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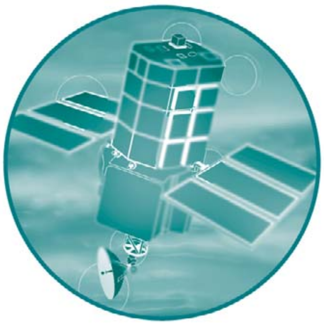

Preliminary rainfall runoff management for developments

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

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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.

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SUMMARY

This Guide is aimed at Regulators, Developers and Local Authorities to advise 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 the Interim Procedure produced by the Environment Agency which is reproduced in this document following on from this summary.

This Guide may be used to form part of a Flood Risk Assessment to comply with PPG25, but it does not address issues such as risk of flooding from a watercourse, effects of changes in floodplain storage or in floodplain conveyance.

The Guide provides flow charts and forms to fill in using look up tables and figures which allow the whole process to be carried out without reference to other documents. It is stressed that the approach provided for sizing of stormwater storage is only to be used at Master Plan stage to assist with defining indicative volumes.

The Guide also touches briefly on a number of related issues such as drainage modelling and detailed design to provide guidance and information for more detailed analysis of stormwater drainage requirements.

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 conservative, but sufficiently accurate to provide a reasonable indication of the storage requirements.

There are three main elements that need to be considered for stormwater design. These are:

- Conveying stormwater on the site
- Treating stormwater runoff
- Controlling stormwater runoff using storage.

Conveying stormwater on site
Attention is drawn to the rule of thumb methods for sizing of pipes and the reasons for carrying out an initial sizing of the conveyance system. Additionally attention is drawn to the need to consider the impact of extreme events and the effects of overland flooding.

Treating stormwater runoff
The Guide provides a method for defining a **Treatment volume** which is aimed at improving the quality of the stormwater runoff. This is normally applied as the dry period volume of water in a pond. It must be stressed, however, that although the Guide does not go into detail on the concept of the treatment train and advice on the use of SUDS, the provision of this Treatment volume in a single pond without the use of other SUDS components, would not generally be regarded as providing sufficient protection in dealing with the treatment of stormwater.

Controlling stormwater runoff from the site using storage
This Guide divides this storage into two components:

- the first is **Attenuation storage** to limit discharge to greenfield, or pre-development, discharge rates
- the second is **Long Term storage** to address the additional volume of runoff generated by the developed area compared to the runoff that previously took place from the greenfield site.

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	Expressed as an index determined by summation of weighted.
Index	daily rainfalls for a period preceding the start of a specific event.
Attenuation Storage	Temporary storage required to reduce the peak discharge of a flood wave. This is by an increase in duration of increased flow.
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 discharge.
Brownfield site	Redevelopment of a previously-developed 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	Permission granted by the appropriate public authority to discharge potentially polluting flow to a watercourse or into the ground, subject to meeting specific conditions.
Contributing area	The area that contributes storm runoff directly to the sewerage 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, such as 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 sufficient dry period for runoff and discharge from the drainage system to cease.
Extreme event	Single occurrence of an event that is likely to occur very infrequently (e.g. long drought or big storm, etc.).
First flush	The initial discharge of active sediments and pollutants generally higher than the average concentration of pollutants caused by rainfall.
Flood Risk Assessment	Technical review of the effects of a development on the risk of flooding on that development, and on adjacent sites upstream and downstream.
Flood Studies Report	Landmark report in UK for catchment Hydrology – Institute of Hydrology 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 at 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 saturation zone from which wells and springs are fed. In a strict sense 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 limit the rate of the 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. Usually a measure of roof and road surfaces in simulation modelling.
Infiltration	(a) The unintended ingress of groundwater into a drainage system. (b) The introduction of rainwater runoff into the ground.
Initial loss	In hydrology, rainfall preceding the beginning of surface runoff. It includes interception, surface wetting, and infiltration.
Intensity-duration-	The relationship between rainfall intensity (amount per unit of time), frequency rainfall duration (total time over which rainfall 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 zoned based on economic, geographic or demographic use of land, such as residential, industrial, farm, commercial.
"Long Term" Storage	Storage of stormwater which is drained by infiltration, or if this is not possible, directly drained at a rate of less than 2l/s/ha.
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 storage area when the capacity of that storage is exceeded.
Overland flow	The flow of water over the ground or paved surface before it enters some defined channel or inlet, often assumed to be shallow and uniformly distributed across the width.
Peak discharge	The maximum flow rate at a point in time at a specific location resulting from a given storm condition.
Percentage runoff	The percentage of the rainfall volume falling on a specified area which enters the stormwater drainage system.
Pervious area	Areas of ground which allows infiltration of water, although some surface runoff may still 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 surrounding 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 such as CSOs.
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 OFWAT due to their legal involvement in controlling Water Companies.
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 material originally carried by water, which has been deposited.
Sewerage Undertaker	An organisation with the legal duty to provide sewerage services in an area. In England and Wales these services are provided by ten Water Service companies, in Scotland by a single Water Authority, and in Northern Ireland by the Water Service of the Department of the Environment in Northern Ireland.
Simulation	The representation of specific conditions during a specific period in a sewerage system, treatment works, river, etc., by means of a computer model.
Soakaway	A 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 in 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 infiltrate. Used in connection with a phenomenon which is either unusual or of great magnitude, rate, or intensity.
Surface water	Water from precipitation which has not seeped into the ground and which is discharged to the drain or sewer system directly from the ground or from exterior building surfaces.
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 events generated artificially or selected real historical events which are representative of the rainfall in that area.

Treatment Storage	Storage provided to enable poor water quality to gain an improved standard.
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	The chemical, physical and biological characteristics of water with respect to its suitability for a particular purpose.
Water quality	The chemical and biological content of water, usually compared to defined standards, many of which are set by the national legislation or European Community directives and enforced by regulatory authorities in member states.
Watercourse	A natural or artificial channel for passage of 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)
M_560	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
PPG25	Planning Policy Guidance 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

SAAR Standard Average Annual Rainfall assessed over a period of years

SfA5 Sewers for Adoption 5th Edition
SMD Sort 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 Urban 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

1. **Procedure status.** This procedure is an interim method, which is expected to be revised as improved tools are developed. It utilises well recognised existing methods, but revision is anticipated to provide a more consistent approach as and when FEH procedures can be extended to catchments at development scale.

2. **Compliance to national guidance.** The objective of this procedure is to assist developers and their designers to conform to PPG25 which states "… where possible to reduce and certainly not to increase flood risk".

3. **Application of the procedure.** This procedure applies to both greenfield and brownfield sites. In the case of brownfield sites, drainage proposals will be measured against the existing performance of the site (although it is preferable for solutions to provide runoff characteristics which are similar to greenfield behaviour). Therefore where greenfield performance is referred to in this document, this should be considered as meaning the existing site conditions for brownfield redevelopment sites. Sites with polluted land will have particular consent requirements and affect the drainage techniques that can be used.

4. **Use of infiltration.** Part H of the Building Regulations requires that the first choice of surface water disposal should be to discharge to infiltration systems where practicable. Infiltration techniques should therefore be applied wherever they are appropriate.

5. **Sewers for Adoption.** Drainage calculations and criteria, where appropriate, should comply with the 5th edition of Sewers for Adoption.

6. **Need for this procedure.** It is recognised that the impact of urban development on greenfield areas increases both the rate of run-off and the volume of run-off in response to rainfall and that the water quality impact on the receiving watercourse is likely to be detrimental.

7. **Procedure philosophy.** The objectives of this procedure are to:
   - stormwater runoff discharged from urban developments to replicate or achieve a reduction from the greenfield response of the site over an extended range of storm probabilities (return periods)
   - manage runoff on site for extreme events.

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

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. 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 merit specific consideration; 100%, 3.33% and 1%. (Note that in many places elsewhere in this Guide return periods are used 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 these more frequent events.

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 5th edition (SfA5). SfA5 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 PPG25 and also recognises it is not practicable to fully limit flows for the most extreme events. Also SfA5 recognises that, during extreme wet weather, the capacity of surface water sewers may be inadequate. SfA5 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.** up to the 1% annual probability event should preferably be contained within the site at designated temporary storage locations unless it can be shown to have no material impact 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 short high intensity rainfall events of between 15 minutes and 1 hour duration.

9. **The calculation of greenfield runoff rate.** The calculation of peak rates of runoff from a greenfield site is related to its size. The values derived should be regarded as indicative due to the limitations of the existing tools. Table 9.1 summarises the techniques to be used.

Table 9.1 Tools to be used for calculation of greenfield run-off criteria

Development size	Method
0 – 50 ha	The Institute of Hydrology Report 124 Flood Estimation for Small Catchments (1994) is to be used to determine peak green field run-off rates. Where developments are smaller than 50 ha, the analysis for determining the peak greenfield discharge rate should use 50 ha in the formula and linearly interpolate the flow rate value based on the ratio of the development to 50 ha. FSSR 2 and 14 regional growth curve factors are to be used to calculate the greenfield peak flow rates for 1, 30 and 100 year return periods.
50 ha – 200 ha	IH Report 124 will be used to calculate greenfield peak flow rates. Regional growth factors to be applied.
Above 200 ha	IH Report 124 can be used for developments that are much larger than 200 ha. However, for schemes of this size it is recommended that the Flood Estimation Handbook (FEH) should be applied. Both the statistical approach and the unit hydrograph approach should be used to calculate peak flow rates. The unit hydrograph method will also provide the volume of greenfield run-off. However, where FEH is not considered appropriate for the calculation of greenfield run-off for the development site, for whatever reasons, IH 124 should be used.

10. **Volumetric criteria.** The stormwater runoff volume from a site should be limited to the greenfield runoff volume wherever possible. The additional runoff volume caused by urbanisation should be controlled using two criteria.

10.1 **Interception.** Where possible, infiltration or other techniques are to be used to ensure minimal 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 2l/s/ha.

10.3 Where compliance to 100 year volumetric criterion, as defined in section 10.2, is not provided, the limiting discharge for the 30 and 100 year return periods will be constrained to the mean annual peak rate of runoff for the greenfield site (Referred to as Q_{BAR} in IH Report 124).

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. Runoff from impermeable surfaces served by effective infiltration systems can be assumed to contribute no runoff for storage volumes assessment.

13. **Detailed design of stormwater runoff.** All network design for stormwater runoff and proof of compliance in meeting peak flow rate discharge criteria, using computer simulation, should use the standard Wallingford Procedure variable UK runoff model using appropriate parameters.

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. **Reliability of SUDS.** At present certain SUDS units are considered to have some degree of risk of medium term hydraulic failure, due to either maintenance or possible change of status. In these situations, to ensure compliance with pipe capacity criteria, they will be deemed not to be effective when calculating pipe sizes and storage requirements. For pipe sizing the current view of the Water Undertakers should apply (see the National SUDS Framework document). For storage sizing of all structures which are not to be adopted by Water Undertakers, the view of the Environment Agency should normally apply.

16 **Climate change factor.** Climate change will be taken into account in hydrological regions by increasing the rainfall depth by 10% for computing storage volumes. The official advice by Defra on river flows is that a 20% increase should be added for climate change. Due to the relationship between rainfall and runoff being non-linear, the use of 10% additional rainfall is considered to approximate to a 20% increase in runoff for larger events. No allowance for climate change should be applied to calculated greenfield peak rates of runoff from the site for any hydrological region. It should be recognised that although climate change is acknowledged as taking place, certainty regarding the hydrological changes, particularly of extreme short duration events, is very low.

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. It is suggested that this is 5 litres per second, using an appropriate vortex flow control device or other technically acceptable flow control device. The minimum size of pipe discharging from a flow attenuation device should be 150mm laid at a gradient not flatter than 1 in 150, which meets the requirements of Sewers for Adoption 5th Edition.

18. **Catchment Flood Management Plans.** CFMPs (Catchment Flood Management Plans), consider the impact of development on flood risk in the catchment based on existing land use plans contained in the local plan published by the Local Planning Authority and projections of development beyond the periods covered by the land use plans. Strategy Plans identified in the CFMPs each cover part of the catchment and may consider the local impact of these developments in more detail. Where these exist for an area proposed for development, their findings must be taken into account in the development proposal.

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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 to assist in the initial sizing of storage elements for the control and treatment of stormwater runoff.

The Guide:

- Provides an easy-to-use method for assessing initial storage volumes for stormwater control and providing simple guidance on the stormwater design process generally
- States the Environment Agency policy on stormwater treatment and discharge
- Provides supporting information on the assumptions used and briefly covers some other important technical issues that are usefully highlighted.

Stormwater system design, with its emphasis on SUDS and limiting discharge rates to receiving waters or drainage systems, now has a major impact on both development costs and planning the layout of the site. This Guide will help in both these issues, providing a quick manual method for initial sizing of storage and also drawing attention to the timing of various stormwater design activities within the project planning cycle. This Guide complements the CIRIA report “Drainage of Development Sites – A Guide” (2003) and also the Interim Code of Practice for Sustainable Drainage Systems document produced by the National SUDS Working Group (2004) which gives the current water industry view on the use of SUDS.

The Environment Agency “Rainfall-runoff management for development – an Interim national procedure” (2003) is current national guidance detailing the approach that must be taken for stormwater design and this is included for reference. This Guide has been developed using the criteria stated in the interim procedure.

2. OVERVIEW OF STORMWATER DESIGN REQUIREMENTS

This chapter briefly describes the key aspects of stormwater design, describing the importance of addressing the subject at an early stage in the development planning process and giving an explanation of the philosophy being applied and reasons for the storage criteria that are stipulated. This is illustrated with flow path diagrams, individual elements of which are discussed to ensure the requirements are understood.

It should be stressed that the storage design tools of this Guide are only to be applied at initial design stages and that computer models would normally be expected to be used for detailed design of all aspects of the drainage system. However, the principles remain valid at all stages of design.

The other key point to stress is that the calculations of these storage volumes are based on the premise that all hard surfaces provide 100% runoff and all pervious areas provide 0% runoff based on the assumptions from Sewers for Adoption. In practice the use of certain SUDS units, particularly pervious pavements, provide highly modified runoff characteristics and the storage provided by these and other units should be taken into account in the detailed design stage.

The calculated volumes 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 storage needed for any site.

Chapter 3 provides a simple step-by-step look-up method for assessing stormwater storage requirements. Explanation of the assumptions used and why this approach has been taken is given in section 2.2. Chapter 5 provides more detailed information on a number of relevant drainage design issues.

2.1 Key stages of stormwater design

This Guide provides a manual method for sizing stormwater storage for managing runoff, and this section provides a summary of the elements of stormwater design for the various stages of the development planning process. The CIRIA report "Drainage of Development Sites – A Guide" (2004) provides a detailed outline of the staged planning process as well as providing detailed advice on all aspects of site drainage design. Table 2.1 summarises the key stages and activities that need to be carried out for stormwater design. Figures 2.1 and 2.2 illustrate the design process for initial and detailed design. Task activities are highlighted in the areas that this Guide provides detailed assistance.

Section 2.2 of this chapter provides a summary of the principles being used and the reasons for the provision of stormwater storage. It then goes on to give a brief explanation of each of the elements of figures 2.1 and 2.2 to assist in understanding the activities of each of the design stages.

Chapter 3 provides a simple step-by-step look-up method for assessing stormwater storage requirements.

Table 2.1 Stormwater drainage design stages

	Stage	Activity
1	Purchase of land for development	Approximate estimation of stormwater storage requirements (to assess costs)
2a	Master Plan definition	Conceptual outline of SUDS components to be used and location of stormwater storage and outfall(s)
2b	Environmental Impact Assessment	Check conceptual stormwater design proposals meet the Environmental Impact Assessment requirements
3	Detailed definition of development site	Detailed design of stormwater storage including detailed design of drainage components (SUDS and pipe network)

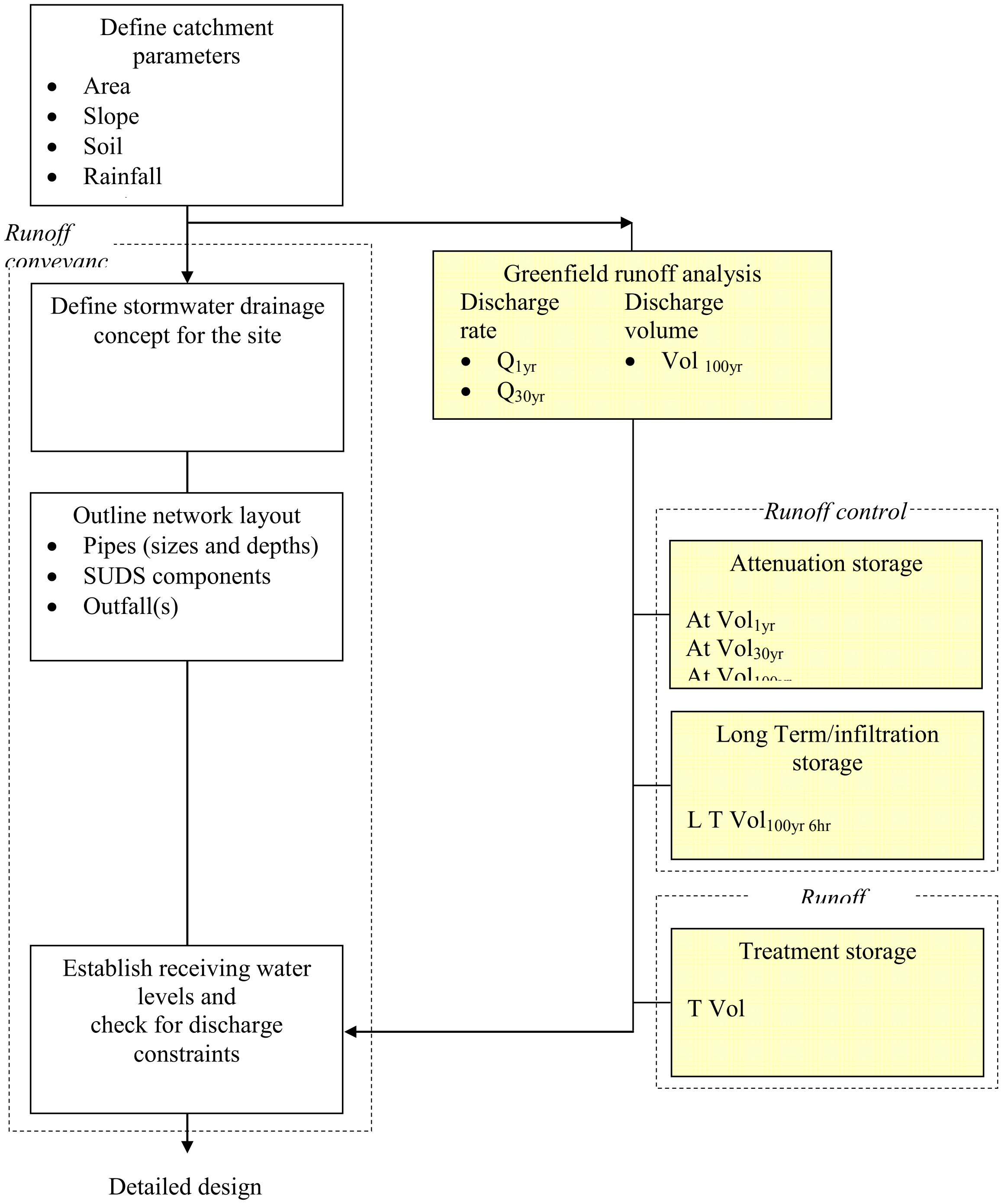
Essentially there are 3 design stages (assuming the Environmental Impact Assessment does not result in a change of stormwater strategy).

Stage 1 – Prior to or during Master Plan development
Nearly all sites will need to provide some form of stormwater storage. The initial estimate of storage volumes can be very large and are a function of climate and physical characteristics of the site. This analysis can be carried out very easily using the manual method provided by this Guide and will assist in assessing global costs and initial discussions with the local authorities and the Environment Agency. Preliminary assessment of flood risk of a site for outline consent will usually need to apply the methodology given in this Guide.

Stage 2 – At Master Plan development / Environmental Impact Assessment
A detailed evaluation of the stormwater constraints imposed by the catchment and the opportunities for using SUDS based on site characteristics should be made during development of the Master Plan. This will allow partial redistribution of storage across the site using a range of SUDS units other than ponds as well as outline design of the conveyance processes to be used. The plan requirements of the storage units and their location should be considered, recognising that there might be downstream constraints created by the receiving water levels.

Stage 3. – Detailed planning of the site drainage
Design of the SUDS units, conveyance system and storage units should be carried out in detail. Assumptions used for sizing of any pipework to be adopted will need to take account of Water Company precautionary requirements due to uncertainty of long-term performance of some SUDS units. Analysis for assessing flooding performance and compliance with discharge consent requirements should use the Wallingford Procedure variable runoff model and detailed representation of all hydraulic components of the drainage system should be made within the limits of existing drainage software capabilities (see Chapter 5).

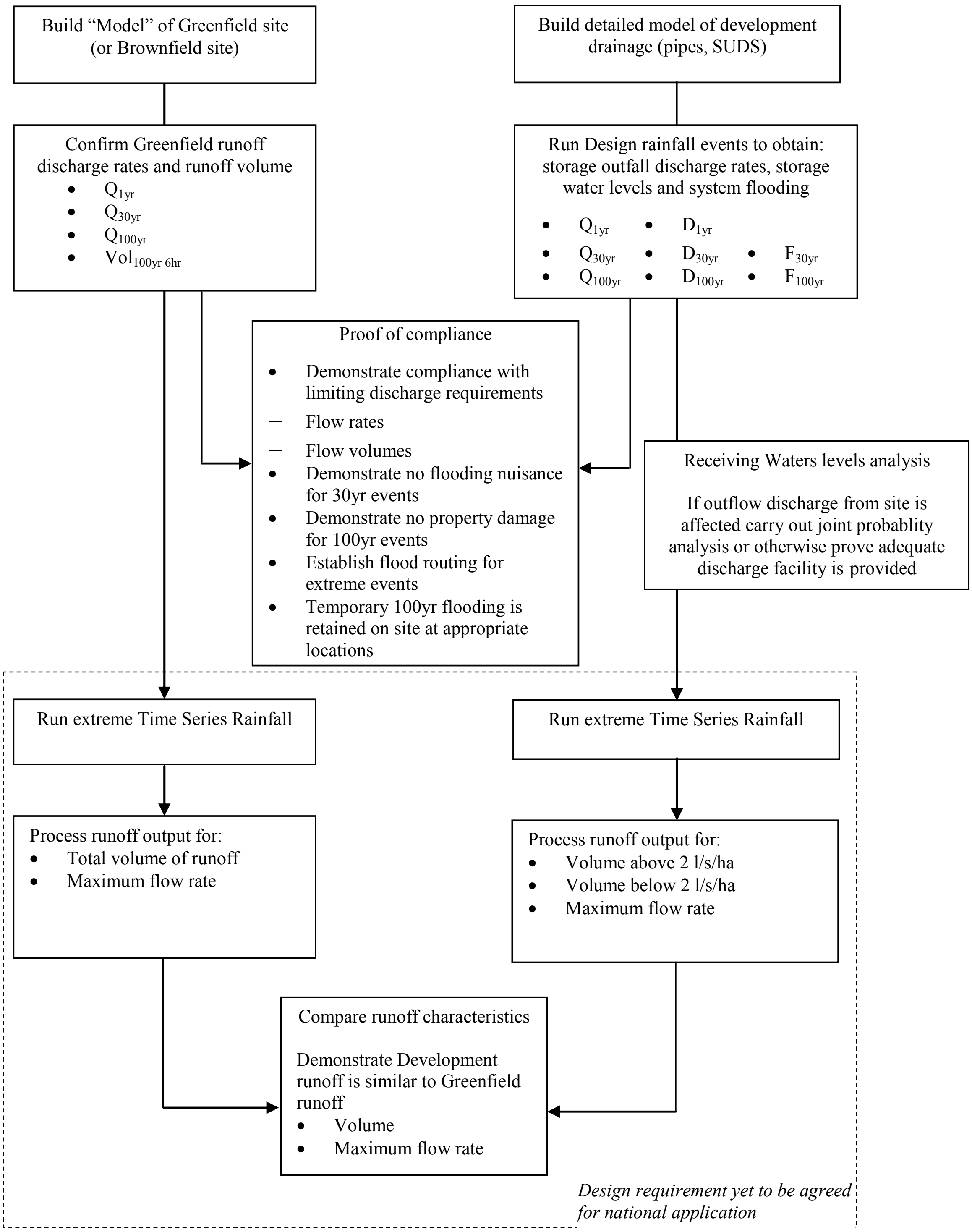
Detailed design normally requires detailed modelling to be carried out, so this initial assessment method for storage sizing should not be used at stage 3.



Notes: *Q_{1yr}* *Peak discharge rate for 1 year return period*
PIMP *Percentage of the catchment which is impervious (roofs and roads)*
Vol_{1yr} *The volume of storage for 1year return period*
Values obtained using this guide

Figure 2.1 Initial design of stormwater drainage for new developments

Detailed Design



Notes:	Q_{1yr}	*Peak discharge rate for 1 year return period*
	D_{1yr}	*Depth of water for 1 year return period*
	F_{30yr}	*Flooding volume (and location) for 30 year return period*
	$Vol_{100yr\ 6hr}$	*The volume of storage for 100 year return period*

Figure 2.2 Detailed design of stormwater drainage for new developments

2.2 The necessity for provision of stormwater storage

Rainfall runoff from greenfield areas (whether agricultural land or virgin land) 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

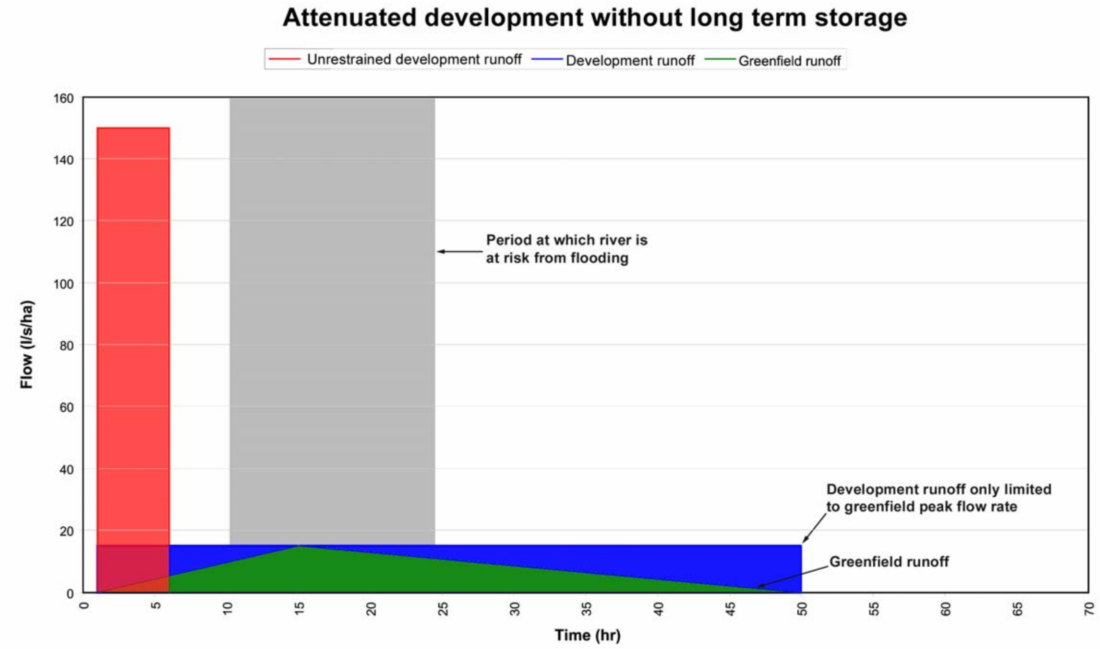
- Quality of runoff
  - Cleaner runoff (BOD, sediment, pathogens, metals, hydrocarbons)

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

Volume of stormwater runoff. – small rainfall events
The volume of rainfall runoff is important at each of end of the rainfall spectrum. Around 30 to 40 percent of rainfall events (probably in excess of 50 events a year in most areas), are sufficiently small that there is no measurable runoff taking place from greenfield areas into receiving waters. By contrast runoff from developments takes place for virtually every rainfall event. The difference means that streams are more “flashy” and groundwater recharge is being reduced, thus reducing base flows in the streams between events. (The related issues of water quality are addressed under quality of runoff). The criterion of provision of storage and release of stormwater dealing with volume of runoff does not specifically address this important issue. However where it is possible to provide replication of this behaviour (described as Interception) in being able to prevent runoff from rainfall of up to 5mm, this should be provided. Certain SUDS features such as Swales and Pervious Pavements do provide runoff characteristics that reflect this behaviour to some degree.

Volume of stormwater runoff. – large rainfall events
In extreme rainfall events the total volume of runoff from a developed site is typically between 1 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 it were released over a similar period to the greenfield runoff, due to the finite storage volume provided by flood plains, by definition there must be greater depths of flooding if more water is discharged (see Figure 2.3).

The criterion for Long Term Storage is a pragmatic approach to calculating an appropriate volume which should be retained and discharged at sufficiently low flow rates to the receiving water, such that there is limited impact on exacerbating flooding downstream. This is achieved by either the use of infiltration or sufficient attenuation that discharge from the development is below 2l/s/ha. Theoretically this form of storage need only be mobilised at times of extreme rainfall. However in practice it is difficult to mobilise this storage only during extreme events. Figure 2.3 illustrates the effect of providing long term storage and demonstrates the reduced volume of runoff contributing to a river at times of flooding. The basis of calculating the Long Term Storage volume is to use a 6 hour 100 year event. The volume derived is largely influenced by the soil type of the site and is particularly onerous in areas of SOIL type 1.



