

**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 4

The Modified Rational Method

**Department of the Environment
National Water Council
Standing Technical Committee reports
No 31**

Hydraulics Research Limited
Howbery Park
Crowmarsh
Wallingford
Oxfordshire OX10 8BA
©HR 1981 ISBN 0 946466 041

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.

ISBN 0 901090 28 X.

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.

ISBN 0 901090 30 1.

Modified Rational Method (Volume 4)

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.

ISBN 0 946466 041.

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.

Contents of Volume 4

Page

1. Background	3
2. The Rational formula	3
3. Scope of the method	3
4. Summary of data requirements	4
5. Application of the method	5
6. References	6
APPENDIX Calculation of rainfall intensities for any location in the United Kingdom.	7

The Modified Rational Method

1. Background

The Rational (or Lloyd-Davies) method is in widespread use and provides a convenient and easily understood tool for design. Studies^{1,2)} have shown the variable accuracy of the method as normally applied in the UK, and recent research has suggested ways in which the method could be improved. A Modified Rational Method has therefore been included in the procedure for the design and analysis of storm drainage networks produced for the DoE/NWC Working Party on the Hydraulic Design of Storm Sewers.

The purpose of this brief volume is to explain the application by hand calculation of the Modified Rational Method. The modifications are concerned with the values used for the coefficient and the time of entry. A simple method for determining the appropriate rainfall intensities for any location in the UK is also recommended. Details of the other methods in the procedure, and of the relevant computer programs, are available in other volumes of this report.

2. The Rational Formula

The method gives the peak discharge from the equation:

$$Q_p = CiA \quad \dots\dots(1)$$

where Q_p is the peak discharge

C is a dimensionless coefficient

i is the average rainfall intensity during the time of concentration

and A is the contributing catchment area.

Additional factors may be necessary to allow for the dimensions used. If Q_p , i and A are expressed in l/s, mm/hr and ha respectively, equation 1 becomes

$$Q_p = \frac{CiA}{0.36} = 2.78 CiA \quad \dots\dots(2)$$

3. Scope of the method

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 conditions. It provides only a value of the peak runoff discharge; the hand calculation presented here cannot deal with sewer structures such as storm overflows. A computer version which can deal with storm overflows is available as part of the larger procedure. Networks incorporating more complex features such as storage tanks or pumping stations should be analysed using one of the hydrograph methods available.

Tests have shown³ that the Modified Rational Method is as accurate for the determination of peak runoff discharge as some more sophisticated urban runoff methods. These tests were limited to urban catchments up to 150ha in area with times of concentration up to about 30 minutes and outfall pipe diameters up to about one metre. The slope and distribution of impervious area in these catchments were reasonably uniform. The accuracy of the method when applied to larger or more irregular catchments is not known, and therefore the method cannot be positively recommended outside these limits.

4. Summary of data requirements

The data required for applying the method are:

(a) Details of the proposed or existing catchment, including the impervious area contributing to each pipe length and the gradient of each pipe length.

(b) A tabulation or graph of rainfall data relating average intensity to duration and the required return period(s). Such data for a specific location in the UK may be obtained from the Meteorological Office (Met 08, London Road, Bracknell) or may be derived by a simple hand calculation described in the accompanying appendix. The relevant return period is usually specified by the drainage authority.

(c) Suitable values for the coefficient C and the time of concentration t_c , determination of which is described below:

Determination of C

The coefficient C may be regarded as a combination of two separate coefficients:

$$C = C_v C_R \quad \dots \dots (3)$$

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

Value of C_v

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

The recommended value of C_v is affected by whether the whole catchment is being considered (impervious areas and pervious areas), or the impervious areas alone. For the purpose of this volume it is assumed that impervious areas (paved and roof) alone will be used; alternative approaches are described in volume 1.

An extensive study of runoff data from sewered urban catchments⁴ showed that the volume of runoff was related to the impervious area, the soil type and the catchment wetness. An approximate result may be obtained by assuming that the runoff derives from a proportion of the impervious area (paved and roof), the proportion varying according to soil type. On this basis the overall average value of C_v is about 0.75, ranging from about 0.6 on catchments with rapidly-draining soils to about 0.9 on catchments with heavy soils.

