THE PERFORMANCE OF CERTAIN COMBINED SEWER OVERFLOWS WITH STORAGE

by

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I certify that this thesis is the true and accurate version of the thesis approved by the examiners.

Signed..[redacted].....
(Director of Studies)

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The work is dedicated to my wonderful parents.

ABSTRACT

The provision of storage at overflows is a commonly used technique for the reduction of pollution from combined sewer systems. Field data were gathered at three combined sewer overflow sites during dry weather and a wide range of high flow events. The overflows incorporated off-line storage which at two sites took the form of rectangular partitioned tanks and at the third was a twin hydrodynamic separator installation. Conventional flow measurement and small bore sampling equipment was employed together with a prototype Gross Solids Sampler (GSS) manufactured by the UK WRc, and visible solids interception devices developed by the author and termed Trash Traps.

A relationship was developed for the variation of visible solids during dry weather flow at the inlet to one site based on the GSS results. Good correlation was found with suspended solids concentrations allowing the relationship to have wider applicability.

Retention of particulate matter during high flow events was found to be more dependent on volumetric considerations than on the treatment provided by the storage. It was found that for all sites studied the measure of pollutant separation at each installation, the treatment factor, did not vary significantly from unity. The Trash Traps provided a method of distinguishing between the performance of the overflows utilising the visible solids intercepted and the degree of blinding of the Traps.

It was concluded from the GSS results that the gross solids arriving at the overflow sites had the same movement characteristics as the type C sediment which is usually found in sewer inverts. A chart which provides a basis for a differentiation between combined sewer categories was prepared. This was developed from the rate of gross solids movement and on the average flow during high flow events. The nomograph showed a clear distinction between a collector and a trunk sewer site and included antecedent dry period as a significant component.

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Glossary of Terms

Average Event Flow Average flow during GSS run.

Baseflow Sewer flow, including infiltration, on days with less than 0.5mm rain.

Blinding Fine sewage solids covering mesh of Trash Traps after removal of visible solids.

Bypass Pipe allowing dry weather flow to pass by an installation.

Combined Sewer Overflow (CSO) A structure to relieve excess flow loading from a sewer.

Continuation Flow Flow passing to treatment during overflow operation.

COPA Sack Open mesh sack made from woven polypropylene. Weave size may vary. See Plate 6.6.

Combined Sewer Sewer carrying both foul and storm sewage.

Dry Weather Flow (DWF) Sewer flow on days with less than 0.5mm rain. Little or no infiltration is assumed.

Epic Sampler Small bore sampler.

Event Mean Concentration Average concentration of pollutant over a high flow event.

Event Proportional Volume Ratio of event flow volume to the average of all volumes measured at that site.

First Foul Flush (FFF) Increased concentration of pollutants at the start of certain high flow events

Flow Survey Monitor Device measuring and logging flow levels and velocities.

Flow Split A measure of the retention of flow volume within the sewer system during a high flow event

Formula A Theoretical flow at which overflow commences at a CSO.

Hydrobrake A vortex flow control device.

Installation CSO, storage and connecting pipes at a site.

Gross Solids Solids greater than 6mm in two dimensions found in a sewer.

Gross Solids Sampler (GSS) Apparatus using pump, and COPA sacks constructed to sample for gross solids.

GSS Concentration Concentration of gross solids derived using GSS.

GSS Load Rate The mass of gross solids captured by GSS expressed as a rate over the duration of a GSS run.

GSS Ratio The ratio of the mass of gross solids captured by the GSS to the mass of TSS in the same duration.

Pollution Separation Efficiency A measure of the ability of an overflow or installation to separate solids from a flow when discharge or spill is actually taking place.

Retention Time The theoretical time taken to fill a storage volume at a given flowrate (normally DWF).

Ring Bag Test A test used to determine the passage of gross solids during dry weather flow.

Scum Board A baffle to prevent floating solids from passing over a weir.

Sediment Accumulations of sewage solids on the bed or sides of sewers or sewer appurtenances.

Sediment Type A method of describing the characteristics of sediments.

Separate Sewer A sewer which carries only foul sewage.

Sewage Solids Any material carried in sewage.

Spillflow Flow from an overflow installation to a watercourse.

Storm Sewer A sewer which carries only surface runoff.

Throttle Pipe A length of pipe, normally at an overflow used to control the rate of continuation flow.

Total Efficiency A measure of the retention of pollutant mass within a sewer system by an overflow or installation during a high flow event.

Trash Trap A device developed by the author to evaluate the performance of overflow installations.

Treatment Factor A measure of the ability of an overflow or installation to separate solids from a flow over the course of a high flow event.

Venturi Flume A flow control device requiring free surface flow.

Visible Solids Gross solids which are identifiable sewage in origin following visual examination, effectively paper and plastic strips.

List of Symbols and Abbreviations

ADWP	Antecedent dry period
BOD	Biochemical Oxygen Demand
C_c	Continuation flow concentration
C_i	Inflow concentration
C_o	Overflow concentration
C_s	Spillflow concentration
D	Diameter (mm)
D_{min}	Theoretical diameter for overflow design $D_{min} = 0.815xQ^{0.815}$
g/hd	grammes per person
EMC	Event Mean Concentration
EPV	Event Proportional volume
FFF	First Foul Flush
l/hd	litres per person
l/s	litres per second
mg/l	milligrammes per litre
m/s	metres per second
mm/h	millimetres per hour
GSS	Gross Solids Sampler **or** gross solids measured by Gross Solids Sampler
LGSS	load of gross solids
LTSS	load of suspended solids
NSW	net COPA sack weight
PIL	total storm inflow pollutant load
PLR	total storm pollutant load retained
PSL	total storm spill pollutant load
q_c **(also q)**	continuation flow
q_i **(also Q)**	inflow
q/Q	flow ratio
q_s	spill flow
q_t	flow to storage
q_{thr}	throughflow
SAAR	standard annual average rainfall
TIV	total storm inflow volume
TVR	total storm volume retained
TSS	suspended solids concentration
u_o	mean inflow velocity at an overflow
w	terminal settling velocity of a particle

CHAPTER 1 INTRODUCTION

> **For my part I travel not to go anywhere, but to go. I travel for travel's sake. The great affair is to move.**
> R.L. Stevenson Travels with a Donkey

1.1 SCOPE OF THE RESEARCH

The study described in this thesis developed from a three year field investigation in which three combined sewer overflow installations were monitored. The installations each incorporated an on-line diversion structure and off-line storage. The storage comprised partitioned rectangular tanks for two of the CSOs with the other being a hydrodynamic separator. Data were gathered using three distinct methods of sampling for sewage solids. Conventional small-bore samplers and novel solids intercepting devices termed Trash Traps were located at all three installations, and the prototype Water Research Centre (WRc) Gross Solids Sampler was installed at two of the CSOs. The sampling programme was supported in all cases by flow and rainfall measurement.

Analysis of the data has resulted in the production of novel information on the movement of pollutants within sewer systems and at combined sewer overflows. The volume of storage installed at a site was found to be the primary factor in retaining pollutants within sewer systems. An understanding has been gained on the behaviour of gross solids which include the aesthetically unpleasant visible solids which, once discharged into watercourses, are immediately recognisable as sewage pollution. Methods have been developed for predicting rates of discharge of gross and visible solids during both dry and wet weather flows. Strong evidence was found to suggest that the gross solids were subject to the same hydraulic influences as other sewer sediments.

1.2 RESEARCH AIMS AND OBJECTIVES

The research had the following principal aims:-

i) To gather information from field studies on pollutant behaviour at combined sewer overflows, and in particular on the behaviour of gross and visible solids.

ii) To assess the operation of the WRc Gross Solids Sampler (GSS), sampling during both dry weather and combined sewer event flows.

iii) To evaluate the operation of Trash Traps in identifying the performance of combined sewer overflows.

iv) To explore the operation of hydraulic controls at certain of the overflows.