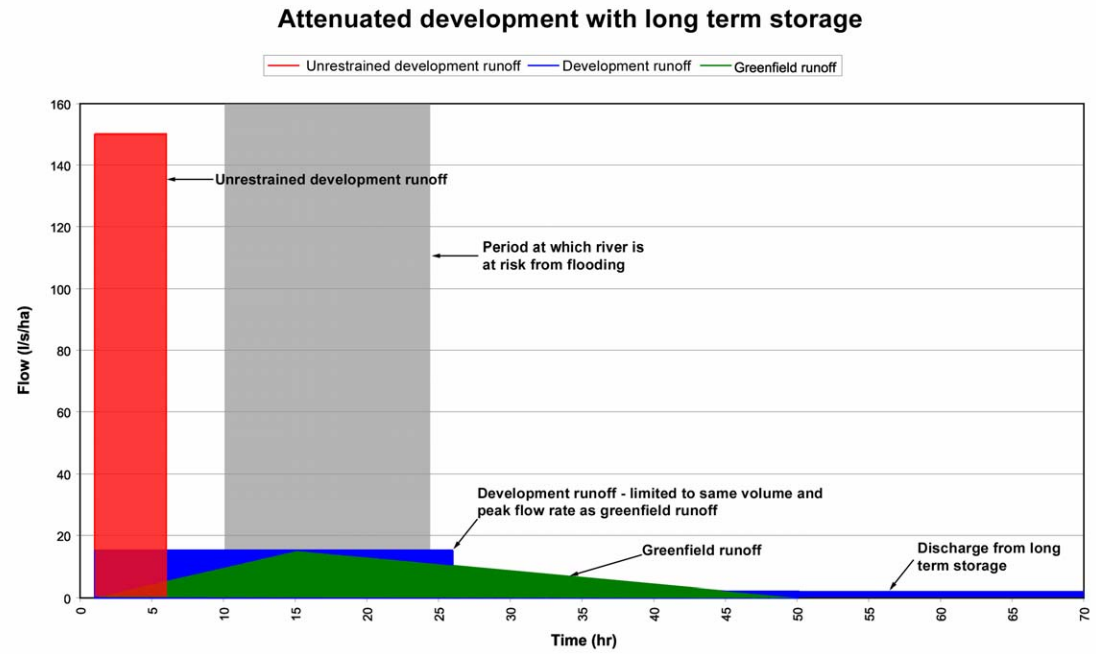


Figure 2.3 Schematic illustrating river flooding protection using Long Term Storage

Rate of stormwater runoff
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. This causes flashy flow in the river which is likely to cause scour and erosion that may seriously affects the morphology and ecology of the stream.

Attenuation storage is provided to limit the runoff from the site to minimise these problems in the receiving water. 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 of individual events would require a very complex approach. This is not justified based on water quality or hydraulic grounds, and due to the limited accuracy of predicting the actual runoff from greenfield sites.

There are two exceptions where the use of this methodology requires modification in its application.

The first exception is in areas of SOIL type 1 where the calculated storage volumes from both Attenuation storage and Long Term storage become very high. Even though the methodology is sound in trying to reflect the actual greenfield flow rates and runoff volumes (that very little runs off from highly porous catchments), it effectively makes development impractical with Q_{BAR} values normally being between 0.1 and 0.2 l/s/ha. These values are out of range of the charts and would produce extraordinary storage requirements. Also the research, which forms the basis of this Guide, generally showed that limiting discharge rates lower than 2 l/s/ha was effective in providing river flood protection. It is therefore proposed to use a Q_{BAR} value of 1 l/s/ha, recognising that the Long Term storage volume (which should be applied in the form of infiltration methods) will generally be large. Note that a Q_{BAR} value of 1 l/s/ha will have a 100 year limiting discharge of 3.5 to 1.9 l/s/ha depending on the hydrological region. The charts therefore need to be used carefully to interpolate an estimate of Attenuation storage requirement.

The second exception is the need to adjust the limiting discharge rate to take account of a minimum practical orifice size and the size of the development, particularly where an orifice is proposed as the method of hydraulic control. This may be an orifice of 150mm diameter for reasons of Adoption, stipulated by the sewerage undertaker. (It should be noted that this does not preclude lower limiting discharge rates where appropriate use of SUDS is applied). Calculations can be made to derive an equivalent Q_{BAR} discharge rate for the site to enable an assessment of the storage volume needed for the site. However this value may well be much greater than an equivalent Q_{BAR} value of 6 l/s/ha for small sites in which case it will be outside of the range of these charts. It will therefore not be possible to estimate the Attenuation storage volume. Although the storage volume is likely to be very small, a simulation model would need to be used to calculate the storage volume.

Quality of stormwater runoff

The quality of stormwater runoff is an issue for small events. 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. For large events or during periods of high river flow, this water quality impact is much reduced, so the key period of concern is the summer months of low river flows and the many small events which take place on a regular basis.

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 volume of runoff from 10 to 15mm of rainfall.

This storage should not be confused with the concept of Interception referred to earlier in this section in the discussion on the volume of runoff. Clearly if no runoff takes place for small events, maximum water quality protection is being achieved.

It should be stressed that drainage of a site should now be designed using the treatment train concept using appropriate drainage mechanisms. Reliance on only a single pond prior to the outfall is not regarded as best practice in providing the best water quality protection for the receiving water. In some cases a wet pond (providing treatment storage) may not be the most appropriate solution. In this situation treatment of surface water runoff would be achieved using other SUDS techniques.

3. PRELIMINARY SIZING OF STORMWATER STORAGE VOLUMES

This chapter provides a simple look-up method for deriving the storage volumes needed to meet Environment Agency recommendations for ensuring minimal impact from stormwater runoff on the environment.

The method provided has been developed to minimise the need for technical expertise and the use of computer tools to arrive at approximate values for stormwater storage. All parameters and factors for any site can be obtained from the figures and tables provided, except where the resolution of A4 figures of the UK are considered inadequate and other well known sources are available. In these cases, references are provided (such as the Wallingford Procedure maps) where the information can be found.

Figure 3.1 provides a flow diagram of the process to follow to obtain the stormwater storage volumes. There are 3 storage volumes to determine. These are:

- Attenuation storage
- Long Term storage
- Treatment storage.

A brief explanation of the need for each of these three elements is given in section 2.2. The distinction between each of these elements can be summarised as follows:

Attenuation storage aims to limit the rate of runoff into the receiving water to similar rates of maximum discharge as that which takes place before the site is developed (greenfield runoff rate). This can be provided at one or several different locations using a variety of SUDS 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.

Treatment storage aims to ensure the water quality of the stormwater is sufficient to cause minimal impact on the flora and fauna in the receiving water. This is normally provided as the dry period volume of ponds.

A maximum limit on the size of the development for using this Guide to determine Attenuation Storage and Long Term Storage is suggested. Although Report IH124 is applicable up to 25km^2, it is considered that the sizes of developments larger than 200 ha would warrant a more accurate first estimation of storage requirement using FEH to determine the maximum greenfield rate of runoff and volume runoff. These figures could subsequently be used to determine storage volumes using this Guide or by building computer models.

Sheets ASV1 to ASV4 are used to provide an assessment of Attenuation Storage Volume. Three values are determined which represent the 1 year, 30 year and 100 year storage amounts together with their respective limits of peak discharge to the receiving water.

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

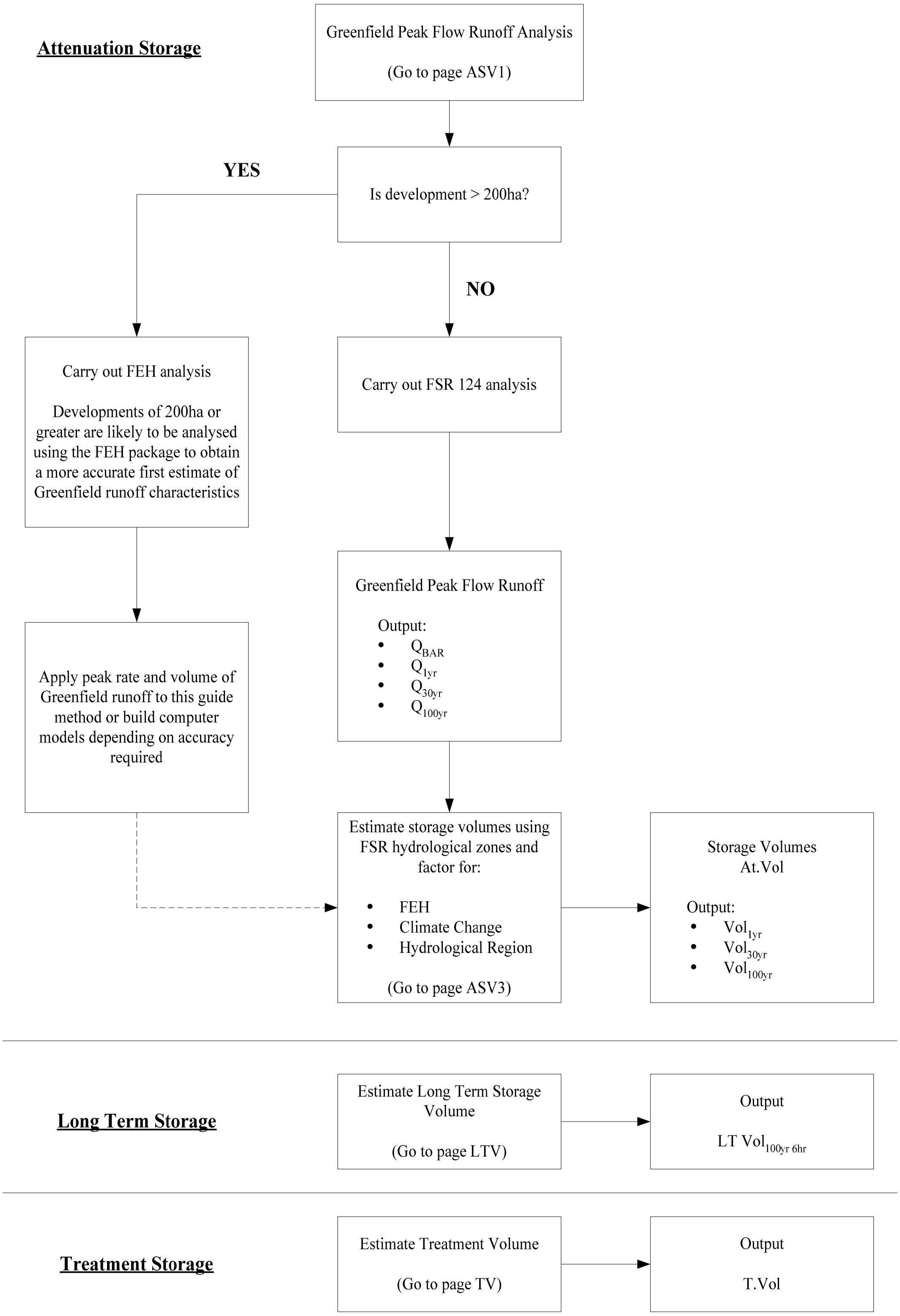


Figure 3.1 Flow chart for storage volumes estimation

3.1 Initial sizing of Attenuation storage volume

ASV1

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 1.1)

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

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. Method of Greenfield analysis

   If development area is 200+ ha a full FEH analysis is recommended to obtain a more accurate estimate of greenfield runoff characteristics.

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

6. Annual Rainfall (SAAR) [ ] mm — SAAR – use either SAAR from FSR or AAR from FEH (Appendix 1, Figure 4)

7. 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

8. Development mean annual peak flow ($1.08\left(\frac{A}{100}\right)^{0.89}$. $SAAR^{1.17}$. $SPR^{2.17}$) ($Q_{BAR} = Q_{BAR50\ hr}$ x (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)
9. 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)
10. Minimum limit of discharge ($Q_{throttle}$)		l/s	Minimum discharge (see note 3)
10.1 100 year flow rate per unit area ($Q_{throttle}$ /A)		l/s/ha	
10.2 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 9.
11. 1yr, 30yr and 100yr peak discharge rate of runoff per unit area			Use the larger of the 2 values of item 9 and 10.2 for calculating 11.1 to 11.3
11.1 Q_{BAR}/A x 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.
11.2 Q_{BAR}/A x GC_{30} Q_{30yr}		l/s/ha	The 30 and 100 year factors are found from Appendix 1, Figure 1.2 from FSSR 14. (Do NOT use the Growth Curve Factors from the embedded table in the figure).
11.3 Q_{BAR}/A x 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 (IH Report 126 – Hydrology of Soil Types)*

***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.*

Assessment of attenuation storage volumes

1. Hydrological Region	(R)	[ ]		Regions 1 – 10 for runoff growth factor (Appendix 1, Figure 1.1)
2. Hydrological rainfall Zone ($M_5 60$, r)	(Z)	[ ]		Zones 1 to 8 based on FSR rainfall characteristics (Appendix 1, Figure 2)
3. Development Area	(A)	[ ]	ha	Excluding large public open space which is not modified and drained by the development
4. Proportion of impervious area requiring Attenuation storage	(α)	[ ]		Impermeable area served by direct drainage / total area of impermeable surface. (see Note 1)
5. Greenfield flow rate per unit area	Q_{BAR}/A	[ ]	l/s/ha	From page ASV 2, use the larger of item 9 or 10.2. See also note 2 on page **LTV**
6. 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.
7. Attenuation storage volumes per unit area	($Uvol_{1yr}$)	[ ]	m^3/ha	Interpolate values based on PIMP and Q_{BAR}/A (Appendix 1, Figures 7.1 – 7.8) Use characteristics from item 2 ($M_5 60$, 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 1.1 (see note 2).
10. FEH Rainfall factor	(FF_{1yr})			Use factors based on the critical duration Q_{BAR}/A and PIMP (Appendix 1, Figure 11 and Appendix 1, Figures 6.1.1 – 6.3.4). (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 8.1 – 8.8 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
	(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 9). 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 1.2)
	(HR_{30yr})			
	(HR_{100yr})			
14. Final estimated Attenuation Storage				
Attenuation Storage Volumes	At. Vol_{1yr}		m^3	Required Attenuation Storage
	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 are providing part or all of the Long Term storage volume) can be assumed to be not contributing runoff for 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.*

Where pervious pavements are used (where a significant proportion of runoff will discharge at less than 2 l/s/ha), a reduction in the contributing paved areas can be made for assessing Attenuation storage volume. For the purpose of this simple method it should be assumed that all hard surfaces passing through such SUDS attenuation units (unless specifically controlled to a specified discharge rate of less than 2l/s/ha) are 25% effective. Therefore hard surface areas served by these units can be reduced by 25% for calculating Attenuation storage. Proof of compliance at detailed design will determine the actual Attenuation storage needed.

2 *The Defra guidance on the impact of climate change on river flows is to apply a factor of 1.2. As there is a non-linear relationship between rainfall and runoff it is suggested that a factor of 1.1 should be applied to rainfall depths in this procedure.*

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 (6.1.1 – 6.3.4) can then be used to determine the FEH rainfall factor.*

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

Refer to 1yr 12hr map (Figure 6.1.4) and determine the appropriate FSR:FEH ratio for the location of the development; lets assume 1.0. Assuming the development characteristics are:

Qbar = 4 L/S/ha, PIMP = 75, $M_5 60$ *= 14:03*

Then using figure 11 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 6.1.3). 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.

3.2 Initial sizing of Long Term storage volume

Estimation for 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 2l/s/ha. This is particularly applicable to catchments that are susceptible to flooding downstream of the proposed development, but should generally be complied with unless there are particular reasons for not providing this storage.

Discussion on the practical provision of Long Term storage is provided in Chapter 5.

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 · 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 10)
5. Rainfall depth	(RD)	☐	mm	Rainfall depth for 100 year 6 hour event (Appendix 1 Figure 3.1). Also see note 1.
6. Long Term storage volume (RD · LTF · AP)	($LTVol_{100yr\ 6hr}$)	☐	m^3	

Note 1 *All use of infiltration units in clay soils (type 3 and 4) should include high level overflows connected to the site drainage system. Suitable consideration should be given to their effectiveness in providing Long Term storage.*

2 *Where Long Term storage is being discharged directly to the receiving water at 2l/s/ha, the values for Q_{30} and Q_{100} for attenuation storage discharge 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 1.0l/s/ha, in which case 1.0l/s/ha Should be used.*

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

3.3 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 12)
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 2).
6. Treatment storage volume (T Vol) T Vol = $9A.M_560{\cdot}(SPR/2 + (1 - SPR/2). \beta\ PIMP/100)$	[ ]	m^3	Treatment volumes 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. Current best practice in UK (CIRIA report C522, 2000) requires between 1 and 4 times T.Vol to be provided. A minimum of 1.0 T Vol would normally be required unless other appropriate forms of treatment are proposed.*

2 *Hard surfaces being drained through infiltration units and pervious pavements can be deemed not to require treatment in terms of Vt for a pond. Vt needs only to be applied to impermeable surfaces with direct runoff. The volume for* β can be less than 1.0 to take account of areas of land surface deemed to have been 'treated' by upstream SUDS components.

3.4 Initial sizing of pipe networks

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. In practice pipe systems are still commonly used to provide the conveyance and drainage connectivity for a site. The reason for doing an initial design for a pipe network is to show the general connectivity arrangement and to check on pipe sizes and depths of the system.

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 simple rules 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 gradients by 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 construction issues. Tables for the capacity of pipes at all gradients are available from HR Wallingford. All pipe networks to be adopted should conform to Sewers for Adoption 5th edition.

If swales are used for conveyance, design is rarely a function of conveyance capacity. Issues such as velocity to protect against erosion and shape to allow easy maintenance usually define their size and prevent their use above a certain contributing area. Linear ponds are more flexible in this regard, by providing storage, infiltration and conveyance within the network.

3.5 Initial sizing and location of temporary storage for high intensity rainfall

During heavy rainfall up to 30mm of rain can occur in 30 minutes with bursts of intensities of up to 150mm/hr for short periods. Pipe networks can absorb rainfall in the region of 50mm/hr from impervious surfaces, but above this rate, rainfall will result in overland flows taking place. In addition to runoff being generated by impermeable surfaces, contribution will also take place from permeable surfaces (which are effectively impermeable under these rates of rainfall). Runoff from these pervious surfaces will generally be significantly delayed compared to those from hard surfaces so it is suggested that usually no allowance need be made specifically for this aspect. However, where it can be seen that a significant contribution from pervious surfaces is likely, due to the steepness of the area and its size, that a significant contribution is likely, some allowance for additional runoff should be made.

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 and phasing of housing construction.

The mechanism of runoff when gullies and pipework are overloaded is for the stormwater to run down roads to low spots. It is important to provide suitable holding areas at these points (rather than dwellings) to provide temporary retention of flood waters. It is relatively easy to identify these locations by examining the site contours and the site layout as flooding is usually channelled down the roads. However flooding across roads is possible and care should be taken to determine all possible flood flow paths. At the initial assessment, flood depths for possible volumes of temporary flooding can be based on assuming 5mm runoff from all impermeable surfaces from contributing uphill areas. However where there is a mix of SUDS units used, the initial evaluation of flood flows cannot be predicted using such rules of thumb.

Unfortunately flooding at certain locations can be exacerbated by the pipe system itself. Water can exit from gullies, thus concentrating floodwater in particularly 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.

4. DETAILED DESIGN REQUIREMENTS FOR STORMWATER STORAGE AND PIPE SIZING

This chapter details the procedure that needs to be followed for carrying out the detailed design of the stormwater system to serve a development. Unless agreed to the contrary, all design of stormwater systems should be modelled explicitly to demonstrate the performance provided by the proposed system.

4.1 Detailed design of stormwater storage

Figure 2.2 provides a flow chart of the detailed design of the stormwater system. Although the principles are the same as for initial design, there are some significant differences that are worth emphasising.

Stormwater storage can be provided as a range of distributed units and need not be a single pond unit at the downstream end of the development. If a single pond solution is proposed, it will generally be regarded as not providing the best practice approach to comply with the SUDS train principle. To establish compliance of the development proposals with the consent requirements for stormwater discharge, all elements of the drainage system will need to be modelled together with an accurate representation of the land use of the areas served.

Current software does not explicitly allow the actual performance of some SUDS units to be accurately represented, though this is likely to be addressed in time. This constraint does not prevent a reasonable approximation of the system, particularly as many drainage elements can be replicated well. This process will require head-discharge curves and pond depth-storage information to be known.

The runoff model used in this Guide for obtaining an initial estimate of stormwater storage assumes 100% runoff from hard surfaces and 0% runoff from pervious areas. These coefficients will not be used in any computer model, where the Wallingford Procedure variable UK runoff model would normally be used with appropriate choice of coefficients and parameter values.

Compliance to the criteria of 1, 30 and 100 year discharge limits then needs to be proven by running the model with a range of rainfall events of various durations for each return period.

The Treatment Volume will not need to be re-assessed (subject to any changes to the site plan), but provision of this element might be redistributed due to the revision of specific drainage elements in the design process.

Long Term storage, if provided as infiltration units, need not be specifically modelled to demonstrate compliance unless they are expected to contribute positive runoff during extreme events. However where it is provided as storage with a throttle to limit the discharge to less than 2l/s/ha, this should be modelled as part of the whole drainage system. Compliance should be demonstrated by showing that the total volume which is discharged when outflows are above 2l/s/ha for a 100 year 6 hour event is equal to or less than the calculated value of the greenfield runoff volume (see Section 5.7).

4.2 Future additional measure of compliance for drainage system design

The second stage of analysis as shown in Figure 2.2 is not applied or required by the Environment Agency at present. The reason for having this stage of analysis, for use at some time in the future, is that:

- Modern drainage systems now have a mixture of units each of which has a design performance based on various different rainfall conditions
- The events which cause peak flows from greenfield sites are not the same events as those used to prove compliance of the development system with limiting discharge requirements.

The use of design storm events provides an efficient way of designing a drainage system. However in order to confirm that runoff from developed sites is similar to that from the site in its undeveloped state, then it is necessary to compare the pre and post development response to the same set of rainfall events. This is not necessary for all events and would be very computationally inefficient to do so. A select series of extreme events would be suitable to demonstrate its performance in extreme wet conditions, while an annual series would test for the system behaviour under a "normal" range of conditions.

Theoretically there are two important issues to demonstrate. These are:

- that runoff from small events shows the same reduction in proportion of runoff (or no runoff) as the greenfield condition
- that the runoff from extreme events does not produce a greater risk of flooding from the receiving water.

The small events' responses will, in fact, normally be very difficult to replicate and the implications of not doing so can be considered to have limited consequences. Therefore it is unlikely that this criterion (the replication of all events) will ever be applied in practice.

The extreme event is clearly more important assuming provision for stormwater treatment has been made. It is suggested that several extreme events which have been recorded are used as a sample set of time series with which to measure the ability of the developed site to replicate the predicted response from the greenfield condition. There are several arguments that can be levelled against this proposal. These are:

- The events will not necessarily be the worse for that site system drainage configuration
- The return periods of the events may not easily be classified
- The events would not occur at that location as they were measured elsewhere
- The greenfield response can only be approximately modelled.

These assertions are all true, but it is the only approach which will provide a good indication of the level of competence of the designed drainage system for protecting against catchment flooding. As this is the focus of this test, the type of event to be used should be those that caused flooding in rivers rather than local flash flooding events caused by short thunderstorms. Examples of events that meet this criterion are those of

November 2000 and the end of December 2002. At this stage there is no definitive methodology agreed yet for this type of analysis.

The final stage of analysis, as summarised in Figure 2.2, is to compare the hydrographs (their volumes and flow rates) in a manner which is simple, but effective in evaluating the characteristics of the site runoff.

4.3 Detailed design of stormwater networks

The detailed design of the stormwater networks, including all non-pipe features such as swales, should comply with the Sewers for Adoption 5th Edition criteria for:

- No flooding at 30 year event
- Consideration of extreme events (normally 100 year event) for flood routing.

As with the design and detailed assessment of storage, a complete representation of the drainage system would normally be made, using the Wallingford Procedure runoff model. There is extensive standard guidance and documentation on drainage design and therefore this is not repeated here.

Sewers for Adoption 5th edition does not specify the return period of what constitutes an extreme event. It is important to consider events of 100 years or greater and take a risk based approach to check on the consequences of any "failure" that might take place.

There is one aspect which needs to be highlighted with regards to SUDS. Pipework which serves SUDS units that is to be adopted by the Sewerage Undertaker will need to be designed on the basis that all hard surfaces contribute runoff in the standard manner even if it is attenuated or reduced in volume by SUDS components. This is a precaution which is being taken at this stage to ensure that long term failure or change of drainage practice in the future will not result in flooding due to pipe capacities being overloaded which will require future modification to the network. Exceptions to this rule will need to be agreed specifically with the adopting authority.

5. TECHNICAL ISSUES

This chapter aims to provide more detailed information to those who would like to understand more about the current status of rainfall runoff with respect to drainage and gain a better knowledge of the basis of the methods being applied.

It is important to stress that the method used here to provide an initial design for the various elements of storage has sacrificed some degree of accuracy at the expense of providing a very simple tool for all to use. It is emphasised that detailed design of stormwater storage should not use this tool (though applying the same principles), but be carried out using computer tools which enable a holistic approach to all elements of the site drainage to be considered.

5.1 Runoff assumptions and criteria used

An explanation of the runoff assumptions and criteria used 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 6.

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 usefullness discussed.

5.1.1 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" 5th ed. (2001) 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 5.1, 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 14, 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 5.1 and 5.2 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 3.1 and 3.2 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 5.1 Parameters used in the New PR equation for Figure 5.1

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 5.2 Rainfall events used in Figure 5.1

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 5.3 100 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

5.1.2 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:

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

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.

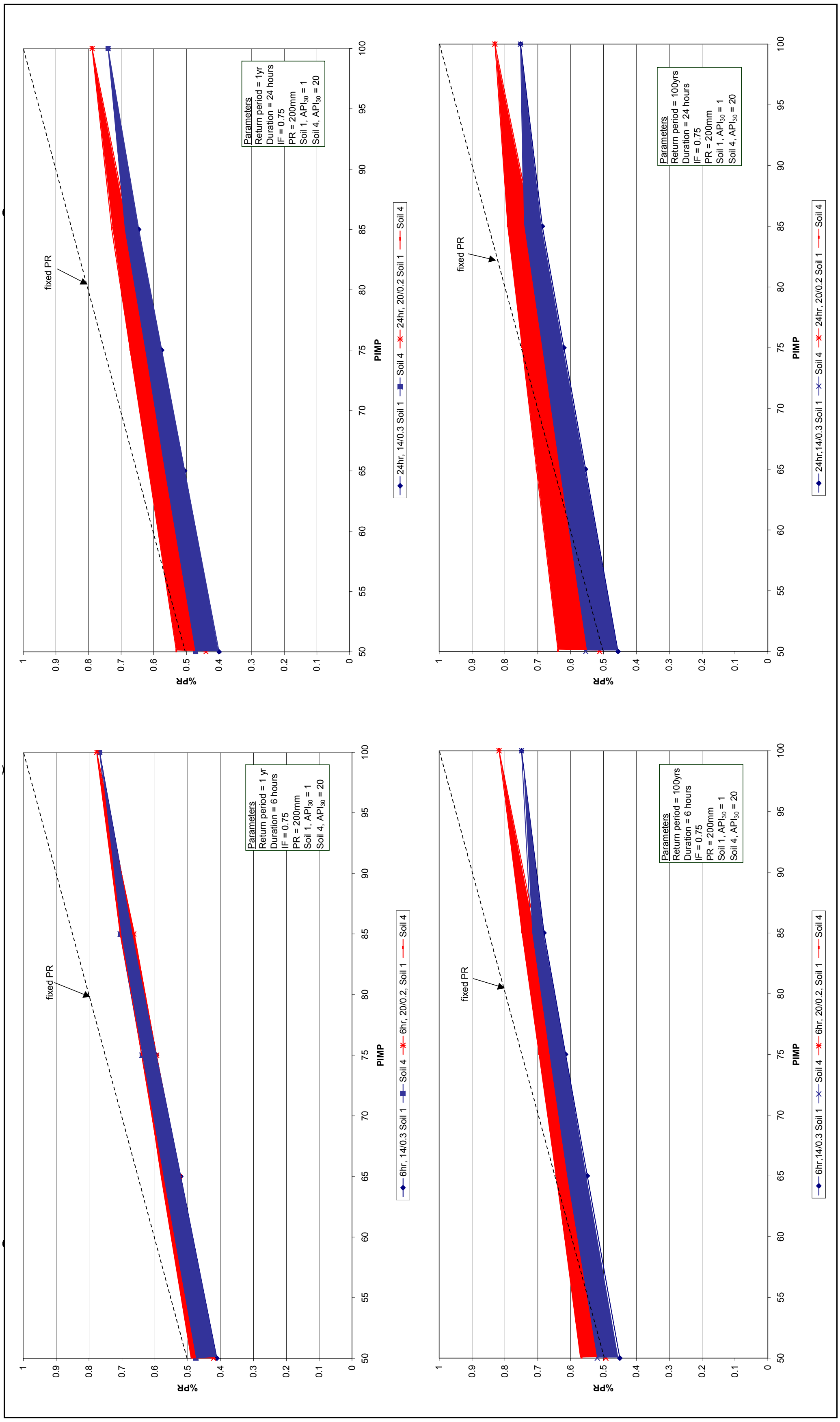


Figure 5.1 Comparison of PR between the variable Wallingford Procedure runoff model and Sewers for Adoption

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 5.4 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.

Further discussion of these equations is covered in the documents referenced at the beginning of this section.

5.1.3 Statistical Percentage runoff models – rural

There are statistical runoff formulae for rural runoff in both FEH and FSR for use with hydrographic analysis of catchments. Section 5.7 provides details of the FSR runoff formula used for an assessment of percentage runoff from a greenfield site which would be used to derive a more detailed assessment than the simple assumption made in this Guide of using SPR.

5.1.4 Statistical peak flow estimation models

There are two equations for predicting peak runoff rate from a catchment other than those available in FEH. These are usually referred to as the ADAS 345 formula (1980) and the IH report 124 (1994) method, which is an extension of the FSR work carried out by the Institute of Hydrology (now CEH, Centre for Ecology and Hydrology). A brief summary of both these methods is provided although only method IH124 is advised for use with this Guide. As with the previous sections, the appropriate documents should be referred to for additional information.

There is debate in the industry as to which method of assessing greenfield runoff is best to use. The advice to use the FSR approach is not based on whether it is better (or worse) than ADAS for small catchments. It should be recognised that:

- both methods only give an approximate estimate of the likely greenfield peak runoff response
- ADAS requires interpretation of the permeability function S_T
- ADAS is based on fewer catchments and aims to evaluate flows for land drainage purposes and is not aimed at predicting extreme runoff conditions
- The FSR formula is simpler to apply.

It is felt that for the purposes of providing a consistent easy to use method, the FSR formula is the most appropriate tool. To cater for the stipulation of a lower limit of 50ha, all flow calculations for sites smaller that 50ha are carried out assuming an area of 50ha and then linearly interpolated. This provides a conservative (lower) estimate of greenfield runoff for these sites.

