enough to give structural protection and allow connections at appropriate gradients from all existing and likely future developments : CP 2005 gives recommendations about this also. Ground conditions such as the presence of rock or ground water can have a strong influence on the choice of profile.

The profile must also take into account existing services and structures and the requirements of sewer ancillaries such as overflows, storage tanks, pumping stations and outlet structures.

In special cases sewers are required to be above ground level. The reasons can be to maintain minimum gradients to avoid pumping in flat terrain or to save cost compared with a longer route; planning approval will generally be required for above-ground sewers and visual appearance may be a constraint on this approach.

The choice of profile and associated gradients will have implications for the velocities in the sewer system. The subject of minimum and maximum velocities has already been discussed in sections 8.7 and 17.7.1. An increase in pipe gradient and the consequent higher velocity of flow tends to increase the peak flow in the system for a given storm return period. This is due, in a calculation by the Modified Rational Method, to a reduction in the time of concentration and consequent increase in the appropriate rainfall intensity. In a calculation by a hydrograph method the same tendency is due to an increase in the speed of the flood wave along the pipeline. A trial and error procedure therefore has to be adopted to adjust the profile and check the velocities until a satisfactory performance is obtained.

It is appreciated that engineers have always given consideration to these factors. However, they have generally been inhibited from a rigorous investigation of the alternatives by the time and cost involved in an iterative design procedure. This is now quickly and effectively performed by the Optimising Method which minimises the capital cost to achieve a design consistent with velocity constraints specified by the user.

17.9 Adjustments for manufactured pipe sizes

The design methods in the procedure select the manufactured pipe size that will accommodate the calculated flow. This generally leads to some over capacity and there may be a case for adjustment to the pipe sizes calculated (for example, if the next smaller pipe size only just fails to contain the design flow, or the selected pipe only just contains it). In these cases the amended network should be examined for surcharge using the Simulation Method.

It would also be helpful to check the amended design for both shorter and longer return periods to identify the effect of the revised diameters on the hydraulic performance and to consider the variations in cost and the hydraulic consequences.

Alternative designs are generally worth considering if an increase in diameter of a short length significantly enhances the capacity of the system or if in the initial design the capacity is significantly in excess of the design flow over relatively long lengths.

17.10 Future research and experience

This chapter has discussed the application of the new procedure in several situations in which current experience is inevitably limited and definitive answers cannot yet be given. In addition, at the time of writing (mid 1981) active research projects are investigating several matters relevant to the contents of this chapter. These include alternative methods for reducing sewer inflow discharges and attenuating peak discharges; the modelling of water quality parameters; sediment transport in sewers; and the site application of sewer flow measurement techniques.

The Standing Technical Committee on Sewers and Water Mains will take note of the results of these investigations, and of the experience of users in applying the new procedure, and will produce further recommendations as necessary.

CHAPTER 18

Application outside the United Kingdom

SEVERAL parts of the procedure are based on conditions and engineering practices within the UK and therefore should not be used in any overseas applications. This restriction affects the following features:

(i) Design rainfall depth is a function of return period and duration, and of parameters derived for any location in the UK (see chapter 6).

(ii) Percentage runoff is a function of percentage impermeable area, a soil index and an urban catchment wetness index (see section 7.4). These variables have been determined for UK conditions and the regression coefficients have been derived from data for a number of UK catchments.

(iii) Percentage runoff factors for paved, permeable and pitched roof surfaces are selected so that the same factors apply to paved and pitched roof surfaces (section 7.5). Runoff from the permeable surface is assumed to be negligible if the percentage runoff factors for the paved and pitched roof surfaces are less than 70 per cent. Any permeable area contributing runoff is regarded as an extension of the corresponding paved area (section 7.7); runoff from a permeable surface is not calculated separately.

(iv) The storage coefficients in the non-linear reservoir model for runoff from a paved surface are non-linear functions of ground slope and area (section 7.7). Also the depression storage is a non-linear function of ground slope (section 7.6). Again the regression coefficients have been derived from data for a number of UK experimental catchments. The storage coefficient and depression storage for a pitched roof surface are constants selected on the basis of UK data. In the procedure a set of nine runoff hydrographs is generated for three standard slopes and three standard paved areas. These standard slopes and areas cover the ranges of the original data set on which the regression equations for the storage coefficient and depression storage were based.