These values reflect the loss of some rainfall from impervious areas through cracks and into depressions and by drainage onto pervious (unpaved) areas. Similarly, any runoff from the pervious areas onto the impervious areas is also incorporated.

The above values of C_v should therefore be used in conjunction with the total impervious area (paved and roof) intended to drain to the storm sewer system.

Alternative methods of determining C_v which take account of specific soil characteristics and regional variations in catchment wetness are described in Volume 1 of this report and in reference 4.

Value of C_R

The routing coefficient C_R depends on the shape of the time-area diagram and on the variation of rainfall within the time of concentration. Examination of typical time-area diagrams, rainfall profiles and rainfall-runoff data led to the recommendation of a constant value for C_R of 1.30 for both design and simulation.

Equation (2) may therefore be rewritten

$$Q_p = 2.78 \times 1.30 C_v i A = 3.61 C_v i A \quad \dots\dots(4)$$

Determination of time of concentration

The time of concentration t_c is defined by:

$$t_c = t_e + t_r$$

where t_e is the time of entry

and t_r is the time of flow through the pipe system to the point under consideration.

The time of entry t_e may be regarded as representing the delay and attenuation of the flow over the ground surface. It 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 research⁵ has shown that times of entry of 8 to 12 minutes are common amongst frequently occurring storms. However the data set used in this exercise was biased towards small rainfall events equivalent to a return period of a few weeks or months. For larger return periods, times of entry were developed which gave surface runoff attenuation similar to that given by the surface runoff part of the more sophisticated hydrograph methods described in Volume 1.

This analysis led to the following recommended values for the time of entry:

<i>Return period</i>	<i>Time of entry (minutes)</i>
5 years	3 – 6
2 years	4 – 7
1 year	4 – 8
1 month	5 – 10

For each return period the larger times of entry are applicable to large, flat subcatchments (area greater than 400m², slope less than 1 in 50) and the smaller values to small, steep subcatchments (area less than 200m², slope greater than 1 in 30). Note that these values of area and slope refer to the subcatchments contributing to each pipe length.

The time of flow t_r through the pipe system may be determined from the pipe full velocity given in design tables⁶. This velocity gives a good approximation to the actual water velocity at all depths likely to occur under design conditions.

5. Application of the method

Given the pipe network and rainfall intensity data and suitable values for C_v and t_e as described above, the Modified Rational Method is applied as described below. A blank copy of a suitable calculation sheet is included in this volume, and may be copied for use.

For ease of reference, the pipe network is given a reference system of branch numbers (b) and pipe numbers (p) such that each pipe has a unique reference b.p. Steps 1 to 8 are repeated for each pipe in the network.

1. If a new pipeline is being designed an initial diameter equal to the minimum permitted diameter is assumed. Further downstream an initial diameter equal to the diameter of the largest incoming pipe at the upstream manhole is assumed.
2. The pipe full velocity is obtained from design tables⁶ and the time of flow along the pipe is determined.

3. The time of concentration is taken as the cumulative time of flow plus a time of entry. The time of flow through long lengths of carrier pipe (i.e. pipes with no contributing sub-catchment) should not be included in the calculation of the time of concentration for that pipe. It should, however, be included in the calculations for any subsequent pipes downstream.
4. A rainfall intensity corresponding to the time of concentration and the required return period is read from the rainfall data.
5. The cumulative impervious area is determined. It is assumed that additional area is added at the upstream end of the pipe length to which it contributes.
6. The peak discharge is calculated from equation (4). Any dry weather flow is added to give the total discharge. Dry weather flow may be determined from local information or standard calculation methods.
7. If the pipe is being designed, the smallest available diameter which will convey Q_1 , is determined from design tables⁶. If this diameter is equal to or less than the diameter originally assumed, the diameter given by the design tables is accepted (if permitted by local restrictions on minimum diameter and non-decreasing diameter). Otherwise, steps 1 to 7 are repeated using a larger assumed diameter.
8. Flow velocities in the pipeline should be checked from design tables⁶ and compared with minimum velocity requirements. If the pipeline is being designed, the diameter or gradient may be adjusted if necessary. Adjustment of the gradient may affect the gradients of adjacent pipes.
9. For the pipe 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.

