Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme

Preliminary rainfall runoff management for developments

R&D Technical Report W5-074/A/TR/1
Revision E

Produced: January 2012

Author(s): R Kellagher

Statement of use
This document provides information for DEFRA and Environment Agency Staff about consistent standards for flood defence and constitutes an R&D output from the Joint DEFRA / Environment Agency Flood and Coastal Defence R&D Programme.

Contract Statement
This Guide was commissioned by the Environment Agency. The original project number was MAS0813. The project manager was Richard Kellagher with technical assistance provided by Ms V Jacot. The Environment Agency manager was Geoff Gibbs supported by Andrew Pepper.

Revision E was guided by Geoff Gibbs (Environment Agency) and produced by Richard Kellagher (HR Wallingford). This revision was an update to ensure relevant references were given along with the inclusion of another method of estimating greenfield runoff rates, using the Q_{MED} equation from the Flood Estimation Handbook (FEH). This report is produced as part of HR Wallingford project MAY0405.

The HR Wallingford reference for this document is SR 744.

Keywords:

Research contractor:
HR Wallingford Ltd
Howbery Park, Wallingford, Oxon, OX108BA
Tel: +44(0)1491835381
(For contractor quality control purposes this report is also numbered EX 6562)

Defra project officer:
Geoff Gibbs

Publishing organisation
Environment Agency
Horizon House
Deanery Road
Bristol BS1 5AH

Tel: 03708506506

Original issue: September 2005

Current version: January 2012

Executive summary

The purpose of this document
This Guide is aimed at Regulators, Developers and Local Authorities to provide advice on the management of stormwater drainage for developments and in particular to assist in sizing of storage elements for the control and treatment of stormwater runoff. The Guide is based on the requirements of this Interim Procedure which was commissioned by the Environment Agency.

This revision is produced to take account of changes and practices that have occurred since the document was first produced. The changes include the issue of PPS25 and the currently on-going development of the National SuDS Standards. In addition, although the basis for the analysis is the use of IH124, an additional method of calculation using the Q_{MED} estimation equation from the Flood Estimation Handbook (FEH) is also included now.

This Guide may be used to form part of a Flood Risk Assessment to comply with PPS25 and a Flood Consequence Assessment to comply with TAN 15, but it does not address issues such as risk of flooding from a watercourse, effects of changes in floodplain storage or in floodplain conveyance.

This Guide is aimed at providing a way of obtaining typical storage volume requirements for stormwater control for development sites by using a simple manual calculation procedure based on look up tables; the whole process is aimed at avoiding having to reference other documents or use software design packages. The objective is to allow any individual or organisation to be able to use a freely available tool to obtain an indication of the size (and therefore costs and space needed) to meet current stormwater control requirements. It is stressed that the approach provided here for sizing of stormwater storage is only to be used at the Planning stage to assist with defining indicative volumes. Detailed design of drainage systems will always require the use of stormwater software.

In addition to this paper-based system, a web based tool (www.uksuds.com) has been produced which is aimed at producing similar results. The information required by the web tool has been simplified to maximise its usability, and therefore the results do not exactly replicate this paper-based procedure. However the use of the web tool is approved by the Environment Agency for use in planning applications. The web based tool provides the facility to use both IH124 and the FEH formula in estimating values for Q_{BAR} and Q_{MED} and also permits a user defined value to be used.

The use of both formulae is approved in this revision, (as is the Revitalised Flood Hydrograph Model (ReFH) assuming suitably experienced users are involved in carrying out the analysis). The storage calculations can use either method, or a user-defined Q_{BAR} value. Growth curve factors are based on FSSR2 and 14. FSSR 14 can also be used to convert Q_{MED} to Q_{BAR} when Q_{MED} is derived).

The Guide is focused on only the storage aspects of stormwater design and does not address many other issues which are to be considered when carrying out drainage design and analysis.

Supporting explanations and examples have been provided to help engineers to use the method. The illustrations are based on five cities in UK which have a range of different hydrological and soil characteristics. Comparisons between the User Guide method and other modelling approaches show that the Guide method is generally conservative, but sufficiently accurate to provide a reasonable indication of the storage requirements.

The use of SuDS for stormwater storage

SuDS are aimed at addressing both treatment and hydraulic management of stormwater runoff. It is stressed that the Floods and Water Management Act 2010 requires the use of SuDS. Although storage can also be provided using underground storage systems, the Environment Agency very much prefer the use of surface level, vegetative systems to be used for conveyance and temporary storage (swales, basins and ponds etc).

Glossary

Adoption of sewers	The transfer of responsibility for the maintenance of a system of sewers to a Sewerage Undertaker.
Antecedent conditions	The condition of a catchment before a rainfall event.
Antecedent precipitation	The relevant rainfall that takes place prior to the point in time of interest.
Antecedent Precipitation Index (API)	Expressed as an index determined by summation of weighted daily rainfalls for a period preceding the start of a specific event.
Attenuation Storage	Temporary storage required to reduce the peak flow rate.
Base flow	Sustained or dry-weather flows not directly generated by rainfall. It commonly constitutes flows generated by domestic and industrial discharge and also infiltration or groundwater fed flows.
Brownfield site	A term used for to denote either the redevelopment of a previously developed site, or a contaminated site.
Catchment	A defined area, often determined by topographic features or land use, within which rain will contribute to runoff to a particular point under consideration.
Consent	Conditional permission granted by the appropriate public authority to discharge flows to a watercourse, into the ground or elsewhere.
Contributing area	The area that contributes storm runoff directly to the stormwater system.
Design storm	A synthetic rainfall event of a given duration and return period. It has been derived by statistically analysing a historical series of rainfall events for a specific location.
Development	The site area which is being considered for the drainage design. For the purpose of these analyses, all unmodified public open spaces should not be included.

Discharge	The volume of liquid flowing through a cross section of conduit per unit of time.
Drainage	A collection of pipes, channels and other engineering works designed to convey stormwater away from a built-up environment.
Erosion	Detachment and movement of soil or sedimentary deposits by the flow of water, caused by the flow of water over the ground surface or in a pipe or channel.
Event (rainfall)	Single occurrence of a rainfall period before and after which there is a dry period.
Extreme event (rainfall)	An infrequent large rainfall event
First flush	The initial discharge of active sediments and pollutants which often has higher than the average concentration of pollutants.
Flood Estimation Handbook	Fundamental revision and update to Flood Studies Report (Institute of Hydrology, 1999) for UK flood prediction.
Flood Risk Assessment	Technical review of the effects of a development on the risk of flooding on that development, and on adjacent sites downstream and sometimes upstream as well.
Flood Studies Report	Landmark report in UK for flood flow prediction (Natural Environment Research Council, 1975).
Flow regime	The typical variation of discharge of a waterway usually over an annual or seasonal period.
Frequency	The number of occurrences of a certain phenomenon per unit time.
Gradient	The angle of inclination (of pipe) which dictates its capacity and velocity of flow.
Greenfield/Greenfield Site	New development, usually on the periphery of existing urban areas. This creates increased rainfall-runoff and has an impact on existing sewer systems and watercourses.

Groundwater	Sub-surface water occupying the saturated zone from which wells and springs are fed. Strictly the term applies only to water below the water table.
Gully	A structure to permit the entry of surface runoff into the sewer system. It is usually fitted with a grating and a grit trap.
Head-discharge	The relationship between a discharge rate and the water level causing that discharge.
Hydraulic Control Unit	A hydraulic device to control the rate of flow.
Hydrograph	A graph showing, for a given point on a stream or conduit, the discharge, stage, velocity, available power, or other property (of water) with respect to time.
Impermeable surface	Surface which resists the infiltration of water.
Infiltration	The introduction of rainwater runoff into the ground.
Initial loss	In hydrology, the rainfall amount before which surface runoff occurs. It can include interception, surface wetting, and infiltration.
Intensity-duration-frequency	The relationship between rainfall intensity (amount per unit of time), duration of a rainfall event (time over which the average rainfall intensity occurs) and frequency (return interval) at which the specific intensity-duration relationship is expected to recur.
Interception	The process by which rainfall may be prevented from reaching the ground, for example by vegetation.
Land use	Catchments or development areas zones categorised by economic, geographic or demographic use.
"Long Term" Storage	Storage of stormwater which is drained slowly (less than 2 l/s/ha), preferably by infiltration.
Model	A series of mathematical equations in a computer developed and used with the aim of replicating the behaviour of a system.

Network	A collection of connected nodes and links, manholes and pipes when referred to in the context of sewers.
Orifice	A constriction in a pipeline to control the rate of flow.
Outfall	The point, location or structure where wastewater or drainage discharges from a pipe, channel, sewer, drain, or other conduit.
Overflow	The flow of excess water from a system when the capacity of that system is exceeded.
Overland flow	The flow of water over the ground, or paved surface before it enters some defined channel or inlet.
Peak discharge	The maximum flow rate at a point in time at a specific location.
Percentage runoff	The proportion of rainfall which becomes runoff, usually associated with a specific surface type.
Pervious area	Areas of ground which allows infiltration of water, although some surface runoff may also occur.
Pollution	The addition to a natural body of water of any material which diminishes the optimal use of the water body by the population which it serves, and has an adverse effect on the environment.
Rainfall intensity	Amount of rainfall occurring in a unit of time, generally expressed in mm/hr.
Rational Method	A simple method, used throughout the world, for calculating the peak discharge in a drainage system for pipe sizing.
Receiving waters	Water body (river or lake) which receives flow from point or non-point sources (outfalls).

Regulator

(1) A structure installed in a sewer, conduit or channel to control the flow of water or wastewater at an intake, or overflow or to control the water level along a canal, channel or treatment unit.

(2) The term used in UK to refer to the Environment Agency and other organisations which are involved in stipulating or applying legal requirements to the activities of organisations.

Return period

The reciprocal of the average annual probability of exceedence of a specific flow value or event.

Runoff

Water from precipitation which flows off a surface to reach a drain, sewer or receiving water.

Runoff coefficient

The proportion of total rainfall that appears as total runoff volume after subtracting depression storage, infiltration and interception.

Sediment

Organic or inorganic solid material originally carried by water, which has been deposited.

Sewerage Undertaker

An organisation with the legal duty to provide sewerage services in England and Wales.

Simulation

The representation of specific conditions during a specific period for one or more loading conditions (rainfall) in a drainage system, treatment works, river, etc., by means of a computer model.

Soakaway

An underground hole or gravel filled pit into which surface water is drained to infiltrate into the ground.

Soil Moisture Deficit (SMD)

A measure of soil wetness, calculated by the Meteorological Office in the UK, to indicate the capacity of the soil to absorb rainfall.

Storage

The impounding of water, either on the surface or in underground reservoirs.

Stormwater

The product of a meteorological event, often of rainfall, snow or hail, when it forms runoff due to an inability to permeate into the ground or falls onto impermeable surfaces.

Surface water	Water from precipitation which has not seeped into the ground.
Swale	The term given to a grass channel for stormwater collection with shallow side slopes and which is normally dry except during rainfall.
Time series rainfall	A continuous or discontinuous record of individual rainfall events generated artificially, or a selection of real historical events which are representative of the rainfall in that area.
Treatment Storage	Storage provided to enable polluted stormwater to be improved before discharging to receiving waters.
Urban drainage	Pipe systems and other related structures to serve an urban environment.
Wallingford Procedure	A design and analysis procedure for urban drainage networks. Produced by HR Wallingford and the Institute of Hydrology in 1981. Funded by DoE.
Wash off (of pollutants)	The transport of pollutant mass from a surface during a rainfall event.
Water quality	A measure of the chemical, physical and biological characteristics of a water body or sample.
Watercourse	A natural or artificial channel which conveys water.

Abbreviations

ADAS Agricultural Development and Advisory Service

API_5 Antecedent Precipitation Index (over previous 5 days)

CEH Centre for Ecology and Hydrology

CEN Comité Européen de Normalisation (European Committee for Standardisation)

CIRIA Construction Industry Research and Information Association

CWI Catchment Wetness Index from FSR

Defra Department for Environment, Food and Rural Affairs

FEH Flood Estimation Handbook (Centre for Ecology and Hydrology (CEH), 1999)

FSR Flood Studies Report (Institute of Hydrology, 1975)

FSSR Flood Studies Supplementary Reports (Institute of Hydrology, 1985)

IDF Intensity – Depth – Frequency (relationship)

IF Effective Impervious Area Factor

IH Institute of Hydrology (replaced by Centre for Ecology and Hydrology)

LTS Long Term Storage

$M_5 60$ The 5 year 60 minute depth of rainfall

NAPI New Antecedent Precipitation Index

NERC Natural Environment Research Council

PF Porosity Fraction (soil storage depth)

PIMP Percentage Impermeable proportion of a catchment or development contributing to runoff– see PR equation

PPS25 Planning Policy Statement 25 ‘Development and Flood Risk’, applicable to England and Wales

PR Percentage Runoff

Q_{BAR} An FSR term denoting the Mean Annual Flood flow rate for a river. This approximates to a return period of 2.3years

Q_{MED} A term used in FEH denoting the median annual maximum flow rate for a watercourse.

ReFH model The Revitalised Flood Hydrograph model is the latest FEH rainfall runoff method for UK design flood estimation (Kjeldsen, 2005)

SAAR Standard Average Annual Rainfall, an FSR parameter, assessed over a period of years – 1941 - 1970

SfA Sewers for Adoption (published by WRc); current version is the 7th Edition (2012)

SMD Soil Moisture Deficit

SOIL Soil type classification used by Institute of Hydrology, FSR, 1975 and the HR Wallingford and Institute of Hydrology, Wallingford Procedure, 1981

SPR Standard Percentage Runoff. Used in FSR and FEH equations

SuDS Sustainable Drainage Systems

TSR Time Series Rainfall

UCWI Urban Catchment Wetness Index – describes the wetness of the catchment, usually calculated for the start of a rainfall event

WRAP Winter Rainfall Acceptance Potential (used by the HR Wallingford and Institute of Hydrology, Wallingford Procedure, 1981)

Rainfall Runoff Management for Developments - Interim National Procedure Principles

1. **Procedure status.** This procedure will be replaced / updated as and when improved tools are developed. The web site www.uksuds.com provides a web based approximation to this tool.

2. **Procedure philosophy.** The principles under-pinning this procedure are that:
   - stormwater runoff discharged from urban developments should approximate to the site greenfield response over an extended range of storm frequencies of occurrence (return periods)
   - manage runoff on site for extreme events.

   This requires:
   - the **peak rate** of stormwater run-off to be limited
   - the **volume** of run-off to be limited
   - the **pollution** load to receiving waters from stormwater runoff to be minimised
   - the assessment of **overland flows and temporary flood storage** across the site.

3. **Compliance to national guidance.** The objective of this procedure is to assist developers and their designers to conform to PPS25 which requires the development to be 'safe, without increasing flood risk elsewhere' and, where possible, to 'reduce flood risk overall'. Sustainable drainage is the suggested approach to managing flood risk resulting from surface water runoff from the site.

4. **Application of the procedure.** This procedure applies to both greenfield and previously developed sites. It is in conformance with the anticipated requirements of the SuDS Standards and the Code for Sustainable Homes. Drainage proposals may be measured against the existing drainage performance of the site (although it is preferable for solutions to provide runoff characteristics which are similar to greenfield behaviour).

5. Sites with contaminated land will have particular consent requirements and affect the drainage techniques that can be used.

6. **Use of infiltration.** SuDS Standards and Part H of the Building Regulations require that the first choice of surface water disposal should be to discharge to infiltration systems where practicable. This should even be applied where infiltration can only account for a proportion of the runoff from the design event.

7. **Sewers for Adoption.** Drainage calculations and criteria, where appropriate, should comply with the latest edition of Sewers for Adoption, published by WRc.

8. **Discharge rate criteria.** The Environment Agency will normally require that, for the range of annual flow rate probabilities, up to and including the 1% annual probability (1 in 100 year event) the developed rate of runoff into a watercourse should be no greater than the undeveloped rate of runoff for the same event based on the calculation of Q_{BAR} or Q_{MED} and the use of FSSR growth curves. Exceptions only apply where it is not practical to achieve this due to either constraints on the size of the hydraulic control unit (see point 17), or excessive storage volumes. The purpose of this is to retain a natural flow regime in the receiving watercourse and not increase peak rates of flow for events of an annual probability greater than 1%. Three annual probabilities are used to define discharge compliance limits though the critical criteria are for the lowest and highest frequency events; 100% (1 year), 3.33% (30 year) and 1% (100 year). (Note that in many places elsewhere in this document that return periods are referred to instead of annual probabilities, as much historic nomenclature and many formulae use return periods).

8.1 **The 100% annual probability** (once in one-year event) is the highest probability event to be specifically considered to ensure that flows to the watercourse are tightly controlled for frequent events to provide good morphological conditions.

8.2 **The 3.33% annual probability** (once in 30 years event) is of importance because of its linkage with the level of service requirement of Sewers for Adoption 7th edition (SfA7). SfA7 requires that surface water sewers should be capable of carrying the 3.33% annual probability event within the system without causing flooding to any part of the site.

8.3 **The 1% annual probability** (once in 100 years event) has been selected since it represents the boundary between high and medium risks of fluvial flooding defined by PPS25 and also recognises that it is not practicable to fully limit flows for the most extreme events. Also SfA7 recognises that, during extreme wet weather, the capacity of surface water sewers may be inadequate. SfA7 requires that the site layout should be such that internal property flooding does not result, by demonstrating safe above ground flow paths. The return period for this analysis is not specified, but it is recommended that 1% annual probability event (i.e. an event with a return period of 100 years) is used.

8.4 **Flood flows.** Runoff up to the 1% annual probability event should preferably be managed within the site at designated temporary storage locations unless it can be shown to have no material impact by leaving the site in terms of nuisance or damage, or increase river flows during periods of river flooding. Analysis for overland flood flows within the site will need to use appropriate duration events which may be different to critical events for designing stormwater control storage structures.

9. **The calculation of greenfield runoff rate.** The methods appropriate for calculation of peak rates of runoff from a greenfield site are constrained by the data and analysis used to derive the various methods available. The sizes of most sites are far smaller than the area for which these equations were produced, therefore the values derived should be regarded as indicative due to these limitations. Table 1 summarises the techniques to be used. Discussion on the various methods is provided in this document and must be referred to.

Table 1 Methods to be used for calculation of greenfield run-off peak flow rates

Development size	**Method**
0 – 50 ha	One of two approaches can be used: 1. The Institute of Hydrology (IH) Report 124 Flood Estimation for Small Catchments (1994) method can be used to estimate the greenfield site flow rate, Q_{BAR} (the Mean Annual Flood). 2. The Index Flood, Q_{MED} (the median of the set of annual maximum flood peaks) regression equation that forms part of the FEH statistical method can also be used where the appropriate parameters are known or can be derived/ estimated. Where developments are smaller than 50 ha, the analysis for determining the greenfield index flood flow rate should use 50 ha in the formula and linearly interpolate the flow rate value based on the ratio of the development area. FSSR 14 can be used to convert Q_{MED} to Q_{BAR}. FSSR 2 and 14 regional growth curve factors can be used to calculate the greenfield peak flow rates for 1, 30 and 100 year return periods.
50 ha +	IH Report 124 or the FEH Q_{MED} equation from the statistical method can be used to calculate the greenfield site peak flow rates. FSSR 2 and 14 regional growth curve factors can be used to calculate the greenfield peak flow rates for other return periods. Where the site is hydrologically similar to the catchment in which it lies, additional FEH techniques can be used to predict flow peaks. Catchment-scale flow predictions will then need to be scaled appropriately to the site area. Where FEH techniques (the statistical method and/or the ReFH) are used, appropriate data, tools and skills must be applied.

10. **Volumetric criteria.** Theoretically the stormwater runoff volume from a site should be limited to the greenfield runoff volume for all event frequencies. However this is technically extremely difficult to achieve and therefore compliance to two criteria on runoff volume is required.

10.1 **Interception.** Where possible, infiltration or other techniques are to be used to try and achieve zero discharge to receiving waters for rainfall depths up to 5mm.

10.2 **Additional runoff due to development.** The difference in runoff volume pre- and post-development for the 100 year 6 hour event, (the additional runoff generated) should be disposed of by way of infiltration, or if this is not feasible due to soil type, discharged from the site at flow rates below 2 l/s/ha.

10.3 Where compliance to 100 year volumetric criterion is not provided, as defined in section 10.2, the limiting discharge for any return period up to the 100 year event shall not be greater than the mean annual peak rate of runoff for the greenfield site (Referred to as Q_{BAR} in IH Report 124) or 2 l/s/ha, whichever is the greater.

11. **Percentage runoff from greenfield sites.** The percentage runoff of the rainfall on a greenfield site can be assumed to be approximately equal to the SPR value of the soil type of the site. The SPR value can be used from either the Flood Studies Report (FSR) or the Flood Estimation Handbook (FEH).

12. **Percentage runoff from developments.** Calculation of the run-off volume from the developed site for preliminary assessment and design of drainage facilities will assume 100% run-off from paved areas and 0% run-off from pervious areas. This presumes that sites are developed with a degree of impermeability greater than 50%. Runoff from impermeable surfaces served by effective infiltration systems can be assumed to contribute no runoff for storage volumes assessment assuming the infiltration systems are designed to address the runoff from at least a 10 year event.

13. **Detailed design of stormwater runoff.** All network design for stormwater runoff and proof of compliance in meeting peak flow rate discharge criteria should use appropriate drainage simulation software tools and runoff models in accordance with current best practice.

14. **SuDS for water quality.** SuDS units should be used to achieve water quality improvements and amenity benefits as well as achieving compliance to these hydraulic criteria. Best practice in achieving water quality protection should be used.

15. **Climate change factor.** Climate change will be taken into account in all hydrological regions by increasing the rainfall depth by the amount recommended by the Environment Agency. Current recommendations, based on UPCP09, are presented in their document 'Advice to Flood and Coastal Erosion Risk Management Authorities' (Environment Agency, 2011) which advises an uplift on extreme rainfall intensities of 20% when designing to 2080's and beyond, for extreme events (ie > 1 in 5 years). There are also tables within the document that provide regional factors to be applied to river flows, for assessing flood levels. No allowance for climate change should be applied to calculated greenfield peak rates of runoff from the site for any hydrological region. It is recognised that an increase in rainfall will result in an increase in runoff, but that if the greenfield runoff formula is increased proportionately, that storage volumes will remain largely the same and this is not considered to be a suitable precautionary position.

16. **Urban Creep**. Urban creep is now an acknowledged issue which results in an increase in runoff from an estate over time. An allowance should be made by factoring the impermeability percentage by 1.1 (10% increase) unless a more precautionary requirement is specified by the local planning authority.

17. **Minimum limit of discharge rate.** A practicable minimum limit on the discharge rate from a flow attenuation device is often a compromise between attenuating to a satisfactorily low flow rate while keeping the risk of blockage to an acceptable level. This limit is set at 5 litres per second, using an appropriate vortex or other flow control device. Where sedimentation could be an issue, the minimum size of orifice for controlling flow from an attenuation device should normally be 150mm laid at a gradient not flatter than 1 in 150, which meets the requirements of Sewers for Adoption 7th Edition.

    A second minimum discharge limit based on 1l/s/ha for Q_{BAR} is also applied where soil types produce lower calculated values. This limit is applied to prevent the size of storage systems becoming unacceptably large and expensive.

18. **Local Flood Risk Management Strategies** (LFRMs) or Catchment Flood Management Plans (CFMPs) may exist for the catchment. These plans consider the impact of development on flood risk in the catchment based on existing land use plans published by the Local Planning Authority and projections of development. Where these explicitly define stormwater discharge limits for development sites which are relevant for the proposed development, these discharge limits will take precedence over the requirements of this document.

Acknowledgements

Guidance with regard to the contents of this first version of this Guide was provided by:

Mervyn Bramley	Environment Agency
Geoff Gibbs	Environment Agency
John Packman	CEH
Andy Pepper	Consultant to Environment Agency
Mervyn Pettifor	Environment Agency
Suresh Surendran	Environment Agency

Advice on the latest revision, Version E, was provided by Geoff Gibbs.

Particular thanks must go to UKWIR for allowing the use of FEH rainfall maps (which were produced for them by the Met Office), which enables this generic procedure to be applied using FEH rainfall characteristics for any location in the UK.

Contents

Tables

Figures

1 Scope of this User Guide

This User Guide is aimed at Developers and Local Authorities to advise on the requirements for stormwater drainage design and in particular for setting stormwater discharge limits from a site and to assist in the initial sizing of storage elements for the control and treatment of stormwater runoff which will be needed to meet these discharge limits. The reason that the Environment Agency commissioned this document was to enable the non-drainage expert, who does not have the appropriate industry standard tools, to obtain a quick assessment of the principal drainage requirements needed for a proposed development.

The Guide:

- Provides an easy-to-use manual method for assessing initial storage volumes for stormwater control for a development site;
- States the current Defra/Environment Agency position on stormwater management discharge control
- Provides some supporting advice, information on the assumptions used and some important technical issues.

Stormwater system design, with its emphasis on the use of SuDS and applying limiting discharge rates to receiving waters or drainage systems, has implications on both development costs and planning the layout of the site. This Guide is aimed at helping provide this information, recognising that other documents should be referred to for more detailed information.

This guide provides a quick manual method for initial sizing of storage for those who do not have access to industry standard drainage design tools. There is now also a simplified internet based tool which is based on this approach and aims to provide similar results – www.uksuds.com.

Those using this Guide must recognise its limited objectives and that other documents should be referred to as appropriate. These include:

- The Code for Sustainable Homes (2010)
- The SuDS Standards (2011)[1]
- Guidance on the SuDS Standards (2011)[2]
- The SuDS Manual CIRIA C697 (2006)
- The Interim Code of Practice for Sustainable Drainage Systems document produced by the National SuDS Working Group (2004)
- Drainage of Development Sites – A Guide (2003)

[1] *At Consultation stage at the time of issuing this report.*
[2] *Yet to be developed.*

This document is official Environment Agency policy on rainfall-runoff management for developments. This is an Interim national procedure (2011) version E as further research is taking place on updating best practice methods for assessing greenfield runoff. Version E has been produced to take into account changes that have occurred in science and policy since version D was produced.

2 Philosophy of current stormwater control requirements

The basic principle underpinning the requirements for controlling stormwater runoff and the use of the greenfield flow rate estimation methods are explained here.

2.1 Why is runoff control needed?

A development which does not control the runoff from rainfall results in greater volumes of runoff than the undeveloped site, and which discharges rapidly to receiving waters if drained using normal piped systems.

The principle being applied is that the post-development site should aim to try and replicate the un-developed state. Arguments can be made for an alternative strategy for runoff control to protect against flooding in a catchment, especially in situations where the catchment is heavily urbanised or there is a Local Flood Risk Management Strategy in place which defines regional flood risk control measures based on a detailed hydrological assessment.

There are 3 volume or flow rate criteria which should be met. These are:

- Interception – no runoff for up to 5mm of rainfall
- 1 year return period flow control – aimed at morphological protection of receiving streams
- 100 year return period flow and volume control - aimed at flood protection of those living downstream

2.2 Why is IH124 still used for defining the site limiting discharge rate?

IH 124 is a document that was issued in 1994 by the Institute of Hydrology. It was an extension of the FSR work aimed at providing a better estimate of peak runoff flow rates for small catchments (small in terms of river catchments) than had been previously developed. It is a correlation equation based on three parameters, (SOIL, SAAR and AREA) all of which are easily measured / obtainable. It is therefore very easy to use.

Since then FEH and ReFH have been developed and there is currently more research being carried out in this topic area. These tools are acknowledged to often provide better estimates of peak river flow rates than IH124, though none of the methods can be assumed to provide the “right” value – particularly at the development or plot scale (often less than 1 ha). However the following reasons are provided for the current continued use of IH 124:

1. An easy to use and consistent method of approach needs to be applied nationally.
2. There are copyright issues associated with the FEH parameters, which means that the FEH tool would have to be purchased if these methods are to be used.

3. The implementation of FEH and ReFH methods must be carried out by competent hydrologists otherwise the estimates cannot be considered to be robust.

Most importantly, the critical requirement is to use a method which is:
Objective, fair and practical, and which is effective in providing runoff flood protection.

Theoretically, any control rule which achieves this objective can be used and need not necessarily accurately reflect the greenfield runoff characteristics for a site. Furthermore, it should be noted that setting discharge limits as required by this document – 5mm Interception, 1 year and 100 year control rates – will not result in the runoff from a site replicating the undeveloped state. An accurate assessment of greenfield runoff is therefore not fundamental to the flood protection of catchments, it is just an approach which is believed to provide a consistent, simple and effective method for controlling stormwater runoff.

It is therefore considered important not to relinquish these principles of simplicity and objectivity by adopting a more accurate formula for estimating greenfield runoff rates unless the replacement approach also achieves the fundamental objectives of ease of use, and access by all stakeholders at minimal cost.

2.3 Why has the correlation equation from the FEH statistical method now been included for defining the site limiting discharge rate?

Not withstanding the arguments for the continued use of IH124, it is now proposed that an estimate of Q_{MED} is permitted as this approach largely complies with the same principles of simplicity and ease of use. In addition, recent research has shown that it is generally more accurate than IH124.

Although FEH soil parameters are used in this equation (see section 5), these are available in report IH126 which looks at the classification of soil types and provides the values for SPRHOST and BFIHOST for all soil categories found on soil maps.

In principle it is therefore to be used in just the same way as IH124 in that it is a simple correlation equation which can be applied to site specific parameters. It is stressed that in allowing the use of this equation that the pooling techniques and calibrating to flow data is not needed for sites less than 50ha and should only be carried out by competent hydrologists.

Section 5 in this document details both the Q_{MED} estimation equation from the FEH Statistical method as well as Q_{BAR} for IH124, and also provides suggestions on the use of the parameters. A brief overview and discussion of other greenfield runoff estimation methods are also provided.

3 Stormwater design and this document

This chapter briefly describes the key aspects of stormwater design with respect to the storage requirements needed to meet criteria related to limiting discharge from the site, and also water quality. It does not address all aspects of drainage design. It does not provide detailed guidance on the planning process, though it is worth stressing that many of the issues associated with drainage should be addressed at an early stage in the development planning process.

It should be noted that the storage design tools of this Guide and the associated website are only to be applied at initial planning and design stages and that detailed computer models would normally be expected to be used for detailed design of all aspects of the drainage system.

Rainfall runoff from greenfield areas has very different characteristics to development runoff. These differences can be summarised under three main categories:

- Volume of runoff
  - No runoff for small events
  - Less runoff for large events
- Rate of runoff
  - Slower, later runoff for all events
- Quality of runoff
  - Cleaner runoff (BOD, sediment, pathogens, metals, hydrocarbons)

The objectives of the storage criteria are to address these three aspects and to control the urban runoff to mimic, as practicably as possible, the greenfield behaviour of the site. To do this, storage is specifically and separately calculated to address each of these criterion, and the means by which this may be achieved is briefly explained below.

3.1 Volume of stormwater runoff – very small rainfall events

<u>Interception storage</u>

The volume of rainfall runoff is important at each end of the rainfall event spectrum. Around 50 percent of rainfall events (probably in excess of 70 events a year in most areas), are less than 5mm and cause no measurable runoff from greenfield areas into receiving waters. In contrast, runoff from a development takes place for virtually every rainfall event. This difference means that streams receive frequent discharges with polluted washoff from urban surfaces (hydrocarbons, suspended solids, metals etc). Replication of the greenfield runoff from small events will result in many fewer polluted discharges so limiting the potentially damaging impact on the receiving environment.

The concept of Interception storage to prevent any runoff from rainfall depths up to 5mm, should therefore be provided. Certain SuDS features such as Swales and Pervious Pavements provide runoff characteristics that reflect this behaviour depending on their design.

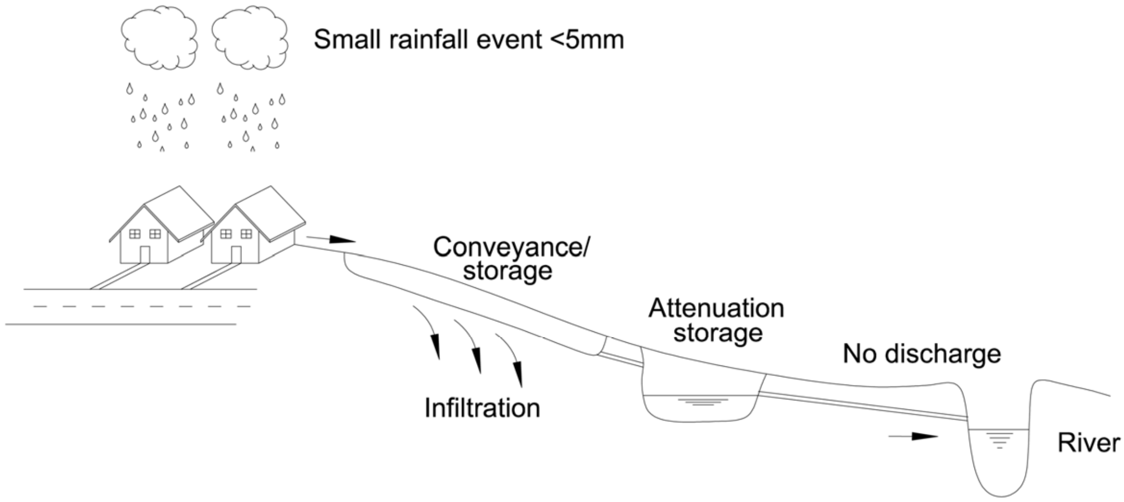


Figure 3.1 for Interception runoff

3.2 Volume of stormwater runoff – extreme rainfall events

<u>Long Term Storage</u>

In extreme rainfall events, the total volume of runoff from a developed site is typically between 2 and 10 times the runoff volume from the same site in a greenfield state. It is important to control this additional volume from the developed site for two reasons. Firstly a large proportion of runoff tends to be released much more quickly than the greenfield runoff (even where attenuation storage is provided to address the difference in the rate of runoff). Secondly, even if this volume was released at the peak rate of the greenfield runoff, due to the finite storage volume provided by floodplains, flood depths and extents will be greater. Figure 3.2 schematically illustrates this aspect.

In theory, therefore, the 100 year flood runoff from a site should be controlled to greenfield volume and rates to ensure the same conditions occur downstream after development. In practice this is virtually impossible to achieve.

The criterion for Long Term Storage (LTS) is a simple pragmatic approach aimed at addressing this issue without too much complexity. The 100 year 6 hr event is used to estimate the runoff volumes both before and after development, and this volume difference needs to be captured and theoretically prevented from being discharged at all. In principle, it would therefore be preferable to control this volume using infiltration on the site, but often this is not feasible. Perhaps as many as 80% or more of developments do not have the opportunity to use infiltration methods. If infiltration is not possible, this volume of storage (LTS) needs to be specifically designed for and discharged at a maximum rate of 2 l/s/ha. If this option of direct runoff of the Long Term Storage volume is utilised, the runoff from the attenuation storage to meet the greenfield rate of discharge (based on the critical duration event – which is likely to be significantly longer than the 6 hour event), needs to be reduced by the same amount.

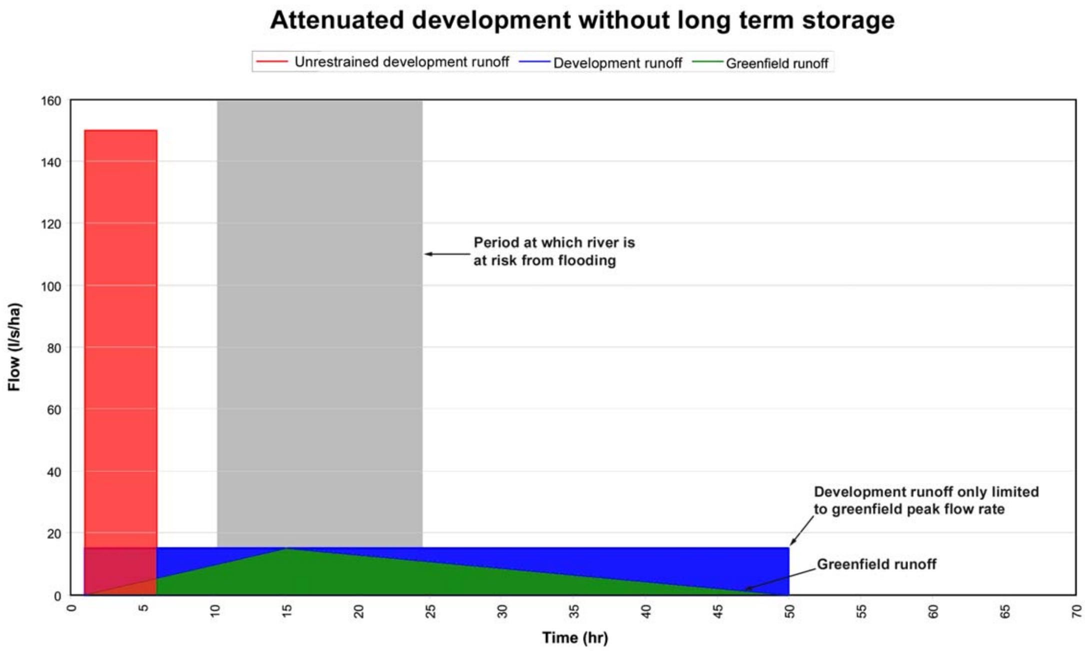


Figure 3.2 Schematic illustrating relative runoff volumes before and after development

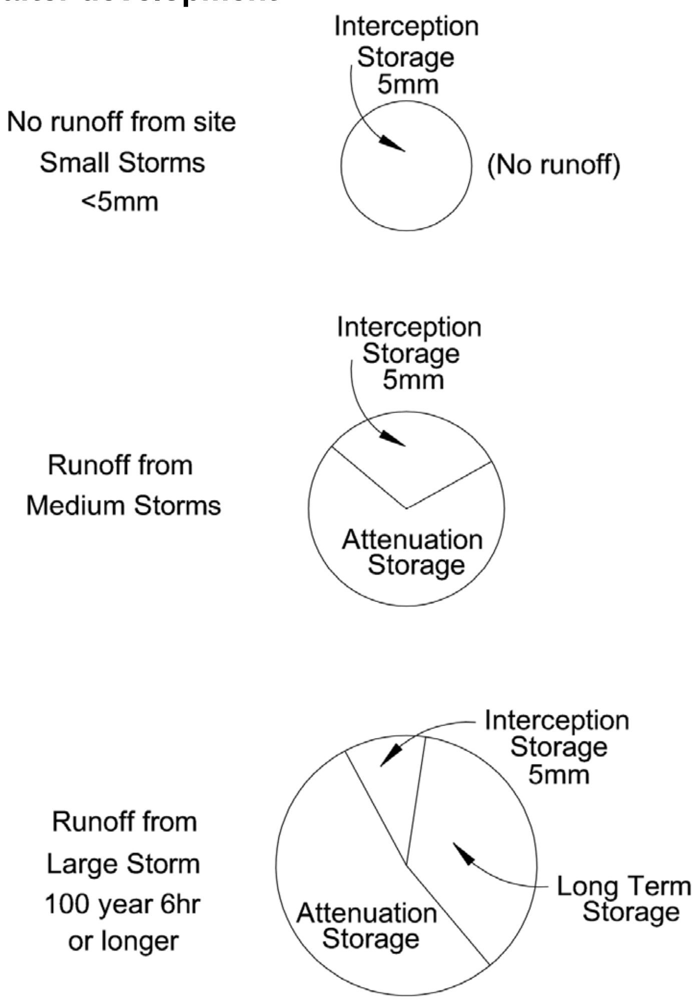


Figure 3.3 Schematic illustrating the concept of Long Term storage

3.3 Rate of stormwater runoff

Attenuation storage

Whatever the event, development runoff through traditional pipe networks, if allowed unchecked, will discharge into receiving waters at orders of magnitude faster than the undeveloped site. For significant rainfall events this causes flashy flow in the river which is likely to cause scour and erosion that may seriously affect the morphology and ecology of the stream, and for extreme events this may exacerbate flooding downstream. Both of these aspects are therefore addressed by applying a limit of discharge from the developed site, thus requiring attenuation storage.

The design principle is to limit the runoff for events of similar frequency of occurrence to the same peak rate of runoff as that which takes place from greenfield sites. There is a basic limitation in this philosophy in that the rate for any given event will not actually replicate the same rate of runoff, due to the difference in runoff characteristics between the developed and undeveloped site. However to achieve an exact replication would be impossible and attempting to get a relatively good approximation of runoff from any site would require the use of a continuous rainfall series, advanced modelling and considerable effort. Even if this effort was made, there is little justification for this approach as the actual runoff from greenfield sites cannot be sufficiently accurately predicted.

This equivalence rule is therefore applied at both the 1 year event and the 100 year event using the critical duration storm to find the maximum attenuation storage volume.