The ADAS 345 method
The Agricultural and Development Advisory Service (ADAS) report number 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)
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). 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 1.2) is unlikely to give exactly the same answer as the 10 year curve from the ADAS nomograph.

Table 5.5 summarises all the parameters used.

Table 5.5 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 (Appndix 1, Figure 1.1). 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 Figure 4)

Flood estimation for small catchments (Institute of Hydrology report no. 124)
The 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 index found from the Flood Studies Report soil maps or the WRAP map of the Wallingford Procedure

The 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.

5.2 Methodology used for deriving storage volumes assessment

It is important to provide a summary of the approach used to derive the data in the various graphs and figures which have been produced for this Guide.

5.2.1 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 5.1 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 9) 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.

5.2.2 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 5.3 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 6.1.1 to 6.3.4). 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.

5.3 Provision of Long Term Storage

Long Term storage is to be provided to cater for the additional runoff caused by the development compared to the volume that would have been contributed from the site in its greenfield state. In principle it needs only to be mobilised for the longer 100 year storms. In practice this is very difficult to achieve so it tends to be designed to come into effect for all storms for all or part of the site. However if Long Term storage can be mobilised in the form of temporary flooding during extreme events, this would be equally suitable. It should be noted that although the 100 year 6 hour event is used to define the storage volume needed, the type of events which require this volume to be mobilised (to protect rivers during flooding) are generally much longer, low intensity events. It is likely that the only mechanism available for achieving this is to allow flooding from the downstream pond, when full, to spill to an adjacent location. Careful analysis at detailed design would be needed to demonstrate the effectiveness of such a proposal.

The intention of Long Term storage is to only allow the volume equivalent to greenfield runoff to discharge at greenfield rates, while retaining the rest of the runoff to lower rates so that the river downstream is protected from flooding. This can be provided as a mix of infiltration and very slow release of stored runoff. However this complicates what is intended to be a simple concept and a simple initial volume assessment. To provide a simple criterion, this storage volume is based solely on the 100 year 6 hour event. The selection of the 100 year return period is related to fluvial river flooding criterion from PPG25, while the 6 hour duration is a relatively arbitrary criterion, albeit based on providing protection primarily for the smaller streams and rivers where critical rainfall durations are around 6 hours. This should also provide significant benefits for larger catchments.

This volume must be provided either as infiltration storage or as storage with the ability to be discharged at a rate of less than 2l/s/ha with an equal reduction in Q_{30} and Q_{100} for discharges from Attenuation storage. The latter is generally less satisfactory, and has practical difficulties in limiting flows without using orifice sizes which are considered to be too small and at risk of blockage. However it is recognised that SOIL types 3 and 4 are not generally suitable for infiltration. Developments on such soils, therefore, would normally require some form of overflow facility to be built into the infiltration units to prevent flooding in long wet winter periods.

It should be noted that infiltration units are normally only designed to 10 year rainfall. However, assuming that adequate capacity is provided, Appendix 1, Figure 10 can be read in reverse to determine the minimum area that needs to be designed for infiltration. Thus for a development with a PIMP of 50% of SOIL type 1, there is a need to have at least 35% of the hard surfaces draining to the infiltration system. This rises to 85% for a totally paved development.

The alternative of providing attenuation storage such that the additional runoff volume discharges at less than 2 l/s/ha is more difficult. Firstly, the minimum area that could be served by a simple throttle unit of a 150mm diameter pipe (which has a discharge of 13 l/s) is around 6.5 ha (to limit the discharge to 2l/s/ha). Secondly, it would be very difficult to assess whether a mix of swales, pervious pavements, attenuation pond and other units will result in the additional runoff volume being discharged at 2 l/s/ha or less.

The unique nature of each site with its arrangement of drainage units and their performance will result in different event durations being critical. For simplicity it is felt that as the criterion is based on the 100 year 6 hour event, that this event should therefore be used for demonstrating compliance with this requirement and show that:

- At preliminary design an equivalent volume in the form of infiltration units and 25% of pervious pavement storage has been provided
- At detailed design that the volume discharged from the site when flows from the site are greater than 2 l/s/ha is not more than the estimated volume generated by the greenfield condition.

This discussion does highlight the need for the second part of Figure 2.2 to be implemented to show the development really complies with a 'near greenfield' response.

5.4 Ownership of SUDS

The issue of SUDS ownership and the responsibility for maintenance of these units is a subject that is causing some difficulties at present in the industry. This document is not the appropriate forum for discussing the legal position of drainage responsibilities and the current stance being taken by the various relevant organisations. This is all summarised in the draft consultation SUDS Framework document issued by the SUDS national working group. This section is provided to draw attention to those who are less familiar with this ongoing debate, that it is important to address the subject of adoption and ownership of all drainage features which are proposed to be included in new developments.

5.5 Current software limitations

Computational drainage software has developed over the last 20 years to being able to analyse pipe-based systems with great capability and accuracy. However it is important to draw attention to the current limitations of these tools as there are implications for detailed design requirements of stormwater systems and their ability to be able to predict the actual performance of the system.

There are three main international drainage packages, as well as a number of lesser known products which are used for detailed evaluation of drainage networks. The three main packages are Micro-drainage, InfoWorks, and MOUSE, although this last package is not commonly used in UK. The comments below are known to be true for the InfoWorks package, and are also likely to be true for the other two products. However where it is important to obtain clarification on packages, the relevant software houses should be approached.

The subject areas of interest are itemised as follows:

1. Gully capacity
2. Overland flow
3. SUDS units representation
4. Runoff models.

Gully capacity
The linkage between overland runoff (from paved and other surfaces) and receiving networks is via conceptual routing models which do not actually represent the actual physical processes. Thus runoff simply "enters" the pipe system with a suitable delay to account for the flow time above ground. This is perfectly reasonable in most circumstances and has been proven in practice to provide accurate results for predicting system performance.

However, the ability of gullies to convey water into the drainage network is limited for extreme events so where models predict all the water passing into the pipework, in practice, it often passes on down the road. Thus there are situations where flooding can take place that are not predicted by the model.

Overland flow
Overland flow in designing drainage systems has only recently become a stated requirement. Overland flood flow modelling has been carried out for a number of years by representing the road network as a secondary drainage system. Although roads can be represented with reasonable accuracy, the issues related to model stability require very careful attention, as it is not unusual to derive spurious values from such models. In addition, the representation of flood storage depths at low points in the development, are far from ideal and may not relate to the topography that actually exists.

The use of 2D models to simulate overland flow, which is not constrained by the road network, is extremely rare at present. Due to the density of developments, this is often not an important consideration, but it is necessary to be aware of the possibility of flows passing across roads rather than being constrained to flow down them.

In summary, although models may improve in their ability to accurately represent overland flows, care must always be taken in evaluating the possible flow paths and predicted flooding when using these tools.

SUDS units representation
The representation of SUDS units in models and being able to replicate their hydraulic behaviour ranges from relatively good through to fairly poor. It is important to be aware of the model approximations being used in assessing their predicted drainage performance.

Runoff models
There are a number of runoff models that have already been described. However even the variable UK Wallingford Procedure model, generally regarded as the best at present, is considered to be deficient for applying to SUDS where the soil saturation is an important element of their performance. Research to produce a new runoff model is currently being carried out by CEH and should provide an improvement that will enable the analysis of SUDS systems to be carried out more effectively.

5.6 Treatment of surface water

The simple approach of providing treatment storage (Vt) is only one of a number of SUDS features which provide stormwater treatment. The amount of storage for treatment should take into account the use of these other SUDS features which are proposed to be used on the site.

5.7 Greenfield runoff volume

At detailed design stage, the effectiveness of the provision of Long Term storage should be demonstrated by showing that the volume of runoff for the 100 year 6 hour event above 2 l/s/ha is equal to or less than the greenfield volume of runoff. The use of FEH techniques is regarded as best practice, but this estimate of volume can also be easily calculated by using the formulae provided in FSSR16. Although this estimate is better than the simple assumption of percentage runoff being equal to SPR, it should be recognised as still being an approximate estimate for any given site. The volume of the runoff per unit area from greenfield areas can be calculated from the product of the rainfall depth and the percentage runoff. The percentage runoff for a site (PR_{RURAL}) is given by the following equation:

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

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 5.2.

$DPR_{CWI} = 0.25 (CWI – 125)$

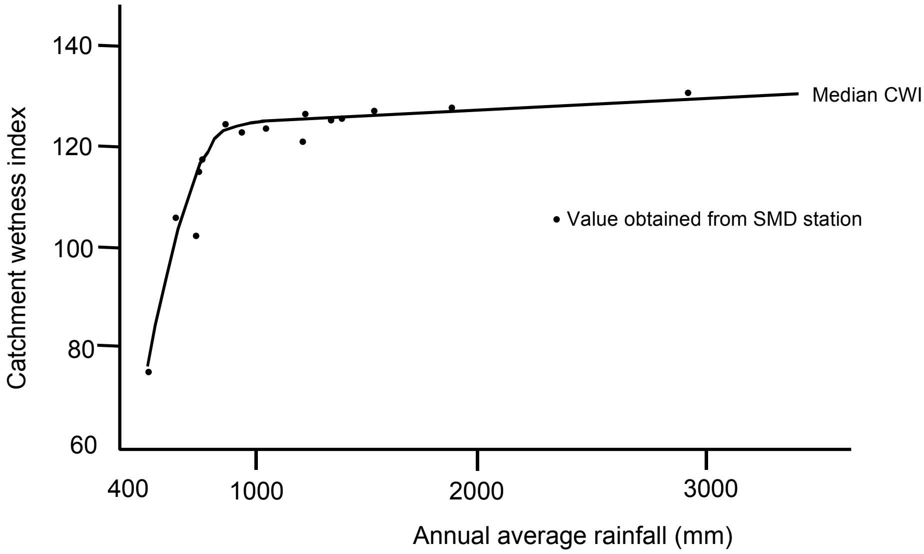


Figure 5.2 CWI vs SAAR – Flood Studies Report

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.

6. 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.

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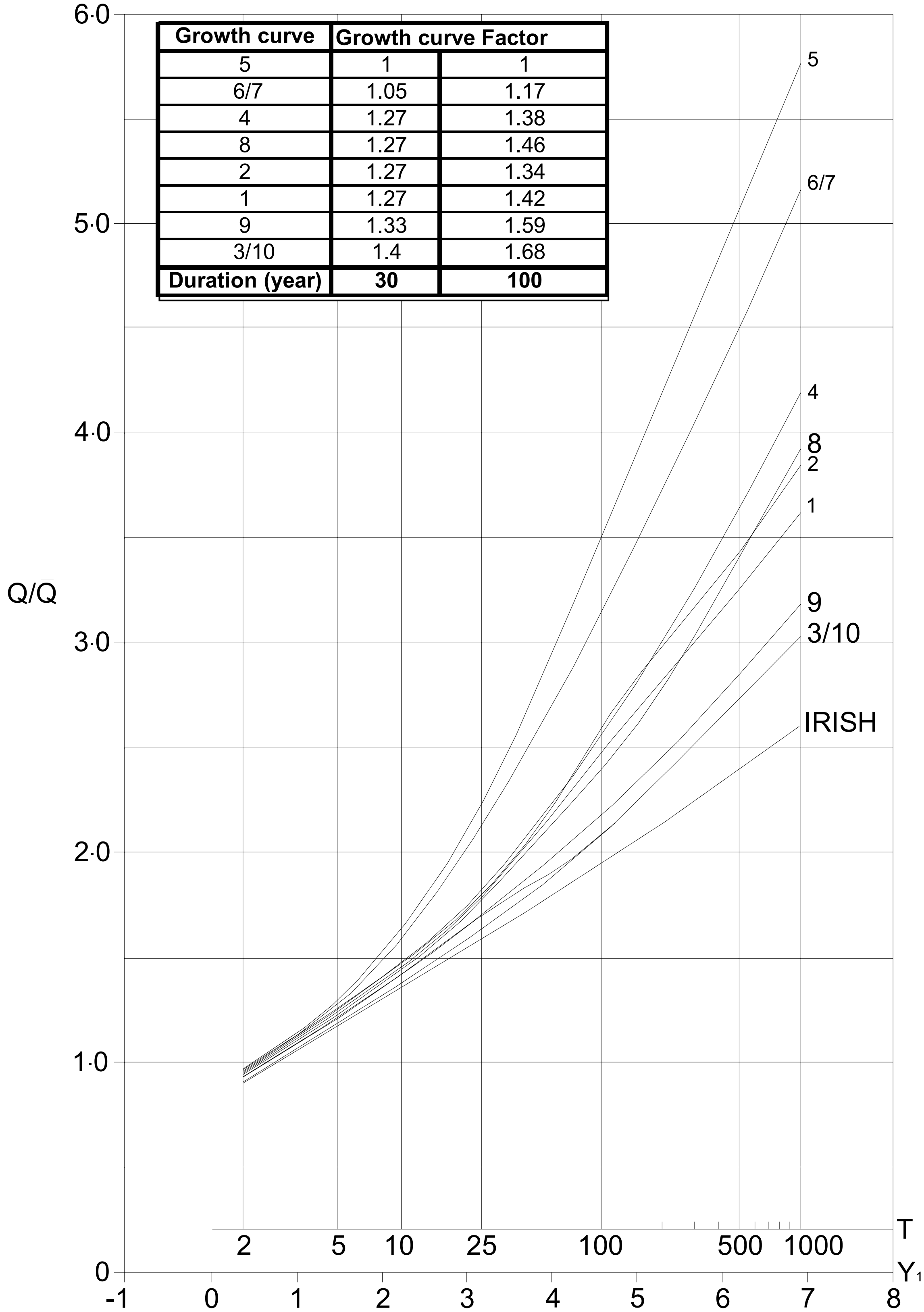
Water UK 2001, Sewers for Adoption 5th edition, A design and construction guide for developers published by WRc

APPENDICES

Appendix 1

Figures and graphs

Figure 1.1 Hydrological regions of UK



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 1.2 Peak flow growth curves of UK (from FSSR 14)

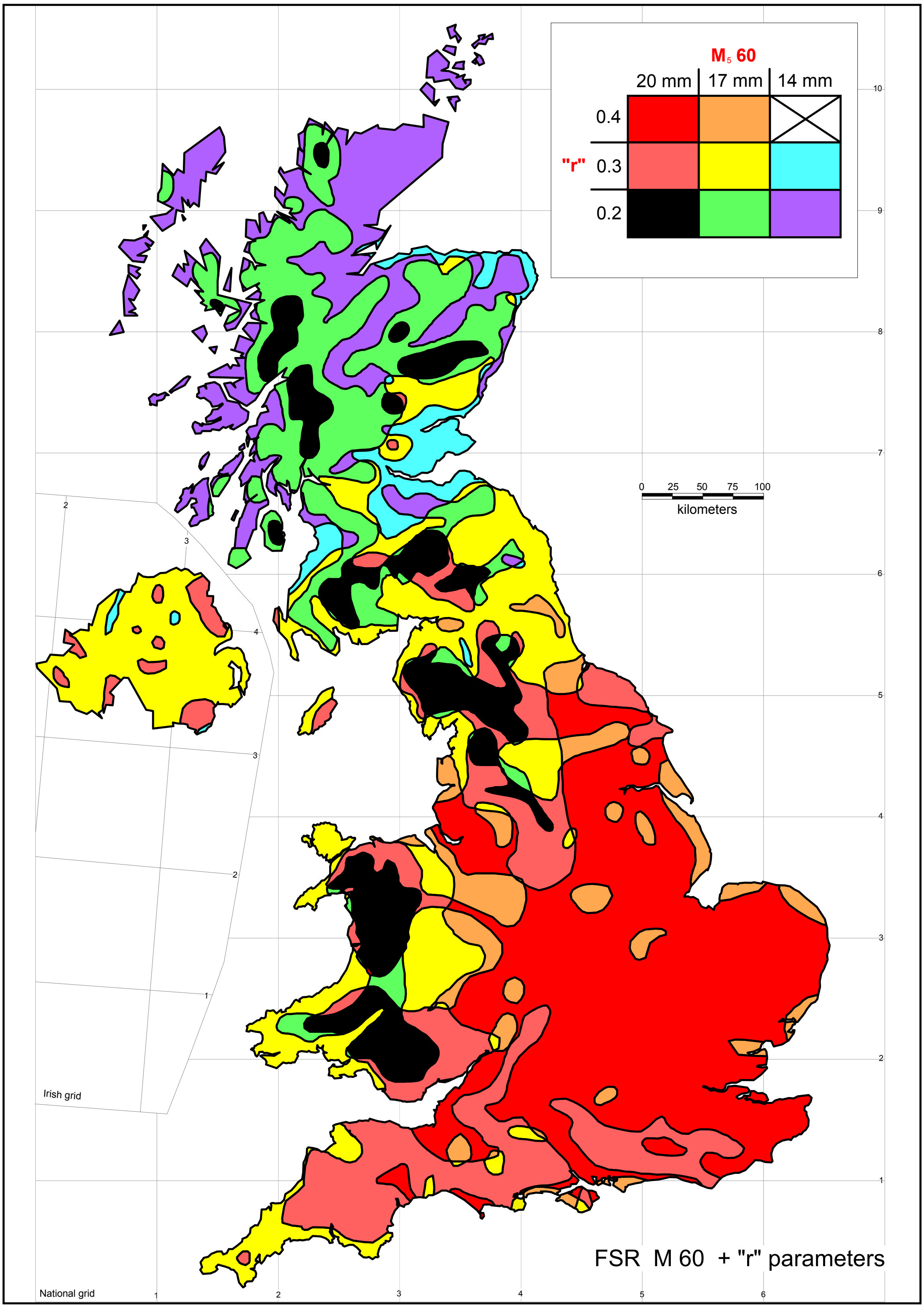


Figure 2 Hydrological rainfall zones of UK

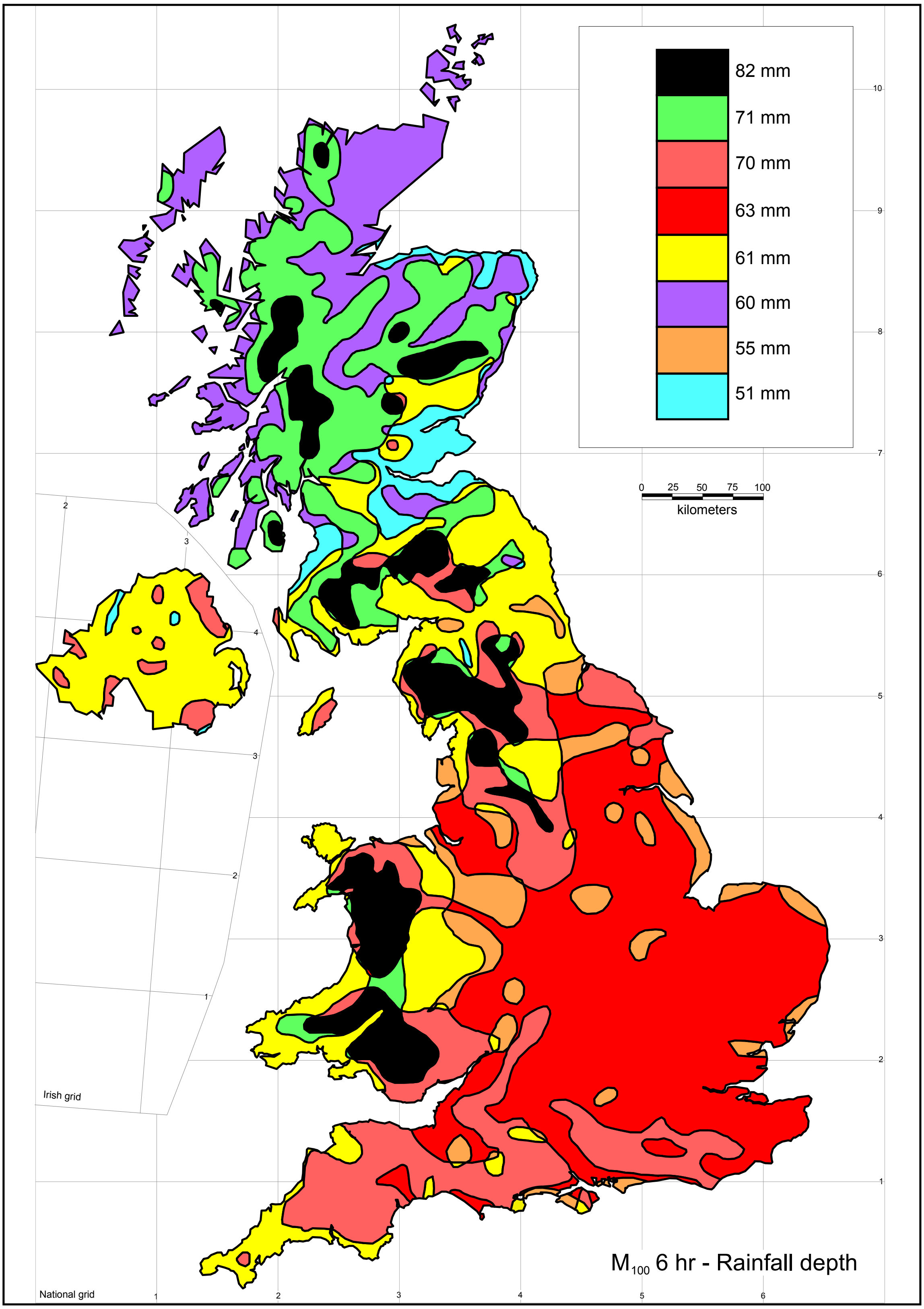


Figure 3.1 100 year 6 hour rainfall depths of UK

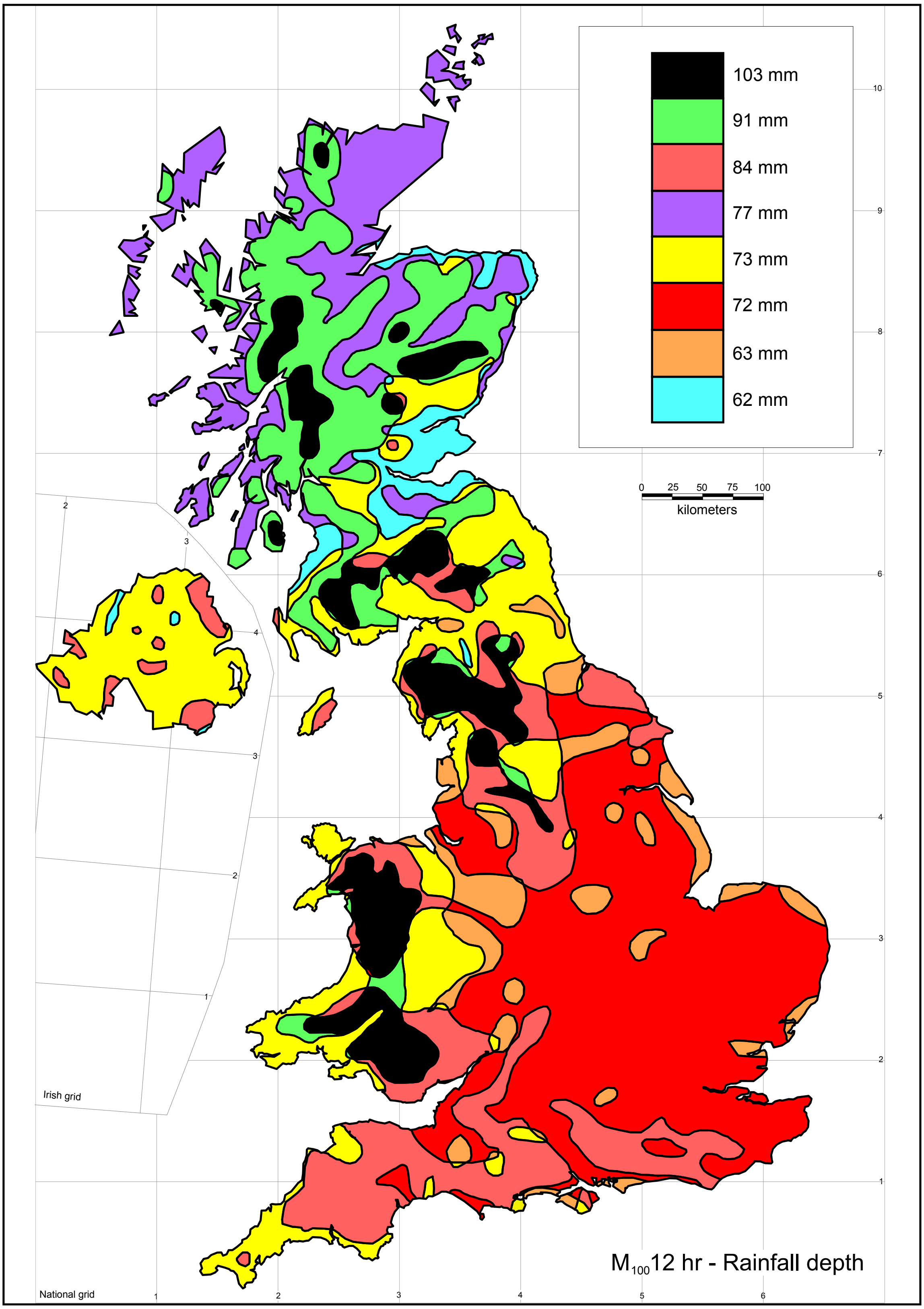


Figure 3.2 100 year 12 hour rainfall depths of UK

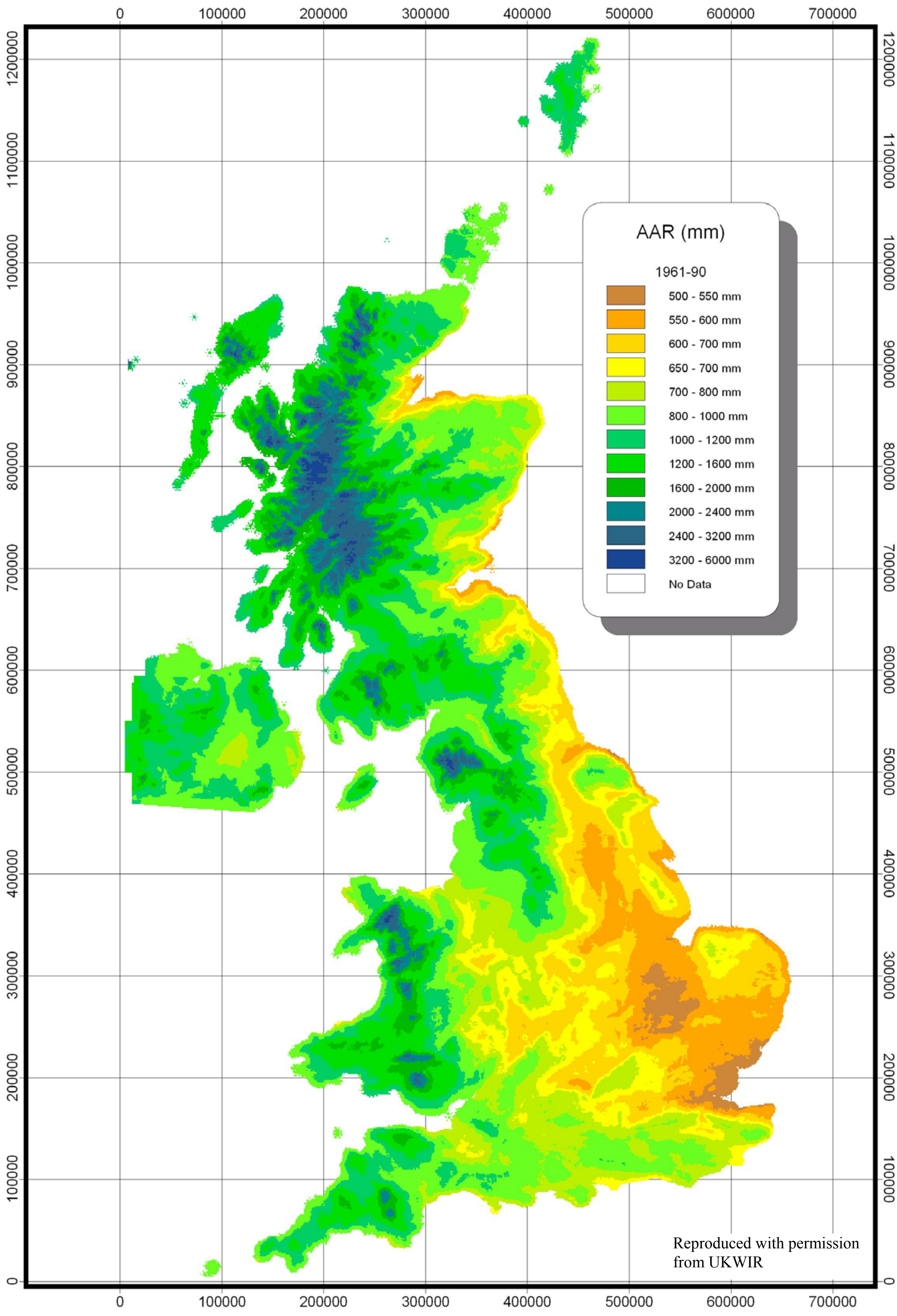


Figure 4 Average annual rainfall (1961 – 1990) (from FEH)