6. References

1. P. J. Colyer. Performance of storm drainage simulation models, Proc ICE, 63, Part 2, June 1977, pp 293-309.
2. C. H. R. Kidd and J. C. Packman. Selection of design storm and antecedent condition for urban drainage design, Institute of Hydrology, Report No 61, 1979.
3. R. W. Pethick. Comparative testing of urban drainage models, Hydraulics Research Station report, in preparation.
4. C. H. R. Kidd and M. J. Lowing. The Wallingford urban subcatchment model, Institute of Hydrology, Report No. 60, 1979.
5. C. H. R. Kidd (ed). Rainfall-runoff processes over urban surfaces, Proceedings of an International Workshop, Institute of Hydrology, Report No. 53, 1978.
6. Hydraulics Research Station. Tables for the hydraulic design of pipes, Third Edition, HMSO, 1977.

APPENDIX

Calculation of rainfall intensities for any location in the United Kingdom

Introduction

The following manual method for calculating rainfall intensities for urban drainage design is reproduced from volume 1 of this report. The method was developed by the Meteorological Office as a simplified version of a computerised method; output from the computerised version may be obtained for a specific location either from the Meteorological Office (Met 08, London Road, Bracknell) or by application of the computer programs for storm drainage design and analysis.

Summary of the method

The manual method permits the calculation of rainfall intensities for durations between 5 minutes and 48 hours, and return periods between one year and 100 years. A graph of intensity against duration may be produced from about ten values at each required return period. The steps involved are:

1. Read from maps the value of the five year – 60 minute rainfall depth (described by the notation M5-60min) and the ratio (r) of the five year – 60 min rainfall depth to the five year – two day rainfall depth.
2. Determine the rainfall depths of five year return period for all required durations (described as M5-D).
3. Convert the five year rainfall depths to rainfall depths of the alternative return period(s) required (MT-D).
4. Convert the rainfall depths into point intensities.
5. Apply an areal reduction factor if required.

Application of the method

1. Determination of M5-60 min and r

Values of M5-60min and r are obtained from Figures A.1 and A.2. M5-60min should be estimated to the nearest mm (or half mm in areas of weak gradient); r should be estimated to ± 0.01 .

2. Determination of M5-D

M5-D is obtained from the relationship:

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

The factor $Z1$ is read from Figures A.3a or A.3b for values of r between 0.12 and 0.45 and for durations between 5 minutes and 48 hours. $Z1$ should be read from the graphs to an accuracy of about 0.01.

3. Determination of MT-D

MT-D is obtained from the relationship:

$$MT-D = Z2(M5-D)$$

The factor Z2 is read from Table A1 for England and Wales and Table A2 for Scotland and Northern Ireland.

4. Determination of point rainfall intensities

The rainfall intensity i of return period T and duration D is obtained from:

$$i = \frac{MT-D}{D}$$

5. Application of areal reduction factor

The intensities determined in the previous step are multiplied by the appropriate areal reduction factor read from Figure A.4. In most urban drainage catchments the areal reduction factor will be greater than 0.9.

Example

The following example illustrates the use of the method to provide the two year 30 minute rainfall intensity for a location in Oxford (GR 45002100)

1. From Figure A.1 M5-60min = 20 mm
from Figure A.2 r = 0.42
2. From Figure A.3b Z1 = 0.79
therefore M5-30 min = 0.79 × 20 = 15.8mm
3. From Table A1, for two year return period
Z2 = 0.80
therefore M2-30min = 0.80 × 15.8 = 12.64 mm
4. Average point intensity = $\frac{12.64}{30/60} = 25.3$ mm/hr
5. If the catchment with a 30 minute time of concentration has a total area of 2 km²,
Figure A.4 gives an areal reduction factor of about 0.94.
therefore average areal intensity = 0.94 × 25.3 = 23.8mm/hr.