(v) The return period of flow is made the same as the return period of rainfall by an appropriate choice of the urban catchment wetness index in the percentage runoff equation (section 9.4). The value of the index for a catchment in the UK depends on the standard average annual rainfall.

(vi) The construction cost of a pipe and its upstream manhole is based on UK data (section 12.3).

(vii) The sewered sub-area model uses the non-linear reservoir model for the surface runoff and routes flows through a set of equivalent pipes whose gradient and diameters are derived from UK data (chapter 10).

(viii) The sewer system network has a tree-like structure with few loops and no reversed free surface flow in any pipe (section 8.3). Manhole head losses are neglected until surcharging occurs.

The UK version of the procedure includes all of the above features. The user can however supply his own rainfall hyetograph and percentage runoff as input data. Changes to the distribution of runoff, the storage coefficients, depression storage and aspects of the construction cost calculations would require changes to the programs. A preliminary overseas version would require the user to introduce his own data relating to these features. Further overseas versions would incorporate alternative design rainfall, percentage runoff and surface runoff models and a construction cost routine appropriate to the country concerned. The representation of flow in looped networks would require substantial changes to the structure of the programs and to the design philosophy.

Acknowledgements

Primary acknowledgement for the contents of the computer programs and of this report goes to the staff of the main research organisations involved: the Hydraulics Research Station, the Institute of Hydrology and the Meteorological Office. The research team leaders were Dr R.K. Price (HRS), Dr M.J. Lowing (IH) and C.K. Folland (Met. Office). This report was edited by P.J. Colyer (HRS). In addition the Working Party recognises that many individuals and organisations have contributed to this wide-ranging project. The organisations employing the Working Party members listed in Appendix 3 have been generous in their provision of time and supporting facilities. Invaluable assistance was also provided by staff of the following organisations involved in the trials of the procedure:

Water Research Centre
North West Water Authority
Severn Trent Water Authority
Thames Water Authority
Welsh Water Authority
Wessex Water Authority
Borough of Blackburn
City of Bristol
Central Lancashire Development Corporation
Telford Development Corporation
Central Regional Council
Strathclyde Regional Council
John Taylor and Sons
Watson Hawksley

The Working Party also acknowledges the support and encouragement of the Standing Technical Committee on Sewers and Water Mains, in particular its subcommittee on Hydraulic Design and Planning.

APPENDICES

Glossary of terms

The definitions given below refer to the usage in this report and are not necessarily of general application.

Adoption of sewers. Acceptance by the appropriate authority of statutory rights and duties relating to sewers.

Air entrainment. The process by which bubbles or pockets of air are caught within the fluid and transported with the flow.

Antecedent conditions. The wetness of a catchment before a particular rainfall event; see urban catchment wetness index.

Antecedent precipitation index. An indicator of rainfall depth over a period preceding a particular event; the five-day antecedent precipitation index (API5) is used in this report.

Areal reduction factor. A factor applied to point rainfall depths or intensities to give values applicable to an area.

Attenuation. The reduction of peak flows in a flood wave, accompanied by an increase in its duration, as the flood wave progresses downstream.

Autographic raingauge. A raingauge recording the variation of rainfall intensity with time.

Back drop manhole. A manhole at which a vertical drop in the longitudinal profile of the pipeline occurs.

Backwater effects. The effect of water flows or depths on hydraulic conditions upstream; backwater effects can only occur in subcritical flow.

Battered trench. A trench with sloping sides.

Bedding factor. The ratio of the superimposed load to the crushing strength [12] of a pipe.

Benefit-cost ratio. The ratio of benefits to costs in an economic analysis.

Biochemical oxygen demand (BOD). The oxygen absorbed by a water sample in five days at 20°C.

Birmingham curve. A rainfall depth-duration curve developed by Lloyd-Davies [57] from rainfall data at Birmingham.

Branch. A number of pipes in series, numbered consecutively in a downstream direction.

Catchment. An area served by a single drainage system.

Catchment wetness index. An index of the wetness of a catchment before a rainfall event; see urban catchment wetness index.

Cavitation. A potentially damaging condition, occurring at high flow velocities, in which dissolved oxygen is released from solution at low pressure.

Chemical oxygen demand (COD). The oxygen required to oxidise all organic material in a water sample.

Class B granular material. Uniform readily compactible material free from tree roots, vegetable matter, building rubbish and frozen soil, and preferably excluding clay lumps retained on a 75mm sieve and stones retained on a 40mm sieve.