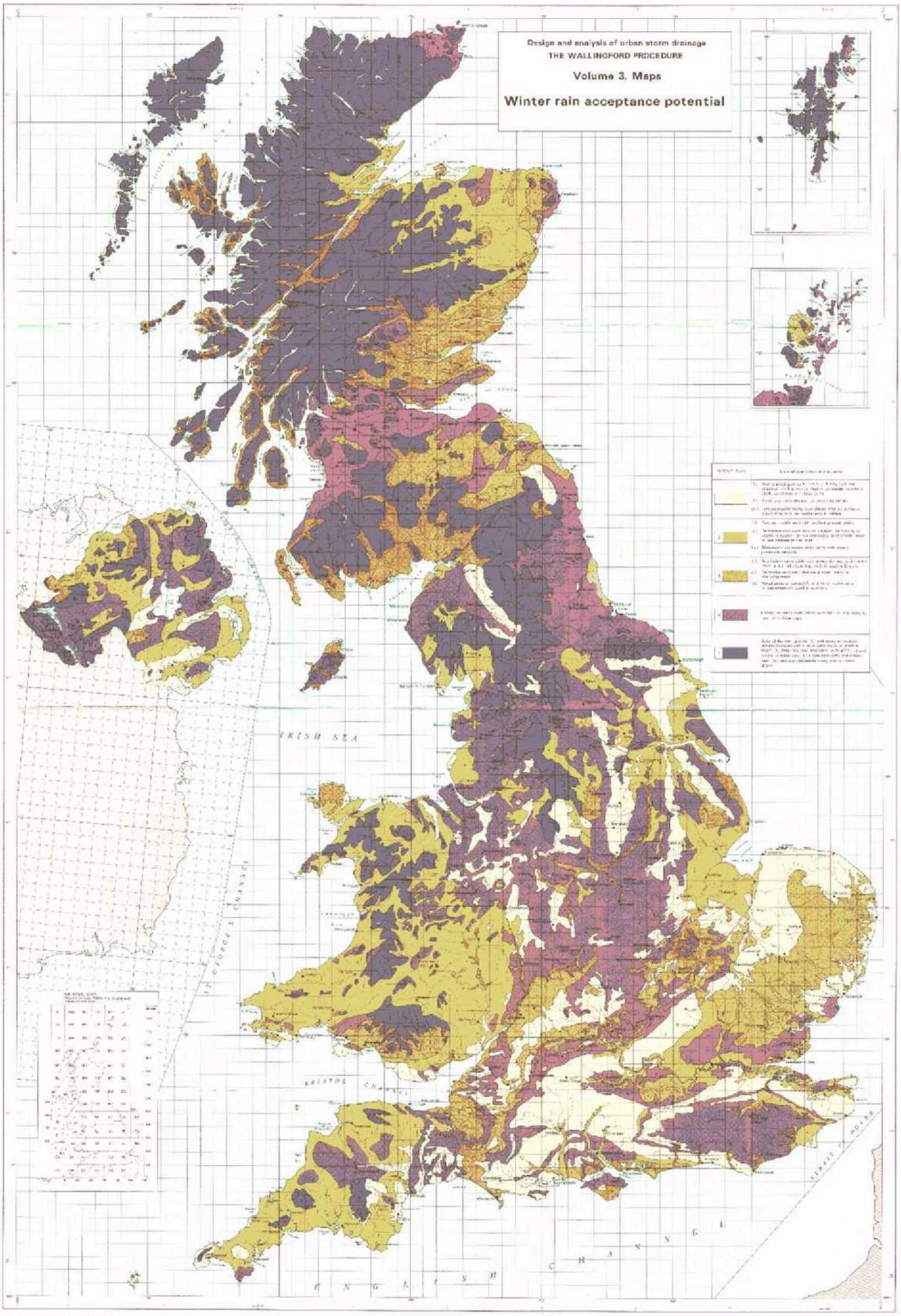


Figure 5 WRAP map of SOIL type from the Wallingford Procedure

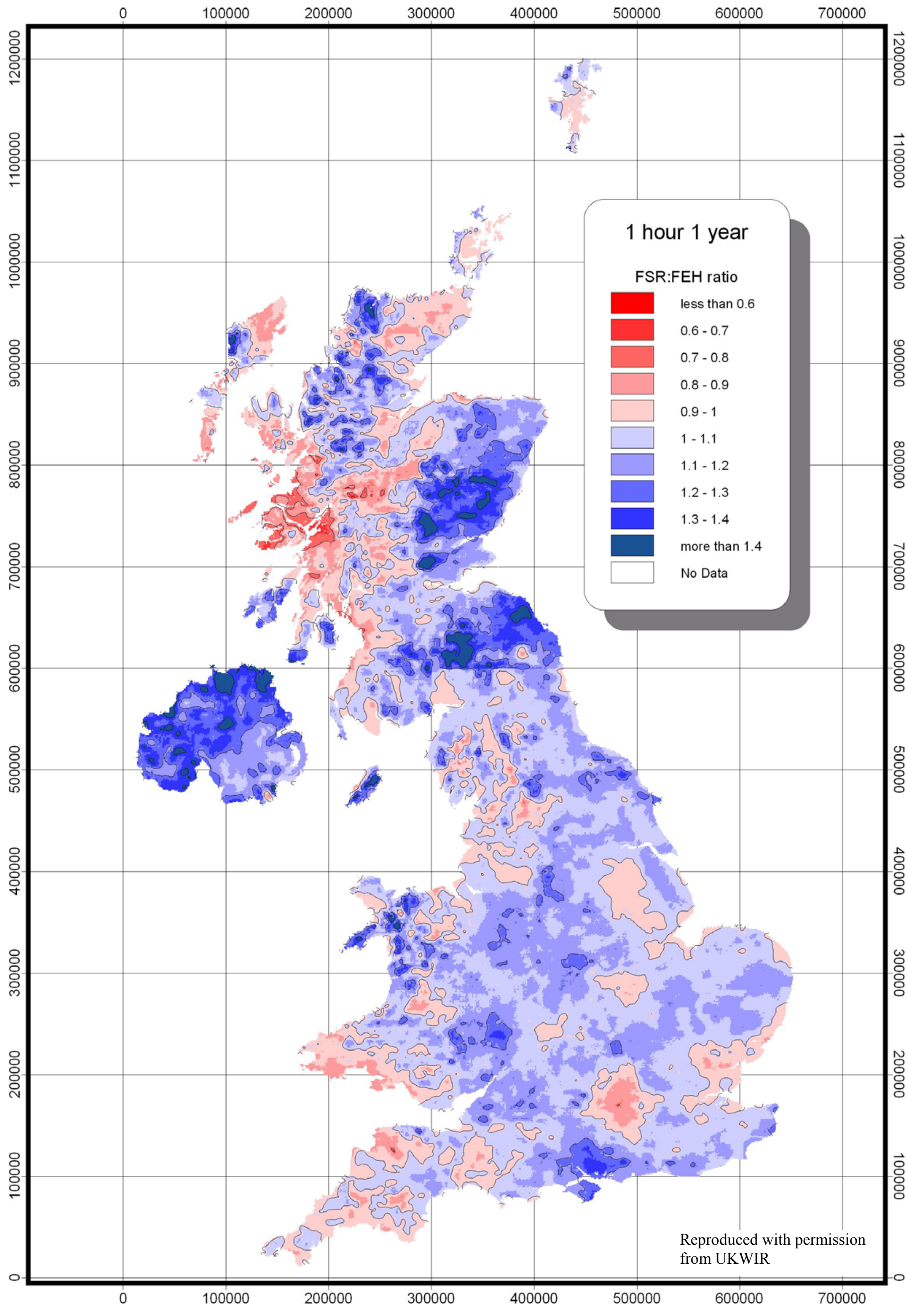


Figure 6.1.1 FSR/FEH rainfall depth ratios

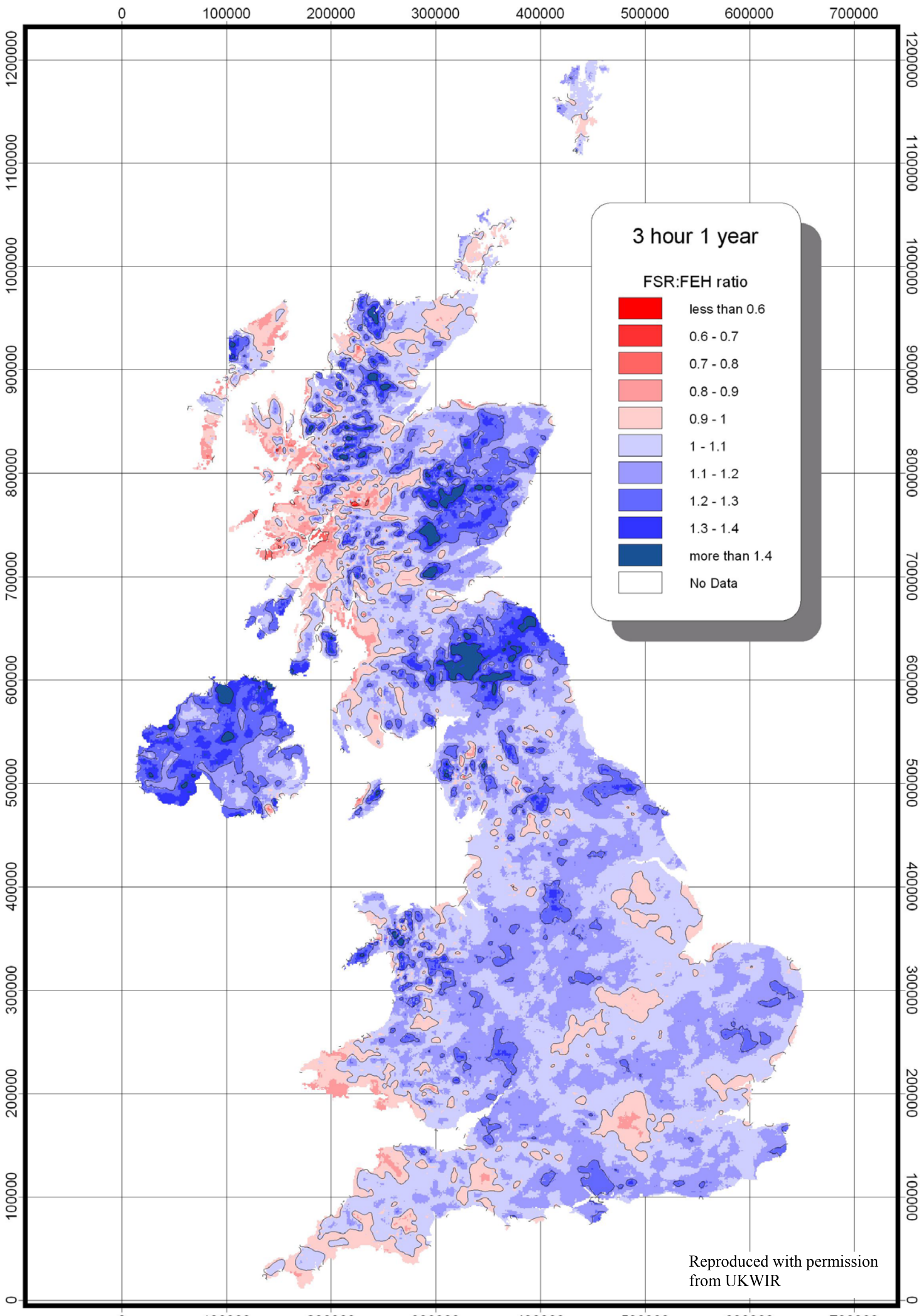


Figure 6.1.2 FSR/FEH rainfall depth ratios

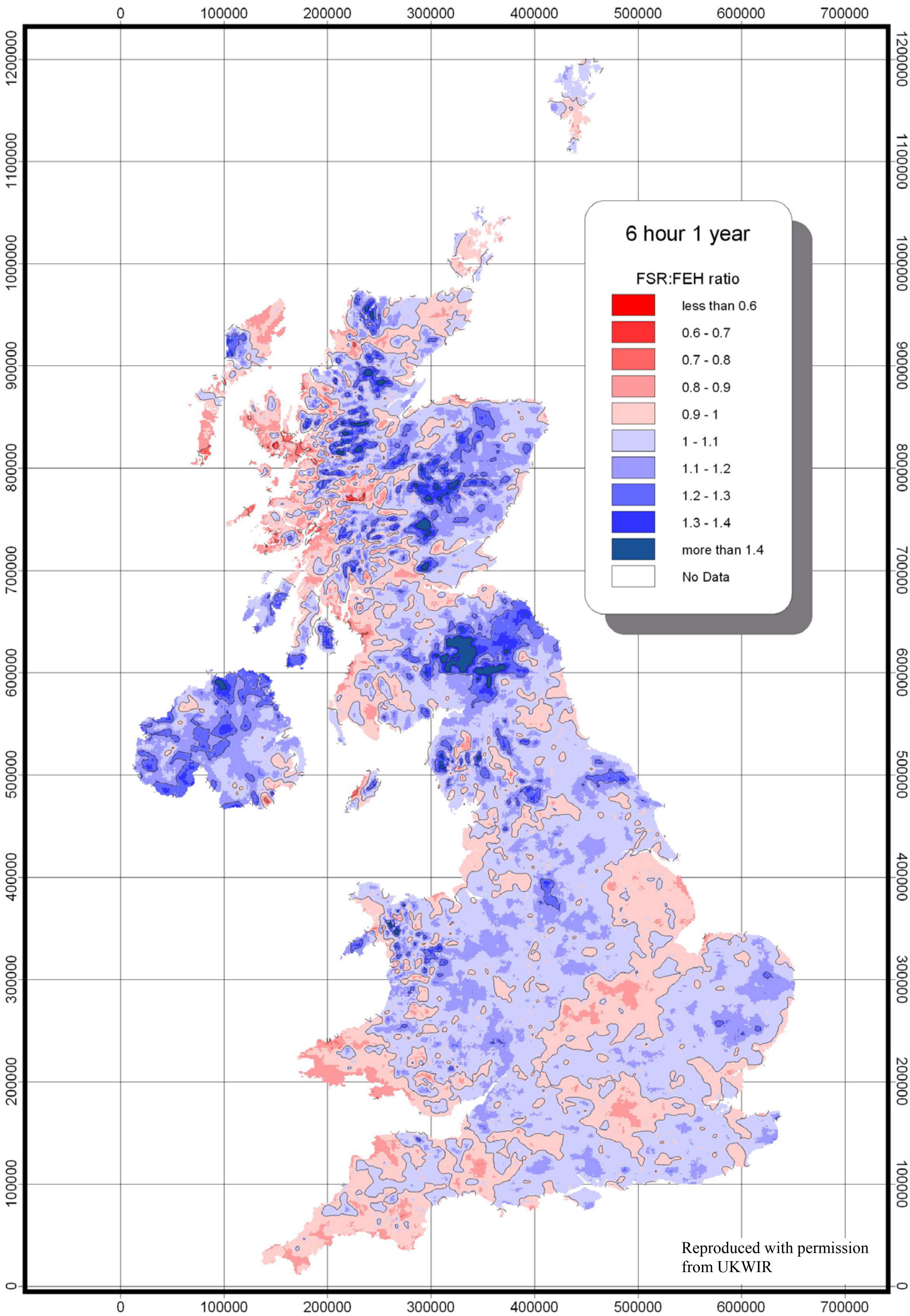


Figure 6.1.3 FSR/FEH rainfall depth ratios

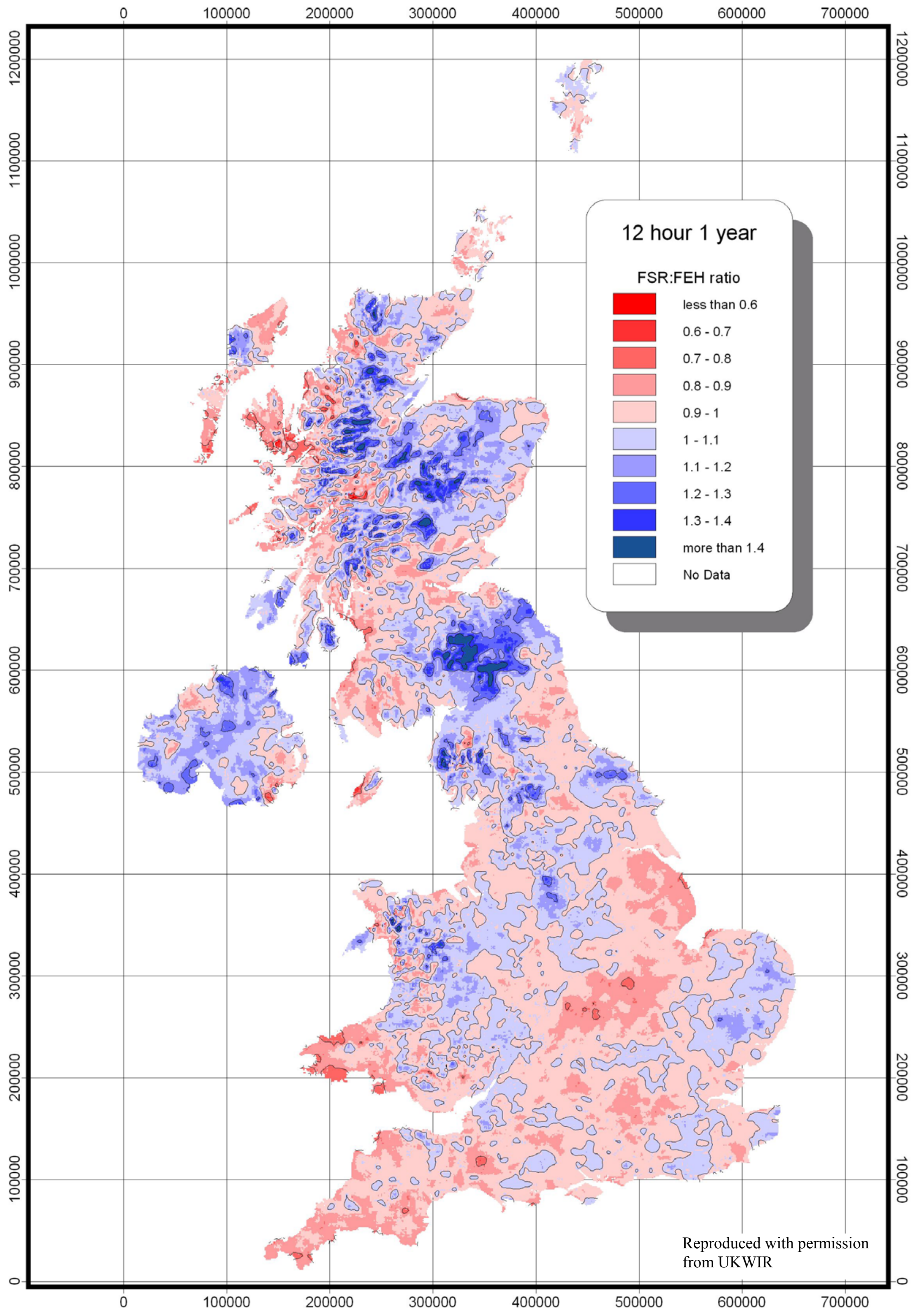


Figure 6.1.4 FSR/FEH rainfall depth ratios

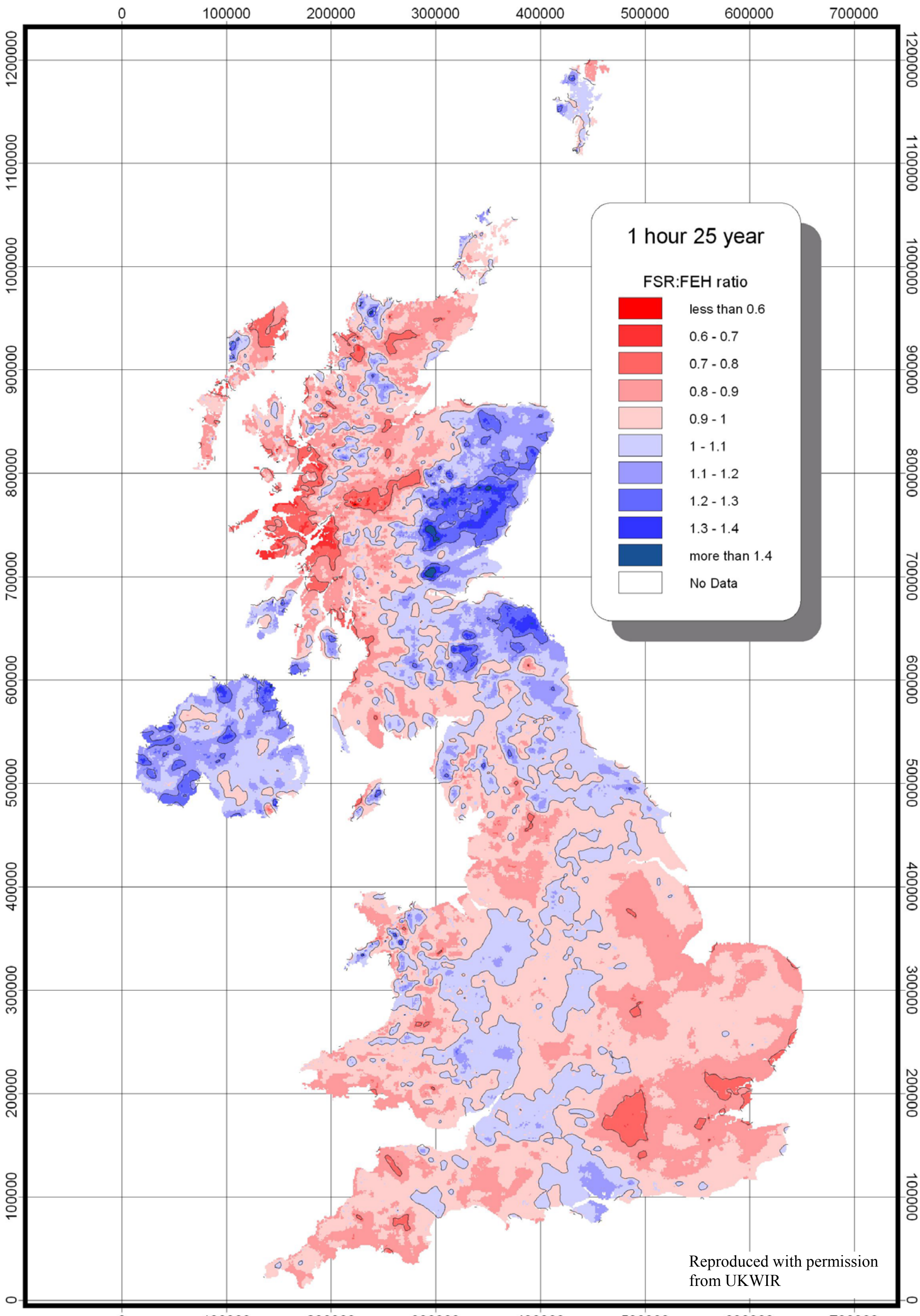


Figure 6.2.1 FSR/FEH rainfall depth ratios

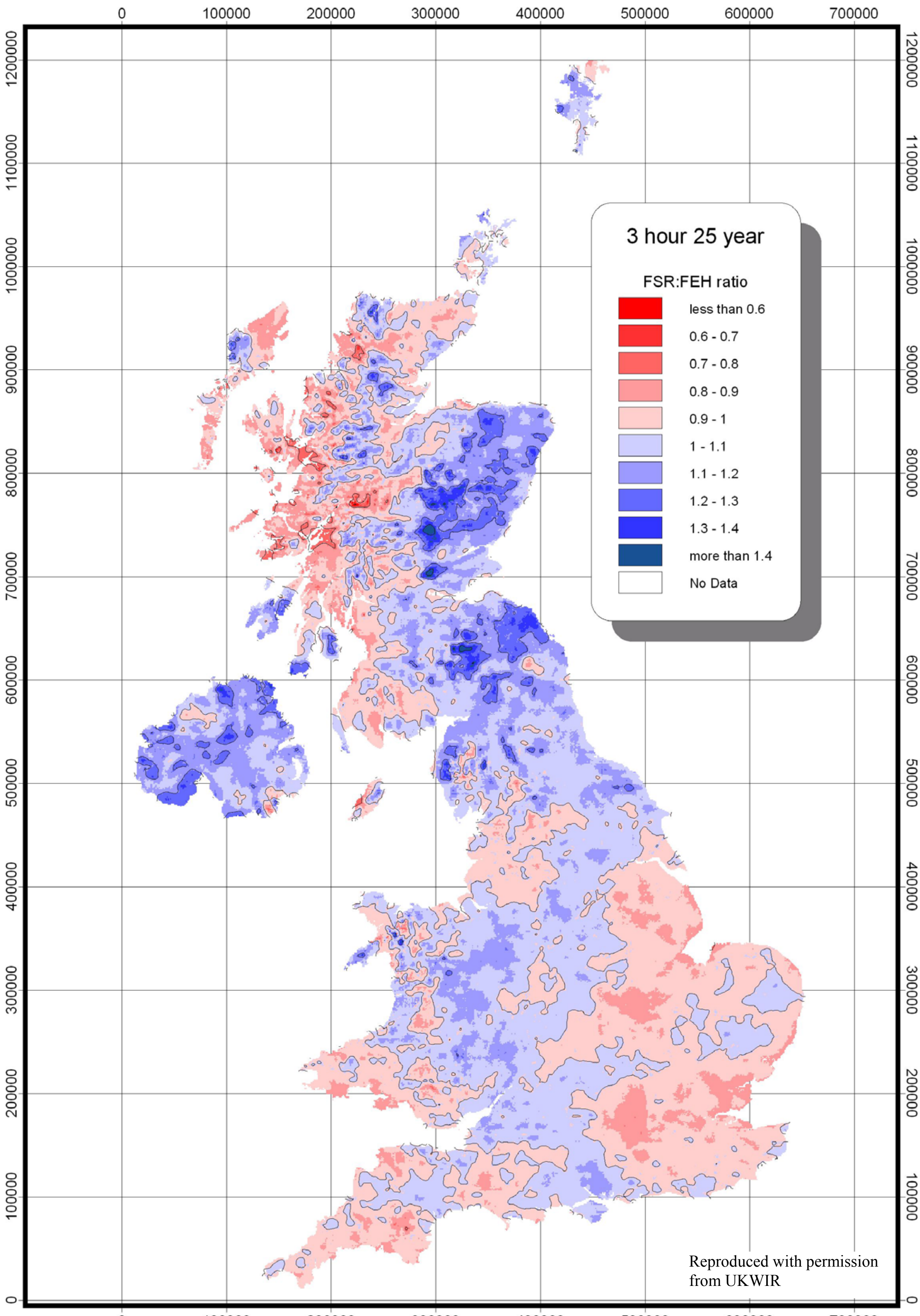


Figure 6.2.2 FSR/FEH rainfall depth ratios

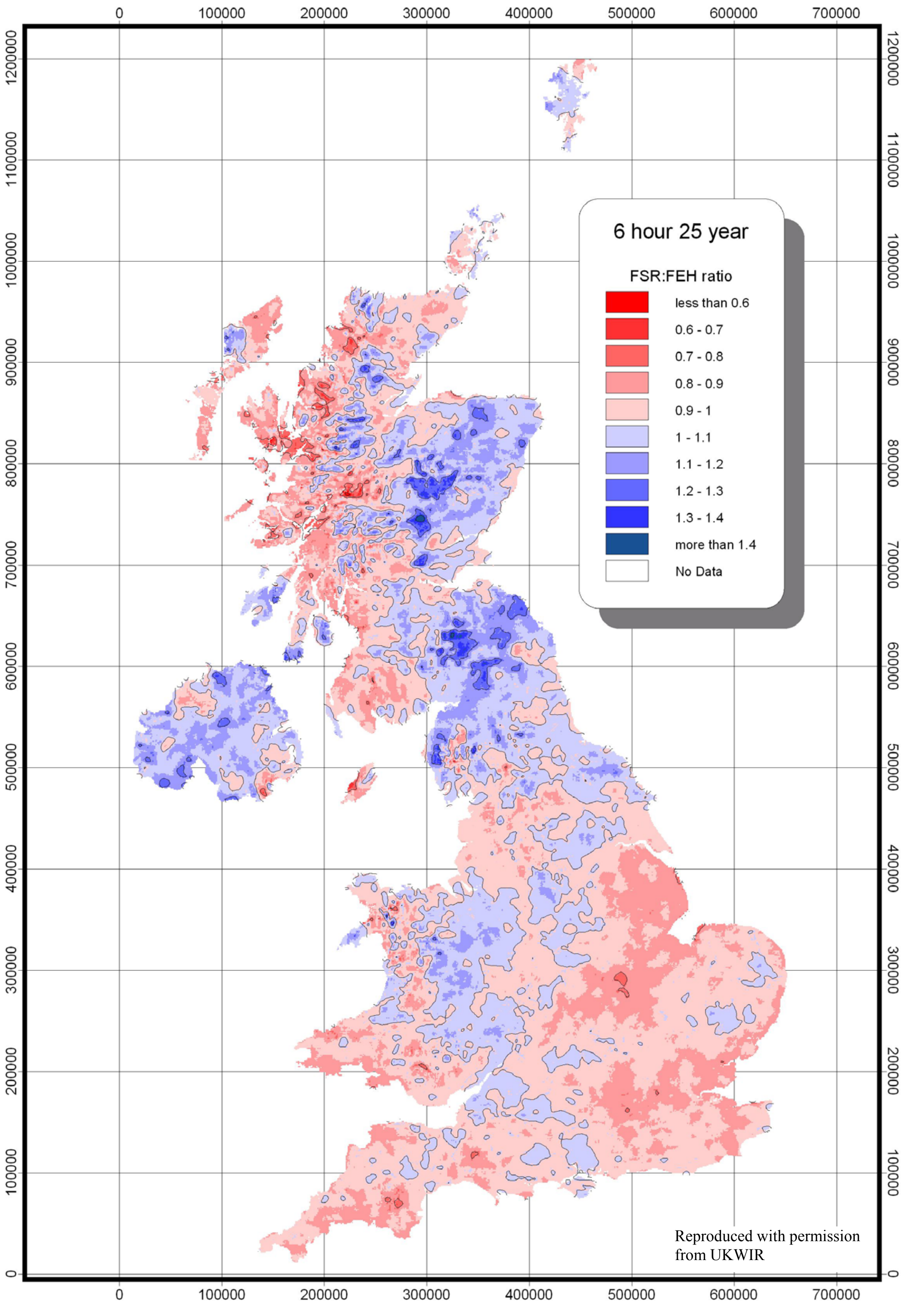


Figure 6.2.3 FSR/FEH rainfall depth ratios

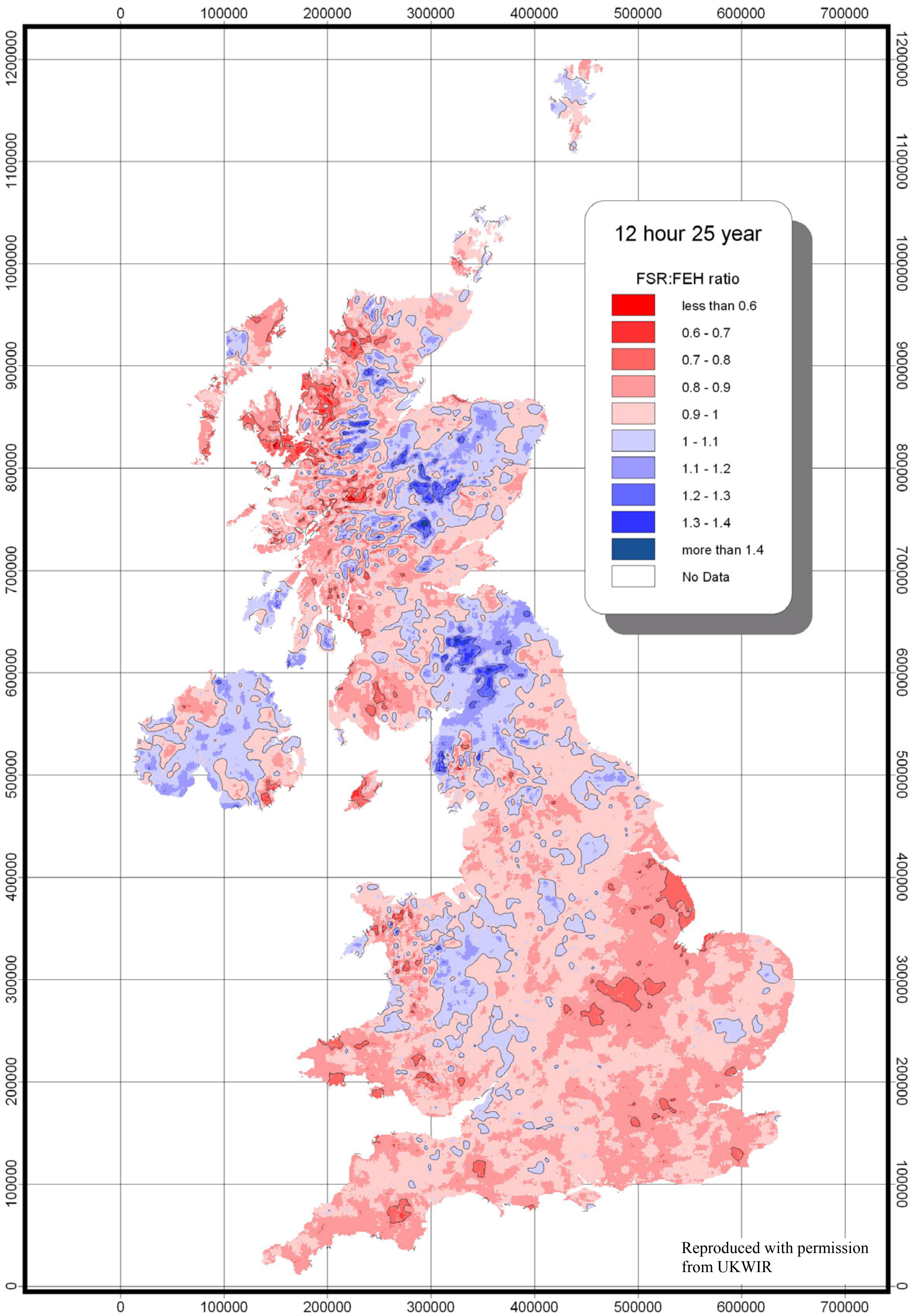