In practice there are two conditions where the greenfield flow rate is not actually applied to define the limiting discharge rates. These are:

1. The limit of discharges based on Q_{BAR} that are less than 1 l/s/ha for permeable sites as this is seen as being an unreasonable requirement (producing very large storage volumes). Q_{BAR} is then set to 1 l/s/ha;
2. Small sites would require impractically small controls to achieve the required flow rates where these are calculated to be less than 5 l/s and therefore in this case a minimum flow of 5 l/s is used.

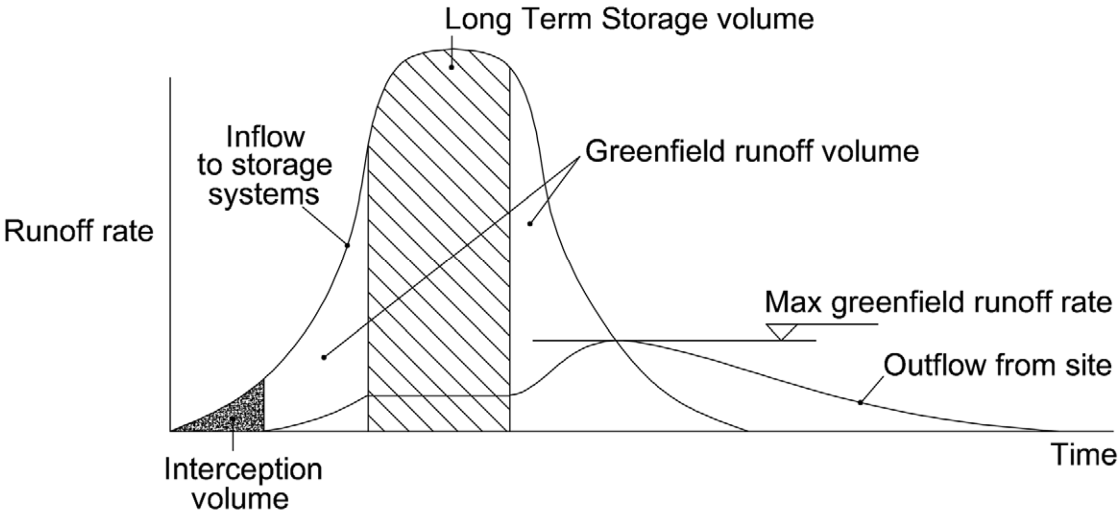


Figure 3.4 Hydrograph schematic: Interception, attenuation and volume control

3.4 Non-compliance with providing volume control of runoff

There are situations where applying the concept of LTS to a site and ensuring the drainage system operates to meet this criterion is difficult. In this situation an alternative peak rate of discharge is applied to the site for sizing the attenuation storage without using LTS. Instead of using the rates for the 1 year and 100 year events, the discharge limits are set to the 1 year and Q_{BAR} (approximating to the 2.3 year event) flow rates for sizing the storage. This results in a greater storage volume being required.

Again there is an over-ride where permeable catchments result in a Q_{BAR} value which is less than 2 l/s/ha. In this situation the higher value (i.e. 2 l/s/ha) is used for events up to the 100 year return period.

3.5 Treatment of stormwater runoff

Stormwater runoff for all events is contaminated to some degree. This is due to the flush of debris and sediment from paved surfaces being "washed off" in the first part of the event together with any sediment deposits in the pipe network. This is compounded by the fact that this highly concentrated initial flow enters the receiving water which is still flowing at base flow conditions, thus providing a minimum level of dilution. Although the Interception criterion should address much of this pollution, there is a need to deal with the runoff for events larger than this.

The concept of Treatment Storage is to provide a body of water in which dilution and partial treatment (by physical, chemical and biological means) of this runoff can take place. This is effectively the volume of water which remains in ponds during the dry weather periods between rainfall events. The amount of storage normally provided is the equivalent volume of runoff of 15 mm of rainfall.

It should be stressed that drainage of a site should be designed using the treatment train concept using appropriate drainage mechanisms throughout the site. Reliance on only a single pond prior to the outfall is not regarded as good practice in providing the best water quality protection for the receiving water. In fact in some instances a final pond may not be needed if water quality issues and stormwater runoff control are adequately addressed by extensive use of vegetative drainage systems upstream.

4 Using the storage sizing tool

This chapter provides a simple look-up method for estimating the storage volumes needed to comply with the discharge limits which the Environment Agency / Local Planning Authority would normally set for a greenfield site development.

A website also exists which provides an automated and simplified form of this tool. www.uksuds.com. A number of simplifications have been made in the web tool, and therefore there will be some differences found when comparing the manual method with the web tool. The web tool is much easier to use than this manual approach, but for those who might not have access to the website, this method still allows users to estimate the storage volumes needed for any site.

The method provided here has been developed to minimise the need for technical expertise and avoid the use of computer tools to arrive at approximate values for stormwater storage volumes. All parameters and factors for any site can be obtained from the figures and tables provided. Soil parameters should be based on the use of FSR maps or the Wallingford Procedure maps.

Figure 4.1 provides a flow diagram of the process to follow to obtain the stormwater storage volumes. There are 4 storage volumes to determine. These are:

- Interception storage
- Attenuation storage
- Long Term storage
- Treatment storage.

Interception storage may not require explicit provision of storage, but is the volume of runoff which must be prevented from leaving the site for up to the first 5mm of a rainfall event.

Attenuation storage aims to limit the rate of runoff into the receiving water to similar rates of discharge to that which took place before the site was developed (greenfield runoff rate). This can be provided at one or several different locations using a variety of SuDS or other storage techniques.

Long Term storage is similar to attenuation storage, but aims to specifically address the additional volume of runoff caused by the development. This is either infiltrated into the ground or, if this is not possible due to soil conditions, attenuated and discharged at very low rates of flow to the receiving watercourse so as to minimise the risk of exacerbating river flooding (see chapter 6 for more information).

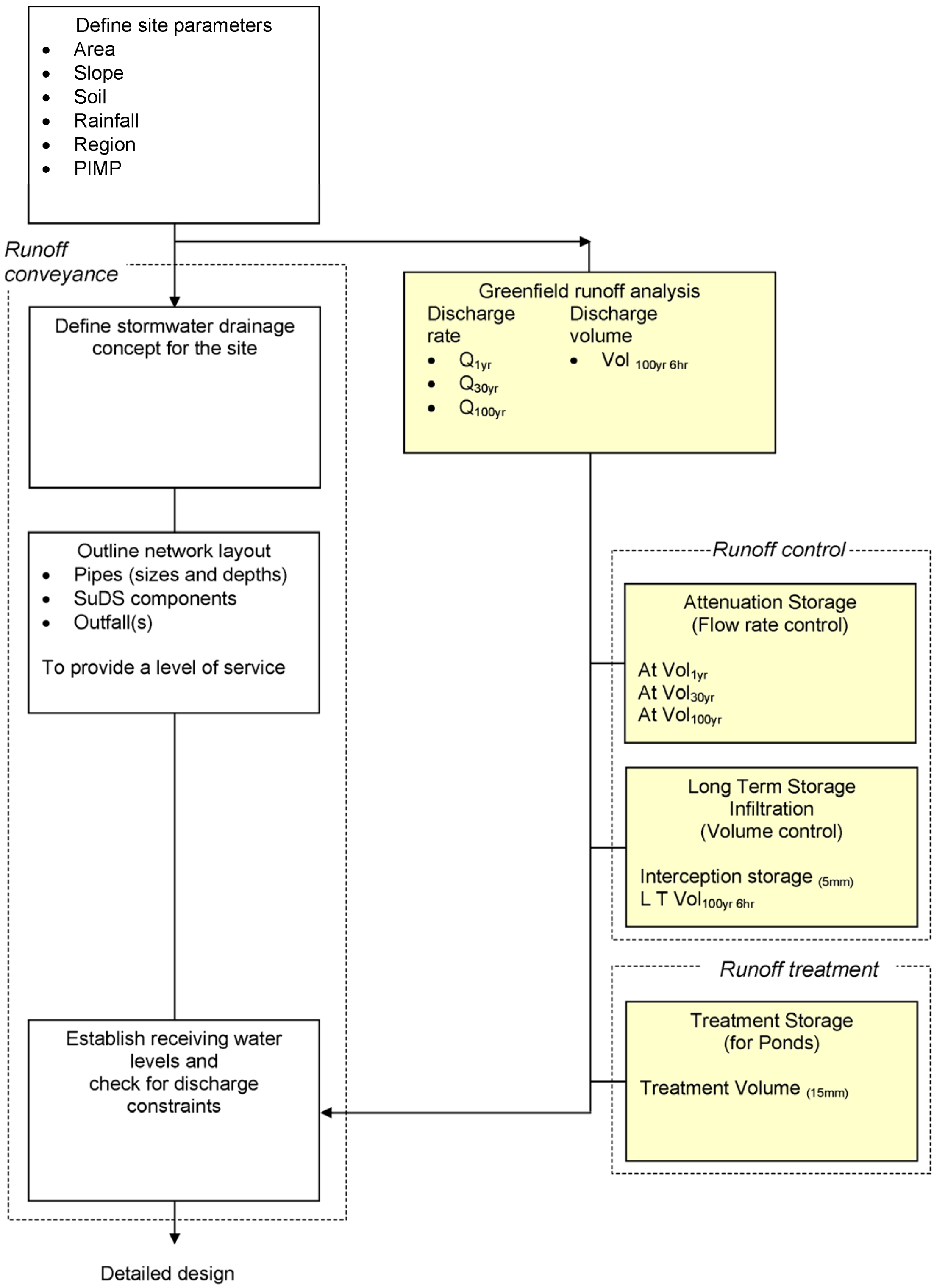
Treatment storage aims to ensure the water quality of the stormwater is sufficiently improved to minimise its impact on the flora and fauna in the receiving water. This is normally provided as the dry period volume of one or more ponds.

The calculated volumes estimated using the tool do not take account of head-discharge characteristics of pond outfalls. There are a number of safety factors built into the method used in this Guide which should ensure that the storage volumes are not under sized and it is intended that this tool will provide an estimate of volume which is within 20% of that determined by detailed design. However the opportunity to use a range of different SuDS units, all with their own runoff and storage characteristics, will significantly affect the actual performance of the drainage system for any site.

Figure 4.1 illustrates the elements of drainage design, showing that there are two distinct flow paths; firstly the need to deal with the conveyance of stormwater safely through the site, and secondly the assessment of greenfield runoff rates and the subsequent estimation of storage volumes.

Figure 4.2 shows how the tool process is structured. Sheets ASV1 to ASV2 are used to provide an assessment of the greenfield site discharge limits and attenuation storage volumes. The methodology is based on complying with 3 stages of discharge limits: the 1 year, 30 year and 100 year flow rates.

Calculation pages TV and LTV are used to find the Treatment volume and Long Term storage volume respectively.



Notes: *Q_{1yr}* — *Peak discharge rate for 1 year return period*
At Vol_{100yr} — *The attenuation volume of storage for 100 year return period*
LT $Vol_{100yr\ 6hr}$ — *The Long Term volume of storage for 100 year return period*

Figure 4.1 Initial design of stormwater drainage for new developments

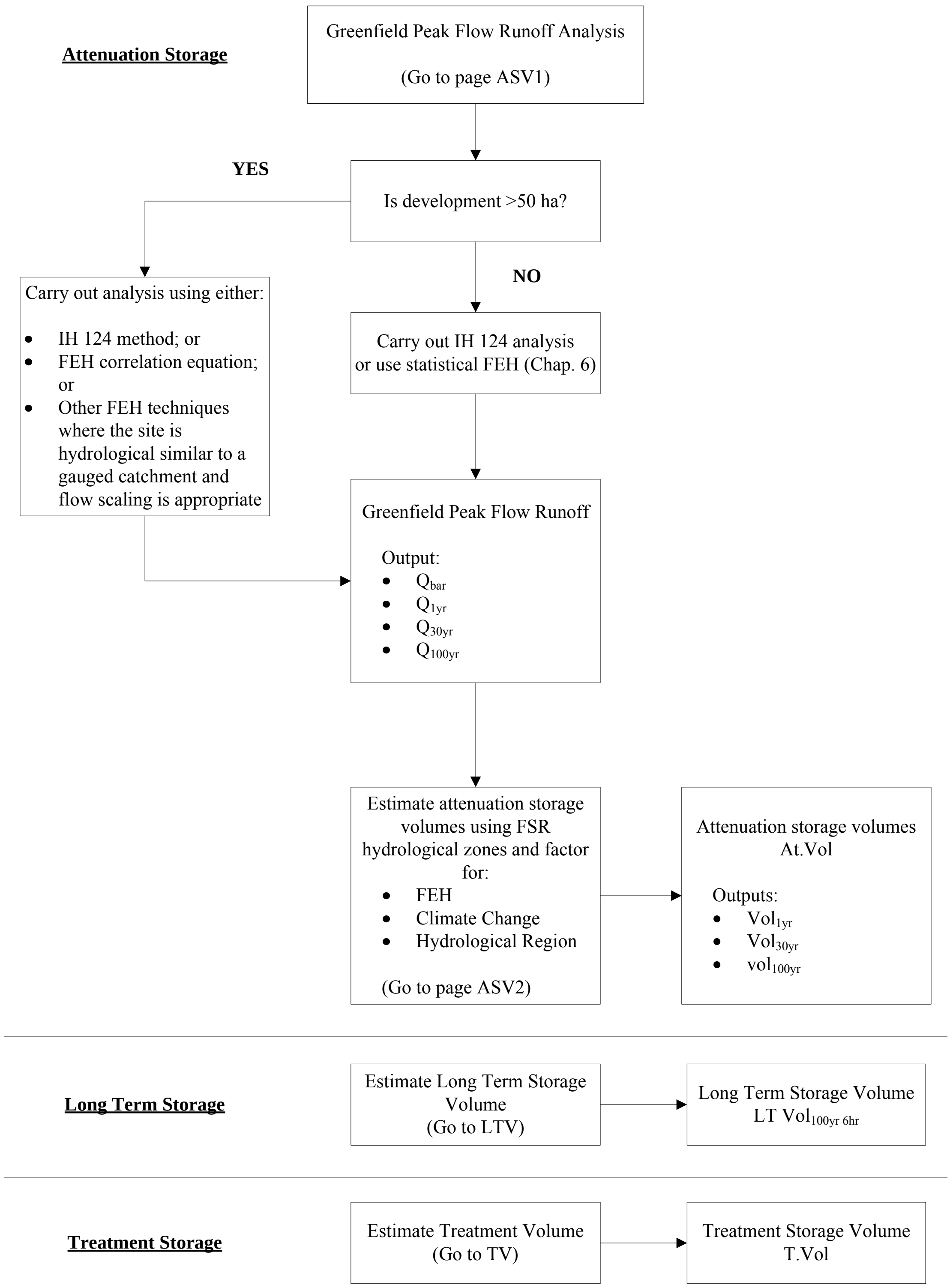


Figure 4.2 Tool flow chart for estimating storage volumes

4.1 Greenfield estimation of peak flow rate of runoff

The aim of this first section is to determine the peak discharge rate of the greenfield site runoff for 1, 30 and 100 year return period events.

Site characteristics

1. Hydrological Region (1 – 10) (R) [ ] — UK is divided up into 10 hydrological regions reflecting the different flood frequency growth curves. (Appendix 1, Figure A1.1)

2. (SOIL) type (1 – 5) (S) [ ] — Refer to Wallingford Procedure WRAP map or FSR maps (Appendix 1, Figure A5.1)

3. Development size (A) [ ] ha — The size of the gross development excluding large parkland areas being allocated as public open space which remain unmodified.

4. Area (A) [ ] ha — Excluding public open space not modified by the proposed development

5. Annual Rainfall (SAAR) [ ] mm — SAAR – use either SAAR from FSR or AAR from FEH (Appendix 1, Figure A4.1)

6. Soil runoff coefficient (SPR) [ ] — SPR value for SOIL – this is not the FSR index class value for SOIL (1 to 5), but its corresponding runoff coefficient (SPR) as follows:

SOIL	1	2	3	4	5
SPR	0.10	0.30	0.37	0.47	0.53

Also see note 1 below.

7. Development mean annual peak flow $(1.08\left(A/_{100}\right)^{0.89}.\ SAAR^{1.17}.\ SPR^{2.17})$
$(Q_{BAR} = Q_{BAR50\ hr} \times (A/50))$
(Q_{BAR}) [ ] l/s — For development sites of 50 ha or less, use 50 ha when applying the formula. Subsequently factor the resulting value by the ratio of the site area to 50 ha. (i.e. if the site is 10 ha divide the answer by 5)
For using the equation from the Statistical FEH see note 4

8. Mean annual peak flow per unit area	(Q_{BAR}/A)		l/s/ha	For SOIL type 1 and occasionally type 2 Q_{BAR}/A will generally have a value less than 1. If so use 1 l/s/ha (see note 2)
9. Minimum limit of discharge	($Q_{throttle}$)		l/s	Minimum discharge (see note 3)
9.1 1:100 year flow rate per unit area	($Q_{throttle}/A$)		l/s/ha	
9.2 Equivalent mean annual peak flow per unit area	($Q_{throttle}/3.5A$)		l/s/ha	Use this value as (Q_{BAR}/A) if it is greater than item 8.
10. 1yr, 30yr and 100yr peak discharge rate of runoff per unit area				Use the larger of the 2 values of item 8 and 9.2 for calculating 10.1 to 10.3
10.1 $Q_{BAR}/A \times 0.85$	Q_{1yr}		l/s/ha	GC_{30} and GC_{100} are the growth curve ratios $Q/\overline{Q}$ for the 30 year and 100 year events for the relevant hydrological region.
10.2 $Q_{BAR}/A \times GC_{30}$	Q_{30yr}		l/s/ha	The 30 and 100 year factors are found from Appendix 1, Figure A1.2 from FSSR 14.
10.3 $Q_{BAR}/A \times GC_{100}$	Q_{100yr}		l/s/ha	

Note 1 *HOST classes for soil also have SPR values. Although derived a little differently, these values can also be used – read chapter 6 (IH Report 126 – Hydrology of Soil Types). As the ranges used in FSR soil types are very coarse it is preferable to use the HOST categories. (Appendix 1 Figure A12.1)*

Note 2 *Very low values of Q_{BAR} /A result in excessive storage volumes. As Long Term storage for SOIL type 1 is large, a minimum value of Q_{BAR} /A of 1 is to be used.*

Note 3 *Minimum sizes of an orifice may limit the minimum hydraulic control flow rate. This allows the derivation of an equivalent value of a Q_{BAR} /A.*

Note 4 *The use of the correlation equation of the FEH Statistical equation can be used, together with FSSR 14 growth curve factors to convert Q_{MED} to Q_{BAR}. See section 5 for the correlation equation.*

4.2 Assessment of attenuation storage volume

Hydrological Region	(R)		Regions 1 – 10 for runoff growth factor (Appendix 1, Figure A1.1)
Hydrological rainfall Zone (M_560, r)	(Z)		Zones 1 to 8 based on FSR rainfall characteristics (Appendix 1, Figure A2.1)
Development Area	(A)	Ha	Excluding large public open space which is not modified and drained by the development
Proportion of impervious area requiring Attenuation storage	(α)		Impermeable surfaces served by direct drainage / total area of impermeable surface. (see Note 1)
Greenfield flow rate per unit area	Q_{BAR}/A	l/s/ha	From page ASV 1, use the larger of item 8 or 9.2. See also note 3 on page **LTV**
Estimate of development (PIMP) percentage impermeable area		%	For developments where the PIMP value is less than 50% (i.e. where pervious area is the main surface type) a more detailed study should be made as the storage estimates may be undersized or 50% can be used.
7. Attenuation storage volumes per unit area	($Uvol_{1yr}$)	m^3/ha	Interpolate values based on PIMP and Q_{BAR}/A (Appendix 1, Figures A7.1 – A7.8) Use characteristics from item 2 (M_560, r).
	($Uvol_{30yr}$)	m^3/ha	
	($Uvol_{100yr}$)	m^3/ha	
8. Basic storage volumes (U.Vol . α A)	(BSV_{1yr})	m^3	Storage units may serve areas of different densities of development. If necessary calculations should be based on each development zone then cumulated.
	(BSV_{30yr})	m^3	
	(BSV_{100yr})	m^3	

9. Climate Change factor	(CC)			Suggested factor for climate change is 1.2 (see note 2).
10. FEH Rainfall factor	(FF_{1yr})			Use factors based on the critical duration Q_{BAR}/A and PIMP (Appendix 1, Figure A11.1 and Appendix 1, Figures A6.1.1 – A6.3.4). (See note 3)
	(FF_{30yr})			
	(FF_{100yr})			
11. Storage Volume ratio (CC/ FF)	(SVR_{1yr})			Calculate item 9 / item 10 then use Appendix 1, Figures A8.1 – A8.8 to obtain storage ratios
	(SVR_{30yr})			
	(SVR_{100yr})			
12. Adjusted Storage Volumes (SVR x BSV)	(ASV_{1yr})		m^3	Storage volumes adjusted for Climate Change and FEH rainfall. Item 11 x item 8.
	(ASV_{30yr})		m^3	
	(ASV_{100yr})		m^3	
13. Hydrological Region volume storage ratio	(HR_{1yr})			Adjustment of storage volumes for hydrological region using Volume Storage Ratio (Appendix 1, Figure A9.1). The values are based on growth curve factors - the ratio of growth curve factor of the region of site with hydrological region 5 (Appendix 1, Figure A1.3)
	(HR_{30yr})			
	(HR_{100yr})			
14. Final estimated Attenuation Storage				
Attenuation Storage	At. Vol_{1yr}		m^3	Required Attenuation Storage
Volumes	At. Vol_{30yr}		m^3	
(HR x ASV)	At. Vol_{100yr}		m^3	

Note 1 *Hard surfaces draining to infiltration units (which are considered to be effective for extreme events and therefore provide part or all of the Long Term storage volume) can be assumed to not contribute runoff for the estimation of Attenuation storage with α being calculated accordingly. (The assessment of PIMP should still be based on the total area of hard surfaces to the total area of the development).*

Where Long Term storage is being provided as off-line temporary storage from the Attenuation pond (drained by either infiltration or direct discharge), α should be a value of 1, but the estimated Attenuation volume will then be reduced by the value of the Long Term storage volume.

2 *The Environment Agency guidance on taking account of future increases in rainfall intensities under climate change scenarios is to apply a factor of 1.2 for developments with a design life extending to 2080 and beyond (which is usually applied to residential developments) (Environment Agency, 2011).*

3 *Appendix 1, Figure 11 assists in using the appropriate factor which is a function of the critical duration event. This is necessary to know as the rainfall depth relationship between FSR and FEH varies with both return period and duration. Having established the approximate duration from figure 11, the appropriate duration map (A6.1.1 – A6.3.4) can then be used to determine the FEH rainfall factor.*

The process is iterative. The following is an illustration of the procedure.

Refer to 1 yr 12 hr map (Figure A6.1.4) and determine the appropriate FSR:FEH ratio for the location of the development; lets assume 1.0. Assuming the development characteristics are:

Q_{BAR} *= 4 l/s/ha, PIMP = 75,* $M_5 60$ *= 14 R0.3*

Then using figure A11.1 and referring to the appropriate cell for the table for an FEH factor of 1.0, the critical duration lies somewhere between 5 and 10 hours. This means that the map that should be referred to is the 6 hour 1 year map (Figure A6.1.3). The FEH factor may then change to 0.9. The process is then repeated to check that the critical duration is still around 6 hours to confirm that the FEH factor is 0.9. This process is then repeated for the 30 and 100 year return period FEH factors.

Maps for a return period of 30 years do not exist – use those for 25 years.

FSR / FEH maps are unavailable for durations greater than 12 hours. Use 12 hours in this situation.

4.3 Initial sizing of long term storage volume

Long term storage is to be provided to cater for the additional runoff caused by the development compared to the volume that would be contributed from the site in its greenfield state. This volume must be catered for as either infiltration storage or in storage with the ability to be discharged at a rate of less than 2 l/s/ha. Designing the drainage system with Long Term storage will result in less total storage being required than not providing LTS.

Discussion on the equation and practical provision of Long Term storage is provided in Chapter 7.

1.	Development area	(A)		ha	Excluding public open space which is not modified by the development
2.	Estimate of PIMP (percentage impermeable area)	(PIMP)		%	
3.	Impermeable area ($A \cdot PIMP/100$)	(AP)		ha	All hard surfaces in the development
4.	Long Term storage factor	(LTF)			Storage volume per unit area per mm of rainfall (see Figure A10.1 or A10.2) see note 3.
5.	Rainfall depth	(RD)		mm	Rainfall depth for 100 year 6 hour event (Appendix 1 Figure A3.1).
6.	Long Term storage volume ($RD \cdot LTF \cdot AP$)	($LTVol_{100yr\ 6hr}$)		M^3	See note 2

Note 1 *Where Long Term storage is being discharged directly to the receiving water at 2 l/s/ha, the values for Q_{30} and Q_{100} for attenuation storage discharge rate should be reduced accordingly. If this is the case, the calculation for Attenuation storage should be based on (Q_{BAR}/A – 0.5) l/s/ha unless this reduces below a value of 1.0 l/s/ha, in which case 1.0 l/s/ha should be used.*

2 *LTF is defined such that the equation of item 6 uses rainfall depth in millimetres and area in hectares.*

3 *There is a choice of LTF factor based on the assumption as to whether the pervious area contributes to the 100 year runoff based on its SPR value. The conservative assumption would be to assume it does unless the site is designed to minimise the runoff from green areas (using Figure A10.1). 80% runoff is assumed to occur from pervious surfaces. Other options can be assumed working from first principles using the formulae in Chapter 7.*

4.4 Initial sizing of Treatment storage volume

1. Development area	(A)		ha	Excluding public open space which is not modified by the development
2. Estimate of PIMP percentage impermeable area	(PIMP)		%	
3. Proportion of impervious area requiring Treatment storage	(β)			(see note 2)
4. Soil runoff coefficient	(SPR)			From the Wallingford Procedure WRAP map or FSR SOIL maps (Appendix 1, Figure A12.1)
5. 5 year / 60 minute rainfall depth	(M_560)		mm	5 year 60 min rainfall depth. From the Wallingford Procedure M_560 map or FSR rainfall maps (Appendix 1, Figure A2.1).
6. Treatment volume 6.1 Treatment volume (England and Wales) (Tv) T Vol = A.15.PIMP/100			m^3	Treatment volume is calculated using the formula See note 1
6.2 Treatment storage volume (Scotland and N. Ireland) (Tv) T Vol = $9A.M_560\cdot(SPR/2 + (1 - SPR/2).\ \beta\ PIMP/100)$			m^3	Treatment volume is calculated using the formula See note 1

Note 1 *The concept of treatment volume is to provide sufficient volume to provide partial treatment of the stormwater effluent. There are no specified water quality discharge criteria to comply with. Accepted best practice in Scotland is based on (CIRIA report C522, 2000) requiring 1 times Tv except in certain circumstances when up to 4 times Tv is needed. See SEPA guidance. Elsewhere in UK the volume equivalent to 15mm of rainfall is used.*

It should be noted that the emphasis on the importance of ponds to provide treatment has changed to some degree in favour of emphasising the need to use vegetative units to provide treatment or pre-treatment. This therefore has implications on treatment volume requirements and the need for flexibility in pond sizing.

2 *Compliance with the SuDS Manual or the National SuDS Standards, if different, in terms of providing a treatment sequence of 1, 2 or 3 treatment units, makes any rule of thumb for modifying the sizing of the treatment volume difficult. It is unlikely that a Tv of less than 10mm of rainfall runoff from impermeable areas draining to the pond would be acceptable.*

5 Initial sizing of conveyance systems

The use of pipes as part of a stormwater system on a site should not be presumed with the current emphasis on the use of SuDS for all new developments. Although pipe systems are still commonly used to provide the conveyance and drainage connectivity for part or all of a site, efforts should be made to use surface vegetative forms of conveyance.

The reason for needing to do an initial design for a pipe network is to show the general connectivity arrangement across the site and the methods proposed.

The sizing of the stormwater network is often carried out using the Rational Method and subsequently checked using hydrograph methods. This can be done initially by very a simple rule of thumb using a constant rainfall intensity of 35mm/hr. Historically 50mm/hr has been used and provides a more conservative solution.

Similar rules of thumb for gradients exist for pipe gradients. Pipes must be at least 150mm in diameter and these should not be laid flatter than 1 in 150. As larger pipes are required, pipes can be laid at minimum gradients using the inverse of the pipe diameter, so a 225mm pipe can be laid at 1 in 225 or steeper and a 300mm pipe at 1 in 300. For pipes larger than 500mm, gradients should not generally be laid flatter than 1 in 500 due to limitations on construction accuracy. Tables for the capacity of pipes at all gradients are available from HR Wallingford.

If swales are used for conveyance, design is rarely a function of conveyance capacity. Issues such as velocity to protect against erosion and maximise treatment, and a shape which allows easy maintenance usually define their size and characteristics. The SuDS Manual should be referred to for design guidance.

Linear ponds/ vegetative channels are also good alternatives by still providing treatment, storage, infiltration and conveyance but taking less land. However there are issues associated with safety and maintenance that need to be considered.

5.1 Designing for exceedence - temporary storage and flood routing

Extremely heavy rainfall, either in terms of intensity or total depth, can result in elements of the drainage system (storage or conveyance) being overloaded; either intentionally or otherwise. The consequence is that overland flow takes place within the site and off-site and ponding occurs at low points in the site.

The CIRIA guidance 635 (Designing for Exceedence in urban drainage – Good Practice) should be referred to for more information on this topic, but in principle design of systems should take these aspects into account. In the case of SuDS systems, conveyance exceedence is rarely an issue. However due to the sizing rules for both pipework and gullies, overland flow on roads where these are used is much more likely to occur for brief periods during intense storms. Careful consideration of overland flow routing and threshold levels should be considered at the appropriate design stage.

In addition to paved surfaces and the drainage system, flooding from steeper areas of permeable (unpaved) areas can also generate flooding. Again consideration of flood routes should also be made for these situations.

Sites are particularly at risk during the construction period, as areas stripped of topsoil can effectively act as impermeable areas. This has implications for temporary bunding of stormwater flooding, phasing of housing construction and temporary storage of flood waters.

The mechanism of runoff when gullies and pipework are overloaded is for the stormwater to run down roads to low spots. This characteristic can be explicitly used in drainage design to temporarily utilise suitable temporary holding areas to retain flood waters.

Unfortunately flooding at certain locations can be exacerbated by the pipe system itself. Water can exit from gullies, particular where steeper sections flatten out, thus concentrating floodwater in possibly vulnerable locations. However at the stage of initial evaluation, identifying these locations is difficult and they can only be effectively determined during detailed design using computer models.

Where routing of water is assessed to take place, particular consideration of floor thresholds and risk to property flooding in these locations needs to be made.

6 Estimation of greenfield runoff

There has been a continuing debate over the years as to the merits of the various methods of estimating greenfield runoff. To be strictly correct, the debate is about which approach is appropriate that gives the most accurate stream flow for small catchments which is then assumed to also provide the best estimate of greenfield runoff from a site, usually by linear interpolation. This chapter and Chapter 2 provide an explanation as to why the use of IH124 is still thought to be appropriate, and where other methods may be used as alternatives. The following text provides a brief overview of the main greenfield runoff estimation techniques and their relative merits and draw-backs.

6.1 The principal methods used for estimation of river flow / site greenfield runoff rates

It is important to note that the debate on which method is most appropriate for greenfield runoff assessment has focused on the estimation of flow rate. This debate has not extended to the estimation of runoff volume which is of equal importance, both for small rainfall events and large ones. Research has shown (EDAW/AECOM, 2009) the importance controlling runoff into streams and rivers to preserve the morphology to “natural” states and this is not just a function of peak flow control. Clearly, at present, the criteria on volume control is extremely simplistic (Interception of the first 5mm, and then, at the other extreme, the use of the 100 year 6 hour event). When criteria extend eventually to infiltration volumes and the consideration of the site response to all rainfall events, runoff volume will receive much greater attention, including the need for a significant amount of additional research.

This chapter first addresses the methods for estimating peak flow and then discusses runoff volume.

There are many UK guidance documents available on greenfield runoff estimation. These include:

- Institute of Hydrology Report 124 (Marshall and Bayliss 1994)
- A number of other FSR based runoff equations (FSR 6 parameter equation - 1975, FSSR 6 - 1978)
- Prudhoe and Young (TRRL LR565)
- Flood Estimation Handbook (Institute of Hydrology 1999)
- ADAS Report 345 (ADAS 1982),
- the Rational Method (Shaw 1994)
- and in Northern Ireland, the Poots & Cochrane formula (Civil Engineering Branch, Department of Finance, Northern Ireland 1980).

Although there is much debate on the relative merits of each of these methods, there has been no definitive guidance about which methods give the most reliable design flood estimates for small catchments and greenfield runoff in the UK. In general there has been acceptance that there is general preference for the use of IH124 or FEH methods and that other methods should generally not be used. (This does not apply to land drainage of fields where ADAS 345 is probably still the preferred method). However as ADAS 345 is still used by some, information on this equation is also provided.

In general there has been an acceptance by the water industry that IH124 is suitable for estimating greenfield runoff since this document and the Interim SuDS Code of Practice came out in 2004 which aimed to provide a pragmatic approach to meet the various needs of the water industry and which was applied consistently across the country. As a result the recommendation is repeated in several subsequent guidance documents:

- Design Manual for Roads and Bridges - Volume 4, Drainage (Highways Agency 2006), for catchments larger than 0.4 km^2,
- Exceedance in Urban Drainage (Balmforth *et al.* 2006);
- The SuDS Manual (Woods-Ballard *et al.* 2007);
- Code for Sustainable Homes Technical Guide (Department for Communities and Local Government 2010).

However in 2011 a paper "Greenfield runoff and flood estimation on small catchments" (draft pending publication in Journal for Flood Risk Management) was produced by Faulkner. D et al which provided a detailed comparison of IH124 against both the Q_{MED} equation from the statistical FEH method and the ReFH hydrograph method on a number of small river basins. In summary this showed that:

- When catchments with SAAR depths of > 800mm were excluded from the analysis (generally representative of development sites in the UK), IH124 and the FEH statistical equation performed very similarly. ReFH performed least well;
- When all catchments were considered, the FEH method performed best;
- The ReFH performed very poorly for catchments with high permeability.

Further work by Andy Young of Wallingford Hydrosolutions (Estimating flows for small catchments – a review of techniques and future research, unpublished 2011) has shown that IH124 has a bias towards under-prediction for catchments which are less than $10km^2$. This therefore results in larger storage volumes than might be required if another method was used. It is worth noting that all of the above methods tend to underestimate Q_{MED} for catchments with low annual rainfall.

6.2 Statistical FEH model

6.2.1 Q_{MED} estimation

This is a simplified summary of this method. The FEH method correlation formula produces a value for the Index flood (Q_{MED}) which is the median of the set of annual maximum flow peaks and is equivalent to approximately the 1 in 2 year flow rate.

FEH has been produced in a way to enhance the accuracy of estimating flows by giving the user the opportunity to also incorporate the use of “Donor” catchments and “Pooled” growth curves using information on gauged flows from catchments of similar characteristics.

Where a catchment is ungauged, the user can either derive a Q_{MED} based on flows from other hydrologically similar catchments, or else derive it from a correlation formula given below. It would seem that, for very large sites, that form a substantive part of a catchment, and for which there are reasonable flow datasets available for hydrologically similar catchments, the first approach could be considered (assuming the user has access to the FEH tools and a qualified hydrologist is available to use the tool).

If the catchment in which the site lies is gauged, the site is greater than 50 ha, and generally has similar soil characteristics and other hydrologically properties, then it would be appropriate to assess the Q_{MED} value against the observed Q_{MED}, scaled by area.

The FEH statistical method correlation formula (revised in 2008) is:

$$Q_{MED} = 8.3062AREA^{0.8510} \cdot 0.1536^{(1000/SAAR)} \cdot FARL^{3.4451} \cdot 0.0460^{BFIHOST\wedge 2}$$

The original equation in FEH also incorporated the parameter SPRHOST but this is no longer used in the equation. FARL is a measurement of water bodies in the catchment so that their attenuation effects are considered. If the equation is applied to development sites, it is unlikely that FARL will be relevant so this term becomes 1.0 and therefore drops out. BFIHOST is a measure of base flow runoff. For more information on BFIHOST see report IH126 (Hydrology of soil types: a hydrological classification of the soils of the United Kingdom) or FEH documentation.

As with IH124, this equation comes with the warning that it should not be used for catchments less than 50ha in size.

The disadvantage of this equation is the need to obtain site specific data for SAAR and BFIHOST. For users with the FEH software, this data can be estimated by selecting characteristics of a small catchment close to the site in question. However, the FEH tool only outputs catchment averaged values for catchments greater than 0.5 km^2 so site specific characteristics cannot be generated.

Fortunately every soil class is attributable to the HOST soil categories and this can be found in IH126, Hydrology of Soil Types: a hydrologically based classification of soils of the United Kingdom (1995). Therefore if one obtains a soil map of the site area, the soil class can be found and used to obtain the value for BFIHOST by referring to chapter 5 of IH126 and using Table 3.4 of the same document.

6.2.2 Extreme flood flow estimation

Within the FEH tool, growth curves are computed by merging gauged data from hydrologically similar catchments. This process requires the use of Winfap FEH – a component of the FEH suite of tools which requires implementation by a skilled hydrologist. The use of such an approach has the benefit of using the longest flood peak series available.

However, an alternative approach is to use the original growth curves produced in FSSR 14 (Review of Regional Growth Curves, 1983) (Appendix 1, Figure 1.2). These generally provide a reasonable estimate for all return periods including 2 years to convert Q_{MED} values to Q_{BAR}. FSSR 14 gives a range of 2 year factors ranging from 0.88 to 0.94 depending on the hydrological region. A value of 0.9 is suggested as being sufficient for the purpose of using this methodology.

6.3 ReFH hydrograph method

The ReFH methodology is not explained here in any detail as it requires the use of the ReFH tool and its application by a skilled hydrologist.

It is a hydrograph method and therefore has the advantage of providing both peak flow and runoff volume for any rainfall event.

It should be noted that it significantly under-predicts peak flow rates for permeable catchments (much more so than IH124) and therefore should not be used for sites with these characteristics anyway.

6.4 IH 124 – peak flow estimation

The document Flood estimation for small catchments (Institute of Hydrology report no. 124) was published in 1994 and describes further FSR research on flood estimation for small catchments. The research was based on 71 small rural catchments. However these catchments are not small relative to typical developments as these are defined as having areas less than 25 km^2. The report advises that the method should not be applied to catchments which are smaller than 50 hectares. A new regression equation was produced to calculate $Q_{BAR(rural)}$ the mean annual flood for small rural catchments. $Q_{BAR(rural)}$ is estimated from the three variable equation shown below:

$$Q_{BAR(rural)} = 0.00108AREA^{0.89}SAAR^{1.17}SOIL^{2.17}$$

Where:

$Q_{BAR(rural)}$	is the mean annual flood (a return period in the region of 2.3 years)
AREA	is the area of the catchment in km^2
SAAR	is the Standard Average Annual Rainfall for the period 1941 to 1970 in mm
SOIL	is the soil index, which is an value found from the Flood Studies Report soil maps or the WRAP map of the Wallingford Procedure

Q_{BAR} can be factored by the UK Flood Studies Report regional growth curves to produce peak flood flows for any return period using FSSR 14.

A particularly poor aspect about the use of IH124 is the estimation of SPR from the selection of the SOIL category. It is suggested that the FSR soils map is not used and reference is made to a soils class map of the site. From this there are two ways forward to obtaining a value for SPR.

The first is the use of IH126 in getting the relationship between soil class and HOST category and then using Table 3.4 in that document to obtain a value for SPRHOST. Although one must immediately state that SPRHOST is not the same as SPR based on the WRAP map classification, this approximation is likely to be better than using the coarse categorisation of the 5 SOIL categories. The second approach is still to use the soil class but then to try and apply the process used in developing the WRAP classes used in FSR (Table 4.5). However there is little guidance on this and it requires expert judgement in making the appropriate classification so this is advised against except where experts are involved and where the value of SPRHOST may be considered to be inappropriate.

It is suggested that the default approach should be based on the use of SPRHOST as it is felt that any “error” based on the difference in principle is more than compensated for by the improved accuracy of the classification process.

Growth curves will be required to make predictions of extreme flood peaks and the available methods are as described in Section 6.2.2 above.

6.5 The ADAS 345 method

The ADAS method is based on limited data (both sites and duration or records) and measured flows in land drains, so for both these reasons ADAS is strongly advised against. Also it must be acknowledged that the data was collected for the purpose of land drainage and support for farming and not for urban drainage or river flow calculation. However it has one great advantage; it remains the only field research on flows at the site scale. For this reason it has merit in being summarised in this report, even though the method is not preferred.

The Agricultural and Development Advisory Service (ADAS) report 345 details a technique which is primarily aimed at providing information to determine the size of pipes required for field drainage systems. The method is based on measurements taken from a number of small rural catchments.