The general objective of the research was to advance knowledge of the performance of combined sewer overflows, particularly with respect to the retention of gross solids. The specific objectives were as follows:-

i) To establish whether Trash Traps could be used to evaluate the performance of combined sewer overflows, and to develop a method for the interpretation of Trash Trap results based on the retention of small sewage particles and visible solids;

ii) To establish the same for the WRc gross Solids Sampler;

iii) To develop a Gross Solids Rate chart from which a classification of catchments by their wet weather gross solids production potential may be inferred;

iv) To demonstrate that a dry weather period of 24 hours duration is highly significant in the accumulation of gross solids;

v) To produce evidence that gross solids are subject to the same hydraulic influences as type C sewer sediments;

vi) To derive values for the performance indicators Total Efficiency and Treatment Factor for the installations studied; and,

vii) To show that Flow Split is the most appropriate measure available for comparing the performance of combined sewer overflows with storage.

1.3 LIMITATIONS

The underlying tenet governing the research was that it should be based on fieldwork, theoretical approaches being unrealistic. The approach was pragmatic - to gain the best information possible using the resources available from the installations during fixed periods in time when the sampling programmes were being carried out. Within the overall study period from January 1989 to February 1992 equipment was installed for shorter durations at each site and all data obtained were restricted to these periods.

The advantage of being able to claim that the data were derived from observed events is countered by the difficulty of being able to ascribe particular observations to particular conditions. This highlights the advantage of model studies where one set of criteria may be maintained for considerable durations, and each parameter may be altered in specific, predetermined ways. Control over inputs to the overflows was not possible in this study and the interpretation of the data and resultant conclusions reflect this limitation.

In addition to the pollutant loads, flowrates and all other data being site specific, there was also the problem posed by fixed point sampling by both the small-bore Epic and the Gross Solids Sampler. Sampling points were chosen to be as representative of conditions within the flow streams as possible, however, there was no means available of carrying out checks of variations within the flows and it has been assumed that samples were indeed representative. Floating solids could only sampled by the Trash Traps.

Every effort was made to relate the determinations herein to previous studies. The ability to make comparisons was limited by a decision early in the study to restrict the physico-chemical analysis of every small bore sample to suspended solids only and more complete analyses were restricted to limited numbers of samples. This limitation is discussed further in sections 3.2.3 and 4.5.

1.4 THESIS OUTLINE

A review of current knowledge of sewage related pollutant production and retention is presented in Chapter 2. Definitions of pollutant concepts and an examination of their measurement are followed by a review of suitable and available pollution separation technology, comparisons being made between previous work and this research wherever appropriate.

The background to and basis of the field investigations are presented in Chapter 3. The study catchments and the extensive rehabilitation works undertaken by the sewerage authority are described. The items of equipment mobilised, together with their methods of operation and faults are reviewed. Finally a description of each of the study sites is included.

In Chapters 5, 6 and 7, the results from the field studies of each of the major items of equipment are presented and evaluated. The chapters deal in turn with the Trash Traps, the Gross Solids Sampler and the small-bore sampler results. The three sampling methods have been separated because of their different methods of operation, definitions of events and applicability of results. In each chapter, the results, conclusions drawn, and suggested future work for each of the methods are presented.

The practical application of the findings together with recommendations for future research are contained in Chapter 8. In Chapter 9 the principal conclusions of the research and the accuracy of the results are reviewed.

Seven Appendices are included containing references, field data, relevant papers by the author, further information on combined sewer overflow operation and an assessment of the accuracy of the flow data.

1.5 PRINCIPAL RESULTS

The advancement of knowledge is demonstrated in the thesis to be in four specific areas:-

i) Studies using the Gross Solids Sampler at one combined sewer overflow site enabled the daily variation of gross solids during dry weather flow to be determined. This result, presented as Figure 6.3 shows that there was close correspondence between the variations of gross and of suspended solids in the sewage. A relationship was developed between the load of GSS and that of TSS passing the observation point in dry weather. This relationship (Equation 6.4) is presented for use with other predictive methods.

ii) A chart has been developed (Figure 6.8 & 6.9) which differentiates with a high degree of reliability the gross solids production of two different types of catchments, one being a collector sewer catchment, and the other a trunk. This chart has been based on the rate of gross solids production over events. This rate is considered to be a critical factor in differentiating catchments. A consistent and further differentiation was also derived on the basis of ADWP.

differentiation was also derived on the basis of ADWP. Durations greater than 24h were found to lead to no increased accumulations than shorter dry weather periods.

iii) A technique is presented for comparing overflow and spill discharges based on measurements from Trash Traps. This technique has shown that there was no difference in performance between two of the overflows. The third overflow studied, the hydrodynamic separator, gave significantly different performance, with less gross solids discharged per unit of suspended solids.

iv) The small bore sampler results showed that little treatment took place within either the overflows or tanks, and performance measurement by volume was the only relevant method. Treatment Factors for suspended solids were found to be near unity for all overflow and tank combinations and one overflow and storage installation, together with the hydrodynamic separator had average values for treatment factor of 1.12. It has been concluded that the resources required for routine quality performance monitoring at combined sewer overflows are likely to be excessive for routine sampling purposes.

1.6 A PERSONAL REFLECTION ON THE RESEARCH

The work reported in this thesis was some seven years in development followed by three in execution. For many years the subject, principally aspects of the Dunfermline sewer system, had been of great interest to me. The infrastructure was old and of interest from a descriptive point of view (Ashley et al 1986) and in parts complicated (Au Yeung 1990). It consumed much of my time and effort in providing ad hoc answers to a variety of engineering problems. The problems were interesting, the solutions were challenging, the experience was fulfilling, but it was hardly more intellectually challenging than the appropriate application of engineering practice based on past experience and the lessons gained on the way. The work has been a journey of discovery, the thesis a record of what was found: a traveller's logbook.

At the start, in spite of there being many problems to be addressed there was no objective, and without one is a journey worthwhile? Or are temporary experiences, however exciting or fulfilling en route, sufficient? Shackleton, in his epic voyage in an open boat (Lansing 1959) had the aim of navigating through wild seas and mountains to reach safety, but this was only gained once his objective, the South Pole, was abandoned. Life rarely produces such extremes of endeavour as Shackleton experienced, especially in the abandonment of an objective. Equally, the objective may seemingly be so trivial that the journey may appear to be too impossible to be worthwhile. The study of Emperor penguins laying eggs in the middle of an Antarctic winter hardly seems sufficient cause to make the Worst Journey in the World (Cherry-Garrard 1929). Yet three dependable men trekked to make their observations, nearly perishing with the effort, and all this virtually in their time off. A study of the sewerage of South Fife may seem prosaic in comparison and lacking such excitement, but a journey it was and the objectives only became clear after some considerable period of time.

The objectives for the research became clear after legislative changes required significant public investment to be committed to the reduction of pollution from the sewers in the area. A programme of construction of sewage retention tanks was initiated in Dunfermline and at Lochgelly and the determination of the performance of these tanks became the principal research objective. However this was no shining star to follow, nor a south pole to be reached, it was modest engineering for which the questions asked required careful study, the real interest lay in determining what could be found on the way.

CHAPTER 2 DISCHARGE AND CONTROL OF POLLUTANTS FROM COMBINED SEWER SYSTEMS

Discharge (n) Unloading; firing off; omission; release.
Control (n) Function power of directing and directing and regulating.
Pollute (v) Make foul or filthy.
Oxford English Dictionary. (1970)

2.1 CSO POLLUTION - A PROBLEM OF PERCEPTION

Combined sewer overflows are essential for the operation of most sewer systems, discharging excess flows and allowing smaller downstream pipe sizes than would otherwise be the case. Water pollution from such discharges is caused by both particulate and dissolved pollutants and may lead to deoxygenation, increased toxicity, deposition of sediments, aesthetic pollution or increased nutrient loads. Each of these factors, either individually or in combination might lead to an impairment of the water quality as has been reported in a number of studies (eg. Clifforde et al 1990). However, in many locations, and in spite of frequent overflow operation, the ecological effects may be minimal. Working in Switzerland for example, Gujer & Krejci (1987) found no documented evidence of ecological damage in the context of rain events and in Scotland, despite regular discharge of combined sewage, many streams have high classifications (FRPB 1990).

