THE WATER BALANCE OF URBAN IMPERMEABLE SURFACES:

CATCHMENT AND PROCESS STUDIES

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Abstract

An examination of research and information needs in urban hydrology suggested the investigation of urban water balances and micro-hydrological processes. This should facilitate more accurate modelling of the rainfall-runoff process from urban impermeable surfaces.

Greater London data produced annual water balances for 5 heavily urbanized Thames tributaries and estimates of the annual yield ranging from 12-72%. Mean annual runoff for largely rural basins in South East England in comparison was 15-44% of rainfall. The inadequacy of the data for water balance studies led to the instrumentation of a small urbanized catchment at Redbourn, Hertfordshire.

Standard meteorological measures were recorded. New instrumentation was designed to measure runoff from shallow pitched roofs while commercially produced instruments were adapted and installed to monitor runoff from a block of flat asphalt-and-chippings garage roofs, and runoff from asphalt roads and pavements at the highway drain outfall.

Runoff from these impermeable surfaces is less than 100% even during winter months when evaporation is low. Percentage runoff is 76% for both the pitched and flat roofs while that from the paved surfaces is only 17%. Despite differences in slope, runoff volumes from the pitched and flat roofs are almost identical suggesting that the flat roof does not afford much greater depression storage and evaporation losses. The flat roof does however attenuate storm runoff producing lower flow rates and longer runoff duration than the pitched roofs. Road runoff is very low because of infiltration. The calculated depression storage is 0.25 mm for both roof types and 1.00 mm for the road surface. An average water balance compiled for the roofs gave evaporation as the residual 19% of rainfall. Using an average roof evaporation rate in the road surface water balance gave infiltration as 36% of rainfall with 17% runoff, 21% evaporation and 26% depression storage.

Runoff from metre-square roof samples produced slightly different percentage runoff figures for the same winter period. Average percent runoff from red Redland 49 tiles (set at 30°) was 98%, grey Stonewold tiles (set at $17\frac{1}{2}^{\circ}$) produced 85% and asphalt roofing felt produced 38% runoff. These results are evaluated in the light of probable errors in measurements.

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Chapter 1. INTRODUCTION

Recognition and investigation of man's impact on the environment has grown over the last two decades. One specific area of this broad field generating research interest is man's impact on the land phase of the hydrological cycle. The International Hydrological Decade (IHD), 1965-1974 attempted to provide a rational framework for international hydrological research by setting up sub-groups and working parties. The IHD Working Group on the influence of man on the hydrological cycle (Unesco, 1972) collected international case studies of the effects of land use change and from reviewing this impact suggested positive action under the headings of forest lands, grasslands, arable lands, irrigation and salinity, swamp drainage, landslides and road construction, the location of hazardous areas in large catchments and urbanization and water pollution. Forestry and urbanization land use changes tend to dominate the literature. Sopper & Lull (1965) reviewed the hydrological problems of forested areas while Moore & Morgan (1969) edited a more complete selection of land use changes affecting watersheds in the USA. Hollis (1979) edited a similar volume of UK papers covering most of the IHD subject headings such as drainage, reservoirs, forestry, groundwater, storm runoff, sediment control and water quality subdivided into rural and urban sections. Penman (1963) summarised an international selection of work on the effects of land use changes on hydrology.

Focussing on simply man's impact on the hydrological cycle as a result of urbanization still leaves much scope for investigation. The IHD/ Unesco Subgroup on the Effects of Urbanization on the Hydrological Environment (Unesco 1974) made recommendations for international action and set out priorities from the results of a questionnaire enquiring into the major subjects considered to need serious research and development attention within the field of urban hydrology. The current state of national programmes was presented together with special case-studies. The international co-operation started by the IHD is being continued in the International Hydrologic Programme (IHP) which has several project headings relating to urban hydrology.

Investigations into various aspects of the influence of urbanization on the hydrological cycle fall into several overlapping categories. Engineering solutions to urban storm drainage and flood control are a fairly narrow field linking with the economics of providing such schemes. The costs and benefits of providing additional safety from floods may be a political decision and flood plain zoning and planning may require legislation. Longer term water resource management of both surface and groundwater supplies needs detailed knowledge and experience to forecast future water requirements and estimate the hydrological consequences of major changes in land use. In order to improve knowledge and predictive models of both water resources and engineering drainage design, greater efforts at understanding the actual urban hydrological processes are being made. These scientific programmes examine the full range of hydrological effects caused by urbanization including changes to flood flows, runoff regimen and yield, soil erosion and sediment production, stream morphology, water quality, groundwater reservoirs and other stores and transfers of water in the hydrological cycle such as evaporation,infiltration, depression storage, and interception. These process studies are then integrated into simulation and planning models usually of only parts of the urban hydrological cycle eg groundwater abstraction forecasting or flood hydrograph simulation.

Several international and national conferences and reports have covered some of these economic, political, social, management, planning, engineering, scientific and modelling aspects of man's impact on the hydrological cycle in urban areas. The American Society of Civil Engineers (ASCE) has published recommendations for basic information needs in urban hydrology (ASCE 1969) and reviewed the effects of urbanization on low flows, total runoff, infiltration, and groundwater recharge (ASCE 1975). A very broad collection of USA papers edited by Whipple (1975) emphasised the increasing public awareness of urban runoff problems and covered most of the above topics of investigation. The presentations at a one-day workshop sponsored by the Water Resources Center at the University of Illinois (1978) covered subjects such as groundwater and surface water supplies, drainage, flood management, the quality of storm runoff water, and planning methods together with assessments of the problems and the research needed to solve them. An international Unesco-IAHS Conference (IAHS 1977) at Amsterdam limited itself to the effects of urbanization and industrialization on the hydrological regime and water quality. A national symposium on urban rainfall, runoff and sediment control held at the University of Kentucky (1974) concentrated on modelling applications to storm runoff prediction, soil erosion and sediment control in urban areas. Urban storm drainage alone was the subject matter of an international conference at Southampton. The papers concerning storm runoff control and modelling, data collection and the costs and benefits of storm water drainage systems were edited by Helliwell (1978). At the instigation of the ASCE and IHP, 12 national reports on the 'state-of-the-art' in urban catchment research and hydrological modelling have been prepared, (Unesco, 'Research on Urban Hydrology', Vol I 1977 and Vol II 1978). These describe active research projects investigating urban runoff (quantity and quality) and the resulting progress in urban hydrological modelling. McPherson and Zuidema (1978) have summarised these reports.

INFORMATION NEEDS IN URBAN HYDROLOGY

From the above reports and other work, and by limiting the scope to urban runoff quantity, the fundamental and essential information needs in urban hydrology can be stated. To improve scientific knowledge and understanding of the processes involved requires 1) more field data collection in order to 2) improve water resource management and 3) develop improved urban rainfall-runoff models.

1) Field Data Acquisition

The major finding of the ASCE Urban Water Resources Council was that the acquisition of much more field data was the greatest research need in urban hydrology. Unfortunately data requirements inevitably exceed the data available.

A lack of foresight in data collection programmes means that the current data base is often of poor quality or collected for only one purpose so that it is of severely limited transfer value. This latter fault may also partially result from the complexities of an urban landscape, (McPherson 1977).

Field data is required for a multitude of purposes such as planning,model building, theory advancement and technique development. The current data base may not satisfy requirements in these fields as Marsalek (1977), details - "The lack of urban runoff data seems to impair progress in the development, testing, verification and calibration of runoff models" while Anderson (1971), is somewhat more scathing: "Because of the lack of comprehensive urban water data collection systems, theories and techniques for urban water management have been based on fragmentary information and antiquated data." McPherson and Zuidema (1978)conclude that none of the 12 national data programmes examined were adequate in providing a national data base for validating and supporting tools of analysis. Obviously a vital need is the creation of a good urban water resources data base.

This lack of a good data base has been long recognised in the U K and the Institute of Hydrology (IOH) was established in 1968 with one of its main aims being "to obtain and develop high quality catchment data," (McCulloch 1969). More recently the IOH has embarked on an urban runoff data collection programme owing to the scarcity of hydrological data covering a wide spectrum of research interests. The data generated by this programme and other investigations into sewered catchments in the UK reviewed by Lowing (1977), have been collated into a computer-based archive, readily available to any user (Makin and Kidd 1979). A similar archive is being set up in the U.S.A. sponsored by the Environmental Protection Agency (Huber and Heaney 1978).

2) Water Balance Models And Water Resource Inventories

The use of a water balance model should provide a rational framework for data acquisition and thus avoid earlier mistakes of poorly organised programmes. Within them, individual subsystem phenomena or physical processes can be identified. Edwards and Rodda (1970) suggest that "one of the most satisfactory ways of studying (the components and processes of the hydrological cycle) is by compiling a complete water balance for a watertight drainage basin." From these studies, the quantities of water involved in each stage of the cycle can be calculated, and relationships between processes examined.

For a metropolitan area, investigation of the artificial and natural water cycles is necessary if a successful water balance inventory is to be compiled. Such a survey should include for the natural cycle, measurements of rainfall, runoff, evapotranspiration and groundwater changes; while for the artificial cycle, measurements should be made of any imports of water and their transfer by pipe systems together with an investigation of any leakages causing interaction of the 2 cycles, for instance, groundwater and storm runoff may enter foul-water sewers and foul or fresh water may leak from pipes and thus supply groundwater or rivers. Such a 2 cycle investigation has been carried out by Carlsson and Falk (1977) for the metropolitan areas of Sweden.

Water balance models have other advantages in that they are a useful tool for water resource management and planning. Unesco (1974) recommend that "More metropolitan scale water balance inventories and their analysis should be undertaken as a means for improving overall water resources planning and management (because of)the inter-relation, interdependence and interconnection of the elements of the water resources of a metropolis."

These types of study would also satisfy the ASCE Task Committee which highlighted a need for more comprehensive and more systematized investigations of hydrological changes in urban areas.

Urban water balance inventories are therefore multi-purpose since they provide a framework for both data collection and physical process studies while also being a satisfactory management tool for resource and environmental planning.

3) Urban Hydrological Modelling

While the data base remains unsatisfactory and inadequate with respect to certain applications, water resource managers and engineering designers turn to urban hydrological modelling as a means to produce a range of alternative planning outcomes or an adequate design within set constraints.

For modelling progress to be made, more data must be forthcoming as currently "the state of the art in urban hydrological modelling seems to surpass the available calibration/verification data base," (Marsalek 1977). The power and utility of even the advanced simulation models is greatly reduced in specific applications because of a lack of co-ordinated field calibration data for validating and supporting tools of analysis, (Wenzel 1975).

Further improvements in modelling would be achieved by a better knowledge of the processes involved: "the application of present methods for research in hydrology, in particular, mathematical modelling of processes of the hydrological cycle in nature is impossible without investigating their physical parameters," (Green and Dreyer 1973). McPherson (1977) adds that these physical processes must be independently verified otherwise the model remains simply "a hypothesis with respect to its internal locations and transformations."

With a detailed model, planning decisions need no longer be based on intuition, experience and poor theory but can be based on known responses, (Raoul 1978). With the data collector and modeller working as a flexible team, Lystrom and Jennings (1978) suggest that new types of data can be supplied as the urban modeller's knowledge of cause and effect progresses.

The Natural Environment Research Council (NERC) Working Party on Hydrology (1976) have identified the need to "synthesise descriptions of hydrological processes to form a model which will describe catchment behaviour .... (in order to) .... predict the effect of major changes such as .... urban development", and also suggested that the identification of the effects of urbanization are a more immediate research aim.

If this last proposal and all other information needs are to be satisfied it would appear that the ultimate goal of urban hydrological research is to produce a catchment scale model which incorporates fully field-tested processes involved in the hydrological cycle which can be used for scientific or planning purposes.

BACKGROUND AND RATIONALE

Considering the above research recommendations, a number of lines of research were suggested that would help make progress towards the proposed ultimate goal of urban hydrological research.

Existing work on urban water balance studies reveals a range of complexity in approach from the highly theoretical, where many elements are estimated (eg Lvovich and Chernogayeva (1977) for Moscow), and the simplistic, where only rainfall and runoff are measured (eg Balek (1978) for Pojbuky, Czechoslovakia and Lvovich and Chernishov (1977) for Kursk, Russia), to the very detailed measurement of several elements in the natural and artificial urban water cycles (eg Malmquist and Svensson (1977) for Goteburg, Sweden; Carlsson and Falk (1977) for Sweden and Van den Berg (1978), for Lelystad, Netherlands). Because of the as yet very experimental nature of urban water balance results, a theoretical water balance computation on the lines of the Russian work seems unjustifiable as any conclusions cannot be substantiated. Unfortunately, the results obtained from the more detailed water balance studies seem only to apply to each city or area and its unique set of conditions. More complete water resource inventories are necessary before any definite conclusions about the effect of a city on the hydrological cycle can be made and before water resource planning on the basis of these inventories is possible.

One research proposal was therefore to compile a water balance for London using available data. A basis for this part of the research project was present in the rainfall and runoff records for 5 heavily urbanized tributaries of the Thames in Greater London. Water balances from this data partially fulfilled one of the above recommended information needs in water resource management, namely "More metropolitan-scale water balance inventories and their analysis should be undertaken as a means for improving overall water resources planning and management," (Unesco 1974).

Experimental urban data collection programmes have concentrated almost exclusively on measuring rainfall and storm runoff and modelling the flood wave. The fact that the runoff volume is significantly less than 100% of the rainfall on the impermeable area has been largely ignored (eg TRRL, Watkins 1962). The losses from rainfall have been ascribed to depression storage (taken to be an initial loss) and the possibility of infiltration through the surface of roads has been recently recognised. The Wallingford Urban Subcatchment Model (Kidd and Lowing 1979) allows depression storage as an initial loss, infiltration through 'impermeable' road surfaces at a constant rate and uses a regression equation to estimate % runoff from a small urban catchment. Evaporation is considered negligible during a storm but may influence the catchment conditions inbetween rainstorms. The importance of infiltration through paved surfaces was demonstrated by Van den Berg (1978), whose water balance for a car parking lot produced 40% subsurface runoff indicating substantial infiltration through the 'impermeable' surface. Falk and Niemczynowicz (1978) found losses of 0·17% to 7·4% from asphalt road surfaces which could not be explained by depression storage and as the largest losses were from the oldest and most irregular surface, they were ascribed to infiltration. It is possible that evaporation from paved surfaces over a longer time span than during a storm is not a negligible quantity and further separation and explanation of the losses from rainfall is necessary.

A further research proposal therefore entailed the setting up of a small experimental urban catchment to satisfy another of the above recommendations by providing data to improve knowledge and understanding of processes in an urban hydrological cycle. By investigating some of the losses from rainfall through even smaller-scale process studies it proved possible to separate them and estimate their size and relative importance.

The data thus collected could be used to test and extend existing models eg the Wallingford Urban Subcatchment Model, Kidd and Lowing (1979), and to develop an urban water balance model thus ultimately aiding progress towards the proposed goal of urban hydrological research.

SUMMARY OF THESIS CONTENTS

A review of the literature and current research frontier in relation to the proposed goal for urban hydrological research (Chapters 1 and 2) suggested several new research proposals. The areas requiring greatest research attention in urban runoff quantity studies appeared to be in the fields of urban water balances and process studies so that a realistic and field-tested model of the urban hydrological cycle could be developed.

The initial project was to compile a water balance for the London area using available data (Chapter 3). Although rainfall and runoff data have been collected for many years, little analysis has been carried out and no previous attempt at a water balance has been made. The water balances for 5 heavily urbanized Thames tributaries in the Greater London area were unfortunately not very successful and annual measures of runoff and evapotranspiration from pervious surfaces only accounted for an average 70% of the incoming precipitation. The mean annual runoff from these basins ranged from 12-72% of rainfall with no statistically significant difference between the mean value and that for rural basins in south east England with similar geology and climate, ie urbanization could not be said to increase annual runoff in this area as is the usually anticipated result. Despite allowances being made for known imports of water into these rivers, it was obvious that the errors involved were large due to unquantified sources of inputs, outputs and transfers (by leakage) between the natural and artificial drainage systems.

It was considered that a better urban water balance could only be achieved by close monitoring at a small scale. Therefore a small scale urban catchment was instrumented to produce measurements of some of the elements involved in an urban water balance and to improve knowledge and understanding of some of the processes. The site chosen was a small housing estate north of London in Redbourn, Hertfordshire. Meteorological measures were recorded at a secure site close to the catchment. New instrumentation was designed to measure runoff from tiled pitched roofs while commercially produced instruments were adapted and installed to monitor runoff from a block of flat asphalt-and-chippings garage roofs, and runoff from asphalt roads and pavements at the highway drain outfall. Details of the site and instrumentation are given in Chapter 4.

To ensure the usefulness of the data collected, tests were made of the precision of the commercially made instruments which measured rainfall, evaporation and road surface runoff (Chapter 5). The weir tanks and tipping bucket instruments designed to measure pitched and flat roof runoff respectively needed calibration as well as an assessment of their precision. These tests indicated that any errors in measurement were random and of low absolute or % value (under 6%) so that the recorded measures could be considered a good estimate of the true values.

Runoff data from the 3 types of urban surfaces were collected and analysed from 88 storms occurring between October and December 1979, (Chapter 6). The most important result was that runoff from these 'impermeable' surfaces was less than 100% and average percentage runoff varied from 76% for the pitched and flat roofs to only 17% for the road surface. Road runoff volumes were thought to be exceptionally low because of infiltration through into the highly permeable chalk subsoil. This theory was supported by % runoff doubling from dry to wet antecedent conditions consistent with reduced subsurface infiltration capacity. Storm runoff volumes from the pitched and flat roofs were almost identical despite the differences in slope and surface texture. The flat roof did not allow the expected greater losses from evaporation and depression storage but if anything produced slightly higher runoff volumes. This lower runoff from the pitched roof could have been a function of a reduced rainfall catch through roof-level turbulence and eddying in comparison with the ground level reference raingauge. Depression storage for the 3 surfaces was calculated from a regression technique as 0·37 mm for the flat roof, 0·25 mm averaged for the 2 halves of the pitched roof and 0·02 mm for the road surface. This latter value seemed unacceptably low and revised depression storage values estimated from an analysis of storms which did and did not produce runoff, gave values of 0·25 mm for both roof types and 1·00 mm for the road surface. These depression storage, runoff and rainfall values were used in water balance calculations for the 3 surfaces. As it can be safely assumed there is no infiltration through roofs, the residual in the roof balances was accounted for by evaporation. Rainfall on the roofs was therefore on average distributed as 76% runoff, 19% direct evaporation and 5% depression storage. An average roof evaporation rate was calculated and applied to the road surface figures so that any residual in the water balance was due to infiltration. Rainfall on the road was distributed as 17% runoff, 21% direct evaporation, 26% depression storage and 36% infiltration.

To extend the results from the experimental catchment, a series of small scale process studies were devised (Chapter 7). A range of roofing materials were selected and installed as metre-square runoff plots under natural meteorological conditions. From the same winter period, red Redland 49 granular tiles (set at 30°) produced 98% runoff while smooth grey Stonewold tiles (set at $17\frac{1}{2}^{\circ}$) produced 85% runoff. The smoother texture of the Stonewold tiles did not counteract the gentler slope to produce similar runoff volumes from the 2 surfaces. A very low % runoff of 38% was recorded from a sample of asphalt roofing felt because of large losses from the edges. The higher runoff coefficients from these pitched roof samples in comparison with the catchment roof areas could be because of differences in the reference rainfall measurement. The catchment used a ground raingauge exposed under recommended Meteorological Office exposure conditions while the roof samples had a raingauge exposed at the same height which reflected the reduced catch of roofs caused by turbulence.

Infiltration tests were carried out on the road surface at the catchment site. Results suggest that the road had an infiltration capacity in excess of 14 mm/hr but more detailed experiments are needed to confirm this and measurements of ambient meteorological conditions are necessary to assess the evaporation rate included in this figure.

This thesis contributes to knowledge about urban hydrology in several of the recommended research areas. Employing a range of respected techniques in hydrology and applying them specifically to an urban situation has produced either results contradictory to previous work or suggested that some processes have a larger importance than previously suspected. In addition, the development of new measuring systems has allowed thorough investigation of some processes for the first time.

Thus while water balance compilations are well documented for rural catchments, they have rarely been calculated for large metropolitan areas, and never for London. Those few studies carried out seem to indicate substantial increases in annual discharge as a result of urbanization, with some of the largest proposed increases coming from the more theoretical studies (Lvovich and Chernogayeva (1977) and James (1965). However, the findings of this investigation are in contradiction to this. The average annual yield from these heavily urbanized catchments is not significantly different from that produced by rural catchments on similar geology experiencing the same climate. The humid climate may be partially responsible for the minimal changes in yield as suggested by Hollis (1977) working in Essex. Although no great confidence can be placed in the London data this may be a true result and care should be taken if predicting large increases in runoff as a result of urbanization.

The monitoring of a very small urban catchment while not a new concept has rather different data collection aims in this case. An emphasis in previous similar studies has been on the measurement and reproduction of storm peak flows and hydrographs to produce ultimately a sewer design method or simulation model (eg Watkins 1962; Kidd and Lowing 1979). These studies used only the fairly high intensity rainstorms and corresponding peak flows and measured runoff from combined roads and roofs. In this thesis, the emphasis is placed on measuring runoff volume accurately from the whole range of rainstorms regardless of intensity or size as this should give a more complete picture of each urban surfaces response to rainfall. Novel methods of measurement allowed separate monitoring of pitched and flat roofs and road runoff and demonstrated that, on this catchment at least, roof runoff would form the largest contribution to total runoff with important consequences for possible pollutant dilution. Measurement proved that the road surface was not impermeable and that further separation of the losses from rainfall into evaporation, depression storage and infiltration (through roads) is possible. A new estimation technique for depression storage using low rainfalls that did and did not produce runoff from each surface gave slightly higher values from the road surface (1·0 mm) than suggested by other work (eg Kidd, 1978b - 0·67 mm), and gave estimates of depression storage from pitched and flat roofs from actual data (0·25 mm for both roof types). A water balance calculation allowed the estimation of evaporation as the residual from the roof water balances and proved that contrary to expectations, (Stoneham and Kidd 1977; Van den Berg 1978), it is not a negligible quantity but accounted for 19% of the rainfall. Employing an average roof evaporation rate in the road surface water balance allowed the estimation of infiltration as the residual and largest single loss - 36% of rainfall. The high infiltration makes the runoff from the roads of minor significance - 17% of rainfall - and demonstrates the possible importance of chalk subsoil in aiding infiltration, so this runoff coefficient is excessively low in comparison with other studies carried out on other soil types. The actual infiltration rate was also tested experimentally and discovered to be in the order of 14 mm/hr.

Further process studies on samples of roof materials exposed under natural meteorological conditions gave further support to the runoff coefficients calculated from the monitored catchment. Although similar micro-scale studies have been and are being carried out at present (Trent Polytechnic) these all rely on laboratory facilities and artificial simulation of rainfall, a technique which can only reproduce the higher intensity storms. Investigations using the full range of meteorological conditions influencing the urban fabric are perhaps more justifiable and conclusive.

The results from the 3 scales of experimentation are summarised in Chapter 8. Suggestions for further work include the extension of these urban hydrological process studies so as to provide a wider range of information from a wider set of surfaces that can ultimately be used in an urban water balance model.

Chapter 2 DIRECTIONS OF RESEARCH ACTIVITY IN URBAN HYDROLOGY

The general effects of urbanization on the hydrological cycle are fairly well understood, but detailed research into the reality of these effects and their mechanisms is still required. In 1961, Savini and Kammerer recommended that "there is a need for a program directed towards investigation of the effects of urbanization on hydrology," but more than a decade later Unesco, (1974) could still write "Because of the increase in complexity of water problems brought about by urbanization, there is a need for more comprehensive and more systematic investigation of hydrological changes in urban areas." Only piecemeal progress has been made. While man's influence on water quality is dramatic and important, this review of urban hydrology research will be limited to the equally important evaluation of the effects of hydrological changes induced by urbanization on water quantity which will be vital for the development of strategies for resource management and environmental planning, (Jens and McPherson 1964; McPherson 1977).

From the survey in Chapter 1 of the basic information needs in urban hydrology, 3 main recommendations were made. The most important of these was the requirement of more field data acquisition because of its determining influence on model building and theory advancement. Water balance inventories and models were seen as a suitable rational framework for this data collection and for physical process studies while also being a satisfactory management tool for resource and environmental planning. Improvements in urban hydrological modelling were also recommended but as these depend on the available calibration and verification data base the modeller and data collector must work together.

It was proposed that if these information needs in urban hydrology are to be satisfied, the ultimate goal of urban hydrological research is to produce a catchment scale model incorporating fully field-tested processes involved in the hydrological cycle which can be used for scientific or planning purposes. Existing urban hydrological research is therefore reviewed with reference to this proposed goal.

THE CURRENT POSITION OF URBAN HYDROLOGICAL RESEARCH

An initial survey of the literature involved with urban hydrology revealed several apparently unrelated areas of research. Many engineering design methods were highly mathematical in approach and seemed far removed from the simpler urban water balance studies. The scale of approach also ranged from the complete catchment down to the individual process. To try and make sense of the achievements to date and to identify those areas requiring greater research attention, a closer examination showed that while some of the areas remained distinct, other branches of the subject had grown from another so that there was a partial chronological order to reach some of the current research fields. For instance, early work in the U.S.A. on urbanizing catchments used comparative data from urbanized and control rural basins to try and detect the effect of urbanization on yield and storm runoff. This work has now been largely superceded by attempts to synthesise these results to give predictive equations of the effect of urbanization on flood flow based on one or two catchment characteristics. This area is the still prolific field of unit hydrograph and conceptual model techniques. Dissatisfaction with the limited applicability of these results to other regions has led to the development of more sophisticated models which simulate a range of processes in the urban hydrological cycle. Much effort is being currently expended on individual process studies such as overland flow and depression storage which once successfully measured and adequately modelled can be transplanted back into the catchment scale models. The models need to have wide applicability so as to not only simulate but also predict the effects of urbanization. These effects have longer term influence on the use of water resources and planning and water balance inventories are now being compiled to aid these decisions.

As a visual and academic aid to understanding the directions of research in urban hydrology a table has been constructed along the partially chronological and dendritic lines outlined above (Table 2.1). The research quoted in it is limited to quantitative studies and does not include the recent developments in urban runoff quality measurement and modelling which form a separate field of research. The current research frontier is towards the foot of the table within the "Sewered Urban Catchment Models", "Processes" and "Urban Water Balance" sections. The apparent deficiency in the most recent work is caused by the burgeoning of work in the "Processes" section and a consequent lack of space on the table. These studies are dealt with in full in the text.

A fuller description of the separation of studies into each research category follows. The individual studies in each category are then discussed and summaries made of the main research conclusions and the chain of progress to another category detailed.

THE STRUCTURE OF RESEARCH ACTIVITY IN URBAN HYDROLOGY (TABLE 2.1)

Urban hydrological research falls into several categories, some of which

ACTIVITY IN URBAN HYDROLOGY

SEWERED URBAN CATCHMENT MODELS (flow through pipes simulated)	PROCESSES (In order of frequency of use of hydrological cycle elements)		URBAN WATER BALANCES (In order of number of measured elements)
STORM RUNOFF	ESTIMATED	MEASURED	
		Precipitation	Lvovich & Chernogayeva 1977
- RATIONAL (Lloyd-Davies 1906) INLET TIME-AREA SARGINSON METHOD (Sarginson 1973, Nussey & Sarginson 1978) - UNIT HYDROGRAPH (Eagleson 1962) TRRL (Watkins 1962)	Runoff / Sewer Flow		
		Runoff	Balek 1978 Lvovich & Chernishov 1977 Lull & Sopper 1969
	Evapotranspiration		
ILLUDAS (Terstriep & Stall 1974)	Soil Infiltration		
MIT (Leclerc & Schaake 1973)) CHICAGO HYDROGRAPH (Tholin & Keifer 1960)) UCUR (Papadakis & Preul 1972)) LOS ANGELES HYDROGRAPH (Hicks 1944))	(Road Gutter Flow (Gutter Storage (Overland Flow (Depression Storage		
WALLINGFORD URBAN SUBCATCHMENT MODEL (Kidd & Lowing 1979)	Soil Moisture Deficit		
SWMM (Metcalf & Eddy Inc. 1971)		Groundwater Levels	Franke & McClymonds 1972
HSP (Crawford 1971)		Pan Evaporation	
		Soil Moisture	
		Soil Infiltration	
		Storm Water Flow	Van den Berg 1978 Malmquist & Svennson 1977
		Atmospheric Humidity	
		Pipe Leakage	Carlsson & Falk 1977
		Imported Water	
		Abstracted Water	
	Terra Incognita	(Overland Flow (Roof Gutter Flow (Interception & Absorption by buildings (Dew & Hoar Frost (Depression Storage (Anthropogenic Moisture Sources (Infiltration through 'Impermeable' Surfaces (Paved Surface Evaporation	

Increasing Generalisation and Ease of Application of Methods necessarily overlap. Table 2.1 illustrates the structure of this research. The table is ordered vertically downwards according to the increasing number of processes (either estimated or measured) used by each model or study. Therefore the more sophisticated models fall nearer to the bottom of the table (eg HSP: Crawford (1971) and Carlsson and Falk (1977)) and consequently nearer to the proposed goal of this research. The table is also split according to the main research interest - either using actual catchment data (Urbanizing Catchment Studies and Urban Water Balances) or attempting to model one or more of the processes involved in an urban hydrological cycle (Urbanized Catchment Models and Sewered Urban Catchment Models). Urbanizing Catchment Studies use comparative data to detect the effect of urbanization on runoff. Interest is divided between measuring the increased annual discharge (yield) and the changes to storm runoff. Attempts to predict these changes to storm runoff as defined by catchment characteristics are detailed in Urbanized Catchment Models while Sewered Urban Catchment Models attempt to simulate flow through pipes rather than open channels (some models may do both) and may simulate more processes although storm runoff is still the main focus of research attention. Urban Water Balances generally incorporate more processes and attempt to quantify more components of the hydrological cycle. They can consequently produce the most complex models of all. Fuller details of each section follow.

The process scale is ordered on the basis of decreasing knowledge of the actual quantity of water involved and decreasing knowledge of the process itself. Therefore, rainfall is the most easily measured quantity while the set of processes labelled collectively "Terra

Incognita" at the bottom of the table have unknown quantities and are poorly simulated (if at all) by current models. Suggestions for future research in this area are detailed later.

URBANIZING CATCHMENT STUDIES use comparative catchment data to detect the effect of urbanization on runoff - either comparing an urbanized basin with a nearby rural one which acts as a control or detailing the "before and after" effects of urbanization on one catchment. The only elements measured are rainfall and runoff. The references are a representative selection of this type of research and are simply listed in chronological order on Table 2.1.

The studies can be subdivided into firstly, yield and regimen studies which examine the increases in annual percentage runoff and total discharge resulting from urbanization, and secondly, storm runoff studies which document the often dramatic increases in peak flood flow and storm runoff volume from urban areas and other changes in hydrograph characteristics such as time to peak and flood duration.

Yield And Regimen

The extensive paving of a catchment will reduce infiltration of rainfall into the ground. This will produce reduced rates of evapotranspiration owing to the smaller areas of vegetation and soil. Recharge of groundwater will be halted so that the low flows of streams will diminish but become more frequent. As a result of increased percentages of precipitation becoming direct runoff, the total annual flow of streams is likely to be increased. In general, the regime of the rivers should become more 'flashy' with frequent and rapid changes from a low flow situation to moderate or high flow and back again. A fuller qualitative account of these expected changes is given by Savini and Kammerer (1961).

Three studies have been conducted in California on urbanization and water yield. Harris and Rantz (1964) using double mass analysis on data from the upstream rural section and from the downstream partly urbanized catchment of Permanente Creek, discovered a four fold increase in flow as a result of urbanization. This supported earlier findings by Waananen (1961) on the same creek where intensive urban development caused an increase in out-flow from the area from 76% of the inflow 1939-1955 to 123% of the inflow 1956-1958. Crippen and Waananen (1969) examined the effects of surburban development on Sharon Creek, Palo Alto, California using the rural Los Trancos Creek tributary as a control. Irrigation of the area caused the ephemeral stream to become perennial while on average 30% of the rainfall became runoff compared with 5-10% before development - an increase of between 3 to 6 times in yield.

Studies in more humid climates have been conducted in America and England. Ramey (1959) reported flows increasing 2½ times in the Chicago area over a 30 year period. Waananen (1969) detailed Stall and Smith's work comparing the 38% urban Boneyard Creek with the rural Kaskaskia River near Urbana, Illinois, which indicated a doubling of total runoff from the urban basin for dry years, but near equal annual discharge for wet years. Sawyer (1963) reported an increase in total runoff of 15·8% from the urbanized East Meadow Brook, Long Island, New York compared with an increase of only 7.2% in total runoff for the nearby rural Mill Neck Creek following a 9·4% increase in precipitation over the study period. Hollis

(1977) working on the Canon's Brook, Harlow, Essex calculated a 30% increase in water yield (1·3 times the pre-urban quantity) as a result of urbanization.

Overall, urbanization does increase yield. The range of increases in annual discharge is from 1·3 to 6 times the rural equivalent and Hollis (1977) suggests that climate may influence this increase. Urbanization in a humid climate (Essex) may produce minimal changes in yield, whilst the same type of development in a drier clime (California) would increase the total stream-flow significantly. However, this larger increase in yield may be entirely man-made as a result of importation of water rather than increased percentages of rainfall becoming runoff. Little work has been done on flow regimes after urbanization because of the short records but work by Hollis (1977) suggests that very low flows decrease in frequency and are superceded by slightly larger low flows. This is contrary to expectations that low flows should decrease in both volume and frequency and the likely explanation is the increased ressponse to even small rainstorms.

Storm Runoff

The most noticeable and drastic changes brought about by urbanization are the effects on storm runoff. Urban storm water drainage collection systems reduce the amount of time taken for rain falling on a built-up area to reach the gutter inlets and the smooth concrete pipes speed up underground flow in comparison with natural drainage channels so that the time characteristics of the hydrograph, such as the lag time and the time base, are shortened. The larger volumes of runoff and greater velocities of flow result in higher peak rates of runoff. However, the effect of urbanization decreases as the flood magnitude increases, as at higher rainfall intensities, infiltration capacities are often exceeded so that rural and urban catchments tend to react in the same manner. A number of studies have addressed the measurement of the above anticipated changes in basin runoff response.

Reductions in lag time (the time interval between the centroids of rainfall and direct runoff) range from 50% as recorded by Gregory (1974) working on the urbanizing Rosebarn catchment on the margin of Exeter to 90% reduction from the Sharon Creek basin, Palo Alto, California (Crippen and Waananen 1969). Other work suggests very high reduction factors. Wiitala (1961) working on the Red Run and Plum Brook basins at Detroit demonstrated that lag time in the sewered basin was reduced from 9·5 to 3 hours - a reduction of approximately 70%. Stall and Smith (reported by Waananen 1969) found that the time of concentration in Boneyard Creek, Illinois was 50 minutes compared with the 3hrs 45 mins. for nearby rural Kaskaskia River. Anderson (1970) recorded a reduction in lag time of as much as 85% in his study near Fairfax, Virginia.

Increases in the volume of direct runoff are equally dramatic but more variable. From analysis of one single storm, Mills (1968) calculated 92% of the rainfall became runoff from a wholly urbanized basin within Dallas, Texas compared with 85% from a partly urbanized basin nearby. Crippen and Waananen (1969) found that 58% of the rain became runoff at Sharon Creek, Palo Alto, California while the rural control basin produced only 36% runoff from the same storm. Syntheses of several storms by Sawyer (1963) showed that urbanization of East Meadow Brook, Long Island, New York, increased direct storm runoff by 123·1% compared with a 6.1% increase for the nearby control basin as a result of a 9.4% increase in precipitation. While Seaburn (1969), working on the same basin found an increase in direct runoff of 27% following an increase in the sewered area. Gregory (1974) found runoff volumes increased by factors of 1·1 to 3 and an increase in % runoff from precipitation of 0·9% (low intensity storms) to 22·7% (high intensity storms) for an urbanizing catchment in Exeter. Taylor (1977) discovered seasonal differences in direct runoff volumes between urban and rural portions of a catchment in Peterborough, Ontario. Summer rainstorms in the urban area produce runoff volumes 1·2 x the rural value; a 2·3 x increase for autumn rainstorms and a 7·5 x increase for spring snowmelt and rain-on-snow events.

The effect of urbanization on flood magnitude and frequency seems more marked for lower magnitude high frequency events but the range of recorded increases is wide. Hollis (1974) records increased peak flows of from 1·5 to 11·5 times those recorded before urbanization for storms with a return period of 5 years or less on the Canon's Brook, Harlow, Essex. Taylor (1977) also noted seasonal effects on storm peak increases. The effects of urban development in Peterborough, Ontario were most strongly felt in the spring under snowmelt conditions. (7·1 times increase) and least strongly in the summer (4·5 times increase).

The mean annual flood for Red Run, an urban watershed, was found to be 3 times as large as that indicated from a flood frequency study for an undeveloped basin (Wiitala 1961). Waananen (1961) used records from Onondaga Creek, Syracuse, New York to calculate that urban runoff peaks were on average 3 x greater than the average of corresponding peaks of regulated flow from Onondaga Reservoir. Wilson (1967) found that the mean annual peak flows were 2-3 times greater for urban streams near Jackson, Mississippi, than for streams in adjacent rural areas. Gregory (1974), working at Exeter, recorded increased peak discharges approximately double those recorded before further residential development had occurred. Cech and Assaf (1976) compared patterns of annual peak runoff within the Houston-Galveston Bay area and discovered the greatest differences in runoff occurring in regions of most intense urbanization and industry with peak flows increasing 3-5 times the background magnitude. From these studies of high probability events such as the mean annual flood, the storm peak is increased by an average factor of 3 times the equivalent rural value. However, work by Stall and Smith (reported by Waananen 1969) in East Central Illinois records mean annual flood peak flows up to 8 times the rural value.

The effect of urbanization on high magnitude low frequency events is less marked. Hollis (1974) working in Harlow, Essex discovered that urbanization had little effect on storm peak magnitude for events with a recurrence interval of 20 years or more. Stall and Smith calculated a quadrupling of the storm peak for the 50 year flood from Boneyard Creek, Illinois, while Wilson (1967) estimated a 3 fold increase in the 50 year flood peak flows for a fully urbanized catchment near Jackson,Mississippi. A USGS study on Little Sugar Creek, North Carolina, reported by the ASCE Task Force (1969) calculated a reduced influence by urbanization on higher return period floods. Urbanization had increased the mean annual flood by 58%,the 10 year flood by 30% and the 20 year flood by 17%. Skelton (1972) reported thaturbanization had a reduced influence on greater frequency storms from St. Louis, Missouri - he discovered the

25 year flood to be 2·4 times greater than the 2 year flood, whereas the same flood from a rural basin was about 3·4 times the 2 year flood.

Attempts to make generalisations about the effects of urbanization on floods have been made by Leopold (1968) the ASCE Task Force (1969) and more recently by Riordan, Grigg and Hiller (1978). Although the information gathered by the many researchers is informative, the variability of the results means that no definite answers to questions of the effects of urbanization are available. The works substantiate the notion of increasing peak storm water runoff with increasing urbanization with a rough ratio of 3 times the equivalent rural mean annual flood but with a range of 1·5 to 11·5. There is less (or little) effect on the greater magnitude low frequency storms. Lag time is shortened by between 50% and 90%, but there are too few works to attempt an average value. The small quantity of information and even greater variability of storm runoff volume means that one can still only generalise and say that the volume is usually increased by urbanization.

URBANIZED CATCHMENT MODELS have been developed to try and predict the effect of urbanization on flood flow. Storm runoff therefore provides a linking theme between the 2 categories of research. These studies may use actual catchment data but the aim is to generalise the flood statistics to give a simple model using 1 or 2 catchment characteristics such as the % urban area or % paved area to represent the extent of sewered areas. These simple relationships can then be applied to ungauged catchments or used for long term prediction of flow regime (which is impossible from the generally short hydrometric records from urban catchments), or used for evaluating the effect of alternative development schemes (ie a planning model). Because these largely empirical methods are fitted to observed rainfall-runoff data, there may be errors when the model is applied to non-comparable conditions.

One of the earliest models is that by Bigwood and Thomas (1955) who developed a flood flow formula for Connecticut. The mean annual flood was predicted from basin area and slope and a "watershed coefficient," describing the degree of imperviousness. Carter (1961) proposed a method for determining the effect of urbanization on the mean annual flood, which was extended by Anderson (1970) to include estimation of the urbanized growth curve. Based on 44 catchments in the Washington-Northern Virginia area, the mean annual flood was calculated from basin area and lag time (dependent on basin slope and channel length) and a coefficient of imperviousness.

Martens (1968) employed the same approach as Carter for determining the mean annual flood and extended his analysis to floods of higher recurrence interval on the basis of simple ratios - if the 50 year peak rainfall intensity for a given duration were twice the mean annual peak rainfall, the 50 year flood for a given urban catchment would be twice its mean annual flood. Putnam (1971) extended Anderson's and Marten's work on defining lag time for ungauged catchments and showed that only one curve (and not a family) was necessary.

The Rational or Lloyd-Davies (1906) method is a well-established pipe-design method which gives an estimate of the peak rate of runoff from a given catchment (or subcatchment) due to a rainstorm of given average intensity. There is no information on hydrograph shape. It is usually expressed in the form:-

$$Q = CiA$$

Where Q is the peak runoff rate; C is a runoff coefficient; i is the design rainfall intensity and A is the catchment area. There are published tables of the coefficient C which in urban areas primarily takes account of the percentage of directly connected impermeable area. Experience is necessary for the satisfactory evaluation of this coefficient and in applying the model generally. Its accuracy is however questionable as its simplicity cannot adequately represent the physical processes involved. It has been widely used to design pipe-sizes in sewered catchments but rarely considered as a tool to predict the effects of progressive urbanization on a catchment. Da Costa (1970) modified the hydrograph and peak discharge according to the degree of urbanization - the runoff coefficient C, being a function of the degree of imperviousness and artificial canalization. McCuen and Piper (1975) used the Rational method to compare with the results they derived from their "linked process hydrologic model" which estimated the hydrologic impact of various land use configurations and stormwater management practices.

In order to be able to predict the post-urbanization hydrograph shape, unit hydrograph theory has been applied quite extensively. This is one form of a linear reservoir model first proposed by Sherman (1932). The unit hydrograph may be defined as the surface runoff hydrograph resulting from a unit depth of excess rainfall generated uniformly over the catchment area at a constant rate during a specified period. Because of the scarcity of flow records spanning urbanization within a catchment area, rainfall and river flow records from catchment areas at different stages of urban development within the same hydrologically homogeneous region have been used to derive unit hydrographs of a pre-determined duration. Parameters describing the shape of the unit hydrograph are then correlated with catchment characteristics and extent of urbanized area. Van Sickle (1969) presented a basin factor which depends on a length-slope relationship and the degree of development of the catchment. This also provided the best correlation with peak discharge and time to peak parameters. The method proposed by Espey, Winslow and Morgan (1969) has gained general acceptance. Multiple linear regression analysis of data from 33 urban and 17 rural catchments in Texas defined the 30 minute unit hydrograph characteristics of peak and rise time. The independent variables used were catchment area, main channel length, main channel slope, % impervious cover and a channel roughness factor. Their equations can be used to estimate pre- and post-urbanization unit hydrograph parameters. Crippen (1965) has also used unit hydrographs to represent conditions before and after suburban development on Sharon Creek, California.

A further simplification of the unit hydrograph concept was put forward by Rao, Delleur and Sarma (1972). Their instantaneous unit hydrograph (IUH) method was based on storm events from 5 rural and 8 urban catchments in Indiana and Texas. They concluded that for catchments smaller than 13 km^2 the single linear reservoir model gave an adequate description of the response hydrograph but for larger areas, the Nash cascade of linear reservoirs was preferable.

Using records from a group of urbanized catchment areas in the South East of England, Hall (1973) derived finite period unit hydrographs (TUH) and showed that their response could be described successfully by a one-parameter dimensionless one-hour unit hydrograph. The parameter, lag time, was a function of the length and slope of the main channel and the % impervious area. Later work by Packman (1974) and Hall (1977) on 2 adjacent catchment areas within North London found that the original definition of lag time was inadequate to explain the variations in lag time as the catchments have undergone urbanization. The explanation may lie with the type of urban development where urban infilling makes little difference to the drainage network and runoff response and because the sewerage system may contain additional storage which would account for the underestimate of predicted lag times.

The unit hydrograph studies above use a rainfall separation technique to derive excess rainfall that bears little relationship to the actual processes operating. A more physically-based separation of these losses from rainfall is attempted in unit hydrograph studies by Brater (1968) and Brater and Sangal (1969) on watersheds in the Detroit metropolitan area. An initial value for retention (defined to include interception and depression storage) is abstracted and the infiltration process is modelled for the pervious portions of the catchment. Hydrograph studies by Viessman (1966) and Willeke (1966) carry the separation further and suggest simple models for the calculation of depression storage. A fuller discussion of losses from rainfall is contained in the next section. Because of the more physically-based treatment of the runoff process, the studies by these workers are situated at the level of the process they attempt to model on Table 2.1 – ie soil infiltration and depression storage.

All the above methods for predicting the changes in flood response of a catchment undergoing urbanization rely on hydrometric data from catchments in the proposed region of study at the various stages in urban development and apply linear regression analysis assuming that suitable catchment parameters can be used as the independant variables to describe these changes. The regional regression equations therefore apply only to the original area of study and have limited transfer value, (eg Hall 1973 and 1977).

More complex models which require computer solution have been developed and the techniques they use are discussed in the following "Sewered Urban Catchment Models" section. One particular application of such a model - The Stanford Watershed Model (Crawford and Linsley 1966), the original rural version of the Hydrologic Simulation Program (HSP), (Crawford 1971), - is that by James (1965) who generates long term continous hydrographs (1905-1963) for a wholly rural and wholly urbanized situation for Morrison Creek, Sacramento Valley, California. The model simulates depression storage, infiltration and interception losses and routes overland and channel flow. It requires extensive field data to calibrate the model to the particular catchment and, because of its water balance computation capabilities, requires pan evaporation figures - a rare model requirement. Other equally complex physically-based simulation models exist but their ability to estimate the effect of urbanization on flood fow has not been adequately tested against real data.

SEWERED URBAN CATCHMENT MODELS are generally more complex models of the urban rainfall-runoff process which have the same aims as the previous section's models-notably determining the peak flood flow and hydrograph characteristics - but attempting to simulate this flow through storm sewer pipe systems for fully sewered catchments rather than open channels (some models having the capability to do both). They also can simulate the response from subcatchments rather than producing a single hydrograph at the design point. The models range from the simpler design models which require few catchment statistics to the highly complex continuous simulation models. As the proposed goal for urban hydrological research is a catchment-scale model with fully field-tested processes, the models using the greatest number of physically-based processes are considered nearest this goal. On Table 2.1 the Storm Water Management Model (SWMM) and Hydrologic Simulation Program (HSP) models which simulate a continuous water balance use the greatest number of catchment parameters and estimated physical processes and therefore come closest to this proposed goal. A full review of these models and their applications is supplied by Torno (1975) and Colyer and Pethick (1977).

The rainfall-runoff processes over an urban catchment are complex so it is necessary to represent the processes by simpler concepts. It is appropriate to discuss the more frequently used ways of estimating or modelling those processes which remain inadequately measured and the attempts to derive more general descriptions of the catchment processes. The following sections provide a description of the processes involved and detail the techniques used to simulate them. A comparison of model characteristics is given in Table 2.2 which is ordered, as before, in terms of increasing number of processes used and according to their

Table 2.2 Comparison of Model Characteristics

Model	Author	MA	Losses: DS	Losses: Infiltration	Losses: ET	Main Loss Model	Overland Flow Routing	Channel/Pipe Routing
Rational	Lloyd-Davies 1906	L	-	-	-	%	Te	T_F
Inlet	Kaltenbach 1963	L	-	-	-	%	UH	UH
Time Area		L	-	-	-	-	Te	T_F
Sarginson a	Sarginson 1973	L	c	-	-	Φ	SR	SR
" b	Nussey & Sarginson 1978	L	c	-	-	$\propto$	SR	SR
Unit Hydrograph		L	c	-	-	V	UH	UH
TRRL	Watkins 1962	L	c	-	-	%	Time-Area	SR
Illudas	Terstriep & Stall 1974	L	c	Holtan	-	V	Time-Area	SR
Non-Linear) Urban Runoff)	Kidd & Helliwell 1977	L	c	-	-	Φ	SR	KW
MIT	Leclerc & Schaake 1973	D	c	Eagleson	P	V	KW	KW
Chicago Hydrograph	Tholin & Keifer 1960	D	c	Horton	-	V	SR	Time-Offset
UCUR	Papadakis & Preul 1972	D	L	Horton	-	V	SR	Time-Offset
Los Angeles Hydrograph	Hicks 1944	D	c	Local Experiment	-	$\propto$	UH	T_F
Wallingford Urban) Subcatchment)	Kidd & Lowing 1979	L	c	-	SMD	$\propto$	SR	-
SWMM	Metcalf & Eddy 1971	D	c	Horton	-	V	SR	KW
HSP	Crawford 1971	C	L	Cumulative	EP	$\bar{V}$	SR	KW

(MA) Modelling Approach:
L = Lumped parameter (conceptual) models
D = Distributed parameter (physically-based) models
C = continuous simulation models

Main Loss-Model:
% = runoff as a % of rainfall
Φ = Phi-Index, constant loss
$\propto$ = constant proportional loss
V = variable proportional loss

Routing:
Te = Time of Entry
T_F = Time of Flow
UH = Unit Hydrograph
SR = Storage Routing
KW = Kinematic Wave

Losses:
DS = Depression storage
L = Linsley
c = a constant value, initially abstracted
ET = Evapotranspiration
P = Penman
SMD = Soil moisture deficit
EP = Evaporation pan

See text for fuller explanations.

treatment of losses from rainfall and flow routing.

Modelling Approach:

Modelling of these processes has 2 broad approaches, the conceptual (or lumped parameter) and the deterministic, physically-based (or distributed parameter) model. The deterministic model attempts to simulate all the particular aspects of a process while the conceptual model attempts to simulate the process by a simpler concept of the physical system, using a reduced number of parameters. The deterministic approach to modelling requires a heavy data collection programme and much computing time. It has therefore been more generally used in the USA where the uniform urban design means simulation of one urban standard "block" can be repeated many times (eg Chicago Hydrograph, Tholin and Keifer 1960; UCUR, Papadakis and Preul 1972). With appropriate inputs the deterministic model can provide a continuous simulation for a whole catchment (eg HSP, Crawford 1971). The conceptual model with its smaller data requirements is more suitable for urban design purposes (eg TRRL, Watkins 1962; Wallingford Urban Subcatchment Model, Kidd and Lowing 1979).

Modelling of the rainfall-runoff process generally has 2 stages firstly, a determination of the volume of runoff after extraction of losses from rainfall, and secondly, routing of the runoff first overland and then through a pipe system. The losses to be abstracted from rainfall are broken down into interception, wetting, depression storage, infiltration and evapotranspiration.

Hydrological Losses

1. Interception:

Interception is the precipitation caught by vegetation and buildings before it reaches the ground. No estimates or measurements of rainfall intercepted by buildings have been made because it is considered a negligible quantity in fully-sewered catchments with little vegetation (Stoneham and Kidd 1977). Because of the general lack of interception data, the common practice is to deduct an estimated interception volume from the beginning of a storm as part of an initial abstraction. This however ignores the absorptive capacity of bricks and tiles of the urban fabric (Givoni 1969).

In forested areas, interception was also considered a negligible quantity until it was experimentally measured. Recent repeats of Law's 1957 experiments by the IOH (IOH, report 33, 1976) showed that up to 45% of the rainfall was intercepted by the pine tree canopy in a study of the headwaters of the Wye and Severn rivers. A high building density with high-rise blocks may also intercept large quantities of rainfall and this possibility should be investigated.

2. Wetting/Surface Detention:

For runoff to occur, a thin film of water must first be built up - surface detention - which overcomes surface tension effects. Jacobsen and Falk (1979) report work by Pecher which ascribes values for surface wetting of 0·2 to 0·5 mm for impervious areas and 0·2 to 2·0 mm for pervious areas. This loss is usually abstracted as part of a set of initial losses and is not considered separately in most models.

3. Depression Storage:

Depression storage is that volume of water retained on the ground surface in minor depressions until it evaporates or infiltrates. The magnitude of depression storage on impermeable surfaces has not been directly measured because of its obvious great variability. In modelling depression storage, in most cases, a constant value is used which will also include interception and wetting losses (eg Hicks 1944; Tholin and Keifer 1960; Papadakis and Preul 1972).

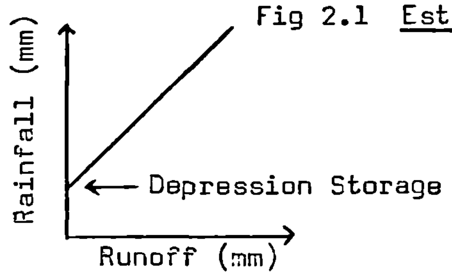
Linsley, Kohler and Paulhus (1949) indicated that the volume of water stored by surface depressions can be regarded as a process varying with time using:

$$V = S_D \ (1 - e^{-k P_e})$$

Where V is volume in storage at a specific time; P_e is rainfall excess; and k is a constant equivalent to $1/S_D$. This relationship has been used by Papadakis and Preul (1972) and Crawford (1971).