Table A1: 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 A2: 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

Modified Rational Method

Calculation sheet

SCHEME

1 Pipe number	2 Pipe length (m)	3 Pipe gradient	4 Assumed diameter (mm or m)	5 Pipefull velocity (m/s)	6 Time of flow (min) $\frac{(2)}{60 \times (5)}$	7 Time of concentration (min)	8 Rainfall intensity (mm/hr)

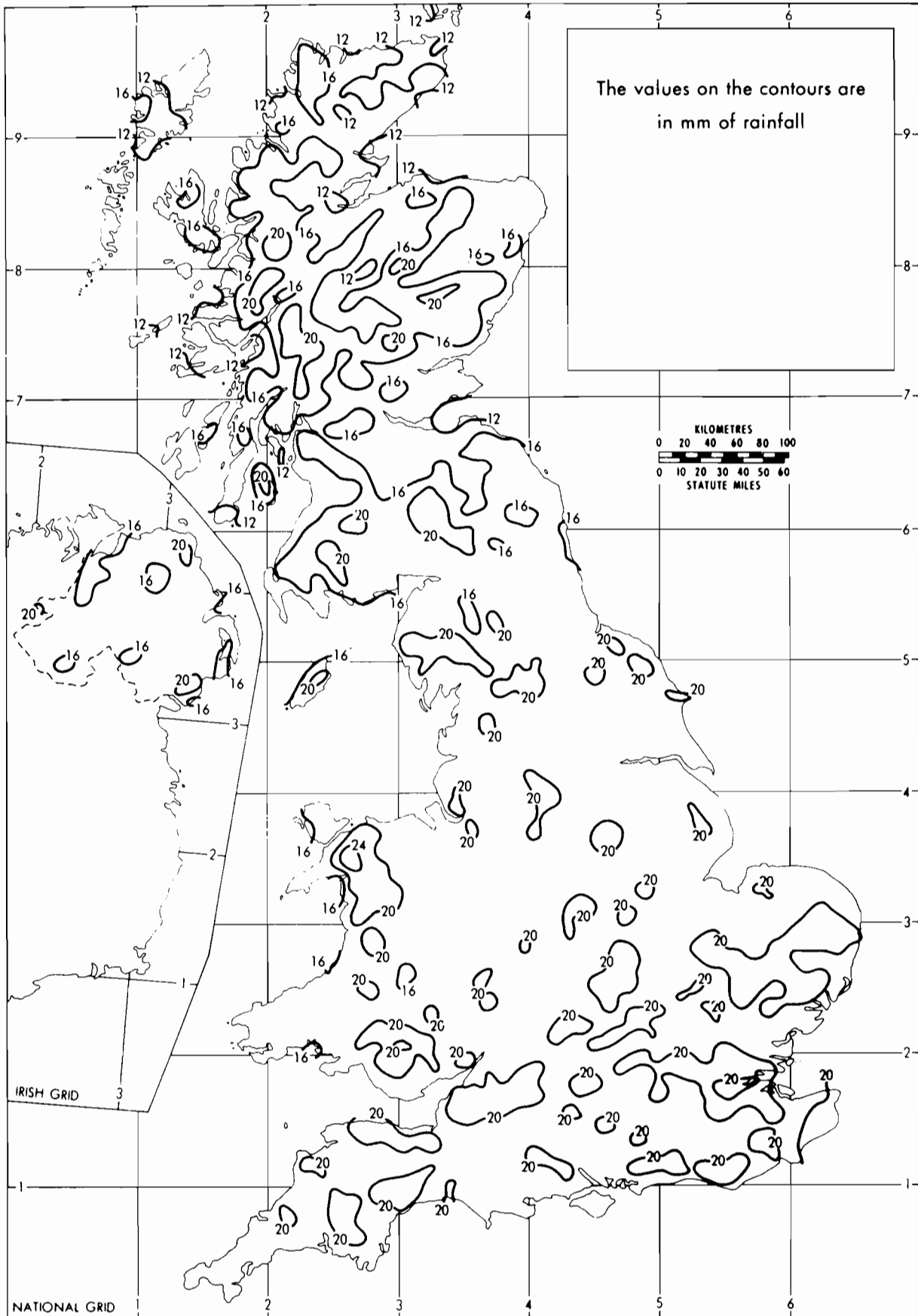
Rainfall return period..... years

Pipe roughness..... mm

Time of entry min

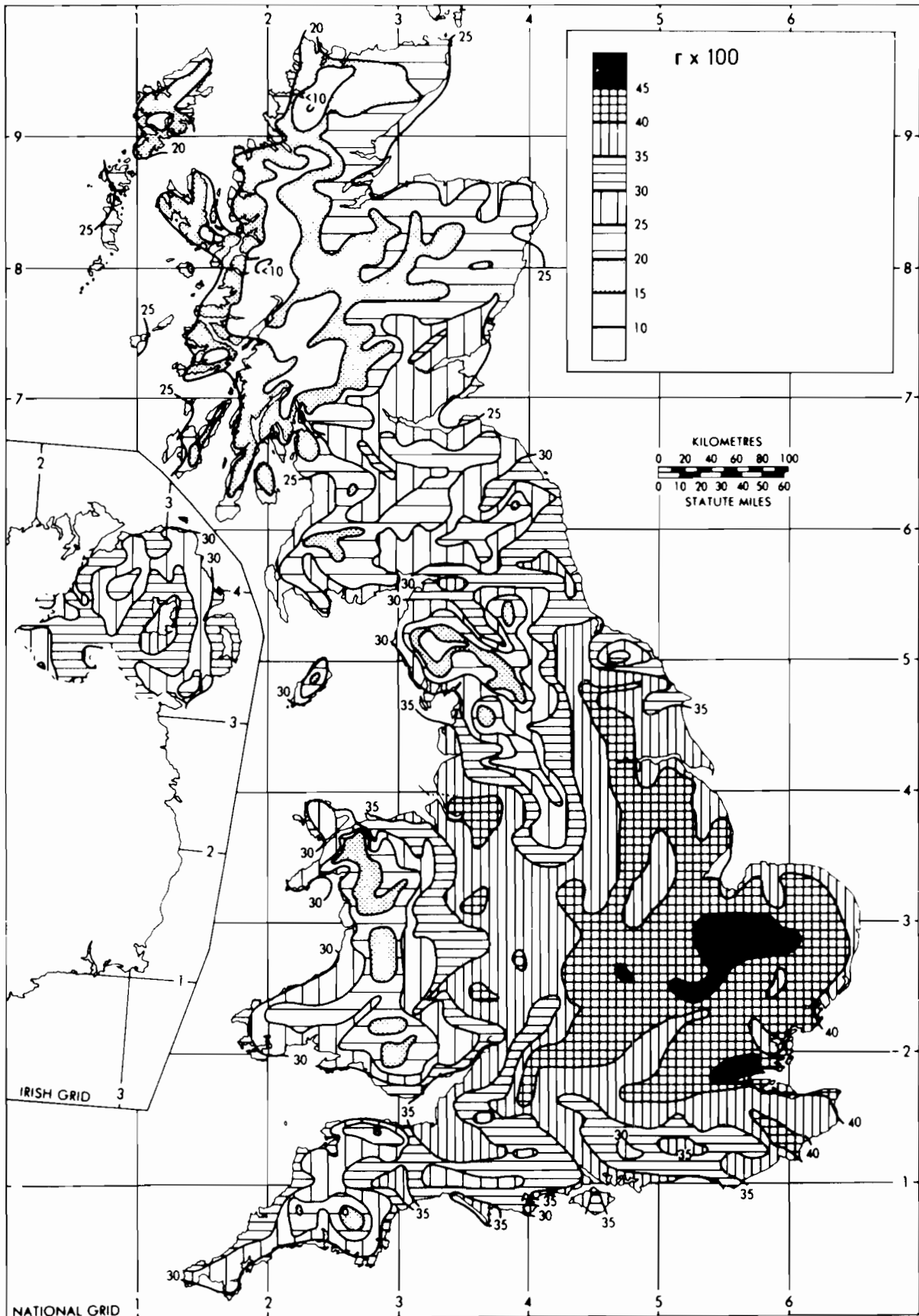
Coefficient C_v

9 Impervious area (ha)	10 Cumulative impervious area (ha)	11 Calculated discharge (l/s) 3.61C _v (8)(10)	12 Dry-weather flow (l/s)	13 Total discharge (l/s) (11) + (12)	14 Required diameter (mm or m)	15 Comments