The equation to estimate runoff from a site is of the form:

$Q = S_T F A$

Where:

- Q is the 1 year peak flow in l/s
- S_T is the soil type factor which ranges between 0.1 for a very permeable soil to 1.3 for an impermeable soil. (This does not directly correlate with SOIL, and the original MAFF 5 document would need to be referred to for more information)
- F is a factor which is a function of the following catchment characteristics: average slope; maximum drainage length; average annual rainfall. The value of F can be obtained from a nomograph included in the ADAS report
- A is the area of the catchment being drained in hectares

The slope of the catchment is used to derive a second coefficient “C”. The additional parameters of height, catchment length and Average Annual Rainfall (AAR) are also needed. The formula to calculate this second coefficient is:

$C = 0.0001 L / S$

Where:
S = slope
L = catchment length (m)

The slope and length functions are normally dictated by the highest and lowest points on the site. However where the site has multiple outfalls or very different gradients across the site, appropriate consideration of these factors needs to be made.

Guidance on the values of the above variables is given in the ADAS report, together with a nomograph which can be used to estimate the flow (Appendix 1, Figure 11 of ADAS 345). It is advised in the report that the method should not be used for catchments that exceed 30 ha. The predicted peak flow resulting from the ADAS equation should be taken as being the one year return period flood and not the mean annual flood for the catchment. Flow rates for higher return periods can be calculated using the appropriate Flood Studies Supplementary Report regional growth curves (FSSR 2, FSSR 14).

It is generally believed that greenfield runoff calculated for small developments using the ADAS formula provides relatively conservative values, but that it gives a useful and consistent rule of thumb for use across the UK for developments of this nature.

Having derived the value for the 1 year runoff rate (or one of the other return periods which are also provided), this has then to be factored using a growth curve for flood return period to determine the 30 and 100 year peak flow rates. Note that using the 1 year event and factoring it for the 10 year event using FSSR 14 (Appendix 1, Figure A1.2) is unlikely to give exactly the same answer as the 10 year curve from the ADAS nomograph.

Table 6.1 summarises all the parameters used.

Table 6.1 Parameters used in calculating greenfield runoff

Parameter		Units	Comment
Area (A)	(A)	ha	The site, excluding areas of greenfield which are to remain unmodified.
Soil type (S_T)	(ST)		Permeability factor does not directly correlate with SOIL. Agree values to be used.
Length	(L)	m	Between highest and lowest point of the site.
Slope	(S)		Defined by length and change in height
F = Coefficient 1 (1 year) (10 year)	(F)		From nomograph.
C = Coefficient 2	(C)		From nomograph.
Highest level	(H_1)	m	Highest point on site
Lowest level	(H_2)	m	Lowest point on site
Hydrological Regional growth curve (FSSR 14) 1 – 10	(HR)		The growth curve applied is that for the relevant region (Appendix 1, Figure A1.1). 1 year to Q_{BAR} use values from FSSR 2 or approximate using a ratio of 0.85
Standard Average Annual Rainfall	(SAAR)	mm	Use either SAAR from FSR map or Wallingford Procedure map (1941 – 1970), or FEH AAR (1961 – 1990) (see Appendix 1, Figure A4.1)

6.6 Greenfield runoff volume

Calculation of the volume for the provision of Long Term storage is based on the 100 year 6 hour event and a simple formula is available (FSSR 16, The FSR rainfall-runoff model parameter estimation equations updated) Although FEH / ReFH also provides methods for assessing this volume, as it is acknowledged that this design requirement is a very arbitrary (though very important) one, the proposed approach is perfectly adequate for the present. It should be noted that although it can also be accused of being a simple and relatively inaccurate method, it is probably also true that any other method is also likely to have great difficulty in getting a more accurate value.

It should be noted that this equation again uses SOIL to provide an estimate. It is suggested that the same refinement of using Soil classes and the use of SPRHOST would again be useful in getting a more accurate approach (see Section 6.4)

The percentage runoff for a site (PR_{RURAL}) is given by the following equation (FSSR 16):

$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$

Where:
SPR is the standard percentage runoff which is a function of the five soil classes S_1 to S_5

$SPR = 10S_1 + 30S_2 + 37S_3 + 47S_4 + 53S_5$

DPR_{CWI} is a dynamic component of the percentage runoff. This parameter reflects the increase in percentage runoff with catchment wetness. The catchment wetness index (CWI) is a function of the average annual rainfall. The relationship is shown in Figure 6.1.

$DPR_{CWI} = 0.25\ (CWI - 125)$

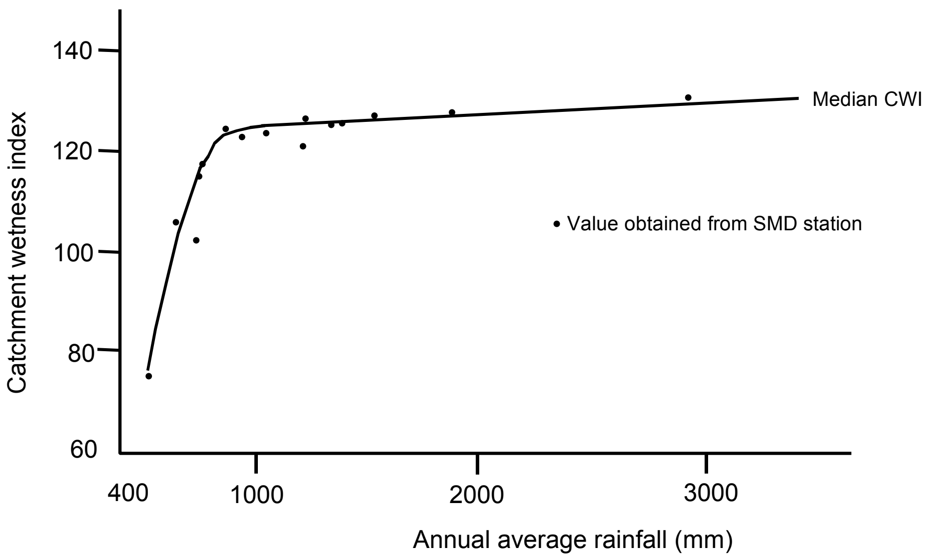


Figure 6.1 CWI vs SAAR – Flood Studies Report

The DPR_{RAIN} is the second dynamic component that increases the percentage runoff from large rainfall events.

$DPR_{RAIN} = 0.45(P - 40)^{0.7}$ for P > 40 mm

$DPR_{RAIN} = 0$ for $P \leq 40$ mm

Where P is the rainfall depth

The derivation of this equation is for extreme events and for catchments which are larger than those of development sites. Its accuracy therefore is to be treated with caution. However if account is to be taken of the volumetric effects of development, this is one of the accepted methods for assessing runoff volumes. It has the advantage of simplicity and therefore a rapid assessment of the impact of development can be made with respect to runoff.

The key feature of this formula is the important influence of soil type. In practice it indicates that developments on sandy soils create massive additional runoff, but development on clays do not. This is obvious, but it has very significant implications for the cost of developments. Other parameters have very little influence.

7 Long Term Storage: Design practicalities and estimation

7.1 Estimation of the difference between greenfield runoff and development runoff

This section discusses and provides a method for estimating the difference in runoff between a site before and after its development.

Figure 7.1 Development plan of a modern high-density housing estate *(courtesy of Linden Homes Chiltern Limited)*

Examination of the formula in FSSR 16 demonstrates that an approximation to SPR is valid for extreme events for the respective soil type. Unfortunately this means that SOIL types 4 or 5 which have SPR values around 50% can show a higher runoff volume than the post-development situation using the standard modelling equations (see Appendix 3).

However, careful consideration of the built environment provides some support for this result. Developments involve the construction of not only buildings and roads, but also involve the re-contouring of the area. Runoff from pervious areas may not be possible from back gardens or low areas created by the development process. Therefore where the pre-development situation reasonably assumed that the whole catchment contributed to the runoff, this may no longer be the case once development has taken place. Figure 7.1 shows a housing development plan (designed to comply with current rules on housing density) which illustrates that much of the green area is to be found in back gardens behind terraced houses.

It is therefore important to be able to differentiate between areas that can and cannot contribute runoff to the drainage system. Similarly, when using infiltration systems, the proportion of the paved area that drains directly to a watercourse may be significantly reduced. These issues, together with the rather complex and sometimes awkward issues of using the appropriate runoff equation, means that a simple and easy to use approach to quantify the difference between runoff volumes before and after development is desirable.

Equation (7.1) has been derived to achieve this. It assumes that extreme events are being considered, as SPR is only a reasonable assumption for the soil runoff factor for this situation. It also assumes that only 80% runoff occurs from paved areas as it is generally recognised that 100% is a conservative assumption which aims to take into account some pervious runoff element.

$$Vol_{xs} = 10.RD.A\left[\frac{PIMP}{100}(\alpha 0.8) + \left(1 - \frac{PIMP}{100}\right)(\beta.SOIL) - SOIL\right] \quad (7.1)$$

Where:

Vol_{XS} = the extra runoff volume (m^3) of development runoff over Greenfield runoff

RD = the rainfall depth for the 100 year, 6 hour event (mm)

PIMP = the impermeable area as a percentage of the total area (values from 0 to 100)

A = the area of the site (ha)

SOIL = the “SPR” value for the relevant FSR soil type

α = the proportion of paved area draining to the network or directly to the river (values from 0 to 1)

β = the proportion of pervious area draining to the network or directly to the river (values from 0 to 1)

0.8 = the runoff factor for contributing paved surfaces

If all the paved area is assumed to drain to the network and all the pervious areas are landscaped <u>not</u> to enter the drainage system or river, this formula simplifies to:

$$Vol_{xs} = 10.RD.A\left(0.8\frac{PIMP}{100} - SOIL\right) \quad (7.2)$$

But where all pervious areas are assumed to continue to drain to the river or network the formula becomes:

$$Vol_{xs} = 10.RD.A\left(0.8\frac{PIMP}{100} - \frac{PIMP}{100}.SOIL\right) \quad (7.3)$$

Figures 7.2 and 7.3 illustrate the difference in runoff volume for these two extremes (fully disconnected / fully connected pervious surfaces) for the five different soil types for any development density. To obtain a volume all that is required is to multiply the X axis value by the catchment area and the rainfall depth.

These graphs demonstrate the very great difference in soil type, the importance of using infiltration to disconnect impermeable areas from the drainage network and the need to be efficient in designing the general landscape to disconnect pervious areas.

It is felt that the assumption should be made that all pervious areas continue to generate runoff for an extreme storm event, unless areas are specifically shown to have been designed not to contribute because of the topography of the site. This method of analysis provides a rapid and robust easy-to-use method for assessing the additional volume of runoff generated by any development for any large rainfall depth.

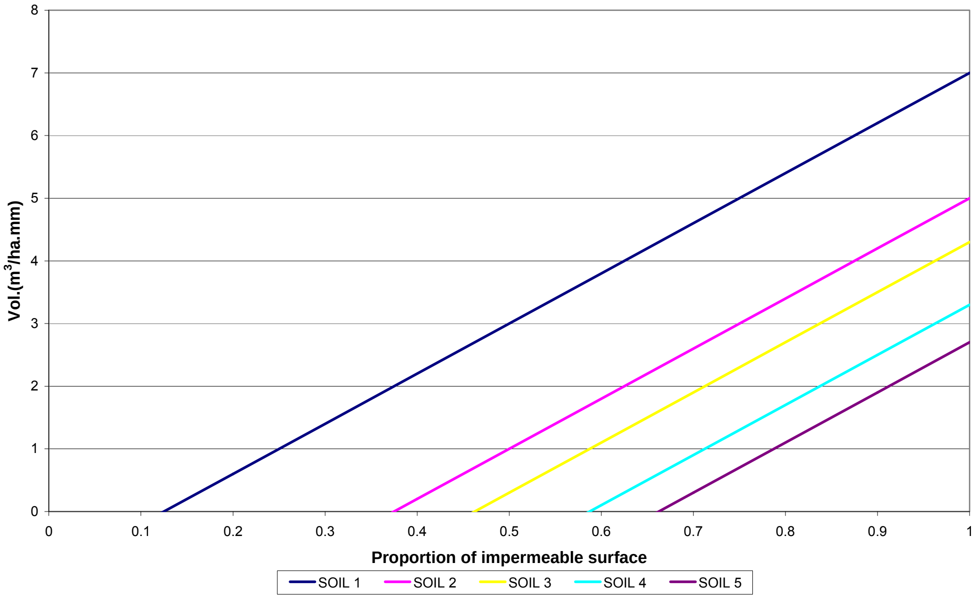


Figure 7.2 Difference in runoff volume for developments where all pervious areas are assumed <u>not</u> to drain to the drainage network

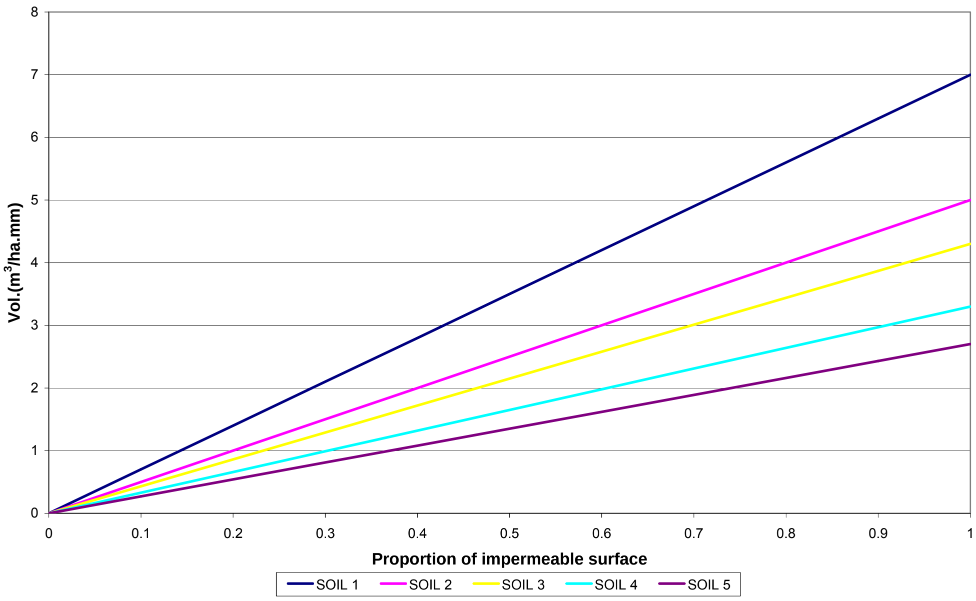


Figure 7.3 Difference in runoff volume for developments where all pervious areas are assumed to drain to the drainage network

7.2 The practicalities of activating Long Term Storage (LTS)

Long Term Storage (LTS) can be provided to store the additional runoff caused by the development compared to the volume that would have been contributed from the site in its greenfield state. The current design rule of using the 100 year 6 hour event to determine this volume is very simple to use. However the practicalities of designing a drainage system to achieve this element of stormwater control is not particularly simple and is discussed here.

Perhaps it is worth stressing that any element of reduction of runoff volume such as topographic management of runoff from indirectly drained areas, interception storage, use of rainwater harvesting (where they are designed to achieve stormwater control) and even where ground conditions are poor, but some use of infiltration can be presumed, will help in minimising the volume needing to be retained by Long Term Storage on a site.

There are three basic approaches to activating Long Term Storage. These are:

1. Ensure that the drainage system provides sufficient infiltration volume to capture the 100 year 6 hour volume difference,
2. Design a storage system which comes into effect for all events (effectively an on-line design) that captures the required volume difference for the 100 year 6 hour event which is specifically designed to empty at a rate of less than 2 l/s/ha (using land drains or a throttle),
3. Design a storage system which only comes into effect above a storm size threshold (effectively an off-line design) which captures the required volume and discharges at the appropriate rate.

The problem with the first is that many sites do not have the soil characteristics to enable infiltration to provide this volume of stormwater control.

The problem with the second (illustrated by Figure 7.4) is that a significant area needs to be provided which comes into operation in all events. This results in a much greater area being required to manage stormwater runoff than option 3.

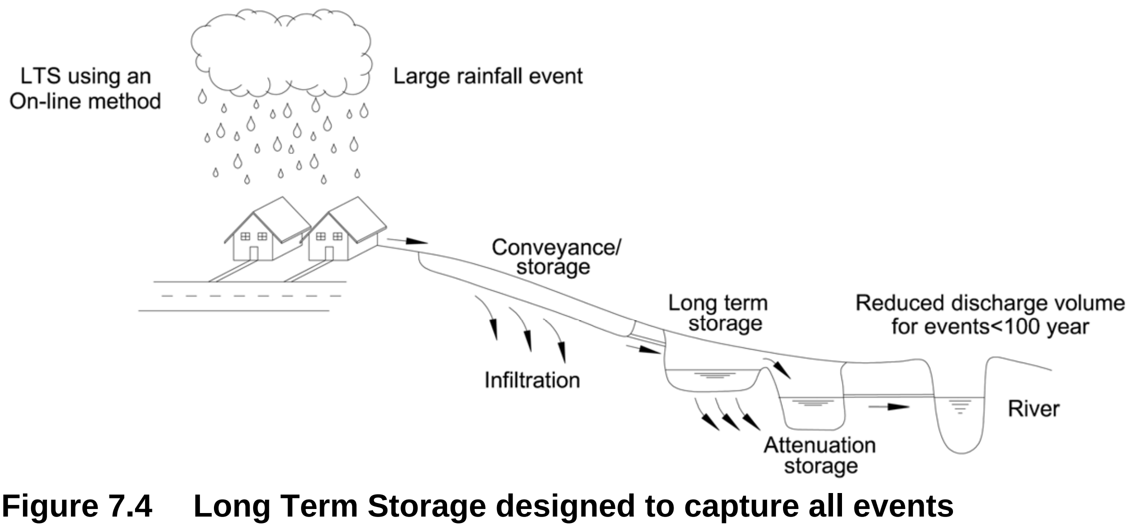


Figure 7.4 Long Term Storage designed to capture all events

The third approach is to fill the attenuation storage system (which is designed to discharge flows at greenfield rates), which “spills” water to the Long Term Storage area. Although this is the most efficient use of land, there are difficulties in achieving this which requires a specific and careful design process.

The critical duration event for the attenuation system for a site may be less than 6 hours, but is more likely to be a lot longer than this. There are therefore two design stages required; firstly to determine the storage volume and arrangement for the attenuation storage system, and then one also needs to demonstrate that the 100 year 6 hour event will spill the requisite volume to the Long Term Storage area. This will therefore have to come into effect well before the attenuation storage system reaches its design capacity.

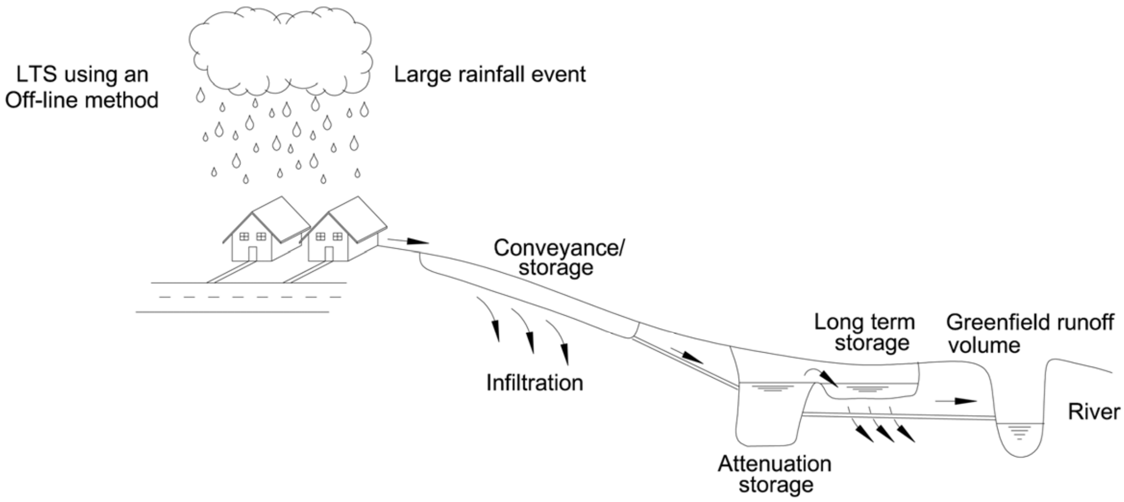


Figure 7.5 Activation of Long Term Storage from the Attenuation system.

The activation of Long Term Storage will therefore also need to be checked for when it comes into effect for the most frequent event. It is likely that this will be of the order of 5 years. This is still sufficiently rare for say flooding of a football field or even a car park, but in the case of the car park a maximum depth of 150mm or so would probably be required to avoid damage to vehicles for any size of event.

Finally there is the issue of public expectation, draining the area and clean up. If it is a dual use area, it is important that suitable notices exist to inform the public and that maintenance procedures are in place so as to ensure the community accept the flooding as part of normal practice. The drain-down of the area may be by infiltration or by direct throttle control, but any solution must be practical and robust in coming into effect in a proper manner even though it only has to operate rarely. As a direct discharge rate must be less than 2 l/s/ha, a simple throttle will rarely be effective as it would need to be impractically small unless the Long Term Storage unit served a development larger than about 6ha.

References for further guidance on stormwater design and planning

Several of the following references are referred to in the body of this report. Others have been included as they are useful documents which are relevant to drainage design.

ADAS, 1980. MAFF Report 5, Pipe size design for field drainage

ADAS, 1981, MAFF Report 345, The design of field drainage pipe systems

BRE, 1991, Digest 365, Soakaway design

British Standard, 1994, BS EN 752 –2, Drain and sewer systems outside buildings – Part 2 Performance requirements

British Standard, 1998, BS EN 752 -3, Drain and sewer systems outside buildings – Part 3 Planning

British Standard, 1998, BS EN 752 -4, Drain and sewer systems outside buildings – Part 4 Hydraulic design and Environmental considerations

British Standard, 2000, BS EN 12056, Gravity drainage systems inside buildings – Part 3 Roof drainage, layout and calculation

CEH 2006, Revised Flood Estimation Handbook, rainfall-runoff method.

CIRIA, 2002, Report C582, Pratt, Wilson, Cooper: Source Control using constructed pervious surfaces; hydraulic, structural and water quality performance issues

CIRIA, 2001, Martin, P, *et al,* Report 523, Sustainable Urban Drainage Systems – Best Practice Guide

CIRIA, 2000, Martin, P, *et al,* Report 522, Sustainable Urban Drainage Systems – Design manual for England and Wales

CIRIA, 2000, Martin, P, *et al,* Report 521, Sustainable Urban Drainage Systems – Design manual for Scotland and Northern Ireland

CIRIA, 1996, Report 156, Bettess R, "Infiltration drainage – Manual of good practice". Construction Industry Research and Information Association, ISBN 0 86017 457 3.

CIRIA, 1993, Hall, Hockin and Ellis, "Design of Flood Storage Reservoirs".
Copas, B. A., 1957, "Stormwater Storage Calculation". J. Inst. Public Health Eng. 56(3) pp137 – 162.

CLG, 2010 (first ed. 2006), PPS25, Planning Policy and Guidance – Development and Flood risk (Replacing PPG25).

DETR, 2000, PPG3, Housing

DOE, 1981, The Wallingford Procedure - Design and analysis of Urban Storm Drainage. HR Wallingford

DOE, 1981, Volume 4 Modified Rational Method, The Wallingford Procedure. HR Wallingford

DOE, 2002, Building Regulations part H – Drainage and Waste disposal

DOE, 1994, PPG23, Planning and Pollution control

EDAW / AECOM, 2009, Urban Stormwater Best Practice Environmental Management (BPEM) Guidelines, Review of the principles, objectives and targets – A Discussion Paper. Prepared for EPA Victoria

Institute of Hydrology, 1999, Flood Estimation Handbook (FEH), 5 volumes, Reed, Faulkner, Bayliss.

Institute of Hydrology, 1994, IH Report 124, Flood estimation for small catchments

Institute of Hydrology, 1994, IH Report 126, Hydrology of Soil Types: a hydrologically based classification of soils of the United Kingdom.

HR Wallingford, 2000, Kellagher, R, The Wallingford Procedure for Europe: Best Practice Guide to Urban Drainage Modelling

HR Wallingford, D. Barr, 1997, Tables for the hydraulic design of pipes, sewers and channels. 7th edition

HR Wallingford, 2002, Kellagher, R, Report SR591, Overview summary – Storage requirements for rainfall runoff from green field development sites

HR Wallingford, 2002, Kellagher, R, Report SR574, Drainage of development sites – a guide

JBA, 2011, "Greenfield runoff and flood estimation on small catchments" Faulkner. D et al

NERC, 1975, Flood Studies Report, by Institute of Hydrology.

NERC, 1977, FSSR 2 The estimation of low return period flows – Flood Studies Supplementary Report, Institute of Hydrology

NERC, 1978, FSSR 6, Flood prediction for small catchments - Flood Studies Supplementary Report, Institute of Hydrology

NERC, 1983, FSSR 14, Review of Regional Growth curves, Flood Studies Supplementary Report, Institute of Hydrology

NERC, 1985, FSSR 16, The FSR rainfall-runoff model parameter estimation equations updated - Flood Studies Supplementary report, Institute of Hydrology

Water UK 2012, Sewers for Adoption 7th edition, A design and construction guide for developers published by WRc

Appendices

Appendix 1 Figures and graphs

Figure A1.1 Hydrological regions of UK

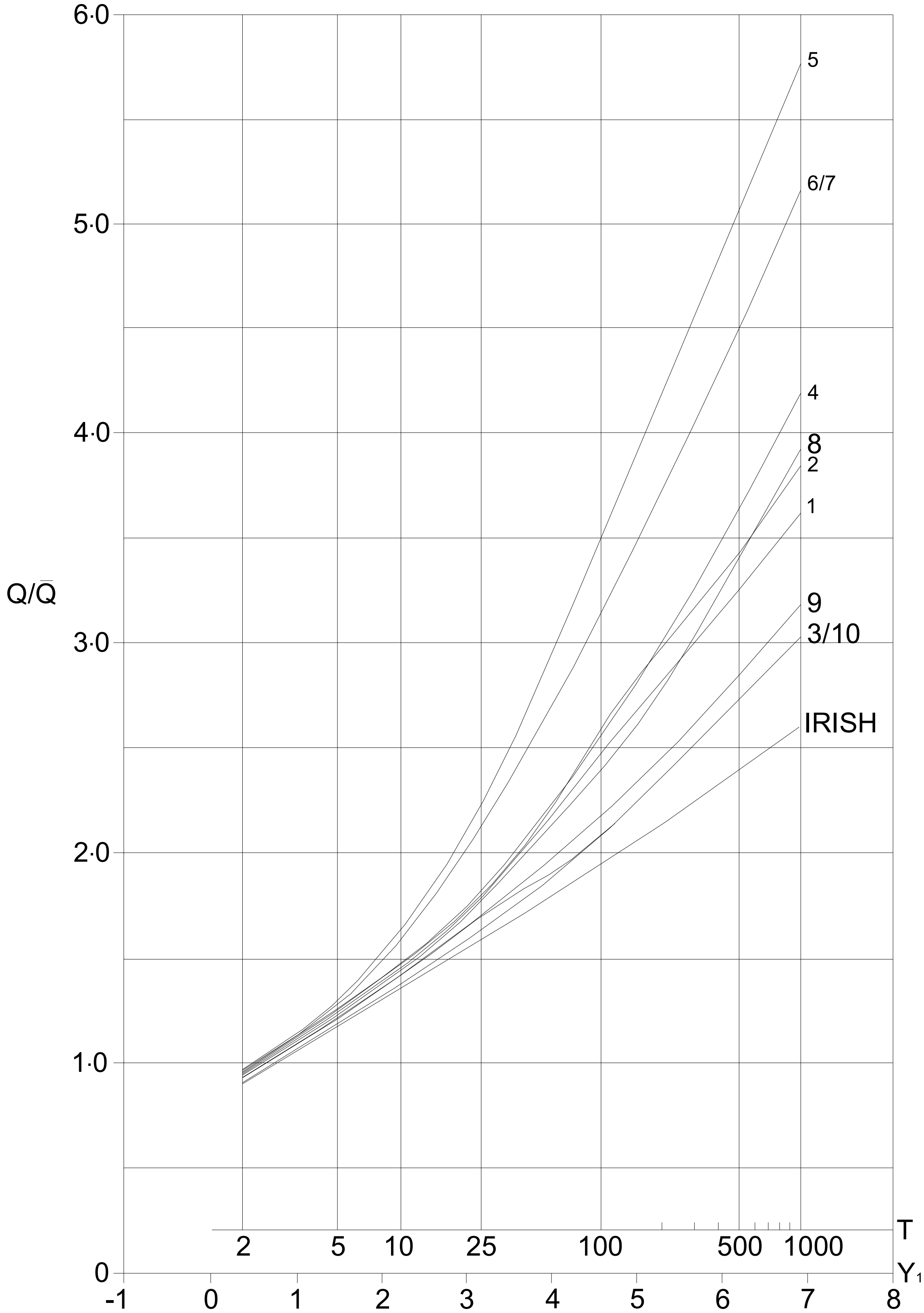


Figure A1.2 Peak flow growth curves of UK (from FSSR 14)

Growth curve	Growth curve Factor	
5	1	1
6/7	1.05	1.17
4	1.27	1.38
8	1.27	1.46
2	1.27	1.34
1	1.27	1.42
9	1.33	1.59
3/10	1.4	1.68
Duration (year)	**30**	**100**

Figure A1.3 Growth curve factors

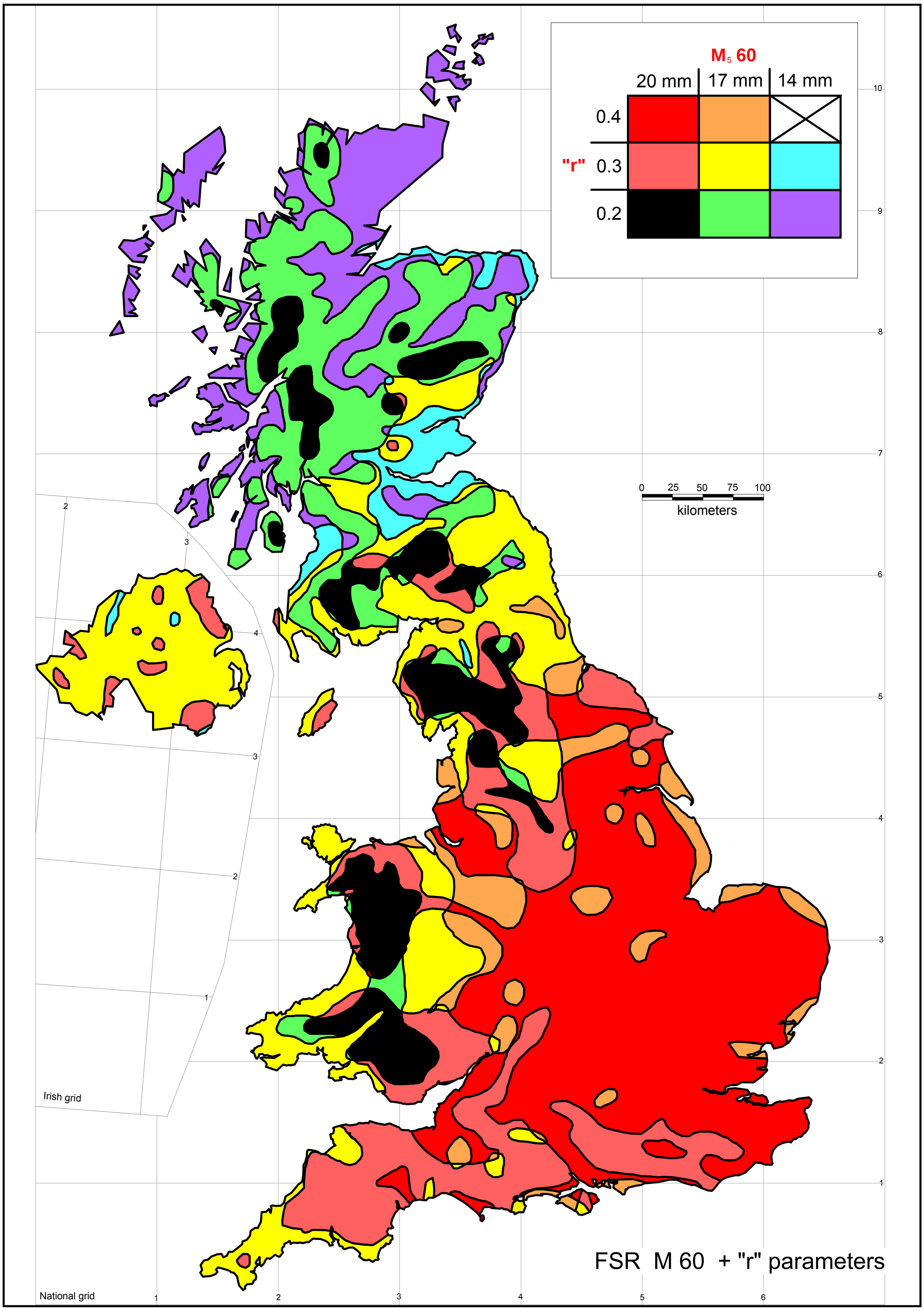


Figure A2.1 5 year 60 minute rainfall depth parameters of UK

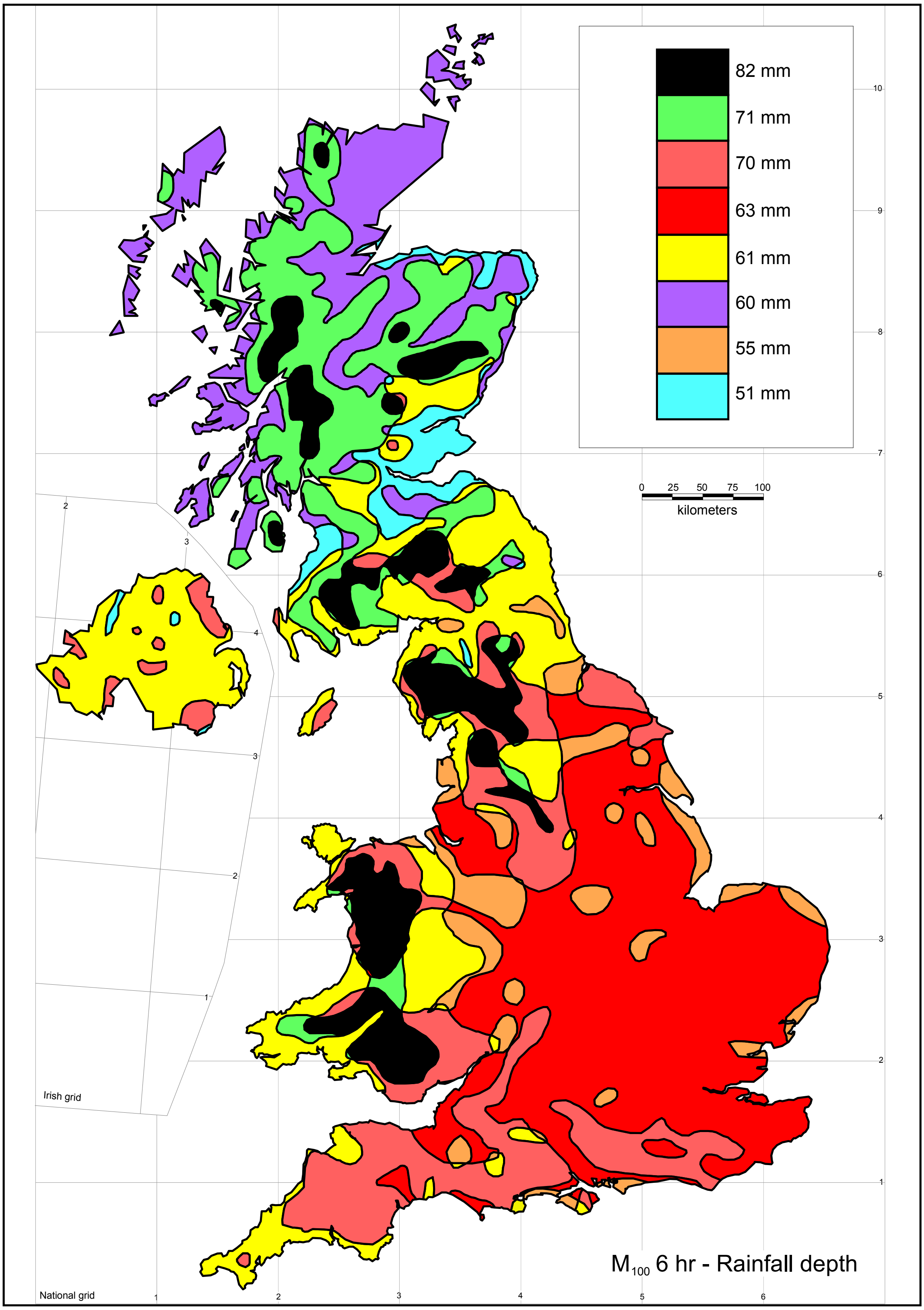


Figure A3.1 100 year 6 hour rainfall depths of UK

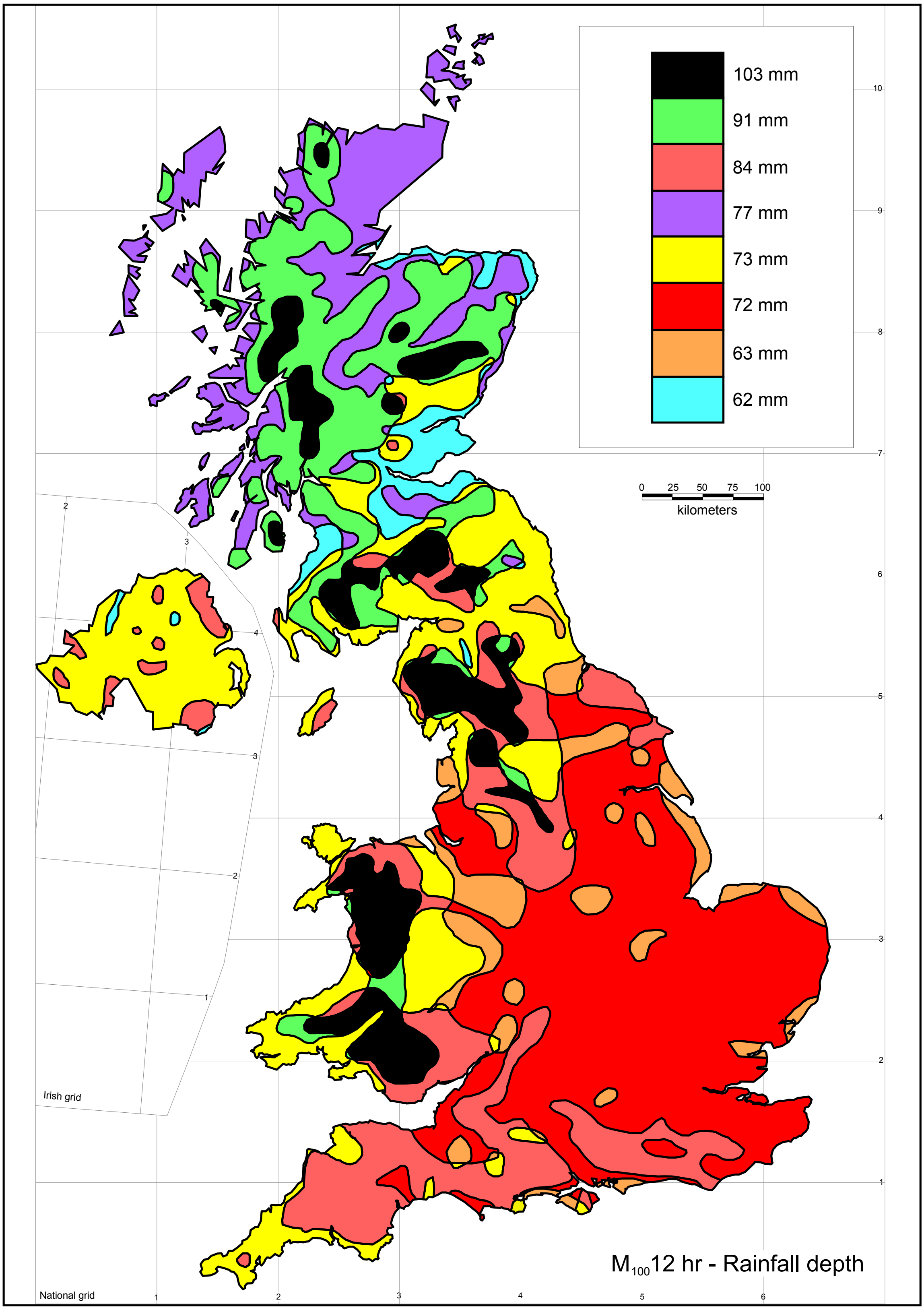


Figure A3.2 100 year 12 hour rainfall depths of UK

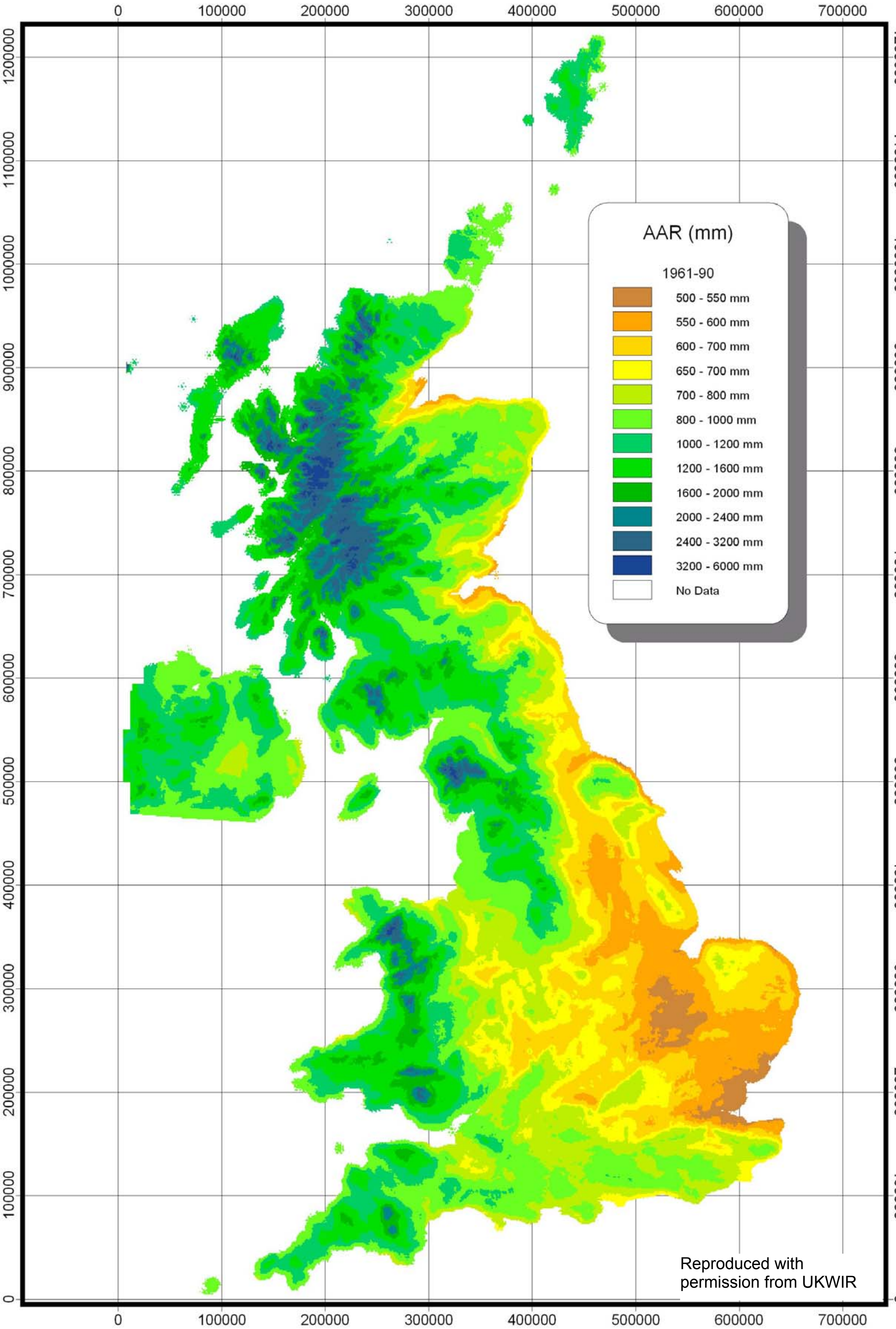


Figure A4.1 Average annual rainfall (1961 – 1990) (from FEH)