Recent work on the size of depression storage on impermeable surfaces suggests a strong relationship with ground slope, (Falk and Niemczynowicz 1978; and Kidd 1978$_a$). Rainfall and runoff volumes were plotted and the intercept of the regression line with the rainfall axis considered to represent depression storage (Fig 2.1).

Fig 2.1 Estimation of Depression Storage



This gives a linear relationship between depression storage (DS) and slope (%) of the form

$$DS = C - K \times SLOPE \quad \text{where C and K are constants.}$$

This is similar to early work by Viessman (1966) and Willeke (1966) on small runoff plots.

A combination of the Swedish (Falk and Niemczynowicz 1978), English (Kidd 1978a) and Dutch (Van den Berg 1978) work on depression storage yielded a non-linear relationship between depression storage and slope (Kidd 1978b):-

$$DS = 0{\cdot}77\ SLOPE^{-0{\cdot}49} \qquad (r = 0{\cdot}85)$$

from data giving DS values of 1·5 to 0·13 mm for slopes of 0·25 to 4·0%.

4. Infiltration:

Infiltration through pervious surfaces has been described by several hydrologic models that take account of such variables as the pervious area, rainfall duration, amount and intensity, soil structure and texture, any vegetation cover and long term antecedent wetness of the catchment.

The most generally adopted formula for infiltration is Horton's negative exponential function (1940), for instance used by Tholin and Keifer (1960); Papadakis and Preul (1972); Metcalf and Eddy Inc. et al (1971).

$$f = fc + (fo - fc)\ e^{-kt}$$

Where f is infiltration capacity at time t; fo and fc are the initial and final infiltration rates and k is a constant, a function of soil and vegetation.

Other formulae include that by Holtan (1961) who based an equation on the premise that soil moisture storage, surface-connected porosity and the effect of root paths are the dominant factors influencing infiltration rate. Philip (1957) and Green and Ampt (1911) both employ soil constants to describe infiltration capacity while the Massachusetts Institute of Technology (MIT) model (Leclerc and Schaake 1973) uses filter theory following work by Harley, Perkins and Eagleson (1970).

Hicks (1944) based infiltration losses on experimental data from the local urban area for his Los Angeles Hydrograph method. The HSP continuous simulation model (Crawford 1971) has to employ cumulative infiltration rates.

Stoneham and Kidd (1977) suggest that impervious surfaces, as traditionally understood, may not live up to their description. Small cracks, joints or faults or even the nature of the surface itself may allow infiltration. The extent of this loss will depend as much on local engineering practices as on the meteorological conditions prevailing. For instance, a yearly water balance calculated for a Dutch parking lot (Van den Berg, 1978), shows that 40% of the incoming rainfall infiltrates the paved surface, probably due to the fact that about 80% of the area is laid with bricks. Conversely, Swedish use of asphalt (Falk and Niemczynowicz 1978) gives very low losses ranging from 0·17% of rainfall on the newest surface to 7·4% from an older surface. The nature and temporal distribution of this infiltration is difficult to determine but is assumed to be constant (Jacobsen and Falk 1979; Stoneham and Kidd 1977). This is done on the assumption that the permeability of the underlying pervious material is far larger than that of the asphalt. This means that no decay in infiltration rate with time may be expected during rainfall.

5. Evaporation and Transpiration:

Various attempts have been made to satisfactorily model evapotranspiration both by hardware scale models (evaporation pans and lysimeters) and mathematical approaches based on theories of the operation of the process. Only one simulation model, the HSP (Crawford 1971) uses evaporation pan coefficients as it produces a continuous water balance rather than a single storm hydrograph as do most other urban hydrological models.

The usual method of estimating evapotranspiration is indirect using measured climatological elements. The Penman method (1948 and 1963) and its several modifications is one of the most widely known and respected systems. It is a combination of the energy budget and aerodynamic approaches. From this, Penman developed the concept of potential evapotranspiration and soil moisture deficit in his study of the Stour (1950). Soil moisture deficits (SMD) are calculated by the Meteorological Office for Britain and they have been used to predict urban storm runoff volume (Stoneham and Kidd 1977) whose "urban catchment wetness index" (UCWI) is defined by:-

$$UCWI = 125 + 8\ API5 - SMD$$

(Where the API5 is the 5 day antecedent precipitation index (NERC 1975)). This is one of the independent variables in a multiple regression equation subsequently used in the Wallingford Urban Subcatchment Model (Kidd and Lowing, 1979).

Alternative methods for calculating evaporation include those by Thornthwaite (1948) Thornthwaite and Mather (1957) and Blaney and Criddle (1950). The Thornthwaite method has been used in a water balance study for Lelystad, Netherlands by Van den Berg (1978).

Evaporation is considered a negligible quantity in urban hydrological modelling as most models are concerned with single storm events during which evaporation will be of minor concern. Evaporation between rainfalls will be important for the depletion of depression storage and soil moisture storage and so will influence the immediate antecedent conditions.

6. Other Losses:
The great variability of surfaces in urban areas means that juxtaposition of pervious next to impervious surfaces may well prevent runoff contributing to gutter inlets. This produces doubts as to the exact size and nature of the contributing area from any one catchment.

Buildings will also absorb a certain amount of rainfall and roofs are thought to have a reduced rainfall catch because of increased turbulence.

Another potential loss is below ground where cracked pipes may allow leakage into or out of the storm sewer system. This will only be a source of error when the storm runoff is measured at the outfall of the drainage system.

Each of the above factors is small individually but when added together may be a major source of error when trying to simulate the above-ground phase of runoff.

Main Loss-Models
The individual losses from rainfall detailed above are generally only modelled separately by the deterministic models (Table 2.2). The conceptual models employ a lumped-parameter loss-model to distribute the net rainfall through time, of which 3 are widely known - the Phi (Φ) - Index (Constant loss), Constant Proportional Loss and Variable Proportional Loss.

An alternative and traditional approach is to develop a runoff coefficient (runoff as a % of rainfall). The Rational method coefficient requires subjective judgement to apply correctly to get the correct runoff volume while a simpler approach takes the runoff coefficient as being 100% from impervious surfaces and 0% from pervious surfaces (eg TRRL, Watkins 1962). Both assumptions are doubtful but the errors have a tendency to cancel each other out. The Inlet Method (Kaltenbach 1963) also relates % runoff to % impervious area. Further statistical treatments to calculate runoff from rainfall have been developed by the IOH for natural catchments (NERC 1975) and for urban catchments (Stoneham and Kidd 1977) where:-

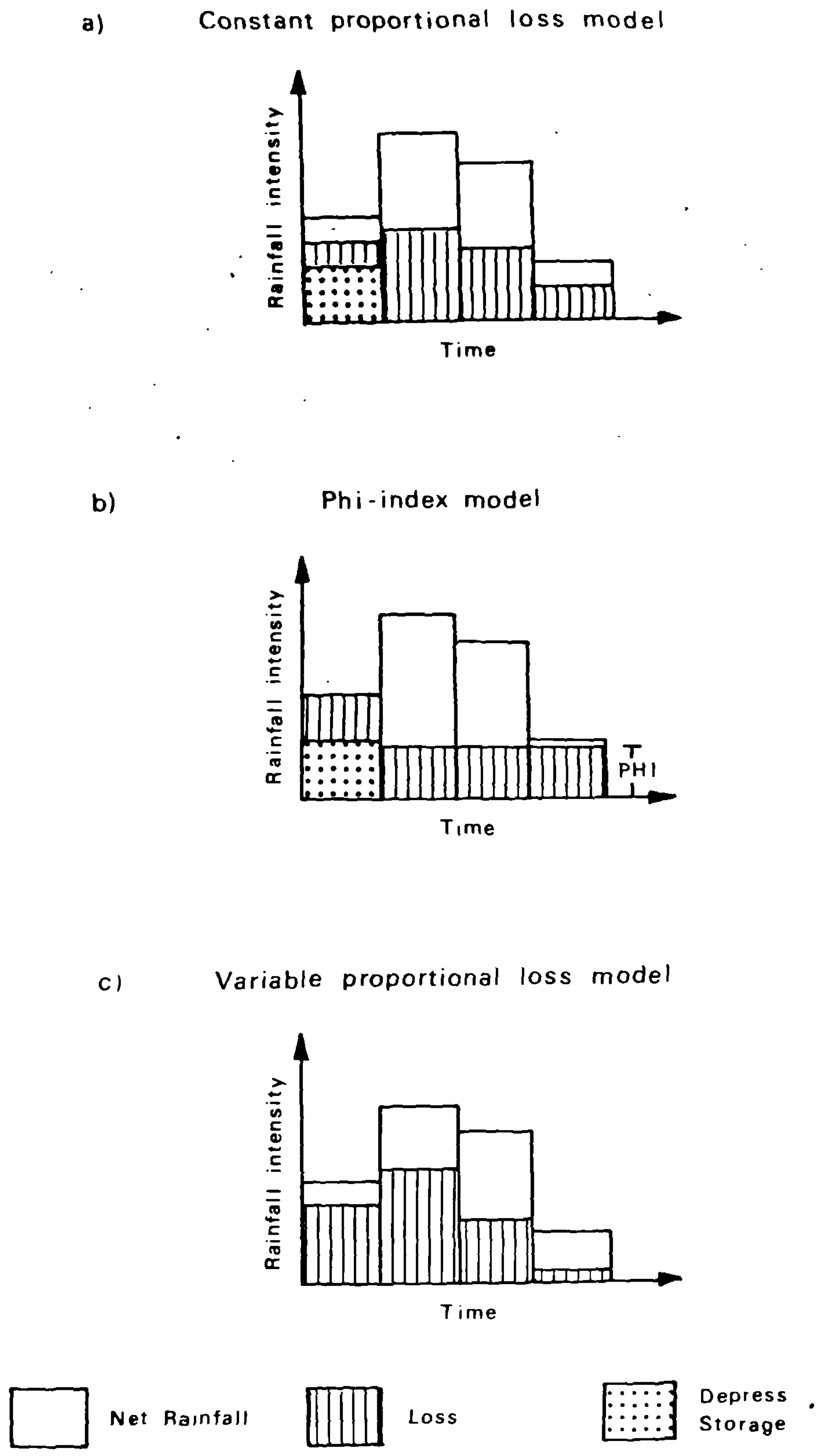
$$PRO = 0{\cdot}92\ PIMP + 53\ SOIL + 0{\cdot}65\ UCWI - 33{\cdot}6$$

Where PRO is the % of runoff volume; PIMP is the % of impervious area; SOIL is a soil index (NERC 1975); and UCWI is an urban catchment wetness index (see before).

1. Constant Proportional Loss Model Fig 2.2a
Depression Storage is first subtracted, the remaining losses are distributed as a fixed proportion of the rainfall intensity to give the correct total volume. This method has been used by Nussey and Sarginson (1978), Hicks (1944) and Kidd and Lowing (1979). It has no physical explanation in catchment process terms.

2. Phi-Index Loss Model Fig 2.2b
Depression storage is first subtracted after which loss takes place at a constant rate (Φ) for the remainder of the storm. It may have some physical basis as infiltration through impermeable surfaces will operate at a constant rate. Its simplicity has led to its wide application eg Viessman (1966), Willeke (1966), Sarginson and Bourne (1969), Brater (1968), Brater and Sangal (1969) and Kidd and Helliwell (1977).

Fig. 2.2 MAIN LOSS MODELS



3. Variable Proportional Loss Model Fig 2.2c

This uses a Horton type equation to allow for a higher proportion of loss at the beginning of the storm than at the end. The loss is a negative exponential decay function:-

$$Z_j = Z_E + (Z_o - Z_E)\, e^{-\alpha r_j}$$

Where Z_j is fraction of loss in interval j; Z_o is fraction of loss at start of the storm; Z_E is fraction of loss at end of the storm; α is a constant;

$$\text{and } r_j = \frac{\sum_{i=1}^{j} P_i}{P}$$

P_i is rainfall in interval i; P is total rainfall

The losses vary and decrease with time in the same way as infiltration rate and the equation therefore models a physical process. This type of loss model has been employed in a variety of simulation models together with some constant value for depression storage eg Illudas (Terstriep and Stall 1974), the Chicago Hydrograph Method (Tholin and Keifer 1960), MIT (Leclerc and Schaake 1973), UCUR (Papadakis and Preul 1972), SWMM (Metcalf and Eddy Inc et al 1971), HSP (Crawford 1971).

Runoff Routing

The urban runoff process is split into the above-ground phase and the below-ground phase. The below-ground phase in pipes has well-defined boundaries and well-known properties governing fluid flow. The combining and routing of the inlet hydrographs through the sewer system to the outfall can therefore be treated deterministically as a hydraulic process. The above-ground phase involves the description of overland flow and the conversion of rainfall into the runoff contribution at the inlet or collecting point in the sewer system. The overland flow process is complex with the flow breaking down into small rivulets and with greater depths of water the flow becomes turbulent and chaotic. However attempts have been made to model this process deterministically by assuming uniform sheets of water travelling across uniform planes (The St. Venant equations). The necessary equations - one for the conservation of mass and one for the conservation of momentum - are too complex for analytical solution. Numerical solutions are possible but time consuming. To avoid the complete equations of momentum and continuity which can be used to deterministically model the above- and below-ground phases of runoff, less sophisticated approximation theories have been developed. These theories can be divided into the following classes which can be applied to both runoff phases:-

1. Kinematic wave theory
2. Storage routing
3. Unit hydrograph methods
4. Time-offset (pipe-flow routing)
5. Time of entry combined with either Time of Flow or Time-Area methods.

These theories are given in order of sophistication but the boundaries between them are not sharp.

1. Kinematic Wave Theory

The best approximations to the complete equations of momentum and continuity are theories based on the kinematic waves, as first postulated by Wooding (1965). The conservation of momentum equation is simplified to one of steady uniform flow (eg the Manning or Chezy equations) and combined with the continuity equation for the conservation of mass. The kinematic wave theory has parameters directly related to the physical characteristics of the catchment. Because of the heavy demands in terms of both computer time and data collection to describe the sub-catchments, it has been little used to model the above-ground phase of runoff (eg Gunst and Kidd 1979; Leclerc and Schaake 1973). In addition, this theory is only a poor approximation of the overland flow process as described above. It has been more frequently used to route pipe flow because of the easier definition of conditions (eg Kidd and Helliwell 1977; Leclerc and Schaake 1973; Metcalf and Eddy Inc et al 1971; Crawford 1971).

2. Storage Routing

Storage routing is essentially a conceptual model with storage as the lumped parameter. The basic equations governing the storage routing concept are the equations of continuity and dynamic storage. These equations are a further simplification of the St. Venant equations with the extra assumption that the depth of stored water is constant all over the catchment.

$$\frac{dS}{dt} = i - q \quad - \quad \text{equation of continuity}$$

$$S = Kq^n \quad - \quad \text{dynamic storage}$$

Where S is the storage in the system; i is input discharge; q is output discharge; t is time and K and n are 2 model parameters.

There are 2 possible solutions to the combined equations, the first for $n = 1$ and the other for $n \neq 1$. The first of these gives a linear relationship between storage and output discharge and is the case of the linear reservoir. It has been used for the simulation of surface runoff by several researchers - Viessman (1966), Tholin and Keifer (1960), Papadakis and Preul (1972), Metcalf and Eddy Inc et al (1971), and for the above and below ground phases by Sarginson (1973) and Nussey and Sarginson (1978). The second case $n \neq 1$ gives a non-linear reservoir model, used to simulate overland flow by Kidd and Helliwell (1977), Falk and Niemczynowicz (1978) and Kidd and Lowing (1979).

3. Unit Hydrograph Methods

The linear reservoir is one mathematical description of unit hydrograph theory. The idea of the unit hydrograph was first developed by Sherman (1932) to predict flood peaks in natural catchments. The unit hydrograph is the hydrograph of direct runoff which results from a unit depth of effective rainfall falling uniformly over the catchment area at a uniform rate during a specified unit of time. They can be developed in a variety of ways. Some methods estimate a peak flow rate and time to peak and then 'sketch in' the actual shape of the hydrograph. Some methods employ a mathematical function to define the shape and some simply use a triangular shape (eg Kaltenbach 1963). The method has been widely applied to river catchment simulation (see Urbanized Catchment Models section) and to a lesser extent to urban sewer simulation and design (Eagleson 1962). It has been used to describe overland flow in urban areas when the principle of superposition is applied. (Kidd 1976; Falk and Niemczynowicz 1978) while Hicks (1944) developed a set of normalised inlet hydrographs for local conditions.

4. Time-Offset Method

This was developed by Tholin and Keifer (1960) and also used by Papadakis and Preul (1972) as a means of routing flow through the sewer system. The whole unchanged hydrograph is offset by a representative time, calculated from representative discharge as applied to Manning's Formula. The total outflow hydrograph is then determined by summation.

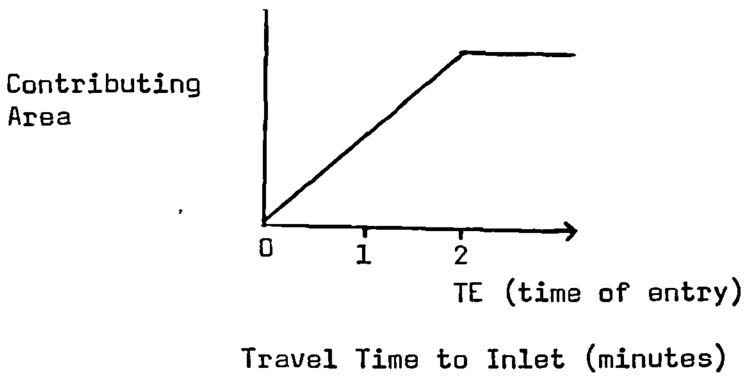
3. Time of Entry, Time of Flow and Time-Area Methods

By applying rates of runoff across the ground surface as calculated by researchers such as Izzard (1946) and Hicks (1944) it is possible to estimate the time of travel of runoff for subsections of a catchment and to divide a catchment by lines of equal travel time. This time of entry of runoff has also been determined subjectively, for instance, UK practice recommends the use of a 2 minute time of entry overland flow model.

The time of entry is then combined with a time of flow down the sewer to give a time of concentration for the Rational Method. The routing velocity is assumed to be full bore.

The contributing areas for each travel time are estimated from survey and a time-area curve can be constructed. This 'S' - shaped curve is usually simplified so that the increase in area with time is linear, and the whole area is contributing at the designated time of entry (Fig 2.3).

Fig.2.3 <u>Time-Area Diagram</u>



To derive the inlet hydrograph the rainfall excess is applied to the time-area diagram. Such a model is simple to use and is incorporated in several models eg TRRL (Watkins 1962); ILLUDAS (Terstriep and Stall 1974).

The Sewered Urban Catchment Models detailed in Tables 2.1 and 2.2 achieve their purposes, namely design of major drainage works, accurate simulation of the flood hydrograph, prediction of the effects of urbanization on the flow regime or in some cases a continuous simulation based on the water balance concept, by the use of the above range of process modelling techniques. Because of this concern largely to produce an engineering solution to deal with storm runoff, the studies are not greatly interested in improving understanding of the individual processes operating. The more complex models employ terms for infiltration, depression storage and a few include evaporation, soil moisture or groundwater storage and flow indices but none consider the urban area as an integrated hydrological system incorporating other processes with, for instance, terms for changes in river and ground water levels caused by discharge of imported water, groundwater abstractions or pipe leakage. If accurate simulation or prediction of the full range of changes to the hydrological cycle induced by urbanization is to be possible, these and other processes must be examined. The most satisfactory way of evaluating these processes is within the context of a metropolitan-scale urban water balance which provides a framework for data collection and physical process studies

URBAN WATER BALANCES should thus be capable of quantifying and proving the reality of the effects of urbanization on all components of the hydrological cycle. A water balance model would be a satisfactory management tool for resource and environmental planning.

The water balance studies listed in Table 2.1 become increasingly sophisticated the lower down they are placed alongside the measured process scale. Thus the study by Lvovich and Chernogayeva (1977) uses only one measured variable - precipitation, while the study by Carlsson and Falk (1977) uses actual data for pipe leakage and measures for many of the processes listed above its position on the Table.

The fewer elements measured by a water balance study, the greater the error contained by each part of the equation. In the simpler cases, precipitation and runoff are measured for a city area and the remaining quantity (which includes errors) is ascribed usually to evapotranspiration. Balek (1978) adopted this approach when preparing water budgets for 4 land-use category catchments - partly urbanized (20% paved), agricultural, forested and drained catchments. By preparing annual balances he was able to ignore the groundwater and soil moisture terms which showed no appreciable difference between the beginning and end of the study period. The percentage runoff from the partly urbanized catchment was consistently at least twice that from the other catchments, and varied from 49% for a dry year to 60% for a wet year. Similarly, the work by Lvovich and Chernishov (1977) used only measures for rainfall and runoff when comparing the water balances from Virgin Steppe, ploughed lands and the city of Kursk. In this case, however, the residual quantity from the water balance was attributed to infiltration. No runoff occurred from the Steppe region because the deep-rooted grasses allowed complete infiltration of rainfall while 85 to 97% of rainfall infiltrated into the ploughed lands. The more heavily urbanized areas of Kursk produced 26% runoff indicating that urbanization increased runoff.

Although Lvovich and Chernogayeva (1977) put forward a complex model for calculating surface and subsurface flows from Moscow based on intuitively derived runoff coefficients for the different land use areas, their water balance used only rainfall as a measured quantity and therefore the mean annual runoff coefficients of 0·43 for the Moscow city area and 0·73 for the central city area remain simply hypothetical. A more detailed model by Lull and Sopper (1969) allowed computation of daily water balances for an urbanized catchment (with varying degrees of imperviousness) in comparison with actual rainfall and runoff data for a forested watershed. They estimated that annual potential evapotranspiration was reduced by 19, 38 and 59% by 25, 50 and 75% impervious cover, and annual runoff was increased 15, 29 and 41% respectively.

The above type of water balance study has much the same aim as the Yield Studies under the Urbanizing Catchment Studies category - namely a desire to quantify the effects of urbanization on annual yield and runoff in comparison with other land uses. They reinforce the view that urbanization increases runoff but do little to improve knowledge about the other components of the hydrological cycle.

A detailed attempt to quantify man's influence on the complete hydrologic system of Long Island, New York, was undertaken by Franke and McClymonds (1972). From measurements of groundwater levels, rainfall and runoff they assessed the changes in the annual water balance. The net effect of man's use of the groundwater was a lowering of the water table. The estimated total loss from the system in 1965 was 125 mgd of which 60% was discharge of sewage to the sea and the remainder caused by export of water to New York City and disposal of direct runoff from urban areas to streams and subsequent outflow to the sea. Artificial recharge from cesspools and tanks (125 mgd), recharge basins (85 mgd), injection wells (55 mgd) and leaking pipes (45 mgd) totalling 310 mgd was not sufficient to counteract the abstractions from groundwater.

Exact field measurements of precipitation, storm runoff and subsurface runoff allowed Van den Berg (1978) to calculate 2 small scale water balances for a housing area (44% paved area) and parking lot (99·6% paved area) at Lelystad in the Dutch polders region. He estimated potential evapotranspiration from the unpaved areas according to the method by Thornthwaite and Mather (1957). The runoff elements were measured in the sewers, the subsurface runoff figures being essentially an indirect measurement of soil infiltration approximately 1 metre below ground level. Five annual water balances for the housing area gave average values of 24% for surface runoff, 45% for subsurface runoff and 34% for actual evapotranspiration from the unpaved area. 4 annual water balances for the parking lot produced average values of 50% for surface runoff and 40% for subsurface runoff. This latter value would seem to indicate substantial infiltration through the supposedly impermeable paved surfaces and the incomplete balance achieved – only 90% of the rainfall accounted for – suggests the possibility of a missing factor, perhaps paved surface evaporation. Another detailed field study by Malmquist and Svensson (1977) for a suburban housing area of Göteburg, Sweden, took a broader view of the urban water budget by including measurements of drinking water consumption (averaged at 400 m^3/d for the 5000 inhabitants). Measurements of rainfall, sewage and storm water flow were also made.

The most complete water balance study is that by Carlsson and Falk (1977) who produced a water budget model for a typical Swedish urban area (Fig. 2.4). Using actual catchment data and figures for domestic and industrial water use from the Swedish Water and Sewerage Works Association, they estimate typical values for the elements involved in the outer (or "natural") cycle and for the inner city-distribution system.

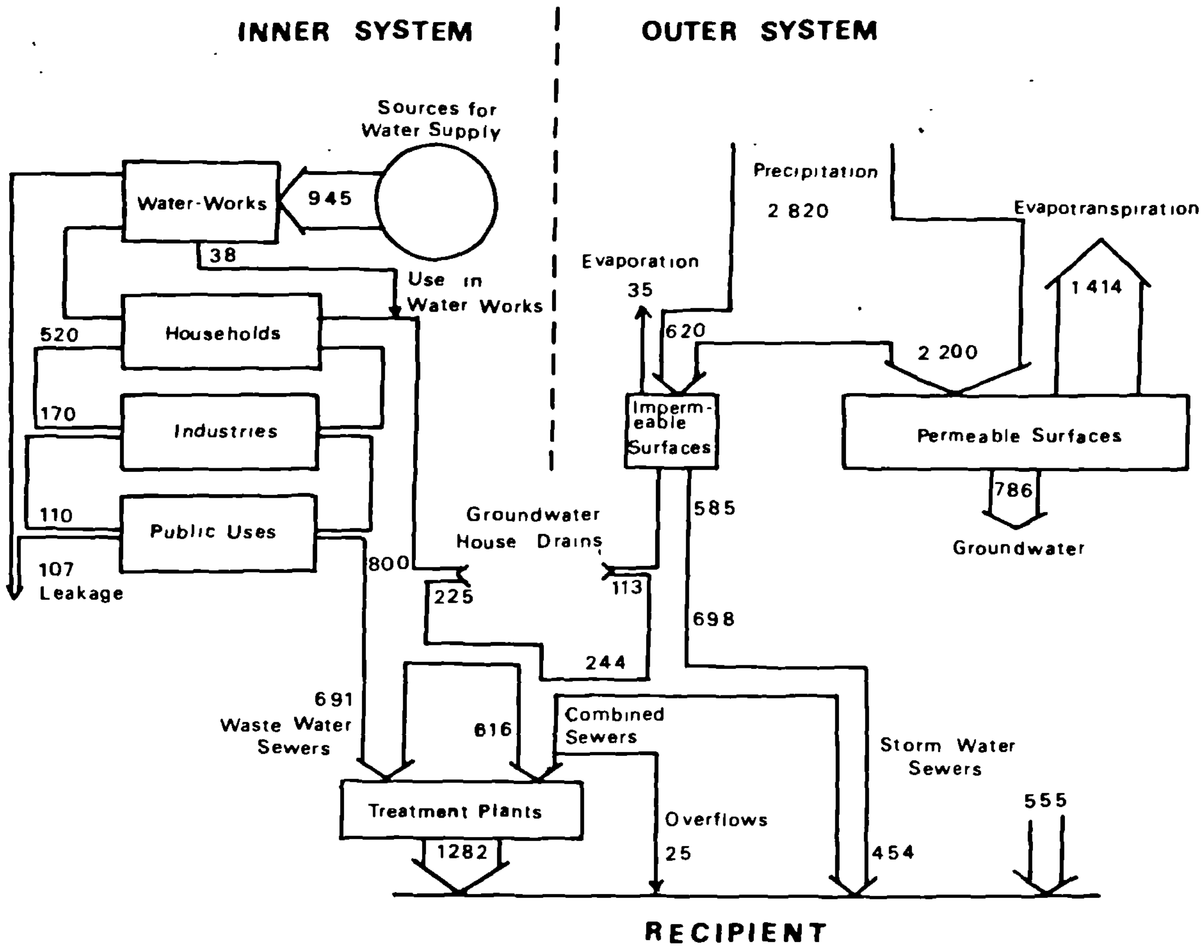


Fig. 2.4 General Urban Area Water Budget for Sweden
(from Carlsson and Falk, 1977)

Inventories of the type made by Carlsson and Falk (1977) are extremely useful for water resource planning. In all the above studies, the quantities of water involved in each of the major components of the urban hydrological cycle really only apply to each city and its unique set of conditions. No predictive set of equations of the effect of urbanization on a metropolitan water budget can be achieved until more complete inventories have been made and until all the processes have been investigated thoroughly. Detailed small scale studies of the type by Van den Berg (1978) have indicated the possible importance of processes which until now have been largely dismissed for urban areas. Therefore, to improve scientific understanding of the processes involved and to provide adequate information of the respective quantities of water for each element for better resource planning and management, further investigations of the changes in processes induced by urbanization is suggested.

PROCESSES Measured or anticipated hydrologic changes due to urbanization are expressed qualitatively on Table 2.3.

Table 2.3 Measured or Anticipated Hydrologic Changes due to Urbanization (adapted from Oke, 1974).

Element	Comparison with Rural Environment	Remarks
Precipitation	More ?	Thermal and mechanical uplift, nuclei, combustion
Imported water	More	Piped water supply
Water released by anthropogenic activities	More	No rural counterpart
Surface and subsurface runoff	More	Lower permeability and channelling
Leakage into or from pipes	More	No rural counterpart
Actual evapotranspiration	Less ?	Reduced evapotranspiring surfaces
Evaporation from paved areas	More	No rural counterpart
Soil moisture storage	Less	Reduced infiltration and interception
Groundwater storage	Less	Reduced percolation
Storage in city fabric	Variable	Interception high in forests, low on grasslands

The dramatic increases in surface runoff are fairly well documented as detailed in previous sections but the less well-known effects include changes to precipitation, evaporation and soil and groundwater storages.

Urban influences on precipitation are rather speculative and sparsely reported. Intuitively cities may be expected to increase precipitation for a variety of reasons such as:

1) their higher temperatures (due to greater radiation and combustion processes) leading to upward motions of air to initiate or enhance convection,

2) greater concentrations of condensation and ice nuclei which should encourage the formation of clouds and raindrops,

3) additional sources of water vapour from combustion and industrial processes which could increase cloudiness and precipitation potential,and

4) the roughness of the city surface which would produce low level mechanical turbulence increasing upward vertical velocities and again increase precipitation potential.

It has proved difficult to separate the strictly urban effects on precipitation from the more normal orographic influences, especially since the main techniques for examining the effect are empirical and rely either on statistical analyses of long climatological records or the detailed case-study approach of single storms (eg Atkinson 1970, 1971, 1975 and 1977). This former approach is typified by American and European studies of various cities summarised by Changnon (1969), Huff and Changnon (1972) and Landsberg (1956 and 1970). Changnon (1968) reported the now well-known La Porte phenomenon where marked increases in precipitation, moderate rainy days, thunderstorm days and hail days occurred downwind of the urban-industrial area since 1925. However, other investigators have reported decreases in precipitation. An analysis of precipitation records for Central Park, New York City, by Spar and Ronberg (1968) showed a significant decreasing trend of precipitation from 1927 to 1965.

These contrasting results indicate the difficulty in determining whether the urban-industrial complex actually influences precipitation. The massive data collection programme of Project METROMEX (Changnon et al 1971) has produced results that have largely dispelled scepticism about the reality and mechanisms of urban enhanced precipitation, (Changnon 1972 and 1978), Changnon et al 1972, Semonin and Changnon 1974, Beebe and Morgan 1972, Huff and Vogel 1978, Huff 1974).

Although measured increases of rainfall in and downwind of urban areas generally fall within the range of 5-15% of annual rainfall amounts,there are as yet no forecasting rules. Not every city appears to have this effect and city size must be an important variable. Kuprianov (1977 and 1979) allows for this effect when predicting annual runoff from the Minsk city area and any hydrological planning model should also consider this potential local increase in precipitation as a result of urbanization.

The water balance equation potentially provides a means of obtaining evaporation from the city. Unfortunately, other than for gross annual figures as seen in the above Urban Water Balance section, the errors in measurement of the other variables make this impractical. Evaporation from cities has long been assumed to be drastically less than from neighbouring rural areas because much of the urban surface is sealed by concrete, asphalt and other impervious or semi-impervious materials,which thus reduces the areas of soil and vegetation releasing moisture. However this ignores the often large areas of parks and gardens (Detwler and Marcus 1972) and the absorbtive capacities of bricks and tiles of the urban fabric (Givoni 1969).

An attempt to approximate evapotranspiration from urban areas has been made by Eagleman et al (1972) who mapped potential evapotranspiration variations and suggested there were increased moisture demands for urban plant life in central Kansas City. Evaporation remains a most difficult process to measure directly or indirectly.

Evaporation is the process which supplies atmospheric humidity and rather more successful attempts to measure this quantity have been made. Until comparatively recently, and in the absence of good measurements, urban air was considered to be drier than rural air, the reasoning being that runoff would be higher in urban areas due to the impervious surfaces and so evaporation rates must be less. Some investigators have suggested the city to be drier on the basis of lower urban relative humidity values, failing to recognise that the higher urban temperatures (the heat island) are controlling them and their distribution. Chandler (1967), Bornstein et al (1972) and Kopec (1973) have shown that although relative humidities were lower, the absolute humidities were 2-3 mb higher than in the surrounding rural areas. Mapping of these real moisture differences (as expressed by specific humidity values) by Shea and Auer (1978) and Sisterson and Dirks (1978) has revealed a day time dry plume of urban air over metropolitan St. Louis, characterised by specific humidities of $\leqslant$ 15 g/kg, only fractionally lower than those for the rural air at $\geqslant$15 g/kg. Several local regions of relatively high or low specific humidities were attributed to the abundance or lack respectively, of natural evaporating surfaces (Auer 1978). Sisterson and Dirks (1978) also noted an 8% increase in the total moisture of a column of air as it passed over the urban area on a fair weather summer day. This suggested continued evaporation of surface moisture into the air column.

Whether the evaporation rate supplying the often higher absolute humidities in cities is above or below the comparable rural rate is unknown, but these atmospheric humidity values suggest that evaporation from a city is not a necessarily negligible quantity. It is likely that the main source of this moisture is the areas of evapotranspiring vegetation but additional moisture can be supplied by industrial emissions of water vapour and combustion of petrol from cars, while just after rainfall, paved surfaces and bricks will supply moisture for evaporation. With the higher urban temperatures, potential evaporation rates are much higher in cities and actual evaporation may approach this value after rainfall when sources of moisture are available.

Owing to reduced infiltration of precipitation and increased runoff rates from the urban impermeable surfaces, it is assumed that ground water and soil moisture storages will not be replenished. This is not necessarily the case since some uncontrolled recharge through soakaways and leaking pipes will occur.

Sawyer (1963) in his study of Long Island, New York, concludes that recharge to the ground water reservoir by East Meadow Brook has been cut by 2% (equivalent to 63,000 gpd) from 1952-60 as a result of urbanization. Franke (1968) also supported these findings with evidence of a lowering of groundwater levels by 1-2 ft for the urbanized areas of Nassau County, Long Island. Kuprianov (1977) reports that constant diversions of sewage effluent and runoff outside the catchment boundaries and directly to the sea have caused an ever decreasing deficit of groundwater and consequent reduction of river flows in Denmark.

Leaking pipes have proved to be a major source of ground water recharge and several studies have estimated its size. Cotton and Delaney (1971) found that leaks from the water-mains in Boston, Massachusetts were equivalent to a recharge of 0·73 mgd/sq.ml. over the Boston peninsula. Franke and McClymonds (1972) estimate that 15% of the total public supply leaks from pipes and recharges the groundwater reservoir of Long Island, New York. Phillips and Kershaw (1976) found annual losses for the Malvern area of 24-31% from the metred water supply. Additional data gathered for other countries suggested that this was not unusual with France, Holland and Scotland all having an approximate 30% quantity of 'unaccounted' water lost from the mains supply.

A useful summary of the situation by the ASCE Task Committee on the Effects of Urbanization on Ground-Water Recharge (ASCE 1975) showed that depending on the situation, ground water recharge has not been affected, has decreased or has increased in the urban environment. The effects of urbanization on ground water levels are therefore fairly unpredictable and only groundwater models calibrated for a particular area will work eg Schicht (1978) reports the use of a groundwater management model in north eastern Illinois that can predict water level declines for different management schemes. Without adequate information on ground water level fluctuations and their causes, ground water management models will be hampered and prediction of the effects of different water use schemes impossible.

FUTURE RESEARCH NEEDS

The overall conclusion to be drawn from this review of the current position of urban hydrological research is that the precise effects of urbanization on even the major components of an urban hydrological cycle - precipitation, evaporation, runoff, groundwater storage - are unpredictable or even unmeasurable. Modelling progress has been made but has been heavily directed to predicting storm runoff while other equally important quantities in the metropolitan water balance model/inventory for resource planning and evaluation have been largely neglected. Knowledge about additional processes involved is sparse and their relative importance speculative. However, until these processes have been actually measured, they cannot be dismissed as unimportant quantities. Investigation of processes and components involved within the '<u>Terra Incognita</u>' process section of Table 2.1 would appear to be a necessary step to complete the hydrological water balance required, while additional data to verify and test the current models would also be useful.

Those processes that require investigation or additional research investment include overland flow and roof gutter flow which have as yet been poorly simulated because of the complexity of the surfaces involved. Urban humidity levels are supplied with moisture by evaporation from a variety of sources all as yet unquantified. Urban surfaces intercept and absorb unknown quantities of rainfall. Some of this moisture is stored on the surface as depression storage, dew and hoar frost, while some may infiltrate through roads and pavements to the soil below. Evaporation from these surfaces together with evapotranspiration from vegetation, and supplies of moisture from man-made sources such as the burning of fossil fuels and the use of cooling towers contribute to the humidity levels of cities. While humidity levels themselves can be measured, the processes supplying the moisture - evapotranspiration and paved surface evaporation - become the hardest unknown quantities to assess, yet they are probably a substantial part of the urban water balance equation.

If a realistic metropolitan scale water balance is to be achieved, additional information about the size of imports of water, abstractions and returns made by industry and sewage treatment works must be incorporated with the natural drainage cycle in an urban area. The scale of any transfers by leakages between the natural and artificial drainage systems must also be estimated as groundwater and storm runoff may enter foul-water sewers to be measured as treated effluent while foul or fresh water may leak from pipes and thus supply groundwater or rivers and be measured as part of the response to rainfall.

Work carried out to try and fill some of these gaps in knowledge is detailed in the following chapters of the thesis, under the proposals outlined in Chapter 1. If additional studies of this kind are made, the proposed goal of producing a catchment scale model which incorporates fully field-tested processes involved in the urban hydrological cycle which can be used for scientific or planning purposes should be nearly within our grasp. Sensible resource management and drainage design need no longer be based on inadequate information and poor knowledge of the processes involved.

Chapter 3. WATER BALANCES OF URBANIZED CATCHMENTS IN GREATER LONDON

INTRODUCTION

Hydrological data has been collected in the London region for many years (Butters and Vairavamoorthy 1977), but little or no effort has been made to co-ordinate this information to produce a metropolitan water balance as recommended by Unesco (1974). The data has been used to assess the effects of urbanization on storm runoff in North London (Hall 1977). Analyses of the water balances of 5 subcatchments of the river Thames are therefore made here using some of the data available. These water balances are calculated over different periods of record. Monthly balances for one catchment are detailed while annual water balances are made for all 5 catchments. The average rainfall and runoff records are compared with records from rural basins in South East England which are also cut into London clay and chalk. Similar large scale water balances using rainfall and runoff data for urban areas have been constructed by Balek (1978) for Pojbuky, Czechoslovakia and by Lvovich and Chernishov (1977) for Kursk, Russia. The information obtained by these types of study may prove useful for water resource management, amenity use and planning and indicate the scale of impact that urbanization has on the hydrological cycle for the reasons as suggested in Chapters 1 and 2.

CATCHMENT AREAS

The Brent, Crane, Beverley, Wandle and Ravensbourne rivers are tributaries of the river Thames whose catchments lie wholly or partly within Greater London. Greater London covers an area of 1580 km^2 with a population of 7·45 million. Inner London is densely urbanized and each of the 5 catchments has more than 30% of its area covered by impervious surfaces. The catchment headwaters are still rural (except for the Beverley Brook) so that stretches of the watercourses are in a natural or semi-natural state. Large areas of open space, parks and gardens exist although extensive development continues within London's boundary. The Brent reservoir is important for flood storage and recreation use as are the smaller lakes, flood plain riverside parks and walks of the other catchments.

The geology of the London Basin is relatively simple in its influence on river hydrology. The thick syncline of chalk is largely covered by London clay with superficial deposits of alluvium, terrace gravels and sands. The chalk is exposed in the headwaters of the 3 southerly catchments whose base flow is consequently reduced by the pumping of the chalk aquifer.

Industrial usage of these Thames tributaries is now largely historical, most water for production being obtained from the mains supply and wastes being treated by the major sewage treatment works. However some wastes reach these water courses through industrial spillages. From a management point of view, the future use of these rivers will be for amenity projects, while greater attention is given to their flood risk potential. Pumping of the chalk aquifer has had the effect of drying up many attractive small streams and ponds but steps have been taken to counteract this, for instance, by recycling of water within the Wandle river system and concreting the base of ponds to prevent percolation.

SOURCES OF DATA

More than 120 rainfall stations throughout the whole of Greater London supply regular rainfall statistics to the Greater London Council (GLC). Seventy of these (Fig 3.1), within or near the catchment boundaries, were

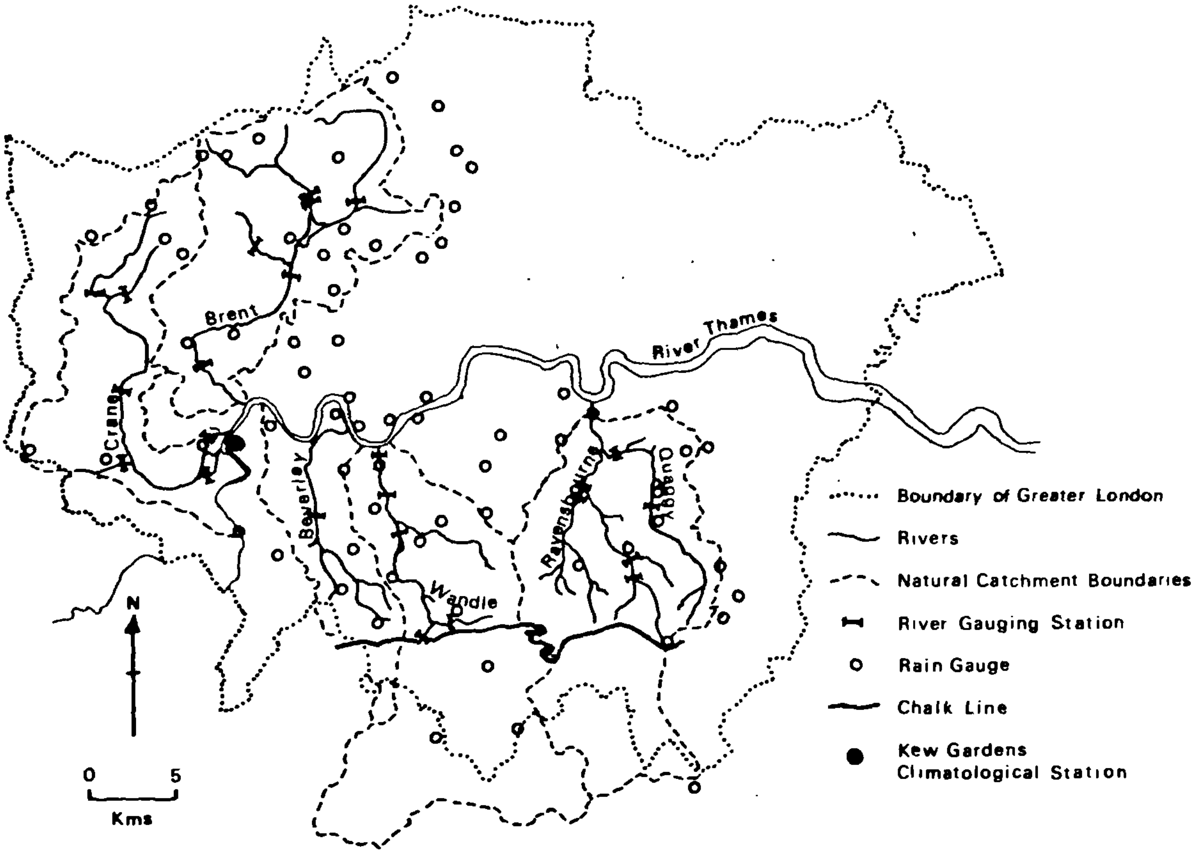


Fig. 3.1 GREATER LONDON COUNCIL HYDROMETRIC NETWORK

operating in 1977 producing a dense network of approximately 1 gauge every 10 km^2. Twenty seven flow gauging stations are currently in operation on the 5 rivers and their tributaries. The data amounts to over 300,000 daily rainfall charts and 26,000 weekly river flow charts, dating back in some cases to 1928. Owing to the difficulty of handling such a backlog of valuable information it has been little used. A programme of digitising this data was begun but has now halted as a result of financial cutbacks and the data are therefore still largely inaccessible. The GLC prepare monthly and daily summaries of the rainfall totals from their raingauges for the Meteorological Office, so that the data is in useful form for water balance calculations. The limiting factor on the number of annual water balances that could be prepared for the 5 rivers was the quantity of analysed flow data. The lowest reliable downstream gauge on each river or tributary with daily mean flow data was used. These gauges are referenced in Table 3.1.

Table 3.1 GLC River Gauging Stations

River	Station Name	TQ Grid Reference	Analysed Record Length (years)
Wandle	Connolly's Mill	266 706	8
Beverley Brook	Wimbledon Common	216 717	13
Brent	Hanwell	151 801	4
Crane	Marsh Farm	154 734	4
Ravensbourne	Catford Hill	373 732	1
Ravensbourne East	Bromley South	405 687	3
Quaggy	Manor House Gardens	394 748	4

Planimetering of charts, was necessary to obtain flow records of the Crane, Brent and Ravensbourne (plus tributaries) so that a shorter length of record was available.

The GLC has calculated catchment areas based on the storm sewer network. These largely correspond with the areas of London clay as soakaway drainage is employed within the more lightly built-up chalk areas. The urbanized area of each catchment was obtained from GLC analyses or the Flood Studies Report (Vol 1V, NERC 1975) and is calculated as the percentage of 'pink' area on the 1 in 50,000 map series.

The Thames Water Authority (TWA), keeps records of treated water discharge from sewage treatment works and industrial abstraction and discharge. The River Wandle and Beverley Brook receive water from minor sewage treatment works and the River Wandle has one industry extracting water for cooling purposes. Unfortunately, in the hand-over of responsibility to TWA, records from the Beddington Sewage Treatment Works on the Wandle, prior to 1970, have been lost. No other industry officially disposes wastes into these 5 watercourses.

The Meteorological Office maintains 2 climatological stations in Central London. Records from Kew Gardens of precipitation, potential evapotranspiration and soil moisture deficit (based on Penman's method, 1950 and 1963) were employed in this study.

ANALYSIS OF DATA

Initially a detailed monthly water balance of the Wandle catchment was attempted. The density of the raingauge network allowed the use of a simple arithmetic average to produce a figure for catchment average rainfall. Reliable flow records from the lowest downstream gauging point were used for the period 1970–1977. Adjustments to flow were necessary because of the upstream influence of industrial abstraction and additions by a small sewage treatment works. Using monthly figures of rainfall (P) and potential evapotranspiration (Ep) (which is derived from climatological data), Penman's water balance technique (Penman,1950) produces an estimate of soil moisture deficit (SMD),(the amount of soil moisture content less than field capacity) and uses this to calculate actual evapotranspiration (E_a). The balance for 2 sample years is shown in Table 3.2, these years were selected as no soil moisture deficit existed in January. The effective rainfall is defined as rainfall minus potential evapotranspiration ($P-E_p$) and where this is positive, actual evapotranspiration can operate at the same rate as potential ($E_a = E_p$). Where the effective rainfall is negative, an equivalent soil moisture deficit is created up to selected levels which are root constants of 76•5 mm for short-rooted vegetation (eg grass) and 203•2 mm for long-rooted vegetation (eg trees). The reduced ability of short and long-rooted vegetation to draw moisture from deeper in the soil when the potential soil moisture deficit has reached their respective root constants

Table 3.2 MONTHLY WATER BALANCE WANDLE CATCHMENT

1970 32.2% Urban Catchment Area 176.12 km^2

	P	E_p	$P-E_p$	Potential SMD	SMD Grass	SMD Trees	Areal SMD	E_a	ΔS	$\bar{Q}$	Additions by ST.	Extractions	$P-\bar{Q}-E_a \neq 0$ $-\Delta S$
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
J	64.3	4.8	59.5	0.0	0.0	0.0	0.0	3.3	0.0	19.1	14.6	0.64	27.9
F	43.9	18.5	25.4	0.0	0.0	0.0	0.0	12.6	0.0	19.0	12.8	0.64	0.1
M	43.2	36.6	6.6	0.0	0.0	0.0	0.0	24.9	0.0	21.8	14.0	0.64	-16.9
A	65.2	54.6	10.6	0.0	0.0	0.0	0.0	37.1	0.0	25.3	14.7	0.64	-11.3
M	23.1	96.5	- 73.4	73.4	73.4	73.4	39.6	65.6	-39.6	20.4	13.8	0.64	-36.5
J	26.2	131.8	-105.6	179.0	116.8	179.0	75.5	68.5	-35.9	15.6	12.6	0.64	-34.0
J	57.3	102.1	- 44.8	223.8	120.7	217.2	84.5	54.6	- 9.0	16.3	13.5	0.64	-17.5
A	56.4	84.6	- 28.2	252.0	123.2	221.0	86.1	43.8	- 1.6	18.0	13.2	0.64	-16.4
S	47.4	54.1	- 6.7	258.7	124.0	222.2	86.6	33.8	- 0.5	27.3	12.5	0.64	-25.1
O	16.6	25.9	- 9.3	268.0	124.5	223.5	87.0	13.0	- 0.4	9.7	12.7	0.64	-17.8
N	157.7	10.9	146.8	121.2	0.0	76.7	15.3	7.4	71.7	23.0	16.5	0.64	39.7
D	50.0	5.6	44.4	76.8	0.0	32.3	6.5	3.8	8.8	19.9	14.7	0.64	3.4
	651.3	626.0						368.4	- 6.5	235.4	165.6	7.68	-104.4

1974

	P	E_p	$P-E_p$	Potential SMD	SMD Grass	SMD Trees	Areal SMD	E_a	ΔS	$\bar{Q}$	Additions by ST.	Extractions	$P-\bar{Q}-E_a \neq 0$ $-\Delta S$
J	68.1	20.8	47.3	0.0	0.0	0.0	0.0	14.1	0.0	15.8	15.2	0.64	23.6
F	82.3	19.9	62.4	0.0	0.0	0.0	0.0	13.5	0.0	18.9	16.0	0.64	34.5
M	34.8	33.7	1.1	0.0	0.0	0.0	0.0	22.9	0.0	16.9	14.9	0.64	-19.3
A	13.9	67.6	- 53.7	53.7	53.7	53.7	29.0	46.0	-29.0	15.2	13.6	0.64	-31.3
M	27.7	96.6	- 68.9	122.6	109.2	122.6	61.6	61.1	-32.6	15.6	13.5	0.64	-29.3
J	73.4	104.0	- 30.6	153.2	115.0	153.2	69.7	62.3	- 8.1	16.8	13.8	0.64	-10.8
J	39.5	109.6	- 70.1	223.3	120.7	213.4	83.7	50.6	-14.0	15.3	13.5	0.64	-25.3
A	72.2	89.1	- 16.9	240.2	120.7	218.4	84.7	52.4	- 1.0	13.9	13.4	0.64	- 5.9
S	130.3	54.0	76.3	163.9	44.4	142.1	43.5	36.7	41.2	18.8	15.4	0.64	18.8
O	79.9	27.2	52.7	111.2	0.0	89.4	17.9	18.5	25.6	17.1	16.2	0.64	3.1
N	128.9	14.6	114.3	0.0	0.0	0.0	0.0	9.9	17.9	31.3	20.4	0.64	50.0
D	41.9	21.3	20.6	0.0	0.0	0.0	0.0	14.5	0.0	28.7	18.4	0.64	-19.1
	792.9	658.4						402.5	0.0	224.3	184.3	7.68	-11.0

P = Catchment Average Precipitation (Kew Gardens 1941-70, 599 mm)

E_p = Potential Evapotranspiration (Kew Gardens)

E_a = Actual Evapotranspiration

SMD = Soil Moisture Deficit

$\bar{Q}$ = Gauged discharge - Additions + Extractions

ΔS = Soil Moisture Changes

Q = Gauged Discharge

STW = Sewage Treatment Works

is taken into account to give the actual soil moisture deficits as read from a graph. The arealsoil moisture deficit is traditionally calculated from a land use composed of 50% grass, 30% trees and 20% riparian area (where the permanent ground water is so close to the surface that evaporation is always assumed to take place at the potential rate and no deficit is set up). To take account of the paved areas in this urban catchment, the areal soil moisture deficit is weighted according to 32% urban area, and the 68% remaining, weighted as before (5: 3: 2) as 34% grass, 20% trees and 14% riparian land. When a soil moisture deficit exists, actual evapotranspiration is calculated as the amount the soil has dried from the previous month added to the rainfall for that month. All actual evapotranspiration values are reduced to come from the 68% vegetated areas.