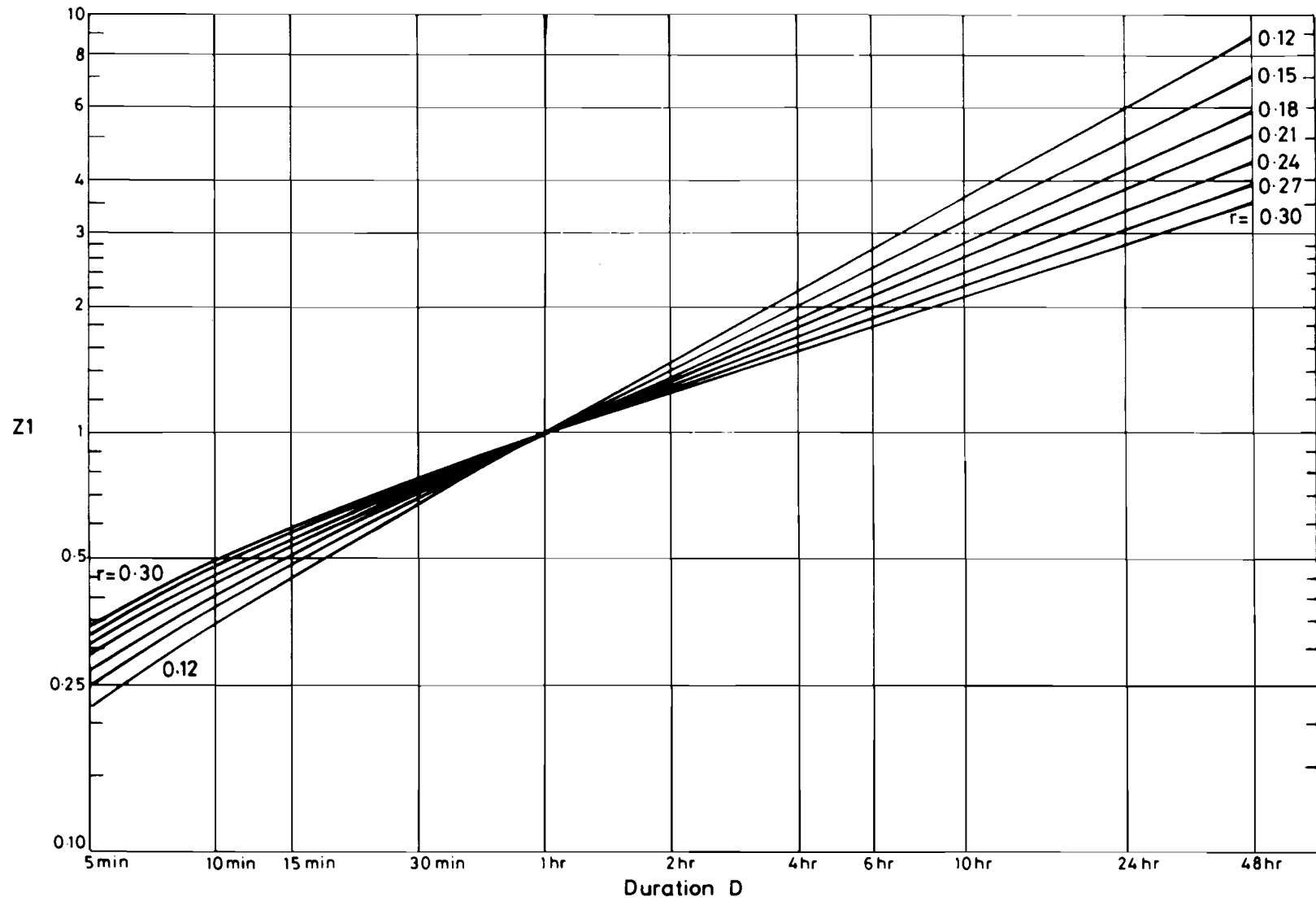
Rainfall depths of five year return period and 60 minutes duration (M5-60 min)

Fig.