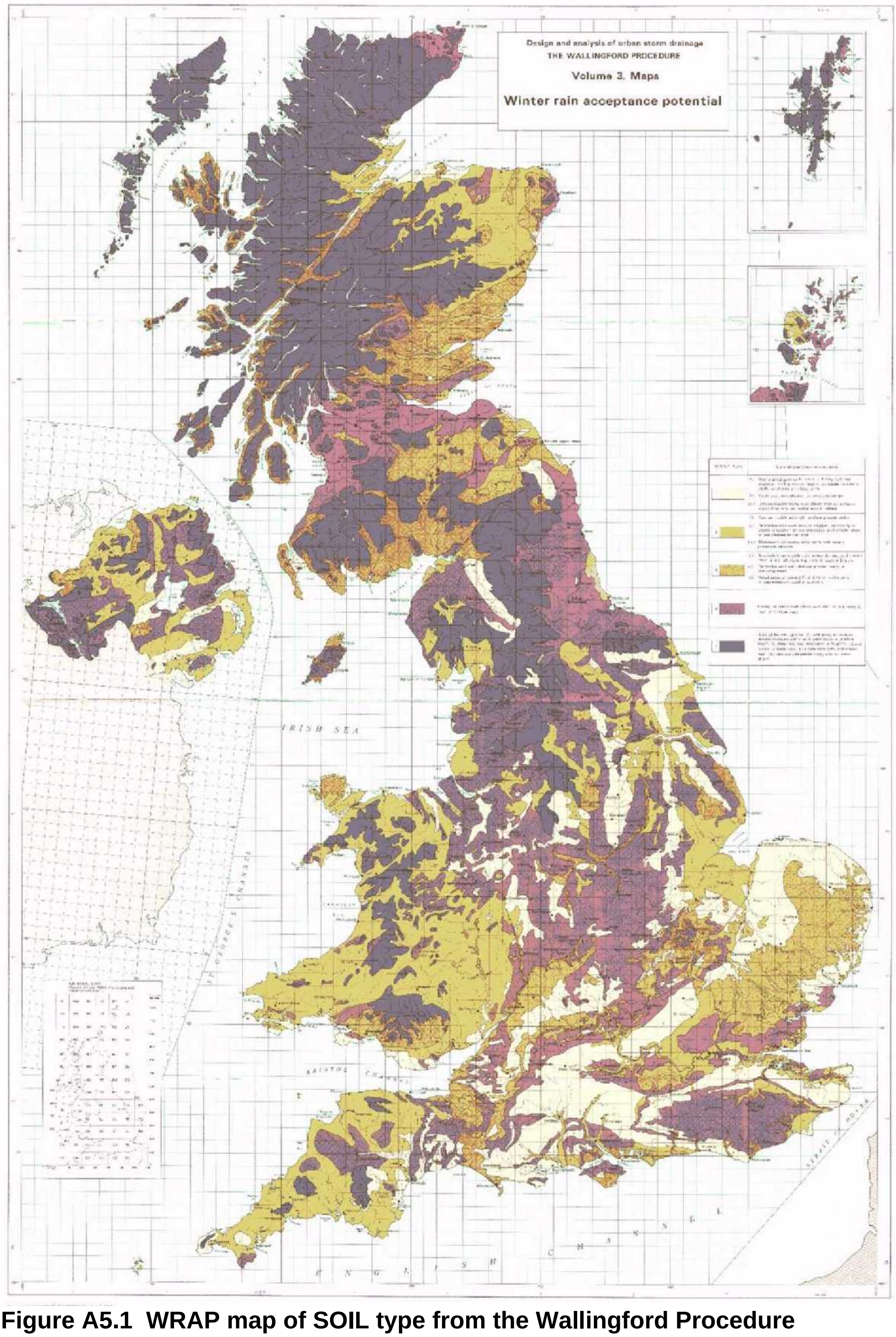


Figure A5.1 WRAP map of SOIL type from the Wallingford Procedure

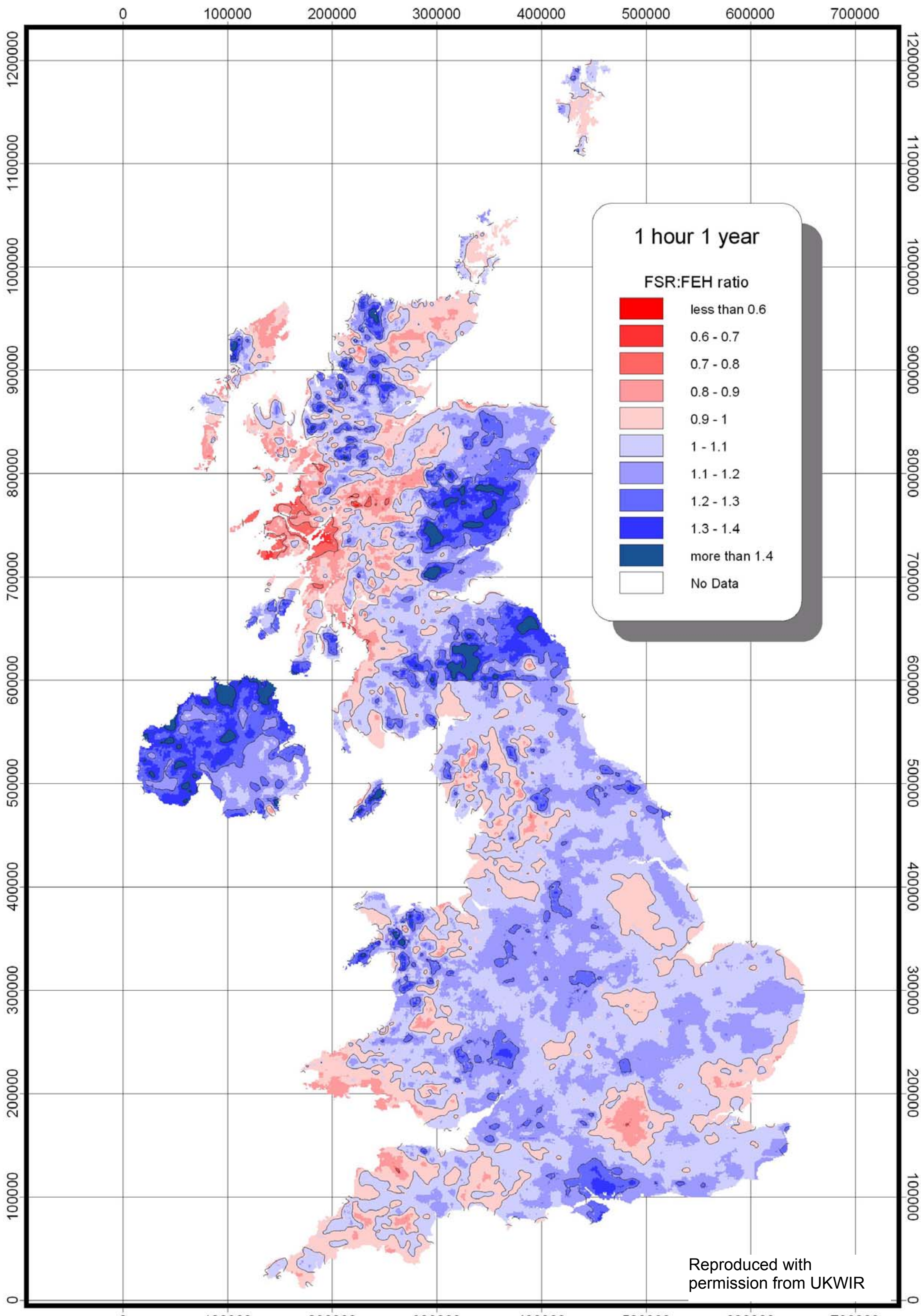


Figure A6.1.1 FSR/FEH rainfall depth ratios

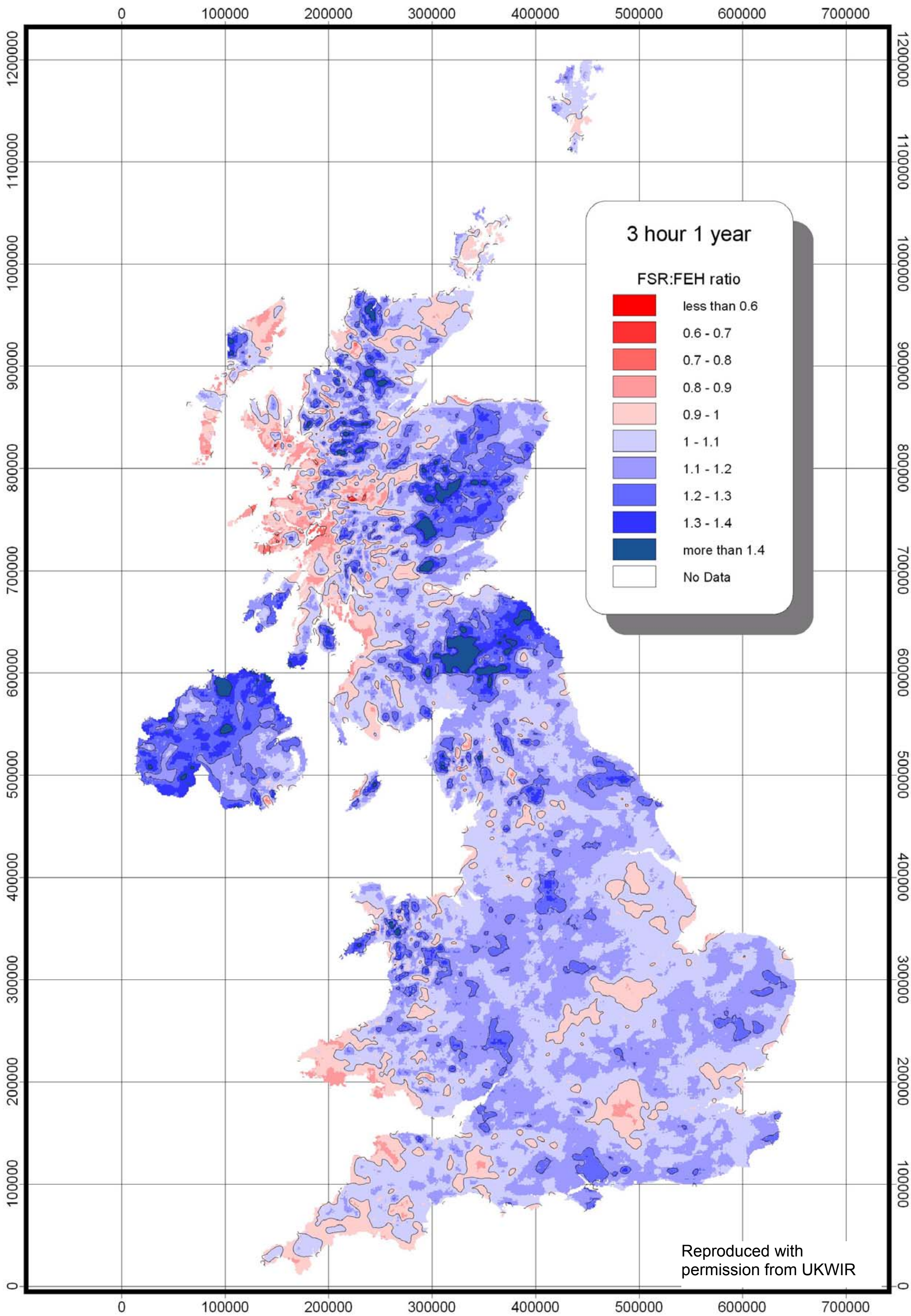


Figure A6.1.2 FSR/FEH rainfall depth ratios

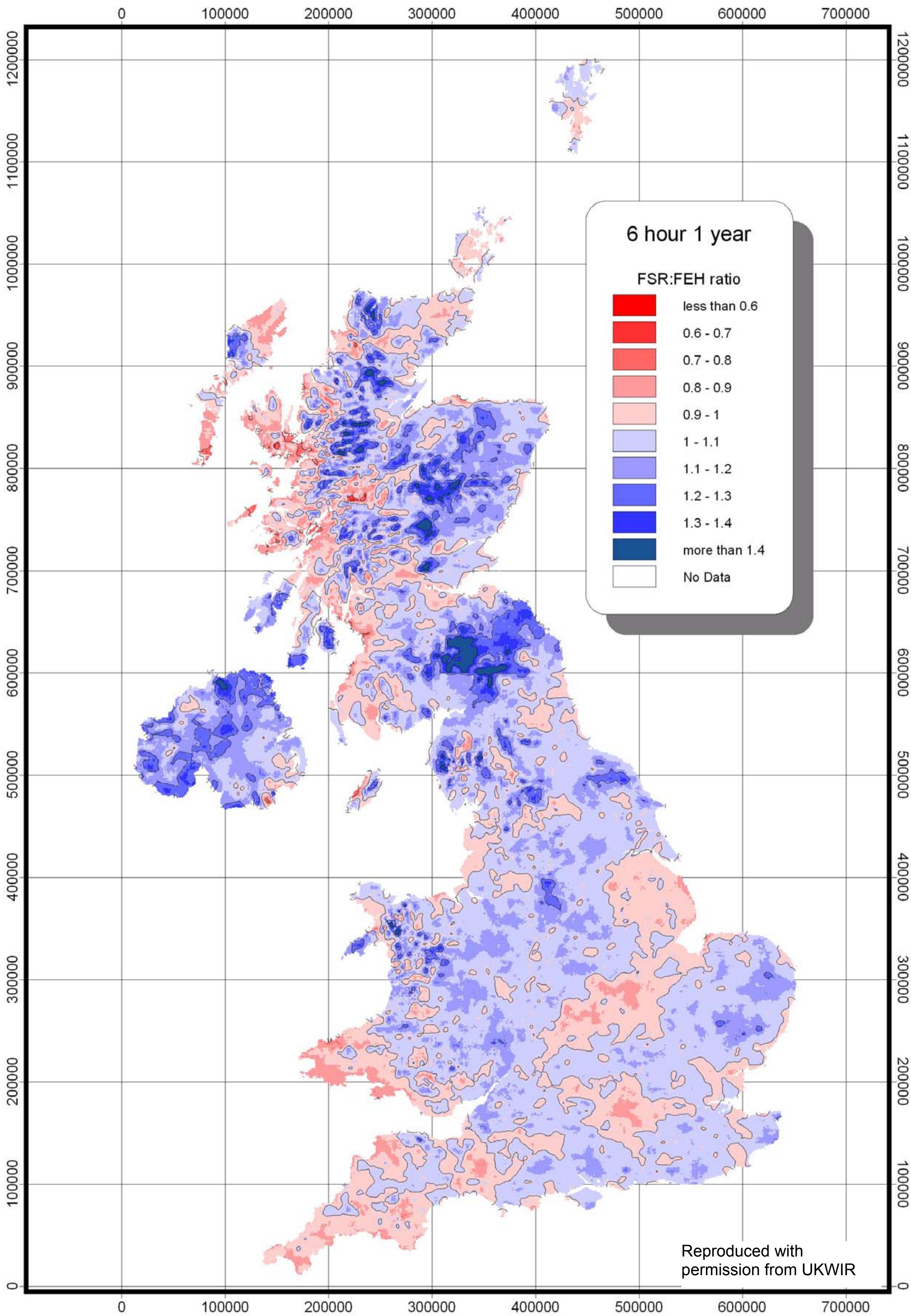


Figure A6.1.3 FSR/FEH rainfall depth ratios

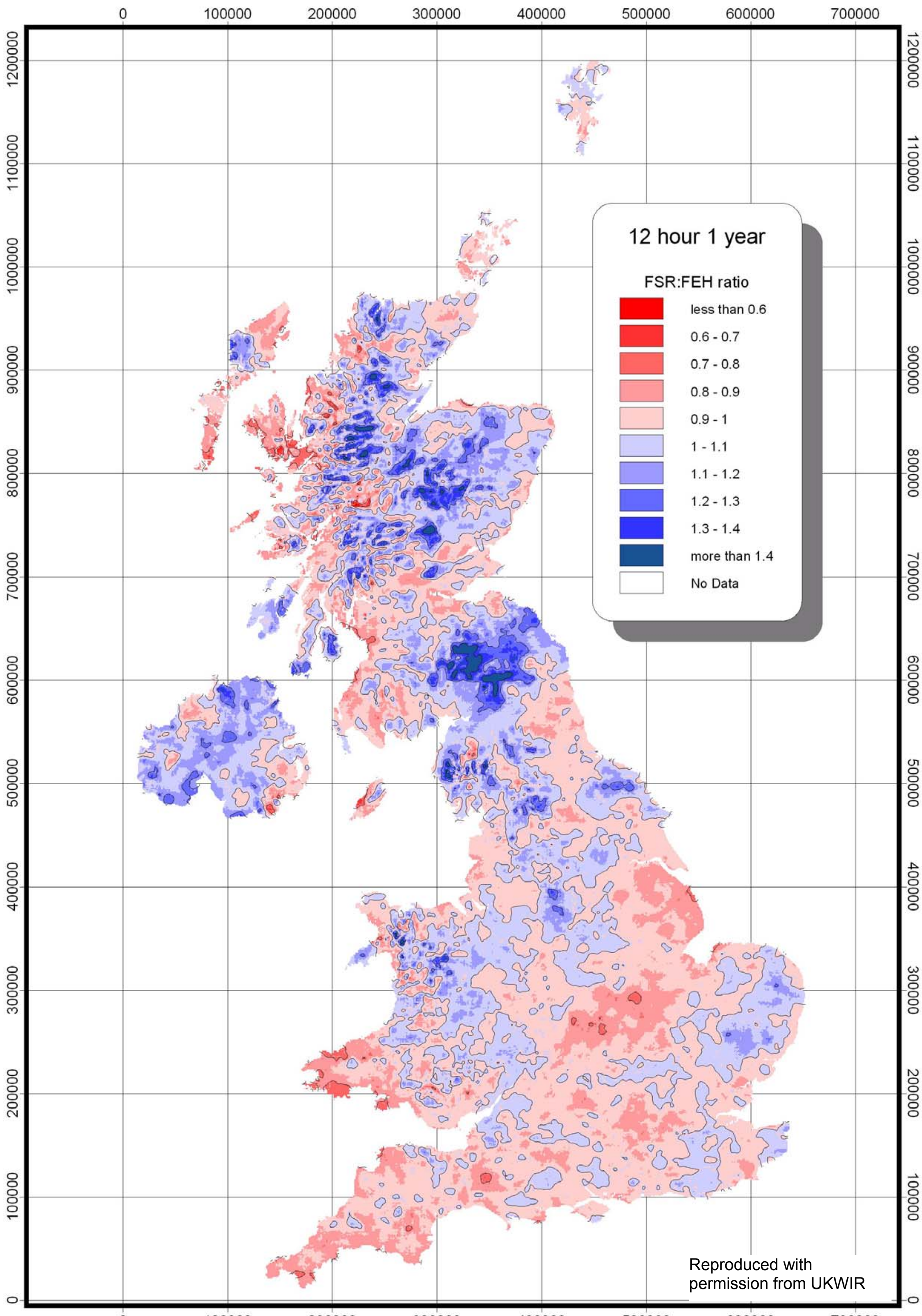


Figure A6.1.4 FSR/FEH rainfall depth ratios

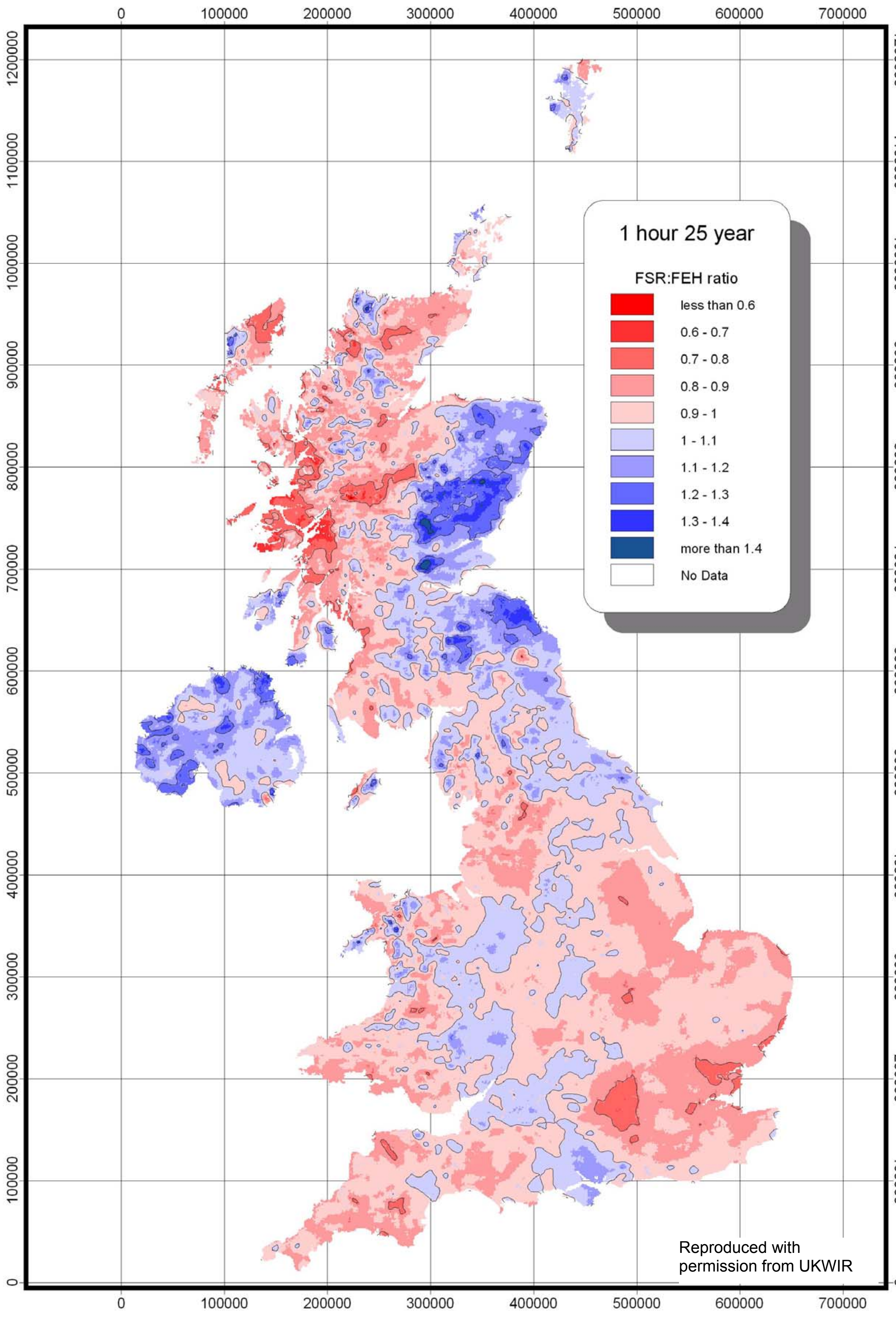


Figure A6.2.1 FSR/FEH rainfall depth ratios

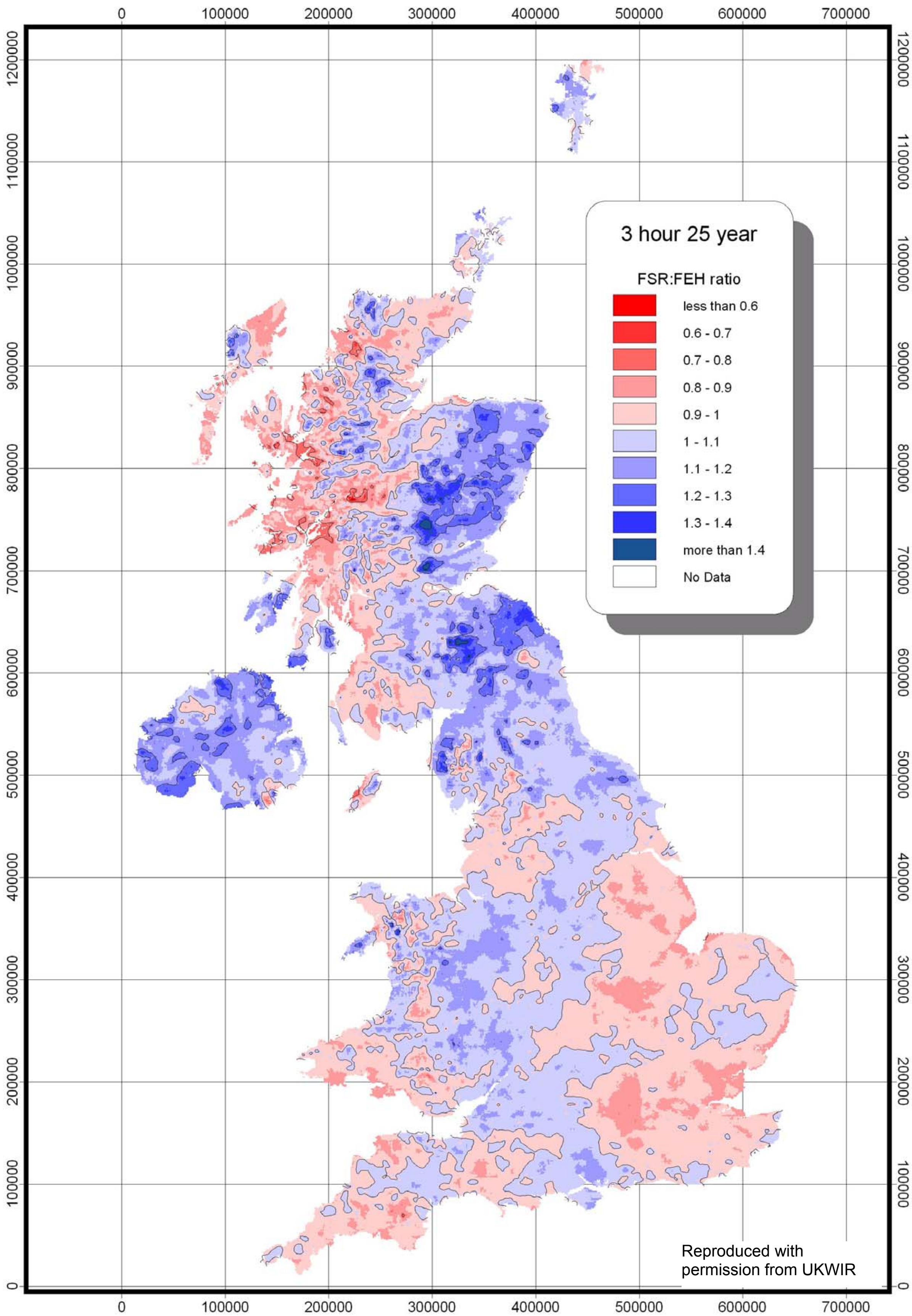


Figure A6.2.2 FSR/FEH rainfall depth ratios

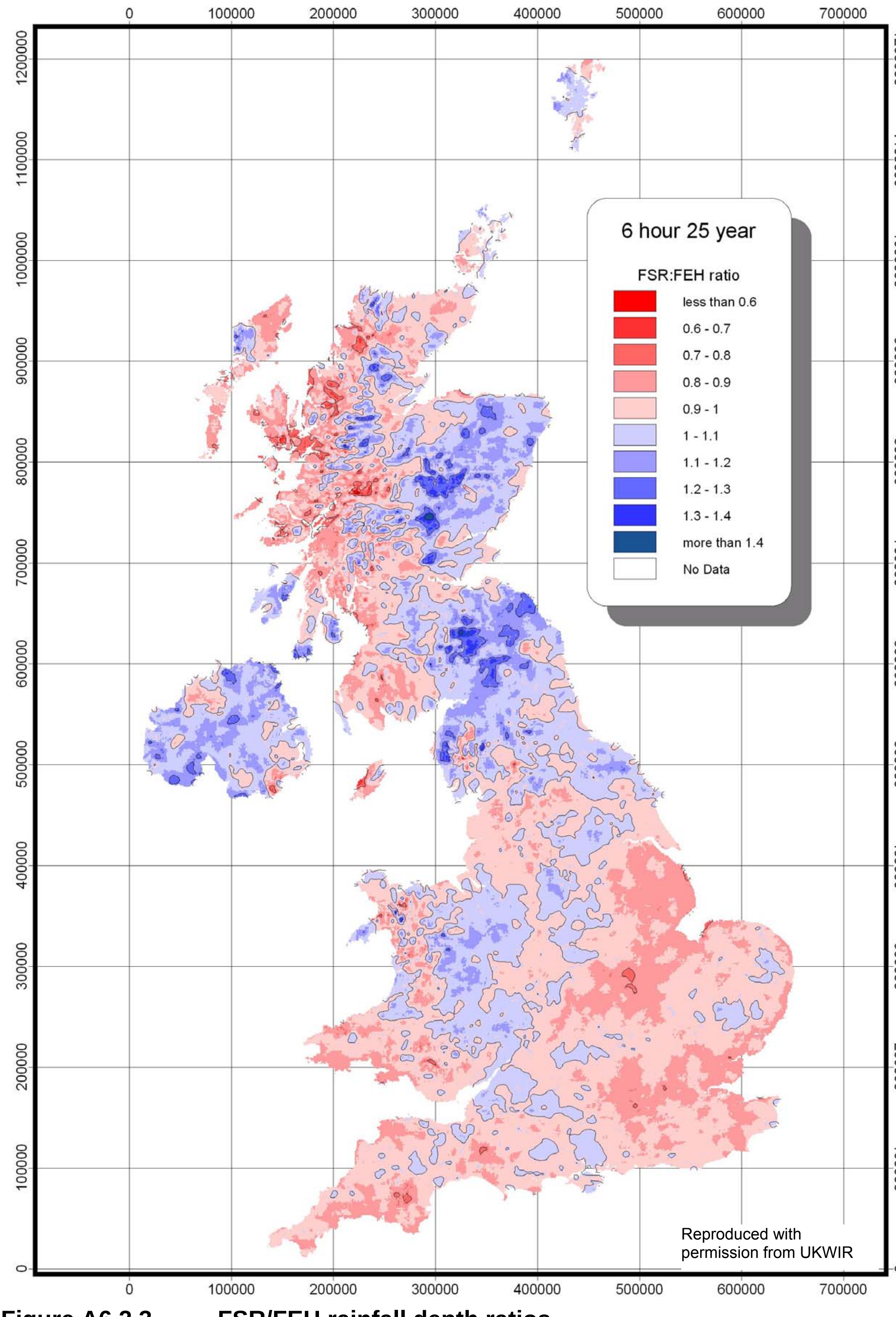


Figure A6.2.3 FSR/FEH rainfall depth ratios

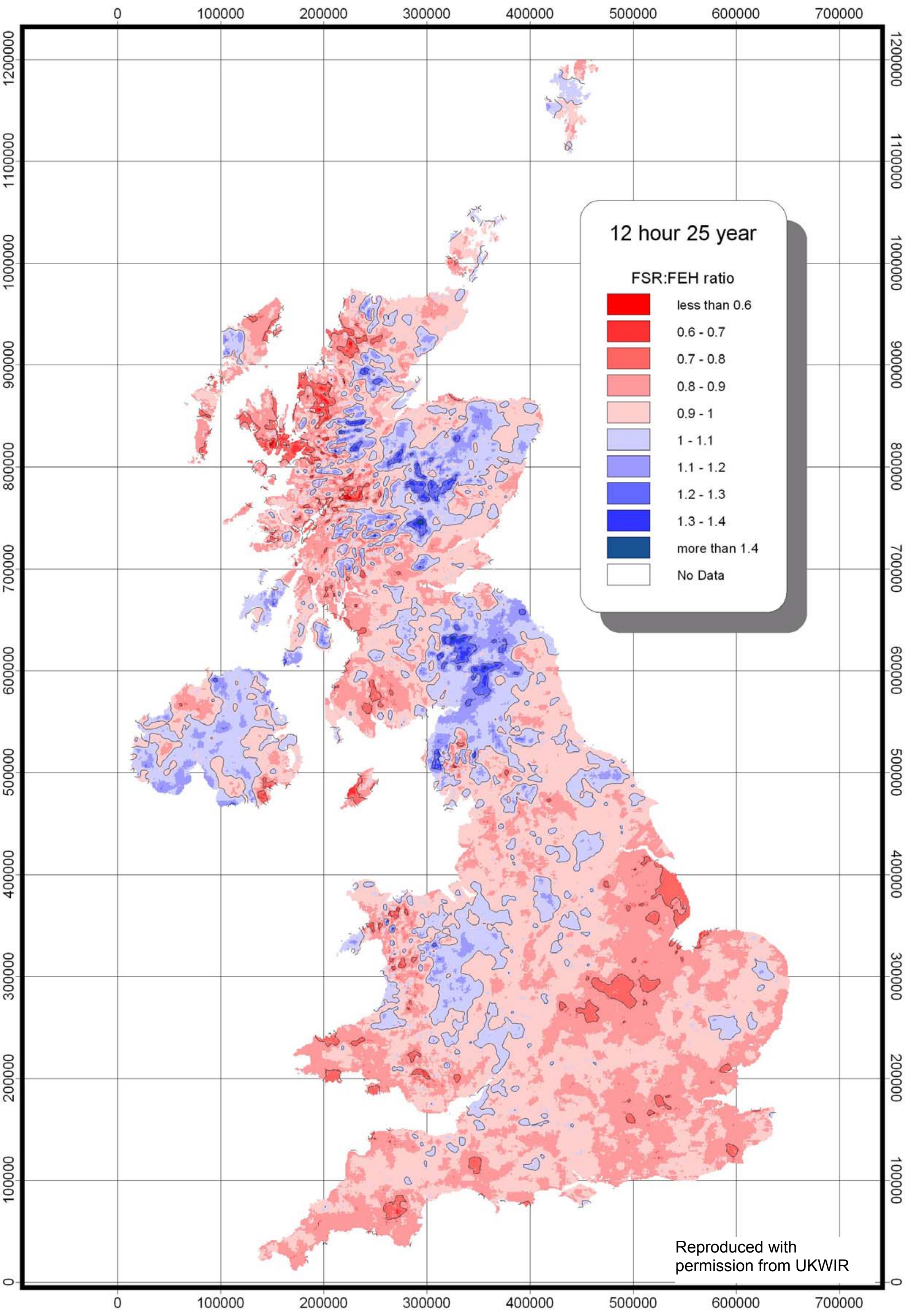


Figure A6.2.4 FSR/FEH rainfall depth ratios

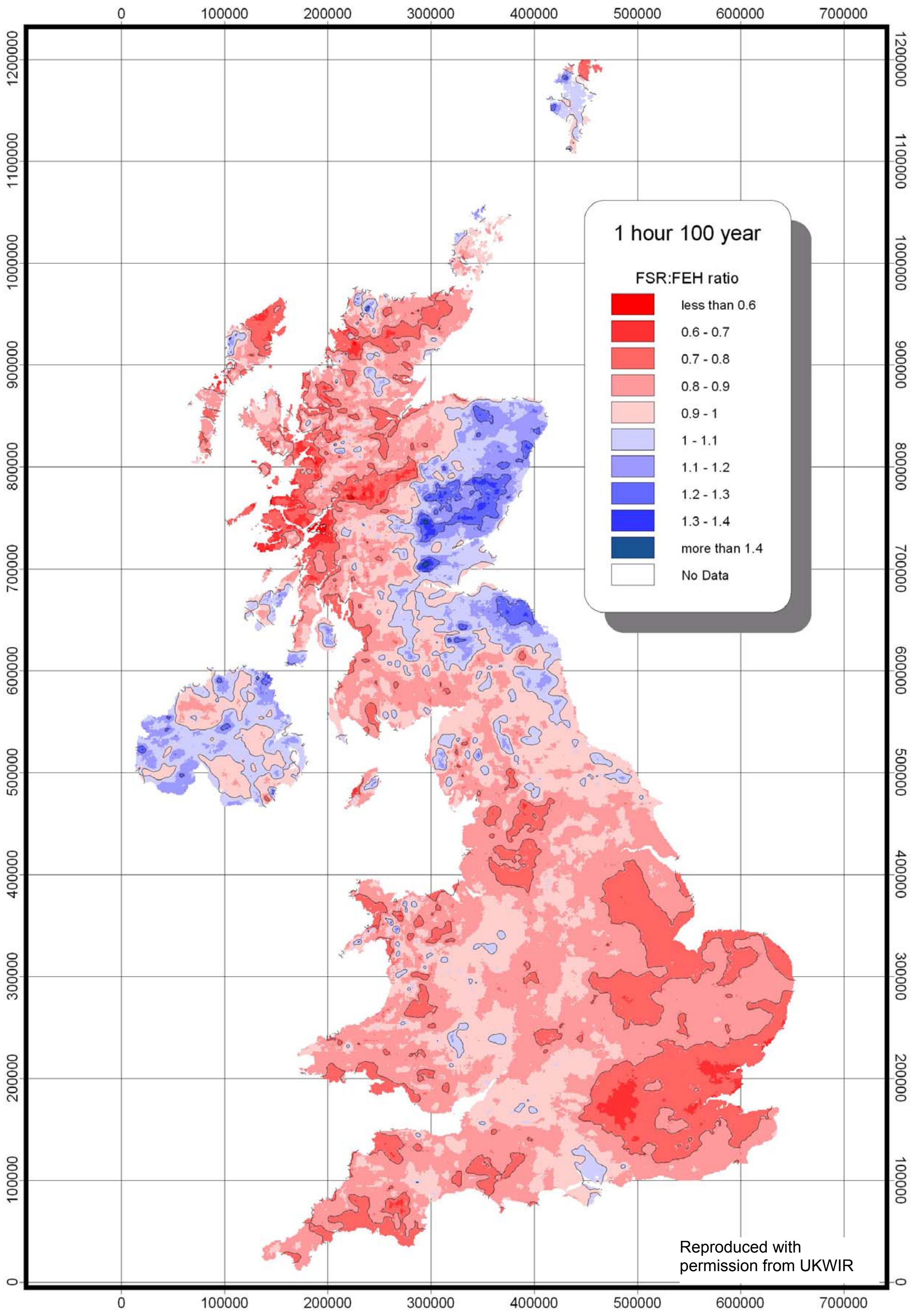


Figure A6.3.1 FSR/FEH rainfall depth ratios

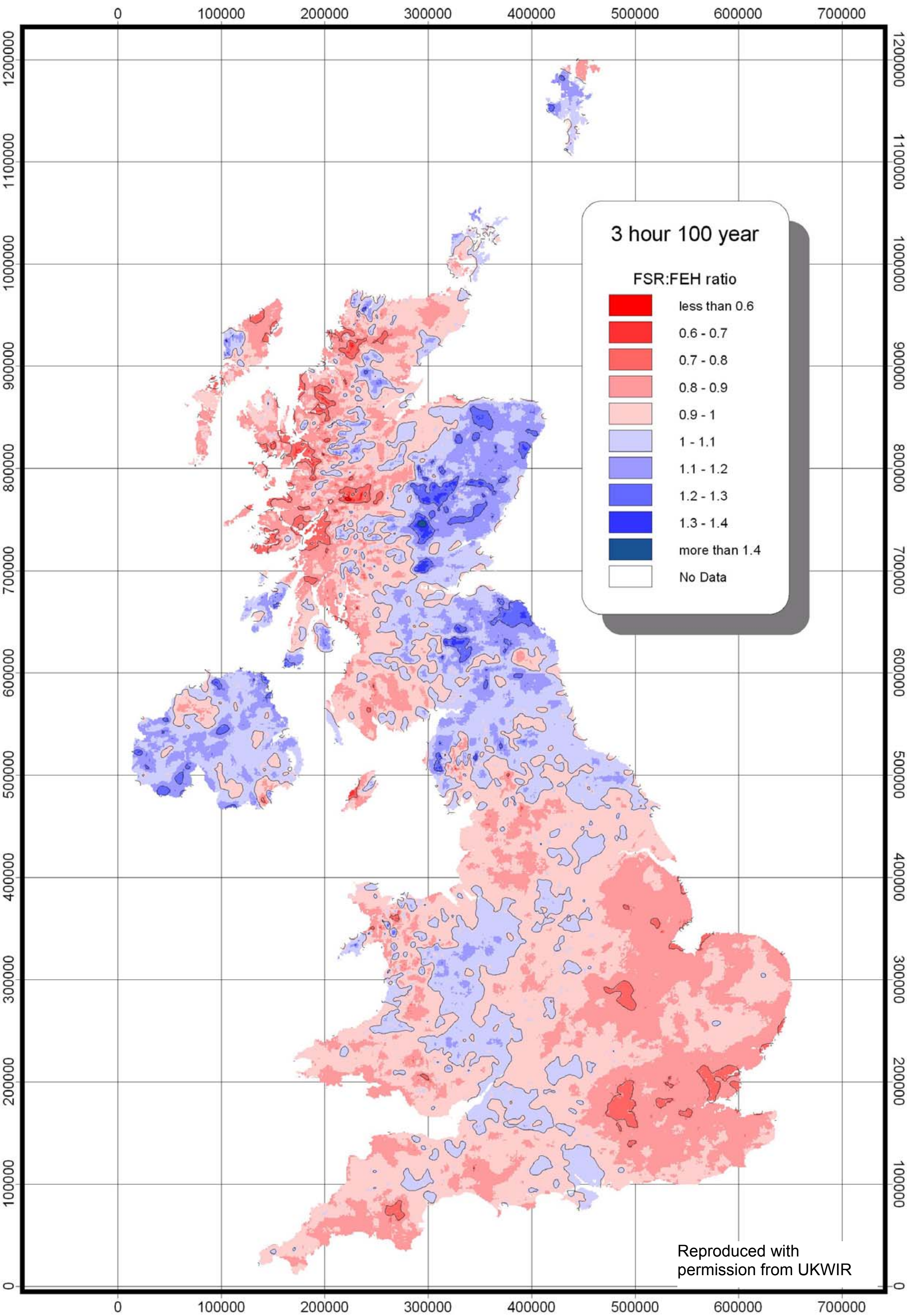


Figure A6.3.2 FSR/FEH rainfall depth ratios

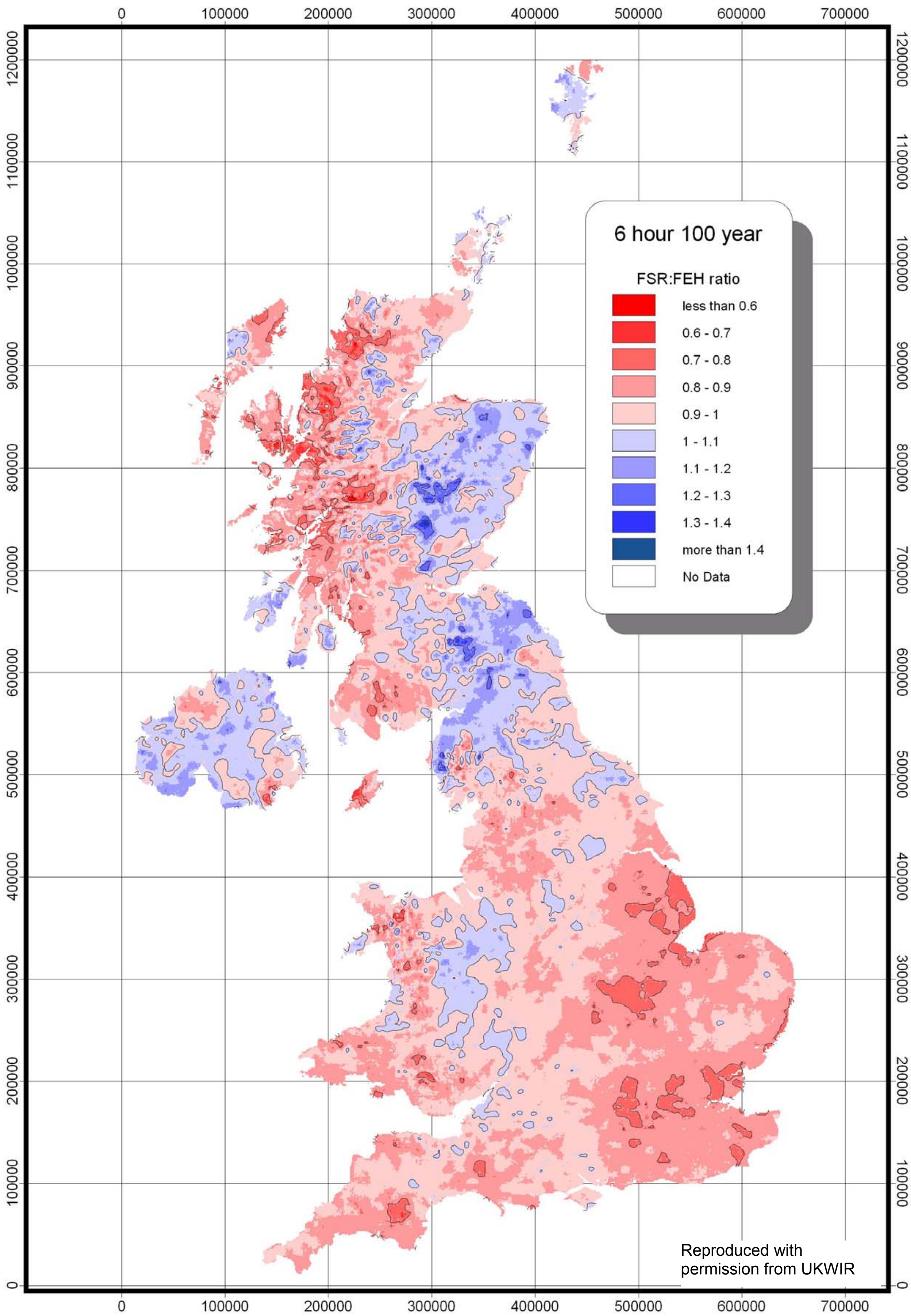


Figure A6.3.3 FSR/FEH rainfall depth ratios