Soil moisture changes (ΔS) are calculated as the change in monthly areal soil moisture deficit. The error term in the balances is calculated as the residual when actual evapotranspiration, soil moisture changes and discharge (gauged discharge minus sewage treatment works flow plus extractions) are subtracted from rainfall, ($P - \bar{Q} - E_a \pm \Delta S$).

Annual water balances for the river Wandle and Beverley Brook were calculated including any additions or extractions from flow (Table 3.3). The "X" column for the Beverley being simply sewage treatment work flow additions while for the Wandle, the industrial abstractions are subtracted from the sewage treatment flow figures to give net positive figures. Therefore the "Q_a" column is gauged discharge (Q) minus these net additions to flow (X).

Annual water balances for the remaining 3 catchments are given in Table 3.4. This time span removed the necessity to consider changes in soil moisture storage which can be assumed to balance out over a period of a year.

Catchment mean annual precipitation was again an arithmetical average of raingauge stations. Kew Gardens provided independent estimates of potential (Ep) and actual evapotranspiration (Ea). This latter figure being then adjusted according to the percentage of pervious area supplying the moisture. Evaporation from impermeable surfaces is ignored as it is incalculable and considered a negligible quantity (Van den Berg 1978).

The annual water balances were averaged over their period of record and the individual water balance elements compared in both absolute and

Table 3.3 ANNUAL WATER BALANCES FOR 2 LONDON URBANIZED CATCHMENTS AFFECTED BY ADDITIONS OR EXTRACTIONS

Beverley Brook (81% Urban)
Catchment Area 42.42 km^2

Worcester Park Sewage Treatment Works serves 80,000 people
Sutton Sewage Treatment Works serves 40,000 people

	P mm	Q mm	X mm	Q_a (Q–X) mm	Q_a/P %	$P-Q_a$ mm	E_a mm
1965	638.4	357.9	221.5	136.4	21	502.0	90.2
1966	704.6	631.8	249.0	382.8	54	321.8	101.3
1967	739.2	414.1	235.2	178.9	24	560.3	99.1
1968	698.2	439.1	248.5	190.6	27	507.6	99.4
1969	468.5	359.7	214.6	145.1	31	323.4	83.7
1970	625.2	275.2	261.7	13.5	2	611.7	89.9
1971	611.8	481.5	239.9	241.6	39	370.2	99.6
1972	452.3	280.3	191.4	88.9	20	363.4	65.4
1973	508.8	252.8	207.8	45.0	9	463.8	91.5
1974	732.6	477.4	241.1	236.3	32	496.3	85.2
1975	596.0	396.6	275.9	120.7	20	475.3	74.6
1976	488.9	321.2	204.3	116.9	24	372.0	60.1
1977	617.5	415.5	214.4	201.1	33	416.4	83.8
Average (mm)	606.3	(392.5)	(231.2)	161.4			86.4
(%)	100			27			14

$$P \neq Q_a + E_a$$
$$100 \neq 27 + 14$$

River Wandle (32% Urban)
Catchment Area 176.12 km^2

Beddington Sewage Treatment Works serves 350,000 people
(records prior to 1970 lost)
Merton Board Mills extracts 15.6 cumecs annually

1970	674.3	235.3	157.9	77.4	11	596.9	321.9
1971	611.0	244.6	171.2	73.4	12	537.6	356.5
1972	497.3	152.0	152.2	– 0.2	–	497.5	234.1
1973	546.6	145.9	154.0	– 8.1	–	554.7	327.4
1974	791.2	226.4	176.7	49.7	6	741.5	304.9
1975	638.2	366.8	212.9	153.9	24	484.3	266.8
1976	517.9	213.8	143.2	70.6	14	447.3	215.2
1977	655.5	326.5	173.1	153.4	23	502.1	299.7
Average (mm)	616.5	(238.9)	(167.7)	71.3			290.8
(%)	100			12			47

$$P \neq Q_a + E_a$$
$$100 \neq 12 + 47$$

P = Mean annual catchment precipitation
E_a = Actual evapotranspiration from pervious surfaces (Kew Gardens)
Q = Mean annual gauged runoff
X = Total additions and extractions from flow

Table 3.4 ANNUAL WATER BALANCES FOR LONDON URBANIZED CATCHMENTS

River Brent (75% Urban)
Catchment Area 132.1 km^2

	P mm	Q mm	Q/P %	P-Q mm	E_p mm	E_a mm
1974	783.3	512.8	65	207.2	658.4	112.1
1975	641.5	472.1	74	111.1	652.2	98.1
1976	512.1	522.7	102	-75.1	736.4	79.1
1977	712.6	413.4	58	248.2	630.1	110.2

River Crane (48% Urban)
Catchment Area 81.07 km^2

	P mm	Q mm	Q/P %	P-Q mm	E_p mm	E_a mm
1974	768.2	385.1	50	383.1	658.4	233.2
1975	608.8	333.7	55	275.1	652.2	204.1
1976	505.6	76.9	15	428.7	736.4	164.6
1977	707.3	356.5	50	350.8	630.1	229.2

P = Mean annual catchment precipitation
Q = Mean annual gauged runoff
E_p = Potential evapotranspiration (Kew Gardens)
E_a = Actual evapotranspiration (Kew Gardens) from pervious surfaces

River Ravensbourne (49% Urban)
Catchment Area 30.6 km^2

	P mm	Q mm	Q/P %	P-Q mm	E_p mm	E_a mm
1977	704.4	343.9	49	360.5	630.1	224.8

Ravensbourne East (37% Urban)
Catchment Area 10.3 km^2

	P mm	Q mm	Q/P %	P-Q mm	E_p mm	E_a mm
1974	828.8	495.8	60	333.0	658.4	282.5
1975	663.2	361.5	55	301.7	652.2	247.3
1976	542.4	192.9	36	349.5	736.4	199.4

River Quaggy (72% Urban)
Catchment Area 33.5 km^2

	P mm	Q mm	Q/P %	P-Q mm	E_p mm	E_a mm
1974	828.8	331.4	40	497.4	658.4	125.6
1975	663.2	268.2	40	395.0	652.2	109.9
1976	542.4	175.4	32	367.0	736.4	88.6
1977	704.4	307.2	44	397.2	630.1	123.4

percentage terms (Table 3.5). Data obtained by Van den Berg (1978) from a small (2.0 ha) residential area of Lelystad, Netherlands with 44% impervious area are given for comparison. This latter study used detailed measurements of rainfall, stormwater discharge and subsurface drainage discharge and an independent estimate of evapotranspiration from the pervious area to calculate annual water balances.

The averaged annual water balances were then compared with the rainfall and runoff figures for predominantly rural catchments in South East England which have also been cut into London clay and chalk (Table 3.6).

DISCUSSION OF RESULTS

The monthly water balances for the River Wandle were not very successful. The poor balance could be due to the over or under-estimation of several

Table 3.5 AVERAGE ANNUAL WATER BALANCES FOR LONDON URBANIZED CATCHMENTS

		P	Q	Ea	Q+Ea
Beverley Brook		(81% Urban)			
Average	(mm)	606	161	86	247
	(%)	100	27	14	41
River Wandle		(32% Urban)			
	(mm)	670	71	309	380
	(%)	100	12	47	59
River Brent		(75% Urban)			
	(mm)	662	479	100	579
	(%)	100	72	15	87
River Crane		(48% Urban)			
	(mm)	647	288	208	496
	(%)	100	45	32	77
River Ravensbourne		(49% Urban)			
	(mm)	704	347	225	572
	(%)	100	49	32	81
Ravensbourne East		(37% Urban)			
	(mm)	678	359	243	602
	(%)	100	52	36	88
River Quaggy		(72% Urban)			
	(mm)	685	270	112	382
	(%)	100	39	16	55
Van den Berg 1978		(44% Urban)			
	mm	666	457	230	687
	%	100	69	34	103

of the components. The actual evapotranspiration figure may be too small because it assumes the urban area is totally impervious whereas there are significant areas of back gardens and open space producing evapotranspiration that are included in the area. A fault of the technique is that soil moisture deficits may be overestimated since "observation shows that even moderate falls of rain can give appreciable increases in stream flow even though substantial deficits exist over the catchment generally," (Grindley and Singleton 1969). Consequently the changes in soil moisture storage will also be overestimated, since these rely on the soil moisture deficit values. The treated discharge figure from the sewage treatment

Table 3.6 RAINFALL AND RUNOFF FIGURES FOR HEAVILY URBANIZED CATCHMENTS IN GREATER LONDON

River	Catchment Area km^2	Standard Average Rainfall 1916-50 mm	Period of Record Average Rainfall mm	Period of Record Mean Gauged Discharge m^3/s	Mean Annual Runoff mm	Runoff as a % of Rainfall
Hogsmill	69.2	691	696	0.94	428	61
Wandle •	176.1	754	533 671	0.95 1.44	170 71	12
Beverley •	42.4	640	628 612	0.53 0.53	384 161	27
Brent •	132.1	678	- 662	- 2.01	- 479	72
Crane •	81.1	-	- 647	- 0.74	- 288	45
Ravensbourne •	30.6	-	- 704	- 0.34	- 347	49
Ravensbourne East •	10.3	657	- 685	- 0.12	- 359	52
Quaggy •	33.5	661	- 685	- 0.28	- 270	39

Range 12-72%

RAINFALL AND RUNOFF FIGURES FOR PREDOMINANTLY RURAL CATCHMENTS IN SOUTH EAST ENGLAND

River	Catchment Area km^2	Standard Average Rainfall 1916-50 mm	Period of Record Average Rainfall mm	Period of Record Mean Gauged Discharge m^3/s	Mean Annual Runoff mm	Runoff as a % of Rainfall
Roding	303	635	622	1.66	172	28
Chelmer	190	595	570	0.96	159	28
Chelmer	534	602	590	1.63	96	16
Colne	238	599	568	1.01	133	23
Lee	1040	649	639	3.74	113	18
Medway	1260	759	773	10.97	274	36
Rother	206	851	907	2.11	323	35
Great Stour	230	749	764	2.13	292	38
Eden	224	775	814	1.89	266	33
Darent	191	754	726	0.66	109	15
Cuckmere	135	825	884	1.68	392	44
Ouse	182	859	874	1.94	336	38
Adur	109	785	806	0.98	283	35
Thames	9950	735	736	67.40	213	29

Sources: Surface Water Year Book UK 1971-3 and survey of GLC data (marked •)

Range 15-44%

works is excessive since it includes a proportion of storm runoff that enters storm flow balancing tanks that are then discharged through the meter. Industrial abstraction makes little significant difference to the flow. These additions and extractions are of a fairly constant flow rate so that on a monthly basis they tend to mask any variation in gauged river discharge in response to rainfall. The chalk headwaters may also make some unquantified delayed contribution to streamflow as a result of changes in storage.

Unfortunately the underestimates of evapotranspiration and over-estimates of soil moisture deficit, soil storage changes and additions of treated effluent did not cancel each other out so that the error term ($P-\bar{Q}-E_a \pm \Delta S$) in some cases was greater than the input rainfall.

By considering annual values of rainfall, runoff and evapotranspiration, any changes in soil and groundwater storage can be ignored as they tend to balance out over a year. Inclusion of figures for additions by sewage treatment works and industrial abstraction in the annual water balances of the Beverley Brook and River Wandle (Table 3.3) gave only a very poor balance. For the Wandle, the subtraction of the net treated effluent and industrial abstraction figures would theoretically produce a negative flow in 2 years. The sum of the adjusted net flow (Q_a) and actual evapotranspiration from pervious surfaces (E_a) accounts for only 41% and 59% of the precipitation for the Beverley and Wandle respectively when averaged over their period of record. One reason for this is that the treated effluent figures include some storm runoff which therefore exaggerates the sewage treatment works metered flow. In addition some allowance may be necessary for evaporation from the urban area, – this latter quantity deriving both from the unaccounted-for areas of gardens included in the % urban fraction and from the roads, roofs and pavements themselves which are capable of storing rainfall both on the surface and to a limited extent within their fabric for later evaporation. The average percentage annual runoff is fairly constant for the 2 rivers. The Beverley achieves an average of 27 ± 7·2% at the 95% confidence level while the Wandle averages 12 ± 5·3% runoff. The influence of urbanization has already reached a steady state by the beginning of the periods under consideration and the artificial drainage system has been little altered by an increase in density of housing. The eventual digitisation of the GLC data will allow examination of far longer records for all the catchments.

Annual water balances for the 3 other catchments monitored by the GLC are equally variable with simply the gauged river discharge being used,(Table 3.4). The annual water balances achieve some extremely large percentage runoff figures with a range of 15% for the Crane to 102% for the Brent in 1976 (a drought year), while virtually all percentage runoff figures are greater than 40%. An average runoff figure with confidence limits cannot be calculated for these catchments because of the short record length. There is no obvious difference between the runoff from the chalk and clay catchments in Tables 3.3 and 3.4

The high percentage runoff figures may cast some doubts upon the quality of the data. Strict quality control is carried out by the GLC and is backed up by current metering through the whole range of flows to ensure that the gauging structure and recorder measure accurately. A possible explanation for these figures lies with imports of water. Only the Beverley Brook and River Wandle have sewage treatment works with metred flow, while the Wandle has the only surviving industrial user. All other water in Greater London is obtained from the mains supply (15% groundwater and 85% river abstractions). There remains the possibility of illegal imports of water in the form of leakage from cracked mains and sewage pipes, faulty pipe connections between supposedly separate storm runoff and waste disposal systems and accidental spillages of wastes from industry. These may seem minor factors, but dozens of observed running storm runoff outlets and wastes flowing from industrial estates during periods of dry weather must contribute significant quantities to the annual runoff and these unquantified unofficial imports are the only logical explanation for the 102% runoff from the Brent in 1976.

When the annual water balance figures are averaged over their period of record (Table 3.5) the sum of the gauged discharge and actual evapotranspiration from pervious surfaces ($Q + E_a$) only accounts for an average 70% of the precipitation for all 5 catchments and ranges from 41% to 88%. This incomplete balance suggests the possibility of some missing factor in the water balance equation, perhaps paved surface evaporation. Alternatively, it may be that the estimate of paved surface area is exaggerated so that a more detailed survey is necessary to include all areas of vegetation such as gardens and grass verges. This may explain the better balance obtained by Van den Berg (1978), included in Table 3.5, whose figures for runoff and actual evapotranspiration were from detailed measurements of flows and more exact estimation of the pervious area contributing evapotranspiration at a much smaller scale of investigation.

Averaging of the annual water balance figures reduces the range of percentage runoff to 12 - 72% (Table 3.6). These figures are within the same range (15 - 44%) as the rainfall and runoff figures for predominantly rural catchments in South East England which have similar geology and climate. Applying a student's 't' test to the 2 samples reveals that there is no statistical difference between the means of the urban and rural catchment percentage runoff values at the 95% confidence level. (Table 3.7). The lowest urban runoff values are from the Beverley and Wandle which have records of imports of water (in this case treated effluent). The net runoff (gauged discharge minus these imports) produces similar runoff coefficients to those for the South East region which suggests that urbanization has had little effect on increasing runoff, as is usually anticipated, (eg Savini and Kammerer 1961). Runoff figures from the other rivers reflect unmonitored illegal inputs which may well be of the same order as the sewage treatment works flows. It is therefore possible that the annual flows from all 5 rivers are little different from the regional runoff values. This tentative conclusion from actual data is at variance with that theoretically derived by Lvovich and Chernogayeva (1977) who estimatea runoff coefficient of 0.73 for central Moscow, and James (1965) whose work in California indicated an urban water yield of 2•29 times the rural.value. From the London data it cannot be concluded that urbanization greatly increases annual yield.

Table 3.7 Comparison of Urban and Rural Percentage Runoff

		Mean % Runoff	2 Standard Errors
Urban	(Greater London)	44•63	± 13•36
Rural	(S.E.England)	29•71	± 4•78

CONCLUSION

The standard measurements of rainfall, runoff and evapotranspiration which are generally sufficient to produce a reasonable balance for a rural catchment are inadequate for an urban catchment. Knowledge of the sources of all inputs and outputs to the system is necessary if any sensible quantities for the different components of an urban hydrological cycle are to be found. This will only be achieved by close monitoring at a small scale of the artificial as well as the natural drainage system.

No definite conclusions about the effect of urbanization on runoff can be drawn as the water balances calculated merely indicate that the errors involved are large. Annual water balances for 5 London urbanized rivers have been achieved but only an average of 70% of the rainfall can be explained by gauged runoff and independent estimates of actual evapotranspiration from pervious surfaces. There is no statistical difference between the average percentage runoff from urban and rural catchments in South East England based on similar geology so that it cannot be concluded that urbanization increases runoff.

Because of the poor balance obtained even using annual figures of rainfall, runoff and evapotranspiration so that changes in storage of soil and ground water can be ignored, small scale studies were instituted to provide estimates of the size of other elements involved. The instruments necessary were set up on a small residential housing estate and details of the catchment and the results obtained follow.

Chapter 4

INSTRUMENT SELECTION & DESIGN FOR MEASURING URBAN HYDROLOGICAL PROCESSES

INTRODUCTION

The range of instruments available for monitoring hydrological processes is wide but choice can be reduced by individual site requirements, budgetary constraints and recommendations made by experienced field workers (eg Marsalek 1976). Such processes may include precipitation, evapotranspiration, runoff, soil moisture and groundwater storage. In an urban context, additional processes such as runoff from pitched and flat roofs and other paved surfaces, paved surface evaporation, depression storage, infiltration through 'impermeable' surfaces, interception, absorption and storage of moisture by urban fabrics would also ideally be measured.

This chapter details measurements of precipitation, open-water evaporation, paved surface runoff, pitched and flat roof runoff and the indirect derivation of figures for depression storage and the combined losses of paved surface evaporation and infiltration within a typical residential housing estate.

The reasons for these measurements include efforts to establish runoff coefficients for a set of different urban surfaces and to develop better assessment of effective rainfall (ie the percentage of rainfall becoming runoff) which would aid engineering design. The estimation of the size of the different components of the hydrological cycle involved in an urban water balance would help advance theory, model building and eventually improve water resource management.

The following sections discuss the range of instruments available to monitor the chosen processes and detail the actual instruments either bought or designed to accomplish the above aims.

SITE SELECTION

Choice of site was restricted by the following requirements:

1. Easy access for care and maintenance of instruments making the distance from home or college important.
2. Secure site with restricted access to avoid unauthorised interference placing preference on a privately owned housing estate with the approval and cooperation of residents.
3. Separate storm sewer system serving about 50 to 100 houses with recent detailed plans with the further proviso that there should be no dry weather flow which would otherwise indicate connections with the foul sewer system or seepage.

Inspection of several possible sites in Greater London did not produce one sewer system without dry weather flow. The site finally chosen was part of The Park housing estate, Redbourn, Hertfordshire, (Figs 4.1 and 4.3), where the three preliminary site requirements could be met. The sewer system outfalls into the River Ver and is supplied by runoff from the roads and one block of garages. All other roofs drain to soakaway in this area of chalk. Most of this information was available from original drawings but a detailed survey was undertaken to give spot heights for contouring and accurate levelling of the manhole depths. Actual road catchment boundaries were delineated by survey and by use of a hose pipe and water to provide accurate limits over gently changing slopes. Because the infiltration capacity of this chalk soil is very

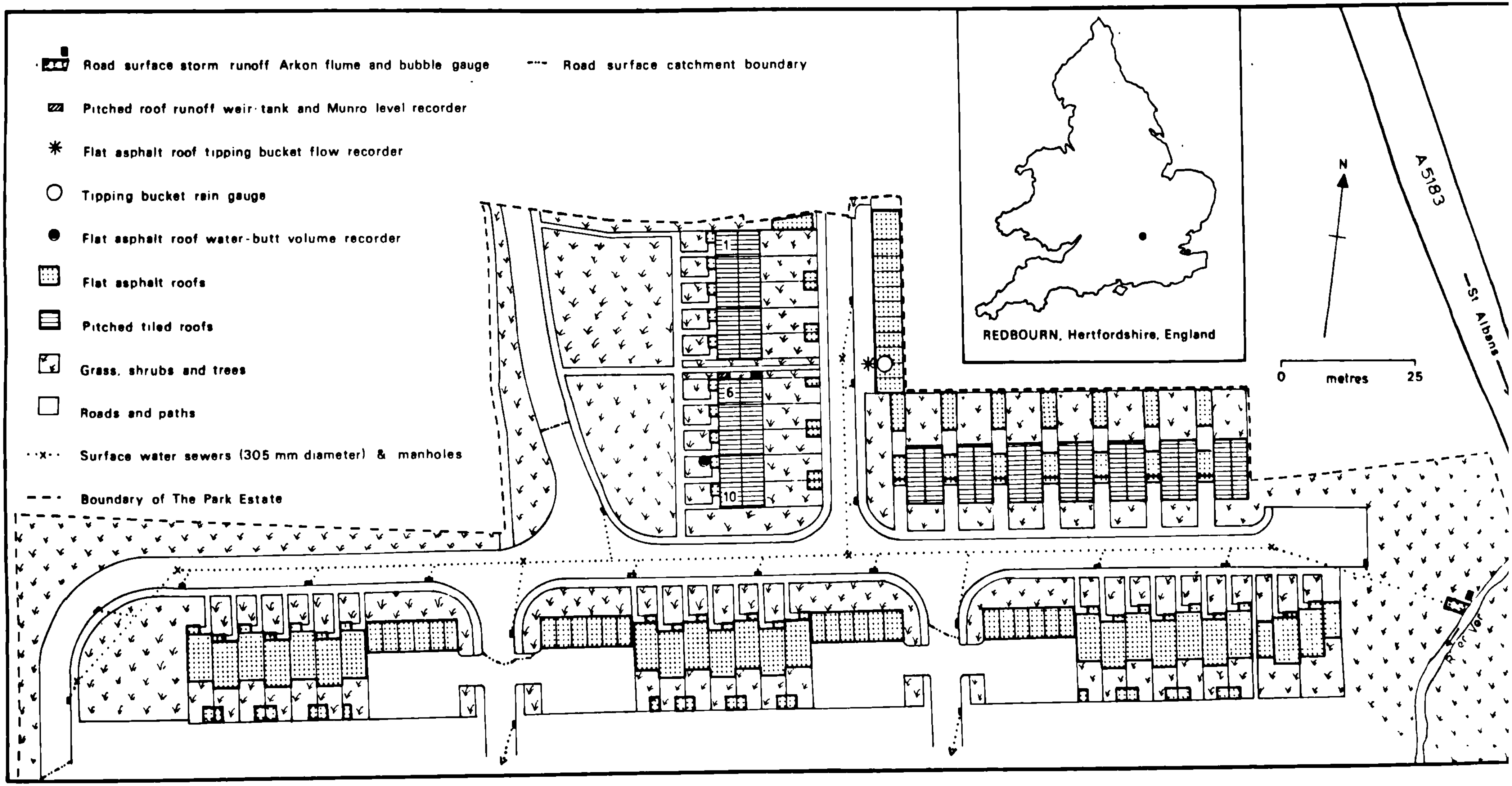


Fig 4.1 Catchment and instrument position information The Park Estate, Redbourn, Hertfordshire.

large it was considered that overland flow from pervious areas was unlikely to occur and contribute to the sewer. Therefore the road catchment area is limited to the impermeable areas of road surface, pavements and any garden sheds or garages that contribute directly to the road and excludes any driveways which shed water to the side onto gardens.

The impermeable areas of roads, pavements and the one block of garages draining to the highway drains is 3465•4 m^2, the 2 pitched roof areas of monitored flow draining to soakaway are 109•4 m^2, monitored flat roof flow area is 48•7 m^2 from a portion of garage roof and 4•8 m^2 from the porch roof. The catchment has a fairly steep central section but the overall road ground slope is 1•37%. Pipe slope, depth and length information are given in table 4.1.

DATA LOGGING

The type of data recording system chosen will partly determine the measuring instruments that can be used. For instance, if synchronous recording of all measurements onto one time base is required, the instruments will need some form of electrical output. However most instruments can be adapted where necessary. While synchronous logging is the ideal system, central logging can be extremely expensive (£20,000 upwards) and computer back-up facilities are necessary to translate the data into useable form. From a brief review of the available data logging devices with respect to financial and practical limitations,the only logical cheap but workable system entailed the use of autographic chart recorders. These have one main advantage over magnetic and electrical recording systems in that they allow immediate visual assessment of the data and any instrument malfunction can be quickly identified and remedied.

Table 4.1 Storm Sewer Pipe Information

Manhole No.	Manhole Above	Diameter (m)	Slope (%)	Length (m)
6	0	0	0	0
5	0	0	0	0
4	5	0.30	1.16	27.5
3	4	0.30	0.57	61.5
2	3	0.30	3.52	58.0
2	6	0.30	1.03	35.0
1	2	0.30	3.14	76.0
Flume	1	0.30	0.68	34.0

Average Main Pipe Slope 2.07%
Average Ground Slope 1.37%

Therefore all the instruments selected and detailed below use chart recorders.

MEASUREMENT OF PRECIPITATION

Accurate measurement of precipitation is very important. The precipitation caught by a single raingauge at a point is assumed to represent a sample of the same depth of rain falling on a large area surrounding the gauge. Because of the variability of areal rainfall, Linsley (1973) recommends the use of at least 2 raingauges even for the smallest catchment, with preferably at least one within the catchment boundaries.

For urban catchment studies recording raingauges are essential. Depending on the use of the data and the size of the catchment, Marsalek (1976) offers guidelines as to the ideal time resolution of precipitation data. The range is from a time resolution requirement of 1-2 mins. for experimental watersheds to 60 mins. for planning data. There are 3 types of recording raingauge generally available based on the principles of a tipping bucket, siphon or weighing. The last is more commonly used in the USA and Canada. Details of their operation may be obtained from the HMSO Handbook of Meteorological Instruments (1956).

For urban runoff studies, the tipping bucket gauge is considered preferable (Smoot 1971), mainly because of its superior actuating mechanism and greater accuracy over medium intensity rainfalls. The tilting of the bucket creates an electrical signal which can be logged as a pulse on one channel of a magnetic tape and so this type of raingauge can be used where synchronous recording of data is required. However it suffers from the disadvantage that rainfalls of less than bucket capacity are not recorded and are added on to the next storm. Part buckets-full provide a large surface area for evaporation losses. Both tipping bucket and tilting siphon raingauges will under-record rainfall during the tipping or siphoning mechanisms.

Despite the advantages of the tipping bucket raingauge, a Casella Tilting -Siphon Recording Raingauge (W5552) was used since it was available from college store. The chart moves at a speed of 11·4 mm/hr recording rainfall increments of 0·1 mm. The siphon is activated after a rainfall of 5·0 mm. As 0·1 mm is a standard bucket capacity for tipping bucket raingauges used in urban runoff studies (eg the Rimco 0·1 mm raingauge used by the IOH), total rainfall records would be as accurate. The chart speed is only sufficient for reading 5 minute intervals whereas a tipping bucket can provide resolutions of 30 seconds, but, as accurate volume and not timing was the main requirement this was not felt to be a major disadvantage. To prevent frost damage to the float, thin sheets of polystyrene were wrapped round the inside of the raingauge as insulation and a 40 watt light bulb operated on a thermostat which was activated when the temperature dropped to 3°C (Fig 4.5).

Precipitation was monitored using 3 gauges located within the grounds of the neighbouring Brooke-Bond factory (Figs. 4.2 and 4.4). One being the tilting-siphon recording raingauge detailed above and the remaining 2 gauges being daily check gauges. The site originally contained only one check gauge, a black-painted Gallenkamp Snowdon pattern raingauge with the later addition of a splayed base Meteorological Office Mark 2 copper standard raingauge. This new gauge was then used for the reference rainfall catch and the catch was distributed according to the record from the tilting-siphon raingauge. The Brooke-Bond Oxo factory is patrolled by security guards and access is limited to authorised personnel so that the gauge site is a compromise between adequate gauge protection and proper gauge exposure. The site is not ideal but most nearly meets Meteorological Office specifications that any obstacle (trees, buildings, walls etc) is at a distance equal to twice its height away from the raingauge. One or two trees may fall within this distance so that the gauges suffer from under-exposure. As access to these gauges was impossible at weekends, a further raingauge was sited on the garage roof block behind No's. 1-10 The Park. This gauge, a Casella Tipping Bucket (W5699) tilting every 1·0 mm with a chart speed of 11·25 mm/hr, is used to separate the rainfalls from the other gauges where necessary.

MEASUREMENT OF STORM RUNOFF FROM PAVED SURFACES

Flow can be monitored either in the sewer system, or whenever possible, at the sewer outfall.

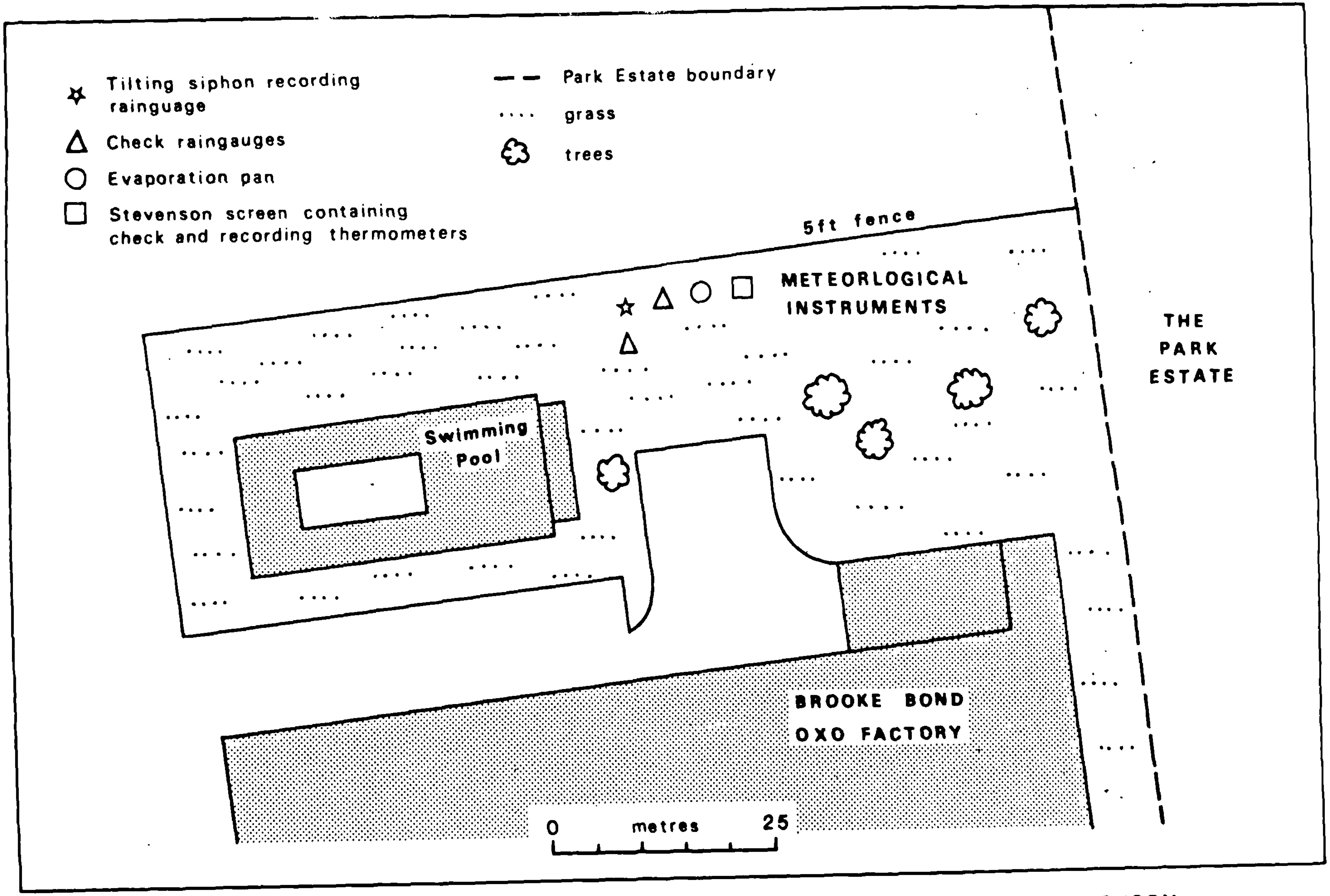


Fig. 4.2 METEORLOGICAL INSTRUMENT LAYOUT WITHIN THE BROOKE-BOND OXO FACTORY, REDBOURN, HERTS

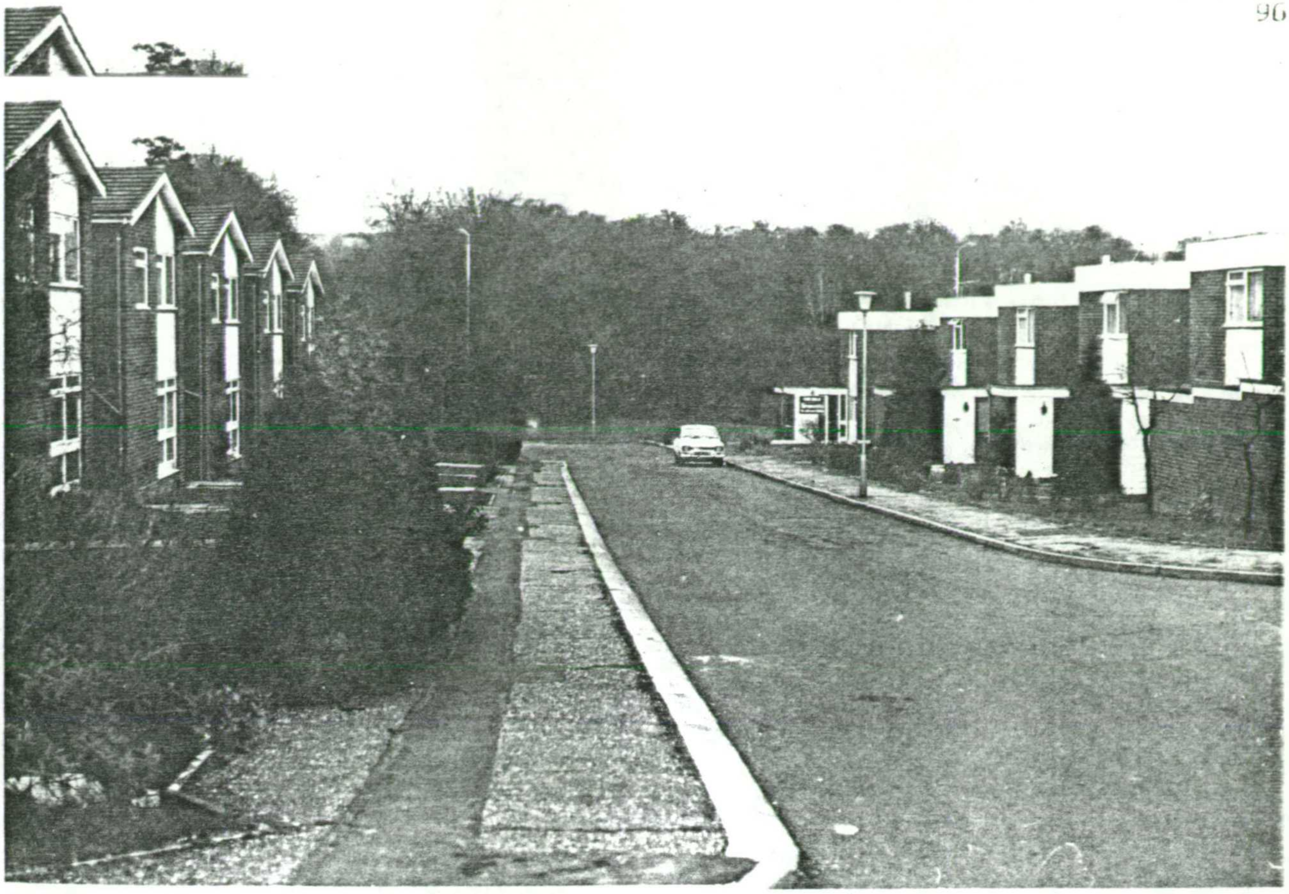


Fig. 4.3 View of the catchment looking towards the flume.

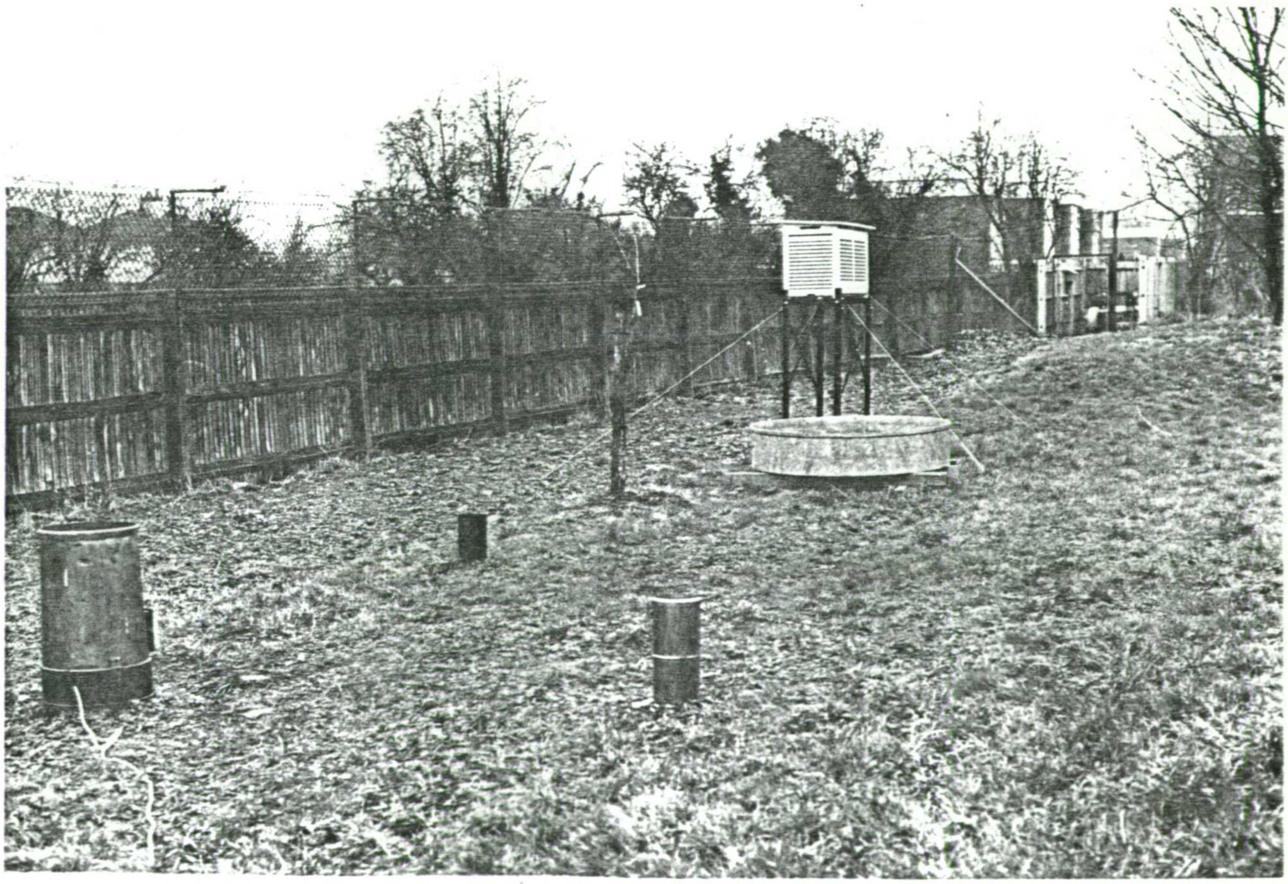


Fig. 4.4 View of meteorological instruments - recording raingauge, 2 standard raingauges, evaporation pan, Stevenson Screen.

Fig. 4.5 Opened tilting-siphon raingauge showing 40 watt light bulb and polystyrene insulation to prevent frost damage.

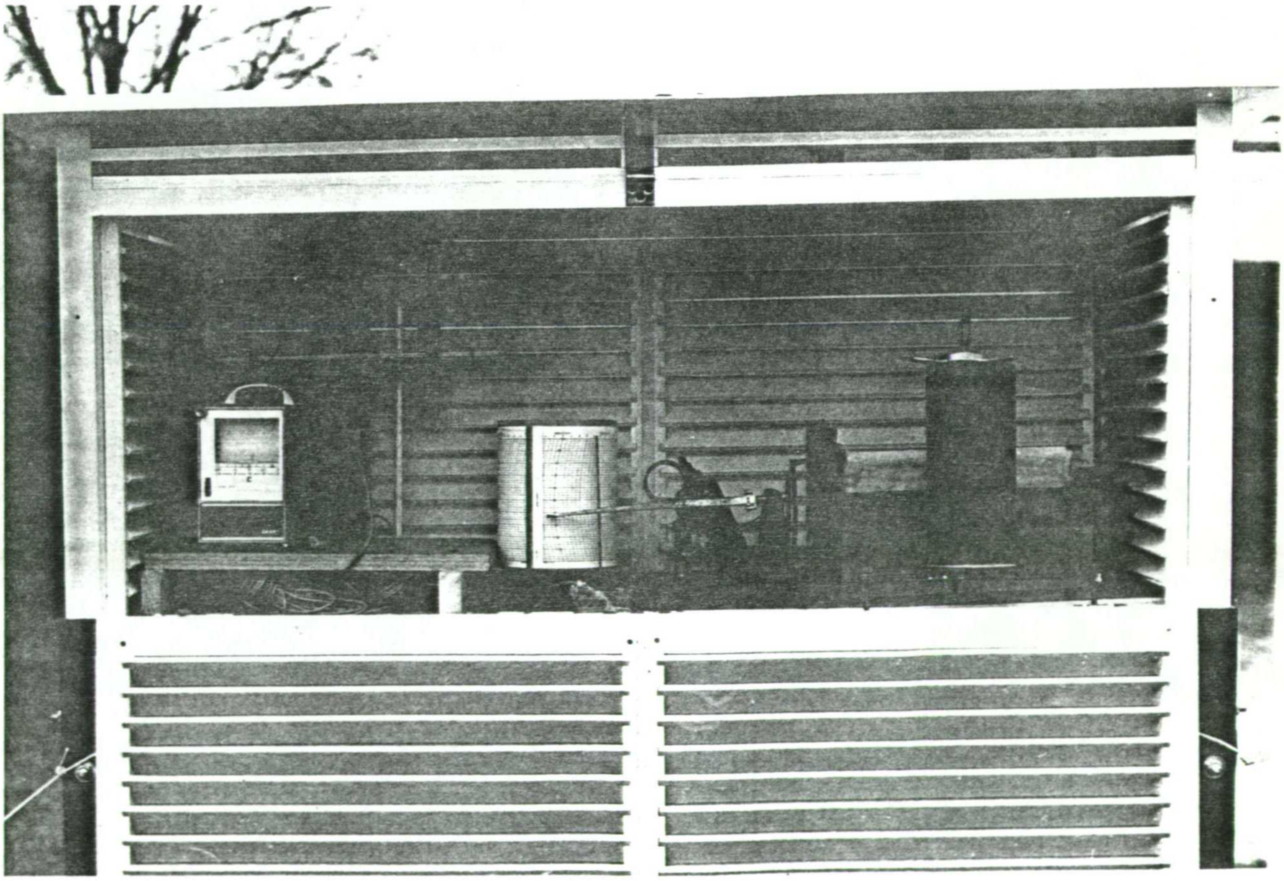


Fig. 4.6 Contents of Stevenson Screen.

Kirkpatrick (1975), lists the following primary design goals for storm runoff measuring equipment:-

The system must

a) be capable of monitoring flows of 0·1 to 30 ft^3ps (0·03 to 9·1 m^3/s)

b) have an accuracy of ± 5 to 8% at the readout point

c) maintain its accuracy under accelerated flow conditions of depth and flow velocity

d) be capable of measuring a full range of open channel flow in a closed or open conduit and flow with the conduit flowing full and under pressure

e) not be seriously affected by the movement of solids, such as sand, gravel and debris within the fluid flow

f) have no direct interference with flow (non-intrusive) to avoid flow blockages

g) have the capability for installation in confined, corrosive and moisture laden spaces, eg sewer manholes

h) require minimum power to operate and be adaptable for a system of multiple flow measuring points and one readout.

Marsalek (1976) reviews the techniques for sewer flow measurement and reports on both the laboratory and field test performance of several commercially available instruments. The main techniques for sewer flow measurement are as follows:- 1) depth of flow measurement; 2) measured velocity (point and mean) and area methods; 3) weirs and flumes.

1. <u>Depth of Flow Measurement</u>

By measuring the depth of flow in the sewer pipe it is possible to calculate the discharge assuming uniform flow for a known cross-sectional area.

The Manning equation is used for the flow rate calculation:-

4.1 $$Q = A \frac{m^{\frac{2}{3}} \, i^{\frac{1}{2}}}{n}$$

Q = discharge (m^3/s)
A = cross - sectional area (m^2)
m = hydraulic mean depth (m)
= $\frac{\text{area of cross section of water}}{\text{length of wetted perimeter}}$
i = slope of channel (dimensionless)
n = Manning's roughness coefficient

This technique suffers from several errors. Unsteadiness and non-uniformity of flow and the variability of Manning's n may combine to produce errors of 15 - 20% (Marsalek 1976).

Depth of flow measuring instruments consist of the following types:-

Resistance or capacitance probes

Dipping probes

Floats

Bubble gauges

Ultrasonic probes

Resistance or capacitance probes

These gauges generally consist of twin conductors which are mounted vertically in the water. Variations in water levels cause changes in the electrical path between the electrodes and are interpreted in terms of changes in either resistance or capacitance. Potentiometers have been used in conjunction with a flow measuring structure such as a weir or flume by several countries. Verworn (1978) describes the use of a 10 turn potentiometer in conjunction with a horizontal sharp-crested weir built into a conduit at an experimental catchment in West Germany. Experimental urban catchment studies, in England by the IOH have used water level sensors incorporating a barrel potentiometer (Makin and Kidd 1979) and similar studies in the Netherlands (Van den Berg et al 1977) have made use of potentiometers to measure both storm water discharge (with a Thompson 30^{o} V-notch) and rainfall. Swedish experimental work on storm runoff makes use of the changes in electrical conductivity of 2 parallel platinum wires in an electrolyte in response to changes in water level, (Falk and Niemczynowicz 1978).

Dipping Probes

A thin stainless steel probe is lowered on a wire controlled by a precision motor. When the probe makes contact with the water surface this completes an electrical circuit and the probe retracts slightly and then repeats the cycle. Changes in water level produce shortening or lengthening of the wire. These probes cannot be used if the pipe should ever be completely dry as the wire spool will be emptied without the wire making the necessary water contact. Trials by the IOH (Makin and Kidd 1979) on one commercially available model also indicated that waves and excessive turbulence of the water surface when no 'stilling' of the water in the pipe was possible caused large errors in the recorded depth at high flows.

Floats

The float is the most commonly used instrument for measuring water-surface level. The float is attached to a cable which passes round a pulley and then to a counterbalance weight. The float rises and falls with the water surface. They are very reliable, inexpensive and easy to maintain. The float needs to be housed in a stilling well but there is rarely space for this in a sewer pipe.

Bubble Gauges

The bubble gauge consists of a tube which is mounted with its open end below the water surface. A supply of gas under pressure bubbles through the end of the tube. The pressure required to overcome the head of water above the outlet of the tube is measured. This gives a direct measure of the water surface level. These instruments are relatively simple, do not obstruct flow and are the only ones suitable for operation in surcharged sewers. Bubble gauges have been used in experimental urban runoff studies in Canada (Tupper and Waller 1978) and England (Kidd 1976) in conjunction with other flow measuring structures.

Ultrasonic Probes

The sensor is located above the liquid and emits a signal which is reflected back (echoed) to the sensor where it is translated into a measure of the depth of the liquid.

2. <u>Velocity-Area Methods</u>

The second main group of techniques for measuring runoff involve measurement of the flow rate multiplied by the flow area (inferred from pipe geometry). The flow rate should be the mean velocity.

Current meters may supply a measure of the point velocity which in itself is unsatisfactory but current meters are also unsuitable for use in sewers where they suffer from clogging or disruption by floating debris.

Mean velocity estimates can be obtained by ultrasonic, electromagnetic and tracer techniques.

The ultrasonic method involves the use of 2 sonic sensors which can transmit and receive sound pulses placed inside the pipe. They produce echo paths diagonally across the flow. The difference between the time of travel of the pulses travelling upstream and those travelling downstream is directly related to the average velocity of the water at the depth of the sensors.

Pipe flow meters are based on Faraday's Law of electromagnetic induction. Here a magnetic field is created inside the pipe and the liquid flowing past cuts the field and induces an electrical potential which is detected by 2 probes. The electrical potential induced is proportional to the average velocity of flow.

These last 2 methods can only be successfully applied to completely filled pipes and therefore they have been little used in sewers. Their high price is another limitation.

Dilution gauging provides a further means of determining mean velocity. The 2 main techniques are gulp and constant rate injection of some tracer material (usually a salt) and the resulting concentration wave is measured by a conductivity probe. Most gauging has been for calibration of pipe stage-discharge relationships and has involved manual techniques which have worked well in wastewater sewers. But in storm sewers it is seldom possible to be on site during a high discharge and only rarely can similar flows be introduced by pumping. Developments in sewer gauging have been described by Blakey (1969) and work on an automatic injection and downstream sampling system is proceeding at the IOH (Harvey et al 1976). The choice of a suitable tracer is highly dependent on local conditions – the nature of the suspended load and sorptive processes - which means that extensive tests are necessary before dilution gauging can be implemented (Neal and Jordan 1978).

3. Measuring Weirs and Flumes

These are structures built across the direction of flow which, if standard designs are chosen, have a specific stage discharge relationship. The main types of structure are as follows:-

Thin plate weirs	:	rectangular, V-notch and trapezoidal
Long base weirs	:	streamlined, triangular profile, rectangular profile and compound
Critical-depth flumes	:	triangular, trapezoidal, U-shape and Venturi.

Ackers et al (1978) and Herschy (1978) provide detailed reviews of the types of weir and flume used for measuring flow as do the British Standard (BS 3680) publications on the subject.

The main advantages of weirs are that they are low-cost, reliable and accurate. However, they are unsuitable for installation in sewers which have debris because they may block the flow and reduce pipe capacity. Some of these problems may be overcome by the use of a slot-type weir where the bottom section is removed to allow passage of material but this reduces their range of performance. Weirs cannot operate in surcharged or drowned conditions.

Flumes are better suited for use in sewer pipes as they merely provide a constriction in the pipe and do not block the flow. They are as reliable and accurate as weirs but more expensive. Critical depth flumes require only a measure of upstream depth, but again they become inoperable under submergence and surcharging. Venturi flumes are capable of operating under surcharged conditions and are a combination of the standard critical depth flume and a venturi meter. Double gauging is necessary, ie measuring water levels upstream and in the throat. Two have been recently developed for use in urban runoff studies, one by the USGS (Smoot 1975) and one by the University of Illinois (Wenzel 1975). Melanen (1978) reports on the use of a venturi to measure stormwater flow in Finnish urban storm drainage projects.

Both weirs and flumes require some measure of head and any of the previously mentioned liquid level monitors may be suitable. Table 4.2 provides a summary of the flow measurement techniques with recommendations as to their use.

Owing to the relative accessibility of the storm sewer pipe outfall (beneath a grass cover in open space) a flow measuring structure was chosen as the most suitable monitoring system following Marsalek's recommendations (1976). Choice of structure type was guided by Ackers et al (1978) whose logic diagram in Chapter 1 led to the final selection of a U – shaped or semi-circular bottomed flume. Besides the advantages that all flumes have (namely no obstruction to any floating debris or flow) this shape flume is more accurate at gauging low flows because of the narrower base and is the most suitable shape for joining on to pipe sections.

Runoff from the road, pavements and garage block draining to the storm sewer system is monitored at the outfall using a glass-fibre pre-cast flume manufactured by Arkon Instruments of Cheltenham. To determine the size of flume necessary, 2 of the very simple design procedures were used.

Table 4.2

Overview of Sewer Flow Measurement Techniques

Technique	Free Flow	Free & Pressure Flow	APPLICABLE Outfall	Manhole	Sewer Pipe	Estimated Accuracy	Cost Range	Recommended
Depth Measurement Only	✓	✓[1]	✓	✓		20%	L	No
Velocity - Area	✓	✓[2]	✓	✓	✓[2]	3-5%	M-H	Yes [3]
Weirs and flumes	✓	✓[4]	✓	✓	✓	5%	L-M	Yes

L = low, M = medium, H = high

1 Measuring pressure drop between 2 manholes

2 Not current metering

3 Except dilution gauging

4 Venturi flumes only

(Adapted from Marsalek, 1976)

The Rational method is the simplest.

$$Q = ciA \qquad \begin{aligned} Q &= \text{discharge} \\ c &= \text{runoff coefficient} \\ i &= \text{rainfall intensity} \\ A &= \text{catchment area} \end{aligned}$$

The design rainfall intensity was calculated from the Flood Studies Report (Vol II, NERC 1975) and a 15 minute rainstorm with a return period of 20 years selected (known as the 15 min M20 rainfall) which gave a rainfall of 17·82 mm equivalent to 71·28 mm/hr. The impermeable catchment area supplying the storm drains was 3465 m^2 so that the runoff coefficient could be assumed as unity (an assumption often made in runoff calculations eg Watkins 1962). The peak runoff rate was therefore 68·6 l/s.

