ON-SITE REDUCTION AND ATTENUATION OF URBAN STORMWATER RUNOFF

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ON-SITE REDUCTION AND ATTENUATION OF URBAN STORMWATER RUNOFF

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ABSTRACT

The continued construction of impermeable surfaces, such as roads and roofs, has led to overloading and flooding of conventional urban drainage systems by stormwater runoff. Natural recharge of groundwater through infiltration is also limited. One possible solution to such urban drainage problems is to reduce or attenuate all or part of the storm inflow. This may be achieved by either: infiltrating the runoff directly into the ground; or, routing the runoff through a permeable pavement and thence to groundwater or conventional drainage. U.K. field research in this subject has been extremely limited.

The research project reported herein, based at The Nottingham Trent University, evaluated a number of reduction and attenuation structures suitable for 'on-site' use. The structures built were: a $160m^2$ permeable pavement used as a car park; and three infiltration devices which received roof runoff, (a stone-filled soakaway, a dual-chambered soakaway and an infiltration trench). The rainfall, runoff and storage relationships of these structures were monitored for a period of two years.

Real-time observation and testing of the hydrological characteristics of an engineered permeable pavement, surfaced with concrete blocks, demonstrated the structure to be effective at both reducing and attenuating the rainfall hyetograph. 'Runoff' was monitored as discharge from a sealed sub-base 'tank'. Incident rainfall was subject to absorption into the concrete block surface and upon the sub-base stones: evaporation returned these 'rainfall losses' to the atmosphere. Attenuation of the runoff peak was achieved as flow percolated through the device: runoff frequently continued for many hours or days after rainfall. The sub-base stones affected the magnitude of runoff by virtue of their texture and grading: of the stones trialled, blast furnace slag proved particularly effective. 'Runoff' was typically an average of 30% to 50% of rainfall.

Analysis of the monitored data, and the results of several small-scale tests, enabled the generation of a physical/conceptual loss model which demonstrated that rainfall depth was the most significant determinant of runoff volume: rainfall duration was also significant as 'loss processes' were time-dependent. Antecedent conditions controlled the volume of storage available at the beginning of an event and were also demonstrated to be significant in determining runoff volume. Statistical regression analysis indicated that the data set contained strong predictive qualities which may be used for determining runoff parameters. However, the natural rainfall 'input' to the model was generally far from extreme, and care must be taken before the models are used for 'design' events on similar urban surfaces.

Long-term monitoring of roof runoff inflow to, and storage within, infiltration devices has established that such devices may successfully infiltrate runoff in poor, silty-clay, soil conditions. The calculated infiltration rates exceeded those indicated by borehole percolation tests, (commonly used as dimensioning methods). Repeated borehole testing showed large seasonal variations to calculated infiltration rates.

The design, maintenance and philosophy relating to on-site stormwater reduction and attenuation practice for various countries is examined in the light of the experimental data and experience. Wider use of stormwater reduction and attenuation methods are recommended as part of urban stormwater management strategies.

ACKNOWLEDGMENT

I would like to thank several people who have helped towards the completion of the project. Firstly the staff of Nottingham Trent University: Dr Hawkins, Head of the Department of Civil Engineering, for the generous allowance of the use his Departments facilities; and the Departments technical staff, for their help with so many 'little jobs'. Large forbearance being shown by Colin Chambers, Duncan Nash, Steve Goodman and Roy Gregory.

On the home front, I am extremely grateful for the support given by my family, especially during the times when the task seemed never-ending. Thank-you Mother, Father and my wife/editor Jo.

Finally, I owe a large debt of gratitude to my supervisor, Professor Chris Pratt, and my colleague Paul Schofield. Chris, for his expertise, encouragement and cheerful leadership, and Paul for his support and discussion - (but mostly for his drawings of the research kit).

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CHAPTER 1

INTRODUCTION

1.1 Background

Natural drainage systems, such as rivers and streams, evolve through time, they are dynamic in both their volume of flow and their course. Since the industrial revolution there has been an extremely rapid rise in the number of rural catchment areas which have been subject to urbanisation. The resulting growth in impermeable surfaces has not only encroached onto the flood plains of rivers, but also altered their flow regime by the addition of urban stormwater runoff.

The fundamental purpose of urban storm drainage systems is to ensure the rapid removal of rainfall, as runoff, from impermeable surfaces (principally roads and roofs). Stormwater is often conveyed, via 'separate' storm sewer systems, to a local watercourse at its nearest point to the impermeable, drained areas. Alternatively, 'combined' sewerage systems, which normally discharge stormwater and sewage to treatment works, will frequently contain overflow structures which at times of heavy rainfall, also discharge stormwater to a watercourse.

Both separate and combined systems produce the same results with respect to the quantity of runoff discharged to a watercourse: impermeable surfaces produce a large increase in the volume of runoff in comparison to the pre-development natural drainage; the time of concentration, or time interval before the arrival of the peak flow rate, is reduced; and there is an increase in the frequency of peak discharge of any given magnitude. Large urban areas frequently encroach upon the over-bank storage areas to natural drainage systems: the concentration of flows from the urban area can therefore result in flooding, either to the same urban area, or downstream.

Secondary problems are also evident. The natural water cycle is further disrupted by urbanisation as groundwater recharge is reduced. Urban areas tend to grow, either on 'green' spaces within the built-up area, or on the outskirts of the conurbation, in the form of industrial parks or new housing. Thus the original dimensions for the old storm drainage system may become inadequate, and the added total flow can help to reduce the design life of the system.

Until recent years, the answer to downstream flooding and storm sewer overloading has been through structural solutions, often involving large capital works. For example, in 1984 the Water Companies in Britain, (then the Water Authorities), spent over 100 million pounds on dealing with hydraulic overloading of its sewer systems alone, (Fiddes, 1984).

However, in cases of hydraulic overloading, a stormwater runoff strategy aimed at reducing inflow to the upstream drainage system could prove a cost effective solution, extending the operating life of some sewers and reducing peak flows to the natural drainage system downstream. A literature review of inflow reduction techniques by Hydraulic Research Ltd, (commissioned by Water Research Centre, Report EX1049, (Anon. 1982)), surveyed various methods and their effectiveness. The report concluded that,

> 'the most potentially effective means of flow reduction are the disconnection of contributing roof and paved areas, and increased use of permeable pavements and detention storage. It should be noted that the design of a drainage system more in keeping with the natural drainage of the environment may also involve less expenditure on the drainage system...greater use of infiltration, soakaways and permeable areas might lead to a smaller drainage network either in areal extent or in the dimensions of its components. In these ways environmental requirements would coincide'.

No field studies were undertaken as part of the investigation for the report.

Although the limitation of runoff volume and peak discharges is widely considered as a useful goal, several questions remain about the implementation of such a strategy. Foremost amongst these are: by whom should such a strategy be implemented and regulated?; and explicitly, what structures should engineers be building, i.e. how, to what dimensions, and for how long will the structures operate? The main aim of this research was to begin to bridge the gap between these drainage philosophies and the economic, engineering and social requirements for reliable urban storm drainage.

1.2 Terminology

In the field of stormwater infiltration there are three broad categories of structure or method, (although some systems may fall into more than one category) , these are:

A. Plane infiltration, e.g. simple, flat or gently sloping areas, such as grassed lawn, which have little or no storage facility. These areas may require no formal construction, but for design purposes the infiltration rate of the surface should be at least equal to the design rainfall intensity;

B. Flat basin infiltration, e.g. swales or grassed retention basins. These structures are a combined form of above ground detention and infiltration: this implies that the surface is contoured to provide storage.

C. Below ground or pit infiltration, e.g. soakaways and infiltration trenches. These structures usually require some form of excavation followed by re-surfacing upon a porous fill or a constructed chamber, the porosity or cavity provides storage whilst infiltration takes place. Stormwater inflow may be via a drainage pipe, or by overland flow, (entering through the top of the structure).

The various methods of flow reduction can be defined in terms of their effects upon stormwater input to the urban drainage system. Structures can 'attenuate' the discharge hydrograph, or 'reduce' the volume of flow, (or both). Attenuation, within the field of storm drainage, generally refers to the reduction in peak and extension in duration of the runoff hydrograph. Detention ponds can be considered, primarily, as attenuation devices. Structures which aim at a 'reduction' in runoff are designed to prevent, all or part of, the impermeable surface runoff from entering the conventional drainage system. This implies infiltration to groundwater or evaporation of runoff. A soakaway is primarily a 'reduction' device.

1.3 Previous research

The development of infiltration theory dates back to Green and Ampt, (1911), and Horton, (1940). However, the main requirement for the scientific and engineering study of infiltration devices, such as soakaways, lies in the dimensioning of the structure so as not to flood. This subject has not been widely researched, but based on a limited amount of infiltration theory, a number of recommendations for infiltration practice are available. These recommendations, (and their background theory and current practice), are discussed in-depth in relation to the experimental work from this project within Chapter 8.

Porous pavements contain elements of both 'plane infiltration' and 'below ground infiltration', but it is possible that the devices may be built purely as attenuation structures with no facility for infiltration.

The earliest research in the field of porous pavements dates from the 1970s, when the Franklin Institute (U.S. A.) conducted the first extensive study of porous pavement systems, (Anon, 1972). Since that time, research in the United States has continued, (Diniz, 1976, and Go forth et al, 1984), but many of these studies included few field trials and concentrated purely on theoretical analysis of systems. However the latter study by Goforth et al (1984) did show porous asphalts were economic when compared to the long term costs of a conventional highway and associated drainage.

Little research was reported in Europe until Jacobsen and Harremoes (1982) related studies of a semi-pervious surface of granite sets. This was one of the very few projects not using permeable asphalt. During the last 10 years, numerous porous asphalt structures have been built and monitored in Sweden, (Hogland et al, 1987). This work on the 'unit superstructure' was one of the first to raise questions about the clogging of porous asphalt and to propose methods for maintenance. The Swedish project has also commented on the excellent cold weather operation of permeable pavements and has examined the build-up of pollutants within such a structure , (Hogland et al, 1990).

In Japan, several studies have taken place: one of the largest and most comprehensive projects was implemented to overcome flooding problems caused by extreme 'urban density' in the city of Tokyo, (Fujita, 1987). The E.S.S. (Experimental Sewer System) was not, primarily, a research project but a solution to flooding problems, implemented by the Sewerage Bureau of Tokyo Metropolitan Government: aspects of this practice are discussed in Chapter 8.

British research has been extremely limited such that no other field studies of porous pavements are known to the author. In addition to Report EX1049 (Anon, 1982), CIRIA (Construction Industry Research and Information Association) has instigated a further desk study, (Anon, 1992), to provide an overview of the technology, design and legislative framework with respect to stormwater reduction/attenuation systems. Current infiltration practice and available design literature for the U.K. and elsewhere are discussed in Chapter 8 in the light of the observations from this research.

1.4 Research aims

Research from various countries has shown permeable pavements and infiltration practices to be effective methods of controlling runoff as part of an urban stormwater reduction/attenuation strategy. However, the main thrust of previous studies has been towards the use of asphalt/ tarmac surfaces which can be prone to clogging. Also, previous research has produced only limited analysis of a long-term, natural rainfall series upon such structures.

The specific aims of this research were to further the work contained in report Ex1049 (Anon, 1982) by conducting both field and laboratory experiments upon a range of methods designed to reduce and/or attenuate urban stormwater discharges. The study intended to examine aspects of the design and maintenance of structures, such as soakaways, infiltration trenches, and permeable pavements through their construction and operation, rather than by desk study and computer simulation. All of these devices are suitable for 'on-site' drainage, i.e. the attenuation or reduction of runoff at or near the site of runoff generation. The main body of the experimental work was directed towards long term monitoring of the hydrological response, and its variation to measured rainfall, for the different devices.

This research examined a paving block, permeable surface and monitored its performance over 2 years. Infiltration practice in the U.K. was also examined in the light of a series of field trials in poor soil conditions .

1.5 Associated work

This research project was based within the Storm Drainage Research Group of the Department of Civil Engineering of The Nottingham Trent University, (formerly Trent Polytechnic). Many aspects of this research have implications for the quality of urban stormwater runoff. Therefore, in addition to the research on the hydrological quantity performance of the devices constructed for the field work reported here, parallel project examined the quality of all the waters input, stored and output from each system. This ' quality' study had aims distinct from those for this research, but the use of the same structures for both projects necessitated a degree of cooperation for the implementation of monitoring systems and the administration of the projects. In the following Chapters, where necessary, areas of 'co-operation', or influence between the two projects will be referred to as the 'quality project', Schofield (1991).

CHAPTER 2

DESIGN CONSTRUCTION AND MONITORING OF A PERMEABLE PAVEMENT

An engineered permeable pavement is designed to allow all incident waters, either rainfall or rainfall and runoff from nearby surfaces, to immediately percolate through its surface to its lower layers. Permeable surfaces should thus prevent any surface accumulations of water by ponding, provided that the rates of percolation through the surface exceed any likely rates of input.

There are two common surfacing materials: an open-textured macadam, (Hogland et al 1987); and block paving, (Jacobsen and Harremoes, 1981). The former is porous across the whole of its surface by virtue of its uniform grading, which allows pores to remain open whilst keeping the appearance and texture of a conventional, impermeable macadam surface. The latter consists of nominally impervious blocks of granite, concrete or other material, which may be shaped and laid so that gaps between the blocks allow for the passage of water through the surface.

Originally, porous macadam was only laid onto existing surfaces to prevent skidding and aquaplaning by aircraft on runways prone to flooding, (Johnson and White, 1976). More recently both asphalt and porous block paving have been used in more conventional, load bearing, roles in the urban environment, such as in car parks and pedestrianised roads as part of stormwater management strategies.

No U.K. based research into the hydrological characteristics of permeable pavements is known. In order to gain accurate data relating to rainfall-runoff relationships for a permeable pavement, a concrete block surface and hydrological monitoring system was constructed as part of this research. This Chapter explains the design and construction of the major part of the experimental work for this research, a permeable surfaced car park. The last

Section describes how hydrological data were collected and processed. The final part of the design and the construction took place during the last three months of 1986 and data collection began in April 1987.

2.1 Design Notes

In October 1986 The Nottingham Trent University gave permission for an existing area of un-surfaced car park at its Clifton Campus to be used for the construction of a permeable pavement. Once built, the pavement area would resume its former use as a car park. The designated area proved suitable for a strip of surfacing 40 metres long by 5 metres wide: this corresponded to a parking area for 16 cars side by side.

When planning to build a permeable pavement, a major design decision is whether to allow percolating waters to drain naturally through to the underlying sub-grade, or to place the sub-base within an impermeable membrane and provide sub-base drainage. The choice will largely depend on how well draining the sub-grade is, although a moderately well drained sub-grade may warrant a combination of infiltration and sub base drainage. In each case the sub-base performs the tasks of load bearing and water storage when the rates of input are greater than may be drained from the structure or infiltrated to the underlying soil.

The poor draining sub-grade at the Clifton Campus, combined with a research requirement to study the attenuation effects and quality parameters of the permeable pavement upon stormwaters, led to the adoption of a design of a sub-base collection and drainage system. This entailed using an impermeable membrane to separate the percolating waters from the sub-grade beneath the construction, and required the provision of sub-base drainage. A polyvinylchloride (P.V.C.) material used for damp-proof courses was selected as the most hard-wearing material available.

To enable an evaluation of several different types of sub-base stone, four different types of stone were chosen: each to be contained within a separate tank structure. It was thought that the evaluation would prove useful for comparative purposes from both a quantity and qualitative viewpoint. The main considerations were to choose stones and gradings which were readily available and which would provide both adequate drainage and load bearing characteristics. The stones used were gravel, blast furnace slag, granite and limestone. The gradings were chosen so as to include the main types used in drainage constructions. However, the common road sub-base grading, Type 1, conforming to the Department of Transport clause 803, (Anon 1986), was considered to contain too many fine particles which were likely to prevent free drainage.

The final choice of stone type and gradings for the sub-bases were:

Location	Stone	Grading
Bay 1	Gravel	Type A (10mm)
Bay 2	B.F.S.	Type B (clause 505, Anon 1986)
Bay 3	Granite	Type 1X (a type 1 with no fines)
Bay 4	Limestone	Type 1X

The grading 'envelopes' defined by the specifications for Type A, Type B, and Type 1 sub base stones, (Anon, 1986), are shown in Figure 2.1.

The splitting of the study area into four units necessitated the construction of four separate P.V.C. tanks, with each tank, or bay, being drained separately. A single location for all the monitoring equipment also required that the drainage from the higher, uphill bays was laid along the length of the car park beneath any subsequent bays down to the lowest end of the structure. Each sub-base drain entered a 'measuring pit' containing the monitoring and sampling equipment before being discharged to an infiltration trench adjacent to the car park.

To ensure that the sub-base stone remained free from silt, and to screen out gross solids from any effluent, a geotextile (trade name 'Terram'), was used to cover each sub-base stone in the four bays. Geotextiles are designed to allow waters to pass through whilst preventing solid particles, (of size greater than the manufacturers design limit), passing into, and gradually clogging, free-draining materials.

Above the geotextile was placed a bedding layer of gravel upon which were laid the pre-cast concrete blocks. The blocks, (made by EEC Quarries Ltd), were designed for this particular structure. The blocks were oblong in plan, with the corners and part of the middle 'cut away', the dimensions of a block are given in Figure 2.2. When laid in a conventional herringbone pattern, the gaps between blocks account for 15% of the plan surface area, and form a pattern of cylindrical voids, or pores. The blocks also have two raised 'discs' on their uppermost surface, which are designed to bear most of the trafficking, thereby preventing heavy loads from compacting the free draining void spaces. A cross-section of the final design for the car park is shown in Figure 2.3.

2.2 Construction Notes

The excavation of the site of the car park was achieved using a JCB excavator. To a large extent the excavation for the pavement followed the natural fall of the ground. The width of the car park had a fall of 1:100 and the length a fall of 1:40. The composition of the excavated formation varied widely, as the site cut across the foundations of some old buildings. However, where found, the natural formation was weathered silt and clay.

The PVC material used to line the 'tanks', and thus retain the stormwater was cut to size from rolls 4x25 metres in size. The same basic layout for the sub-base drainage was used for each bay: a 110mm diameter perforated pipe placed inside the bay was connected to two right-angled bends joined by a 110-82mm reducer. The drainage from the outside of the bay down to the instrument pit was via 82mm pipe.

To ensure that there were no leakages from the point where the drain from each bay came out through each P.V.C. tank wall, the following system was devised. A 150mm square plastic plate was heat welded on to the reducer (110-82mm), which in turn was connected via a right-angled bend to the perforated pipe. This assemblage, shown in Plate 2.2, was placed inside the P.V.C. tank. The reducer assembly was then pushed through the P.V.C. at the lowest corner of the tank, and a second plate was offered up from the outside of the tank and bolted onto the first plate. The in-situ assemblage is shown in Plate 2.3, (this assemblage was repeated for all four bays).

The final tonnages and types of stone placed in each bay were:

Bay 1:	4 tonnes of 20mm rounded gravel placed around the perforated pipe and 19.5 tonnes of 10mm rounded gravel, (Type A);
Bay 2:	20.2 Tonnes of Type B, blast furnace slag;
Bay 3:	23.0 Tonnes of Type 1X (5-40mm) granite; and,
Bay 4:	4.0 Tonnes of 50mm dolomitic limestone, and 19.3 Tonnes of Type 1X (5-40mm) carboniferous limestone.

When the sub-base stones were delivered, they were not tipped directly into each bay because of the danger of puncturing the P.V.C. membrane. Instead, the stone was tipped to one side and the JCB was used to gently tip the stone into place, (Plate 2.4). The excavator arm of the JCB was used to roughly level each sub base: finally, hand shovels and rakes were used to obtain a well graded surface across all four bays in preparation for compaction by a vibrating plate whacker.

The depth of sub-base ranged from 275-350 mm across the graded formation. Plate 2.5 shows all the sub base stones in place: limestone in the foreground; followed by granite; blast furnace slag; and gravel in the distance.

The sides of the P.V.C. tanks were folded over, level with the top of each sub-base. Kerbstones (150mmx150mm) and concrete 'haunching' were built around the of the whole structure's edge, thus sealing the sides against ingress of water.

The geotextile was rolled out over the surface of all the levelled sub-bases. Approximately 18 tonnes of 2-6mm crushed gravel was then placed on the geotextile along the whole length of the car park, giving a depth of about 80mm as a bedding layer for the concrete blocks. The gravel was roughly levelled with a shovel, then pulled to level with a plank, using the kerbstones as a guide.

The concrete blocks were laid directly onto the levelled gravel in a conventional herringbone pattern. The excess gravel resulting from the levelling was thrown back onto the newly laid blocks to fill the gaps between them: this helped to 'lock' the surface together, aiding stability. Plate 2.6 shows the process of laying the blocks taking place. When completed, the whole surface was compacted using the plate vibrator.

Finally, individual car parking spaces were painted onto the surface, four to each sub-base stone type, 16 in total. A line representing the division between each bay was painted right across the car park to discourage drivers from parking across the division. The painting of spaces on the surface also ensured, (to a degree), that the same parts of each parking space were being trafficked each time: this will be shown to be important from a structural viewpoint later.

Plate 2.7 shows the pavement soon after construction. Before it was open to use, the surface was surveyed so that accurate monitoring of any movement of the blocks due to loading could be made.

The initial survey, and the results of follow-up surveys, are covered in Section 5.1. The surface area for the whole car park was 159.54 square metres, that is the total of those areas deemed to contribute incident rainfall to the permeable surface.

The areas for each individual bay were calculated as:

1. Gravel = 40.30 sq. metres;
2. B.F.S. = 41.97 sq. metres;
3. Granite = 40.42 sq. metres; and,
4. Limestone = 36.85 sq. metres * .

* Footnote. Although the limestone sub-base area was constructed to be the same area as the rest, approximately 4 sq.metres was not covered with permeable paving so that sub-base temperature thermistors could be installed and remain accessible. The unused portion of sub-base was covered instead with P.V.C. and soil.

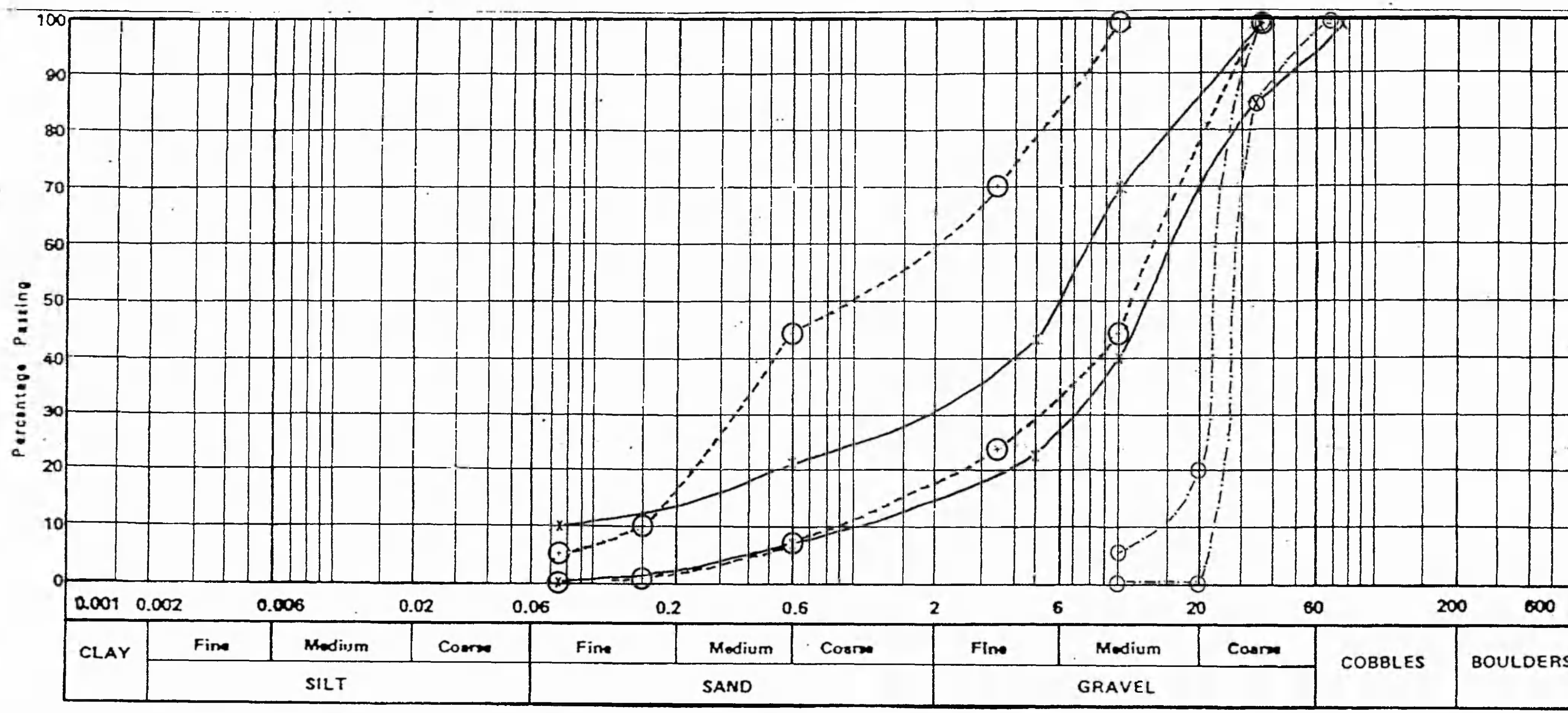


Figure 2.1 Specifications for stone gradings as used for sub-bases.

Type B
Type 1X
Type A

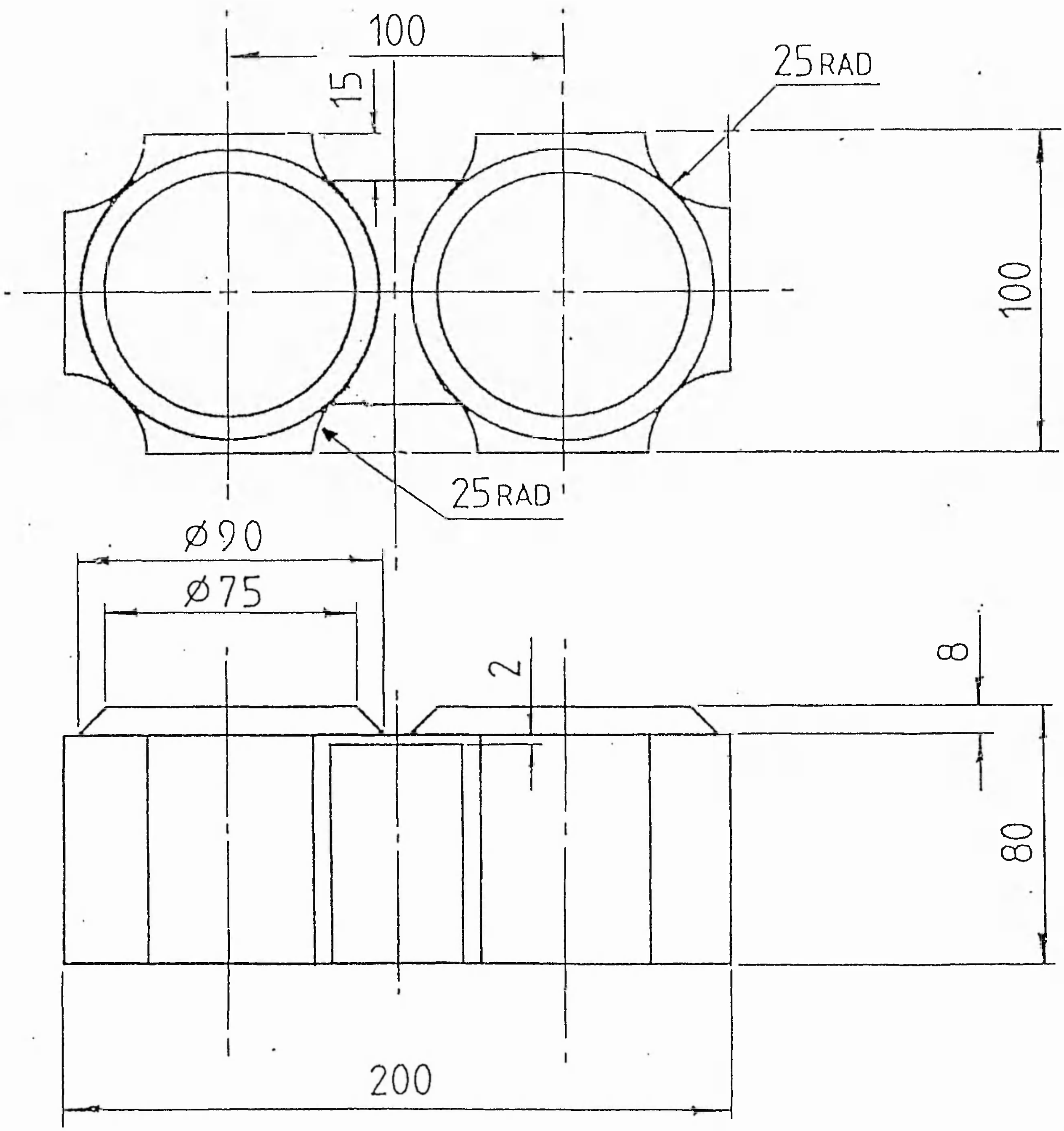


Figure 2.2 Design of concrete block used for surfacing the permeable pavement.

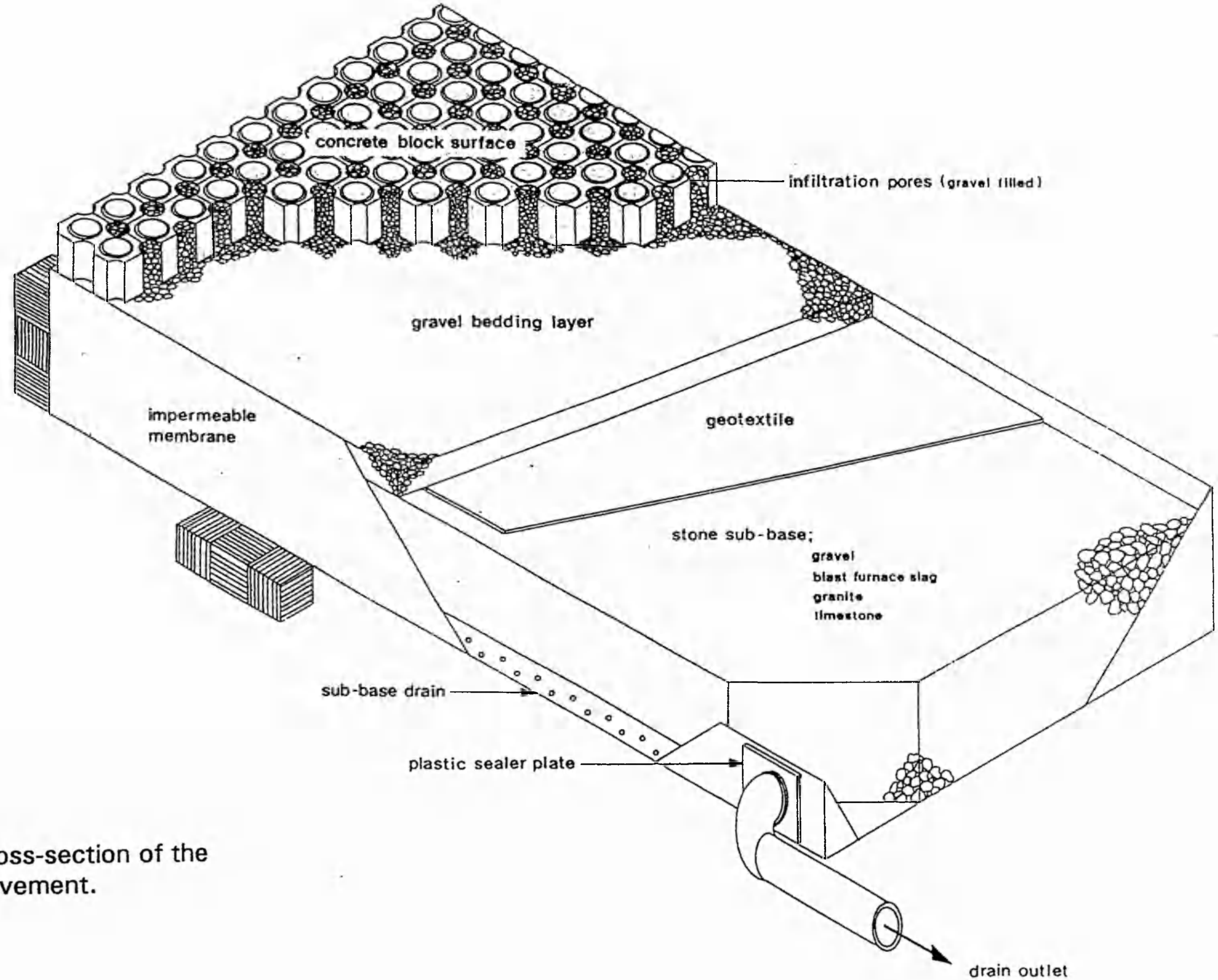


Figure 2.3 Schematic cross-section of the permeable pavement.

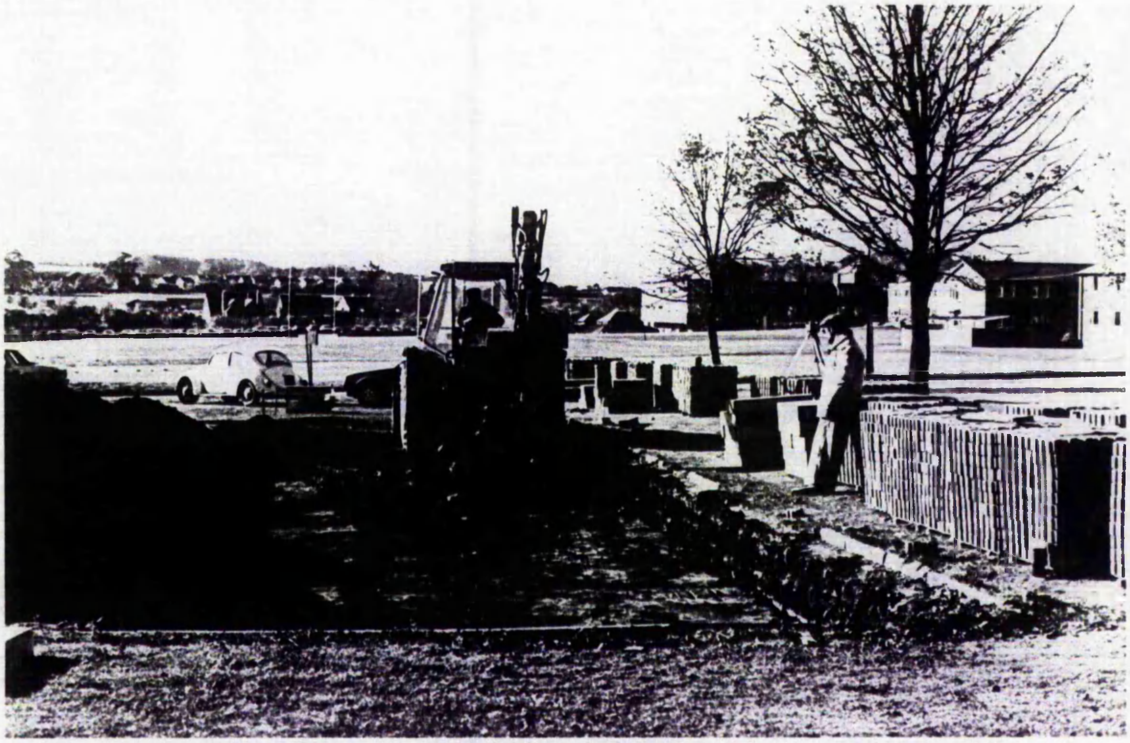


Plate 2.1 Excavation for the car park at Clifton Campus

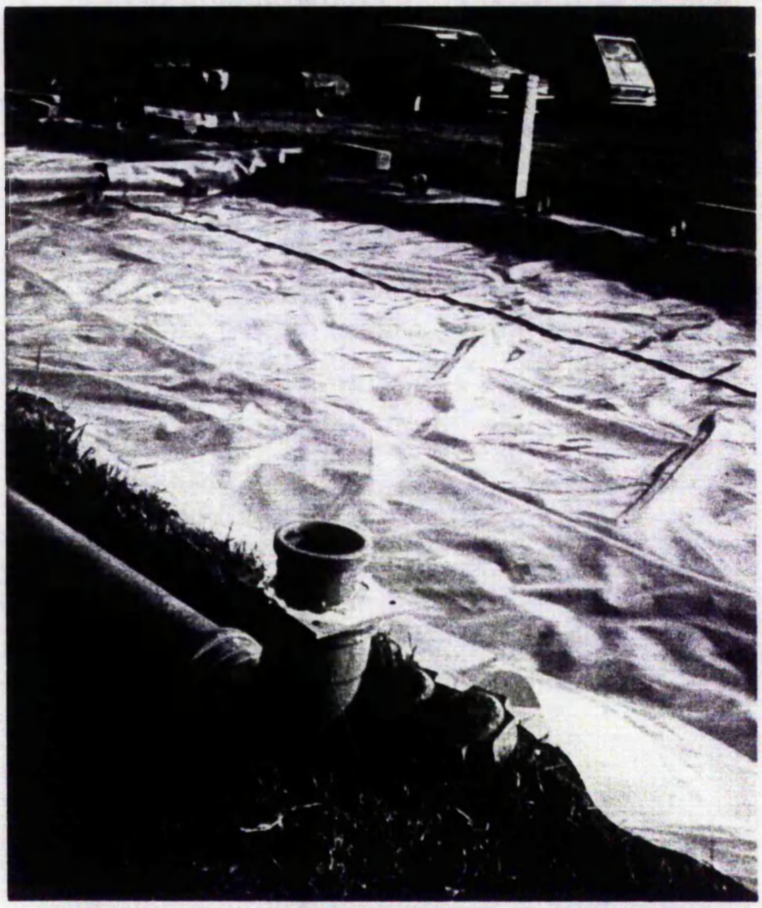


Plate 2.2 Arrangement of perforated pipe, reducer and plastic plates before installation.

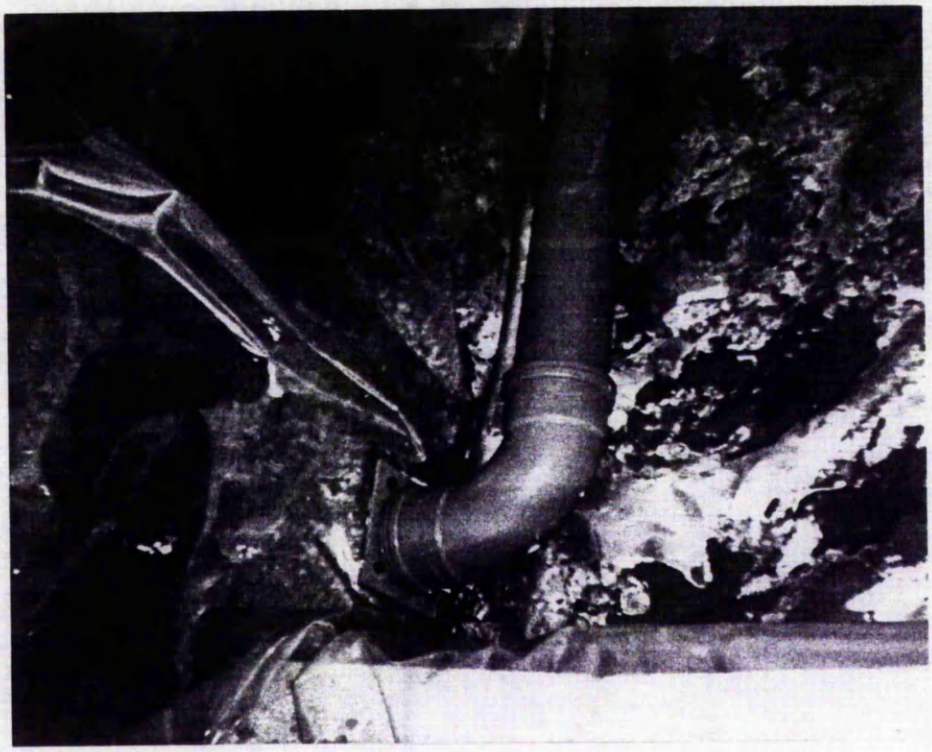


Plate 2.3 In-situ sub-base drainage.

Plate 2.4 JCB emplacing sub-base stone.

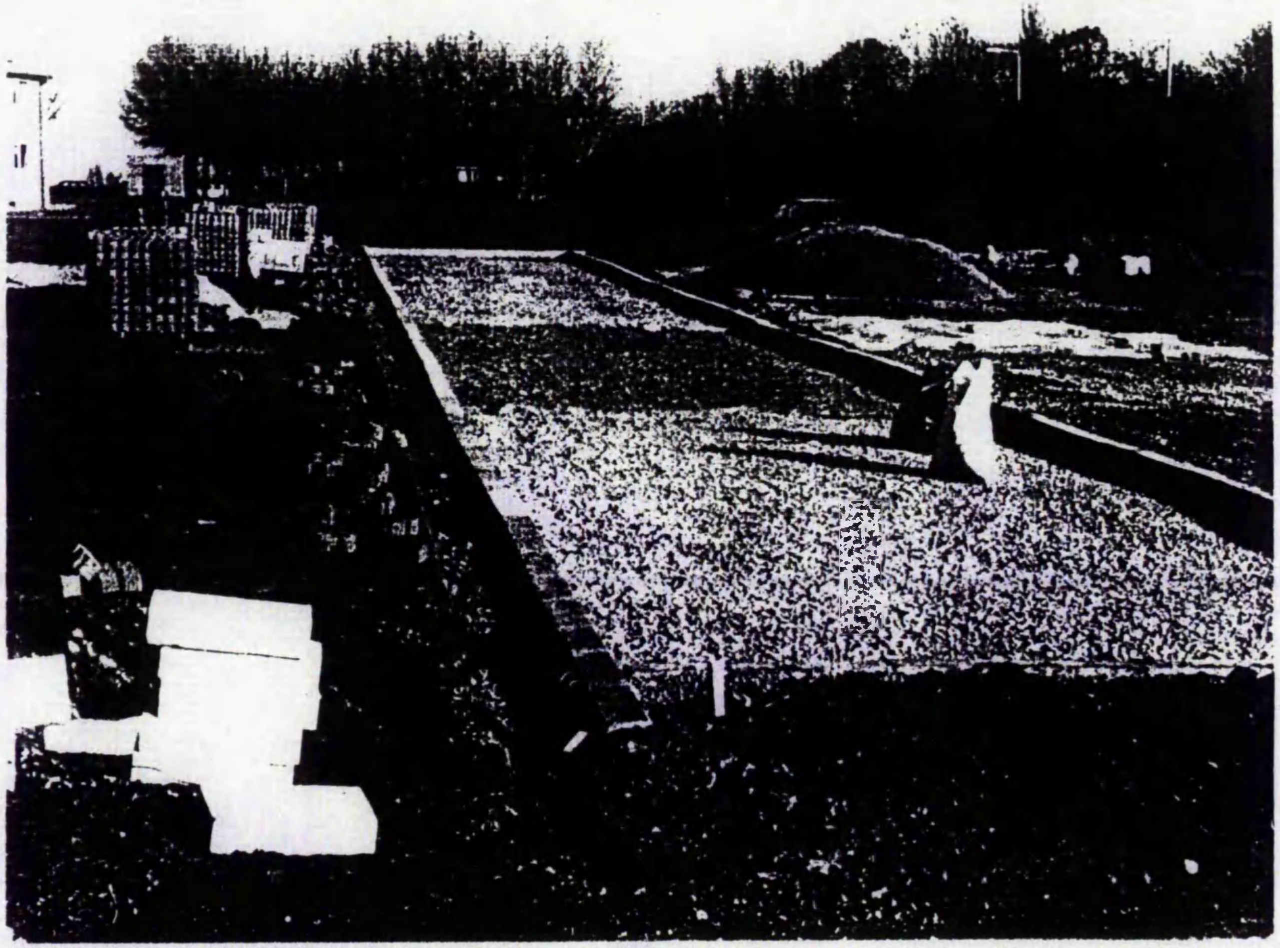


Plate 2.5 The four sub-base stones in place. Limestone in the foreground, followed by granite, b.f.s. and gravel.

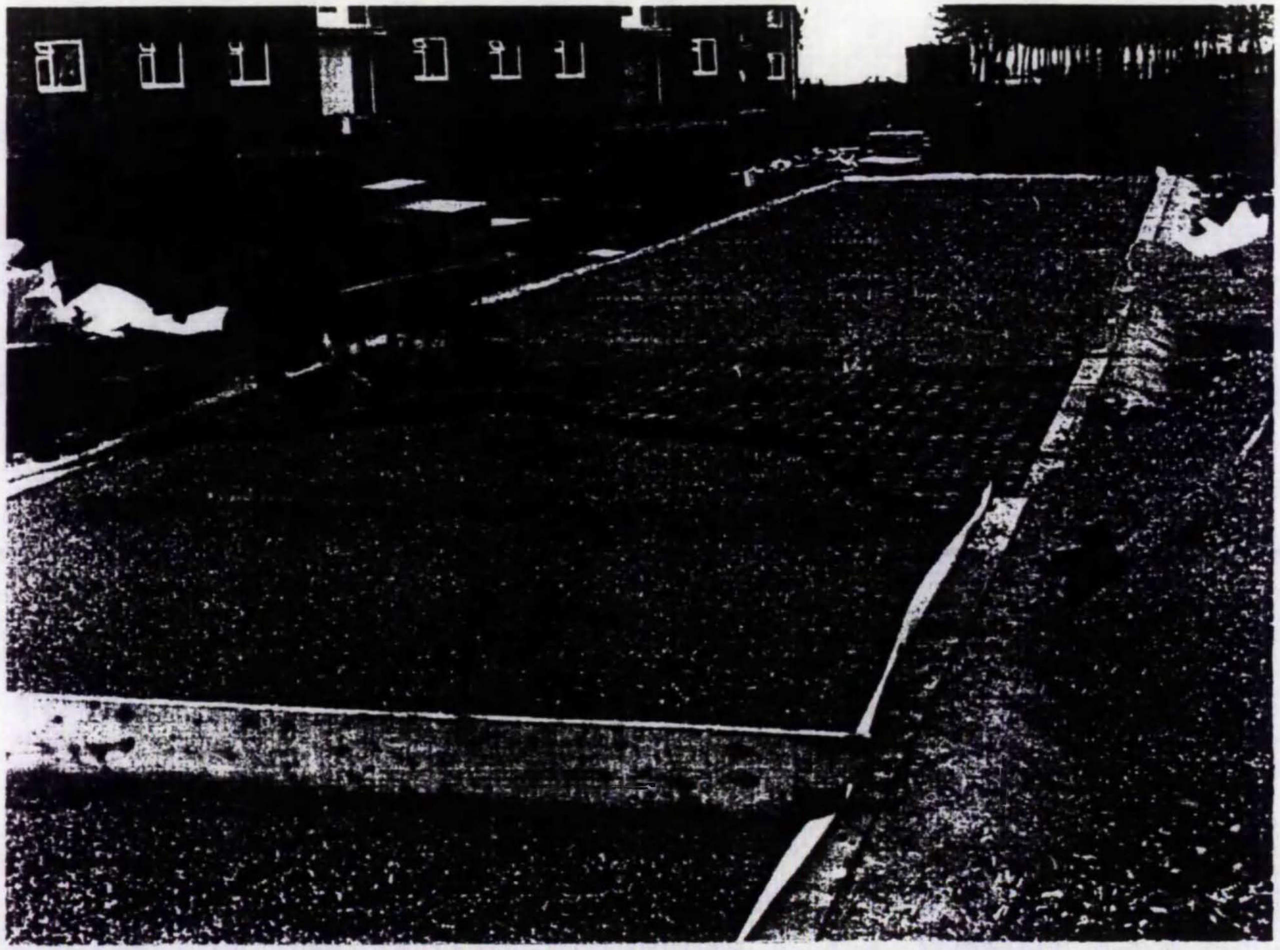


Plate 2.6 Laying of concrete blocks upon gravel bedding layer.

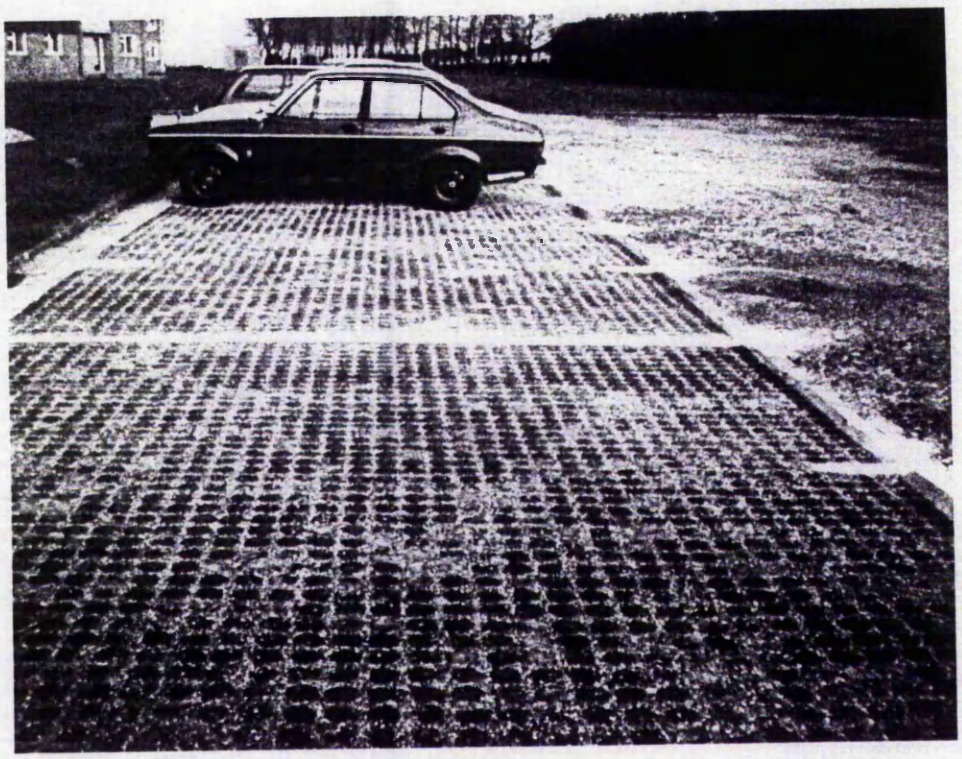


Plate 2.7 Completed pavement soon after construction.

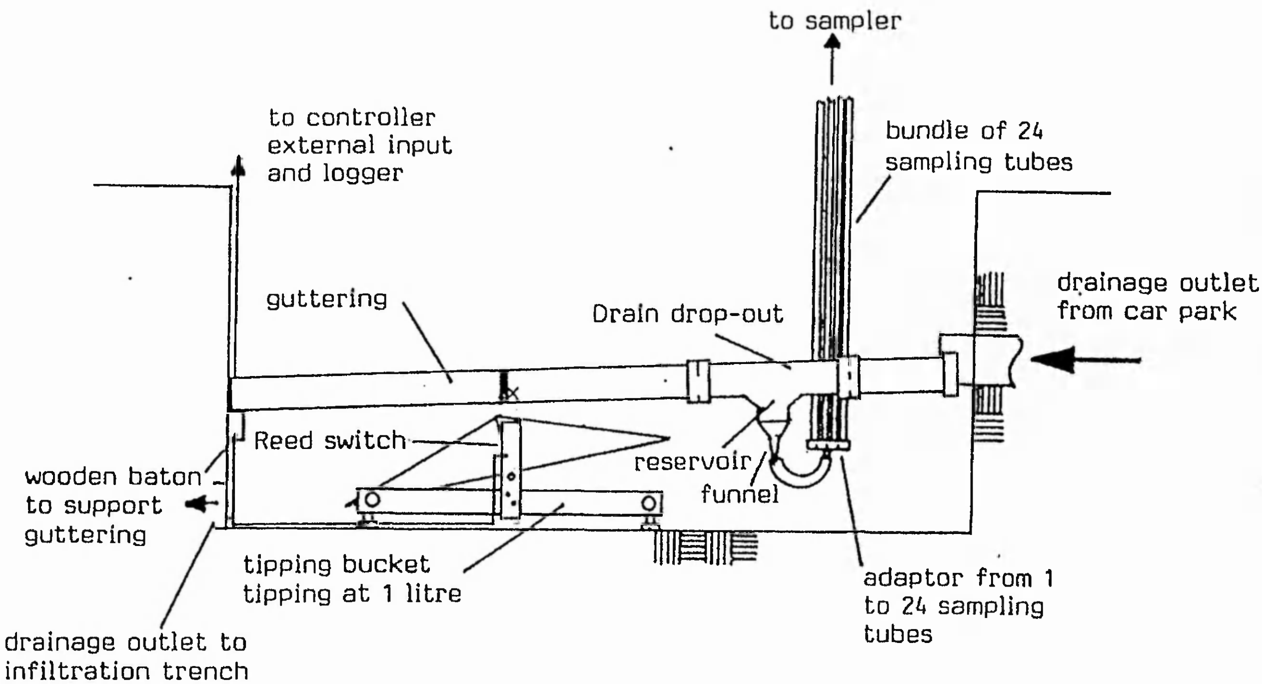
2.3 Monitoring Equipment and Stormwater Disposal

During the design process it became clear that the integration of systems for monitoring the quantity and quality of runoff from the pavement, with a need to dispose of the runoff afterwards, would present problems. The following Sections relate the methods used to achieve this integration, and give a description of the equipment used for data collection.

2.3.1 Monitoring equipment

Runoff monitoring equipment. As outlined above, the runoff from each of the 4 bays was discharged into an instrument pit (Plate 2.8). From each sub-base drainage pipe the runoff moved along a gutter to a retaining plate and down through a drainage hole into a tipping bucket device which measured the volume and hence the rate of runoff. Figure 2.3 shows a cross-section of the arrangement for one of the four sets of apparatus in the instrument pit. It can be seen that before the runoff left the guttering it passed over a sampling reservoir. During times of sampling this reservoir was periodically 'sucked' empty to provide a small sample for quality analysis. (Discussion of the effects of the sampling on runoff volume considerations are made in 2.4.2).

A great deal of time and effort was committed to ensuring that all the runoff fell correctly into the tipping buckets. Liberal use of silicon rubber, silicon gel, wire and elastic bands minimised the occurrence of leaks etc.
The tipping bucket measuring device was constructed at Nottingham University from a design by the Institute of Hydrology (Calder and Rosier, 1976) and was similar to, but larger than, a standard tipping bucket raingauge.



x - drainage hole backed by perspex plate

reservoir holds approx 350 mls.

Figure 2.4 Cross-section of the instrument pit.

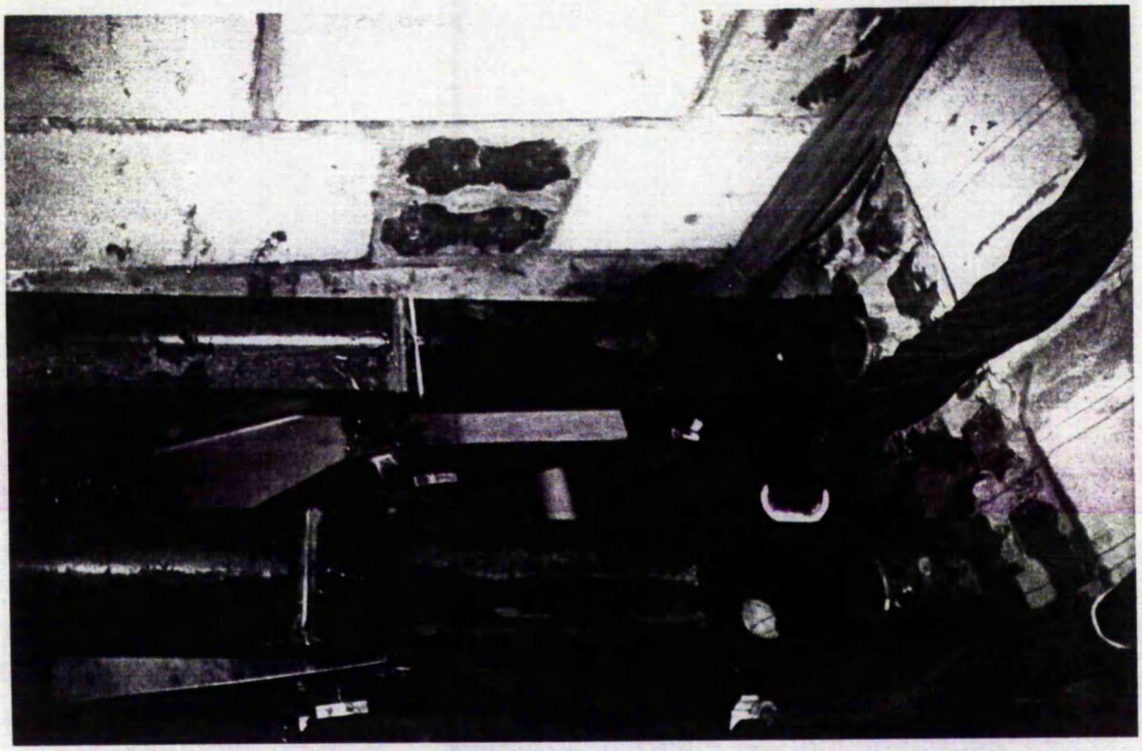
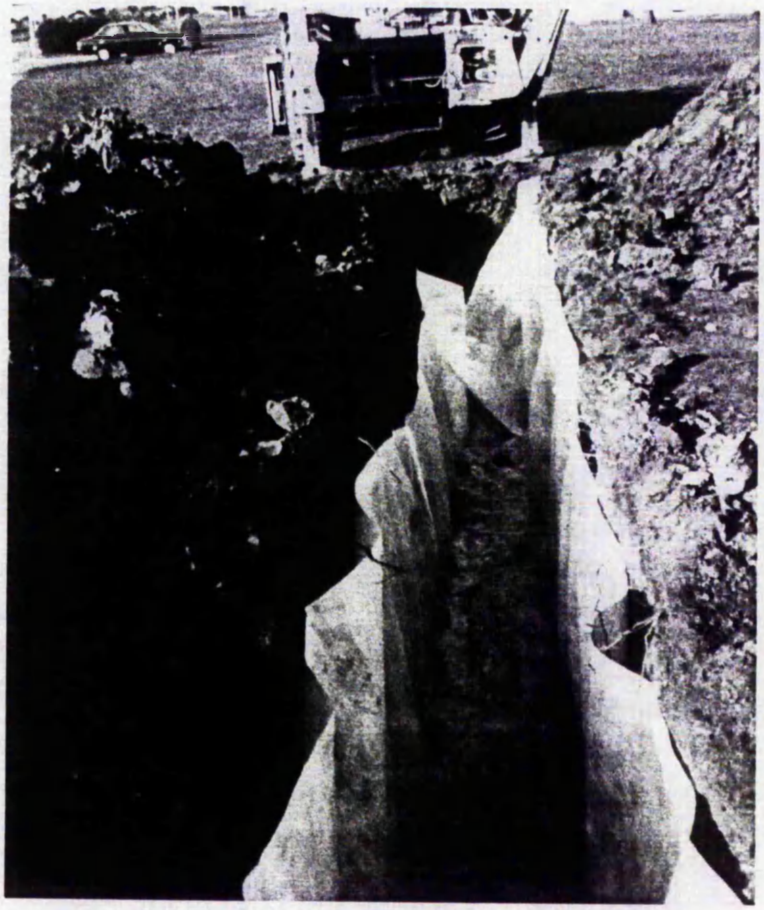


Plate 2.8 Drainage pipes discharging into guttering within the instrument pit.

Plate 2.9 Geotextile lining the infiltration trench.



For each of the four sets of equipment in the instrument pit, each time a bucket tipped a magnetic reed switch was closed and opened again by the movement of a magnet (on the bucket) past an actuator switch on the frame supporting the bucket. The electric pulse thus generated passed to a solid state data logger. The logger, a Campbell Scientific Instruments' 21X logger, was programmed to store the time in hours, minutes and seconds (to the nearest ten seconds) of the tip of each of the four buckets.

During most rainfall events all four bays would be discharging to their respective buckets, and times of each tip would be recorded. To obviate problems during analysis, each ten second period during which there was a bucket tip was given a four digit code, each digit corresponding to the tip of a bucket collecting from a particular bay. Thus a code of '0011' at a particular time indicated that in the preceding ten seconds both bay number 1 (gravel) and bay number 2 (b.f.s.) activated (i.e. tipped). Similarly a code of '2010' indicated that the number 4 bucket (i.e. bay number 4, limestone), had tipped twice in the previous ten seconds whilst the number 2 bucket had tipped once. The granite sub-base was designated bay number 3. During a general overhaul of systems in June 1988, the programming of the logger was changed so that the time of tip was recorded to the nearest two seconds.

The logger was also connected to 3 thermistor probes, of which two were used to measure the outside air and ground temperatures, whilst the third was installed within the pavement sub-base. During the June 1988 refit, two more thermistors were positioned within the sub-base so that the temperature profile within the car park could be monitored for any frost penetration. The logger was programmed to activate all the thermistors every hour, on the hour, and record the time and temperature in degrees Celsius for each.

The logger was programmed to record the 'Julian day' number at midnight. This was the number corresponding to the day of the year, i.e. 1st January = 1, 31st December = 365). An example of the output from the logger is given in Table 2.1. (A garden shed was placed over the instrument pit to contain all the monitoring and logging equipment, as well as the large amount of ancillary research paraphernalia).

Tipping bucket calibration. During the operation of the tipping buckets, as the rate of flow increased, a certain volume of runoff entered the already full and tipping bucket. In the fraction of a second that it took for the bucket to over-balance, flip past the vertical and bring the empty bucket into line, a volume of water related to the runoff rate went 'unmeasured'. Therefore, as with tipping bucket raingauges (Calder and Kidd, 1978), the gauges in this system were dynamically calibrated to account for the non-linear gauge response as part of the runoff analysis.

The method used for a static calibration assumed the relation of flow rate (Q), bucket volume (V) and time between tips (T) to be:

$$Q = V/T$$

However, as outlined above, runoff will enter the already full bucket as it is tipping. Assuming the time taken to tip (t), once full, was independent of the flow rate, then the dynamic equation for calibration may be assumed to be:

$$Q = V/(T-t)$$

The method used on the tipping bucket gauges for this research was to complete a laboratory based dynamic calibration before installation in the monitoring pit, followed by regular, in-situ, dynamic calibration throughout the research period. In short, the gauge parameters V,T and t were determined by measuring the tipping rate of the bucket at known flow rates.

The above equation can be re-arranged to:

$$T = (V/Q) + t,$$

then, if the time between tips (T) is plotted against the reciprocal of the flow rate (Q), 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.

Rainfall data collection. Previous research work at the Clifton Campus had resulted in the installation of a Rimco 0.1mm raingauge at a slightly exposed, quiet location some 250m to the west of the car park pavement. This equipment was utilised for this project with a second Campbell Scientific Instruments 21X logger being programmed to log the time of every tip of the raingauge. A second 'box' at the site contained a rain collector which was used for calibration purposes.

2.3.2 Stormwater disposal

All four sets of sub-base drainage discharged independently into the instrument pit. The pit was built by digging a one cubic metre hole by the side of the lowest corner of the car park construction. The outlet from the pit led, via porous piping to an infiltration trench.

As discussed previously, one of the design options for the pavement would have been to allow all percolating waters to infiltrate directly to the sub-grade. It was therefore deemed appropriate that an attempt be made to infiltrate the discharged runoff after measurement, and so an infiltration trench was installed. The advantage of the trench system was that it could provide valuable information on the infiltration potential of the local sub-grade, so that an assessment of the viability of a 'free-draining' permeable pavement could be made.

However, the outlet from the instrument pit was approximately 1 metre below ground surface, which required that the trench, (10 metres long with a 1 metre square cross-section), be buried 1 metre below the ground surface. The trench excavation to a depth of approximately 2 metres revealed undisturbed formation throughout. The soil conditions were logged as:

a. A mid-brown sandy-silt top soil to a depth of 0.3 to 0.5 metres.
b. An underlying band of firm silty clay to a depth of about 1.5 metres.
c. The bottom 0.5 metres of the trench in firm bands of clay.

The geology of the site conformed to the British Geological Field Maps (Numbers 142 and 126) which classify the area as predominantly Zone III Keuper Marl. During the construction (in November 1986) there was no indication that the water table had been intersected.

After excavation, geotextile was placed on the bottom and pinned to the walls of the trench, (Plate 2.8), whilst 50mm dolomitic limestone fill was carefully placed using the tipping bucket of the JCB. During the construction of the trench, a vertical inspection pipe was installed which consisted of a 300mm diameter sectional pipe, extending from the bottom of the trench to the ground surface.

The volume of the voids in the infiltration trench was designed to be 4.5 cubic metres, in accordance with the specifications in British Standards BS8301, (Anon 1985) and British Research Establishment Digest 151, (Anon, 1973), i.e. to contain the runoff from the car park surface of a storm event of 2 hours duration and 15mm per hour rainfall intensity. However, over particularly wet periods this volume could easily be exceeded, which combined with the low infiltration rates into the clay formation at this location, lead to problems of overflow of the infiltration trench. This in turn lead to a partial flooding of the instrument pit, at times disabling the monitoring equipment.

To reduce the frequency of this problem the inspection tube was used as access for a submersible pump into the infiltration trench so that the trench could be pumped out. The final effluent was discharged into the gutter of a nearby road. This procedure was satisfactory for occasional disposal of the stormwater from the experimental facility.

2.4 Data Collection and Processing

The permeable pavement and the monitoring equipment was operational by April 1987. Systems were quickly implemented to collect 'raw' data from the site of the permeable pavement and from the raingauge station. In general, data collection took place during a weekly visit to the Clifton Campus comprising a half or full day 'down-loading' data from the loggers, taking measurements and general maintenance of the equipment. Periodically, extra experimental procedures, described in Chapters 5 and 6, or calibration of equipment would necessitate an extra day or so on site. Periods of heavy or prolonged rainfall would also require a quick visit to ensure that the structure did not become flooded and that everything was working correctly.

The integrity of the results of the project was dependent on the quality of the 'raw' data. Therefore, many hours of work were involved in small adjustments to various sets of equipment, most notably the tipping buckets, and in the implementation of checks to ensure that the data analysed was an accurate reflection of what was happening in the field.

The aim of this Section is to explain the methodologies and processing techniques used to gather the raw data, process it into a readable form, and combine the results into a database so that general characteristics could be easily identified.

2.4.1 Data collection

Runoff data. The 21X logger on site had 40 kilobytes of Random Access Memory, (RAM), for recording: the times of tipping buckets tips; the codes indicating which bucket(s) had tipped; the temperature data; and the Julian day count. This memory was adequate for holding all the data resulting from a week of heavy rainfall. To 'collect' the data the RAM was down-loaded to the magnetic tape storage of an Epson HX20 portable computer.

A recurring cause of data loss was due to circuitry connected to sampling equipment. One of the sampling methods for the parallel study of quality parameters was based on the runoff flow rate as measured by the tipping buckets. This required the connection of a second circuit from each of the tipping bucket actuator switches to the control box of the vacuum samplers installed in the equipment shed. For reasons never quite understood, it became apparent that when the batteries for the control boxes on the samplers were 'flat', then the tips of the buckets were not recorded on the second circuit to the 21X logger. This was remedied by strict rotation and changing of the batteries for the control boxes.

The major source of data loss, or rather inability to record data, was caused by very heavy or prolonged rainfall resulting in the flooding of the instrument pit, i.e. the infiltration trench overflowing. The 4.5 cubic metre capacity of the infiltration trench would become full after approximately 35mm of effective runoff from the whole car park surface. This equated to an even larger figure for incident rainfall, but during a period of prolonged rainfall this figure could be exceeded, resulting in data being lost as the tipping buckets became submerged.

The final stages of data transfer were from the Epson portable computer to an Apricot PCXi, the data being stored on 3.5 inch disks. A print-out of the data was always made immediately after transfer so that the integrity of the data and any interesting events could be identified.

The library of files comprising the complete data set from April 1987 to April 1989 represented almost 5 megabytes. It would have been possible to process or condense the data during the initial logging, but it was judged important to have an original copy of what actually happened on site. This made the raw data 'readable', in that it was possible to quickly identify any system malfunction.

Rainfall data collection. The raingauge logger was visited on the same days as the car park logger. The Rimco 0.1mm raingauge has a siphon between the funnel and bucket, which is designed to empty waters at a constant rate into the tipping bucket and eliminate problems of non-linear gauge responses to varying rainfall intensities, as described in Section 2.3. The raingauge was calibrated statically, however, tests on this type of raingauge, by Niemczynowicz (1986), have shown it to have a very slight non-linearity.

To ensure that the correct rainfall volumes were being used in the study a second 'gauge', a rainfall collector, was used to calibrate the raingauge, the volume collected being converted to an equivalent rainfall depth.

Assuming that the raingauge did have some degree of non-linear response, then this blanket change in the static calibration had the effect of altering the true values for the rainfall intensity. The calculated values for high intensity rainfall would be slightly depressed and those for low intensity rainfall slightly increased. However, this manipulation of the data would ensure that the rainfall depth calculated from the raingauge corresponded exactly with the rainfall depth measured at the site, subject only to non-homogenous rainfall patterns, (Niemczynowicz, 1984).

2.4.2 Data processing

The initial aim of data processing was to generate a simple presentation of the data to enable comparison of the hydrological performance of the car park sub-bases with each other and with the rainfall input. The suite of computer programs used to process and analyse the raw data evolved over a period of months as different requirements of analysis and standards of presentation were met. The very early results were obtained by laborious use of pen, paper and calculator, but this was a useful learning period during which methodologies evolved. Also, the results obtained were used later to check computer derived results.

Accounting for sampled volumes. The automatic sampling of the runoff, for the purposes of quality analysis, was accomplished by extracting runoff from the guttering in the instrument pit before measurement by a tipping bucket. The first stage of processing the raw runoff data was to account for this volume taken by the samplers and not recorded by the tipping buckets. As soon as the samplers were removed, a list was obtained giving the exact volume of each sample. In general, the volume removed by the automatic liquid samplers was about 500ml for each of 24 bottles.

Accounting for the sampled volumes required the calculation of the volumes removed and a knowledge of the period during which each sampler was activated. The volumes removed, as a percentage of the total runoff, and the period during which samples were taken, were recorded for each set of samples taken from each of the four discharges for later use as input to the analysis calculations.

Computer processing. Table 2.1 shows the format of the output from the data loggers once transferred to the personal computer. The recorded parameters displayed for a bucket tip are shown on a single line and numbered from '01' to '04'. The first parameter is a code showing from what part of the logger program the data array originated. The second and third parameters are the time in hours, minutes and seconds of the tip of the bucket. The fourth parameter is the code showing which of the buckets had tipped , (two buckets tipping at the same time were encoded as described in Section 2.3.1). The output format from the raingauge logger was essentially the same but with just the one bucket code.

The next stage in processing the data was to break down the raw data files into 4 separate files containing the times of tips of each of the four buckets, with the times converted to days and decimal fractions of a day for ease of computing. A second program processed the rainfall data into files of exactly the same format. The result was five separate files containing a list of the times when each bucket tipped.

An examination of the original rainfall and runoff data files highlighted the periods of interest, i.e. rainfall events with significant runoff or significant rainfall with little runoff. A data processing program, named ' RIROP', (an acronym for Rainfall Input Runoff Output Processor), was used to complete the bulk of the data processing. RIROP required the following input:

a. The start and finish times for data processing;

b. A time increment, the program tabulated the calculated flow rates for each time increment, (commonly chosen increments were 1, 5 and 15 minutes);

c. The start and end time of any sampling, and the percentage volume of the runoff that the sampling comprised; and

d. A calibration factor for the raingauge, (as calculated from a comparison of the number of raingauge bucket tips and total rainfall depth calculated for that particular period).

The program opened the data files containing all the times of bucket tips, including the raingauge, and produced the output as shown in Table 2.2. The Table is annotated to explain its various parts. Briefly these are:

1. A print-out of the parameters required as input;
2. Two small listings of the times and durations of rainfall and runoff;
3. A long list of columns comprising the time in Julian day, hours and minutes; the rate of discharge (in mm/h) from each of the sub-bases, and lastly, the rainfall rate (mm/h) for the same period; and,
4. Three tables giving the summarised data for rainfall and runoff for the duration of the event.

Table 2.1 **Example of data recorded by the Campbell 21X logger.**

01+0302.	02+2018.	03+050.0	04+1.000	
01+0302.	02+2039.	03+0000.	04+100.0	
01+0308.	02+2100.	03+1.938	04+6.046	05+1.053
01+0302.	02+2109.	03+050.0	04+1.000	
01+0302.	02+2124.	03+0000.	04+1000.	
01+0302.	02+2133.	03+010.0	04+100.0	
01+0308.	02+2200.	03+1.090	04+6.196	05+1.069
01+0302.	02+2214.	03+010.0	04+1.000	
01+0302.	02+2224.	03+0000.	04+1000.	
01+0302.	02+2229.	03+0000.	04+100.0	
01+0308.	02+2300.	03+1.257	04+6.347	05+1.081
01+0302.	02+2323.	03+030.0	04+1.000	
01+0302.	02+2335.	03+010.0	04+100.0	
01+0302.	02+2343.	03+050.0	04+1000.	
01+0308.	02+0000.	03+0.566	04+6.356	05+1.098
01+0109.	02+0064.			
01+0302.	02+0043.	03+050.0	04+100.0	
01+0302.	02+0051.	03+050.0	04+1.000	
01+0308.	02+0100.	03+0.921	04+6.428	05+1.111
01+0302.	02+0103.	03+030.0	04+1000.	
01+0308.	02+0200.	03+0.631	04+6.481	05+1.118
01+0302.	02+0201.	03+040.0	04+100.0	
01+0302.	02+0222.	03+030.0	04+1.000	
01+0302.	02+0246.	03+020.0	04+1000.	
01+0308.	02+0300.	03+0.945	04+6.506	05+1.131
01+0302.	02+0320.	03+050.0	04+100.0	
01+0308.	02+0400.	03+0.797	04+6.498	05+1.142
01+0302.	02+0408.	03+030.0	04+10.00	
01+0302.	02+0419.	03+050.0	04+1.000	
01+0302.	02+0426.	03+040.0	04+1000.	
01+0302.	02+0450.	03+010.0	04+100.0	
01+0308.	02+0500.	03+0.452	04+6.468	05+1.154
01+0308.	02+0600.	03-0.290	04+6.420	05+1.165
01+0302.	02+0610.	03+0000.	04+1.000	
01+0302.	02+0625.	03+040.0	04+100.0	
01+0302.	02+0646.	03+040.0	04+1000.	
01+0308.	02+0700.	03-1.036	04+6.356	05+1.171
01+0308.	02+0800.	03+0.527	04+6.432	05+1.182
01+0302.	02+0809.	03+0000.	04+100.0	
01+0302.	02+0823.	03+0000.	04+1.000	
01+0308.	02+0900.	03+2.073	04+6.456	05+1.191
01+0302.	02+0908.	03+030.0	04+1000.	

Table 2.2 Example of the output from a 'RIROP' processed event.

```
HISTOGRM DATA PREPARATION PROGRAM
STARTING & FINISH POINTS ARE  31.916666                        32.416666
INCREMENTS (ie RES code) are 3
START & END TIMES FOR SAMPLING ARE  0      0
% BY VOL TAKEN FOR SAMPLING IN THIS PERIOD =  1
THE CALIBRATION FACTOR FOR RAINGUAGE=  1.21

EV8031C

RO/RF START                       (RO START)-(RF START)
 1.32. 2.18. 1. 0. 4.10.
 2.32. 2.40. 2. 0. 4.32.
 3.32. 1.53. 3. 0. 3.46.
 4.32. 2.12. 4. 0. 4. 5.
 5.31.22. 8. 5. 0. 0. 0.
RO/RF END                      (RO END)-(RF END)
 1.32. 5. 9. 1. 0. 2.12.
 2.32. 3.55. 2. 0. 0.58.
 3.32. 5.37. 3. 0. 2.39.
 4.32. 5. 4. 4. 0. 2. 6.
 5.32. 2.58. 5. 0. 0. 0. 5. 0. 4.50.
D  h  m       1     2     3     4     5
31.22.15. 0.000 0.000 0.000 0.000 1.062
31.22.30. 0.000 0.000 0.000 0.010 2.123
31.22.45. 0.000 0.000 0.010 0.000 1.062
31.22.60. 0.010 0.000 0.000 0.000 1.062
31.23.15. 0.000 0.000 0.105 0.000 0.531
31.23.30. 0.055 0.000 0.092 0.000 0.000
31.23.45. 0.000 0.000 0.093 0.000 1.062
31.23.60. 0.071 0.000 0.088 0.000 0.000
32. 0.15. 0.060 0.000 0.077 0.000 0.000
32. 0.30. 0.000 0.000 0.000 0.035 0.000
32. 0.45. 0.069 0.000 0.071 0.000 0.531
32. 0.60. 0.000 0.000 0.081 0.000 1.062
32. 1.15. 0.094 0.000 0.000 0.000 0.531
32. 1.30. 0.117 0.000 0.094 0.072 0.531
32. 1.45. 0.149 0.000 0.141 0.000 2.654
32. 1.60. 0.186 0.000 0.240 0.115 1.062
32. 2.15. 0.237 0.000 0.310 0.208 3.185
32. 2.30. 0.469 0.000 0.640 0.444 4.247
32. 2.45. 0.887 0.404 1.053 0.796 4.247
32. 9.30. 0.089 0.052 0.101 0.000 0.000
32. 9.45. 0.089 0.000 0.106 0.097 0.000
32. 9.60. 0.081 0.049 0.094 0.095 0.000

TOTALS IN MM FOR RAINFALL DURATION
 BUCKET No  5  =  6.901067    100  %
 BUCKET No  4  =  .7004276    10.14955  %
 BUCKET No  3  =  1.0949      15.86567  %
 BUCKET No  2  =  .276938     4.012974  %
 BUCKET No  1  =  .8774339    12.71447  %

TOTALS IN MM FOR RF UNTIL RO START
BUCKET No  4  =  3.954843     57.3077  %
BUCKET No  3  =  3.196905     46.32479  %
BUCKET No  2  =  5.860009     84.91454  %
BUCKET No  1  =  4.302846     62.35044  %

TOTALS IN MM FROM RF START UNTIL RO END
 FOR No  5  =  6.901067       100  %
 FOR No  4  =  2.146153       31.09886  %
 FOR No  3  =  2.869722       41.58375  %
 FOR No  2  =  .9860592       14.28851  %
 FOR No  1  =  2.263895       32.80501  %
```

D – Julian Day
h – hour
m – minutes
1 – gravel runoff mm/h
2 – b.f.s.
3 – granite
4 – limestone
5 rainfall mm/h

Definitions of rainfall and runoff. After a few weeks of monitoring the discharged runoff from each of the four sub-base bays of the car park, it became obvious that definitions for the beginning and end of rainfall and runoff would have to be made, so that all events could be summarised, compared and statistically analysed from a common base.