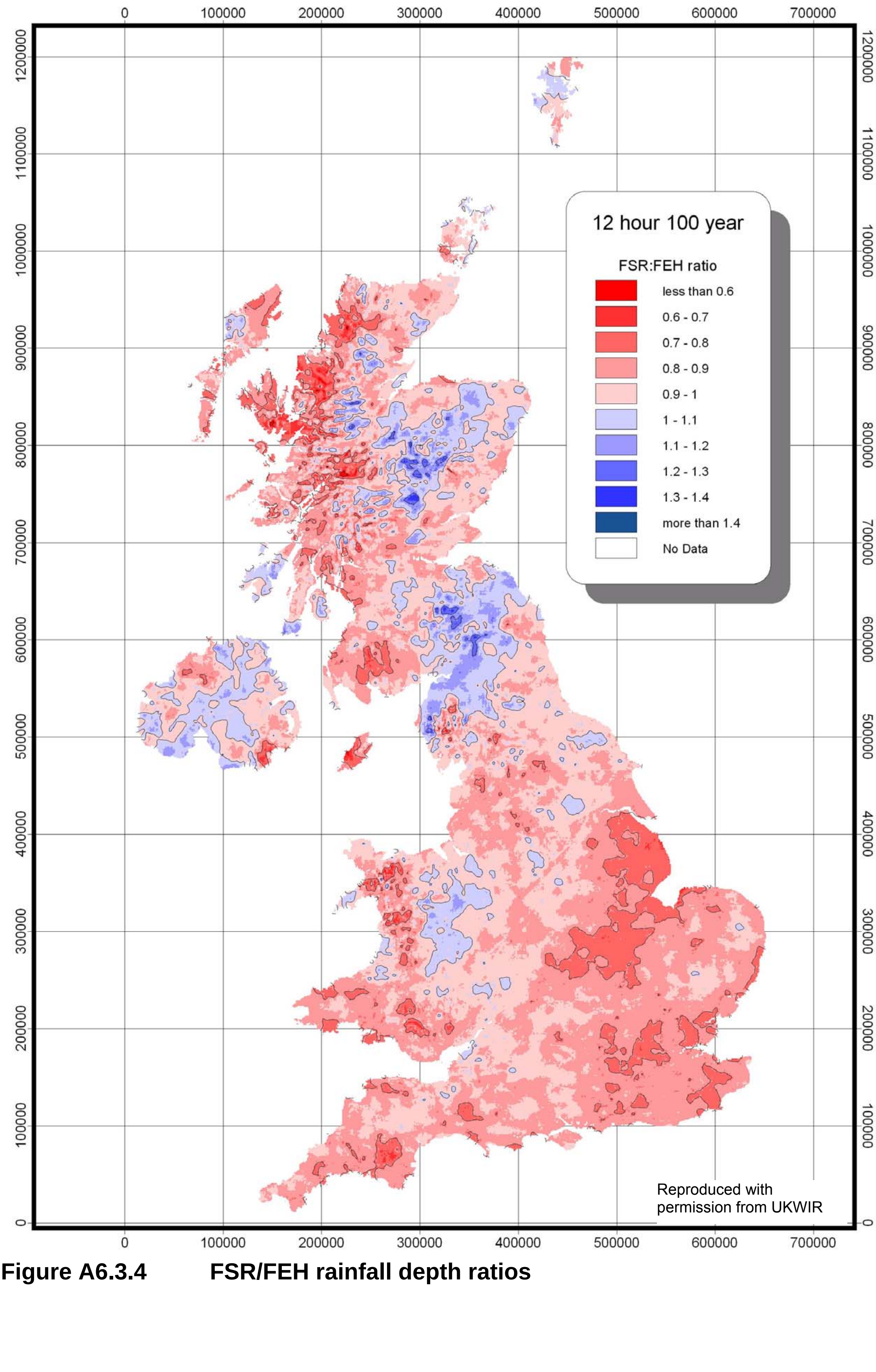


Figure A6.3.4 FSR/FEH rainfall depth ratios

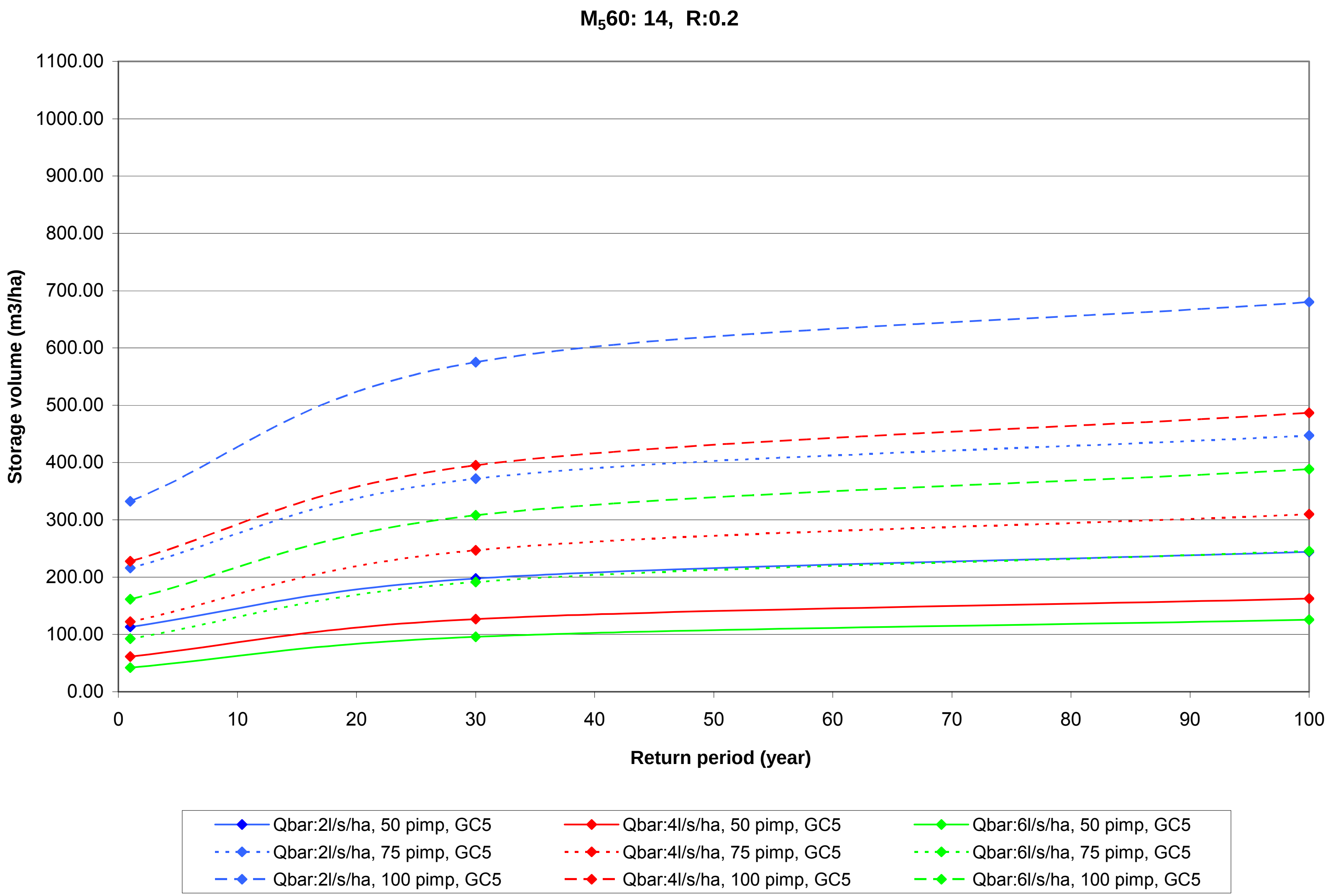


Figure A7.1 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:14, “r”:0.2)

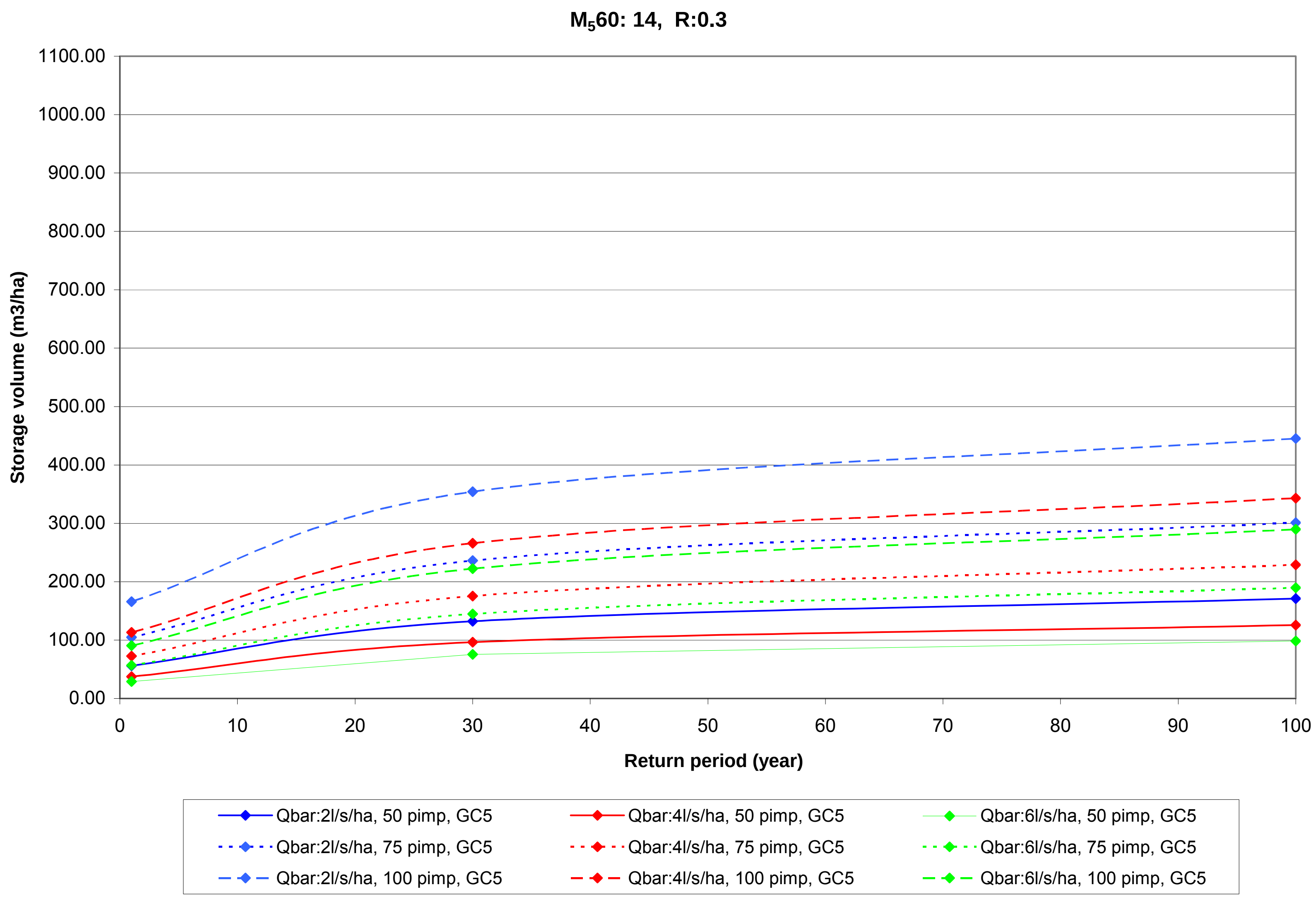


Figure A7.2 Attenuation storage volume as a function of Q_{BAR}/A and PIMP (M_560:14, "r":0.3)

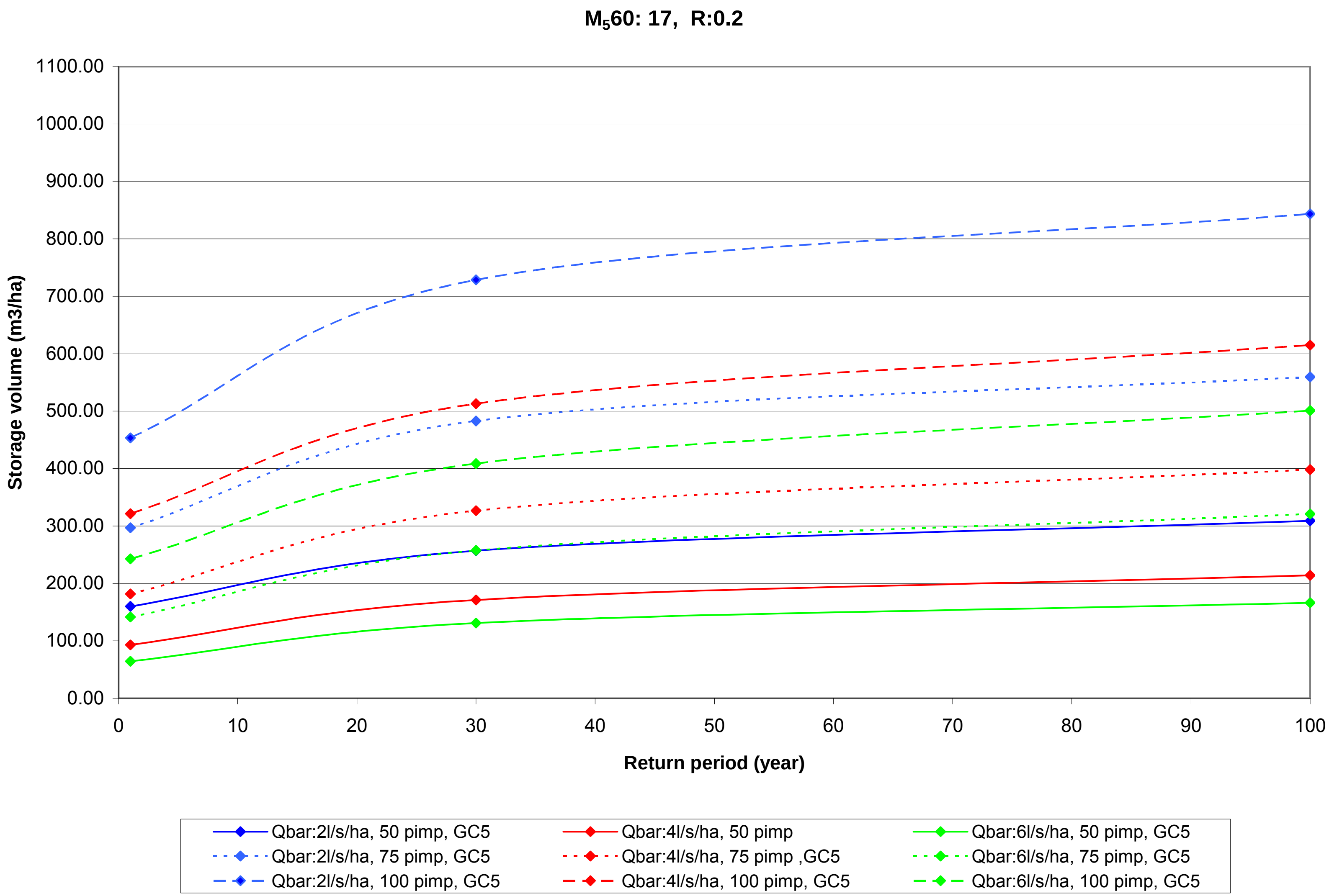


Figure A7.3 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.2)

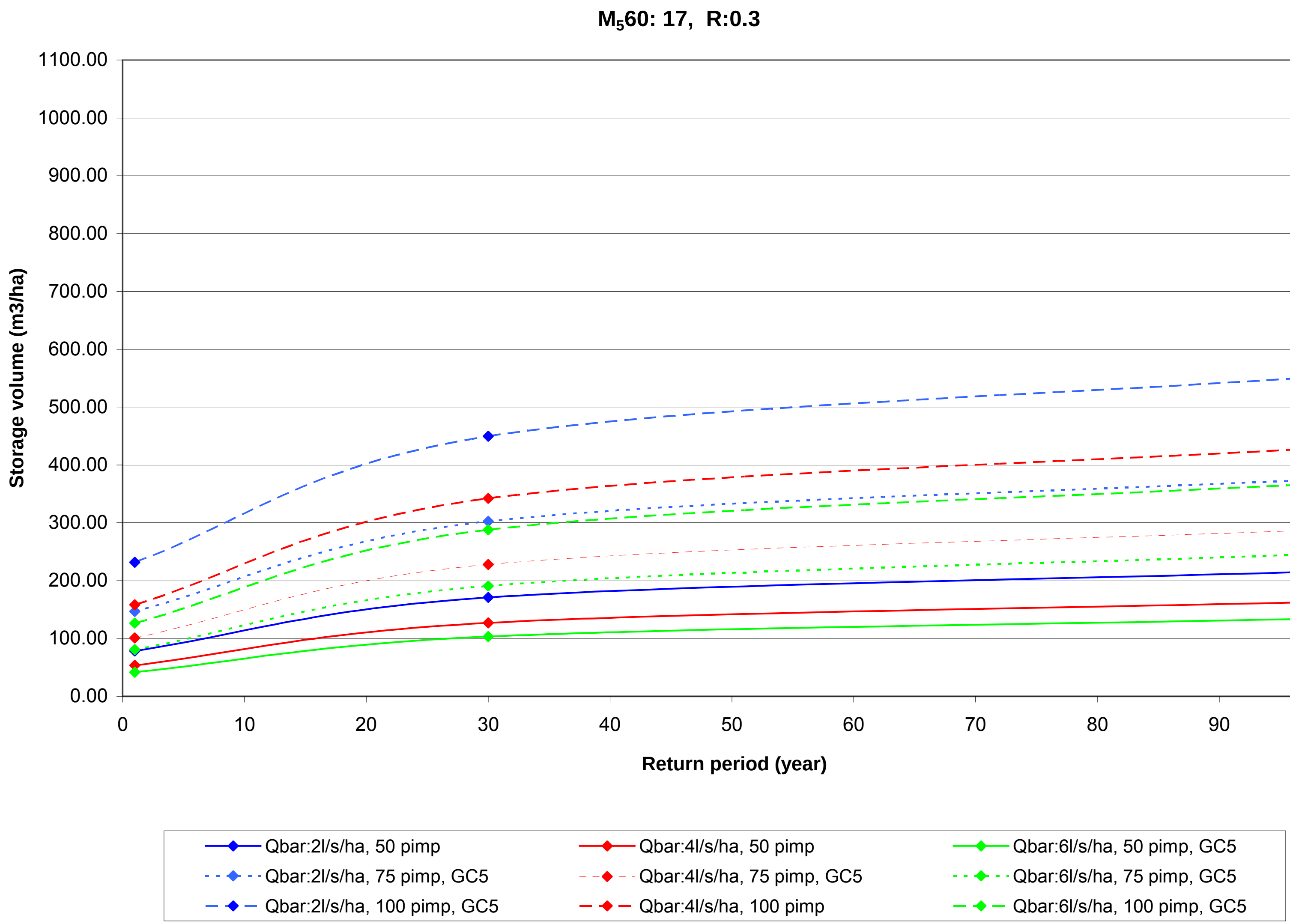


Figure A7.4 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.3)

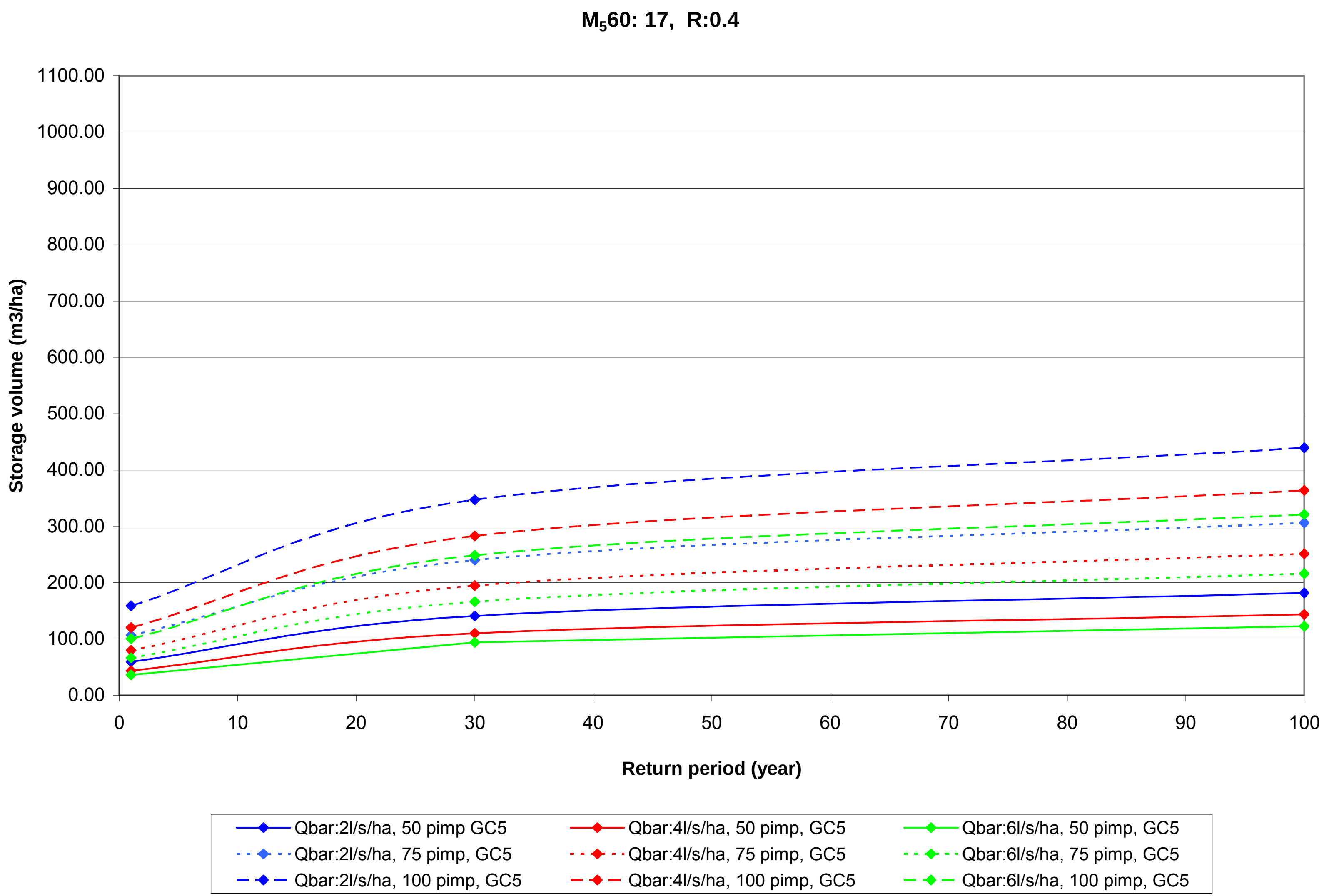


Figure A7.5 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.4)

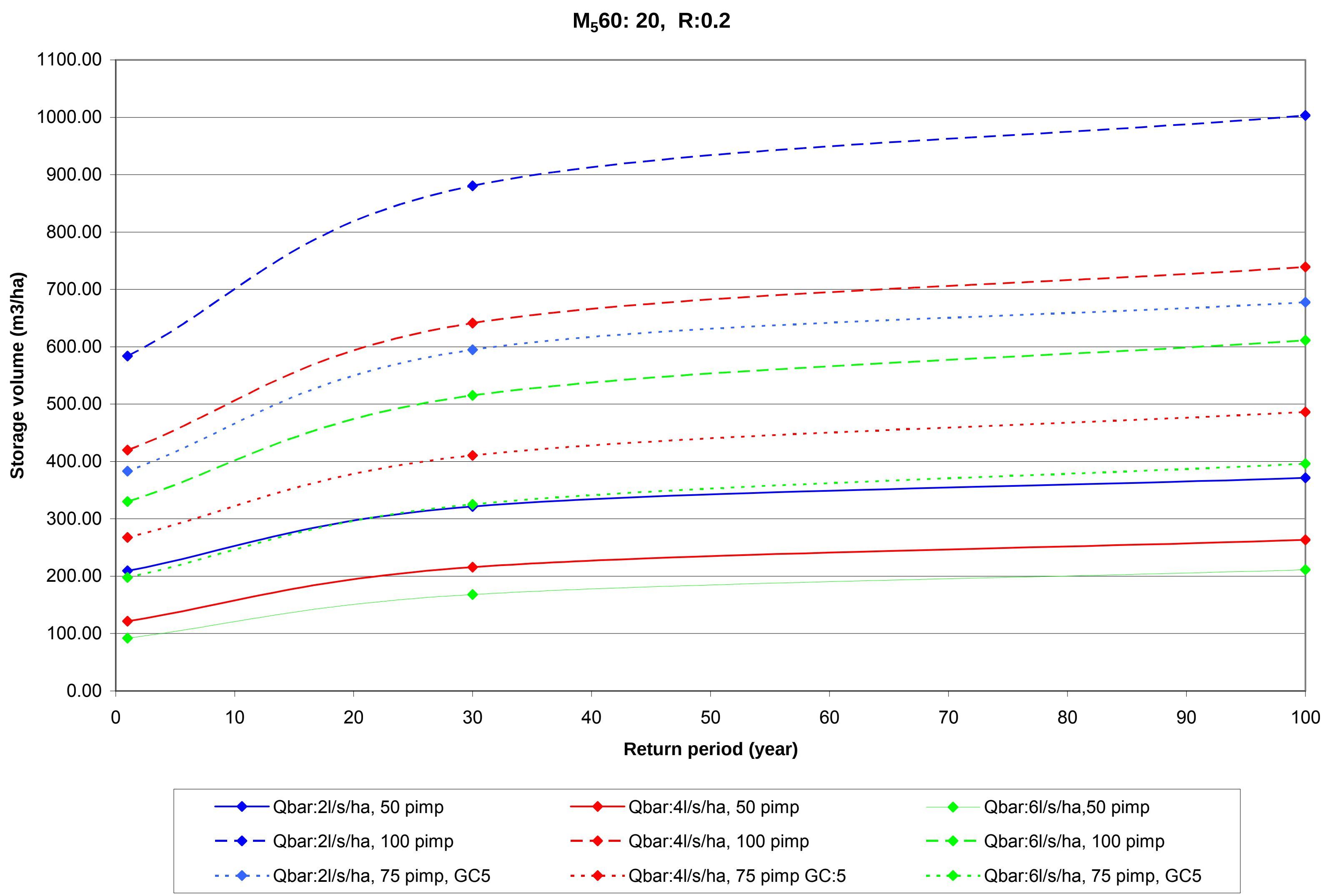


Figure A7.6 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, "r":0.2)

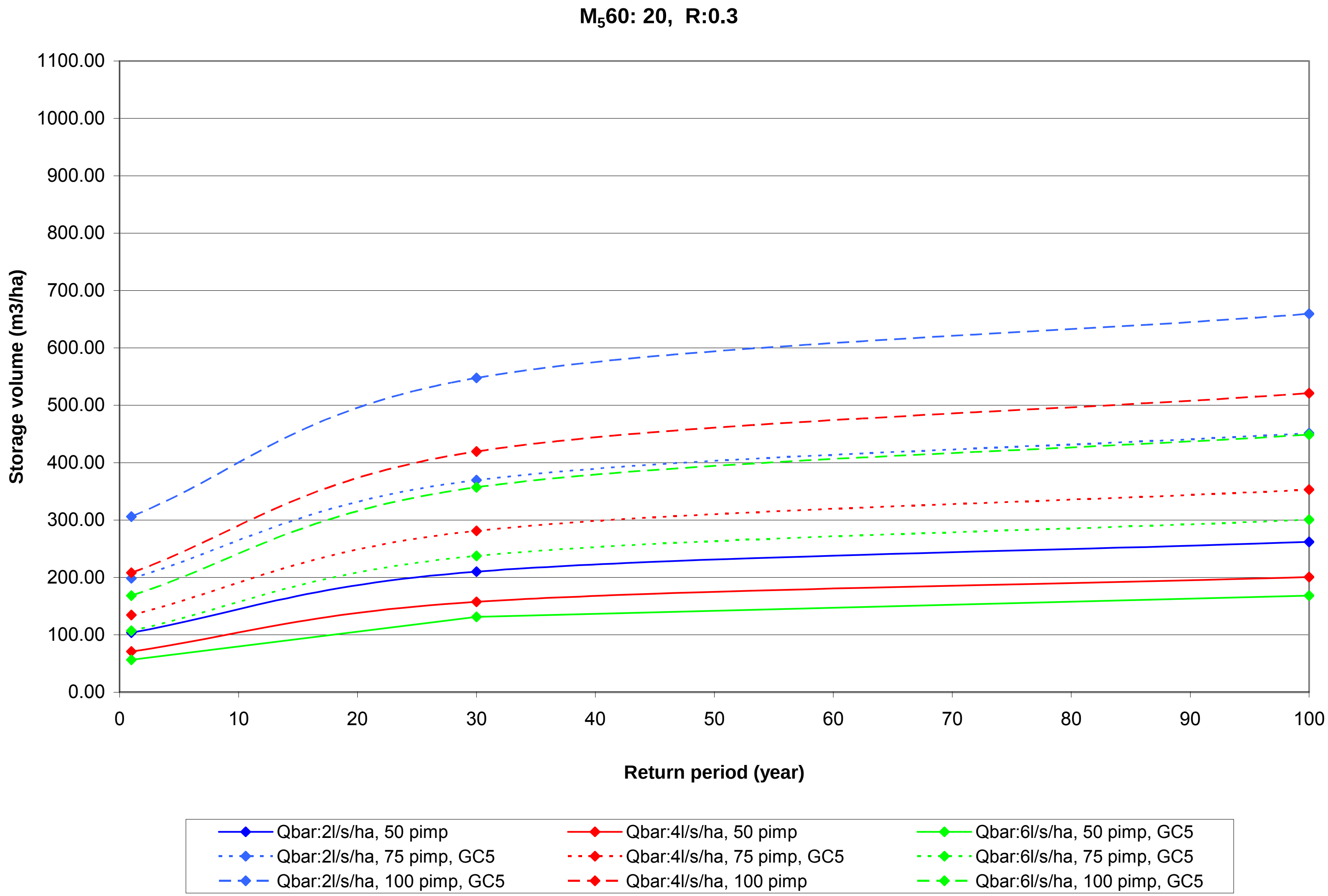


Figure A7.7 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, "r":0.3)

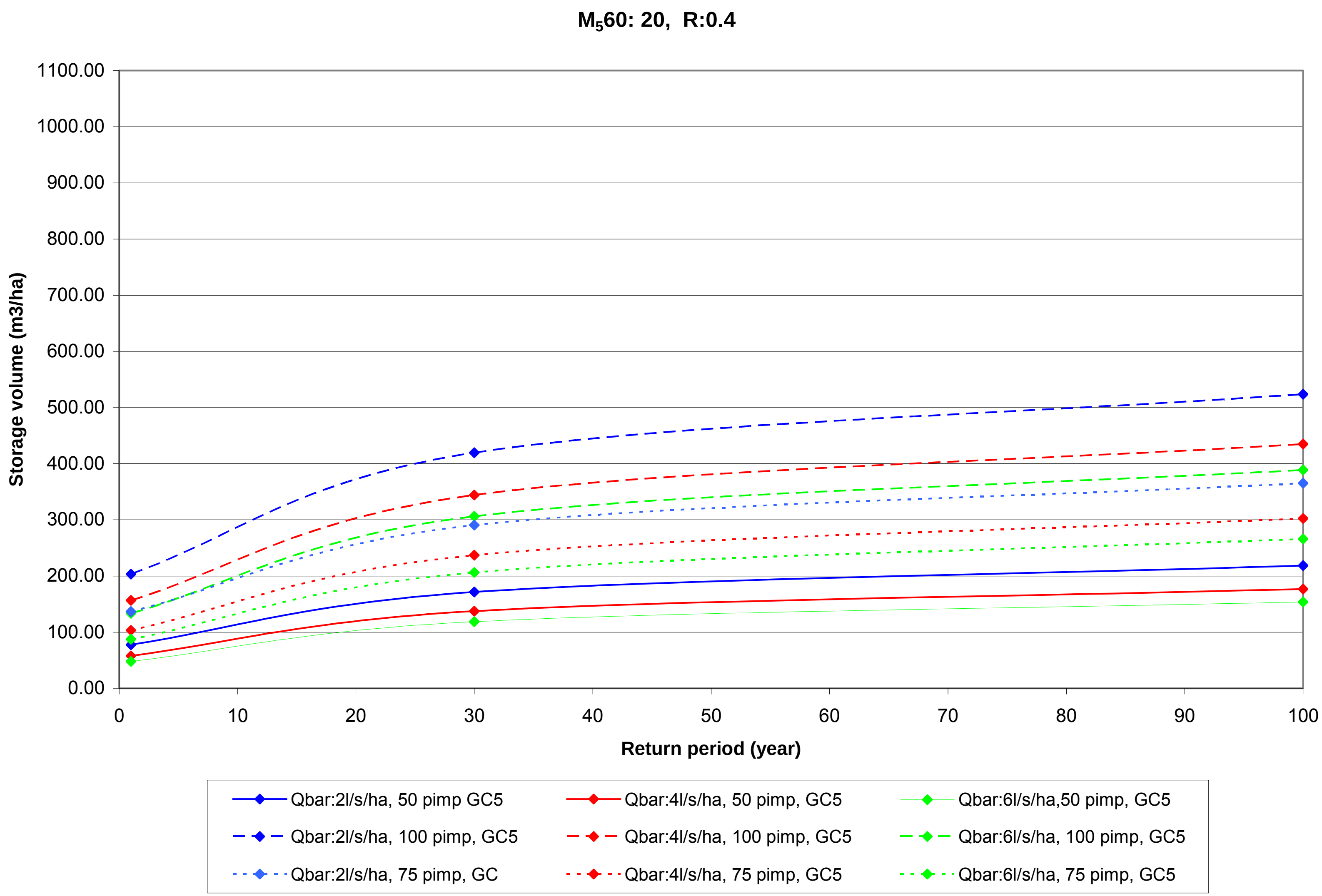


Figure A7.8 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, "r":0.4)