Figure 6.2.4 FSR/FEH rainfall depth ratios

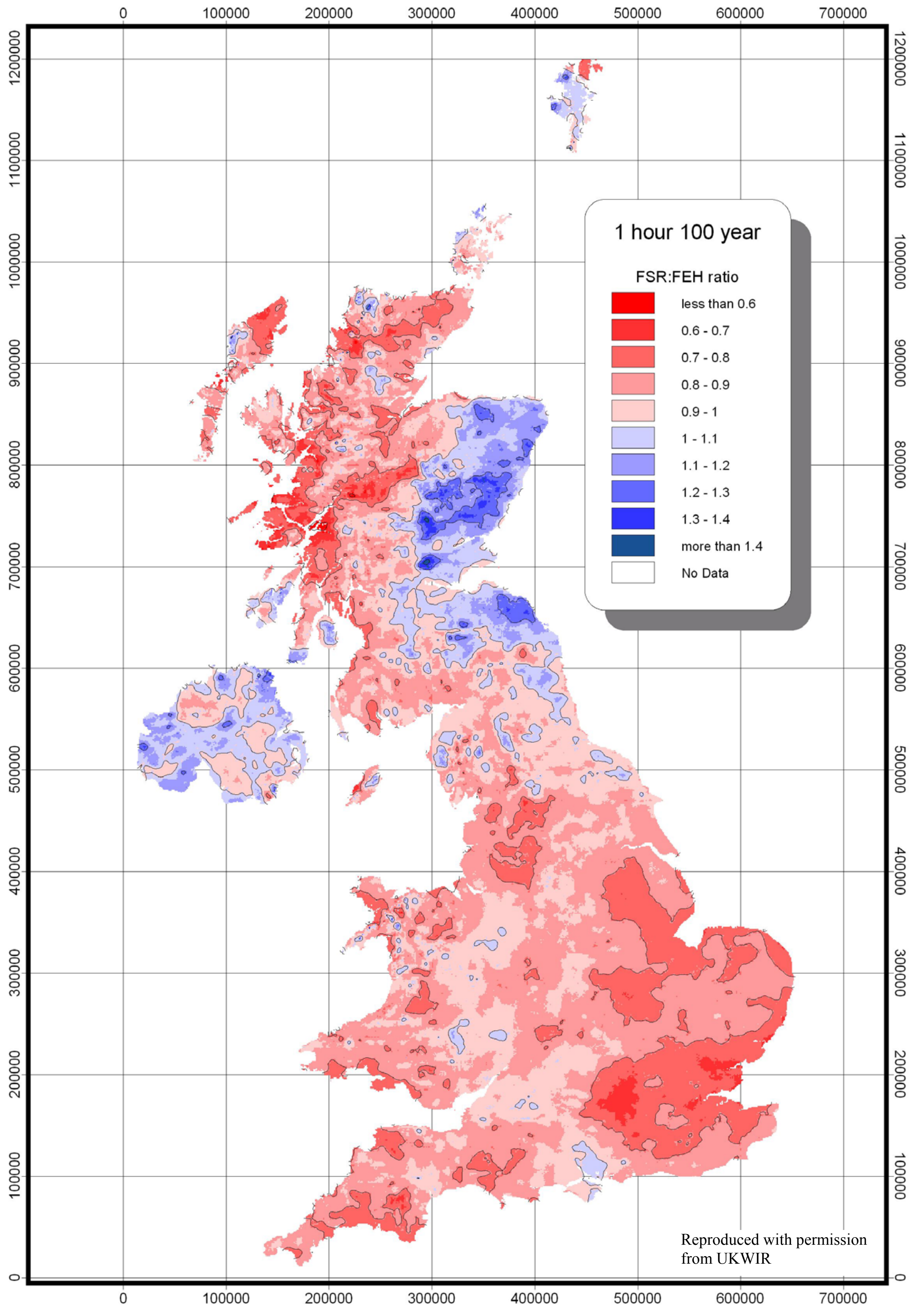


Figure 6.3.1 FSR/FEH rainfall depth ratios

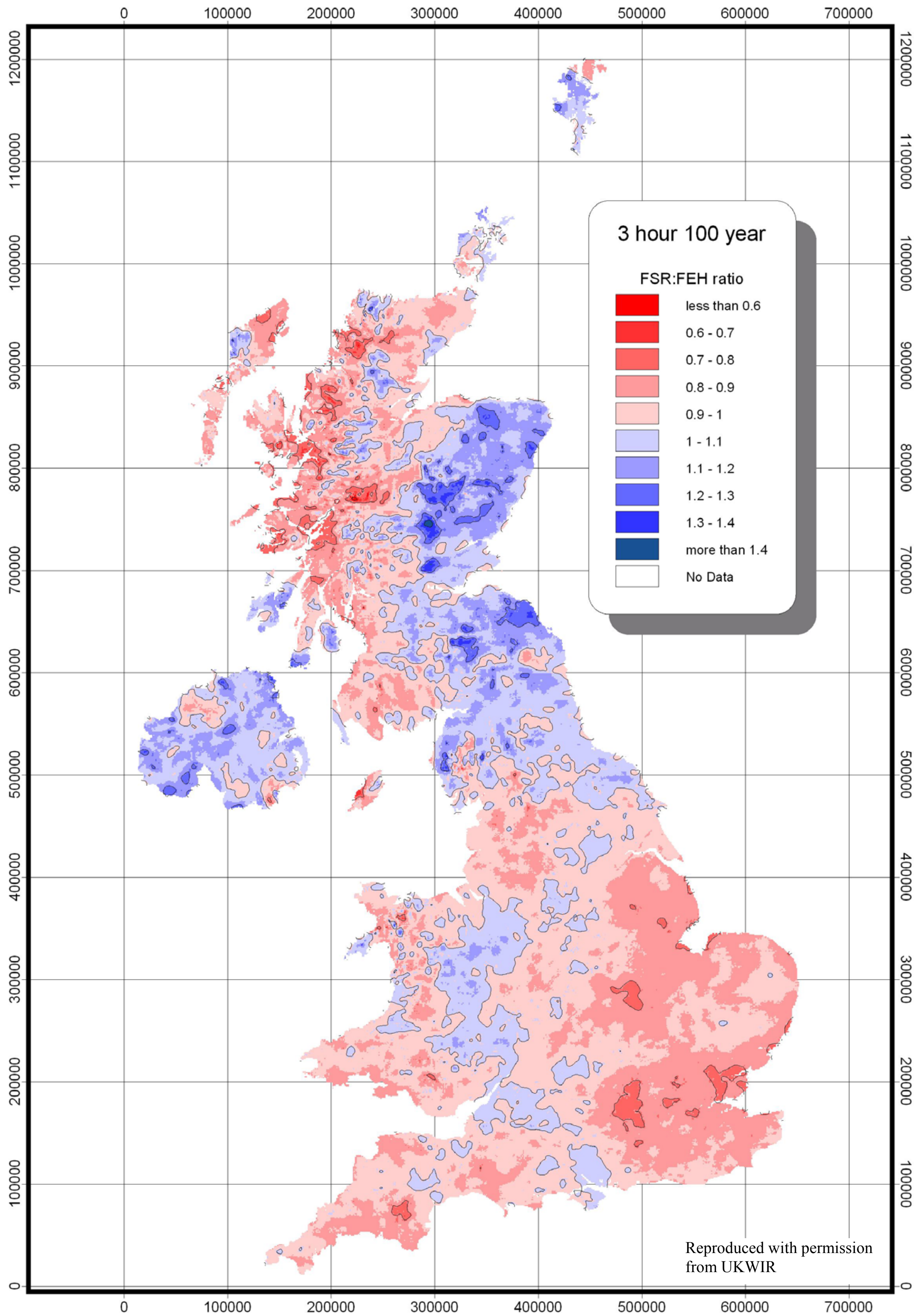


Figure 6.3.2 FSR/FEH rainfall depth ratios

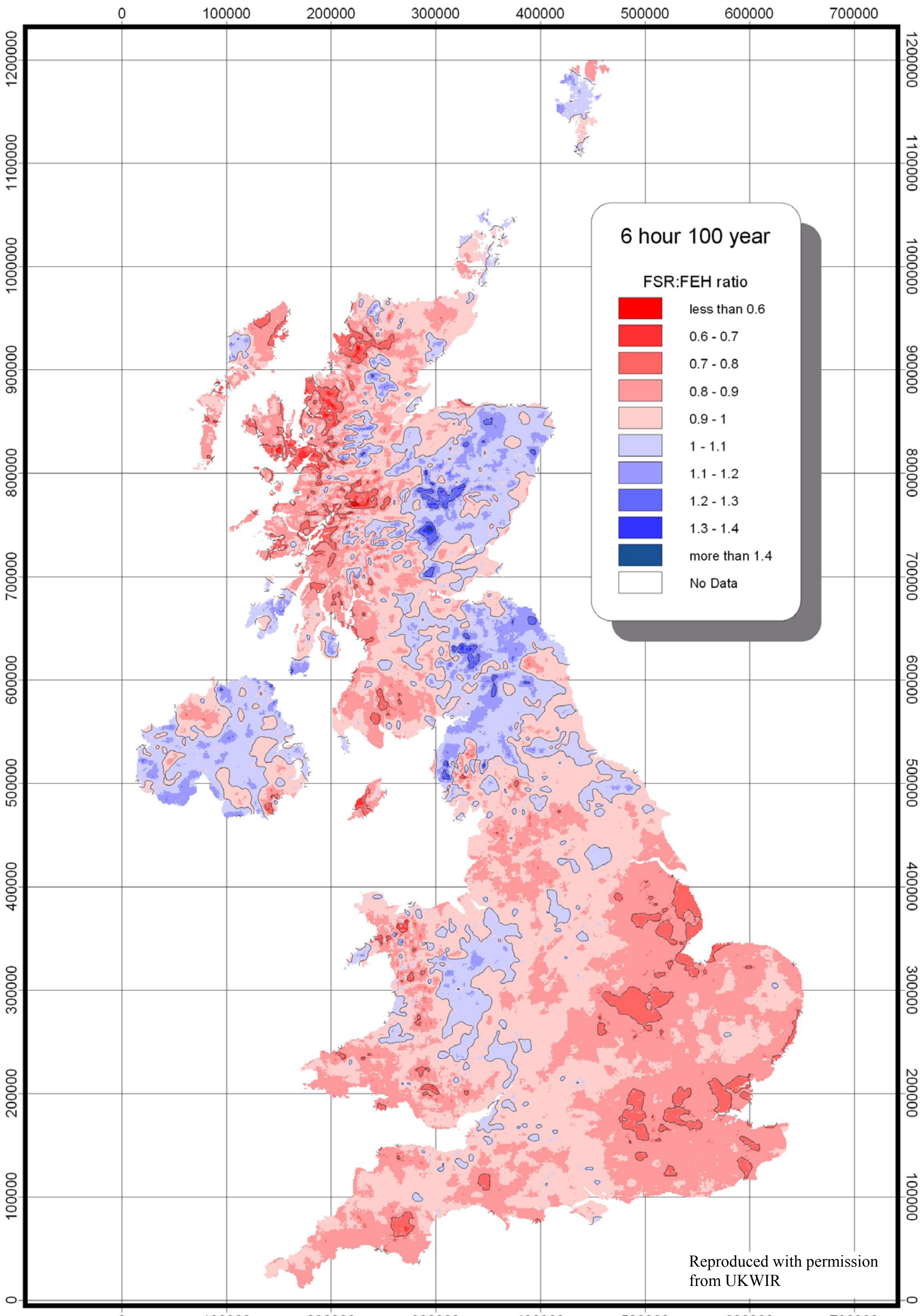


Figure 6.3.3 FSR/FEH rainfall depth ratios

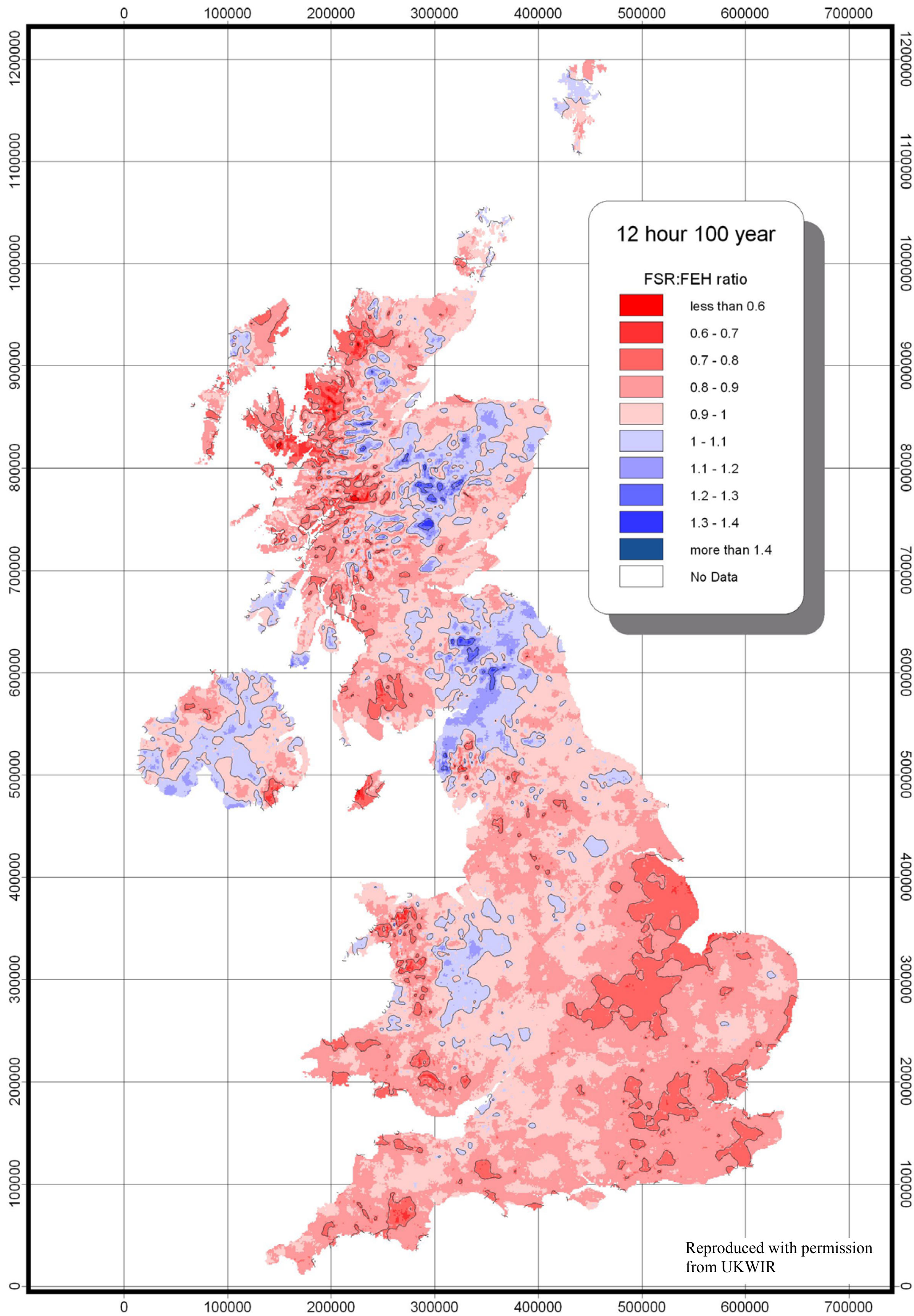


Figure 6.3.4 FSR/FEH rainfall depth ratios

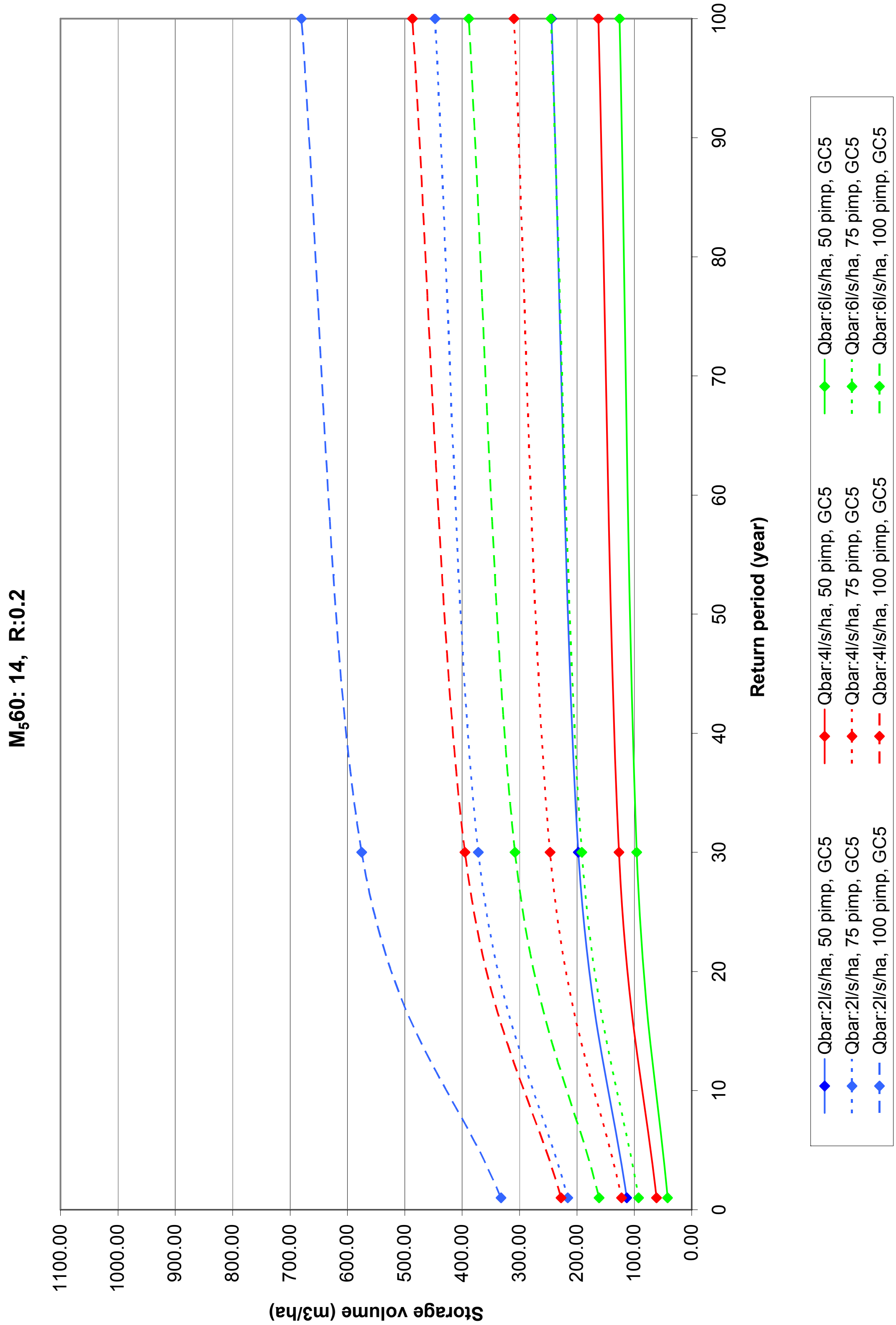


Figure 7.1 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:14, "r":0.2)

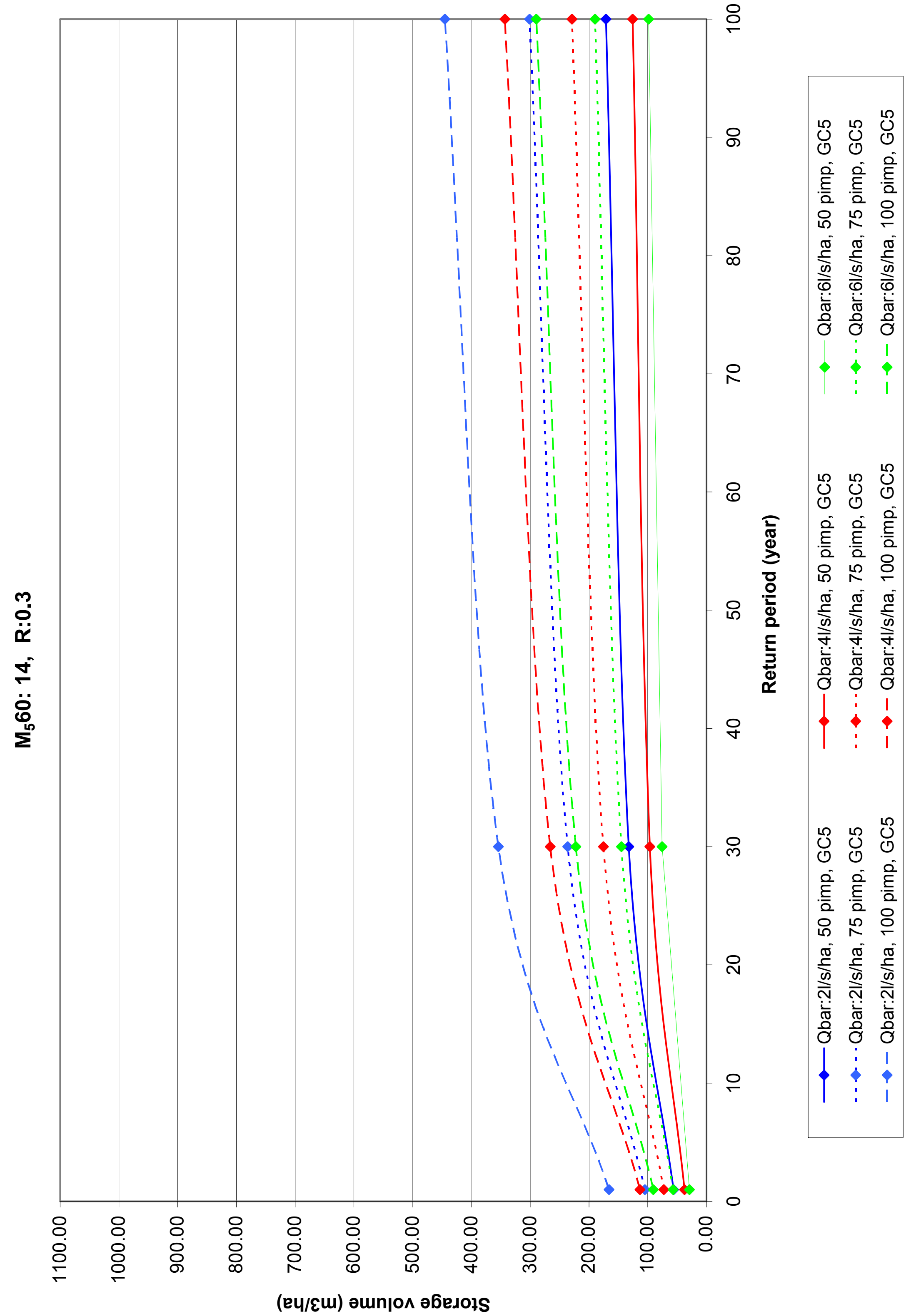


Figure 7.2 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:14, "r":0.3)

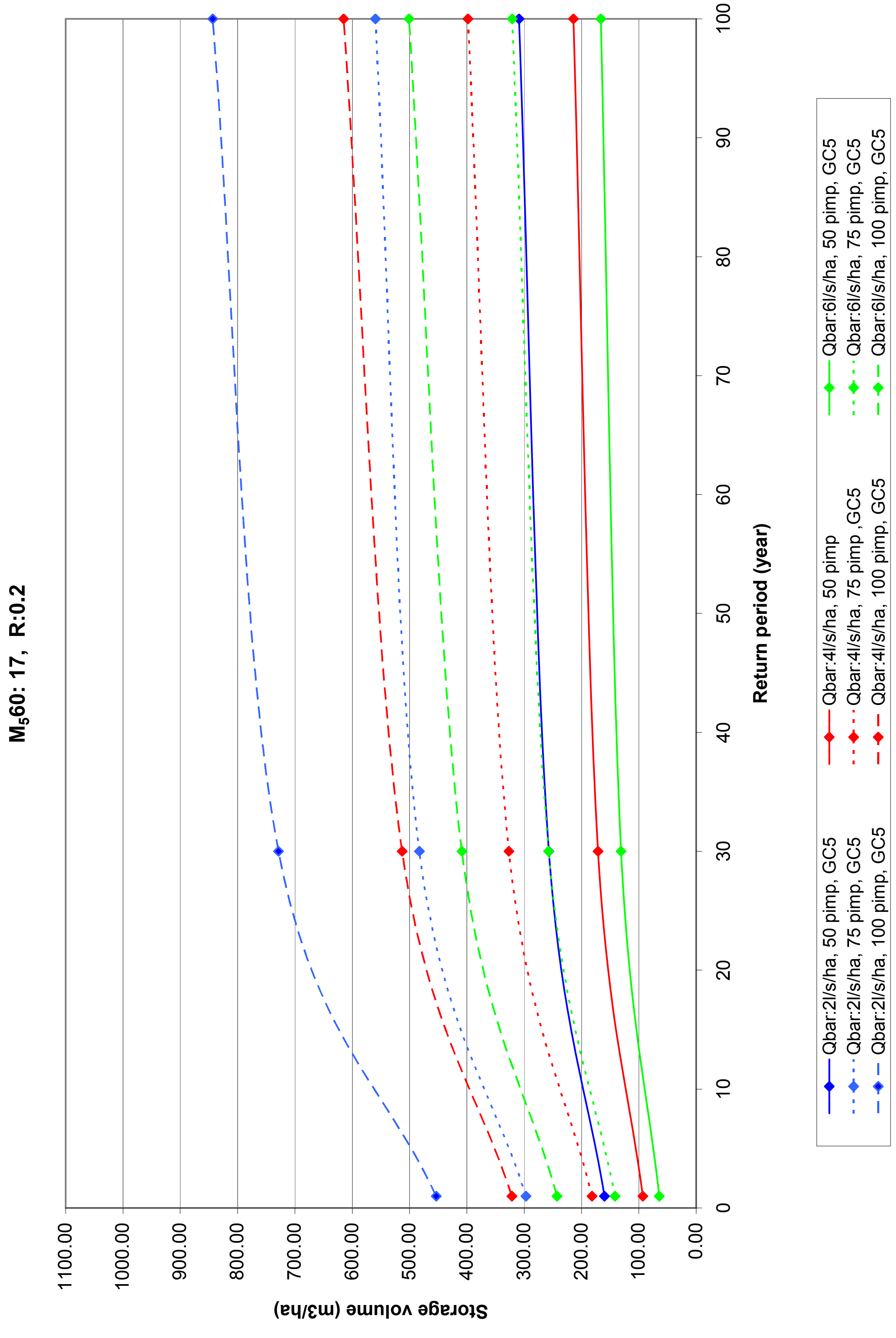


Figure 7.3 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.2)