Absolute terms such as rainfall begins at the time of the first tip of the raingauge and runoff ends when no tips are recorded for one day would clearly have proved unworkable: showers and drizzle could go on for many hours before and after a significant rainfall event. Also, in the case of the permeable pavement studied here, a 'trickle' of runoff was seen to continue for several days after some rainfall events. Both of these effects made it impractical to account for every drop of rainfall and runoff.

The definitions chosen were related to observations for the car park, and may not be directly applicable elsewhere. For defining the start of rainfall, a rainfall intensity of 1.0 mm/h was chosen. Any rainfall before this threshold was ignored by the program, but it would be included in the total for rainfall antecedent to the period of interest. After this rainfall threshold, the total amount of rainfall in the event was summed, and the end of the event defined as that time when there had been no rainfall for a period of 90 minutes. The choice of this limit was derived from observations of significant events which had some periods without rainfall, but of a length which did not justify splitting the event into two or more individual storms.

For the analysis of runoff, a threshold level of 0.25 mm/h was chosen for the effective start and finish of runoff, which was equivalent to a flow of approximately 9 litres per hour from one of the bays. This value, equal to one quarter that of the rainfall intensity threshold, was chosen to take into account the attenuation effects of the sub-bases upon the incident rainfall.

Units of flow. The choice of 'millimetres per hour' as units for the runoff rate was made for two reasons. Firstly, figures in litres per hour, or similar, would render comparisons between the different bays difficult, as they all had different surface areas. Secondly, as the project was investigating specific attenuation effects, it would be useful to compare the figures for rainfall and runoff directly. This line of thought was followed through in the presentation of the data summaries, (Table 2.2), in which runoff volumes are given in millimetres equivalent depth for the surface area of each bay, (1mm is approximately equal to 40 litres of runoff). The data summaries also show the values for runoff as a percentage of the equivalent values for rainfall.

Presentation. The final stage of data processing was to present the results in graphic form, an example of which can been seen in Figure 2.5. It is important to note the conventions used in these Tables and Figures. The time given in each row of output from RIROP is that of the end of the period for which the data has been calculated, e.g. in Table 2.2 the first line of output gives a rainfall rate of 1.06 mm/h for the time of 22:15 hrs. This means there were 1.06/4mm of rainfall in the period from 22:00hrs to 22:15hrs, (the time resolution for the analysis being 15 minutes, therefore 1.06mm/h for 15 minutes is 0.27 mm). Thus, on the graphs this volume is recorded as the bar of a histogram (of height 1.06mm and base 15 minutes) similar to a conventional hyetograph, but because rainfall and runoff are plotted with the same scales, and the two parameters are to be compared, they are superimposed rather than having the rainfall plotted upside down.

The values calculated for the runoff by RIROP are average rates of flow during the previous time period, hence to plot the correct curves for the runoff, the values given are plotted in the centre of the time period to which they refer.

Antecedent conditions. Before experimental data were collected, it was appreciated that the climatic conditions that preceded any event could have an effect on the runoff. To monitor the effects of these 'antecedent conditions', a continuous record was kept of the 'antecedent rainfall' or dry period and of the temperatures before and during an event. The records were checked for every event analysed, so that antecedent conditions could be examined and compared for similar events.

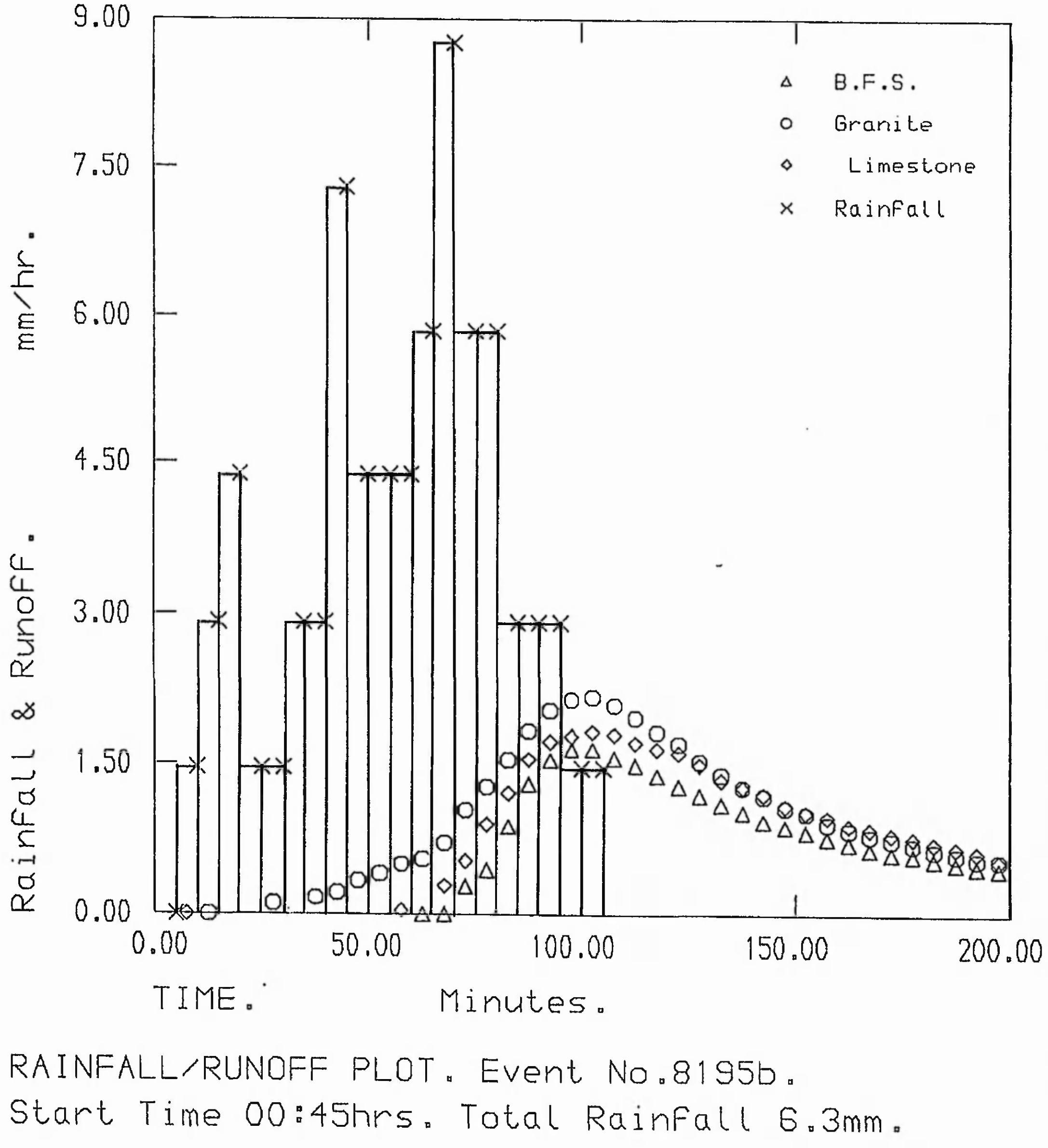


Figure 2.5. An example of the final plot of a rainfall histogram and resulting runoff curves. The event suffix 'b' indicates a time resolution of 5 minutes for intensity (mm/h) calculations.

CHAPTER 3

RUNOFF INFILTRATION DEVICES

The permeable pavement, which formed the main subject of this research, had all runoff directed to a rather ineffective infiltration trench. The poor performance of the infiltration trench was due the low level of the final discharge from the measuring pit which required that the trench was 'buried' one metre into the local clay soils, (Chapter 2). To determine the effectiveness of more 'realistic' infiltration devices, which could then demonstrate the potential for infiltration from the permeable pavement, a second series of infiltration devices was constructed near to the pavement. To add a second dimension to the research of runoff from urban surfaces an area of roof was chosen as the runoff source. There are several characteristics which make roof runoff of particular interest to on-site storage and attenuation.

Roofs generate some 60% of stormwater runoff in the urban environment, (Pratt and Harrison, 1982), but for the majority of rainfall events, i.e. those less than 10mm, they may produce a higher percentage of the total urban runoff. In terms of the relationship between rainfall and runoff, roofs differ from roads in three important ways:

a. They are often smoother, or less rough, than roads and therefore have a higher runoff coefficient. (A low depression storage is generally limited to ponding in gutters);
b. Many roofs are pitched, causing a much faster runoff response than roads, (i.e. they have a short 'time of concentration'); and,
c. The height of most roofs means that they do not accumulate many solids and the runoff is noticeably less polluted than road runoff.

Report HX1049, (Anon, 1982), emphasised the relative effectiveness, (in reducing stormwater discharges), of an areal reduction in roof runoff to the sewerage system, i.e. a reduction in the area of roofs contributing runoff is more effective in preventing hydraulic overloading of sewers than an equal reduction in the area of roads that contribute runoff.

Furthermore, it is important to emphasise the difference in relative pollutant loadings between typical roof and road runoff. In many circumstances it makes good sense not to 'contaminate' relatively 'clean' roof runoff with 'dirty' road runoff, so that the former can be disposed of directly to 'natural' drainage systems, whilst the latter is treated as required.

Although the use of infiltration devices such as soakaways is thought to be common, there is little published research data on their hydrological performance. In order to gain research data on the construction and performance of a variety of designs, a suite of infiltration devices was built and monitored. Although the devices are considered on their own merit as part of a roof runoff detention system, the hydrological results could be used to assess the suitability of such structures, in the local soils, for receiving the permeable pavement runoff. This Chapter describes the construction, in unfavourable ground conditions, of three infiltration structures designed to dispose of roof runoff by infiltration. The final Section describes the monitoring equipment used for data collection from the experiments.

3.1 Construction

The Keuper Marl geology at the Clifton Campus has been described in Section 2.3.1 and consists mainly of clay/silty-clay overlain by silty soils. The soil horizons were thought to be largely unsuitable for the siting of infiltration devices. However, it was advantageous to investigate the limitations of various designs in this difficult environment.

The specific site chosen for the investigation was next to a student residential block at the Clifton Campus. The south-west facing part of the sloped roof of the building provided the stormwater runoff supply. The area of the roof, equivalent to a flat roof area of 128 square metres, or approximately three-quarters of the area of the permeable pavement, was drained via a single gutter with three water leaders, (down pipes), into a standard below-ground pipe sewerage system.

Three separate designs for infiltration devices were chosen for the investigation, the designs ranged from the traditional to the more modern type. All three devices were to be supplied with runoff via the existing down pipes from the gutter. To ensure that all devices received an equal quantity of runoff, semi-circular plastic plates were o inserted into the gutter to act as barriers and divide the drainage into three equal lengths, (all serving an equal area of the roof). Details of the construction and monitoring of the three devices: a stone-filled soakaway; a dual-chambered soakaway; and an infiltration trench, are given below.

3.1.1 Stone-filled Soakaway

The first structure to be considered for the roof drainage site was a design similar to most traditional soakaways currently in use. A stone-filled soakaway pit is the simplest of infiltration devices as it consists only of a pit back-filled with stone or building rubble into which runoff from impermeable surfaces is discharged.

The design of the soakaway constructed at Clifton is given in Figure 3.1. This shows two minor improvements to the traditional design. Firstly, the stone fill was enclosed within geotextile to prevent movement of the surrounding soil into the voids between the stones; and secondly, a 100mm diameter inspection pipe was installed at the end of the inflow pipe, extending from the ground surface to the base of the device, so that water levels could be monitored. This inspection pipe would not, in practice, allow for any serious maintenance, but would forewarn of reduced rates of discharge from the soakaway, or of silting up of the voids indicating progress towards failure. Frequently, the owners of these devices do not know of their existence until such time as the device fails and causes flooding. Therefore, the inspection pipe also served to provide an aid to the location of the soakaway. The back fill for this device was 50mm clean limestone.

This type of design of soakaway is simple to construct and may provide for cheaper unit costs of construction than more complex designs such as those described below. However, effective maintenance is not really viable. Any signs of hydraulic failure due to clogging would require complete re-excavation to remove all the stone fill and any clogging matter, or the construction of a new soakaway adjacent to the existing one. One reason why this device was constructed was to monitor any difference in quality effects between this and the chambered soakaway (described below), by having larger stone surface area with which any pollutants might react.

The effective storage volume of this construction was 1.6 cubic metres, with an internal area, excluding the base, of 9.0 square metres available for infiltration, (when full to the invert of the inflow drainage pipe): a surface area/volume ratio of 5.63.

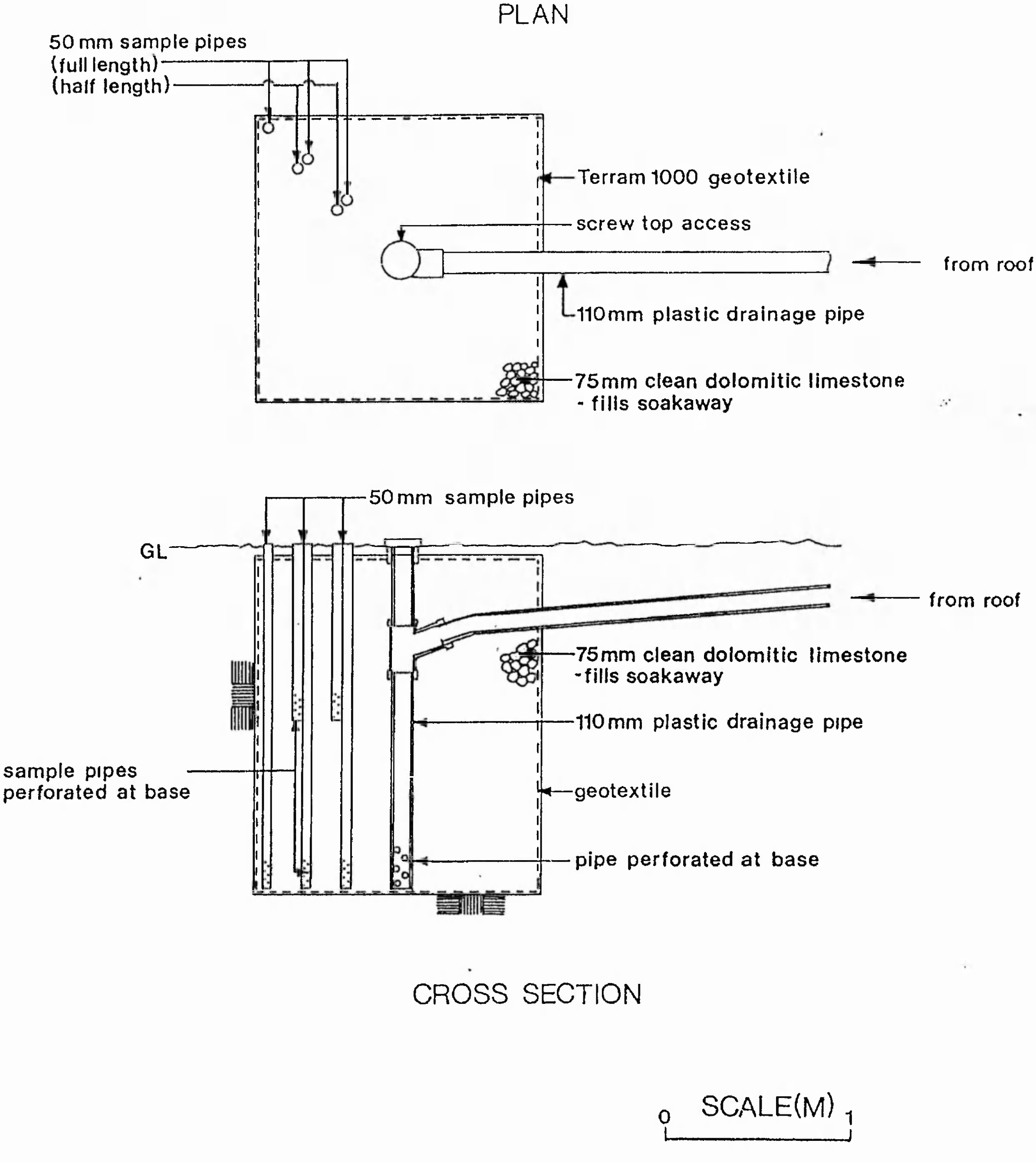


Figure 3.1 Design for the stone filled soakaway.

3.1.2 Dual-chambered soakaway

The traditional structure of a soakaway, described above, allows for the storage of storm runoff and facilitates the infiltration of the runoff into the ground. The development of large pre-cast concrete structures has enabled the design of soakaways with open chambers which provide a greater storage volume, relative to the excavation size, and also allow for some degree of maintenance.

The design chosen for this investigation, with dual soakaway chambers, incorporated modern techniques and structures used in present day infiltration systems. The main chambers were formed from perforated concrete ring units which provided a cylindrical space for water storage, whilst the perforations allowed for movement of waters to the soil interface. To provide the estimated required storage volume, whilst keeping the sizes of the excavations to a reasonable depth, two chambers were installed to receive runoff 'in series'. The details of the design are given in Figure 3.2.

Construction commenced with de-turfing the areas to be excavated, Plate 3.1. The excavation for the first soakaway was 1.60 metres square and 1.80 metres deep, for the second 1.80 metres square by 1.90 metres deep. A 150mm diameter pipe was used to connect the first soakaway chamber to the roof drainage and a second pipe, 2.5 metres long, to connect the two chambers. This connection between the two pits was made using a plastic pipe from the outside of the first concrete ring to the inside of the second.

After excavation the soakaway pits were lined with geotextile before the pre-cast concrete rings were placed in the centre of each pit. Plate 3.2 shows the pre-cast concrete rings in each pit with concrete cover slabs and galvanised steel manhole covers in place. The cavity around each set of concrete rings was back-filled with clean 50mm dolomitic limestone.

Plate 3.3 shows the dual-chambered soakaway site some months after construction and re-turfing.

As runoff drained into the first soakaway, solids and suspended matter should settle onto the floor of the soakaway. When the first soakaway became filled with waters the overflow was from the outside edge of the soakaway pit, so that only 'filtered' runoff entered the second chamber. This system was designed to ensure that the infiltration capacity of the second chamber remained as high as possible and did not suffer from clogging.

The use of pre-cast concrete rings adds considerably to the cost of construction but this is offset to a degree by an increased storage volume for the same size of excavation and a reduced requirement in stone for back-fill material. A major advantage in this design was the provision of access for maintenance, i.e. the removal of solids. (Although not strictly within the scope of this research, care must be taken with the provision for access, if these large, often water-filled structures are built in an urban environment, or wherever children can gain access).

The total volume available for storage within the two units when full (taking into account stone porosity and the invert level of the drainage pipes) was 3.3 cubic metres and the internal surface area available for infiltration to the soil was 14.4 square metres, a surface area to volume ratio of 4.36.

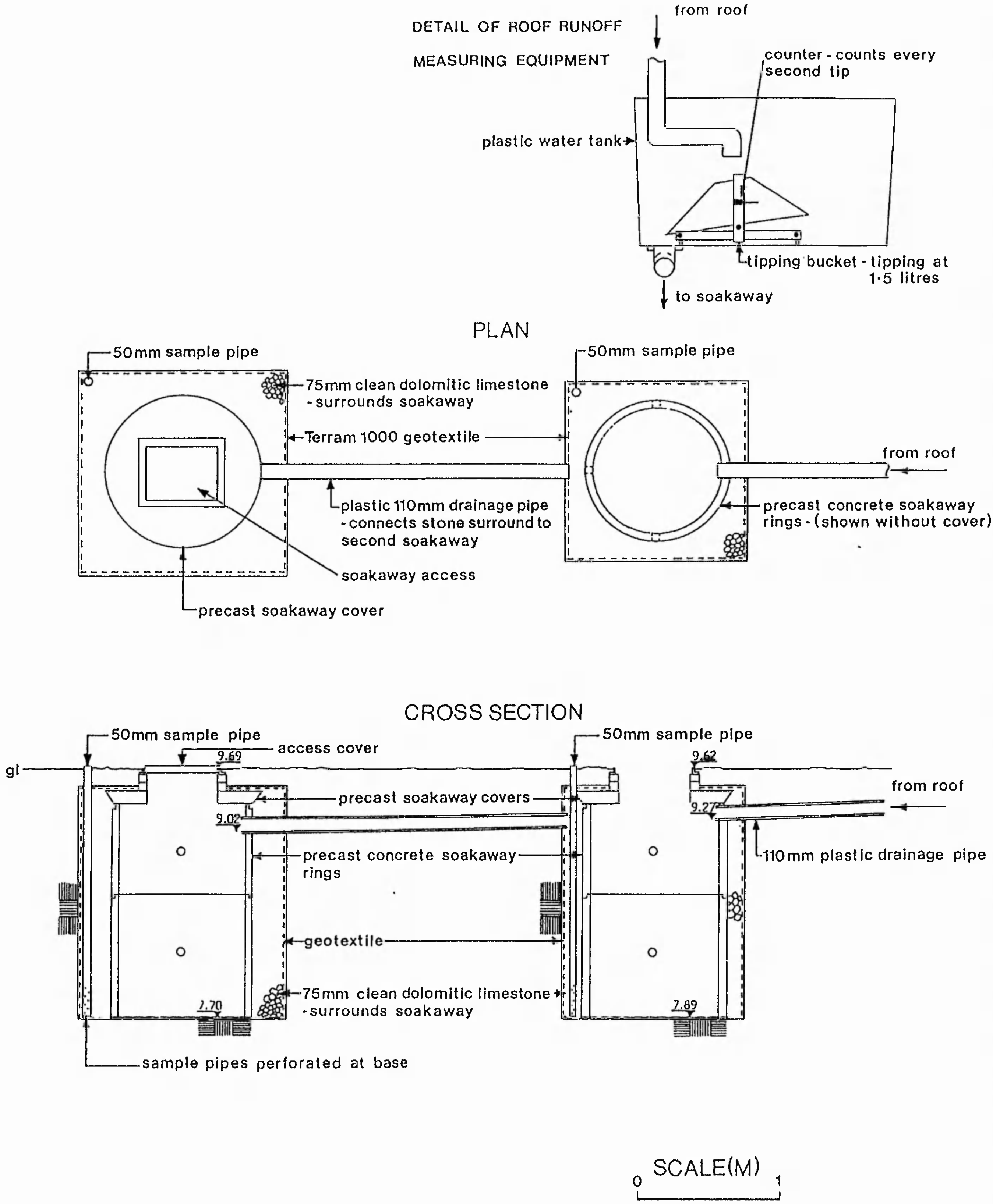


Figure 3.2 Design for the dual-chambered soakaway.

Plate 3.1 De-turfing prior to construction of the dual-chambered soakaway.

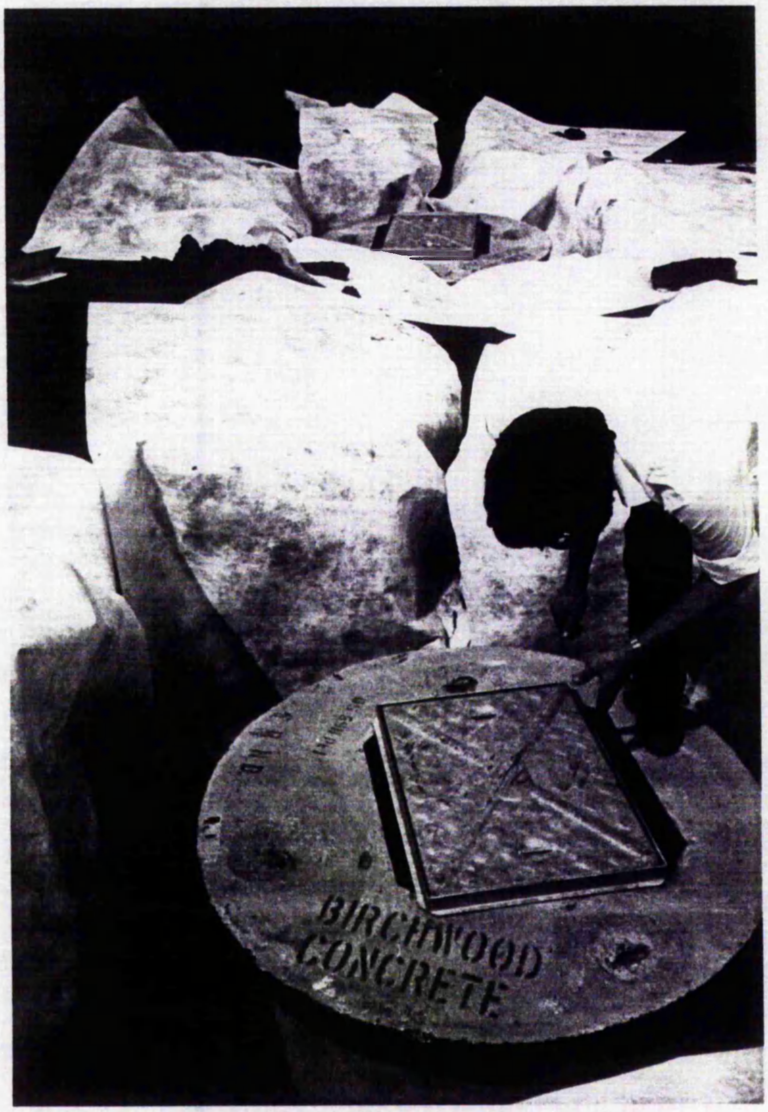


Plate 3.2 Dual-chambered soakaway nearing completion, geotextile and concrete rings in place.

Plate 3.3 Dual-chambered soakaway site some months after construction.

3.1.3 Infiltration trench

An infiltration trench consists of a shallow excavated trench, of required length, back-filled with a coarse stone aggregate and supplied with stormwater, either via a perforated pipe along its upper surface, or via direct inflow from the ground surface.

The obvious difference between an infiltration trench and a stone-filled soakaway (or dry well), is one of shape: the trench is typically a linear structure and is often shallower than the soakaway; whilst the latter, traditionally, has similar dimensions for length, width and depth. This difference suggests that the (internal surface area)/ (storage) ratio is much higher for the trench, producing higher rates of discharge from the trench to the surrounding soil. Also, a trench may be used in areas with low permeability soils at depth. However, a trench of the same storage capacity as a stone-filled soakaway may have a greater internal area for infiltration, but it will occupy a larger surface area of ground.

The infiltration trench constructed as part of this investigation was similar in design to that already described in Section 2.3, although it was not required to be built to such a depth. It was excavated to a depth of 0.95 metres and was 0.5 metres wide and 6.5 metres long, the full design is given in Figure 3.3.

The excavation was lined with geotextile to keep the void spaces open by providing a separation layer between the soil and the stone fill. The stormwater discharge to the trench was via a 300mm diameter inspection chamber with an access cover.

The outlet from the inspection chamber was via a porous distribution pipe along the length of the trench near the top of the aggregate. Plate 3.4 shows the back-filling of the trench, with 50mm clean limestone, over the porous piping. The spare geotextile shown in the plate was folded over the top of the stone fill before back-filling of the trench was completed with top soil and the turf replaced.

Taking into account the porosity of the stone and the discharge pipe invert level, the volume available for storage in this device was 0.9 cubic metres. The trench had a relatively large area for infiltration, 12 square metres, when full to the invert of the drainage pipe, and a surface area/storage ratio of 13.33. It is important to note that as the trench was only excavated to a depth of about 1 metre it provided greater scope for infiltration into the upper soil horizons than either of the two devices described above.

Table 3.1 provides a summary of the dimensions of the infiltration devices installed at Clifton Campus. BRE 151, (Anon, 1973), recommends a storage equivalent to 30mm of over the area of the metres impermeable area to be drained. This is equivalent to 1.28 cubic metres for each device at Clifton. Table 3.1 shows that only the dual chambered soakaway provided more storage than the recommendation, whilst the stone-filled soakaway and the infiltration trench were approximately 30% smaller than the recommended volume.

Table 3.1. Summary of infiltration device dimensions.

Device	Effective Area m^2	Effective Volume m^3	Area/Volume Ratio m^2/m^3
Stone-filled soakaway	9.8	0.92	5.63
Dual-chambered soakaway	20.3	3.35	4.36
Infiltration trench	12.0	0.9	13.3

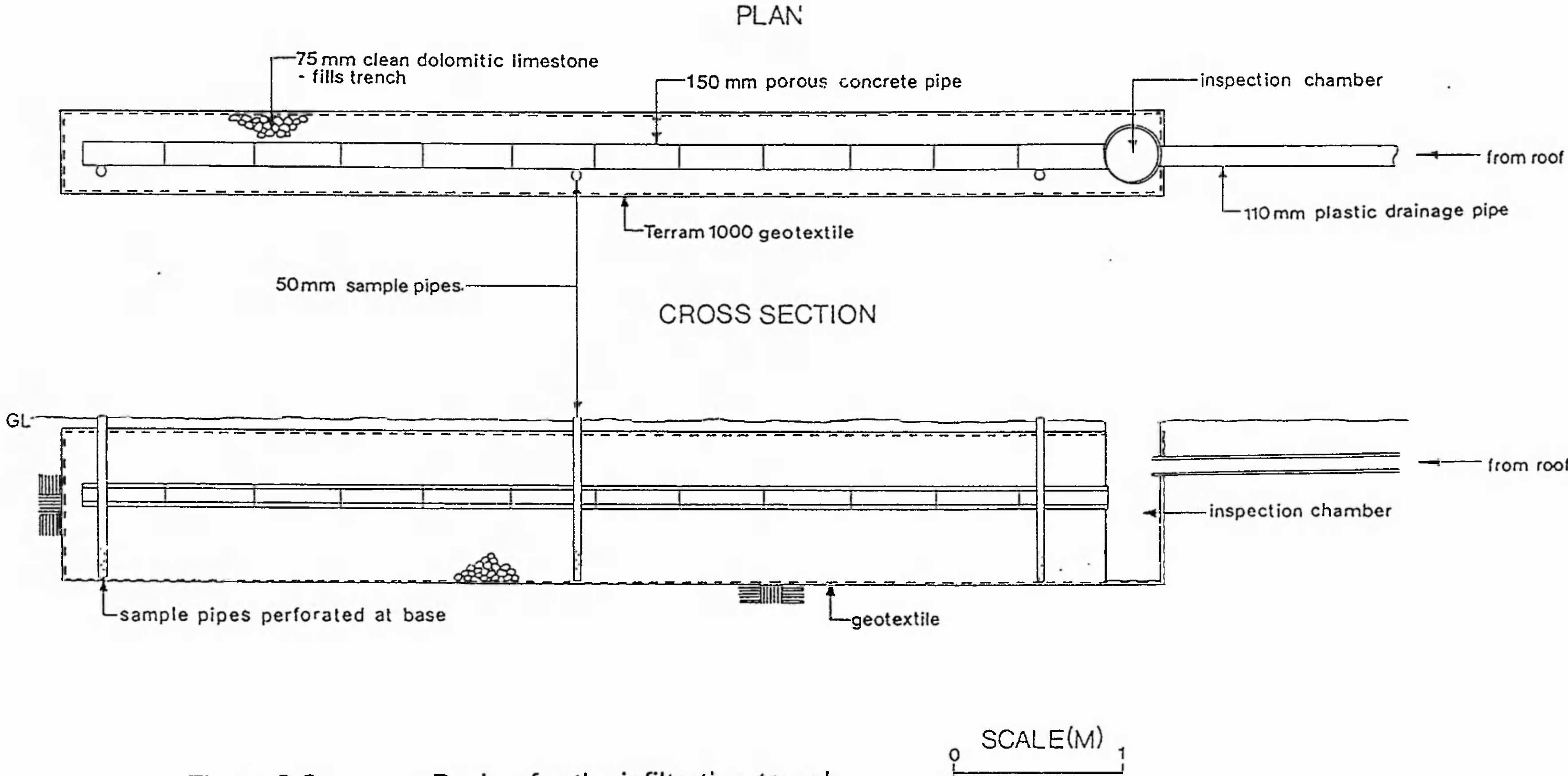


Figure 3.3 Design for the infiltration trench.

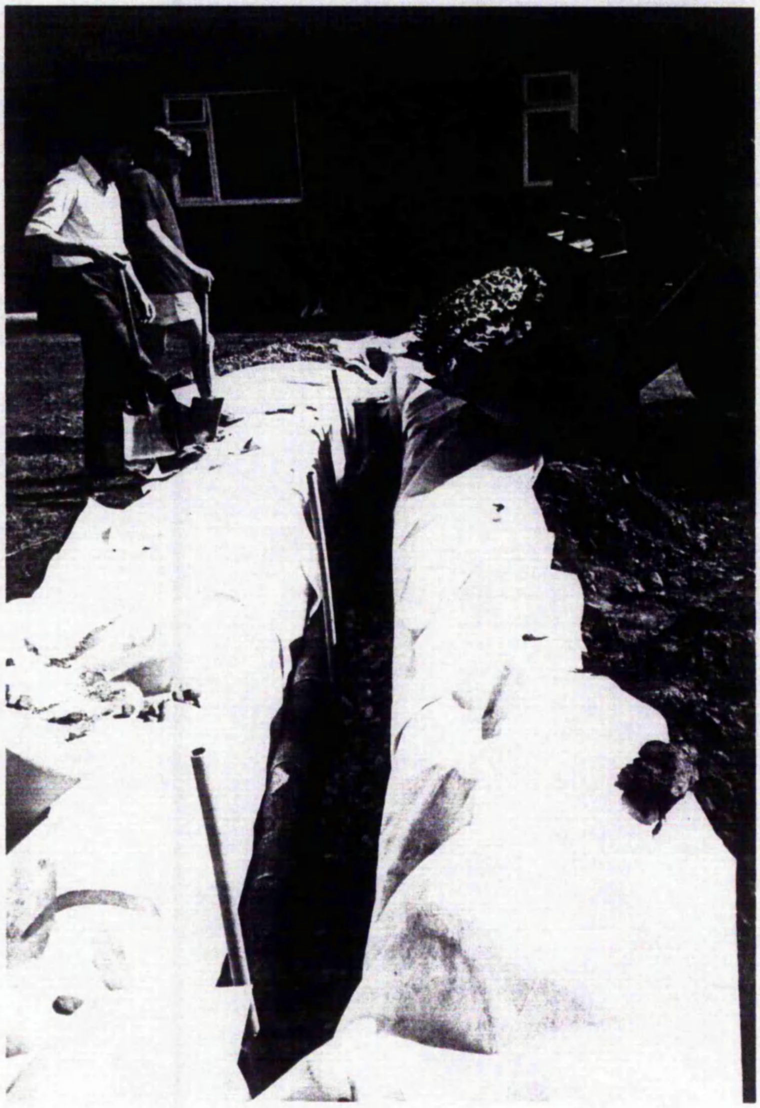


Plate 3.4 Back filling of infiltration trench.

3.2 Monitoring Equipment and Data Collection

The devices described above were, together, designed to receive runoff from one half of the roof of a student accommodation block. The monitoring of the performance of the devices included the stormwater input to each device and the storage volume occupied within each device, thus allowing the 'losses' through infiltration and other mechanisms to be calculated, (the input minus the change in storage for a given time interval equals the losses).

As described previously, the gutter servicing the roof area was divided into three equal sections, requiring only one rainwater leader to be equipped to measure runoff volumes. The volumes of runoff received by the other two devices were assumed to be the same as that measured as inflow to the dual-chambered soakaway.

The runoff was measured using a tipping bucket system similar to that described in the monitoring of the permeable car park (in Section 2.3). Plate 3.5 shows the interior of the grey water tank seen in Plate 3.3. Problems with site security and lack of a suitable method for real time data collection resulted in the fitting of a simple trip meter to a tipping bucket. The roof runoff recorded was taken to be the same for all three devices.

The lack of real time data logging meant that it was only necessary for the bucket to be statically calibrated, (see Section 2.3). This would y have resulted in an under estimation of the runoff during high intensity rainfall events. Calculations from calibrations of the tipping buckets which were dynamically calibrated, and logged in real time, suggested an estimated under recording of between 5% and 10% for the yearly totals of runoff from the roof. However, as the individual 'under-recording' for f each event could not be estimated, no manipulation of the data was undertaken. This meant that whatever the margin of error within the calculations was, it was on the 'safe' side, i.e. any prediction about the infiltration capabilities of any of the devices will be an under-estimate.

Each time the runoff totals were taken, the level of water in each device was measured using a 'dip tape'. When the end of the tape was immersed in water a connection was made and a light indicated to the user that the end of the tape was touching water, the distance from the water surface to the ground could then be read from the tape.

Using the measurements of the levels of the water in each device and the dimensions of the construction enabled the occupied storage in each device to be calculated. This data was compared with the previous value of storage and the volume of runoff in the intervening period to give a value for the volume of runoff 'lost' from each device between readings.

Experiments were also undertaken with a 'maximum depth gauge'. This 'homemade' instrument was designed to show the maximum level to which the water had risen within the device between measurements. The maximum depth gauge consisted of a vertical, screw threaded, steel rod placed inside the infiltration device, with a short cylindrical float made of balsa wood which 'climbed' up the steel rod as the water level rose in the device. As the water level fell, (due to infiltration after the end of the storm), a small clip on the base of the float caught in the grooves of the threaded rod to prevent the float moving back down the rod. Thus the height of the float would record the highest water level reached during the event. The instrument was reset to the prevailing water level on each visit to the site.

The maximum depth gauge, with its casing, can be seen in-situ, in the inspection chamber for the infiltration trench in Plate 3.6. The stainless steel probe on the grass is the end of the dip tape used for measuring the depth to water.

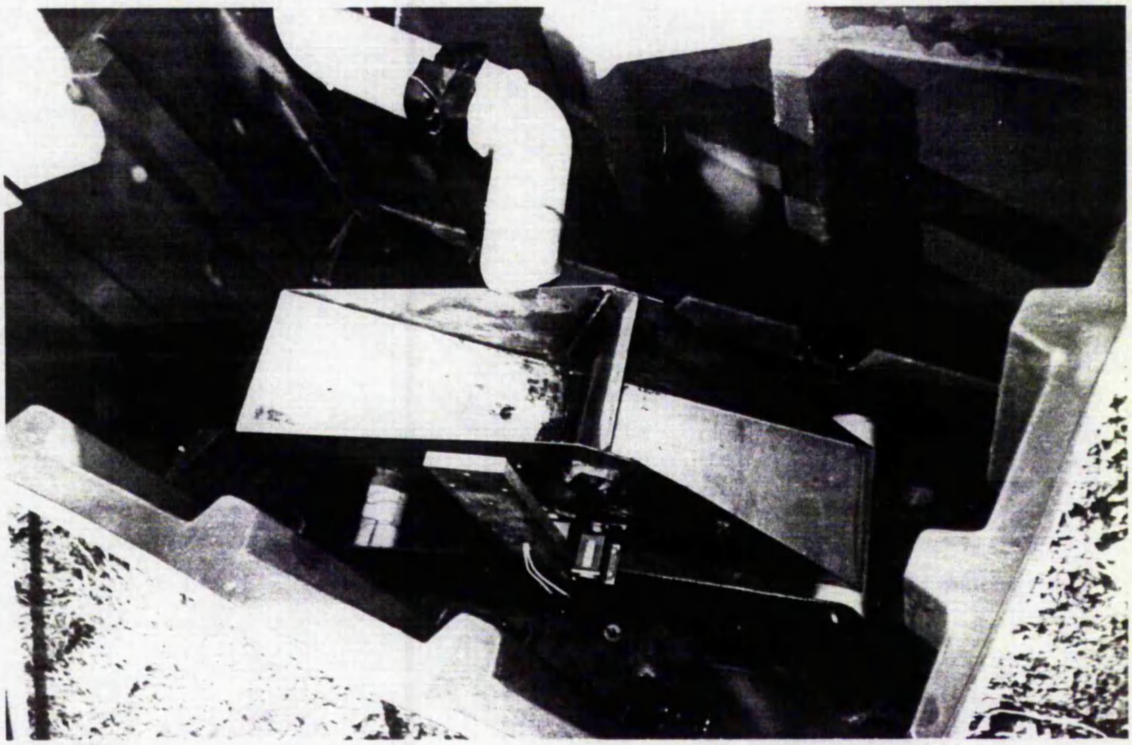
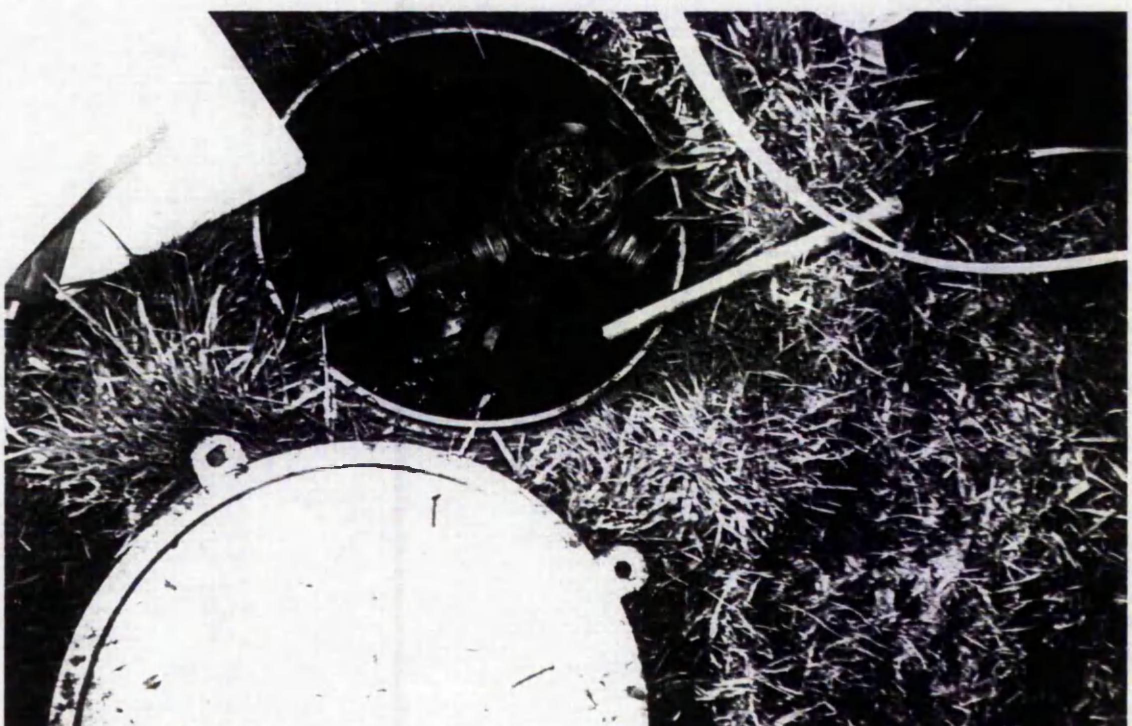


Plate 3.5 Tipping bucket used for measuring runoff at the 'roof site'.

Plate 3.6 Inspection chamber of the infiltration trench and view of the 'maximum depth gauge'.



The aim of the use of this gauge was to be able to calculate long term infiltration rates. Assuming that the infiltration rate at any time was less than the typical runoff rate from a storm event, then it would be possible, using the data from the raingauge on the Campus, to estimate at what time the infiltration device was at its fullest. This would give a time at which the maximum depth was set, which could be used in conjunction with the dip tape level measurements to calculate the infiltration for that device in the time between the end of the event and the reading of the maximum depth gauge.

Maximum depth gauge equipment was fitted into both of the chambers of the dual-chambered soakaways and into the inspection chamber of the infiltration trench. The results of measurements taken are discussed in Section 4.2.

As infiltration devices are intended to facilitate the movement of water into the surrounding ground, there may be various changes in groundwater levels around such devices. To monitor these changes a number of piezometers were installed into the ground in the vicinity of the devices.

The piezometers were installed using a 50mm diameter auger to drill holes to a depth of 1.5 meters. Perforated plastic tubes of 50mm diameter were inserted into the holes, the perforated part of each tube was surrounded with geotextile. To prevent ingress of surface waters into these holes, the outside of the tubes was sealed by back-filling with clay, and screw tops with ventilation holes were fitted to the tops of each tube.

To enable a sufficiently detailed contour map of the surface of the groundwater to be constructed, the seven installed piezometers were clustered solely around the end of the infiltration trench. Dip tape measurements were taken from each piezometer on a regular basis to establish the depth to water in each.

CHAPTER 4

HYDROLOGICAL PERFORMANCE OF EXPERIMENTAL FACILITIES

The permeable pavement and the infiltration devices at the 'roof site' were all operational by the end of the summer of 1987. The systems for collecting data from the permeable pavement site and from the raingauge station were explained in Section 2.4, however, the full set of monitoring equipment at the roof site was not operational until January 1988. Data collection for both sets of experiments continued until April 1989.

This Chapter presents the basic hydrological data obtained from the two experimental sites and summarises the results from these different structures by drawing together the similarities between the processes observed. This Chapter also comments upon the complimentary nature of the two systems and on implications for the design of attenuation/infiltration devices.

4.1 Permeable Pavement Results

The data for the car park covered a period of over two years, however, during the last 6 to 9 months of the research alterations were made to the sub-base drainage in some of the bays to evaluate various alternative arrangements. These alterations resulted in the removal of the P.V.C. lining from bay number 1, (the gravel), and in the height of the drainage inlet in the gravel bay being increased.

These changes to the gravel bay resulted in almost nil runoff for the rest of the monitoring programme, and hence the data presented below shows the gravel bay as having missing values for the last 20 events presented. These 20 events were, on average, of above mean rainfall volume and rainfall intensity, and therefore comparison between gravel and the other sub-bases are not strictly valid when considering the whole data set.

Data loss. The results presented here span 15 months from the summer of 1987 to autumn 1988. The results in Table 4.1, in six parts, represent the completed processing of 62 rainfall events during this period, an asterisk represents a missing data value. During the whole of these 15 months there were 15 events of over 5mm rainfall missed. Approximately half of these events (seven) were lost due to flooding of the instrument pit. The rest were caused by the logger being switched off/disconnected, either accidentally (twice) or during the Christmas holidays (six events).

A feature of this 'lost' data is that the flooding was related to high rainfall, so the lost events tended to be the largest during the period of monitoring. Figure 4.1 shows a frequency distribution curve for the size of the rainfall events given in Table 4.1. This shows a typical rainfall volume distribution with the curve skewed towards the smaller events. The events lost due to flooding (with rainfall depths of 11, 15, 18, 18, 28, 32 and 40mm) would have extended the positive tail of the curve, and extended understanding of the system under high rainfall conditions. However, the results presented are thought to be a fair representation of the response of the car park structure to typical rainfall events over a prolonged period.

4.1.1 Rainfall and runoff parameters

From Table 4.1 the largest rainfall event recorded was 22.6mm, which produced between 73.3% and 84.8% runoff in the bays monitored. The smallest rainfall event processed was 2.75mm for which no runoff was recorded.

The averages for all the events monitored were:

Mean rainfall = 8.08mm

Mean runoff:	Bay 1, gravel	= 3.36mm (36.6% of rainfall)
	Bay 2, b.f.s.	= 3.28mm (34.0%)
	Bay 3, granite	= 4.27mm (46.7%)
	Bay 4, limestone	= 3.97mm (45.0%).

This seems to indicate that bay 2, (with the blast furnace slag sub-base), was the most efficient at retaining stormwater and attenuating the discharge hydrograph, followed by the gravel, limestone and lastly granite, with an average difference of 1.0mm (or 14% of average rainfall) between the b.f.s. and the granite.

Individual plots of rainfall versus runoff for each of the 4 bays are presented in Figures 4.2, 4.3, 4.4, and 4.5 for the gravel, b.f.s., granite and limestone respectively.

* Footnote: These runoff figures relate to the runoff (and percentage runoff) from the events that were recorded for that bay, whereas the average rainfall is for all 62 recorded events. The figure for the gravel is flattering as the 20 events at the end of the data set, for which no data was recorded for the gravel bay, generally produced high runoff.

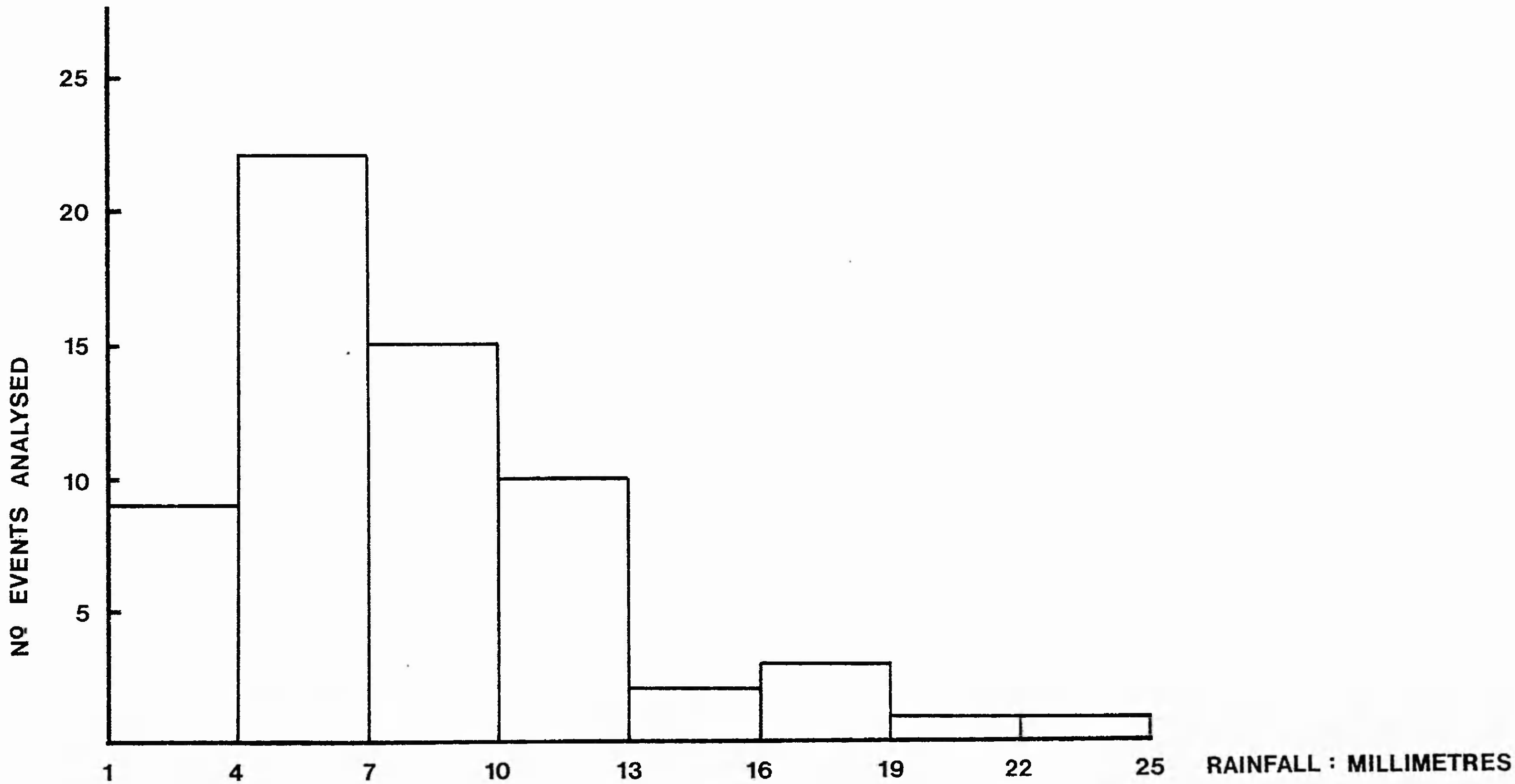


Figure 4.1 Frequency distribution histogram for rainfall-runoff events recorded.

Table 4.1 Full data set for the 62 rainfall-runoff events monitored for the permeable pavement.

EVENT	=	Event series No. and Julian day code e.g. 8023 is Julian day 23, 23rd January, 1988. (.5 = 2nd event, for the same day).
Rf	=	Rainfall, recorded in millimetres.
GVL%RO	=	Gravel bay. Runoff as a % of rainfall
BFS%RO	=	B.F.S bay. Runoff as a % of rainfall
GNT%RO	=	Granite bay. Runoff as a % of rainfall
LST%RO	=	Limestone bay. Runoff as a % of rainfall
24ANTMM	=	Antecedent rainfall in the 24 hours before observation event. Expressed in mm.
48ANTMM	=	Antecedent rainfall in the 48 hours before observation event. Expressed in mm.
ANTMM	=	Rainfall depth of antecedent event, in mm.
ANTHRS	=	Time from mid-point of antecedent event to beginning of rainfall for observation event.
24TEMP	=	Average temperature, in degrees celsius for the 24 hours before the reported event.
168TEMP	=	Average temperature, in degrees celsius for the week before the reported event.
GVLMMNRO	=	Gravel bay. Total losses, volume not recorded as runoff, in mm.
BFSMMNRO	=	BFS bay. Total losses, volume not recorded as runoff in mm.
GNTMMNRO	=	Granite bay. Total losses, volume not recorded runoff in mm.
LSTMMNRO	=	Limestone bay. Total losses, volume not recorded runoff in mm.
GVLMMRO	=	Gravel bay. Recorded runoff in mm.
BFSMMRO	=	BFS bay. Recorded runoff in mm.
GNTMMRO	=	Granite bay. Recorded runoff in mm.
LSTMMRO	=	Limestone bay. Recorded runoff in mm.
DURATION	=	Duration of rainfall, in minutes.
RFPINT	=	Rainfall peak intensity, in mm/hr, for 15 min resolution.
GVLINT	=	Gravel, runoff peak intensity, in mm/hr, for 15 min resolution.
BFSINT	=	B.F.S., runoff peak intensity, in mm/hr, for 15 min resolution.
GNTINT	=	Granite, runoff peak intensity, in mm/hr, for 15 min resolution.
LSTINT	=	Limestone, runoff peak intensity, in mm/hr, for 15 min resolution.

Table 4.1, part ii.

EVENT	RF	GVL%RO	BFS%RO	GNT%RO	LST%RO
1 7207	9.2	35.8	*	*	38.9
2 7221	3.9	0	0	0	0
3 7225	21.5	70.8	55.6	66.9	*
4 7230	10	41.2	39.4	52.2	*
5 7235	10.1	51.9	51.8	59.5	65.9
6 7238	8.5	44.1	41.5	49.2	59
7 7249	12.4	51.7	48.9	61.1	69.5
8 7262	7.6	36.6	35.7	48.4	55.3
9 7277	11	25.8	28.8	34.2	35
10 7280	3.3	20.4	48.4	59.5	39.6
11 7282	13.3	48	48.8	54.8	61.2
12 7283	16.5	71.1	62.4	71.2	83.3
13 7287	13.1	55.7	48.4	59.5	66.7
14 7293	6.4	23.4	32.3	51.1	34.1
15 7300	12.5	52.9	49.8	60.4	*
16 7304	7.5	27.6	33.2	44.8	41.1
17 7312	2.75	0	0	0	0
18 7314	3	0	4.7	20.1	6.9
19 7315	8.6	54.9	51.3	59.6	62.5
20 7319	5.4	13	0	25.8	30.8
21 7323	9.1	45.3	32.8	41.4	50.7
22 7328	3.75	50.9	40.1	48	67.8
23 7349	9	22.1	18	24.6	23.6
24 7350	4.7	53.2	56.6	66.8	66.9
25 7350.5	3.1	44.1	12	61.6	21
26 8008	6.4	34.2	26.2	37.5	51.9
27 8022	8.2	22.4	12.8	38.8	41.6
28 8031	6.9	32.8	14.3	41.6	31.1
29 8035	10.3	50.3	37.6	*	45.2
30 8035.5	6.1	60.8	44.7	68.6	64.2
31 8038	5.35	19.8	3.8	28.4	10.1
32 8045	4.4	48.5	35.9	68.1	56.7
33 8063	3	2.4	0	3.3	0
34 8069	5.6	18.4	0	16	15.2
35 8072	8.4	20.2	11.3	36.8	14.4
36 8073	8.3	70.3	46	87.2	85.8
37 8078	2.7	0	0	0	0
38 8078.5	4	42.6	30.8	61.4	57.4
39 8086	6.7	31	15.7	*	12.2
40 8094	17	49.2	41.6	62.3	53.7
41 8107	6.6	29.3	25.5	47.4	61.9
42 8125	4.7	65.9	64.3	96.3	85
43 8152	8.2	*	*	59.2	50.2
44 8155	6.9	*	*	51.6	68.3
45 8160	8.4	*	*	54.4	50
46 8178	10.9	*	*	40.6	27.5
47 8186	10.5	*	100.5	92	100.3
48 8186.5	2.9	*	36.9	49	40.1
49 8287	9.4	*	24.6	37.2	35.2
50 8188	4.7	*	32.6	51.2	43.3
51 8188.5	6.6	*	54.6	68.1	72
52 8189	6.9	*	54.6	69.3	67.4
53 8195	6.6	*	36.9	55.1	49.1
54 8195.5	5.7	*	46.7	52.7	56
55 8198	17	*	53.9	64.2	62.2
56 8203	4.4	*	*	11.8	11.7
57 8203.5	22.6	*	73.3	80.1	84.8
58 8204	10.7	*	48.7	55.1	55
59 8212	5.2	*	0	0	0
60 8231	4.6	*	0	4	3
61 8268	8.8	*	43.3	46.5	52.5
62 8271	9.24	*	48	56	59.6

Table 4.1, part iii.

EVENT	24ANTMM	48ANTMM	ANTMM	ANTHRS	24TEMP	168TMP
1 7207	0	0	1	53	13.7	14.5
2 7221	0	0	1	170	13.5	14.2
3 7225	2	2	2	22	18.9	15.1
4 7230	0	0	21	110	19.7	19.2
5 7235	3.3	3.3	3	20	16.1	20.6
6 7238	0	2.5	2	28	13.6	17.6
7 7249	1	4	4	26	13.7	16.7
8 7262	0	5	15	60	12.3	14.8
9 7277	0	0	1	240	12.6	12.4
10 7280	2	2	2	7	14.4	12.8
11 7282	1	1.5	1	13	10.9	11.9
12 7283	14.3	14.8	12	8	10.9	11.9
13 7287	0	0	20	56	9	8.9
14 7293	0	2.7	2	31	12.9	11.6
15 7300	3	0	3	10	11.3	8.9
16 7304	0	0	12	86	11.3	9
17 7312	0	0	9	200	5.7	7.7
18 7314	0	3	3	39	6.4	7.4
19 7315	5	5	2	10	7.4	7.4
20 7319	0	0	9	87	8.8	6.9
21 7323	2	2	2	10	9.3	8.9
22 7328	6.5	6.5	6	10	7	7.9
23 7349	0	0	1	250	2.3	2.9
24 7350	9	9	9	12	9.5	3.9
25 7350.5	14	14	5	5	9.5	3.9
26 8008	0	0	7	57	5	5.5
27 8022	2	0	2	7	2.5	5
28 8031	0	0	2	60	5.2	5.2
29 8035	0	0	6	74	5.1	5.9
30 8035.5	11.9	11.9	10	9	5.1	5.9
31 8038	0	0	6	72	3.7	5.2
32 8045	2.3	2.3	2	20	9.4	5.4
33 8063	0	0	2	125	4.4	4.5
34 8069	0	0	1	77	9	6.1
35 8072	4	4	3	5	5.9	7.5
36 8073	6	9	7	20	4.7	7.1
37 8078	0	5	2	62	7.3	6.5
38 8078.5	2.7	7.7	3	4	7.3	6.5
39 8086	0	12	12	40	7.8	8.3
40 8094	0	0	2	108	11.6	9.2
41 8107	2	2	1	8	14	9.9
42 8125	3	4	1	2	11.6	11.5
43 8152	3	5	1	4	12.5	14.4
44 8155	0	0	8	72	12.6	14.4
45 8160	0	0	1	84	13.2	14.6
46 8178	1	1	1	420	16.6	19.3
47 8186	7	7	3	9	16.4	17.2
48 8186.5	4	6	2	6	16.4	17.2
49 8287	10	18	10	22	16.6	17.3
50 8188	10	20	10	23	15.5	16.9
51 8188.5	5	15	5	4	15.5	16.9
52 8189	7	12	7	22	15.5	16.1
53 8195	3	3	3	3	17	15.9
54 8195.5	7	7	6	9	17	15.9
55 8198	0	2	2	44	14.6	15.7
56 8203	0	0	17	96	18.3	16.9
57 8203.5	5	5	5	17	18.3	16.9
58 8204	14	23	23	22	18.5	17.3
59 8212	0	.7	1	24	14.2	15.4
60 8231	0	0	1	96	18.6	17.5
61 8268	2	5	3	7	13.9	15.5
62 8271	2	6	1	8	15.9	13.7

Table 4.1, part iv.

EVENT	GVLMMNRO	BFSMMNRO	GNTMMNRO	LSTMMNRO
1 7207.0	5.9	*	*	5.6
2 7221.0	3.9	3.9	3.9	3.9
3 7225.0	6.3	9.5	7.1	*
4 7230.0	5.9	6.1	4.8	*
5 7235.0	4.9	4.9	4.1	3.4
6 7238.0	4.8	5.0	4.3	3.5
7 7249.0	6.0	6.3	4.8	3.8
8 7262.0	4.8	4.9	3.9	3.4
9 7277.0	8.2	7.8	7.2	7.2
10 7280.0	2.6	1.7	1.3	2.0
11 7282.0	6.9	6.8	6.0	5.2
12 7283.0	4.8	6.2	4.8	2.8
13 7287.0	5.8	6.8	5.3	4.4
14 7293.0	4.9	4.3	3.1	4.2
15 7300.0	5.9	6.3	4.9	*
16 7304.0	5.4	5.0	4.1	4.4
17 7312.0	2.8	2.8	2.8	2.8
18 7314.0	3.0	2.9	2.4	2.8
19 7315.0	3.9	4.2	3.5	3.2
20 7319.0	4.7	5.4	4.0	3.7
21 7323.0	5.0	6.1	5.3	4.5
22 7328.0	1.8	2.2	2.0	1.2
23 7349.0	7.0	7.4	6.8	6.9
24 7350.0	2.2	2.0	1.6	1.6
25 7350.5	1.7	2.7	1.2	2.4
26 8008.0	4.2	4.7	4.0	3.1
27 8022.0	6.4	7.2	5.0	4.8
28 8031.0	4.6	5.9	4.0	4.8
29 8035.0	5.1	6.4	*	5.6
30 8035.5	2.4	3.4	1.9	2.2
31 8038.0	4.3	5.1	3.8	4.8
32 8045.0	2.3	2.8	1.4	1.9
33 8063.0	2.9	3.0	2.9	3.0
34 8069.0	4.6	5.6	4.7	4.7
35 8072.0	6.7	7.5	5.3	7.2
36 8073.0	2.5	4.5	1.1	1.2
37 8078.0	2.7	2.7	2.7	2.7
38 8078.5	2.3	2.8	1.5	1.7
39 8086.0	4.6	5.6	*	5.9
40 8094.0	8.6	9.9	6.4	7.9
41 8107.0	4.7	4.9	3.5	2.5
42 8125.0	1.6	1.7	0.2	0.7
43 8152.0	*	*	3.3	4.1
44 8155.0	*	*	3.3	2.2
45 8160.0	*	*	3.8	4.2
46 8178.0	*	*	6.5	7.9
47 8186.0	*	-0.1	0.8	-0.0
48 8186.5	*	1.8	1.5	1.7
49 8287.0	*	7.1	5.9	6.1
50 8188.0	*	3.2	2.3	2.7
51 8188.5	*	3.0	2.1	1.8
52 8189.0	*	3.1	2.1	2.2
53 8195.0	*	4.2	3.0	3.4
54 8195.5	*	3.0	2.7	2.5
55 8198.0	*	7.8	6.1	6.4
56 8203.0	*	*	3.9	3.9
57 8203.5	*	6.0	4.5	3.4
58 8204.0	*	5.5	4.8	4.8
59 8212.0	*	5.2	5.2	5.2
60 8231.0	*	4.6	4.4	4.5
61 8268.0	*	5.0	4.7	4.2
62 8271.0	*	4.8	4.1	3.7

Table 4.1, part v.

EVENT	GVLMMRO	BFSMMRO	GNTMMRO	LSTMMRO
1 7207.0	3.3	*	*	3.6
2 7221.0	0.0	0.0	0.0	0.0
3 7225.0	15.2	12.0	14.4	*
4 7230.0	4.1	3.9	5.2	*
5 7235.0	5.2	5.2	6.0	6.7
6 7238.0	3.7	3.5	4.2	5.0
7 7249.0	6.4	6.1	7.6	8.6
8 7262.0	2.8	2.7	3.7	4.2
9 7277.0	2.8	3.2	3.8	3.8
10 7280.0	0.7	1.6	2.0	1.3
11 7282.0	6.4	6.5	7.3	8.1
12 7283.0	11.7	10.3	11.7	13.7
13 7287.0	7.3	6.3	7.8	8.7
14 7293.0	1.5	2.1	3.3	2.2
15 7300.0	6.6	6.2	7.6	*
16 7304.0	2.1	2.5	3.4	3.1
17 7312.0	0.0	0.0	0.0	0.0
18 7314.0	0.0	0.1	0.6	0.2
19 7315.0	4.7	4.4	5.1	5.4
20 7319.0	0.7	0.0	1.4	1.7
21 7323.0	4.1	3.0	3.8	4.6
22 7328.0	1.9	1.5	1.8	2.5
23 7349.0	2.0	1.6	2.2	2.1
24 7350.0	2.5	2.7	3.1	3.1
25 7350.5	1.4	0.4	1.9	0.7
26 8008.0	2.2	1.7	2.4	3.3
27 8022.0	1.8	1.0	3.2	3.4
28 8031.0	2.3	1.0	2.9	2.1
29 8035.0	5.2	3.9	*	4.7
30 8035.5	3.7	2.7	4.2	3.9
31 8038.0	1.1	0.2	1.5	0.5
32 8045.0	2.1	1.6	3.0	2.5
33 8063.0	0.1	0.0	0.1	0.0
34 8069.0	1.0	0.0	0.9	0.9
35 8072.0	1.7	0.9	3.1	1.2
36 8073.0	5.8	3.8	7.2	7.1
37 8078.0	0.0	0.0	0.0	0.0
38 8078.5	1.7	1.2	2.5	2.3
39 8086.0	2.1	1.1	*	0.8
40 8094.0	8.4	7.1	10.6	9.1
41 8107.0	1.9	1.7	3.1	4.1
42 8125.0	3.1	3.0	4.5	4.0
43 8152.0	*	*	4.9	4.1
44 8155.0	*	*	3.6	4.7
45 8160.0	*	*	4.6	4.2
46 8178.0	*	*	4.4	3.0
47 8186.0	*	10.6	9.7	10.5
48 8186.5	*	1.1	1.4	1.2
49 8287.0	*	2.3	3.5	3.3
50 8188.0	*	1.5	2.4	2.0
51 8188.5	*	3.6	4.5	4.8
52 8189.0	*	3.8	4.8	4.7
53 8195.0	*	2.4	3.6	3.2
54 8195.5	*	2.7	3.0	3.2
55 8198.0	*	9.2	10.9	10.6
56 8203.0	*	*	0.5	0.5
57 8203.5	*	16.6	18.1	19.2
58 8204.0	*	5.2	5.9	5.9
59 8212.0	*	0.0	0.0	0.0
60 8231.0	*	0.0	0.2	0.1
61 8268.0	*	3.8	4.1	4.6
62 8271.0	*	4.4	5.2	5.5

Table 4.1, part vi.

EVENT	DURATION	RFPINT	GVLPINT	BFSPINT	GNTPINT	LSTPINT
1 7207.0	342.0	3.3	1.0	*	*	1.5
2 7221.0	147.0	3.7	0.0	0.0	0.0	0.0
3 7225.0	266.0	12.1	7.6	5.0	6.3	*
4 7230.0	219.0	20.2	1.8	1.5	2.3	*
5 7235.0	145.0	12.6	2.7	3.2	3.2	4.2
6 7238.0	189.0	5.5	1.9	1.9	2.2	3.0
7 7249.0	438.0	6.0	1.8	2.0	2.2	2.8
8 7262.0	327.0	5.8	1.2	1.3	1.7	2.4
9 7277.0	309.0	10.0	1.4	1.6	1.9	2.3
10 7280.0	51.0	5.5	0.5	0.4	0.6	0.6
11 7282.0	311.0	14.3	1.8	3.0	2.4	3.5
12 7283.0	949.0	3.0	1.7	1.7	1.6	2.1
13 7287.0	461.0	4.5	2.4	2.4	2.6	2.9
14 7293.0	330.0	2.0	0.6	0.6	0.8	0.8
15 7300.0	236.0	11.6	3.8	3.4	4.3	*
16 7304.0	497.0	3.3	0.7	0.8	0.9	1.3
17 7312.0	125.0	2.0	0.0	0.0	0.0	0.0
18 7314.0	103.0	3.0	0.2	0.3	0.3	0.6
19 7315.0	343.0	7.5	2.2	2.4	2.2	2.1
20 7319.0	210.0	3.5	0.3	0.2	0.4	0.4
21 7323.0	188.0	4.0	0.8	0.6	0.9	1.0
22 7328.0	188.0	2.5	0.8	0.6	0.9	1.0
23 7349.0	592.0	3.1	0.9	0.7	0.9	1.2
24 7350.0	133.0	4.1	1.1	1.1	1.1	1.3
25 7350.5	131.0	3.6	0.4	0.5	0.7	0.6
26 8008.0	139.0	4.2	1.2	1.1	1.3	1.2
27 8022.0	293.0	4.8	0.6	0.4	0.7	0.9
28 8031.0	245.0	4.2	1.3	1.0	1.5	1.4
29 8035.0	216.0	9.3	2.8	2.1	*	3.6
30 8035.5	44.0	12.4	3.1	3.0	3.5	4.1
31 8038.0	195.0	5.3	0.6	0.3	0.3	0.6
32 8045.0	224.0	2.7	0.8	0.8	1.0	1.1
33 8063.0	40.0	4.6	0.2	0.0	0.3	0.1
34 8069.0	247.0	5.1	0.5	0.1	0.4	0.5
35 8072.0	795.0	4.2	0.3	0.3	0.5	0.4
36 8073.0	390.0	3.2	0.8	0.7	0.9	1.0
37 8078.0	178.0	1.5	0.1	0.1	0.1	0.1
38 8078.5	263.0	1.5	0.5	0.5	0.7	0.7
39 8086.0	301.0	6.1	0.5	0.4	*	0.3
40 8094.0	490.0	9.9	1.8	2.0	2.7	2.8
41 8107.0	182.0	4.2	0.9	0.7	1.3	1.6
42 8125.0	137.0	11.8	2.2	3.2	4.1	4.0
43 8152.0	241.0	9.6	*	*	2.0	1.7
44 8155.0	25.0	21.1	*	*	2.5	3.6
45 8160.0	265.0	6.1	*	*	2.9	2.5
46 8178.0	259.0	13.0	*	*	2.5	1.5
47 8186.0	49.0	33.1	*	13.2	7.2	8.8
48 8186.5	122.0	4.9	*	0.5	0.6	0.6
49 8287.0	124.0	21.7	*	1.5	1.8	1.8
50 8188.0	94.0	8.7	*	0.9	1.3	1.1
51 8188.5	116.0	12.5	*	2.6	2.7	2.5
52 8189.0	104.0	15.0	*	3.6	4.0	3.0
53 8195.0	117.0	6.3	*	1.6	2.0	1.8
54 8195.5	30.0	19.5	*	1.9	1.9	1.8
55 8198.0	519.0	4.3	*	3.7	3.8	3.7
56 8203.0	133.0	3.2	*	*	0.2	0.3
57 8203.5	446.0	13.2	*	6.8	6.8	7.0
58 8204.0	296.0	16.7	*	3.4	4.0	3.4
59 8212.0	78.0	11.2	*	0.0	0.1	0.0
60 8231.0	134.0	3.5	*	0.0	0.1	0.1
61 8268.0	178.0	31.1	*	4.0	3.8	3.7
62 8271.0	222.0	8.5	*	4.1	4.0	3.4

```
        -
    15.0+                                                        *
        -
GVLMMRO -
        -                                          *
        -
    10.0+
        -                             *
        -                                            *
        -                               **
        -                   *          *  *
     5.0+                    *   2
        -            *      *  *  *
        -        2        *   *    *
        -      2 *    *2** ** *
        -    2     2* *
     0.0+   22 *
          ----+---------+---------+---------+---------+---------+--RF
            3.5       7.0      10.5      14.0      17.5      21.0
       N* = 20
```

Figure 4.2 Plot of runoff against rainfall for the gravel sub-base bay. Both axes in mm.

```
        -
    18.0+
        -                                                      *
BFSMMRO -
        -
        -
    12.0+                                                   *
        -                       *              *
        -                           *            *
        -
        -                                        *
     6.0+                            2 2
        -                   ** * *
        -         *   **   **  ** *
        -         * **2  2   **
        -    ******   22  ***
     0.0+    4* ** 3*
          ------+---------+---------+---------+---------+---------+RF
              4.0       8.0      12.0      16.0      20.0      24.0
       N* = 7
```

Figure 4.3 Plot of runoff against rainfall for the b.f.s. sub-base bay. Both axes in mm.

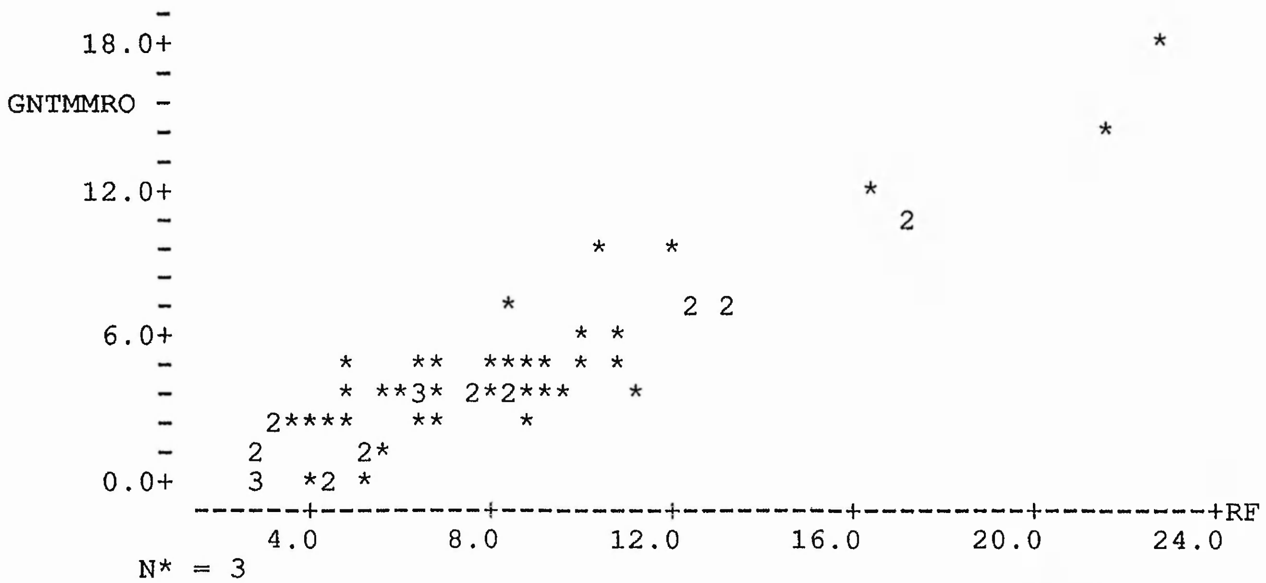


Figure 4.4 Plot of runoff against rainfall for the granite sub-base bay. Both axes in mm.