The second method used was Manning's formula for uniform flow in open channels and pipes, to calculate the maximum capacity of the pipes. (Equation 4.1). For a 305 mm diameter pipe, with a pipe slope (i) of 2·07% and assuming uniform flow in full pipes and n of 0·010 (for glazed stoneware) the maximum discharge was 189·1 l/s.

The Rational Method is known to grossly overestimate the peak discharge (Colyer and Pethick 1977) and yet it calculated a third of the volume of runoff that the pipe could carry. Presumably the original design allowed for a much larger contributing area. Use of the wrong size flume could either result in frequent flooding out if too small or inaccurate gauging of low flows and greater expense if too large a flume was chosen. Advice was sought from both experienced field workers and Arkon Instruments. The final recommended solution was a nominal 25 l/s flume which could accomodate flows of 50 l/s which was specially designed to fit on 305 mm diameter pipes.

The flume has an approach section of 305 mm width narrowing to 203 mm in the throat. A 305 mm square precast stilling-basin is connected 533 mm upstream of the throat section. A U-shaped approach section of 10 times the width of the approach section (3·05 m) was made out of glass fibre at University College London according to BS 3680, part 4c, in order to provide relatively smooth, non-turbulent approach flow conditions. After removal of the necessary length of existing pipe, the approach section and flume were connected to the pipe and levelled throughout their length and set in concrete. The structure is covered with 20 mm marine ply boards fitted over 12 mm bolts with nuts to prevent vandalism and accidents, (Fig.4.7).

To prevent high water levels in the River Ver causing backing up of water in the flume, frequent weeding of a downstream watercress bed was undertaken until frosts achieved the same effect in winter.

Water level is monitored in the stilling-well by means of an Arkon bubble-gauge Model 63TN housed in a 12 mm marine ply case. (Fig.4.8). The instrument is connected by an impulse pipe to a dip tube immersed in the water (Fig. 4.9). A pneumerstat controls the air flow and provides continuous bubbles from the dip tube. As the head varies so the pressure to the instrument varies and this information is recorded on a chart as a pen-trace of water level. The chart is clockwork driven and has a speed of 20 mm/hr. The conversion of water level to discharge is achieved by a stage-discharge relation, (Fig. 4.10).

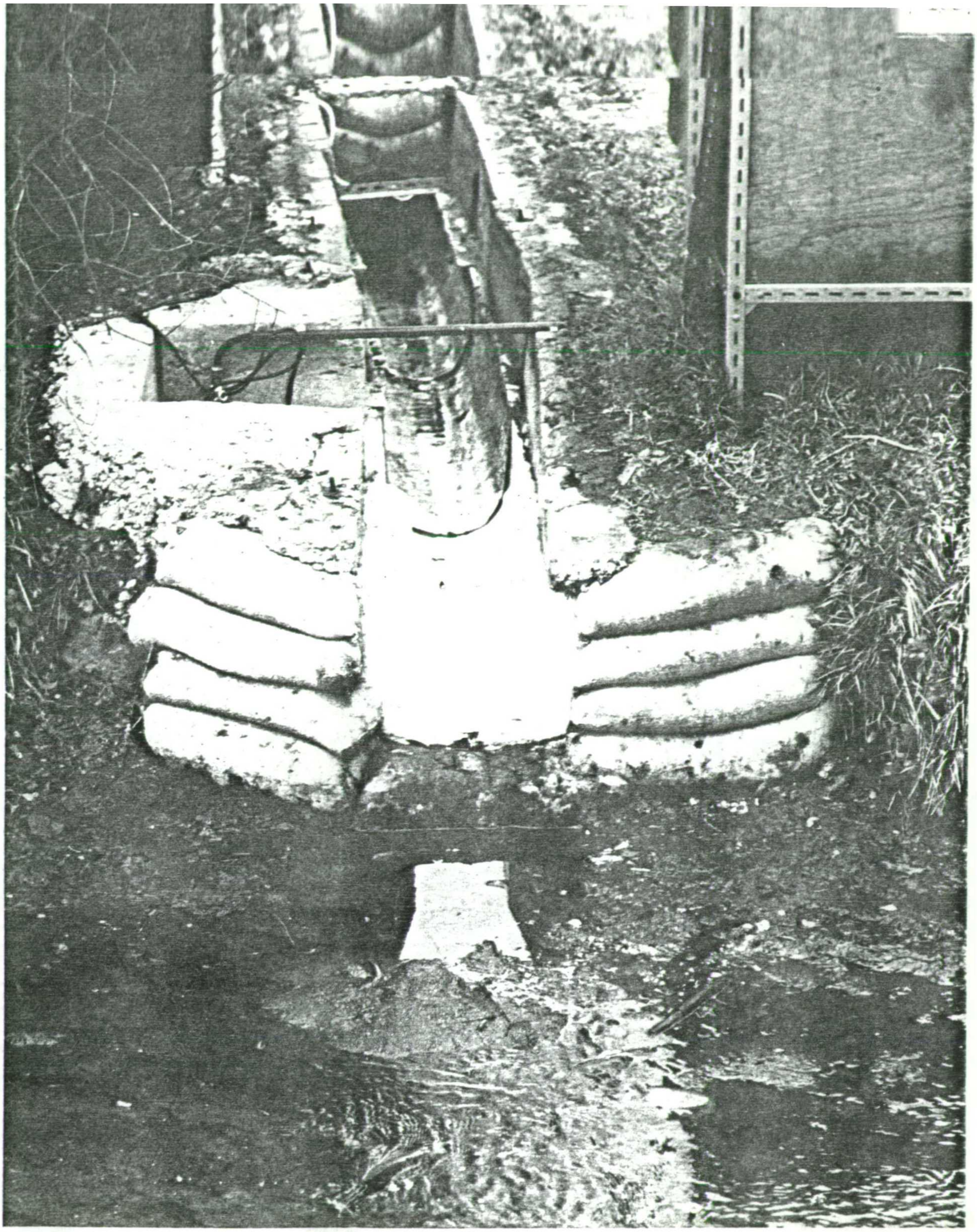


Fig. 4.7 Approach channel and flume at outfall (with covers removed).

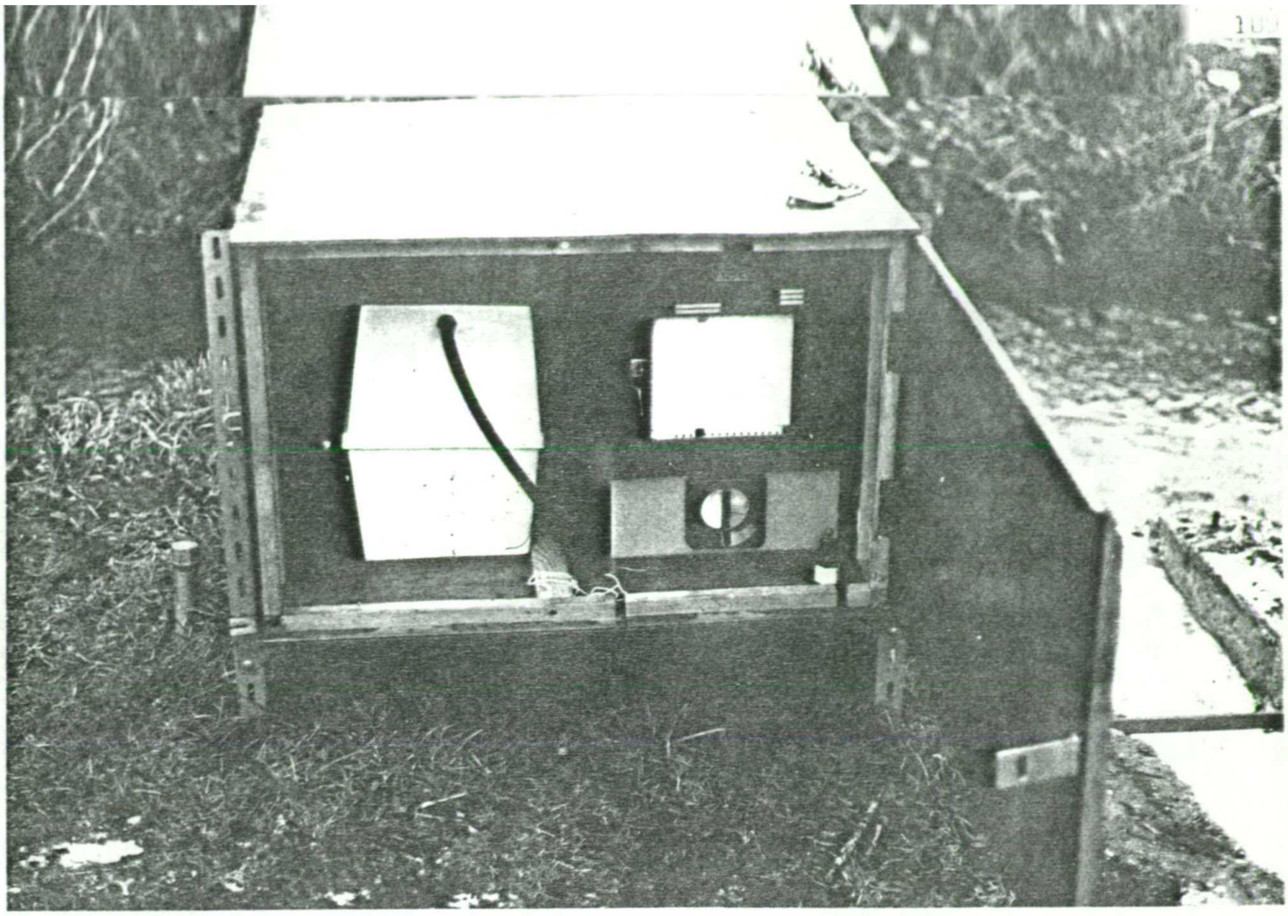


Fig. 4.8 Arkon recorder and pneumerstat.

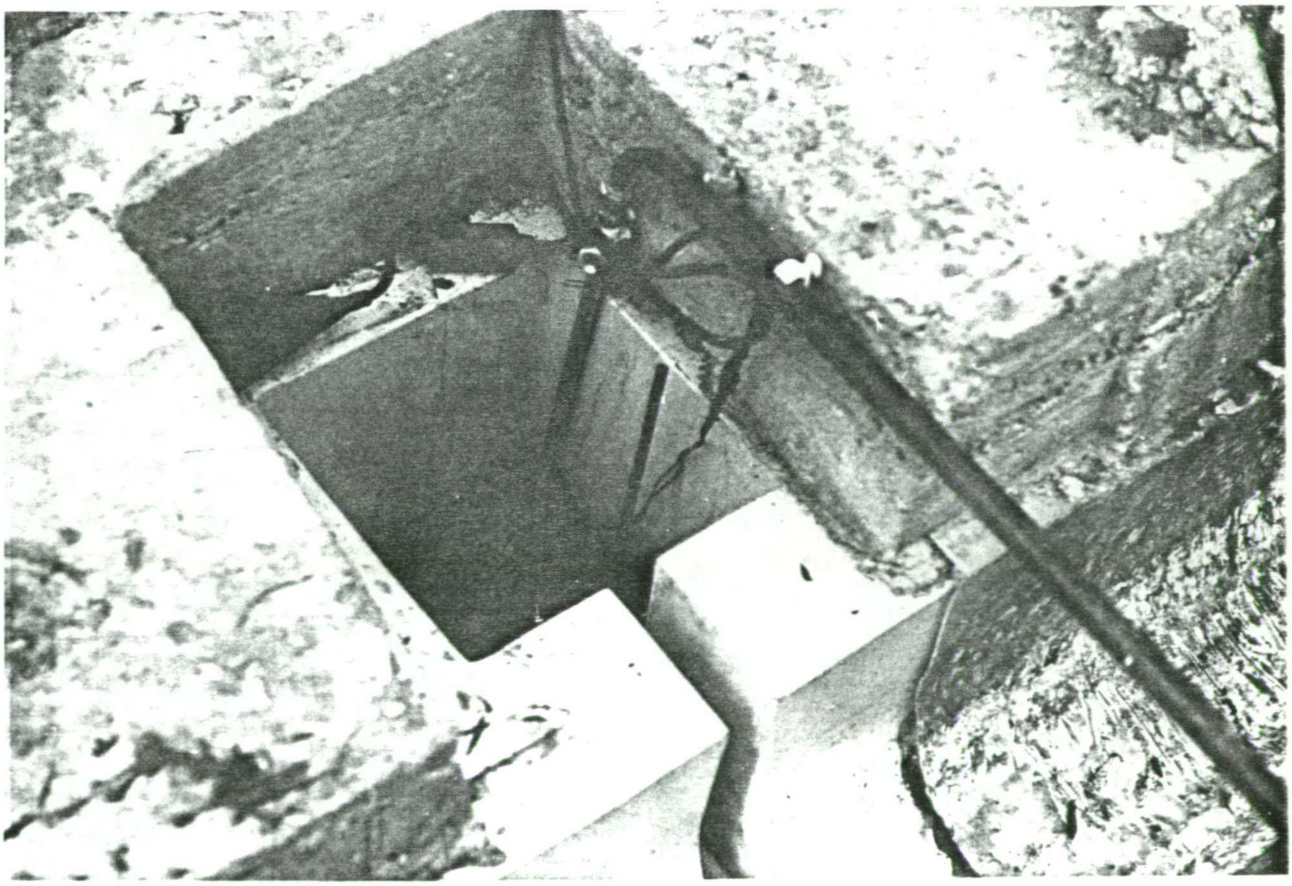
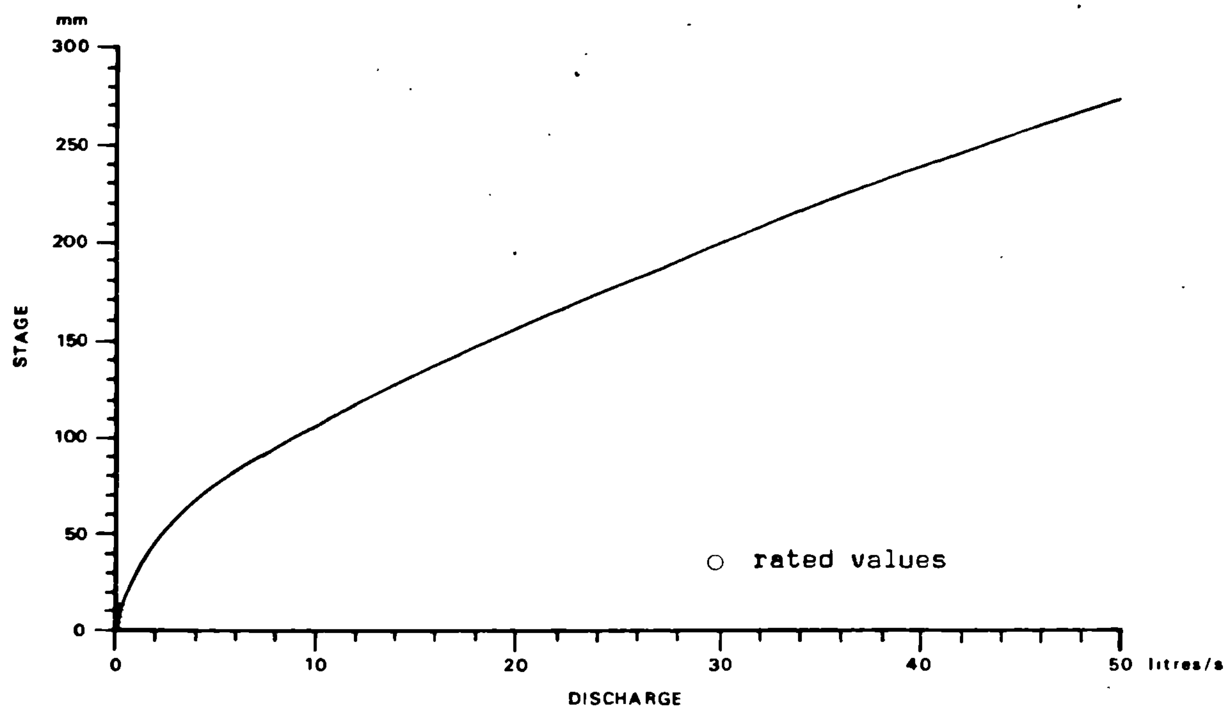


Fig. 4.9 Arkon bubble gauge in stilling-well.

Fig. 4.10 STAGE-DISCHARGE GRAPH FOR ARKON SEMI CIRCULAR FLUME



SEPARATION OF RUNOFF INTO ITS CONTRIBUTING SOURCES

Further advances in knowledge of urban runoff processes requires that the above-ground and below-ground phases be considered separately, (Kidd, 1976). This has led to several countries developing meters which are capable of gauging flows at the phase-boundary in the gully pots, for the UK: Blyth and Kidd (1976); USA: Schaake (1969); Netherlands: Zuidema (1978); Canada: Tupper and Waller (1978); Sweden: Lindh (1976); and Denmark: Jacobsen (Unpub. Report 1978).

Sweden, USA, Canada and the Netherlands employ laboratory calibrated weir plates inside the gully inlet while the UK uses a hinged gate where the angle of opening caused by water flowing from the gully pot into the sewer system is proportional to the discharge. This difference in design was largely necessitated by the small size of gully pots in the UK in comparison with those of other countries. These systems are based on measuring discharge which gives relatively low accuracy at low flows. Makin and Kidd (1979) report that the UK design gully meter cannot record flows below 0·08 l/s causing difficulty in the exact definition of the start and end of runoff. The Danish gully meter measures volume which gives improved accuracy at low flows.

The IOH did offer to hire some of the UK design gully meters for this experimental catchment study together with the tape translation facility. However the flume had already been installed on the storm runoff sewer outfall and funds were not available for this additional equipment. Also it was felt that the chosen equipment would give sufficient separation of the runoff into that from roofs and roads as the roofs drained to soakaway and the total runoff recorded at the outfall would be that from the roads alone (plus one garage block) and not from combined roofs and roads.

MEASUREMENTS OF ROOF RUNOFF

Further division of storm runoff into flows from roofs and roads is infrequently done in urban field studies. An attempt to measure roof runoff has been made at Lelystad, in the Netherlands (Zuidema 1978) by installing a 30° Thompson V-notch weir plate in the sewer pipe at a manhole. In the UK, 2 studies have used a 1·2 litre tipping bucket of IOH design (IOH report 43, 1977) to collect runoff from sloping tiled roofs. As yet the data collected by Middlesex and Trent Polytechnics is unpublished.

Owing to a lack of research on this area of hydrology, new forms of instrumentation were developed to monitor flow from roofs.

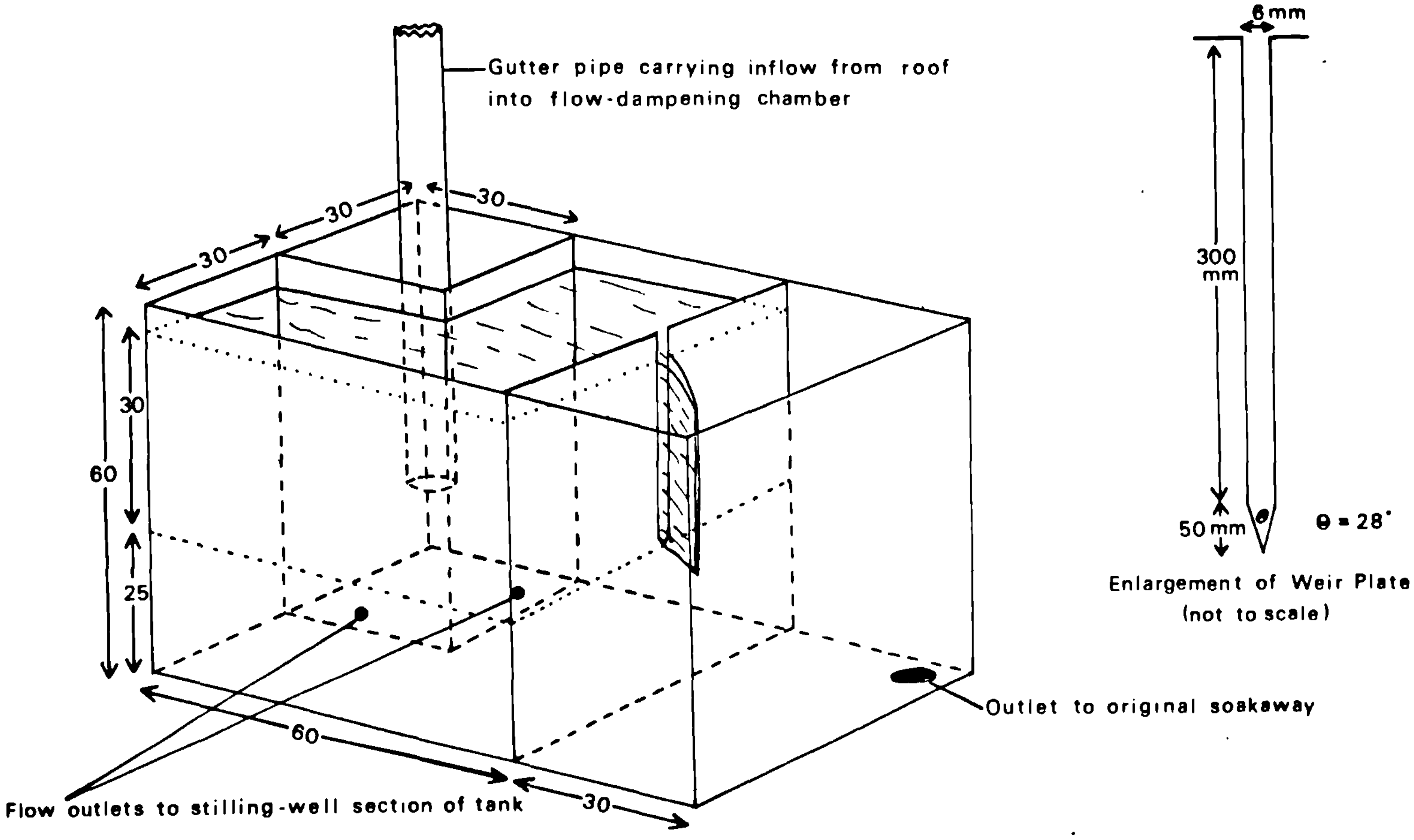
To estimate the flows from a block of tiled roof houses on The Park Estate, the Flood Studies Report (Vol II, NERC 1975) was used to obtain design rainfall intensities. Using the 15 min M20 for this area of England and the plan area of the house roofs from which runoff was to be measured, the peak discharge was calculated as 0·84 l/s.

The Building Research Establishment has issued a code of practice with which to determine the sizes of gutter pipes for roof drainage (Digest 107, HMSO 1972). This recommends the adoption of a 75 mm/hr rate of rainfall and suggests that this rate may occur for 6 minutes once in 5 years, 9 minutes once in 10 years and for 13 minutes once in 20 years, which is fairly similar to the Flood Studies value of 71·28 mm/hr lasting 15 minutes recurring once every 20 years. For roof pitches up to 50^o, the angle of pitch is ignored and the flow load in litres per minute is 1·25 x actual roof area (in m^2). Thus the flow load is 1·14 l/s for each 54 m^2 roof area - the front and rear halves of the block of houses.

New instrumentation was therefore made to these design values. As the roofs drained to soakaway in this area of chalk, it was not possible to insert a weir plate in the drains as in the Dutch experiments (Zuidema 1978). Two iron zinc-painted weirs were designed according to BS 3680 part 4 (Fig. 4.11).

A brass weir plate was screwed into position (Fig. 4.13) and was based on an original design by the Field Drainage Experimental Unit (Annual Report, 1978), which is capable of measuring flows from 0·01 to 8 l/s These weir plates were later replaced by narrower plates capable of producing a greater head range over a smaller range of discharge of 0·006

Fig. 4.11 THIN-PLATE WEIR AND TANK FOR GAUGING FLOW FROM PITCHED ROOFS



All tank measurements in cms

to 0·8 l/s. All work was carried out by the Chemical Engineering Department workshops of University College London.

The tanks are placed at the bottom of the roof down-pipes in an alley by the side of No. 6, The Park (Figs. 4.12 and 4.14). One tank collects runoff from half the front roof of the terrace Nos. 6-10 The Park and the other collects runoff from half the back roof of the terrace. The remaining 2 down-pipes from No.10 The Park are left to drain to soakaway.

Flow is baffled in a sectioned-off part of the tank as it enters and is allowed into the rest of the tank through small holes at the base, thus the rest of the tank acts as a stilling well and debris is prevented from reaching and blocking the weir. The whole tank is covered by a lid and instrument housing made of 12 mm marine ply. The water level is measured by a 150 mm diameter float attached to a Munro IH89 water level recorder (Fig. 4.15) with a chart speed of 17·7 mm/hr. Figs. 4.16 and 4.17 give the stage-discharge calibrations for each tank.

A volumetric method of measurement was adopted as being more accurate for the lower flows to be expected from part of the flat asphalt-and-chippings garage roof which drains to the highway drains. Flow is monitored using a Casella Tipping Bucket (W5699) adapted by the Chemical Engineering Workshops to carry larger buckets which tip when on average a volume of 70 ml. has filled the bucket (Fig. 4.18). Each tilt produces an electrical pulse recorded on a tilt-counter and by pen on a recorder (W5709) with a chart speed of 11·25 mm/hr. (Fig. 4.20). Fig 4.19 illustrates the equipment and its position at the base of the downpipe from part of the flat garage roof. The instrument is housed in 12 mm marine ply.

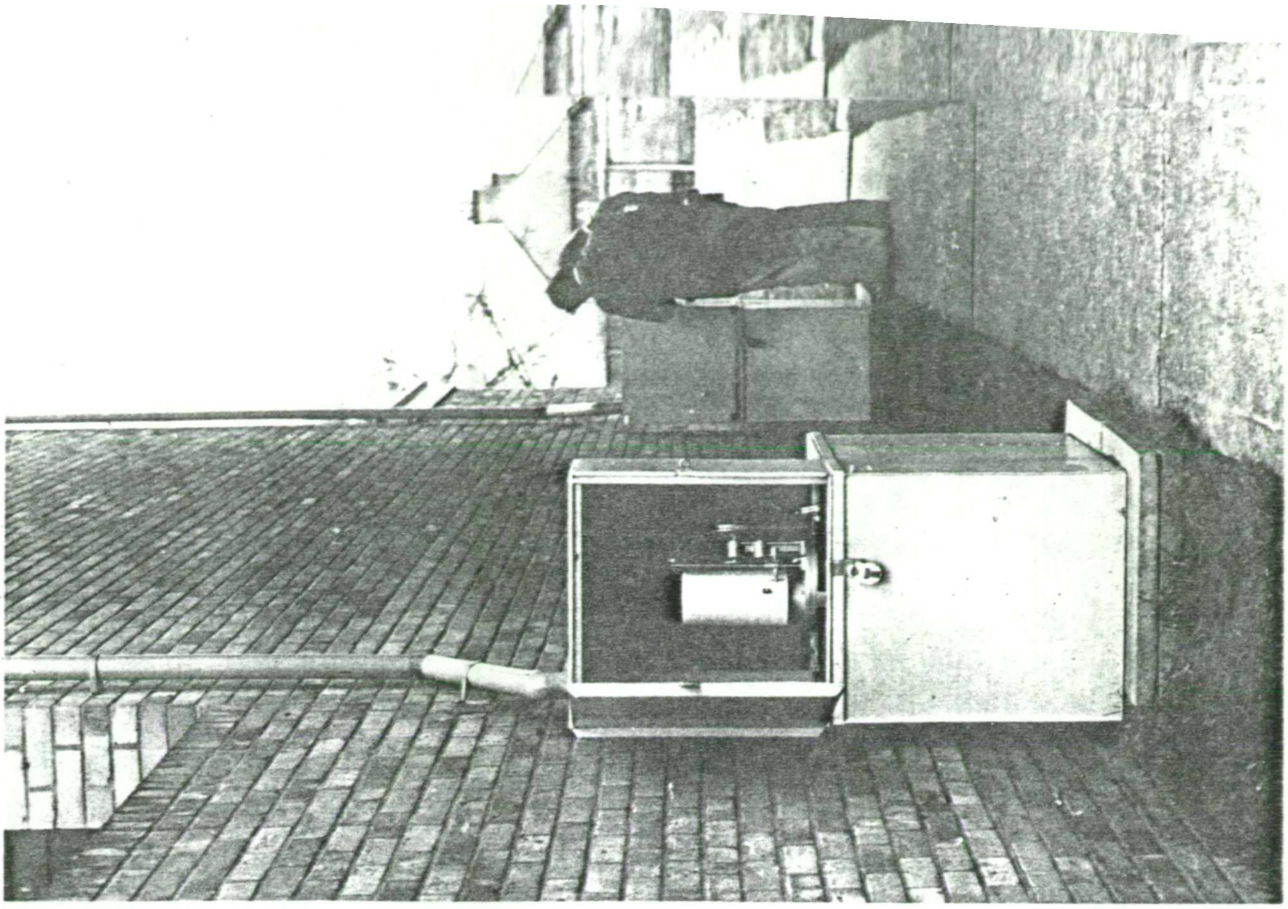


Fig. 4.12 View of the 2 weir tanks in position in alley by side of 6, The Park.

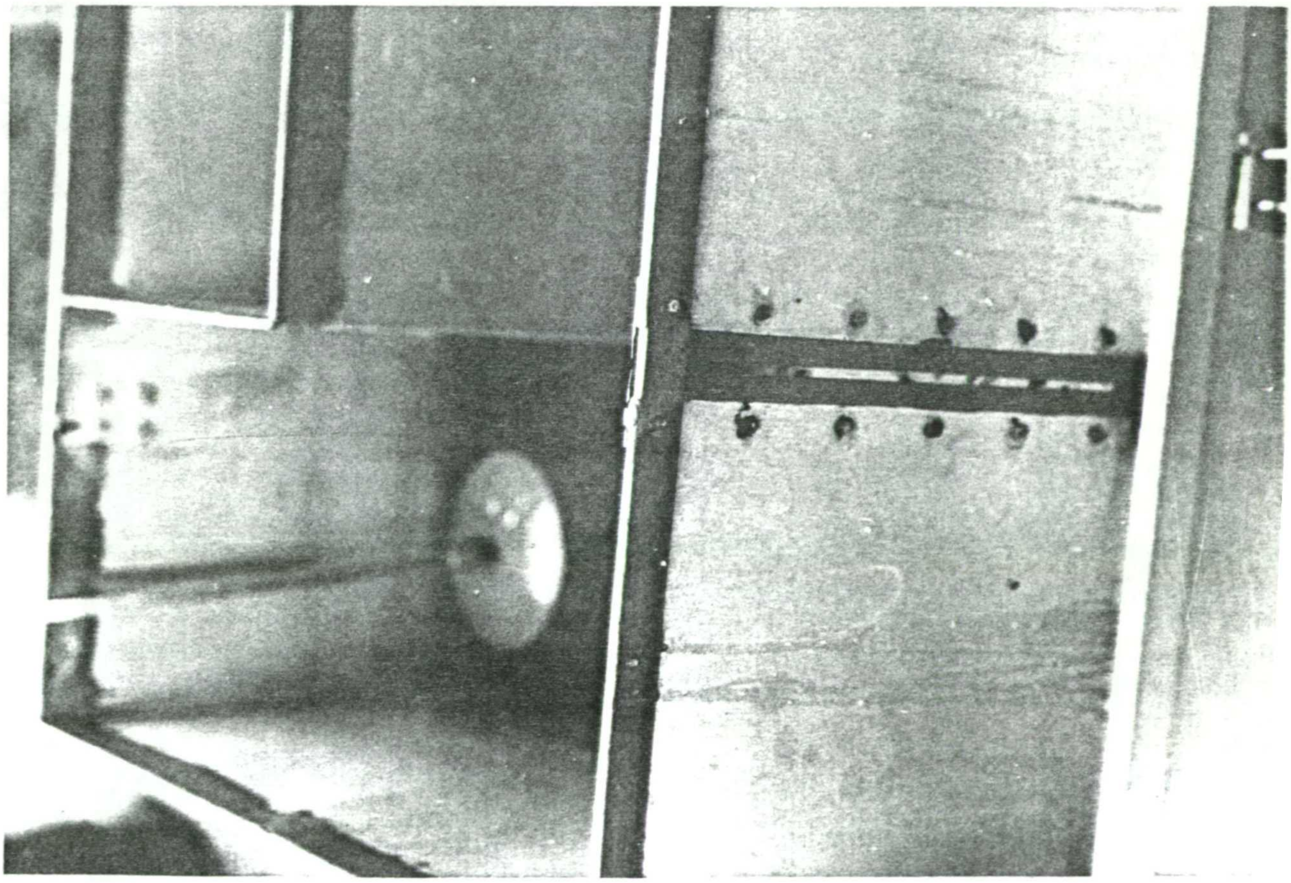


Fig. 4.13 Inside of weir tank showing rear view of weir plate with float in position in stilling-well section of tank.

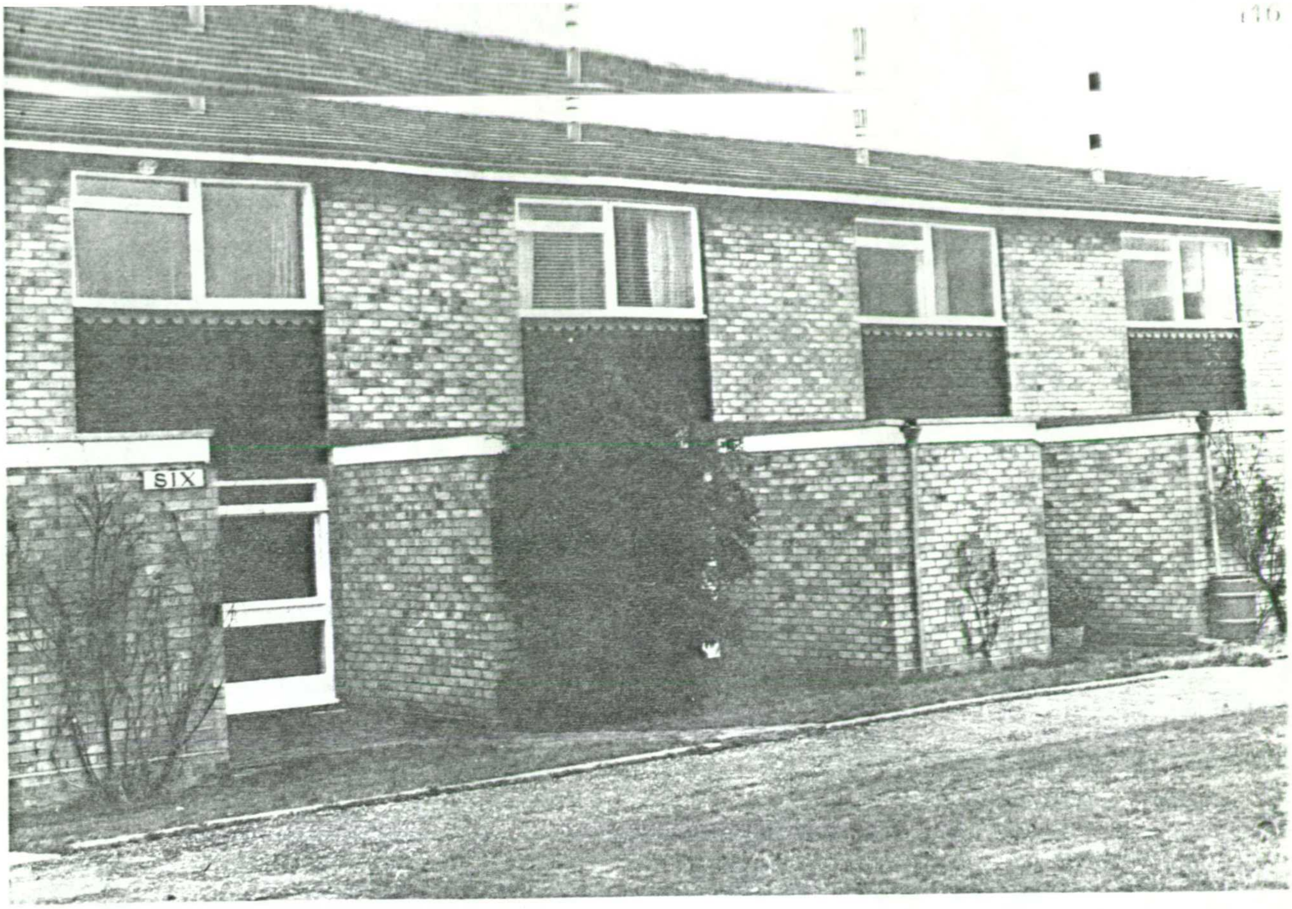


Fig. 4.14 View of houses whose roofs are monitored by the weir tanks.

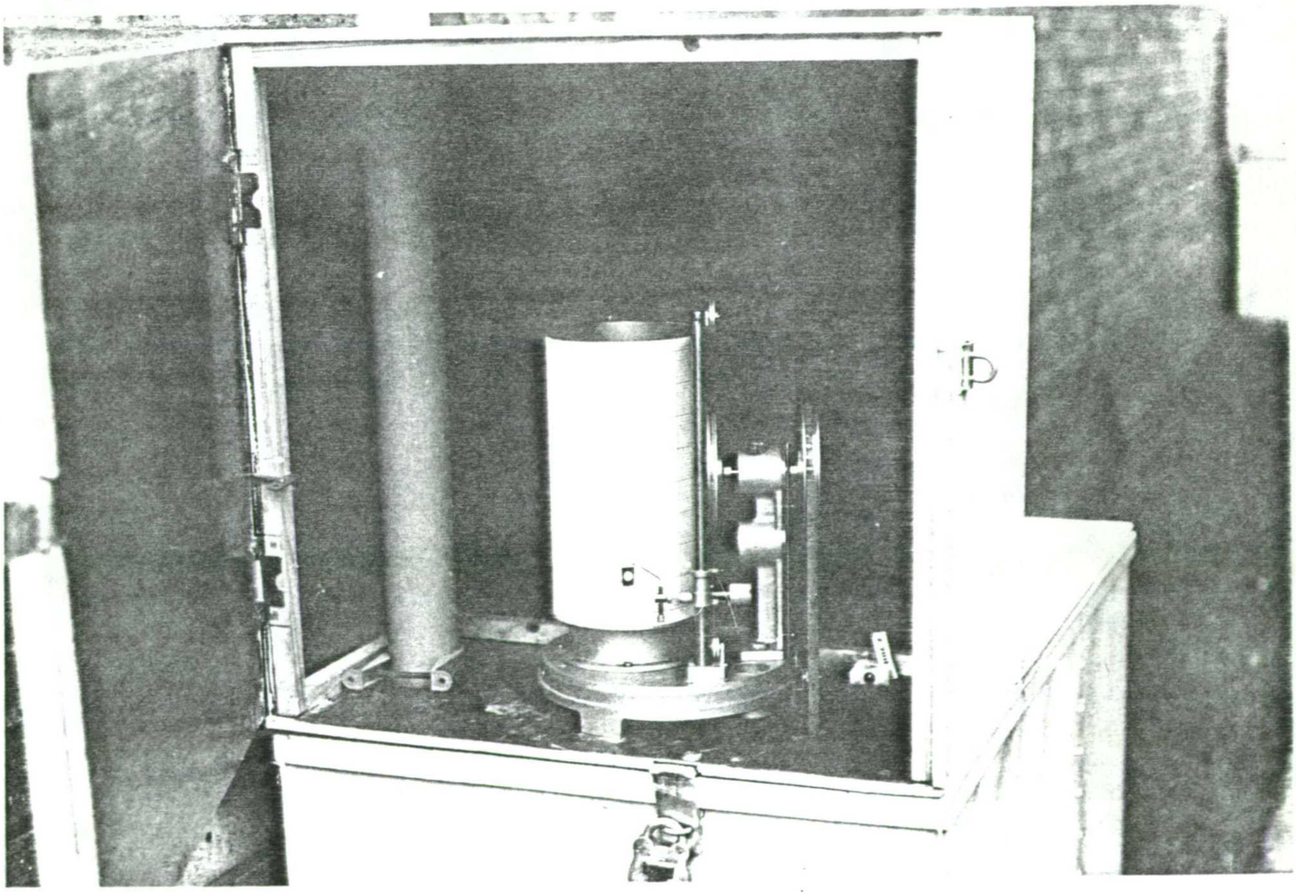


Fig. 4.15 Munro IH89 water level recorder in position.

Fig. 4.16 STAGE - DISCHARGE RELATION GARAGE WEIR TANK

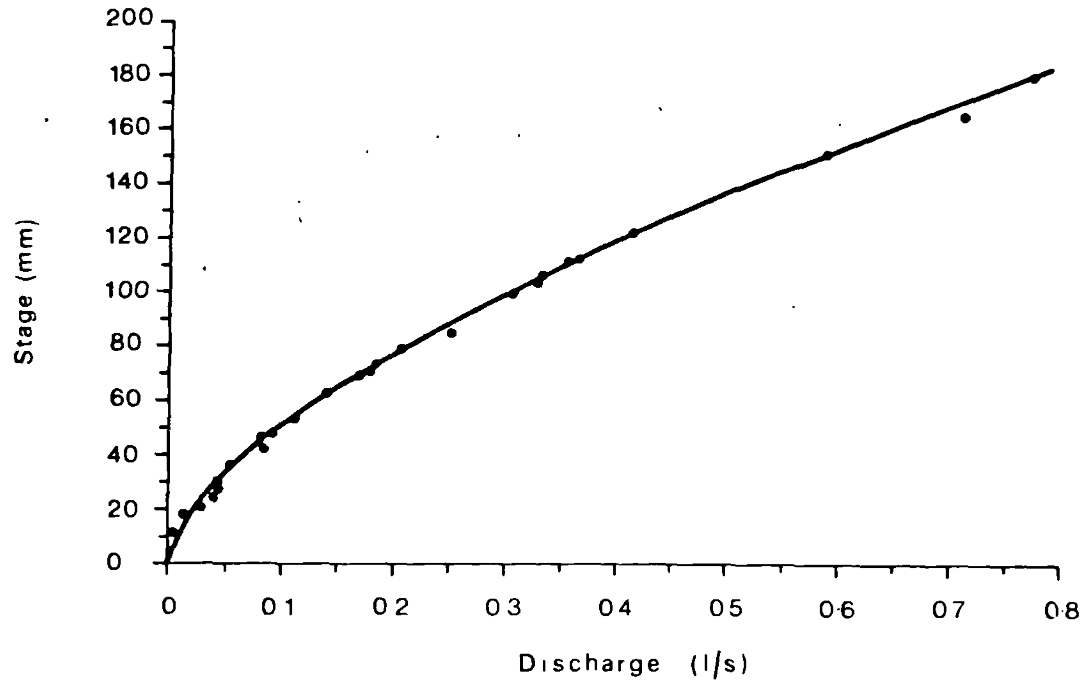
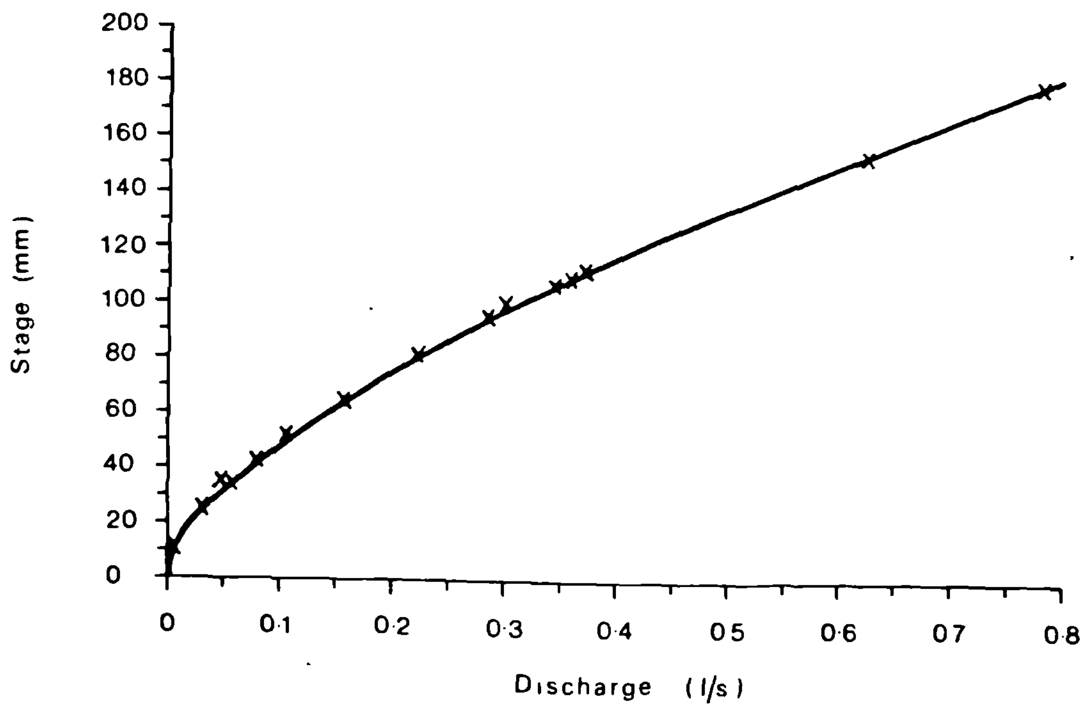


Fig. 4.17 STAGE- DISCHARGE RELATION FRONT WEIR TANK



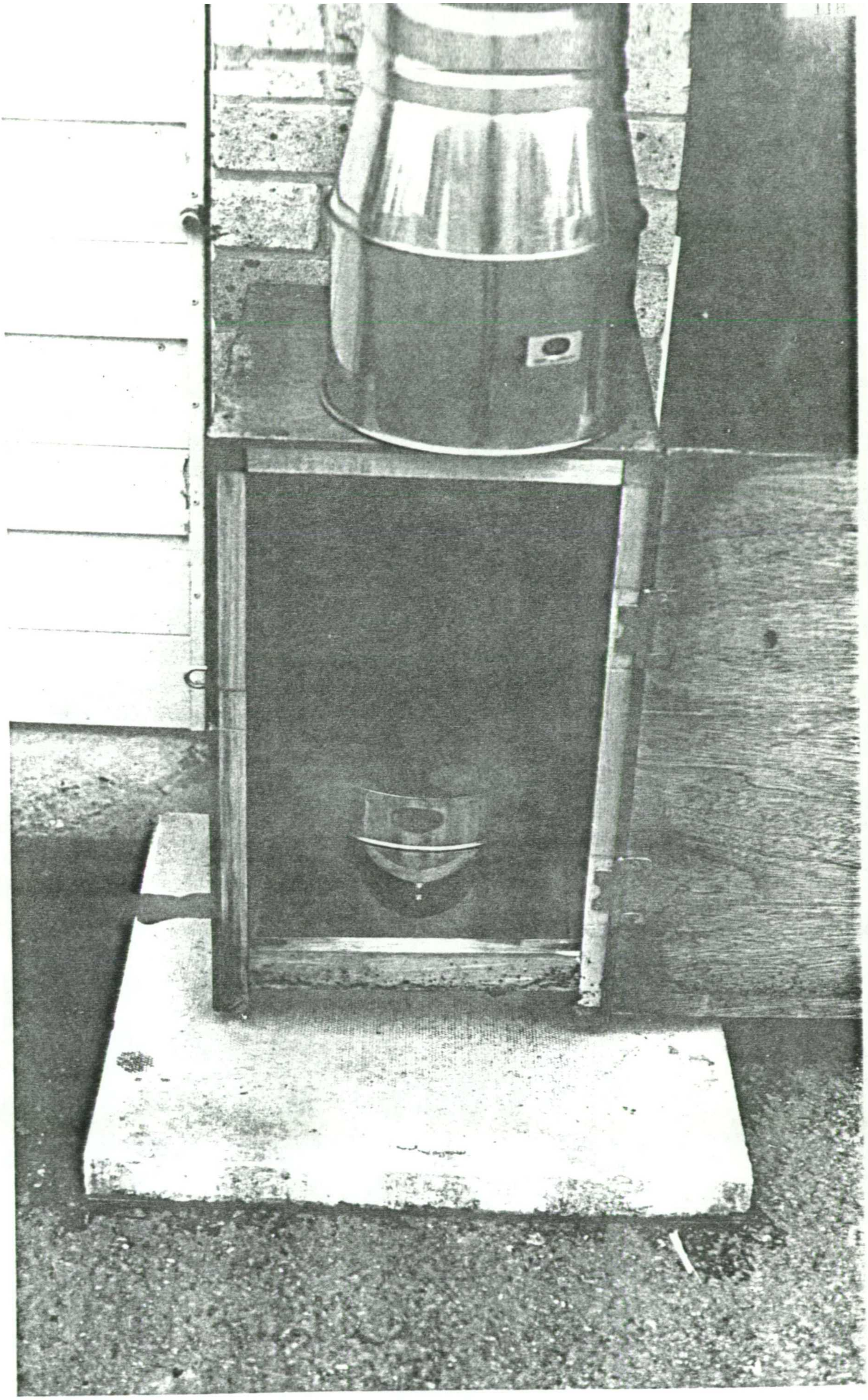


Fig. 4.18 Interior of tipping bucket flowmeter.

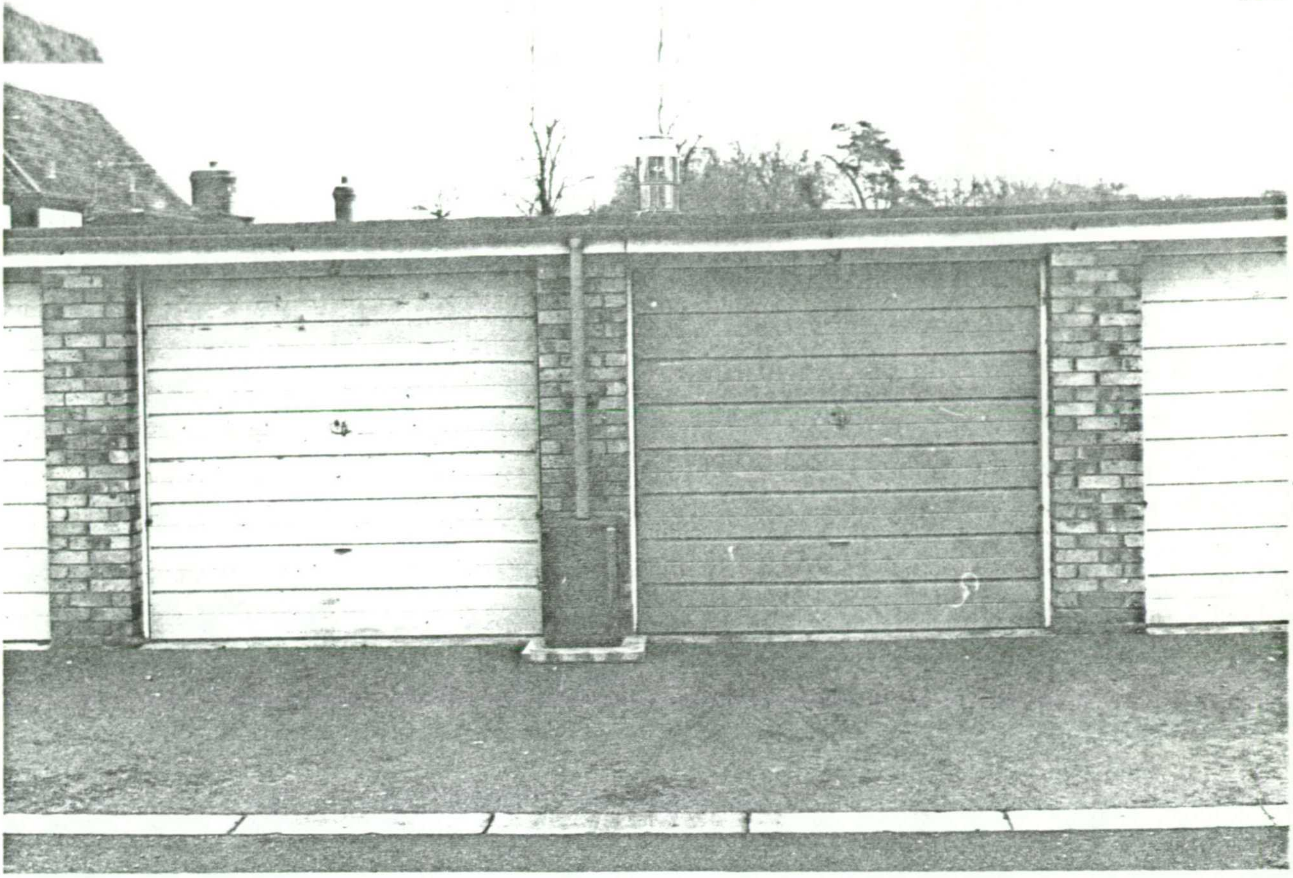


Fig. 4.19 Tipping bucket flowmeter in position on bottom of garage roof downpipe with 1.0 mm tipping bucket raingauge on roof.