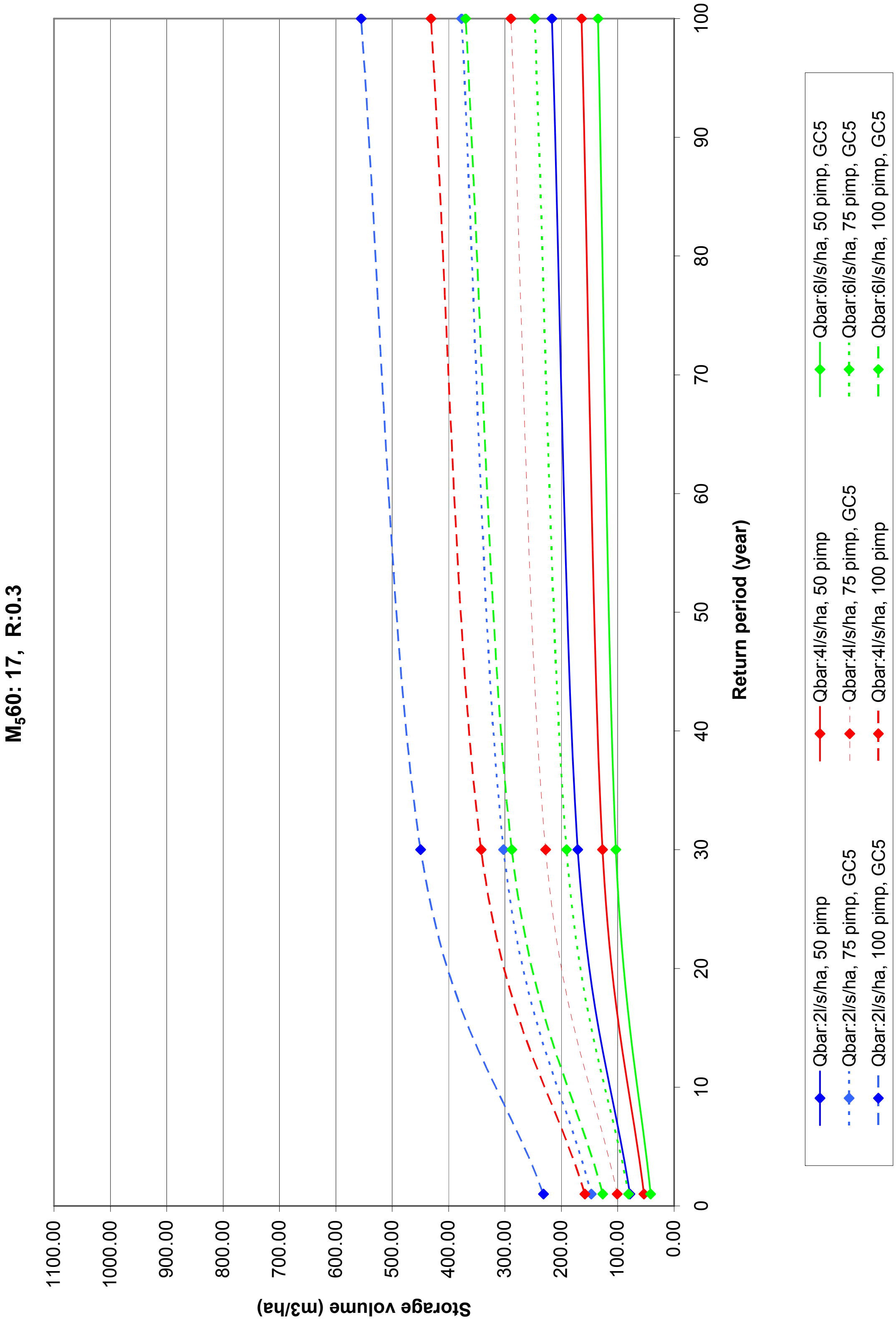


Figure 7.4 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.3)

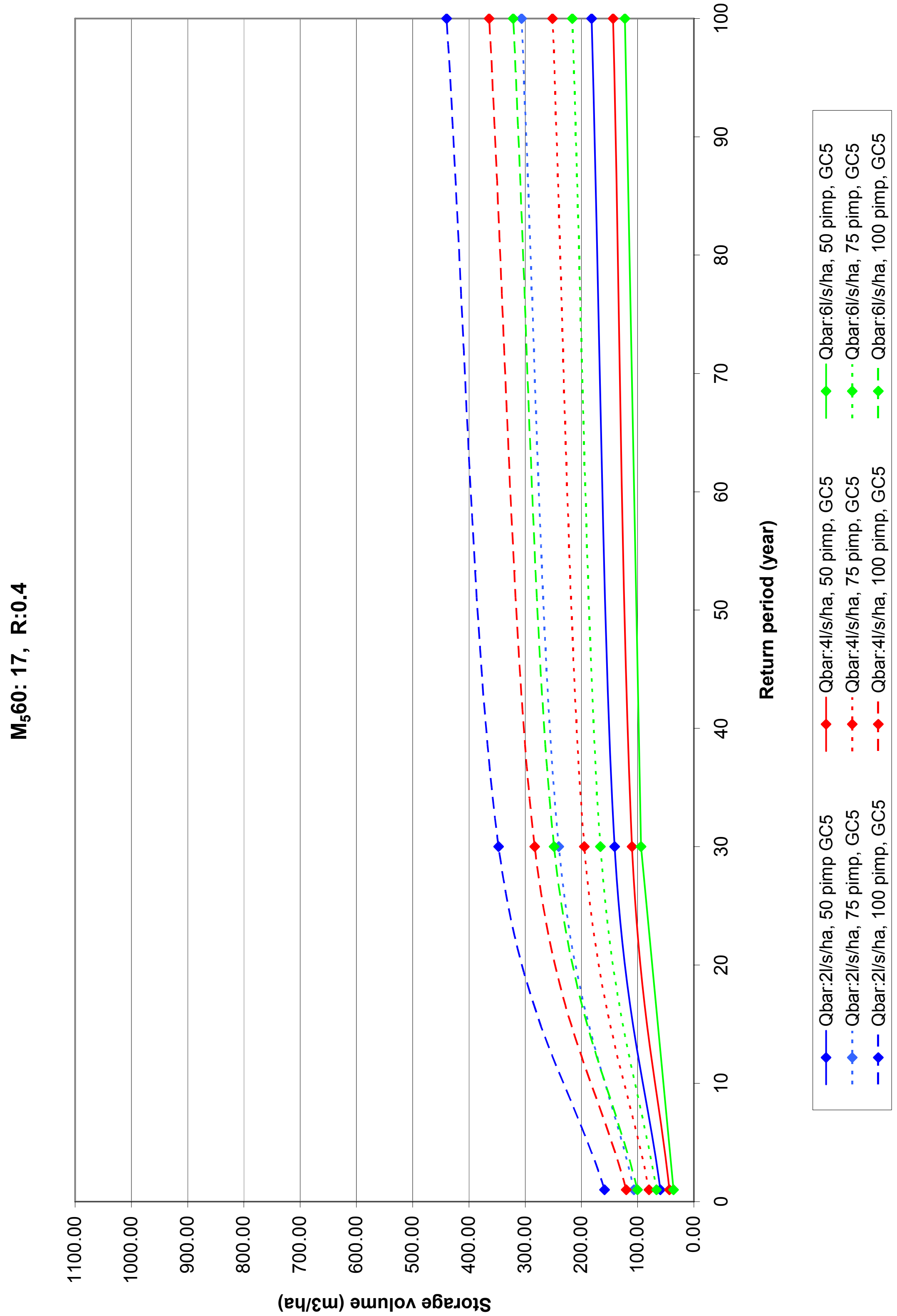


Figure 7.5 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:17, "r":0.4)

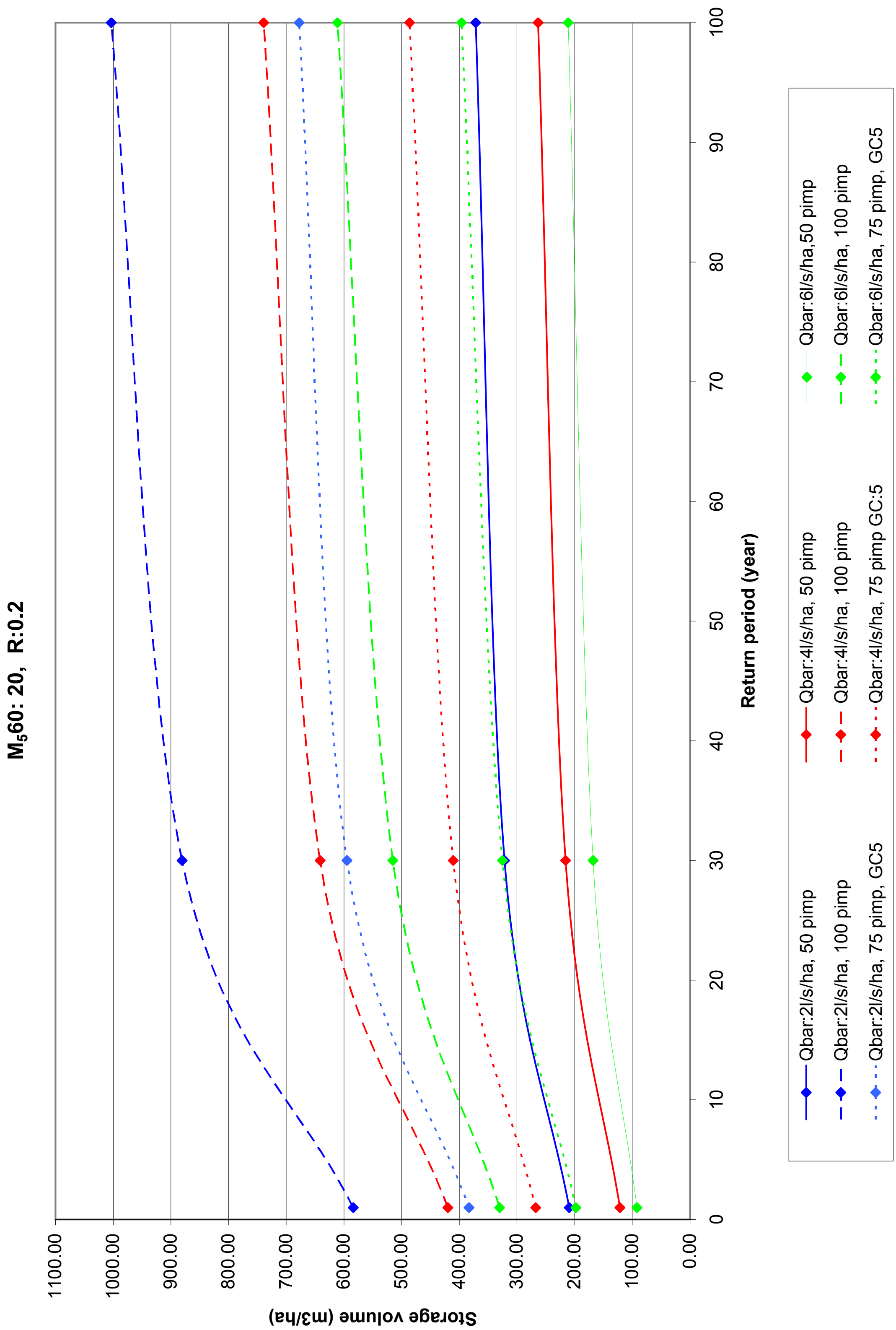


Figure 7.6 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, "r":0.2)

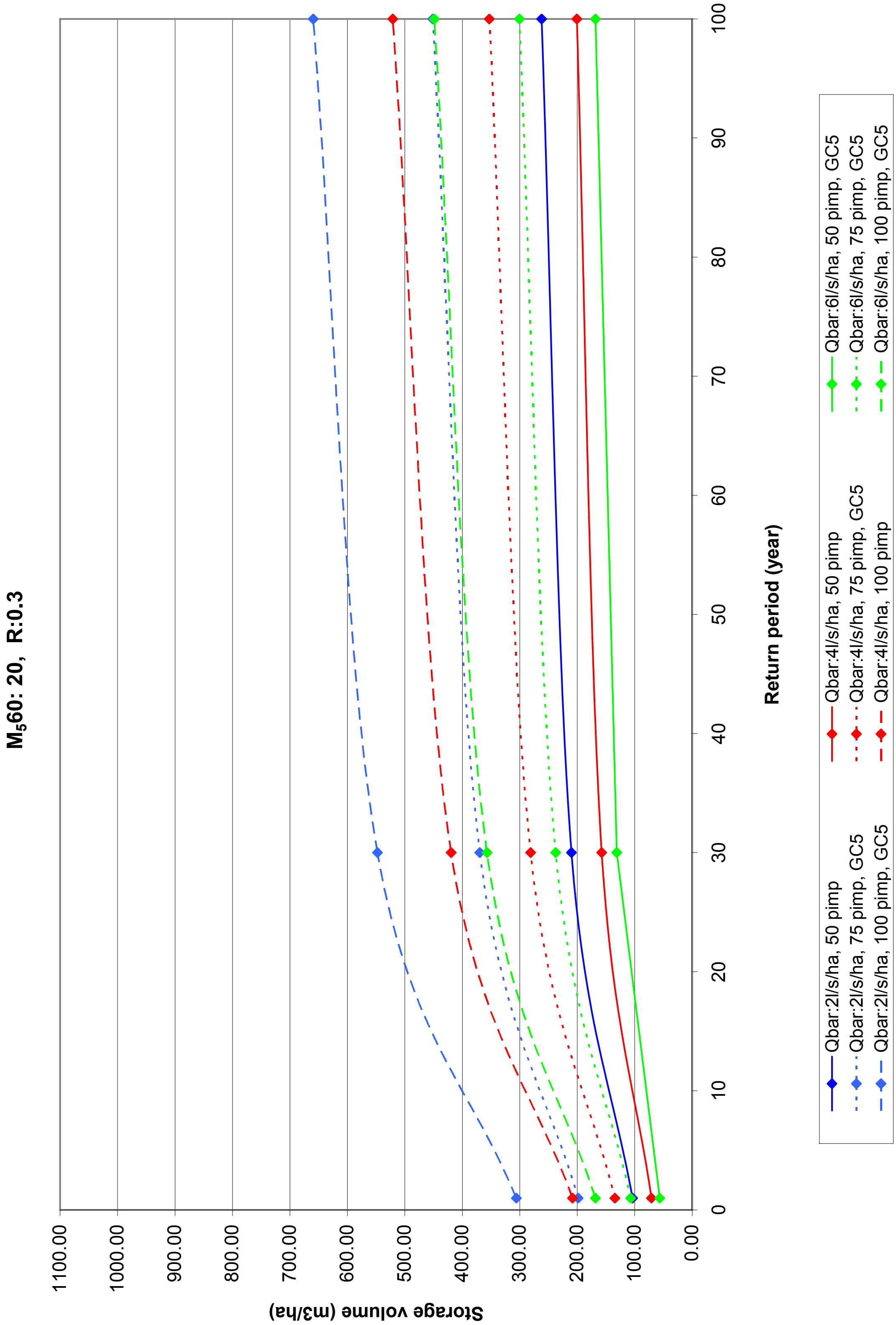


Figure 7.7 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, “r”:0.3)

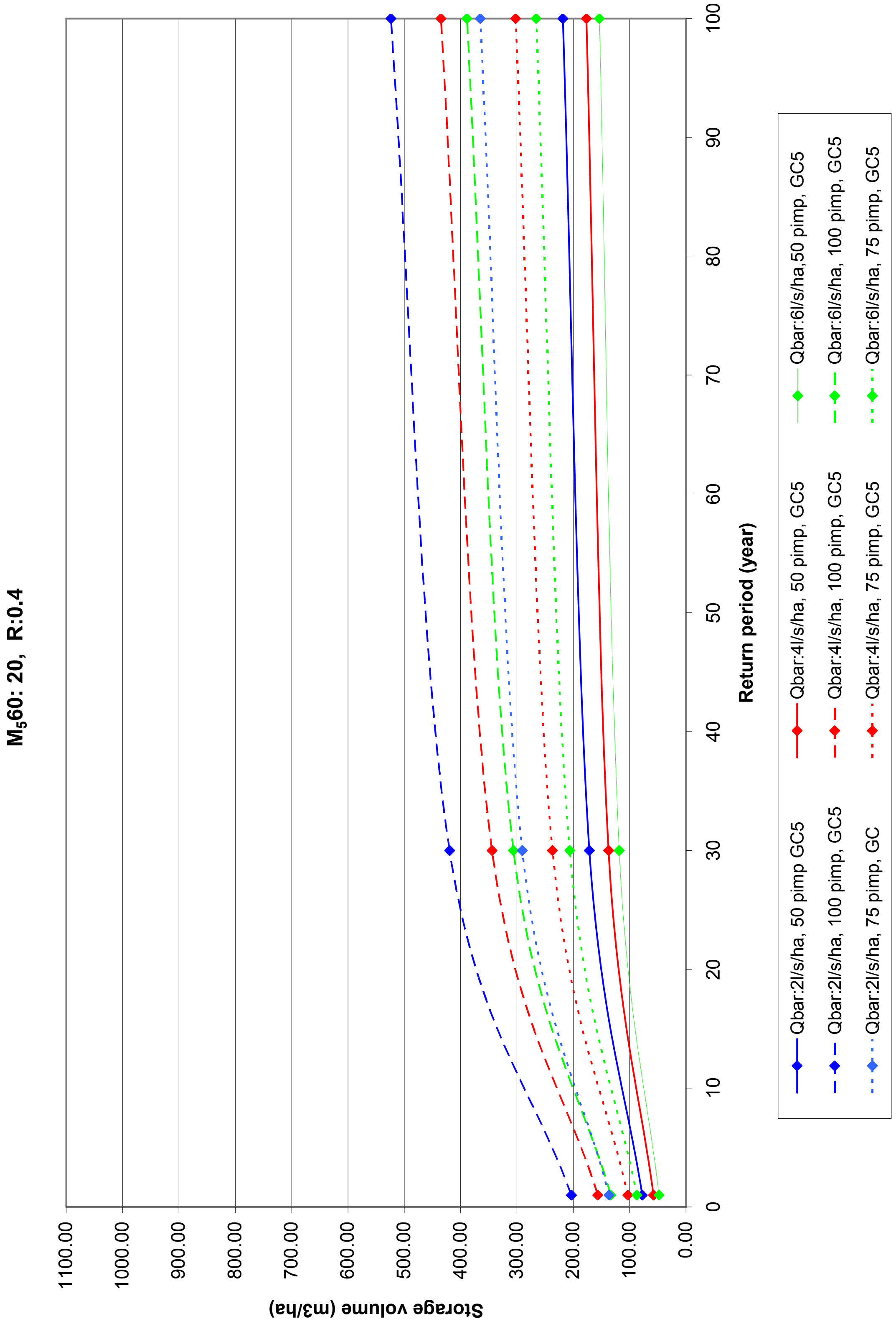


Figure 7.8 Attenuation storage volume as a function of Q_{BAR}/A and PIMP ($M_5$60:20, "r":0.4)

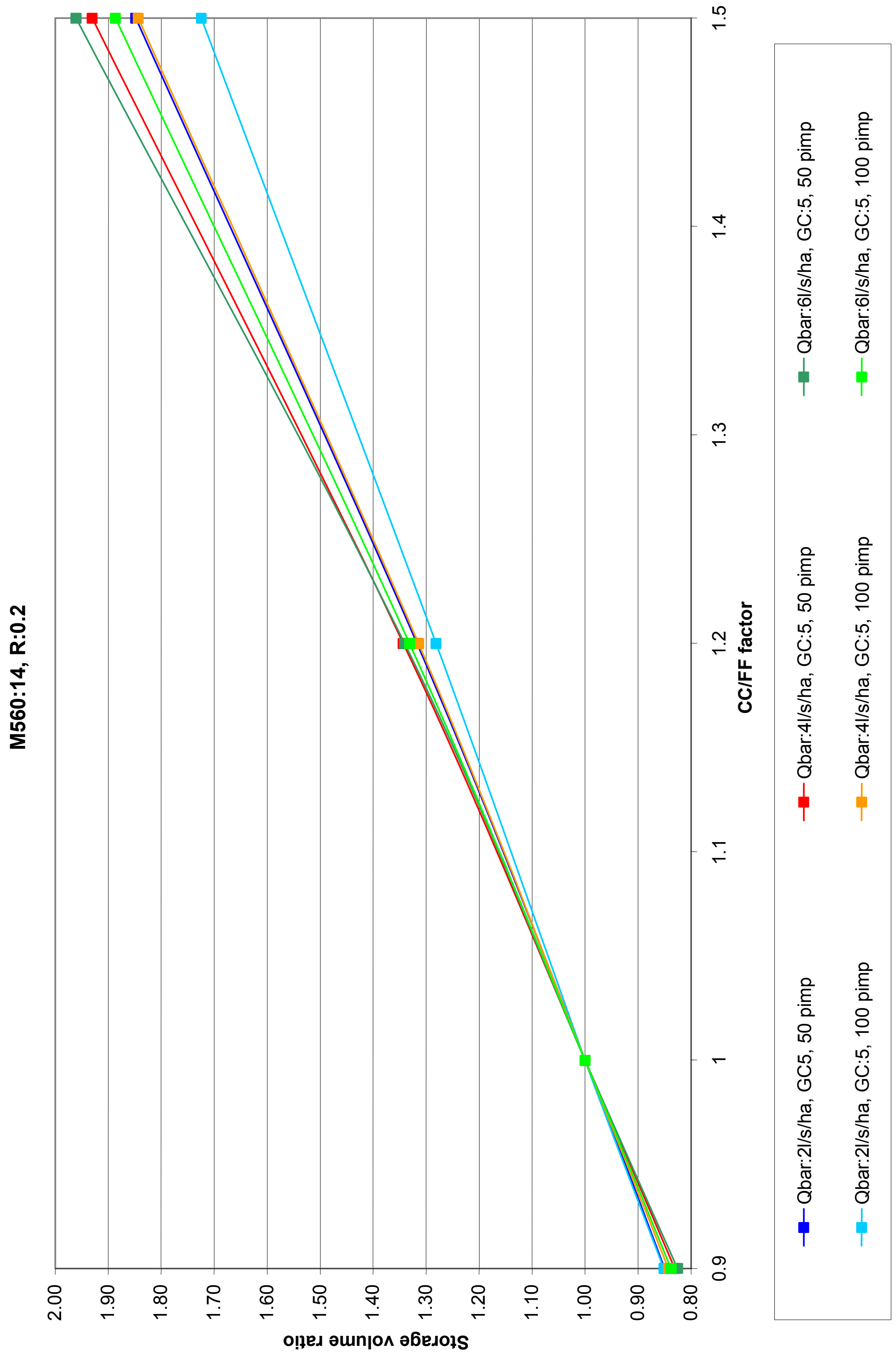


Figure 8.1 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios ($M_5$60:14, "r":0.2)

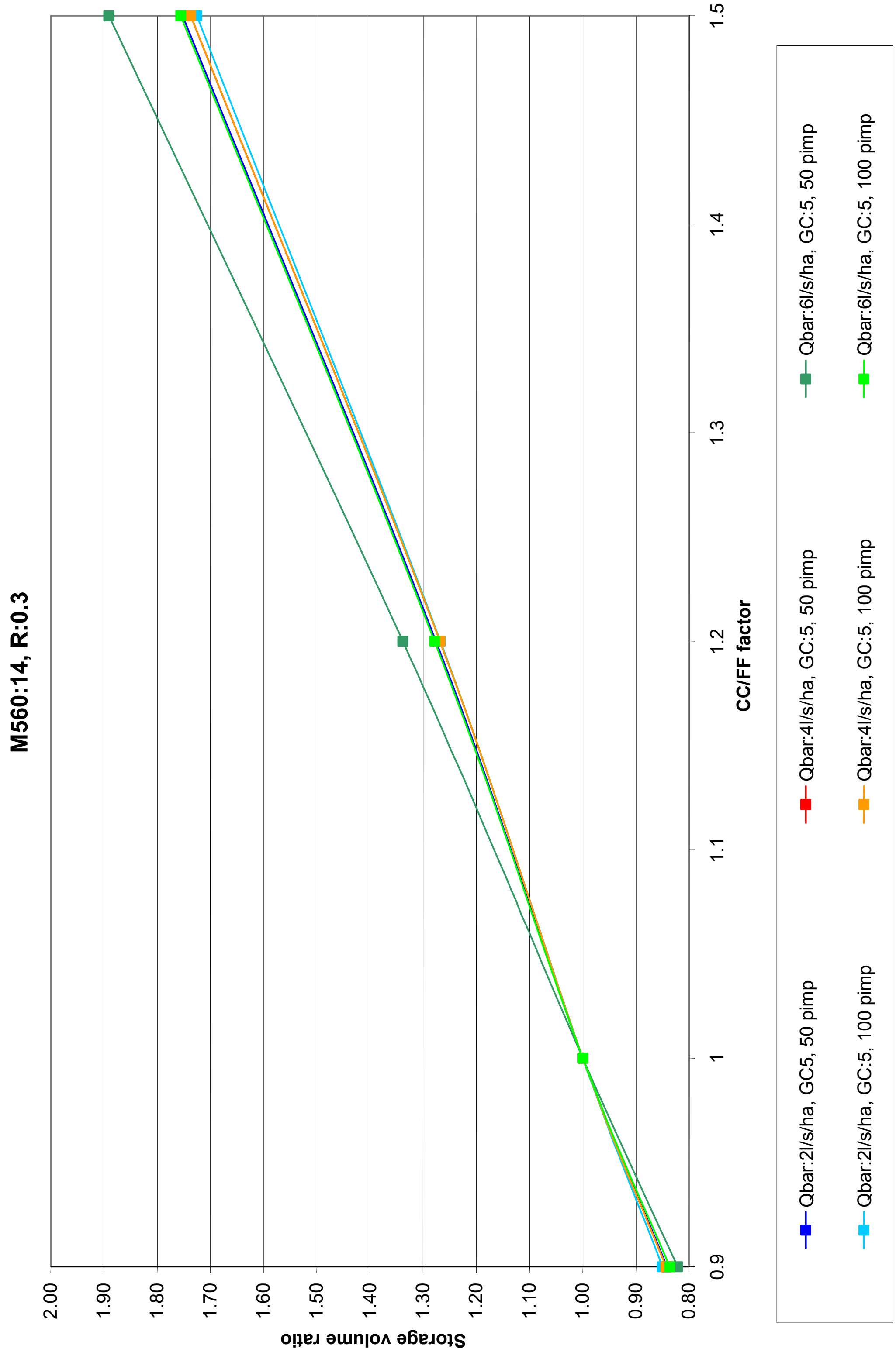


Figure 8.2 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios ($M_5$60:14, "r":0.3)

Figure 8.3 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:17, R:0.2)

Figure 8.4 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:17, R:0.3)

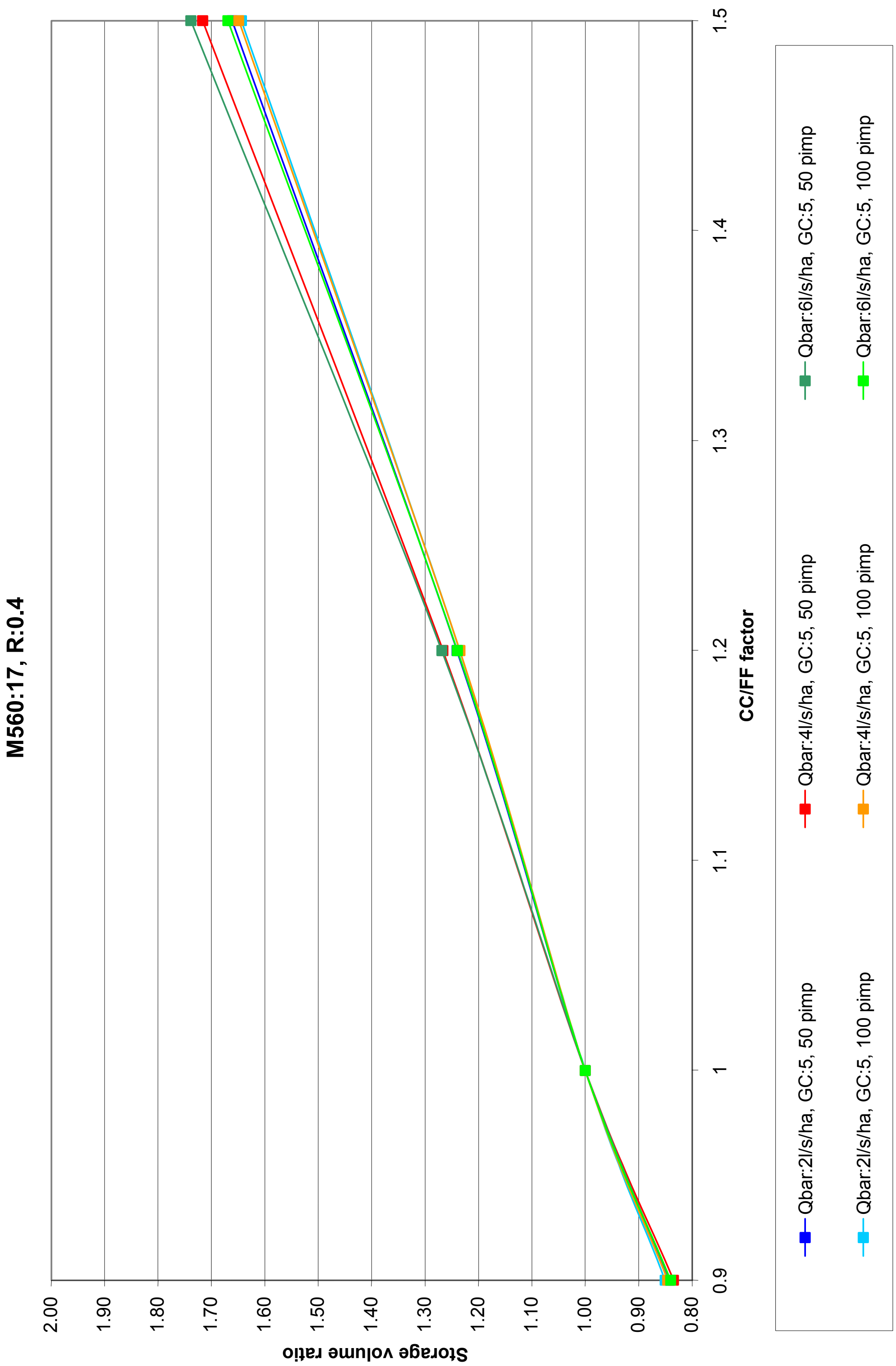


Figure 8.5 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.4)