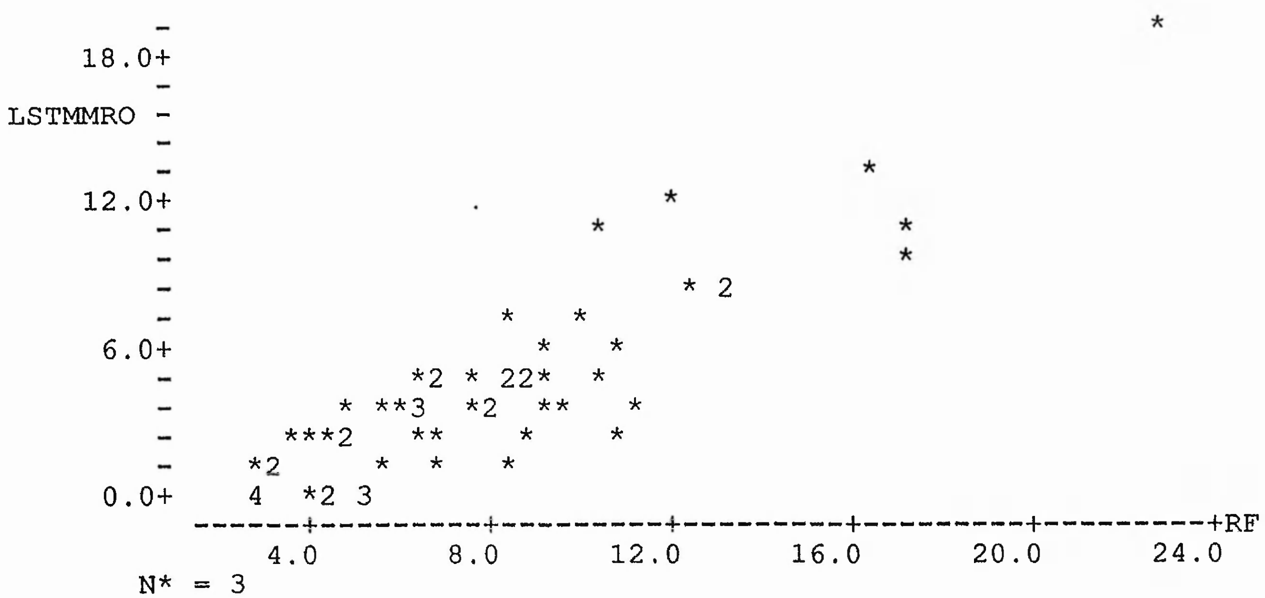


Figure 4.5 Plot of runoff against rainfall for the limestone sub-base bay. Both axes in mm.

A least squares regression analysis of each plot using a computer-based statistical package, (Anon, 1986), resulted in the following sets of equations for the relationship between the rainfall and runoff (as defined above):

GVLMMRO = 0.69*Rf - 2.10	R^2 = 85.1%
BFSMMRO = 0.68*Rf - 2.20	R^2 = 81.5%
GNTMMRO = 0.76*Rf - 1.82	R^2 = 86.4%
LSTMMRO = 0.81*Rf - 2.27	R^2 = 79.8%

(Where: 'GVL','BFS','GNT', and 'LST' are abbreviations for the sub-base stones, gravel, blast furnace slag, granite and limestone respectively; 'MMRO' is short for 'millimetres runoff' as an equivalent depth for the area of each bay, (thus 'GVLMMRO' is the volume runoff from the gravel bay expressed as the equivalent depth in mm for the area of the bay); and 'Rf' is the depth of rainfall (in millimetres). The coefficient of determination (R^2) for the equations is shown to be between 78.7% and 86.1%, i.e. approximately 80% of the variation in the monitored runoff can be explained by the variation in rainfall depth.

The coefficient for rainfall for each of the four bays is seen to vary between 0.68 for the B.F.S. bay to 0.81 for the limestone bay. Substituting nil as the value for the 'MMRO' of each bay allows the calculation of the depth of rainfall that can be accommodated, on average, before allowing any runoff, the resulting values are:

gravel	= 3.0 mm
b.f.s.	= 3.2 mm
granite	= 2.4 mm
limestone	= 2.8 mm

These figures are, in effect, a graph average of the observations of the depth of rainfall required before runoff began. Event numbers 7221, 7312, 8063 and 8212, with rainfall depths of 3.9, 2.75, 3.0 and 5.2mm respectively, all failed to produce runoff in some or all of the bays. Conversely, the events 7280, 7350.5 and 8078.5 had rainfall depths of 3.9, 3.1 and 4.0 mm, and yet resulted in between 15% and 60% runoff. This shows that, although there was a good relationship between the depth of rainfall and the volume of runoff, other factors were required to be taken into account and examined further.

Throughout this and other Chapters, the figures for average rainfall and runoff refer to a mean of the discrete events for which data was analysed. In between many of these events were scores of smaller episodes of drizzle or light rainfall which produced little or no runoff. Therefore, over a long period of time, such as a year, the total percentage runoff from all rainfall would have been less than the figures for individual events presented here.

4.1.2 Antecedent rainfall

The antecedent rainfall conditions were seen to play an important part in the relationship between rainfall and runoff. In the examples above, for those small events producing almost no runoff, there had been no rainfall for the preceding 24 hours, and only 0.7 mm rainfall for event 8212 in the preceding 48 hours. For those small events producing between 15% and 60% run off, between 2 and 9mm depth of rainfall had been recorded within the previous 12 hours. These figures indicated that conditions antecedent to the rainfall event were important in the resulting volume of runoff from that event.

The effects of long, dry periods on larger rainfall events was less certain, but it was useful to compare events of similar magnitudes but with varying antecedent conditions. Events 7230 and 7235 were of 10.0mm and 10.1mm rainfall depth respectively, (and occurred within 5 days of each other): however, the average percentage runoff from each bay was between 7.4% and 12.5% greater for the second event than the first. A comparison of the antecedent conditions for both events showed that for 7230 the preceding 4.5 days were dry, whilst for 7235, 3.3mm rainfall was recorded in the previous 24 hours. A further example is given in Table 4.2.

Table 4.2. Comparison of events of similar rainfall volume but with differing antecedent conditions.

Event No.	Rainfall	Runoff for bay:				Antecedent Event:	
		1	2	3	4	rainfall	time separation
	mm	%				mm	hours
7323	9.1	45	33	41	51	2	10
7349	9.0	22	18	25	24	1	250

An examination of the relationship between antecedent conditions the rainfall-runoff relationship is made in Section 7.1.4.

4.1.3 Temperature

A second important factor in the relationship between rainfall and runoff was thought to be temperature (and sunshine). A high ambient temperature should have assisted in the evaporation of water held in the structure and generally increased the volume of precipitation returned to the atmosphere. However, the difference in rainfall patterns between summer and winter (high and low temperature seasons) make it difficult to compare events from the two periods: summer events tend to be of short duration and high intensity, winter events being longer and of relatively low intensity. (A more rigorous examination of the relationship between runoff and evaporation is made in Section 7.1.2).

4.1.4 Rainfall and runoff intensity

In all cases rainfall and runoff intensity was calculated and recorded in millimetres per hour. However, it is important to always define the time resolution for the given data. Thus, in Table 4.1 part vi, the quoted peak rainfall and runoff intensities are averages over 15 minute time periods: different values would result from calculations for a different time resolution. This part of the Table serves to show how successful the permeable pavement was at attenuating peak flows, delaying their occurrence and reducing their intensity. The mean value, for the maximum rainfall intensity, for all events was 7.8mm/h, whilst the mean of all the peak runoff rates from the bays was below 2.0 mm/h or approximately 8 litres per hour. This compares favourably with urban areas of similar dimensions, (Watkins 1962). Again, the performance of the gravel bay is not strictly comparable with the others, as the 20 events not monitored for the gravel had an average higher than the 75th percentile of maximum rainfall intensity.

No detailed analysis of peak flow reduction and attenuation is described here, because of analysis in Chapter 5 which shows how discharge control may be used to modify peak flows to suit individual site requirements. N.B. strictly speaking, rainfall and runoff intensities used here are the peak rates, however, in all but a few cases, the events with high maximum intensities were those with high mean intensities. The exceptions are those events of long duration and low mean intensities, which had a period of high intensity rainfall.

After processing a limited amount of data it became apparent that rainfall intensity within an event had an effect on the percentage runoff, i.e. an event with a high rainfall intensity was likely to result in a greater percentage runoff than a similar sized event with a low rainfall intensity. Initially, this was perceived intuitively, but as data accumulated it could be shown quantitively: Table 4.3 compares two events of similar rainfall depth but with very different maximum rainfall intensities, (the pairs of events had similar antecedent conditions).

Table 4.3. Comparison of two pairs of similar events with differing maximum rainfall intensities.

Event No.	Rainfall (mm)	% Runoff for bay: 1	2	3	4	Max.Rainfall Intensity (15min, mm/hr)
7235	10.1	52	51	59	66	12.6
8186	10.4	*	100	92	100	33.1
8031	6.9	33	14	42	31	4.2
8155	6.9	*	*	52	68	21.1

Much of the effect of varying rainfall intensities was frequently masked by the antecedent rainfall conditions. It is difficult to find many examples such as those in Table 4.3 because of the various combinations of rainfall, antecedent rainfall and rainfall intensity.

After reviewing the reasons for the apparent runoff increase and after examination of the data, it was noticed that the high intensity rainfall events were also those of somewhat shorter duration. Mechanisms to explain the observed results were postulated. This is best explained by reversing the original statement: thus, longer rainfall events, commonly of lower rainfall intensity, give rise to lower runoff volumes than the shorter, higher intensity events of the same rainfall depth. To account for these observations the following processes were proposed:

1. The artificial values chosen for the definition of the start and end of runoff served to depress the runoff volumes calculated for low intensity events, whilst high intensity, 'peaky' runoff has more of its runoff captured by the computing process.

2. One or more processes were operating which resulted in a 'continuing loss' function whilst rainfall proceeded.

These proposals are examined in the following two sections: firstly, by examination of the runoff decay curves, (Section 4.1.5); and secondly, by reference to possible continuing loss processes, (Section 4.1.6).

4.1.5 Runoff decay curves

The first argument, regarding the 'artificial' definition of 0.25mm/h for the start and end of runoff, can be examined by looking at the 'unrecorded' part of the runoff curves i.e. that runoff occurring after the flow rate drops below 0.25 mm/h. The initial examination of runoff decay or recession curves was stimulated by early attempts at modelling the behaviour of the runoff. For this examination, the 'post-rainfall' runoff rate from events covering the full range of recorded rainfall and runoff intensities and volumes, (twenty events in all), was plotted against time, in an attempt to find a single recession curve for each sub-base stone of the type exhibited for the drainage for larger streams and natural catchments by streams and rivers.

The runoff, in millimetres per hour, was plotted against time, on semi-log graph paper. A straight line relationship would have indicated a typical, natural log decay , (which could have been incorporated as an equation into a computer model). Straight line relationships were sometimes found for those parts of the 'decay curve' above the threshold for recording runoff, but they only continued for periods of approximately 2 hours, then the recession seemed to adopt a secondary, lower gradient, straight line relationship for another 2 hours, possibly followed by a third.

An unsupported theory to explain this series of apparent relationships was that each straight line reflected the drainage from a different part, or parts, of the pavement: the early line related to the simple throughput of the last part of rainfall over the saturated surfaces of stones; a later line reflecting the drainage of the sub-base; and a still later line exhibiting the drainage of the bedding layer and sub-base. However, the variety of the curves and the lack of straight line relationships for many recessions implied that such a simplistic explanation was probably a poor representation of the processes involved.

By and large, all recession curves below the threshold of 0.25mm/h, for the same sub-base, were of the same gradient and of a similar shape. Figure 4.6 shows the 'average' curves obtained by 'eye' from the collection of curves for each sub-base. Using these curves enabled a calculation of the 'average, unaccounted for runoff', i.e. that runoff occurring at a rate less than 0.25 mm/h. Summarising these were:

A. Gravel: a time of 1.75 hours for runoff rate to decay from 0.25 to 0.1 mm/h, during which time a further 0.25mm runoff was discharged, and from 0.25 to 0.01mm/h an estimated 0.35mm;

B. B.F.S.: a time of 2 hours for runoff rate to decay from 0.25 to 0.1 mm/h, during which time a further 0.27mm runoff was discharged, and from 0.25 to 0.01mm/h an estimated 0.37mm;

C. Granite: a time of 2.25 hours for runoff rate to decay from 0.25 to 0.15 mm/h, during which time a further 0.48mm (plus or minus 0.1mm), runoff was discharged, and from 0.25 to 0.01mm/h an estimated 0.68mm was discharged;

D. Limestone: a time of 2.25 hours for runoff rate to decay from 0.25 to 0.15 mm/h, during which time a further 0.41mm runoff was discharged, and from 0.25 to 0.01mm/hr an estimated 0.61mm was discharged.

The variation in the bands of runoff rate given above, (0.25 to 0.1 or to 0.15 mm/h), are related to the levels of confidence with which the lower levels of the average runoff are plotted. Below these levels, i.e. down to 0.01mm/h the totals are only estimated as the degree of variability for the decay curves below 0.1mm/h was marked. The larger figures for the granite and limestone reflect the often extended lag, or residual, runoff which could be observed continuing for several days after some events or particularly wet periods.

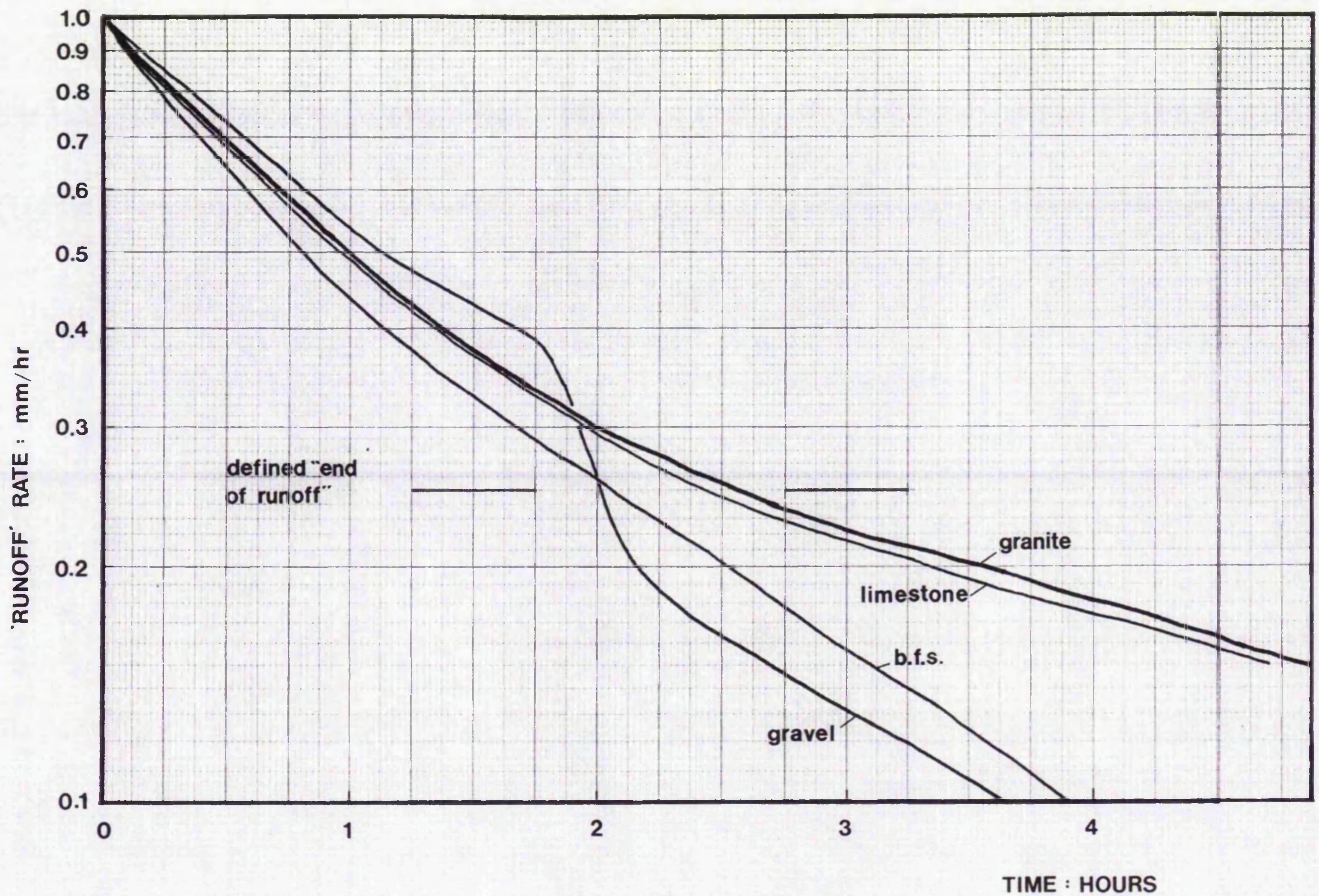


Figure 4.6 Post-rainfall, runoff decay curves.

The range qualifier of plus or minus 0.1mm for granite is given because the variation in the recession curves for granite was far greater than that for the other curves, (this variation is not shown in Figure 4.6). Also, the recession curve for the gravel runoff may appear to be a function of a mathematical quirk of the averaging process. In fact, all the decay curves for the gravel displayed the same 'breaks' in gradient, i .e. a steepening of the recession curve when the runoff rate fell below 0.4 mm/h, and a 'shallowing out' of gradient between 0.2 and 0.1 mm/h. The initial break in curve may have been due to aspects of the gravel grading, or texture, as discussed in Section 7.2.4, which had different attenuation characteristics from the other, larger and rougher sub-base stones.

The figures for the additional, unrecorded, runoff varied between 0.25mm and 0.48mm for up to 2.25 hours after the defined 'end of runoff', at which time the runoff rates were at, or below, 0.15mm/h, or approximately 6 litres per hour. In some cases, runoff may have continued, at an ever-decreasing trickle, for several days, but this would not have significantly altered the figures for 'runoff' as defined in this Chapter. Moreover, the variation in the additional runoff, if included in the runoff for each sub-base and for each event, would only have served to increase the differences between the 'better' and 'poorer' sub-base stones, with respect to their water attenuation/ retention characteristics.

Finally, with respect to the argument that the definition of start and end of runoff may have served to depress the calculated runoff volumes from low intensity events, examination of 'unrecorded' runoff showed that the decay in the rate of runoff for all events in each sub-base was similar. The definition chosen did not alter any derived relationships between rainfall and runoff, nor did it disguise any meaningful difference in the retention characteristics of the individual sub-base stones.

4.1.6 Continuing losses

The second argument proposed in Section 4.1.4 contended that several 'continuing loss' processes that could be causing high losses in rainfall events of longer than average duration. Processes that could be considered were:

- Evaporation;
- Filling of block or stone porosity;
- Increasing wetted areas or wetting of stone surfaces; and,
- Experimental error.

Each one of these processes describes a method by which rainfall may not have been 'logged' as runoff and, therefore, may account for the calculated losses. All four of these possibilities form the basis of the further experiments and analysis discussed in Chapter 6.

4.2 Roof Runoff Infiltration Results

The various infiltration devices were constructed in July 1987: a stone-filled soakaway; a dual chambered soakaway; and an infiltration trench. The development of measuring equipment and of observation techniques evolved over a period of time, as an understanding of the fluctuations of the system also evolved. The final methodologies for data collection were described in Section 3.2.

The following Sections describe the processing of the collected data and compare the hydrological performance of each structure. The last Section gives the results from infiltration tests carried out at the site. Care must be taken when interpreting or extrapolating from the results presented throughout this Section to more general applications. Only one site was studied and some variation in lithology and hydrological parameters may occur across the site. In the first instance, the results are therefore site specific, although the generalisations from the data should have wider application.

4.2.1 Data processing

The roof runoff site was visited every 7 to 10 days , at which time a full set of water level and gauge readings were taken. Full tabulation of a section of the data from the beginning of June 1988 to early 1989 are presented in Tables 4.4, 4.5 and 4.6 for the stone-filled soakaway, dual-chambered soakaway and infiltration trench respectively. The columns of data in Tables 4.4 to 4.6 are:

- **a** Serial number and date of observation;
- **b** Number of days since the last observation was made;
- **c** The total rainfall, in millimetres, in the intervening period between 'a' and 'b';

d The dip tape reading, in centimetres, of distance to the surface of the water in the device from the datum at ground level, (a higher number indicated a lower water level);

e The change, in centimetres, between the water level in 'd' and the last observation;

f A conversion of the change in observed level to a calculated change in volume, (in litres), contained within the device;

g The number of tips recorded on the counter of the tipping bucket gauge measuring the roof runoff into the dual-chambered soakaway; h. a conversion of the number of tips in 'g' into the volume of roof runoff, in litres. The static calibration of the bucket indicated 3.24 litres were required to tip both buckets, equivalent to an increment of one on the tipping bucket counter;

h The volume calculated from multiplying the number of tips (G) by themeasured volume of the tipping bucket.

j The calculated loss, or volume of infiltrated water, in litres, from the device for the observation period. This was obtained by subtracting the input volume , in 'h', from the change in volume within the device, given in 'f' i.e. if the input from the roof runoff was calculated as 500 litres and the rise in level within the device only accounted for a change in volume held of 200 litres, then 300 litres was the 'loss', (or infiltrated volume), from the device.

During the period from the 3rd to 10th of November 1988, a series of infiltration tests were completed on the infiltration trench. A full discussion of the results of these tests is given in Section 4.2.4. The additional water used in testing the trench was drawn from the dual-chambered soakaway, which resulted in the observations from this period being un-useable for the general discussions below. Therefore, comparisons of individual readings and totals are made without inclusion of the above period.

Table 4.4 **Results from the stone-filled soakaway, June 1988 to February 1989. See notes for full explanation of the columns of data.**

Ser.Date A x: ddmmyy	Days B	Rf. C mm	Reading D cm	Change E cm	Vol. F l	Tips G	Vol. H l	Loss J l
1: 090688	36	50	80	-4	-45	462	1497	1542
2: 280688	19	15	103	-23	-265	120	390	680
3: 040788	6	25	80	23	265	174	564	299
4: 120788	8	78	45	35	403	766	2481	2078
5: 210788	9	36	46.5	-1.5	-17	435	1409	1426
6: 220788	1	23	40	6.5	75	258	836	761
7: 090888	18	16.5	72	-32	-368	166	538	906
8: 120888	3	10	77	-5	-58	83	269	327
9: 180888	6	2.4	74	3	35	7	23	-12
10:250888	7	11	75	-1	-11.5	81	262	250
11:310888	6	14	63	12	138	114	369	231
12:260988	26	34	83	-20	-230	260	842	1072
13:131088	17	39	66	17	196	330	1069	873
14:191088	6	2.5	88	-22	253	11	36	289
15:271088	8	23	74	14	161	216	700	539
16:031188	7	0.5	103	-29	-334	0	0	334
17:101188	7	1.8	125	-22	-253	3	10	263
18:291188	19	11	155	-30	-345	65	211	556
19:011288	2	33	85	70	805	259	839	34
20:211288	20	26	85	0	0	185	599	599
21:050189	16	6.5	120	-35	-403	56	172	575
22:240189	19	24	108	12	138	225	729	591
23:140289	21	21	*					

* - no data, unable to gain access to monitoring tube.

Table 4.5 **Results from the dual cavity soakaway, June 1988 to February 1989. See notes for explanation of the columns of data, n.b. D1, E1, & F1 refer to the first chamber, D2, E2, & F2 the second.**

Ser.Date	Days	Rf.	Reading	Change	Vol.	Tips	Vol.	Loss
A	B	C	D1/D2	E1/E2	F1/F2	G	H	J
x: ddmmyy		mm	cm	cm	1		1	1
1: 090688	36	50	55/77	-1/-7	-11/-105	462	1497	1613
2: 280688	19	15	62/110	-7/-33	-72/-495	120	390	957
3: 040788	6	25	53/92	9/18	99/270	174	564	195
4: 120788	8	78	38/30	15/62	165/930	766	2481	1386
5: 210788	9	36	45/37	-8/-7	-82/-105	435	1409	1596
6: 220788	1	23	23/16	22/21	242/315	258	836	279
7: 090888	18	16.5	73/72	-50/-58	-550/-870	166	538	1958
8: 120888	3	10	61/80	13/-8	143/-120	83	269	246
9: 180888	6	2.4	73/94	-13/-14	-143/-210	7	23	376
10:250888	7	11	74/109	-1/-15	-11/-225	81	262	498
11:310888	6	14	65/120	10/-11	105/-165	114	369	429
12:260988	26	34	84/158	-20/-38	-215/-570	260	842	1627
13:131088	17	39	72/170	12/-12	132/-180	330	1069	1117
14:191088	6	2.5	91/167	-19/3	-209/45	11	36	200
15:271088	8	23	78/162	13/5	143/75	216	700	482
16:031188	7	0.5	105/164	-27/-2	-297/-30	0	0	327
17:101188	7	1.8	*/168	*/-4	*/-60	3	10	70
18:291188	19	11	165/185	-60/-17	-660/-235	65	211	1106
19:011288	2	33	105/178	61/-7	665/-105	259	839	279
20:211288	20	26	113/162	-9/16	-94/240	185	599	453
21:050189	16	6.5	140/167	-27/-5	-297/-75	56	172	544
22:240189	19	24	131/172	9/-5	99/-75	225	729	705
23:140289	21	21	141/176	141/176	-110/-60	155	502	672

* - data lost because of infiltration tests.

Table 4.6 Results from the infiltration trench, June 1988 to February 1989. See notes for full explanation of the columns of data.

Ser.Date A x: ddmmyy	Days B	Rf. C mm	Reading D cm	Change E cm	Vol. F 1	Tips G	Vol. H 1	Loss J 1
1: 090688	36	50	36	0	0	462	1497	1500
2: 280688	19	15	40	-4	-60	120	390	450
3: 040788	6	25	15	25	375	174	564	189
4: 120788	8	78	38	-23	-345	766	2481	2800
5: 210788	9	36	36.5	1.5	23	435	1409	1400
6: 220788	1	23	22.5	14	210	258	836	615
7: 090888	18	16.5	51	-28.5	427	166	538	965
8: 120888	3	10	47	4	60	83	269	210
9: 180888	6	2.4	48	-1	-15	7	23	38
10:250888	7	11	47	1	15	81	262	247
11:310888	6	14	42.5	5.5	83	114	369	286
12:260988	26	34	39	3.5	53	260	842	789
13:131088	17	39	38	1	15	330	1069	1054
14:191088	6	2.5	48	10	150	11	36	186
15:271088	8	23	43	5	72	216	700	638
16:031188	7	0.5	55	*	*	0	0	*
17:101188	7	1.8	*	*	*	3	10	*
18:291188	19	11	55.5	-0.5	-7	65	211	218
19:011288	2	33	35.5	20	300	259	839	539
20:211288	20	26	41	-4.5	-67	185	599	666
21:050189	16	6.5	56	-15	-225	56	172	397
22:240189	19	24	40	16	240	225	729	489
23:140289	21	21	42	-2	-30	155	502	532

* - data lost because of infiltration tests.

4.2.2 Rainfall-runoff and infiltration relationships

The main control on the accuracy of the data presented was a mass balance consideration of input, storage and output to give an appraisal of the degree of confidence in the results. 'Input' was measured as runoff by the tipping bucket. 'Storage' was derived from measurement of the water levels in the devices. And 'output' or loss, i.e. the volume infiltrated by each device, was calculated as the difference between the 'input' and 'storage'.

Due to the loss of data from one or more of the devices from periods 17, 18 and 23 in Tables 4.4 to 4.6, all 'totals' given below exclude figures for these periods so that the totals for each device can be compared.

The total depth of the rainfall recorded for the period under consideration was 470mm. This corresponded to a volume of 20.05 m^3 incident on an area equivalent to the plan area of roof served by one of the infiltration devices. The total runoff volume calculated from the tipping bucket measurements for the same period was 16.62 m^3, which was approximately 68% of the rainfall.

A plot of the individual readings, for the rainfall against calculated runoff into each device, for the period is shown in Figure 4.7. The relationship is seen to be essentially linear, with a correlation coefficient between rainfall and runoff of 0.97 and the following regression equation:

$$\text{Runoff} = 0.75 * (\text{Rainfall}) - 1.89$$

A coefficient of 0.75 derived for rainfall can be considered to be lower than expected, (Parkar and Pratt, 1987). This could be due to 3 reasons. Firstly, only one side of the pitched roof of the building was included in the study, so that the prevailing winds at the time of rainfall could cause the depth of incident rainfall on the south-west facing half of the roof to be less than that on an equivalent flat or plan area. Secondly, as stated in Section 3.2, the tipping bucket measuring device was only calibrated statically, and this was likely to produce an under-estimation of runoff by up to 10%. Thirdly, the rather low runoff coefficient could be an accurate reflection of the particular roof concerned, indicating a 'depression' storage of over one millimetre which was evaporated between events.

All three of the above could be valid, although the first reason can be countered by the observation that the roof faces into the general direction of the prevailing winds. Further examination of the data showed that the period of highest percentage runoff (92%) occurred during a time of quite intense storms, whilst the periods of lowest percentage runoff, of 22% and 34%, occurred during periods of less than 1mm per week of rainfall. Also, an examination of the roof tiles themselves showed they were rough with a 'sandy' textured surface which would inhibit runoff and facilitate evaporation from their surface. These facts seem to support the latter theory of the rather low runoff coefficient being a property of the roof itself.

A further check on the accuracy of the data was obtained from the calculation of the balance of water entering, and the water lost, from each of the three devices. Tables 4.4 to 4.6 show that during the period discussed the calculated water losses from the stone-filled soakaway, dual-chambered soakaway and infiltration trench were 13.41, 15.27 and 13.65 m^3 respectively. However, the figure of 15.27 for the dual-chambered soakaway contains discrepancies as a result of two related factors: firstly, the infiltration tests during November 1988 resulted in the pumping of approximately 0.9 m^3 from the first chamber into the infiltration trench; and secondly, the levels of water within the two chambers at the end of the monitoring period indicated a difference, (loss), of 2.5 m^3 from the start of the monitoring period, of which 0.9 m^3 was caused by the pumping mentioned above. (The difference between the water levels for the stone-filled soakaway and the infiltration trench for the same period indicated losses of 0.32 m^3 and 0.09 m^3 respectively).

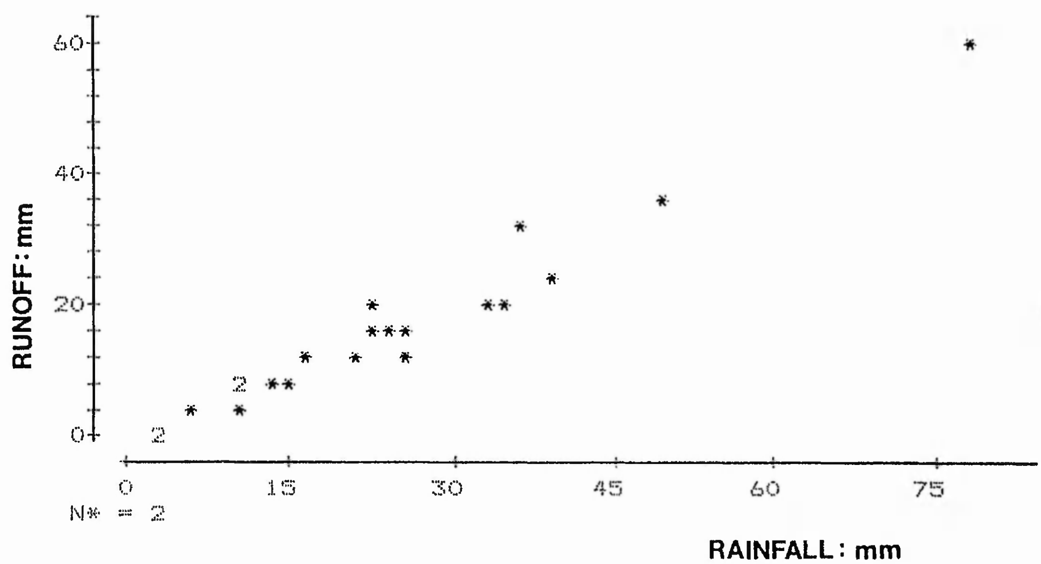


Figure 4.7 Plot of the relationship between rainfall and measured runoff for the roof site. Both axes in millimetres.

The various totals given above can now be corrected to give:

Input: Rainfall = 470mm = 20.05 m^3

Runoff = 13.63 m^3

And, calculated infiltration volumes:

Stone-filled soakaway	= 13.09 m^3
Chambered soakaway	= 13.45 m^3
Infiltration trench	= 13.56 m^3

With the corrections for change in depth over the monitoring period included, the figures for 'runoff' and 'losses' should, theoretically, be equal. The largest disagreement, of 4% of runoff, between runoff and infiltration from the stone-filled soakaway is considered to be small enough to be within expected measurement errors.

The most striking observation from the results was the similarity in the calculated hydrological performance between the widely varying constructions. Each received about 13 m^3 of runoff and apparently was able to store and infiltrate these waters perfectly satisfactorily. At no time was any surface flooding recorded or reported, even though the whole of the campus area is considered to be of poorly draining soils. In fact, conversations with members of the cleaning staff for the block served by the devices revealed that prior to the beginning of this research work, the area where the construction took place was prone to flooding due to overflows from the conventional stormwater drainage system serving the building.

A comparison and discussion of the individual results from each device in turn, is made below. However, one of the observation periods worthy of special comment, concerning the collective performance of the devices, is for period number 6 (as designated in Tables 4.4 to 4.6), when 23mm of rain fell during one night between successive readings taken about 20 hours apart. The rainfall event generated approximately 2.5 m^3 of runoff from the roof of the whole block: 66% of this had been infiltrated by the three devices only 10 hours after the majority of the rainfall.

Taking into account the effective infiltration areas for each device, the average infiltration rates for the period between the middle of the rainfall event and the observations made 10 hours later, were:

Stone-filled soakaway	$= 2.16 \times 10^{-6}$ m/s
Chambered soakaway	$= 5.52 \times 10^{-7}$ m/s
Infiltration trench	$= 2.00 \times 10^{-6}$ m/s

the variation in infiltration rate highlights some of the differences between the designs. The reasons for these differences is discussed below.

The specific performance of each device is considered below. This assessment examines the individual losses calculated for each device, especially for those times when the performance of the devices were seen to differ markedly.

Stone-filled soakawav. The most notable aspect of the results from this, more traditional, design of soakaway was the similarity of the results with those for the other, less traditional, designs. In retrospect, it should not have been expected that the old and new designs would differ fundamentally in their hydraulic properties: after all, each design is just a different form of hole in the ground. The major difference between the designs, i.e. the access, is considered with regard to maintenance in Section 8.3, but would not affect hydraulic performance.

In some ways the performance of the stone-filled soakaway could be expected to be an improvement on structures such as the dual-chambered soakaway. For any given volume of input, a stone-filled soakaway will have a greater rise in depth of water than a chambered soakaway of similar dimensions, thereby increasing both the surface area available for infiltration and head available to overcome soil pore pressures at depth.

Specific results that require comment are those from observation periods numbered 4, 9 and 19, (in Table 4.4). Firstly, period number 4, (which will also be commented on for the other two devices below), indicated an infiltration loss of just over 2 m^3. The extremely large rainfall event (45mm) that took place within the 48 hours of these readings was expected to have filled the soakaway. However, there was no evidence of this, and there were no reports of flooding.

The most important point here was that the volume infiltrated was much greater than that recorded for the chambered soakaway because the device was tested 'in extremis'. Any filling/flooding of the device would have greatly facilitated the rate of infiltration at depth due to the large head, and more importantly, within the near surface horizons of the soil.

The result from period number 9 indicated a negative loss of 12 litres. There were three possible causes of this figure. Firstly, it could have been due to a mis-reading of about 1 centimetre for the depth of the water; secondly, but least likely, the figures for volume per unit depth change were wrong by a factor of 50%; and thirdly, the groundwater level could have risen above the level of water in the device and resulted in the seepage of water from the surrounding soil into the device, again unlikely in the middle of summer. The actual cause of this 'negative loss' was not clear, but it is not considered to be significant and is commented on here for the sake of completeness.

During the period number 19, when over 33mm of rainfall fell in two days, only 34 litres (of the 840 litres of runoff) was infiltrated, which was significantly lower than for the other devices. Also, a preliminary comparison with period number 6, during which 23mm of rainfall in one day resulted in the infiltration of over 0.75 m^3, seemed to indicate a drastic change in the hydraulic capabilities of the stone-filled soakaway.

The simplest explanation for these results lies in the actual levels recorded for this and the previous period. In the three weeks prior to observation number 19 the levels of water within the device had receded to over 1.5 metres below the ground surface, the lowest recorded. The runoff during the period under discussion only filled the device to just under 1 metre from the surface. It would seem that below a depth of 0.8 metres below ground level the infiltration rate was greatly reduced, which if true, provides an interesting counter-point to the observations from period number 4, when extremely large infiltration rates were obtained when the device was full (or overflowing).

Furthermore, as there was almost always over 0.5 metres depth of water within the device at any time, it is probable that any construction of infiltration devices below 1.5 metres, at this particular site, was ineffective. The examination of effective depth of construction, and other matters of design and maintenance, are covered in Chapter 8.

Dual-chambered soakaway. Period number 6 in Table 4.5 shows a loss of 0.96 m^3 for the dual-chambered soakaway, much more than the losses for the other devices. This was caused because the infiltration volumes from the chambered soakaway were split fairly evenly between the two chambers, 0.46 m^3 from the first and 0.5 m^3 from the second. This highlighted a particular feature of the dual-chambered system: the water level observations indicated that the second chamber was not receiving any waters via the overflow pipe from the top of the first chamber into the second. Therefore, the second chamber was behaving as an independent soakaway. The slightly higher loss from the second chamber was probably due to the larger surface area available for infiltration.

After the middle of August the second chamber was not hydraulically connected to the first for the rest of the study period. Therefore, it received no further input of runoff, so the general decline in water levels through the winter were a good indication of falling groundwater levels (during the dry winter), although, some interference from the infiltration of waters from the first chamber may have taken place.

Two other sets of interesting observations occurred during the periods numbered 4 and 6. Period number 4 had losses from the dual-chambered soakaway of between 0 .7 and 1.5 m^3 less than from the other devices, and for period number 6 the loss was less than half of that from the other devices. During the periods corresponding to these observations there was a considerable amount of rainfall within the 48 hours preceding each reading. This would seem to indicate that the low figures for the dual-chambered soakaway were related to the short time between the rainfall and the time of observation.

Conversely, the periods following both period numbers 4 and 6 show a much larger calculated loss for the dual-chambered soakaway than for either of the other devices; in the case of period number 7 over 1.0 m^3 more. Following on from the comments of the previous paragraph, these periods of somewhat lower rainfall suggest that the dual-chambered soakaway system was much slower at infiltrating the waters it received than either of the other devices. Thus, after periods of recent, heavy rainfall the chambers had low figures for losses, but in the following, drier, periods the chambers were able to 'catch up' with the other devices.

Other occasions of wide variation of the observations of total losses between the dual-chambered system and the other devices are due to the 'extra' infiltration occurring via the disconnected second chamber, which was not acting as a dynamic part of the system: for example, during period number 12, the larger infiltration volume for the dual-chambers was due to that loss from the second chamber which had received no runoff.

The failure of the second chamber to fully empty over a period of 6 months indicated that it was probably constructed to too great a depth, i.e. it was over-designed and the construction intersected either an impermeable horizon or the local water table.

Infiltration trench. A discussion of the results from the piezometers installed around the trench and of the infiltration tests on the trench is made in Section 4 .2.2.

The most notable results for the losses calculated for the infiltration trench, in Table 4.6, were from period number 4, a loss of 2.8 m^3 , a volume over 3 times its designed effective capacity (of 0.9 m^3). The maximum depth gauge reading was at its highest possible level, within a centimetre of the ground surface. The storm which generated most of the runoff for that period had happened within 48 hours of the reading and was measured as producing approximately 46mm depth of rainfall. Conservatively, this would have generated 1.4 m^3 of runoff for each device, which should have produced an increase in the depth of water within the device of over 0.9 metres - it was only excavated to that depth and it was estimated that it was half-full before the event commenced.

The invert level of the pipe providing the in-flow to the trench was at a depth equivalent to a water level of 33 centimetres from the surface. The maximum depth gauge reading showed that the water level exceeded this on numerous occasions. Thus, it is almost certain that, not only did the runoff frequently 'back up' along the in-flow pipe, but that on the occasion of this large event, described above, the trench either over-flowed or water was 'backed up' along the length of the in-flow pipe (and possibly up the rainwater leader from the roof).

It is not clear whether or not the trench could have overflowed because the lid to the inspection chamber was screw-fitted and water-tight and the other means of egress for water was via sampling tubes along the length of the trench, the tops of which may or may not have been water tight. If no such egress was possible then the water would have been forced to seep into the upper layers of soil and grass.

The main point about this heavy rainfall during period number 4 (and period number 6) was that a much larger volume of runoff was lost compared to the dual-chambered soakaway. This was most likely to have been caused by the relatively high level to which the trench filled, which enabled infiltration to take place at much higher zones within the soil profile, which generally have a greater infiltration capacity.

4.2.3 Implications for disposal of pavement runoff

As discussed previously, one of the aims of the 'roof runoff' study was to ascertain of the infiltration devices would have proved an effective method for final disposal of the pavement runoff. After the removal of the gravel bay from the pavement discharge, (Chapter 5), the effective area of the car park was approximately the same as the contributing roof area to the infiltration devices. (The pavement runoff also had a slightly higher concentration of suspended solids, Schofield 1991).

The method and time resolution of the data collection for the infiltration devices prevented an event by event simulation or comparison of the pavement runoff versus storage and infiltration capacity of the soakaways and infiltration trench. However, the roof site was situated only a few hundred metres from the pavement and the infiltration devices would have received an equivalent or greater volume of runoff than that discharged from the pavement, (given the reduction and attenuation capabilities of the pavement compared to the roof).

Therefore a basic analysis confirms that, as the infiltration devices were fully functional for the duration of the research, the stone-filled soakaway, dual-chambered soakaway and infiltration trench, would have proved perfectly adequate devices for installing a fully operational on-site stormwater disposal system for the pavement. (Direct infiltration through the base of the pavement is considered in Chapter 5).

4.2.4 Additional testing

In an attempt to further quantify specific properties of the infiltration devices and the groundwater regime of the site, two further experiments were carried out.

<u>Infiltration tests</u>. Several sets of infiltration tests were conducted on the infiltration trench in an attempt to physically measure infiltration rates for the device, instead of extrapolating from the results of random rainfall and intermittent observations. Due to technical problems, the simplest and most reliable way of providing a constant flow at known rates was to pump water from one of the chambers of the dual-chambered soakaway into the infiltration trench. It was not possible to pump into the stone-filled soakaway or pump from one of the other devices into the chambered soakaways because of their relative positions and small diameter access tubes.

The main type of test undertaken was a constant head infiltration test, (Reynolds et al, 1983). This required that the level of water in the device be kept steady for a period of time by the addition of water at a known rate. To achieve the maximum possible rate of infiltration in the soil conditions at the site the trench was filled to capacity.

Several constant head tests were conducted after filling the infiltration trench. Short tests of approximately 20 minutes duration achieved infiltration rates of about 4 litres/minute. However, longer tests of up to 4.5 hours showed that the trench was capable of a sustained infiltration rate of 2.4 litres/ minute (4.8 x 10-6 m/s). This lower rate was equivalent to an input of runoff of approximately 3.4mm/h from the roof surface served by the infiltration trench. Given a runoff coefficient of 0.75 this approximated to a rainfall rate of 4.5mm/h.

Thus, when full, the infiltration trench should have been able to function satisfactorily at this sustained rainfall intensity. However, rainfall records for this project indicated that many events exceeded this intensity and it is in consideration of this that the engineer must be able to balance the storage requirement against the infiltration rate for a particular device. This is covered in more detail in Section 8.2.

Piezometer monitoring. The construction of a small network of piezometers was described in Section 3.2. The original intention was to monitor the groundwater levels in the area of the infiltration trench so that the variation in the local water table levels could be mapped. Thus allowing the direction of any prevailing, or induced, groundwater flow to be plotted. Monitoring the levels and calculated flows in the network provided an opportunity to account for the observed changes in the volumes in the infiltration devices. Furthermore, it was hoped that, in conjunction with the infiltration tests, it would be possible to monitor the head loss across the formation and hence calculate the hydraulic gradient and hydraulic conductivity at the site.

After a few observations it became apparent that the results were not what was expected. The site was generally flat: a survey showed that there was only a few millimetres between the heights of the datums for the piezometers. However, the first sets of observations gave variations in depth to the water table of between 0.20 and 0.30 metres, which on the area covered by the network indicated unreasonably large groundwater gradients.

The results of eleven sets of readings taken from October 1988 until February 1989 are plotted in Figure 4.8, an inset plan of the positions of the seven numbered piezometers relative to the end of the infiltration trench is also shown.

All of the readings showed a similar trend throughout the winter period of monitoring a sharp peak in December followed by a steady decline through to the middle of February. These general trends could be related to the rainfall: moderate to high rainfall through December, much lower through November, January and February.

Beyond the general similarity there are some striking dissimilarities in the degree of change observed in different piezometers. Taking the piezometers numbered 1, 2 and 3 as an example, which the plan shows were not more than 2 metres apart, between readings for 28th November and 1st December, the piezometers 1 and 3 showed a rise of over 40 centimetres each, whilst for piezometer 2 the level only rose by 6 centimetres. Similar variations in the individual recorded levels can be seen throughout the plot.

The readings for periods at the beginning of November were taken before, during and after, some of the infiltration tests upon the trench described above. These readings failed to show any discernible movement caused by the infiltration of 1,500 litres of water.

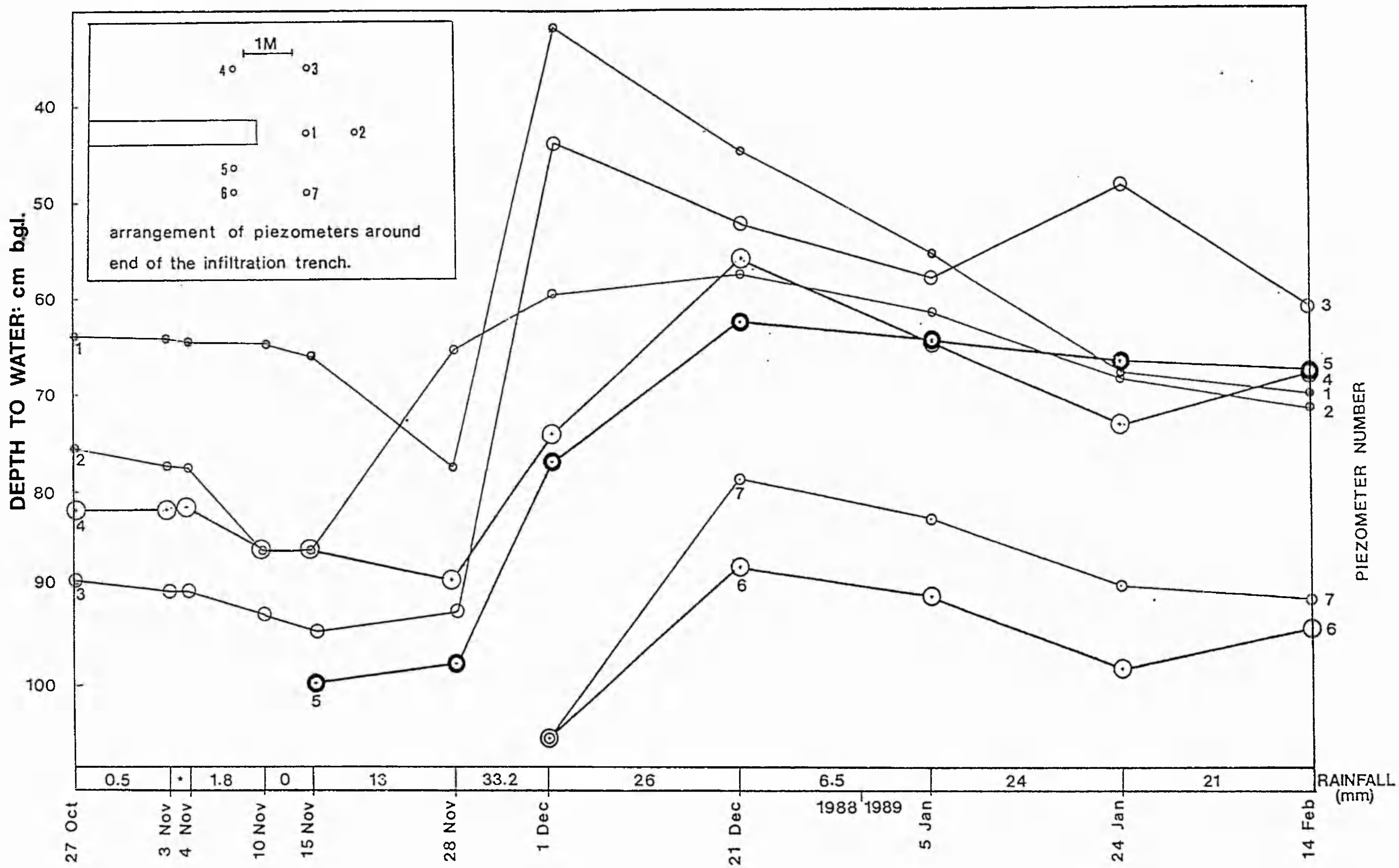


Figure 4.8. Plot of the piezometer data, (piezometer locations inset).

There are several possible interpretations of these results. Firstly, the main avenues of permeability within the sub-surface horizons may have been by fissure flow through fractures in the clays rather than an inter granular permeability. Each piezometer would thus have been in a different position relative to these fissures which accounted for a variation in the results.

Secondly, the piezometers may not have been recording groundwater movement at all. This could be the case if the installation methods resulted in the establishment of simple pathways for surface runoff to enter each bored hole. Alternatively, the auguring of each hole may have resulted in varying amounts of 'smearing' of the walls of each hole restricting the ability of some of the piezometers to accurately reflect changes in groundwater levels.

Lastly, a far more complicated arrangement of flow horizons, perched water tables and inaccurate installation may have combined to produce the observations.

In summary, it is probably wise not to make any inferences about the methods of groundwater flow from this particular study. However, a more thorough investigation of this side of the stormwater disposal equation is needed to supplement, and integrate with, designs for different devices.

4.3 Summary and Design Implications

This Chapter has reported the basic hydrological observations from the permeable pavement and roof infiltration experiments, and made simple deductions and generalisations about the results from each. Both systems represent alternative and complimentary methods of on-site stormwater management. For example, the collection and monitoring system installed for the permeable pavement effectively prevented the use of infiltration techniques for that particular experiment, but the philosophy behind that design would normally require that any runoff from such a surface be disposed of in devices similar to the soakaways used for the roof runoff, if not through the base of the pavement itself.

This Section summarises the results from both systems and draws together conclusions on areas common to both, with emphasis on the implications for the design and specification of attenuation and infiltration devices.

<u>Data collection</u>. Throughout the experimental work to collect the results presented in this Chapter great care was taken to ensure that the quality of data obtained was sufficiently high to justify the conclusions drawn from it. In the case of the pavement this was ensured through rigorous monitoring and calibration of the equipment, and for the infiltration devices by balancing all input and output (i.e. rainfall and infiltration). As illustrated with the collection of rainfall data, readings and calculations were always made so that uncertainties erred towards an underestimation of rainfall intensity incident upon the car park. Similarly, runoff input to the soakaways was underestimated.

These efforts were a requirement of the scientific study. However, they may have an important bearing on the collection of design parameters for the engineer. For infiltration measurement and for runoff estimates the engineer will wish to have accurate data, or make accurate assumptions, as part of the design process for stormwater infiltration/attenuation systems. However, in many situations there will be little opportunity to collect data, so the engineer will have to rely on estimates and should therefore include a factor of safety in his design, i.e. 'err on the safe side', if he wishes to meet the design specification and provide the level of service required.

Rainfall and runoff coefficients. The average amount of rainfall before the beginning of effective runoff was calculated to be 2.3 to 3.5 millimetres for the different bays in the pavement, and 1.9 millimetres for the roof. The totals of the percentage runoff were 33% to 45% for the four sub-base stone types in the pavement, and 68% for the roof site.

These figures show that when considering specifications for the design of systems to handle urban runoff the assumption of 100% runoff from surfaces, which superficially seem incapable of storage/loss processes, could lead to serious over-design. (This point is elaborated in Chapter 8).

Weather conditions. The effects of wet conditions antecedent to any particular storm were shown to be important to both attenuation and infiltration facilities. In both cases an earlier rainfall event meant that 'effective storage' was taken up for a period of time, reducing the effectiveness of attenuation. Explicitly, this refers to: porosity storage and wetness in the permeable pavement, (discussed in detail in Chapter 6); the wetness of the roof tiles; and the taking up of storage within infiltration devices. In the last case there is a trade-off between the loss of storage capacity and higher, induced, rates of infiltration caused by the increased head of water.

This consideration of antecedent conditions implies a requirement for a 'time to empty' specification within the design of such devices: i.e. the time required by a permeable car park to 'dry out' when saturated; or, the time required for infiltration devices to 'empty' once full.

Rainfall intensity. For both sets of experiments, high intensity rainfall events were seen to produce a higher than average percentage runoff. Thus, several short events would have a smaller total percentage runoff than a single event of the same total rainfall and total duration: this results from the initial interception losses being abstracted from each event. However, when comparing a single, high intensity rainfall event, to a low intensity, long duration event with the same total rainfall, then some form of 'continuing loss' processes reduced the effects of the longer event, giving it a much lower percentage runoff. These continuing loss processes were: infiltration in the case of the roof site; and, evaporation; or paving block porosity; or stone wetting/soaking in the case of the car park, (these processes are examined in Chapter 6).

However, infiltration/attenuation designs must take account of the likely maximum rainfall intensity to be catered for at any particular site. So, for infiltration devices the maximum infiltration rate may need to be specified: this should be related not simply to rainfall intensity but resultant runoff intensity as outlined above. For permeable surfaces, a consideration of the higher percentage runoff from high intensity events may lead to specifications which relate to the runoff coefficient from these events rather than an assumed average calculated from a broad range of intensities.

Flooding. Both sets of experiments were subject to some degree of flooding during the study period. For the pavement study, several events caused flooding of the instrument pit, and for the small volume infiltration devices, (the stone-filled soakaway and infiltration trench), both were calculated as having been filled beyond capacity. Although the flooding of the instrument pit did not form part of the design of the experiment, and no problems were encountered at the roof site, these observations force some consideration of the likely effects of flooding or failure by infiltration and attenuation systems.

Given the stochastic nature of rainfall, and the likely loss of performance of infiltration and attenuation systems with time, all designs for such devices should consider the effects of hydraulic failure i.e. where will over-flowing water go?. Also, given an estimation of resultant amenity (and possibly economic) problems caused by flooding, designers of such devices may wish to consider the cost-effectiveness of over-sizing constructions such that they are calculated not to fail in the lifetime of the surface they service, whether the surface is a road or a roof.

<u>Economic construction</u>. It is considered possible that both the pavement and the infiltration systems may have been over-designed to some degree. It was not clear what the effects would have been on the permeable pavement if smaller volumes of sub-base stone had been used i.e. a thinner and hence cheaper pavement. The two soakaway devices showed evidence of being excavated too deeply by not being able to fully empty, (or by being built into the water table).

Furthermore, all three infiltration devices performed satisfactorily, yet the dual-chambered soakaway had a storage capacity over three times greater than the other two devices: also, these had less storage capacity than recommended by BRE 151 (Anon, 1973).

However, bearing in mind the observations from the paragraphs regarding flooding, the cost of investigation work required to 'trim' the designs for individual sites to the most economic, commensurate with the specification, may far outweigh the savings from consequent reductions in design dimensions.

Finally, in support of the robustness of the systems built and monitored for this study the following should be made clear:

1. A roof area of 128 m^2 was disconnected from the normal drainage system and serviced by 3 infiltration devices, constructed in poorly draining soils, for a period of eighteen months. No flood damage or loss of amenity was observed.

2. A permeable pavement of 160 m^2 was shown to have extensive capabilities of water retention and attenuation. The structure was still functioning trouble free 4.5 years after construction. For the last 2 years no maintenance had been carried out and during that time all 'runoff' through the pavement was infiltrated to the poorly draining sub-grade soils.

CHAPTER 5
PERMEABLE PAVEMENT HYDROLOGICAL AND STRUCTURAL REFINEMENTS

Soon after the permeable pavement had been constructed, but before it had been subjected to trafficking, an assessment of structural performance of the pavement was undertaken. This consisted of an initial survey of levels of parts of the surface followed by subsequent surveys to monitor any deflection in the pavement due to its use. After eighteen months of hydrological data collection, a planned reconstruction of parts of the pavement took place to allow for the incorporation of some new structural elements within the car park, and to allow for the implementation of changes to the drainage system of the structure. Some of these changes were expected to alter both the quality and quantity parameters of the monitored runoff from that date.

This Chapter details the structural monitoring of the car park and the structural changes implemented in the summer of 1988. Various refinements to the drainage system of the permeable pavement are also reported. The changed runoff characteristics are discussed and further design improvements are outlined.

5.1 Pavement Structural Monitoring and Modifications

5.1.1 Deflection monitoring

Upon completion of the construction of the car park an engineering survey of parts of its surface was completed, so that later level surveys would enable any deflection in the surface to be quantified. As explained in Section 2.2, the boundary between each sub-base bay was delineated by white lines on the surface of the car park.

The initial level survey was of one-third of a car parking space for each of the gravel, granite and limestone bays, and of a full parking space for the blast furnace slag (b.f.s.). Assuming that, in general, cars parked within the spaces delineated, then the 'one-third space' surveys were designed to cover the area within which two wheels of any car parking in that space would come to rest.

Throughout, the surveys were conducted using an automatic engineering level and a staff, employing conventional observation and booking techniques, (Allen et al, 1973). The 'one third space' surveys covered a rectangular area 0.9m by 1.8m: each rectangle being divided to produce a grid 0.2m by 0.3m with an observation made at each intersection, (a total of 40 observations). The 'full space' survey on the b.f.s. was divided into a grid of 300mm by 300mm squares covering an area of 2.4m by 4.2m, (135 observations). It was not necessary to mark each individual coordinate, only one corner of each grid, as all coordinates were designated as the centres of the raised discs of individual blocks in the pavement surface.

The level survey was repeated a year after the initial one and the changes in the recorded levels between surveys calculated. Figure 5.1 is a contoured plot of the average of the calculated settlements for the b.f.s. parking space. The plot presented has had a smoothing function applied to it in order to simplify the results: all points used in the contours were at the centres of a square of four observed levels, the points plotted being given the value of the average of those four measurements. The actual observed level changes varied between a rise of 3mm and a settlement of 17mm. A rise in the level of some of the blocks was expected because some were seen to rotate or pivot about their short horizontal axis after pressure had been applied to only one end of the block.

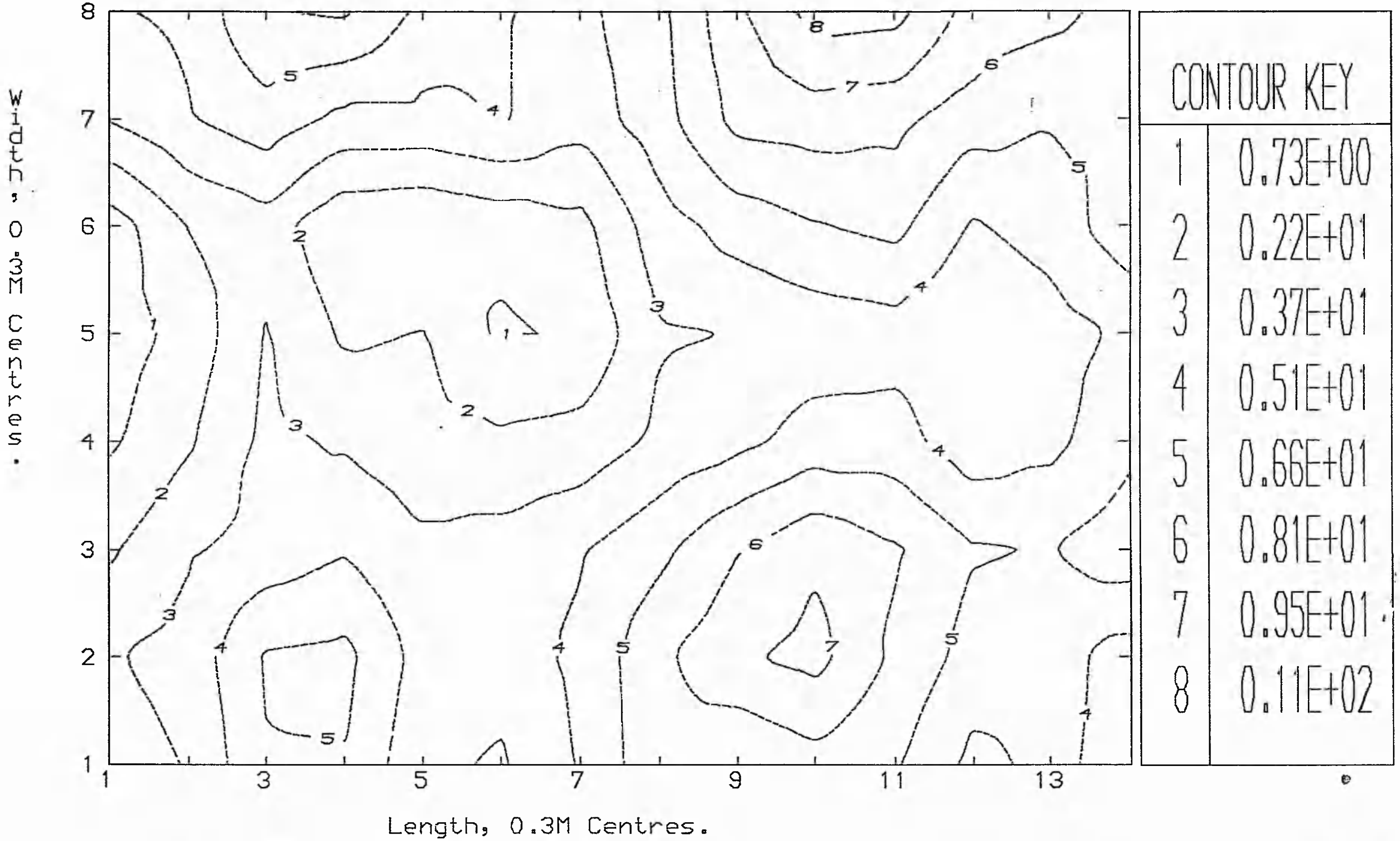


Figure 5.1 Contoured plot showing averaged depressions on the surface of a parking space above the b.f.s. sub-base, (all contours in millimetres).

Figure 5.1 demonstrates that four areas had higher than average settlement. It was obvious that these areas corresponded to the general locations in which car tyres came to rest. Furthermore, the greatest settlement generally was seen to occur at the right hand side of the Figure which corresponded to where cars would drive onto and off the car park.

The monitoring of the levels for the other sub-base bays did not produce such graphic displays of the effect of the differential loading. This was partly due to the smaller surface areas monitored and partly because the areas chosen did not quite match up with the centre of the average tyre loading as delineated by the b.f.s. survey.

The average and maximum settlements of the 40 coordinates of each bay monitored for the first year are given in Table 5.1, with values for the equivalent area of the b.f.s. bay also included. The figures show that there was a degree of settlement in the surface of the pavement. It also appeared that the degree of settlement was influenced by the type of sub-base stone, with little settlement in the pavement above the granite sub-base and just under 4mm, on average, in the pavement above the limestone sub-base. However, given the precision of the levelling it is considered that the figures in Table 5.1 are only a guide to the relative performance of each of the different sub-base areas.

Table 5.1. Permeable pavement - calculated average and maximum settlements for part of each bay during the first year of use.

Sub-base	Average settlement mm	Maximum settlement mm	Number of observations over 2mm
gravel	3.9	10	29
granite	2.0	5	12
limestone	4.9	13	37
b.f.s.	3.2	8	24

5.1.2 Structural modifications

After the initial results of the structural monitoring and after collecting 15 months of data on the hydrological performance of the pavement, the research was extended by implementing a series of structural and hydrological modifications to various parts of the system.

It was expected that many of the parameters monitored up to that time would be altered. In order to provide a control to compare the pre- and post-alteration data, the blast furnace slag bay was not modified. This Section deals with the structural changes which set out to investigate: firstly, the effects of the removal of the impermeable membrane between the sub-grade and sub-base; and secondly, the effects of the addition of man-made 'geogrid' structures into the bedding layer or sub-base to enhance pavement stability.

Permeable pavements are usually designed to allow on-site infiltration of incident rainwater, some designs also aim to infiltrate rainwater directly to the pavement sub-grade, Hogland et al, (1987). The notion of saturating a pavement sub-grade will be anathema to many highway engineers to whom the protection of the sub-grade against excess moisture is of fundamental importance to good highway design. It is generally considered that if water enters a sub-grade then there would be an appreciable risk that the strength of the sub-grade would be reduced, lowering its resistance to vertical strain.

In order to test part of the structure in such 'unsuitable' conditions, the whole of the impermeable membrane forming the 'tank' for the gravel bay was removed. This entailed lifting out the blocks forming the pavement surface and using a JCB to excavate, firstly, the gravel bedding layer, and then, after removal of the geotextile, the gravel sub-base. The PVC membrane was removed and the drainage pipe for the bay altered so that the invert of the drain was lifted by approximately 50mm, (acting as a safety overflow by providing positive drainage if there was very heavy rainfall which could not infiltrate promptly).

After removal of the impermeable membrane, the reconstruction of the gravel bay brought the gravel sub-base into direct contact with the silty-clay sub-grade. It is likely that the CBR (Californian Bearing Ratio, Powell et al, (1984)), was 2 to 3% for the cohesive sub-grades encountered during construction of the car park, (i.e. appropriate values for a silty-clay with a plasticity index of between 30 and 40%).

To aid stability of the sub-base, the gravel was placed within a commercially available 'grid confinement' system, provided by Ardon International, called 'Geoweb'. Figure 5.2 gives the manufacturers specifications and a diagram of the grid. Filling the 'pockets' of the web with granular sub-base is reported to inhibit sub-base failure. This is achieved because the grid confinement increases the bridging effect between adjacent 'pockets' thereby decreasing the pressure on soft sub-grades. This should greatly reduce the dynamic loading of the sub-base/sub-grade interface which can cause large permanent deformations through the phenomenon known as 'pumping'.

A second set of structural alterations was made to the granite and limestone bays. In both of these bays a change to the bedding layer was made in an attempt to reduce any problems with settlement, (or rotation), of the blocks.

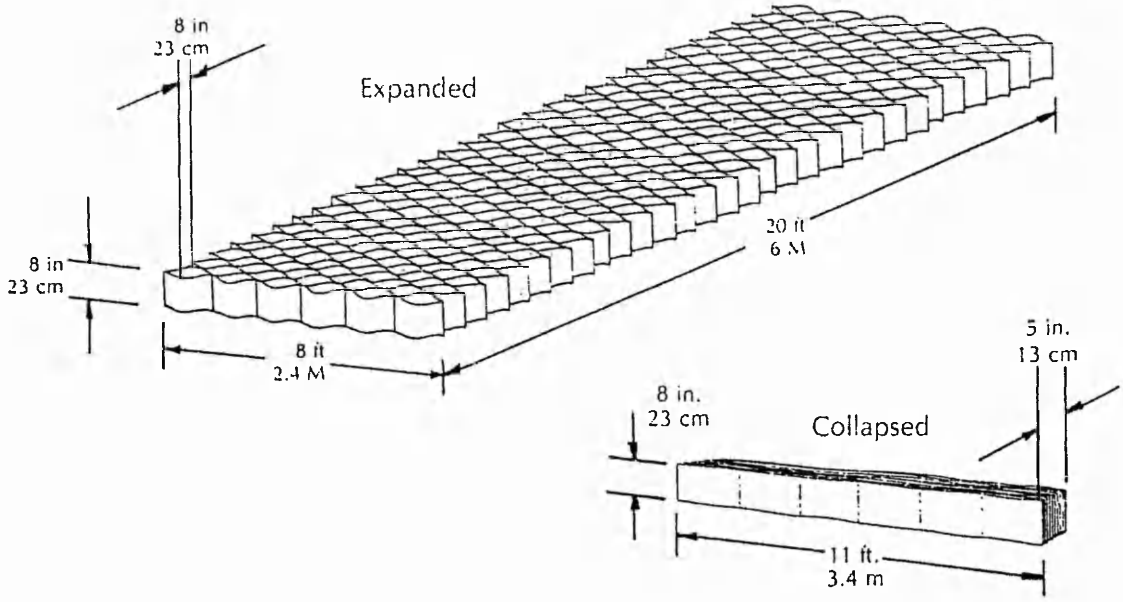
A commercially available 'mesh element' system produced by Netlon Limited was chosen for this purpose. The mesh elements were 'discrete pieces of orientated polypropylene mesh, typically 100mm x 50 mm with 10mm x 10mm mesh apertures', which resembled small squares of stiff netting. Thousands of the mesh elements were mixed with the gravel bedding layer. This is designed to produce enhanced stabilisation by the interlock of the elements , which have a very high tensile strength. After mixing with the bedding layer, the elements became linked in random orientations, inhibiting movement in all directions. There was a small difference between the alterations to the limestone and granite bays in that the former had slightly longer mesh elements, (100mm x 75mm), mixed with its bedding layer.

<u>Results</u>. After reconstruction of the gravel, granite and limestone bays the whole pavement was surveyed in a similar manner to that described in Section 5.1. A final set of levelling was completed in April 1989 so that the effects of 9 months of vehicle use on the modified bays could be measured. The average and maximum depressions for the bays are given in Table 5.2.

Table 5.2. Permeable pavement - calculated average and maximum settlements for 9 months to April 1989.

Sub-base	Average settlement mm	Maximum settlement mm	Number of observations over 2mm
gravel	1.1	9.0	10
granite	0.6	6.0	4
limestone	1.1	5.0	10

The comparison of the results presented in Table 5.1 for the settlements recorded during the first 13 months after construction, with these results shows a significant improvement in structural performance. However, the average value for settlement should be treated cautiously, as the majority of calculated settlement for each bay (112 coordinates per bay) were less than 2mm which is inside the margin of error for such levelling. The maximum settlements and the number of results over 2mm are a better indication of the structural improvement monitored. These results show a reduction in the degree of settlement for all the sub-base bays, especially for the limestone bay.



GEOWEB Structural Properties	Metric System
1. Expanded Dimension	2.5 m x 6 m x 2.3 cm
2. Collapsed Dimension	3.4 m x 13 cm x 23 cm
3. Panel Thickness Nominal	0.119 cm
4. Weight	3.1 kg/m²
5. Cell Area	265 cm²
6. Cell Seam Node Pitch	33 cm
7. Welds/Seam	7
8. Seams Tensile Peel Strength	69 kg
9. Installation Temperature Range	-27°C to 43°C

Polymer Material: High Density Polyethylene
Color: Black
Carbon Black Content: 2%
Chemical Resistance: Superior

Figure 5.2 Illustration of 'Geoweb' and technical data.

In summary, these structural modifications were a useful indication of how these types of modern 'geo-structures' can be integrated with constructions such as the permeable pavement. The wetting of the sub-grade beneath the gravel sub-base did not show any effects at the pavement surface during the monitoring period of 9 months. Although more thorough research on the structural implications should be conducted before the implementation of such a drainage system is incorporated into pavements designed to support more frequent, or greater static and dynamic loading.

5.1.3 Deflection tests

A standard test for the structural performance of a flexible pavement is the measurement of the pavement deflection under a specified load. Significant relationships between the deflection of roads, measured under a standard rolling wheel load and their structural performance have been described by Kennedy and Lister, (1978). The test, known as the Benkleman Beam Test, is designed for, and usually applied to flexible bitumen surfaces and the validity of the application of such a test to a concrete block surface is debatable. The equipment for the Benkleman Beam Test, (Kennedy et al, 1978), was made available to the research project by the Department of Planning and Transportation of Nottinghamshire County Council, and it was thought useful to perform this standard measurement for future reference and comparison.

Briefly, the equipment for a deflection test consisted of a thin beam, 3.66m long, one end of which had a pointer which rested on the surface whose deflection it was to measure. The other end of the beam was attached to a gauge or transducer. The beam was pivoted about a point giving a 1:2 length ratio either side of the pivot such that a 2mm deflection of the pavement resulted in a 1mm reading on the gauge or transducer. During the test the beam was situated between a pair of wheels on one side of the rear axle of a lorry, the rear axle load being 6.35 tonnes. The test consisted of the lorry moving forward at a 'creep speed' of 0.5m/s along the length of the beam and past the pointer. Three readings were taken: the initial and final gauge readings; and the maximum deflection of the gauge as the wheels of the lorry pass by the pointer.

For the purposes of the test on the permeable pavement a transducer was used to measure the deflection. This was connected to a Campbell Scientific 21X logger to display and record the data. From the readings taken the 'depression' during the movement of the lorry was calculated from the difference between the initial and maximum gauge readings, multiplied by two to account for the ratio caused by the location of the pivot. The 'deflection' is given by subtracting the final reading from twice the initial reading, Table 5.3 gives the full set of results.

Table 5.3. Results of Benkleman beam deflection tests.

Sub-base & trial No.	Readings Max. (A) mm	Final (B) mm	A+A-B mm	Rut Depth mm	Average Deflection mm
Gravel 1	3.1	0.8	5.4	1.6	5.1
Gravel 2	2.6	0.4	4.8	0.8	
B.F.S., 1	1.75	0.15	3.35	0.3	2.97
B.F.S., 2	1.3	0.02	2.58	0.04	
Granite 1	1.2	0.1	2.3	0.2	2.27
Granite 2	1.14	0.04	2.24	0.08	
Limest. 1	1.44	0.2	2.68	0.4	2.67
Limest. 2	1.38	0.1	2.66	0.2	

The observations presented in Table 5.3 show a degree of deflection within the pavement which would be compatible with its design as an area for private car parking, especially as the surface is primarily subject to static loading, (Powell t al, 1984).

A comparison of the results also serves to compliment the observations of the surveys presented in Section 5.1.1, in that the load bearing and crushing strengths of granite are evident. The relatively high depression and deflection of the gravel sub-base is probably related to its uniform grading, (a grading normally specified for use as drainage material).

After the structural modifications described in Section 5.1.2 it was hoped that a second set of deflection tests could take place to ascertain the relative performances of each of the 'improved' bays, however, a heavy work-load during the summer of 1989 prevented the Department of Planning and Transportation from providing their services. It was thought that the results from the levelling presented in Section 5.1.2 were an indication of an improved structural performance of the car park and that further deflection testing would confirm this at some later date.

In summary, the levelling and deflection tests did show some degree of movement within the surface of the pavement, due to applied loads. The differing sub-base stones were shown to affect the magnitude of movement observed; the granite and b.f.s. showing better load bearing characteristics than the limestone and gravel sub-bases.

The long-term effects of any structural deterioration would depend on the loadings incident on such a surface. The provision of structural elements to aid the permeable pavement withstand repeated loadings was shown to reduce the degree of settlement in the surface blocks.

5.1.4 Cold weather susceptibility

Open-textured surfaces, such as the permeable pavement, which allow water to percolate to their base, and possibly accumulate upon the sub-grade, may be susceptible to 'frost heave'. To monitor the sub-surface air temperature of the permeable pavement, three thermistors were installed in the limestone sub-base bay to measure the temperature profile of the sub-base.

The thermistors were connected to the on-site Campbell logger which also recorded the runoff from the pavement, and the air temperature above the car park. The logger activated the thermistors every hour, and recorded the temperature, in degrees Celsius, as part of the usual data file, (see Table 2.1).

Figure 5.3 illustrates the relationship between the above ground air temperature and the air temperature at the bottom of the limestone sub-base. The period shown was one of the coldest recorded during the course of the research. The plot shows that the outside air temperature fluctuated much more rapidly than the internal sub-base temperature. Close scrutiny of the relationship between the results from the two thermistors shows that the sub-base temperature followed the trend of the surface temperature, but the daily maxima and minima lagged behind the surface temperature by between 8 to 12 hours: often the daily maximum sub-base temperature would coincide with the coldest time of day for the surface temperature.

No observations were made during a persistent period of below freezing temperatures, but the low thermal conductivity of the paving blocks, gravel bedding layer and sub-base was thought to provide a moderate level of thermal insulation to the bottom of the sub-base and the sub-grade. It was estimated that over one week of sub-zero temperatures would have been required to bring the sub-base temperature down from 5° C to freezing point.

Operation in a cold climate. The prospect of frost heave would worsen if the pavement was subjected to long periods of sub-zero temperatures, as experienced in Scandinavia. In such cases the depth of the sub-base may have to be increased to provide lower thermal conductivity from the ground surface to the base of the structure. Restrictions may also be placed on percolation rates for sub-grade infiltration to reduce the likelihood of having standing water at the base of the pavement.

A singular advantage of porous surfaces, over impermeable surfaces, in cold climates, is the rapid removal of melt-waters following snowfall. Commonly, on impermeable surfaces, snow or ice will remain for several days even when the temperature allows for thawing. The snow itself, often blocks effective drainage of melt-waters, leading to a re-freezing and icing of the road or pavement surface. However, the pores and gaps between the concrete paving blocks provide for rapid drainage as the snow melts.