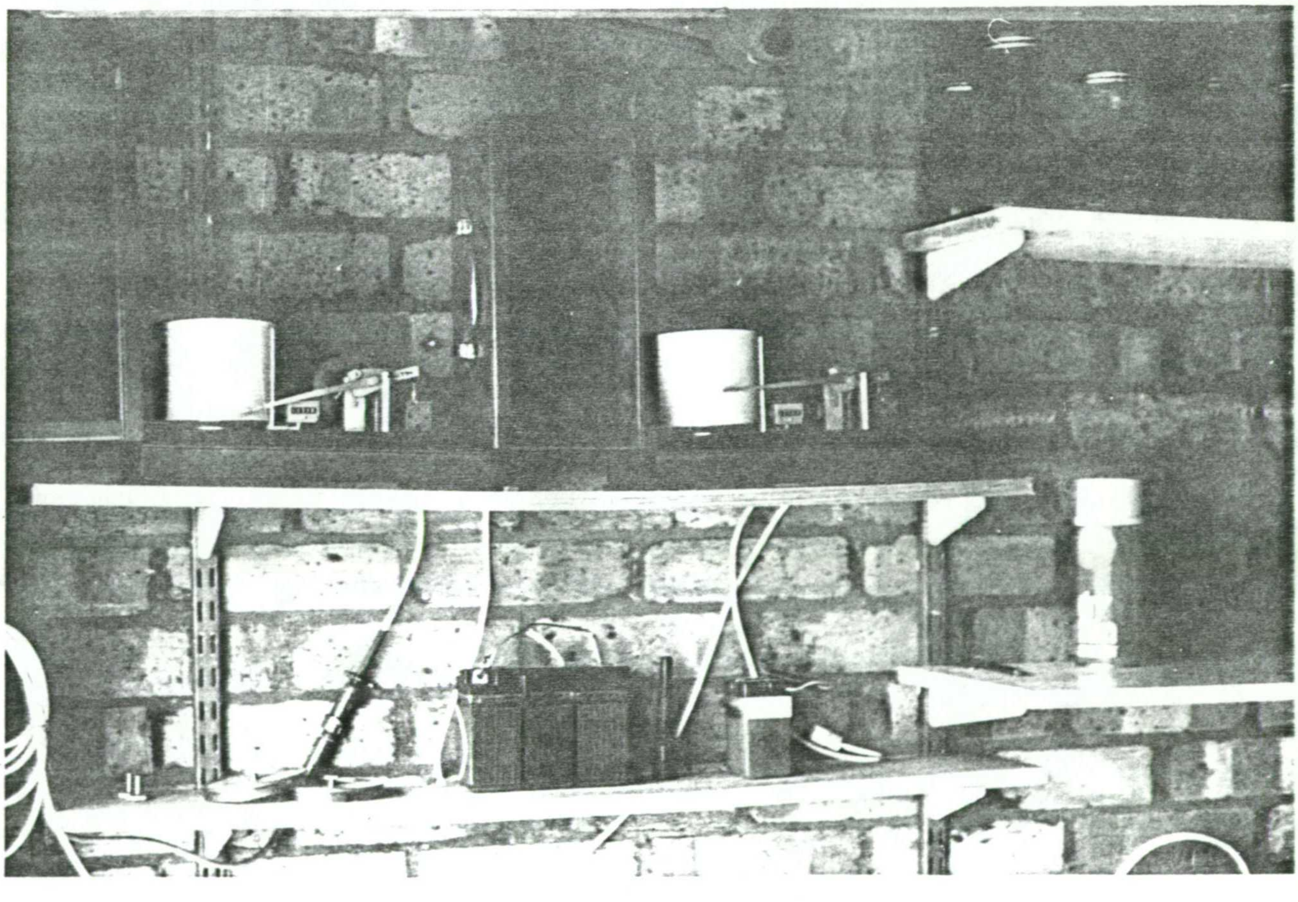


Fig. 4.20 Casella rainfall and garage roof runoff recorders.

OTHER MEASUREMENTS

Storages and transfers involved in an urban hydrological cycle include precipitation, runoff from roads and roofs, paved surface evaporation and infiltration and depression storage. As this catchment contains only monitored segments of nominally impermeable surfaces, measures for soil moisture storage and infiltration, ground water storage and percolation and evapotranspiration are unnecessary. The above sections detail the direct measurements of precipitation and runoff from roads, pitched and flat roofs. The remaining elements can only be measured indirectly.

Depression storage can be calculated by a regression technique developed by Kidd (1978a) and Falk and Niemczynowicz (1978). Rainfall and runoff volumes are plotted and the intercept of the rainfall axis with the regression line is considered to represent depression storage. Alternatively, detailed mapping of the road surface microtopography could give an estimate of potential depression storage backed-up by field measurements during rainfall. Both these approaches assume that depression storage is an initial loss while recent work by Verworn (1978) suggests that it is in fact a time varying loss depending on both the surface and the rainfall intensity. This study adopted the regression technique proposed by Kidd (1978a) to provide estimates of depression storage for the 3 monitored surfaces - roads, pitched and flat roofs - since no method exists to allow its calculation as a variable loss through a storm.

Evaporation from paved surfaces is assumed to be unmeasurable, even indirectly, by any currently available technique. Ward (1967) and Chandler (1976) suggest that a significant proportion of precipitation may be held by and evaporated from the surfaces of buildings and paved surfaces but Van den Berg (1978) believes this is an unmeasurable quantity. Most urban hydrological studies are concerned with modelling storm runoff and can therefore afford to ignore evaporation since it can be assumed that evaporation losses during rainfall are negligible. Evaporation rates may exercise control over the total volumes of runoff since high surface temperatures before rain and high evaporation rates at the end of a storm could cause significant losses from rainfall. Since total volumes of runoff were to be measured, some estimate of evaporation was considered essential. An examination of techniques for estimating water losses from water and vegetated surfaces suggested ways in which these methods could be adapted for urban impermeable surfaces.

In water balance studies, the evaporation term is often left as the residual figure which also contains any errors from measurement of the other elements.

$$P - Q \pm \Delta S = E + \varepsilon$$

P - precipitation
Q - runoff
S - storage
E - evaporation
ε - errors

Since it can safely be assumed that there is no infiltration through roofs, this water balance technique could be used to give a rough measure for evaporation from roof surfaces by elimination of the storage term. This evaporation term would however also contain the quantities of rainfall used for interception, wetting, depression storage and absorption. Unfortunately this technique cannot be applied to road surfaces where some infiltration occurs through any cracks and through the fabric itself. If independant estimates can be made of these additional losses, the residual would be a reasonable approximation of evaporation from urban surfaces. Efforts were therefore made to calculate evaporation using this technique (Chapter 6).

A further extension of this water balance technique would be to set up urban 'lysimeters' or runoff plots where a watertight block of road surface or a section of roof could be irrigated and any runoff collected together with any percolate from the road section. The residual from rainfall minus runoff would then be due to evaporation from the surface. Microhydrological process studies using urban runoff plots were set up and are detailed in Chapter 7.

The energy balance and aerodynamic approaches have been developed by Penman (1948) to allow calculation of open water evaporation and evapotranspiration from vegetated surfaces. It relies on the measurements of temperature, humidity, radiation and wind which influence the rate of evaporation. In theory this technique could be used when measures of the urban surface temperatures are substituted. As this technique requires a large investment in meteorological equipment and was strictly developed for water and vegetated surfaces it was not used.

An independent direct measurement of evaporation can be made through the use of an evaoration pan. Even though this would be a measure of open water surface and not impermeable surface evaporation, it could provide an extra explanation for variations in storm losses, and some idea of of the daily maximum evaporation rate.

Evaporation was measured directly from a Casella US Weather Bureau Class A land pan (W5826) supported 150 mm above the ground on a wooden frame. The tank is made of galvanised iron and is 1·22 m diameter and 254 mm deep. The water level is maintained 50–75 mm below the rim and is measured by means of a hook gauge in a small stilling well.

Additional meteorological information was obtained by measurements made by daily maximum and minimum thermometers which were housed in a Stevenson Screen (Fig 4.6). A 7-day recording thermograph (made by Negretti and Zambra) and a Grant temperature recorder gave readings for dry and wet bulb temperatures. It was hoped that the temperature graphs could be used to distribute the pan evaporation total throughout the day but the instruments proved unreliable, despite recalibration, and so were not used.

Infiltration through paved surfaces is a further unknown quantity that can only be measured indirectly. If independent estimates can be made of rainfall, runoff and evaporation from paved surfaces together with combined losses of depression storage, interception and surface wetting, any residual in the water balance could reasonably be ascribed to infiltration through the asphalt road surface. Since the asphalt roof would have a similar reflection coefficient and therefore similar evaporation rate, the evaporation rate determined for the flat asphalt roof could be applied to the paved surface water balance equation and the remaining balancing volume would be attributable to infiltration. Variations in infiltration rates would be caused by changes in the road surface infiltration capacity as affected by antecedent conditions. Therefore an examination of the effect of antecedent conditions on individual storm runoff volume was made to assess changing infiltration capacity and an urban water balance for the paved surface attempted. The results are given in Chapter 6. A variety of techniques to directly or indirectly measure infiltration are considered in Chapter 7 and the results of experiments on a section of road surface are presented.

CONCLUSIONS

A range of urban surfaces have been monitored for runoff. The choice of instruments was determined by recommendations made by other field workers and financial limitations. The instruments are capable of measuring rainfall, runoff from roads, pitched and flat roofs and open water evaporation. A set of summary statistics to describe each surface's response to rainfall can be developed while further analyses of the data will allow estimates to be made of depression storage, evaporation from impermeable urban surfaces and infiltration through paved surfaces. To ensure confidence in these statistics, calibrations of the newly developed instruments and assessments of the precision of all the instruments used are essential and therefore these tests are detailed in the following chapter.

Chapter 5. PRECISION OF THE INSTRUMENT TESTS AND CALIBRATIONS

Introduction

The usefulness of any of the data collected lies with the precision with which the rainfall and runoff quantities are measured. As with any system, problems arise in measuring the variables since any attempt at measurement automatically interferes with the system. For instance, the value recorded by a raingauge is not necessarily the value which the measured variable would have assumed in the instrument's absence. Provided the inaccuracies are of a simple kind, such as random errors, it is possible that by calibration of the instrument by independent means and an assessment of the precision rather than the accuracy of the instrument, the measures can at least be considered a good estimate of the true or absolute values. Therefore a range of tests of the urban runoff recording instruments and independent checks of the raingauge and evaporation pan measurements were carried out.

PRECISION OF THE ESTIMATE OF PRECIPITATION

The measurements produced by a well-maintained and well-exposed raingauge are sufficiently near to the true rainfall for most purposes. As stated above, the raingauge will interfere with the system so that sources of error produced by the wind through turbulence and eddies; splash in and out of the gauge; evaporation and condensation and gauge inclination will be introduced. Thus all rainfall measures are relative.

The observations made by a single raingauge may not truly represent the catchment mean areal rainfall. Linsley (1973) recommends the use of at least 2 raingauges for urban runoff data collection programmes at this catchment scale. The two (and then 3) raingauges operating at the same site thus provided independent checks on each other. Although the catch of a raingauge set with its orifice horizontal to the ground surface will not be a true reflection of the rain falling on pitched or flat roofs it is a standard measure which means the conclusions about runoff from this catchment can be applied to other similar sites where standard rainfall data is available. The possibilities of actually measuring rainfall on roofs were rejected as impractical and because of the great disruption to the hydrological process itself through trying to measure it.

The Casella tilting-siphon recording raingauge was used to distribute temporally the total rainfall depths caught by the standard Snowdon raingauge. A new copper check gauge was added to the site in early December 1979 and its readings were then taken as the reference. Paired sample Student's t-tests of the discrepancies between raingauge catches were applied to the data collected from October to December. Table 5.1 shows the arithmetic means of the total catches of the Casella recording and standard gauges together with the mean differences and standard error at the 95% confidence level. Although the Casella tilting siphon raingauge differed from the check gauges in that it was taller, had a larger orifice and gave a continuous rainfall record rather than daily totals, this statistical comparison of catches for unlike raingauges has been previously satisfactorily used on the study of the headwater catchments of the Wye and Severn (IOH, Report 33, 1976).

The means given by the 3 gauges during their respective periods of operation did not differ significantly. The agreement between raingauge catches appears to be satisfactory and acceptable.

Table 5.1 Comparison of Raingauge Catches

	Mean Catch (mm)		Difference (mm)
4/10/79 - 19/12/79	Black	Casella	
	5.17	5.13	0.04 ± 0.098
7/12/79 - 19/12/79	Black	Copper	
	14.28	14.04	0.24 ± 0.642
7/12/79 - 31/12/79	Casella	Copper	
	22.35	22.23	0.12 ± 0.662

Despite lack of access to the raingauges on the Brooke-Bond Oxo site at weekends, it was possible to separate the rainfall into individual storms from the chart record so that it was not necessary to use the record from the tipping bucket raingauge sited on the roof of the garage block behind No's. 1-10 The Park. Therefore no checks have been made on the performance of this fourth raingauge.

PRECISION OF THE ESTIMATE OF PAN EVAPORATION

To provide an independent check on the pan evaporation and mean monthly temperature, records from Kew Gardens climatological station were compared with those from Redbourn. The 2 pans were of different type - a British Standard pan being used at Kew and a US Weather Bureau Class A pan at Redbourn - but the figures are not so different that the minor differences between figures cannot be explained by differences in the pans themselves and local conditions (Table 5.2). The lesser evaporation rate at Redbourn for October can be explained by the lower mean monthly temperature. These figures can be used to partly explain any differences in monthly runoff as they are a measure of evaporation rate, albeit from an open water surface. They cannot be used to calculate evaporation from the impermeable surfaces being monitored.

Table 5.2 Comparison of Pan Evaporation And Temperature Figures

	KEW		REDBOURN	
1979	Mean Daily Pan Evaporation (mm)	Mean Monthly Temperature (°C)	Mean Daily Pan Evaporation (mm)	Mean Monthly Temperature (°C)
October	1.1	12.5	0.97	7.4
November	0.4	6.9	0.50	5.1
December	0.6	7.0	0.74	7.8

PRECISION OF THE ESTIMATE OF PAVED SURFACE RUNOFF

Roads and pavements drain to a separate storm sewer system and the runoff is monitored at the outfall by a 25 l/s pre-cast glass fibre flume and bubble-gauge both manufactured by Arkon Instruments. The flume is made to the very precise limits set by BS 3680 part 4c and was installed also according to this standard. It is supplied with a rating curve according to the head and flow specified.

Ideally, this rating curve would be checked over a wide range of flows but local water supplies were insufficient to provide these and any attempt to use a fire hydrant and hose could have resulted in the reduction of water supplies to other users in the water supply area. The possibility of using storm events for dilution gauging was considered but these inevitably occur at inconvenient times. Using a Kent Water Meter installed by the Colne Valley Water Company, a stop-watch and a long length of hose it was possible to check the flume rating for 2 low flow rates - 0.088 l/s and 0.267 l/s. These 2 rated values are plotted on the stage-discharge graph Fig.4.10. It must be assumed that this rating curve supplied by Arkon instruments is accurate for all other flow rates. To remove doubts about the accuracy of the water meter itself, known volumes of water were poured at varying flow rates into an independently calibrated water butt.

The volumes recorded were well within the 2% error that the manufacturers of the meter suggest; (Table 5.3).

Table 5.3 Kent Water Meter Tests

	Kent Meter (litres)	Water Butt (litres)	Flow Rate (l/s)	Volume Error %
	40	40.0	0.320	0.00
	57	57.2	0.345	0.35
	45	45.2	0.320	0.44
	43	43.0	0.190	0.00
	30	30.0	0.188	0.00
	30	30.5	0.12	1.67
	20	20.0	0.075	0.00
Total	265	265.9	Average	0.35

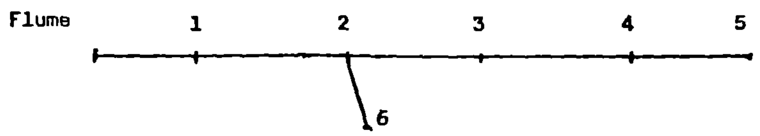
As recorded volumes of runoff were of major interest, any possible pipe leakage was investigated. Using the Kent water meter and hose pipe, water was poured down each storm sewer manhole in turn in the road catchment at ca 0.28 l/s and the level of steady state in the flume recorded and compared with the rated value. The total volume poured in was also compared with the calculated volume from the hydrograph, Table. 5.4.

In all cases the head recorded by the bubble-gauge and flume was close to the rated head at 0.267 l/s. The average flow rate used in these tests was 0.280 $\pm$ 0.016 l/s with a corresponding average head of 9.857 $\pm$ 0.649 mm both at the 95% confidence level. From a total input of 2400 litres for 3 manholes (the topmost, the cul-de-sac and the junction, manhole numbers 5, 6 and 2 respectively) the loss was 149 litres over an initial dry length of pipe of 292 m. This is equivalent to 6% of the input. Volume errors can be accounted for by simple wetting of the pipe or instrument errors caused by factors such as the damped response to

Table 5.4 <u>Arkon Flume And Pipe Leakage Error Tests</u>

Input Point (manhole number)	Inflow Rate (l/s)	Head (mm)	Hydrograph Calculated Volume (litres)	Known Input Volume (litres)	Volume Error (litres)	Volume Error (%)
Flume	0.267	10.9	rating curve tests			
Flume	*0.088	*6.8				
Flume	0.270	9.9	1078.4	400	21.6	1.96
1	0.321	10.1		700		
5	0.251	10.7		950		
6	0.286	9.9	2201.3	675	148.7	6.33
2	0.268	9.4		725		
Gully between 1 & 2	0.298	8.1	610.5	669.2	58.5	8.74
Average (excepting *)	0.280	9.86				

<u>Pipe Layout Diagram And Manhole Numbers</u>



rising or falling water levels, the thickness of the pen trace making chart interpretation difficult and the limited sensitivity of the pressure gauge to slight changes in water level.

In addition a test was made of possible leakage from gully pots and their connection to the sewer system by pouring a known volume of water at $\simeq$ 0.28 l/s into a selected gully pot. The loss was 9% of the input volume (Table 5.4) on a low volume test when percentage errors will be high.

These tests suggested that the gully pots and pipes were sound with few cracks or poor joints and that the losses recorded in the tests could be the result of pipe wetting and instrument errors combining to produce errors in volume recording of fairly low magnitude.

To determine the true area contributing to the storm sewer system, water from a hose was poured over the surface at the suspected catchment boundaries. Those driveways which drain onto adjacent gardens are not included but those garden sheds, garage roofs and driveways that direct runoff onto pathways and thence to gutters are included in the contributing area. In this way the recorded runoff can be accurately "spread" over the area to give an equivalent depth of rainfall which can be compared with recorded rainfall.

The tests carried out are of insufficient number and flow range to enable precise confidence limits to be applied to the data for either flow rate or hydrograph volume but they indicate that the potential errors are of fairly small magnitude (a maximum of 9% on a low volume test) so that the paved surface runoff figures can be taken as a reasonable estimate of the true amount of water entering the drainage system.

PRECISION OF THE ESTIMATE OF PITCHED ROOF RUNOFF

Pitched roof flow was measured on the downpipes from 2 roof halves of a block of townhouses using weir tanks and Munro water level recording equipment before the runoff was returned to soakaway.

Because the weir tanks were of a new design, no standard rating curve was available and calibration was necessary. The usefulness of the record depends on the accuracy of this calibration which was carried out in situ. The rating curves were derived by pouring water from the Kent water meter at various flow rates and dipping the water level in the tank with a portable audio well-dipper when steady state was achieved. As the Kent water meter merely recorded the volume of water flowing, it was necessary to time the output at the period of steady head to obtain flow rates in litres per second. Because of the difficulty of maintaining steady in-flow rates less than 0.006 l/s and because of the severe surface tension effects of water flowing over the weir at the low head of 11 mm, this is considered the lower limit of accurate rating. The tanks are capable of recording flow rates from 0.006 to 0.8 l/s.

As total runoff volumes are the main focus of research attention it was necessary to check the accuracy with which they were recorded. Several known-volume tests were run on each weir tank over a range of flows to discover the confidence limits of the data and to determine if the experimental errors were random or systematic.

The possible sources of error in these tests were considered to be:-

1. Incorrect level recording which would be of more importance at peak flows as the tail-end of the hydrograph is of little volumetric significance.

2. Incorrect calibration which would be checked by the volume tests.
3. The damped response of the level recorder because of tank storage but this will apply equally on the rising and falling limbs of the hydrograph.
4. Inaccuracy of the time-interval selected for hydrograph analysis.
5. Instrument errors caused by clock inaccuracy and an occasionally sticky pen column, but these can be kept to a minimum by regular maintenance.

The first 2 sources of error would be checked by the volume tests themselves as any large discrepancy in volumes would obviously be caused by these 2 factors. Selection of a finer time-step than the 5 minutes used would only minimally improve the analysis and would greatly add to the work load.

The Kent water meter was again used to provide the known inflow rates and total volume poured into the tanks. After each test at a set flow rate the weir tank was allowed to drain to the lower limit of the rating when the remaining tank volume was siphoned out until the water level reached the bottom of the weir. The average tank volume siphoned out, with the float in position, was 4.00 $\pm$ 0.34 litres for the garage weir tank and 4.01 $\pm$ 0.16 litres for the front weir tank, both at the 95% confidence level.

The water level record from each test was converted to discharge and the area under the hydrograph calculated to give the volume.

This was compared with the known input volume and any absolute error in volume, whether in excess or falling short of the input, plotted against the input (Fig. 5.1 A). This error was also plotted against the peak flow rate (Fig. 5.1 B). The absolute error in litres was converted to a percentage error of the known input volume and then plotted against peak flow rate (Fig.5.1 C). These % volume errors were then used to calculate the accuracy and confidence limits of the data by means of a standard error test about the data mean.

The arithmetic mean of 12 runs on the front weir tank was 100.576% ± 4.370% of the input volume and from 11 runs on the garage weir tank the mean was 102.518% ± 4.094% of the input volume both at the 95% confidence level. This means that when interpreting the runoff data, it must be borne in mind that both tanks have a slight tendency to overestimate the runoff volume and only % runoff figures below 91.836% and above 109.316% for the front weir tank and below 94.330% and above 110.706% for the garage weir tank are significant.

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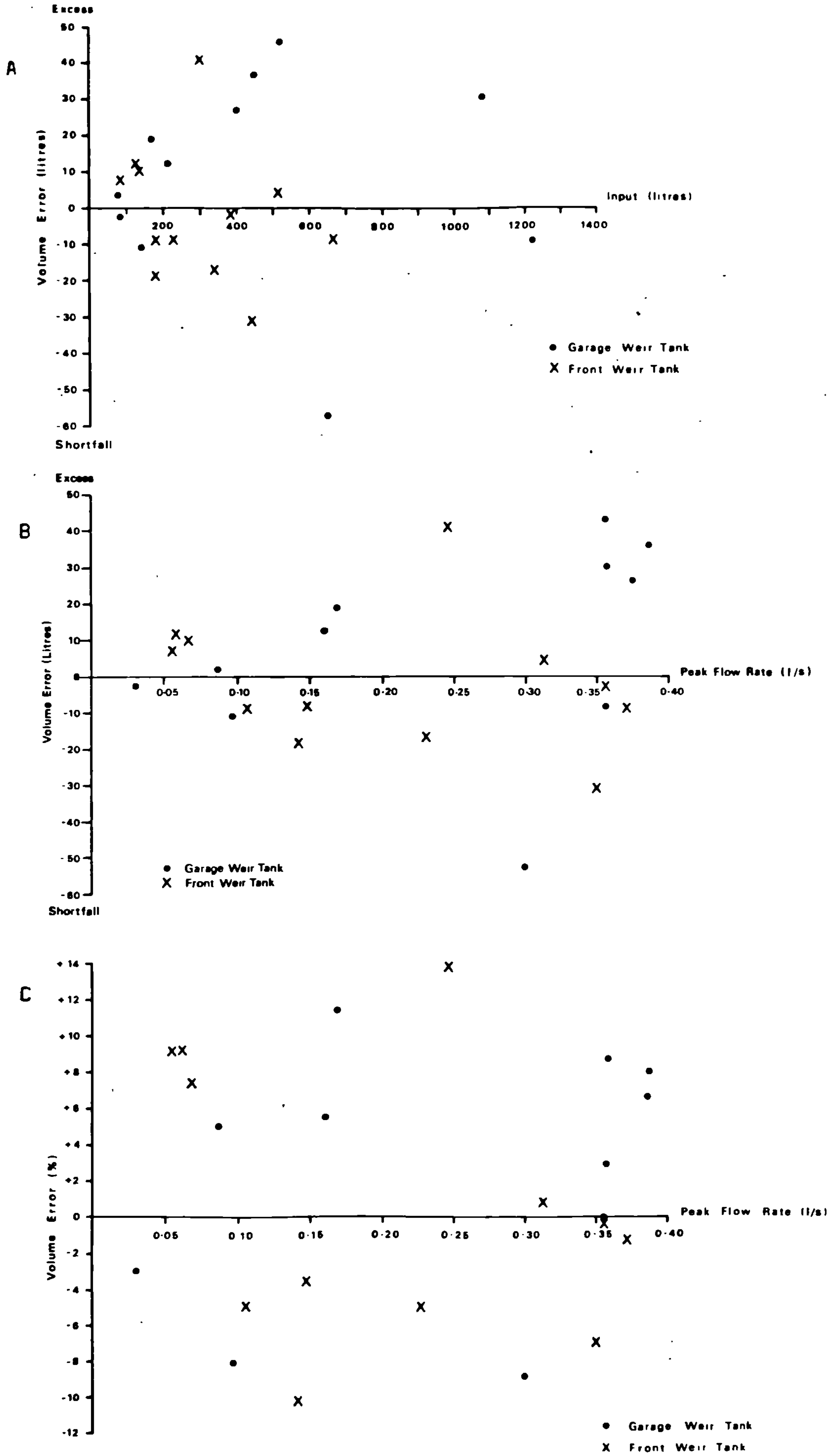
					<u>Mean</u>		
<u>91.836</u> + 4.370	=	96.206	=		100.576 − 4.370	}	Front Weir
<u>109.316</u> − 4.370	=	104.946	=		100.576 + 4.370	}	Tank
<u>94.330</u> + 4.094	=	98.424	=		102.518 − 4.094	}	Garage Weir
<u>110.706</u> − 4.094	=	106.612	=		102.518 + 4.094	}	Tank

If any systematic error in volume recording existed, such as might be caused by an incorrect zero setting, this would be shown by a constant positive or negative bias on Fig 5.1 A. For both weir tanks the absolute errors are scattered fairly evenly on both sides of the axis of no error which confirms that there is no systematic bias.

Fig. 5.1 WEIR TANK ERROR TESTS



Constant over-or under-estimate of volume at set flow rates would indicate incorrect calibration. On Fig 5.1 B there is a slight tendency for flow rates between 0.05 and 0.10 l/s to be over-recorded by the front weir tank and flow rates between 0.10 and 0.20 l/s to be under-recorded. As these flow rates are low they are unlikely to cause serious error in volume calculations. Of more interest is the apparent tendency for overestimation of volumes at flow rates of $\simeq$ 0.37 l/s by the garage weir tank. Rechecking of the calibration confirmed its accuracy however and no alterations to the rating curve of either weir tank were found to be necessary when these additional points were applied. As the input volume is often larger for higher peak flow rates there is a slight increase in absolute errors and therefore an increase in the scatter of points with increasing flow rates. In general the errors are fairly evenly distributed about the X-axis of no error which suggests that the calibration is satisfactory.

On Fig 5.1 C the % volume errors are scattered nicely on both sides of the X-axis. This axis is a true reflection of the mean of the front weir tank errors but as the mean of the garage weir tank errors was +2%, rather more of the points are to be found on the positive error side. The maximum % errors are + 13.7% and - 10.4% both recorded by the front weir tank and this slightly greater scatter is reflected in the standard error calculation above, although the garage weir tank recorded the largest absolute errors. There is no tendency for bigger % errors with the smaller inputs caused by low peak flow rates which might have been expected and indeed the size of error does not seem to depend on peak flow rate in any way.

These tests therefore suggest that any volume errors are randomly caused.

Confidence limits on the data have been set for each weir tank so that significant and non-significant events can be identified.

PRECISION OF THE ESTIMATE OF FLAT-ROOF RUNOFF

Runoff from the flat asphalt-and-chippings garage roof was monitored on the downpipe using a tipping bucket flowmeter.

Tipping bucket flowmeters require calibration as when they are operating, flow will continue to pour into the discharging bucket after tipping has commenced. This means that the volume metered in a single tip is a function of inflow rate, and errors will increase with increasing inflow rate. Another major problem in small gauges is to ensure proper emptying owing to surface tension effects, but the impact shock on the bucket stops aids emptying.

A static calibration for each side of the tipping bucket was carried out in the laboratory. Water from a burette was dripped into one bucket half until it just started to tip. This was repeated for the other bucket half and the whole procedure continued until sufficient values near an identifiable mean were achieved. The average bucket volume for both sides was calculated as 0.07365 ± 0.00287 litres (at the 95% confidence level).

As however a non-linear relationship exists between flow rate and tipping rate because of the variable quantity of water lost according to the inflow rate (Calder and Kidd 1978), a further dynamic calibration was attempted with the flow meter in situ.

Inflow from a calibrated 100 litre water-butt placed on the garage roof was poured into the tipping bucket flowmeter. The volume poured in for each run was timed to give the average inflow rate and the total number of bucket tips recorded to give an outflow volume. The number of tips was also timed by stop watch to give a tipping rate. The chart was analysed to give the respective recorded flow rates as when dealing with rain events, only chart flow rates would be available. From these measurements it was possible to derive 3 alternative calibrations.

The calibration method suggested by Calder and Kidd (1978) was adopted. Theory indicates that the inflow rate (Q) can be related to the bucket volume (V) according to the time between bucket tips (T).

$$Q = \frac{V}{T}$$

This however is a static calibration. When the tipping time (t) taken for the bucket to move from rest until the central bucket division is directly beneath the flow inlet is also included as a constant, then a first-order dynamic calibration equation results:

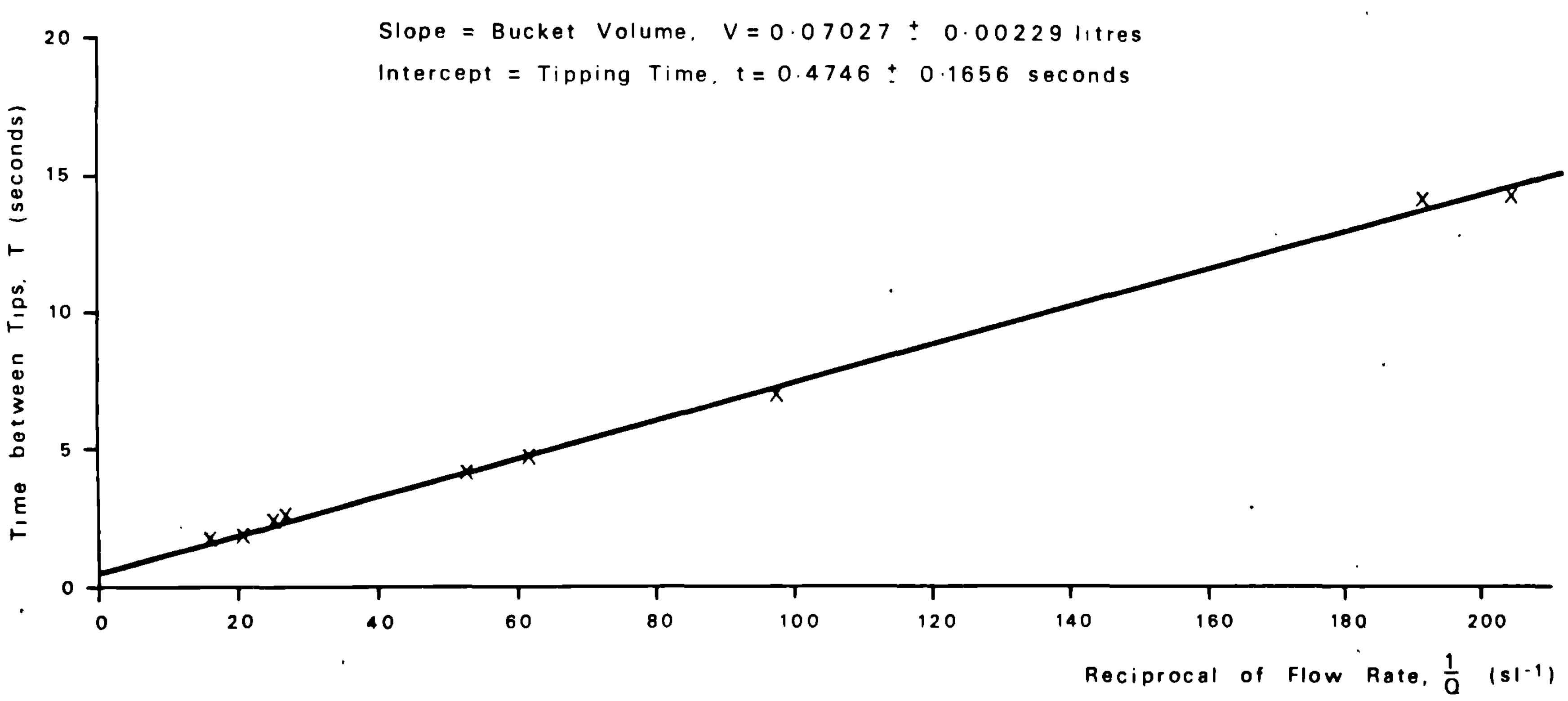
$$Q = \frac{V}{(T-t)}$$

The dynamic calibration only becomes significant at high flow rates as at low flow rates, when T is large compared with t, the above equation reduces again to the static flow rate equation.

From the experimental data, a plot was made of the reciprocal of the inflow rate (Q) against the time between successive tips (T). From Fig 5.2, the volume of the bucket, (V) is given by the slope of the line and the tipping time, (t) by the intercept on the T axis. This gave a

Fig. 5.2

CALIBRATION OF 0·07 LITRE TIPPING BUCKET FLOWMETER



bucket volume value of 0.07027 ± 0.00229 l and tipping time of 0.4747 ± 0.1655 seconds both at the 95% confidence level.

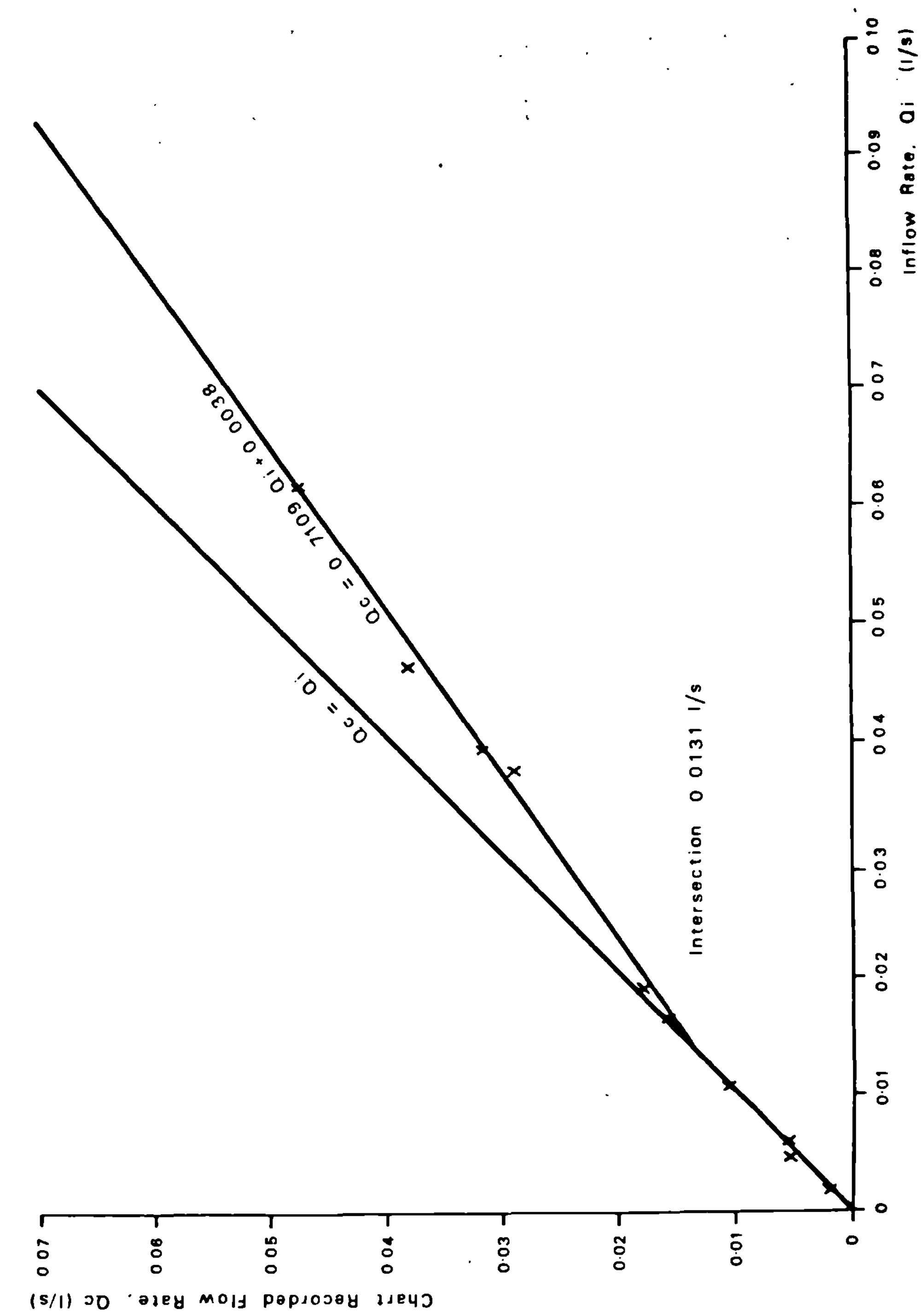
As this bucket volume was derived dynamically and in situ, it was used in chart analysis of total runoff volume in preference to the static calibration volume of 0.07365 litres.

A second calibration involved the plotting of the inflow rate, (Q_i) against chart recorded flow rate, (Q_c), (Fig 5.3). It is clear that at low flow rates the record was quite accurate, ie $Q_c = Q_i$. By calculating the regression line through the remaining points, it was possible to determine the point at which the tipping bucket became inaccurate - 0.0131 l/s (the intersection of the 2 lines), and to obtain a simple read-off of the true inflow rate according to the equation $Q_c = 0.7109\ Q_i + 0.0038$.

This method of flow rate calibration was considered more suitable for the modified Casella Tipping Bucket Raingauge and tip counter and chart recorder being used than that proposed by Calder and Kidd (1978) detailed above originally used for the IOH 1.2 l flowmeter and the Rimco 0.1 mm recording raingauge which both record pulses onto magnetic tape.

The final calibration method used was that suggested by Edwards, Jackon and Fleming (1974) which allows corrections to be made to the tipped static volume. By plotting the percentage error by which the tipped volume is underestimated against inflow rate (Fig. 5.4) it is then simple to adjust tipped volumes accordingly. The volume error becomes unacceptably large (> 15%) at flow rates of 0.033 l/s and more.

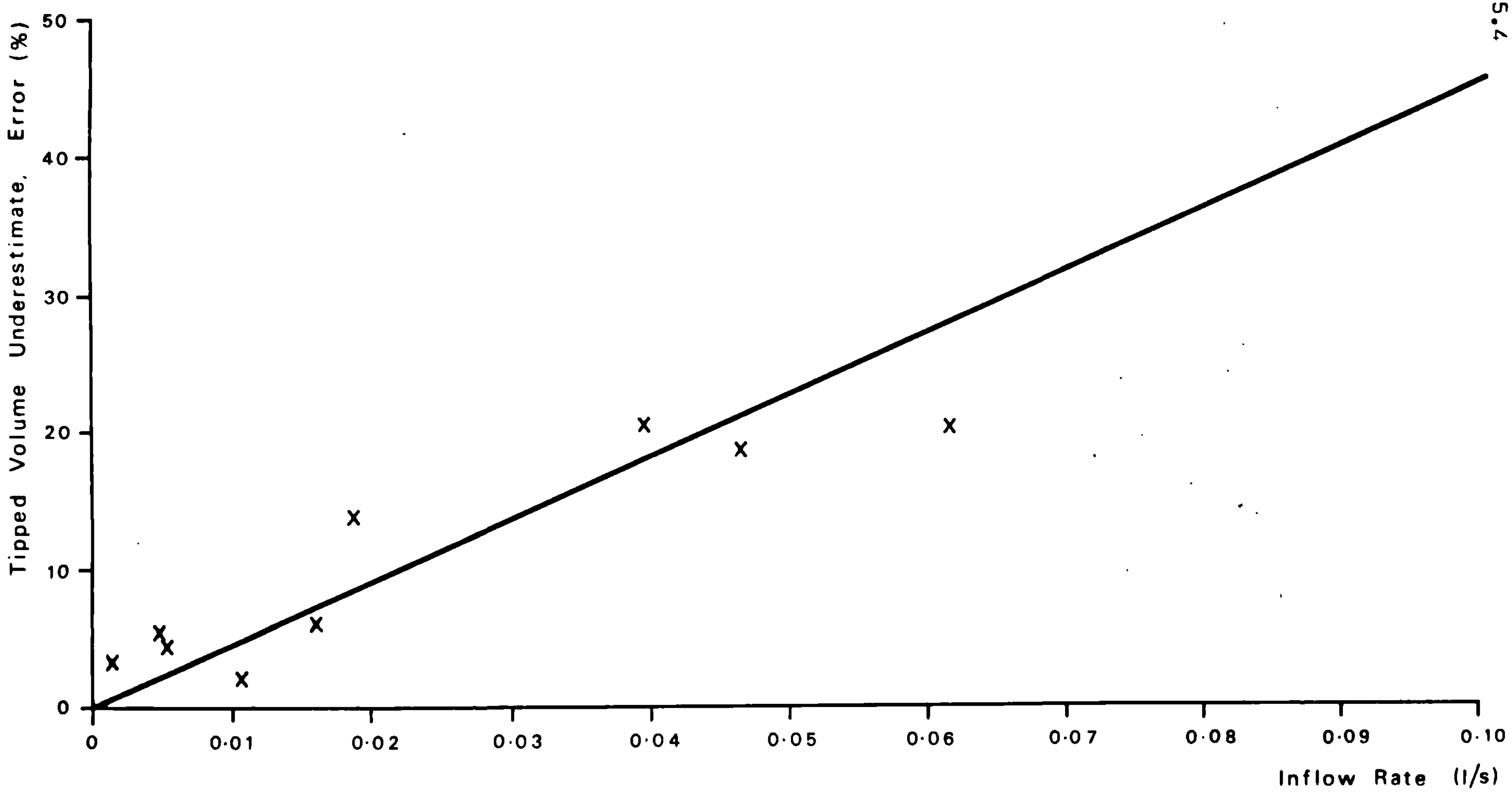
Fig. 5.3



CALIBRATION OF 0·07 LITRE TIPPING BUCKET FLOWMETER

Fig. 5.4

CALIBRATION OF 0·07 LITRE TIPPING BUCKET FLOWMETER



The flow rates recorded by this tipping bucket may be beyond its recommended capacity and accuracy - the WMO (1961) recommend 30 bucket volumes per minute for raingauges as a maximum rate which would correspond to a flow rate of 0.035 l/s on this equipment. However, when installation was considered no flow rates from flat roofs were known. With the addition of some collecting device beneath the tipping bucket, total storm volumes could be accurately recorded and the chart flow rate adjusted according to the dynamic calibration detailed above.

With the calibrations of the tipping bucket flowmeter detailed above it is possible to adjust the recorded flow rate to the true inflow rate and consequently apply corrections to the under-estimate of the tipped volume caused by high tipping rates.

CONCLUSIONS

By examining the sources and types of instrument error it is possible to place reasonable confidence in the recorded variables. The raingauge catch would appear to be a good estimate of the true rainfall with only minor differences of low statistical significance being recorded between the average catch of the 3 gauges. Difficulties in achieving the necessary test range of flow rates meant that the rating curve of the flume and bubble gauge could not be fully checked. The limited number of hydrograph volume tests on this equipment also mean that no precise limits can be set on the variability of the paved surface runoff data. They do however indicate that the errors are likely to be low in absolute and percentage terms (eg 149 litres or 6%) so that pipe leakage can be considered negligible and the runoff figures of generally adequate accuracy. Pitched roof runoff is reliably measured with any possible recording error being of a random nature and within narrow confidence limits. Both weir tank recorders have a tendency to slightly over-estimate the runoff volume so that the average volume recorded by the front weir tank is 100.576 $\pm$ 4.370% and 102.518 $\pm$ 4.094% by the garage weir tank. Possible under-recording of flat roof runoff by the tipping bucket flowmeter is compensated for by detailed calibrations and estimates of the error at high flow rates (eg $>$ 15% error at 0.033 l/s). These calibrations and estimates of the instrument error for all recording devices mean that the data and the results of the data analysis are reliable within known limits. Any unexpected results or anomalies will be the result of the variability of the process and not of the instrument.

Chapter 6

DATA ANALYSIS AND RESULTS FROM THE INSTRUMENTED URBAN CATCHMENT

INTRODUCTION

In response to calls for the acquisition of field data for use in urban hydrology research to further model building, water resource management and theory advancement and in view of the limited number of efforts to establish a water balance for an urban area, data from 88 storms occurring between October and December 1979 were analysed to give a range of information about runoff and losses from urban surfaces. The runoff hydrographs from the paved surfaces, pitched and flat roofs were analysed to give storm runoff volume, peak flow rate and storm duration. The storms were categorised to determine the effect of antecedent conditions on storm runoff volume and to calculate depression storage. Water balances for each surface allowed the determination of other losses such as evaporation from impermeable surfaces and infiltration through the road. The results of these analyses are detailed in this chapter.

COLLECTION AND PREPARATION OF THE DATA

Rainfall and runoff records were collected from the Casella tilting-siphon raingauge (daily), Arkon level recorder (weekly), Munro level recorders (daily) and Casella tilting-bucket flowmeter (daily).

A separate rain event was defined as having a minimum of one hour between events. Rain events were further categorised according to the amount of time between storms. Antecedent conditions were considered dry if more than 24 hours without rain had preceded the storm (XD), and wet (W) if less than 6 hours separated the storms leaving several events in the in-between category (D).