Combined system. A sewerage system in which foul sewage and storm water are carried in the same pipes (compare separate system and partially separate system).

Conceptual. A description of a process in equivalent or notional terms, which do not directly represent the physical forces involved (compare deterministic).

Consent conditions. The conditions imposed by the appropriate public authority before permitting the discharge of a potentially polluting flow to a watercourse.

Constraint. A limit imposed on the range within which a solution may be sought.

Correlation coefficient. The (multiple) correlation coefficient r_c is a measure of the association between the observed (x) and predicted (x') values in regression analyses:

$$r_c = \sqrt{1 - \frac{\Sigma (x - x')^2}{\Sigma (x - \bar{x})^2}}$$

r_c^2 is a measure of the proportion of the variance in x which is explained by the regression analysis.

Cost-benefit analysis. A method of economic analysis in which all identifiable costs and benefits are quantified (if possible) and compared.

Cost effectiveness. A method of economic evaluation in which the main tangible benefits of the project are evaluated to ensure that the expenditure is worthwhile.

Culvert. A covered channel or pipeline.

Depth-duration-frequency relationship. A table or graph showing the way rainfall depth at a particular location is related to duration and frequency (or return period).

Depression storage. The depth of water retained on the ground surface in puddles or other depressions.

Detention tanks. Tanks constructed within a sewerage system to store temporarily a volume of water during peak flows (see off-line tanks and on-line tanks).

Deterministic. The representation of a process by the physical laws of cause and effect, such that a change in the input would accurately reproduce an observed output (compare conceptual).

Discharge coefficient. A numerical value, determined experimentally, included in an equation relating discharge to upstream head and the physical characteristics of a weir, orifice, etc.

Discount rate. An annual percentage rate used in economic studies to reduce costs and benefits occurring in the future for comparison with present costs and benefits.

Discounted cash flow. A method of evaluating cash flows in which future income and expenditure are reduced to present values by applying a discount rate.

Filter. A smoothing procedure to convert point rainfall profiles to areal profiles.

Finite difference equations. Equations describing continuous functions in terms of values at discrete points.

Formation level. The level to which a trench is excavated before bedding is placed.

Froude number. The ratio of flow velocity to the speed of a wave in shallow water $\sqrt{V^2B/(gA_f)}$; the flow is described as subcritical if this ratio is less than unity and supercritical if it exceeds unity.

Free surface flow. Flow conditions which include a water surface subject to atmospheric pressure (compare with surcharged flow).

Gradually varying flow. Flow conditions in which the discharge varies gradually with distance along the pipe or channel.

Ground condition. An index used in the resource cost model (see Table 12.1) to describe the sub-surface conditions encountered during excavation (see also site condition).

Gulley. A structure, usually incorporating a grating and a grit trap, to permit the entry of surface runoff into the pipe system.

Haunching. The material surrounding a buried pipeline, up to the depth at which the pipe width is a maximum.

Hydraulic gradient. In an open channel, the gradient of the water surface; in a pressurised pipe, the gradient joining points to which water would rise in pressure tappings.

Hydraulic radius. The ratio of cross sectional area of flow to the wetted perimeter of a channel or pipe.

Hydrograph. A series of values, in either numerical or graphical form, of flow rate varying with time.

Hyetograph. A series of values of rainfall intensity varying with time; same as rainfall profile.

Impermeable, impervious. Description of a surface type which resists the infiltration of water; in practice some infiltration occurs through pores and cracks.

Infiltration (a) to the ground. The loss of rainwater into the ground.
(b) to pipelines. The entry of groundwater into pipelines.

Inflow. Surface runoff entering a sewerage system from areas not originally intended to be connected to the system.

Inlet. An entry point to a sewerage system, usually a gulley.

Inlet hydrograph. The hydrograph generated by surface runoff at the entry points to a sewerage system.

Intangible. Description of costs or benefits to be considered in an economic evaluation, which cannot be expressed in monetary terms (opposite: tangible).

Intensity-duration-frequency relationship. A table or graph showing the way rainfall intensity at a particular location is related to duration and frequency (or return period).

Interception. The process by which rainfall may be prevented from reaching the ground, for example by vegetation.

Invert. The lowest point on the internal bore of a pipe (opposite: soffit).