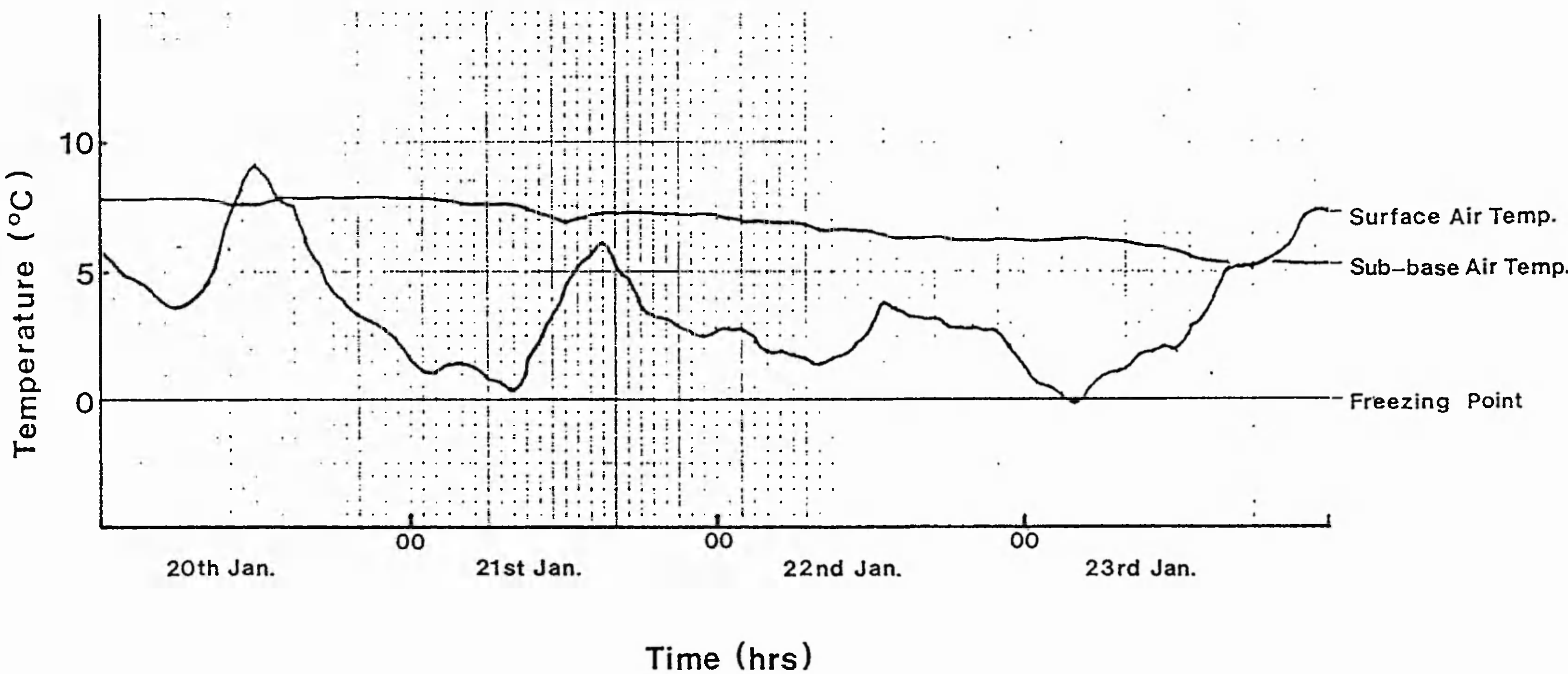


Figure 5.3 Comparison of the sub-base and surface temperature fluctuations.

5.2 Flow Control

During the structural modifications to the permeable pavement several changes to the drainage system of the pavement were made. These alterations served to moderate the runoff from some of the bays. This Section describes the changes to the drainage system and the resultant runoff, and then outlines other drainage modifications which may prove useful as part of a stormwater attenuation system.

5.2.1 Drainage modifications

Gravel Bay. The alterations to the gravel bay included removal of the P.V.C. tank containing the sub-base and the raising of the drainage outlet for the bay by 50mm. It was expected that most of the 'runoff' would infiltrate through the base and sides of the bay.

This was confirmed after several weeks with no runoff through the conventional drainage to the measurement pit. Subsequently, the gutter and tipping bucket for this bay were removed to facilitate access to the other tipping buckets in the measurement pit. In place of the tipping bucket a two gallon plastic bucket was hung beneath the drainage outlet of the gravel bay into the measurement pit. The bucket was inspected on each visit to the site to see if any runoff had occurred, indicating that the lowest, discharge corner of the gravel bay had had over 50mm of standing water at the sub-grade at some time.

During the last 10 months of monitoring the bucket contained water twice, both times after heavy rainfall. On neither occasion was the bucket filled, indicating a total discharge equivalent to less than 0.25mm rainfall over the area of the gravel bay.

These simple results show that even in the silty-clay formation at this site there was ample capability for the operation of a fully infiltrating system, (Section 5.1.2 comments on the advisability of this from a structural perspective). The runoff that was collected could only indicate a degree of ponding within the system.

Furthermore, the grid confinement system installed into the gravel bay as a structural aid, (see Section 5.1.2 and Figure 5.2), could possibly have acted as an internal system of 'reservoirs' within the sub-base depending on the effectiveness of the seal between the bottom of the 'Geoweb' and the underlying formation.

Limestone bay. The discharge corner of the limestone bay was excavated to reveal the drainage pipes (and the thermistor which monitored the temperature profile within the pavement). The perforated pipe which acted as a collector within the bay was removed and replaced by a 'fin drain'.

Fin drains are a relatively new method of providing filtered drainage. Figure 5.4 shows a cross-section from a typical design: the filter fabric is a geotextile of the required specification; the pipe, a conventional perforated pipe for conveyance of flow; and the fin or core a polythene mesh which has an open texture preserved by the filter fabric. Any waters moving horizontally across the direction of the fin drain will enter through the filter fabric, flow along the core to the pipe and be conveyed away by the drain.

The installation of this new drainage was aimed at altering some of the parameters of runoff quality that were being monitored. The suspended solids that were found within the runoff were mostly derived from within each bay, Schofield, (1991), i.e. most of the solids that were washed from the surface of the pavement by rainfall were filtered-out by the geotextile beneath the bedding layer: however, as the rainfall continued through the sub-base, particles would be washed from the surfaces of the stones giving a range of values for suspended solids for the runoff from the different bays.

The installation of the fin drain did not result in any observed changes to the quantity of runoff from the limestone bay but it is described here to show one method of removing suspended solids derived from within the sub-base. A second method formed the basis of research into throttles, described below.

<u>Hydraulic throttles</u>. A further part of the upgrading of the permeable pavement was the construction of drainage inspection chambers. Four chambers, one for each sub-base drain, were constructed, approximately 1.5 metres from where the drains discharged into the instrument pit. The chambers facilitated a number of experiments, including the use of pumped waters from the infiltration trench at the site into the chambers, for dynamic calibration of individual tipping buckets. Their main use was for putting 'inserts' into the drainage channel in the bottom of the chambers in an attempt to alter some of the quality parameters of the runoff.

The 'inserts' were large cylindrical, plastic mesh sachets into which various 'fillings' could be placed to filter the runoff as it passed through the drainage pipes in the base of an inspection chamber. They are commented on here because of their effects as hydraulic throttles of the runoff.

Two types of 'inserts' were used. One was filled with granular activated carbon, the second with wood shavings from a joiners workshop. The sachets, filled with material as required, were placed in the drainage channel at the base of an inspection chamber. The cylindrical, plastic mesh sachets were hand-made from sheets of commercially available mesh and were designed to be the same size as the drains (80mm diameter), so that all discharged waters passed through the filter material. The different inserts were moved between the drains of the different bays throughout the latter part of 1988 until the end of data collection in April 1989.

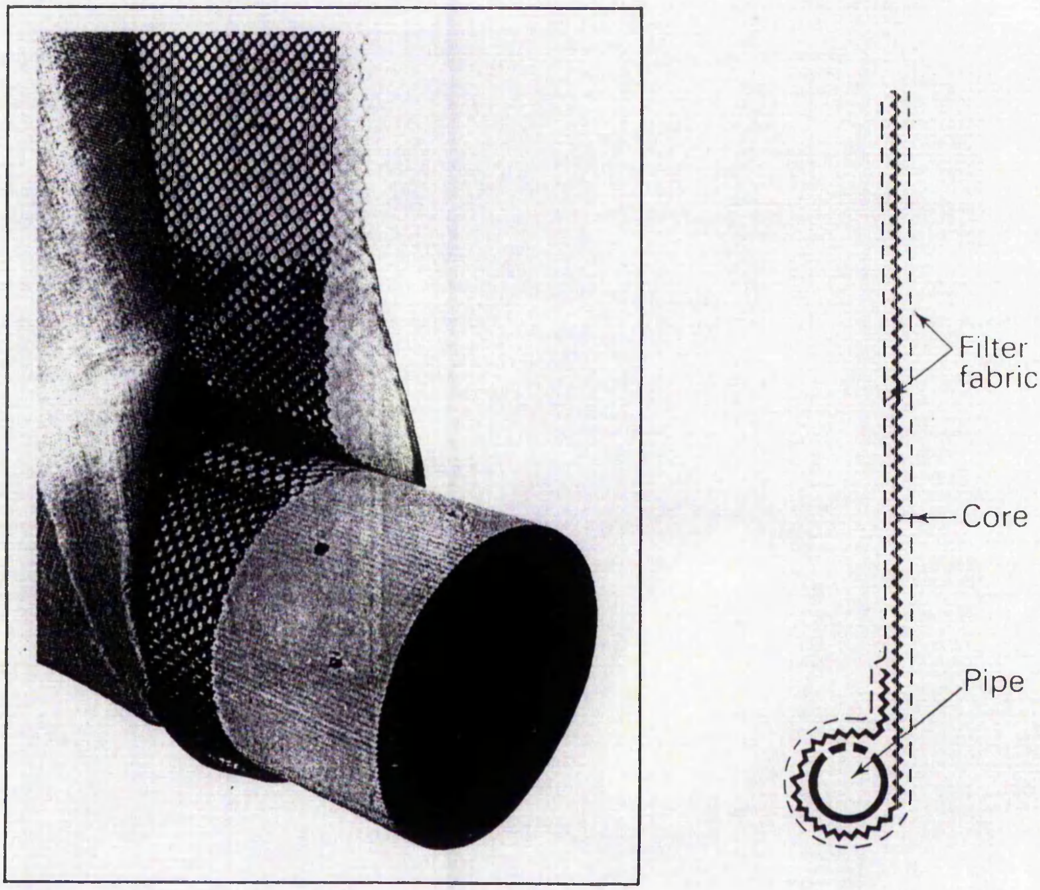


Figure 5.4. Cross-section of typical design for a 'fin drain'.

Although this particular avenue of research was primarily directed at the 'runoff quality' aspects of the research project, there were some very important 'runoff hydrology' implications. Briefly, in monitoring the runoff that had passed through these inserts the rate of runoff through an insert of granular activated carbon showed no discernible change, whilst the rate of runoff through an insert containing wood shavings was greatly reduced and the duration of runoff greatly increased.

The lack of attenuation by the activated carbon was thought to be due to its high porosity and permeability. The high attenuation from the wood shavings was due to its low permeability which was a function of the orientation of the shavings giving a convoluted pathway for any waters passing through.

Figure 5.5 shows the recorded rainfall-runoff relationships for event number 9065. The three runoff curves displayed are for b.f.s., granite and limestone. In the limestone bay the attenuation effects of the wood shavings insert is obvious: a greatly depressed peak runoff and extended duration of runoff. (The b.f.s. bay was being filtered through the activated carbon insert).

These changes had various effects on the calculations for runoff, etc. For event number 9065, in Figure 5.5, the attenuation of the runoff hydrograph from the limestone bay was such that the rate of runoff did not reach the defined 'start of runoff' rate, resulting in a computation of 'nil runoff' for the 13mm rainfall event. The insert was acting as a hydraulic throttle so that the rate of runoff was reduced below 0.25 mm/h, the threshold defined for the calculation of runoff. The runoff from the other bays was approximately 40% or 5mm.

It should be clearly stated that these reductions were a feature of the reduction in runoff rate to a level below the definition of runoff used in this research. No change in the total volume of runoff was evident through the use of the inserts.

For event number 8334, Figure 5.6, the total runoff recorded was approximately the same for all three of the bays monitored. This was due to the large rainfall, 28mm, which resulted in a storage of over 300 litres behind the throttle, (and therefore within the bay). This volume was able to provide sufficient head to keep the rate of runoff above 0.25mm/h for much longer. Runoff was still continuing some 11 hours after the rainfall event when the monitoring pit became flooded. However, at no time did the runoff from the limestone bay exceed 1mm/hr, (0.01 l/s)

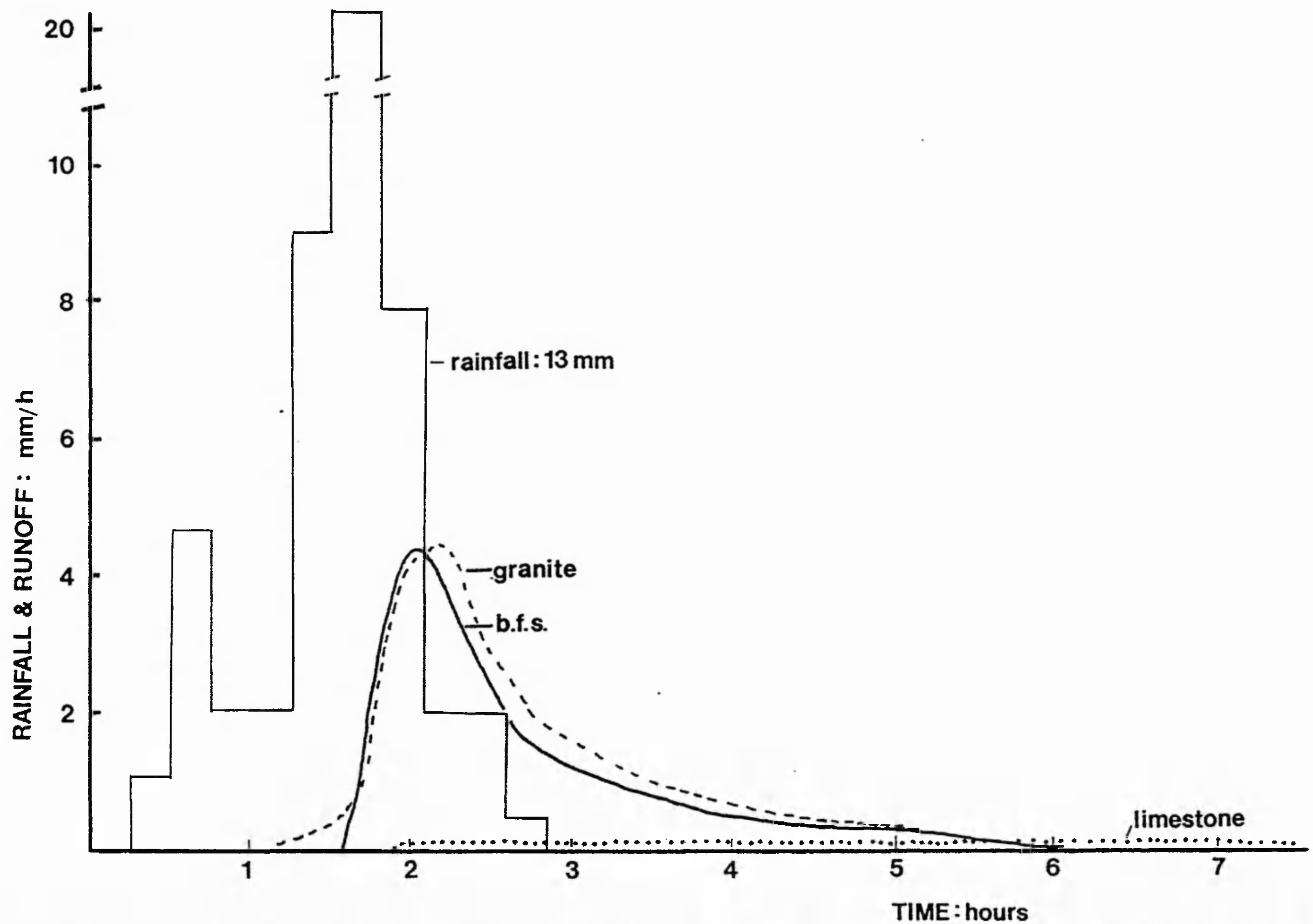


Figure 5.5 Rainfall-runoff relationships for event 9065. Wood shavings 'insert' attenuating limestone runoff.

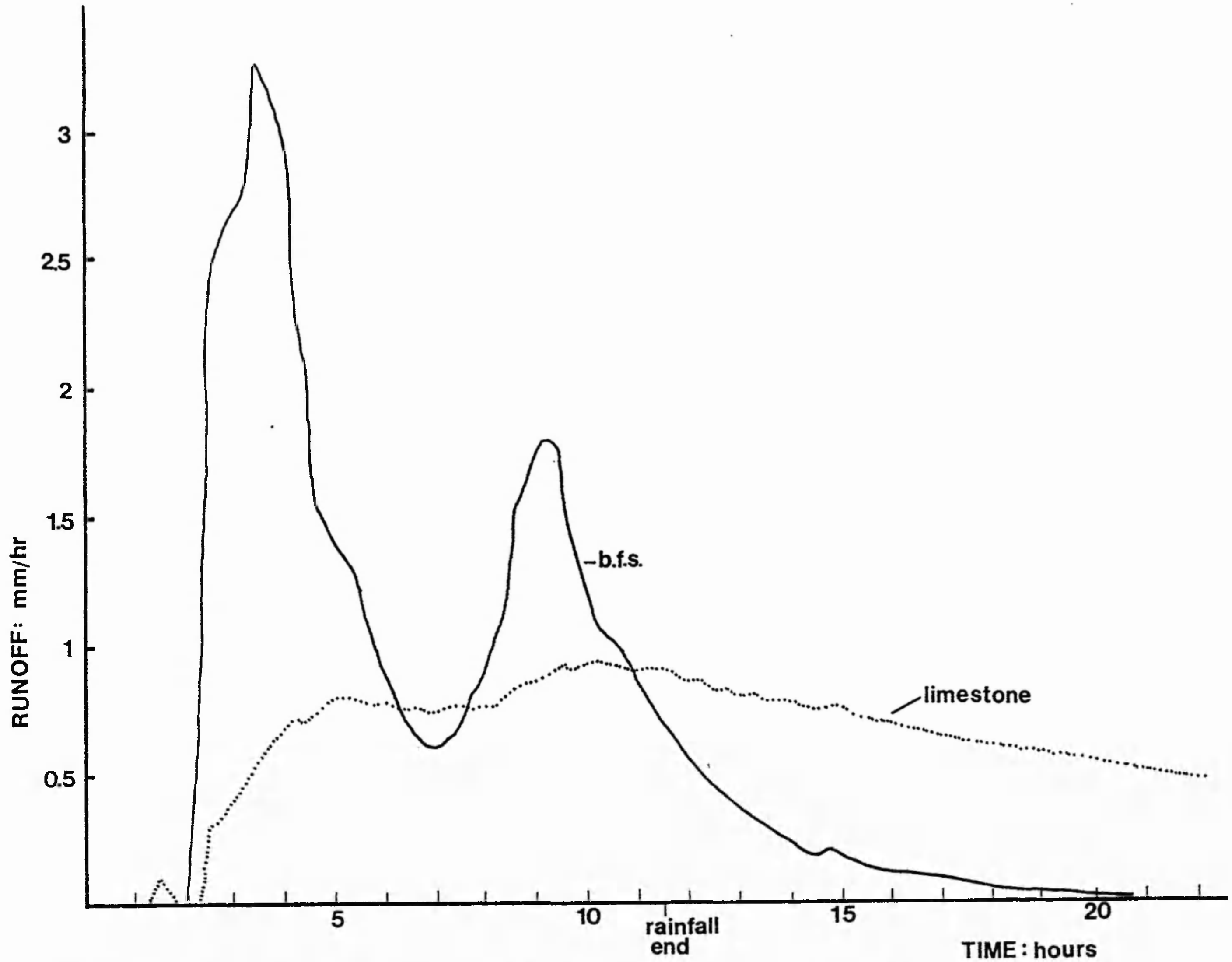


Figure 5.6 Comparison of runoff from event 8334, limestone runoff attenuated.

5.2.2 Further application of flow control

The results with the inserts prompted the consideration of the wider use of throttles within stormwater attenuation devices. Hydraulic throttles usually imply a requirement for a small orifice and storage upstream of the throttle. In the case of the former, this normally rules out the use of throttles in storm drainage because of the problems of clogging and cleaning. Work by Balmforth and Bailey, (1985), on roof storage using throttles as a method of stormwater attenuation relied on the relatively clean roof runoff for its long-term serviceability. In the case of road runoff, hydraulic throttles would not normally be considered because of the relatively high loadings of gross and suspended solids.

However, as stated above, most of the solids found in the runoff from the car park were derived from within the sub-base. If the design had been altered so that the geotextile had been used to intercept not only waters entering the sub -base, but also leaving the sub-base, i.e. by covering the drainage pipe inlet with geotextile, there would have been no gross solids and only a very small suspended solids fraction discharged.

The other implication of using hydraulic throttles to control runoff from a permeable pavement is storage. The throttle aperture would theoretically have to be large enough to prevent flooding by keeping the volume of any stored waters lower than the storage volume of the pavement. However, in practice, this is unlikely to be a problem as a typical depth of sub-base and bedding layer of 300mm and 80mm respectively should provide storage for over 150mm of rainfall. The real concern would be over the structural integrity of the sub-base tank should the loadings become excessive. This study did show that a runoff rate of less than 1mm/h for the car park could be established satisfactorily by the use of simple throttles, this is equivalent to approximately 100 l/s per hectare, (well below that for a comparable urban area, (Anon, 1976).

The hydraulic throttles described above have implied the use of a 'reduced orifice' i.e. a reduction in the diameter of the drain severely restricting the flow of waters. An alternative method would be to further the original idea behind the 'inserts', as methods of filtering the runoff, by examining filter materials which may also provide an effective method of flow control. Ichikawa and Harada, (1990), have described the use of sand within 'drainage infiltration strata' which, depending on grading, can prove extremely effective at attenuating peak flows when incorporated into such a drainage system. The primary concern with this type of throttle would be the clogging of the system due to sediment derived from within the sub-base stones and the difficulty of maintenance. Both of these concerns are addressed in Chapter 8.

In summary, this work did show that discharge control was viable so that flows could be modified to suit individual site requirements, something which is not possible with conventional drainage from impermeable surfaces. The choice of whether to adopt flow control for peak discharges should rest with regulatory authorities so that the site discharge can be integrated with broader stormwater management requirements. For example, there would be no point in attenuating peak flows so that they coincide with peak discharges arriving from upstream, thus compounding the problem of flooding.

5.3 Summary and Design Implications

From the point of view of a highway engineer the structural investigation carried out during the research could be considered to be somewhat limited with regard to the long-term effects of a saturated sub-base and sub-grade. However, within its design limits as a car park, the permeable pavement proved more than adequate, and no signs of distress or structural failure occurred up to March 1991, some four years after construction.

Care must be taken when considering similar designs which allow direct infiltration to the underlying formation to ensure that the design loadings such a surface may receive are appropriate and will not be exceeded in practice. The permeable pavement at the Clifton Campus only received loading from private cars, and occasionally, light goods vehicles. All dynamic loading was confined to vehicle movement from a conventional, hard core surface, at low speed, onto or off the pavement.

If much larger areas were to be used for the installation of permeable surfaces, then consideration should be given to providing conventional surfaces for vehicles to traffic before they move onto a permeable pavement. The impermeable surfaces could be sloped to drain into the permeable surfaces.

If concern regarding the stability of the sub-grade in a wet environment is such that infiltration through the base of the structure is judged untenable, then it may be prudent to consider the draining of a permeable pavement into infiltration on-site, but 'off-structure', in a nearby soakaway/infiltration trench in a similar way to the combination of the pavement and infiltration devices discussed in Chapters 3 and 4.

The use of modern polypropylene 'geogrids' was shown to be effective at stabilising the permeable pavement by reducing the degree of deflection in individual blocks in the pavement surface. Products such as those tested are widely available and could prove economic in terms of reduced requirements for sub-base, or in prolonging the effective design life of the structure. However, further work on the relationship between sub-grade strength, sub-base depth, the use of geogrids and the loading of the surface would be required to quantify the economics.

One of the most important results from the period of 'structural improvements' of the pavement was the simplicity and ease with which the de-commissioning and the reconstruction of the pavement was conducted. The installation of the mesh elements into the bedding layer of a bay could have been achieved by two men with forks and shovels within a day, requiring only a vibrating plate whacker to bed down the blocks.

The changes to the drainage of the pavement were aimed primarily at altering the quality of the runoff. However, the hydraulic throttle effect of the wood shavings has shown one of the many different ways in which the attenuation effects of such structures could be enhanced. The ability to dictate a maximum runoff rate, within the constraints of not flooding the system, should enable the capabilities of such a combination of water storage and runoff attenuation to satisfy the most rigorous planning authority.

CHAPTER 6

FURTHER HYDROLOGICAL INVESTIGATION OF PERMEABLE PAVEMENT

At the end of Section 4.1, which contained a preliminary examination of the results from the permeable pavement, four 'continuing loss' processes were postulated as possible causes for increased losses for longer duration rainfall events. A conventional, 'impermeable', road surface can also display similar loss characteristics but this is normally attributed to infiltration through cracks in the road surface, (Davies & Hollis 1981). However, for the particular design of the permeable pavement at Clifton Campus, such an explanation was not applicable.

A series of small scale experiments were devised and conducted in an effort to understand some of the possible continuing loss processes. Additionally, other experimental work focussed on trying to interpret some of the effects of the stochastic elements within the results already obtained. These experiments consisted of full-scale simulations of rainfall across the surface of parts of the car park and monitoring the consequent runoff. This Chapter describes the results from these experiments and discusses the implications for a thorough understanding of the processes involved, and for future investigations.

6.1 Investigation of Mechanisms of Water Loss

Results in Section 4.1 showed how the total loss, (rainfall minus runoff), from rainfall events was proportionately higher for those events of long duration. That is, losses continued even after runoff had begun, and were not simply an initial loss due to the wetting of surfaces. The various factors which could have given the observed results were: evaporation; paving block porosity; some method of continued surface wetting; and experimental error.

These factors are dealt with, in turn, below.

6.1.1 Evaporation

During the summer of 1988 an 'evaporometer' was constructed next to the car park site in an effort to measure what volumes of rainfall may have been lost due to evaporation. The 'evaporometer' comprised an excavation, $1m^2$ in plan by 0.5m deep, into the centre of which a sealed section of plan area $0.5m^2$, of the same cross-sectional design as the car park, was built. The sealed section provided a collecting basin for any incident rainfall: in many ways the instrument was similar to a lysimeter.

A section of the installed device is illustrated in Figure 6.1. The surface area around the collecting basin was constructed of the same materials as the car park to simulate the effects found at the car park surface, this included accounting for splash effects as rain-drops impacted on the block surface and an attempt to mirror the temperature profile of the car park.

The sealed section was constructed from planks of wood nailed together to form a box, (without a lid), into which a single sheet of P.V.C. material, (as used in the car park), was placed. The edges of the P.V.C. were trimmed level with the top of the box and pinned tight against the sides. The box was gently filled with a mixture of the four sub-base stone types used in the car park. Two 25 millimetre diameter inspection tubes were installed to stand vertically from the base of the box to the surface so that the depth of any water within the structure could be monitored using a dip-tape.

After construction, the device was 'primed' by adding a known volume of water to the collecting basin via the inspection tubes, thus creating a reservoir at the base of the device. The purposes of this priming was to enable measurements of any change in level of water within the basin to be taken with the dip-tape; and to calculate the porosity of the stones within the basin. This was a simple empirical method of finding out what depth of rainfall finally percolated to the bottom of the basin, which could then be compared with depths measured by raingauges to allow calculation of rainfall lost by evaporation.

The usual schedule of monitoring, i.e. weekly or when required, continued from August 1988 until February 1989. The observed levels together with the raingauge recordings were used to calculate the depth of rainfall lost through evaporation, assumed to have been the difference between the rainfall depth input in a period and the change in effective depth of water in the collecting basin. The full table of readings and calculated evaporation loss is presented in Table 6.1. This data shows values of evaporation of up to 5.5 mm per day.

Analysis of the relationship between total rainfall and the calculated totals for evaporation, during the periods monitored, showed a correlation coefficient of 0.47, which meant that only 21% of the variance in the volume of evaporation could be explained by the rainfall depth. An alternative avenue of investigation was suggested by the observations that there was only a marginal loss from the reservoir during periods of no rainfall and that the largest volume of evaporation occurred during long periods of rainfall rather than periods of heavy rainfall.

This suggested that the water was not only evaporating from the volume held in the bottom of the collecting basin but must have been 'held' within the surface layers of the pavement after rainfall thus making it available for evaporation. If this was the case then the longer it rained, or more precisely, the more often it rained, the more often the surface layers would be recharged with water that could be evaporated.

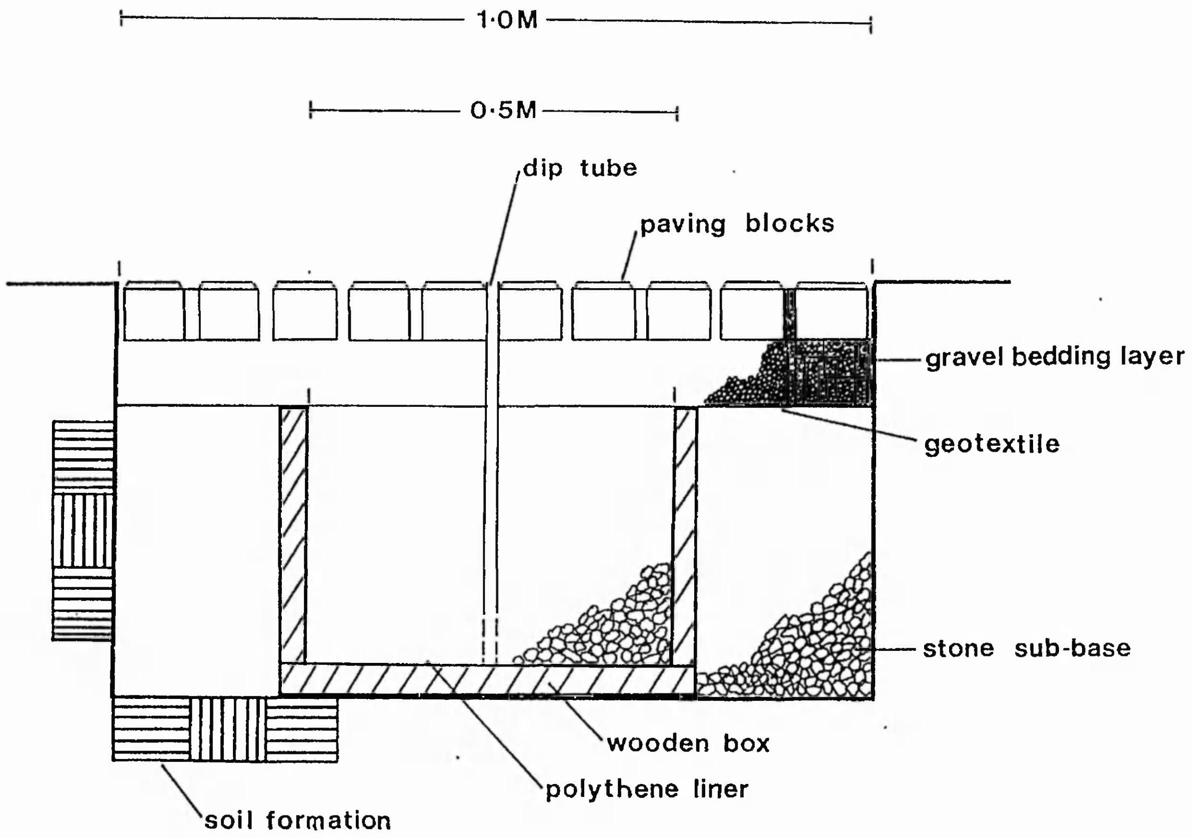


Figure 6.1 Cross-section of the 'evaporometer'.

Table 6.1 Evaporometer measurements and calculations

Columns:
- A. Date
- B. Depth to water
- C. Change in depth from previous reading
- D. Equivalent rainfall depth, (D=C*4/9)
- E. Rainfall for period of reading
- F. Evaporation total, (F=D+E)
- G. Average daily temperature for period
- H. Evaporation per day
- J. Total number of hours of rainfall for period
- K. Hours of rainfall per day for period

Date	Read.	Chng.	=Rf.	Rf.	Evap.	Temp.	E/d	hRf.	h/d
A	B	C	D	E	F	G	H	J	K
ddmmyy	cm	mm	mm	mm	mm	^oC	mm	h	h
090888	37.8								
110888	37.9	1	0.4	0.6	1.0	18.0	0.5	0	0
120888	36.9	-10	-4.4	9.2	4.8	15.0	4.8	3	3
180888	37.3	4	1.8	2.4	4.2	17.5	0.7	3	0.5
240888	36.7	-6	-2.7	10.3	7.6	15.9	1.3	3	0.5
310888	34.6	-21	-9.3	16.2	6.9	15.3	1.0	6	1
260988	39.0	44	19.6	33.3	52.9	15.3	2.0	17.5	0.7
131088	36.9	-21	-9.3	41.3	32.0	11.0	1.9	11.5	0.7
201088	33.1	-38	-16.9	21.0	4.1	13.2	0.6	11	1.6
271088	32.0	-9	-4.0	5.0	1.0	13.4	0.14	1.5	0.2
031188	32.0	0	0	0.3	0.3	5.8	0.04	0	0
101188	32.2	2	0.9	2.1	3.0	7.8	0.4	0.3	0
281188	35.2	*	*	13.9	*	5.5	*		
011288	31.2	-40	-17.8	34.0	16.2	7.3	5.5	13.8	4.6
041288	31.0	-2	-0.9	15.2	14.3	6.2	3.6	7.8	1.9
151288	32.2	12	5.3	4.2	9.5	8.4	0.9	5	0.5
211288	31.9	-3	-1.3	8.0	6.7	8.3	1.1	2.3	0.4
050189	32.6	7	3.1	0	3.1	8.1	0.2	2.3	0.16
240189	31.8	-8	-3.6	30.3	26.7	6.4	1.4	13.5	0.7
140289	31.6	2	0.9	21.1	22.0	7.0	1.0	10	0.5

* - data lost and device rebuilt after being disturbed by a tractor during grass cutting.

The rainfall records were inspected to calculate the number of hours of rainfall that fell per day in each period of monitoring, (this is included as a column of Table 6.1). An analysis of the relationship between the evaporation, (in millimetres per day), and the number of hours of rainfall per day showed a correlation coefficient of 0.89, i.e. almost 80% of the variation in daily evaporation loss could be explained by the duration of rainfall rather than the depth of rainfall.

The simplest interpretation of this was that a volume of rainfall was 'held' in the surface layers of the evaporometer from where it evaporated. Several rainfall events, or a longer period of rainfall, would allow this process to continue. However, a short, more intense, rainfall event would saturate the upper layers, and/or bypass their capacity to hold or absorb the water.

An estimate of the loss to evaporation from the reservoir at the base of the device can be made from those periods of no, or low, rainfall. For the two day period up to 11th August an evaporation total of 1.0mm was calculated during which time the rainfall was 0.6mm. This indicates that the evaporation rate from the base of the evaporometer was at least 0.2mm/d. Similarly, for the 15 day period to the 5th of January a daily evaporation loss of 0.2mm/day is recorded during a period of no rainfall. These observations allow for an order of magnitude assessment of the losses to evaporation from the sub-base, and will be discussed further in Section 7.1.2.

The results from the evaporometer were a confirmation that evaporation was one of the 'continuing loss' processes which could account for an increased loss with increased rainfall duration. The use of the evaporometer and its results enabled the first steps towards an understanding of the internal processes of the pavement affecting runoff. The next stage was to examine how the rainfall was 'held' in the surface layers.

6.1.2 Paving block porosity

The results from the evaporometer pointed towards the paving blocks and/or the gravel bedding layer being involved in the 'storage' of water, thereby making it available for evaporation. This storage was initially interpreted as a surface accumulation of water on stone surfaces and at the contact points between the surfaces. An examination of the effects due to the wetting of sub-base stones is made below. An examination of the hydrological impact of the pre-cast paving blocks on incident waters was made in the laboratory.

Water absorbency tests were conducted in the laboratory on an individual paving block by adding measured volumes of water to its surface and measuring the 'runoff ' from the block. Briefly, the experimental design was as follows: the base of the block was supported on a stage attached to a retort stand; rainfall was simulated by using two burettes; and the runoff was collected in a drip tray beneath the paving block. Measurements were made of 'rainfall' and 'runoff' at regular intervals, the difference between the two values being that volume on or within the block.

As soon as the initial trials began it became obvious that the paving blocks were far from impermeable. As the initial drips, (rainfall), from the burettes landed on the surface of the block they soaked into it. A typical set of results is presented in Table 6.2, (the figures discussed here are described in terms of rainfall and runoff). This shows that for a 90 minute simulated rainfall event of average intensity 12mm/h, (a 17.5mm event in total), 3.9mm or 22.2% was absorbed by the block. This gives a porosity of 5.6% for the blocks, (ignoring the cut-away portions of the blocks, see Figure 2.2). The porosity within the blocks is not visible to the eye and capillary forces prevent waters draining through the blocks once absorbed.

A study of Table 6.2 shows that 'runoff' began within 15 minutes of the start of the test whilst 'loss' or absorption was still continuing at the end of the test. Therefore, as 'runoff' and 'loss' processes continued simultaneously, the block must have had a finite absorption rate.

Figure 6.2 shows superimposed histograms of the volumes added and volumes absorbed, and it clearly shows the rate of loss into the blocks declining with time. This is indicative of high losses whilst the porosity of the outside edge of the blocks is taken up, which declines as the block 'fills up' from the outer edges. The variation of 'rainfall' shown in Figure 6.2 is due in part to the difficulty in maintaining a constant rate throughout the test, but the high rates towards the end of this particular test served to emphasise that the losses decline with time and that the rate of absorption was independent of the rate of rainfall.

It was estimated, from several of these tests, that each block could absorb a volume of water equivalent to approximately 6.0mm of rainfall over its full, rectangular, surface area, but this would only be possible if: the block was totally dry to begin with; and the rainfall event was of sufficient duration. Evaporation after, (and possibly during), the event would 'empty' the pores of the block before enabling the block to absorb rainfall from the next event.

A plot of the volume absorbed against time on a semi-log plot shows that the rate of absorption was related to a log-linear function, (an exponential decay of the form $Y = k \, \mathrm{Log} X$), which would allow the 'loss' to a rainfall event to continue for several hours. An estimation of the best fit of several decay curves gave a value of 0.87 for k, (when Y is measured in mm and X in minutes).

This storage within 'block porosity' gave another explanation for some of the observed losses and, indeed, could also explain some of the continuing losses.

Table 6.2 Concrete block soaking test

Time elapsed min.	Volume Added Period cc	Volume Added Total cc	Volume Collected Period cc	Volume Collected Total cc	Volume Absorbed Period cc	Volume Absorbed Total cc
2	1.9	1.9	0	0	1.9	1.9
15	27.9	29.8	6.5	6.5	21.4	23.3
21	17.8	47.6	10.0	16.5	7.8	31.1
30	31.2	78.8	19.5	36.0	11.7	42.8
40	24.7	103.5	18.0	54.0	6.7	49.5
50	36.1	139.6	28.0	82.0	8.1	57.6
60	37.7	177.3	31.6	113.6	6.1	63.7
70	38.8	216.1	33.3	146.9	5.5	69.2
80	64.9	281.0	60.0	206.9	4.9	74.1
90	70.0	351.0	66.0	272.9	4.0	78.1
Totals:	1.5 hour event, 351cc added, 273 cc collected. Therefore 78cc absorbed. Area 200cm^2					
Equivalent Rainfall	1.5 hour event. 17.5mm rainfall, 13.65mm runoff, Therefore 3.9mm, or 22.2% of rainfall, storage.					

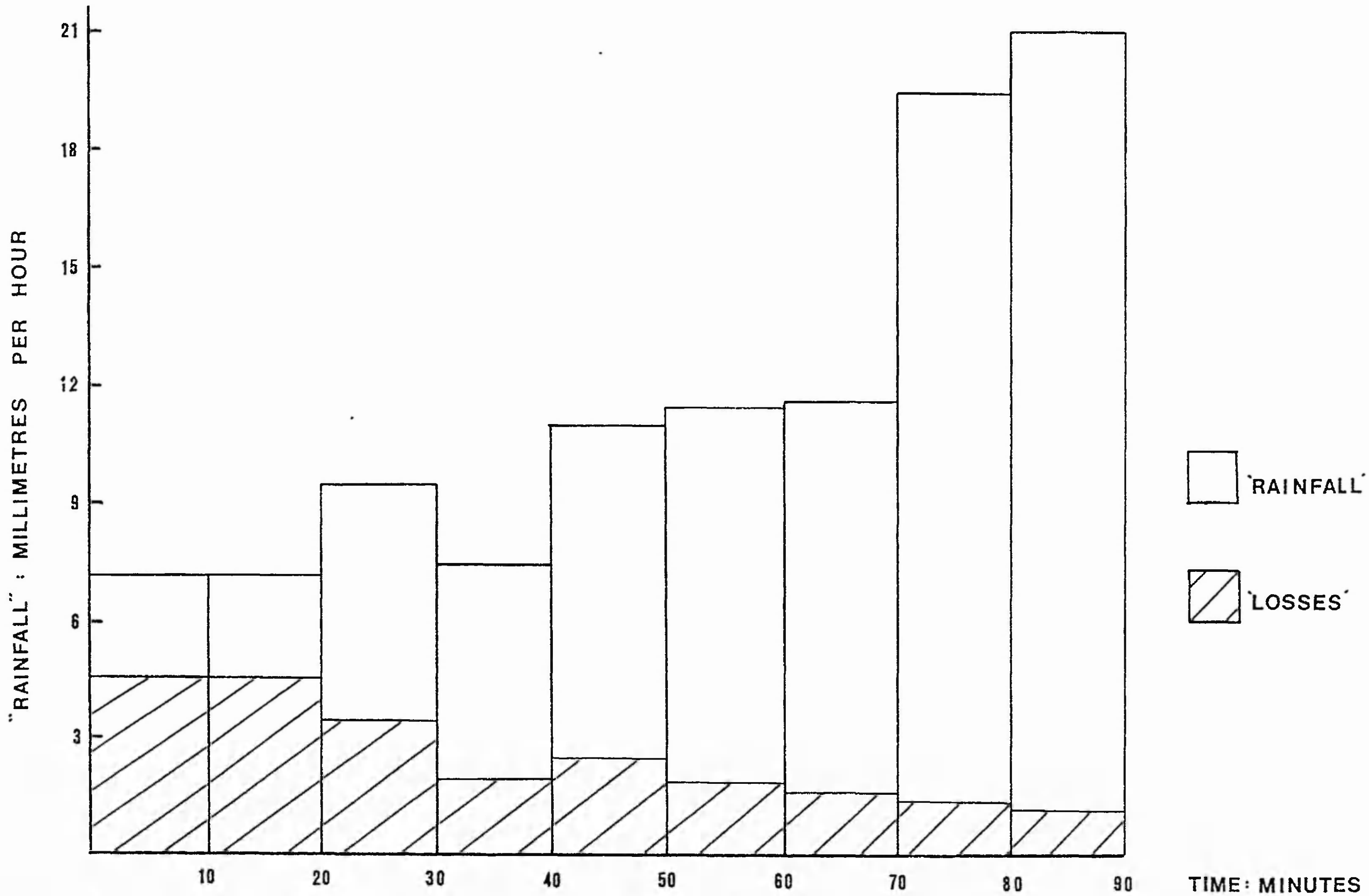


Figure 6.2 Histograms showing 'rainfall' and calculated losses for block soakage test, (see Table 6.2).

6.1.3 Stone surface wetting characteristics

A third area of water loss between rainfall and subsequent runoff follows from the examination of absorption by the paving blocks: loss due to wetting of the surfaces of the stones within the sub-base and bedding layer of a permeable pavement.

The nature of these 'losses' are illustrated in Figure 6.3 which shows how a mass of coarse rock particles is able to retain water: as a thin film on the surface of the rocks; in drops at contact points between rocks; and in small pools on the upturned surfaces of the rocks. Within soils, most water is retained by surface tension, or capillary forces resulting from attraction from within the water body itself. On rock surfaces the same phenomenon is present, the precise volumes of water held by surface tension will depend on many variables including: surface roughness, (dependent on rock type); inclination of surface; and temperature (or viscosity) of the water.

Boushi and Davies, (1969), found that the largest amount of water retained by coarse, (>100mm), non-saturated angular rocks was on near horizontal faces that retained small pools of water. Also, for rock particles with diameters smaller than about 30mm, most water storage occurred at contact points between the particles (smaller diameters give more contact points per unit volume). Apart from the gravel bedding layer and sub-base used on the car park, the other sub-base stones used in the research fitted between these two limits.

Examination of the 'micro' scale retention characteristics did not form part of this research. However, an attempt was made to examine the specific differences between the sub-base stones, especially with relation to water retention. Firstly, by a simple examination of the stones in 'hand specimen', and secondly, by a simple laboratory test.

Gravel. The gravel was a typical alluvium derived stone, which was, in the grading selected, a very uniform collection of smooth, hard, rounded, impervious stones. Each individual stone would itself have had poor retention qualities. However, the grading would have given several advantages over the other stones examined. The packing ratio for the stone would ensure many stone to stone contact points, possibly in excess of 8 per stone, and as shown by Boushi and Davies, (1969), this would have allowed for a high level of retention within the menisci spanning contact points. The passage of water through the unsaturated layers would have been highly convoluted, over many stone surfaces and through many stone-stone contact points aiding the attenuation performance of this sub-base.

B.f.s. Individual stones were observed to have varying amounts of a 'honeycombed' surface texture. The sub-rounded clasts often had a sponge-like surface texture with numerous sub-millimetre pores, evidence of gasses within the slag when it solidified. Many stones had several much larger holes (>4mm) which, if orientated correctly, would hold a comparatively large volume of water.

Granite. The large angular/sub-angular fragments comprising the granite sub-base were typical of that derived from an igneous deposit. Each fragment of this medium grained rock had several level surfaces almost at right angles, each surface having numerous indentations related to crystal boundaries. It is likely that many individual stones, when in-situ , had one of their level surfaces in a near horizontal position enabling water to rest in a pool on its surface. Also, pairs of stones that came to lie with a flat side resting against each other would have had multiple contact points increasing further the sub-base's retention/attenuation performance.

Limestone. The sub-rounded fragments of this hard sedimentary formation had sharp edges. However, their water retention/attenuation characteristics were limited by their fine grained texture which gave the surface of each stone a smooth texture. Another, significant difference between the limestone and the granite was the more rounded shape of the individual stones, so that, unless a granite fragment had its long axis vertical, it would present more surface area to any percolating water. In summary, the above descriptions cannot serve to quantify the coefficients of runoff from each of the sub-base stone but they do serve to indicate that the relative order of performance, namely, b.f.s., gravel, granite and lastly limestone, can be justified in terms of the textural characteristics of each stone type.

Upon delivery to site of the sub-base stones, up to 50kg samples of each stone were taken for analysis. Firstly, each stone type was put through a series of sieves to establish the particle size distribution curves for each stone. Each conformed to the grading specifications given in Figure 2.1. Further examination yielded the density, bulk density, void ratio and porosity of each sub-base stone type, Table 6.3.

Finally, a short test was conducted on each stone type to gauge its relative water retention capabilities. This test comprised filling a large bucket, (which had a perforated bottom), with one of the sub-base stones, to a depth of 30cm. The bucket and stones, once weighed, were immersed in water for a few seconds before being lifted out of the water and allowed to drain for a period of one minute. The dry weight of the bucket and stones, subtracted from the final, combined weight of the bucket, stones, and retained water, gave a value for the weight and hence volume of water retained by the stones. This test was repeated for each stone type.

The results from this test are presented in the last columns of Table 6.3 which shows that the b.f.s. was able to retain far more water than any of the other stone types. Limestone retained the least volume of water. These figures may be translated to equivalent rainfall loss on each bay as follows: gravel 12mm, b.f.s. 18mm, granite 12mm, and limestone 10mm. Although these figures appear to be quite high, in the field they are likely to be reduced by:

- Failure to wet all stone surfaces by percolating rainfall; and,
- The stones not drying out completely between storm events.

The first assumption comes from common sense and is illustrated in Figure 6.3 where the 'undersides' of stones remain dry. The second, from the evaporometer data which only moderate rates of evaporation from the pool of water beneath the geotextile, indicating that the primary method of removing water from the sub-base stones was draining by gravity through the drain outlet.

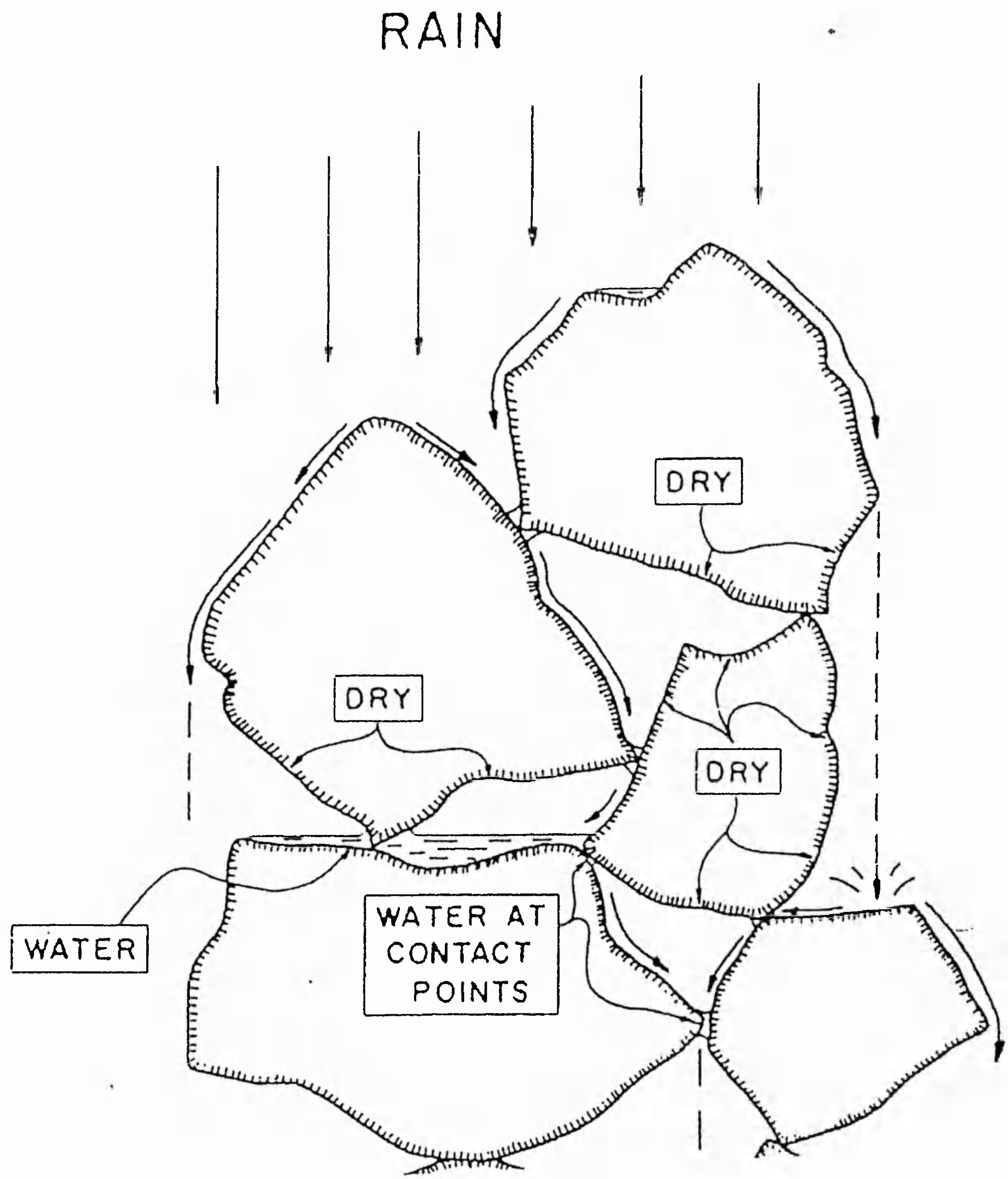


Figure 6.3 Water retention characteristics of coarse rock particles.

Table 6.3 Comparison of parameters for sub-base stones.

Stone	Grading	Density	Bulk Density	Void Ratio	Porosity	Water * Retention	
GVL	Type A	1.97	1.36	0.45	31	21	483
BFS	Type B	2.16	1.12	92.0	48	38	757
GNT	Type1X	2.82	1.64	0.73	42	21	483
LST	Type1X	2.68	1.53	0.75	43	16	371

* Water retention: 'l/bay' is an extrapolation to the number of litres of water that could be retained by the total weight of stone used in each bay

This seems to offer two conflicting observations: firstly, that water was retained in the sub-base and secondly, that there was a lack of evaporation from within the sub-base. This situation could be reconciled if evaporation is assumed to be the main cause of observed water loss between the total rainfall and runoff: the main method for accounting for evaporation being from the porosity of the paving block. However, if the blocks were the only influence on loss and attenuation, all the bays would have displayed very similar runoff and loss characteristics, which they evidently did not. Moreover, the runoff characteristics, (presented and discussed in Section 4.1), in terms of the order of performance of each stone, did appear to be related to the observations from the water retention of the sub-base stones, (presented in Table 6.3).

These observations could be reconciled further if it was assumed that after saturation a minor, but significant, evaporation loss was taking place from at least some of the sub-base stone surfaces. This would account for some of the discrepancy between observed 'losses' and the potential losses, as indicated by water retention tests. Also, it was unlikely that the wetting process was uniform, as the pathways through the surface of the car park to the base would have been numerous, therefore not all stone surfaces would have become wet before runoff had commenced. As wetting progressed and the wider exploitation of pathways for drainage took place, so the attenuation of the runoff hydrograph continued. This would produce a 'continuing loss', of some degree, until all available stone surfaces were wetted, however the time taken for this to happen throughout the sub-base is uncertain.

6.1.4 Experimental error

The last investigation to account for the observed 'continuing loss' processes was that of experimental error. There were two areas that required consideration: firstly, loss of water before measurement; and secondly, under-measurement or mis-calibration. (An over-measurement of rainfall was thought improbable because of the elaborate checks on rainfall volume described previously).

Water loss from the drainage system could have occurred via: holes in the P.V.C. membrane which formed the tank containing the stones; along the pipes from the tanks to the measuring pit; or by losses due to water not entering the tipping bucket. Some of the structural modifications described in Section 5.2 required the excavation of the sub-base stones in the gravel and limestone bays. This enabled an inspection of the P.V.C. membrane, one and a half years after installation, and in both cases no holes or puncture marks were observed.

The modern 'push-fit' drainage assembly from the tanks to the measuring pit are considered unlikely to have failed and no gross loss would appear to have taken place. In retrospect, for experimental control, it would have been prudent to conduct some form of 'drop test' on these lengths of pipe to have full confidence in their integrity. (In a drop test, the out-flow end of a pipe is sealed, the pipe filled with water, and the level of water at the in-flow end observed. If the level drops, this indicates leakage from the pipe -this is a common test in the water supply industry).

Occasionally, during a site visit, the drainage system within the measuring pit was found to be leaking from one of the pipe-gutter joints. When this was the case a note was made in a field note book so that any processed results from that bay would be treated with caution. The occurrence of leaks of this type received much attention, and as the project continued the number of instances declined.

The second area of experimental error which could have led to the observed losses was under-measurement or mis-calculation from the tipping buckets. The calibration of the tipping buckets has been covered in Section 2.3, and it is emphasised here that the frequent dynamic calibrations of all the tipping buckets resulted in a consistent set of results which gave a high degree of confidence in the observations. Also, checks on the calculated results by pen, paper, and calculator prevented any programming error undermining the results.

This analysis into possible experimental error as a cause of reduced runoff concluded that, within the limits of the available experimental techniques, no significant experimental error was reflected in the observations and results.

6.2 Experimental Simulations

One of the major problems with monitoring systems that interact with nature is that inputs to the system, such as rainfall, are strongly stochastic phenomena. Attempts to quantify and generalise the capabilities of the system are complicated by the variations in natural parameters. The simulations described in this Section were an attempt to control various factors affecting the performance of the permeable pavement, and so provide an understanding, by simplification, of the effects caused by changes of individual parameters.

6.2.1 Artificial rainfall events

As part of the effort to gain an understanding of the mechanisms affecting runoff, and to be able to observe directly a discrete event, several artificial events were simulated upon the car park. These events were performed by pumping waters on to the surface of one of the bays at a measured rate, for a pre-determined time, and monitoring the resultant runoff. The source of the water supply was the runoff held in the infiltration trench which was supplied via a submersible pump through a length of hose, the end of which was held above the car park for discharge. The rate of discharge was checked volumetrically before, during and after the test.

With only a point source of discharge the hose was constantly moved around the area of one bay in an effort to simulate rainfall. The total area of the bay was covered approximately every minute. The rate of discharge was determined by the power of the pump, and the head it was required to overcome. Some coarse flow control could be effected by the use of a pipe clamp and screw, fitted to the end of the hose, but in general, a discharge rate of some 20 litres per minute was achieved. This equated with a rainfall intensity of approximately 32.5 mm/h over the area of the limestone bay of the car park.

The length of hose limited the use of this experimental technique to the lowest end of the car park (nearest the infiltration trench). Plate 6.1 shows the application of water to the limestone bay during a simulated event.

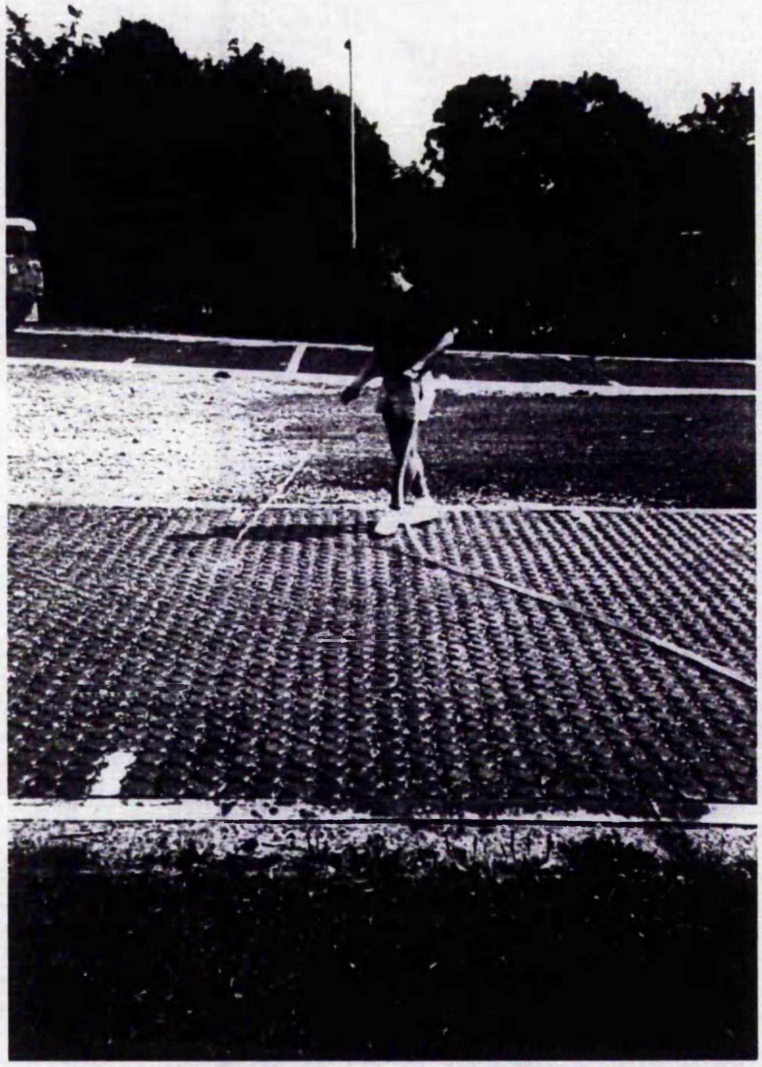


Plate 6.1. Application of artificial rainfall to limestone sub-base, two views.

The main advantage of this avenue of research lay in the repeatability of the 'rainfall', a secondary advantage was the general confirmation of the system constructed: known volumes of water could be applied to the pavement and the attenuation effects and operation of the system accurately observed.

In earlier discussions in Chapter 4 comparisons were made between similar events in an effort to draw conclusions on how different conditions before and during each event resulted in widely varying observed runoff. The major problem with this form of comparison is that no two events are exactly the same in their antecedent conditions, duration, and distribution of rainfall intensity, so that, at least some of the observed differences in runoff are due to the variation in natural rainfall. This was why the artificial event was useful: the rainfall duration and intensity was controlled, leaving any major differences between observed runoff from two artificial events, being the result of the antecedent conditions, (or conditions at the time of rainfall).

The major problem with this form of experiment was the difficulty in guaranteeing that the natural and artificial rainfall would have produced the same response from the car park. Although the artificial technique produced an average application of water which was taken as an equivalent rainfall event, the actual instantaneous effect, was of one small area receiving a thorough drenching for a second or two, whilst the rest of the car park received no 'rainfall'. This fact was mitigated to some extent by the constant movement of the hose to all parts of the pavement.

The likely effects of this method of simulating rainfall was to hydraulically overload, for a short time, each small area upon which the water from the hose was incident (probably less than a second). This probably resulted in a reduction of the attenuation capabilities of the stones within the car park, as water cascaded, rather than trickling down over them, and exploiting any circuitous pathways over the stone surfaces.

Section 6.1.2 described how the paving blocks were able to soak up a relatively high volume of water but only at a moderate rate, (approximately 3mm in the first hour of rainfall), with any excess supply passing over the block and on through the system. The twenty minute artificial rainfall greatly exceeded this rate, thereby reducing the potential for water 'loss' from the event into the porosity of the blocks. More water would probably have been able to soak into the blocks had the artificial rainfall been applied at a constant rate to the whole surface, but the time and materials needed to create a more sophisticated simulation of rainfall were not available. However, this means that the observed runoff rates from the artificial events was likely to be greater than those which could have been expected from natural events, with exactly the same parameters.

The first set of artificially generated storms took place in April 1988. An event of twenty minutes duration at an equivalent intensity of 34.2mm/h, designated number 8105, was applied to the limestone bay. The same event was repeated 5 days later, event number 8110. As a high degree of confidence could be placed on the similarity of the rainfall between the two events, it is useful to superimpose the two runoff curves upon one another, Figure 6.4. Besides the general similarity of the form of the curves the most noticeable feature was the higher peak generated by the second event.

The total computed runoff for the events was 51.9% for No. 8105 and 65.5% for No. 8110, a difference equivalent to 1.6mm of rainfall. The comparison of rainfall conditions antecedent to the events is given in Table 6.4 which gives some indication of how wet antecedent conditions result in a greater runoff volume for comparable events.

Another interesting difference between the two events was the time taken between the start of rainfall and the beginning of effective runoff. For event number 8105 the runoff from the limestone bay only began just as the application of the 20 minute rainfall ended, meaning that the whole bay had all 11.4mm, or 420 litres, of 'rainfall' held within the system, (although there is no doubt that most of this volume was moving and not 'held' stationary). For the second artificial event, with the wetter antecedent conditions, effective runoff began 5 minutes before the end of 'rainfall'.

Table 6.4 Antecedent conditions for artificial events, 8105 & 8110.

Event No.	Antecedent Rainfall (in mm)				Runoff
	0-24h	0-48h	0-72h	0-168hrs	(mm)
8105	0	0	0	5.5	5.9
8110	2.2	2.2	9.6	20.0	7.5

Finally, with regard to the plot in Figure 6.4, the peaks of the runoff of both artificial events occurred after the end of the simulated rainfall, not during rainfall. With a continuation of the constant input, the rate of runoff would have eventually achieved a constant rate approximately equal to the rainfall rate, (the runoff rate may have been slightly less than the 'rainfall' rate by an amount equivalent to any prevailing evaporation rate). If 'rainfall' had continued until such time that the runoff rate equalled the rainfall rate, then the difference between the rainfall and runoff totals would have yielded the total storage capacity within the pavement for the prevailing set of antecedent conditions.

Furthermore, a repetition of such a test for a series of different antecedent conditions may have provided valuable information relating to the variability and significance of wet and dry antecedent periods. These questions were not answered during the experimental work carried out at the pavement site and remain an interesting direction, and possibly a valuable tool, for future investigations.

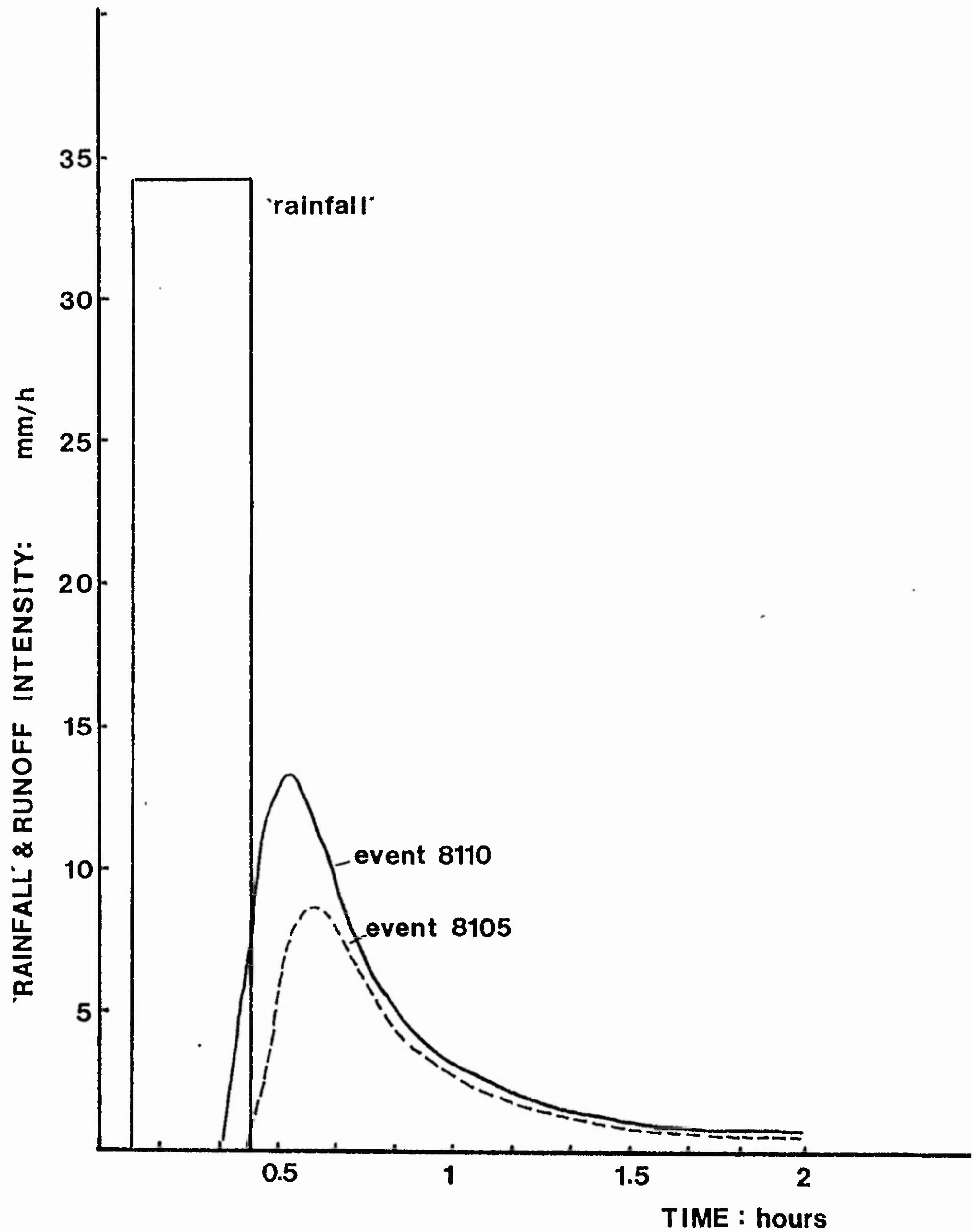


Figure 6.4 Rainfall-runoff plot for two artificial rainfall events: 8105 and 8110. (The simulated rainfall is common to both.

Attempts at producing artificial events on more than one type of sub-base bay, for comparison, would have been difficult for two reasons. Firstly, there was a restriction on the length of hose available; and secondly, the surface areas of each bay of the car park was different. This latter fact meant, for instance, that as the granite bay was almost 10% larger than the limestone bay, the same rate of flow for both bays would have delivered 10% less equivalent rainfall depth to the granite bay than to the limestone bay. It was thought unnecessary to compare the bays further as they could be compared with each other after every natural rainfall event.

Further restrictions on the use of this experimental technique were caused by the needs of the runoff quality monitoring system (Schofield, 1991), which would have been adversely affected by frequent applications of waters from the infiltration trench which contained a cocktail of waters from several events which had already drained through all four bays.

In retrospect, it would have been useful to have conducted a long running, and frequent, series of artificial events throughout the duration of the study period. If pursued, this would enable the effects of various antecedent conditions to be defined more precisely, and possibly enable accurate determination of evaporation rates and storage capabilities of the structure.

6.3 Summary and Implications for Further Study

Evaporation has been shown to be a significant factor in the relationship between rainfall and runoff. The rate of evaporation would have been somewhat dependent upon variables that were not monitored as part of this project, namely: water vapour saturation of the air; wind-speed; and solar radiation before and during rainfall. However, evaporation was not proven to be one of the processes producing a continuing loss as rainfall duration increased, only as a factor in drying out the pavement between events.

Research by Davies and Hollis, (1981), has indicated the potential for evaporation during rainfall. Therefore, the correlation coefficient between rainfall duration and evaporation from the evaporometer, of 0.89, may have been an indication, not only of the way in which storage in the paving blocks was filled, but also related to evaporative process taking place during rainfall. The experimental evidence for this is far from conclusive and further discussion on previous research and known phenomena is presented in Section 7.1.2. Given the available data this factor should be treated as an indistinguishable and undefined part of the pavement storage.

The pores within the surface layer of paving blocks were shown to be able to contain up to 6.0mm depth of rainfall. As the pores filled, (during the first few hours of rainfall), they also acted as a continuing loss process. After rainfall, evaporation from the blocks took place to restore their effectiveness for water storage. The time taken for evaporation from the pores would be very important in consideration of antecedent conditions for any subsequent event.

Initial observations pointed towards the paving blocks as the store from which the process of evaporation took place causing the observed losses from individual events. However, the results presented above only showed that the paving blocks provided a store from which evaporation took place and that the paving blocks could not fully account for all the observed losses.

The full data set for the permeable pavement, Table 4.1 columns 17 to 20, shows that the losses from rainfall events frequently exceeded 6.0mm. Taking into account the following: runoff not included in the calculated figures in Table 4.1, (see Section 4.1.5, 'runoff decay curves'); and, estimated potential storage within the block porosity, that still left accounted for 'losses' to either evaporation or storage capacity within the surface of the sub-base stones.

Short experiments on the surface wetting characteristics of the sub-base stone indicated that, when dry, they would be extremely effective at retaining and attenuating runoff, although in 'field conditions', the stones were probably subject to only a low rate of evaporation. However, the time taken for natural rainfall to percolate down and saturate all available surfaces, and stone-stone contact points, presented a third important method which accounted for observed continuing losses.

In all the 'continuing loss' processes a low intensity, long duration rainfall would enable each process to reach its optimum effectiveness at reducing runoff. However, as continued rainfall inevitably increases rainfall depth, this does not necessarily mean that the percentage runoff decreases with time, only that the total volume of runoff not measured, (as 'runoff'), increases.

The use of artificial events was a useful tool for confirming the operation of the system, and for an elementary examination of the effects of differing antecedent conditions. However, full exploration of this line of investigation was limited and it is considered that work of considerable value remains to be completed in this field.

The experiments and analysis presented in this Chapter were very important in gaining an understanding of the assortment of inter-related phenomena affecting rainfall-runoff relationships. However, there are many opportunities for further work which could refine, separate and accurately quantify these intimate processes. Such work, apart from its scientific value, would enable improved design proposals to be fully modelled by the researcher/engineer.

CHAPTER 7

MODELLING THE PERFORMANCE OF THE PERMEABLE PAVEMENT

The main aim of modelling the hydrological characteristics of the permeable pavement was to define the controlling variables and range of parameters for the rainfall-runoff relationship. This could then be used to establish the framework for design procedures for small on-site attenuation and storage devices. The modelling of such a small structure as the permeable pavement can have serious limitations in terms of its general uses and wider applicability. However, the method of construction of such a model, and the interpretation of the reasons for significance of individual parameters, should prove to be of wider use in small urban catchments.

Given the prodigious volume of data collected from the permeable pavement, the significant parameters in the rainfall-runoff relationship could be readily appraised using numerical and statistical modelling techniques. However, to gain an understanding of the systems and storage relationships for such a structure requires the derivation of a conceptual model. The formulation of a deterministic conceptual model for hydrological systems, if accurate, would enable the basic or fundamental controls upon such engineered systems to be understood, (and modifications to the systems to be modelled).

This Chapter is divided into two sections, each details the results from a different approach to model construction. In the first Section a physical conceptual model is developed from experimental results and observations gained during this research. In the second Section, statistical research models are generated from a sub-set of the data gathered during this research and, where appropriate, tested against a second 'test' data set.

The initial aim of both types of modelling was to predict the volume of runoff. This is at variance with many modelling approaches which aim at producing peak flows and hydrograph profiles. However, this is a requirement for the type of water management strategy being investigated. For on-site detention devices the critical design variable must be the size or volume of the receiving device. In addition, the results of the outlet or flow control devices (Chapter 5) has demonstrated that if the device is discharging to a sewer the peak flow can be moderated as required. For design purposes this would therefore require that the volume of runoff detained within the sub-base be accurately predicted. Analysis of peak flows does, however, form part of Section 7.2 which attempts to provide a simple method for predicting peak flow from the data set generated.

7.1 Physical Model

The aim of developing a physical conceptual model was to demonstrate the significant variables, and their parameters, which control the rainfall-runoff relationship for the permeable pavement, whilst also providing an understanding of the physics and hydrology of such a system.

Excluding any experimental errors, the input (rainfall) to the pavement eventually occurred either as runoff at the base of the pavement or was evaporated back to the atmosphere. At any given time a quantity of water will also be stored within the pavement and therefore, for a particular event, a portion of the rainfall will remain within the pavement causing a change in the storage. The following equation may therefore be used to describe the water balance of the pavement during a given time interval:

$$Ro = Rf - E - dS \qquad (7.1)$$

(where Ro is measured runoff, - as defined for this research; Rf is Rainfall; E is Volume evaporated; and, dS is the change in storage).

The depletion of the storage will take place either as drainage, to occur as runoff, or as water vapour through evaporation. It is useful to point out that in terms of hydraulic effect, the storage may therefore be acting as both an attenuation and a reduction factor. This demonstrates a particular inter-relationship which confuses the calculation of reduction and attenuation.

The following Sections provide analysis to show how each of the component parts of equation 7.1 may be accounted for.

7.1.1 Effective Runoff

Effective runoff is that runoff occurring directly as a result of the observed rainfall event. In cases of large and/or recent antecedent events, the receding or decaying limb of the antecedent event runoff may continue, such that measurement of its runoff becomes part of the measurement for the observed event. The runoff from the antecedent event is analogous to the base-flow from a stream hydrograph, and the procedure adopted for removal of this non-effective runoff is similar to the separation of base-flow within hydrograph analysis to determine effective rainfall.

Calculation of effective runoff was achieved using the base-flow or decay functions described in Section 4.1.5. The decay curves provided an estimation of the total remaining runoff from each stone type once the runoff rate had dropped to less than 0.25mm/hr. Therefore, the runoff rate immediately prior to each recorded event was examined and, if greater than zero, an estimation of the total remaining runoff for the antecedent event was made. This figure, calculated for each stone type, was then subtracted from the runoff total for each event. The effect of this adjustment of measured runoff was, of course, to lower the runoff from several events with significant antecedent events. The resulting adjusted, or effective runoff, calculations are included in Table 4.1.

7.1.2 Evaporation

As described above, that rainfall which does not occur as runoff will eventually be returned to the atmosphere through evaporation. Time series data for evaporation was not gathered, and calculations of prevailing evaporation rates were not possible with the variables measured. Direct measurements for evaporation rates were not made and calculations of likely evaporation rates are fraught with potential errors. However, model construction was possible by deriving assumptions from data gathered.

Firstly, it is sensible to consider the main factors controlling the evaporation process. If natural evaporation is considered as an energy-exchange process, it can be demonstrated that solar radiation is by far the most important single factor in controlling the rate of evaporation, (Shuttleworth, 1979). This, of course, highlights one of the main deficiencies in the data set for providing a model which can account for evaporative losses: the measured variables give poor/no indication of potential evaporation. At a given temperature, the evaporation rate is also subject to influence from wind speed and vapour pressure of the overlying air. The relationship between energy input and meteorological conditions can be understood in the following manner: if radiation/energy-exchange and the meteorological factors remain constant, the evaporation rate would become constant. If the wind speed doubled the evaporation rate would also double momentarily, however, the increased rate of evaporation would immediately begin to extract heat from its surrounding environment. As the temperature reached a new, lower equilibrium value, the rate of evaporation would diminish. This complex relationship should also be considered in relation to the surface and sub-surface environments of the permeable pavement and their exposure to evaporative processes.