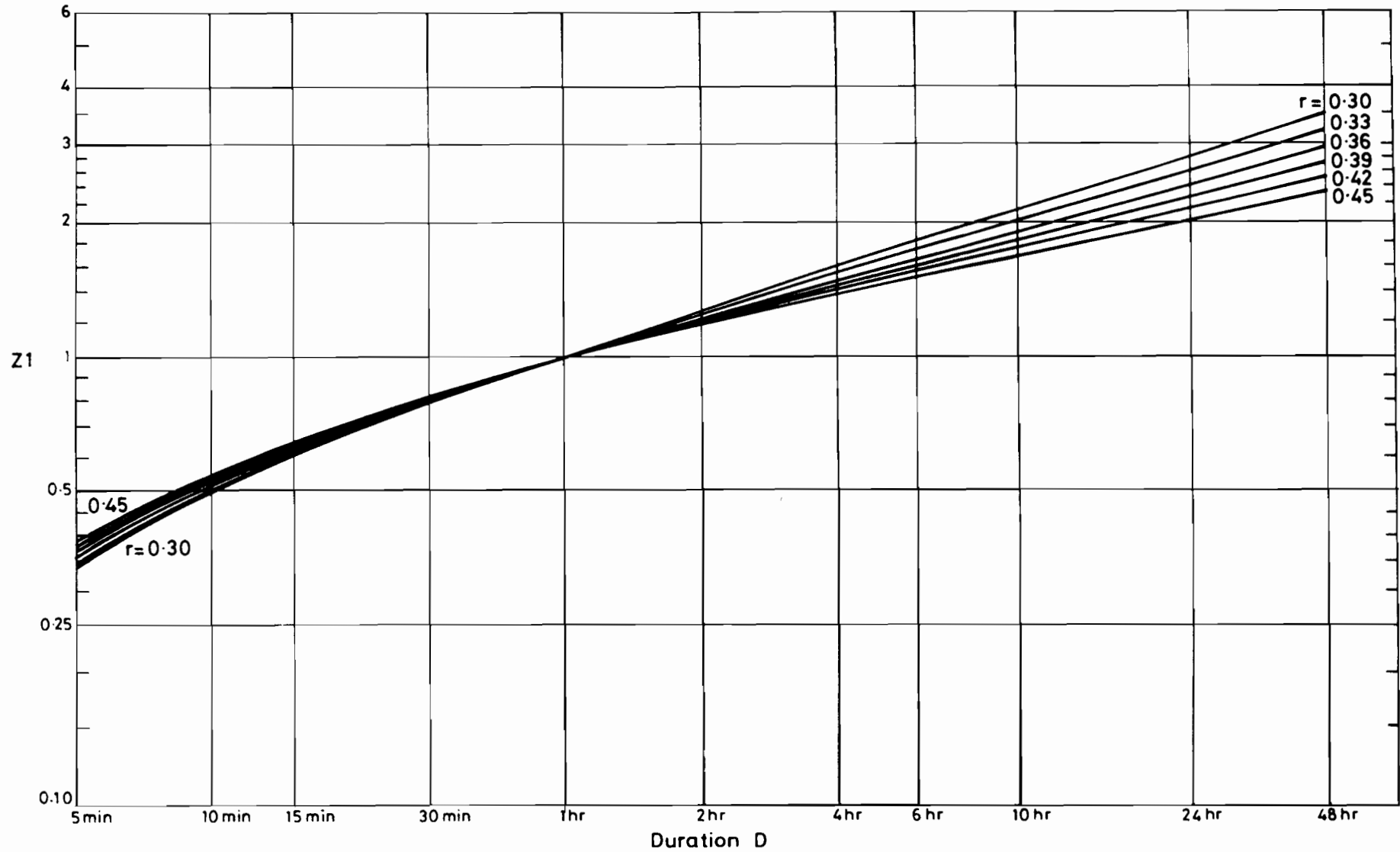
Ratio of sixty minute to two day rainfalls of five year return period (r)