Inverted syphon. A pipeline carrying sewage or stormwater beneath an obstacle such as a river channel or a road in cutting.

Jardi rainfall recorder. An instrument capable of measuring short duration, high intensity bursts of rainfall.

Kinematic viscosity. Absolute viscosity divided by fluid density (equal to $1.141 \times 10^{-6} m^2/s$ for water at 15°C).

Lateral inflow. Flow entering a channel uniformly along its length.

Linear. A description of the relationship between two or more variables which vary in proportion to one another (compare non-linear).

Lloyd-Davies method. An adaptation by Lloyd-Davies [57] of the Rational Method for storm drainage design.

Ministry of Health formulae. Equations recommended by the Ministry of Health in 1930[65] to provide rainfall intensities for storm drainage design.

Muskingum-Cunge routing method. A method of routing flows in channels and pipes, first applied on the Muskingum River in the USA and subsequently modified by Cunge [21].

Non-linear. A description of the relationship between two or more variables, which has the form of a power law rather than a straight line (compare linear).

Normal depth. The water depth in normal flow conditions, ie with the hydraulic gradient equal to the gradient of the pipe or channel.

Off-line tanks. Detention tanks which are physically separated from the flow of water along the pipeline.

On-line tanks. Detention tanks which form part of the pipeline system, so that water flows through the tank between incoming and outgoing pipes.

Optimisation. The process by which a preferred solution is sought amongst several alternatives.

Orifice. A constriction in a pipeline to control the rate of flow.

Overland flow. Flow over the ground surface, including both paved and unpaved surfaces and roofs.

Over-under factor. A factor used in the WRC study of sewerage costs [97] to describe the difficulty of a particular construction scheme.

Overflow chamber. A stilling chamber incorporated in some designs of storm overflow.

Partially separate system. A sewerage system in which part of the storm runoff is carried with the foul sewage in a combined system, and part is carried in a separate system.

Peakedness. A measure of the sharpness of a rainfall profile, in terms of the ratio of the maximum to the mean rainfall intensity; percentile peakedness gives the percentage of storms of a specified duration and return period with a peakedness less than or equal to that of a given profile.

Percentage runoff. The percentage of the rainfall volume falling on a specified area which enters the stormwater drainage system.

Percentile. The percentage of occurrences within a stated range; for application to rainfall profiles, see peakedness.

Performance-cost. A method of economic analysis in which costs and in-service performance are compared.

Permeable, pervious. Description of a type of ground surface through which water may infiltrate; some surface runoff may occur if the ground becomes saturated.

Pumping station. A structure included within a sewerage system to pump water when drainage cannot be achieved by gravity.

Rainfall intensity. The rate of rainfall, expressed in mm/hr or ins/hr.

Rainfall profile. A series of values of rainfall intensity varying with time; same as hyetograph.

Rational method. A simple method, in well-established use throughout the world, for calculating the peak discharge in a drainage system.

Recession. That part of a flood event or hydrograph when the flow is reducing after the peak.

Regression analysis. A statistical technique by which a dependent variable is expressed in terms of one or more independent variables.

Reservoir storage. The phenomenon by which a volume of flow has to be stored temporarily on a surface or in a length of pipe or channel as the depth and rate of flow increase; the storage is depleted during the recession.

Resource cost. The cost of resources used in sewerage construction (materials, plant and labour); the resource cost is appreciably less than the final cost of construction (see section 12.2.).

Return period. The average period between occurrences of an event greater than or equal to a given value.

Reynolds number. The ratio of inertia force to viscous force in a flowing fluid (Vd/ν). The magnitude of the Reynolds number determines whether the flow is laminar or smooth-, transitional or rough-turbulent (see turbulent flow and reference 1).

Routing coefficient. A component part of the coefficient used in the Modified Rational Method.

Separate system. A sewerage system in which foul sewage and storm water are carried in different pipes (compare combined system and partially separate system).

Sewerage system. A network of pipes or channels to convey foul sewage and/or stormwater from a developed area.

Side weir. A weir constructed in the side of a pipe or overflow chamber to permit the spill of high flows into a relief system.

Simulation. The representation of specified conditions in a sewerage system using a rainfall-runoff calculation method.

Site condition. An index used in the resource cost model (see Table 12.1) to describe the surface condition encountered during sewer construction (see also ground condition).

Soakaway. A pit, usually filled with large stone, into which surface water is drained to infiltrate into the ground.