The existing UK river quality classification is biased towards continuous rather than intermittent discharges (Clifforde et al 1990) which in part explains why CSO discharges apparently do not give rise to poorer water qualities than is generally the case. Detailed studies at specific sites invariably show marked local biotic and water quality changes requiring complex explanation. Seager & Milne (1990) and Davis & Parkinson (1990) for example show contrasting approaches to the study of pollution from two overflows based on macroinvertebrate counts and water quality classification respectively. In general, however, as both Mance (1981) and Ellis (1986) report, discharges from combined sewer overflows do not normally result in severe ecological damage to inland waters.

To draw the conclusion that the operation of CSOs is satisfactory would be entirely inappropriate. The aesthetic impact of discharges may be extensive, and sewage discharges can lead to sudden shocks on a watercourse, particularly when storm events result in significant first foul flushes. The impact on the stream biota may be severe as shown by Seager & Milne (1990) but such effects tend to be short lived and not measurable by the routine monitoring programmes required for river quality monitoring (National Water Council 1977). It is well documented (SDD 1977 for example) that the discharge of visually offensive paper and plastic material, rather than water pollution, gives rise to the most complaints from the public. This material frequently accumulates on riverbank vegetation and is aesthetically revolting as it is clearly sewage in origin and also slow to degrade.

The public perception of CSO discharges centres around the presence or otherwise of visible solids in a watercourse. There is even a body of opinion suggesting that the effect of CSO discharges on water quality may be ignored. Krejci & Baer (1990) for example have gone as far as to propose that "..water pollution control measures should be primarily viewed as a solution of local aesthetic problems..". As part of this strategy it is also suggested that CSO discharge should only be permitted once in two years and undoubtedly this would also improve water quality significantly, however, the focus of attention would remain the removal of visible solids.

Despite being at the forefront of public perception of pollution, the behaviour of gross and visible solids has been little studied and a discussion on the nature of pollution from combined sewer systems follows. A principal conclusion is that a wide range of tools are available for studying the behaviour of suspended particulate and dissolved pollutants, whereas little has been similarly determined for gross particulate and aesthetically offensive material.

2.2 POLLUTANTS WITHIN COMBINED SEWER SYSTEMS

Combined sewer systems collect waste from almost the full range of human activity whether in buildings or out of doors. The following are identifiable as relevant sources and reservoirs of potentially polluting material;

Road surfaces	Street cleaning, highway drainage and faeces
Roofs	Airborne deposition,
Permeable Surfaces	Washoff of soil and other light material
Below ground	Gulley pots and in-pipe deposits
Human waste	From washing, cooking and sanitation
Industrial	Effluent from processing and manufacturing

The range of industrial discharges is so wide and the behaviour of pollutants produced so disparate that any effort to account for their behaviour at CSOs is beyond the range of this research. No combined sewer overflow included in this study had any significant industrial discharge upstream.

2.2.1 Categorisation and Measurement of Pollutants

Pollution is relevant when extraneous matter is present in unacceptable amounts within what is generally considered to be natural environments. Ellis (1986) and Moffa (1990) among others have presented general categories of aquatic pollution, and stress six quality problems associated with combined sewer overflow discharges;

Aesthetics	Visual and odour evidence of sewage discharges
Solids	Organic and inorganic, colloidal and particulate.
Oxygen Demand	Due to decomposition of carbonaceous matter.

Toxicity	Rarely found in storm sewage at concentrations acute to freshwater aquatic life.
Bacteria	Derived from human waste and creating public health nuisance.
Nutrients	Generally low loads compared with the continuous loads of treatment plant effluents, however phosphorus and nitrogen loadings discharged from CSOs can give rise to eutrophication problems.

The relative importance of each item above will vary from site to site, the principal concern from the overflows in this study being aesthetics and solids emissions.

The quality of rivers and other natural water bodies is principally measured using inorganic physical and chemical parameters (National Water Council 1977, House 1989). It is also established practice to measure the strength of pollutant flows using primarily inorganic tests. The range of compounds in combined sewage is potentially vast and it is only in detailed studies at specific sites that testing of more than a few can be contemplated. Hogland et al. (1984) list 26 measured determinands in Scandinavian and US studies, all but two of which were inorganic. This number of determinands is rare and in less specific research, particularly of CSOs, a more limited range of measurements is normal. To reduce the number of variables to be examined, it was appropriate, in the context of this study, to categorise the pollutants principally in terms of their physical characteristics thus;

Dissolved & Colloidal:-	Measured by chemical analysis.
Fine Particulate; and:-	Measured by mass.
Gross/Visible Solids :-	Measured by mass or counting.

Table 2.1, from Crabtree et al (1991) with additional information from the present study, is included to illustrate the range of pollutant concentrations recorded in UK and European studies. The parameters listed are non-specific and indicate the general pollutant loads. In such

presentations, oxygen demand and solids concentration measurement predominate, although, reviewing European and US data, Geiger (1986) additionally stresses the importance of nitrogen and phosphate loads.

Average pollutant concentrations as shown in Table 2.1 tend to be of limited value due principally to temporal variation during events which may be very large. However, they can be useful for comparison between catchments and the high flow event mean concentration is used

Site	**TSS**		**COD**		**AmmN**		**BOD_5**	**Flow**
	(mg/l)	**(g/hd)**	**(mg/l)**	**(g/hd)**	**(mg/l)**	**(g/hd)**	**(mg/l)**	**(l/hd)**
Dundee;								
Summer	173	54.4	517	163	21	6.6	143	315
Winter*	80	20.7	41	10.4	27	7.0	-	254
Winter*	195	49.6	41	10.4	91	23.0	-	258
Dunfermline;								
Broomhead	182	47.9	689	181	36.6	9.6	116	182
Elgin St.	188	50.8	535	160	14.6	4.5	106	313
Dixon St.	193	44.7	366	73.3	18.1	4.0	106	174
Lochgelly	137	28.6	461	95.8	23.6	4.9	115	252
England**	231	44.3	348	66.7	19.8	3.8	-	191.8
Germany*	177	-	443	-	45	-	199	-
Brussels*	-	90	-	135	-	12	-	180

All data represent yearly average values apart from Dundee
* - During Winter Salting (From Crabtree at al 1991)
** - MOSQITO Data (Henderson 1988)

Table 2.1 Dry Weather Flow - mean values

for return period determination. Recent work (Crabtree et al 1991) has suggested that on certain catchments there are marked concentration differences between summer and winter conditions. Such marked variations must affect pollutant loads, particularly from CSO structures, however, in this study differences attributable to flowrate variations have been observed but seasonal effects have not.

At CSO devices, particulate rather than dissolved pollutants are potentially removed. Consequently, the solids concentration as total suspended solids (TSS) is considered to be the most pertinent determinand. The most common measure of oxygen uptake is chemical oxygen demand (COD), although Aalderink et al. (1990) among others have shown that frequently a relationship exists between COD and TSS. Biochemical Oxygen Demand (BOD) is the most appropriate measure of oxygen demand but it is difficult to use in intensive studies due to the resource requirements of the analysis. It also tends to be unreliable, and as a result the number of BOD tests carried out is usually limited. Consequently COD is the measure of oxygen demand most frequently used with Ammonia concentrations (as NH_3) considered to be indicative of the loading of dissolved pollutants present.