It is likely that the pavement acted as a 'heat-sink' during periods of high solar radiation and this could have facilitated the evaporation from its surface. The sub-surface evaporation rate is more problematical, and was probably subject to large influence from the surface wind speed to facilitate the removal of water vapour. As a general point, if it is assumed that all un-measured runoff was eventually returned to the atmosphere, then evaporation from permeable pavements will be of benefit in 'urban climates' by lowering temperatures.

It is also prudent to point out that evaporation during rainfall events may have presented a significant loss to the observed rainfall. The mechanism of this process, which acts as an interception loss, is well understood and documented, (Merriam 1960, & Horton 1919). Indeed, meteorologists are aware that rainfall can evaporate after condensation from a cloud and before reaching the earth's surface. When such rainfall evaporates totally before reaching the earth's surface the rain is termed 'virga' (Austin-Miller & Perry, 1958). However, if such losses did occur, no specific data was gathered which could quantify the volume or rate at which such evaporation may have taken place, or differentiate it from other 'interception' losses. For the purposes of this investigation, this 'potential loss' through evaporation during an event is included in the change in storage for the event, 'dS'. (N.B. if evaporation was taking place during rainfall this would add another variable component to dS).

Following from this argument, the evaporation parameter within equation 7.1 is now only applicable for a water balance over an extended period of time, (when evaporation is significant). Thus for individual events we may concern ourselves only with the change of storage, and equation 7.1 becomes:

$$Ro = Rf - dS \qquad (7.2)$$

However, the consideration of evaporation is significant in relation to the effect of antecedent events, i.e. evaporation 'restored' or 'reset' the storage after rainfall. Therefore, account must be made of this loss as a change to storage <u>between</u> events. To assist in this we may re-examine the evaporometer data.

The evaporometer provided valuable insights into the operation of the pavement and the modelling concepts. Firstly, the relationship of rainfall 'loss' to rainfall duration was demonstrated, and secondly an estimation of the daily evaporation loss from the sub-base stones was made. This empirical data indicated that a daily loss of 0.2mm may take place. The variability of this figure is not known but it does provide a clear 'order of magnitude' assessment. The volume of rainfall taken into storage during rainfall is addressed in the following paragraphs on rainfall duration, and consideration of the loss to evaporation between rainfall events is expanded upon below in Section 7.1.4 on antecedent events.

The 0.2mm/d evaporation rate was obtained from the level changes in the pool of water at the base of the evaporometer during periods of no rainfall. It is difficult to estimate how this may reflect on the sub-base evaporation as a whole. The pool would have had a surface area of approximately 44% of the evaporometer, (equivalent to the stone porosity). As a first approximation, the surface area of this pool was only a fraction of that presented by the sub-base stones which, when wet, would have also been subject to the same rate of evaporation. Also to be included in the evaporation calculation would be the loss from the blocks and the gravel bedding layer, (and any sediments accumulated above the geotextile). For the purposes of an initial estimate for the conceptual modelling an evaporation rate of 2.0 mm/d was assumed. Some limitations to this figure were realised at the outset, however, the effects of recent antecedent events did appear to decay rapidly during the first 24 hours, so a 'significant' figure was chosen as a start point.

7.1.3 Rainfall Duration and Storage

The data from several areas of the research indicated that rainfall duration played an important part in the storage and runoff mechanisms. The original analysis for the runoff data demonstrated an increased loss with longer storms, and the evaporometer data showed a clear correlation between the length of an event and the (subsequent) loss to evaporation. From a conceptual viewpoint this indicates that rather than parts of the pavement storage acting as linear reservoirs, such as in a simple tank model, it actually has a limited inflow rate, i.e. a large event would not take-up or fill all the storage capacity during the early part of the event. This agrees with the data from the block soakage test which showed a clear 'absorption rate' unrelated to the volume or rate of rainfall applied. Furthermore, this rate of absorption was seen to decay with time and was attributed to the gradual filling of the block storage capacity, i.e. porosity, and the rate of decline being related to 'filling' of the storage from the outer edges to the interior of the block.

Analysis of the volume absorbed against time, (Section 6.1.2), demonstrated that the rate of decline observed in the absorption tests was of an exponential decay. Equation 7.3 describes the general function from a number of tests:

$$V = b.\mathrm{Log}\ (\mathrm{Duration}) \qquad (7.3)$$

where V is the volume adsorbed and 'Duration' is the elapsed time of the test. The best fit of the curves, by eye, gave a value of 0.87 for the slope estimate 'b'. This concept may be readily incorporated into the required model, and if we now re-examine equation 7.2 the storage variable may be equated to the volume or losses given by equation 7.3. Thus the storage variable may be substituted for, to give:

$$\mathrm{Runoff} = a.\mathrm{Rainfall} - b.\mathrm{Log}\ (\mathrm{Duration}) \qquad (7.4)$$

Where 'Duration' is the duration of the rainfall event. (This equation ignores any effect from antecedent rainfall).

There are, however, limitations upon the general use of this concept as it is based only upon the data for the paving blocks. The rate of storage uptake within other parts of the pavement was not tested. For example, on those areas or surfaces which could act as depression storage there would probably have been a linear filling of the storage. Alternatively, the extension of the 'wetting front' as rainfall progressed could have brought more areas of storage 'into play', thus making storage uptake analogous to the block uptake, i.e. logarithmic. In support of the last proposition we can again refer to the 'continuing loss' observations which support a gradual uptake of storage rather than a linear filling of numerous areas of depression storage.

In summary both types of storage uptake may have occurred, and limitations and improvements to the model are discussed in Section 7.1.8.

7.1.4 Antecedent Rainfall

Rudimentary observations in Chapter 4 outlined the effects antecedent rainfall had upon a rainfall event. In order to accurately model these effects the interaction of the antecedent events with the variables in equation 7.2 need to be examined. Obviously, an antecedent event will not differ from any other event in its effects on the system: the antecedent rainfall will produce runoff and a change in storage within the pavement. And it is therefore the occupation of this storage which affects the following or observed rainfall event.

From this observation three simple, additional assumptions can be made:

1. The total rainfall depth of the antecedent event is not directly related to the 'antecedent effect' on the subsequent or observed event.
2. The volume of the antecedent event remaining within the pavement is directly related to the 'antecedent effect', i.e. the loss, (or 'MMNRO', see Table 4.1), from the antecedent event; and,

3. The time interval between the antecedent event and the observation event will allow the storage to empty, primarily by evaporative processes.

From these observations we may now formulate some equations to model the 'antecedent effect'. Given that the total volume not occurring as runoff (MMNRO) is described by:

$$\mathrm{Ro} = \mathrm{Rf} - \mathrm{MMNRO} \qquad \text{and that;}$$

$$\mathrm{Ro} = \mathrm{a.Rf} - \mathrm{b.\ Log\ (Duration)}$$

then substituting for runoff gives,

$$\mathrm{MMNRO} = (1\text{-}a).\mathrm{Rf} + \mathrm{b.Log\ (Duration)} \tag{7.5}$$

This equation therefore describes the volume passed into storage at the end of an antecedent event.

During the time between the antecedent event and the observation event the storage within the pavement will be 'reset' by evaporation. The evaporation rate will depend upon many factors, as discussed above, but for a given set of atmospheric conditions the evaporation will be linear (subject to the supply of water from storage).

The effect of the antecedent event may now be calculated as the storage occupied at the beginning of the observation event. Thus the original volume not occurring as runoff will decay in a linear fashion through time, this may be stated as:

$$\mathrm{dS} = \mathrm{MMNRO} - \mathrm{c.ANTHRS} \tag{7.6}$$

where dS is the storage still occupied by the antecedent event at the beginning of the observation event, (the so called 'antecedent effect'); MMNRO as previously defined, (Table 4.1); and ANTHRS is the time interval between the end of the antecedent event and the beginning of the observation event, (in hours).

Substituting for MMNRO from equation 7.5:

$$dS = (1\text{-}a).Rf + b.\text{Log (Duration)} - c.\text{ANTHRS} \qquad (7.7)$$

This equation may now be added to equation 7.4 to take account of any antecedent effect, (and renaming the appropriate antecedent variables), Equation 7.8:

$$Ro = a.Rf - b.\text{Log (Duration)} + (1\text{-}a).\text{ANTRf} + b.\text{Log (DurANT)} - c.\text{ANTHRS}$$

where ANTRF and DurANT are the depth duration of the antecedent rainfall. This equation therefore constitutes a conceptual model to describe the runoff from an individual event using the following variables as input: event rainfall; event duration; antecedent event rainfall; antecedent event duration; and the period between the observation and antecedent events.

For those antecedent events where the calculation for evaporation given by ANTHRS is greater than the sum of ANTRF and DurANT, a negative antecedent value cannot be passed into the equation and must therefore be set to zero.

7.1.5 Parameter Estimation and Sensitivity Analysis

Equation 7.8 provides a theoretical-conceptual model for the rainfall runoff relationship of the pavement. It is based upon an understanding of the processes observed and tested. It does not include a variable to describe or differentiate between the stone types: all estimations and calculation in this Section will be based on data for the b.f.s. sub-base and the relative effects between the stones will be considered in Section 7.1.7.

To be able to use the model effectively, and to allow for an understanding of the relative effects or weight of the variables, an estimate and optimisation of the parameters is required. This may in the first instance be provided from data already obtained and optimised by iteration to find the 'best fit' the model can provide for the data set collected for the research, and then subjected to sensitivity analyses.

Logically and intuitively, the effect of rainfall depth for the observation event will be the 'strongest' variable. An initial method of parameter estimation was made from an understanding of runoff coefficients for urban surfaces and a simple rainfall-runoff plot for all events monitored. Equation 4.2 presented a coefficient of 0.69, (N.B. the constant or intercept in this equation is related to rainfall losses which are accounted for in the duration and antecedent variables). Modelling procedures make the use of this coefficient somewhat 'taboo', however, expectations were that the model parameter should be between 0.6 and 0.8, and therefore as a first estimate the parameter 'a' was designated as 0.70.

The parameter for the duration of rainfall should be directly related to those estimates provided during the block soakage tests i.e. 0.87. However the rate of absorption/storage occupation within the lower layer of the pavement was likely to progress at a slower rate due to the functions such as routing and the expansion of the wetting front. As an initial estimate 'b' was assigned a value of 0.60.

For an assessment of the antecedent rainfall parameters, consistency dictates the same values for 'a' and 'b' to be used for ANTRF and DurANT. Therefore an estimate of 0.70 for 'a', the rainfall parameter, dictates that the antecedent rainfall storage parameter is 1 - 0.7, i.e. 0.3. Similarly, the antecedent rainfall duration parameter should equal the 0.6 as assigned to DURATION. For the decay of the antecedent storage the parameter was estimated at 0.08mm/hr from the evaporation data, (Section 7.1.2).

Equation 7.8 may now be restated:

$$Ro = 0.7Rf - 0.6Log(Duration) + 0.3ANTRf + 0.6Log\ (DurANT) - 0.08ANTHRS \tag{7.9}$$

For the simplicity of the calculation and to provide for a sensible adjustment of the 'antecedent terms' the last three terms in the model were lumped to form an antecedent effect term, ANTEFF, where;

$$ANTEFF = 0.3ANTRf + 0.6Log\ (DurANT) - 0.08ANTHRS \tag{7.10}$$

It was realised at an early stage that the 'antecedent effects' calculation was the weakest and least significant part of the model, both from the significance of the parameters and the 'goodness' or 'correctness' of their predictive strengths. For this reason the optimisation of the antecedent effects term was conducted on a lumped approach.

Using equation 7.9 the optimisation process then consisted of scores of iterations to find the parameters which produced a 'best fit' model when compared to the data set for the b.f.s. runoff. The chosen procedure was to hold two of the three parameters constant and calculate the runoff resulting from a matrix of values of the third parameter for every event within the b.f.s. data set. These values were then compared to the measured runoff for each event within the data set. For the first few 'passes' each value from each matrix computation was compared, however, as the simulation grew more accurate and the differences between the measured and predicted values smaller, a set of model assessment variables were developed. Some details from the procedure are described below.

For the first model simulations a matrix of rainfall parameters of 0.65, 0.70 and 0.75 were computed whilst keeping the other parameters constant. Almost half of the measured or recorded values were less than all the predicted values, and most of these had significant (non-zero) antecedent effects. It was immediately apparent that the ANTEFF value was far too large. This was substantially reduced in the following iterations.

Once the general accuracy of the model was attained, (parameters 'correct' to one decimal place), the assessment of the accuracy of the model was made using a series of simple descriptive variables. For each matrix of runoff predictions the residual or prediction errors were calculated by subtracting the predicted value from the measured runoff. Optimisation of the model was then assessed from: a comparison of the number of positive and negative residuals - the closer the numbers the better; the maximum and minimum residuals; the mean and standard deviation of the residuals; and finally and most importantly, the sum of the squares of the residuals, (and the root mean square error, RMSE).

The optimised parameter values obtained, (to two decimal places), gave the following model:

$$BFSMMRO = 0.75Rf - 0.62Log\ (DURATION) + 0.39(ANTEFF) \qquad (7.11)$$

The correlation coefficient between the two sets of values is almost 0.93, and the mean predicted runoff was 3.26 mm compared to a mean measured runoff of 3.28mm. Another indication of the best fit of the model was given by the sum of the square errors (or residuals) which, for the 56 events was 73.7, this would have been reduced to 56.1 if the event with the largest residual was excluded, (i.e. a RMSE of approximately 1).

The relative importance or strength of each variable was assessed by sensitivity analysis, i.e. calculating the effect on the model due to changes to each variable. Such calculations, when dealing with variables of differing units and from different populations, should be based on the population of the model data set. Therefore, using the means and standard deviations of the observed data set, a 'standard' variation to each variable will yield a change to the predicted runoff. The significance or degree of this change indicates the importance of the variable to the accuracy of the model.

For rainfall, a change of one standard deviation (or 4.3mm as derived from the measured data set) would vary runoff by 3.25mm. A change in rainfall duration of one standard deviation, about the mean duration of 246 minutes, (standard deviation 174 minutes) would result in a change in runoff of 0.76mm for a lower duration or 0.33 mm for a higher duration, (the difference is caused by the natural logarithm function). A similar calculation for the ANTEFF variable indicated a change of 0.56mm runoff for the lumped variable for a change of one standard deviation. The significance of this effect would be 'diluted' when distributed between the three component variables of ANTEFF, and their individual effect would be much reduced by their low coefficients.

It is apparent that the event rainfall is by far the most significant to the model, followed by rainfall duration and the antecedent effects. However, the antecedent effect calculation was significant for those events where it was not zero, i.e. the 'fit' of the model would suffer considerably if the antecedent effects were not modelled.

It is also useful to consider the response of the model to variations in the parameters. These changes may seem to be of academic value only, but from an engineering perspective they could result as part of the clogging of the pavement or by application of slightly different methods of construction or construction materials. For rainfall, a change in the parameter of 0.1, i.e. to 0.65*Rf or 0.85*Rf would increase the mean error of the prediction by 0.8mm for the data set tested. This is equivalent to saying that if the rainfall parameter could be reduced by construction methods to 0.65 then the model predicts that mean runoff would be reduced by 0.8mm or 19%, (of the mean). A similar, but perhaps meaningless calculation, can be performed for the rainfall duration. Whilst the individual variables for the antecedent effect are probably not significant enough to warrant investigation unless the construction location was likely to present greatly improved evaporative conditions.

7.1.6 Residual analysis

To determine the conditions which caused the model to give poor predictions, a further examination of the residuals was undertaken. Table 7.1, (part iv), contains the residuals calculated by subtracting the models' predicted values from the measured runoff for the b.f.s sub-base. This Section briefly examines the possible causes for these large residuals.

Of the 56 events, 15 produced residuals of greater than 1.0mm, and of these only 4 were greater than 2mm. For 23 events the predicted runoff was within 0.5mm of the measured runoff. The largest residual was 4.2mm for event 8186, (row 47 in Table 7.1, part iv). This event was rather exceptional amongst the data set for the pavement as the measured runoff from the three bays monitored was greater than 90%. The antecedent event, occurring 9 hours previously, was of 3mm whilst the event itself was 10.5mm depth and 49 minutes duration, (this equates very closely to a one hour storm of one year return period for the Nottingham area, (Anon, 1975)).

For this event, the greater than average runoff was due to the occupation of the pavement storage by the antecedent event and by the, (low duration and high intensity), observation event 'by-passing' many of the attenuation/retardation mechanisms of the pavement. (As stated in Chapter 6, the pavements moderation of the rainfall is most effective for long, low intensity events). These observations are simply made, the question is, why they are not accounted for by the model.

To obtain, or force, the model to calculate 100% runoff, the 'positive' and 'negative' parts of the model must balance. Thus, the rainfall coefficient, the duration variable, and the ANTHRS variable are all functions which serve to lower the predicted runoff beneath 100% of rainfall, whilst the DurANT and ANTRf variables work in the opposite sense. Therefore, for the model to predict 100% runoff, the reductions caused by the rainfall parameter, rainfall duration and time between antecedent event and observation event, must be balanced by the 'small' DurANT and ANTRf variables. From a cursory examination of the variables and parameters in equation 7.9 it would seem unlikely that the parameters for DurANT and ANTRf could be realistically changed to have the required effect. Rather we must examine the event itself in comparison to the rest of the rainfall data set, or 'population'. If this event is taken out of the data set, the predictive qualities of the model would improve markedly. However, to do so implies that the model should be limited in its use, i.e. it should only be applied to specific ranges of event variables.

This approach would almost certainly lead to a more robust and accurate model, however, this takes the modelling down an 'academic' line. A more useful approach would be to recognise the limitations of the model and try to improve or change the model to account for more extreme events, i.e. those of most interest to the practicing urban hydrologist or civil engineer. For the current model this would require more data, either from full scale field trials, (by event simulation or very long-term monitoring), or from laboratory based simulations, (adequately calibrated). In the interim, the model should only be applied to events which 'fit' into the current data set.

All of the remaining events with large residuals had significant or non-zero antecedent effects except events 7277 and 7349. The average runoff from these events was only between 20 and 30% from 11.0 and 9.0 mm events respectively. These events had the longest dry periods recorded for the b.f.s. data set, over 10 days each. The limitations to the model demonstrated by these events relates to the 'antecedent effect' function, as each of these events only had antecedent events of 1mm. The effect of the ANTHRS function in the model would have negated the antecedent event and resulted in a lumped ANTEFF variable equal to zero. Thus the model did not take account of the prodigious dry period. If we assume that the dry period contributed to the low runoff from these events, then a more accurate model should take this effect into account. This point is elaborated upon in Section 7.1.8.

Other standardised techniques of residual or error analysis were also undertaken. For example, plotting the following functions:

(predicted)/(observed)	against (observed)
SQR{[observed - predicted]^2}	against (observed)

where, SQR is the square root, and observed and predicted refer to the measured and calculated values respectively. In all cases, for all stones, the plots demonstrated little or no correlation and no discernible bias for large events. The largest, (most erroneous), Y-axis values resulted from division by small numbers, i.e. when runoff, and therefore rainfall, was small.

7.1.7 Model predictions for different sub-bases

All of the above analysis refers to models created for predicting the runoff from the b.f.s. sub-base bay. There are two approaches for deriving similar models for the different sub-base stones. A similar exercise to that reported above, involving the iteration of the parameters to match observed values: this would be valid and may bring out useful differences between the response of the individual stones, but it does not take the conceptual modelling in any fresh or useful direction. A second approach would be to examine the differences between the sub-base stones to determine if parameters or 'relative coefficients' that describe their different retention properties may be found.

This Section examines both approaches to estimate the changes in runoff between the sub-base stones. Firstly, deriving runoff predictions from conceptual models as for the b.f.s. above. And secondly by examining the sub-bases physical properties to determine useful functions which may describe the sub-base stones' various retention-attenuation properties.

<u>Individual conceptual models</u>. The derivation of the individual models for the gravel, granite and limestone sub-base stones followed exactly the same procedure as described for the b.f.s above. The iterations and optimisations produced the following models:

$$GVLMMRO = 0.76*Rf - 0.55*Log(DURATION) + 0.40*ANTEFF$$

$$GNTMMRO = 0.80*Rf - 0.50*Log(DURATION) + 0.31*ANTEFF$$

$$LSTMMRO = 0.88*Rf - 0.62*Log(DURATION) + 0.47*ANTEFF$$

(all variables are described in table 7.1, part i).

A comparison of the measured and predicted values for each of the models is given in Table 7.2 part i. The relationship between the predicted and the measured values for each sub-base had a correlation coefficient of almost 0.95.

The parts of these models which are of most interest are the parameters and how comparison of the parameters may indicate characteristics of the sub-base stones, (this assumes that the characteristics are related to the parameters as estimated and that the difference between the parameters is significant).

The coefficient for rainfall varies between 0.75, (for b.f.s.) and 0.88, (for limestone), these figures are certainly the most important in determining the runoff from each bay, (as the sensitivity analysis indicated that the rainfall parameter was the most significant).

The parameters for the duration function are 0.62 for both the b.f.s. and limestone, and 0.5 and 0.55 for the gravel and granite respectively. If the construction of the model reflects physical processes as predicted, this indicates that the granite and gravel are somewhat slower at taking water into storage than the b.f.s and limestone. This actually compares very well with an understanding of the texture of the stones, (Section 6.1.3), with the gravel and granite being much 'smoother' than the b.f.s. and limestone.

The parameters for the antecedent effects 'lumped variable' are 0.39 and 0.4 for the b.f.s. and gravel with the granite being lower, at 0.31, and the limestone higher, at 0.47. No sensible, or significant, inferences can be made from this figure as the 'variable' contains three functions which describe two different processes, i.e. the uptake of storage by the antecedent event and the subsequent loss from storage by evaporation. In retrospect it may have been interesting to perform the optimising procedure on all the component variables instead of the lumped antecedent effect. However, the procedure adopted was to ensure that negative values were not produced for the combined antecedent variables, (and to simplify the processing).

Stone 'relative coefficients'. Descriptions of the physical characteristics of the individual stones were given in Section 6.1.3, which also included a short test to demonstrate the variability of the sub-base stones to allow water to drain through them. The empirically derived volumes that the sub-base bays could store were, 12mm, 18mm, 12mm and 10mm for the gravel, b.f.s., granite and limestone bays respectively. These results were 'scaled-up' from the wetting of approximately 15kg of stones in a bucket, to the 20+ tonnes in the pavement. These storage volumes were not been reflected in the field results but the general relationship between the different stone types to attenuate rainfall were, i.e. the b.f.s. sub-base is the most effective sub-base for reducing the rainfall.

The original aim of this part of the research was to generate coefficients for different stone types and gradings, determined from a simple laboratory test, which could then be used to calibrate the existing model to predict runoff for those stones. The main difficulty in this approach is to separate the losses caused by the storage on the sub-base stones, from the losses to the concrete blocks and bedding layers. It is not expected that these losses occur in the same proportion for each event, especially as the antecedent effects may vary in their effect between the upper and lower layers of the pavement, (and therefore vary between the different stones as well).

After many trials at generating 'conceptually significant' parameters for this purpose, without success, the last alternative was to adopt a simple $Y = mX + C$ relationship between the runoff parameters for the different stone types and generate a model, and test it, using the b.f.s. predicted runoff results obtained previously.

Three equations were generated to predict the measured runoff from the gravel, granite and limestone bays using the b.f.s. predicted values derived and reported in Section 7.1.5. The data set was divided into two sub-sets creating an even and odd data set according to event number, (Table 7.2,ii).

This particular division of the data set is discussed more fully in Section 7.2).

The values from the 'even' sub-set of the predicted values for the b.f.s. sub-base were plotted against the observed measurements, for the same events, from each of the other stone types. The straight line correlation between the b.f.s. values and each of the stones set of values was then used to predict the runoff from the 'odd' events using the b.f.s. values as input.

The equations generated from the 'evens' data set were:

$$GVLMMRO = 0.65 + 0.998*PBFSMMRO$$

$$GNTMMRO = 1.24 + 0.933*PBFSMMRO$$

$$LSTMMRO = 1.02 + 1.03*PBFSMMRO$$

Each of these equations had a correlation coefficient of over 0.85. The equations were then used to calculate the runoff from each sub-base bay given the predicted values from the odd numbered rows. The results of these predictions and the residuals obtained by subtracting the predicted values from the measured values, are presented in Table 7.2, parts ii-iv. The RMSEs (root mean square errors) of the residuals from each test data set were, 1.1, 1.4 and 1.52 for the gravel, granite and limestone bays respectively.

It is evident that these equations are not as accurate as the individual conceptual models. This is almost certainly a function of producing the results via two 'models', the b.f.s. conceptual model and the equation relating this model to the observed flows in the other bays, (the limitations of the data division into sub-sets is discussed in Section 7.2). However, in engineering terms it is likely that the actual difference to the volume of runoff predicted by the different procedures will not be significant. Refinement of the procedure to describe the attenuation capabilities in laboratory-scale tests may therefore prove to be of value.

7.1.8 Conceptual modelling and model improvements

The conceptual modelling exercise proved quite successful in terms of the ease of its use and the qualities of the 'fit' of the model. However, during the course of the research potential improvements to the original data set and possible improvements to the modelling procedures were observed. These are now considered against a background of other conceptual methods and models used in urban hydrology.

The main requirement of a hydrological model is usually the estimation of peak flow for flood prediction for given rainfall parameters, (e.g. depth or return period). Early modelling efforts concentrated on rural catchments and was based upon statistical flood frequency analysis of long observation records and some deterministic modelling based on simulating the observed flows. In the urban environment, deterministic models are used almost exclusively. This is due to the easily defined, (and assumed constant), contributing area which discharges into a well-defined system, (the storm drainage system).

Base-flow and effective rainfall. Hydrological models are designed to demonstrate and calculate the transformation of rainfall to runoff and aim to predict the discharge (Q) parameters, i.e. how much and how quickly. For most catchments the models are concerned with separation of rainfall into 'effective rainfall' and losses (or non-effective rainfall). The effective rainfall is then passed through a routing function and the 'base-flow' for the receiving system is added to give the total runoff, (the first calculation to determine effective rainfall may have been the subtraction of the base-flow from the discharge hydrograph).

With regard to the permeable pavement the calculation of effective rainfall was simply quoted as runoff that was measured above a defined threshold level, (Section 2.4.2), i.e.measured runoff was effective rainfall. For the conceptual model this was refined by the subtraction of the flow caused by the decaying limb of the antecedent event, if present, (these were relatively small volumes and only occurred in a tenth of the data set). This calculation of the remaining flow from the antecedent event was made possible by the runoff decay calculations and the whole operation was analogous to base-flow separation for normal hydrograph analysis.

Loss modelling. As an additional step within effective rainfall calculation, conventional models need to calculate the losses to rainfall and adopt some form of loss function(s) to distribute these losses with time, and account for runoff being less than 100%. One of the most typical loss functions is the 'constant proportional', i.e. a constant percentage loss from effective rainfall. This constant proportional loss may also be termed a runoff coefficient, C, where:

$$C = (Rf - L)/Rf$$

where: C is the runoff coefficient, Rf is the rainfall depth, and L is the total loss.

Typical values for C are quoted in standard texts, (Chow, 1965), and may vary between 0.1 for woodland to 0.95+ for city centres. For urban catchments a typical approach is to designate paved areas to have a runoff coefficient of 1.0 and to designate the pervious areas as zero. This is recognised as an over-simplification, but may be found to be effective, (in areas where hydrometric and sewer/drainage-system flow data is poor), as the errors may have a tendency to cancel each other out.

Several other loss models are used: constant loss; initial and constant; initial and variable; and, variable-proportional, (these variable loss rates can be either simple decay functions or decay functions of proportional loss). A typical 'loss rate' curve was presented by Horton, (1940), who proposed an initial loss to represent interception and depression storage, and a negative exponential equation to represent infiltration.

There are several problems with such loss calculations, some of which are analogous to the pavement models. Firstly, the more involved the loss model the more difficult it is to calculate, especially if several 'field' parameters are require on-site tests that need to be 'scaled-up'. For 'loss rate' curves and initial interception estimates, the initial starting point in relation to antecedent conditions may need to be determined, (considered further below). Several models fail to take account of periods when the predicted rate of loss exceeds the rainfall rate. For the pavement model, it was assumed that the rate of storage uptake, or rainfall loss, was a logarithmic function of the duration of rainfall. At first glance this would therefore seem to suffer from an error generated when the rainfall intensity is lower than the potential rate of storage uptake, (initially about 3mm/hr). This may be true for events that are of long enough duration to apparently take-up the available storage but are of too low an intensity to do so in practice. However, such events are probably the exception rather than the rule. What is judged to be of more importance in improving the quality of the models, is a consideration of how the calculated 'antecedent effect' may have reduced or altered the function for storage uptake.

It is also considered that the 'storage uptake' function could be improved by laboratory testing of the rate of 'rainfall' loss into volumes of the sub-base stones, (in a similar fashion to the block storage test). Although an argument for a similar log-function for storage within the sub-base was discussed in Section 7.1.3, this has yet to be specifically tested and may prove to have a 'linear' component. This could then be incorporated into the model as a linear function of rainfall duration.

A further improvement to the pavement model would have been to take account of the decay curve predictions, (or measurements), for the observation event, (i.e that runoff occurring after the runoff rate had fallen below the defined threshold). This could have been accomplished by re-defining runoff to a lower threshold level, or subtracting the predicted decay volumes from each event. The advantage of this procedure would have been to define more accurately the difference between reduction and attenuation within the pavement. For example, the final runoff measurements and predictions are a combination of the rainfall held in storage, (to be returned to the atmosphere), and rainfall that had yet to occur as runoff, (often many hours or days after the end of rainfall). However, in practice, the extension of the monitoring period would have resulted in many cases of overlapping events in which superficially insignificant showers would complicate the issue.

The case for making additional runoff estimations from the base-flow/decay curve calculations may deserve further consideration: however, the correlation of the volumes discharged by the different stones during this 'post-runoff definition' period did appear 'strong' so that any apportioning of losses between pre- and post- runoff definitions would not have significantly altered the trend of the comparisons between the stones. (For example the b.f.s. runoff have increased by less than the limestone runoff).

For the Flood Studies Report, (Anon, 1975), the loss calculation technique estimates a runoff coefficient in terms derived from a soil index, the total event rainfall and a 'catchment wetness index' (CWI), with losses apportioned during the event using a variable proportional loss dependent on the value of CWI. Such 'wetness' indices are discussed below.

<u>Antecedent effects</u>. Hydrological models frequently take account of the moisture conditions of the catchment at the onset of the storm by application of an index based on the antecedent precipitation. The rate at which moisture is depleted from a particular catchment is roughly proportional to the volume in storage and should decrease logarithmically with time, (Kohler and Linsley, 1951). Thus, it would appear that the linear loss rate to evaporation, used in the pavement model to 're-set' the storage between events (ANTHRS), is non-standard and unrelated to theory.

However, for a given set of conditions the rate of evaporation is constant and the decay function commonly used is based on the reduction of available moisture (often in soils) and the evaporative 'energy' required to overcome growing capillary pressure in soils as they dry, (Hillel, 1980). As part of the investigative work for model derivation, several log-decay functions were applied as part of the 'antecedent effects' calculation: this included numerous storage and decay coefficients. No improvement to the models presented was found. However, one may consider that the logarithmic absorption into the blocks would be complimented by a logarithmic evaporative decay. This may be the case and could be the subject of further laboratory testing. In the opinion of the author, the different layers of the pavement may be subject to both logarithmic and linear antecedent decay functions. The failure of the models to account for long dry periods, discussed in Section 7.1.6, demonstrates some of its limitations with regard to antecedent effects. An alternative approach would have been to remove the ANTHRS variable from the antecedent effect 'lumped variable', (ANTEFF), and compute its value separately. However, this would have moved the model from being a purely conceptual model, as negative antecedent storages would have been generated.

Another obvious omission from the calculations for the antecedent effect is an iteration of all antecedent events. This method entails keeping a constant record of the 'catchment' rainfall and performing a decay function on each event to arrive at a 'storage occupied' figure for the following event. In this way the effects of a large event occurring not as the last event, but as the last but one (etc.), can be included in the equation for the observation event. The procedure for the pavement calculation was only to consider the single event immediately prior to the observation event. This is likely to cause errors when large antecedent events are 'camouflaged' by much smaller events occurring sometime later but whilst the antecedent effects of the large event remain within the catchment. The significance of this omission has not been determined explicitly as the available data set did not lend itself to this examination.

Future work in this field could include: consideration of all aspects of the storage up-take and depletion by evaporation, including the monitoring and calibration of an energy budget for evaporation, (with estimates for the significance of solar radiation). This work is most suited to examination by small-scale laboratory research.

Conceptual catchment models. Many conceptual models have been formulated in recent years and their mathematical derivations are reported in standard texts. A limited review of the structure and operation of some of these models follows.

The O'Donnell model, (Dawdy and O'Donnell, 1965), is constructed around four storages whose contents vary with time:

- A surface storage, (in effect interception storage);
- A channel storage, the volume of water in receiving streams etc.
- A soil moisture storage, within the soil unsaturated zone.
- A groundwater storage, within the saturated zone.

The O'Donnell model has nine control parameters which determine the interaction between the storages resulting in a final total discharge. This particular model and reference is presented because of the parallels in the methodology of 'fitting' the conceptual model with that presented for the pavement models in this Chapter. Dawdy and O'Donnell (1965) introduced the computer to search for the best set of parameters values, not merely to do the model calculations, (including the optimisation procedures which aim to reduce the sum of the mean square error , SMSE).

The Stanford Watershed Model, (Crawford and Linsley, 1966), was developed as a computer study aimed at simulating the whole of the land phase of the hydrological cycle in a catchment. Later versions of the model required over 30 parameters to describe physical catchment characteristics or initial conditions. This model has been applied to catchments worldwide.

Component models are a group of deterministic models which aim to describe a single part of the hydrological cycle. By their nature they are often very well researched so that the physics of each sub-catchment is described explicitly and the models become quite complex. The amount of hydrological data and catchment measurements required to 'drive' these models have restricted their use in the engineering field. In some respects the pavement model can be viewed as a small sub-catchment model which may be suitable for integration to, or as a sub-routine, to larger urban models. However, the author is careful to point out that the aim of this modelling process has been directed towards useable engineering solutions. (And, of course, the use of the model would require that more permeable pavements be constructed as part of urban stormwater management strategies).

Small catchments. Most of the models reported above are generally used on much larger areas than the very small 'catchment' presented by the permeable pavement. Two approaches to the modelling of small urban catchments has been presented by Swinnerton et al, (1972, 1973), for the design of motorway storm drainage, (the smallest of these catchments was 0.7 acres, (0.28 ha)). The first approach at modelling, enabled derivation of a dimensionless hydrograph which could be transformed to a storm hydrograph using values of the peak-flow and the durations of the rise and recession hydrograph. Calculation of these values was performed by regression equations on the data set. A second approach, using the same catchments, was via a conceptual model which contained separate functions for the runoff hydrograph for when there was no rainfall <u>within</u> the event duration. The final, 'best-fit' model resulted from a linear reservoir model with two storage constants - one for each of the different rainfall functions. The storage constants were found in terms of the impermeable area and the length of the catchment. (The approaches outlined for these studies is similar to those presented in this and the following Section).

Rational & TRRL Hydrograph Method. The most widely applied methods of designing stormwater sewerage systems for small urban catchments are the rational method, (originally developed by Mulvaney, 1850), and a development of the time-area concept of catchment response, the TRRL hydrograph method, (Watkins, 1962). The rational method was originally developed as part of land drainage estimations and is known to give over-estimations of runoff in the urban environment. However, as a first approximation it still provides a valuable technique. The TRRL hydrograph method is developed from the time-area method which itself is an extension of the rational method. The major development of the technique was to be able to divide the catchment into areas which 'contribute' to runoff in the same time period, thus enabling the building of a hydrograph from a number of discrete, contributing areas which may receive rainfall at differing intensities. Loss-rate calculations are performed, as discussed above, by the application of a runoff coefficient which is applied to each time unit of rainfall in turn. (Interestingly, the early development of the model included a division of the urban area into impervious areas with a runoff coefficient of 100% and permeable areas with a runoff coefficient of 0%, this 'development' reduced the flexibility of the model and attracted much criticism).

Table 7.1 , part 1. **Variables used for models and model predictions, (runoff calculations are adjusted, all other variables as for Table 4.1).**

EVENT	=	Event series No. and Julian day code e.g. 8023 is Julian day 23, 23rd January, 1988. (.5 = 2nd event, for the same day).
Rf	=	Rainfall, recorded in millimetres.
ANTMM	=	Antecedent rainfall, depth of the most recent event before observation event. Expressed in mm.
ANTHRS	=	Time from mid-point of antecedent event to beginning of rainfall for observation event.
DURANT	=	Duration, in minutes, of the antecedent event.
ANTEFF	=	Antecedent Effect. Defined in Section 7.1.5.
GVLMMRO	=	Gravel bay. Recorded runoff in mm.*
BFSMMRO	=	BFS bay. Recorded runoff in mm.*
GNTMMRO	=	Granite bay. Recorded runoff in mm.*
LSTMMRO	=	Limestone bay. Recorded runoff in mm.*
DURATION	=	Duration of rainfall, in minutes.
RFPINT	=	Rainfall peak intensity, in mm/hr, for 15 min resolution.
BFSPINT	=	BFS bay. Peak runoff intensity, in mm/hr. 15min res.
PGVLMMRO	=	Predicted runoff from model for gravel bay, in mm.
PBFSMMRO	=	Predicted runoff from model for BFS bay, in mm.
PGNTMMRO	=	Predicted runoff from model for Granite bay in mm.
PLSTMMRO	=	Predicted runoff from model for limestone bay in mm.

* - Figures adjusted to give 'effective runoff' after removal of 'base-flow'.

Table 7.1, part ii.

EVENT	RF	GVLMMRO	BFSMMRO	GNTMRO	LSTMMRO
1 7207.0	9.2	3.3	*	*	3.6
2 7221.0	3.9	0.0	0.0	0.0	0.0
3 7225.0	21.5	15.2	12.0	14.4	*
4 7230.0	10.0	4.1	3.9	5.2	*
5 7235.0	10.1	5.2	5.2	6.0	6.7
6 7238.0	8.5	3.7	3.5	4.2	5.0
7 7249.0	12.4	6.4	6.1	7.6	8.6
8 7262.0	7.6	2.8	2.7	3.7	4.2
9 7277.0	11.0	2.8	3.2	3.8	3.8
10 7280.0	3.3	0.7	1.6	2.0	1.3
11 7282.0	13.3	6.4	6.5	7.3	8.1
12 7283.0	16.5	11.5	10.0	10.1	13.1
13 7287.0	13.1	7.3	6.3	7.8	8.7
14 7293.0	6.4	1.5	2.1	3.3	2.2
15 7300.0	12.5	6.6	6.2	7.6	*
16 7304.0	7.5	2.1	2.5	3.4	3.1
17 7312.0	2.8	0.0	0.0	0.0	0.0
18 7314.0	3.0	0.0	0.1	0.6	0.2
19 7315.0	8.6	4.7	4.4	5.1	5.4
20 7319.0	5.4	0.7	0.0	1.4	1.6
21 7323.0	9.1	4.1	3.0	3.8	4.6
22 7328.0	3.8	1.9	1.5	1.8	2.3
23 7349.0	9.0	2.0	1.6	2.2	2.1
24 7350.0	4.7	2.4	2.6	2.9	2.9
25 7350.5	3.1	1.2	0.9	1.4	1.5
26 8008.0	6.4	2.2	1.7	2.4	3.3
27 8022.0	8.2	1.8	1.0	3.2	3.4
28 8031.0	6.9	2.3	1.0	2.9	2.1
29 8035.0	10.3	5.2	3.9	*	4.7
30 8035.5	6.1	3.5	2.7	3.8	3.6
31 8038.0	5.3	1.1	0.2	1.5	0.5
32 8045.0	4.4	2.1	1.6	3.0	2.5
33 8063.0	3.0	0.1	0.0	0.1	0.0
34 8069.0	5.6	1.0	0.0	0.9	0.9
35 8072.0	8.4	1.7	0.9	3.1	*
36 8073.0	8.3	5.8	3.8	7.2	7.1
37 8078.0	2.7	0.0	0.0	0.0	0.0
38 8078.5	4.0	1.7	1.2	2.5	2.3
39 8086.0	6.7	2.1	1.1	*	0.8
40 8094.0	17.0	8.4	7.1	10.6	9.1
41 8107.0	6.6	1.9	1.7	3.1	4.1
42 8125.0	4.7	3.0	3.0	4.2	3.8
43 8152.0	8.2	*	*	4.9	4.1
44 8155.0	6.9	*	*	3.6	4.7
45 8160.0	8.4	*	*	4.6	4.2
46 8178.0	10.9	*	*	4.4	3.0
47 8186.0	10.5	*	10.6	9.7	10.5
48 8186.5	2.9	*	1.1	1.4	1.2
49 8187.0	9.4	*	2.3	3.5	3.3
50 8188.0	4.7	*	1.5	2.4	2.0
51 8188.5	6.6	*	3.5	4.0	4.3
52 8189.0	6.9	*	3.8	4.7	4.6
53 8195.0	6.6	*	2.4	3.6	3.2
54 8195.5	5.7	*	2.7	2.7	3.0
55 8198.0	17.0	*	9.2	10.9	10.6
56 8203.0	4.4	*	*	0.5	0.5
57 8203.5	22.6	*	16.6	18.1	19.2
58 8204.0	10.7	*	5.2	5.9	5.9
59 8212.0	5.2	*	0.0	0.0	0.0
60 8231.0	4.6	*	0.0	0.2	0.1
61 8268.0	8.8	*	3.8	4.1	4.6
62 8271.0	9.2	*	4.4	5.2	5.5

Table 7.1, part iii.

EVENT	RF	DURATION	ANTMM	ANTHRS	DURANT	ANTEFF
1 7207.0	9.2	342.0	1.0	53.0	1.0	0.0
2 7221.0	3.9	147.0	1.0	170.0	1.0	0.0
3 7225.0	21.5	266.0	2.0	22.0	2.0	0.0
4 7230.0	10.0	219.0	21.0	110.0	5.0	0.0
5 7235.0	10.1	145.0	3.0	20.0	3.0	0.0
6 7238.0	8.5	189.0	2.0	28.0	2.0	0.0
7 7249.0	12.4	438.0	4.0	26.0	4.0	0.0
8 7262.0	7.6	327.0	15.0	60.0	8.0	0.0
9 7277.0	11.0	309.0	1.0	240.0	1.0	0.0
10 7280.0	3.3	51.0	2.0	7.0	2.0	1.9
11 7282.0	13.3	311.0	1.0	13.0	1.0	0.3
12 7283.0	16.5	949.0	12.0	8.0	4.0	4.6
13 7287.0	13.1	461.0	20.0	56.0	12.0	0.0
14 7293.0	6.4	330.0	2.0	31.0	2.0	0.0
15 7300.0	12.5	236.0	3.0	10.0	3.0	1.8
16 7304.0	7.5	497.0	12.0	86.0	4.0	0.0
17 7312.0	2.8	125.0	9.0	200.0	7.0	0.0
18 7314.0	3.0	103.0	3.0	39.0	3.0	0.0
19 7315.0	8.6	343.0	2.0	10.0	3.0	1.5
20 7319.0	5.4	210.0	9.0	87.0	7.0	0.0
21 7323.0	9.1	188.0	2.0	10.0	2.0	1.3
22 7328.0	3.8	188.0	6.0	10.0	6.0	3.0
23 7349.0	9.0	592.0	1.0	250.0	1.0	0.0
24 7350.0	4.7	133.0	9.0	12.0	6.0	3.3
25 7350.5	3.1	131.0	5.0	5.0	10.0	4.1
26 8008.0	6.4	139.0	7.0	57.0	4.0	0.0
27 8022.0	8.2	293.0	2.0	7.0	2.0	1.9
28 8031.0	6.9	245.0	2.0	60.0	2.0	0.0
29 8035.0	10.3	216.0	6.0	74.0	2.0	0.0
30 8035.5	6.1	44.0	10.0	9.0	5.0	4.1
31 8038.0	5.3	195.0	6.0	72.0	1.0	0.0
32 8045.0	4.4	224.0	2.0	20.0	2.0	0.0
33 8063.0	3.0	40.0	2.0	125.0	2.0	0.0
34 8069.0	5.6	247.0	1.0	77.0	1.0	0.0
35 8072.0	8.4	795.0	3.0	5.0	3.0	2.8
36 8073.0	8.3	390.0	7.0	20.0	7.0	1.2
37 8078.0	2.7	178.0	2.0	62.0	2.0	0.0
38 8078.5	4.0	263.0	3.0	4.0	3.0	3.0
39 8086.0	6.7	301.0	12.0	40.0	4.0	0.0
40 8094.0	17.0	490.0	2.0	108.0	2.0	0.0
41 8107.0	6.6	182.0	1.0	8.0	1.0	1.0
42 8125.0	4.7	137.0	1.0	2.0	2.0	2.7
43 8152.0	8.2	241.0	1.0	4.0	1.0	1.9
44 8155.0	6.9	25.0	8.0	72.0	2.0	0.0
45 8160.0	8.4	265.0	1.0	84.0	1.0	0.0
46 8178.0	10.9	259.0	1.0	420.0	1.0	0.0
47 8186.0	10.5	49.0	3.0	9.0	5.0	2.3
48 8186.5	2.9	122.0	2.0	6.0	3.0	2.4
49 8187.0	9.4	124.0	10.0	22.0	1.0	0.4
50 8188.0	4.7	94.0	10.0	23.0	2.0	0.6
51 8188.5	6.6	116.0	5.0	4.0	3.0	3.5
52 8189.0	6.9	104.0	7.0	22.0	3.0	0.3
53 8195.0	6.6	117.0	3.0	3.0	3.0	3.2
54 8195.5	5.7	30.0	6.0	9.0	3.0	2.7
55 8198.0	17.0	519.0	2.0	44.0	1.0	0.0
56 8203.0	4.4	133.0	17.0	96.0	9.0	0.0
57 8203.5	22.6	446.0	5.0	17.0	3.0	0.8
58 8204.0	10.7	296.0	23.0	22.0	8.0	4.9
59 8212.0	5.2	78.0	1.0	24.0	1.0	0.0
60 8231.0	4.6	134.0	1.0	96.0	1.0	0.0
61 8268.0	8.8	178.0	3.0	7.0	3.0	2.4
62 8271.0	9.2	222.0	1.0	8.0	2.0	1.5

Table 7.1, part iv.

EVENT	RF	BFSMMRO	PREDRO.	RESIDUAL
1 7207.0	9.2	*	3.3	*
2 7221.0	3.9	0.0	0.0	0.0
3 7225.0	21.5	12.0	12.7	-0.7
4 7230.0	10.0	3.9	4.2	-0.2
5 7235.0	10.1	5.2	4.5	0.7
6 7238.0	8.5	3.5	3.1	0.4
7 7249.0	12.4	6.1	5.5	0.5
8 7262.0	7.6	2.7	2.1	0.6
9 7277.0	11.0	3.2	4.7	-1.5
10 7280.0	3.3	1.6	0.8	0.8
11 7282.0	13.3	6.5	6.5	-0.0
12 7283.0	16.5	10.0	9.9	0.1
13 7287.0	13.1	6.3	6.0	0.3
14 7293.0	6.4	2.1	1.2	0.9
15 7300.0	12.5	6.2	6.7	-0.5
16 7304.0	7.5	2.5	1.8	0.7
17 7312.0	2.8	0.0	0.0	0.9
18 7314.0	3.0	0.1	0.0	0.1
19 7315.0	8.6	4.4	3.4	1.0
20 7319.0	5.4	0.0	0.7	-0.7
21 7323.0	9.1	3.0	4.1	-1.1
22 7328.0	3.8	1.5	0.7	0.7
23 7349.0	9.0	1.6	2.8	-1.2
24 7350.0	4.7	2.6	1.8	0.8
25 7350.5	3.1	0.9	0.9	-0.0
26 8008.0	6.4	1.7	1.7	-0.1
27 8022.0	8.2	1.0	3.4	-2.3
28 8031.0	6.9	1.0	1.8	-0.8
29 8035.0	10.3	3.9	4.4	-0.5
30 8035.5	6.1	2.7	3.8	-1.1
31 8038.0	5.3	0.2	0.7	-0.5
32 8045.0	4.4	1.6	0.0	1.6
33 8063.0	3.0	0.0	0.0	0.0
34 8069.0	5.6	0.0	0.8	-0.8
35 8072.0	8.4	0.9	3.3	-2.3
36 8073.0	8.3	3.8	3.0	0.8
37 8078.0	2.7	0.0	0.0	0.0
38 8078.5	4.0	1.2	0.7	0.5
39 8086.0	6.7	1.1	1.5	-0.4
40 8094.0	17.0	7.1	8.9	-1.8
41 8107.0	6.6	1.7	2.1	-0.4
42 8125.0	4.7	3.0	1.5	1.5
43 8152.0	8.2	*	3.5	*
44 8155.0	6.9	*	3.2	*
45 8160.0	8.4	*	2.8	*
46 8178.0	10.9	*	4.7	*
47 8186.0	10.5	10.6	6.4	4.2
48 8186.5	2.9	1.1	0.1	0.9
49 8187.0	9.4	2.3	4.2	-1.9
50 8188.0	4.7	1.5	0.9	0.6
51 8188.5	6.6	3.5	3.4	0.1
52 8189.0	6.9	3.8	2.4	1.4
53 8195.0	6.6	2.4	3.3	-0.8
54 8195.5	5.7	2.7	3.2	-0.6
55 8198.0	17.0	9.2	8.9	0.3
56 8203.0	4.4	*	0.3	*
57 8203.5	22.6	16.6	13.5	3.1
58 8204.0	10.7	5.2	6.4	-1.2
59 8212.0	5.2	0.0	1.2	-1.2
60 8231.0	4.6	0.0	0.4	-0.4
61 8268.0	8.8	3.8	4.3	-0.5
62 8271.0	9.2	4.4	4.1	0.3

Table 7.2, part i. **Conceptual model predictions for the gravel, granite and limestone sub-base bays. (Variables and unit listed in Table 7.1, part i).**

EVENT	GVLMMRO	PGVLMMRO	GNTMMRO	PGNTMMRO	LSTMMRO	PLSTMMRO
7207.00	3.29	3.78	*	4.44	3.58	4.48
7221.00	0.00	0.22	0.00	0.62	0.00	0.34
7225.00	15.22	13.27	14.38	14.41	*	15.46
7230.00	4.12	4.64	5.22	5.31	*	5.46
7235.00	5.24	4.94	6.01	5.59	6.66	5.80
7238.00	3.75	3.58	4.18	4.18	5.01	4.23
7249.00	6.41	6.08	7.58	6.88	8.62	7.14
7262.00	2.78	2.59	3.68	3.19	4.20	3.10
7277.00	2.84	5.21	3.76	5.93	3.85	6.13
7280.00	0.67	1.11	1.96	1.27	1.31	1.37
7282.00	6.38	7.06	7.29	7.85	8.14	8.27
7283.00	11.48	10.62	10.07	11.20	13.13	12.44
7287.00	7.30	6.58	7.79	7.41	8.74	7.73
7293.00	1.50	1.67	3.27	2.22	2.18	2.04
7300.00	6.61	7.21	7.55	7.82	*	8.45
7304.00	2.07	2.29	3.36	2.90	3.08	2.75
7312.00	0.00	0.00	0.00	0.00	0.00	0.00
7314.00	0.00	0.00	0.60	0.08	0.21	0.00
7315.00	4.72	3.94	5.13	4.44	5.38	4.67
7319.00	0.70	1.16	1.39	1.65	1.65	1.44
7323.00	4.12	4.55	3.77	5.06	4.61	5.37
7328.00	1.86	1.15	1.75	1.30	2.34	1.44
7349.00	1.99	3.33	2.21	4.01	2.12	3.96
7350.00	2.40	2.20	2.93	2.33	2.93	2.65
7350.50	1.17	1.29	1.36	1.30	1.47	1.61
8008.00	2.19	2.15	2.40	2.65	3.32	2.57
8022.00	1.84	3.88	3.18	4.32	3.41	4.60
8031.00	2.26	2.22	2.87	2.77	2.15	2.66
8035.00	5.18	4.87	*	5.55	4.66	5.73
8035.50	3.53	4.17	3.83	4.24	3.56	4.93
8038.00	1.06	1.17	1.52	1.64	0.54	1.44
8045.00	2.13	0.37	3.00	0.81	2.49	0.52
8063.00	0.07	0.25	0.10	0.56	0.00	0.35
8069.00	1.03	1.23	0.90	1.73	0.85	1.51
8072.00	1.70	3.84	3.09	4.26	*	4.58
8073.00	5.83	3.51	7.24	4.03	7.12	4.17
8078.00	0.00	0.00	0.00	0.00	0.00	0.00
8078.50	1.70	1.19	2.46	1.35	2.30	1.49
8086.00	2.08	1.95	*	2.51	0.82	2.36
8094.00	8.36	9.51	10.59	10.50	9.13	11.12
8107.00	1.93	2.57	3.13	3.00	4.09	3.07
8125.00	2.99	1.95	4.18	2.14	3.80	2.36
8152.00	*	3.96	4.85	4.40	4.12	4.69
8155.00	*	3.47	3.56	3.91	4.71	4.08
8160.00	*	3.32	4.57	3.93	4.20	3.93
8178.00	*	5.23	4.43	5.94	3.00	6.15
8186.00	*	6.76	9.66	7.17	10.53	7.91
8186.50	*	0.51	1.44	0.65	1.16	0.69
8287.00	*	4.64	3.50	5.23	3.31	5.46
8188.00	*	1.31	2.41	1.67	2.04	1.60
8188.50	*	3.81	4.00	4.00	4.26	4.52
8189.00	*	2.81	4.73	3.29	4.60	3.33
8195.00	*	3.69	3.64	3.90	3.24	4.38
8195.50	*	3.56	2.68	3.71	2.99	4.20
8198.00	*	9.48	10.91	10.47	10.57	11.08
8203.00	*	0.65	0.52	1.07	0.51	0.84
8203.50	*	14.15	18.10	15.29	19.16	16.50
8204.00	*	6.95	5.90	7.23	5.89	8.18
8212.00	*	1.56	0.00	1.98	0.00	1.87
8231.00	*	0.80	0.18	1.23	0.14	1.01
8268.00	*	4.80	4.09	5.20	4.62	5.66
8271.00	*	4.63	5.17	5.14	5.51	5.47

Table 7.2, part ii. Conceptual model predictions of gravel runoff derived from the b.f.s. model. (All variables and units are listed in Table 7.1, part i).

EVENT	RF	PBFSMMRO	GVLMMRO	PGVLMMRO	RESIDUALS
1 7207.00	9.20	3.28	3.29	3.93	-0.63,
3 7225.00	21.50	12.66	15.22	13.29	1.93,
5 7235.00	10.10	4.49	5.24	5.13	0.11,
7 7249.00	12.40	5.53	6.41	6.17	0.24,
9 7277.00	11.00	4.70	2.84	5.34	-2.50,
11 7282.00	13.30	6.51	6.38	7.14	-0.76,
13 7287.00	13.10	6.02	7.30	6.66	0.64,
15 7300.00	12.50	6.61	6.61	7.25	-0.63,
17 7312.00	2.75	0.00	0.00	0.65	-0.65,
19 7315.00	8.60	3.37	4.72	4.01	0.71,
21 7323.00	9.10	4.03	4.12	4.67	-0.55,
23 7349.00	9.00	2.79	1.99	3.44	-1.45,
25 7350.50	3.10	0.72	1.17	1.37	-0.20,
27 8022.00	8.20	3.30	1.84	3.94	-2.11,
29 8035.00	10.30	4.39	5.18	5.03	0.15,
31 8038.00	5.35	0.74	1.06	1.39	-0.33,
33 8063.00	3.00	0.00	0.07	0.65	-0.58,
35 8072.00	8.40	3.15	1.70	3.79	-2.10,
37 8078.00	2.70	0.00	0.00	0.65	-0.65,
39 8086.00	6.70	1.49	2.08	2.13	-0.06,
41 8107.00	6.60	2.09	1.93	2.73	-0.80,
43 8152.00	8.20	3.40	4.05	*	
45 8160.00	8.40	2.84	3.48	*	
47 8186.00	10.50	6.27	6.90	*	
49 8287.00	9.40	4.19	4.84	*	
51 8188.50	6.60	3.24	3.88	*	
53 8195.00	6.60	3.13	3.78	*	
55 8198.00	17.00	8.87	9.51	*	
57 8203.50	22.60	13.46	14.08	*	
59 8212.00	5.20	1.20	1.85	*	
61 8268.00	8.80	4.23	4.87	*	

Table 7.2, part iii. **Conceptual model predictions of granite runoff as derived from the b.f.s. model. (All variables and units are listed in Table 7.1 part i).**

EVENT	RF	PBFSMMRO	GNTMMRO	PGNTMMRO	RESIDUALS
1 7207.00	9.20	3.28	4.30	*	
3 7225.00	21.50	12.66	14.38	13.05	1.33,
5 7235.00	10.10	4.49	6.01	5.43	0.58,
7 7249.00	12.40	5.53	7.58	6.40	1.18,
9 7277.00	11.00	4.70	3.76	5.62	-1.86,
11 7282.00	13.30	6.51	7.29	7.31	-0.02,
13 7287.00	13.10	6.02	7.79	6.86	0.94,
15 7300.00	12.50	6.61	7.55	7.41	0.14,
17 7312.00	2.75	0.00	0.00	1.24	-1.24,
19 7315.00	8.60	3.37	5.13	4.38	0.74,
21 7323.00	9.10	4.03	3.77	5.00	-1.23,
23 7349.00	9.00	2.79	2.21	3.85	-1.63,
25 7350.50	3.10	0.72	1.36	1.91	-0.55,
27 8022.00	8.20	3.30	3.18	4.32	-1.14,
29 8035.00	10.30	4.39	5.34	*	
31 8038.00	5.35	0.74	1.52	1.93	-0.41,
33 8063.00	3.00	0.00	0.10	1.24	-1.14,
35 8072.00	8.40	3.15	3.09	4.18	-1.09,
37 8078.00	2.70	0.00	0.00	1.24	-1.24,
39 8086.00	6.70	1.49	2.63	*	
41 8107.00	6.60	2.09	3.13	3.19	-0.06,
43 8152.00	8.20	3.40	4.85	4.42	0.44,
45 8160.00	8.40	2.84	4.57	3.89	0.68,
47 8186.00	10.50	6.27	9.66	7.09	2.57,
49 8287.00	9.40	4.19	3.50	5.15	-1.66,
51 8188.50	6.60	3.24	4.00	4.26	-0.26,
53 8195.00	6.60	3.13	3.64	4.16	-0.53,
55 8198.00	17.00	8.87	10.91	9.52	1.39,
57 8203.50	22.60	13.46	18.10	13.80	4.31,
59 8212.00	5.20	1.20	0.00	2.36	-2.36,
61 8268.00	8.80	4.23	4.09	5.19	-1.10,

Table 7.2, part iv. Conceptual model predictions of limestone runoff as derived from the b.f.s. model. (All variables and their units are listed in Table 7.1, part i).

EVENT	RF	PBFSMMRO	LSTMMRO	PLSTMMRO	RESIDUALS
1 7207.00	9.20	3.28	3.58	4.40	-0.82,
3 7225.00	21.50	12.66	14.06	*	
5 7235.00	10.10	4.49	6.66	5.64	1.01,
7 7249.00	12.40	5.53	8.62	6.71	1.90,
9 7277.00	11.00	4.70	3.85	5.86	-2.01,
11 7282.00	13.30	6.51	8.14	7.72	0.42,
13 7287.00	13.10	6.02	8.74	7.22	1.51,
15 7300.00	12.50	6.61	7.83	*	
17 7312.00	2.75	0.00	0.00	1.02	-1.02,
19 7315.00	8.60	3.37	5.38	4.49	0.88,
21 7323.00	9.10	4.03	4.61	5.17	-0.56,
23 7349.00	9.00	2.79	2.12	3.90	-1.77,
25 7350.50	3.10	0.72	1.47	1.76	-0.29,
27 8022.00	8.20	3.30	3.41	4.42	-1.01,
29 8035.00	10.30	4.39	4.66	5.54	-0.89,
31 8038.00	5.35	0.74	0.54	1.79	-1.25,
33 8063.00	3.00	0.00	0.00	1.02	-1.02,
35 8072.00	8.40	3.15	4.26	*	
37 8078.00	2.70	0.00	0.00	1.02	-1.02,
39 8086.00	6.70	1.49	0.82	2.55	-1.73,
41 8107.00	6.60	2.09	4.09	3.17	0.92,
43 8152.00	8.20	3.40	4.12	4.53	-0.41,
45 8160.00	8.40	2.84	4.20	3.95	0.25,
47 8186.00	10.50	6.27	10.53	7.48	3.06,
49 8287.00	9.40	4.19	3.31	5.34	-2.03,
51 8188.50	6.60	3.24	4.26	4.36	-0.09,
53 8195.00	6.60	3.13	3.24	4.25	-1.00,
55 8198.00	17.00	8.87	10.57	10.16	0.41,
57 8203.50	22.60	13.46	19.16	14.88	4.28,
59 8212.00	5.20	1.20	0.00	2.25	-2.25,
61 8268.00	8.80	4.23	4.62	5.38	-0.76,

7.2 Regression Models

The large amount of data often gathered from hydrological systems can be complex and inter-related. Therefore statistical analysis using computers to process the data and produce statistically 'best fitting' models from chosen parameters is a valuable addition or alternative to mathematical/conceptual deterministic models.

The use of regression analyses within hydrological modelling is widespread. In a review of methods to predict flood response to urbanisation, Hall (1974) stated 'a feature common to the methods outlined is the...application of multiple linear regression analyses.' Indeed many of the final equations for the flood studies report, (Anon., 1975) were derived from regression analyses of catchment and rainfall characteristics. However, successful results are dependent upon accurate definitions of the parameters and variables which relate to the physical processes in the system modelled.

The novel approach reported here is based upon the observations of the 'smoothing' of the hydrograph caused by the soaking and percolation through the pavement system. Observations of the hydrographs from the event data showed that the general effect upon the incident rainfall was not only to depress the peak and extend the recession 'limb', but also to 'blend' or 'smooth' the peaks and troughs of the rainfall hyetograph. This is a common feature to drainage systems but is usually related to the routing of the runoff through much larger systems such as storm-sewers and natural, catchments. For the pavement this smoothing is a direct result of the reduction/attenuation processes within the pavement.

Following this thought, for any given set of event parameters a very similar set of smoothing and reduction factors would be applied by the pavement. The most important parameters would have been the rainfall duration and depth. These parameters were therefore combined to produce a set of simple regression models which demonstrate some of the uniformity of the pavement characteristics.

Data selection

The procedure adopted for model generation and analysis was to divide the data set into two sub-sets, one for calculation of the model parameters and a second for testing, (also known as a split record test). The division of such a data set can be problematical in that it is a time-series data record with a number of variables and it may be impossible to find a division of the data set which allows for a representative set of parameters for each variable. After due consideration, and various trials, the full data record for the analysed events was simply divided into two, the 'odds' and 'evens': the odd row numbers were grouped into a sub-set called odd, and the even row numbers were grouped into a sub-set called even, (the row numbers are printed along-side the event numbers in Table 7.1, part i). Table 7.3, part i, lists the basic data set used in the models.

The main difference between the two sub-sets is that the 'odds' had a higher average rainfall than the 'evens'. In retrospect this was found to be a useful addition to the modelling exercise, i.e. to compare the predictive 'qualities' of the models for slightly different/overlapping populations. For the following discussions all models were generated from the even data and the odd data set was the test data set. Also, the predictive qualities of the models were similar for all sub-base stone types, therefore to keep the presentation simple (and consistent with section 7.1) only the models for the b.f.s. sub-base are reported.

Regression analysis procedures. All of the regression models presented in the following sections were systematically checked for their statistical validity. These checks and the regression assumptions are listed below, any exceptions or variations to these procedures are discussed in the relevant sections.

Specification error: no wilful exclusions of relevant independent variables has taken place and it is worthwhile to note at this point, that no correlations were found to exist between the temperature measurements, (air and sub-base), and any of the variables examined. (Except for expected seasonal variations in rainfall patterns). This point is made to support the observations made with respect to evaporation prediction in Section 7.1.2, i.e. that the evaporative processes required much more complex data before accurate predictions could be made. All equations are presented as linear additive models. All standard transformations, (multiplicative variables, polynomials, log and reciprocal), failed to improve the statistical relevance of the models (except in the minor case stated below). It is extremely unlikely that all the relationships are linear: however, it is thought that the linear models adequately represent the overall effects of the assumed non-linear components of the system. In this respect the models are somewhat stochastic in nature, yet examination of the magnitude of the residuals confirms the robust nature of the data set and the ability of simple analysis to provide strong models.

Homoscedasticity: variance in predictive errors were found to be more or less constant across the values of the predicted variables.

Autocorrelation: the use of the antecedent variable acknowledged that autocorrelation may be present but in most cases the time between events is such that autocorrelation is not a factor.

Error normality: for all cases presented the error term can be considered to be normally distributed, indeed, for the sample size used, the central limit theorem indicates normality is approached irrespective of the nature of the distribution in the population.

Statistical significance: all of the parameters in all the presented equations were shown to be statistically significant at the 95% confidence limit, (except where stated), by the 't-ratio rule of thumb', (Lewis-Beck, 1980), which requires that the slope estimate, (or coefficient), divided by the standard error, (or standard deviation of the slope estimate), does not have a value of less than 2.0.

Multicollinearity: for multiple regression to produce the textbook best linear unbiased estimates there must be an absence of perfect multicollinearity (PMC). PMC makes it impossible to arrive at a unique solution for the least squares parameter estimate - and is therefore readily detectable. High Multicollinearity (HMC) can also be a problem as independent variables are virtually always inter-correlated, ie, multi-collinear. Significant indicators of HMC are poor statistical significance and substantial R^2 but statistically insignificant coefficients. Minor indicators of HMC are 'odd' coefficients such as blatantly incorrect magnitudes or the 'wrong' sign. A rule of thumb is not to consider HMC unless the independent variables have a correlation of 0.8 or higher. However, none of the above are conclusive with respect to the presence of HMC, the preferred method for assessing HMC is to regress each independent variable on all the other independent variables. Any resulting R^2 near 1.0 shows HMC. These inter-regressions were performed for all the models presented in this chapter and no R^2 between the independent variables exceeded 0.52 in magnitude, it was therefore concluded that high multicollinearity is not a problem for the models presented.

Residual analysis: visual inspection of the residuals versus the predicted values for each variable in each equation was completed using scatter plots. The presence of outliers, curvilinearity indicating mis-specification, homoskedasticity and specification error were thus evaluated: none was detected.

7.2.1 Runoff duration

The 'best fit' and most significant regressions derived for the prediction of runoff duration, for the evens sub-set, were from an additive linear regression upon the event rainfall depth and duration, and the depth of the antecedent event. The antecedent variable was found to be significant in extending the duration of runoff. The effects of the antecedent event were discussed at length in Section 7.1.4 and a combination of parameters was used to derive the antecedent effect term, ANTEFF. However, for this particular regression the 'ANTMM' variable was found to be marginally more significant than the 'ANTEFF' variable.

The model generated, by the 'even' data sub-set, to predict the duration of runoff from the b.f.s. sub-base was:

BFSDUR = -77.1 + 23.01*Rf + 0.401*DURATION + 11.4*ANTMM
(all variables and units are given in Table 7.1, part i.)

The correlation coefficient for this equation was 0.89, therefore R^2 was 0.79, i.e. 79% of the variation in the independent variable was accounted for by the regression variables. A comparison of the measured and predicted values for the evens sub-set is presented in the first two columns of Table 7.3, part ii.

<u>Results and residuals</u>. The results from the test data set are presented in Table 7.3, part iii, the first two columns of which list the measured and predicted runoff durations for the b.f.s. sub-base, (for the 'odd' or test data sub-set). Table 7.4 presents the 'correlations' and 'multiple regression' report obtained during the data analysis. Of the 28 recorded events, 12 have predicted durations within 30 minutes, (event number 7 was within 3.3 minutes). The largest error or residual, of over 295 minutes, was for event number 35. The reason for this gross error can be traced back to the storm and its relationship to the runoff calculations: the event lasted nearly 13

hours and had a rainfall depth of only 8.4mm. The resulting runoff only peaked at 0.4mm/hr and remained above the runoff threshold of 0.25mm/hr for only 3 hours. This configuration of event parameters and runoff calculation procedures or systematics has therefore combined to 'hide' or distort the measured parameters. Therefore the following models should not be applied to very long storms.

<u>Specification error</u>. During the analysis of the model and its residuals it became apparent that there existed a specification error, i.e. an examination of the residuals (of BFSDUR) against the predicted values showed a moderate positive correlation. This means that those events with the longest duration produced the largest residual error. One interpretation of this is a 'carry-over' from the inherent stochastic nature of the event parameters: the longer the event duration, the greater the variability in the parameters. However, as part of the specification checks, regressions using the squares of all three predictor variables were carried out. This only produced a marginally better model (higher R^2) and reduce some of the residual correlation. (These observations are reported for completeness).

7.2.2 Mean runoff intensity

The best fit and most significant regression for predicting the mean runoff intensity, (for the b.f.s. sub-base and using the evens data sub-set), was derived using: rainfall depth and the peak rainfall intensity, (models using depth together with average intensity would have been subject to problems of high multi-colinearity as the prediction variables are not independent).

The 'intercept' for this regression was found not to be significant using the t-ratio test. This may be accounted for by an examination of the physical systems: the runoff intensity will be zero for a rainfall depth (and peak intensity) of zero, and a linear progression from this will occur.

N.B. care must be taken before disregarding the intercept within regression analysis as this may distort the statistical coefficients. The resulting model is:

BFSAINT = 0.0486*Rf + 0.0405*RFPINT
(all variables and units are given in Table 7.1, part i.)

The R^2 coefficient for this equation was 0.84, i.e. nearly 92% of the variation in runoff intensity was accounted for within the specified variables.

Results and residuals. Table 7.3, (part ii, columns 3 and 4), lists the measured and predicted values for the model data set. Part iii of the same table presents the same data for the test data set. Table 7.5 presents the 'correlations' and 'multiple regression' report obtained during the data analysis. Inspection of the data from these tables confirms the general accuracy of the predictions. The largest residual, or error, results from event number 47, EV8186, which has been commented on above as being exceptional within the data set gathered. Other large residuals occur for event numbers 10, (residual of 0.76mm/h), and 49 (residual of -0.61mm/h). Event 59 also had a high residual as the 5.2mm rainfall failed to generate runoff from the b.f.s bay.

The regressions for the mean intensity are much 'stronger' than for the runoff duration discussed in the last Section. This is probably due to the stochastic nature of the rainfall, but also because the intensity measurements were not 'clouded' by the definitions of rainfall and runoff chosen for this work.

7.2.3 Runoff depth

The generation of the runoff duration and mean runoff intensity models was aimed at producing the simplest example of a model to predict the volume of runoff. As discussed previously, the runoff volume is the main design variable for on-site reduction and attenuation devices. A further test of the intensity and duration models, reported above, can therefore follow from a multiplication of the two, (and division by 60 to bring the units to mm).

Table 7.3, part ii, lists: the measured runoff depths; the predicted values; and the resulting residuals, generated by the regression data set, ('evens'). Table 7.3, part iii, lists the same values for the test data set. The summary statistics show a mean error of the residuals as -0.29, ie on average, the model over predicted the runoff for an individual event by nearly 0.3mm. The RMSE, (root mean square error), for the predicted values of runoff of 1.4: the correlation between the measured and predicted values is 0.93.

The largest negative residuals were for event numbers: EV7225 (row 3, residual -2.04); EV8072 (row 35, residual of -3.68); EV8187 (row 49, residual -4.43); and EV8268 (row 61, residual -2.36). For all four of these fairly large events the model over-predicted the volume of runoff, this may in part be due to the derivation of the model(s) from the 'evens' data sub-set which had a lower average rainfall than the 'odds' test data set. All four of these events also had a 'significant' antecedent event within the 24 hours preceding the observation event. One would suspect that this should lead to positive residuals (model under-prediction of runoff). However, given the size of the events and the wet antecedent conditions, each event did produce rather low runoff (55.6%, 11.3%, 24.6% and 43.3% respectively), the conceptual model also produced negative residuals for these events.

With regard to under-prediction of runoff, we again find that the model produced a large positive residual for event number 47*, (which had the highest percentage runoff of all the data). Interestingly, two other events exhibiting a large volume of runoff, numbers 55 and 57 (17.0 and 22.6mm rainfall respectively), were both predicted fairly accurately.

In summary, this combination of fairly simple regression models has resulted in a model which allows for a robust prediction the major runoff characteristics. This is interpreted as a consistency in the function of the attenuation and reduction systems within the pavement which result in 'predictable' alteration, or smoothing to the rainfall hyetograph. A comparison of the conceptual and regression models is made in the Chapter summary.

* Footnote: As part of this analysis the 'odd' data sub-set was used in the same way as the 'evens', to produce prediction models. The 'odd' sub-set had a higher mean rainfall than the 'evens', and the combination of models to predict runoff from the b.f.s. predicted the runoff from the 'notorious' event 47 to within 0.1mm. The other statistical parameters were similar to those reported for model determination using the 'evens' sub-set.

7.2.4 Peak runoff intensity

Although the production of a model to predict the peak runoff intensity was not a primary aim, or requirement, of this research, (especially as hydraulic throttles are so effective for these structures), it was thought useful to test the versatility of the regression analysis on the data set for such a purpose. The following notes outline a brief examination of the important variables, and their accuracy, of a regression model to predict the peak runoff intensity, or peak flow, from an event. (The data set used is the complete one of 62 events and the prediction reported is for the b.f.s. sub-base, again).

Whilst considering the physical controls upon the peak flow, it seemed obvious that the rainfall depth, duration and peak intensity of the event would impact upon the resulting peak runoff rate: for a given rainfall, a high peak and a short duration indicate a short sharp event resulting in a high peak runoff, whilst a high peak and a long duration, would indicate more moderate runoff. Similarly, a greater rainfall depth for a given duration and peak intensity is likely to result in a greater peak runoff. These simple observations proved correct during the analysis and the most significant variables for predicting the peak runoff were the rainfall depth, peak intensity and duration. The model produced from a regression of these variables is:

BFSPINT = 0.207*Rf + 0.13*RFPINT - 0.0032*DURATION

(all variables and units are given in Table 7.1, part i.)

Table 7.6 details the variables and their respective predicted values for the full data set for which the mean of the peak runoff is 1.78 mm/hr and the mean of the predicted peak runoff is 1.97mm/hr: the correlation coefficient between the two sets of values is 0.87. However the mean of the residuals is 0.8, which can be considered to be rather large. Table 7.7 presents the 'correlations' and 'multiple regression' reports obtained during the data analysis.

Table 7.3, part i. **Variables and their values used for regression models. (All columns are described in Table 7.1, part i)**

EVENT		RF	DURATION	ANTMM	ARFINT	BFSDUR	BFSAINT	BFSMMRO
1	7207.0	9.2	342.0	0.0	1.6	*	*	*
2	7221.0	3.9	147.0	0.0	1.6	0.0	0.0	0.0
3	7225.0	21.5	266.0	2.0	4.8	389.0	1.8	12.0
4	7230.0	10.0	219.0	0.0	2.7	304.0	0.8	3.9
5	7235.0	10.1	145.0	3.3	4.2	263.0	1.2	5.2
6	7238.0	8.5	189.0	0.0	2.7	224.0	0.9	3.5
7	7249.0	12.4	438.0	1.0	1.7	393.0	0.9	6.1
8	7262.0	7.6	327.0	0.0	1.4	261.0	0.6	2.7
9	7277.0	11.0	309.0	0.0	2.1	206.0	0.9	3.2
10	7280.0	3.3	51.0	2.0	3.9	84.0	1.1	1.6
11	7282.0	13.3	311.0	1.0	2.6	360.0	1.1	6.5
12	7283.0	16.5	949.0	14.3	1.0	808.0	0.7	10.0
13	7287.0	13.1	461.0	0.0	1.7	400.0	1.0	6.3
14	7293.0	6.4	330.0	0.0	1.2	268.0	0.5	2.1
15	7300.0	12.5	236.0	3.0	3.2	241.0	1.5	6.2
16	7304.0	7.5	497.0	0.0	0.9	291.0	0.5	2.5
17	7312.0	2.8	125.0	0.0	1.3	0.0	0.0	0.0
18	7314.0	3.0	103.0	0.0	1.7	30.0	0.3	0.1
19	7315.0	8.6	343.0	5.0	1.5	332.0	0.8	4.4
20	7319.0	5.4	210.0	0.0	1.5	0.0	0.0	0.0
21	7323.0	9.1	188.0	2.0	2.9	250.0	0.7	3.0
22	7328.0	3.8	188.0	6.5	1.2	208.0	0.4	1.5
23	7349.0	9.0	592.0	0.0	0.9	184.0	0.5	1.6
24	7350.0	4.7	133.0	9.0	2.1	228.0	0.7	2.6
25	7350.5	3.1	131.0	14.0	1.4	190.0	0.3	0.9
26	8008.0	6.4	139.0	0.0	2.8	183.0	0.5	1.7
27	8022.0	8.2	293.0	2.0	1.7	152.0	0.4	1.0
28	8031.0	6.9	245.0	0.0	1.7	75.0	0.8	1.0
29	8035.0	10.3	216.0	0.0	2.9	231.0	1.0	3.9
30	8035.5	6.1	44.0	11.9	8.3	157.0	1.0	2.7
31	8038.0	5.3	195.0	0.0	1.6	25.0	0.5	0.2
32	8045.0	4.4	224.0	2.3	1.2	181.0	0.5	1.6
33	8063.0	3.0	40.0	0.0	4.5	0.0	0.0	0.0
34	8069.0	5.6	247.0	0.0	1.4	0.0	0.0	0.0
35	8072.0	8.4	795.0	4.0	0.6	186.0	0.3	0.9
36	8073.0	8.3	390.0	6.0	1.3	521.0	0.4	3.8
37	8078.0	2.7	178.0	0.0	0.9	0.0	0.0	0.0
38	8078.5	4.0	263.0	2.7	0.9	183.0	0.4	1.2
39	8086.0	6.7	301.0	0.0	1.3	251.0	0.3	1.1
40	8094.0	17.0	490.0	0.0	2.1	575.0	0.7	7.1
41	8107.0	6.6	182.0	2.0	2.2	197.0	0.5	1.7
42	8125.0	4.7	137.0	3.0	2.1	153.0	1.2	3.0
43	8152.0	8.2	241.0	3.0	2.0	*	*	*
44	8155.0	6.9	25.0	0.0	16.6	*	*	*
45	8160.0	8.4	265.0	0.0	1.9	*	*	*
46	8178.0	10.9	259.0	1.0	2.5	*	*	*
47	8186.0	10.5	49.0	7.0	12.9	255.0	2.5	10.6
48	8186.5	2.9	122.0	4.0	1.4	152.0	0.4	1.1
49	8187.0	9.4	124.0	10.0	4.5	189.0	0.7	2.3
50	8188.0	4.7	94.0	10.0	3.0	243.0	0.4	1.5
51	8188.5	6.6	116.0	5.0	3.4	408.0	0.5	3.5
52	8189.0	6.9	104.0	7.0	4.0	201.0	1.1	3.8
53	8195.0	6.6	117.0	3.0	3.4	197.0	0.7	2.4
54	8195.5	5.7	30.0	7.0	11.4	201.0	0.8	2.7
55	8198.0	17.0	519.0	0.0	2.0	488.0	1.1	9.2
56	8203.0	4.4	133.0	0.0	2.0	*	*	*
57	8203.5	22.6	446.0	5.0	3.0	584.0	1.7	16.6
58	8204.0	10.7	296.0	14.0	2.2	301.0	1.0	5.2
59	8212.0	5.2	78.0	0.0	4.0	0.0	0.0	0.0
60	8231.0	4.6	134.0	0.0	2.1	0.0	0.0	0.0
61	8268.0	8.8	178.0	2.0	3.0	190.0	1.2	3.8
62	8271.0	9.2	222.0	2.0	2.5	203.0	1.3	4.4

Table 7.3 part ii. Model data. Measured and predicted values from 'even' events.