Soffit. The highest point on the internal bore of a pipe (opposite: invert).

Soil moisture deficit (SMD). A measure of soil wetness, prepared regularly by the Meteorological Office, indicating the capacity of the soil to absorb further rainfall.

Standard deviation, σ. A measure of the dispersion of a series of values about the mean; equal to the square root of the variance:

$$\sigma = \sqrt{\frac{\Sigma(x - \overline{x})^2}{n}}$$

To obtain an unbiased estimate n is replaced by n-1.

Standard error of estimate. A measure of the dispersion between observed values (x) and those predicted from regression analysis (x′) equal to:

$$\sqrt{\frac{\Sigma(x - x')^2}{n - m - 1}}$$

where n is the number of observations used in the regression analysis and m is the number of independent variables. If the best estimate of x is its average value $\overline{x}$, then m = 0 and the standard error of estimate is the same as the standard deviation.

Standing wave. A wave formed on a water surface, which does not progress with the flow; usually associated with the occurrence of critical flow conditions (Froude number $\approx$1).

Stilling pond. A type of storm water overflow incorporating a stilling pond, intended to ensure that polluting material is retained within the pipe system.

Storage tanks. Tanks constructed within a sewerage system to store temporarily a volume of water during peak flows (see also detention tanks, off-line tanks, on-line tanks).

Storm profile. A series of values of rainfall intensity varying with time, which may be expressed in terms of percentile peakedness.

Storm sewage. Storm runoff mixed with foul sewage in a combined system (compare surface water).

Storm water overflow. A structure built within a combined sewerage system in order to spill to a watercourse or relief system stormwater which cannot be carried along the pipe.

Sub-area. A group of sub-catchments treated as a single unit for calculation purposes (see chapter 10).

Sub-catchment. The area draining to a single pipe length.

Subcritical flow. Flow conditions in which the Froude number is less than unity.

Supercritical flow. Flow conditions in which the Froude number exceeds unity; surface waves cannot propagate upstream in supercritical flow.

Surcharged flow. Flow conditions in which the hydraulic gradient is higher than the pipe soffits (compare with free surface flow).

Surface runoff. Flow over the ground surface to the drainage system.

Surface water. Storm runoff not contaminated with foul sewage (compare storm sewage).

Suspended solids. Particulate matter carried in suspension by fluid flow.

Tangent methods. Graphical methods of determining peak discharge from the time-area diagram.

Tangible. Description of costs or benefits to be considered in an economic evaluation, which can be expressed in monetary terms (opposite: intangible).

Tender cost. The cost of sewerage construction as estimated in tender proposals.

Time-area diagram. A diagram showing the increase of contributing area with time in a given catchment.

Time of concentration. The time taken for flow to reach the point under consideration from all contributing parts of the catchment; equal to time of entry plus time of flow.

Time of entry. The time taken for surface runoff to reach the entry to the pipe system from all contributing parts of the sub-catchment.

Time of flow. The time taken for flow to reach the point under consideration from the head of the pipe system.

Time offset method. A method of routing flood waves through channels or pipes by displacing the hydrograph by the flow time in the pipe or channel under consideration.

TRRL method. A computer-based method for the determination of flow hydrographs in storm drainage systems and for the sizing of pipes [91], [95].

Turbulent flow (smooth turbulent, transitional and rough turbulent). Flow conditions which occur in pipes when the Reynolds number exceeds about 2300 (see reference 1).

Typical storm methods. A category of methods for determining flows in storm drainage systems which used a single design storm as rainfall input.

Urban catchment wetness index (UCWI). A development of the catchment wetness index for application to urban catchments; see section 7.9 for a numerical definition.

Variance, σ^2. A measure of the dispersion of a series of values about the mean, equal to the square of the standard deviation:

$$\sigma^2 = \frac{\Sigma(x - \bar{x})^2}{n}$$

To obtain an unbiased estimate of the variance, n is replaced by n - 1.

Volumetric runoff coefficient. The proportion of the rainfall on the catchment which enters the storm drainage system.

Vortex overflow. A type of storm overflow which makes use of the spiralling flow in a vortex to retain polluting material within the pipe system.

Water table. The surface within soil or rock strata at which ground water saturation occurs.

Wet well. The entry chamber in a pumping station from which water is pumped to a higher level.