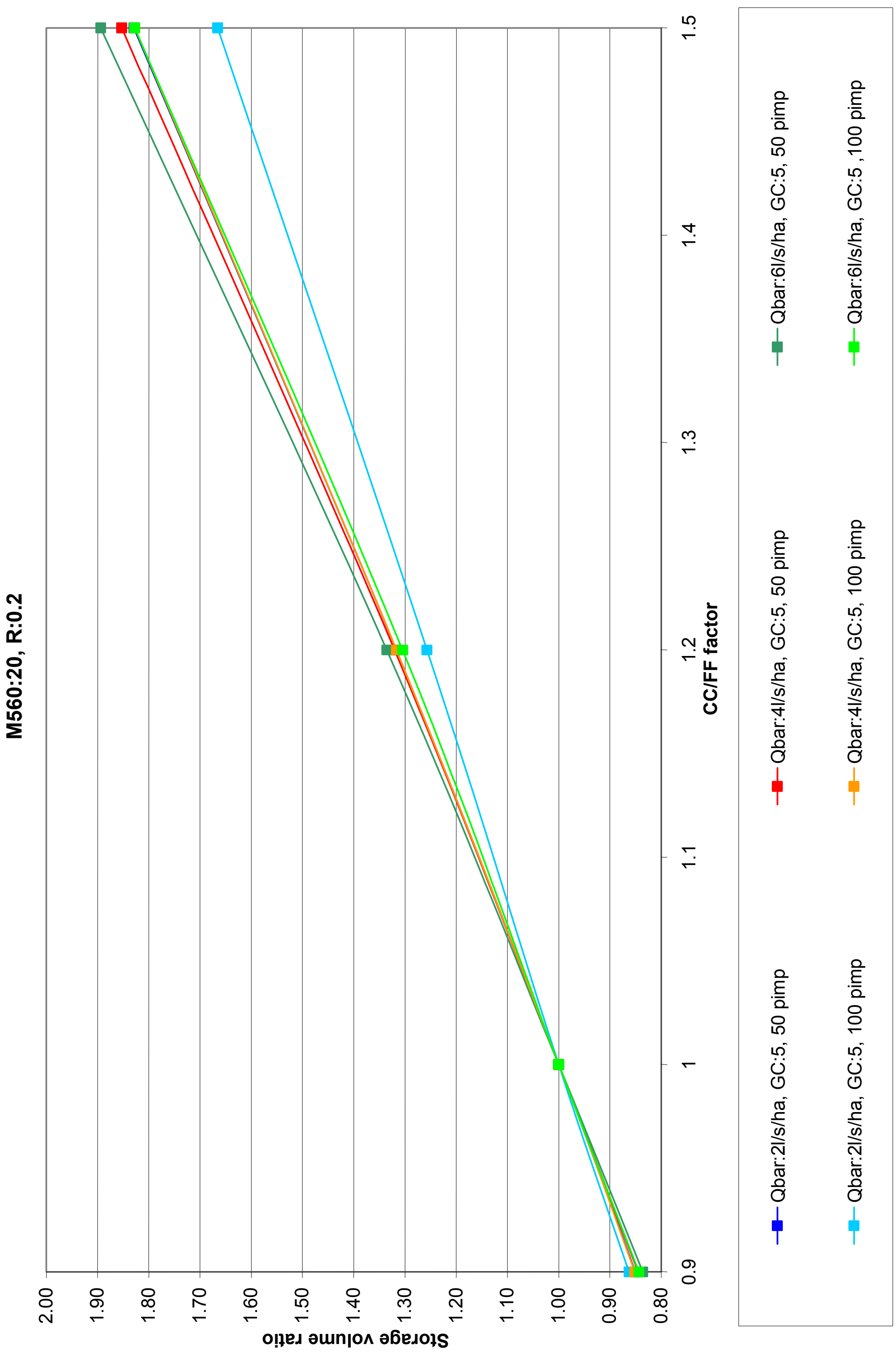


Figure 8.6 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.2)

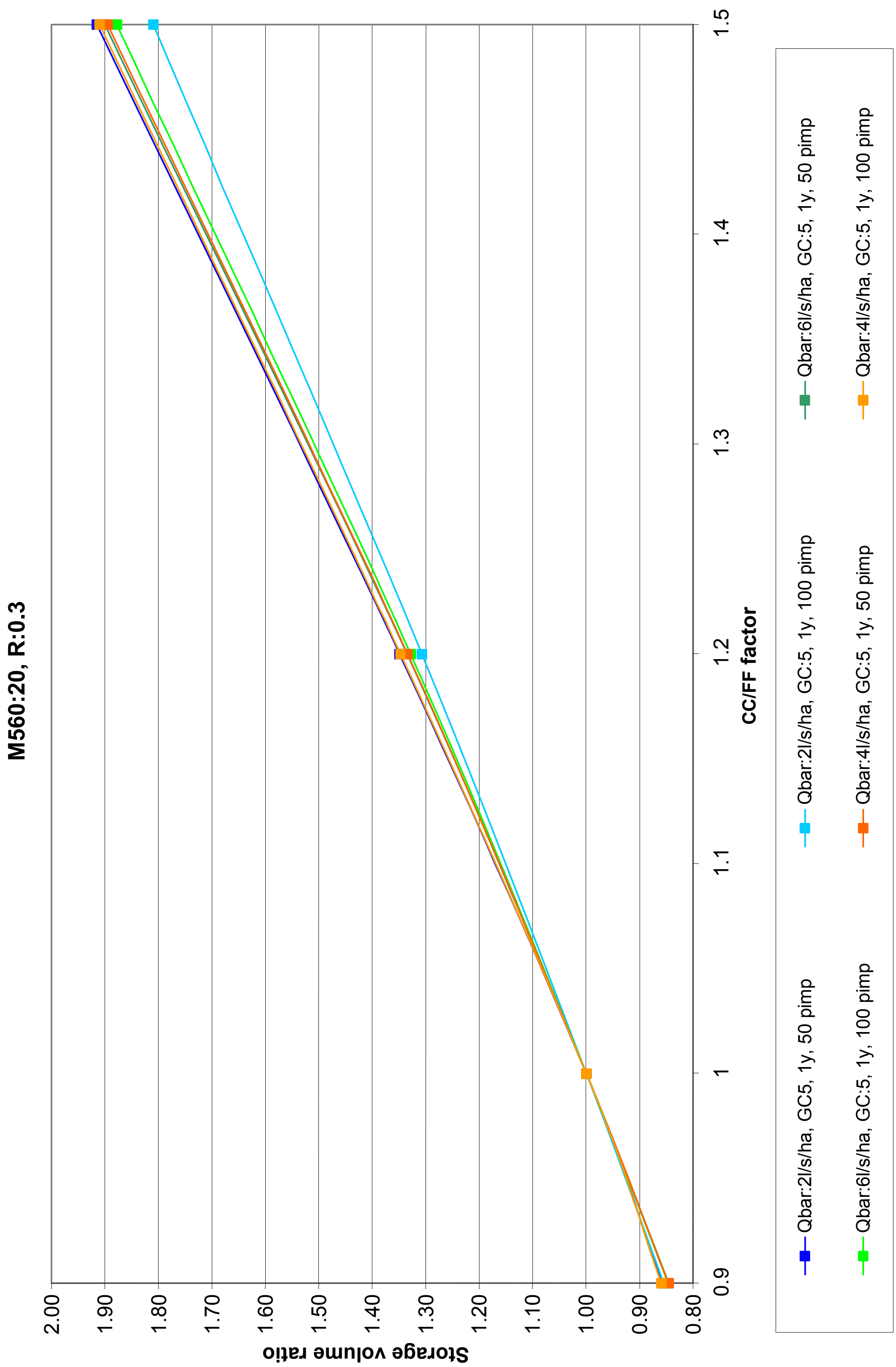


Figure 8.7 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.3)

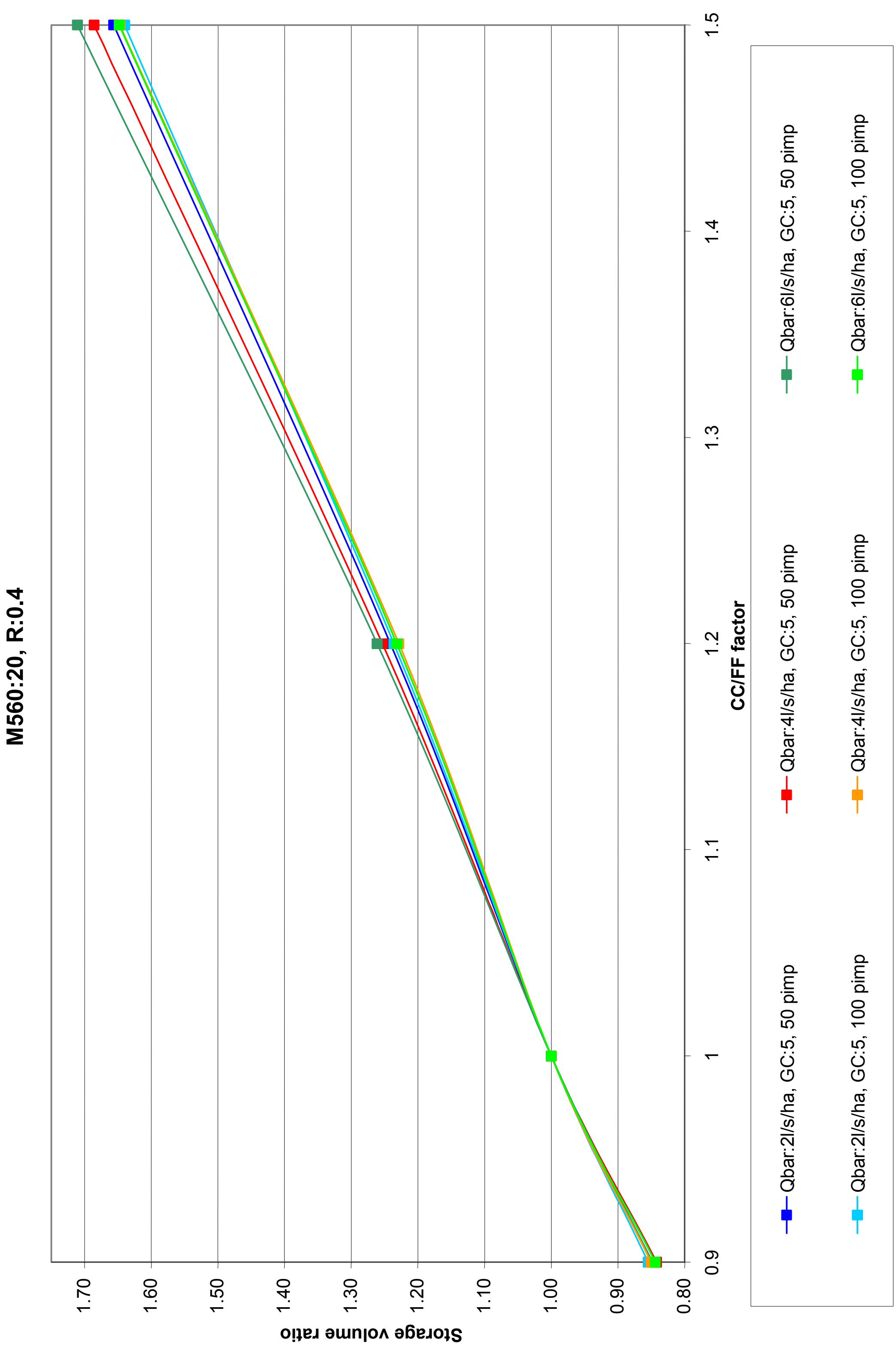


Figure 8.8 Attenuation storage volume adjustment factor to allow for climate change and FEH rainfall depth ratios (M560:20, R:0.4)

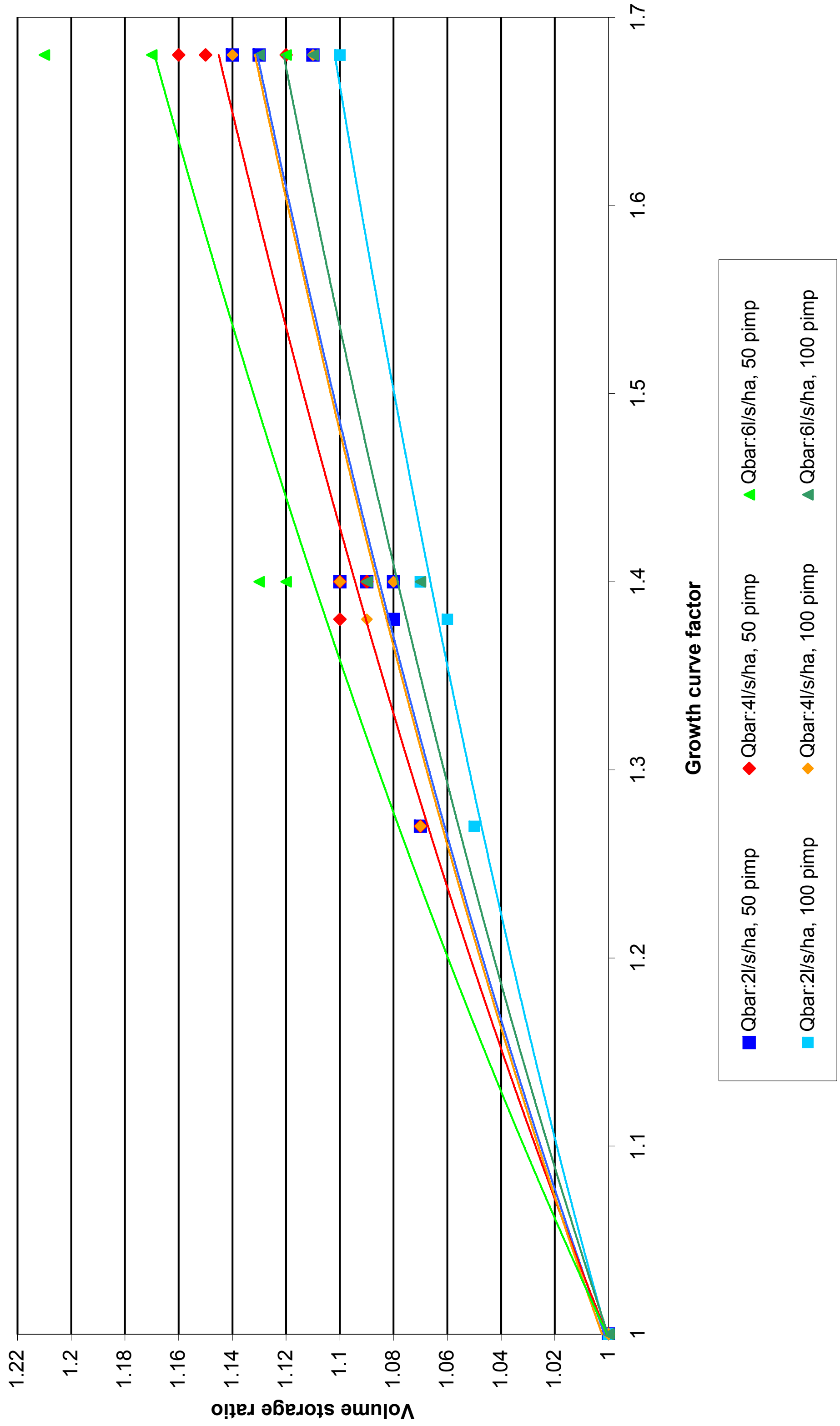


Figure 9 Attenuation storage growth curve adjustment factor hydrological regions of UK (all hydrological zones)

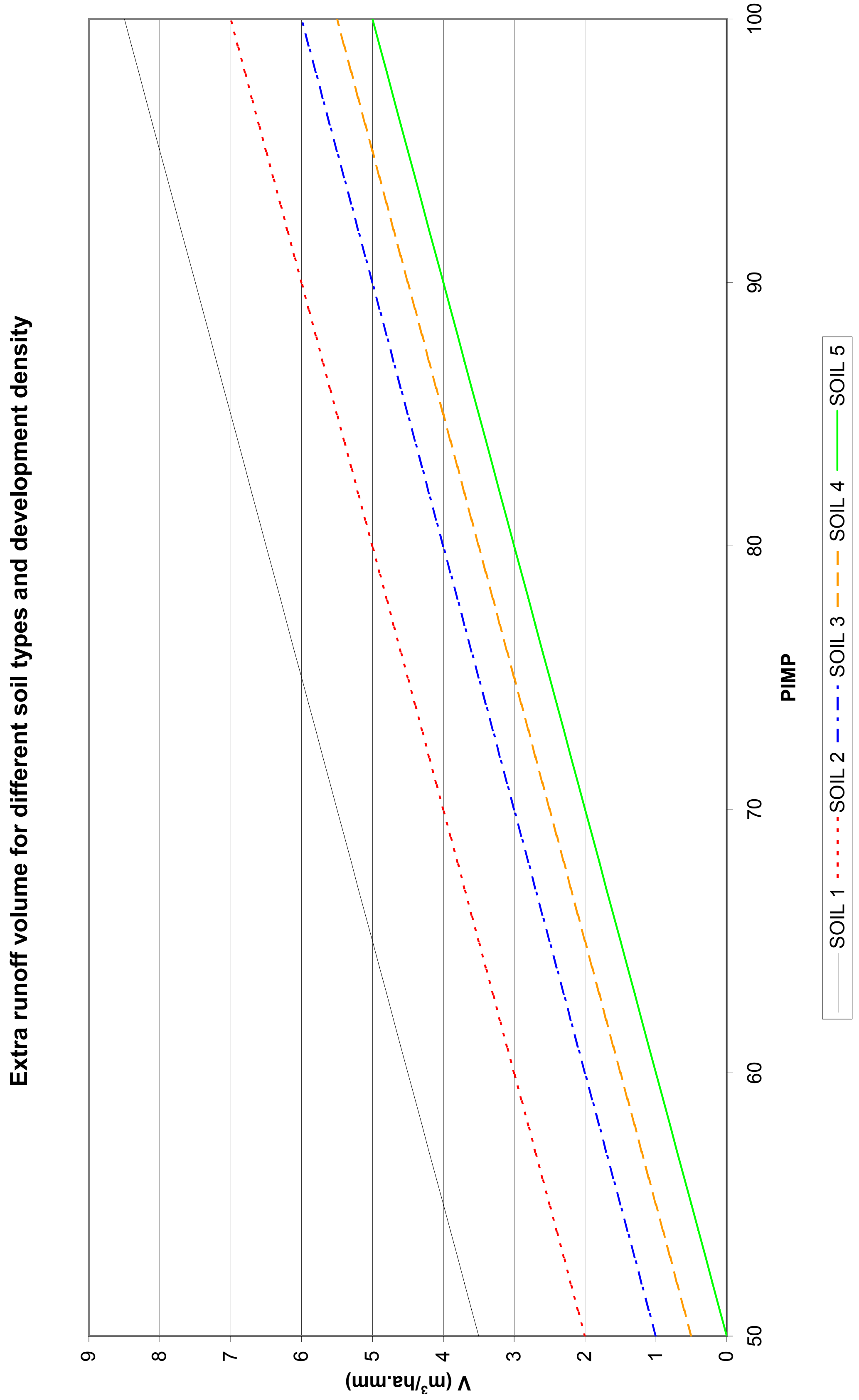


Figure 10 Long Term storage volume based on SOIL type

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 6.1.1-6.3.4

See note 3 in section 3.1 for discussion on the use of Figure 11.

Figure 11 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 10. 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 12 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)

<u>Results</u>
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 1.1)
2. Hydrological rainfall Zone (M_560, r)	(Z)	20/0.3		Zones 1 to 8 based on FSR rainfall characteristics (Appendix 1, Figure 2)
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 2, item 9.
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 7.1 – 7.8)
	($Uvol_{30yr}$)	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 1.1 (see note 2).
10. FEH Rainfall factor	(FF_{1yr})	1.0		Use critical duration based on Q_{BAR}/A and PIMP (Appendix 1, Figure 11 and Appendix 1, Figures 6.1.1 – 6.3.4).
	(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 8.1 – 8.8 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 9). 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 1.2)
	(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	

***Notes:* 1** *Hard surfaces draining to infiltration units (which are considered to be effective for extreme events) can be assumed as not contributing for the purpose of calculating Attenuation storage for determining* α. *(The assessment of PIMP should still be based on the total area of hard surfaces to the total area of the catchment).*

Where pervious pavements are used (where a significant proportion of runoff will discharge at less than 2 l/s/ha), a reduction in the paved areas can be made for assessing Attenuation storage volume. For the purpose of this simple method it should be assumed that all hard surfaces passing through such SUDS attenuation units (unless specifically controlled to a specified discharge rate of less than 2l/s/ha) are 25% effective. Therefore hard surface areas served by these units can be reduced by 25% for calculating Attenuation storage. Proof of compliance at detailed design will determine the actual Attenuation storage needed.

2 *The Defra guidance on the impact of climate change on river flows is to apply a factor of 1.2. As there is a non-linear relationship between rainfall and runoff it is suggested that a factor of 1.1 should be applied to rainfall depths in this procedure.*

3 *Appendix 1, Figure 11 assists in estimating the duration 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, the maps (6.11 – 6.3.4) can be used to determine the FEH rainfall factor.*

Maps for a return period of 30 years do not exist – use those for 25 years. FEH / FSR maps are unavailable for durations greater than 12 hours. Use 12 hours in this situation.

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 %
- Qbar = 6l/s/ha.

Note: FSR/FEH ratio (from the maps of figures 6.1.1 – 6.3.4) 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 %
- Qbar = 6l/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 %
- Qbar = 6l/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 %
- Qbar = 6l/s/ha.

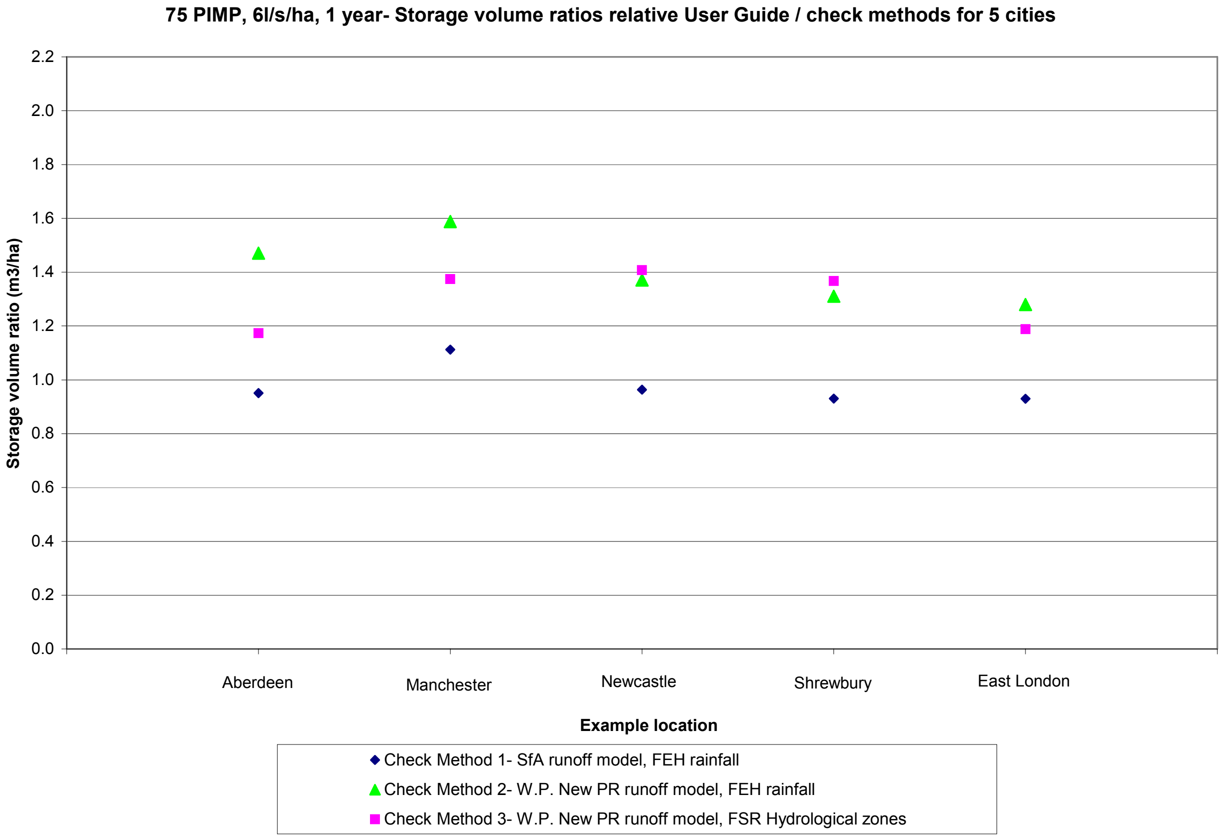


Figure A2.1.1 Check comparison of Attenuation Storage Volume for site type 1 – 1 year

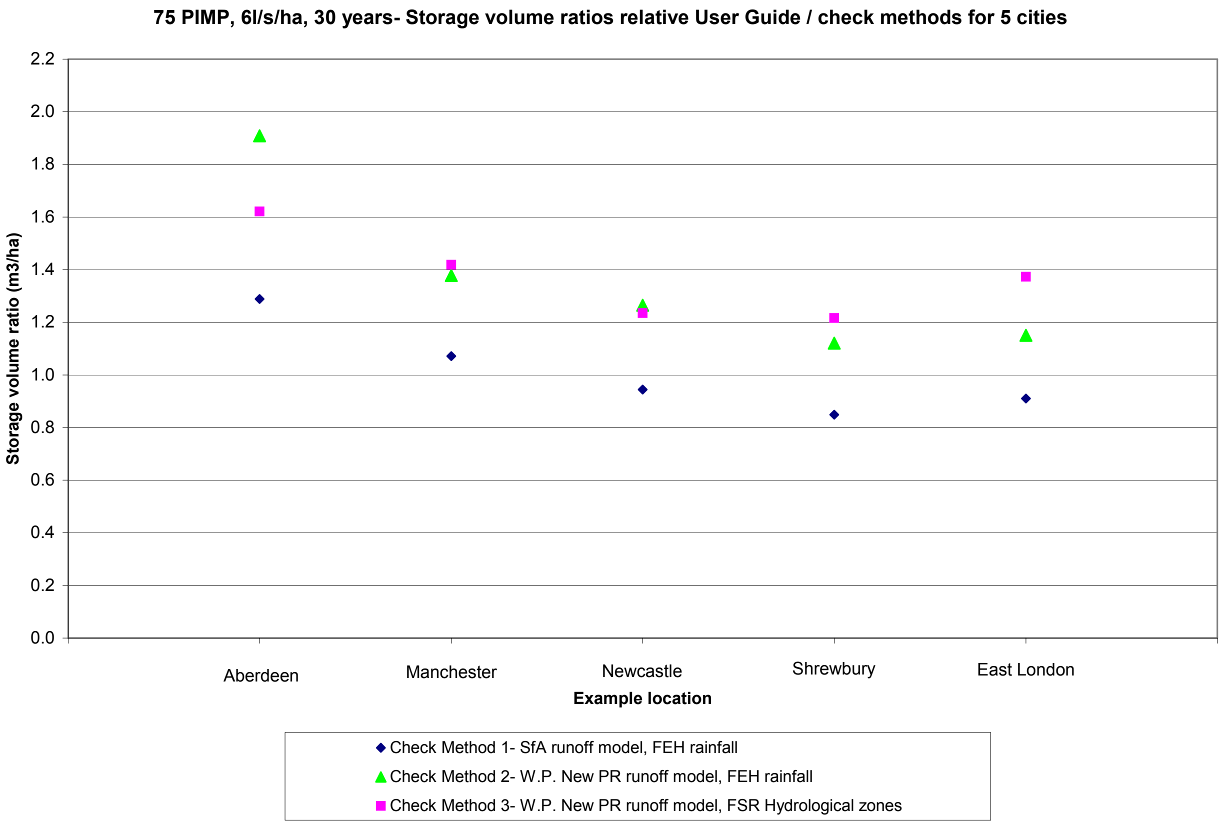


Figure A2.1.2 Check comparison of Attenuation Storage Volume for site type 1 – 30 years

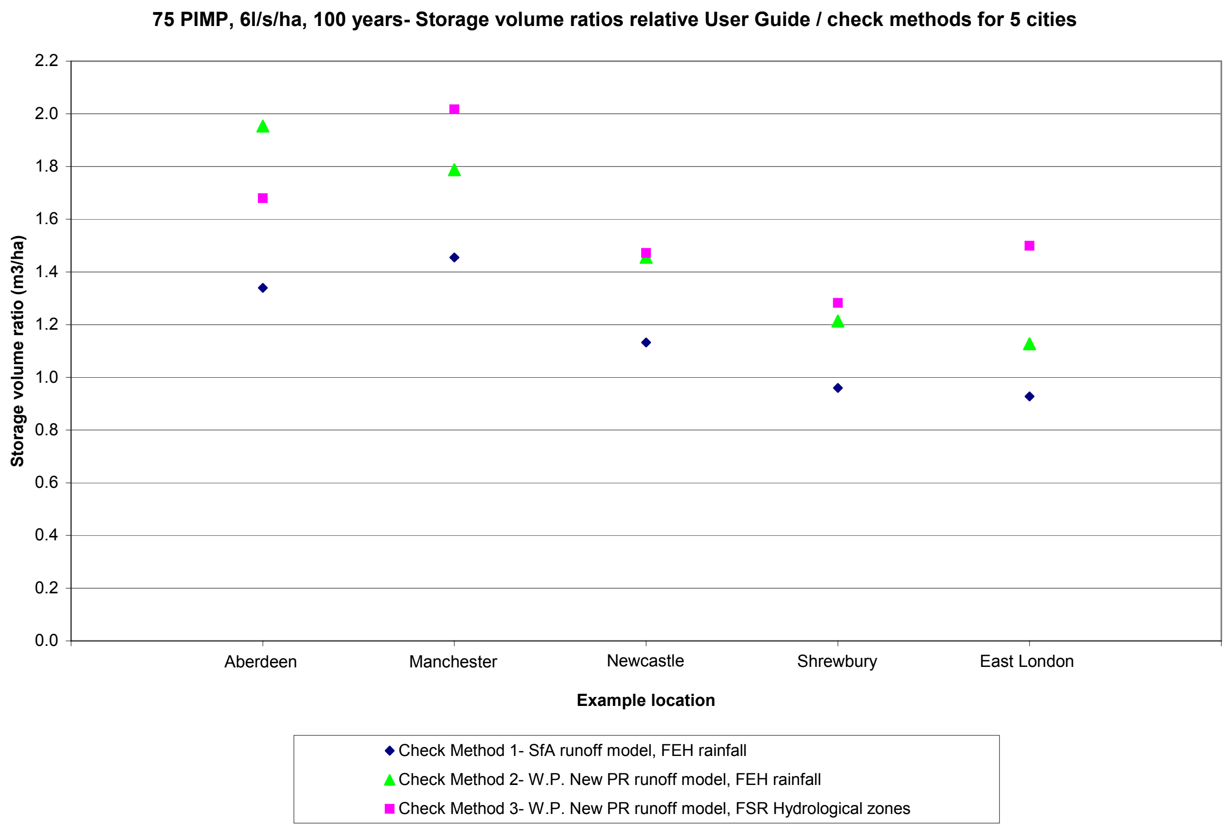


Figure A2.1.3 Check comparison of Attenuation Storage Volume for site type 1 – 100 years

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 %
- Qbar = 6l/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 %
- Qbar = 6l/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 %
- Qbar = 6l/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 %
- Qbar = 6l/s/ha.