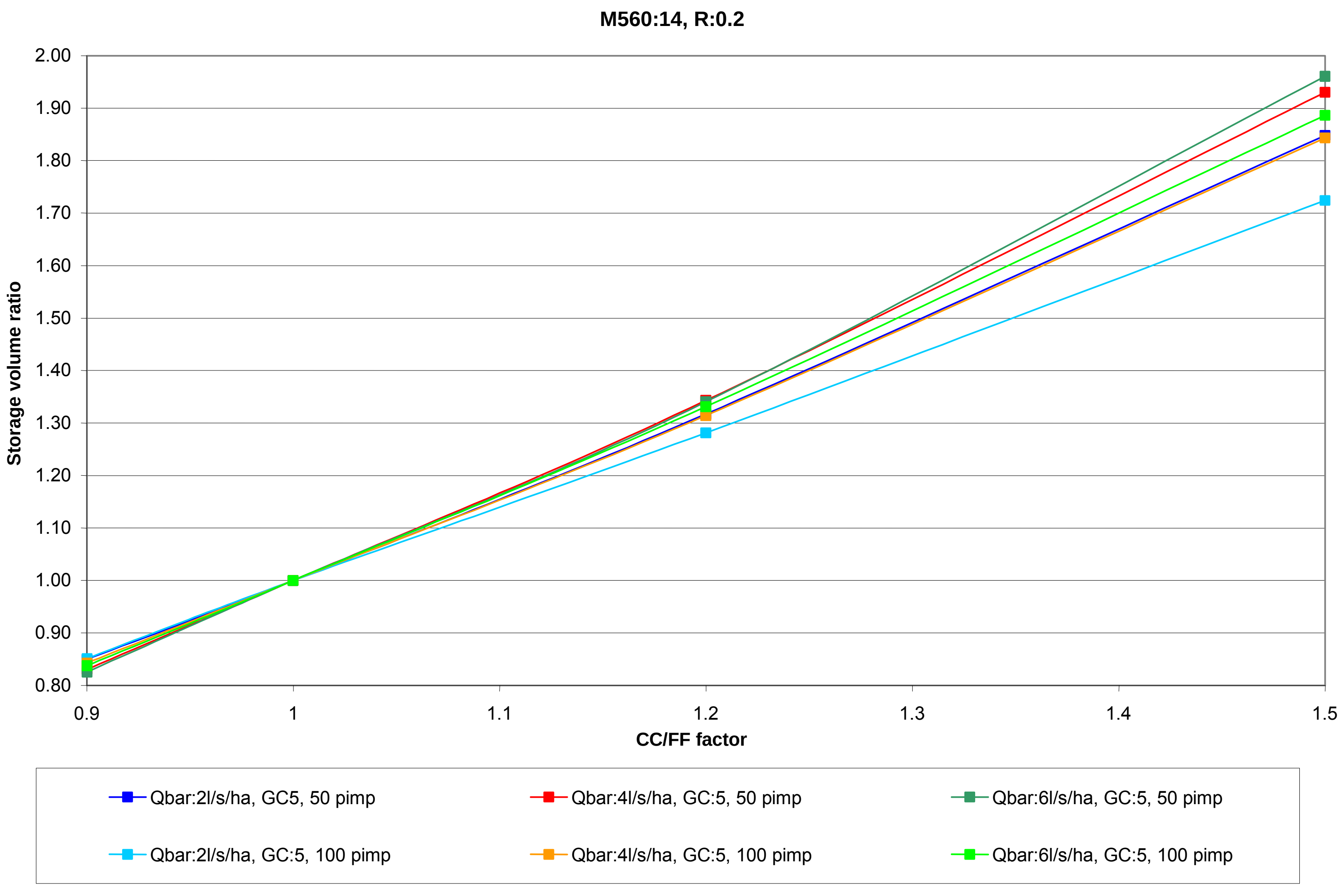


Figure A8.1 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios ($M_5$60:14, “r”:0.2)

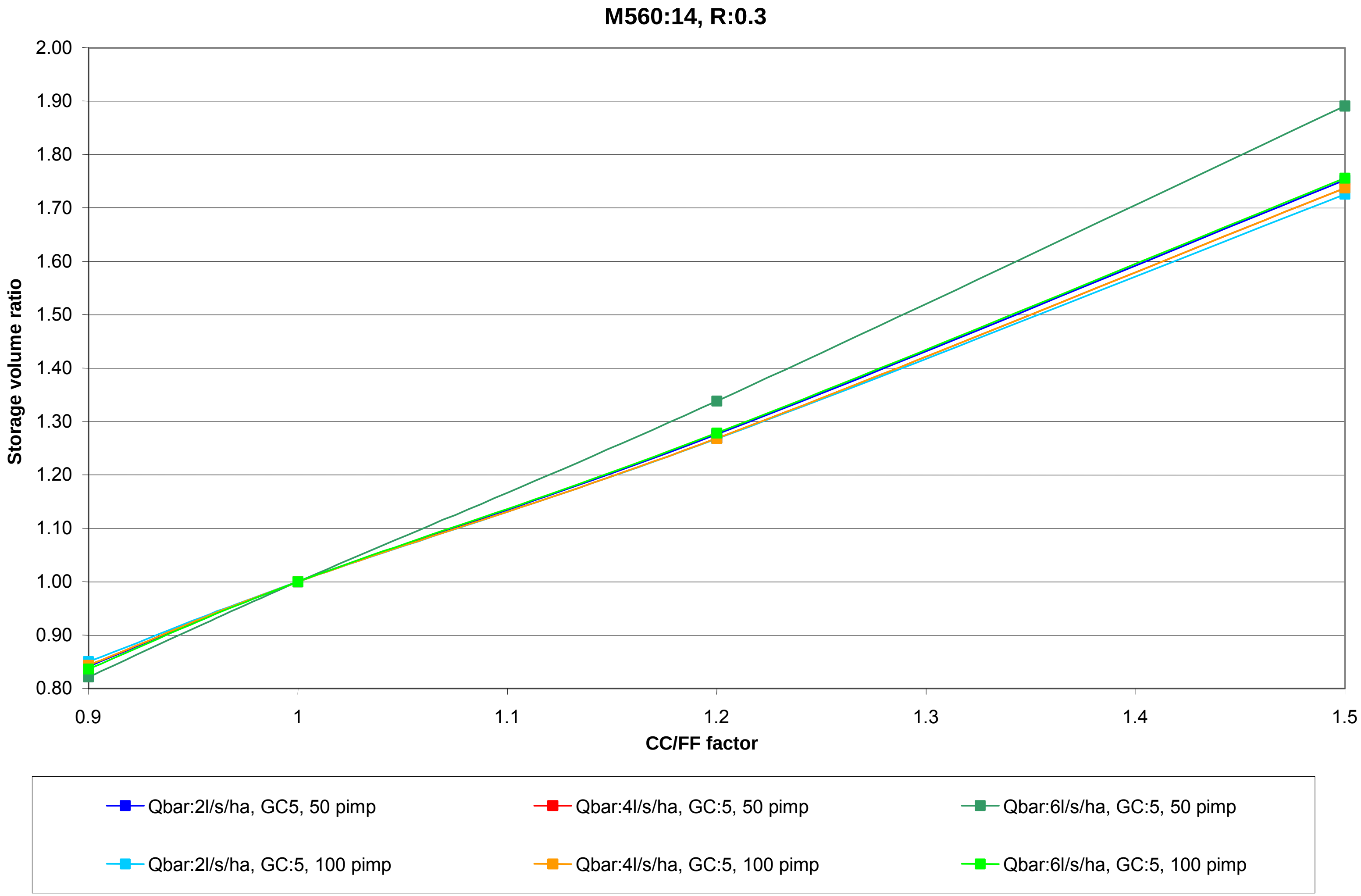


Figure A8.2 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios ($M_5$60:14, "r":0.3)

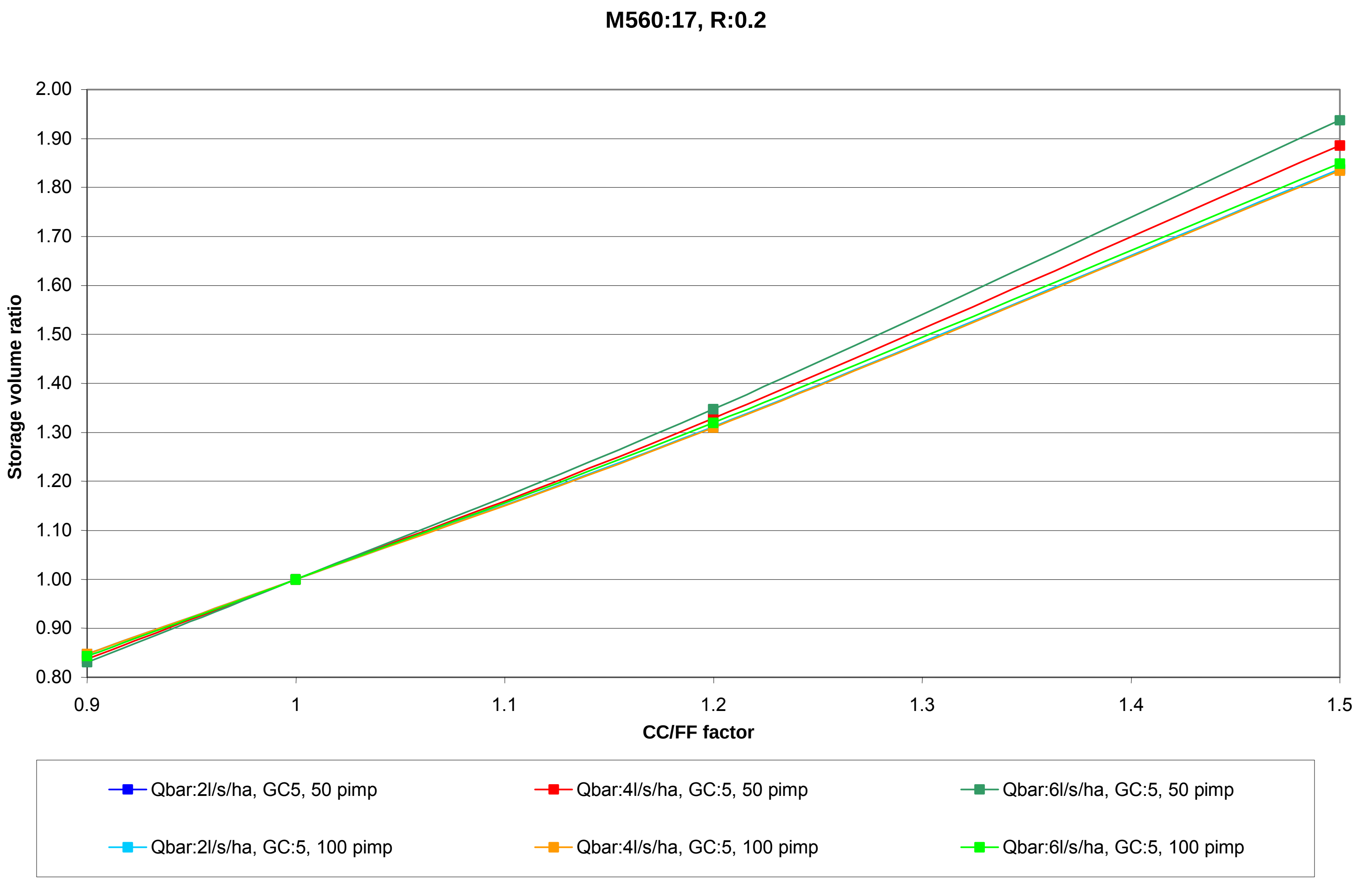


Figure A8.3 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:17, R:0.2)

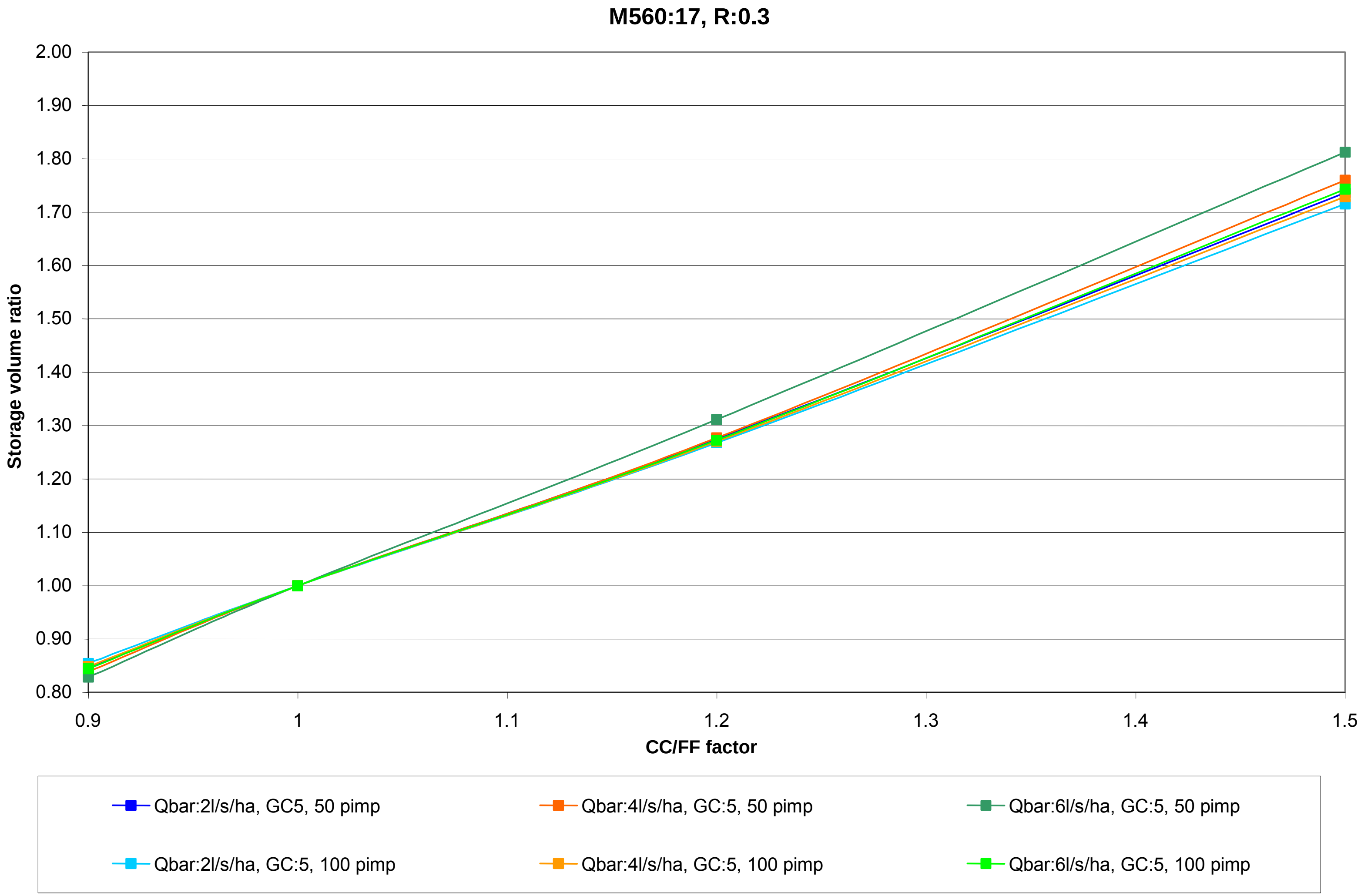


Figure A8.4 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:17, R:0.3)

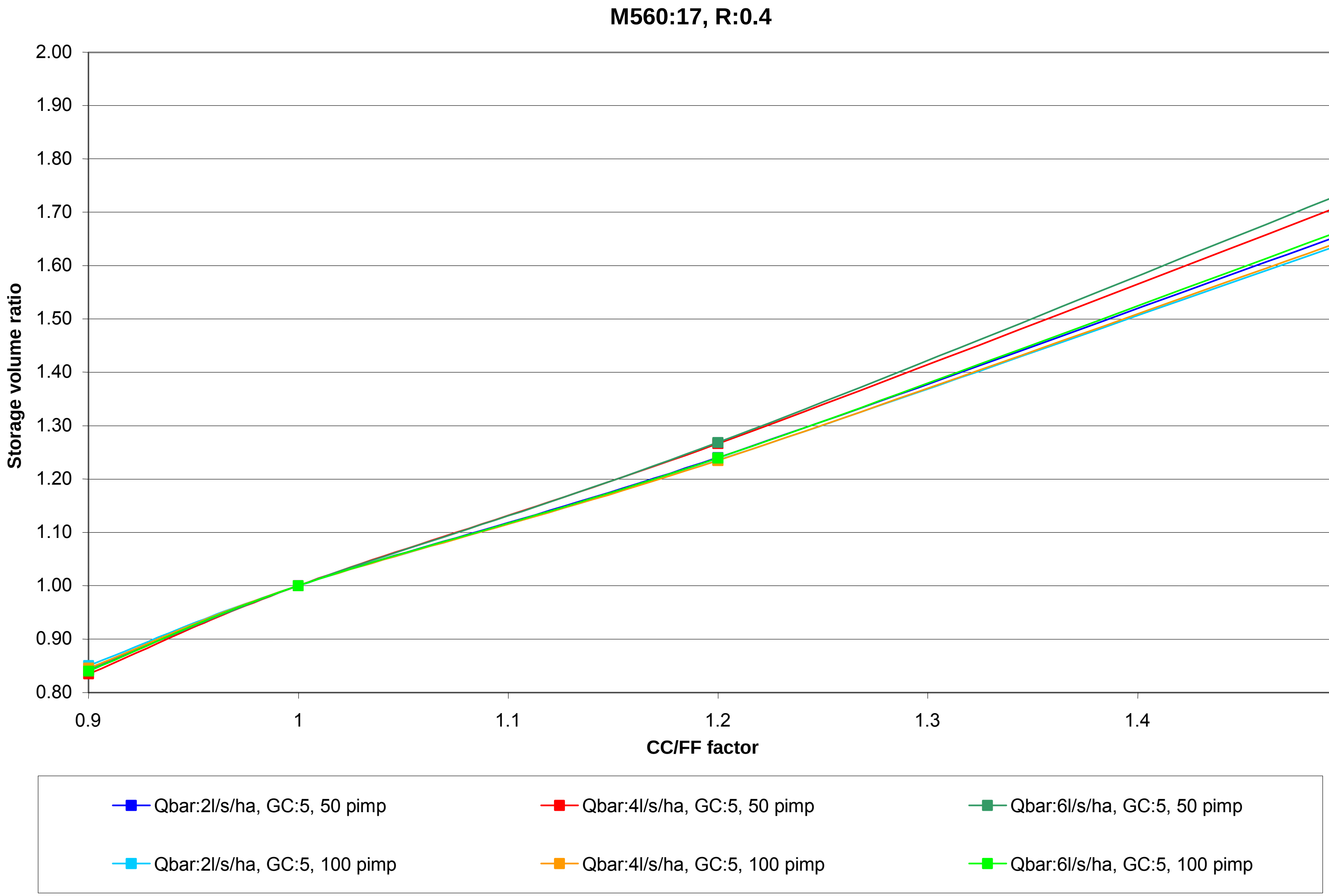


Figure A8.5 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.4)

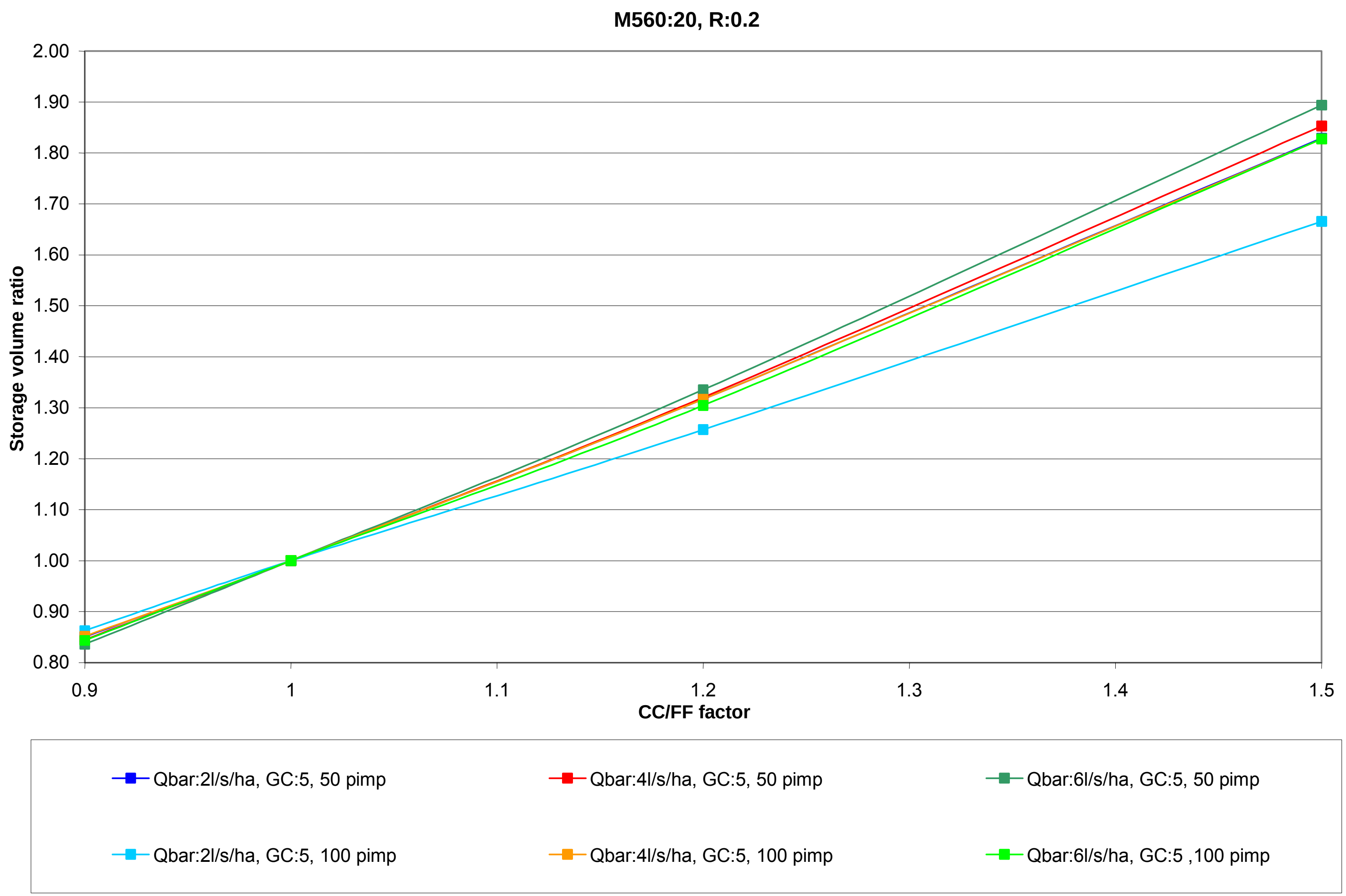


Figure A8.6 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.2)

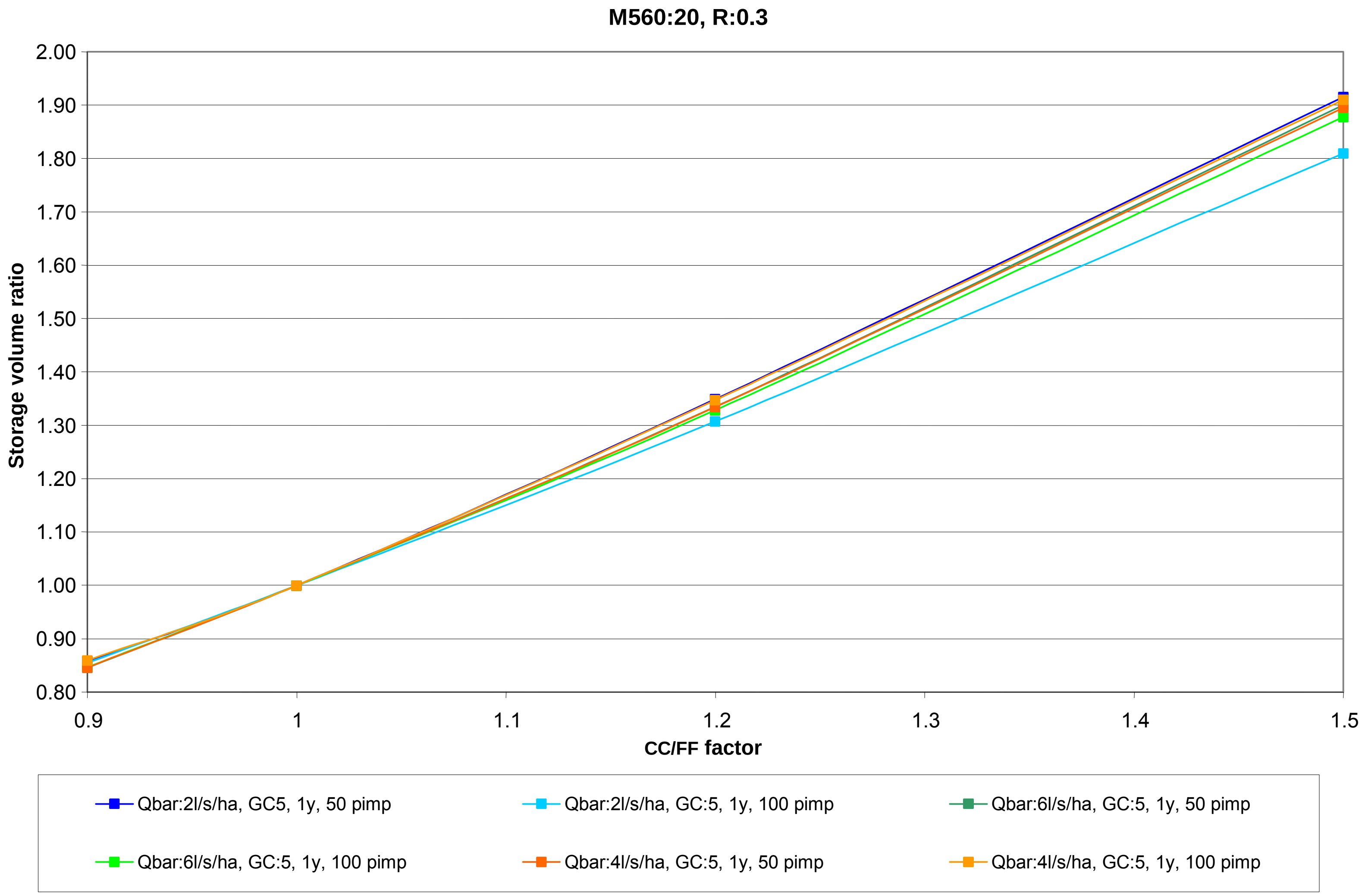


Figure A8.7 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.3)

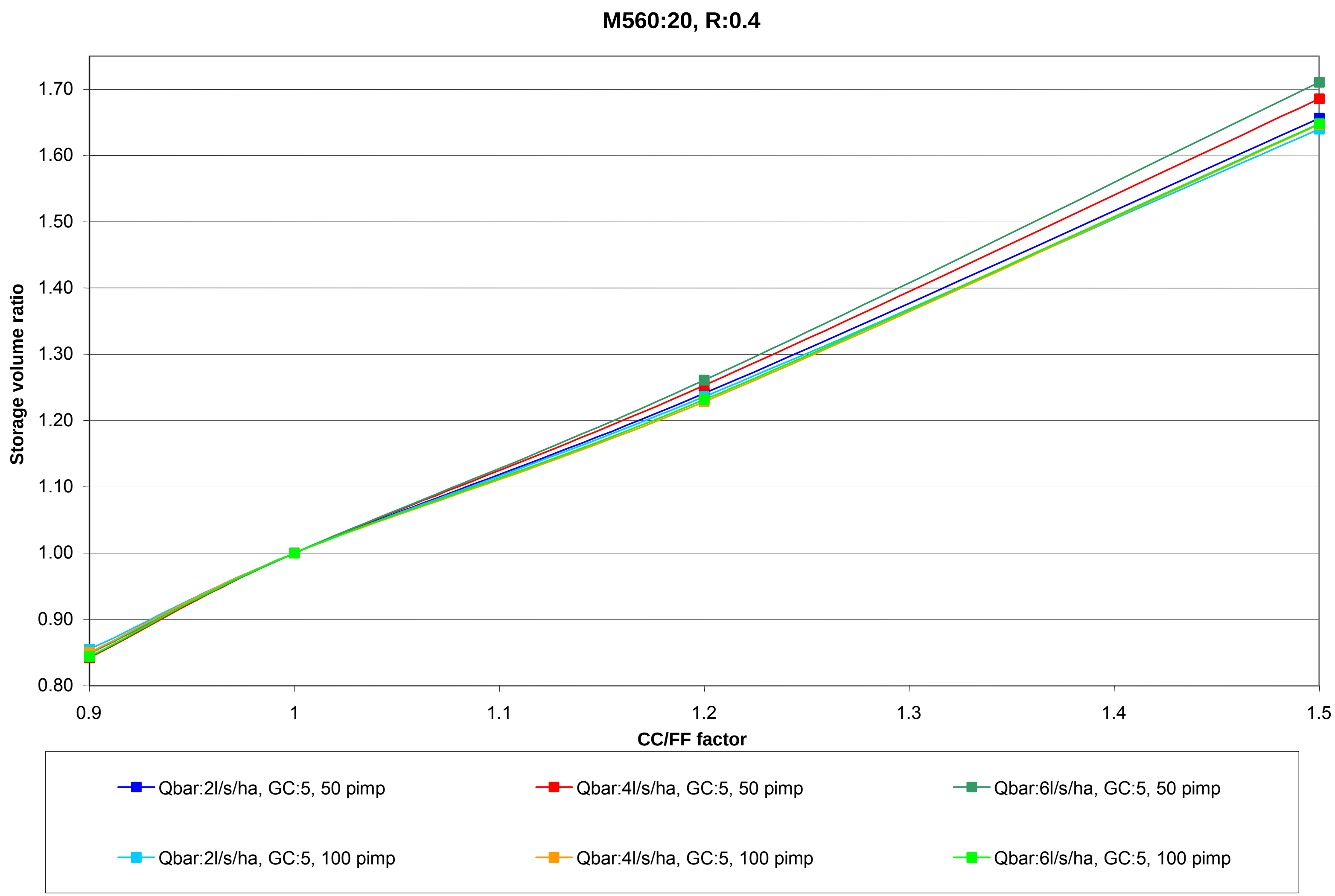


Figure A8.8 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.4)

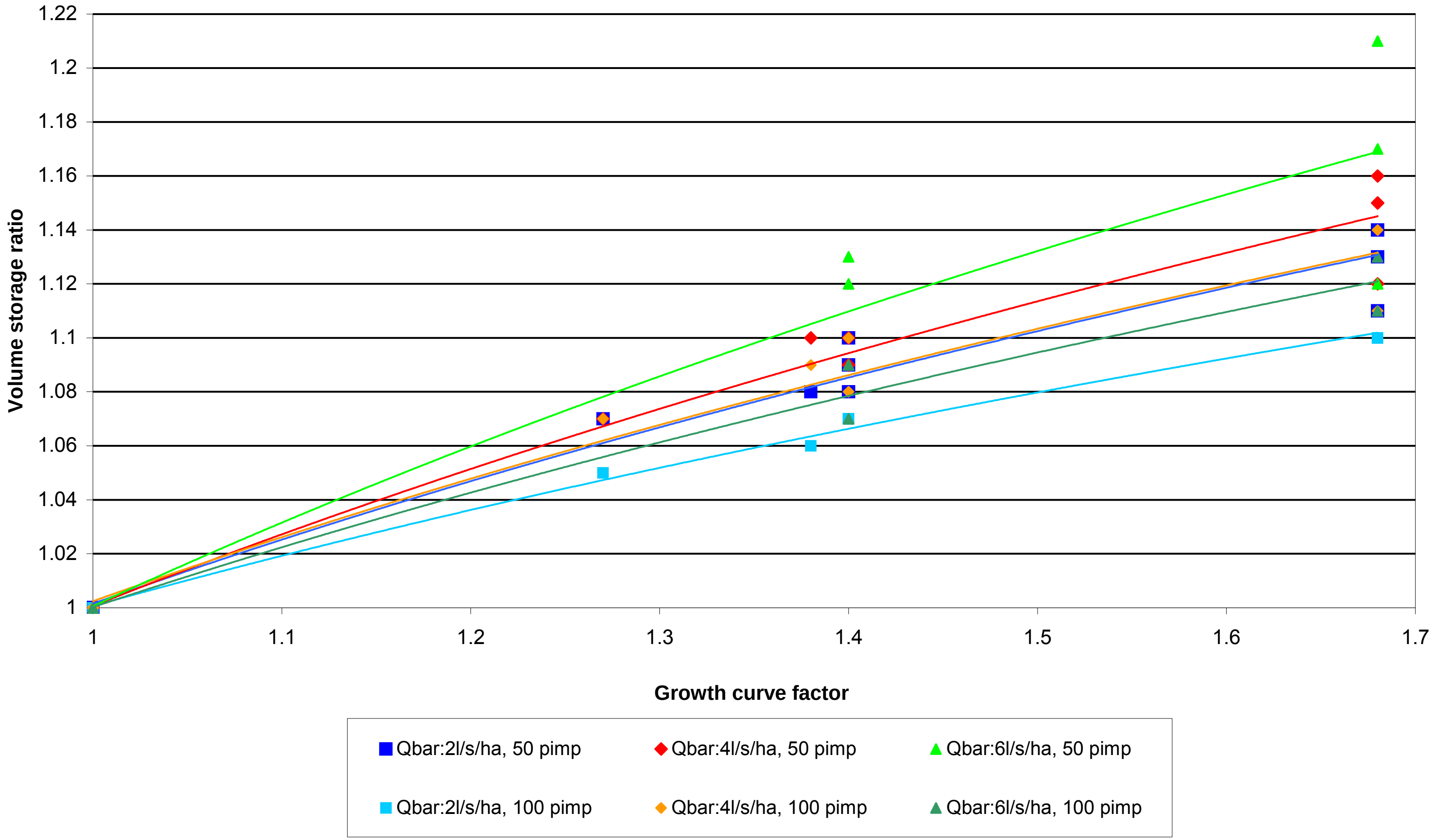


Figure A9.1 Attenuation storage growth curve adjustment factor hydrological regions of UK (all hydrological zones)

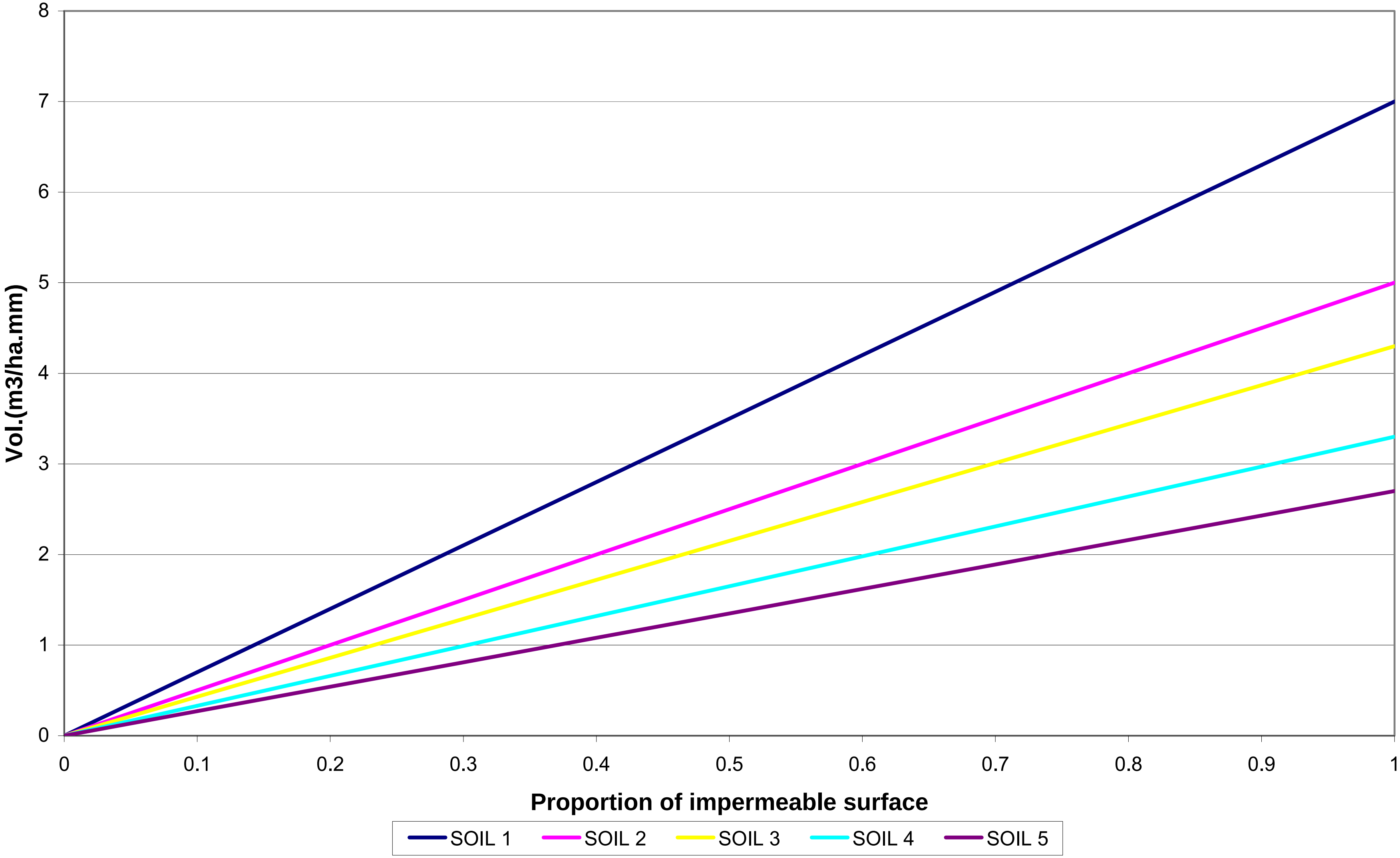


Figure A10.1 Long Term storage volume based on SOIL type

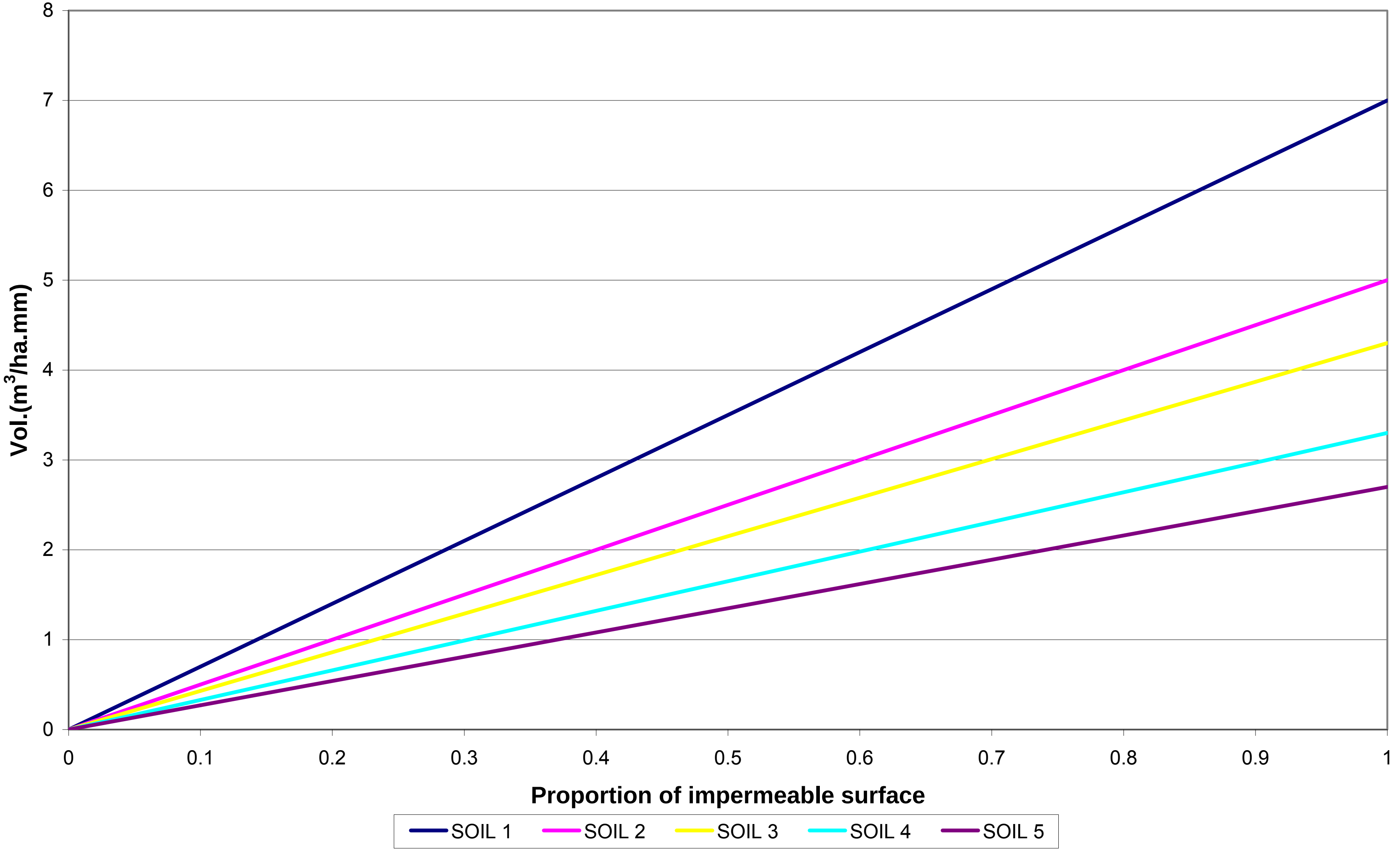


Figure A10.2 Difference in runoff volume for developments where all pervious areas are assumed to drain to the drainage network

FEH factor: 1.1	Critical durations for each hydrological zone (hours)							
Qbar (l/s/ha)	Hydrological Zones							
	M560:14	M560:14	M560:17	M560:17	M560:17	M560:20	M560:20	M560:20
	R:0.2	R:0.3	R:0.2	R:0.3	R:0.4	R:0.2	R:0.3	R:0.4
2 (50pimp/ 100pimp)	28/48	9/26	30/48	13/32	7/12	36/48	16/36	6/13
4 (50pimp/ 100pimp)	11/30	5/9	14/32	5/13	3/7	15/36	6/16	4/6
6 (50pimp/ 100pimp)	5/19	3/7	7/21	4/7	2/4	10/27	3/10	2/4

FEH factor: 1.0	Critical durations for each hydrological zone (hours)							
Qbar (l/s/ha)	Hydrological Zones							
	M560:14	M560:14	M560:17	M560:17	M560:17	M560:20	M560:20	M560:20
	R:0.2	R:0.3	R:0.2	R:0.3	R:0.4	R:0.2	R:0.3	R:0.4
2 (50pimp/ 100pimp)	30/44	10/32	32/48	13/32	6/11	48/48	15/32	6/13
4 (50pimp/ 100pimp)	11/30	5/10	15/32	5/13	4/6	13/40	7/14	4/6
6 (50pimp/ 100pimp)	6/20	4/8	8/26	4/8	2/4	10/27	5/10	2/4

FEH factor: 0.8	Critical durations for each hydrological zone (hours)							
Qbar (l/s/ha)	Hydrological Zones							
	M560:14	M560:14	M560:17	M560:17	M560:17	M560:20	M560:20	M560:20
	R:0.2	R:0.3	R:0.2	R:0.3	R:0.4	R:0.2	R:0.3	R:0.4
2 (50pimp/ 100pimp)	36/48	14/36	40/48	18/40	7/16	48/48	23/44	9/19
4 (50pimp/ 100pimp)	19/36	7/14	20/40	7/18	4/7	19/44	10/23	4/9
6 (50pimp/ 100pimp)	8/27	4/9	9/30	5/11	2/5	14/32	5/13	3/6

FEH factor: 0.65	Critical durations for each hydrological zone (hours)							
Qbar (l/s/ha)	Hydrological Zones							
	M560:14	M560:14	M560:17	M560:17	M560:17	M560:20	M560:20	M560:20
	R:0.2	R:0.3	R:0.2	R:0.3	R:0.4	R:0.2	R:0.3	R:0.4
2 (50pimp/ 100pimp)	44/48	19/44	48/48	24/48	9/20	48/48	30/48	11/20
4 (50pimp/ 100pimp)	25/44	8/9	27/48	10/24	4/9	34/48	13/32	5/10
6 (50pimp/ 100pimp)	13/30	5/11	19/36	5/15	4/6	20/44	7/21	4/7

Note: This set of tables provides assistance in choosing the correct duration map in figures A6.1.1 - A6.3.4

See note 3 in section 3.1 for discussion on the use of Figure A11.1.