OCTOBER RUNOFF RESULTS

Table 6.1 — OCTOBER 1979

STORM RUNOFF VOLUMES

No.	P mm	PS l	PS mm	FAR l	FAR mm	FAR Error %	PR_f l	PR_f mm	PR_g l	PR_g mm
1	0.2	installed		no runoff		-	installed 4/10/79			
2	2.0	12/10/79		70.37	1.44	D/R	36.7	0.67	26.2	0.48
3	0.3			no runoff		-	no runoff		no runoff	
4	0.4			no runoff		-	no runoff		no runoff	
5	13.2			D/R		D/R	469.6	8.61	420.6	7.66
6	6.8			(165.55	3.40)	30.5	229.3	4.20	179.5	3.27
7	0.35			(	)		no runoff		no runoff	
8	14.75			400.33	8.22	27.7	348.9	6.40	328.6	5.09
9	6.2	D/R		D/R		32.8	228.4	4.19	247.3	4.51
10	1.1	no runoff		(44.97	0.92)	4.3	42.4	0.78	58.3	1.06
11	0.2	no runoff		(	)		unknown		unknown	
12	1.0	190.8	0.06	24.03	0.49	3.7	26.8	0.49	46.3	0.84
13	11.0	6217.4	1.79	(	)		505.1	9.26	D/R	
14	14.0	D/R		(1028.68	21.13)	28.2	558.7	10.25	D/R	
15	1.3	D/R		(	)		D/R		D/R	
16	0.1	no runoff		no runoff			no runoff		no runoff	
17	0.1	no runoff		no runoff			no runoff		no runoff	
18	0.3	no runoff		(185.72	3.81)	11.6	unknown		unknown	
19	4.0	855.5	0.25	(	)		204.7	3.75	146.4	2.67
Σ	77.3	7263.7	2.10	1919.65	39.41		2650.3	48.60	1453.2	26.48

PERCENTAGE RUNOFF / DURATION OF EVENT

No.	PS %	FAR %	PR_f %	PR_g %	P hrs m	PS hrs m	FAR hrs m	PR_f hrs m	PR_g hrs m
1		-			8.30		-		
2		72.3	33.7	23.9	1.14		D/R	2.20	1.40
3		-	-	-	4.30		D/R	-	-
4		-	-	-	0.40		D/R	-	-
5		D/R	65.3	58.1	10.12		D/R	10.30	7.30
6		50.0	61.9	48.1	2.43		(34.35)	6.55	2.50
7		-	-	-	5.15		( )	-	-
8		55.7	43.4	40.6	10.14		( )	2.20	5.35
9	D/R	D/R	76.2	81.9	9.56	D/R	D/R	16.15	14.20
10	-	(71.0)	70.7	96.6	2.19	-	(12.20)	16.30	14.33
11	-	( )	-	-	0.10	-	( )	-	-
12	3.5	49.3	49.2	84.4	1.00	4.35	12.00	7.35	7.40
13	16.3	( )	84.2	D/R	11.00	8.30	( )	12.25	11.35
14	D/R	(80.3)	73.2	D/R	9.45	D/R	(53.40)	13.30	10.50
15	D/R	( )	D/R	D/R	11.15	D/R	( )	D/R	D/R
16	-	-	-	-	1.00	-	-	-	-
17	-	-	-	-	0.01	-	-	-	-
18	-	(88.7)	-	-	4.15	-	(20.25)	7.35	7.35
19	6.2	( )	93.9	66.7	6.05	22.40	( )	19.35	11.45

PEAK INTENSITY / FLOW RATE

No.	P 5 min. average mm/hr	PS l/s	FAR 15 min. average l/s	FAR adjusted l/s	PR_f l/s	PR_g l/s	AC	STORM DATE October	No.
1	-		-	-	-	-		4	1
2	3.6		D/R	D/R	0.017	0.021	XD	6	2
3	0.2		-	-	-	-	W	7	3
4	11.4		-	-	-	-	D	7	4
5	15.6		D/R	D/R	0.243	0.275	XD	8–9	5
6	8.8		0.0515	0.0671	0.093	0.085	D	10	6
7	-		0.0009	0.0009	-	-	W	10	7
8	20.8		0.0471	0.0609	0.395	0.285	W	11	8
9	54.5	D/R	0.0550	0.0720	0.410	0.530	XD	13	9
10	0.8	-	0.0095	0.0095	0.009	0.008	XD	17	10
11	0.6	-	0.0008	0.0008	-	-	W	18	11
12	1.0	0.09	0.0088	0.0088	0.008	0.017	XD	24	12
13	6.4	1.64	0.0450	0.0580	0.081	D/R	D	25	13
14	12.7	D/R	0.0479	0.0621	0.100	D/R	W	25–26	14
15	0.4	D/R	0.0016	0.0016	D/R	D/R	W	26	15
16	-	-	-	-	-	-	XD	28	16
17	-	-	-	-	-	-	XD	30	17
18	0.4	-	0.0005	0.0005	0.006*	0.006*	W	30	18
19	6.3	0.25	0.0219	0.0256	0.047	0.043	W	30–31	19

P Precipitation
PS Paved Surfaces
FAR Flat Asphalt Roof
PR_f Pitched Roof, front weir tank
PR_g Pitched Roof, garage weir tank

Error % Tipping bucket maximum volume % underestimate
D/R Defective Record
( ) ( ) Inseparable runoff events
/ / Frost

AC Antecedent Conditions
XD Minimum of 24 hrs previously dry
D Between 6 and 24 hrs previously dry
W Rain occurring in preceding 6 hrs
0.006* Lowest rated flow rate from weir tanks

NOVEMBER RUNOFF RESULTS

Table 6.2 — STORM RUNOFF VOLUMES / PERCENTAGE RUNOFF / DURATION OF EVENT / PEAK INTENSITY / FLOW RATE — NOVEMBER 19

STORM RUNOFF VOLUMES

No.	P	PS		FAR			PR$_f$		PR$_g$	
	mm	l	mm	l	mm	Error %	l	mm	l	mm
20	0.1	D/R		no runoff		-	no runoff		no runoff	
21	5.3	D/R		244.19	5.02	8.4	260.1	4.77	[illegible]24.9	4.09
22	2.9	(135[illegible].9	0.39)	(131.62	2.70)	12.5	(114.8	2.11)	(115.4	2.10)
23	0.2									
24	1.3	608.0	1.23	57.5[illegible]	1.18	8.6	43.9	0.81	42.7	0.78
25	2.8	10[illegible]7.9	0.31	(112.99	2.32)	17.5	141.7	2.60	109.5	2.00
26	0.05	no runoff					no runoff		no runoff	
27	0.1	no runoff					no runoff		no runoff	
28	0.05	D/R		(109.97	2.26)	6.4	(102.1	1.87)	(93.0	1.70)
29	2.6	D/R								
30	6.3	D/R		264.36	5.43	15.4	272.0	4.97	294.1	5.36
31	0.4	no runoff		no runoff			no runoff		no runoff	
32	0.1	no runoff		no runoff			no runoff		no runoff	
33	3.6	1551.8	0.45	130.35	2.58	17.0	130.9	2.40	151.2	2.75
34	1.2	177.4	0.0[illegible]	56.08	1.15	5.3	42.4	0.76	[illegible]2.1	0.95
35	2.6	1110.8	0.32	103.16	2.12	6.9	61.[illegible]	1.12	115.3	2.11
36	2.7	1308.5	0.38	109.55	2.2[illegible]	12.2	10[illegible].1	1.87	118.4	2.16
37	2.4	602.8	0.23	10[illegible].24	2.10	12.5	94.9	1.74	1[illegible]3.3	2.75
38	2.7	D/R		96.85	1.99	2.0	unknown		110.7	2.12
39	0.1	no runoff		no runoff			no runoff		no runoff	
40	0.4	no runoff		3.50	0.07	0.1	unknown		unknown	
41	0.2	no runoff		0.07	0.001	-	no runoff		no runoff	
42	1.1	no runoff		(32.46	0.67)	3.9	(unknown)		(35.2	0.64)
43	0.1	no runoff								
44	/0.1/	no runoff		/2.67	0.05/	-	no runoff		unknown	
45	0.1	no runoff		no runoff			no runoff		no runoff	
46	0.1	(535.5	0.15)	(14.76	0.30)	0.9	(unknown)		(unknown)	
47	0.3									
48	12.4	3934.4	1.14	522.67	10.73	17.8	448.9	8.23	561.2	10.22
49	0.8	no runoff		3.58	0.07	0.1	unknown		unknown	
Σ	53.1	12647.0	3.65	2095.73	43.04		1613.8	33.27	2180.9	39.74

PERCENTAGE RUNOFF / DURATION OF EVENT / PEAK INTENSITY / FLOW RATE

No.	PS %	FAR %	PR$_f$ %	PR$_g$ %	P hrs m	PS hrs m	FAR hrs m	PR$_f$ hrs m	PR$_g$ hrs m	P 5 min average mm/hr	PS l/s	FAR 15 min average l/s	FAR adjusted l/s	PR$_f$ l/s	PR$_g$ l/s	AC	STORM DATE November
20	-	-	-	-	0.01	D/R	-	-	-	-	D/R	-	-	-	-	XD	2
21	D/R	94.6	90.0	77.3	10.26	D/R	25.23	14.00	14.00	2.1	D/R	0.0169	0.0185	0.015	0.025	D	3
22	(12.7)	(87.2)	(68.0)	(67.8)	6.08	(7.50)	(16.30)	(11.40)	(10.35)	3.4	(0.19)	(0.0234	0.0276)	(0.037)	(0.036)	D	4
23					0.37					0.4						W	4
24	17.9	90.9	61.9	59.9	1.24	11.50	11.30	5.55	5.00	3.6	0.31	0.0172	0.0188	0.050	0.045	D	4
25	10.9	70.7	92.8	71.3	1.45	7.50	14.00	7.05	4. 0	3.2	0.37	0.0312	0.0385	0.037	0.048	D	5
26	-	-	-	-	0.05	-	-	-	-	-	-	-	-	-	-	W	6
27	-	-	-	-	0.01	-	-	-	-	-	-	-	-	-	-	W	6
28	D/R	(85.2)	(70.7)	(64.0)	0.04	D/R	(21.00)	(8.10)	(8.00)	-	D/R	(0.0138	0.0140)	(0.015)	(0.013)	W	6
29	D/R				5.26	D/R				2.9	D/R					W	6
30	D/R	86.2	78.9	85.1	3.57	D/R	9. 5	6.40	6.20	3.0	D/R	0.0279	0.0339	0.033	0.045	D	7
31	-	-	-	-	0.15	-	-	-	-	-	-	-	-	-	-	XD	9
32	-	-	-	-	0.14	-	-	-	-	-	-	-	-	-	-	XD	11
33	12.4	74.4	66.7	76.5	3.57	7.39	9.20	9.55	10.00	4.6	0.54	0.0304	0.0374	0.059	0.075	D	11
34	4.3	96.0	64.8	79.2	2.38	8.20	7.55	9.45	6.2	3.4	0.11	0.0117	0.0117	0.028	0.025	D	12
35	12.3	81.5	43.0	81.0	5.15	10.50	10.00	13.30	13.40	2.0	0.15	0.0146	0.0153	0.011	0.023	XD	14
36	14.0	83.3	69.4	79.9	4.07	12.20	13.35	9.50	9.[illegible]	3.4	0.35	0.0229	0.0269	0.032	0.035	D	15
37	9.7	87.5	72.[illegible]	93.6	6.56	[illegible].10	15.20	10.40	10.00	5.1	0.38	0.0233	0.0274	0.037	0.048	D	16
38	D/R	75.7	-	74.7	7.17	D/R	D/R	11.55	14.00	1.7	D/R	0.0043	0.0043	0.006*	0.007	XD	17
39	-	-	-	-	0.01	-	-	-	-	-	-	-	-	-	-	XD	20
40	-	17.0	-	-	6.30	-	12.22	5.20	4.30	0.8	-	0.0002	0.0002	0.006*	0.006*	XD	21
41	-	0.7	-	-	2.00	-	-	-	-	-	-	-	-	-	-	XD	22
42	-	(55.6)	-	(53.4)	2.17	-	(9.00)	(4.30)	(4.20)	1.5	-	(0.0086	0.0086)	(0.006)	(0.009)	XD	23
43	-		-		2.45	-				-	-					W	23–24
44	-	-	-	-	1.13	-	/3.00/	-	-	-	-	-	-	-	(0.006*)	D	24
45	-	-	-	-	0.21	-	-	-	-	-	-	-	-	-		D	25
46	(38.5)	(75.8)	(-)	(-)	0.26	(3.33)	(7.45)	(3.50)	(7.15)	-	(0.22)	(0.0019	0.0019)	(0.006*)	(0.007)	W	25
47					1.24					1.1						W	25
48	9.2	86.6	66.4	82.5	15.52	16.16	21.10	17.50	18.50	5.8	0.67	0.0316	0.0392	0.053	0.085	XD	26–27
49	-	9.2	-	-	0.52	-	7.20	7.15	6.00	1.3	-	0.0002	0.0002	0.006*	0.006*	XD	30
Σ																	

DECEMBER RUNOFF RESULTS

Table 6.3 — STORM RUNOFF VOLUMES — PERCENTAGE RUNOFF — DURATION OF EVENT — PEAK INTENSITY / FLOW RATE — DECEMBER

No.	P	PS		FAR		FAR	PR_f		PR_g		PS	FAR	PR_f	PR_g	P	PS	FAR	PR_f	PR_g	P	PS	FAR	FAR	PR_f	PR_g	AC	STORM DATE
						Error														5 min average		15 min average	adjusted				
	mm	l	mm	l	mm	%	l	mm	l	mm	%	%	%	%	hrs m	hrs m	hrs m	hrs m	hrs m	mm/hr	l/s	l/s	l/s	l/s	l/s		December
50	0.35	no runoff		0.28	0.006	-	unknown		unknown		-	1.6	-	-	0.20	-	2.40	5.00	5.00	4.2	-	-	-	0.006*	0.006*	XD	2
51	0.35	no runoff		6.32	0.13	0.1	unknown		unknown		-	28.9	-	-	1.45	-	10.45	4.05	4.15	2.4	-	0.0003	0.0003	0.006*	0.006	D	3
52	0.2	no runoff		no runoff		-	no runoff		no runoff		-	-	-	-	0.52	-	-	-	-	-	-	-	-	-	-	XD	5
53	1.4	D/R		51.51	1.06	5.3	36.4	0.67	39.55	0.72	D/R	75.6	47.7	51.5	2.17	D/R	15.10	5.15	5.55	1.5	D/R	0.0117	0.0117	0.011	0.013	W	5
54	0.4	no runoff		21.99	0.45	0.7	unknown		unknown		-	112.9	-	-	2.11	-	27.15	7.10	6.25	0.8	-	0.0015	0.0015	0.006*	0.006*	D	6
55	1.2	408.8	0.12	46.73	0.96	7.4	49.0	0.89	43.13	0.78	9.8	80.0	74.4	65.5	0.50	2.15	5.15	4.30	4.15	8.8	0.16	0.0153	0.0162	0.028	0.031	XD	7
56	0.1	no runoff		(26.28	0.54)	2.0	no runoff		no runoff		-	(77.1)	-	-	0.07	-	D/R	-	-	-	-	(0.0043	0.0043)	-	-	W	7
57	0.4	no runoff					(25.0	0.46)	(21.4	0.39)	-		(91.7)	(78.0)	0.45	-	D/R	(4.00)	(4.00)	1.7	-			(0.007)	(0.009)	W	7
58	0.1	no runoff									-				0.15	-	D/R			0.5	-					W	7
59	0.1	no runoff					no runoff		no runoff		-		-	-	0.15	-	D/R	-	-	0.5	-			-	-	W	7–8
60	5.5	D/R		240.39	4.94	8.8	234.1	4.29	253.1	4.61	D/R	89.8	78.1	83.8	8.00	D/R	13.30	13.50	14.25	1.0	D/R	0.0176	0.0194	0.024	0.029	XD	9
61	11.7	D/R		420.92	8.64	36.6	592.6	10.87	530.0	9.66	D/R	73.9	92.9	82.6	4.00	D/R	17.15	7.15	13.00	14.2	D/R	0.0610	0.0805	0.143	0.125	D	9
62	0.3	(979.1	0.28)	(70.90	1.45)	22.6	(112.9	2.07)	(85.1	1.55)	(12.3)	(63.3)	(90.0)	(67.4)	0.22	(2.05)	(5.42)	(4.55)	(5.30)	0.9	(1.05)	(0.0390	0.0496)	(0.135)	(0.085)	D	10
63	2.0														0.37					9.1						W	10
64	12.3	14245.9	4.11	(561.53	11.53)	28.1	518.2	9.50	(670.3	12.21)	33.4	(92.3)	77.3	(97.7)	10.38	18.20	(27.23)	15.10	(15.30)	6.7	1.20	(0.0477	0.0618)	0.067	(0.091)	W	11
65	0.2	no runoff					no runoff				-		-		0.15	-				0.2	-			-		W	11
66	0.25	no runoff		(238.00	4.89)	19.0	(319.3	5.86)	(444.2	8.09)	-	(66.1)	(79.1)	(109.4)	0.26	-	(17.18)	(17.30)	(17.55)	1.1	-	D/R	D/R	(0.053)	(0.125)	XD	12
67	7.15	2701.0	0.64								8.9				5.37	6.57				6.8	0.56	0.0335	0.0417			W	12–13
68	15.3	(15614.9	4.51)	(1016.81	20.88)	27.6	(849.7	15.59)	702.3	12.79	(25.3)	(85.2)	(87.6)	83.6	8.25	(16.45)	D/R	(15.30)	(14.40)	13.7	(4.35)	0.0468	0.0606	(0.186)	0.206	D	13–14
69	2.5					D/R			80.8	1.47				58.9	3.15		D/R			1.7		D/R	D/R		0.013	W	14
70	2.6	784.3	0.23			18.4	103.6	1.90	122.1	2.22	8.7		73.1	85.5	2.17	1.21	5.10	7.15	6.15	5.7	0.38	0.0325	0.0404	0.044	0.060	D	14
71	4.1	1283.6	0.37			D/R	269.8	4.95	231.1	4.21	9.0		120.7	102.7	7.05	7.00	10.17	9.25	9.20	4.6	0.50	D/R	D/R	0.044	0.050	W	15
72	2.5	1796.5	0.52	116.01	2.38	13.4	119.9	2.20	83.0	1.51	20.7	95.3	88.0	60.5	1.35	6.50	4.40	11.10	10.00	5.1	0.49	0.0248	0.0295	0.044	0.030	W	15
73	2.9	821.0	0.24	116.09	2.38	13.2	163.9	3.01	108.3	1.97	8.2	82.2	103.7	68.0	5.26	13.30	11.00	7.30	7.20	2.9	0.15	0.0244	0.0299	0.030	0.029	XD	18
74	0.3	no runoff		21.99	0.45	1.1	unknown		unknown		-	150.6	-	-	0.40	-	10.25	7.05	9.30	1.7	-	0.0023	0.0023	0.006*	0.006*	D	19
75	0.2	no runoff		no runoff			no runoff		no runoff		-	-	-	-	0.40	-	-	-	-	0.4	-	-	-	-	-	XD	21
76	0.1	no runoff		no runoff			no runoff		no runoff		-	-	-	-	0.15	-	-	-	-	0.4	-	-	-	-	-	D	21
77	0.4	no runoff		(86.78	1.78)	1.2	unknown		unknown		-	(118.8)	-	-	2.17	-	(23.20)	(36.00)	(24.20)	0.2	-	0.0025	0.0025	0.006*	0.006*	D	22
78	0.9	no runoff				0.6	unknown		unknown		-		-	-	9.23	-				0.4	-	0.0014	0.0014	0.006*	0.006*	D	22
79	0.2	no runoff				0.6	unknown		unknown		-		-	-	0.10	-				0.8	-	0.0013	0.0013	0.006*	0.006*	W	23
80	0.4	no runoff		28.46	0.58	1.2	unknown		unknown		-	146.1	-	-	2.15	-	10.30		3.15	0.4	-	0.0025	0.0025	0.006*	0.006*	D	23
81	/0.1/	no runoff		/5.83	0.12/	-	/unknown/		/unknown/		-	-	-	-	0.30	-	/10.15/		/10.30/	0.2	-	-	-	0.006*	0.006*	D	24
82	/0.3/	no runoff		/0.70	0.14/	-	no runoff		no runoff		-	-	-	-	5.38	-	/3.20/	-	-	0.6	-	-	-	-	-	D	25
83	0.2	no runoff		no runoff			no runoff		no runoff		-	-	-	-	8.56	-	-	-	-	-	-	-	-	-	-	D	25–26
84	/0.2/	no runoff		/5.20	0.11/	-	/unknown/		/unknown/		-	-	-	-	1.45	-	/7.45/	/12.30/	/13.37/	0.2	-	-	-	0.006*	0.006*	W	26
85	58.7	D/R		1986.04	40.79	30.9	2591.8	47.54	2592.6	47.24	D/R	69.5	81.0	80.5	27.25	D/R	31.15	28.20	28.00	28.5	D/R	0.0521	0.0679	0.177	0.312	D	27–28
86	0.9	1290.0	0.37	25.86	0.53	4.3	39.7	0.73	27.2	0.50	41.4	59.0	80.9	55.2	1.20	2.15	2.20	3.50	3.35	6.3	0.10	0.0094	0.0094	0.032	0.029	D	29
87	/0.3/	no runoff		/15.32	0.31/	-	/unknown/		/unknown/		-	-	-	-	0.30	-	/7.00/	/5.05/	/3.50/	0.6	-	-	-	0.006*	0.006*	D	29
88	/0.2/	no runoff		/1.69	0.03/	-	D/R		D/R		-	-	D/R	D/R	0.10	-	/4.50/	D/R	D/R	1.2	-	-	-	D/R	D/R	XD	31
	136.3	39425.1	11.39	5082.89	104.37		6025.9	110.53	6034.2	109.92																	

The start time, duration and peak rainfall intensity averaged over a 5 minute period for each rain event were extracted from the rainfall charts. The start time and duration of runoff for each remaining instrument were taken for comparison. Peak discharges were read from the converted water level records of the Arkon and Munro recorders while the peak discharge from the Casella tilting-bucket flowmeter was obtained by averageing over a 15-minute period of highest chart-recorded tipping rate. All water level records were converted to discharge and the total volume of runoff per storm calculated as the area beneath the hydrograph. The number of tips per storm recorded by the tilting-bucket flowmeter provided the volume of runoff. Where necessary the tilting bucket peak flow rates were adjusted according to the calibration (Fig 5.3) and a maximum volume error % correction calculated for each storm on the basis of this peak flow rate. All runoff volumes were converted to an equivalent depth of rainfall according to the area supplying runoff and then divided by the amount of rain per storm to give a percentage runoff figure. Where runoff was inseparable into the contributing separate rain events, the rain events were added together also for calculation of the % runoff figure. All analysis was carried out manually and the figures are presented in tables 6.1, 6.2 and 6.3 each one covering one month.

Table 6.4 Instrument Reliability And Record Length

	Paved Surfaces	Flat Asphalt Roof	Pitched Roofs Front	Pitched Roofs Garage
Measurable runoff events	27	53	43	45
No runoff from rainfall	40	16	22	20
Unmeasurable runoff [1]	-	-	20	18
Defective record	13	-	2	4
Partially defective record [2]	-	13	-	-
Frost	-	6	-	-

1 Peak runoff rate less than 0.006 l/s
2 Complete tip-count but defective trace

As the Arkon bubble-gauge and recorder were not installed until 12/10/79 some of the earlier events (Nos's 1 to 8) were not recorded. Some intermittent dry weather flow(of the order of 0.02 l/s) was recorded in the flume over the data collection period. Chemical analysis suggested that this was ground water which was seeping into the pipe through poor pipe connections because of the high water table in the flood plain of the River Ver. The first manhole above the flume remained dry during these periods which confirmed that the flow was groundwater rather than foul water from any misconnections with the foul-water system. Some chart analysis therefore required the removal of this base flow from the storm runoff volume. Base flow separation was achieved by the simple method of drawing a straight line from the start of storm runoff to the point of inflection on the recession limb. Other base flow separation techniques were considered unnecessarily complicated for this analysis.
The Arkon flume and recorder were overdesigned for the majority of flows recorded from the paved surfaces with only 2 storms producing more than 1.0 l/s. Three of the larger storms were lost by backing up in the flume by the River Ver and a total of 13 events were lost (Table 6.4).

Calibrations of the flat asphalt roof tipping-bucket flowmeter performed in Chapter 5 (Fig's 5.2, 5.3 and 5.4) provided adjustments for the recorded flow rate. Flow rates can be calculated as the number of tips over a 15 minute period, but to subdivide each storm into 15 minute average flow rates and to adjust those flow rates above the critical value of 0.0131 l/s was considered very time consuming and impractical for a preliminary stage of analysis. Only when the whole hydrograph is known can corrections be made to the storm runoff volume on the basis of the underestimate caused by high tipping rates.The record is therefore simply presented with the unadjusted volume of runoff and the maximum volume error (as a %) calculated from the storm peak discharge. Several storms have been fully analysed as outlined above and are discussed in a later section. From these it is clear that the maximum volume error correction relating only to the short-duration peak flow greatly exaggerates the under-recording of the volume. The tipping bucket flowmeter gave a near complete runoff record from the flat asphalt garage roof (Table 6.4) with only 13 events having merely a tip-count with no accurate chart trace. This was caused either by a flat battery or where very high intensity rainfalls caused throttling back in the funnel and provided too fast a tipping rate for the pen arm to record. 15 events had a maximum tipped volume error of more than 15% with 9 of these exceeding 20% volume error for peak flow rates. 6 events were produced by thawing frost and are specially marked on tables 6.2 and 6.3. They are excluded from the analysis because the true amount of precipitation supplying the runoff could not be accurately recorded by the raingauges. Melting frost produced peak flow rates of 0.0002 l/s and a total of 31.85 litres from the flat roof.

Conversion of the Munro water-level charts to discharge proceeded to the lowest rated value (0.006 l/s) when the remaining tank volume of 4.0 litres (verified by siphoning the water out during tests on the weir tanks, see Chapter 5) was added to the total runoff volume. This may have produced underestimates of the total runoff where rain had not ceased before this value was reached, but the very low runoff rates (in drips) would only marginally affect the total. The weir tanks used for measuring pitched roof runoff proved sensitive to even light falls of rain. Unfortunately where these rainfalls produced peak runoff rates of less than 0.006 l/s it proved impossible to calculate the volume of runoff. Table 6.4

indicates that a total of 38 events from both sides of the pitched roof could not be used in the analysis because of this.

SUMMARY STATISTICS

Peak Flow Rates

Table 6.5 Peak Flow Rate Summary

Rainfall Intensity (mm/hr)	Paved Surfaces Flow Rate (l/s) f	Range	Mean	Median
Low <1.5	2	0.09 - 0.22	0.16	0.16
Median 1.5 - 10.0	20	0.10 - 1.64	0.48	0.38
High >10.0	1	4.35	4.35	4.35
Rainfall Intensity (mm/hr)	**Flat Garage Roof Flow Rate (l/s) f**	**Range**	**Mean**	**Median**
Low	15	0.0002 - 0.0194	0.0035	0.0015
Median	29	0.0003 - 0.0671	0.0265	0.0269
High	6	0.0606 - 0.0805	0.0673	0.0650
Rainfall Intensity (mm/hr)	**Front Pitched Roof Flow Rate (l/s) f**	**Range**	**Mean**	**Median**
Low	15	<0.006 - 0.024	-	<0.006
Median	32	<0.006 - 0.135	0.036	0.033
High	7	0.010 - 0.410	0.236	0.186
Rainfall Intensity (mm/hr)	**Garage Pitched Roof Flow Rate (l/s) f**	**Range**	**Mean**	**Median**
Low	16	<0.006 - 0.029	-	<0.006
Median	32	<0.006 - 0.125	0.039	0.031
High	6	0.125 - 0.530	0.285	0.280

To summarise the effect of each surface on peak flow rate, a frequency table was constructed (Table 6.5). The rainfalls were categorised according to intensity (low, less than 1.5 mm/hr; medium, 1.5 to 10.0 mm/hr; and high, greater than 10.0 mm/hr) and the corresponding range and average flow rates (both the mean and median) were calculated for each surface. The garage roof and pitched roof areas are both approximately $50m^2$ which allows comparison between their figures. For low and medium intensity storms the peak flow rates from the flat and pitched roofs are very similar (0.027 l/s and 0.037 l/s respectively) while from similar rainfalls, the road surface produces rates 13 times greater (0.48 l/s) from an area 70 times greater. The most marked effect of the flat roof was on high intensity storms where average peak flow rate only achieved 0.067 l/s compared with 0.26 l/s (4 times greater) from the pitched roofs. Therefore the lower slope of the garage roof produces a large reduction in runoff rates from high intensity storms.

The highest recorded rainfall intensity of 54.5 mm/hr on 13/10/79 (storm 9) also produced the highest flow rates from the pitched roofs - 0.53 l/s from the rear half of the roof and 0.41 l/s from the front half. The tipping bucket gauge recorded a high rate of flow - 0.72 l/s from the asphalt garage roof block, but its highest flow rate was 0.0805 l/s recorded on 9/12/79 (storm 61) from a rainfall intensity of 14.2 mm/hr. The Arkon flume suffered backing up by the River Ver for storm 9 and the highest peak flow rate recorded from the paved surfaces was 4.35 l/s from 2 large combined storms (storms 68 and 69) on 13-14/12/79 from a rainfall intensity of 13.7 mm/hr - this latter being the highest rainfall intensity producing a reliable hydrograph from the paved surfaces.

Runoff Duration

Runoff duration proved difficult to determine for all the surfaces either because of an instrument design fault or because of the nature of the runoff itself. Because a certain amount of base flow was also recorded at the flume, runoff duration from the paved surfaces is an approximate figure, roughly determined by the base-flow separation technique. Runoff into the tipping bucket from the flat roof often lasted such a long time that it was still continuing as drips when the next storm began. A rough approximation of the end of runoff can be made at the point where tipping rate increases for the next storm but then a reduced figure for the storm duration is obtained. Some storms proved impossible to separate at all and a combined duration is given in Tables 6.1, 6.2 and 6.3. Severe surface tension effects over the bottom of the weir in the pitched roof runoff weir tanks combined with slight stickiness of the pen column around zero produced overlong recession limbs and therefore an exaggeration of the runoff duration. Allowances were made for this tendency where suspected. In some cases runoff continued until the next storm as with the flat roof so that an underestimate of runoff duration resulted. These errors must be considered when examining the runoff duration figures.

The duration of runoff from the surfaces does not seem to follow any simple pattern. Logically it should be some function of the rainfall duration and rainfall amount with some coefficient to describe the influence of each surface. Owing to the difficulties in storm runoff duration determination detailed above, no statistical relationship was sought.

By ranking the runoff durations for those storms when all the instruments recorded runoff according to the longest and lesser durations it was possible to derive a frequency table (Table 6.6). The flat asphalt roof generally has the longest duration with one exceptional storm (storm 35) having 30 hours of runoff after a 5¼ hour storm. Runoff from the pitched roofs have the 2nd and 3rd longest durations whilst the difference in duration of runoff between the 2 roof types is usually small. Runoff from the paved surfaces is generally of shortest duration except for a few exceptions where the storm is preceeded by wet conditions or has a large volume and high % runoff. This would be consistent with a reduction in infiltration capacity of the road subsurface for these events so that runoff would last longer.

The slightly longer runoff duration from the flat roof is necessary to produce the high % runoff figures from low peak flows. Road runoff with its low runoff coefficient and road surface permeability is of shorter duration than the roofs apart from when permeability is reduced.

Table 6.6. Ranked Frequency Table of Longest Runoff Duration (all instruments recording runoff)

	Paved Surfaces	Flat Asphalt Roof	Pitched Roof front	Pitched Roof garage
Rank 1	3	12	4	2
2	4	3	6	8
3	1	5	9	6
4	13	1	2	5
Most frequent rank	4	1	3	2

Runoff Volume

The total runoff volumes and percentages for each month and totals for the 3 monthly period are presented in table 6.7. As the total runoff volumes are mainly a reflection of the catchment area for each instrument, comparisons between the surfaces are more easily made when these volumes are reduced to an equivalent depth of rainfall and converted to a percentage of the rainfall. The major feature is that the % runoff is well below 100% for all surfaces despite their being "impermeable."

Table 6.7

MONTHLY RUNOFF VOLUME SUMMARY

Month	n	p	Paved Surfaces Flume Area 3465.44 m^2 Slope 1.37%			Garage Roof Tipping Bucket 48.69 m^2 1.67%			Pitched Roofs Front Weir Tank 54.52 m^2 66%			Pitched Roofs Garage Weir Tank 54.88 m^2 66%		
		mm	l	mm	%	l	mm	%	l	mm	%	l	mm	%
October	3	16.0	7263.7	2.10	13									
	12	56.8				1919.6	39.41	69						
	10	74.05							2650.3	48.60	66			
	8	49.05										1453.2	26.48	54
November	12	32.5	12647.0	3.65	11									
	24	52.2				2095.7	43.04	82						
	14	46.5							1813.8	33.27	72			
	17	50.5										2180.9	39.74	78
December	12	53.75	39425.1	11.39	21									
	30	135.6				5082.9	104.37	77						
	19	131.8							6025.9	110.53	84			
	20	132.0										6034.2	109.92	83
Totals	27	102.25	59335.8	17.14	17									
	66	244.4				9098.2	186.82	76						
	43	252.2							10490.0	192.40	76			
	45	231.7										9668.3	176.14	76

p – precipitation totals are only for those storms recorded by the relevant instruments.
n – sample size

The highest figure is 84% for the front pitched roof side in December, and the lowest from the paved surfaces where only 11% runoff was recorded in November. The quantity of runoff from both the roof types is very similar and in percentage terms is identical (76%) over the 3 months suggesting that the flat garage roof does not afford greater depression storage or evaporation than the pitched tiled roofs. The road surface only manages to generate approximately ¼ of the equivalent runoff that the roofs produce – 17% over the 3 month period.

The runoff recorded from the pitched roofs shows an increase in % runoff through the months October to December of 66–84% for the front pitched roof side and 54–83% for the garage pitched roof side. This is presumably because of reduced evaporation with the increasingly cold temperatures. The average pan evaporation figure for October was 1.0 mm per day and 0.7 mm per day for the first half of December. During the latter half of December the water surface often became frozen so that pan evaporation was difficult to record but obviously low. The remaining losses from the pitched roofs can be accounted for by surface wetting and depression storage since it can be safely assumed that there is no infiltration through roofs.

The figures from the flat asphalt garage roof are unadjusted for the underestimate of volume caused by high tipping bucket rates. This means that for the months of October and December when at least half the recorded events had high flow rates, under-recording of the volume of runoff is extremely likely. For December, if the storms with a maximum volume % error of over 20% are removed from the totals, the % runoff figure increases from 77% to 81%. The same action for October would however only leave 3 events on which to calculate the % runoff figure. November has the highest % runoff figure of 82% and as this was a month of low intensity storms little correction to this figure is needed. If the tipping bucket volumes were all corrected they are likely to show the same increase in runoff through the months that the weir tanks recorded from the pitched roofs. Runoff from the flat roof would also be on average a higher % of rainfall than that from the pitched roofs.

The paved surface % runoff is extremely low for all 3 months. The 13% runoff for October is based on only 3 recorded events with one large storm exaggerating the figure. December was a very wet month with 137.4 mm of rain falling which produced saturated soil conditions in the layer of clay-with-flints overlying the chalk of this area. This saturated soil would inhibit infiltration and drainage through the road, and combined with reduced evaporation figures could account for the near doubling of runoff from November to December (11% to 21%).

Because the values in Table 6.7 cover all recorded runoff events and the relevant rainfalls, to achieve strict comparability only those storms where all the runoff recording instruments functioned are totalled for each month in Table 6.8.

As the record for the road surface in Table 6.8 is virtually identical to that in Table 6.7 there is little difference in the figures. The sample size is however much reduced for the roofs and the values are therefore perhaps not such a good indicator of runoff response to rainfall for the monthly periods. The largest discrepancies occur in October when only 2 events were correctly recorded by all instruments. In general, percent runoff is increased from all the roofs while the differences between months for

Table 6.8

MONTHLY RUNOFF VOLUME SUMMARY
FOR IDENTICAL STORMS

Month	n	P	Paved Surfaces Flume Area 3465.44 m^2 Slope 1.37%			Garage Roof Tipping Bucket 48.69 m^2 1.67%			Pitched Roofs: Front Weir Tank 54.52 m^2 66%			Pitched Roofs: Garage Weir Tank 54.88 m^2 66%		
		mm	l	mm	%	l	mm	%	l	mm	%	l	mm	%
October	2	5.0	1046.3	0.31	6.2	209.8	4.30	81.1	231.5	4.24	75.0	192.7	3.51	70.2
November	10	32.1	12111.5	3.50	10.9	1326.2	27.23	84.8	1180.6	21.66	67.5	1398.3	25.32	78.9
December	12	53.8	39425.1	11.39	21.0	2191.9	45.00	83.6	2546.0	46.70	86.8	2597.6	47.31	87.9
Totals	24	90.9	52582.9	15.20	16.7	3727.9	76.53	84.2	3958.1	72.60	79.9	4179.6	76.14	83.8

P - precipitation
n - sample size

the same surface are maintained.

To discover how antecendent conditions affect losses from each surface, the average % runoff from the 3 categories - XD, a minimum of 24 hrs previously dry, D between 6 and 24 hrs previously dry and W, rain occuring in the preceding 6 hours - were calculated and are displayed in Table 6.9.

Table 6.9 Effect of Antecedent Conditions on Percentage Runoff

Antecedent Conditions	Paved Surfaces	Flat * Asphalt Roof	Pitched Roofs Front	Pitched Roofs Garage
XD	9.0	56.1	66.2	73.2
D	15.9	85.6	75.7	73.8
W	20.7	83.4	76.0	78.4
Ratio W:XD	2.3	1.5	1.1	1.1

* Uncorrected values for flow rate.

Runoff increases from all the surfaces from XD to W events but there is a very marked increase recorded from the flat garage roof. Its greater depression storage and initial wetting requirements (see later analysis) have a greater influence on the quantities of runoff produced so making antecedent conditions very important for this surface. Although the increase in % runoff from XD to W events on the paved surfaces is only 11.7% this represents a doubling of runoff. Antecedent conditions are therefore also extremely important for determining the runoff from paved surfaces. The least affected surfaces are the tiled pitched roofs. This is presumably a function of the very low depression storage which once satisfied will make little volumetric difference to the amount of run-off per storm.

In addition, the rain and runoff events were plotted according to their antecedent conditions category (Figs 6.1, 6.2, 6.3, and 6.4). The 'W' events are a series of combined storms, so that the number of points is less than the true number of recorded hydrographs. There does not appear to be any distinct pattern into which the XD, D and W events fall. Some of the W events have very low runoff rates and high losses while some XD events produce very high runoff rates contrary to what would be expected. Further subdivision according to storm characteristics, time of year and more detail about preceding conditions is probably necessary to explain the groupings. Such information is not however available.

The regression line used to calculate depression storage is plotted through the XD events. The line of equal rainfall and runoff represents 100% runoff and apart from 1 or 2 exceptional but low volume events, all runoff events fall above this line indicating less than 100% runoff. The plotting of an envelope line passing through the origin to include all runoff events (apart from those producing 100% runoff) gives a line of minimum losses for each surface. The logic behind this line is that for very small events, especially those with wet preceding conditions with depression storage and other initial losses satisfied, runoff would equal rainfall so the line passes through the axes origin. With longer duration and larger storms, the possibility exists for greater losses through evaporation and infiltration in the case of the road surface, so that the envelope line diverges from the rainfall equal to runoff line. An attempt to calculate a regression line through the combined 'W' events was greatly influenced by the larger storms pulling the intercept with the rainfall-axis above all the smaller volume storms near the axes origin. This was not considered a useful explanation for rainfall and runoff when antecedent conditions are not dry.

Fig. 6.1

PAVED SURFACE RUNOFF, ALL EVENTS

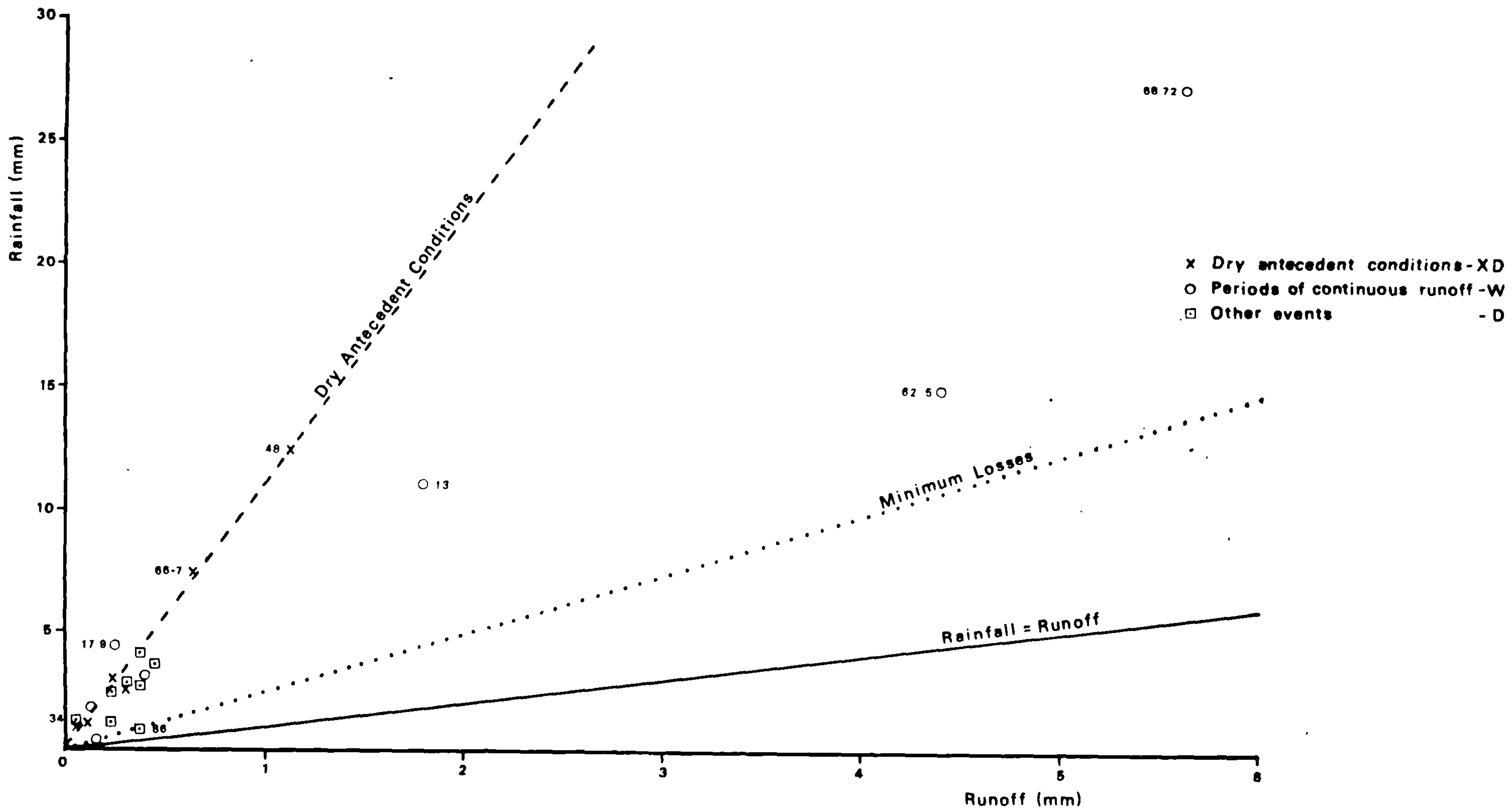


Fig. 6.2 FLAT ASPHALT GARAGE ROOF RUNOFF, ALL EVENTS

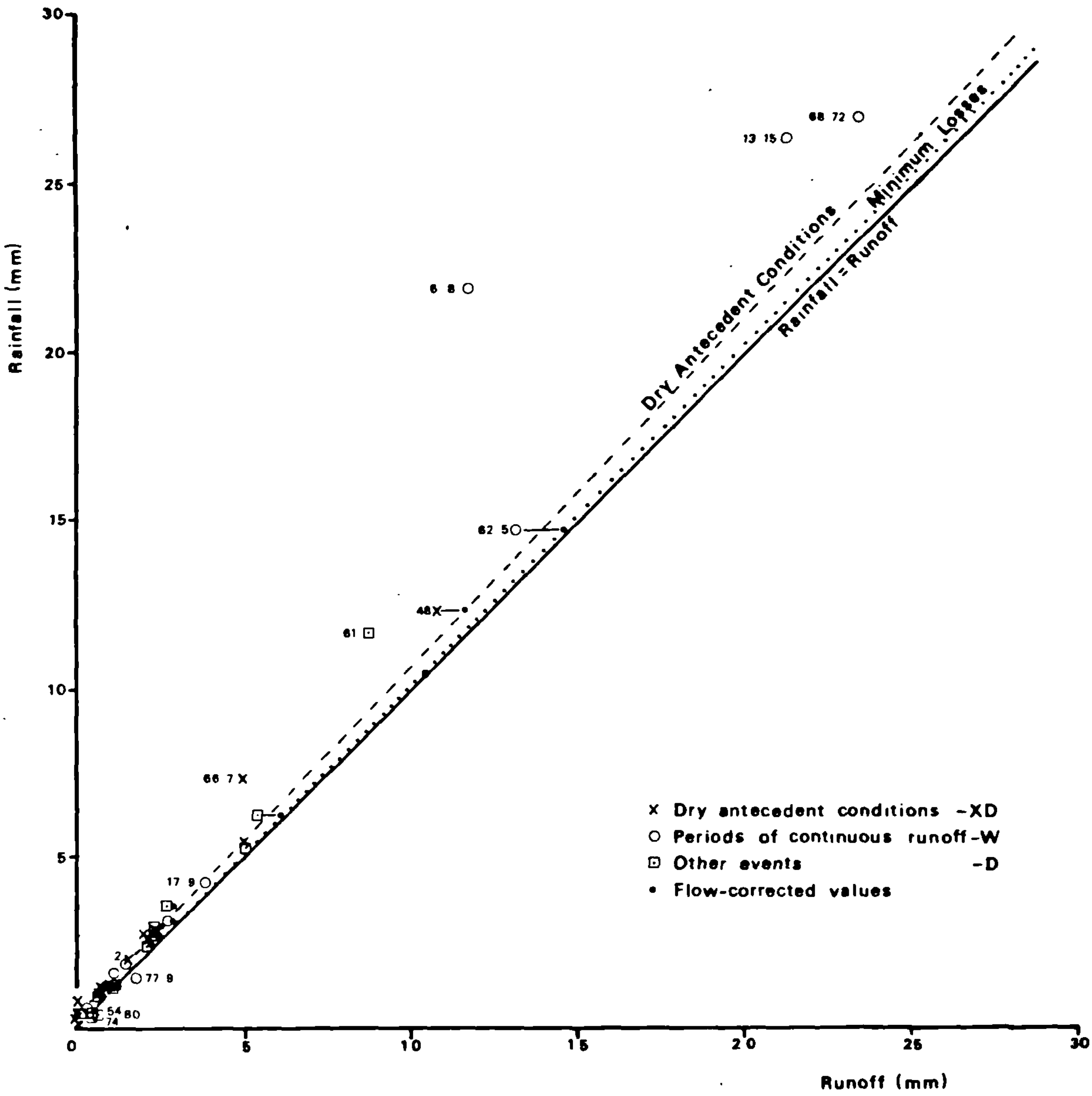


Fig. 6.3

PITCHED ROOF RUNOFF, ALL EVENTS
FRONT WEIR TANK

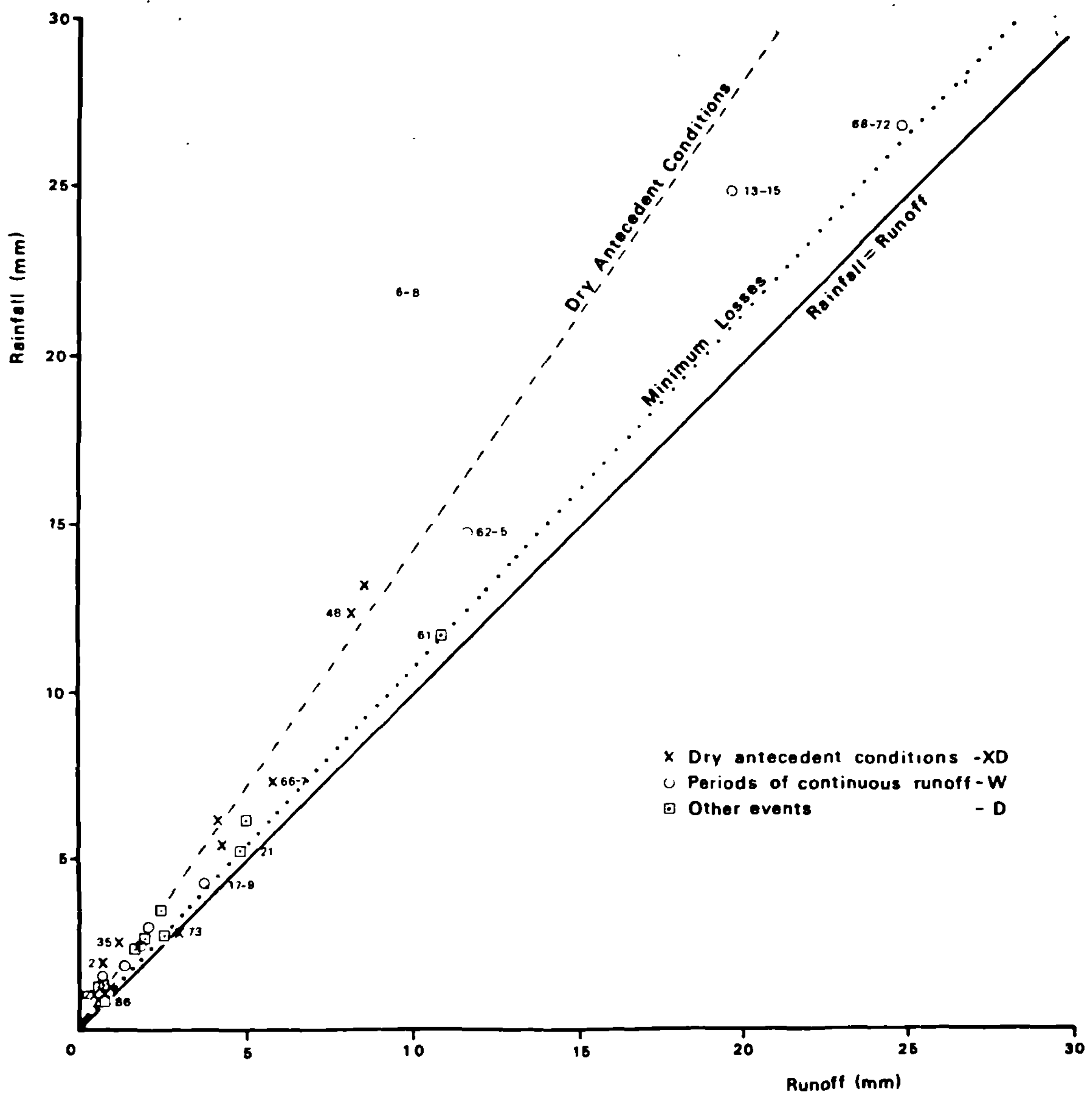
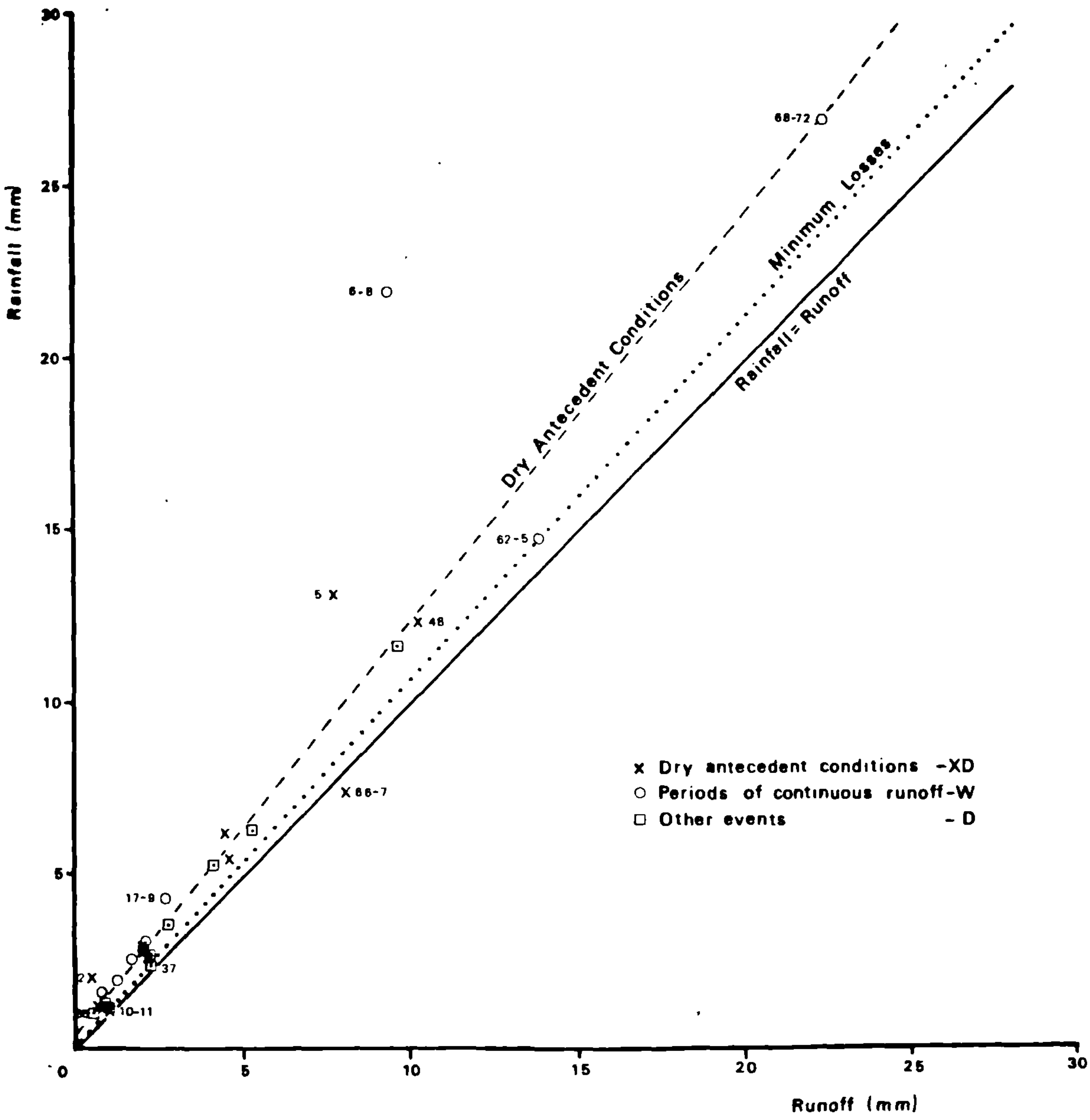


Fig. 6.4

PITCHED ROOF RUNOFF. ALL EVENTS
GARAGE WEIR TANK



The slope of the minimum losses line varies for each surface indicating different total losses. Using only those storms whose volume has been corrected according to the flow rate calibration for the tipping bucket, the minimum losses line for the flat asphalt roof has a slope of 1 in 1.013 which is equivalent to a maximum runoff of 98.7%. The losses from this type of roof are minimal. The minimum losses line for both sides of the pitched roof has a slope of 1 in 1.07 equivalent to a maximum runoff of 93.5%. It may be that the true rainfall incident on the pitched roof is less than that recorded by the horizontally exposed raingauge and caught by the flat roof. This would account for the apparently higher losses from the pitched roof. Alternatively, there may be greater losses of rainfall from the edges of the tiles so that the flat roof with its banked-up edges is a more efficient collector and remover of rainfall towards the gutter.

The minimum losses line for the paved surfaces has a slope of 1 in 2.43. This however includes one exceptional event (storm 86) which produced 41% runoff. The preceding day had suffered 58.7 mm of rain making ground conditions saturated. It is likely that these saturated conditions caused the huge increase in % runoff either by retarding infiltration through the road or by an increase in the contributing area with grass verges and gardens supplying runoff. If this storm is excluded, losses increase so that runoff averages only 30% of rainfall for wet spells. The road therefore produces losses over 3 times greater than those produced by the roofs. Since the evaporation rate will be approximately the same for the asphalt-and-chippings flat roof and the asphalt roads and pavements (they have the same reflection coefficient and surface temperature), the additional losses from the roads must be accounted for by infiltration. The actual road surface geometry is not complicated and no large puddles of water remain on the surface, the extra water simply disappears into the road.

Referring again to Tables 6.1, 6.2, and 6.3, runoff volumes from the surfaces are largely a reflection of the different supplying area. Run-off from both roof types is very similar in volume despite differences in surface texture and slope – the lesser peak flows from the flat roof being compensated for by generally longer runoff duration. Although the road area is nearly 70 times larger than each roof area, total volumes of runoff are only in general 10 times greater. The highest volume of runoff from one storm was 1986.0 litres from the garage roof, 2591.8 litres from the front pitched roof and 2592.6 litres from the garage-side pitched roof, all from a storm of 58.7 mm. Unfortunately the flume suffered backing up from this latter storm so that a storm of 12.3 mm produced the largest volume of 14,245.9 litres recorded from the paved surfaces.

Once the runoff volumes have been corrected for area into mm equivalent depth of rainfall and converted to a % of the rainfall, comparisons can be made between all the surfaces. Runoff from both roofs averages 76% of runoff which is approximately 4½ times that from the road surfaces. Even without a completely corrected set of runoff volumes, it is clear that the flat roof produces very slightly higher amounts of runoff than the pitched roofs.

Some of the extreme events (either very high or very low runoff for the amount of rainfall) are labelled on Figs. 6.1–6.4. October produces some of the lowest % runoff values. For instance storm 2 (2.0 mm) produced 23.9% from the garage-side pitched roof and 33.7% from the front pitched roof. The lowest recorded % runoff from the flat roof is 0.7% as a result of 1 bucket tip from 0.2 mm of rain. 4.3% runoff is the lowest recorded runoff from the paved surfaces from 1.2 mm rain (storm 34) with dry preceding conditions.

As a balance to the concentration on lower runoff coefficients, several events from both roof types recorded over 100% runoff from mid-December onwards. Four light rainstorms between 0.3 and 1.5 mm produced runoff in excess of rainfall amounts from the garage roof. The highest figures from each surface are 109% from 7.4 mm (storms 66 and 67) from the garage-side pitched roof, 121% from 4.1 mm (storm 71) from the front pitched roof, with 41% from 0.9 mm rain (storm 86) recorded from the paved surfaces.

Explanations for these extreme events are difficult with only limited information about preceding conditions and none about meteorological conditions other than average monthly temperature and mean daily pan evaporation (Table 6.10).

Table 6.10 Monthly Temperature and Evaporation Rates at Redbourn

1979	Mean Daily Pan Evaporation (mm)	Mean Monthly Temperature (°C)
October	0.97	7.4
November	0.50	5.1
December	0.74	7.8

Unfortunately the evaporation and temperature values recorded in December apply only to the first half of the month and not to the latter when the high runoff coefficients were recorded from the roofs since the evaporation pan was frozen. It is likely therefore that the evaporation rate and mean monthly temperature should be reduced for December and may provide the explanation for the higher runoff percentages.

Using the average runoff coefficients from the roofs it is possible to scale-up the runoff and calculate volumes for the whole catchment as if all roofs and roads were connected to the drains. As porches and garden sheds are usually left to drain into gardens these areas have been left out of the calculation. The total catchment area would be 6153.16 m^2 composed of 3465.44 m^2 of roads, 1064.68 m^2 of pitched roofs and 1623.04 m^2 of flat roofs. Using the average roof runoff coefficient of 0.76 for both roof types and 0.17 for the roads, from a 1 mm storm there would be 2042.67 litres from the roofs and 589.12 litres from the roads. Thus the roofs while only making up 44% of the impermeable surfaces would provide 78% of the total runoff from the catchment.

Antecedent conditions and time of year therefore have an important influence on the runoff volume from each surface. The greatest influence on the amount of runoff is however the surface itself.

ANALYSIS OF LOSSES

The preceding sections have summarised the main properties of runoff from urban surfaces - peak flow rates, duration and volume according to month and antecedent conditions. Further analysis of the data allows calculations and estimations of the main losses from rainfall - depression storage, evaporation from the impermeable surfaces calculated as the residual from the roof water balances and infiltration through the paved surfaces assuming the same rate of evaporation as the roofs and again calculating a water balance.

Depression Storage

From a cursory examination of the data, runoff is only initiated from the paved surfaces by storms of about 1.00 mm or more while runoff occurs from the roofs from much smaller storms of about 0.2 to 0.3 mm. 40 events produced no runoff from the paved surfaces while only 16 were not large enough to start runoff from the garage roof. The pitched roofs were merely wetted on 22 and 20 occasions for the front and garage sides respectively.