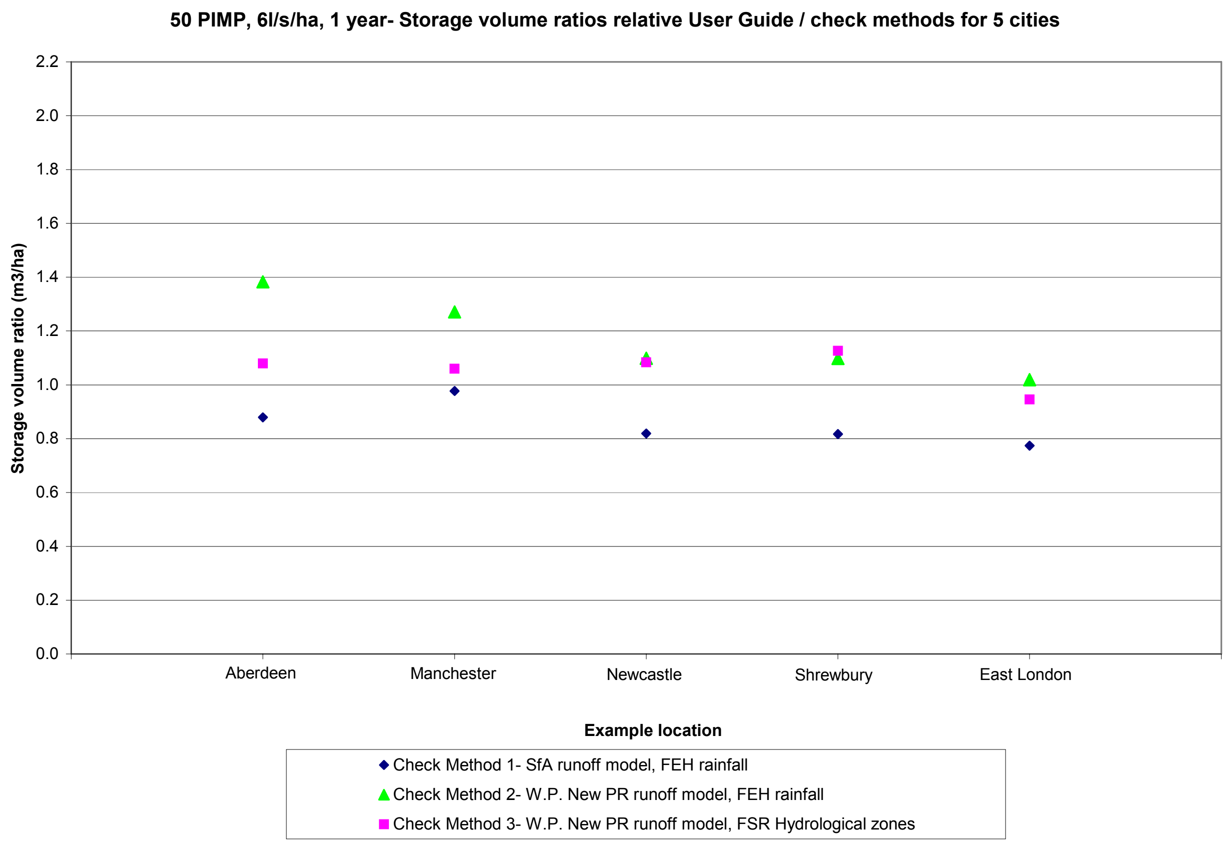


Figure A2.2.1 Check comparison of Attenuation Storage Volume for site type 2 – 1 year

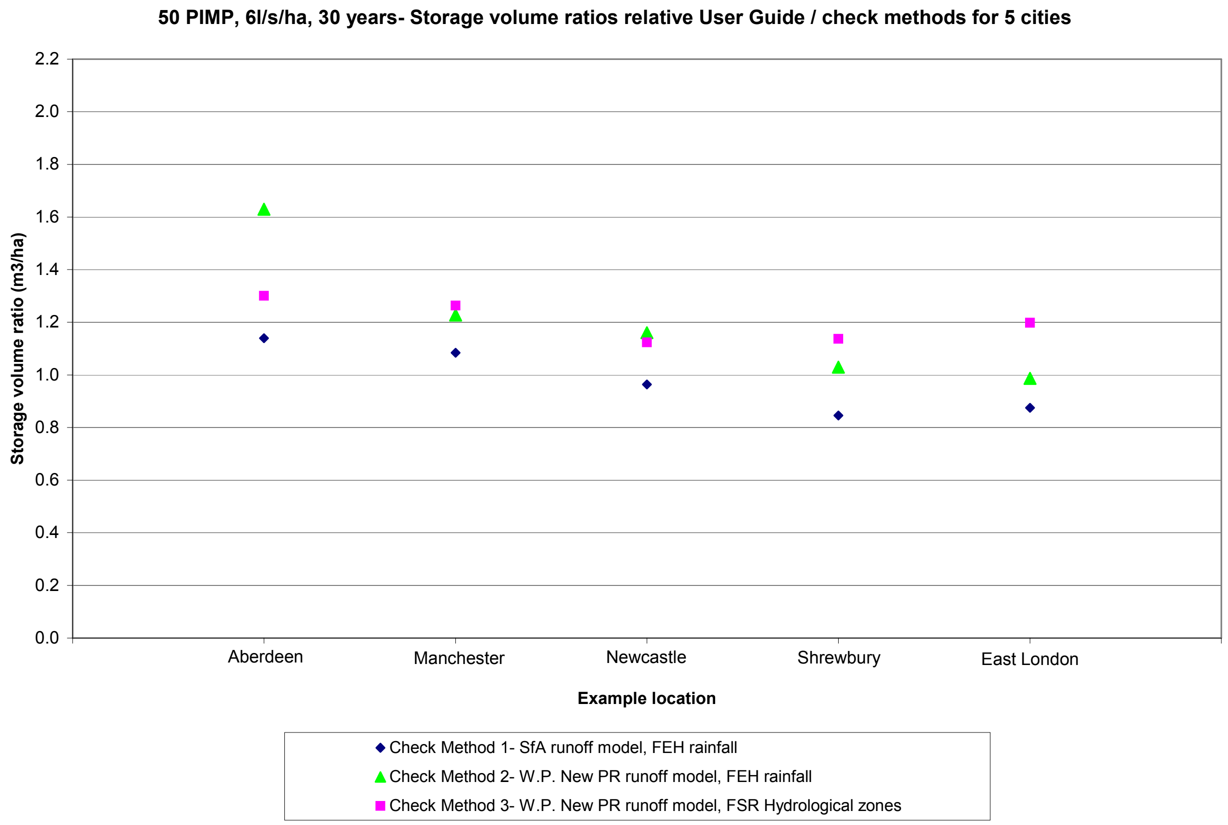


Figure A2.2.2 Check comparison of Attenuation Storage Volume for site type 2 – 30 years

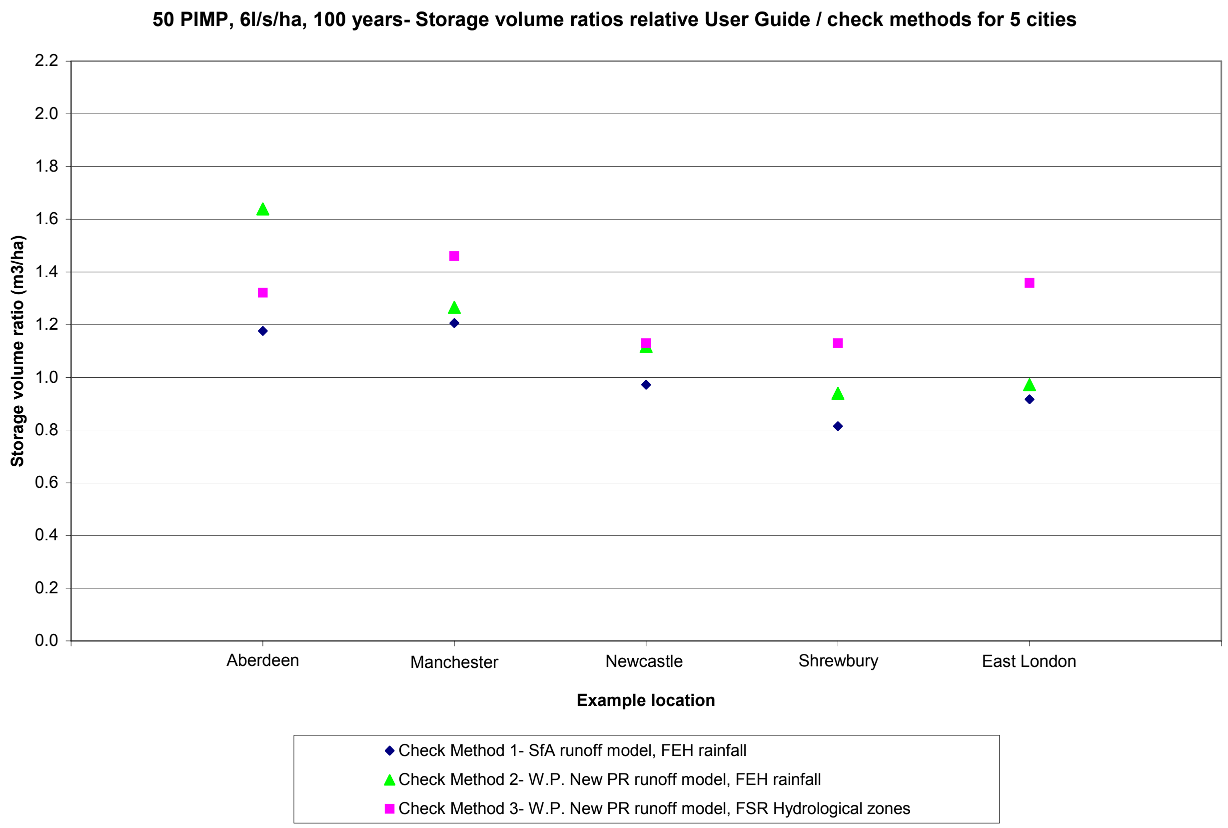


Figure A2.2.3 Check comparison of Attenuation Storage Volume for site type 2 – 100 years

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 %
- Qbar = 2l/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 %
- Qbar = 2l/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 %
- Qbar = 2l/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 %
- Qbar = 2l/s/ha.

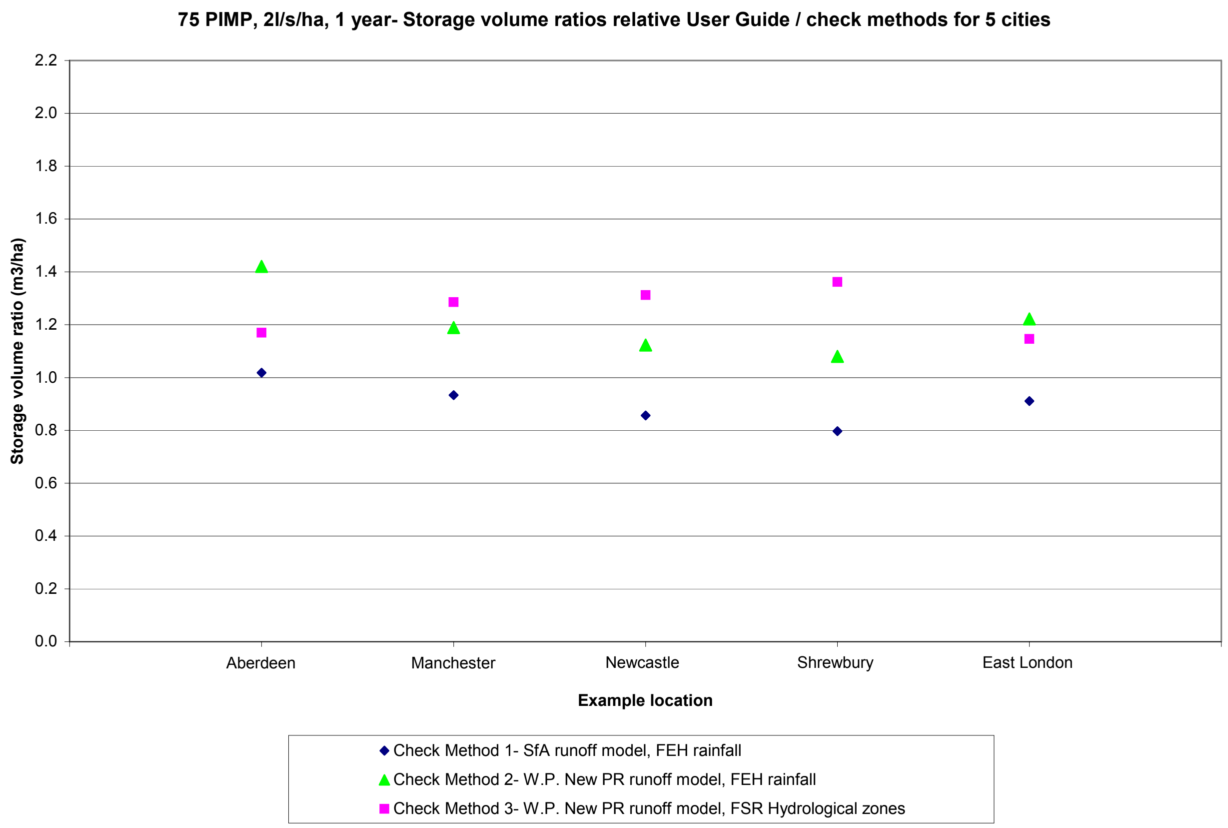


Figure A2.3.1 Check comparison of Attenuation Storage Volume for site type 3 – 1 year

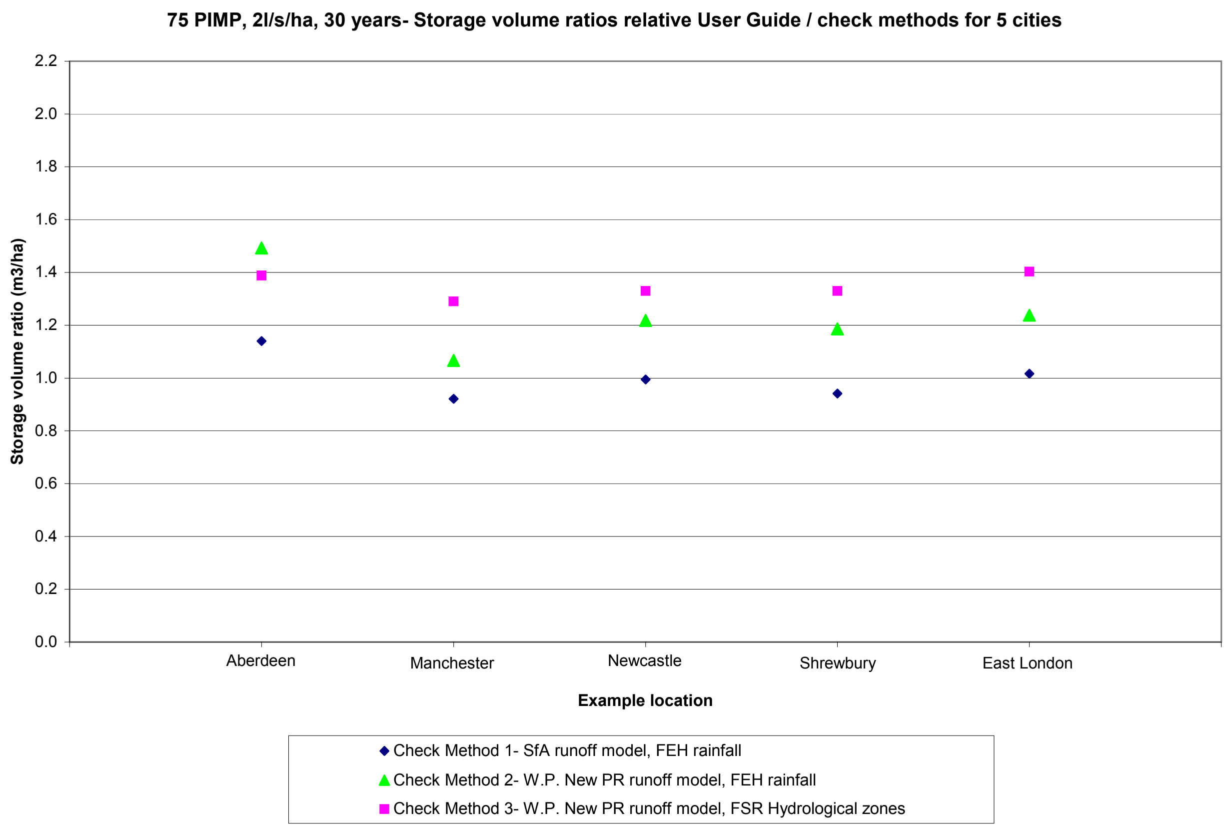


Figure A2.3.2 Check comparison of Attenuation Storage Volume for site type 3 – 30 years

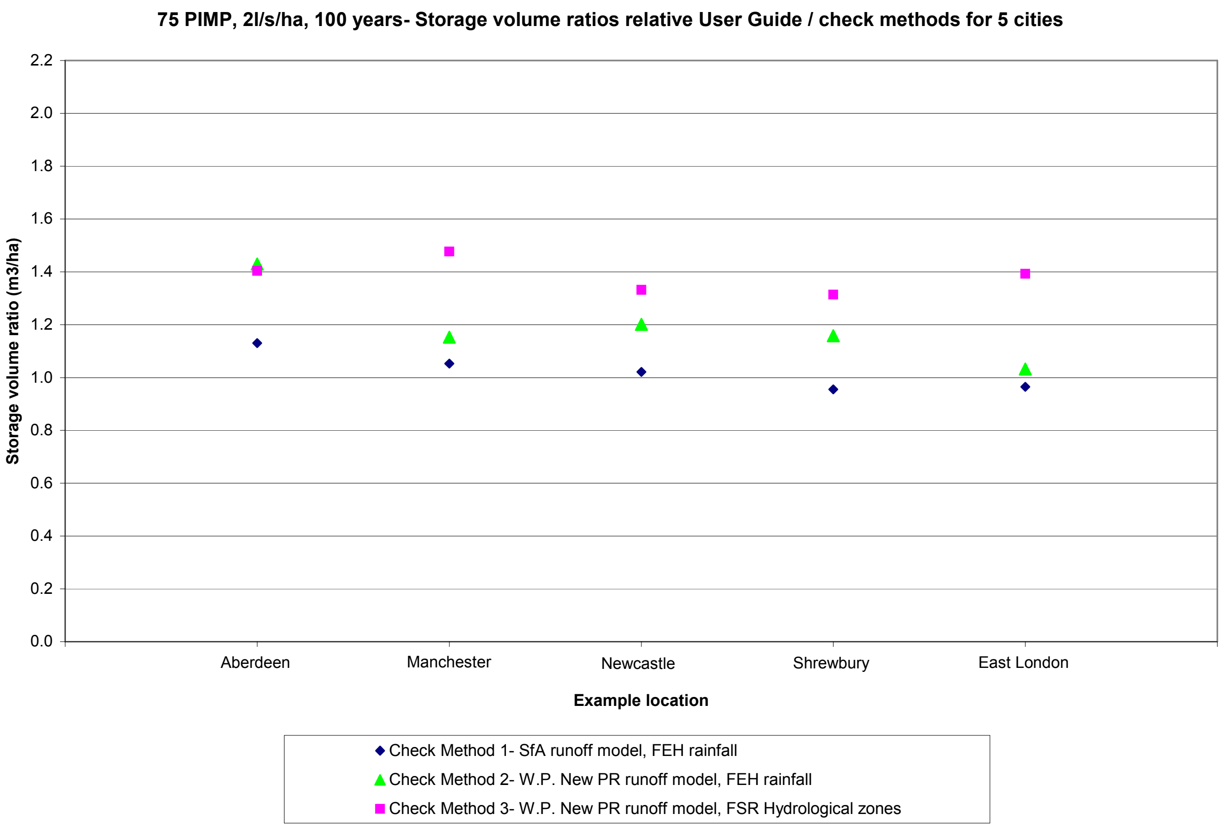


Figure A2.3.3 Check comparison of Attenuation Storage Volume for site type 3 – 100 years

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 %
- Qbar = 2l/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 %
- Qbar = 2l/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 %
- Qbar = 2l/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 %
- Qbar = 2l/s/ha.

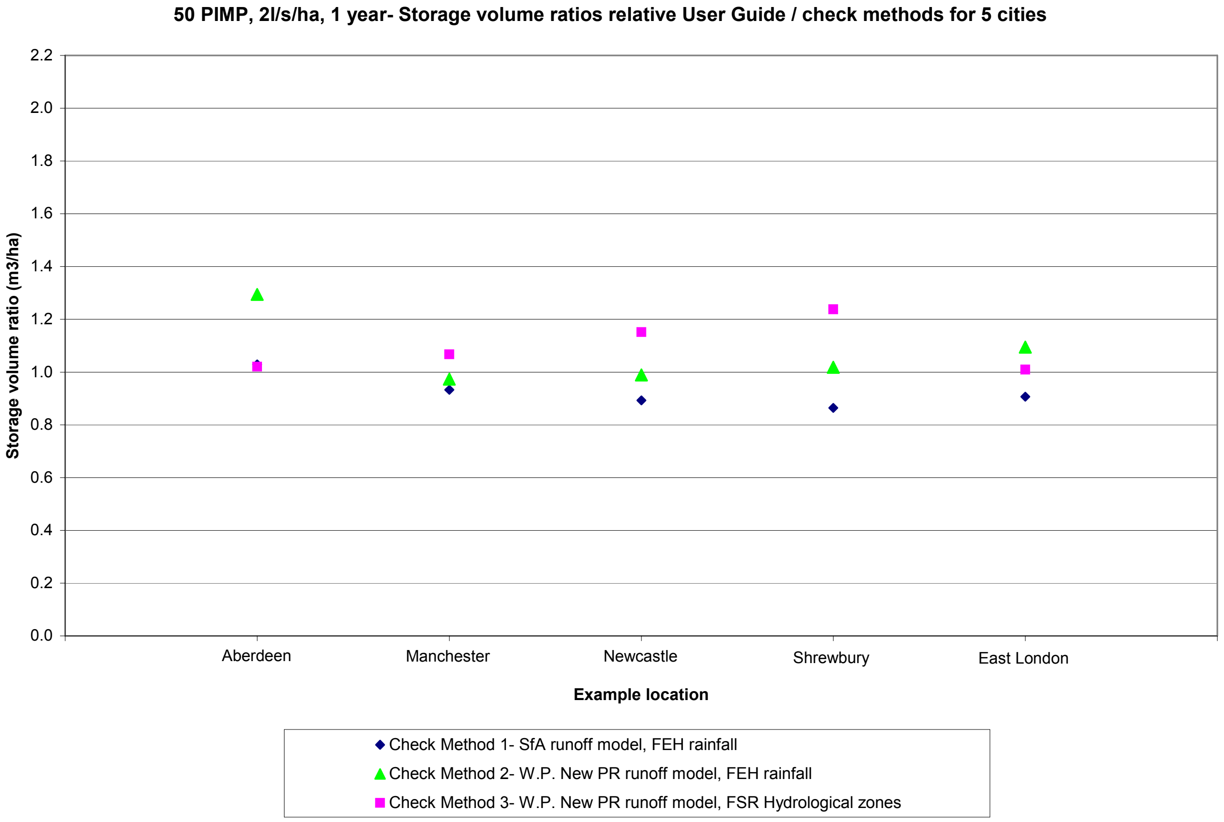


Figure A2.4.1 Check comparison of Attenuation Storage Volume for site type 4 – 1 year

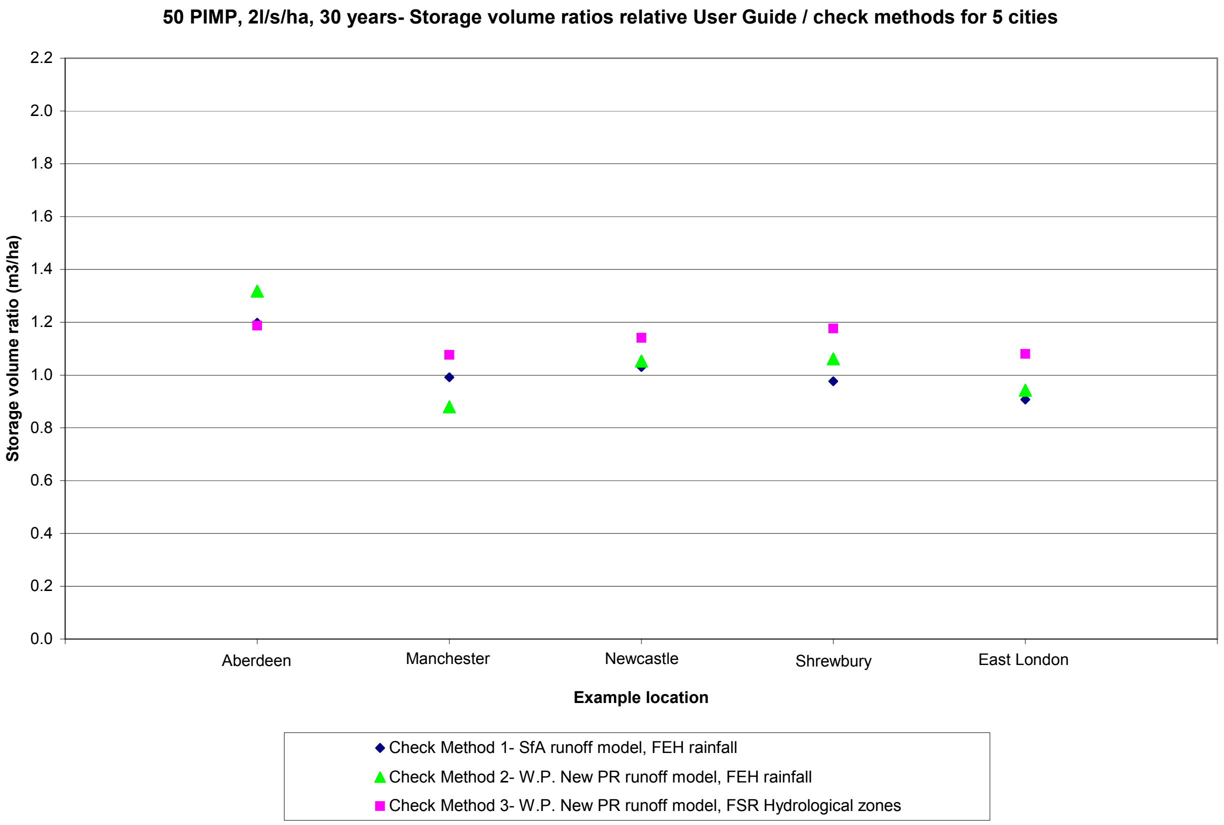


Figure A2.4.2 Check comparison of Attenuation Storage Volume for site type 4 – 30 years

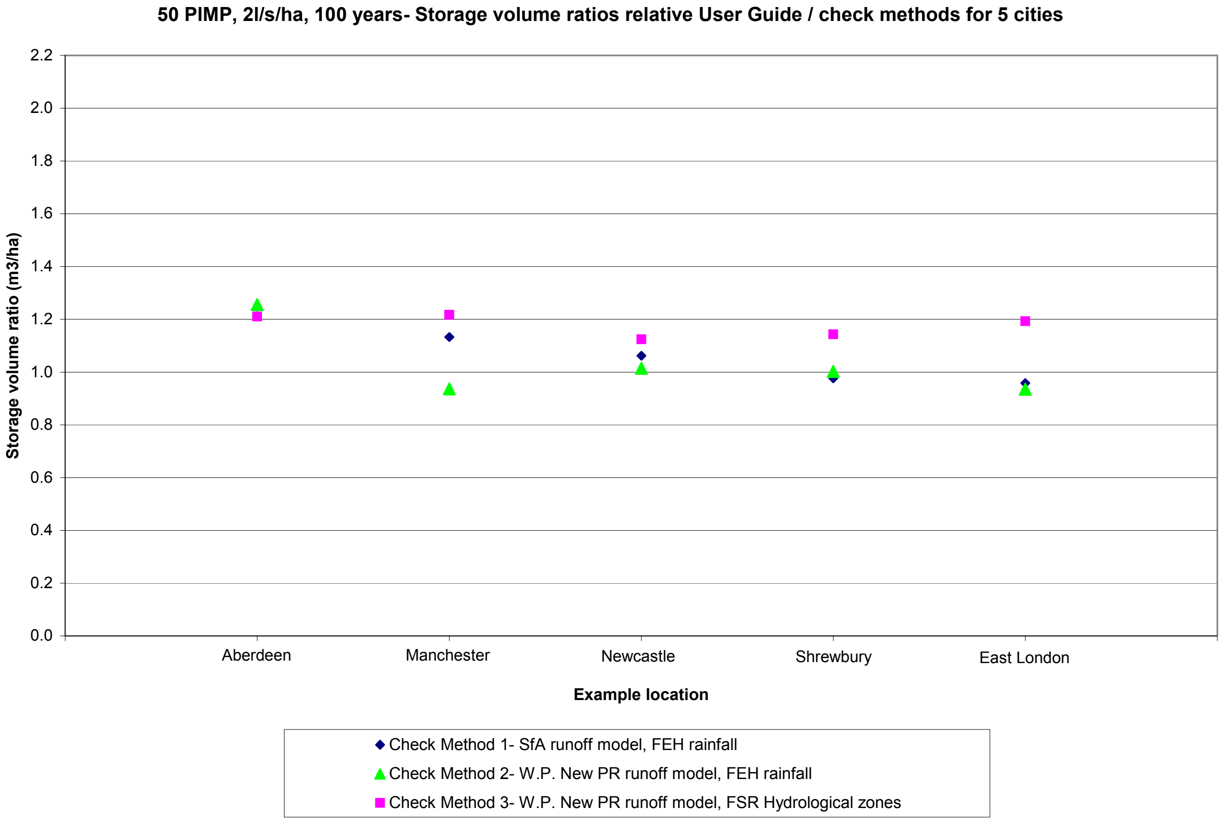


Figure A2.4.3 Check comparison of Attenuation Storage Volume for site type 4 – 100 years