APPENDIX 2

Notation

Dimensions and functions are given first; then follow symbols in alphabetical order followed by Greek symbols and subscripts at the end. Symbols are not always given a particular dimension, since they are sometimes used in theoretical explanations which are valid in any self-consistent set of dimensions.

Dimensions

ha	hectare
hr	hour
ins	inches
l	litre
m	metre
mg	milligram
min	minutes
mm	millimetre
s	second

Functions

exp	exponent
$\log_{10}$	logarithm to base 10
ln	logarithm to base e

Symbols

a_1, a_2, a_3, a_4, a_5, a_6	constants in runoff volume equation
A	contributing catchment area
A_f	cross sectional area of flow (m^2)
A_o	cross sectional area of orifice(s) (m^2)
A_{over}	effective impermeable area contributing to discharge over storm overflow (ha)
A_{st}	plan area of storage tank or wet well at pumping station (m^2)
A_u	impermeable area upstream of storm overflow (ha)
API5	five-day antecedent precipitation index (mm)
$API5_9$	API5 at 0900 hours (mm)
AR	calculated effective subcatchment area (m^2)
AREA	subcatchment area (m^2)
AREAC	total catchment area (ha)
ARF	areal reduction factor
AT	catchment area (km^2)
B	width of water surface (m)
BAXTER	weighted average of Baxter cost indices
BAXTERn	Baxter cost index number n (n= 1, 2, 3 and 5)
C	coefficient in Rational Method
c_b	constant in Bilham formula
C_o	discharge coefficient for orifice
C_p	weighting coefficient in calculation of API5 (= 0.5)
C_r	constant in calculation of MT-D
C_R	routing coefficient in Modified Rational Method
C_v	volumetric coefficient in Modified Rational Method
C_w	discharge coefficient for overflow weir
d	pipe diameter (m)
d_{90}	size of sand grains than which 90 per cent of roughness projections are smaller (mm)

D	rainfall duration (hr)
D'	percentage duration
D_a	average depth to invert (m)
D_e	depth to formation level (m)
DEPSTOG	depth of depression storage (mm)
e	pipe joint eccentricity (mm)
f_1, f_2	constants in calculation of areal reduction factor
F_b	pipe bedding factor
g	acceleration due to gravity (m/s^2)
h	depth of flow
$\bar{h}_w$	mean of switch on and switch off levels in the wet well of a pumping station
H	design head of a pump
H_o	head difference across orifice
H_w	head above weir crest
i	rainfall intensity (mm/hr)
$\bar{i}$	mean rainfall intensity (mm/hr)
I	rainfall intensity (ins/hr)
J_0, J_1, J_2	constants in calculation of C_r
k_m	headloss coefficient at a manhole
k_o	proportion of discharge above Q_s permitted to continue along main pipe at a storm overflow
k_r	coefficient of a non-linear reservoir equation
k_s	equivalent sand roughness (mm)
k'_s	k_s value adjusted for eccentricity (mm)
K_F	storage coefficient in Muskingum-Cunge method
L	pipe length between manholes (m)
Lj	distance between pipe joints (mm)
L_w	length of overflow weir (m)
LENGTH	length of overland flow path in a subcatchment (m)
m	number of independent variables in a regression analysis
MT-D	rainfall depth for a duration D and return period T (mm)
n	an integer number (also used in equation 8.2 for Manning resistance coefficient)
N	number of occurrences of a rainfall event in 10 years
NGULLS	number of gulleys per subcatchment
p	rainfall depth in a time increment (mm)
p'	filtered rainfall depth in a time increment (mm)
P	rainfall depth (mm)
P'	percentage cumulative rainfall depth
$P_{t'-9}$	rainfall between 0900 hours and t' hours (mm)
PAPG	paved area per gulley (m^2)
PIMP	percentage of catchment covered by impervious surfaces intended to drain to the storm sewer
PR	percentage runoff from total catchment area
PR_{pav}, PR_{roof}, PR_{perv}	percentage runoff from paved, roof and pervious areas respectively
q, Q	discharge
Q_d	design flow
Q_{des}	design discharge of a pump
Q_{fb}	pipe-full discharge
Q_{in}	inflow discharge
Q_{max}	maximum permitted discharge downstream of a storm overflow
Q_n	discharge at normal depth
Q_{orfc}	discharge through an orifice
Q_{out}	outflow discharge
Q_{over}	discharge diverted over a storm overflow or out of an off-line tank
Q_p	peak discharge