Using only storms with a preceding period of at least 24 hrs without rain it is possible to calculate depression storage according to the technique described by Kidd (1978a) and Falk and Niemczynowicz (1978). Rainfall and runoff volumes are plotted and the intercept of the rainfall axis with the regression line is considered to represent depression storage. This was done for all 4 recorded surfaces and the resulting graphs are Figs 6.5,6.6,6.7 and 6.8. The depression storage is calculated as 0.02 mm for the paved surfaces, 0.37 mm for the flat asphalt garage roof, 0.11 mm for the front pitched roof and 0.39 mm for the garage-side pitched roof. These latter two give an average value of 0.25 mm. The very low paved surface figure is a reflection of the steepness of the line caused by very low rates of runoff per storm so that the intercept is too low on the rainfall axis. The values used to initially calculate the flat asphalt garage roof depression storage figure were uncorrected for the under-recording of volume according to flow rate. 5 storms were fully analysed and the new values are plotted on Fig 6.2. The intercept was only marginally affected and the new calculated depression storage value becomes 0.32 mm.

Fig. 6 5

LOSSES FROM PAVED SURFACES

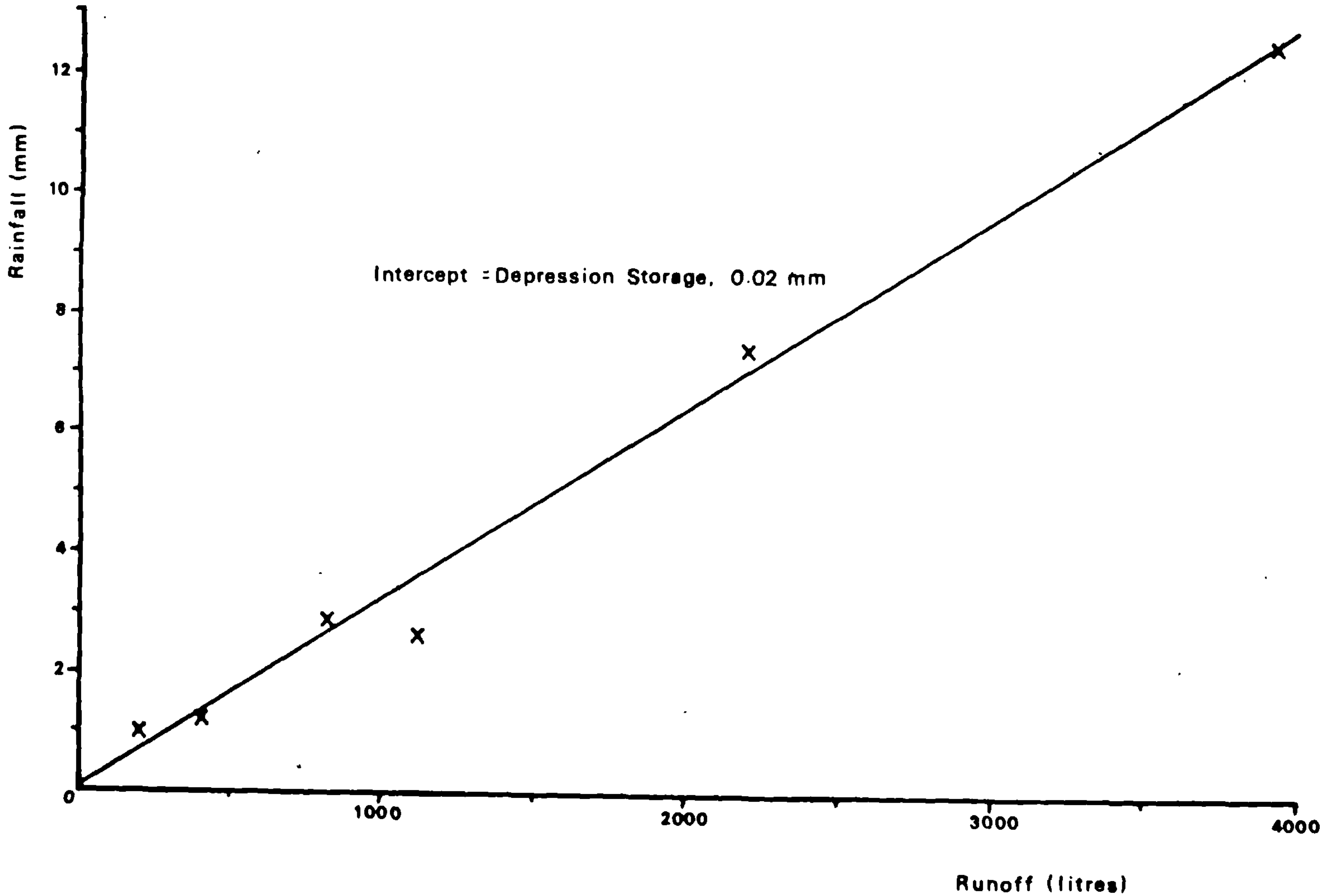
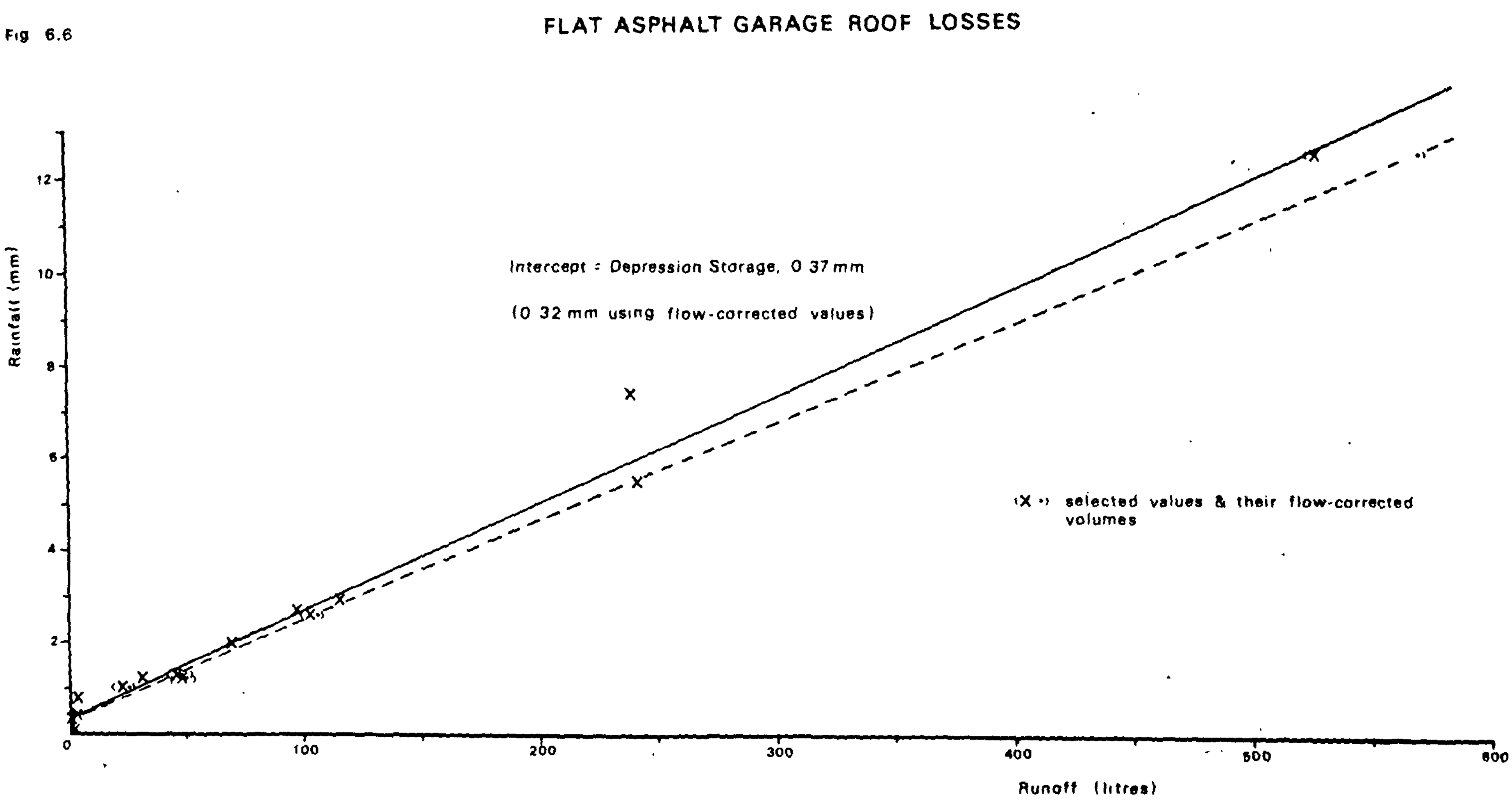


Fig 6.6



FLAT ASPHALT GARAGE ROOF LOSSES
Intercept = Depression Storage, 0 37 mm
(0 32 mm using flow-corrected values)
(X •) selected values & their flow-corrected volumes
Rainfall (mm)
Runoff (litres)
0
100
200
300
400
500
600
2
4
6
8
10
12

Fig. 6.7

PITCHED ROOF LOSSES, FRONT WEIR TANK

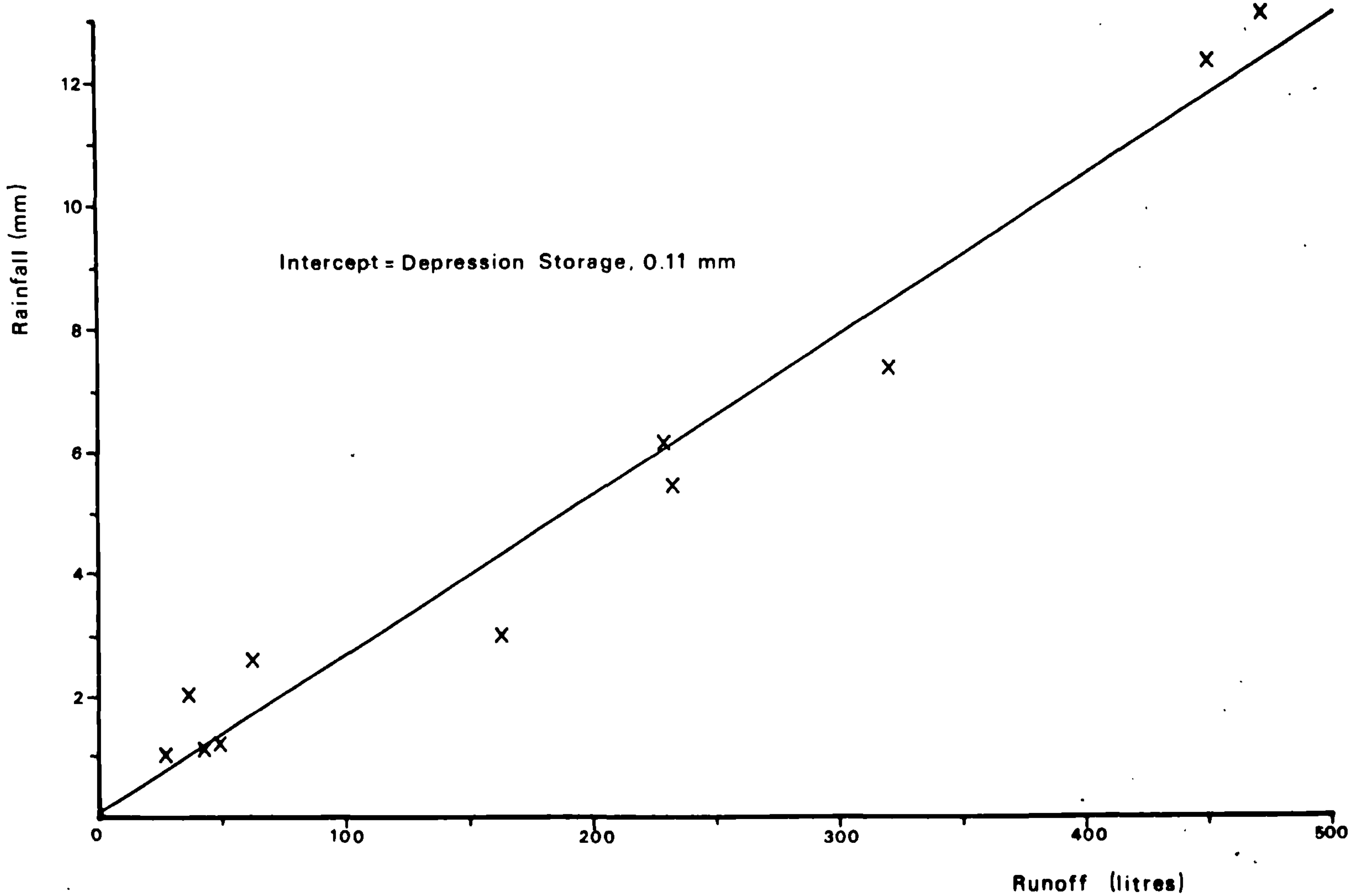
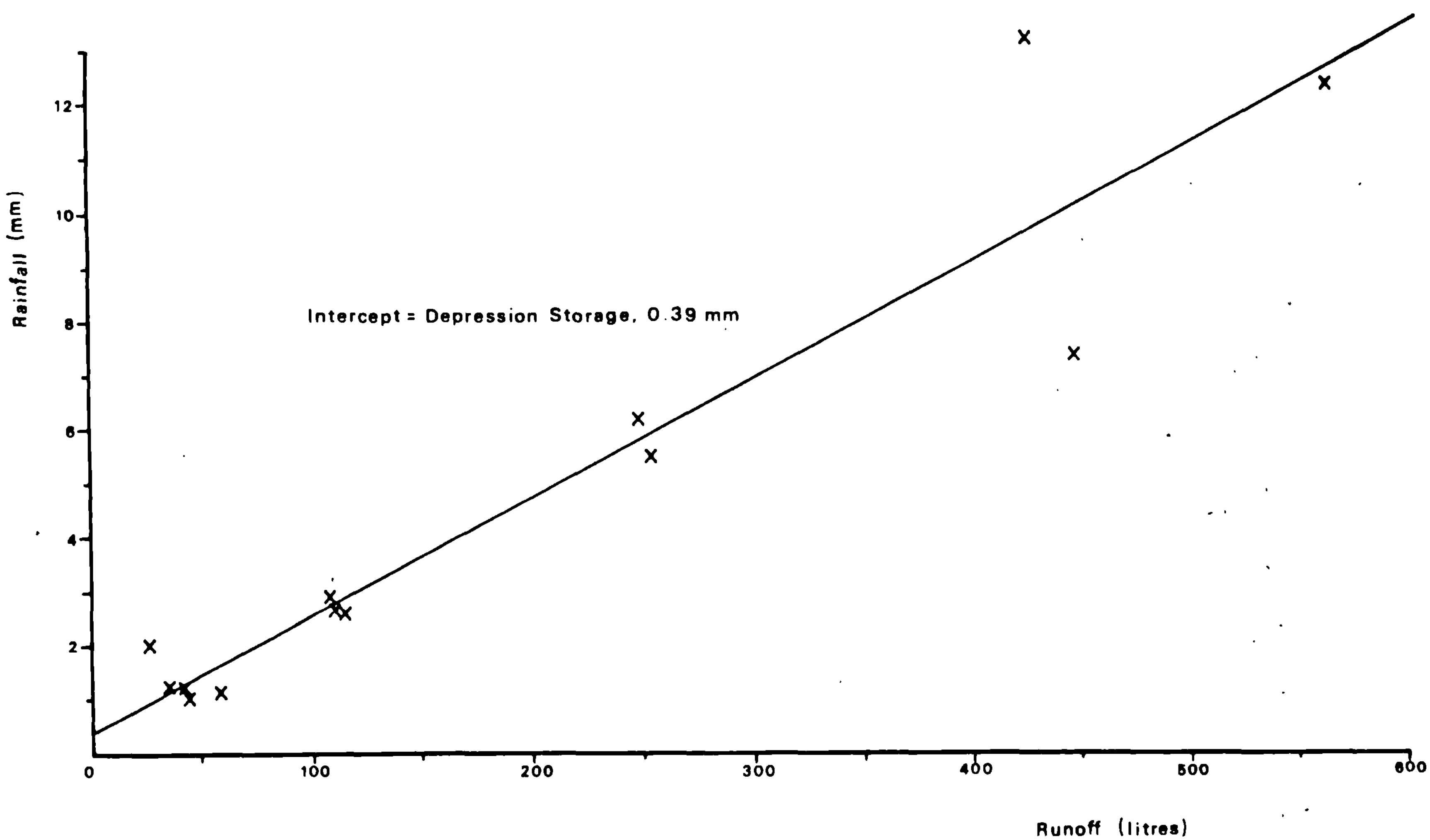


Fig 6.8

PITCHED ROOF LOSSES, GARAGE WEIR TANK



A report by Kidd 1978_b using data from several sources calculates the relationship as being

$$DEPSTO = 0.77\ SLOPE^{-0.49}$$

The corresponding values for each monitored surface according to their slope would be 0.67 mm for the 1.37% sloping roads, 0.60 mm for the 1 in 60 (1.67%) sloping garage roof and 0.10 mm for the 30^{o} pitched roofs. None of the values from the recorded data correspond very well with this statistical relationship, the greatest disparity being shown by the paved surfaces figures. It is likely that the calculated paved surface depression storage figure is influenced by additional losses such as infiltration through the road surface and through cracks in jointed kerbstones.

The response of the surfaces to light rainfall (Table 6.11) is an alternative method of assessing depression storage. Rainfalls not causing runoff could not be included in the first calculation of depression storage as those below the true value would lower the intercept on the rainfall axis. Only those rainfalls with at least 6 hours previously dry were used and any frost figures were excluded.

There is clearly a range of rainfalls between 0.2 and 0.49 mm which may or may not produce runoff from the roofs depending on prevailing conditions. The minimum rainfall which produces runoff appears to be 0.25 mm for both the roof types. The similar depression storage volume for both roof types is unexpected since intuitively the flat roof would store more rainfall. With dry preceding conditions the flat roof usually starts producing runoff after the pitched roofs suggesting a larger initial volume is necessary to overcome surface tension effects and wet the surface. It may be that the roof tiles have weathered and

Table 6.11 Depression Storage Estimation using Low Runoff Producing Rainfalls

Rainfall (mm)	Flat Asphalt Garage Roof		Pitched Roofs Front		Pitched Roofs Garage	
	Runoff	No Runoff	Runoff	No Runoff	Runoff	No Runoff
0 –	0	0	0	0	0	0
0.1 –	0	7	0	7	0	7
0.2 –	2	3	1	4	1	4
0.3 –	3	1	3	1	3	1
0.4 – 0.49	5	2	5	2	5	2
Depression Storage	0.25 mm		0.25 mm		0.25 mm	

Rainfall (mm)	Paved Surfaces Runoff	Paved Surfaces No Runoff
0 –	0	8
0.2 –	0	9
0.4 –	0	6
0.6 –	0	0
0.8 –	1	2
1.0 –	1	1
1.2 – 1.39	3	1
Depression Storage	1.0 mm	

become more absorbent over the 12 years they have been in situ. Alternatively, the pitched roofs may have a reduced rainfall catch caused by turbulence and eddying so the runoff coefficient should be higher and the depression storage lower than the figures derived from comparison with rain caught by a reference ground level gauge. Initial losses from paved surfaces are around 1.0 mm but this is not a constant value. The age of the surface will influence the amount of depression storage. Falk and Niemczynowicz (1978) on experiments on different roads found the lowest depression storage values on the newest and therefore most impermeable surface and highest on the oldest and most worn road surface with complicated geometry. The range of rainfalls producing runoff from the paved surfaces suggest that other factors besides age and preceding conditions may influence depression storage. Depression storage is not perhaps the simple initial deduction from rainfall it is assumed to be but may be a time-varying process dependant on road subsurface infiltration capacity and evaporation both operating during a storm.

Evaporation and Infiltration Estimation Through Water Balances

Further separation of the losses from rainfall is possible if a water balance for each urban surface can be calculated. Table 6.12 gives a breakdown of the various storages and transfers of rainfall on the four urban surfaces. The rainfall used is the total for each month. Depression storage is calculated as the estimated values (0.25 mm for both roof types and 1.0 mm for the road surface) multiplied by the number of events. The volume of rainfall unaccounted for by runoff and depression storage is totalled and divided by the number of events to give a rate of direct evaporation from the roofs. The unaccounted rainfall for the road surface will include a proportion of infiltration.

Table 6.12 The Fate of Rainfall on Four Urban Surfaces

Surface	Month	Rainfall mm	Events	Runoff mm	Depression Storage mm	Unaccounted Rainfall mm	Evaporation mm/event	Average Roof Evaporation mm/event	US Class A Pan Evaporation mm/day
Flat Asphalt Roof	Oct	56.8	12	39.4	3.0	14.4	1.20	2.03	0.97
	Nov	52.2	24	43.0	6.0	3.2	0.13	0.40	0.50
	Dec	135.6	30	104.4	7.5	23.7	0.79	0.84	0.74
Pitched Roof (front weir tank)	Oct	74.1	10	48.6	2.5	23.0	2.30		
	Nov	46.5	14	33.3	3.5	9.7	0.69		
	Dec	131.8	19	110.5	4.8	16.5	0.87		
Pitched Roof (garage weir tank)	Oct	49.1	8	26.5	2.0	20.6	2.58		
	Nov	50.5	17	39.7	4.3	6.5	0.38		
	Dec	132.0	20	109.9	5.0	17.1	0.86		
Paved Surfaces	Oct	16.0	3	2.1	3.0	10.9			
	Nov	32.5	12	3.7	12.0	16.8			
	Dec	53.8	12	11.4	12.0	30.4			

The evaporation rate is averaged for all 3 roof surfaces and compared with the measured US Class A evaporation pan rate. These evaporation figures are in general accordance for each month and they show lowest evaporation during November when pan evaporation was at its lowest. If it is assumed that this monthly rate applies to the road surface, the volume of rainfall lost through evaporation from the road can be calculated according to the number of recorded events, Table 6.13, and the remaining volume of unaccounted rainfall can be ascribed to infiltration through cracks around kerbstones and through the road surface itself.

The water balance in Table 6.13 shows that evaporation is more important than depression storage for the roofs (19% compared with 5% respectively for the 3 monthly period). Infiltration through the road surface is the largest single loss (36%) and is more important than runoff (17%). Depression storage volume will ultimately be transferred by evaporation and infiltration processes but is initially more important than evaporation in determining the low runoff volume from the roads.

Individual Storm Hydrographs

Twenty storms from the winter period were selected where there was a complete record from all the instruments. Storm 30 was also included as it contained steady state rainfall and runoff from the roofs although the record from the paved surfaces was defective. The hydrographs from each surface have been drawn on the same time scale to allow visual comparison of the effect of rainfall. Figs. 6.9 - 6.23 are the results, some of which contain multiple storms.

Table 6.13 Water Balances for Urban Surfaces at Redbourn, Hertfordshire

	1979	Rainfall mm	Runoff mm	Depression Storage mm	Evaporation mm	Infiltration mm
Roofs. Average of 2 pitched roofs & 1 flat asphalt roof	Oct	60.0	38.2	2.5	19.3	
	Nov	49.7	38.7	4.6	6.4	
	Dec	133.1	108.3	5.7	19.1	
	Total	242.8	185.2	12.8	44.8	
	%	100	76	5	19	
Paved Surfaces	Oct	16.0	2.1	3.0	6.1	4.8
	Nov	32.5	3.7	12.0	4.8	12.0
	Dec	53.8	11.4	12.0	10.1	20.3
	Total	102.3	17.2	27.0	21.0	37.1
	%	100	17	26	21	36

Original chart speeds meant that 15 minute average rainfall intensities and flow rates from the tipping bucket flow gauge were practical thus accounting for the stepped hydrographs of the latter. Flow rates every 5 minutes and 3 minutes were taken from the pitched roofs and paved surfaces records respectively. Because the 15 minute interval masks the true peak rainfall intensities, the record from the pitched roofs is probably a more accurate reflection of the variation in rainfall intensities.

These diagrams illustrate previous comments drawn from the tables of data. The peak flows from the paved surfaces are in general 10 times those recorded from the pitched roofs and despite the averageing effect of the 15 minute time-step from the flat roof, the runoff reaches generally ½ the peak flow rate recorded by the pitched roofs since the rainfall intensities are all within the 1.5 – 10.0 mm/hr middle range. Because of the similarity between the recession limb low flow rates from the pitched and flat roofs up to the lowest rated value of 0.006 l/s for the weir tanks it seems reasonable to extrapolate the pitched roof recession limb in a similar form to that from the flat garage roof up to the cessation of runoff. Peak flow rates from the 2 sides of the pitched roof are not consistently greater from one side or the other and this is probably a reflection of wind and storm direction. As no measure was taken of wind direction or roof catch, these differences cannot be fully explained.