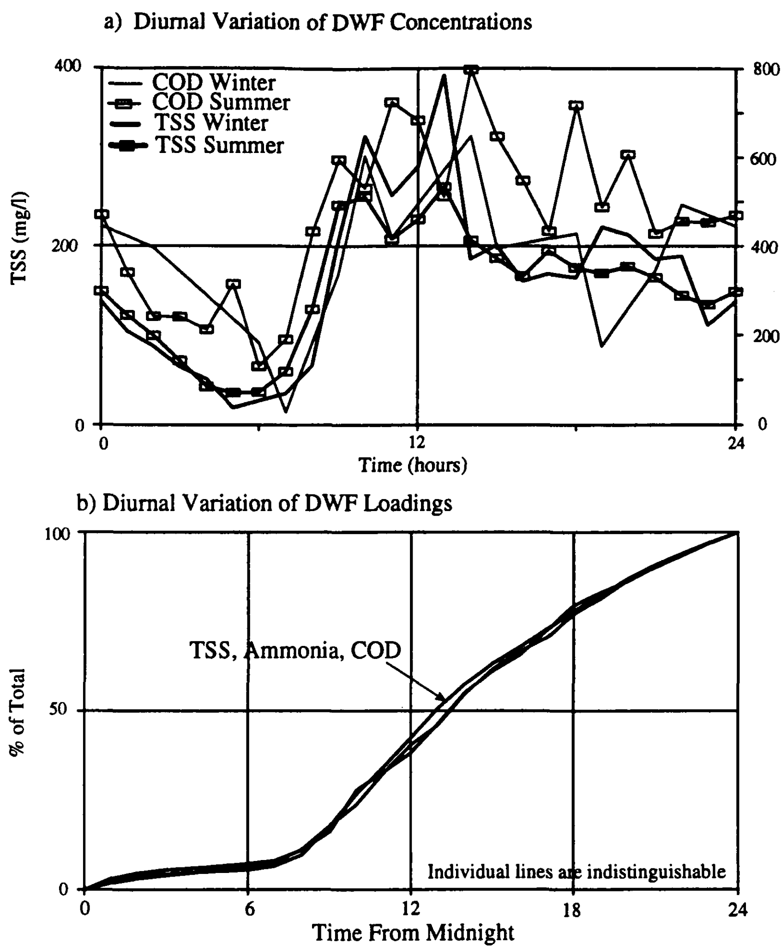
2.2.2 Sources and Reservoirs of Pollutants

Three principal mechanisms have been identified as influencing pollution loads within combined sewer systems (Crabtree et al 1991);

Storm Inputs	-	due to surface runoff from roofs, gulley pots and overland flow.
Sanitary Loadings	-	the diurnal cycle of flow and pollutants linked to human activity.
Deposited material in pipeline	-	derived from the sources listed in 2.2.1 but present in significant quantities due to local deposition.

Sanitary loadings in combined sewer systems during dry weather periods have been studied by many researchers (eg. Thornton & Saul 1986, Ashley et al 1990, Henderson 1988) who have all reported similar variation of flows, concentrations and pollutant loads. Figure 2.1 illustrates typical data from a catchment in Dundee receiving primarily domestic and industrial flows.

Approximately 8% of the total load passes between 1.00 and 8.00am, with from 68% to 82% between 8.00am and 6.00pm. Significant seasonal variation is reported due to road de-icing and Ashley et al (1990) have suggested that other explanations are seasonal population variation and eating habits. In general, DWF conditions are easily measured and provide a useful basis for comparison of catchments, particularly as DWF produces a continuous baseload of pollutant flow. The data in Table 2.1 illustrate how similar are DWF data quality for a range of catchments.



Source: Unpublished data from city centre catchment, Dundee

Figure 2.1 Typical Diurnal Dry Weather Flow Variations

In-pipe deposition has been shown (Lindholm 1984, Stotz & Krauth 1984) to be a significant source of pollutants during first foul flushes (see also section 2.2.3). Deposition of suspended material occurs in sewers either where the pipe slope is slack or at particular locations, such as at CSO structures where the hydraulic gradient becomes reduced. Ashley et al (1992), considering principally in-pipe deposits have hypothesised three types of sewer in terms of their capacity to allow settlement of solids;

Collectors - Small diameter requiring storm inputs to clean out the sediment.

Trunks - Intermediate size, generally with steeper slopes and little potential for sedimentation.

Interceptors - Slack gradients with greatest potential for sedimentation.

At CSO and other structures, deposition may occur both in the overflow device or within upstream sewers where surcharging causes reduced pipeline velocities. Although sedimentation creates pollutant reservoirs, separation of solids is encouraged to improve the quality of CSO spill flows. Recent work by Saul and Ellis (1990) has shown that these conflicting requirements at CSOs can partly be reconciled within on-line tanks by the use of steep benching and long tanks, increasing bottom velocities and reducing consolidation of sediments.

Type	Sediment Description	Percentage Particle Size								
		Gravel(2-50mm)			Sand(0.063-2mm)			Silt/Clay(<0.063mm)		
		Mean	Max	Min	Mean	Max	Min	Mean	Max	Min
A	coarse, loose, granular, predominantly mineral material found in the inverts of pipes	33	90	3	91	87	3	6	30	1
B	as A but concreted by the addition of fat, bitumen cement etc. into a solid mass	Rarely Found and no data available								
C	mobile, fine grained deposits found in slack flow zones, either in isolation or above Type A material.	0	0	0	55	71	5	45	73	29
D	organic pipe wall slimes and zoogleal biofilms around the mean flow level.	6	20	1	62	83	1	32	52	17
E	fine-grained mineral and organic deposits found in SSO storage tanks	9	80	4	69	85	1	22	80	1

Source: Crabtree (1989b)

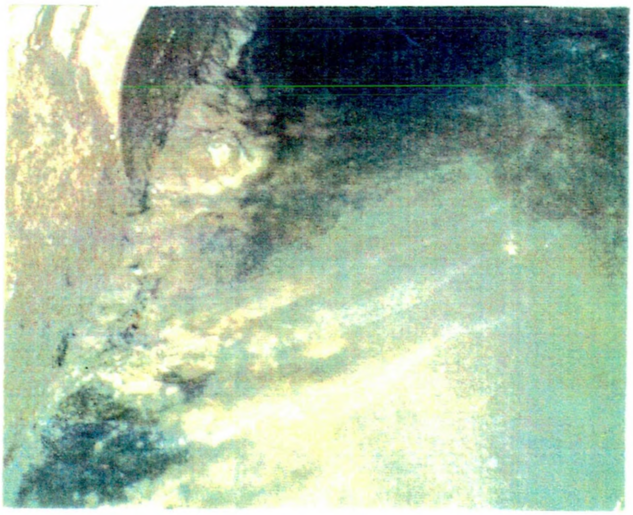
Table 2.2 Sewer Sediment Classification

Crabtree (1989b) defined five classes of sediment material (Table 2.2) and evaluated the potential polluting load of each. Consideration was given to a length of 1000mm diameter sewer with 200mm of Type A sediment covered by 10mm of Type C. With a sewage depth of 290mm it was concluded that, assuming 100% erosion, 87% of the potential pollution, expressed as BOD, would be in the sediment and capable of being entrained, while only 13% would be in the foul sewage. The implication from this example is that Type A deposits constitute the largest pollutant reservoir within a system, principally due to their presence in largest quantities, and may be eroded and deposited at different times during storm flows.

The Type A deposits do require, however, significantly more tractive effort for their removal than Type C material (Ashley et al 1992) for which cycles of night-time deposition, daytime flushing have been suggested (Geiger 1984, Jefferies et al. 1990). Chebbo & Bachoc (1992) have shown that approximately twice as much material of particle size <0.05mm than of size >0.05mm moves during wet weather. In view of the particle sizes of the various sediment Types (Crabtree 1989b) shown in Table 2.2, it is apparent that Type C material forms the principal reservoir for pollutants during storm flows except during extreme flows.

Settling velocity curves have been published from a number of studies along with particle size data (Ashley & Crabtree 1992), however these are currently limited to design procedures (Tyack et al 1992). No clear understanding has yet emerged of the movement of the different particle sizes and types within sewers, this applying equally to the dry weather flow pattern of erosion/deposition as to the first foul flush.