Q_{pump} pumped discharge
Q_{ret} return discharge from an off-line storage tank
Q_s discharge at a storm overflow at which spill begins
$Q_{surcharged}$ discharge along a surcharged pipe
Q_{surf} flow calculated by surface runoff model (l/s)
r ratio of rainfall depths $\frac{\text{M5-60 min}}{\text{M5-2 days}}$
r_b rainfall depth in Bilham formula (ins)
r_c correlation coefficient
R hydraulic radius (m)
RUNVOL volume of runoff (mm over total area)
s hydraulic gradient
s_p pipe gradient
S storage volume
SAAR standard average annual rainfall, 1941-70 (mm)
SLOPE subcatchment slope (per cent)
SLOPEC catchment slope (per cent)
SMD soil moisture deficit (mm)
SMD_9 soil moisture deficit at 0900 hours (mm)
SOIL soil index
SPR constant part of percentage runoff for a given catchment
t time
t′ clock time of start of rainfall event (hours)
t_c time of concentration (min)
t_e time of entry (min)
t_f time of flow (min)
T return period (years)
UCWI urban catchment wetness index
V flow velocity
W width of trench (m)
x values used in statistical analysis
$\bar{x}$ average value of x
x′ value of x predicted from a regression analysis
x_1, x_2 constants in rainfall intensity-duration relationship
y water level at a storage tank or wet well of pumping station
Z1 coefficient in calculation of M5-D
Z2 coefficient in calculation of MT-D
Δh head loss along a pipe
Δt time interval (sec)
ε proportionality coefficient in Muskingum-Cunge method
λ Darcy-Weisbach friction coefficient
μ filter parameter
ν kinematic viscosity
σ standard deviation
ω kinematic wave speed

Subscripts

pav paved area
perv pervious area
pt sum of paved and pervious area
roof roof area

APPENDIX 3

The Working Party on the Hydraulic Design of Storm Sewers

A.3.1 Formation and membership

The need for improvements to available techniques for the design and simulation of storm drainage was voiced most clearly at the research colloquium entitled *'Rainfall, runoff and surface water drainage of urban catchments'* organised by the Construction Industry Research and Information Association in 1973 [20]. Following this colloquium, research projects were initiated by several organisations and the Working Party on the Hydraulic Design of Storm Sewers was established in March 1974 by the Department of the Environment's Directorate General of Water Engineering, with the following terms of reference:

'To examine all aspects of the hydraulic design of systems for the conveyance of storm water from developed areas: to assess and co-ordinate research projects in progress: to promote any necessary new research both in the laboratory and in the field: and to publish guidance and produce a manual of good practice for the design of such systems'.

Membership of the Working Party was drawn from practising engineers, research groups and government departments; the practising design interests were representative of water authorities, local government and consulting firms. Although membership has varied during the life of the Working Party, this balance of interest has been retained. A full list of members is given below.

Following the reorganisation of the water industry, the Working Party was incorporated in 1975 within a new structure of water-orientated committees of the Department of the Environment and the National Water Council. Since then the Working Party has reported to the Standing Technical Committee on Sewers and Water Mains.

A.3.2 Summary of activities

The main Working Party has met approximately quarterly since its inception. Subgroups were formed for several purposes, particularly for drafting sections of this report, and these have met on various other occasions. Occasional progress reports have been issued by the Working Party[102,103,104,16]. The main activities carried out by the Working Party may be summarised as follows:

(i) A survey of current design practice was carried out in 1974 and the results published in the first progress report [102]. This survey provided a valuable nationwide picture which supplemented the detailed knowledge of members of the Working Party.

(ii) A review of available design methods was carried out by the Hydraulics Research Station [18]. This provided guidance concerning the form which the new programs for design and simulation should take.

(iii) Research progress at the main organisations involved was continually reviewed and commented upon, so that the procedure under development would benefit from the experience of practising engineers. Longer-term research needs have also been identified.

(iv) First drafts of this report were prepared under the guidance of subgroups on rainfall, surface runoff, pipe flow, and costs and economics.

(v) The computer programs were tested by practising engineers serving on the Working Party and by other organisations having no previous connection with the research work. In this way the contents and recommendations of the procedure have been refined and tested to the point that they can be confidently recommended.

(vi) Complete drafts of this report were reviewed by the Working Party between 1979 and 1981.