Figure A11.1 Table of critical durations as a function of Q_{BAR}/A and PIMP for Attenuation Storage analysis

HOST/SOIL CLASS	SPR Value % (HOST)	SPR Value SOIL *
1	0.020	0.15 (0.10)
2	0.020	0.30 (0.30)
3	0.145	0.40 (0.37)
4	0.020	0.45 (0.47)
5	0.145	0.50 (0.53)
6	0.338	
7	0.443	
8	0.443	
9	0.253	
10	0.253	
11	0.020	
12	0.600	
13	0.020	
14	0.253	
15	0.484	
16	0.292	
17	0.292	
18	0.472	
19	0.600	
20	0.600	
21	0.472	
22	0.600	
23	0.600	
24	0.397	
25	0.496	
26	0.687	
27	0.600	
28	0.600	
29	0.600	

** Values of SPR for SOIL have been used for deriving Figure A10.1. These SPR values are based on the SOIL coefficients used in the Wallingford Procedure runoff model. The value in brackets is the SPR value for SOIL from the Flood Studies Report. The Wallingford Procedure analysis was carried out by the Institute of Hydrology and resulted in modified SPR values to obtain the best correlation for the percentage runoff equation for urban drainage.*

Note: There is no relationship between the HOST index class and the same index for SOIL

Figure A12.1 SPR Values for SOIL and HOST

Appendix 2 Examples

This section has been provided to illustrate the level of accuracy of the method compared with the results that would have been produced using a modelling approach. Five cities around UK have been used which have a range of hydrological characteristics.

As the percentage impervious proportion of the catchment and the limiting discharge rates are important factors, four sets of comparisons are provided. These are as follows:

Site type 1) Q_{BAR} = 6, PIMP = 0.75
Site type 2) Q_{BAR} = 6, PIMP = 0.50
Site type 3) Q_{BAR} = 2, PIMP = 0.75
Site type 4) Q_{BAR} = 2, PIMP = 0.50

The graphs show the Guide method compared with three other results obtained by modelling.

- The main comparison is between the use of the Guide method against the use of FEH rainfall characteristics of the city using the Variable UK Wallingford Procedure runoff model, which is referred to as Method 2.

In addition, comparisons are also provided for two other sets of modelling assumptions.

- The first is using FEH rainfall, but with the same runoff model (Sewers for Adoption) used in the Guide (Method 1).
- The second is the use of the same FSR hydrological characteristics as that of the Guide, but using the Wallingford Procedure variable runoff model (Method 3).

The parameters used for the Wallingford Procedure variable runoff model are as follows:

IF = 0.75
NAPI = 1 (SOIL type 2)
NAPI = 10 (SOIL type 4)

Results

The results comparing Method 1 and the Guide method are fairly similar with the inaccuracy due to the approximation of the rainfall and the errors introduced due to interpolation.

Methods 2 and 3 use a different runoff model, which generally predicts a slight reduction in storage volume required. However the conservative assessment made by the Guide method is considered to be appropriate for the following reasons:

1. If storage is distributed across the site, the cumulative effects may not be as effective as a single point of control

2. The method of modelling at initial design stage makes a non-conservative assumption on the use of limiting discharge by not taking account of the head-discharge relationship.
3. It is better to be conservative in the initial evaluation of cost and space requirements.

One of the five cities (Manchester) was selected as an example to show the values used in determining the Attenuation storage. It should be noted that no reduction in area was made to take account of any area draining to Long Term storage. This example did not include an analysis of Long Term storage or Treatment storage as these are relatively self explanatory.

In addition, climate change was not included, as the main objective was to demonstrate the level of accuracy of the simple tool in this Guide against computer models at these various locations.

Assessment of attenuation storage volumes

1. Hydrological Region	(R)	10		Regions 1 – 10 for runoff growth factor (Appendix 1, Figure A1.1)
2. Hydrological rainfall Zone (M_560, r)	(Z)	20/0.		Zones 1 to 8 based on FSR rainfall characteristics (Appendix 1, Figure A2.1)
3. Development Area	(A)	1.0	ha	Excluding large public open space which is not modified and drained by the development
4. Proportion of impervious area requiring Attenuation storage	(α)	1.0		Impermeable area served by direct drainage / total area of impermeable surface. (see Note 1)
5. Greenfield flow rate per unit area	Q_{BAR}/A	2	l/s/ha	From page ASV 1, use the larger of item 8 or 9.2. See also note 3 on page **LTV**
6. Estimate of catchment percentage impermeable area	(PIMP)	75	%	For catchments where the PIMP value is less than 50% (i.e. where pervious area is the main surface type) a more detailed study should be made as the storage estimates may be undersized.
7. Attenuation storage volumes per unit area	($Uvol_{1yr}$)	198	m^3/ha	Interpolate values based on PIMP and Q_{BAR}/A (Appendix 1, Figures A7.1 – A7.8)
	($Uvol_{30yr}$)	369	m^3/ha	Use characteristics from item 2 (M_560, r).
	($Uvol_{100yr}$)	451	m^3/ha	
8. Basic storage volumes (U.Vol . α A)	(BSV_{1yr})	198	m^3	Storage units may serve areas of different densities of development. Calculations should be based on each development zone then cumulated.
	(BSV_{30yr})	369	m^3	
	(BSV_{100yr})	451	m^3	
9. Climate Change factor	(CC)	1.0		Suggested factor for climate change is 1.2 (see note 2)
10. FEH Rainfall factor	(FF_{1yr})	1.0		Use critical duration based on Q_{BAR}/A and PIMP (Appendix 1, Figure A11.1 and Appendix 1, Figures A6.1.1 – A6.3.4).
	(FF_{30yr})	1.0		
	(FF_{100yr})	0.9		(See note 3)

11. Storage Volume ratio (CC/ FF)	(SVR_{1yr})	1.0		Calculate item 9 / item 10 then use Appendix 1, Figures A8.1 – A8.8 to obtain storage ratios
	(SVR_{30yr})	1.0		
	(SVR_{100yr})	1.14		
12. Adjusted Storage Volumes	(ASV_{1yr})	198	m^3	Storage volumes adjusted for Climate Change and FEH rainfall
(SVR x BSV)	(ASV_{30yr})	369	m^3	
	(ASV_{100yr})	514	m^3	
13. Hydrological Region volume storage ratio	(HR_{1yr})	1.0		Adjustment of storage volumes for hydrological region using Volume Storage Ratio (Appendix 1, Figure A9.1). The values are based on growth curve factors - the ratio of growth curve factor of the region of site with hydrological region 5 (Table inset in Appendix 1, Figure A1.3)
	(HR_{30yr})	1.08		
	(HR_{100yr})	1.12		

14. Final estimated Attenuation Storage

Attenuation Storage Volumes	At. Vol_{1yr}	198	m^3	Required Attenuation Storage
	At. Vol_{30yr}	398	m^3	
(HR x ASV)	At. Vol_{100yr}	578	m^3	

Table A2.1.1 5 example sites (site type 1)- parameters and results of the User Guide method

User Guide Method						
catchment area	M5-60	R	RP (year)	FSR/FEH Ratio	Critical duration (min)	Cumulative Volume (m3/ha)
Aberdeen	14	0.2	1	1.1	720	76.91
Aberdeen	14	0.2	30	1	720	204.35
Aberdeen	14	0.2	100	1	720	270.02
Manchester	20	0.3	1	1	360	107.14
Manchester	20	0.3	30	1	360	259.85
Manchester	20	0.3	100	0.9	360	486.14
Newcastle	17	0.3	1	1	360	80.68
Newcastle	17	0.3	30	1.1	360	178.17
Newcastle	17	0.3	100	1	360	283.30
Shrewsbury	17	0.4	1	1.1	180	64.68
Shrewsbury	17	0.4	30	1.1	180	151.41
Shrewsbury	17	0.4	100	1	180	235.76
East London	20	0.4	1	1.1	180	74.97
East London	20	0.4	30	1	180	216.69
East London	20	0.4	100	0.9	180	314.34

For all models:

- PIMP= 75 %
- Q_{BAR} = 6 l/s/ha.

Note: FSR/FEH ratio (from the maps of Figures A6.1.1 – A6.3.4) is the inverse of the factor applied to allow for FEH rainfall in this procedure.

Table A2.1.2 5 example sites (site type 1)- parameters and results of Method 1

Check Method 1- SfA runoff model, FEH rainfall				
catchment area	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 1
Aberdeen	1	900	80.88	0.95
Aberdeen	30	480	158.59	1.29
Aberdeen	100	480	201.52	1.34
Manchester	1	1080	96.29	1.11
Manchester	30	360	242.62	1.07
Manchester	100	240	334.07	1.46
Newcastle	1	720	83.76	0.96
Newcastle	30	360	188.82	0.94
Newcastle	100	240	250.12	1.13
Shrewsbury	1	480	69.51	0.93
Shrewsbury	30	120	178.39	0.85
Shrewsbury	100	120	245.60	0.96
East London	1	120	80.64	0.93
East London	30	120	238.07	0.91
East London	100	120	338.87	0.93

For all models:

- PIMP= 75 %
- Q_{BAR} = 6 l/s/ha.

Table A2.1.3 5 example sites (site type 1)- parameters and results of Method 2

Check Method 2- W.P. New PR runoff model, FEH rainfall						
catchment area	Soil Type	NAPI	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 2
Aberdeen	2	1	1	720	52.30	1.47
Aberdeen	2	1	30	480	107.09	1.91
Aberdeen	2	1	100	480	138.20	1.95
Manchester	4	10	1	900	67.46	1.59
Manchester	4	10	30	240	188.65	1.38
Manchester	4	10	100	240	271.96	1.79
Newcastle	4	10	1	600	58.86	1.37
Newcastle	4	10	30	240	140.90	1.26
Newcastle	4	10	100	240	194.58	1.46
Shrewsbury	4	10	1	240	49.35	1.31
Shrewsbury	4	10	30	120	135.12	1.12
Shrewsbury	4	10	100	120	194.26	1.21
East London	4	10	1	120	58.58	1.28
East London	4	10	30	120	188.38	1.15
East London	4	10	100	120	278.63	1.13

For all models:

- PIMP= 75 %
- Q_{BAR} = 6 l/s/ha.

Table A2.1.4 5 example sites (site type 1)- parameters and results of Method 3

Check Method 3- W.P. New PR runoff model, FSR Hydrological Zones								
catchment area	Soil Type	NAPI	M5-60	R	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 3
Aberdeen	2	1	14	0.2	1	900	65.57	1.17
Aberdeen	2	1	14	0.2	30	480	126.09	1.62
Aberdeen	2	1	14	0.2	100	360	160.68	1.68
Manchester	4	10	20	0.3	1	480	77.95	1.37
Manchester	4	10	20	0.3	30	240	183.24	1.42
Manchester	4	10	20	0.3	100	240	240.99	2.02
Newcastle	4	10	17	0.3	1	360	57.30	1.41
Newcastle	4	10	17	0.3	30	240	144.28	1.23
Newcastle	4	10	17	0.3	100	240	192.39	1.47
Shrewsbury	4	10	17	0.4	1	120	47.32	1.37
Shrewsbury	4	10	17	0.4	30	120	124.53	1.22
Shrewsbury	4	10	17	0.4	100	120	183.88	1.28
East London	4	10	20	0.4	1	240	63.08	1.19
East London	4	10	20	0.4	30	240	157.80	1.37
East London	4	10	20	0.4	100	240	209.64	1.50

For all models:

- PIMP= 75 %
- Q_{BAR} = 6 l/s/ha.

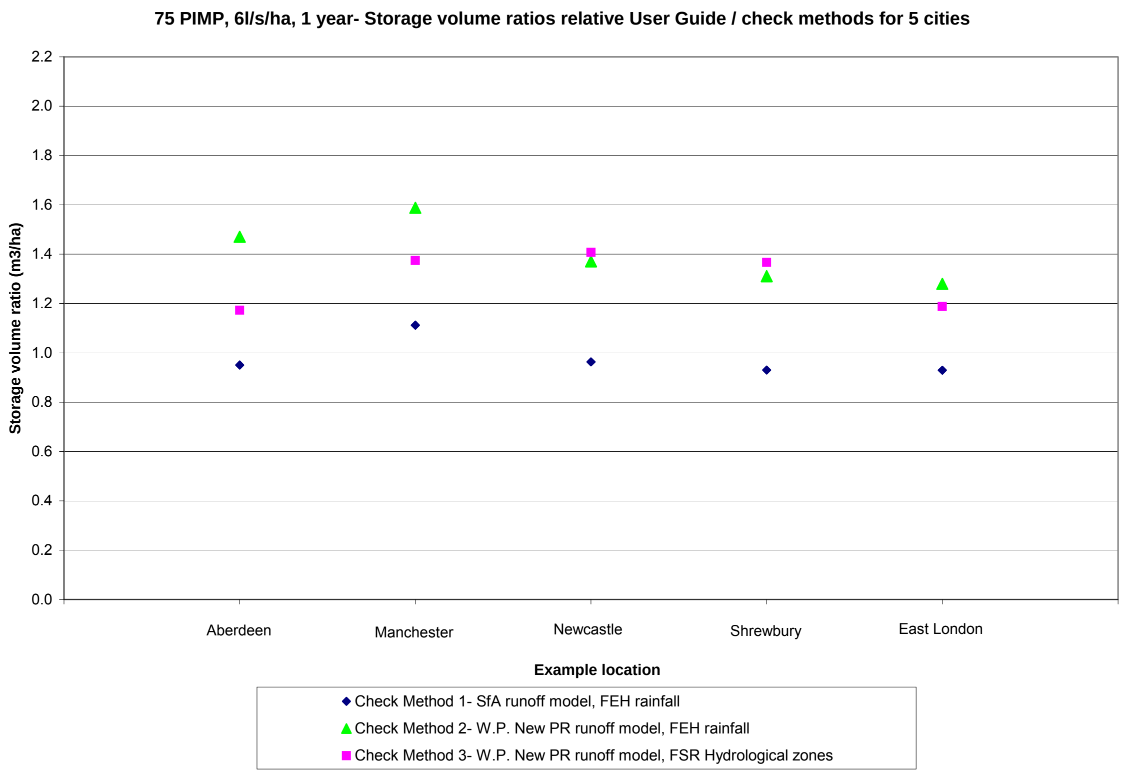


Figure A2.1.1 Check comparison of Attenuation Storage Volume for site type 1 – 1 year

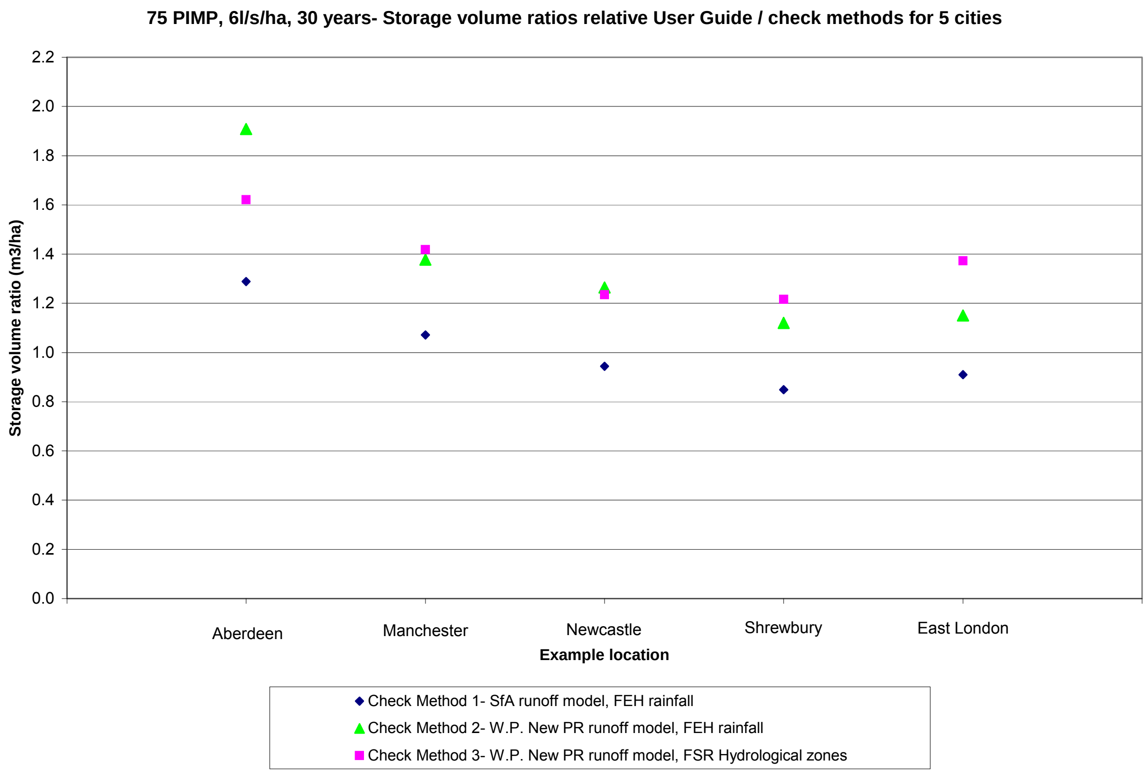


Figure A2.1.2 Check comparison of Attenuation Storage Volume for site type 1 – 30 years

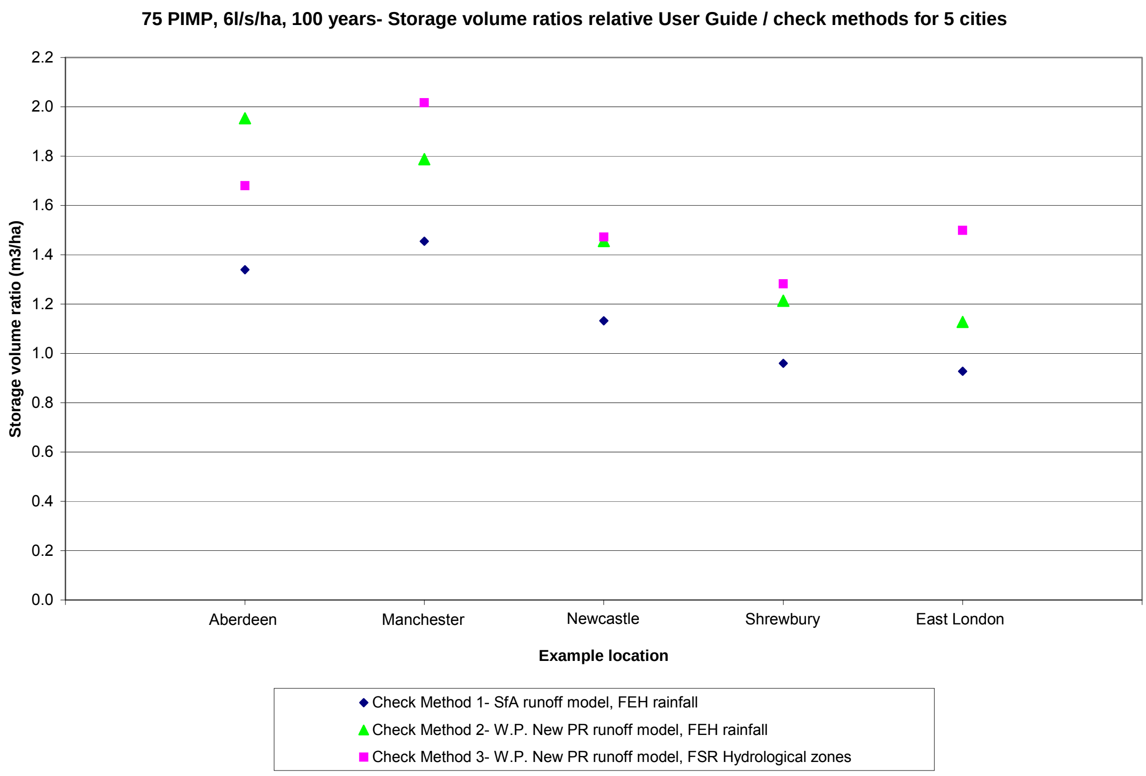


Figure A2.1.3 Check comparison of Attenuation Storage Volume for site type 1 – 100 years

Table A2.2.1 5 example sites (site type 2)- parameters and results of the User Guide method

User Guide Method						
catchment area	**M5-60**	**R**	**RP (year)**	**FSR/FEH Ratio**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**
Aberdeen	14	0.2	1	0.9	360	35.09
Aberdeen	14	0.2	30	0.9	360	87.10
Aberdeen	14	0.2	100	0.9	360	115.58
Manchester	20	0.3	1	0.9	180	48.57
Manchester	20	0.3	30	1	180	145.53
Manchester	20	0.3	100	1.1	180	227.74
Newcastle	17	0.3	1	0.9	180	35.39
Newcastle	17	0.3	30	0.9	180	96.95
Newcastle	17	0.3	100	0.9	180	133.41
Shrewsbury	17	0.4	1	0.9	120	31.03
Shrewsbury	17	0.4	30	0.9	120	86.87
Shrewsbury	17	0.4	100	0.9	120	116.33
East London	20	0.4	1	0.8	120	35.45
East London	20	0.4	30	1	120	120.49
East London	20	0.4	100	1.1	120	184.45

For all models:

- PIMP= 50 %
- Q_{BAR} = 6 l/s/ha.

Table A2.2.2 5 example sites (site type 2)- parameters and results of Method 1

Check Method 1- SfA runoff model, FEH rainfall				
catchment area	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 1**
Aberdeen	1	480	39.92	0.88
Aberdeen	30	240	76.48	1.14
Aberdeen	100	240	98.25	1.18
Manchester	1	360	49.70	0.98
Manchester	30	240	134.26	1.08
Manchester	100	120	188.83	1.21
Newcastle	1	480	43.20	0.82
Newcastle	30	120	100.65	0.96
Newcastle	100	120	137.28	0.97
Shrewsbury	1	120	37.99	0.82
Shrewsbury	30	120	102.72	0.85
Shrewsbury	100	120	142.88	0.81
East London	1	120	45.79	0.77
East London	30	120	137.78	0.87
East London	100	60	201.10	0.92

For all models:

- PIMP= 50 %
- Q_{BAR} = 6 l/s/ha.

Table A2.2.3 5 example sites (site type 2)- parameters and results of Method 2

Check Method 2- W.P. New PR runoff model, FEH rainfall						
catchment area	**Soil Type**	**NAPI**	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 2**
Aberdeen	2	1	1	480	25.38	1.38
Aberdeen	2	1	30	240	53.45	1.63
Aberdeen	2	1	100	240	70.49	1.64
Manchester	4	10	1	360	38.22	1.27
Manchester	4	10	30	240	118.56	1.23
Manchester	4	10	100	240	179.91	1.27
Newcastle	4	10	1	480	32.22	1.10
Newcastle	4	10	30	240	83.54	1.16
Newcastle	4	10	100	240	119.44	1.12
Shrewsbury	4	10	1	120	28.26	1.10
Shrewsbury	4	10	30	120	84.43	1.03
Shrewsbury	4	10	100	120	123.82	0.94
East London	4	10	1	120	34.78	1.02
East London	4	10	30	120	122.06	0.99
East London	4	10	100	120	189.76	0.97

For all models:

- PIMP= 50 %
- Q_{BAR} = 6 l/s/ha.

Table A2.2.4 5 example sites (site type 2)- parameters and results of Method 3

Check Method 3- W.P. New PR runoff model, FSR Hydrological Zones								
catchment area	**Soil Type**	**NAPI**	**M5-60**	**R**	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 3**
Aberdeen	2	1	14	0.2	1	60	32.51	1.08
Aberdeen	2	1	14	0.2	30	360	66.97	1.30
Aberdeen	2	1	14	0.2	100	240	87.48	1.32
Manchester	4	10	20	0.3	1	240	45.83	1.06
Manchester	4	10	20	0.3	30	360	115.16	1.26
Manchester	4	10	20	0.3	100	240	155.99	1.46
Newcastle	4	10	17	0.3	1	240	32.64	1.08
Newcastle	4	10	17	0.3	30	240	86.32	1.12
Newcastle	4	10	17	0.3	100	240	118.14	1.13
Shrewsbury	4	10	17	0.4	1	120	27.53	1.13
Shrewsbury	4	10	17	0.4	30	120	76.43	1.14
Shrewsbury	4	10	17	0.4	100	120	103.03	1.13
East London	4	10	20	0.4	1	120	37.49	0.95
East London	4	10	20	0.4	30	120	100.58	1.20
East London	4	10	20	0.4	100	120	135.81	1.36

For all models:

- PIMP= 50 %
- Q_{BAR} = 6 l/s/ha.

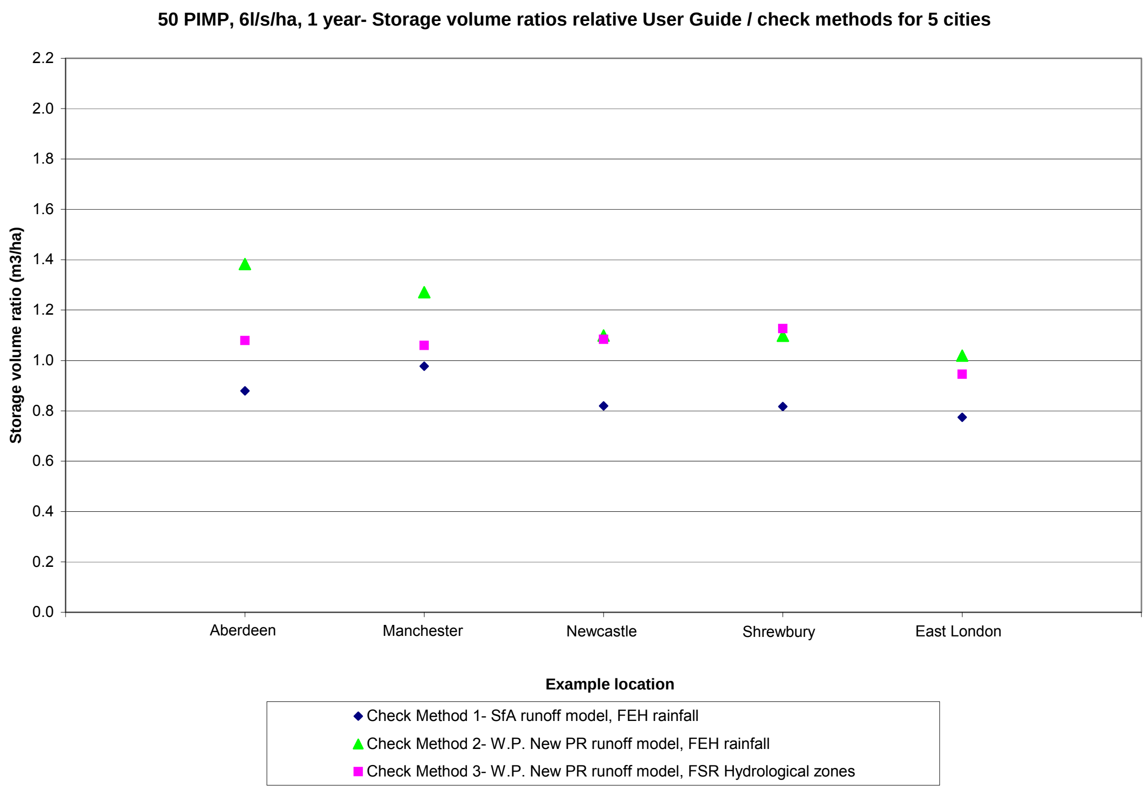


Figure A2.2.1 Check comparison of Attenuation Storage Volume for site type 2 – 1 year

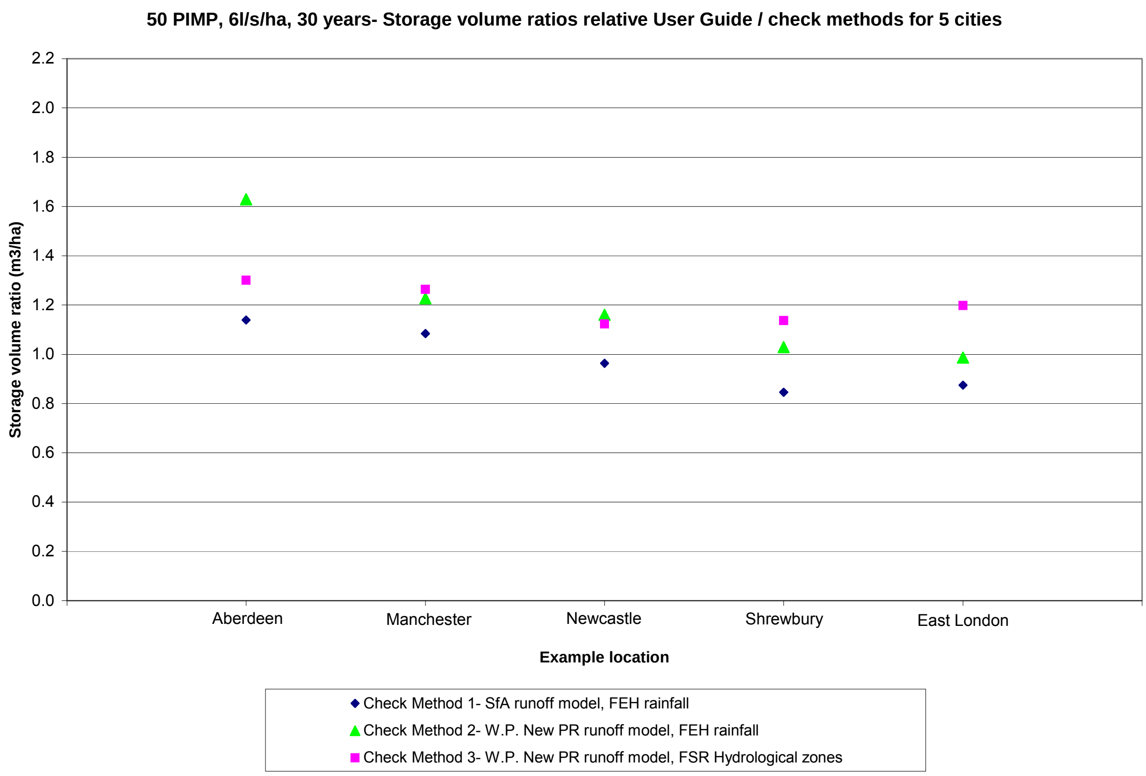


Figure A2.2.2 Check comparison of Attenuation Storage Volume for site type 2 – 30 years

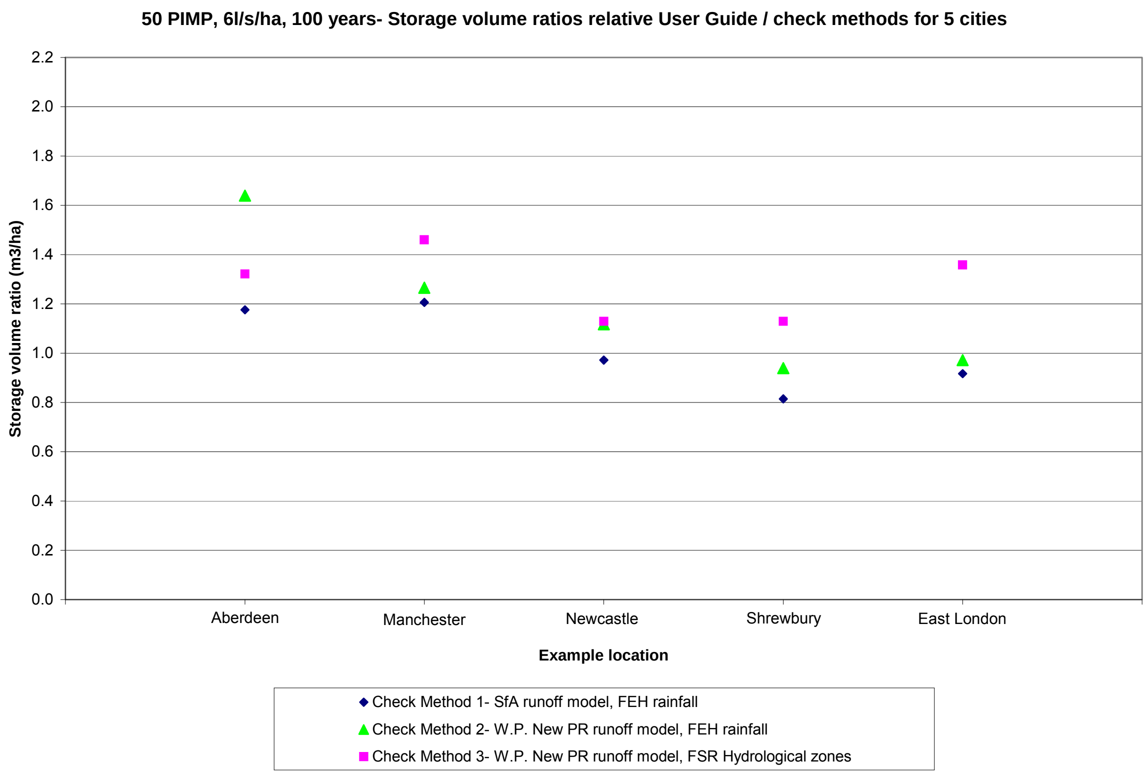


Figure A2.2.3 Check comparison of Attenuation Storage Volume for site type 2 – 100 years

Table A2.3.1 5 example sites (site type 3)- parameters and results of the User Guide method

User Guide Method						
catchment area	**M5-60**	**R**	**RP (year)**	**FSR/FEH Ratio**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**
Aberdeen	14	0.2	1	0.9	720	185.88
Aberdeen	14	0.2	30	1	720	392.28
Aberdeen	14	0.2	100	1	720	482.54
Manchester	20	0.3	1	1	720	198.54
Manchester	20	0.3	30	1	720	397.63
Manchester	20	0.3	100	1.1	720	576.41
Newcastle	17	0.3	1	1	720	146.64
Newcastle	17	0.3	30	1	720	325.29
Newcastle	17	0.3	100	1	720	420.77
Shrewsbury	17	0.4	1	1	720	106.53
Shrewsbury	17	0.4	30	1	720	253.12
Shrewsbury	17	0.4	100	1.1	720	328.63
East London	20	0.4	1	0.9	720	117.41
East London	20	0.4	30	1.1	720	330.73
East London	20	0.4	100	1.1	720	425.87

For all models:

- PIMP= 75 %
- Q_{BAR} = 2 l/s/ha.

Table A2.3.2 5 example sites (site type 3)- parameters and results of Method 1

Check Method 1- SfA runoff model, FEH rainfall				
catchment area	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 1**
Aberdeen	1	2880	182.55	1.02
Aberdeen	30	2880	344.15	1.14
Aberdeen	100	2880	426.75	1.13
Manchester	1	2880	212.63	0.93
Manchester	30	2880	431.61	0.92
Manchester	100	2160	547.54	1.05
Newcastle	1	2880	171.25	0.86
Newcastle	30	1440	327.17	0.99
Newcastle	100	1440	412.06	1.02
Shrewsbury	1	2880	133.66	0.80
Shrewsbury	30	1080	269.03	0.94
Shrewsbury	100	900	344.04	0.96
East London	1	600	128.93	0.91
East London	30	480	325.42	1.02
East London	100	480	441.19	0.97

For all models:

- PIMP= 75 %
- Q_{BAR} = 2 l/s/ha.