Consideration remains to be given to the occurrence of gross solids in sewer sediment deposits. In defining in-sewer deposit types, little heed has been paid to such material. A wide ranging CIRIA (1987) report for example stated that sediment is "any solid material.. including fats..carried by sewage flows". However in defining three types of material as fine, coarser and grits & gravel, the large particles with low settling velocity which typify screenings material were virtually excluded.



From Crabtree (1988)

Plate 2.1 Type C Sediment Overlying Type A

The sediment types defined by Crabtree (1989b) make the same omission, indeed in an earlier document reporting the same research, Crabtree (1988) included photographs of sediments which, although captioned as Type C material, clearly included a significant proportion of gross solids. One of Crabtree's photographs is included as Plate 2.1.

The Type C deposits are 'fine grained', thus excluding gross solids, and 'mobile..' (Crabtree 1989a), which latter characteristic also applies to the gross sewer solids which accompany them. It is concluded from the research reported in this thesis that, in spite of clear differences of size, shape and density, not only are gross solids deposited under similar conditions to Class C material, but the conditions for subsequent resuspension are similar. Further consideration of gross solids is included in section 2.2.5.

2.2.3 First Foul Flush

The existence of a first foul flush (FFF) has frequently been noted (Tucker & Mortimer 1978, Mance 1981, Geiger 1984, Thornton & Saul 1986). Some authors including Ellis (1982) question its existence although it is very likely that catchment differences are responsible for such scepticism. Reviewing data from a number of catchments, Stotz & Krauth (1984) for example concluded that the flush is significantly less marked for larger catchments. Thornton & Saul (1986), in work which is also described in greater detail in Pearson et al (1986), recognised that different magnitudes of flush occur by defining two types of FFF events using TSS and COD concentrations, in addition to events which show no FFF evidence;

Type A - Initial pollutant peak coincides with the flow peak and the concentration may be less than the prevailing DWF values

Type B - Pollutant peak exceeds prevailing DWF concentrations and precedes the flow peak.

It was found that Type B flushes comprised 60-80% of the total in the 41 events studied. In a later paper, Saul & Thornton (1989) produced revised definitions based on cumulative loads rather than concentrations, referring to the flushes as **Enhanced** and **Normal** respectively.

FFF would appear to be limited to solids-related loads and is not observed in dissolved pollutants. Pearson et al (1989) for example have observed that the behaviour of NH_3 during storm events can be explained in terms of DWF concentrations being diluted by storm water. An initial fall of NH_3 concentration to a minimum at peak flowrate was normally followed by a steady return to DWF levels as the event subsided.

Different studies have shown varying correlation between first flush and parameters such as ADWP which might be expected to be of influence. Saul & Thornton (1989) found strong correlation (r^2=0.79) between peak TSS concentration and ADWP for Type B flushes. In contrast Geiger (1984) failed to find any relationship and Mance (1981), reporting on three studies, came to the same conclusion. A conclusion of this research is that part of the differences in flush behaviour may be due to the location and nature of sediments deposited within the catchment. Steep catchments, with fewer sediment deposits will exhibit a different flush behaviour from those with slack gradients.

Stotz & Krauth found that for a flush to occur, deposits actually had to be present in the sewer system and they presented evidence that the extent of the flush was dependent on the preceding dry period. It was also found that a flushing effect on one part of a catchment will not necessarily be reflected by observations at another location. It is suggested that a significant improvement might be found in correlating ADWP and FFF, should sediment deposition be included, although no evidence has been found to support this conclusion.

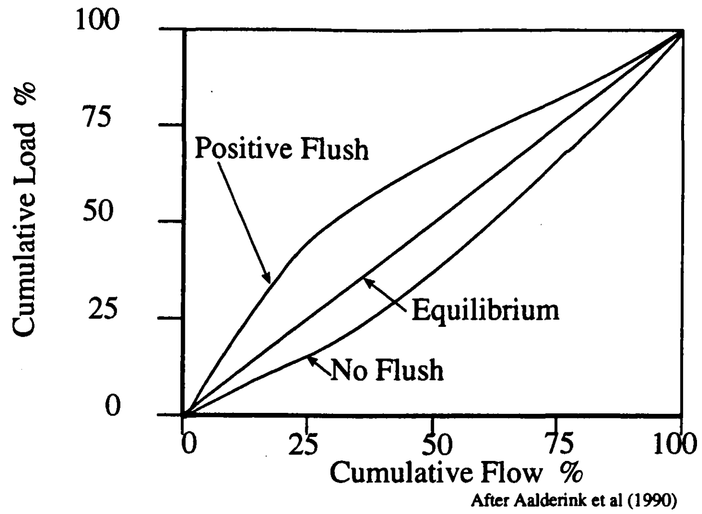


Figure 2.2 First Foul Flush defined by cumulative load curves

Tucker & Mortimer (1978) proposed a classification of FFF based on a plot of cumulative mass discharge against the cumulative square of the flow. This technique has been further developed (Stotz & Krauth 1984, Aalderink et al 1990) by plotting the pollutant mass against the cumulative volume with results expressed as percentages of event totals as illustrated in Figure 2.2. Aalderink recommended that where the resulting curve lay above the 1:1 line, FFF was deemed to be apparent.

2.2.4 Event Mean Concentration

Averages of concentrations provide general estimates of runoff quality and reflect the physical, chemical and biological nature of the catchment studied. It has been noted (Ellis 1986, Geiger 1987, Nakamura 1990) that statistical distributions can be fitted to event mean concentration (EMC) data thus enabling the probability of particular events to be estimated. Geiger (1987) found that all load and runoff figures were distributed log-normally, while Hall et al (1990) found that data from French catchments studied could be fitted to a mixture of two normal distributions.

A further proposal has been made by Aalderink et al (1990) that EMC would assist in relating event characteristics to those of the contributing rainfall, particularly ADWP, however, little success was achieved. There is little doubt that as more data are gathered from a range of different types of catchments, EMC will gain in importance as a tool for the determination of return periods for pollution events.

2.2.5 Estimation of Screenings and Gross Solids

Screenings at sewage works pose continual operational problems principally by virtue of the mass of material requiring disposal. Sidwick (1984) presented tables and charts for the estimation of quantities after studies at 27 sewage works. His data suggested that the total load of screenings in dry weather flows of sewage might range between 0.01 and 0.03 m^3 per m^3 of sewage per day of which some 90% would be paper and rags with less than 5% plastic. During storm conditions, volumes seven times the above were also proposed.

In the operation of a CSO in which settlement takes place, a proportion of heavier solids are removed by gravity. Screenings, which are removed at sewage works from the total flow by traditional bar screens, represent only a part of the total load of gross solids during dry weather due to the proportion which passes through. The use of screenings data from sewage works is thus of limited value for a consideration of CSOs. It has also been shown by Page (1986) that screening load measurements are highly dependent on the screen type and bar spacing in use.

Apart from the methods of estimation of screenings volumes for dry weather flows presented by Sidwick who did not present a review of other findings, surprisingly little information is available for CSO discharges. O'Sullivan (1990) for example found "..a dearth of information relating to the quantities of gross debris discharged.." and "..a lack of field information of different types of overflow structure in terms of their retention of gross solids".

The need for studies into the behaviour of screenings-type material has been established. This material was reported by SDD (1977) as being "The main cause of complaints from the public regarding the operation of overflows..". Few relevant studies have been reported, although Mutzner (1987) described interesting fieldwork carried out during one summer in which the number of solids discharged was estimated in terms of the number per metre visible on the river bank. The data did not suggest any clear relationships with flow rates, volumes or dry weather period, but the study did attempt to make a direct assessment of the problems of visual pollution.

In order to advance the study of the operation of different types of CSO, two extra categories of solids are proposed and used here. The general term **Gross Solids** and as a subset, the more specific **Visible Solids** are introduced.