A.3.3 Future responsibilities

The Standing Technical Committee on Sewers and Water Mains will continue to monitor the use of the procedure during the initial years of its application, and will consider any suggestions for further improvements. The research organisations which have made the major contributions to the development of the procedure will also be available to advise users over any problems experienced. As the results of further research become available their implications upon the present programs will be examined, and any beneficial modifications will be recommended. To avoid excessive variation and consequent confusion, it is hoped that such modifications need be issued at intervals of not less than two years. Information concerning such modifications will be published widely through the National Water Council and the technical press.

A.3.4 Working Party membership

The following were members of the Working Party at the time of the preparation of this report:

	Organisation	*Date of joining Working Party*
Dr D.E. Wright (Chairman)	Sir W. Halcrow and Partners and Halcrow-Balfour Ltd	Member: July 1974 Chairman: November 1976
D.H. Garside (Vice-Chairman)	Central Lancashire Development Corporation	Member: March 1974 Vice-Chairman: Nov. 1976
J. Bonsall	Water Directorate, Department of the Environment	September 1975
H.M.G. Cockbain	Welsh Water Authority	April 1977
W.R. Ferguson	Lothian Regional Council	January 1979
D. Fiddes	Water Research Centre	June 1979
C.K. Folland	Meteorological Office	March 1976
Dr M.J. Hall	Sir W. Halcrow and Partners	March 1974
E.W. Jones	Ministry of Agriculture, Fisheries and Food	March 1974
Dr M.J. Lowing	Institute of Hydrology	August 1974
D.B. Males	National Water Council	October 1979
A.J. Price	John Taylor and Sons	March 1974
Dr R.K. Price	Hydraulics Research Station	November 1975

N.G. Semple	Scottish Development Department	November 1976
P.J. Colyer (Editor)	Hydraulics Research Station	Secretary: March 1974 Editor: January 1980
E.J. Forty (Secretary)	Hydraulics Research Station	January 1978

The following served as members at earlier stages of the Working Party's activities:

	Organisation	*Dates*
T.W.G. Hucker	Directorate of Water Engineering, Department of the Environment	Chairman: March 1974 - November 1976
J.A. Cole	Water Research Centre	November 1975 - June 1979
J. Dugdale	Lothian Regional Council	March 1974 - December 1978
A.J.M. Harrison	Hydraulics Research Station	March 1974 - September 1975
G. Hedley	Severn Trent Water Authority	March 1974 - June 1979
J.F. Keers	Meteorological Office	March 1974 - March 1976
M.V. King	Wolverhampton MBC	April 1977 - September 1980
J.G. Munro	Scottish Development Department	March 1974 - November 1975
Dr R.B. Painter	Institute of Hydrology/National Water Council	March 1974 - June 1975
J.M. Pettigrew	Scottish Development Department	November 1975 - September 1976
H.S. Tricker	National Water Council	May 1974 - June 1977
S.F. White	National Water Council	September 1977 - July 1979
G.M. Wilkinson	Stevenage Development Corporation	March 1974 - September 1975
J. Wilson	Scottish Development Department	September 1976 - November 1976
C.P. Young	Transport and Road Research Laboratory	March 1974 - November 1976

The following served as co-opted members of drafting groups responsible for preparation of sections of this report:

D.M. Farrar	Transport and Road Research Laboratory
R. Giles	Water Research Centre
Dr J.A. Green	Local Government Operational Research Unit
Dr C.H.R. Kidd	Institute of Hydrology
Dr G. Mance	Water Research Centre
Dr E. Penning-Rowsell	Middlesex Polytechnic
J.A. Perkins	Hydraulics Research Station
E.W. Skerry	Severn Trent Water Authority

APPENDIX 4

Sources of information and advice

Advice on the application of the new procedure is being co-ordinated by the Hydraulics Research Station at the following address:

Hydraulics Research Station,
Howbery Park,
Wallingford,
Oxon, OX10 8BA

More specialist advice on the rainfall or above-ground flow aspects of the procedure may be obtained from the Meteorological Office and the Institute of Hydrology respectively, at the following addresses:

Meteorological Office,
Met 08,
London Road,
Bracknell,
Berks

Institute of Hydrology,
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The Standing Technical Committee on Sewers and Water Mains will monitor the use of the procedure and will publish revisions and enhancements as necessary.

APPENDIX 5

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