Where:

BFSDUR = Duration, in minutes of runoff from the bfs sub-base.
BFSAINT = Average runoff intensity, in mm/hr.
BFSMMRO = Runoff, in millimetres from the bfs sub-base.
P..... = Predicted values for the above, same units.
RO.RESIDS = Residuals obtained by subtracting predicted bfs runoff from measured bfs runoff.

Event	BFSDUR	PBFSDUR	BFSAINT	PBFSAINT	BFSMMRO	PREDBFRO	RO.RESIDUALS
2	0.0	71.5	0.0	0.34	0.0	0.40	-0.40
4	304.0	240.8	0.78	1.3	3.94	5.23	-1.29
6	224.0	194.2	0.94	0.64	3.53	2.06	1.47
8	261.0	228.9	0.62	0.6	2.71	2.31	0.41
10	84.0	42.0	1.14	0.38	1.6	0.27	1.33
12	808.0	846.1	0.75	0.92	10.04	13.02	-2.99
14	268.0	202.5	0.46	0.39	2.07	1.32	0.74
16	291.0	294.8	0.51	0.5	2.49	2.45	0.04
18	30.0	33.2	0.28	0.27	0.14	0.15	0.01
20	0.0	131.3	0.0	0.4	0.0	0.88	-0.88
22	208.0	158.6	0.42	0.28	1.45	0.75	0.70
24	228.0	187.0	0.67	0.39	2.56	1.2	1.33
26	183.0	125.9	0.55	0.48	1.68	1.01	0.67
28	75.0	179.9	0.79	0.51	0.99	1.52	-0.53
30	157.0	216.5	1.04	0.8	2.73	2.8	-0.15
32	181.0	140.2	0.52	0.32	1.58	0.76	0.82
34	0.0	150.8	0.0	0.48	0.0	1.20	-1.20
36	521.0	338.7	0.44	0.53	3.8	3.00	0.80
38	183.0	151.2	0.4	0.26	1.23	0.64	0.59
40	575.0	510.5	0.74	1.23	7.07	10.44	-3.37
42	153.0	120.1	1.19	0.71	3.02	1.41	1.61
44	*	91.6	*	1.19	*	1.82	*
46	*	288.9	*	1.06	*	5.09	*
48	152.0	84.1	0.42	0.34	1.07	0.48	0.59
50	243.0	182.7	0.38	0.58	1.53	1.77	-0.24
52	201.0	203.1	1.12	0.94	3.77	3.19	0.57
54	201.0	145.9	0.79	1.07	2.66	1.59	0.07
56	*	77.4	*	0.34	*	0.44	*
58	301.0	447.4	1.04	1.2	5.21	8.92	-3.71
60	0.0	82.4	0.0	0.37	0.0	0.5	-0.5
62	203.0	247.3	1.31	0.79	4.44	3.27	1.16

Table 7.3, part iii. **Test Data. Measured values for 'odd' events, predictions derived from 'even' data set.**

Where:
- BFSDUR = Duration, in minutes of runoff from the bfs sub-base.
- BFSAINT = Average runoff intensity, in mm/hr.
- BFSMMRO = Runoff, in millimetres from the bfs sub-base.
- P..... = Predicted values for the above, same units.
- RO.RESIDS = Residuals obtained by subtracting predicted bfs runoff from measured bfs runoff.

	BFSDUR	PBFSDUR	BFSAINT	PBFSAINT	BFSMMRO	PBFSMMRO	RESIDUALS
1	*	271.7	*	0.58	*	2.63	*
3	389.0	547.1	1.84	1.53	11.95	14.0	-2.04
5	263.0	251.1	1.19	1.0	5.23	4.19	1.04
7	393.0	395.3	0.93	0.85	6.06	5.57	0.49
9	206.0	299.9	0.92	0.94	3.17	4.7	-1.53
11	360.0	365.0	1.08	1.23	6.49	7.46	-0.97
13	400.0	409.2	0.95	0.82	6.34	5.58	0.76
15	241.0	339.4	1.55	1.08	6.22	6.09	0.13
17	0.0	36.3	0.0	0.21	0.0	0.13	-0.13
19	332.0	315.3	0.8	0.72	4.41	3.79	0.62
21	250.0	230.5	0.72	0.6	2.98	2.32	-0.66
23	184.0	367.4	0.53	0.56	1.62	3.45	-1.83
25	190.0	206.4	0.27	0.3	0.86	1.02	-0.16
27	152.0	251.9	0.41	0.59	1.05	2.49	-1.44
29	231.0	246.5	1.01	0.88	3.87	3.6	0.27
31	25.0	124.2	0.49	0.47	0.2	0.98	-0.78
33	0.0	8.0	0.0	0.33	0.0	0.04	0.04
35	186.0	480.6	0.31	0.58	0.95	4.63	-3.68
37	0.0	56.4	0.0	0.19	0.0	0.18	-0.18
39	251.0	197.8	0.25	0.57	1.05	1.89	-0.84
41	197.0	170.5	0.51	0.49	1.68	1.4	0.29
43	*	242.4	*	0.79	*	3.19	*
45	*	222.4	*	0.66	*	2.43	*
47	255.0	264.0	2.48	1.85	10.55	8.14	2.41
49	189.0	302.9	0.73	1.34	2.31	6.74	-4.43
51	408.0	178.3	0.52	0.83	3.5	2.46	1.05
53	197.0	155.9	0.74	0.58	2.44	1.5	0.94
55	488.0	522.2	1.13	1.0	9.16	8.71	0.46
57	584.0	678.8	1.7	1.63	16.57	18.47	-1.91
59	0.0	73.8	0	0.71	0.0	0.87	-0.87
61	190.0	219.6	1.2	1.69	3.81	6.17	-2.36

Table 7.4 **Correlation and Multiple Regression reports for the BFS runoff duration model, (derived from the 'evens' data set).**

BFSDUR = BFS DURation of runoff
RF = RainFall
DURATION = rainfall DURATION
ANTMM = ANTecedent rainfall in MilliMetres

Correlations

Filter: C49 =1

	RF	DURATION	ANTMM	BFSDUR
RF	1.0000	0.7642	0.2346	0.8179
DURATION	0.7642	1.0000	0.1794	0.7932
ANTMM	0.2346	0.1794	1.0000	0.4647
BFSDUR	0.8179	0.7932	0.4647	1.0000

Multiple Regression Report

Filter: C49 =1
Dependent Variable: BFSDUR

Independent Variable	Parameter Estimate	Stndized Estimate	Standard Error	t-value (b=0)	Prob. Level	Seq. R-Sqr	Simpl R-Sqr
Intercept	-77.14253	0.0000	34.22797	-2.25	0.0332		
RF	23.00941	0.4414	7.023006	3.28	0.0031	0.6690	0.669
DURATION	.4010319	0.4041	.1321307	3.04	0.0055	0.7369	0.629
ANTMM	11.40193	0.2887	3.488404	3.27	0.0031	0.8157	0.216

Analysis of Variance Report

Filter: C49 =1
Dependent Variable: BFSDUR

Source	df	Sums of Squares (Sequential)	Mean Square	F-Ratio	Prob. Level
Constant	1	1255904	1255904		
Model	3	748516.8	249505.6	36.88	0.000
Error	25	169118	6764.719		
Total	28	917634.7	32772.67		

Root Mean Square Error	82.24791
Mean of Dependent Variable	208.1035
Coefficient of Variation	.3952261
R Squared	0.8157
Adjusted R Squared	0.7936

Table 7.5 **Correlation and Multiple Regression reports for the BFS average runoff intensity model, (derived from the evens data set).**

BFSAINT = BFS Average INTensity
RF = RainFall
RFPINT = RainFall Peak INTensity

Correlations

Filter: C49 =1

	RF	RFPINT	BFSAINT
RF	1.0000	0.2216	0.2531
RFPINT	0.2216	1.0000	0.5481
BFSAINT	0.2531	0.5481	1.0000

Multiple Regression Report

Filter: C49 =1
Dependent Variable: BFSAINT

Independent Variable	Parameter Estimate	Stndized Estimate	Standard Error	t-value (b=0)	Prob. Level	Seq. R-Sqr	Simpl R-Sqr
Intercept	0						
RF	.0486328	0.4952	.1288E-01	3.78	0.0010	0.7540	0.754
RFPINT	.0405483	0.4826	.1102E-01	3.68	0.0013	0.8477	0.749

Analysis of Variance Report

Filter: C49 =1
Dependent Variable: BFSAINT

Source	df	Sums of Squares (Sequential)	Mean Square	F-Ratio	Prob. Level
Constant	0	0	0		
Model	2	12.32137	6.160684	61.23	0.000
Error	22	2.213392	.1006087		
Total	24	14.53476	.605615		

Root Mean Square Error	.3171887
Mean of Dependent Variable	.7217752
Coefficient of Variation	.4394564
R Squared	0.8477
Adjusted R Squared	0.8408

Table 7.6. **Variables and regression predictions for peak runoff intensity.**

Where: RFPINT = Rainfall Peak intensity
DURATION = Rainfall Duration
BFSPINT = Peak runoff intensity for the b.f.s sub-base.
PBFSPINT = Predicted BFSPINT from regression model, & PINTRESIDS = Residuals: BFSPINT - PBFSPINT.

RAINFALL	DURATION	RFPINT	BFSPINT	PBFSPINT	PINTRESIDS
1 9.20	342	3.30	*	1.24	*
2 3.90	147	3.70	0.00	0.82	-0.82
3 21.50	266	12.10	5.00	5.17	-0.17
4 10.00	219	20.20	1.50	4.00	-2.50
5 10.10	145	12.60	3.20	3.26	-0.06
6 8.50	189	5.50	1.90	1.87	0.03
7 12.40	438	6.00	2.00	1.95	-0.05
8 7.60	327	5.80	1.30	1.29	0.01
9 11.00	309	10.00	1.60	2.59	-0.99
10 3.30	51	5.50	0.40	1.23	-0.07
11 13.30	311	14.30	3.00	3.62	-0.62
12 16.50	949	3.00	1.70	0.77	0.92
13 13.10	461	4.50	2.40	1.83	0.57
14 6.40	330	2.00	0.63	0.53	0.31
15 12.50	236	11.60	3.40	3.34	0.06
16 7.50	497	3.30	0.84	0.40	0.44
17 2.75	125	2.00	0.00	0.43	-0.43
18 3.00	103	3.00	0.31	0.68	-0.37
19 8.60	343	7.50	2.40	1.67	0.73
20 5.40	210	3.50	0.21	0.90	-0.69
21 9.10	188	4.00	0.59	1.80	-1.21
22 3.75	188	2.50	0.59	0.50	0.09
23 9.00	592	3.10	0.73	0.38	-0.35
24 4.70	133	4.10	1.15	1.08	0.07
25 3.10	131	3.60	0.52	0.69	-0.17
26 6.40	139	4.20	1.10	1.43	-0.33
27 8.20	293	4.80	0.42	1.38	-0.96
28 6.90	245	4.20	1.00	1.19	-0.19
29 10.30	216	9.28	2.10	2.65	-0.55
30 6.10	44	12.37	3.00	2.73	0.26
31 5.35	195	5.30	0.25	1.17	-0.92
32 4.40	224	2.70	0.78	0.54	0.23
33 3.00	40	4.60	0.04	1.09	-1.05
34 5.60	247	5.10	0.07	1.03	-0.96
35 8.40	795	4.20	0.26	0.00	0.26
36 8.30	390	3.16	0.73	0.88	-0.15
37 2.70	178	1.50	0.09	0.19	0.10
38 4.00	263	1.50	0.46	0.18	0.28
39 6.70	301	6.10	0.36	1.22	-0.86
40 17.00	490	9.90	2.00	3.24	-1.24
41 6.60	182	4.20	0.73	1.33	-0.60
42 4.70	137	11.80	3.20	2.07	1.13
43 8.20	241	9.65	*	2.18	*
44 6.90	25	21.06	*	4.09	*
45 8.40	265	6.10	*	1.68	*
46 10.90	259	13.00	*	3.12	*
47 10.50	49	33.10	13.20	6.32	6.87
48 2.90	122	4.90	0.50	0.85	-0.35
49 9.40	124	21.70	1.50	4.37	-2.87
50 4.70	94	8.70	0.94	1.81	-0.87
51 6.60	116	12.50	2.60	2.62	0.40
52 6.90	104	15.00	3.60	3.05	0.89
53 6.60	117	6.30	1.60	1.81	0.09
54 5.70	30	19.50	1.90	3.62	-3.44
55 17.00	519	4.30	3.70	2.42	0.79
56 4.40	133	3.20	*	0.90	*
57 22.60	446	13.20	6.80	4.97	1.45
58 10.70	296	16.70	3.40	3.44	0.32
59 5.20	78	11.20	0.00	22.28	-1.96
60 4.60	134	3.50	0.00	0.98	-0.37
61 8.80	178	31.10	3.96	5.30	-0.58
62 9.24	222	8.50	4.10	2.31	2.11

Table 7.7 Correlation and Multiple Regression reports for the BFS peak runoff intensity model.

BFSPINT = BFS Peak runoff INTensity
RF = RainFall
RFPINT = RainFall Peak INTensity
DURATION = rainfall DURATION

Correlations

	RF	RFPINT	DURATION	BFSPINT
RF	1.0000	0.3013	0.5632	0.5521
RFPINT	0.3013	1.0000	-0.2615	0.7129
DURATION	0.5632	-0.2615	1.0000	-0.0231
BFSPINT	0.5521	0.7129	-0.0231	1.0000

Multiple Regression Report

Dependent Variable: BFSPINT

Independent Variable	Parameter Estimate	Stndized Estimate	Standard Error	t-value (b=0)	Prob. Level	Seq. R-Sqr	Simpl R-Sqr
Intercept	0						
RF	.2067299	0.6899	.5902E-01	3.50	0.0009	0.5897	0.589
RFPINT	.1303115	0.5000	.2985E-01	4.37	0.0001	0.7469	0.712
DURATION	-.319E-02	-0.3522	.1343E-02	-2.37	0.0214	0.7712	0.262

Analysis of Variance Report

Dependent Variable: BFSPINT

Source	df	Sums of Squares (Sequential)	Mean Square	F-Ratio	Prob. Level
Constant	0	0	0		
Model	3	330.2583	110.0861	59.55	0.000
Error	53	97.97806	1.848643		
Total	56	428.2364	7.647079		

Root Mean Square Error	1.359648
Mean of Dependent Variable	1.781429
Coefficient of Variation	.7632346
R Squared	0.7712
Adjusted R Squared	0.7626

7.3 Summary and Recommendations

The various approaches at modelling the rainfall and runoff relationships for the permeable pavement produced fairly accurate and useful results. 'Useful', in terms of their descriptive qualities of the physical processes thought to be relevant to the system. The main limitations to the models is their derivation from the given/recorded rainfall data. Prediction beyond the range of observed values, for large design events for example, should only be undertaken with caution as to the accuracy of the results.

The conceptual models were certainly more accurate in their determination of runoff than the regression models. However, the regression models presented were chosen to demonstrate the strong inter-relationship of the measured variables, (rainfall depth, duration and intensity, with runoff depth, duration and intensity), and a direct regression to derive the runoff would have produced a model as accurate as the conceptual models. (Space has not permitted the inclusion of these models, which do not add significantly to the understanding gained from those presented here).

The engineering 'use' of these models remains to be determined. On their own they are of limited value to current engineering practice. However, the embodiment of the concepts within the 'on-site drainage' philosophy could be of significant use for future urban planning and management. For example: the observed and predicted rates of attenuation; the data indicating substantial losses to evaporation; together with the ease of reducing, or preventing, any flow from a paved surface reaching the sewer network, deserves consideration by both planners and engineers.

CHAPTER 8

DESIGN, OPERATION, AND MAINTENANCE OF INFILTRATION DEVICES

Previous Chapters have described and discussed the design, construction, and effectiveness of some methods of providing on-site stormwater retention and attenuation for a particular location. This Chapter aims to discuss the current situation for on-site stormwater disposal by comparing practice in the U.K. and other parts of the world, and by relating how observations from this research project may be used to improve aspects of the current U.K. design philosophy.

8.1 Current Practice for On-site Disposal of Stormwater

A brief outline of research on-site attenuation and reduction systems was given in Chapter 1, this Section goes further by describing current practice in the U.K. and in other countries.

8.1.1 U.K. practice

Traditionally, devices such as soakaways have been employed in areas where there was no mains drainage. The use of pit infiltration in 'remote' areas has had two effects: firstly, the practice has become labelled as a 'rural technology'; and secondly, their remote locations allowed for hydraulic failure without catastrophic effects, therefore, the devices could be allowed to fail before maintenance was undertaken.

The scepticism of many engineers as to the benefit of such small-scale, low-technology systems is undoubtedly one of the barriers to their wider use. It is believed that the 'science' or 'engineering' for on-site disposal and attenuation systems, although limited, is perfectly adequate for more general application. However, unless engineers are 'encouraged' to break away from the traditional practice of connecting urban drainage directly to downstream sewers and water courses, nothing will change. This 'encouragement' could come from several directions:

- Research - improved recommendations, (see Section 8.2.2) and examples of the successful use of on-site disposal. Also a higher profile in professional education would help;

- Financial incentives - the Water Companies are now empowered, under the 1989 Water Act, to charge substantial sums for the connection of supply and drainage facilities to each new property. These charges alone may justify the construction of on-site disposal systems;

- Statutory control - planning and transportation authorities need to be made aware of the possibilities for infiltration, etc, so that they can recommend or demand more imaginative solutions.

Survey of local government practice. As part of this research project, a questionnaire was devised and sent to 13 borough and district councils to elicit their views and hear of their practice with respect to stormwater reduction and attenuation structures. The 13 councils were selected to gain information from areas of the country covering a wide range of five selected parameters:

- Population density. Varied between 75 and 3,200 persons/km^2, covering rural and city areas.

- WRAP Class. The Winter Rain Acceptance Potential, (Farquharson 1978), is a 5 division classification of the hydrological properties of soil types. It was developed by the Soil Survey of England and Wales for use by the Institute of Hydrology. Although this classification is primarily based upon the 'runoff potential' of each soil, it would be related to several hydrological parameters, including infiltration potential. Table 8.1 gives a description of the 5 WRAP classes. For the research questionnaire, councils were chosen so that a variety of WRAP classes were represented.

- Soil moisture deficit is a simple scheme of estimation of soil dryness as defined by Hodgsen, (1974). Figures are in mm for the mean soil moisture deficit. The variation through the council areas chosen for the research questionnaire was from <4mm to 15mm, (U.K. range 2 to 18mm). 4. Average annual rainfall based on a map from the Flood Studies Report, (Anon 1975). Range for questionnaire was 550-1600mm per year, (U.K. range 500-2400mm per year).

- M5-2DAY rainfall. Again this data is from the Flood Studies Report and it gives figures, in mm, for 2 days of rainfall with a return period of 5 years. Range for questionnaire was 45-100mm, (U.K. range 45-150 mm).

Of the 13 questionnaires sent, eight were returned, (one of these was unusable). The results from the 7 remaining sets of answers can be summarised as follows:

Flooding: All the councils had a number of locations, (average 10), which suffered flooding on a yearly basis.

Use of flow reduction/attenuation devices: All councils 'allowed' their use. And 2 councils, (Taunton and W.Oxfordshire), 'demanded' their use on occasion, although this primarily concerned the use of hydrobrakes.

Maintenance: Only Reigate undertook maintenance of soakaways, primarily employing contract labour using suction equipment to remove accumulated sediment.

Design: For those councils that used infiltration methods all, bar one, recommended the use of BS8301 and BRE151, (both of these are discussed in Section 8.2). The one council that did not use these guides was Reigate which, arguably, had the most comprehensive use of infiltration techniques. (Reigate dimensioned soakaways to accept 40mm/h runoff from the impermeable surface they served).

Only Taunton & Deane District Council did not use infiltration methods, because 'soakaways are not a practical alternative...having regard to soil conditions'. Seventy five percent of the Taunton area is designated WRAP class 3 and twenty percent class 2 or better. It would have been useful to conduct a series of field trials in this location, (the results from infiltration tests carried out on various WRAP class soils are reported in Section 8.2).

In summary, the results from the questionnaire indicated a rather conservative approach to the implementation of infiltration or attenuation systems. In areas where the practice was more widespread, (e.g. Reigate and Bandstead), few problems were experienced. In comparison with the implementation in 'less crowded' countries, British practice is less than enthusiastic.

Table 8.1 **Description of the five classes of Winter Rain Acceptance Potential, (Farquharson et al, 1978).**

W.R.A.P. Class	General description
1	(i) Well drained permeable sandy or loamy soils and shallower analogues over highly permeable limestone, chalk, sandstone or related drifts (ii) Earthy peat soils drained by dikes and pumps (iii) Less permeable loamy over clayey soils on plateaux adjacent to very permeable soils in valleys
2	(i) Very permeable soils with shallow ground-water (ii) Permeable soils over rock or fragipan, commonly on slopes in western Britain associated with smaller areas of less permeable wet soils (iii) Moderately permeable soils, some with slowly permeable subsoils
3	(i) Relatively impermeable soils in boulder and sedimentary clays, and in alluvium, especially in eastern England (ii) Permeable soils with shallow ground-water in low lying areas (iii) Mixed areas of permeable and impermeable soils, in approximately equal proportions
4	Clayey, or loamy over clayey soils with an impermeable layer at shallow depth
5	Soils of the wet uplands (i) with peaty or humose surface horizons and impermeable layers at shallow depth, (ii) deep raw peat associated with gentle upland slopes or basin sites, (iii) bare rock cliffs and screes and (iv) shallow, permeable rocky soils on steep slopes

8.1.2 Infiltration practice worldwide

One of the most novel implementations of infiltration practice has been in Sweden, (Holmstrand, 1984), where the use of infiltration was adopted not for stormwater disposal, but to keep clay soils saturated. Previously, after urbanisation, recharge had been reduced causing drying and shrinkage of the clays, resulting in subsidence of the properties in the urban area that had been preventing recharge.

More conventional applications have taken place world-wide. Several other countries have active research programmes, which are likely to lead to more wide-scale adoption of infiltration/attenuation devices, e.g. France - Raimbault, (1990); Finland - Hogland et al, (1990); Sweden - Stenmark, (1990); and Australia - Somaratne and Argue, (1990). However, developments in North America and Japan have far outstripped the activities of other countries.

In the U.S.A. infiltration practice developed so that by 1984 the State Authority of Maryland was able to publish comprehensive design guides and specifications (Anon., 1984). (Section 8.2 comments more widely on the advantages and disadvantages of the Maryland practice).

In Canada, infiltration practice is not widespread but flooding problems in urban areas has led to the development of 'Master Drainage Plans', (MDP's), as part of integrated 'Storm Water Management', (SWM), (Wisner, 1988). However, large centrally controlled detention facilities are preferred to the dissipated, 'chicken pox', implementation of on-site retention/attenuation structures. This is probably a reflection of the outward growth of conurbations in Canada, where large areas can be designated for detention basins etc., whilst in the U.K. the urbanisation within a city or town often requires only small-scale solutions.

Japan has made the most progress at implementing infiltration techniques. In Tokyo, the extreme population density and urbanisation resulted in unacceptable combined sewer overflows during the 1970's. With apparently little research, a system termed E.S.S., (Experimental Sewer System), was implemented in April 1980, (Fujita, 1984). The system contained various infiltration devices and permeable pavements, all of which accepted urban runoff, but which overflow to the conventional drainage should the volume of runoff exceed infiltration capacity. There were few problems regarding the dimensioning of structures: the practice has been to build standardised devices wherever possible so that 'failure' of the infiltration equipment simply resulted in more flow to the combined system. By March 1990, the E.S.S. covered an area of 902 hectares in the suburbs of Tokyo, serving a population of over 100,000. This included 600,000m^2 of permeable pavement , (Fujita, 1990). Construction costs of E.S.S. were estimated at approximately 20% higher than that of a conventional system.

In essence, the Japanese situation of having severe flooding problems, plus a simple applicable technique of trying to infiltrate whatever volume was possible before discharge to a combined sewer, actively encouraged the adoption of infiltration practice. Added to this was the direction given to the implementation by the Tokyo Metropolitan Government. The result was the most comprehensive attenuation/reduction system, using infiltration, in the world.

There are, however, several points to the E.S. S. which would make it unsuitable for direct transplant to the U .K. Foremost amongst these is the labour-intensive 'maintenance practice': each of the 20,000 inlets to the system has a catch bucket AND a catch basket which have to be lifted from beneath a roadside grate and emptied of leaves and sediment. This can be compared with a typical comment obtained from one of the council engineer's reply to the research questionnaire: 'good idea...designs should be aimed at minimising maintenance'.

In contrast to the apparent paucity of technical research at the beginning of the installation of E.S.S. in Japan, there is now a plethora of research activity. This is mainly directed towards problems of silting up or clogging of infiltration capacity, and consideration of pollutant removal, (e.g. Minagawa 1990, Shinoda 1990).

8.2 Sizing Recommendations and Procedures

When attempting to design the 'correct' size of infiltration device there are numerous procedures which could be adopted by an engineer. Several of these procedures adopt different methods which would lead to several alternative recommended dimensions. This Section reviews some of these procedures and advances several general improvements that could be made to design specifications, in the light of the research reported here.

The design of permeable pavements, especially the sub-base thickness and wearing course thickness (for tarmac surfaces), is primarily related to the sub-grade strength and the anticipated loadings for the surface. These areas were outside the remit of this research, and so the main thrust of this Section relates to non-load bearing structures such as soakaways and infiltration trenches.

8.2.1 Current Recommendations

This Section is not a comprehensive review of the 'best' recommendations but an analysis of the main types of sizing procedures that have been used. The different methods are dealt with country by country, all design systems assume that local rainfall frequency duration data is readily available. Some of the methods were discussed in published papers, others (including all those for the U.K.) are extracted from formal design guides.

Sweden. Jonassan, (1984), observed that the choice of 'design rainfall event' for dimensioning infiltration systems depends on the soil hydraulic conductivity at the site of interest. For a soil of high hydraulic conductivity a short duration, high intensity rainfall events should be used, whilst for a soil of low hydraulic conductivity, a low intensity, long duration event should be used. For the former, the emphasis is to size the device to a maximum required infiltration capacity; for the latter, it is to size the device to hold the maximum required volume.

Although this approach is generally correct, the precept is only satisfactory if all storms are easily classified, but events can be of an almost infinite variety of duration and intensity. A more scientific approach would be to examine both extremes of rainfall intensity/duration, (for the required return period), to obtain limiting values for both infiltration rate and storage capacity. Then, on the basis of site investigation, choose the size of device that accommodates both limiting values.

Paus et al, (1974), described a dimensioning method related to the maximum daily precipitation rate. For the two year return period for Stockholm, this was 29mm in 24 hours. Paus et al acknowledge that the required storage volume would be reduced by the volume of water infiltrated during the storm but failed to note that:

- The infiltration rate would vary with the depth of stored water, seriously limiting any assumption of average infiltration rate; and,
- The average rainfall intensity for the design storm given, of only 1.2 mm/h, could be disposed of by almost all reasonably sized infiltration devices, almost regardless of soil type. In effect, the design storm seems wholly inadequate.

In 1977, Cederwall and Eriksson proposed a 'rainfall envelope' method for dimensioning infiltration systems. Statistically generated rainfall intensity curves were used to give a cumulative volume curve from which an assumed and straight line outflow, or infiltration rate, could be subtracted. The maximum volume difference between the two curves gave the required storage volume. Again, the assumption of uniform infiltration rate, unrelated to the depth to which a device was filled, was a possible error.

Development of a 'numerical difference' model by Ericsson, (1978), enabled more accurate dimensioning by processing inflow and outflow for a device in a series of time steps. However, to perform well, the model assumed different hydraulic conductivity of the soil with time: this variation in infiltration rate was likely to have been due to the variation in depth of water within the device rather than any change in the soil itself. Moreover, this model did not lend itself to more general use and application.

Germany. Sieker, (1984), related the standard techniques for on-site infiltration device sizing in Germany. The main method was similar to the 'numerical difference' model of Ericsson, (described above), but the following assumptions were also made:

- The bottom of a infiltration device became impervious after some years of operation;
- The infiltration rate was the coefficient of permeability in unsaturated conditions multiplied by a gradient dependent on the calculated depth of water for any given time interval;
- The 'effective' cross-section which accepted infiltration was taken to be the annular space around the outside of the construction, see Figure 8.1. The dimensions of this area were given as a radius of Z/2 around the circumference of the cross-section of the device itself, where Z was the depth of water in the device.

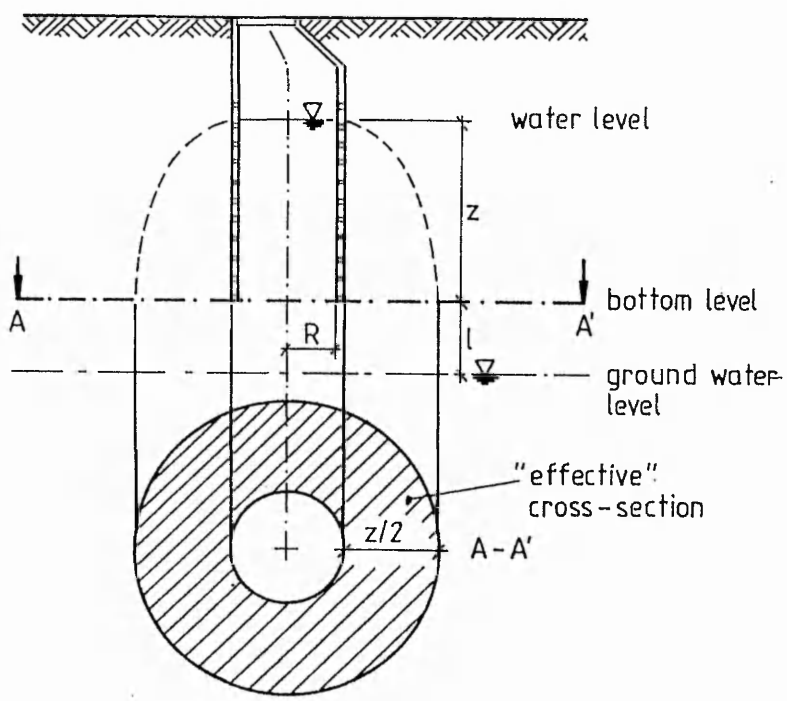


Figure 8.1 Infiltration device sizing in Germany. Effective area for infiltration taken as z/2, where Z is the depth of water in the device, (Sieker, 1984).

The first assumption has been commonly accepted, partly due to the settlement of fine particles which block effective infiltration, and also because of the anisotropic nature of permeability within soil: horizontal permeability has often been observed to be an order of magnitude greater than vertical permeability. The second assumption makes this one of the few recommendations which acknowledged that permeability may take place in unsaturated conditions, however, it is likely that infiltration would take place in both saturated and unsaturated conditions depending on circumstances, Reynolds et al, (1983), and may have even changed during a rainfall event or within the profile of the device. The last assumption may approximate to field observations, but the theory behind such an assumption was not explained and a method for calculating or testing the percolation rates was not given. Lastly, these 'time step' models are liable to a gross, cumulative error if one of the early calculations is incorrect. The design storm used in conjunction with this method was for a storm of 15 minutes duration and a return period of 5 years.

U.S.A. One of the first States to produce thorough standards and specifications for the construction of a range of infiltration devices was Maryland, (Anon, 1984). The primary method for sizing infiltration devices was made simple through the provision of tables, figures and general recommendations. For a site under investigation a simple soil textural analysis, (by sieving), enabled classification and sizing of the infiltration device, the relationship between soil textural class and its infiltration rate was synthesised from an analysis of over 5,000 samples examined by the United States Department of Agriculture, (U.S.D.A.), and discussed in Rawls et al (1982).

A maximum allowable storage time of 72 hours was stated, thus all devices are designed to fully empty within 3 days. The rest of the design procedures are concerned with calculating the exact runoff and dimensioning the device with graphs provided for the soils in each textural class.

The simplicity of these methods was a great advantage in obtaining consistency of results and the elimination of the laborious and costly processes of conducting field and laboratory infiltration and permeability tests. However, this approach was conservative, and its generalisations could result in some degree of over design, for example: minimum infiltration rates were used for all soil types; and any soil whose textural class was finer than a silt loam is counted as being unsuitable for use with infiltration techniques.

The 'trade-off' between simplicity of the use of the recommendations and the blanket dismissal of soils of low infiltration capacity is discussed further below.

U.K. practice.

British Standard B.S.8301, (Anon., 1986), suggested that a common method of designing soakaways was to provide water storage capacity equal to at least 12mm of rainfall over the impermeable area to be drained, but that on some sites tests of permeability through trial boreholes may be needed. To carry out such a test the British Standard then recommended the use of BRE 151, discussed below.

British Research Establishment Digest 151, (Anon., 1973), is one of the most commonly used guides in Britain for the design of soakaways. It clearly states that a successful soakaway is one with sufficient storage capacity to accept the sudden inflow of water and a sufficient rate of dissipation to deal with the average rate of flow. The basis of the BRE 151 recommendation was for a 2 hour event of 15 mm/h intensity, a total of 30mm of rainfall, (but the British Standard refers to a capacity of 12 mm).

The permeability test consisted of a 150mm diameter augured hole from which the time taken to drain 300mm depth of water, (equivalent to 5.5 litres), was measured. This may be repeated for a series of depths of hole if required. BRE 151 gave a graph from which the result of the test, in minutes, was plotted against the area to be drained, in m^2 , the size of soakaway required being taken from one of a series of curves on the graph.

However, the function plotted on this graph assumed an average wetted area and average infiltration rate over the duration of the borehole permeability test, but the decay in levels within the test bore with declining head rendered this assumption invalid. Also, the 'scaling' effects between a 150mm diameter hole and a soakaway of several metres diameter are not simple: the limitations of the test are discussed further below.

PSA 125, (Property Services Agency's design guide No.125, Anon, 1977), recommended the use of BRE 151 for use on areas to be drained of less than $400m^2$. For areas greater than this the PSA method involved the excavation of a rectangular pit which was filled with water and the water level allowed to fall. The time taken for the level to fall a measured distance was then used to calculate the average infiltration rate per unit wetted area.

The prototype soakaway size was found by calculating the wetted area required to disperse the runoff at 1.25 mm/h over the impervious area served. The guide also states that the measured infiltration rate should be multiplied by one third to provide a factor of safety: this would have the effect of tripling the surface area to be excavated during construction. Again, the limitations of such infiltration tests are discussed below.

8.2.2 Improved recommendations

In previous Chapters several observations and recommendations have been made which have a bearing on the design of on-site infiltration and stormwater attenuation devices. This Section considers some of these points in the light of the reviews of design procedures made in the previous Section.

On-site stormwater infiltration/attenuation devices can be built as part of a new development or be installed subsequently, if required, into an existing construction as part of a desire to solve storm drainage flooding problems, (occurring at the site of drainage, or downstream).

In all cases the main design parameters relate to the calculation of the volume of runoff, (for the design storm chosen), and the infiltration rates at the site. These two variables are dealt with first, followed by an examination of the effects of: antecedent conditions; hydraulic failure or flooding; and, awareness policy.

<u>Runoff coefficient</u>. For both the permeable pavement and the roof runoff infiltration devices the percentage runoff was consistently less than 100% for almost all events. However, as almost all design guides assume the runoff from the chosen design event to be 100%, this will lead to an over design in many cases. There are two schools of thought on this matter. The first would assert that parameters such as the runoff coefficient should be accurately estimated and that factors of safety should be built into the design at the end of the design process by increasing the required dimensions of the construction. The second that, in general, the cost of such minor constructions is so small that over-design in this manner will save on the problem of estimating exactly what the correct runoff coefficient is, and provides for an added factor of safety.

It is considered that correct estimate of runoff is a useful piece of information for the engineer and that he should be aware of the probable runoff volumes to be drained from an 'impermeable' surface, but the low cost of small on-site drainage devices should allow for over design. Furthermore, the runoff coefficient for any impermeable surface will be an average for the surface, and high intensity rainfall events that form the basis of many designs are likely to produce a much higher than average percentage runoff, (as shown by the results from the car park and roof runoff data).

Infiltration tests and theory. Several of the recommendations outlined above describe infiltration tests using falling head, borehole or pit methods. Such tests were conducted as part of the initial research into the suitability of the roof runoff site for infiltration devices. Two boreholes were conducted in accordance with BRE 151: the test requires the measurement of the time for the boreholes to become empty after the addition of a known volume of water. In both cases the boreholes failed to fully empty after several days of observations, in the initial stages the fall in level of the water was easily observed, but as the head declined the levels became static.

Another series of BRE 151 infiltration tests were conducted in soil conditions corresponding to various soil WRAP classes as defined by Farquarharson et al, (1978). The four WRAP classes tested ranged from 1 which is a quickly draining soil, to 4 which is poorly draining. The soils at Clifton, corresponding to Class 4, failed to produce useable results as described above. Similarly, the tests on the soils of WRAP Class 3 failed to produce results that could be used with the BRE 151 recommendation because the boreholes were constructed in a low-lying area and they failed to empty.

In stark contrast the results from the WRAP Class 1 site, in Nottingham city centre, failed to produce results because the water drained too quickly: after the addition of over 50 litres as quickly as possible, (approximately 10 times the theoretically required volume), there was no water left within the borehole. (Nottingham city centre's soils are underlain by well sorted, very permeable and poorly cemented sandstone).

A full series of infiltration tests was conducted on a WRAP Class 2 soil in the grounds of a school at Castle Donnington, Derbyshire. The series consisted of 5 sets of falling head tests repeated over the period of a year. The aim of these tests was to observe the differences in results obtained through the year. The results showed a trend in infiltration rates with the time of the year, (Figure 8.2), with the lowest rates, (approximately 4.0×10^{-6} m/s), observed at the end of the winter and the highest rates, (approximately 7.0×10^{-6} m/s),in late summer.

The differences in the results indicated that such tests were not wholly reliable, as tests conducted in summer would probably give rise to a higher infiltration rate, and subsequently, a recommendation for a smaller infiltration device than would a test conducted in the winter. These differences may be more pronounced in finer soils , (Elrick and Reynolds, 1986; Somaratne and Argue, 1990).

An examination of saturated-unsaturated flow theory, (Philips (1985); Reynolds et al (1983); and, Reynolds & Elrick (1985)), indicated that the rate of fall of the water surface, in a cylindrical cavity, depends on many factors besides the permeability (or hydraulic conductivity) of the soil. In fact, the percolation rate depends on the:

- Radius of the hole;
- Depth of water in the test hole;
- Field saturated hydraulic conductivity of the soil;
- Hydraulic conductivity pressure head relationship of the soil;
- Initial pore water pressure head in the soil;
- Depth of the water surface in the test hole below the soil surface; and,

- Depth of the water table or impermeable layer (if present) below the bottom of the test hole.

An equation containing three factors may be used to describe flow from a cylindrical auger hole, these factors contain most of the variables above and may be described as:

A. The 'hydraulic push' of the water into the soil due to the hydrostatic pressure head of the ponded water;

B. The 'gravitational pull' of the water out through the bottom of the whole (N.B. many recommendations discount flow through the bottom of an infiltration device, due to clogging with time, yet they fail to remove its effect from the initial test which is used to assess the infiltration rate);

C. The 'matrix pull' of the water out of the hole due to the capillary forces in the surrounding unsaturated soil.

The first two terms may be thought of as the 'field saturated' component of flow out of the test hole, and the third as the unsaturated flow component. This last factor gives an explanation of the variation in the test results from Castle Donnington. The drier summer months may have higher percolation rates because the contribution of capillarity to flow increased as the soils become drier. Mathematical solutions which attempt to model the above factors can give approximate solutions but, in general, the small-scale percolation tests give little more than an indication of the infiltration rates for a particular test and soil.

There are three obvious solutions to the problem of design based on infiltration rates. Briefly, these are:

1. A thorough experimental analysis of field results and mathematical models aimed specifically at producing a standardised test and conversion formulae which can be incorporated into a clear design brief for the practising engineer;

2. A movement towards the Maryland practice which devolves the specification to the regulatory authorities, the engineer only being required to provide a textural classification of the local soil type. The Maryland example excludes many soils, which it is believed may be suitable for infiltration practice, and probably includes a high degree of over-design. However, the introduction of any kind of clear specification for infiltration practices is extremely useful in promulgating the 'infiltration method' to where it is needed, the developer and practicing engineer.
3. Full scale tests. Given the low unit cost of simple infiltration constructions, it may be appropriate to conduct full-scale tests. Infiltration devices, such as those built for this research project, could be built without the appropriate drainage system from an impermeable surface in a matter of hours. The main problems with this kind of test would be the provision of large quantities of water to test the device. In remote areas the provision of a bowser may be deemed necessary, and the possibility of having to provide a method of shoring up the temporary excavation needs to be considered.

In summary, the assessment of infiltration rates is fraught with difficulty, the science and effort required to understand the relevant quantities and their variation for a particular site may make the use of on-site infiltration seem unsuitable. The Maryland specifications remove the onus of this assessment from the design engineer but precludes the use of many suitable locations. If enough area of a soil with an 'unsuitable' infiltration rate is exposed for use by infiltration devices then the practice may still be suitable.

In the light of the American practice, the way forward for British practice could come through 'user friendly' design criteria such as an extension of the WRAP class criteria or further work on the hydrological classification of British soils, e.g. Painter, (1971). If such recommendations proved to be too general for a specific location, or design requirement, then a second layer of further, more detailed recommendations could be used where more accurate data was required, (e.g. those sites on the borderline of the basic recommendation, or where the site was extremely sensitive to flood failure). These more detailed recommendations could then require a secondary study consisting of full scale-tests. Several recommendations differentiate between the size of impervious area to be drained, giving different design techniques for larger areas. A simpler method would be to have a standard design which is multiplied as necessary to cope with the area to be served making any testing procedure so much simpler and representative.

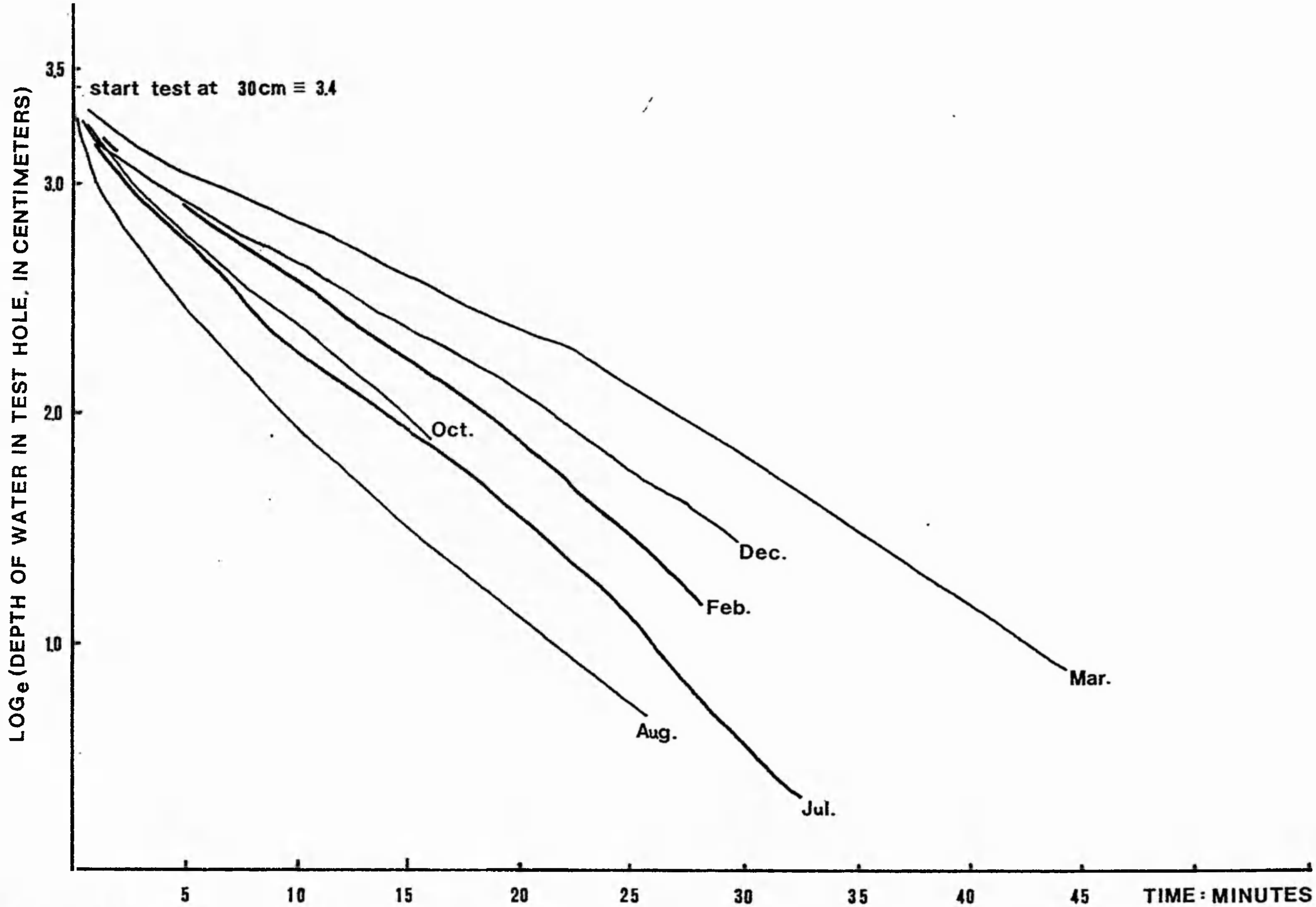


Figure 8.2 Variation in percolation tests over a one year period.

Finally, to resolve many of the problems associated with the calculation of the falling head percolation test described in BRE 151, the maximum infiltration rate should be assessed by performing a 'constant head' percolation test (as described for the infiltration trench in Section 4.2).

Antecedent conditions. This research has shown significant changes in runoff characteristics with varying antecedent rainfall conditions. Wet antecedent conditions have two effects on a stormwater infiltration/attenuation system. Firstly, the wet surface to be drained will produce a larger percentage runoff than if it was dry, and secondly storage within the device receiving the runoff is occupied. These phenomena are easily understood but they are difficult to accurately model for use within design recommendations.

The increased percentage runoff from events following wet periods is covered by assuming runoff as being 100% of rainfall over the area of the impermeable surface. The second effect, occupied storage, is usually dealt with by providing a 'time to empty' factor within design guides. This factor, usually arbitrary, is given as the time in which an infiltration device should empty once full.

For the Maryland specifications, (Section 8.2.1), this time is 72 hours. In Sweden, Paus et al (1973), reported a dimensioning factor of 4 days as the time to empty, this time was 'judged' from rainfall records as that time required to empty a device between two rains of dimensioning size. A draft design for the update of BRE 151, (Pratt, 1990), recommends that for 'satisfactory performance...the soakaway design should discharge from full to half volume in 24 hours', this proposal is an improvement on those that require devices to 'fully empty', as many devices would have low infiltration rates as the head of water is reduced within the device.

Apart from the general indication of a requirement to be able to cope with subsequent events, the 'time to empty' criterion could be seen as a contradiction of the required infiltration rates within many guides i.e. the specification for infiltration rate will often exceed the design guide 'time to empty' criterion. In summary, it is extremely difficult to provide statistically correct data which could be used to give 'time to empty' conditions within a design guide.

Design for failure. The experience from this research project has been that infiltration/attenuation devices may be subject to hydraulic failure. Flooding may occur for a number of reasons: rainfall in excess of the design storm; successive rainfall events exceeding the device capacity; or lack of maintenance leading to clogging and loss of infiltration capacity.

As part of the design process all infiltration/attenuation devices should be considered as susceptible to overflow and surface flooding. The usual practice, when dealing with stormwater drainage, is to plan for failure in relation to rainfall return periods, thus for residential areas this may be a 1:25 year return period. However, what is recommended here is not a review of the frequency of flooding but an analysis of what happens when the device floods: to where would the flood water travel?

This form of analysis may reveal areas which are totally unsuitable for such devices because of a disproportionate level of damage, or a potentially dangerous health hazard resulting from the flooding. In mitigation of the effects of flooding, the following should be borne in mind:

- This research has shown that infiltration devices may perform in excess of their design capacity when over-filled, thus the water level may rise above the inflow invert level, but the increased volume and infiltration rate may keep the water level below the level of flooding, i.e. below ground surface.

- In residential areas, providing the ground surface does not slope towards a residence, it may be perfectly acceptable to 'flood' the garden and suffer, temporarily, the resulting amenity loss. It is unlikely that a garden would be required for use during a wet period.
- Flood analysis may show how landscaping could provide flood ways or swales from an overflowing device to a secondary part of a flood system such as a road, water course or detention system, (Wisner, 1980).
- As a final resort, overflow could be channelled (either by pipe or overground) to the conventional storm drainage.

Much of the current reluctance to the use of soakaways, etc. is based on the notion of the eventual failure of the device. This reluctance should not be compounded by designing infiltration systems that produce unacceptable effects if, or when, they fail.

<u>Awareness policy</u>. Lastly, the design life of infiltration structures is often in excess of the time required for owners, whether public, private or corporate, to forget about their existence. Such a situation arises from: their effectiveness - they work and so attention is not drawn to them; their construction - usually sub-surface and unobtrusive; and their lack of access for monitoring and maintenance. Therefore, the existence of infiltration devices often only comes to light after they display some form of hydraulic failure, or they are revealed by later construction.

The specific details regarding maintenance are discussed in the following Section, but the location of infiltration devices should at least be marked with a monitoring or access tube if not a fully accessible 'maintenance chamber'. Such aids to location will have several benefits. They may: promote understanding of the storm drainage principles involved; allow some form of monitoring of performance; and enable the owner or engineer to plan for additional devices, should they be required due to extension of impermeable areas or loss of infiltration capacity.

8.3 Maintenance

8.3.1 Maintenance philosophy

Lack of maintenance of infiltration devices will often stem from an ignorance of the location and/or function of a retention or attenuation system. The problem of lack of awareness should be addressed at the design stage, with specific recommendations about the level and frequency of any maintenance being given to the owner at the time of commissioning or sale.

In future it may be possible for the presence of infiltration systems to become part of the legal documentation associated with a property. In a similar way to the current practice of the seller of a private dwelling certifying, or providing a contractors guarantee for, work such as damp-proof courses and wood treatment, it is likely that other developments in the energy saving field, (double glazing, solar heating etc) will form part of a solicitors inquiries or even be entered into the deeds of property. Correspondingly, disconnection from the storm drainage facilities provided by a local water company could also benefit from thorough documentation, to assist a claim for a reduction in levied water charges. As water charges are set to increase for some time, and water metering becomes more likely, the chance of reducing charges and having sub-surface storage for possible re-use of the water for gardens, etc. is attractive, especially as consumers are becoming concerned about environmental matters and generally 'green minded'.

8.3.2 Porous pavement maintenance

Hydraulic maintenance. The most commonly cited problem with any infiltration device is that of clogging of pores leading to a loss in infiltration capacity and to hydraulic failure. Problems with infiltration into soil around the device (applicable to the unsealed base of a porous pavement) are dealt with in the next Section but, in general, the geotextile within a porous pavement should confine 'clogging materials', such as sediment, leaves and litter to the upper layers. To prevent partial silting-up of the sub-grade, or to reduce the concentration of suspended solids in waters drained from the sub-base, it is recommended that only 'clean' or 'washed' stone is used in the sub-base.

In the surface of a permeable concrete block pavement there are two areas which may be susceptible to clogging. Firstly, the area around the top surface of the blocks, where the majority of water drains through the gravel-filled 'pores' between the blocks, may collect 'larger' particles which may then develop a crust of finer particles above. Secondly, most of the 'fine' particles draining through the 'pores' and into the bedding layer would collect on the geotextile reducing its permeability.

In the first case, hydraulic failure at an individual pore would cause water to flow between the raised discs of the concrete blocks to nearby pores, (if not down between the blocks themselves). In the latter case the gradual silting-up of the geotextile may eventually lead to it becoming fully clogged, depending on sediment size and loading, but the geotextile would have to become almost impermeable across its whole surface before the 'reservoir' of the bedding layer would overflow to the height of the surface.

In both cases it is worth noting that the hydraulic parameters, (i.e. the retention/attenuation capabilities), of the pavement would improve with time: the clogging or silting of the pores and geotextile helping to slow the passage of water through the upper layers. (The general effects on pollutant removal, i.e. lowering of both dissolved and suspended solids, also improves with time, (Schofield , 1991)).

Permeability tests were conducted on the surface of the car park at Clifton. The tests consisted of separating a 0.1m by 0.1m surface area, (covering a single pore between the blocks), by placing the corners of a hollow box on the centre of four adjacent raised discs on the blocks, and sealing the edges with putty. The tests were conducted two years after the construction of the car park. Only the results from the pavement above the b.f.s. sub-base were from the surface as originally constructed, the other portions of the surface having been removed as part of the 'improvements' described in Chapter 5. The results of the calculated infiltration rates in Table 8.2 are given in millimetres per hour for the 0.01m^2 area tested, although the actual pervious area of the pores between the blocks is only 25% of this area. The results show that the rate of infiltration was considerably in excess of any likely incident rainfall.

Table 8.2. Infiltration test results for 'pores' of the concrete block permeable pavement

Surface	Test Number:				all figures in mm/h	
	1	2	3	4	5	Ave.
A*	3200	12000	18000	7500	7200	9500
B*	22500	15000	16300	16300	-	17500
* 'A': Block paving laid 2.25 years before the test. 'B': Block paving re-laid 10 months before the test.						

During the course of the testing, some pores were found to be much slower at allowing percolation than others: however, repeated testing resulted in much higher infiltration rates for the same pore. The pouring of water onto the surface was observed to loosen the crust of particles on the surface and 'cleanse' the pore. This experience pointed towards a likely remedial technique should the surface of the blocks clog completely: high pressure hose treatment or mechanical disturbance of the clogged pores, (using, for example, a single pronged fork). Such action would dislodge accumulated material which would be carried down by water from the hose, or by subsequent rainfall, to collect on the surface of the geotextile. A third cleansing technique evolved from a sampling technique devised as part of the quality study, (Schofield 1991): a hand-held cordless vacuum cleaner was found to be extremely effective at removing the contents, (gravel and sediment), from within a 'pore'.

The pores in a pervious tarmac surface are immobile, therefore clogging of these surfaces requires movement of the material within the pores themselves. Surface clogging of permeable tarmac constructions has been considered as part of other studies, and in these cases high pressure hose treatment has been found to be moderately effective, (Hogland et al, 1987), but the tarmac surfaces would require more frequent treatment than the concrete block surface.

Where tarmac pavements are less satisfactory than the block pavement is in sub-surface maintenance. At some time it may prove desirable to replace the geotextile, either because of clogging or because of the collection of pollutants at the surface of the geotextile. This operation was performed as part of the changes described in Chapter 5 and was achieved simply by: removal and stacking of the blocks; pulling the gravel bedding layer to one side; and rolling up the geotextile, (Figure 2.3 shows the relative positions of the various layers). A similar operation for a construction with a tarmac surface would involve the total re-construction of the permeable surface.

8.3.3 Maintenance of infiltration devices

Previous discussions have highlighted the major cause of lack of maintenance, and hence failure, of infiltration devices, namely: ignorance of their existence. Obviously, the location of devices is greatly assisted by the use of some form of access, preferably intersecting the invert from the storm drainage and allowing access to the base of the device.

Assuming that the questions of 'awareness policy' have been addressed at the design stage and the owner of an infiltration system knows something of its existence, what is the owner expected to do? Hopefully, the design would be such that the device could perform satisfactorily for many years without maintenance. To achieve this, the problem of clogging of infiltration pores with matter drawn into the device by storm runoff should also be considered at the design stage. For sites such as the roof runoff infiltration site at Clifton campus, the input of sediment, leaves etc. was extremely small and the collection of material at the base of the devices was inconsequential.

However, in areas where roofs (especially flat roofs) are likely to collect leaves etc, or when dealing with road runoff, then consideration should be given to entrapment of solids before they can foul the device soil interface. There are many engineering solutions to this problem, some of them tried as part of this research: the chambered soakaway itself, which allows access into the structure; the use of the dual-chambered system, with the second chamber in the series receiving only filtered overflow from the first; the inspection chamber and porous distributor pipe for the infiltration trench; and finally, to a lesser degree, the monitoring tube within the stone-filled soakaway.

In Tokyo, Japan (Fujita, 1987), where an integrated system of infiltration devices is controlled by local government, each input to the system in the streets of Tokyo contains a 'trash basket' beneath the outlet to collect gross solids. The bucket being frequently emptied by council employees. This system may not be considered suitable in the U .K. where the aim of sediment collecting would probably be to provide a means for cleaning out the system if, and when, it failed.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

General. The broad aims of the research were to build, monitor, and analyse on-site, stormwater runoff attenuation/reduction devices. Primarily: a concrete block surfaced, permeable pavement; but also, a complimentary suite of runoff infiltration devices. Hydrological modelling based on the results of the monitoring was aimed at finding the significant variables controlling runoff volume. On-site stormwater attenuation and reduction practice in the U.K. and worldwide was considered in the light of the experimental data and experience.

Overall, the most significant feature regarding the collective results was that the infiltration and attenuation systems worked extremely well, (and certainly performed beyond the original expectations of the author).

A data set, covering a period of up to two years, was gathered for both systems. For the permeable pavement, a total of 62 rainfall/runoff events were recorded. The mean percentage runoff, for the 4 different sub-base stones of the car park, was between 34 and 47% of the mean rainfall. The volume of runoff from each bay showed a high (>0.9) correlation with the volume of rainfall, and the average rainfall losses, before runoff began, varied between 1.7 to 2.3mm. However, rainfall losses were observed to continue for the duration of the event. For the roof site, the mean percentage runoff was 68%, and the average rainfall interception loss 1.9mm.

Permeable pavement. Rainfall, incident at the surface, drained through to the base of the pavement where the 'runoff' was collected via a sub-base drainage system which prevented infiltration to the sub-grade. Both rainfall and runoff were subject to real-time measurement and recording. The pavement was divided into 4 separate compartments with separate drainage systems. Each compartment was filled with a different sub-base stone type: gravel, blast furnace slag, granite and limestone. This enabled comparison of the hydrological effects that the different stone gradings and textures had on the runoff from the car park.

The mechanisms by which the permeable pavement was able to reduce and attenuate the flow of rainwater were shown to continue with rainfall duration, (rather than being simple interception losses). An analysis of these possible 'continuing loss' processes, which act to increase rainfall losses for the duration of the event, was made. Paving block porosity, possibly accounting for up to 6mm of rainfall depth, and the slow wetting of the stones comprising the bedding layer and sub-base of the structure, were the primary means of both reducing the total runoff and attenuating the storm input. Evaporation was demonstrated as significant in restoring the capability of the pavement to retain water from subsequent rainfall, i.e. the storage occupied by the 'losses' to rainfall was returned to the atmosphere by evaporation. The relative volumes and temporal relationships of these different processes varied between events, depending on the event parameters and the state of the system at the beginning of rainfall, i.e. the storage occupied by the antecedent events.

Briefly, rainfall was subject to the following processes before occurring as runoff:

- Initially rainfall would soak into the surface layer of paving blocks. The porosity of the blocks continued to take up water, at a progressively slower rate, for some hours. However, when the rate of soaking into the blocks was exceeded by the rainfall rate, water would percolate down between the blocks and their gravel-filled pores;

- The sub-base stones retained rainfall upon flat surfaces, rough surfaces, and as menisci at stone-stone contact points. The attenuation potential of the stones lay in their effectiveness at providing convoluted pathways for the water as it moved down through the structure.

Both of these processes were a mixture of retention and attenuation processes. For example, some waters would be retained in storage to be subsequently released back to the atmosphere by evaporative processes: alternatively, the waters were attenuated by the routing processes of percolating through the numerous convoluted surfaces and through water-filled pores and menisci.

As the four bays containing the four sub-base stones were constructed to similar dimensions, the observed variations in runoff parameters were due to the characteristics of the stones themselves. The blast furnace slag sub-base, (at the grading tested), was shown to retain the most water and consistently produced the lowest volume of runoff. This was thought to be mainly as a result of the 'honey combed' texture of many individual stone surfaces, (caused by air bubbles within the molten slag as it solidified).

The gravel and granite sub-base stones displayed similar attenuation-retention characteristics. This was probably for different reasons, such as:

- the uniform, small-graded, gravel stones provided a relatively large number of stone-stone contact points which percolating water had to 'negotiate'; and,
- the large, angular fragments of granite provided upturned surfaces upon which water 'pooled'.

The limestone sub-base generally produced the highest volume of runoff, its relative inability to reduce runoff was probably related to its smooth rounded surfaces.

Two modelling approaches were taken to determine the variables, (and their parameters), important in generating runoff. The first was to design a conceptual model to predict the runoff volume, (for on-site detention devices the volume, not the peak flow, is the most important design factor). The model was generated from experimental data which demonstrated a finite absorption rate for occupation of storage within the surface of the pavement, and the likely rates of evaporation from within the pavement. The model input variables were: the depth and duration of both the observation and antecedent event; and the time between the two events.

After the event, the 'losses' to rainfall were held as storage within the pavement and were subject to evaporation. The time period to the next event therefore dictated the volume of storage still occupied from the initial event; this storage was 'denied' the subsequent event which may have developed a higher runoff as a result. For future research it is recommended that the interplay between antecedent rainfall, storage uptake and subsequent evaporation could be most accurately determined using bench-scale laboratory testing.

The second modelling approach was using multiple linear regression analysis. The event data, for each sub-base stone type, enabled the derivation of computer based models which can give accurate predictions of runoff for a given rainfall. The models generated can be used to predict the mean and peak intensity, and average duration of runoff, from the following variables: rainfall, rainfall duration, peak rainfall intensity and antecedent rainfall. Combinations of predicted runoff duration and intensity enabled accurate prediction of the volume of runoff. Models providing comparison of the relative hydrological performance of the sub-base stones, (as aids to understanding the relative effects of the variables described) were outlined. No statistically significant relationship between the prevailing temperature and the volume of runoff or loss to rainfall was found.

The models are somewhat site-specific, and structures built for a similar purpose, but with different dimensions, grades, or drainage patterns, might produce significantly different rainfall-runoff relationships, especially if there were systematic differences in the collection of data. The opportunity to alter the rainfall-runoff relationship was highlighted by work on the control of discharge from the permeable pavement, which showed that the relatively sediment free runoff could be subject to filtering and a hydraulic throttle.

This research did not set out to advance the development of structural specifications for infiltration/attenuation systems such as permeable pavements and soakaways, although some work on structural improvements to the permeable pavement were made. These 'improvements' were aimed at reducing the degree of deflection or depression observed in the surface layer of blocks, and consisted of integrating polypropylene 'geogrid' elements with the sub-base or bedding layer of the pavement to increase the structures capability to withstand loading. Significant reductions in the amount of deflections were monitored as a result of these improvements. Also, incorporation of these structural elements served to show that the permeable pavement was easily dismembered and reconstructed, entailing no destruction to its constituent parts: thus making maintenance or upgrading of such a structure a relatively simple procedure.

Infiltration devices. The runoff infiltration devices: a stone-filled soakaway; a dual-chambered soakaway; and an infiltration trench, were constructed in poorly draining soil conditions to enable runoff from a roof surface to dissipate into the ground. A simple monitoring system was installed to calculate rainfall, runoff, storage and infiltration.

The infiltration devices were demonstrated to be effective at disposing of the runoff from a pitched roof surface, even though the area was deemed unsuitable for such practice by current recommendations/guidelines. Although the results for the infiltration devices were 'site specific', they did indicate that current sizing methods, for devices such as soakaways, under-estimate the potential for infiltration within 'poorly draining' soils, especially when such devices are subject to large runoff input. A series of borehole percolation tests has demonstrated how such sizing methods could be subject to seasonal variation.

The monitoring of the infiltration devices indicated that they would have performed perfectly satisfactorily as the final disposal method for runoff from the permeable pavement. This practice of 'on-site but off-structure' disposal could be adopted if structural considerations dictated that direct infiltration through the base of the pavement to the sub-grade was undesirable.

Design improvements. Observed runoff percentages of less than 100% for both sets of structures may have led to the recommendation of sub-unity coefficients for use in dimensioning of stormwater infiltration devices. However, many of the larger, more intense rainfall events, those more likely to be chosen for design events, had runoff in excess of the average. It is therefore recognised that the practice of designing systems to accept 100% runoff from 'impermeable' surfaces will, in many cases, lead to an over-design, especially with respect to total volumes received. However, this practice is likely to provide a small, but acceptable, factor of safety for determining the runoff from rainfall events of similar parameters to a 'design' event.

The rainfall conditions antecedent to an event have been shown to be significant in affecting the volume of runoff from that event, and should be reflected in the provision of a 'time to recover' or 'time to empty' in the specification for infiltration/attenuation devices. However, a fuller understanding of the relationship between antecedent conditions and runoff for a subsequent event requires further study, this should include the relationship of likely antecedent conditions to a given design event. Consideration of antecedent conditions are probably most important in geographical areas which receive most of their annual rainfall in short periods or 'rainy' seasons.

There are extensive opportunities for the widespread use of on-site stormwater disposal systems. The generally simple philosophy, and the associated low technology of the systems, should not be seen as a hindrance, but as an aid to the promulgation of the practice, (especially in these environmentally conscious times). Further research in this field should aim to produce examples and simple methodologies that can be adopted, with confidence, by engineers, developers, professional organisations and planning authorities alike.

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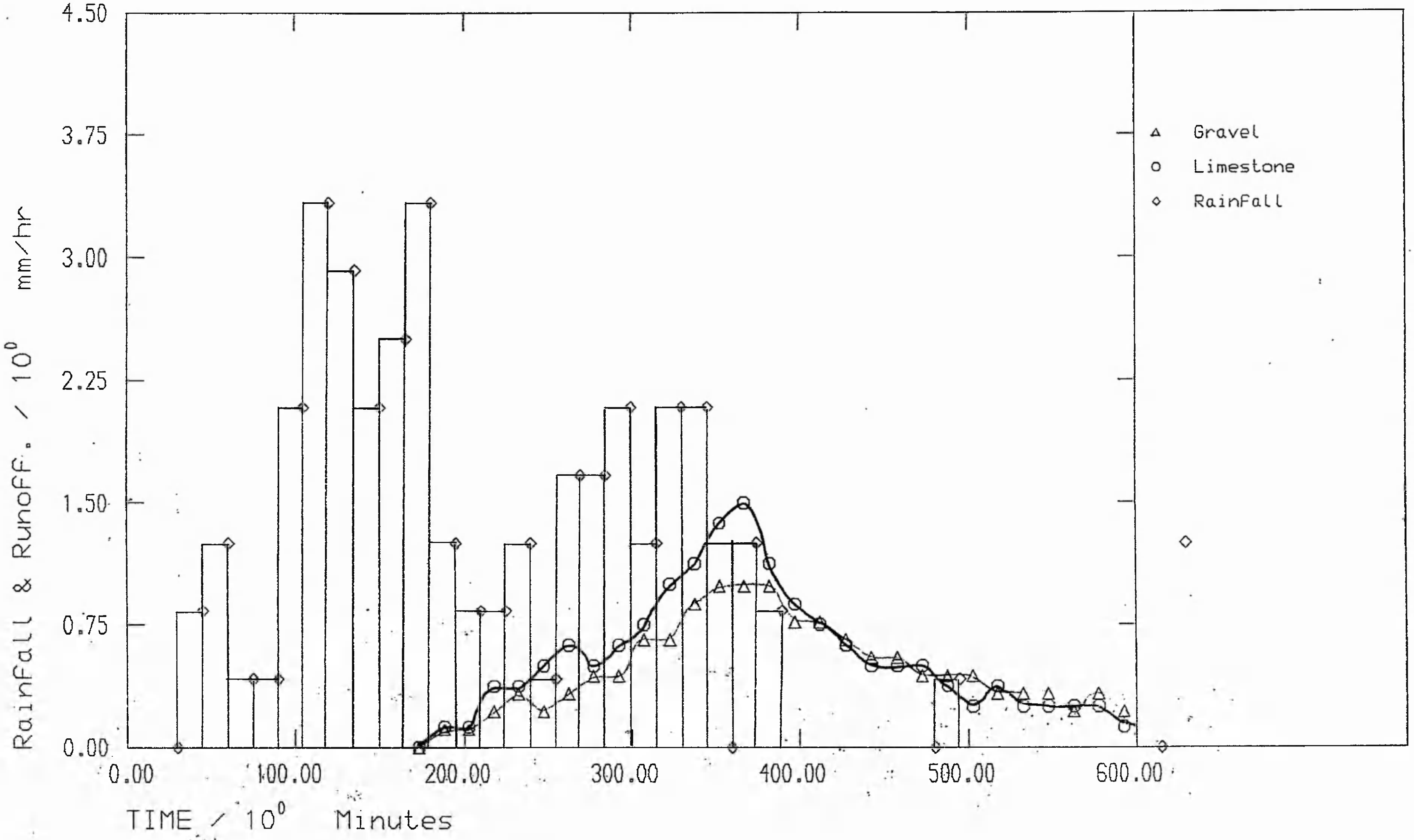
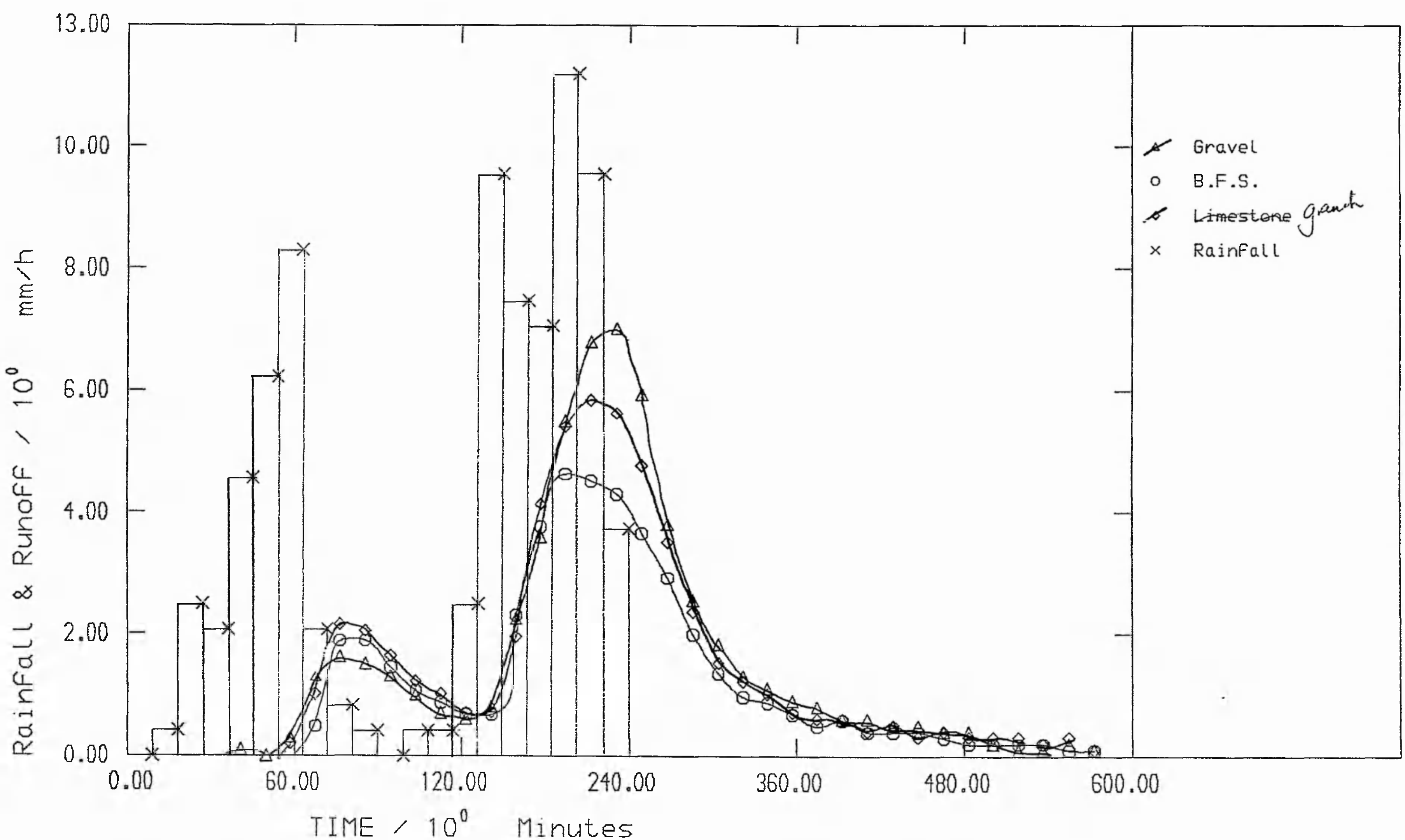
APPENDIX 1

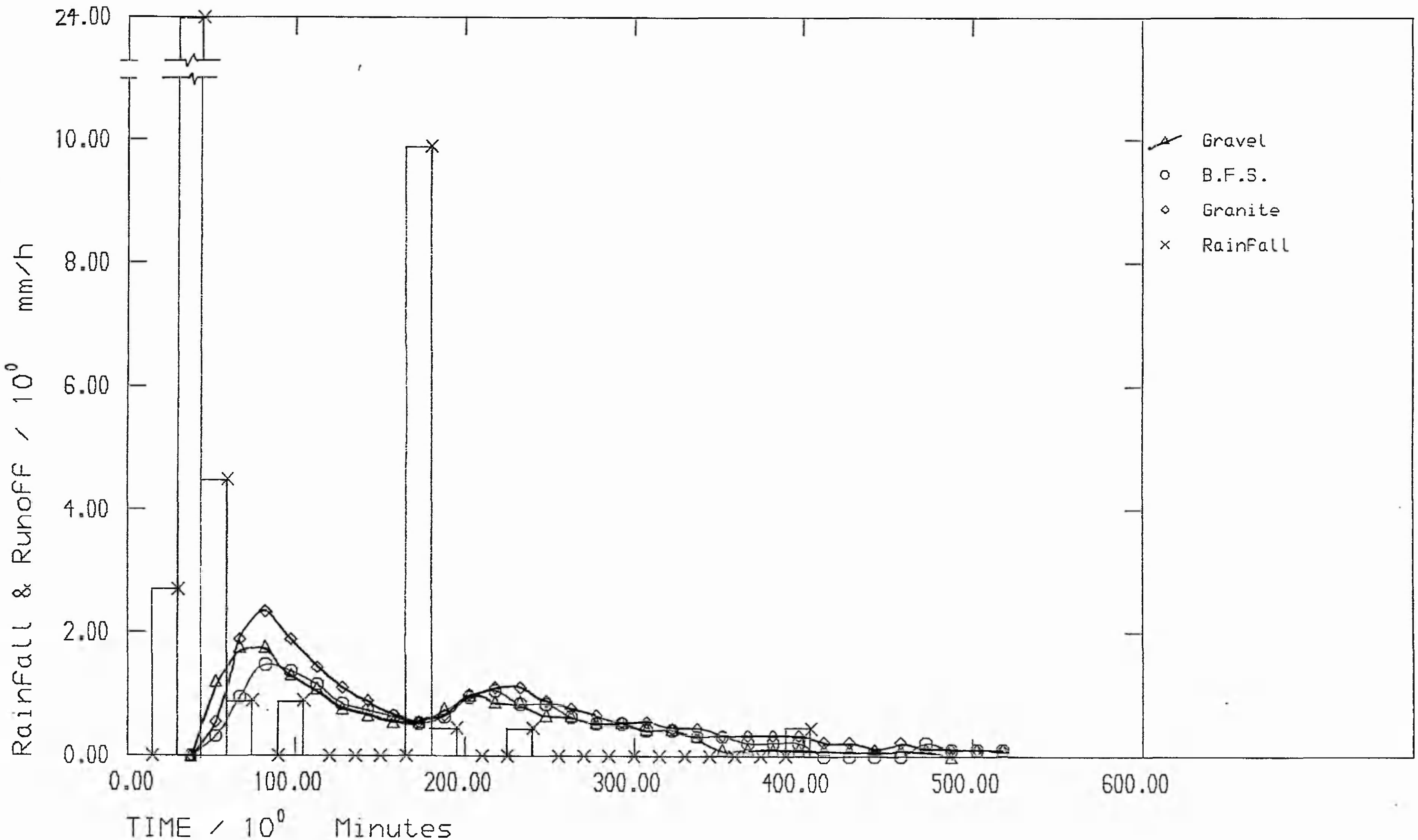
SELECTION OF RAINFALL RUNOFF HYDROGRAPHS FOR THE EVENTS STUDIED



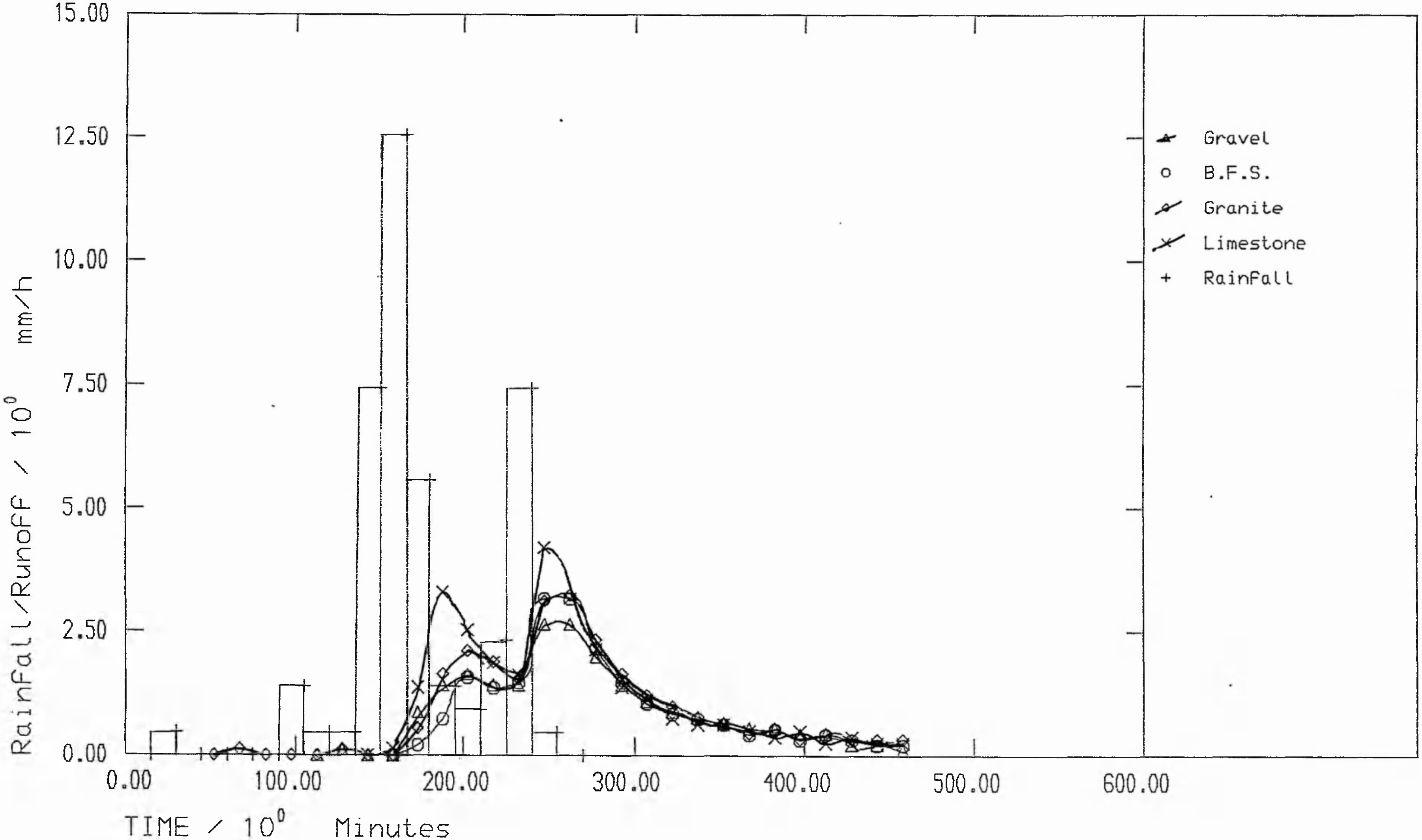
Rainfall/Runoff Plot. Event No. 7207c.
Start Time 16:00hrs. Total Rainfall 9.2mm.