> **Gross solids** can be defined as faecal matter, particles of paper and any other material greater than the arbitrary value of 6mm in any two dimensions with specific gravity close to unity. This definition is required to interpret information obtained using the Gross Solids Sampler described below and may also be used in defining consent standards.
>
> **Visible solids** are material which is identifiably sewage in origin and would be noticed by a casual observer walking on a river bank. The material is in effect plastic and paper strips which have virtually neutral buoyancy and in many respects is the same as screenings material. However, measurement of the latter is dependent on the spacing of screens whereas this is not the case for visible solids.

2.3 SEPARATION STRUCTURES

The combined sewer overflow (CSO) was developed initially as a purely hydraulic device for the reduction of flows downstream within a sewer system to reduce flooding and overloading.

Current practice on CSO devices has been set out by Balmforth & Henderson (1988) who recommended three types of structure which achieve "..effective hydraulic control and good solids separating efficiency..";

Stilling Pond
High Side Weir
Vortex with peripheral spill

The authors comment on two further types which are in effect developments from the above;

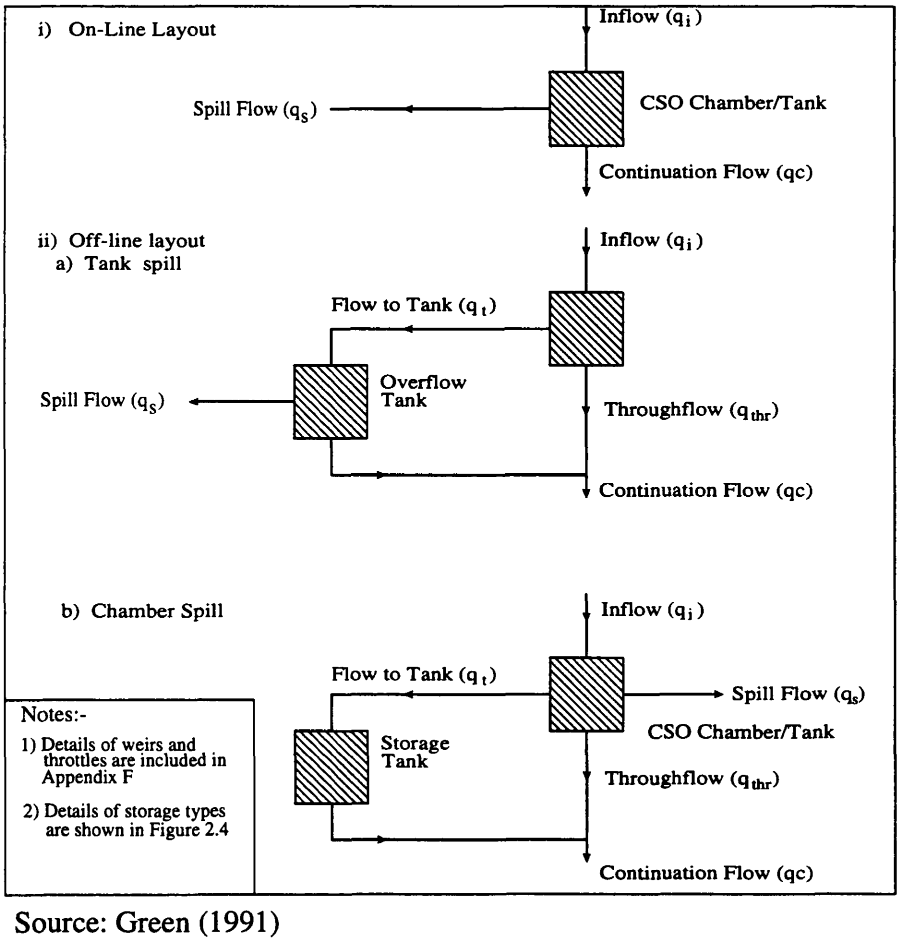
Hydro-dynamic separator (of which the Storm King is a proprietary design)
Compact air-regulated syphon

The performance characteristics of most of the above overflow devices have been determined from laboratory tests using discrete particles to represent the sewage solids. Each series of tests has normally been carried out by a different researcher concentrating on a different type or variation of CSO. The methods of modelling and interpretation of results is of crucial importance when using particulate material. Halliwell and Saul (1980), in an excellent review of hydraulic models demonstrated the importance of using the Froude scale for particulates, thereby modelling the settling velocity correctly, in preference to geometric scaling which suffers from scale effects.

A review of the physical dimensions of the separation devices listed above and an appraisal of the results of their testing in model form are included in this thesis in Appendix F.

2.3.1 Overflow Configurations including storage

Recommended practice in the UK (WRc/WAA 1986) for the design of CSO installations promotes the use of overflow structures with associated storage volume for the retention of pollutants within sewer systems. Overflow devices are currently designed in the UK in accordance with guidelines established after model tests (Balmforth & Henderson 1988). The types of overflow structure for which rational designs are available are discussed in sections 2.3.4 to 2.3.6 and the various arrangement of storage in section 2.3.7, however it is appropriate at this stage to consider the on and off-line configurations in use.



Source: Green (1991)

Figure 2.3 Typical Combined Sewer Overflow Arrangements

Knott & Taylor (1985) outlined four types of tank arrangement in the context of numerical modelling;

- On-line tanks
- Tank sewers
- Off-line tanks with pumped return
- Off-line tanks with gravity return

Tank sewers are the most common type in use (Cant 1990), however they are not included in this study as they do not normally spill to a watercourse. Green (1991), in proposing different definitions for use in field studies of performance, did not differentiate between pumped and gravity return as this feature makes no difference to the pollution performance of the tanks and no such differentiation is considered here. Green however considered the physical location of the spill weir to be important and included a separate category as shown in Figure 2.3.

2.3.2 On and Off-line Arrangements

On-line layout - Figure 2.3 i)

Little or no storage In principle this configuration will only retain a small amount of settleable solids and some floating solids by virtue of scumboards. In practice it is observed that a small amount of storage, together with the volume within the sewers upstream will retain small but frequent minor events.

Including storage Large tanks with a submerged flow control at the downstream end. Spill is direct from on-line tanks, the position of the spill weir being generally close to the inlet.

Offline Layout with Tank Spill - Figure 2.3 ii)a)

Inflow is via an on-line structure which may be a minor diversion chamber, or more normally a conventional overflow with little storage. The design of the storage provided can radically affect the pollution retention performance of the structure and is considered in section 2.3.3.

Offline Layout with Chamber Spill - Figure 2.3 ii)b)

In effect a subset of the overflow tank arrangement with the spill weir located within the CSO chamber.

2.3.3 Storage Arrangements

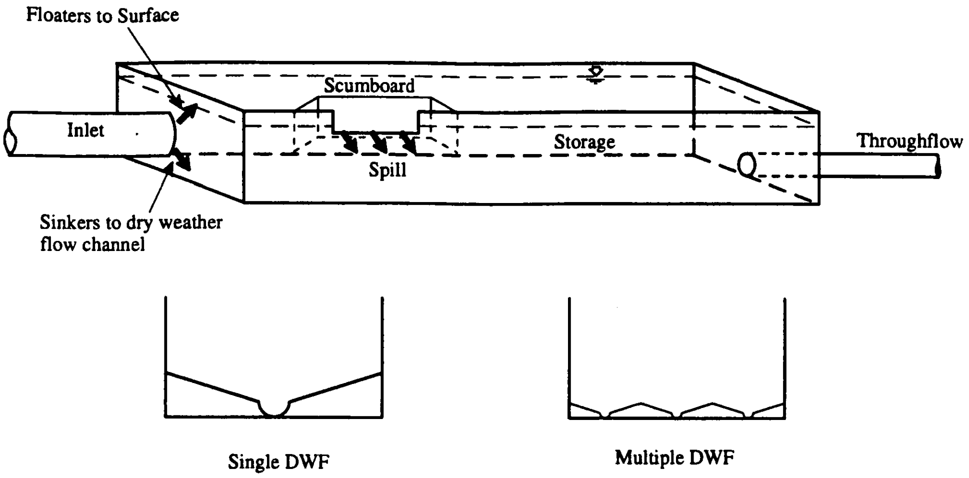
A wide range of storage configurations have been proposed and reported by various authors including Cant (1990) and Crabtree et al (1991). A number of different plan shapes are in use or have been proposed including rectangular, circular and kite shaped. Storage tanks have two conflicting requirements for their operation;