Table A2.3.3 5 example sites (site type 3)- parameters and results of Method 2

Check Method 2- W.P. New PR runoff model, FEH rainfall						
catchment area	Soil Type	NAPI	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 2
Aberdeen	2	1	1	2880	130.86	1.42
Aberdeen	2	1	30	2880	262.86	1.49
Aberdeen	2	1	100	2160	337.58	1.43
Manchester	4	10	1	2880	166.97	1.19
Manchester	4	10	30	2880	372.69	1.07
Manchester	4	10	100	2880	500.08	1.15
Newcastle	4	10	1	2880	130.62	1.12
Newcastle	4	10	30	1440	266.91	1.22
Newcastle	4	10	100	1440	350.22	1.20
Shrewsbury	4	10	1	2160	98.64	1.08
Shrewsbury	4	10	30	1080	213.40	1.19
Shrewsbury	4	10	100	900	283.71	1.16
East London	4	10	1	480	96.04	1.22
East London	4	10	30	480	267.17	1.24
East London	4	10	100	480	412.72	1.03

For all models:

- PIMP= 75 %
- Q_{BAR} = 2 l/s/ha.

Table A2.3.4 5 example sites (site type 3)- parameters and results of Method 3

Check Method 3- W.P. New PR runoff model, FSR Hydrological Zones								
catchment area	Soil Type	NAPI	M5-60	R	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 3
Aberdeen	2	1	14	0.2	1	2880	158.98	1.17
Aberdeen	2	1	14	0.2	30	2880	282.62	1.39
Aberdeen	2	1	14	0.2	100	2160	343.76	1.40
Manchester	4	10	20	0.3	1	2880	154.48	1.29
Manchester	4	10	20	0.3	30	1440	308.20	1.29
Manchester	4	10	20	0.3	100	1080	390.24	1.48
Newcastle	4	10	17	0.3	1	2160	111.73	1.31
Newcastle	4	10	17	0.3	30	1080	244.62	1.33
Newcastle	4	10	17	0.3	100	1080	315.89	1.33
Shrewsbury	4	10	17	0.4	1	480	78.22	1.36
Shrewsbury	4	10	17	0.4	30	360	190.33	1.33
Shrewsbury	4	10	17	0.4	100	360	250.14	1.31
East London	4	10	20	0.4	1	480	102.46	1.15
East London	4	10	20	0.4	30	480	235.74	1.40
East London	4	10	20	0.4	100	480	305.72	1.39

For all models:

- PIMP= 75 %
- Q_{BAR} = 2 l/s/ha.

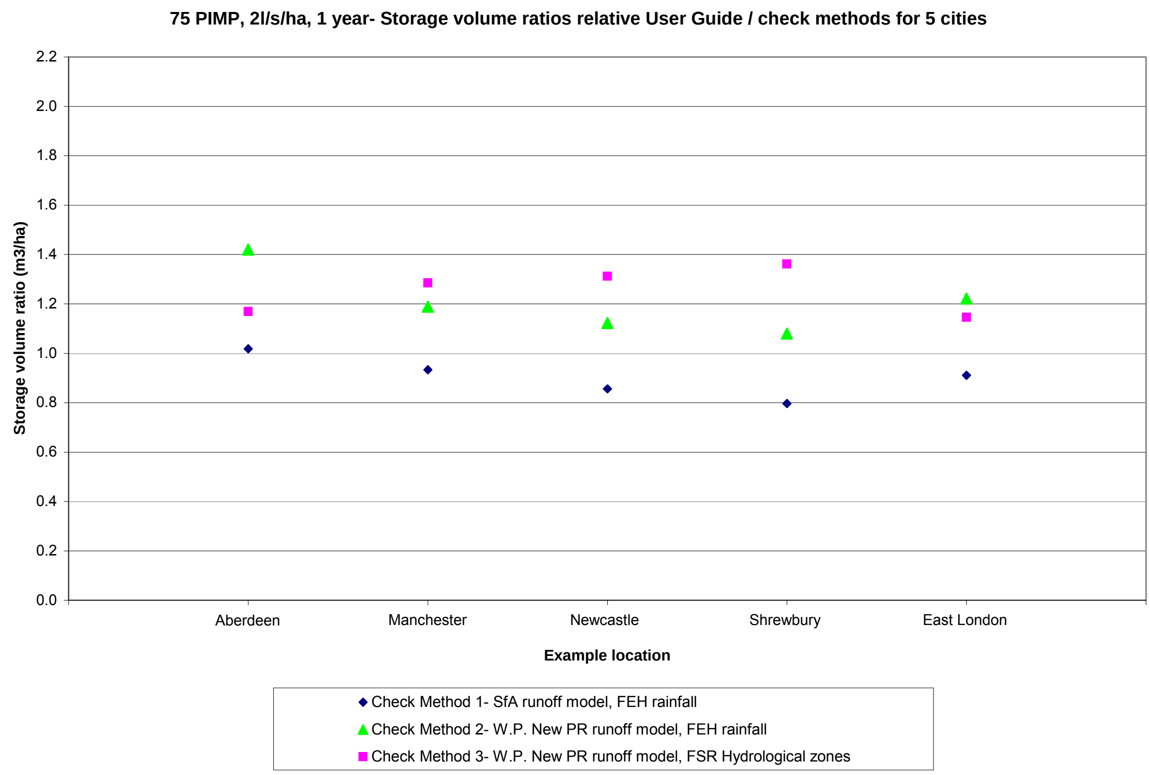


Figure A2.3.1 Check comparison of Attenuation Storage Volume for site type 3 – 1 year

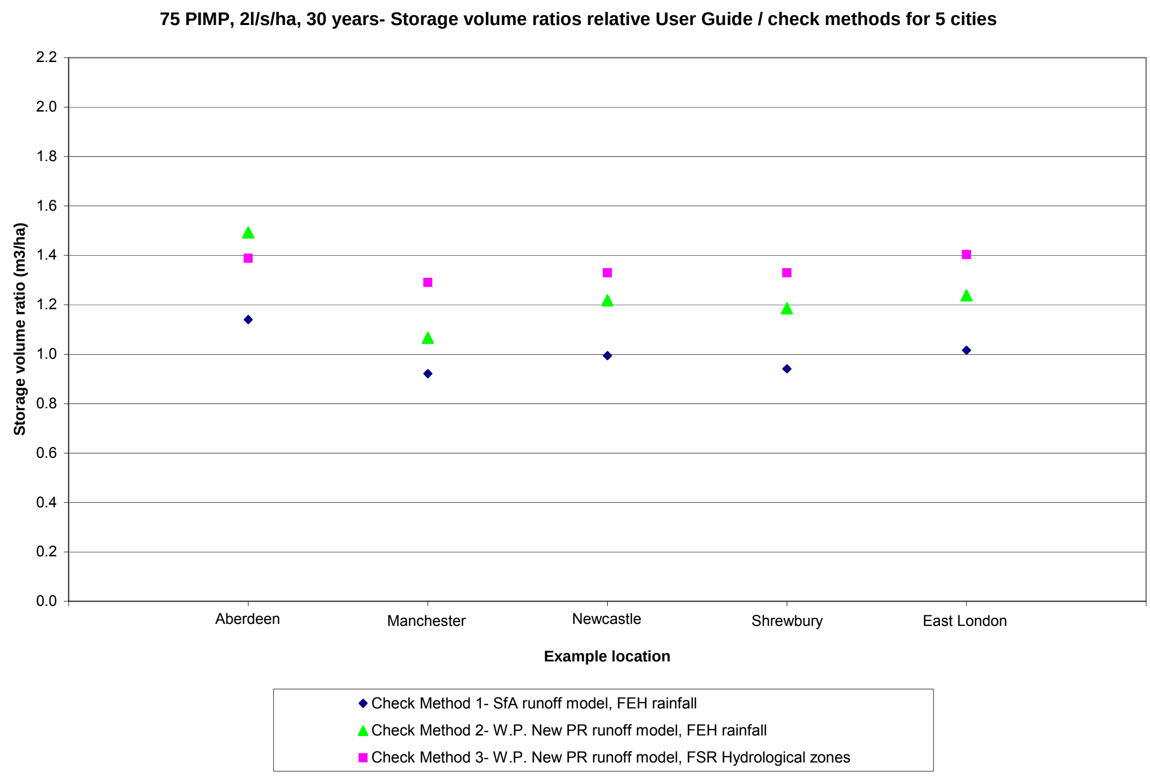


Figure A2.3.2 Check comparison of Attenuation Storage Volume for site type 3 – 30 years

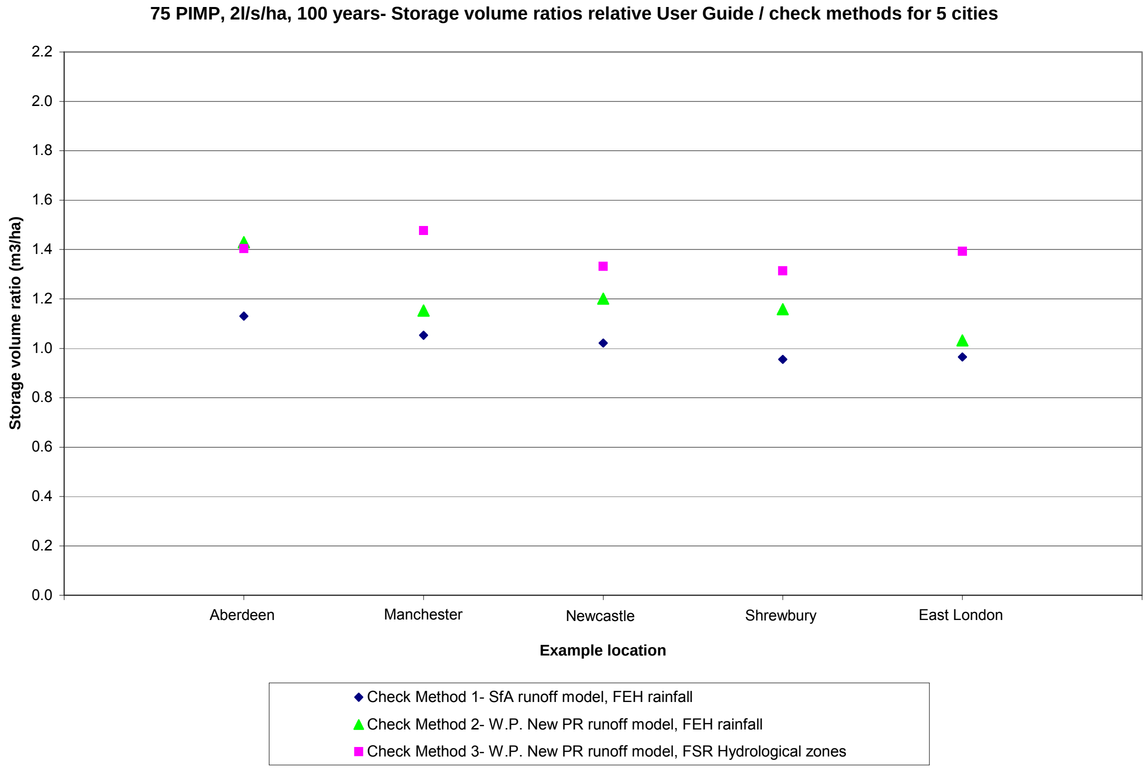


Figure A2.3.3 Check comparison of Attenuation Storage Volume for site type 3 – 100 years

Table A2.4.1 5 example sites (site type 4)- parameters and results of the User Guide method

User Guide Method						
catchment area	M5-60	R	RP (year)	FSR/FEH Ratio	Critical duration (min)	Cumulative Volume (m3/ha)
Aberdeen	14	0.2	1	0.9	720	96.11
Aberdeen	14	0.2	30	1	720	209.57
Aberdeen	14	0.2	100	1	720	265.93
Manchester	20	0.3	1	1	720	103.66
Manchester	20	0.3	30	1	720	228.18
Manchester	20	0.3	100	1.1	720	339.74
Newcastle	17	0.3	1	1	720	78.55
Newcastle	17	0.3	30	1	720	185.33
Newcastle	17	0.3	100	1	720	245.19
Shrewsbury	17	0.4	1	1	360	59.59
Shrewsbury	17	0.4	30	1	360	149.69
Shrewsbury	17	0.4	100	1	360	197.03
East London	20	0.4	1	0.9	360	65.26
East London	20	0.4	30	1	360	173.85
East London	20	0.4	100	1	360	256.98

For all models:

- PIMP= 50 %
- Q_{BAR} = 2 l/s/ha.

Table A2.4.2 5 example sites (site type 4)- parameters and results of Method 1

Check Method 1- SfA runoff model, FEH rainfall				
catchment area	RP (year)	Critical duration (min)	Cumulative Volume (m3/ha)	User Guide / Method 1
Aberdeen	1	2880	93.44	1.03
Aberdeen	30	1440	174.90	1.20
Aberdeen	100	1440	219.49	1.21
Manchester	1	2880	111.16	0.93
Manchester	30	1440	230.21	0.99
Manchester	100	1440	300.03	1.13
Newcastle	1	2160	87.95	0.89
Newcastle	30	900	179.79	1.03
Newcastle	100	720	230.89	1.06
Shrewsbury	1	1440	68.96	0.86
Shrewsbury	30	480	153.34	0.98
Shrewsbury	100	480	201.76	0.98
East London	1	480	72.00	0.91
East London	30	480	191.63	0.91
East London	100	240	268.22	0.96

For all models:

- PIMP= 50 %
- Q_{BAR} = 2 l/s/ha.

Table A2.4.3 5 example sites (site type 4)- parameters and results of Method 2

Check Method 2- W.P. New PR runoff model, FEH rainfall						
catchment area	**Soil Type**	**NAPI**	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 2**
Aberdeen	2	1	1	2880	74.26	1.29
Aberdeen	2	1	30	2160	159.06	1.32
Aberdeen	2	1	100	2160	211.78	1.26
Manchester	4	10	1	2880	106.52	0.97
Manchester	4	10	30	2880	259.55	0.88
Manchester	4	10	100	2880	362.81	0.94
Newcastle	4	10	1	2880	79.41	0.99
Newcastle	4	10	30	1440	176.05	1.05
Newcastle	4	10	100	1080	241.61	1.01
Shrewsbury	4	10	1	1440	58.51	1.02
Shrewsbury	4	10	30	480	141.09	1.06
Shrewsbury	4	10	100	480	196.53	1.00
East London	4	10	1	480	59.62	1.09
East London	4	10	30	480	184.42	0.94
East London	4	10	100	480	274.75	0.94

For all models:

- PIMP= 50 %
- Q_{BAR} = 2 l/s/ha.

Table A2.4.4 5 example sites (site type 4)- parameters and results of Method 3

Check Method 3- W.P. New PR runoff model, FSR Hydrological Zones								
catchment area	**Soil Type**	**NAPI**	**M5-60**	**R**	**RP (year)**	**Critical duration (min)**	**Cumulative Volume (m3/ha)**	**User Guide / Method 3**
Aberdeen	2	1	14	0.2	1	2880	94.14	1.02
Aberdeen	2	1	14	0.2	30	2160	176.47	1.19
Aberdeen	2	1	14	0.2	100	1440	219.71	1.21
Manchester	4	10	20	0.3	1	2880	97.13	1.07
Manchester	4	10	20	0.3	30	1080	211.98	1.08
Manchester	4	10	20	0.3	100	1080	279.19	1.22
Newcastle	4	10	17	0.3	1	1440	68.23	1.15
Newcastle	4	10	17	0.3	30	900	162.40	1.14
Newcastle	4	10	17	0.3	100	900	218.13	1.12
Shrewsbury	4	10	17	0.4	1	360	48.15	1.24
Shrewsbury	4	10	17	0.4	30	360	127.26	1.18
Shrewsbury	4	10	17	0.4	100	360	172.32	1.14
East London	4	10	20	0.4	1	360	64.66	1.01
East London	4	10	20	0.4	30	360	160.93	1.08
East London	4	10	20	0.4	100	360	215.47	1.19

For all models:

- PIMP= 50 %
- Q_{BAR} = 2 l/s/ha.

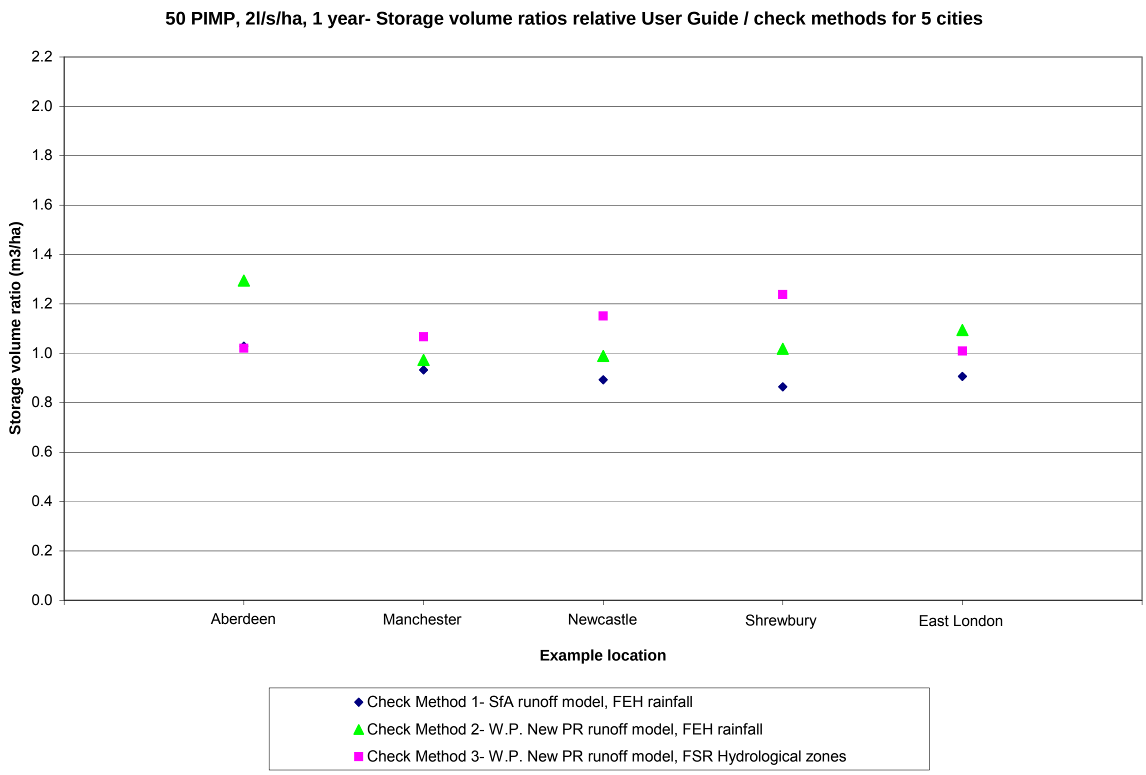


Figure A2.4.1 Check comparison of Attenuation Storage Volume for site type 4 – 1 year

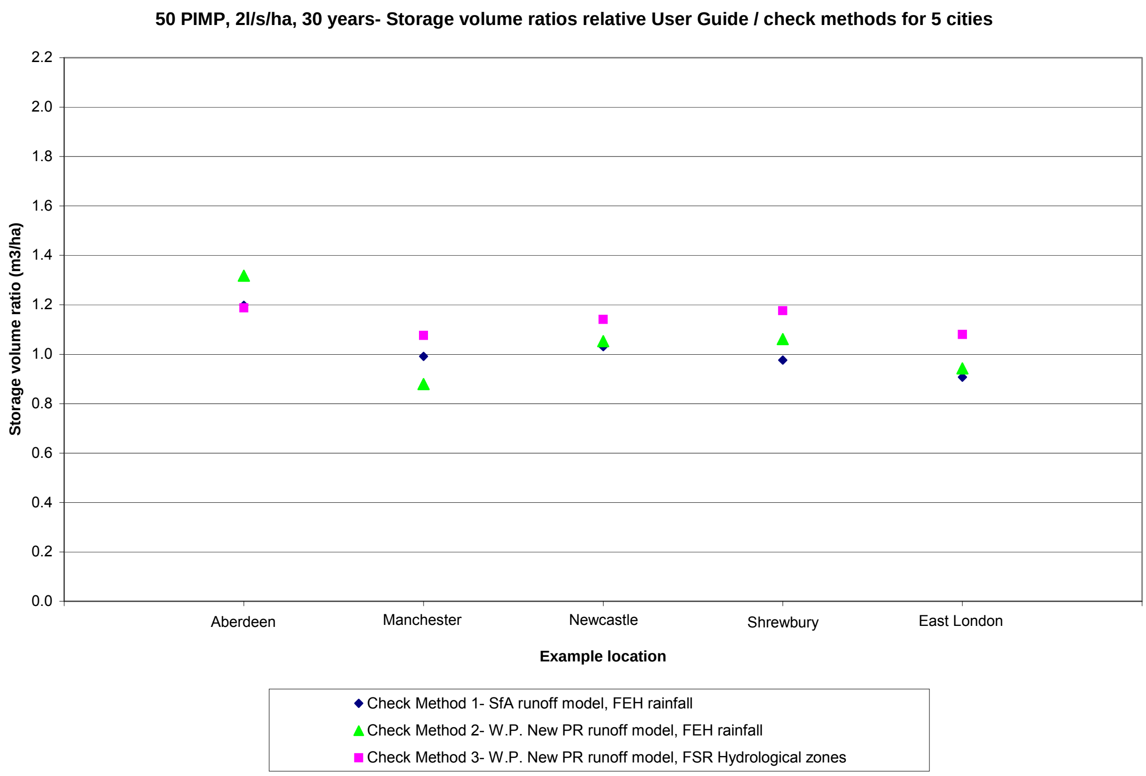


Figure A2.4.2 Check comparison of Attenuation Storage Volume for site type 4 – 30 years

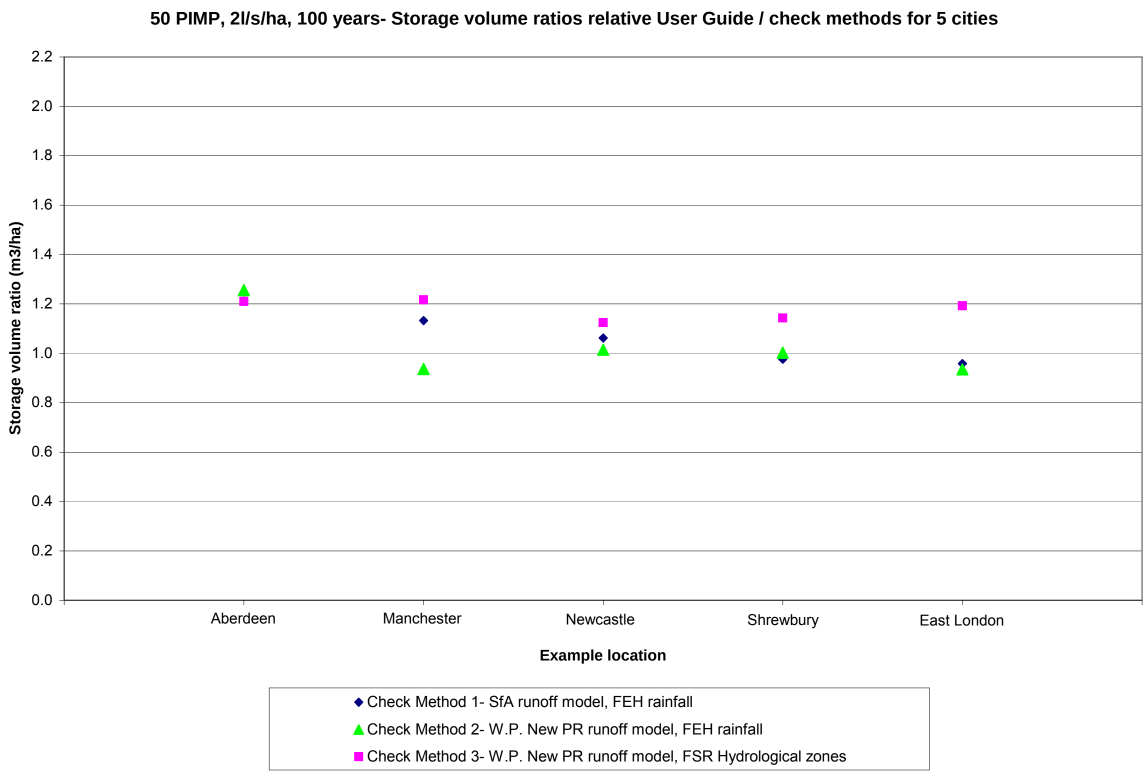


Figure A2.4.3 Check comparison of Attenuation Storage Volume for site type 4 – 100 years

Appendix 3 Methodology used for deriving storage volumes assessment

This section provides a summary of the approach used to derive the data in the various graphs and figures which have been produced for this Guide.

A3.1 Modelling

A large number of models were created and run to derive actual calculated values of storage volumes. For the eight hydrological rainfall zones, three values of Q_{BAR} (2 l/s/ha, 4 l/s/ha and 6 l/s/ha) were assumed for a development of 1ha. For each development two values of PIMP (proportion of contributing impermeable area) 50% and 100% were built. Soil is immaterial in these models as a runoff of 100% is assumed for the hard surfaces and 0% from pervious surfaces. Thus a total number of 48 models were built. If four types of soil were individually considered, the number of models would have been 192. As Figure A4.1 illustrates, the runoff assumptions used in Sewers for Adoption were thought to provide a precautionary set of results, thus avoiding having to consider soil type at this initial stage of storage volume assessment.

Clearly every site will have unique values of PIMP, Q_{BAR} and other parameters. The graphs have therefore been devised to allow interpolation of the values and so provide a value for the storage needed.

Throttle values for each value of Q_{BAR} were calculated for three return periods; 1 year, 30 year and 100 year and applied to each model. The 1 year event was first run, and the storage volume determined. This volume was then added to the model and then run with the 30 year event with 2 throttles (the 1 year and the remainder for the 30 year). The same process was then followed for the 100 year event.

This analysis was carried out for hydrological region 5 (which has the highest flow rate growth curve, FSSR 14). A selection of models from each of the Q_{BAR} families were then rerun using the lowest growth curve 3/10 to compare the difference in predicted volume. This allowed the development of a storage volume correction factor (Appendix 1, Figure A9.1) for all regions. A check was made using region growth curve 4 to be sure that the method was generic for all curves.

A more important correction factor needed to be developed for modifying the storage to compensate for the difference in rainfall from that used for the eight FSR rainfall zones. This is primarily to address the fact that the eight rainfall areas are based on FSR characteristics (see the discussion in the next section) and that FEH rainfall should really be used across the country. However it also allows the introduction of factors for climate change or compensation to take account of the actual FSR rainfall depth rather than the generalised parameter values used for that area. The development of these curves for each of the eight rainfall zones were carried out by rerunning most of the models again three times, each time by factoring the rainfall hyetographs by 0.9, 1.2 and 1.5 times respectively. The results were examined and curves simplified (to avoid too many lines on the graphs) to produce the correction factor graphs for rainfall depth.

A3.2 Rainfall

The method of approach of using FSR parameters when FEH rainfall should be used needs explanation. FEH is a digital tool with every point (1 sq. km) in the country having its own rainfall parameters defining rainfall depth for any duration and return period. This makes it virtually impossible to have a generic approach. FSR, by contrast is well known, but, more importantly, it has two parameters ("$M_5 60$" and rainfall ratio "r") which allows the derivation of rainfall depth for any return period and duration. To avoid excessive work, no differentiation was made between England and Scotland (which is a distinction made in FSR and the Wallingford Procedure). Table A4.3 shows, for a range of return periods and durations, that the differences are less than 5%.

To convert to FEH rainfall, (and possibly to also cater for the actual FSR rainfall depths and climate change factor), maps have been provided which show the difference between the FSR rainfall depths and FEH values for a range of return periods and durations (Appendix 1, Figures A6.1.1 to A6.3.4). The event duration adds a degree of uncertainty as the critical duration for any site will increase as the rainfall is factored (upwards). Some of these durations are in excess of the durations that currently exist on the rainfall ratio maps and even if the critical duration is 12 hours or less, choosing the correct duration map can only be found specifically for each site (with its own characteristics) by modelling. The simple rules provided will therefore only be approximate in their accuracy, and the degree of error introduced will be a function of the rate of change of the rainfall factor across the range of durations at that specific site. However it is believed that this does not introduce inaccuracies sufficient to invalidate the method.

Appendix 4 Overview of urban runoff models

A4.1 Runoff assumptions and criteria used

An explanation of the runoff assumptions and criteria used in developing this method will assist in understanding the level of accuracy that this tool provides together with a general appreciation of the tools currently available to the industry with regard to drainage design. This section is written to provide an overview and more detailed knowledge will require inspection of other documents which are given in the references in chapter 8.

There is a range of runoff models used in UK, including a number of empirical formulae for deriving attenuation storage, the most well known example being the COPAS (1957) formula. However there are three types universally used in the UK. These can be itemised broadly as:

- Simple fixed percentage runoff models
- Statistical percentage runoff models
- Statistical peak flow estimation models.

The important models used under each category are briefly explained and their usefulness discussed.

A4.2 Simple fixed percentage runoff models

The Rational Method approach to drainage usually uses a simple assumption of the percentage runoff contributing from each surface type. The Water Industry manual "Sewers for Adoption" 7th ed. (2012) specifies that 100% runoff should be assumed for paved surfaces and 0% from pervious areas. The choice of these values can be criticised, but these assumptions are both pragmatic and fairly safe, providing a simple approach to drainage design. These assumptions are very reasonable for the purpose that they were originally intended to address, which was the design of a drainage system under pipe-full conditions using 1, 2, or 5 year return periods. The assumption of no runoff from pervious surfaces is less appropriate for extreme events, particularly long duration rainfall, which is needed for storage assessment. However comparison with the more complex and accepted variable Wallingford Procedure runoff model (described in the next section), as illustrated in Figure A4.1, shows that these assumptions still generally provide a reasonably cautious approach, particularly for sites with a high proportion of hard surfaces. It can also be seen that where the contributing hard surface proportion is around 50% that more runoff can be predicted from the variable runoff model for certain site and event characteristics. Therefore it is recommended that the use of the runoff model for Sewers for Adoption should not be used for developments with values of PIMP less than 50% and that for PIMP values in this area that a degree of caution is exercised particularly where SOIL types 4 or 5 are applicable.

The proportion of runoff from the variable runoff model depends on the rainfall depth and soil type, so four comparison graphs are shown with each graph showing the range of results for SOIL types 1 to 4 for two hydrological rainfall zones (14/0.3 and 20/0.2) for 1 year and 100 year events. It should be noted that these rainfall characteristics are the extremes of the spectrum available. The lower bound results (from $M_5 60$ of 14mm, and rainfall ratio of 0.3) will not be dissimilar to the results for the hydrological zone of 20, 0.4 which covers most of South and East England. The values for NAPI are considered to be reasonably cautious, but an official national position on the design values for NAPI has yet to be determined. Tables A4.1 and A4.2 summarise all the relevant parameters for the graphs.

For information the rainfall depths for the hydrological zones for 6 and 12 hours across the country for the 100 year return period are shown on Figures A3.1 and A3.2 in Appendix 1 and also summarised in Table 5.3 for a range of durations. The table also summarises the differences between rainfall depths for England and Wales to Scotland and Northern Ireland. The procedure in this Guide is based on the England and Wales rainfall, which is a conservative assumption.

Table A4.1 Parameters used in the New PR equation for Figure A4.1

Soil types	IF	PF (mm)	Initial NAPI (mm)	PIMP
1	0.75	200	1	(50 – 100)
2	0.75	200	3	(50 – 100)
3	0.75	200	10	(50 – 100)
4	0.75	200	20	(50 – 100)

Table A4.2 Rainfall events used in Figure A4.1

Graph	Duration (hr)	Return period (yr)	Rainfall zones ($M_5$60, ratio "r")
1	6	1	14, 0.3 & 20, 0.2
2	24	1	14, 0.3 & 20, 0.2
3	6	100	14, 0.3 & 20, 0.2
4	24	100	14, 0.3 & 20, 0.2

Table A4.3 100 year rainfall depths for various durations comparing England and Wales with Scotland and Northern Ireland

Duration	1h			4h			12h			18h		
FSR parameters	England/Wales	Scotland/N Ireland	% diff	England/Wales	Scotland/N Ireland	% diff	England/Wales	Scotland/N Ireland	% diff	England/Wales	Scotland/N Ireland	% diff
20/0.4	40.51	38.7	4.47	57.37	53.68	6.43	72.21	67.9	5.97	78.36	73.89	5.70
20/0.3	40.51	38.7	4.47	62.5	58.62	6.21	83.99	79.47	5.38	93.13	88.78	4.67
20/0.2	40.51	38.7	4.47	70.55	66.31	6.01	103.04	99.15	3.78	117.37	114.58	2.38
17/0.4	34.16	33.33	2.43	49.66	46.51	6.34	62.91	59.02	6.18	68.42	64.26	6.08
17/0.3	34.16	33.33	2.43	54.44	50.87	6.56	73.47	69.13	5.91	81.72	77.22	5.51
17/0.2	34.16	33.33	2.43	61.43	57.61	6.22	90.71	86.28	4.88	103.79	99.94	3.71
14/0.3	27.7	27.78	0.29	45.39	42.84	5.62	62.33	58.46	6.21	69.56	65.35	6.05
14/0.2	27.7	27.78	0.29	51.99	48.61	6.50	77.49	73.04	5.74	89.12	84.65	5.02

A4.3 Statistical Percentage runoff models – urban

Statistical runoff models are being classified in this context to mean the use of a correlation equation to define the proportion of runoff. In UK there are only two urban runoff models that are widely used across the UK and these are both referred to as Wallingford Procedure runoff models. This discussion is provided here for information as detailed design of drainage systems should be carried out using one or other of the Wallingford Procedure models. It should be stressed that the initial assessment of storage in this Guide is based on the runoff model from Sewers for Adoption.

The phrase "The Wallingford Procedure" - is regularly encountered by those seeking to obtain consent for proposed drainage systems. The Wallingford Procedure originated in 1981 when HR Wallingford, with assistance from the Institute of Hydrology, completed a DoE funded project by producing a document of five volumes and a range of software called the WASSP suite of programs. This was called "The Wallingford Procedure". This suite of programs, which included a simulation programme, is now long obsolete and have been replaced over time by new products which do the same thing in a much improved way.

Thus when authorities ask for the Wallingford procedure to be applied, this is now generally taken to mean the use of a simulation tool together with the UK calibrated runoff model. Current versions of drainage simulation software are effectively applying this same technique to network design and analysis and are considered as complying with "the Wallingford Procedure".

There are now two versions of the runoff model used in the software and both are still in use throughout UK. A very brief summary is given here, but for more in-depth information, reference should be made to the Wallingford Procedure for Europe (2000) or the CIRIA report "Drainage for development sites – a guide" (2003).

The fixed UK runoff model

The first runoff model is referred to as "the fixed UK runoff model" (or the Old runoff model), and the second as "the variable UK runoff model" (or New runoff model).

The fixed runoff model assumes losses are constant throughout a rainfall event (percentage runoff does not increase as the previous surfaces get wetter) and is defined by the equation:

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

where:

PR = percentage runoff
PIMP = percentage impermeability
SOIL = an index of the water holding capacity of the soil
UCWI = Urban Catchment Wetness Index.

The PR equation was derived by statistical analysis from data from 33 catchments. It should be noted that the equation is entirely statistical and takes no account of ground contouring.

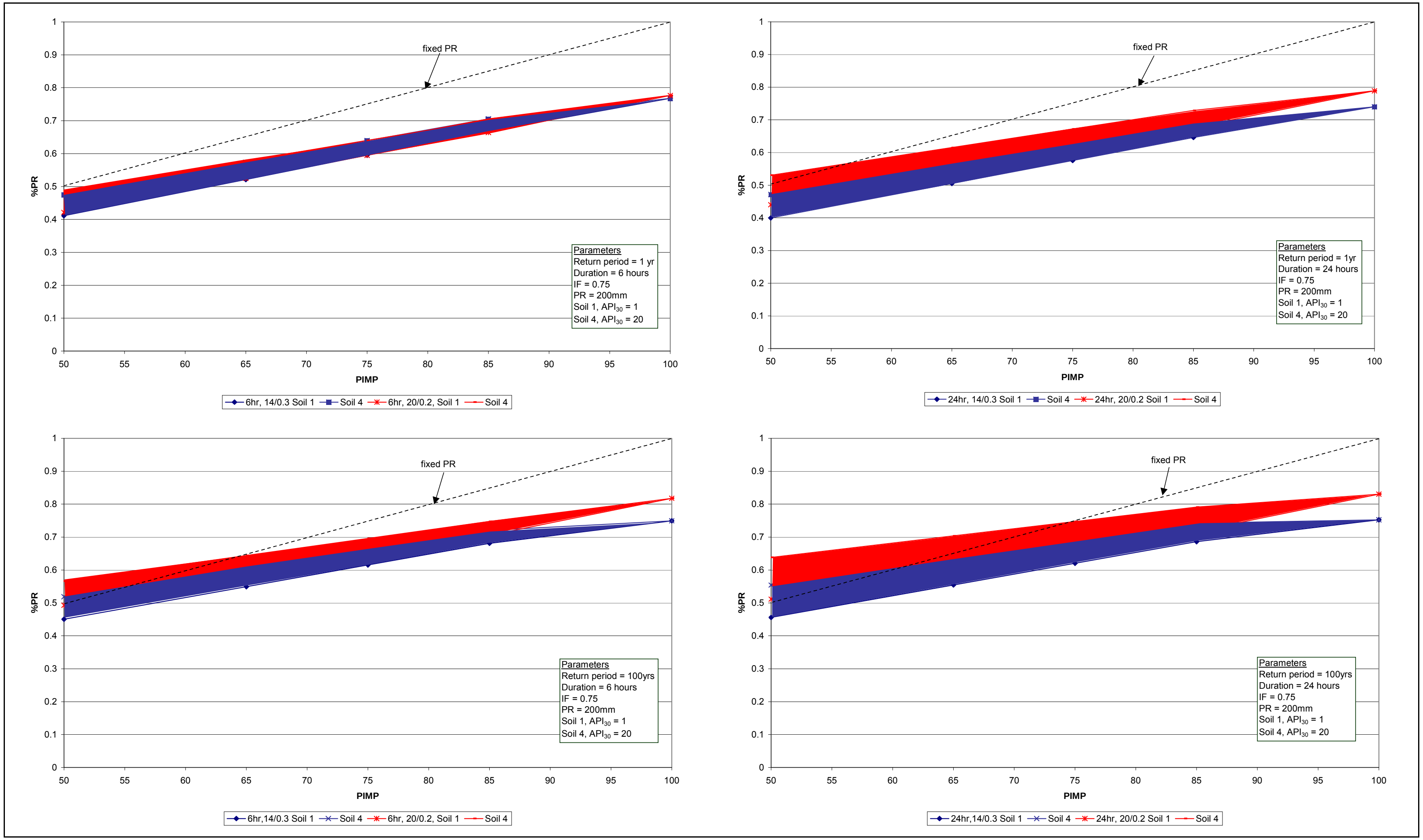


Figure A4.1 Comparison of PR between the variable Wallingford Procedure runoff model and Sewers for Adoption

The Variable UK runoff model

The variable UK PR equation was developed jointly by HR Wallingford, the Water Research Centre and the Institute of Hydrology with support from North West Water plc. It was designed as a replacement to the fixed UK PR equation. Although it was developed several years ago, choice as to which equation should be used is still being debated and is not discussed here, but the key point being that both are still generally accepted.

The new equation was designed primarily to overcome some of the difficulties experienced in practical application of the fixed runoff model, namely:

- The old equation defines PR as being a constant throughout a rainfall event irrespective of catchment wetness. Clearly for long duration storms, lower losses towards the end of the event may be significant in terms of urban drainage design for storage.
- Problems have been encountered in applying the PR equation to partially separate drainage systems and to areas with low PIMP and low SOIL values.

The new model was produced in the form:

$$PR = IF * PIMP + (100 - IF * PIMP) * \frac{NAPI}{PF}$$

where:

IF = effective impervious area factor
PF = moisture depth parameter (mm)
NAPI = 30 day antecedent precipitation index

This equation divides PR into two elements. First, the impervious area runoff is obtained by using an effective contributing area factor, IF. After initial losses on impervious surfaces, remaining losses are given as a constant fraction of rainfall volume. Recommended values of IF are indicated in Table 5.4. One of the principal features of this equation (and a possible drawback) is that engineers have to choose a value.

Table A4.4 Recommended values of IF

Surface Condition	Effective impervious area factor, IF
POOR	0.45
FAIR	0.60
GOOD	0.75

The losses on pervious surfaces and also non-effective impervious areas are represented by the second term of the equation. The first part of this term represents the total percentage of the area occupied by pervious and non-effective impervious surfaces. The losses from this area are dependent on the function NAPI/PF.