RAINFALL AND RUNOFF FROM URBAN IMPERMEABLE SURFACES

Fig 6.9 STORMS 10+11 17-18/10/79

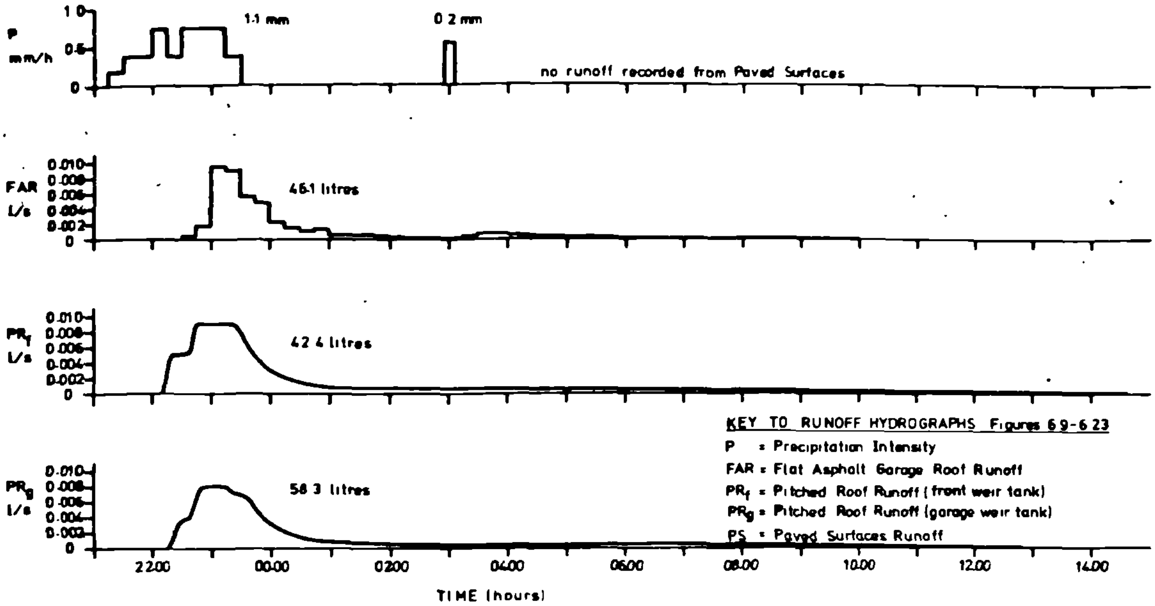


Fig 6.10 STORM 12 24/10/79

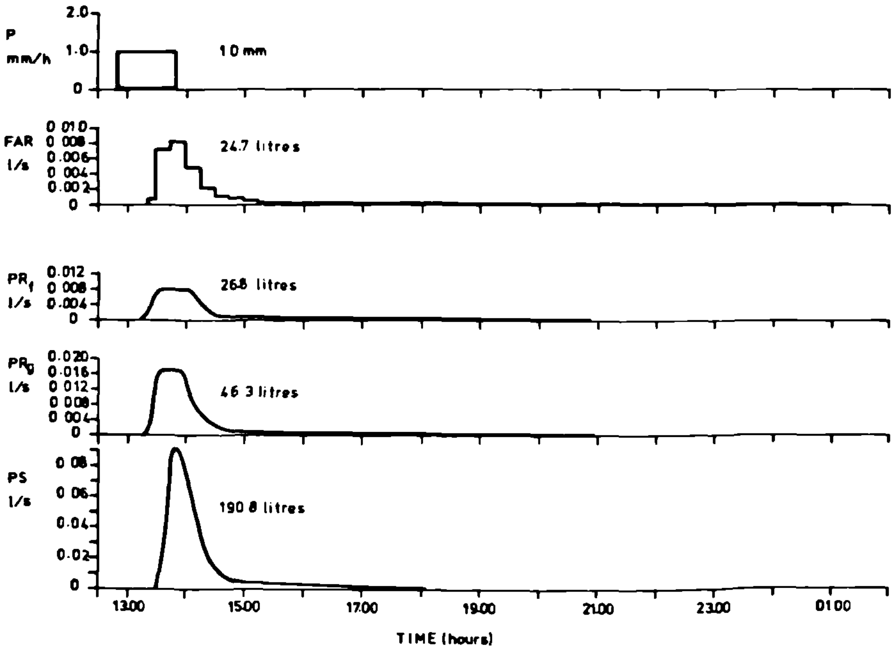


Fig 6.11 STORMS 22-23 4/11/79

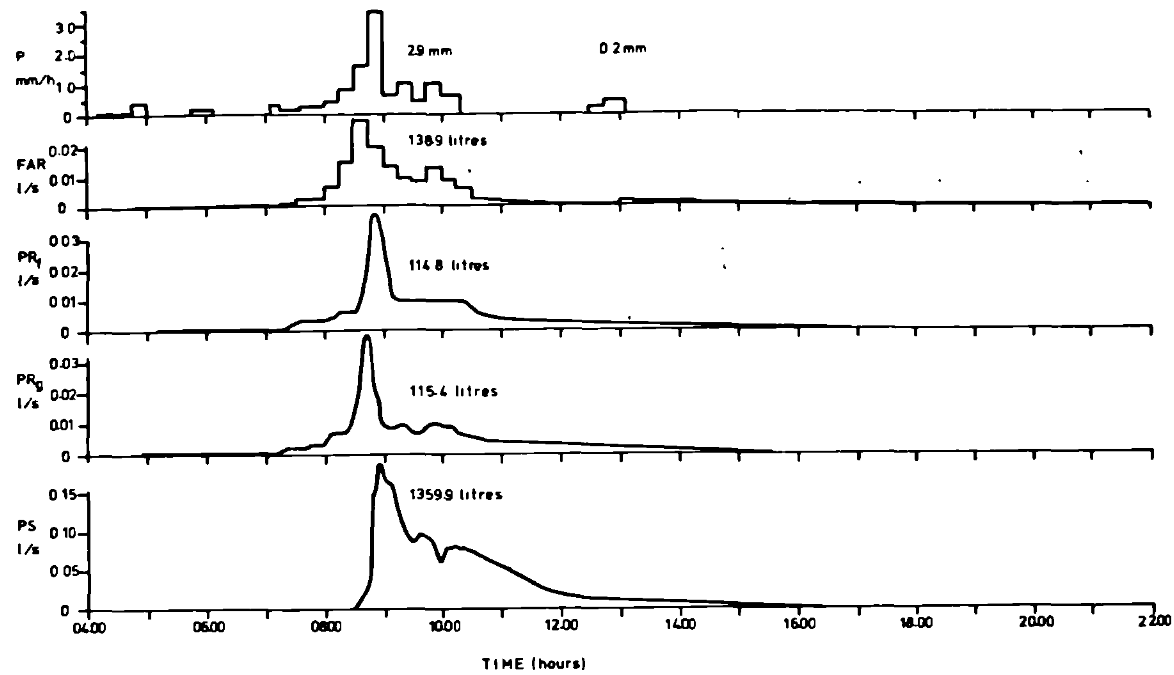


Fig 6.12 STORM 24 4/11/79

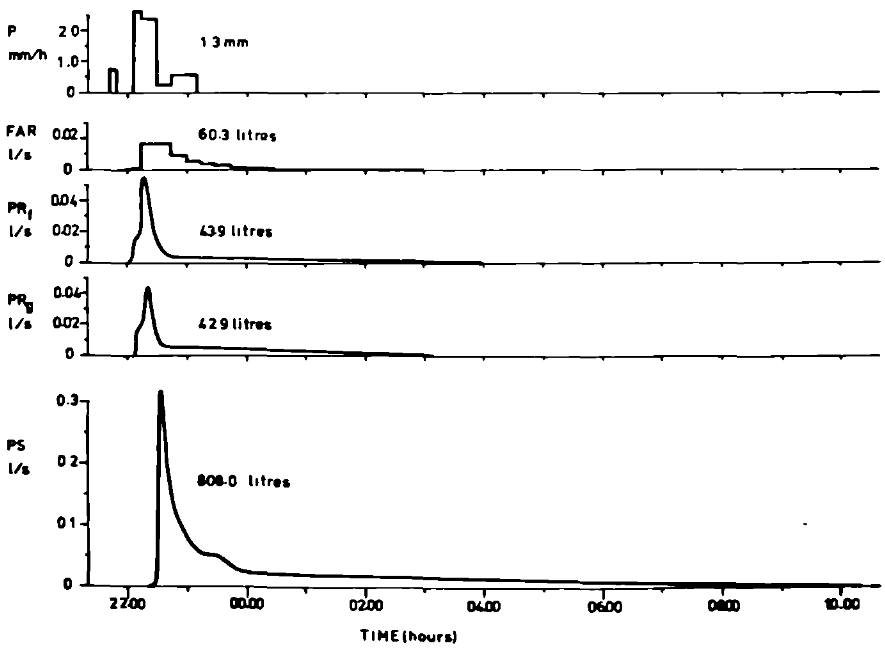


Fig. 6 13 STORMS 25-27 5/11/79

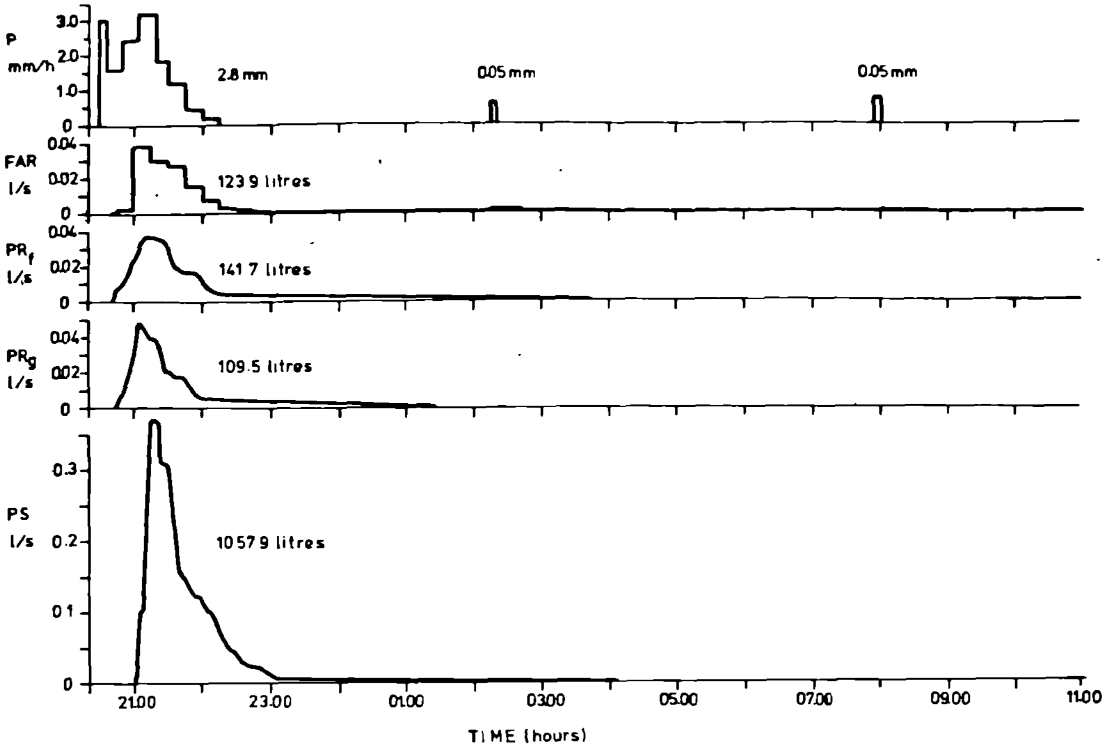


Fig 6 14 STORM 30 7/11/79

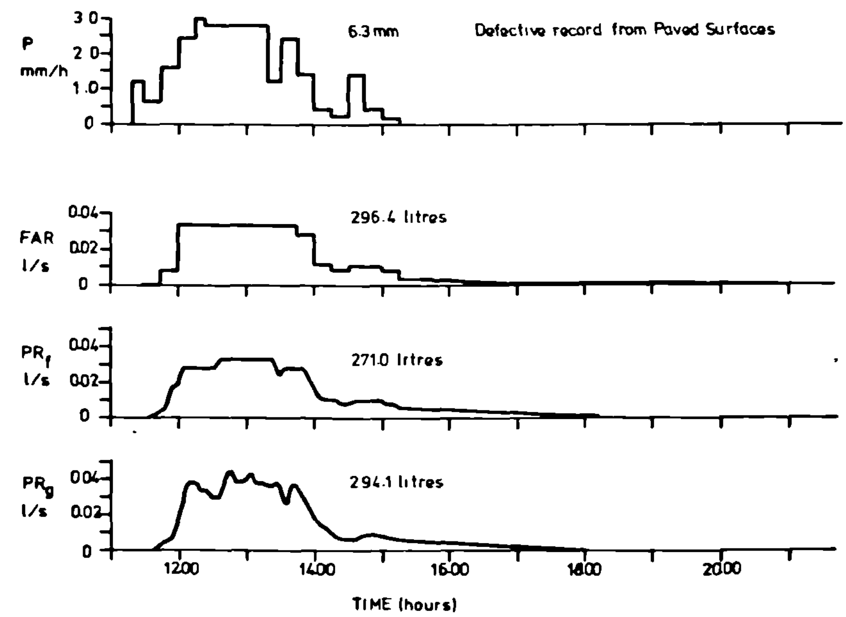


Fig 6.15 STORM 33 11/11/79

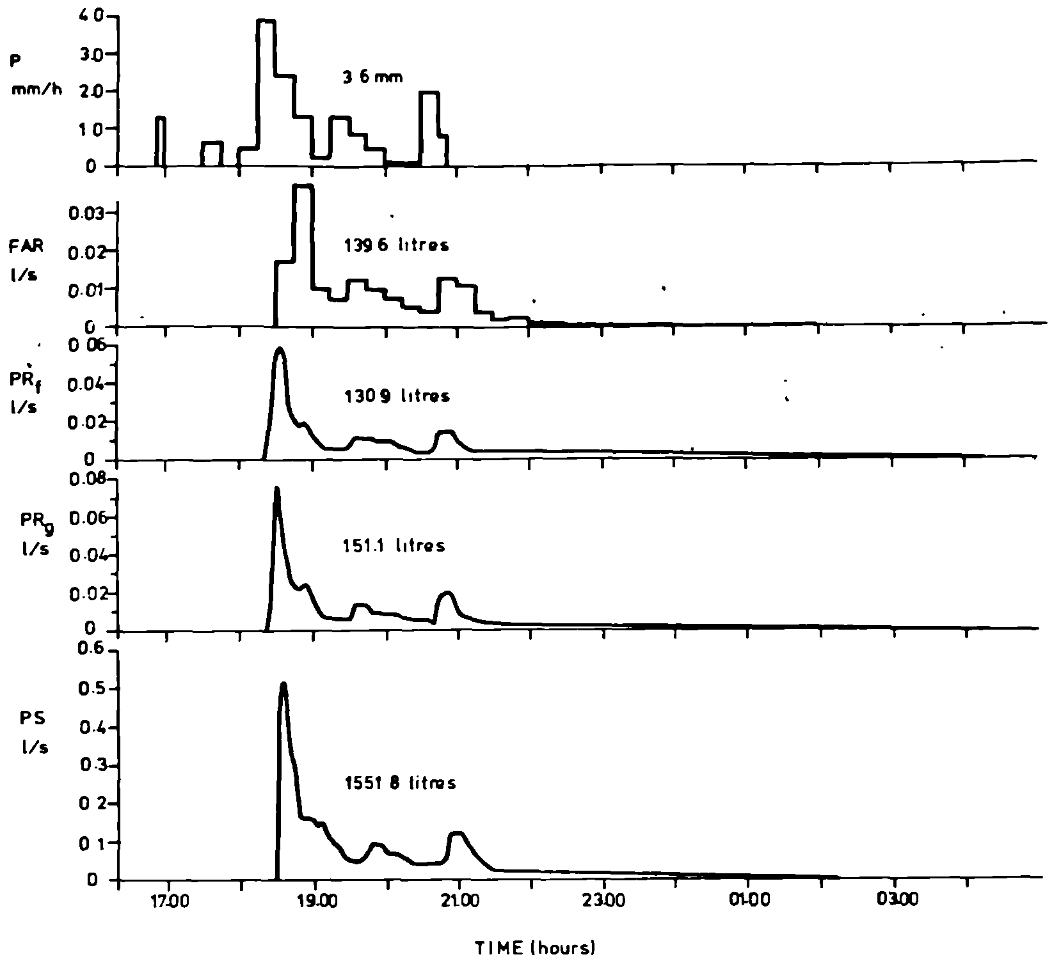


Fig 6.16 STORM 34 12/11/79

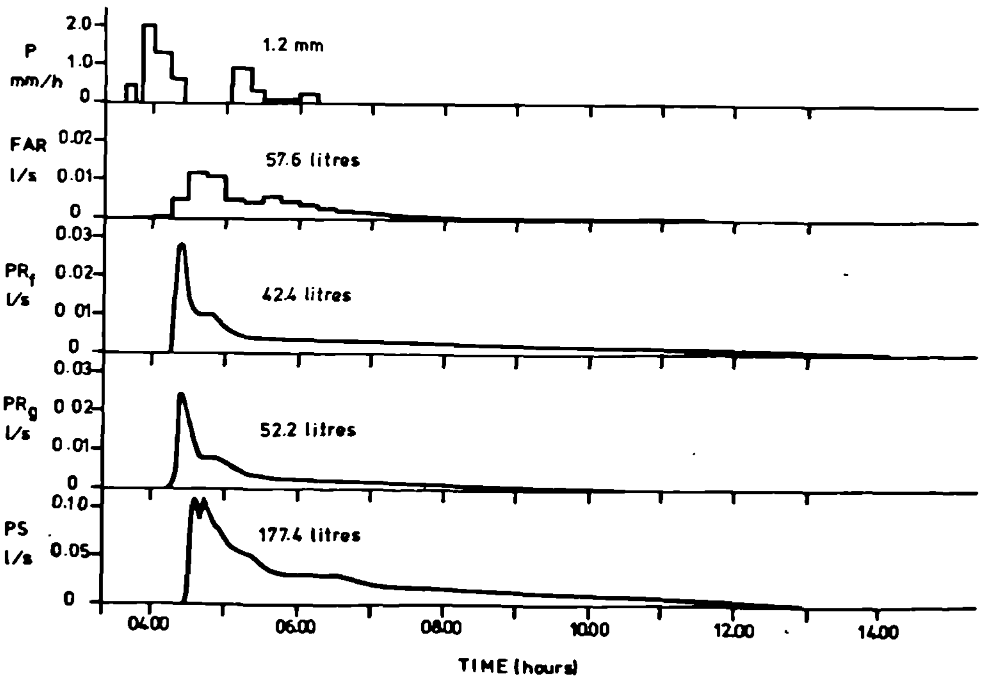


Fig 6.18 STORM 36 15/11/79

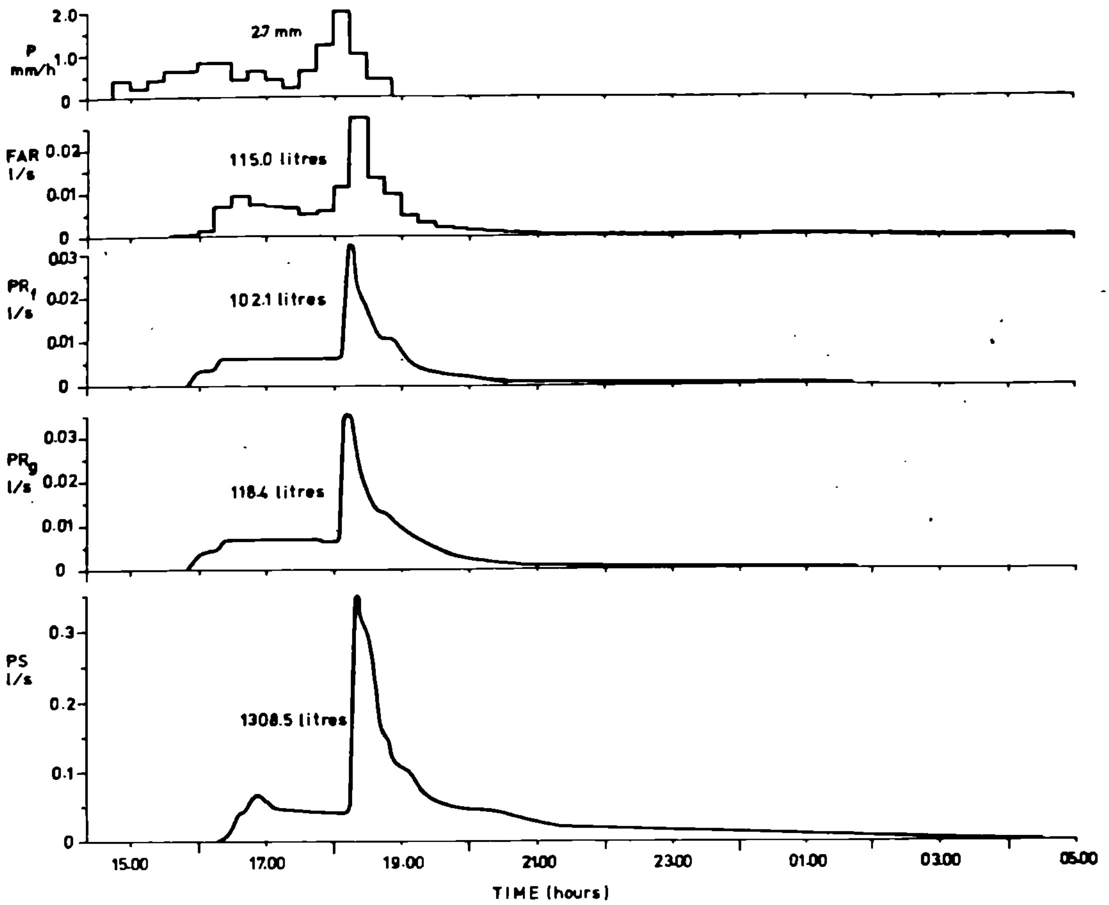


Fig 6.17 STORM 35 14/11/79

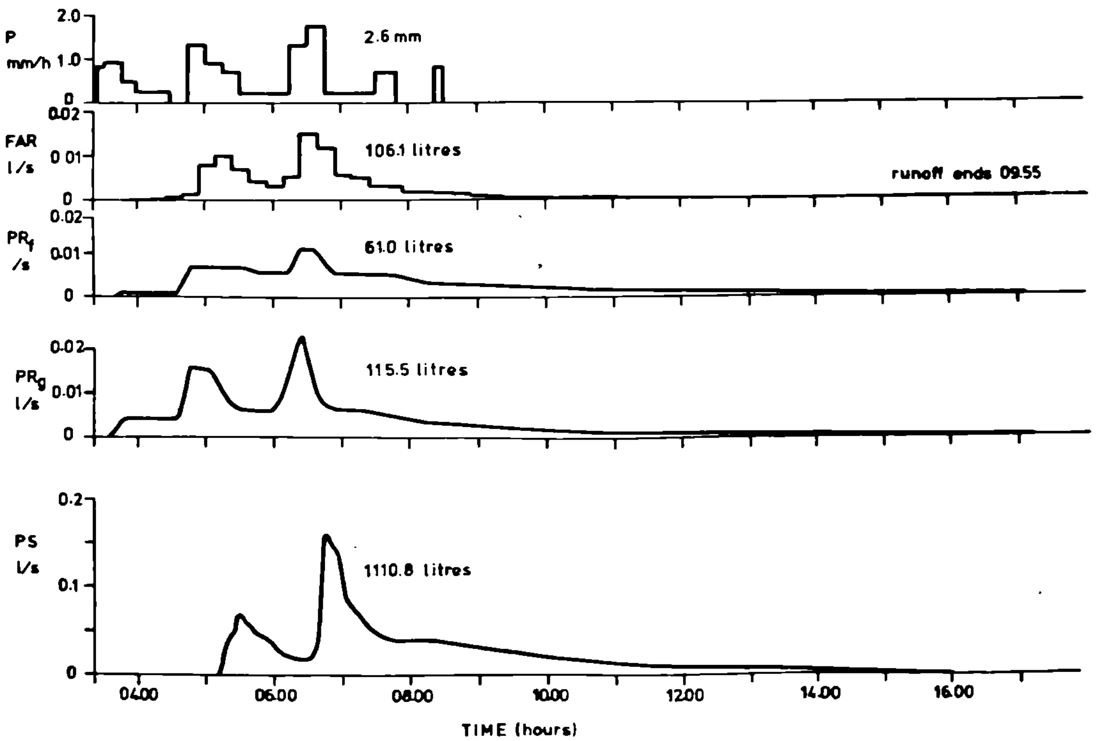


Fig 6 19 STORM 37 16/11/79

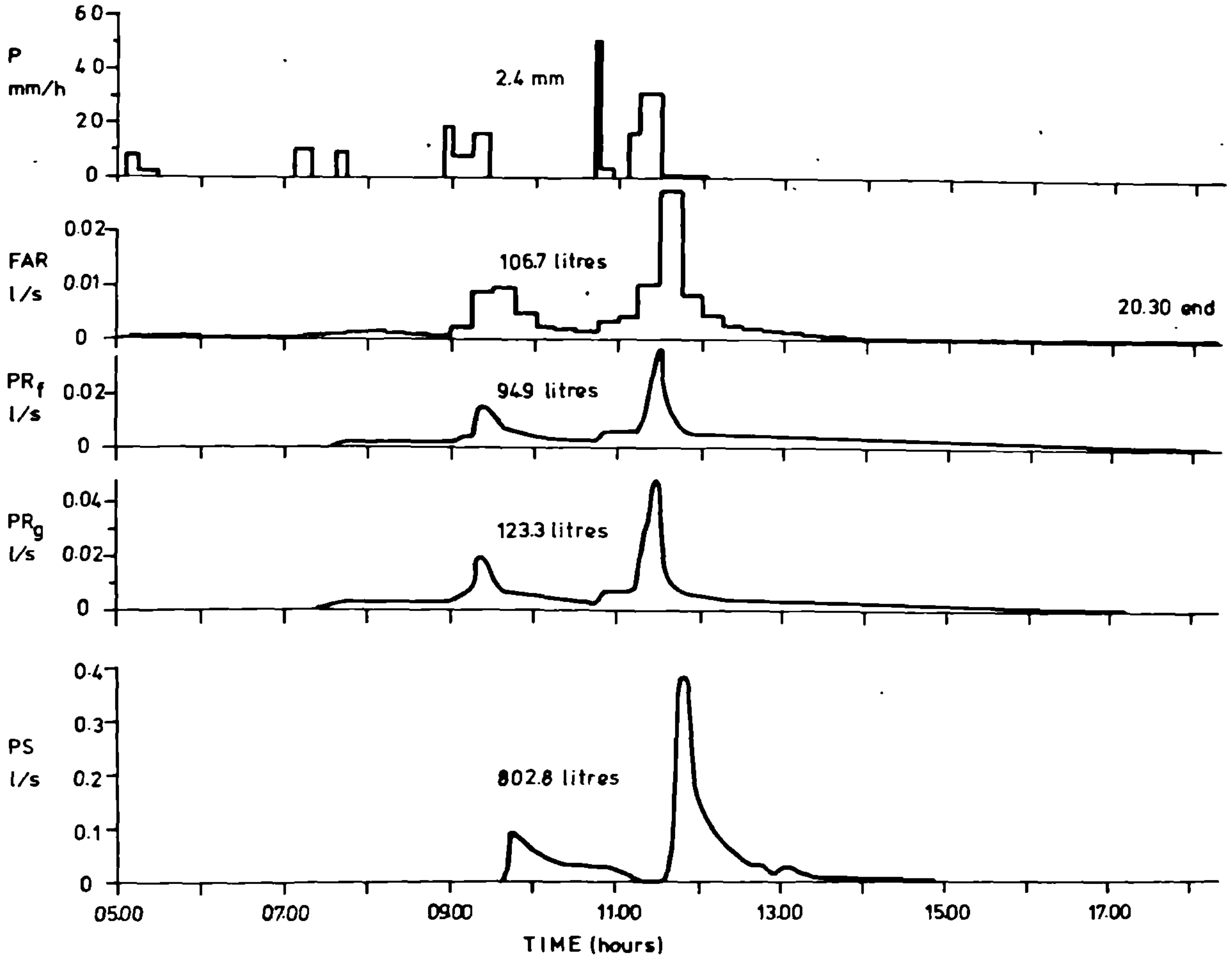


Fig. 6.20 STORM 48 26-27/11/79

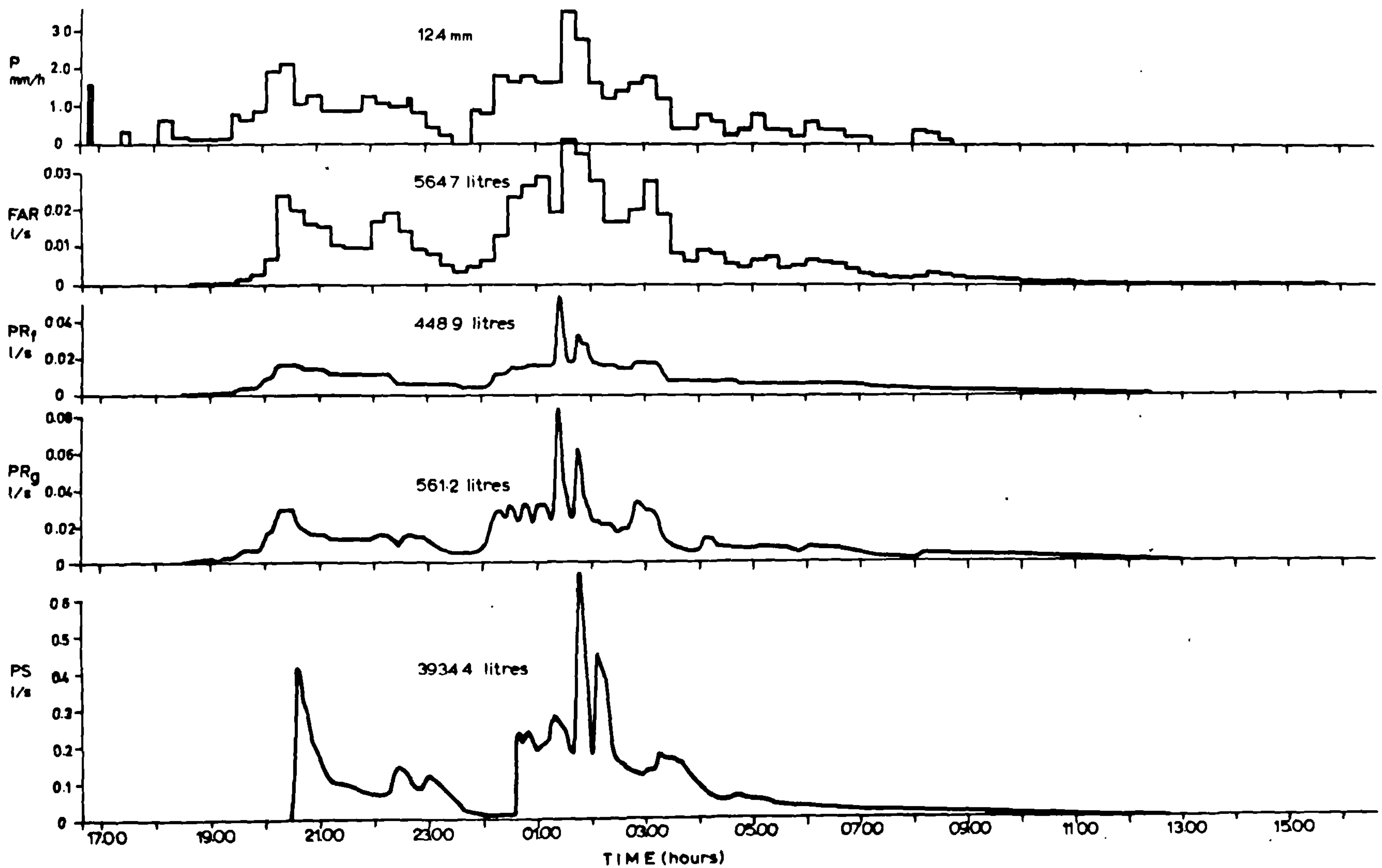


Fig 6.21 STORM 55 7/12/79

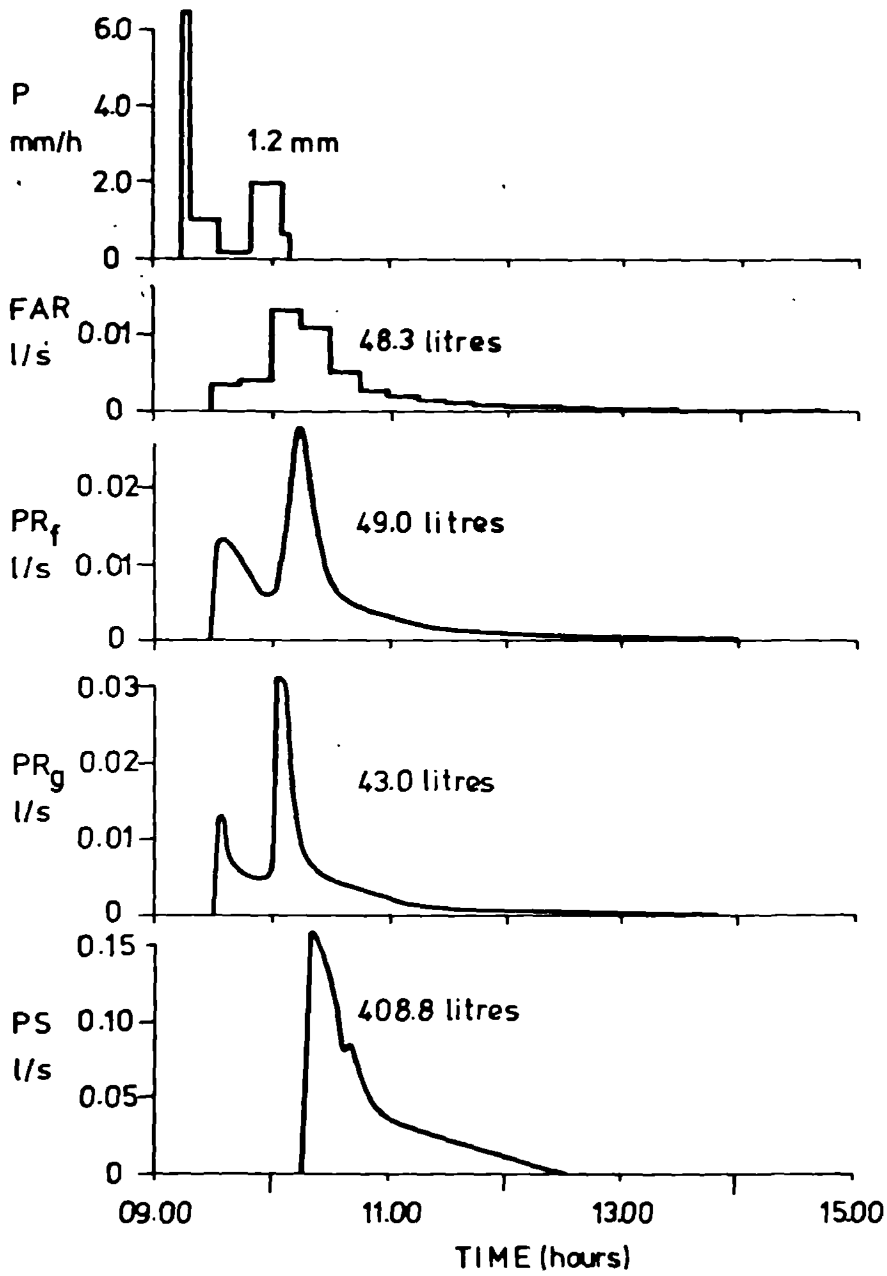


Fig. 6.22

STORMS 62-65 10-11/12/79

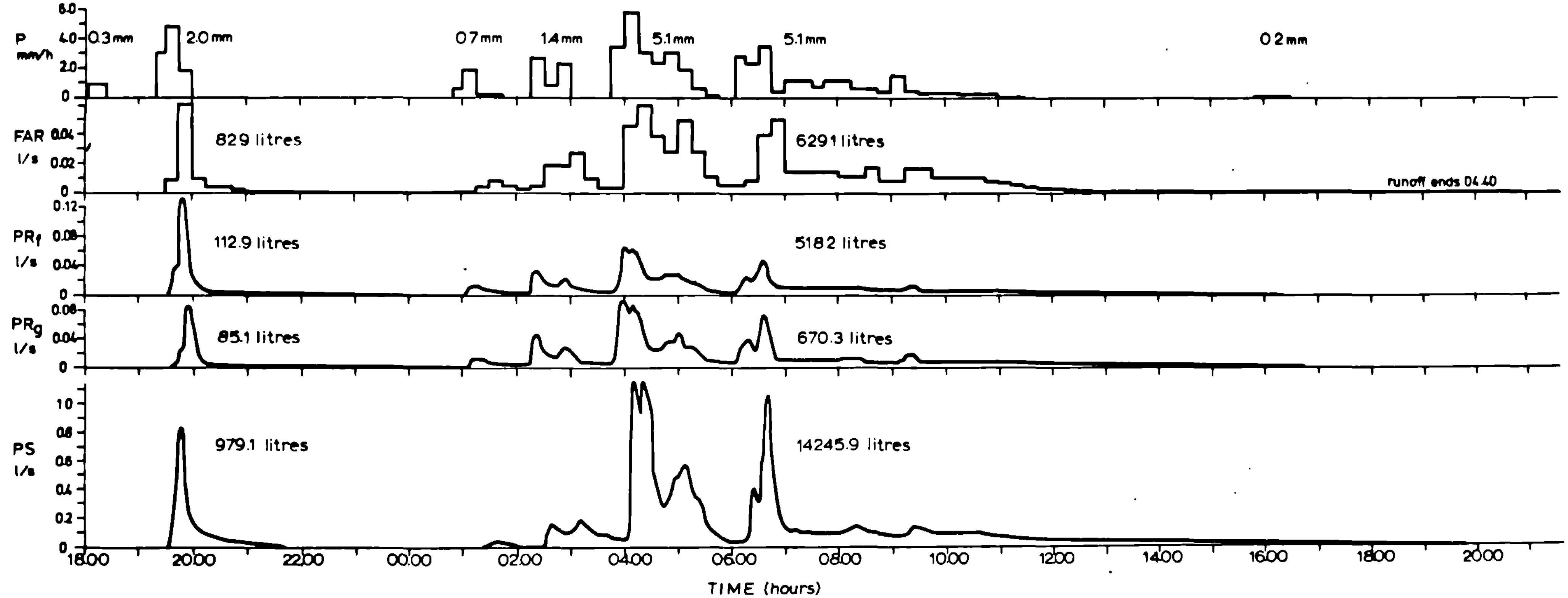
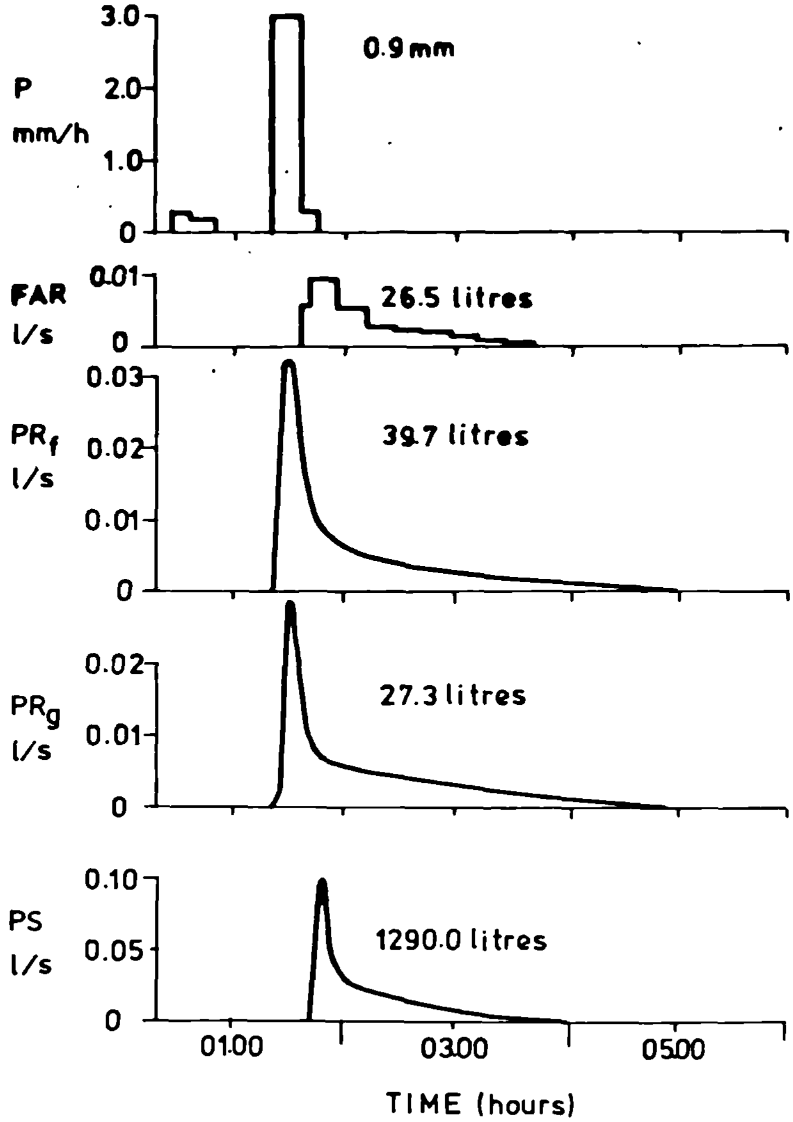


Fig 6.23 STORM 86 29/12/79



The weir tanks prove very sensitive to even slight variations in rainfall intensity and produce very peaked pitched roof runoff hydrographs. The paved surface runoff hydrographs are very similar in form to those from the roofs despite a slightly longer response time caused by the larger supplying area and pipe storage. Within the limits of the record time-step, Figs 6.15, 6.18. 6.20 and 6.22 amply illustrate the sensitivity of all the surfaces to very variable rainfall intensities. The garage roof responds to even minor falls of rain especially when they occur while runoff from a previous storm is finishing. Showers or drizzle can produce a slight increase in the flow rate of the recession limb eg Figs 6.9, 6.11, 6.13, 6.20 and 6.22. In some cases slight increases in runoff from the pitched roofs are also recorded but the true flow rates are unknown because of the difficulty of rating these extremely low flows. Despite the faster flow rates possible from pitched roofs, these extra bursts of rain often did not produce any extra runoff, and never produced any from the paved surfaces.

Volumes of runoff from the roofs are very similar. From these 15 multiple events, runoff from the flat roof (once corrected for flow rate and area) is greater than that from the pitched roofs in the majority of events. The actual volume % underestimate caused by errors in the tipping bucket flowmeter at high flow rates is on average 2½ times less than the maximum volume error % estimate calculated only from the peak flow rate (Table 6.14).

These new volumes on Figs 6.9 – 6.23 are representative of the range of flow rates from the flat roof and therefore the percentage runoff figures would be above those from the pitched roof for most of the 3 month record.

Table 6.14 FLAT ROOF TIPPING BUCKET VOLUME CORRECTIONS

STORM NO.	Maximum Volume Error % Underestimate	Actual Volume Underestimate %
10 & 11	4.3	1.7
12	3.7	1.3
22 & 23	12.5	4.9
24	8.6	4.3
25	17.5	7.6
30	15.4	10.4
33	17.0	5.3
34	5.3	2.6
35	6.9	2.3
36	12.2	4.2
37	12.5	3.8
48	17.8	6.9
55	7.4	2.6
62 - 65	28.1	11.0
86	4.3	1.4

The response times of the surfaces provide another rough measure for depression storage and any other initial losses. From storms with dry antecedent conditions, runoff from the pitched roofs starts first followed by the flat roof and then the roads. Storm 12, (Fig 6.10) is perhaps the ideal storm with 1 mm of rain falling with a constant intensity of 1 mm/hr. The delay to start of runoff in minutes can be converted to the amount of rain that has fallen before runoff starts. From the pitched roofs, a time delay of 25 mins. is equivalent to 0.42 mm of rain used to fill depression storage and initial wetting while 0.50 mm of rain is needed before runoff starts from the flat roof and 0.83 mm for the paved surfaces. These figures are slightly greater than the regression-calculated depression storage, which is strictly the volume of water stored on the surface after the end of runoff, as a certain amount of water is needed to initially wet the surface and overcome any surface tension effects.

It is possible to calculate loss rates from the different surfaces during a storm by comparing periods of steady state rainfall and runoff, 3 events proved suitable - storms 10 and 11, 12 and 30 (Figs 6.9, 6.10 and 6.14.) The loss rate as a % can be calculated from the difference between the rainfall intensity, reworked as a flow rate over each surface area, and the recorded flow rate (Table 6.15). No steady state of runoff was recorded from the paved surfaces for these events.

Table 6.15 LOSS RATES FROM ROOFS

Storm No.	Steady State Rainfall Intensity mm/hr	Steady State Rainfall Intensity l/s	Flat Asphalt Roof Runoff l/s	Pitched Roof Runoff Front l/s	Pitched Roof Runoff Garage l/s	Loss Rate %
10 & 11	0.743					
		0.0100	0.0095			5
		0.011		0.009		18
		0.011			0.008	27
12	1.0					
		0.0135	0.0078			42
		0.015		0.008		47
		0.015			0.017	+ 13
30	2.8					
		0.0378	0.0339			10
		0.042		0.035		17
		0.043			0.038	12

Assuming the same meteorological conditions operate over both types of roof during the storm, any differences in loss rate must be accounted for by differences in the evaporation rate (caused by varying wind speeds and different reflection coefficients) and losses from the roof edges (caused by different slopes and banked side edges in the case of the flat roof). There is no obvious pattern of losses from Table 6.15. Logically one would expect higher loss rates from flat roofs if evaporation is the major control, but this is not the case. With more steady state storms a general trend may appear. 2 of these 3 storms depend heavily on the accuracy with which flow rate is measured to the 3rd decimal point making any conclusions tentative. In one storm (No.12),a loss rate is even converted to a gain. If this figure is ignored,losses from the pitched roofs are fractionally higher and this agrees with previous conclusions drawn from the total data set supporting the greater efficiency of the flat roof asphalt in removing runoff towards the gutter.

The runoff duration from these 15 figures is representative of the mixed response from all the recorded storms with flat roof runoff generally lasting longer than the pitched roofs which lasts longer than the paved surfaces. This rule does not hold in all events. Exceptionally long periods of runoff from the flat roofs are illustrated by Figs. 6.17, 6.13, 6.22, 6.18 and 6.19. Runoff between these last 2 events never actually ceases for the 10.20 hrs dry and runoff from Fig 6.13 lasts for 25.25 hrs after the end of rainfall. Figs 6.13 and 6.22 have small showers following the main rainfall, and runoff from the flat roof never ceases between them unlike the other surfaces. Figs 6.9, 6.15, 6.16 and 6.23, show pitched roof runoff lasting marginally longer than flat roof runoff, but never by much more than 1 hour. Runoff from storm 48 (Fig 6.20) ends at approximately the same time from all the surfaces perhaps because of road surface saturation so that it can respond in similar fashion to the impermeable roofs. From Figs 6.22, 6.12 and 6.18, road runoff ends after that from the pitched roofs and because of either heavy rainfall or short duration between one storm and the next it is likely that the road's infiltration capacity is reduced. Three brief storms with dry antecedent conditions, Figs 6.10, 6.21 and 6.23 are perhaps the best examples of the different responses of the surfaces (summarised in Table 6.16). All 3 have $\backsim$ 1.00 mm of rainfall and short rainfall duration of $\backsim$ 1 hour, but their peak rainfall intensity varies between 1.0, 8.8 and 6.3 mm/hr respectively which may account for the varying durations between storms.

Table 6.16 DURATION OF RUNOFF FROM 3 "UNIT" STORMS

Storm No	Rain Duration (hrs min)	FLAT ROOF	PITCHED ROOFS FRONT	PITCHED ROOFS GARAGE	ROADS	Peak Rainfall Intensity mm/hr
12	1.00	12.00	7.35	7.40	4.35	1.0
55	0.50	5.15	4.30	4.15	2.15	8.8
86	1.20	2.20	3.50	3.35	2.15	6.3

The 2 storms with short and sharp bursts of rain (No's 55 and 86) produce shorter duration storms than 1 steady low intensity storm (No 12), which also produces the greatest differences in runoff duration between the surfaces.

CONCLUSIONS

Overall average % runoff from the surfaces for 3 autumn-winter months varied from 76% for the pitched and flat asphalt roofs to only 17% from the paved surfaces. Monthly % runoff was found to increase from October to December as a result of reduced evaporation.

An investigation of losses from the surfaces according to their antecedent conditions revealed that minimum losses from the flat roof would give a maximum runoff of 98.7%, 93.45% from the pitched roof and only 30% from the roads. These additional losses from the roads were accounted for by infiltration. Antecedent conditions were found to have the greatest effect on the paved surfaces with % runoff doubling from dry to wet preceding conditions which would be consistent with reduced infiltration into the road caused by a saturated subsurface.

The lower slope of the flat garage roof produced a major difference in flow rates from the 2 roof types and greatly reduced high rainfall intensities by as much as 7 times for rainfalls over 10.0 mm/hr. Middle intensity rainfalls of 1.5 to 10.0 mm/hr were only very slightly reduced while low intensity storms produced very similar flow rates from the roofs. Peak flow rates from the paved surfaces were very much less than predicted by design techniques and were only 10 times greater than those from the roofs from an area 70 times larger.

Runoff lasted in general longest from the flat roof which was consistent with the high % runoff figures from the low peak flows. Pitched roof runoff duration was almost as long. Runoff from the roads was of shortest duration except where preceding conditions had reduced road surface permeability.

Separation of the losses from rainfall into some of the components was achieved by derivation of depression storage and a water balance technique. Depression storage was calculated as 0.37 mm for the flat roof, 0.25 mm averaged for both pitched roof sides and 0.02 mm for the paved surfaces. As this last value was considered too small for this 1.37% sloping road an alternative technique for estimating depression storage based on an examination of low runoff-producing rainfalls for each surface was developed. This gave depression storage values of 0.25 mm for the 2 roof types and 1.0 mm for the paved surfaces. These latter values were used in a water balance equation for each surface. Rainfall, unaccounted for by runoff and depression storage from the roofs, was attributed to evaporation. Using this average evaporation rate and applying it to the roads, the volume of water lost through infiltration could be calculated as the residual in the water balance. Infiltration occurred through the many cracks apparent particularly around the kerbs and was aided by the highly permeable chalk subsoil. Of the 83% average loss from rainfall from the roads, 36% was accounted for by infiltration while depression storage and direct evaporation took almost equal amounts of the remaining losses (26% and 21% respectively). Evaporation from the roofs removed 19% of the incoming rainfall leaving 5% for depression storage and 76% as runoff.

The apparently equal volumes of runoff from the pitched and flat roofs may be misleading as the runoff from the flat roof was uncorrected for high tipping-bucket rate errors. By fully analysing the runoff hydrographs from 20 selected storms, it became clear that the % runoff from the flat roof was slightly greater than from the pitched roofs for the majority of the storms. The flat roof is perhaps a more efficient collecter and remover of runoff toward the gutter allowing fewer losses from the roof edges than the roof tiles. Alternatively the rainfall caught by the pitched roofs may be less than that recorded by the ground level raingauge due to roof level turbulence and eddying which could account for the apparently lower runoff coefficient.

By scaling up the roof runoff to apply to the total catchment roof area and assuming all runoff drained into the storm runoff sewers it was possible to calculate the relative contribution of each surface. The roof areas while only making up 44% of the impermeable surfaces would provide 78% of the total runoff from this catchment. Thus while antecedent conditions and prevailing meteorological conditions have an important influence on the amount of runoff from each surface, the greatest influence on the total amount of runoff is the surface itself.

Chapter 7 MICROHYDROLOGICAL PROCESS STUDIES

INTRODUCTION

In order to extend the field data derived from the catchment site at Redbourn, several small scale experiments were devised to further test the runoff coefficients from a fuller range of different roof types and to determine the size of road infiltration. Ideally the range of materials tested on runoff plots or lysimeters should include those most commonly found in urban areas. The possibility of testing road materials such as macadam, rolled asphalt and brushed concrete was considered, but the technical difficulties of either making a suitably sized and adequately rolled block or obtaining a large enough sample from road excavations rendered this impractical. Therefore the sample runoff plots were limited to 3 of the most typical roofing materials - smooth and granular tiles set at different pitches and roofing felt. In addition an asphalt-and-chippings porch roof was monitored at the catchment site. A section of the road at the catchment site was used for fuller investigation of paved surface infiltration through irrigated infiltration plot tests. These process studies should provide additional information that complements the results obtained from the catchment so that a more widely applicable model of the urban rainfall-runoff process can be developed.

RUNOFF PLOTS

Experimental work on the urban rainfall-runoff process has relied on test rigs with simulated rainfall. The nozzles available for rainfall simulators can only successfully reproduce higher intensity rainfalls (Thorpe 1974) typically over 10 mm/hr. Because of this limitation and the time and money required to develop a good laboratory test rig with a rainfall simulator, it was decided to use natural meteorological conditions for the small scale roof runoff experiments. This would also make the data more comparable with the catchment field data.

An early urban runoff experiment was set up by the Road Research Laboratory (TRRL Report LR 236, 1968) which investigated the depth of rainwater on road surfaces during steady simulated rainfall. A large tilting platform 11 x 5.5m capable of a range of slopes of 0.25% to 4.2% was covered with 2 typical motorway grade surfaces - asphalt with chippings and brushed concrete. The rainfall rate from a simulator was deduced from the rate of rise of the runoff level in the collecting tank and ranged from 10 to 200 mm/hr. During steady state rainfall of approximately 100 mm/hr the brushed concrete retained a greater depth of rainfall as detention storage (4.2 mm compared with 3.5 mm) at the shallower slope (1%) possibly because of the slightly greater surface roughness while at maximum pitch (4.2%) the asphalt with chippings had the greater detention storage depth of 3.1 mm compared with 2.8 mm. Because no independent assessment of the input was made (simulated rainfall rate was deduced from runoff output) calculations of runoff coefficients or any losses from these surfaces cannot be made.

A further experimental data set was obtained from a laboratory catchment at the Imperial College of Science and Technology (Johnston and Wing 1978). The original specification was to investigate the rainfall-runoff process over a range of storm inputs, surfaces (concrete, asphalt, grass and combinations thereof), surface slopes and areas. This programme was curtailed by technical difficulties. A rainfall simulator was used to generate artificial rainfall inputs over 5 different catchment areas (ranging from 18 m^2 to 45 m^2) on 2 different catchment slopes (0.7% and 1.4%) on a concrete surface. The catchments contained a single overland flow segment with a single gutter flow segment down one edge. This data has been archived by the IOH (Makin and Kidd 1979) and used in the development of a kinematic wave model to describe overland flow (Gunst and Kidd 1979).

Work is currently in progress at Trent Polytechnic, Nottingham on a laboratory rig containing specimen roof and road surface panels on which storm rainfall is simulated and the corresponding runoff hydrographs recorded. The aim is to develop a model for overland flows from impervious surfaces in urban catchments using this and field data.

With the aim of keeping this series of urban runoff experiments simple and cheap, 3 roof runoff samples each approximately 1 metre square were set up on the roof of the Geography Department building of University College London in central London using natural meteorological conditions. Although the conditions on top of a 6-storey tower block are not typical or ideal, comparisons between the surfaces operating under identical weather conditions are possible.

Rainfall was measured by 2 standard raingauges positioned at each end of the row of roof-types. The average daily catch was used for comparison with the runoff from the roof samples. Each gauge was set under a metre-square piece of aluminium louvre (as recommended by the IOH, Report 43, 1977) with walled sides to correspond as closely as possible to a ground level raingauge. The mouth of each gauge was exposed horizontally at the same level as the louvre and sited on gravel to reduce splash in or out of the gauge. The louvre was used to reduce the effects of turbulence and eddying on the rainfall catch caused by strong winds at the top of the building. A better design may have been achieved by setting a raingauge at the same angle as each roof sample but there would have been no checks on each raingauge catch (unless 6 gauges were available). The pair of raingauges used give a representative figure for the rainfall caught by the roof samples as they are at least exposed at the same height.

Pan evaporation was read daily from a Casella US Weather Bureau Class A land pan (W5826) supported 150 mm above the ground on a wooden frame. The water level was maintained 50–75 mm below the rim and measured using a hook gauge in a small stilling well. This gave a measure of daily open water evaporation. Evaporation from urban impermeable surfaces could not be calculated from the monitored elements and as yet no technique has been developed to measure it even indirectly. Research being carried out by the Atmosphere Road Surface Interaction Study Group at University College London (Wood 1977 and 1979) has led to the production of a road surface energy balance model using road termperature, road surface wetness, dry and wet bulb air temperatures and windspeed measurements. Fuller development of this model could lead to an independent assessment of evaporation from impermeable urban surfaces. The pan evaporation figures obtained could at least indicate the rate of evaporation and weather conditions and be used to partly explain differences in monthly runoff.

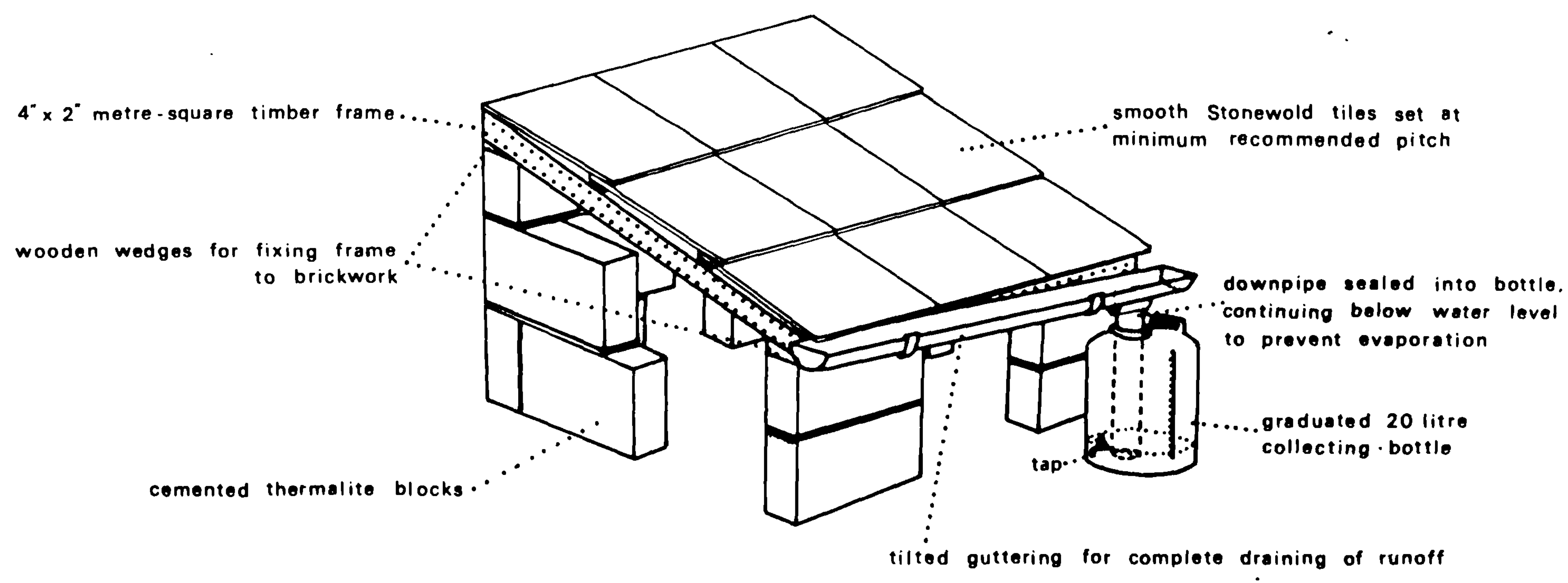
Rainfall was measured at Redbourn using a standard Snowdon raingauge. Pan evaporation, maximum and minimum air temperatures were recorded daily. Further details of this instrumentation and the Meteorological site are given in Chapter 4.

The 3 roof samples were set up to function as runoff plots facing into the prevailing wind. One of the most popular tiles – Redland 49, a red granular tile – was used and set at its minimum recommended pitch of 30^o. Smooth, grey Stonewold tiles which are larger and have a lower pitch requirement of $17\frac{1}{2}^o$ and asphalt roofing felt (set at a slope of 1 in 50) were also used. Fig 7.1 illustrates the construction used for the Stonewold tiles which is typical of that used for the other 2 roofing materials. Guttering attached to the timber frame drained the runoff into a calibrated plastic bottle which could be emptied using a tap. The volume of runoff caught was read off on a daily basis along with the rainfall and evaporation pan.

To estimate the size of bottle required, a design rainfall with a return period of 5 years lasting 3 days was used – the 72 hr M5 from the Flood Studies Report (Vol II, NERC 1975). This gave a runoff of 56.8 litres from a 1 m^2 area, which would require a container approximately half the size of a typical water butt. For small rainfalls, the increase of water depth in this size container from this area of runoff would be negligible. So for greater accuracy in measurement, a smaller bottle of 20 litres capacity was used and in the event proved quite adequate. The possibility of evaporation from the bottle was eliminated by sealing the downpipe into the neck of the bottle with bath caulk and continuing the pipe down to the water remaining below the level of the tap outlet thus greatly reducing the exposed surface area.

The asphalt-and-chippings porch roof area of 4.79 m^2 drained into a calibrated 100 litre water butt which was read daily and emptied by a tap when necessary. The runoff was measured from October to December 1979.

Fig. 7.1 ROOF–RUNOFF SAMPLE MEASURING SYSTEM



The Redland 49 and Stonewold tile plots operated during June 1979 and from mid October 1979 to mid March 1980 while the asphalt roofing felt sample was not installed until December 1979.

As a check on the performance of the College roof raingauges and evaporation pan, precipitation figures from the nearby London Weather Centre and pan figures from Kew Gardens (the nearest pan site) were obtained, (Table 7.1).

RESULTS

The catches of the London Weather Centre and College raingauges are very different but with approximately the same difference for the complete months of operation. This disparity could be because of the difference in height of the 2 buildings - the London Weather Centre measures rainfall twice a day from a check gauge 54 metres above street level while the College building about ¾ mile distant is only 21 metres high, also the London Weather Centre gauge is sited on boards not gravel and is not protected by louvres. The 2 raingauges operating on the College roof gave very similar catches and a paired sample Student's t-test of the discrepancies revealed that the arithmetic means of the total catches did not differ significantly and the agreement between rainfall catches at the 95% confidence level was satisfactory. This roof rainfall can therefore be taken as representative of that caught by the roof samples.

Mean Catch (mm)		Difference (mm)
Gauge 1	Gauge 2	95% confidence level
9.48	9.37	0.11 ± 0.17

No paired sample t-test was attempted on the catches of the London Weather Centre and College raingauges because they were so obviously different and collection times were not always the same to allow comparison of individual catches.

Kew Gardens Climatological Station uses a British Standard Pan which is square and sunk into the ground as opposed to the raised US Class A Pan used both on the College roof and at Redbourn. Wilson (1974) reports the results of comparative tests carried out by Law who found that the ratio of evaporation from the Class A pan to that from the square British pan ranged between 1.17 and 1.40 with an average of 1.32. Even if this marginal reduction to the pan readings from the roof is carried out, the open water mean daily pan evaporation rate (Table 7.1) is very much higher than the rate at Kew except for March 1980. These exceptional evaporation rates are thought to be caused by the very much windier conditions on the College roof and by warm air escaping through the roof in winter.

Checks on the operation of the raingauge and evaporation pan at Redbourn are detailed in Chapter 5.

Table 7.1 COMPARISONS OF RAINFALL AND PAN EVAPORATION RECORDS, CENTRAL LONDON

Month	College Roof Total Rainfall (mm)	London Weather Centre Total Rainfall (mm)	College Roof Mean Daily Pan Evaporation (mm)	Kew Gardens Mean Daily Pan Evaporation (mm)
June 1979	47.9	37.5	-	2.6
Oct	48.1 *	51.2	-	1.1
Nov	57.4	45.2	1.7	0.4
Dec	83.4	62.6	2.6	0.6
Jan 1980	36.4	22.4	0.8	0.1
Feb	53.4	37.8	0.9	0.5
Mar	23.3 *	52.9	0.8	1.0

* Part-month

Monthly summaries of the rainfall and runoff from each samples total area (in mm equivalent and as a % of rainfall) are given in Table 7.2 for the Collge roof samples and Table 7.3 for the porch roof at Redbourn. Because no continuous runoff record from each surface was made that could be used to separate the daily volume of runoff into individual events, the record includes some small storms that produced only low % runoff. Therefore the record is given for monthly totals which would minimise the effect of the small rainfalls.

Table 7.2 ROOF SAMPLE RUNOFF RESULTS

Month	Total Rainfall (mm)	Redland 49 30° mm	%	Stonewold $17\frac{1}{2}$° mm	%	Asphalt 1 in 50 mm	%
Pitch / Areas $1\ m^2$							
June 1979	47.95	39.9	83	36.5	76		
Oct	23.75 *	19.9	84	17.9	75		
Nov	57.35	57.2	100	48.8	85		
Dec	46.90 *	49.4	105	41.6	89	16.4	35
Jan	11.85 *	9.8	83	8.5	72	2.9	24
Feb	53.40	49.1	92	43.4	81	21.1	40
Mar	23.30	24.3	104	21.7	93	11.1	48
* Part-month							
Σ Oct-Mar	216.55	209.7	97	181.9	84		
Σ Dec-Mar	135.45	132.6	98	115.2	85	51.5	38

Table 7.3 ASPHALT-AND-CHIPPINGS PORCH ROOF RUNOFF, REDBOURN

Month	Total Rainfall mm	Runoff 1 in 50 mm	%	Mean Temp. °C	Mean Daily Pan Evap. (mm)
Pitch / Area $4.79m^2$					
October 1979	76.7	58.6	76	7.4	0.97
Nov	51.6	39.7	77	5.1	0.50
Dec	119.8	118.2†	99	7.8	0.74
Σ	248.1	216.5	87		
† Includes some frost thaw					

The average winter runoff coefficient when the 3 roof samples were all operating (Dec - Mar) gives 98% runoff from the granular Redland 49 tiles, 85% for the smooth Stonewold tiles and only 38% from the roofing felt. The reduction in runoff is presumably caused by the reduction in surface slope allowing greater depression storage and therefore evaporation losses to occur. Despite the smooth texture of the Stonewold tiles, their shallower slope resulted in lower runoff volumes compared to the steeper but rougher Redland tiles. The runoff from June and a fairly mild October is slightly lower at 83% from the Redland 49 and 76% from the Stonewold tiles as a result of warmer air and surface temperatures. January has the lowest runoff values of 83% for the Redland 49, 72% for the Stonewold tiles and 24% for the roofing asphalt, which may be the result of some of the precipitation occurring as sleet or heavy frosts and most of the rain falling in small storms some of which only just satisfied the initial wetting and depression storage requirements of the surfaces and caused low runoff volumes. Runoff totals from November, December and March reach 100% or more from the Redland 49 tiles. The steeper tiles may have intercepted a greater quantity of rain than the horizontal raingauge as they provide a surface onto which obliquely falling rain would hit the surface at angles close to 90°.

The very much lesser runoff from the flat asphalt could be the result of more runoff escaping from the edges of the sample as well as a larger depression storage. With the steeper tiles, runoff has only one major direction of flow but with the gently inclined roofing felt some crossfall flow can develop as shorter flow routes to the edges may have the same slope as the overall block. When roofing felt is used on housing, the back and side edges of the roof are normally built-up to prevent runoff escaping other than into the gutter but this was not duplicated on the sample block. Therefore the % runoff figures from the porch roof (Table 7.3) which was made with built-up sides are probably a truer result for this type of surface. The runoff from the porch roof gives % monthly runoff figures of 76, 77 and 99% for October, November and December 1979 which are nearly double those for the asphalt roof sample although recorded from different winter months. The 99% runoff from December includes some thawed frost which is not accurately recorded by standard raingauges and is therefore rather too high a value. The figures for October and November compare favourably with the overall % runoff figure of 76% (Table 6.4) recorded from the block of garages at Redbourn roofed with identical asphalt-and-chippings.

True comparisons between the roof samples and the whole monitored pitched roofs at the catchment site cannot be made because of the different meteorological conditions and different tiles. However, the tiles used on the houses at Redbourn are similar to the grey Stonewold tiles but set at a steeper pitch of 30°. Runoff from these tiles could therefore be expected to exceed that given by the granular Redland 49 tile sample of the same pitch. However the winter 3 monthly average runoff is only 76% instead of over 96% from the Redland 49 tiles for the same 3 months. The rainfall losses are very much greater in practice than would have been predicted from the roof runoff samples. Perhaps with age the tiles have weathered and become more absorbent or greater losses occur from the edges of the tiled roof or runoff overflows from the guttering. More simply, the rain caught at the same level as the roof samples may be more representative than the greater amount caught by a less exposed gauge at ground level.

CONCLUSIONS

Since the conditions are not entirely typical of a residential housing estate where these materials would be used, the runoff volumes recorded from the 3 runoff plots on top of the College roof cannot be used to give standard runoff coefficients for each surface. The weather conditions on the top of the College building are more extreme with stronger winds and warmer temperatures from heat escaping from the building in winter causing higher evaporation rates. These factors should therefore reduce runoff but the overall figures remain very high possibly as a result of a greater catch of rain on the angled surfaces. The differing slopes were the major cause of differences in runoff under identical conditions. 98% runoff was produced by the 30^{o} Redland 49 tiles while the smoother Stonewold tiles at $17\frac{1}{2}^{o}$ produced 13% less runoff. The shallower slope either allowed greater depression storage and absorption despite the smoother texture and more evaporation to occur or greater losses from the edges. This latter effect was the cause of the very low % runoff figures of 38% for the asphalt roofing felt. The asphalt-and-chippings porch roof at Redbourn was made with built-up edges to prevent these losses from the sides and therefore the % runoff figure average of 87% is more accurate for this type of surface when in actual use. The runoff predicted by the tiled roof samples is very much higher than was recorded from a whole pitched roof at Redbourn. The extra losses could be a result of ageing of the tiles or simply the different weather conditions where the rain gauges at the same height as the roof samples more truly reflect the greater amount of rain caught by the roofs than a raingauge set at ground level under recommended Meteorological Office conditions.

ROAD SURFACE INFILTRATION

Attempts to measure the amount of infiltration through the road surface were undertaken at Redbourn. Several experimental methods were considered:-

1. As previously stated, runoff plots of macadam, rolled asphalt and brushed concrete would have been very useful for determining the different depression storages, interceptions and possible

infiltration rates. However, such samples besides being difficult to make were considered unlikely to reproduce real road conditions where cracks have developed along jointed kerb-stones and in the main surface. The minor areas of subsidence for depression storage would also have been difficult to recreate.

2. A road "lysimeter" modelled on the same lines as a turf lysimeter could give measures of the drainage through the road and the amount of evaporation from it. Similar problems to the soil lysimeter would be encountered - how to install a suitable container with the minimum disturbance. An existing piece of road surface would be fractured by attempting to dig it up and would be difficult to seal back into position and a newly rolled block would have the same problems of representativeness as discussed above.

3. A road "infiltrometer" similar to a soil ring-infiltrometer would be a useful instrument if some means of either driving the steel cylinder through the road surface without damage to the equipment or the road or of adequately sealing it to the road could be devised. A constant head of water maintained from a feeder bottle would indicate the infiltration capacity of the road and its subsurface.

4. An irrigated infiltration plot was another alternative. With a known rate of water being sprayed onto a known area and any runoff collected it is possible to calculate the maximum infiltration capacity of a surface.

This technique was adopted as the necessary equipment was already available and no disturbance to the road surface was necessary. The Kent water meter supplied a steady flow of water which when passed through a length of perforated hosepipe gave a very fine spray of water. The road surface in front of a block of garages (an area of 90 m^2) was sprayed and runoff collected by 3 gully pots along 1 side of the road. The road camber ensured drainage into these gully pots, and the Arkon flume at the outfall could record the volume of runoff. Further details and tests on the precision of these instruments are given in Chapters 4 and 5.

Two tests were carried out in May 1980. The first test was carried out on the 6th May on an overcast day. 1470 litres of water was sprayed for 1 hour at a rate of 0.41 l/s equivalent to a rainfall rate of 16.3 mm/hr. This produced 170.9 litres of runoff thus the road (and the pipe) absorbed 1299.1 litres equivalent to an infiltration capacity of 14.43 mm/hr. This rate is slightly less than the infiltrometer results from heavy compacted soils of $\backsim$ 23 mm/hr (Hills 1971). A second test on 12th May was undertaken on warm, sunny and windy conditions. A total of 1721.6 litres was poured onto the road surface for 1 hour giving a rate of 0.48 l/s equivalent to a rainfall rate of 19.1 mm/hr. No runoff reached the gully pots and all water was absorbed into the road surface or evaporated from it. The road infiltration capacity and evaporation rate combined was therefore greater than 19 mm/hr.

These tests confirm that the road surface can absorb large quantities of water but further tests are necessary to give an exact measure for road infiltration capacity. Detailed meteorological measures are necessary to extract the rate of evaporation from these figures.

CONCLUSIONS

These small-scale process studies have provided additional information that could be used in more detailed simulations of the rainfall-runoff process or in an urban water balance model employing a range or combination of urban surfaces.

Chapter 8 CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

This thesis contributes to knowledge about quantitative studies in urban hydrology by working within the context of recommended basic information needs. The most important of these recommendations was the need for more field data acquisition because of its determining influence on model building and increasing knowledge of the processes operating. Water balance models and surveys provide a suitable rational framework for this data collection and for physical process studies while also at the large scale being a satisfactory tool for resource and environmental management. By using a range of respected hydrological techniques and applying them specifically to an urban situation has produced results either contrary to previous work or suggested that some processes have a larger importance than previously suspected. In addition, the development of new measuring systems has allowed thorough investigation of some processes for the first time.

Water balances have been successfully compiled for many rural catchments but they have rarely been calculated for large metropolitan areas, and never for London. In this case, water balances for 5 Greater London Thames tributaries were calculated over different time periods. The least successful balances were achieved for one river using a monthly interval, with, in some cases, the error term being greater than the input rainfall. Annual water balances using measures of rainfall, runoff and evapotranspiration from pervious surfaces were somewhat better but could only explain on average 70% of the incoming precipitation suggesting either the existence of an additional factor (eg paved surface evaporation) or the underestimation of the evapotranspiring area. On first examination, the high percentage runoff figures from the 3 catchments with no official industrial discharge or abstraction seemed to agree well with other workers conclusions eg Lvovich and Chernogayeva (1977) for Moscow, James (1965) for Sacramento Valley, California, and Balek (1978) for Pojbuky, Czechoslovakia, but closer inspection revealed that these high London yields were more likely to be a reflection of unmonitored discharge of imported water through pipe leakage or industrial spillages - this being the only logical explanation for 102% runoff from one river during a drought year. This was further supported by the fact that when net additions by minor sewage treatment works were subtracted from the gauged discharge of the 2 rivers so affected, the resulting runoff coefficients were little different from the regional values for South East England. In addition,the average annual yield from these heavily urbanized catchments was not significantly different from that produced by rural catchments on similar geology experiencing the same climate.

In conclusion, although the sources of error in this study are large, these apparent minimal changes in yield may be a true result for a humid climate (eg Hollis 1977) and care should be exercised if predicting large increases in runoff as a result of urbanization. Measures of the 'natural' cycle - precipitation, runoff and evapotranspiration from pervious areas - are insufficient to produce a satisfactory balance for a metropolitan area as too many unknown and unquantifiable sources of error exist. A more sophisticated analysis along the lines of that produced by Carlsson and Falk (1977) for Swedish urban areas which includes measures for the 'artificial' drainage system is desirable but the necessary data to complete this for the London area is unavailable.

The monitoring of a very small urban catchment is not a new concept but the data collection aims were rather different in this case. An emphasis in previous similar studies has been on the measurement and reproduction of storm peak flows and hydrographs to produce ultimately a sewer design method or simulation model (eg Watkins 1962, Kidd & Lowing 1979). These studies used only the fairly high intensity rainstorms and corresponding peak flows produced by combined road and roof runoff. In this thesis, the emphasis was placed on measuring runoff volume accurately from the whole range of rainstorms regardless of intensity or size as this should give a more complete picture of each urban surface's response to rainfall. New measuring systems allowed separate monitoring of pitched and flat roofs and road runoff. The most important result was that runoff from these 'impermeable' surfaces was less than 100% and average percentage runoff varied from 76% for the pitched and flat roofs to only 17% for the road surface. Road runoff volumes were thought to be exceptionally low both in comparison with the roofs and other work (eg Watkins 1962, Stoneham and Kidd 1977) because of infiltration through into the highly permeable chalk subsoil. This theory was supported by percent runoff doubling from dry to wet antecedent conditions consistent with reduced subsurface infiltration capacity. Storm runoff volumes from the pitched and flat roofs were almost identical despite differences in slope and surface texture. The flat roof did not allow the anticipated greater losses from evaporation and depression storage but if anything produced slightly higher runoff volumes.

This lower runoff from the pitched roof could have been a function of a reduced rainfall catch through roof-level turbulence and eddying in comparison with the ground level reference raingauge. Depression storage for the 3 surfaces was calculated from a linear regression technique (Kidd 1978a) as 0.37 mm for the flat roof, 0.25 mm averaged for the 2 halves of the pitched roof, and 0.02 mm for the road surface. This latter value seemed unacceptably low and was at variance with the value for a 1.37% slope of 0.67 mm from the non-linear regression formula calculated by Kidd (1978b) from several sources of data. A new estimation technique was therefore developed using low rainfalls that did and did-not produce runoff from each surface. This gave values of depression storage of 0.25 mm from pitched and flat roofs and 1.00 mm from the road surface. These depression storage, runoff and rainfall values were used in water balance calculations for the 3 surfaces. As it could be safely assumed that there was no infiltration through roofs, the residual in the roof balances was accounted for by evaporation. Contrary to expectations (Stoneham and Kidd 1977, Van den Berg 1978), evaporation was not a negligible quantity but accounted for 19% of the rainfall. Employing an average roof evaporation rate in the road surface water balance allowed the estimation of infiltration as the residual and largest single component of the water balance - 36%. This high infiltration makes the road runoff of minor significance - 17% of rainfall - and demonstrates the possible importance of chalk subsoil in aiding infiltration. The actual infiltration rate was tested experimentally and found to be in the order of 14 mm/hr.

Further process studies on samples of roof materials exposed under natural meteorological conditions gave further support to the runoff coefficients calculated for the monitored catchments. Average winter runoff coefficients for 3 metre-square roof samples were 98% for granular Redland 49 tiles (30° pitch), 85% for smooth Stonewold tiles ($17\frac{1}{2}^{\circ}$ pitch) and 38% for asphalt roofing felt (1 in 60 pitch). The reduction in runoff was presumed to be caused by the reduction in surface slope allowing greater depression storage and therefore evaporation losses to occur. A very low runoff of 38% was recorded from the asphalt roofing felt sample because of large losses from the edges. A more typical result of 76% was obtained from a porch roof covered with asphalt-and-chippings correctly made with built-up edges monitored at the catchment site. The higher runoff coefficients from these samples in comparison with the complete roofs monitored at the catchment were considered to be because of differences in the reference rainfall measurement. Rainfall was monitored at the same level as the roof samples and would therefore be more representative of the reduced catch on roofs caused by additional high level turbulence than the ground level catchment site gauge with its lesser exposure.

Similar micro-scale studies have been and are being carried out at present (RRL Report LR 236 1968, Johnston and Wing 1978, Trent Polytechnic.) These all rely on laboratory facilities and artificial simulation of rainfall, a technique which can only reproduce the higher intensity storms. The results from any samples cannot be used as standard coefficients because of their non-typical use. Their value lies with comparisons between the samples, and the use of natural meteorological conditions in this instance was perhaps more justifiable as the main research interest was runoff volumes rather than peak flows.

Further Work

This work has developed a useful urban runoff data set with summary statistics that can be used for a variety of purposes. Once digitised it can contribute to the Institute of Hydrology's urban runoff archive with data for urban surfaces on roofs and FSR SOIL type I. It can be used to test and extend existing models such as the IOH developed

Wallingford Urban Subcatchment Model (Kidd and Lowing 1979) which currently uses a 'lumped' modelling approach to simulate the physical processes. The data may indicate the need for a separate treatment of urban catchments underlain by Chalk (SOIL type 1) instead of the current inclusion of all soil groups in the statistical relationship to explain runoff volumes from rainfall (Stoneham and Kidd 1978).

An urban water balance model for a range of urban surfaces could be computed if additional separation of the losses from rainfall is undertaken. The enumeration of such a water balance would relate to the dilution of pollutants in urban streams and the larger volumes of runoff from urban areas once quantified, could prove a useful extra resource in water management and planning decisions.

Fuller explanations of the rainfall-runoff relationship and urban water balance could be achieved by further small scale process studies, in particular, investigations of evaporation and infiltration. The infiltration characteristics of roads, pavements and grass verges could be assessed by ring infiltrometers and as a residual when rainfall, runoff, depression storage and evaporation are known. Evaporation could be assessed by the measurement of water levels in puddles (from rainfall and irrigation) and from the development of a surface heat balance model which uses surface temperature, windspeed and humidity gradient as its main inputs and from the monitoring of surface wetness. Evaporation from urban surfaces could be related to pan evaporation, and a Penman estimate of E_0. A range of urban fabric lysimeters (eg various roof types and road surfaces) with measures for rainfall and runoff could be varied in slope to examine variations in losses with slope and season.

Proposals to cover this range of suggestions have been put forward and the work will be carried out by the Geography Department of University College London in association with the IOH and the University of Birmingham Meteorological Services Unit.

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