- Removal of pollutants, principally by sedimentation, should be maximised to improve the quality of spill discharges; and

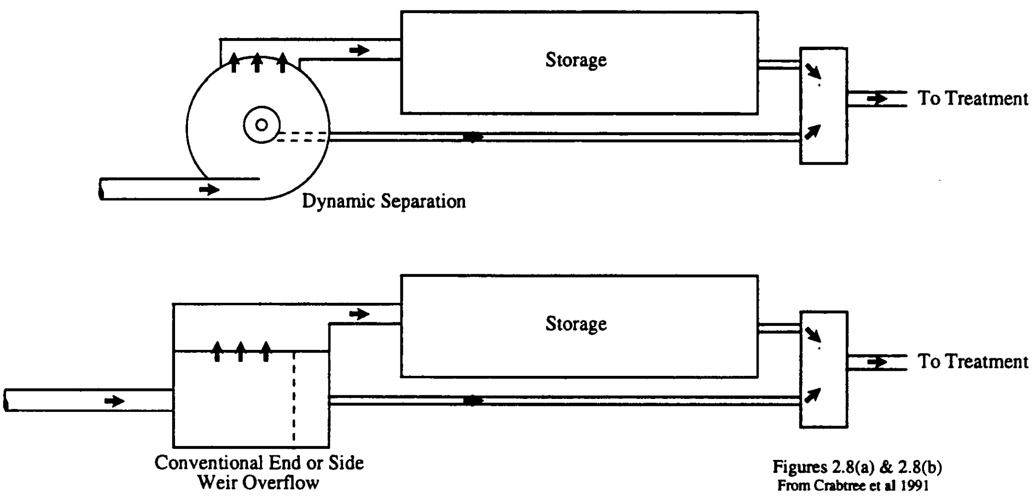
- As much material should be retained in the continuation flow as possible to minimise sediment build up on the tank floor.

Typical storage tank arrangements are illustrated in Figure 2.4. A number of internal arrangements to reduce build up of sediment in tanks have been recommended by Saul & Ellis (1990) & Crabtree et al (1991) It was found that long narrow chambers were best since velocities in the near-bed region remained highest, and benching was helpful but not essential. A drop at inlet was also recommended to assist in the generation of higher velocities.

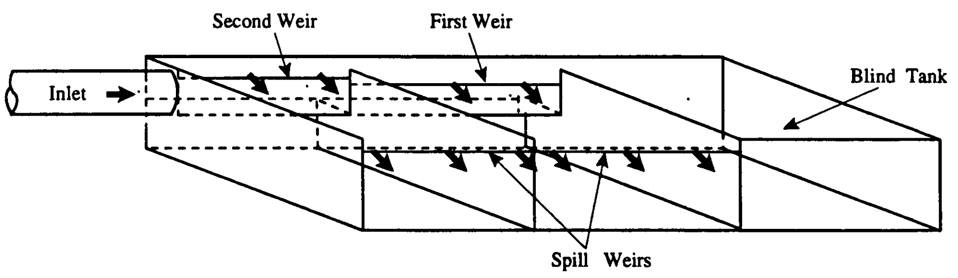
Knott & Taylor (1985) found that the most common arrangement was rectangular, a shape for which there are a number of subdivisions. Sectional rectangular tanks were considered by SDD (1977) and found to be effective in reduction of pollutants by Geiger (1986). A recent study by Brechenmacher et al (1992) came to the same conclusion. Knott & Taylor (1985) however considered this configuration to be unnecessarily complicated and not particularly successful.



(a) Basic Rectangular on-line Storage Tank with side weir overflow



(b) Conventional Overflow with Off-line Storage



(c) Off-line Partitioned Tank Arrangement With Blind Compartment

Figure 2.4 Storage Arrangements

The sectional tank, installed at two of the locations used in this study and illustrated in Figure 2.4 (c) has at least two parts, the first has no spill weir and retains the first foul flush. The remaining parts are filled as the event proceeds and small events may be retained entirely within the blind tank. Off-line structures incorporate a separate flow split device which frequently is a conventional CSO structure or hydrodynamic separator as illustrated in Figure 2.4 (b). The overall pollutant retention performance is improved by good separation at the CSO device.

2.4 EFFICIENCY OF POLLUTANT SEPARATION FROM MODEL TESTS

The range of available CSO structures was reviewed in section 2.3. A review of the extensive modelling work, primarily using steady flows, which has been carried out to determine operational characteristics and to set design parameters is included in Appendix F. The conclusions from this review of published efficiency data are;

(i) Poorer efficiencies are reported for low rates of rise than low fall particles, although improved devices reported by Balmforth show smaller differences. The hydrodynamic separator has poor performance for rise particles.

(ii) The minimum efficiency always occurs for material with low rise/fall velocity and depends upon the **q/Q** ratio chosen in each study.

(iii) Generally there is little difference in reported efficiency between the high-side weir and stilling pond overflows.

(iv) The vortex, swirl and separator overflows have greater efficiencies, particularly for falling material.

(v) The flow ratio **q/Q** has an effect on device efficiencies which exceeds all others.

2.5 FIELD EFFICIENCY DETERMINATION

2.5.1 Monitoring Programmes

Field monitoring programmes to determine the performance of CSO devices contrast markedly from the laboratory procedures described in Appendix F. It is not generally possible to carry out field steady state flow tests with particulate matter of well-defined characteristics, although some tests have been reported (SDD 1977, Hedges et al 1992). Instead, reliance must be placed upon observing naturally occurring events which produce time variant hydrographs and pollutographs. Antecedent conditions vary between events, and storage, even in small overflows, must be filled prior to overflow. In monitoring programmes the measured determinands are flowrates and concentrations in some or all of the flow streams identified in section 2.3.2.

Most monitoring programmes have relied on a combination of flow monitoring and sampling and a wide variety of approaches have been used. The variety of installation types and control devices in Germany has been highlighted by Brombach (1989) where monitoring of level, and in particular, durations of spill is routine. Requirements for compliance of spill discharges (Dohman et al 1992) have required particular attention to be paid to durations of spill and monitoring programmes there concentrate on volumetric considerations. Only limited results of CSO monitoring in the USA have been published and studies reviewed by Pisano (1988) were inconclusive, a not unexpected outcome since at one location no flow monitoring was carried out.

Studies in the United Kingdom have tended to be more complete. SDD (1977) describes monitoring at three storm tanks at sewage works during which flows were monitored and sewage samples taken, while other studies have been described by Thornton & Saul (1986), Cootes et al (1989), Saul & Marsh (1990b), Hedges et al (1992) and Bennett & Rosbrook (1992). Most reports have made the unqualified claim that hydraulic considerations have been monitored satisfactorily, while some have carried out composite sampling only for parts of the study (eg Bennett & Rosbrook (1992). Most studies have used small bore sampling at various times during high flow events with automatic triggering of the samplers. In addition to reporting on a CSO study, sampling techniques and time intervals sufficient to monitor the first foul flush are set out in detail by Saul & Marsh (1990b) whose recommendations have been followed in the study reported here.

Sampling for gross solids has only been reported from one study in addition to the work presented in this thesis. Work at a site in Sheffield has been reported by Cootes et al (1989) at which the Gross Solids Monitor was installed. This equipment was developed in parallel with the Gross Solids Sampler, pumping the combined sewage through 100mm pipes past an infra-red light source to produce an image for a video camera. As with the present study, the system was triggered by high flows but was dogged by an extended dry period and difficulties with image analysis.