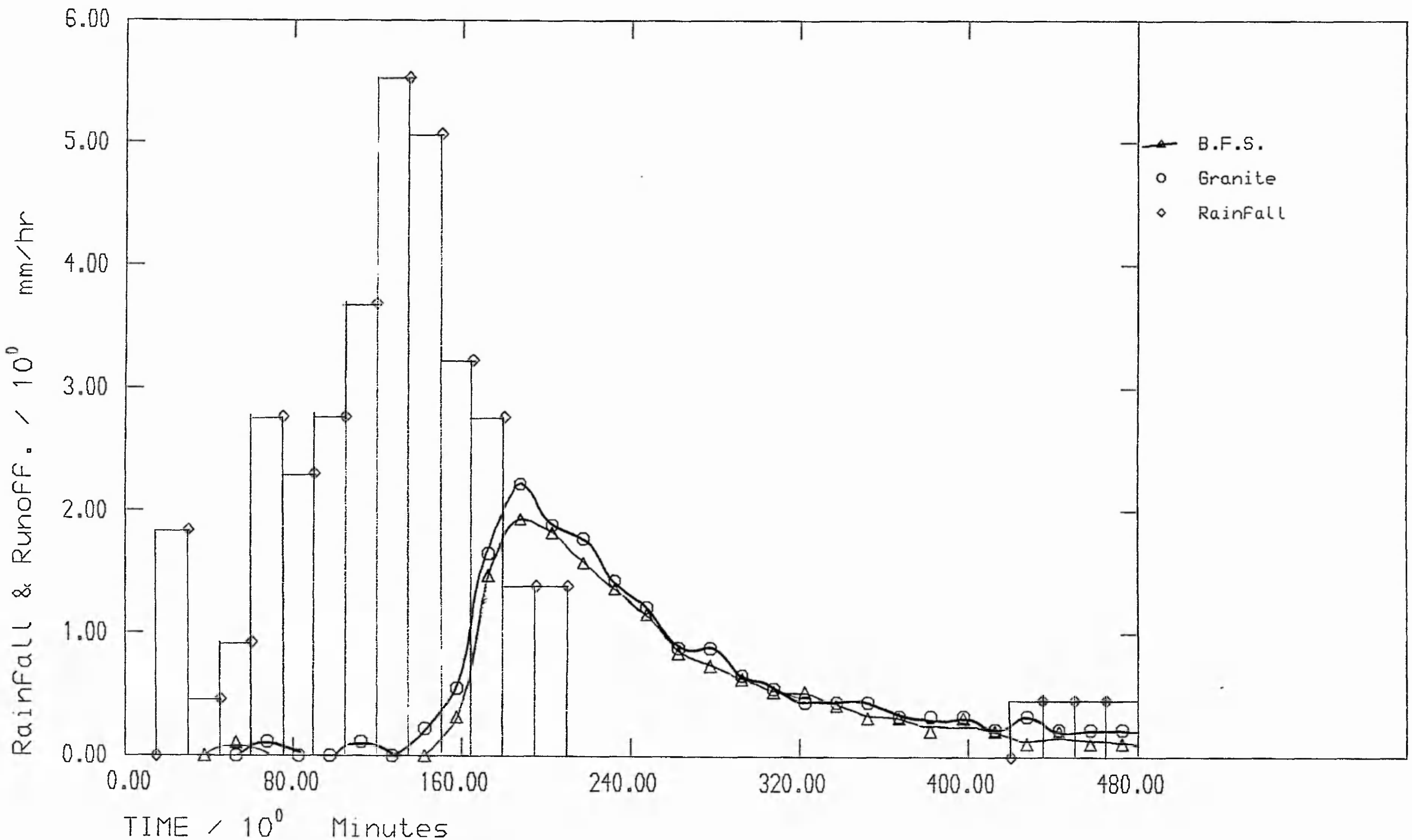
Rainfall/Runoff Plot. Event No. 7225c. Start Time 04:45hrs
Total Rainfall 21.5mm



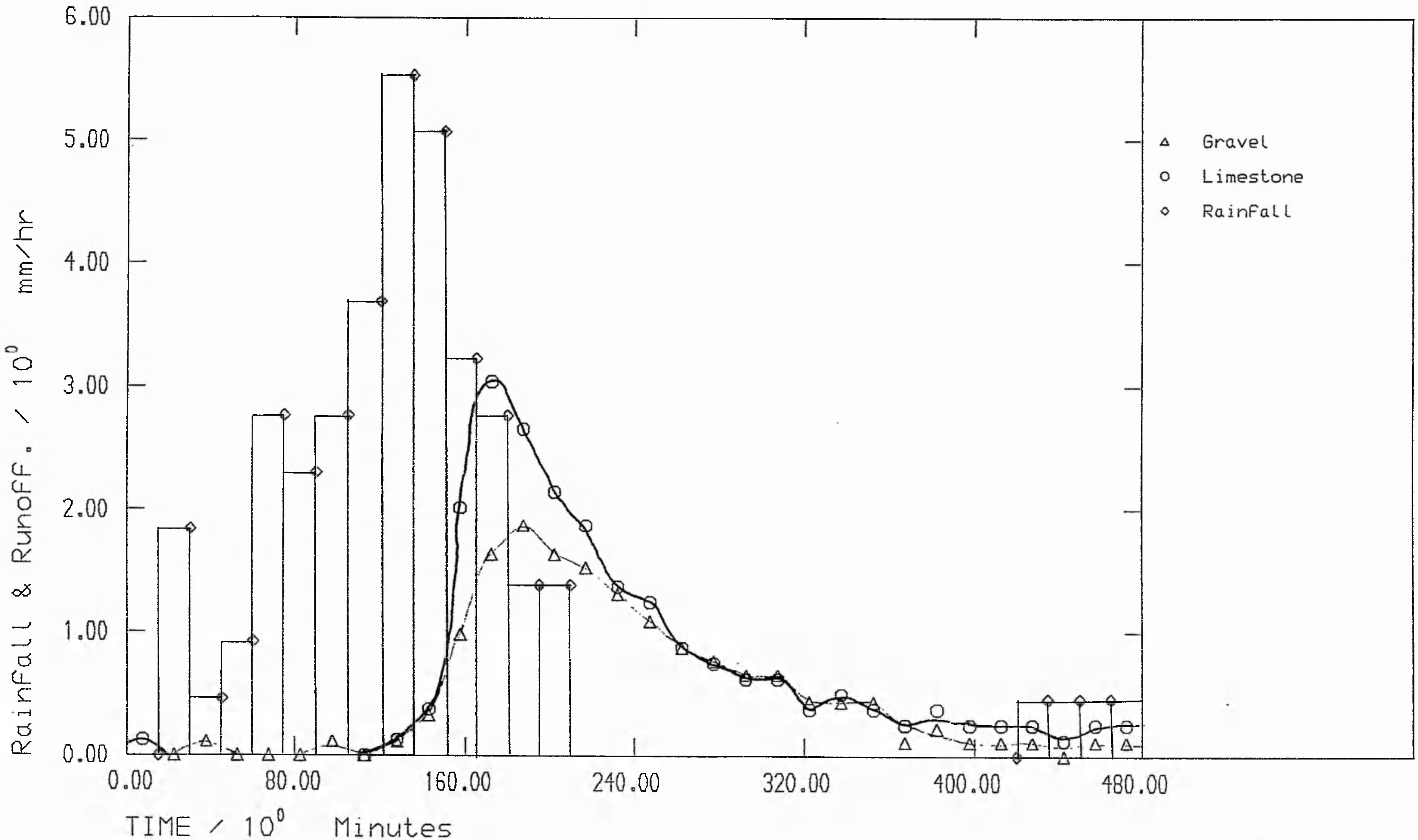
Rainfall/Runoff Plot. Event No. 7230c.
Start Time 00:30hrs. Total Rainfall 10.0mm



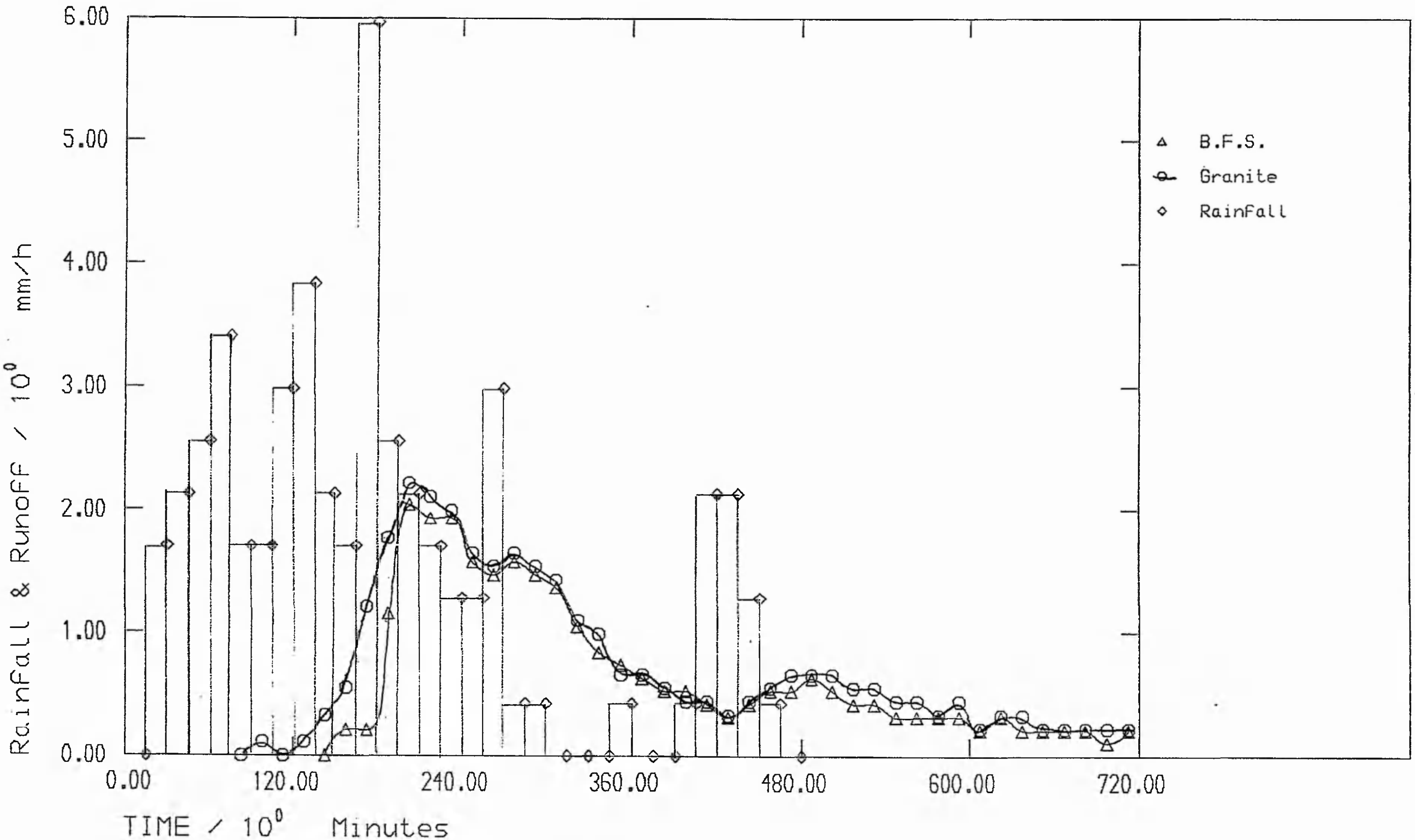
RainFall/RunoFF Plot. Event No. 7235c
Start Time 07:30hrs. Total RainFall 10.1mm



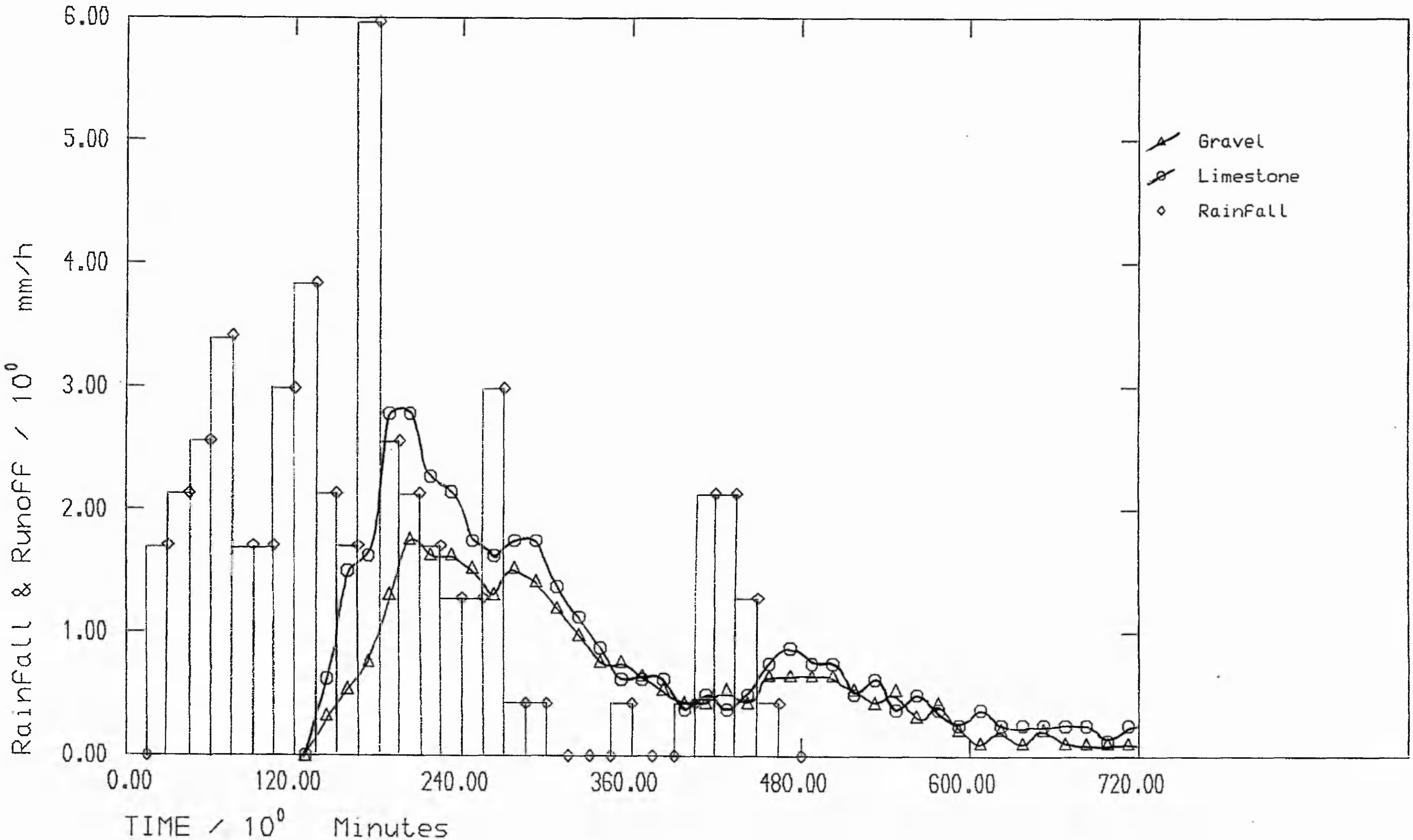
Rainfall/Runoff Plot. Event No. 7238c.
Start Time 13:00hrs. Total Rainfall 8.5mm.



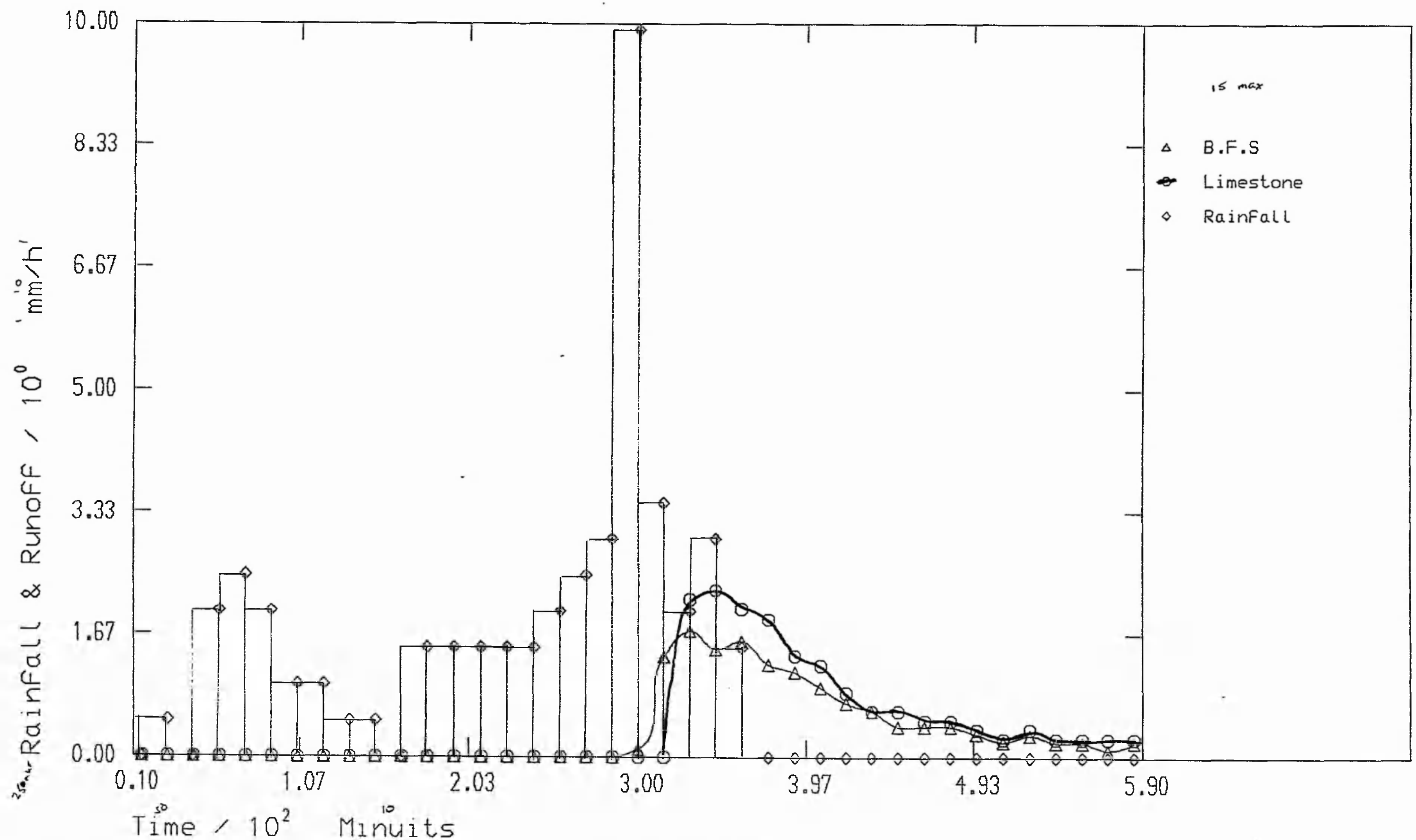
Rainfall/Runoff Plot. Event No. 7238c.
Start Time 13:00hrs. Total Rainfall 8.5mm.



Rainfall/Runoff Plot. Event No. 7249c.
Start Time 13:00hrs. Total Rainfall 12.4mm

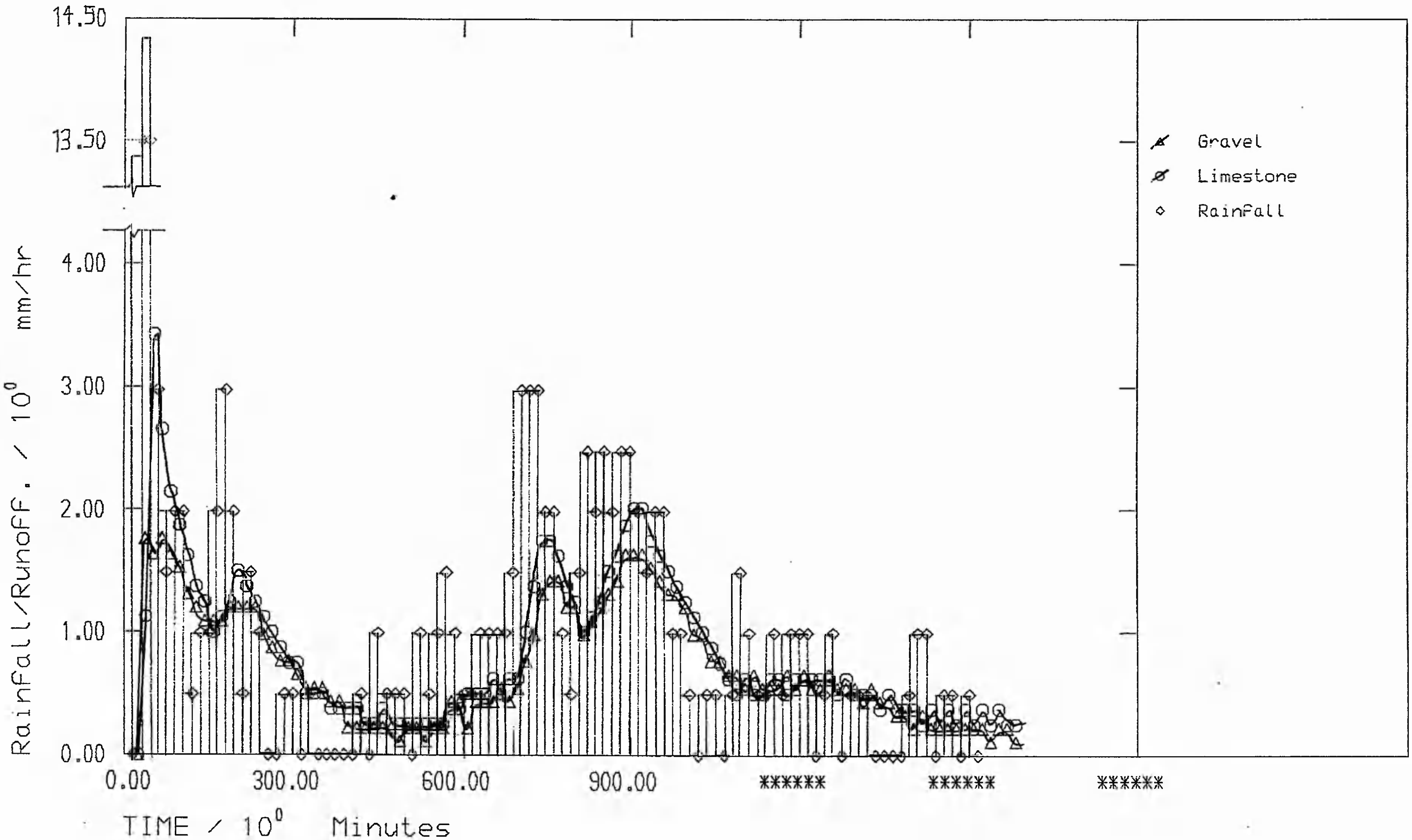


Rainfall/Runoff Plot. Event No. 7249c.
Start Time 13:00hrs. Total Rainfall 12.4mm

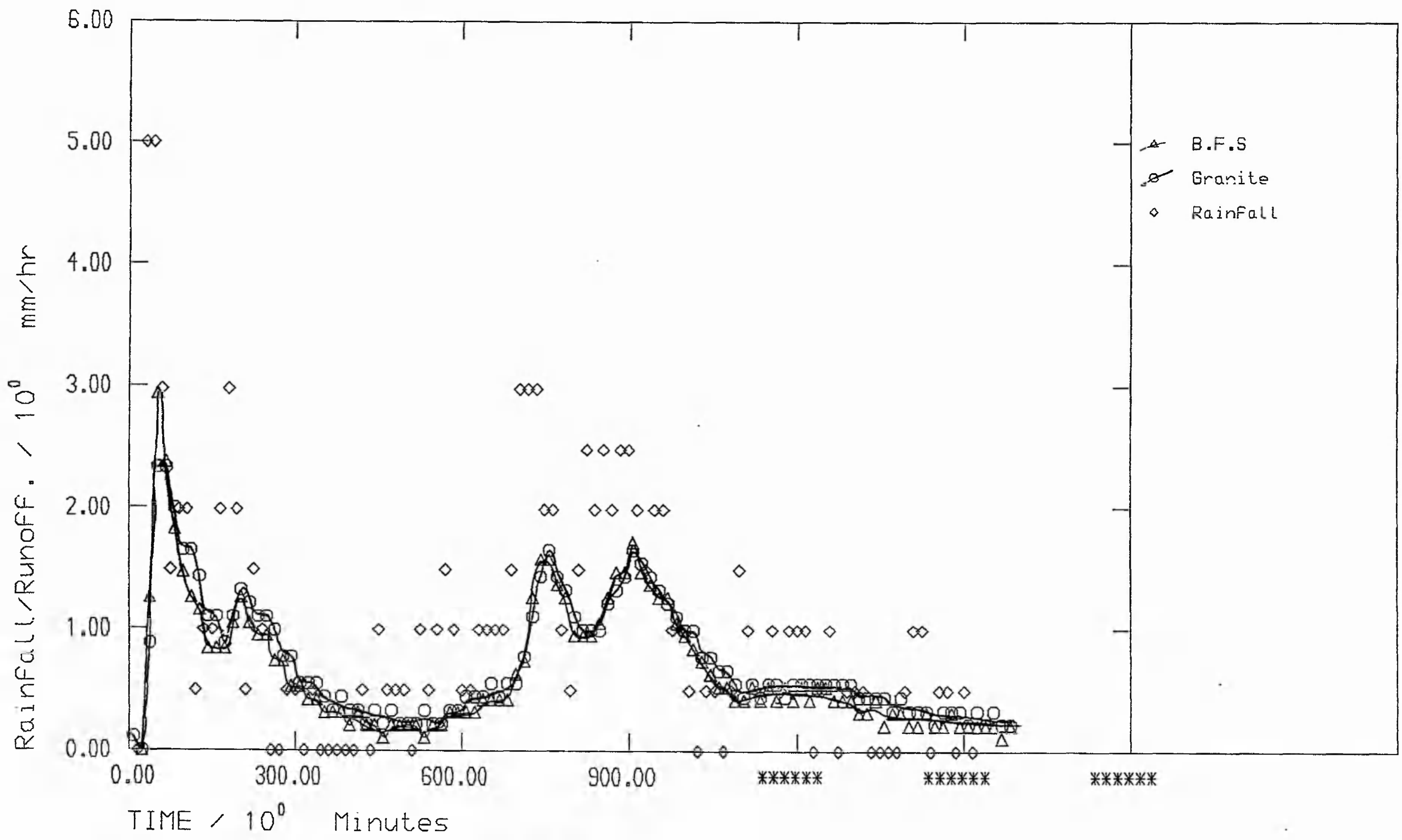


Rainfall/Runoff Plot

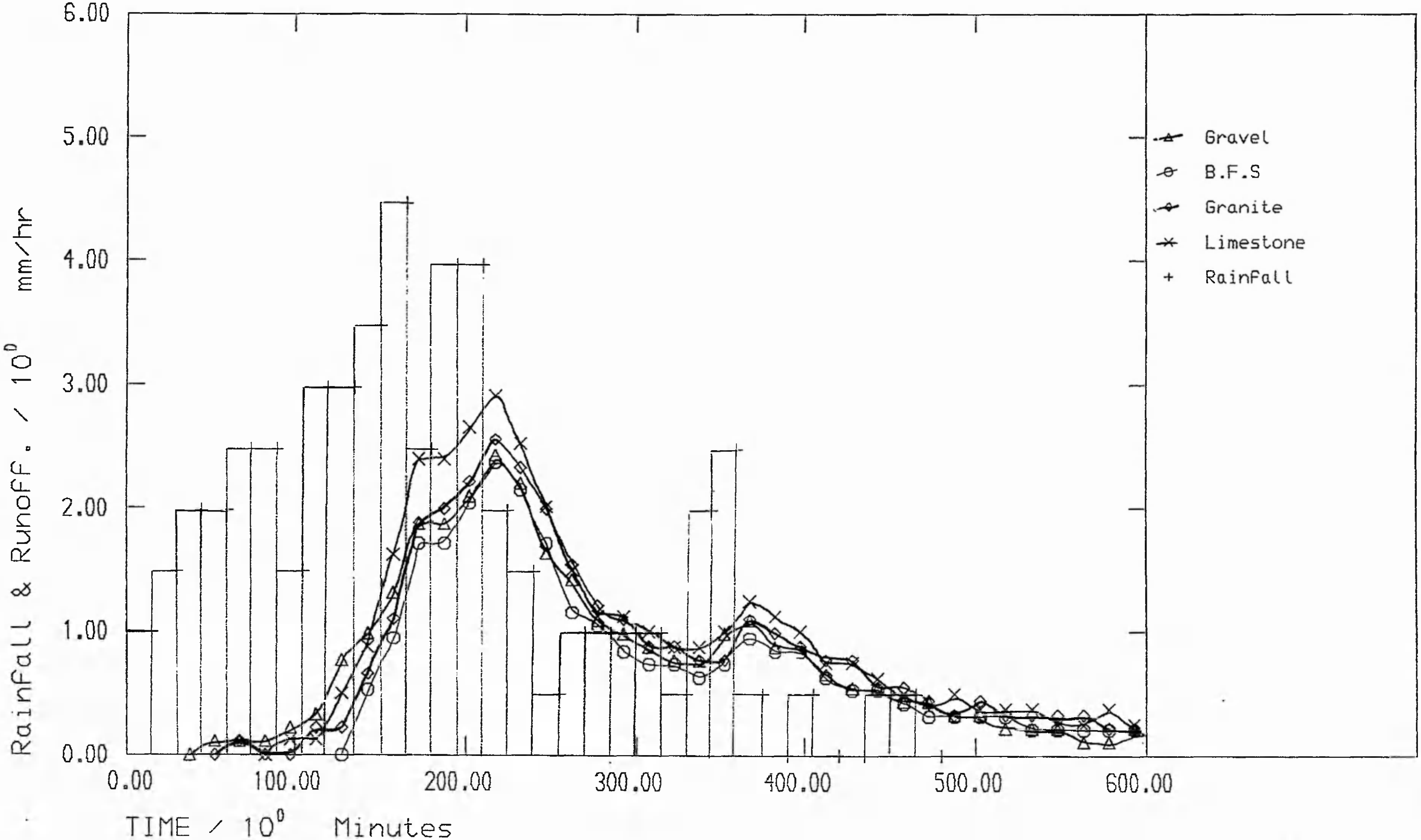
Event No 7277. Start time 02:30



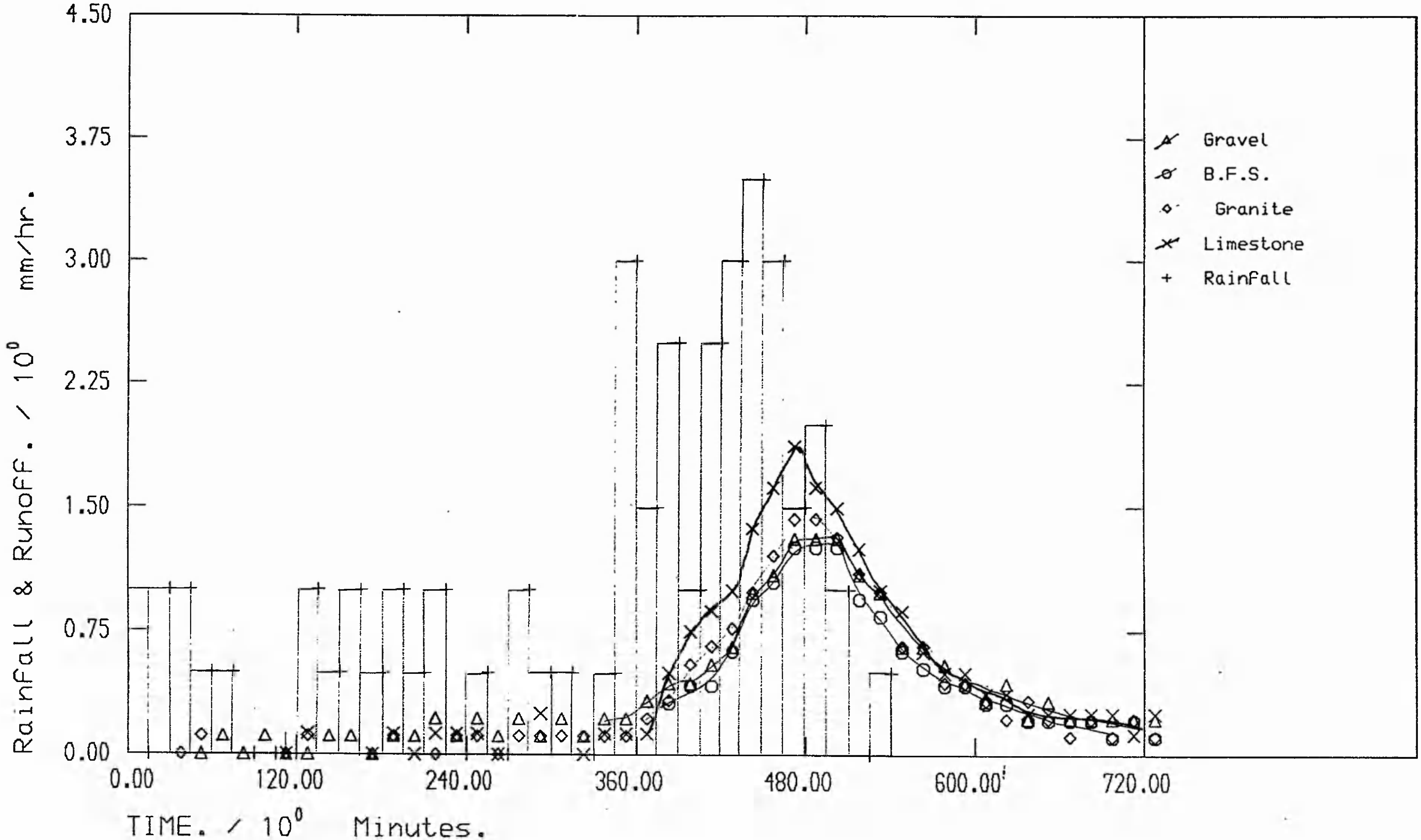
Rainfall/Runoff Plot. Event No. 7282c
Start Time 21:30hrs. Total Rainfall 30.1mm



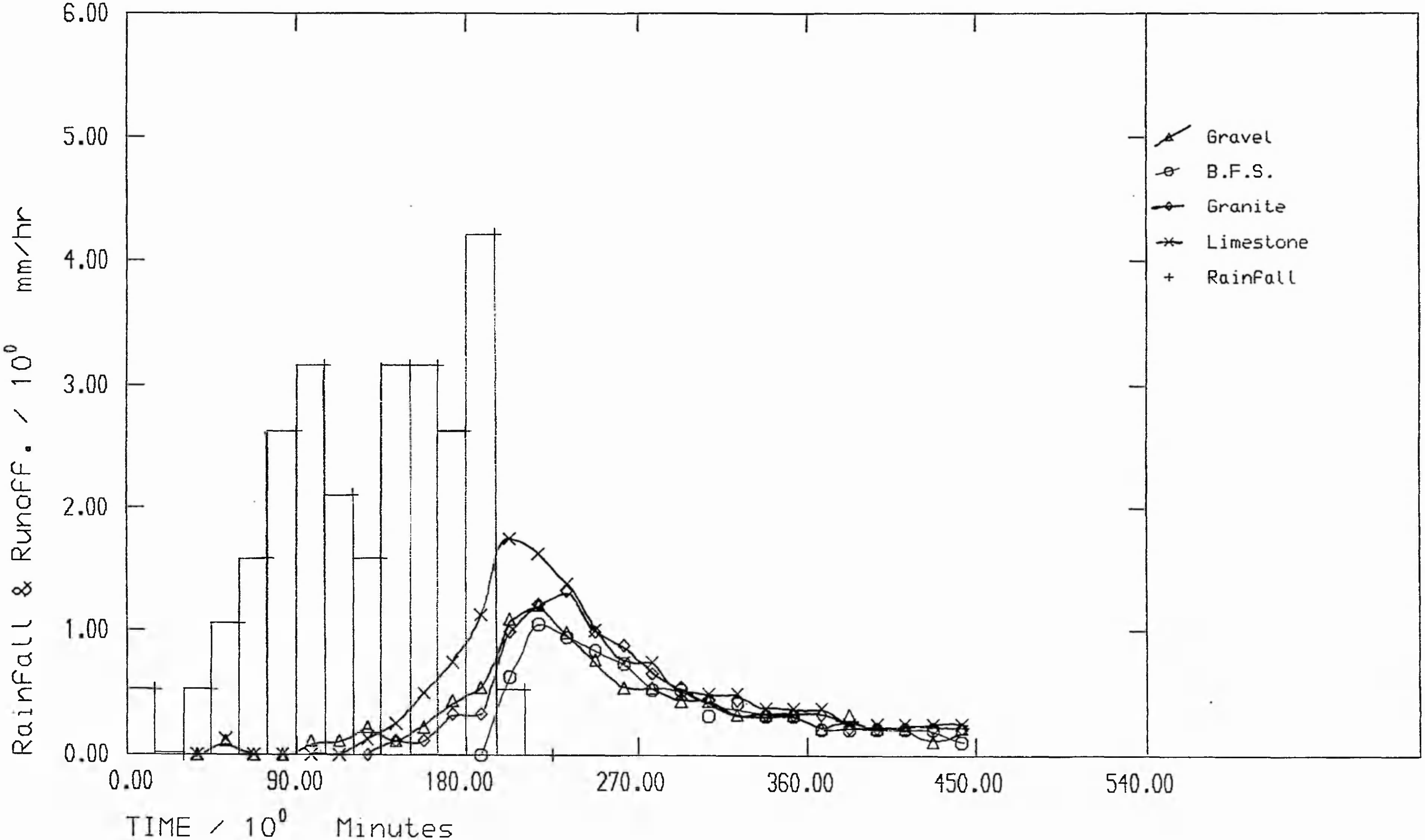
Rainfall/Runoff Plot. Event No. 7282c
Start Time 21:30hrs. Total Rainfall 30.1mm



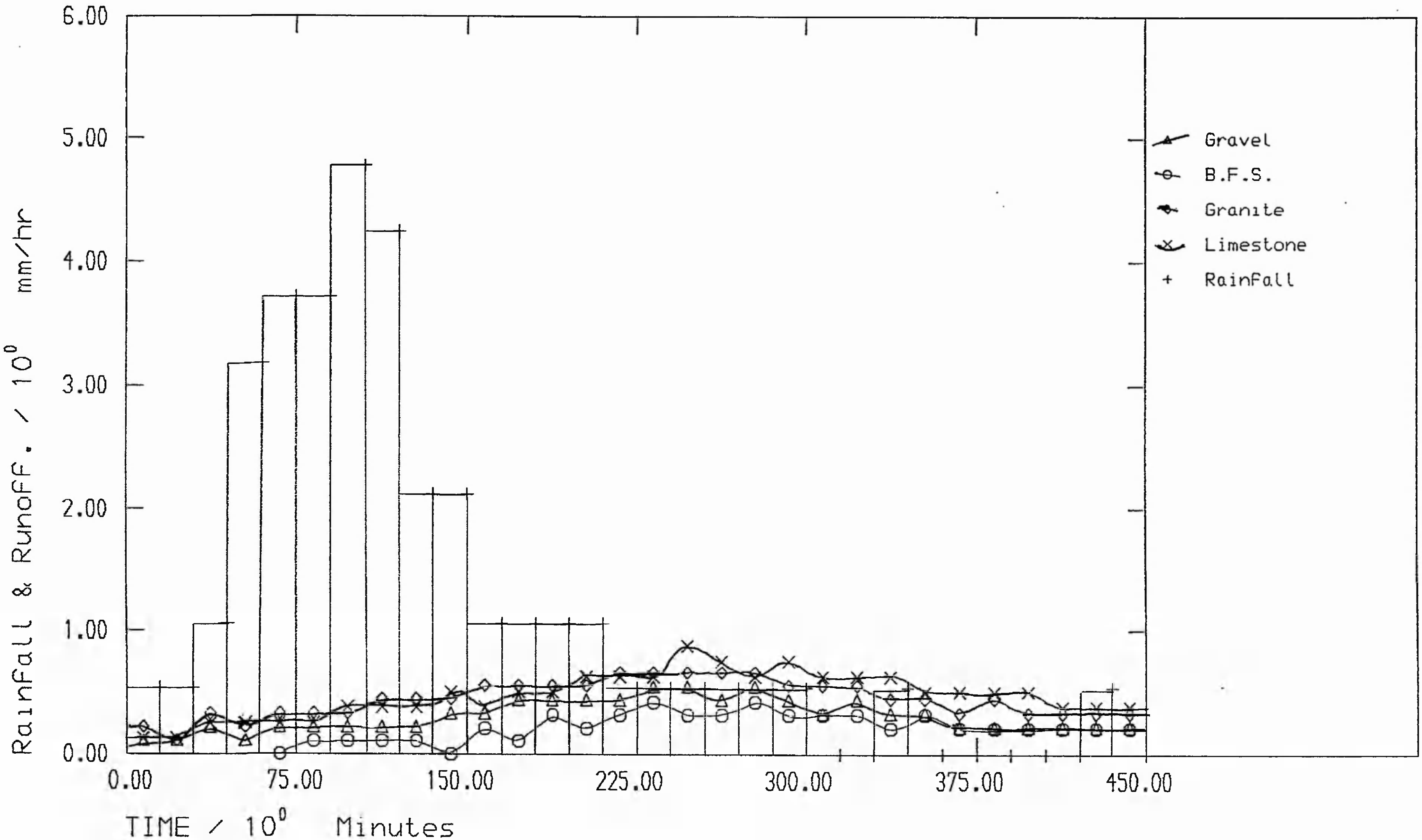
Rainfall/Runoff Plot. Event No. 7287c.
Start Time 23:00hrs. Total Rainfall 13.0mm.



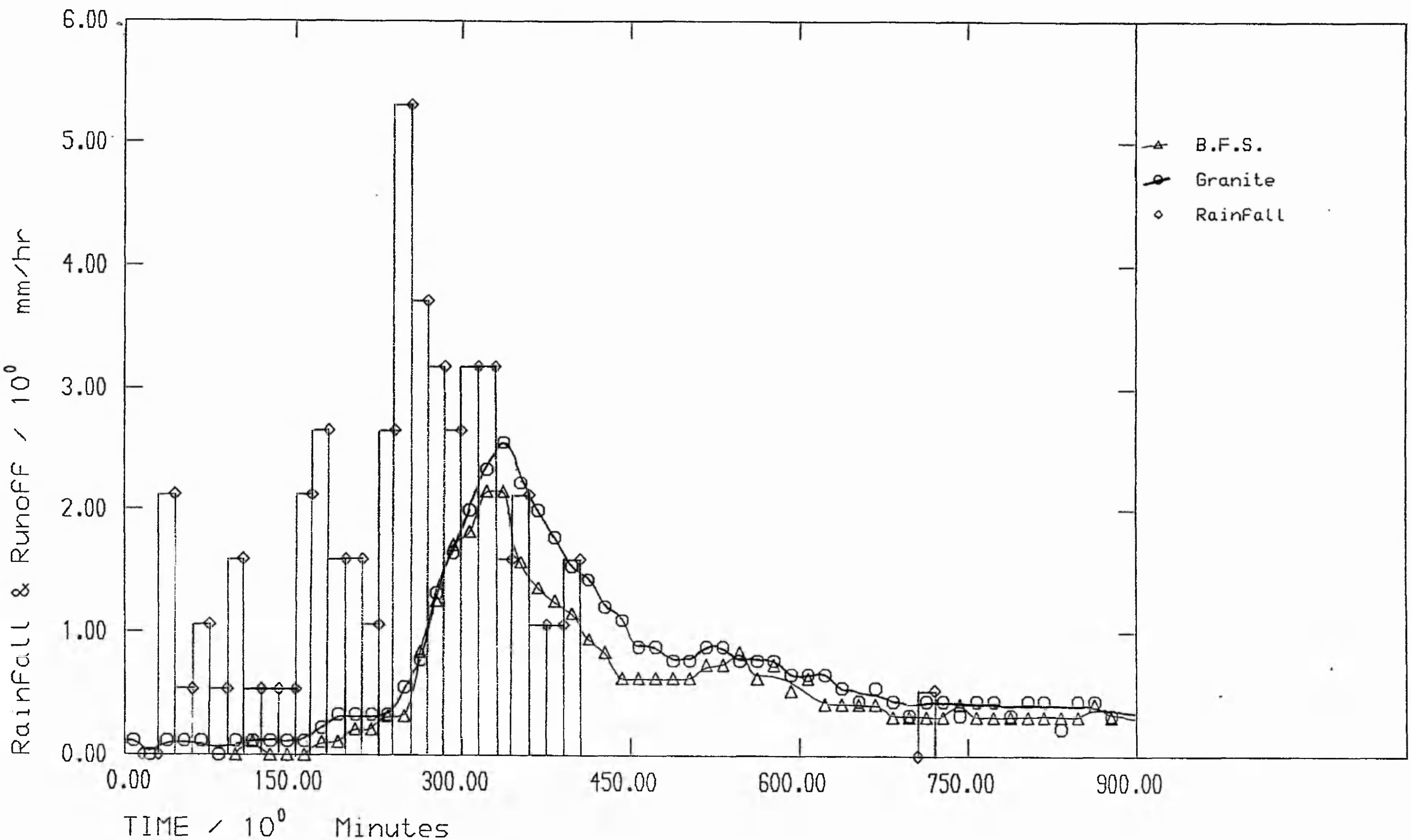
RAINFALL/RUNOFF PLOT. Event No. 7323c.
Start Time 04:30hrs. Total Rainfall 9.1mm.



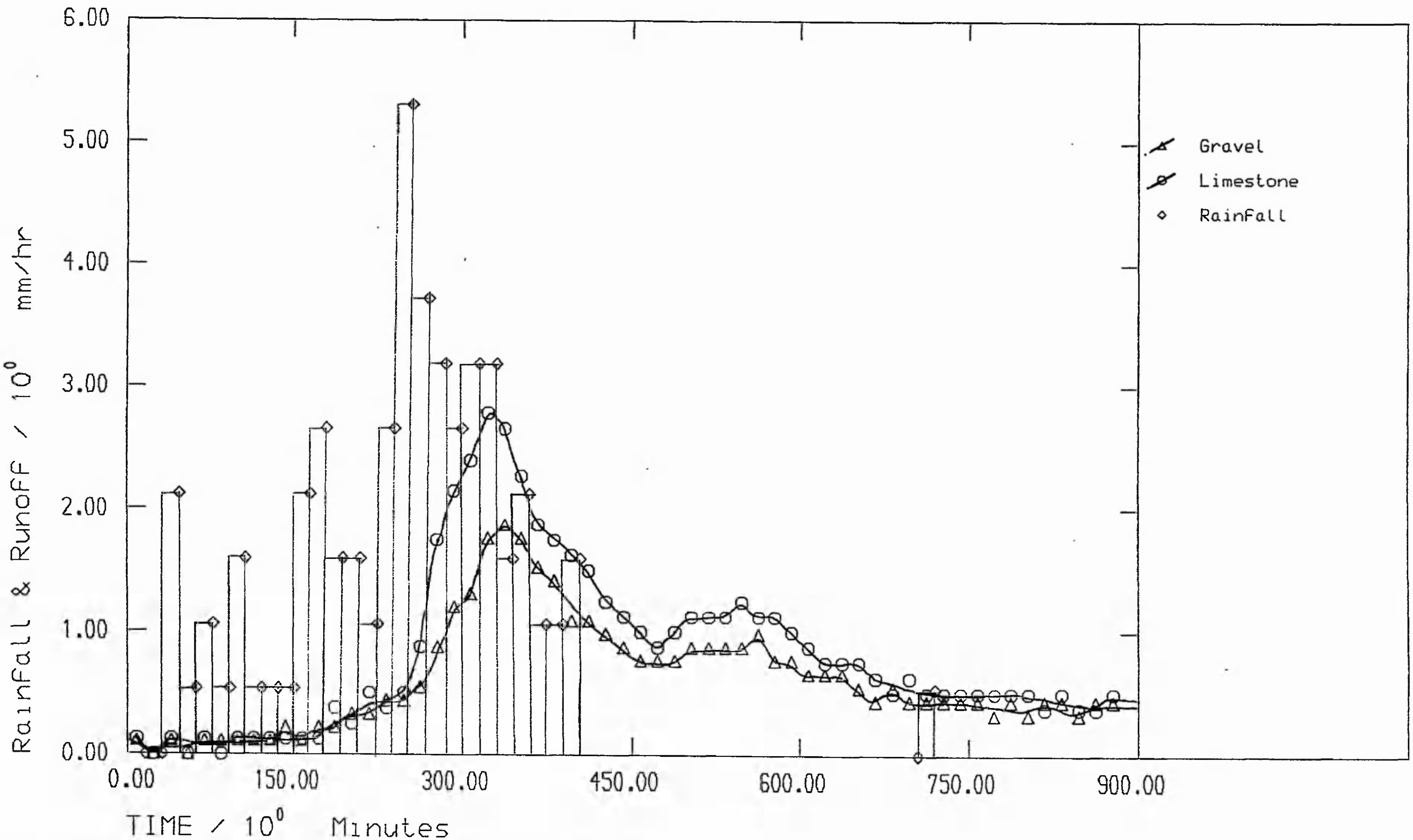
Rainfall/Runoff Plot. Event No. 8008c.
Start Time 19:00hrs. Total Rainfall 6.5mm



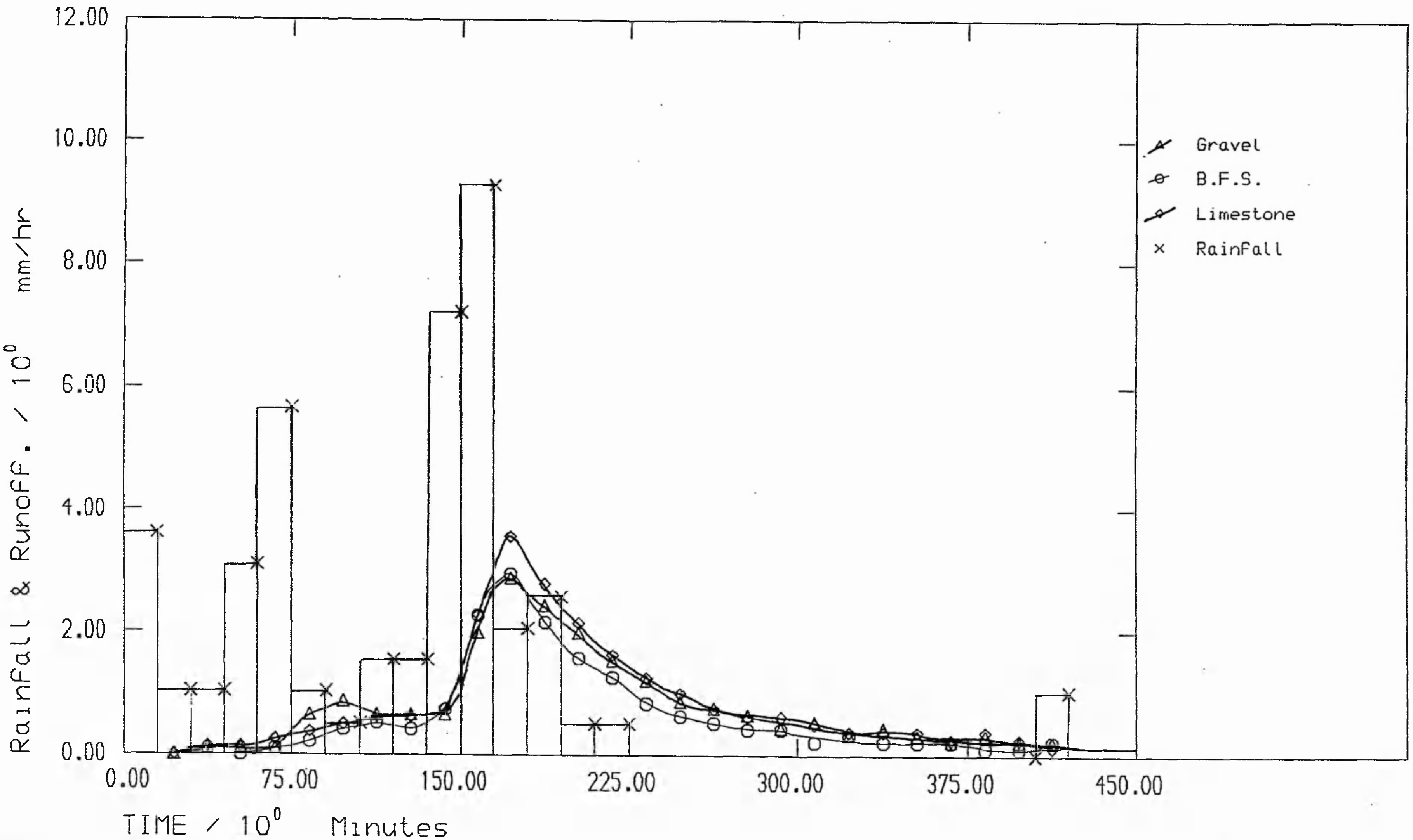
Rainfall/Runoff Plot. Event No. 8022c.
Start Time 11:00hrs. Total Rainfall 8.2mm



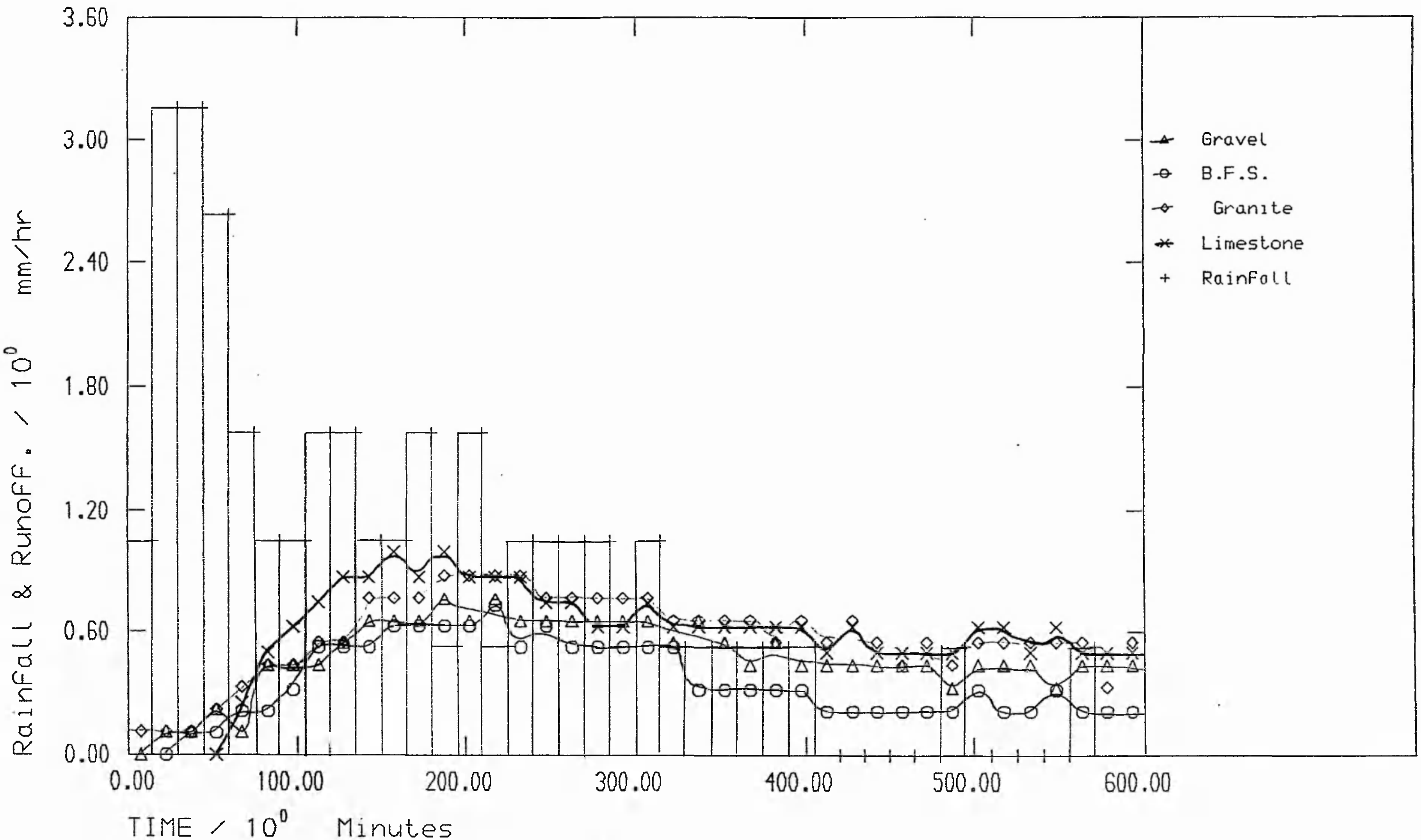
Rainfall/Runoff Plot. Event No. 8023c.
Start Time 08:00hrs Total Rainfall 11.9 mm



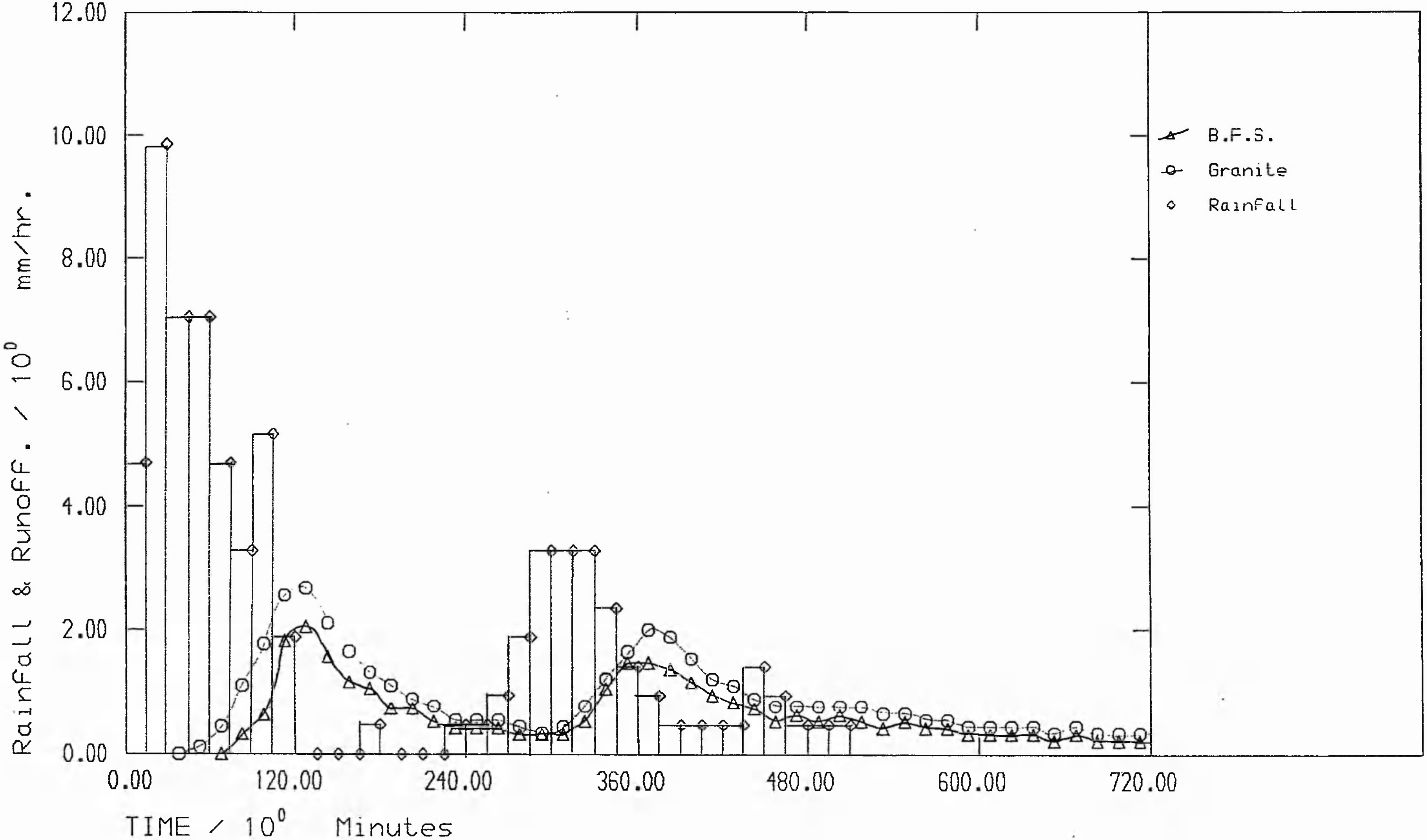
Rainfall/Runoff Plot. Event No. 8023c.
Start Time 08:00hrs Total Rainfall 119mm



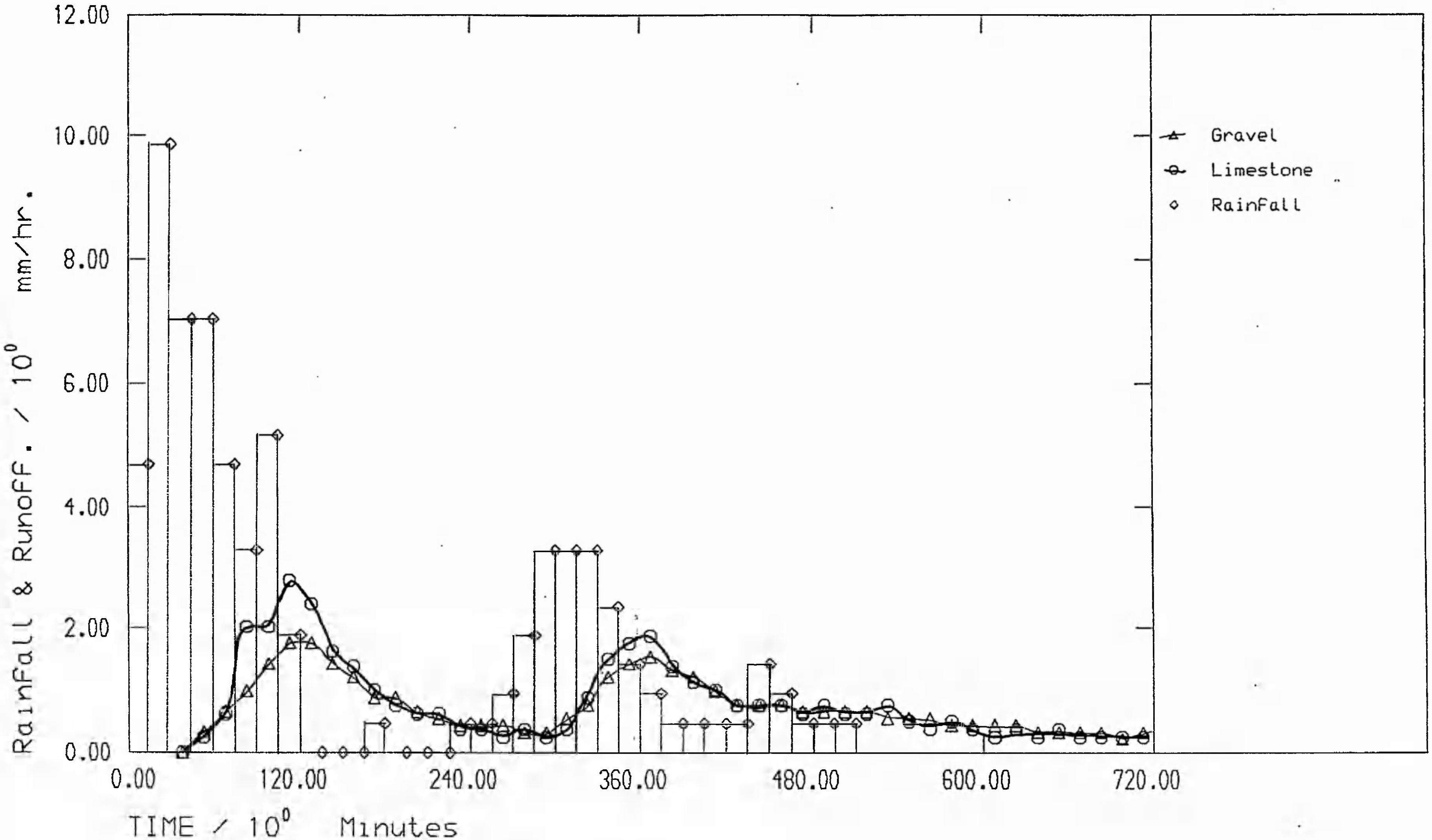
Rainfall/Runoff Plot. Event No. 8035c.
Start Time 04:15hrs. Total Rainfall 10.3mm.



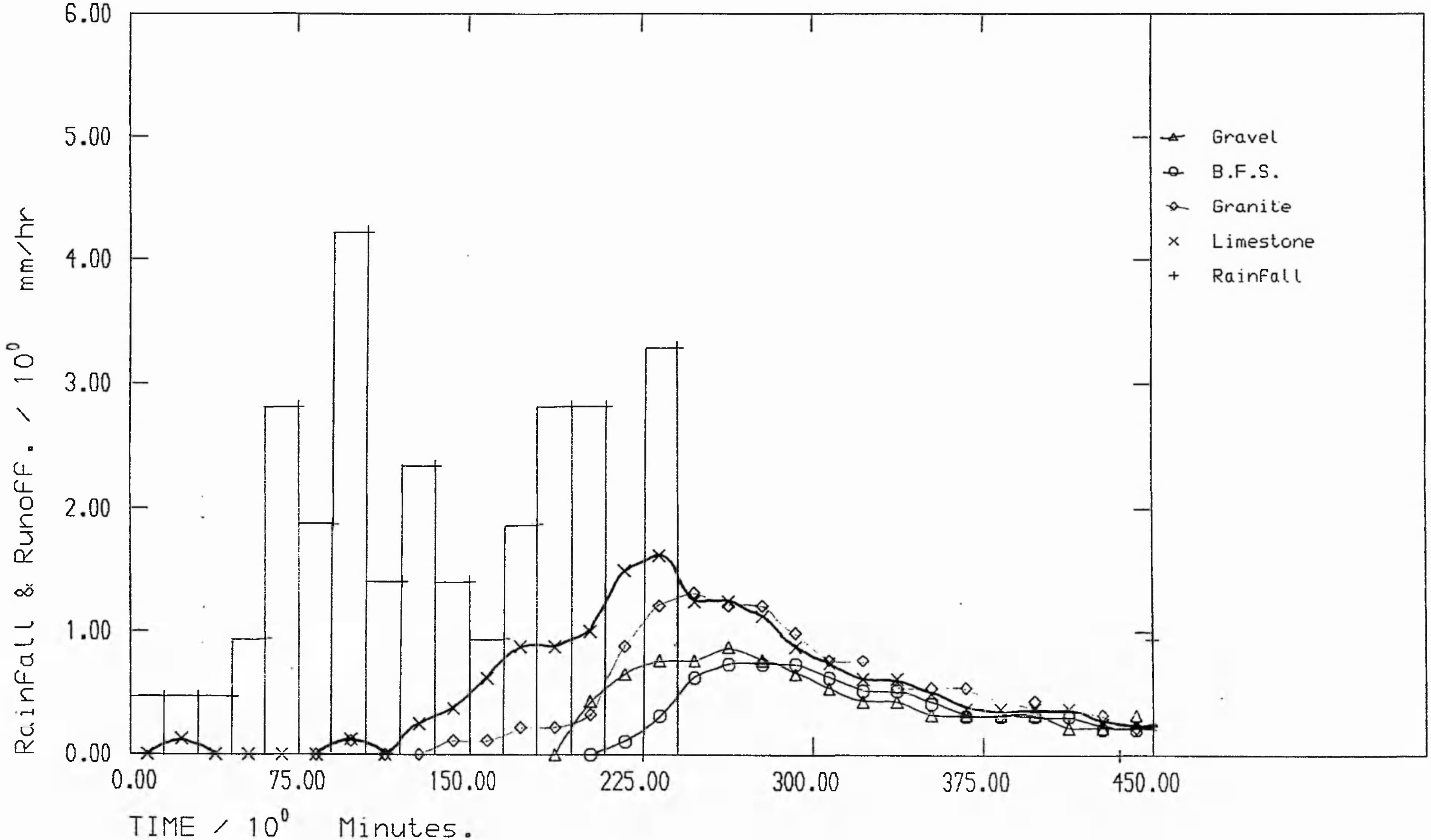
Rainfall/Runoff Plot. Event No. 8073c.
Start Time 14:00hrs. Total Rainfall 8.0mm.