Fig. A.2



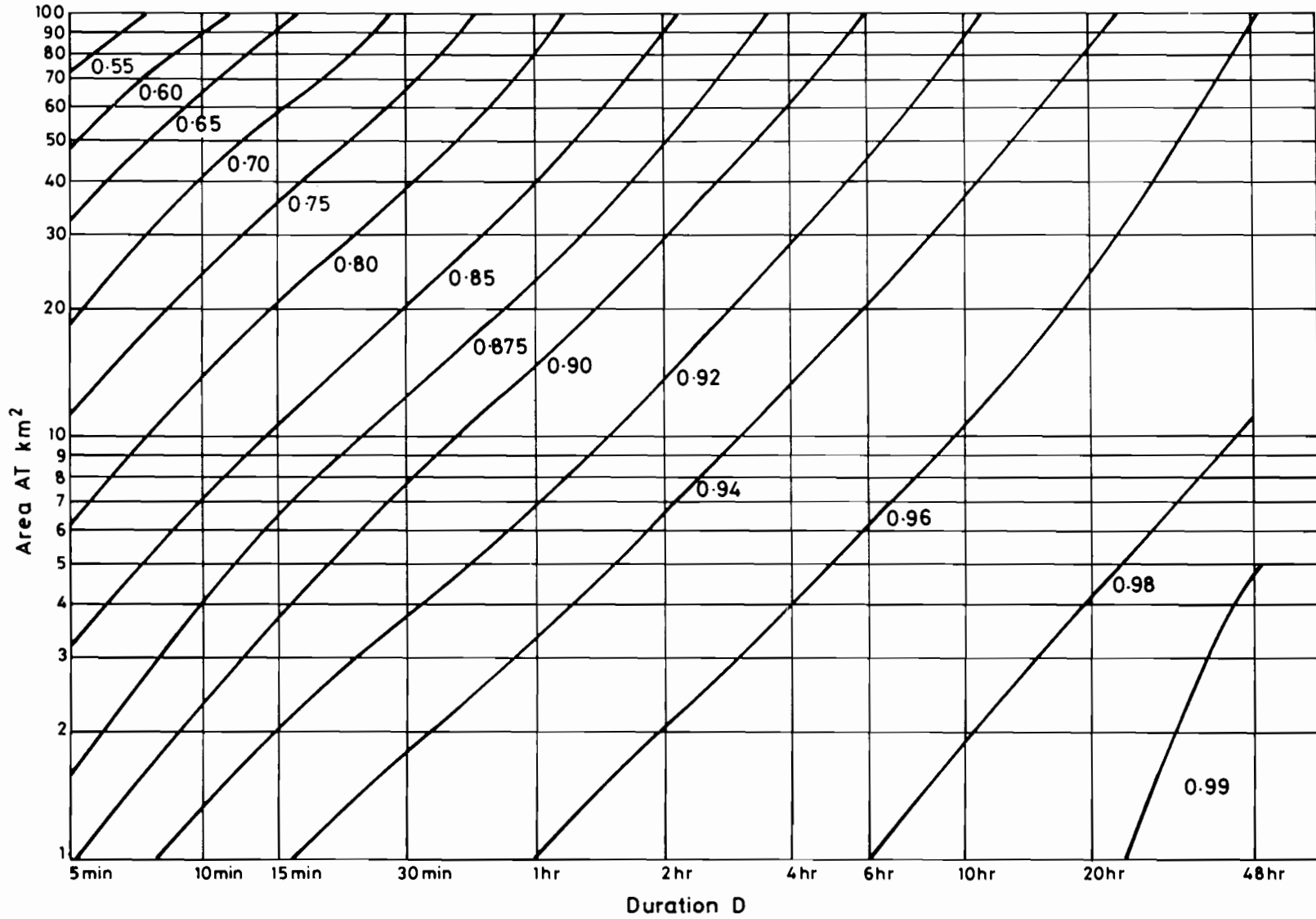
Relation between Z1 and D for different values of r. ($0.12 \leq r \leq 0.30$)

Fig. A.3a



Relation between Z1 and D for different values of r. ($0.30 \leq r \leq 0.45$)

Fig. A.3b



Areal reduction factor ARF related to Area AT and Duration D