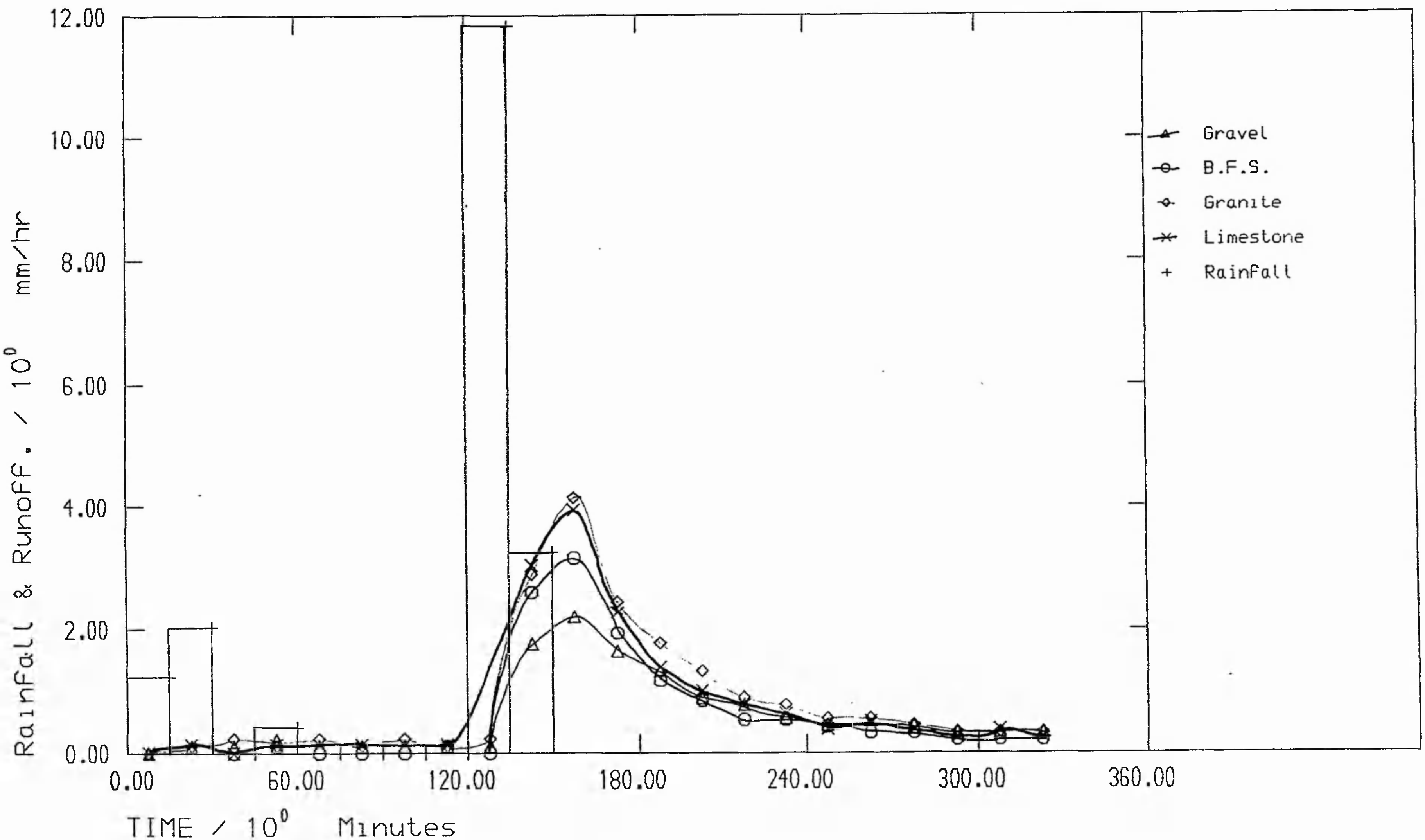
Rainfall/Runoff Plot. Event No. 8094c.
Start Time 15:00hrs. Total Rainfall 17.0mm



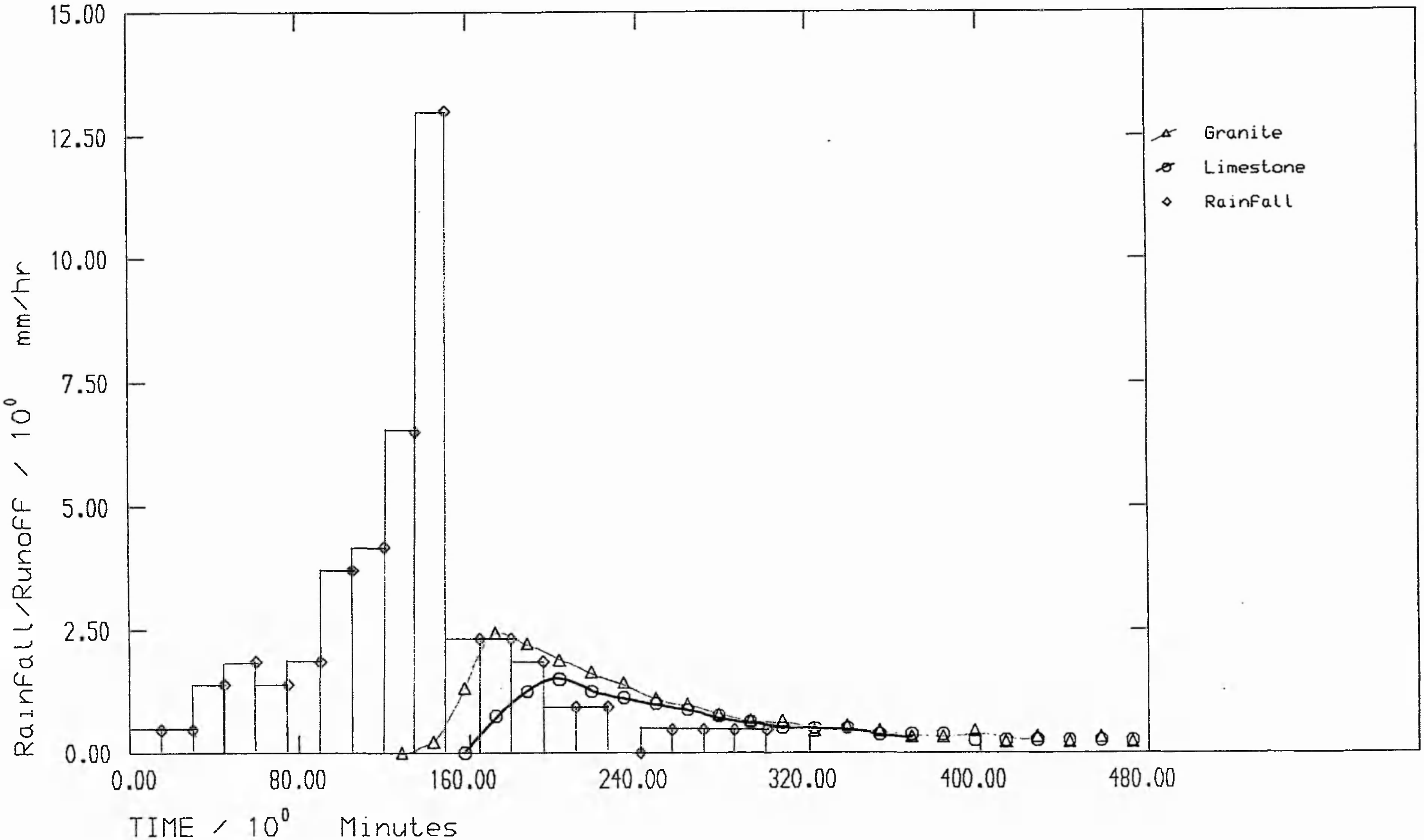
Rainfall/Runoff Plot. Event No. 8094c.
Start Time 15:00hrs. Total Rainfall 17.0mm



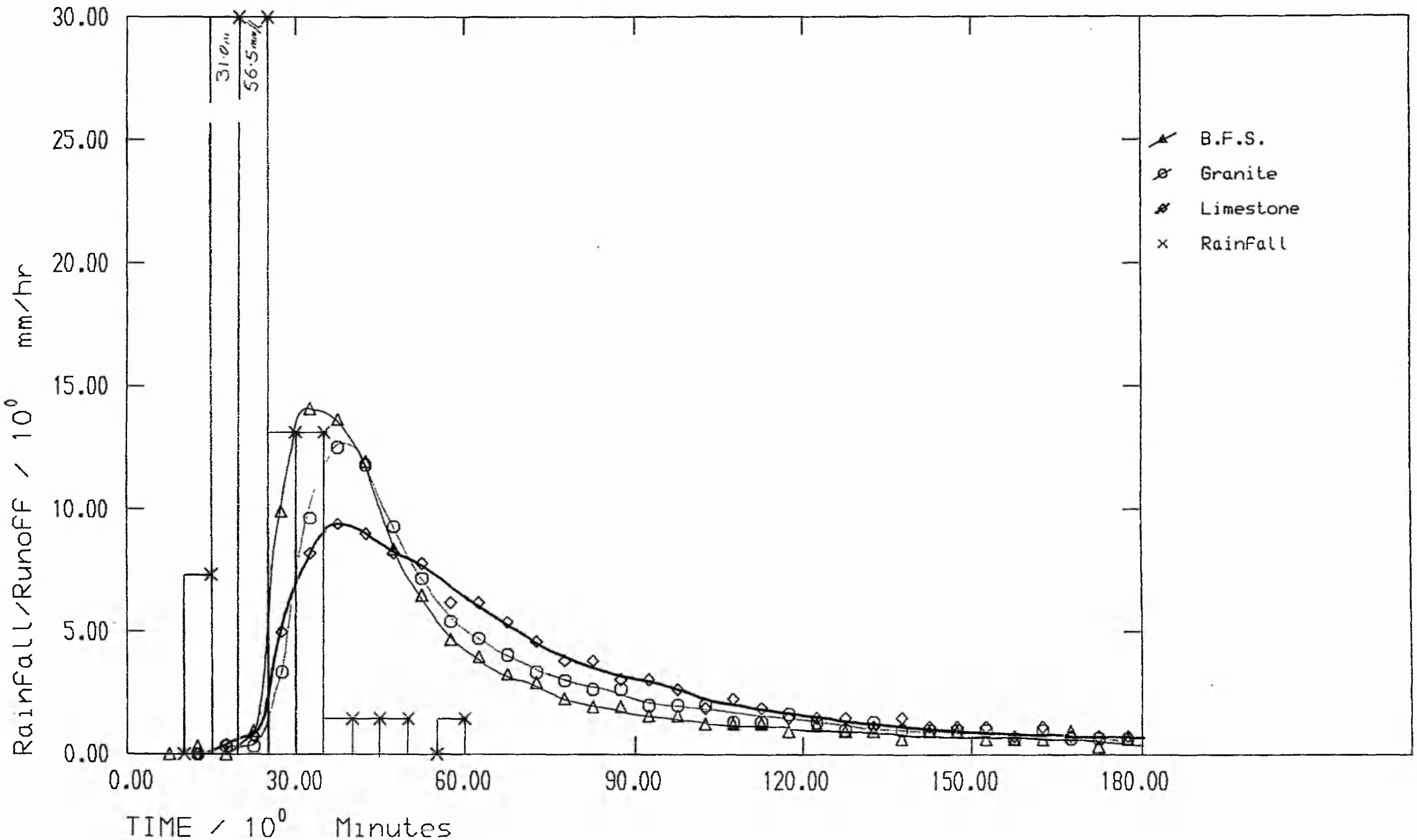
Rainfall/Runoff Plot. Event No. 8107c.
Start Time 07:15hrs. Total Rainfall 6.7mm



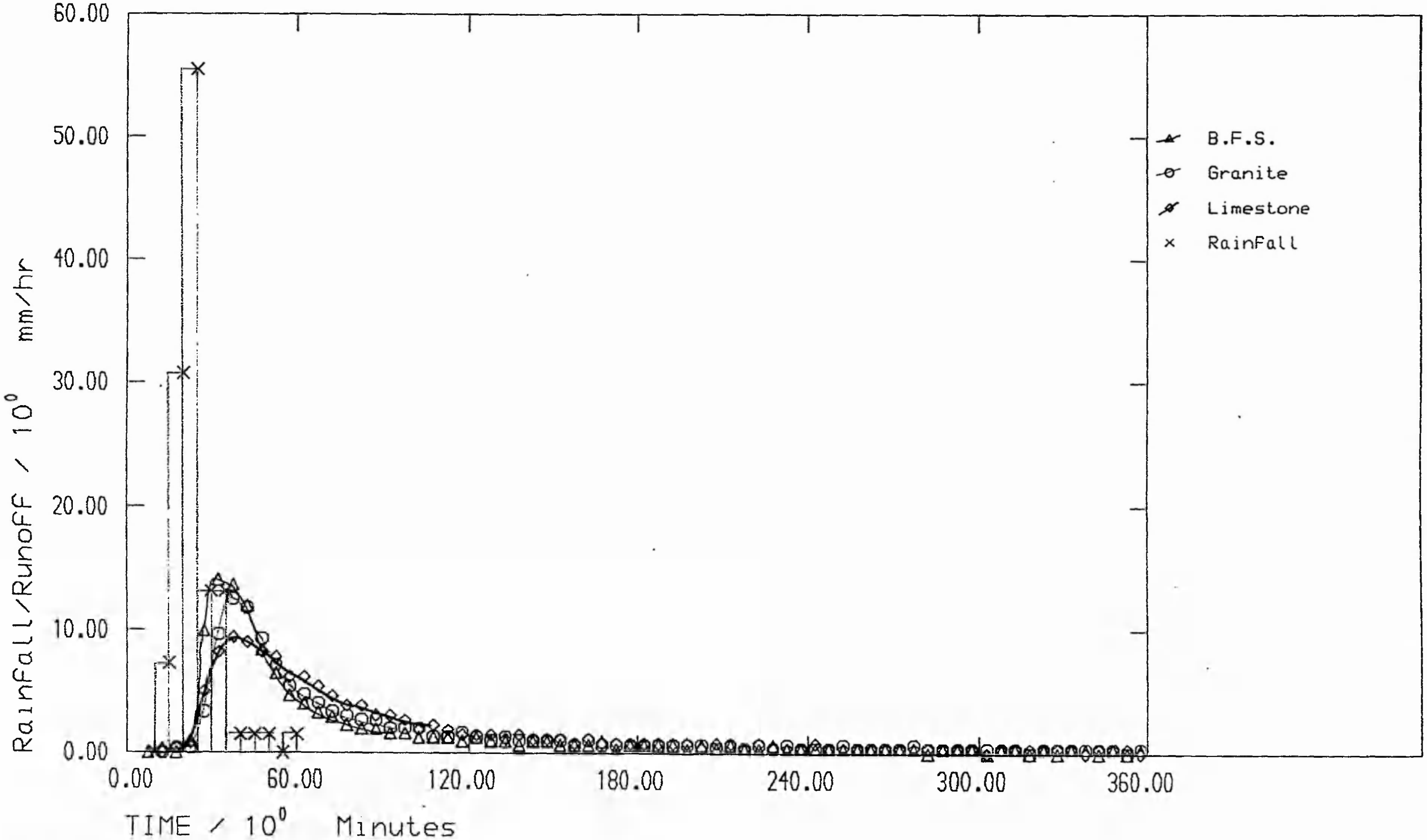
Rainfall/Runoff Plot. Event No. 8125c.
Start Time 11:30hrs. Total Rainfall 4.7mm



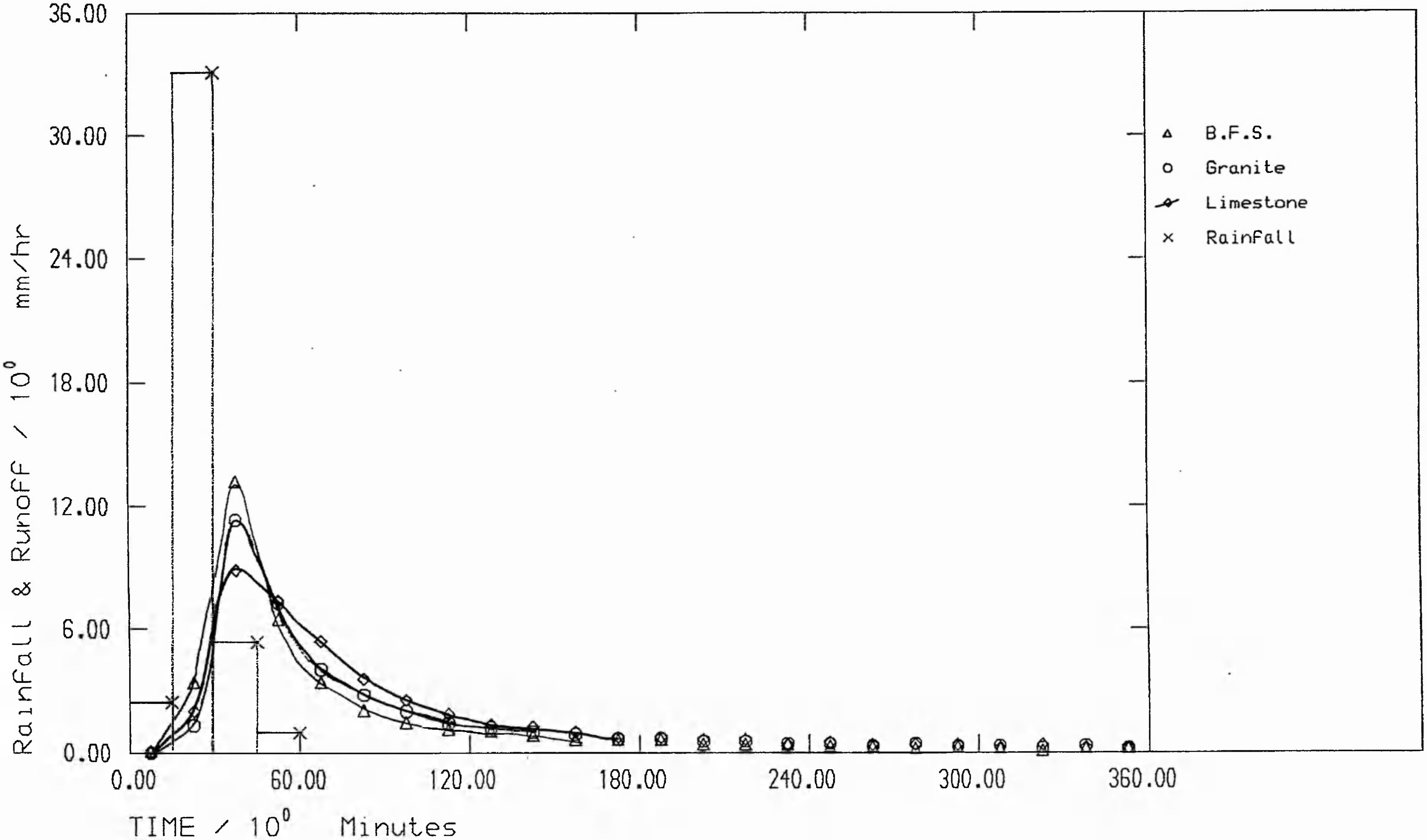
RAINFALL/RUNOFF PLOT. Event No. 8178c.
Start Time 02:15hrs. Total Rainfall 11.0mm



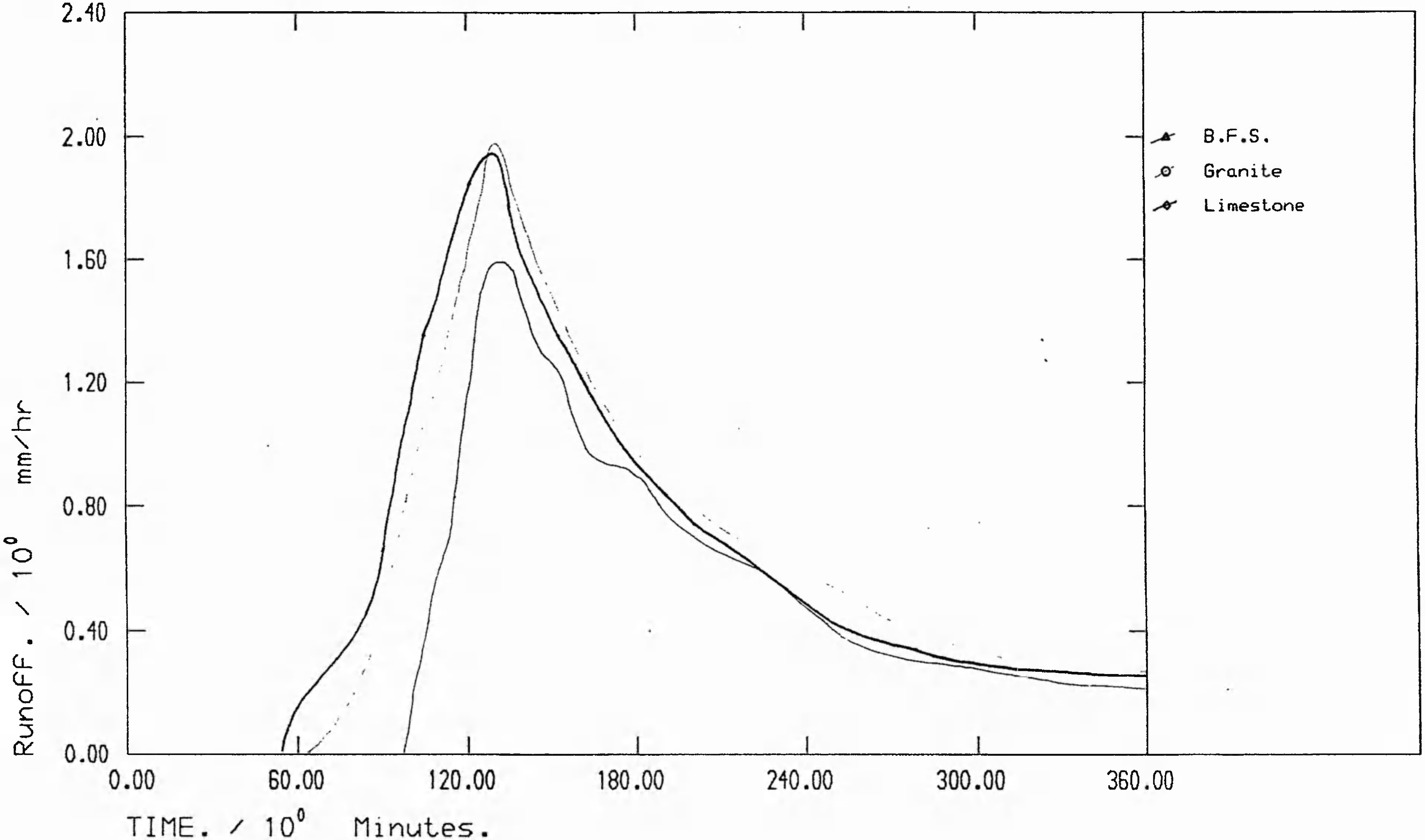
RAINFALL/RUNOFF PLOT. Event No. 8186b.
Start Time 13:15hrs. Total Rainfall 10.5mm



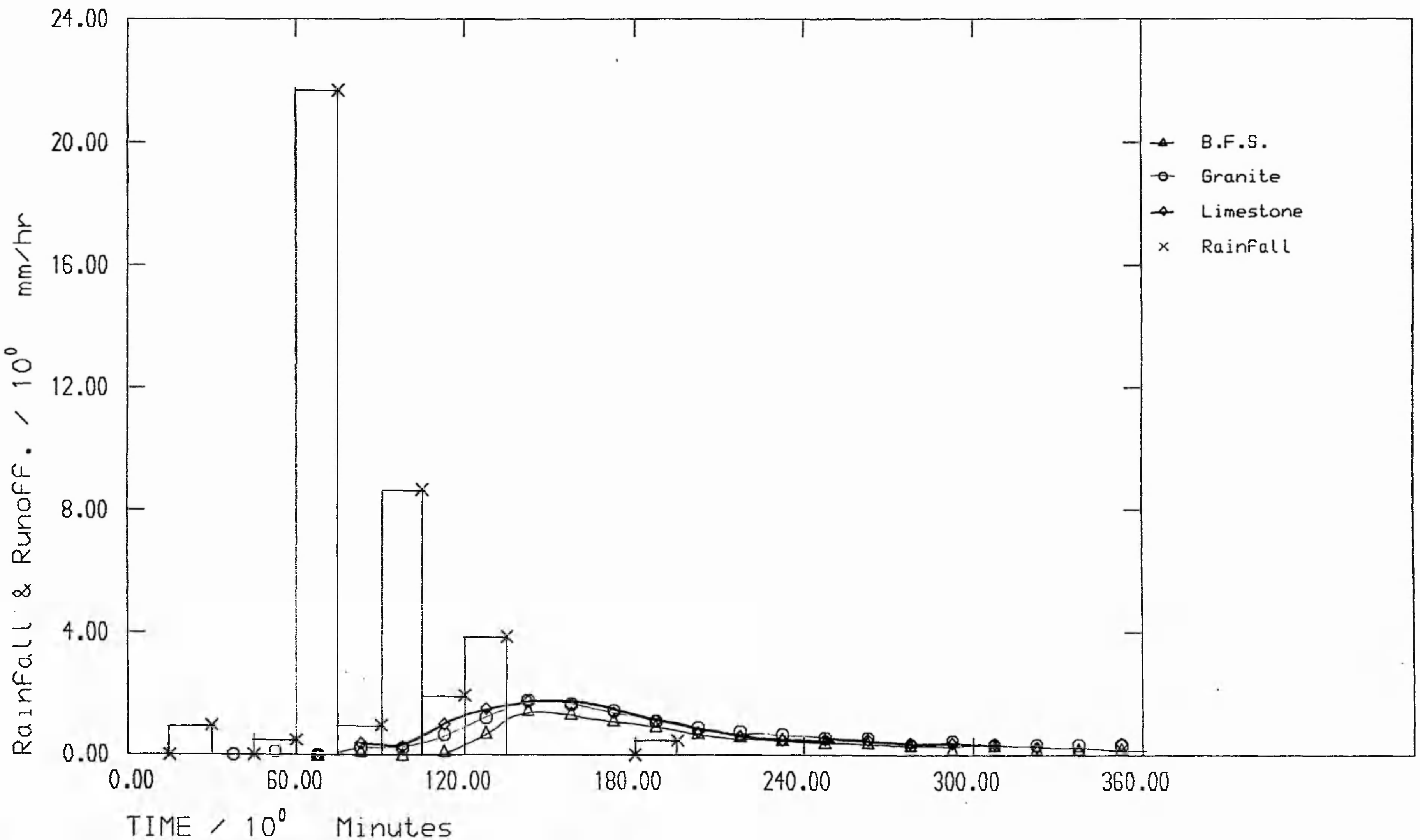
RAINFALL/RUNOFF PLOT. Event No. 8186b.
Start Time 13:15hrs. Total Rainfall 10.5mm



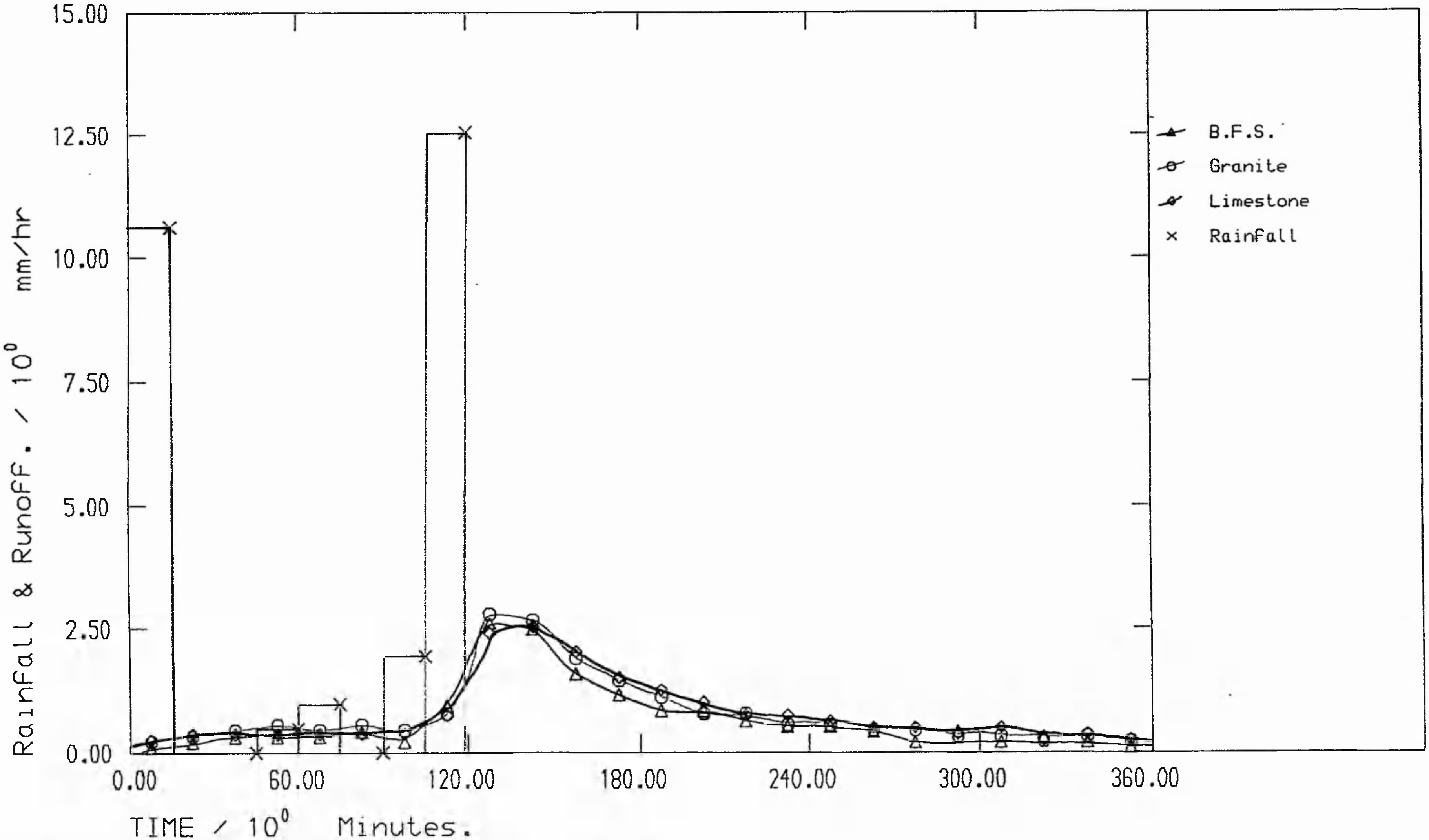
RAINFALL/RUNOFF PLOT. Event No. 8186c.
Start Time 13:15hrs. Total Rainfall 10.5mm



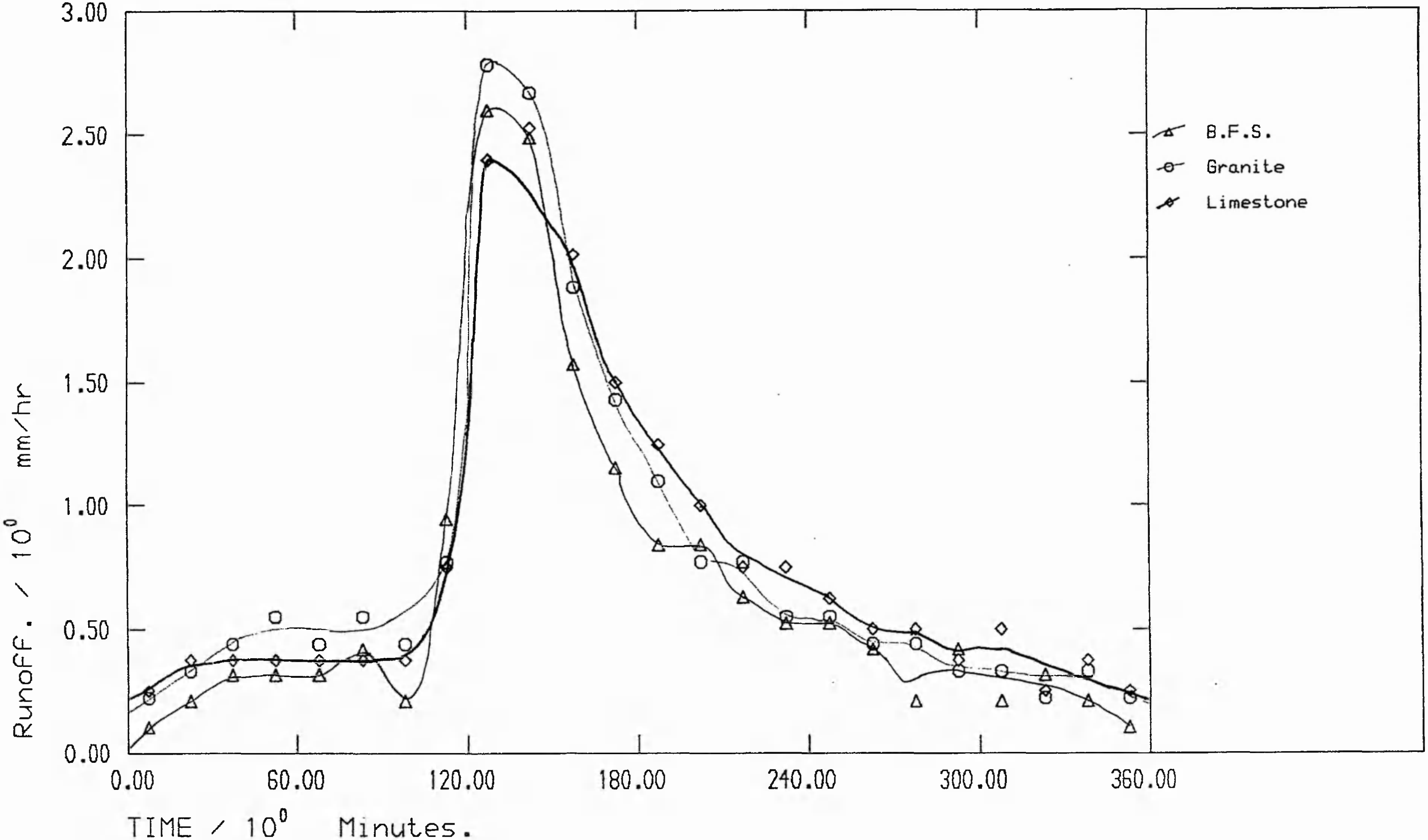
RUNOFF PLOT. Event No. 8187b.
Start Time 15:15hrs. Total Rainfall 9.4mm.



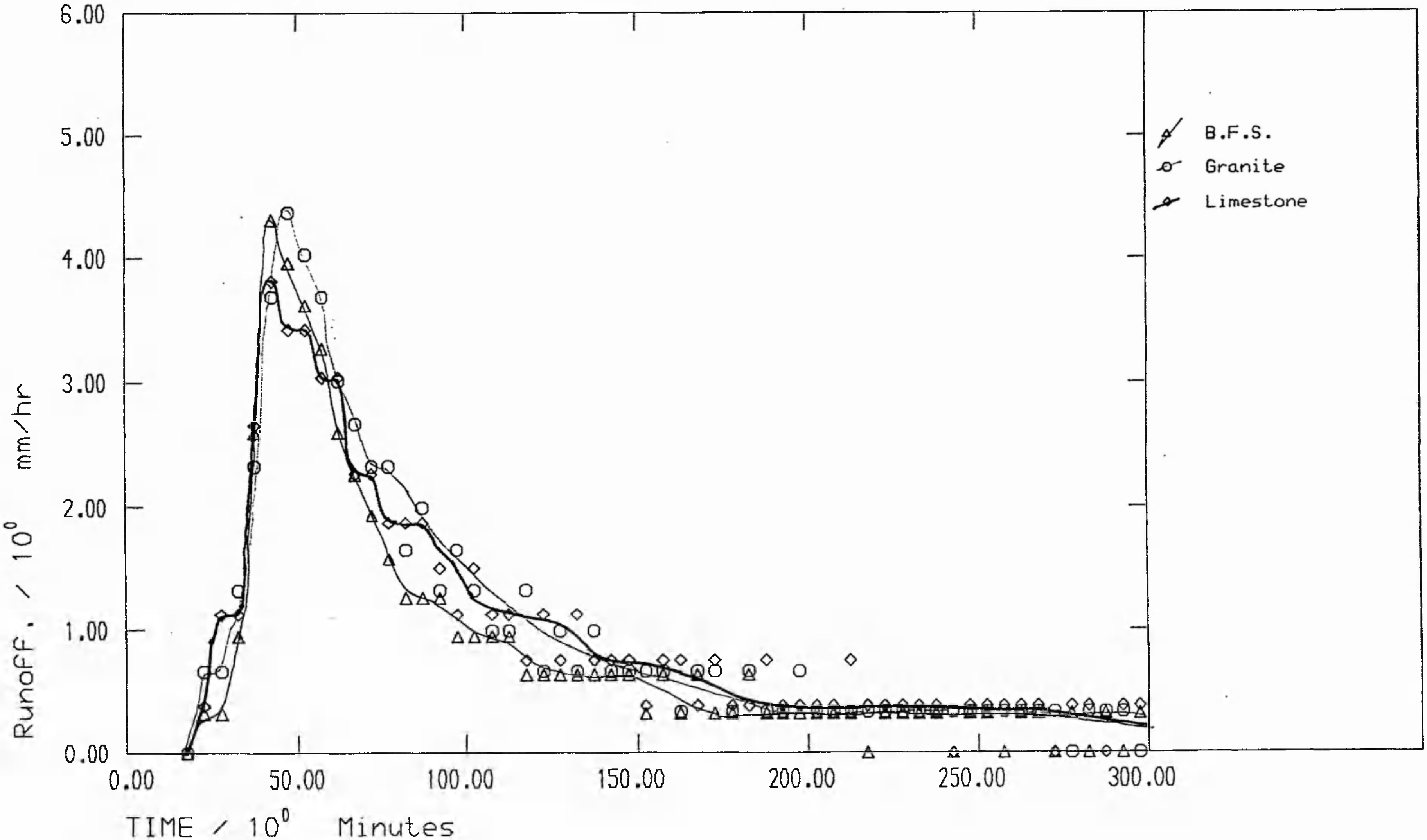
RAINFALL/RUNOFF PLOT. Event No. 8187c.
Start Time 15:00hrs. Total Rainfall 9.4mm.



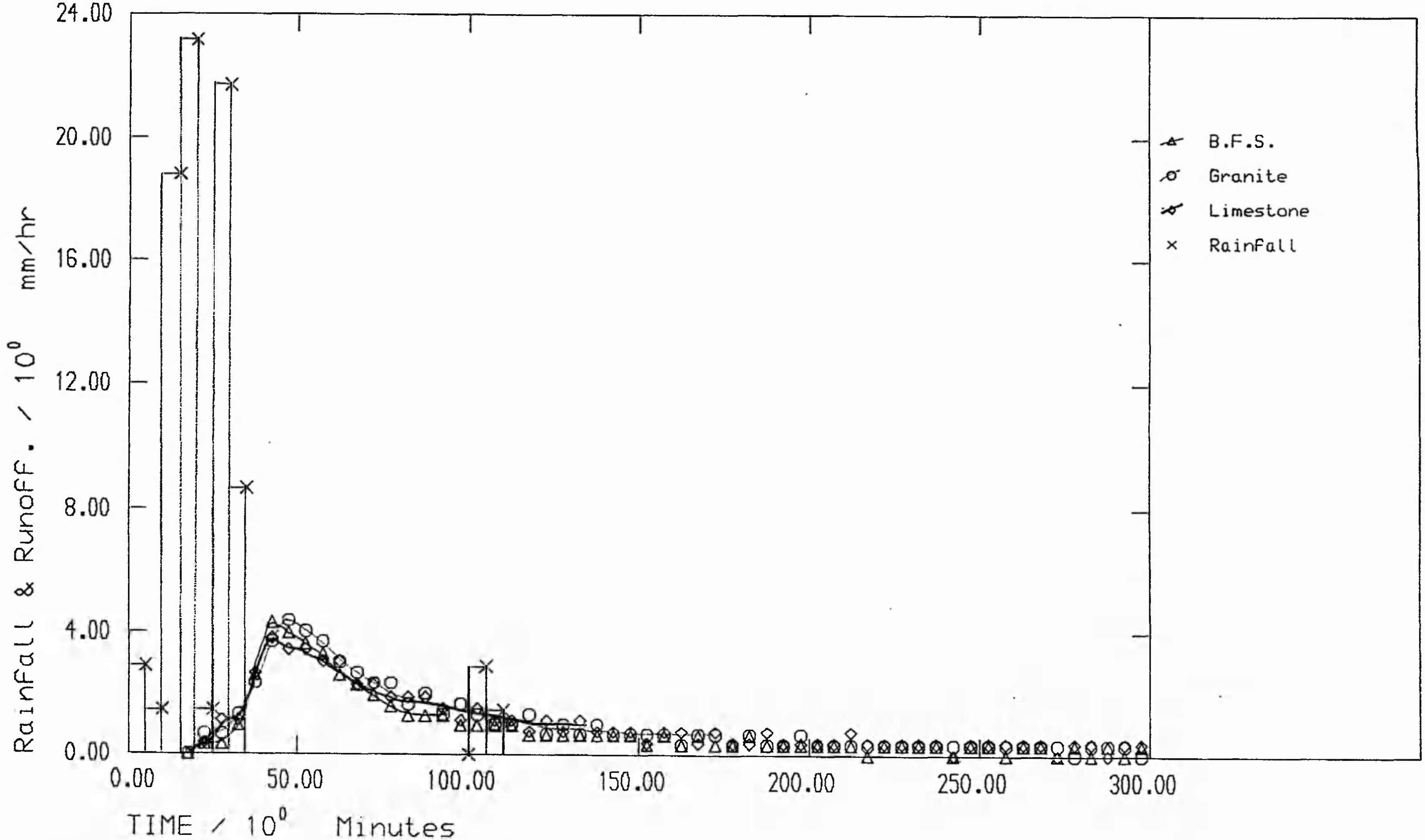
RAINFALL/RUNOFF PLOT. Event No. 8188F.
Start Time 17:30hrs. Total Rainfall 6.6mm.



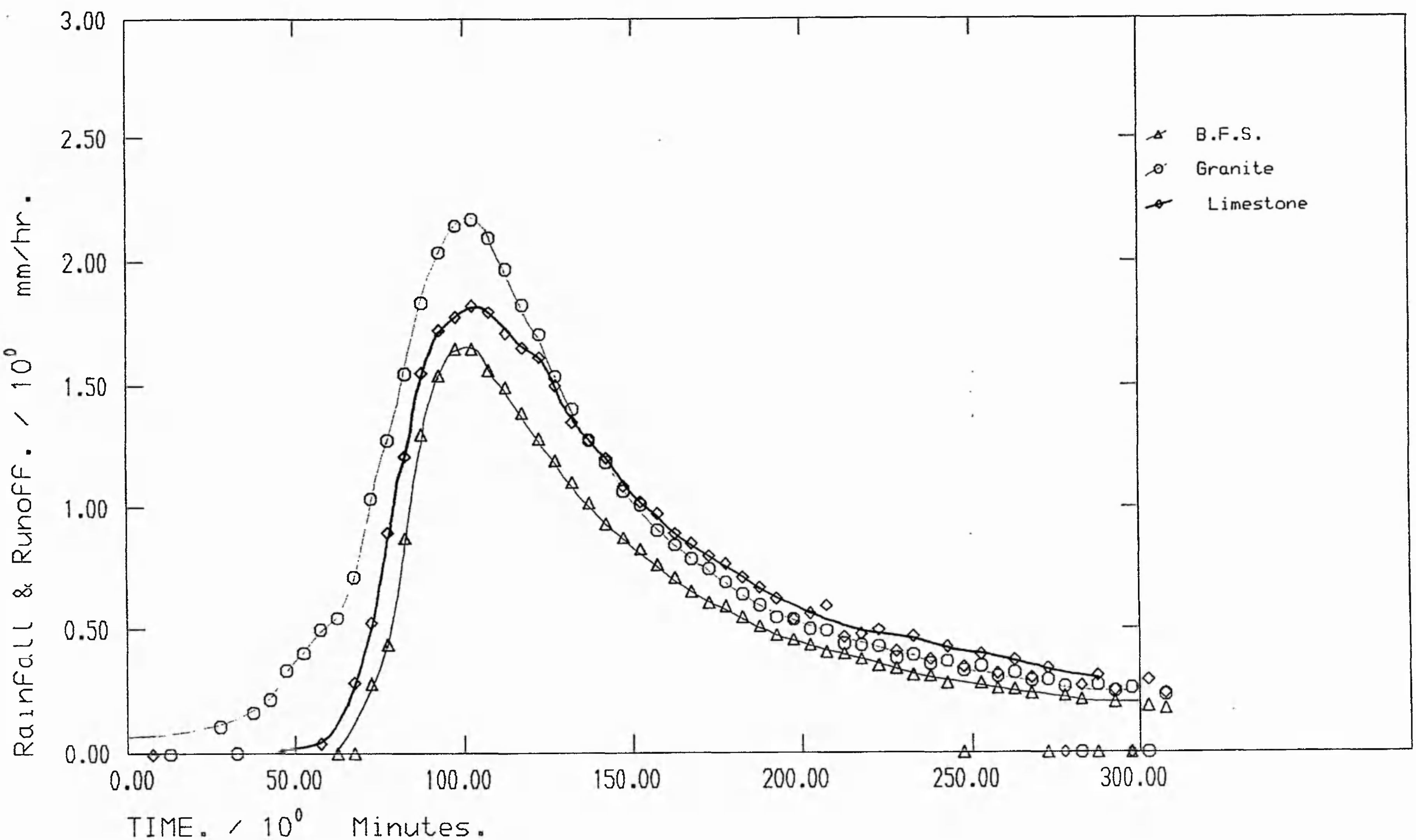
RUNOFF PLOT. Event No. 8188F.
Start Time 17:30hrs. Total Rainfall 6.6mm.



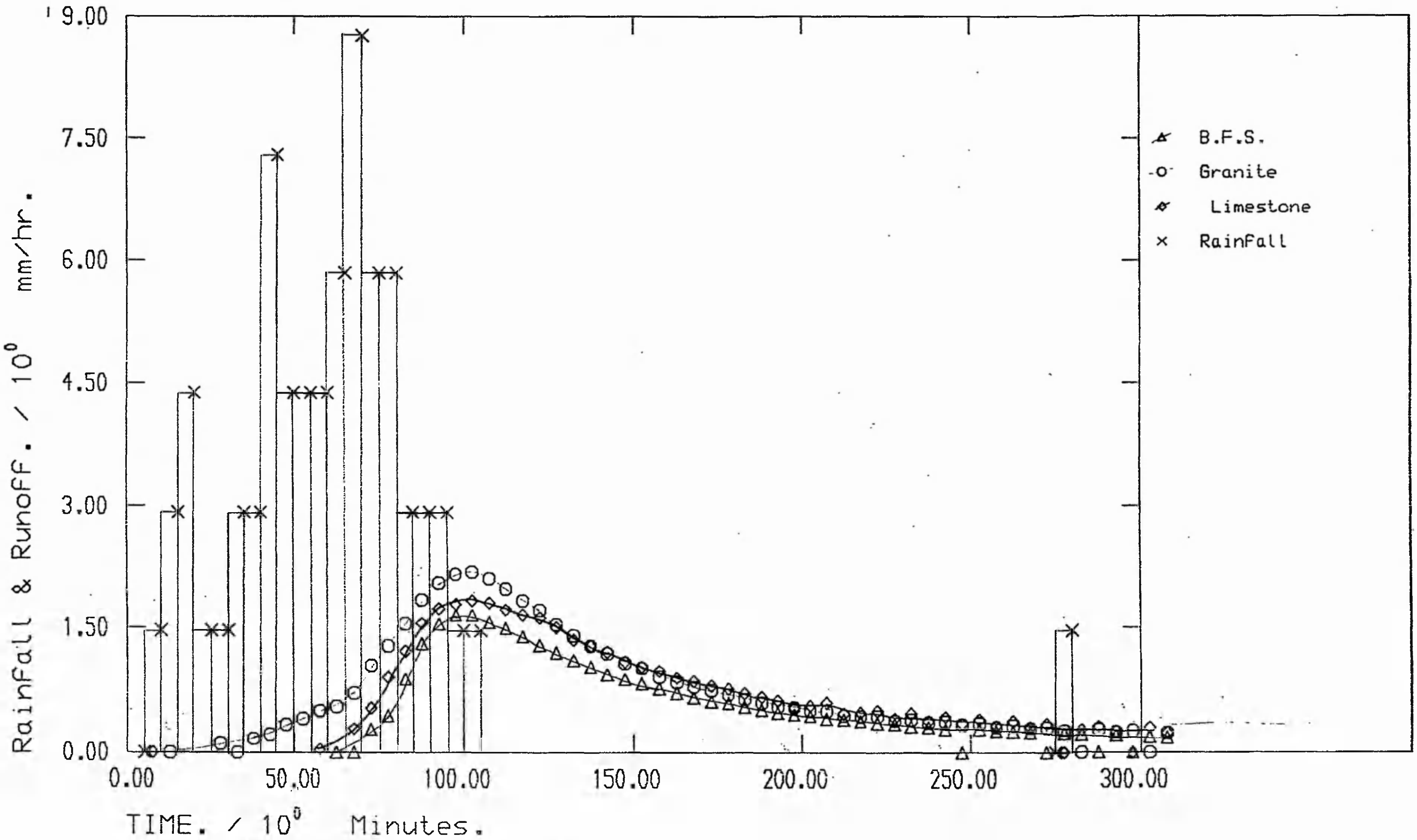
RUNOFF PLOT. Event No. 8189b.
Start Time 17:45hrs. Total Rainfall 6.9mm.



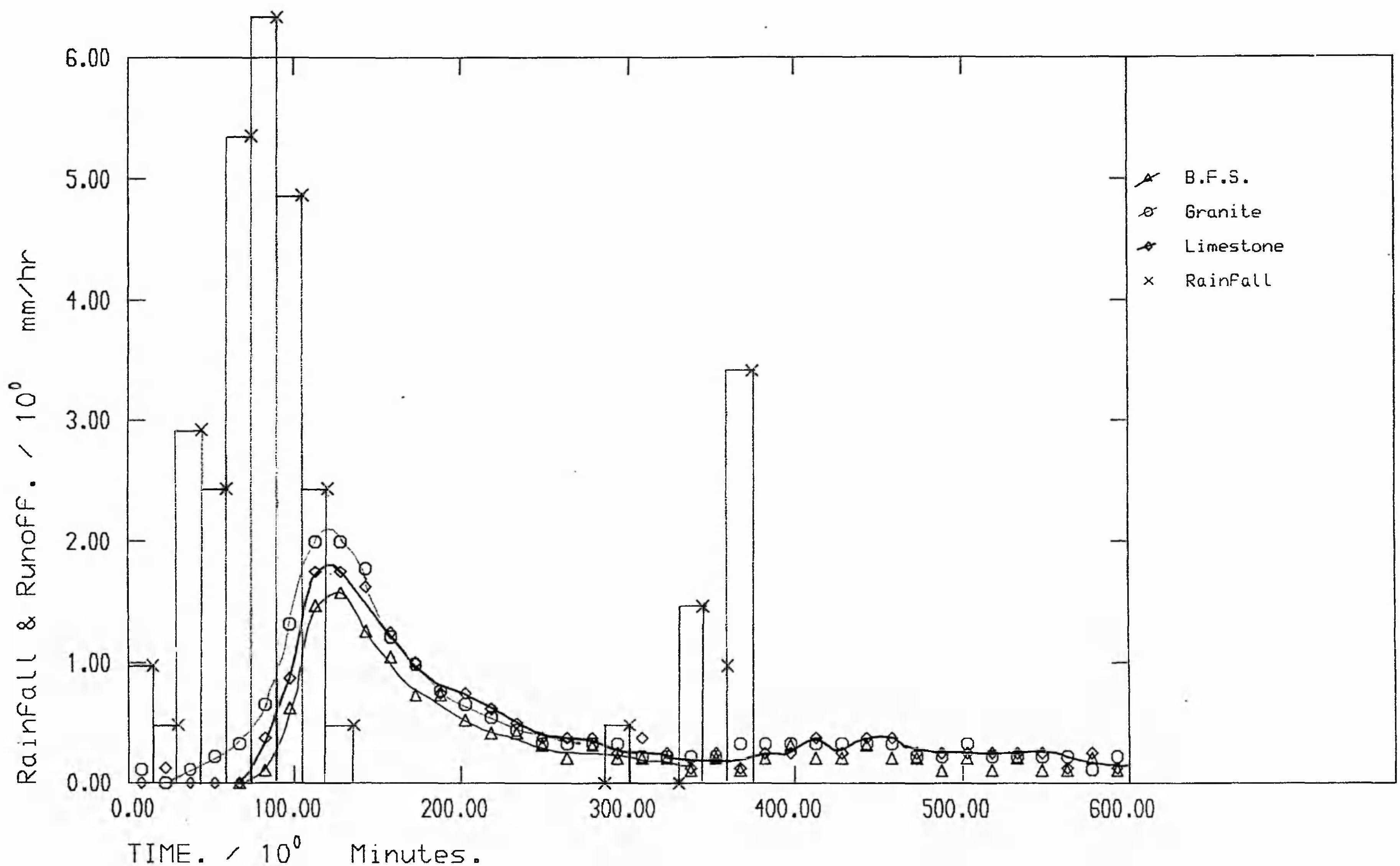
RAINFALL/RUNOFF PLOT. Event No. 8189b.
Start Time 17:45hrs. Total Rainfall 6.9mm.



RAINFALL/RUNOFF PLOT. Event No.8195b.
Start Time 00:45hrs. Total Rainfall 6.3mm.

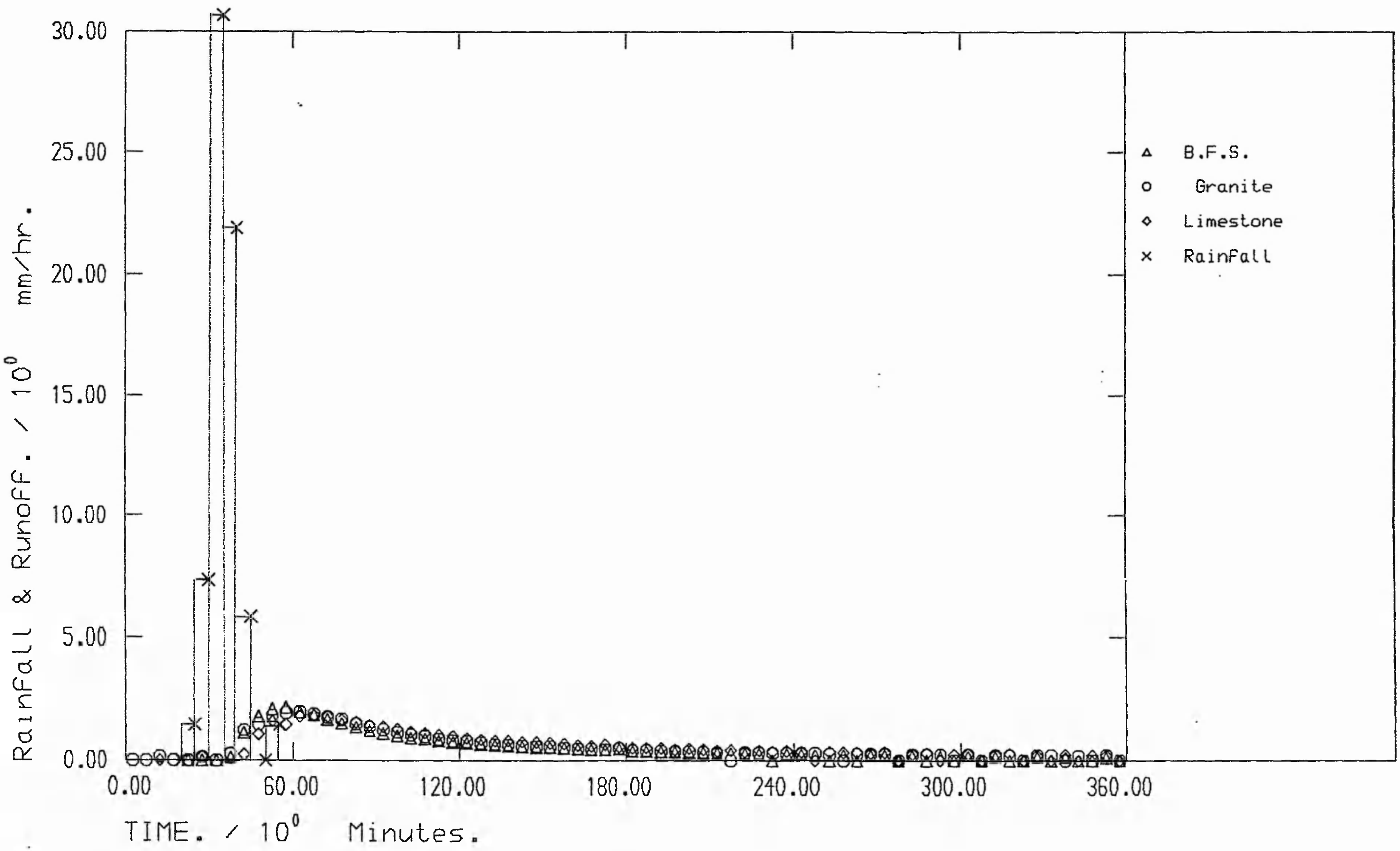


RAINFALL/RUNOFF PLOT. Event No.8195b.
Start Time 00:45hrs. Total Rainfall 6.3mm.

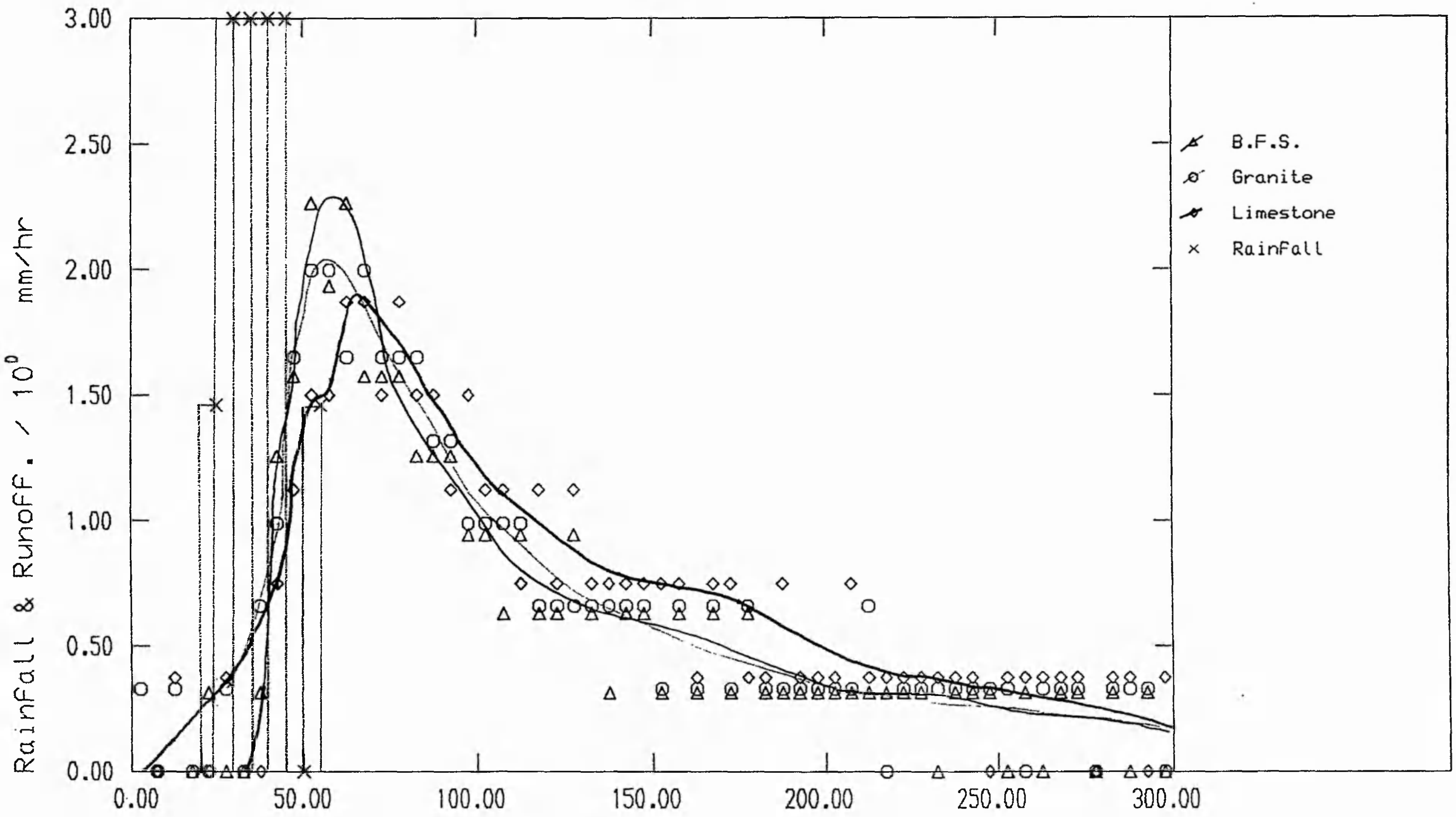


RAINFALL/RUNOFF PLOT. Event No. 8195c.

Start Time 00:30hrs. Total Rainfall 6.6mm.



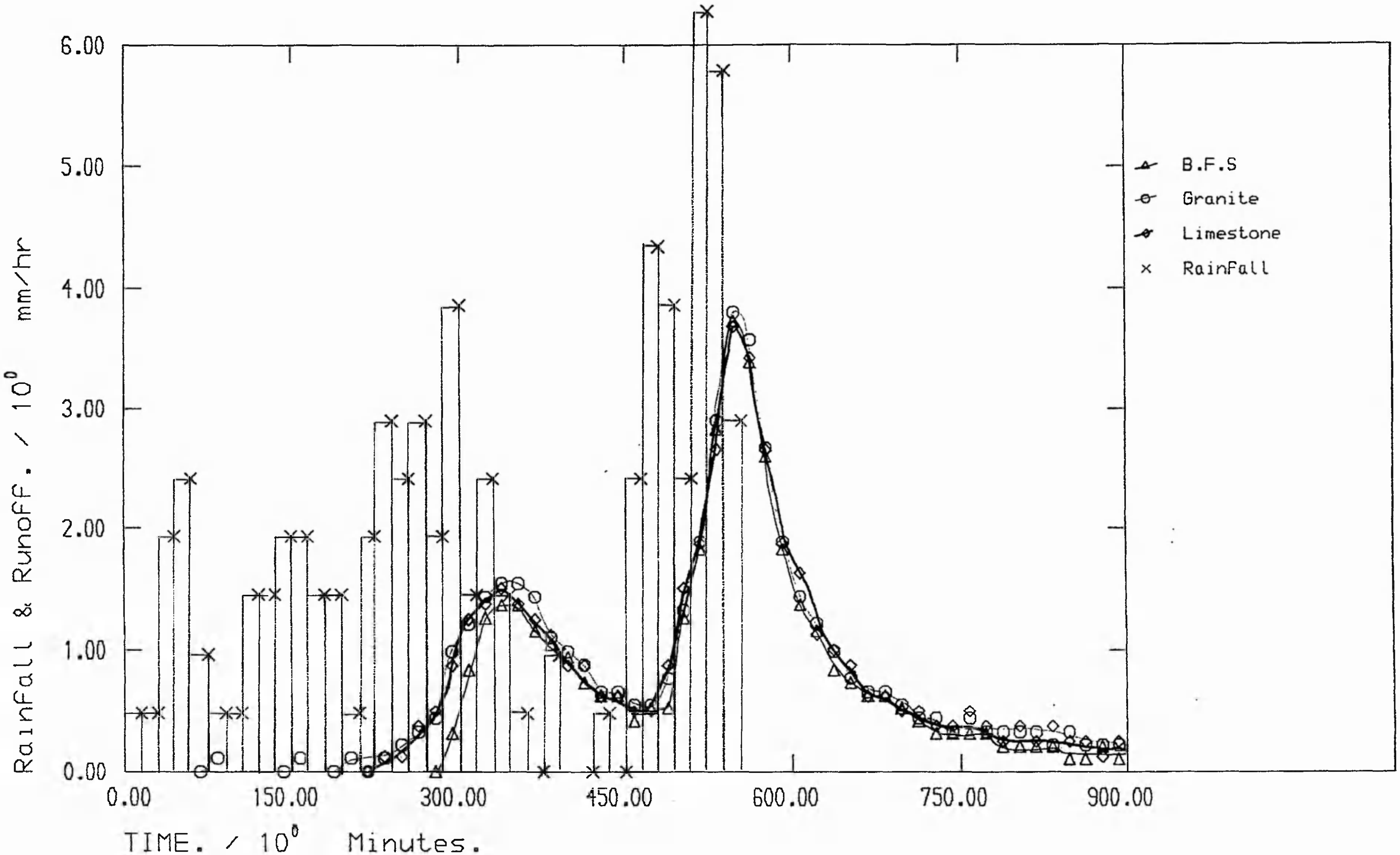
RAINFALL/RUNOFF PLOT. Event No. 8195e.
Start Time 12:00hrs. Total Rainfall 5.7mm.



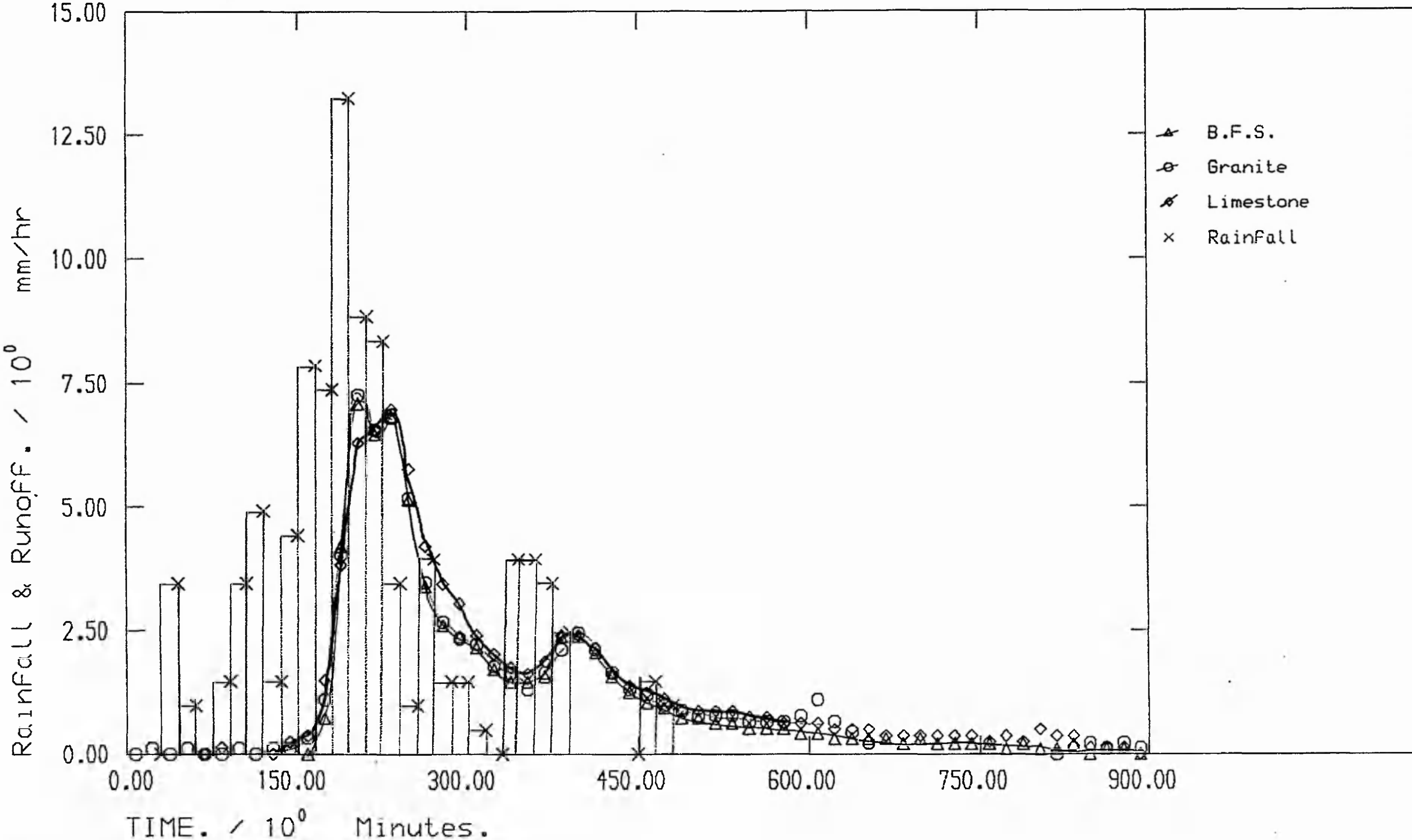
TIME. / 10⁰ Minutes.

RAINFALL/RUNOFF PLOT. Event No. 8195e.

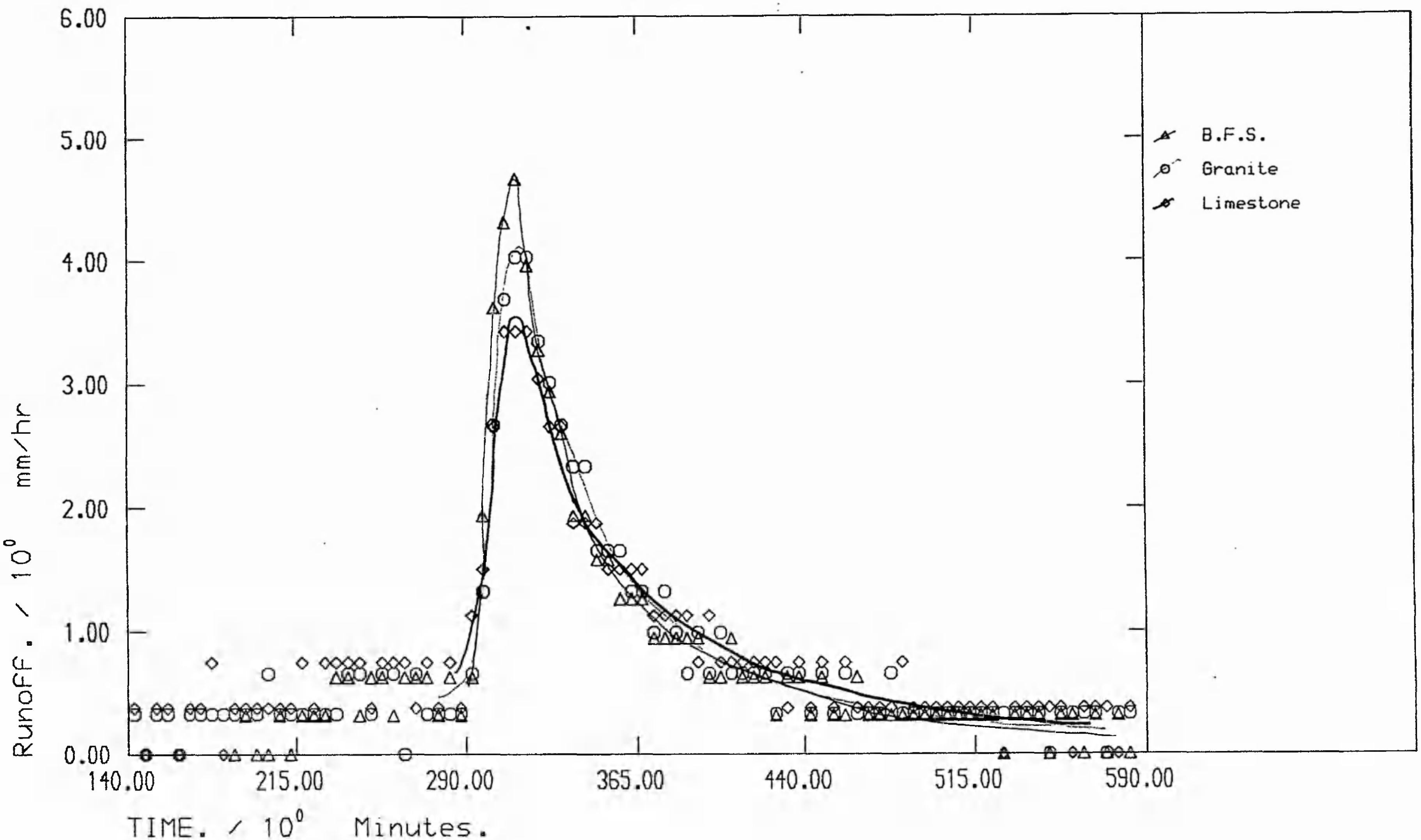
Start Time 12:00hrs. Total Rainfall 5.7mm.



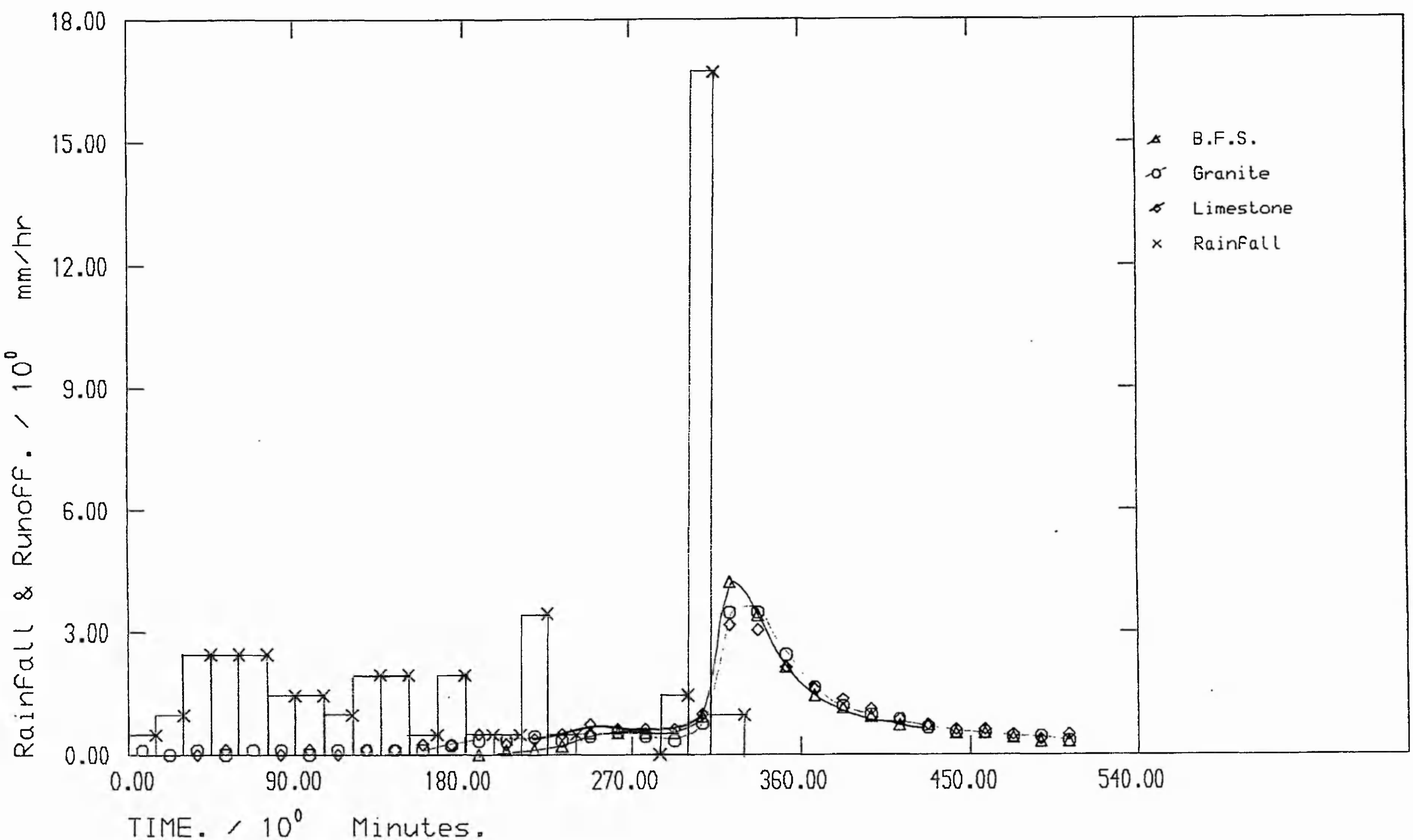
RAINFALL/RUNOFF PLOT. Event No. 8198c.
Start Time 15:45hrs. Total Rainfall 17.0mm.



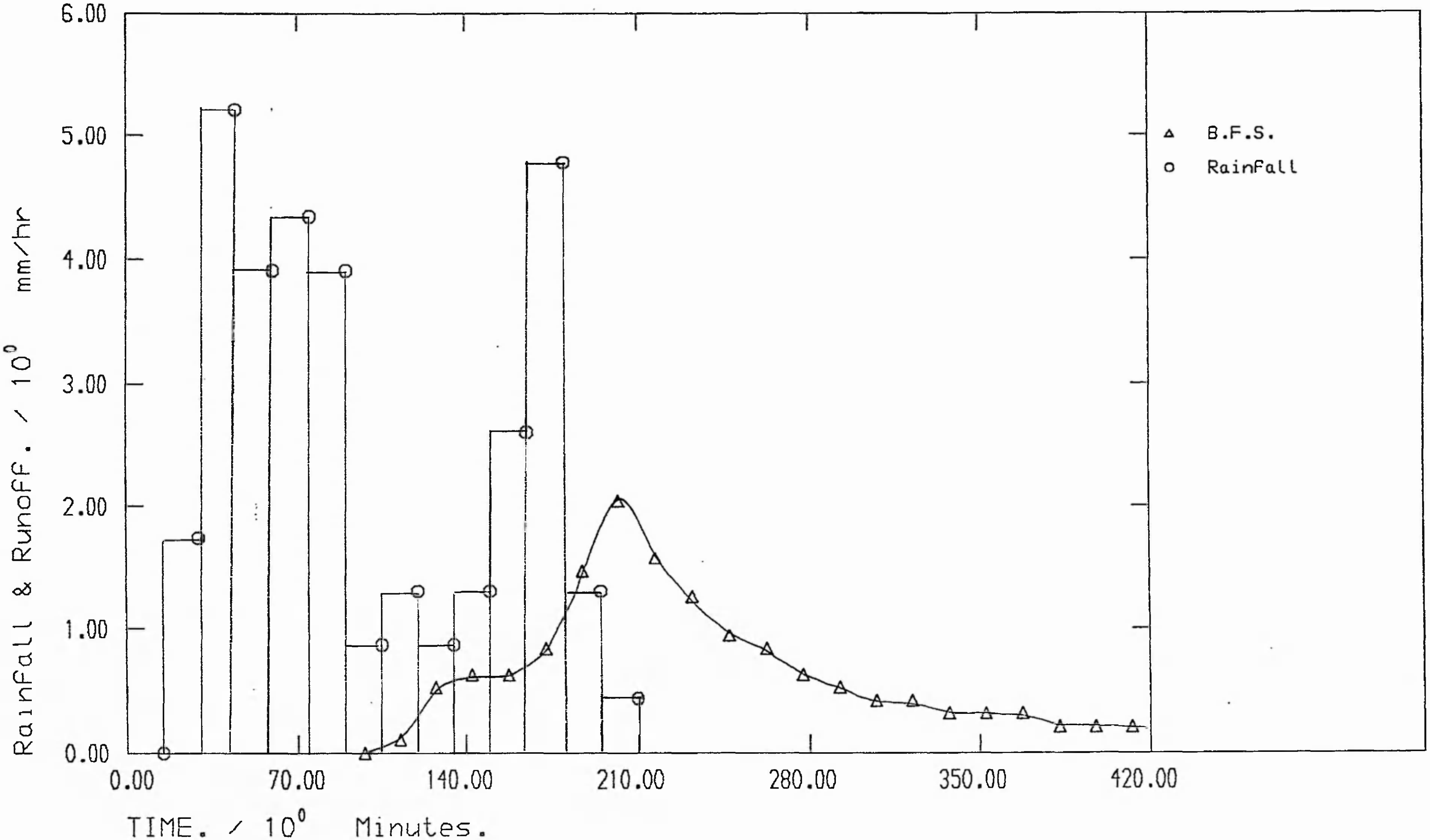
RAINFALL/RUNOFF PLOT. Event No. 8203F.
Start Time 20:00hrs. Total Rainfall 23.5mm.



RUNOFF PLOT. Event No.8204b.
Start Time 01:05hrs (day 205). Total RainFall 10.7mm.



RAINFALL/RUNOFF PLOT. Event No. 8204c.
Start Time 22:30hrs. Total Rainfall 10.7mm.



RAINFALL/RUNOFF PLOT. Event No.8241c.
Start Time 07:00hrs. Total Rainfall 8.1mm.