2.5.2 A Common Basis for Efficiency Definitions

In order to provide a common basis for comparison of CSO monitoring exercises, Green (1991) has set out a procedure and definitions for interpreting results. His report is a synthesis of work by others including the author. Consequently definitions of the various efficiencies are included here in some detail. Typical hydrographs and pollutographs are illustrated in Figure 2.5 and the terms used are defined in Table 2.3.

Location	Instantaneous Values	
	Flowrate	Concentration
Inflow	q_i	c_i
Overflow*	q_o	c_o
Continuation	q_c	c_c
Spill**	q_s	c_s

* On-line devices - Equates to flow to off-line tank (q_t in Figure 2.3)
** On or off-line devices

Table 2.3 Variables in Efficiency Definitions

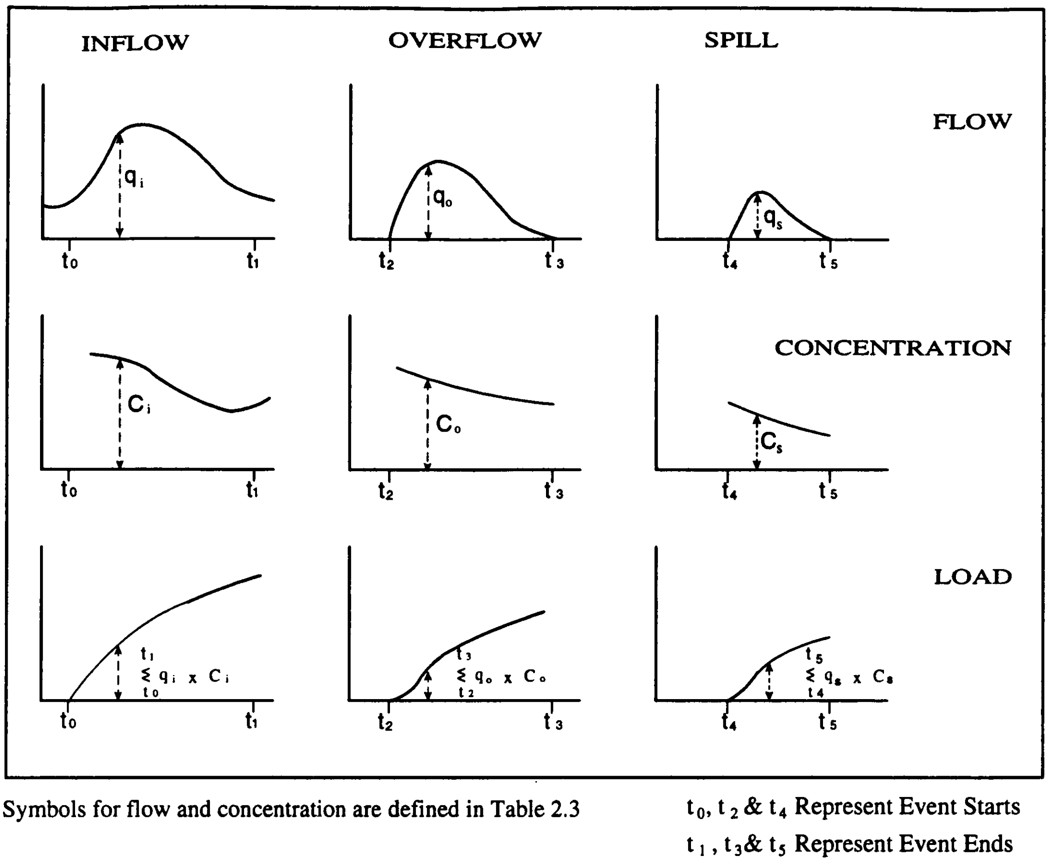


Figure 2.5 Pollutographs for efficiency definitions

2.5.3 Flow Split

Flow split (**FS**) is a purely volumetric ratio reflecting the proportion of flow retained within the system. It is defined as follows;

$$\mathbf{FS} = \frac{\text{Total Storm Volume Retained } (\mathbf{TVR})}{\text{Total Storm Inflow Volume } (\mathbf{TIV})} \qquad \mathbf{2.1}$$

Since the volume retained must include the storage volume within the tank and sewer system, the following formulations have been used;

$$\mathbf{TIV} = \sum_{t_o}^{t_1} q_i \qquad \mathbf{2.2}$$

$$\mathbf{TVR} = \sum_{t_o}^{t_1} q_i - \sum_{t_2}^{t_3} q_o \quad \text{(For On-Line devices)} \qquad \mathbf{2.3}$$

Or

$$\mathbf{TVR} = \sum_{t_o}^{t_1} q_i - \sum_{t_4}^{t_5} q_s \quad \text{(For Off-line devices)} \qquad \mathbf{2.4}$$

2.5.4 Total Efficiency

Total efficiency measures the overall capacity of the CSO to retain pollutants within the sewer system. Although it does account for pollutant separation within the device, the determination of total efficiency is dominated by the retention of flows within the system and thus is highly event and location specific. It is important for the determination of the treatment factor as discussed in section 2.5.5.

$$\textbf{Total Efficiency} = \frac{\text{Total Storm Load Retained } (\textbf{PLR})}{\text{Total Storm Inflow Load } (\textbf{PIL})} \qquad \textbf{2.5}$$

PLR and **PIL** are defined as follows;

$$\textbf{PIL} = \sum_{t_0}^{t_1} q_i \times C_i \qquad \textbf{2.6}$$

PLR can have more than one definition, depending on the measured determinands. When the continuation flow is measured;

$$\textbf{PLR} = \sum_{t_o}^{t_1} q_c \times C_c \qquad \textbf{2.7}$$

In cases where the continuation flow is not measured;

$$\textbf{PLR} = \sum_{t_o}^{t_1} q_i \times C_i - \sum_{t_2}^{t_3} q_s \times C_s \qquad \textbf{2.8}$$

When equation 2.8 is applied to on-line devices discharging to off-line tanks the spill term would be replaced by an overflow term.

2.5.5 Treatment Factor

Treatment Factor (**TF**) is defined as follows;

$$\textbf{TF} = \frac{\text{Total Efficiency}}{\text{Flow Split}} \qquad \textbf{2.9}$$

Treatment factor measures the ability of a device or system to separate pollutants from the flow as compared with the mere split of volume. The following amplifies the meaning of the term treatment factor;

TF < 1 indicates relatively a lesser volume of flow discharged or spilled than pollutant load and thus the device or system concentrates pollutants towards the overflow or spill.

TF = 1 indicates equal proportions of flow and pollutants continuing and being discharged. Under these conditions the system merely acts as a flow splitter.

TF > 1 indicates relatively more pollutants than flow in the continuation pipe and the device or system is positively treating the combined sewage.

2.5.6 Pollution Separation Efficiency

Pollution Separation Efficiency (PSE) is computed from the data only at the time when overflow (for on-line) or spill (for off-line) is occurring. Whilst neither flows nor concentrations can be considered to be steady state, PSE is calculated only after all storage has been filled and as such is the field-determined parameter which most closely relates to laboratory determined values.

$$\mathbf{PSE} = 1 - \frac{\mathbf{PSL}}{\mathbf{PIL}} \qquad \mathbf{2.10}$$

Where; **PSL** = Spill Load <u>over duration of spill</u>
and **PIL** = Inflow Load <u>over duration of spill</u>

$$\mathbf{PSL} = \sum_{t_4}^{t_5} q_s \times C_s \qquad \mathbf{2.11}$$

$$\mathbf{PIL} = \sum_{t_4}^{t_5} q_i \times C_i \qquad \mathbf{2.12}$$

2.6 SUMMARY

The behaviour and nature of suspended particles within a flow of fluid is extremely complex and subject to a high degree of variability. Successful testing either of models or prototypes requires that a large number of observations should be made. This is illustrated by tests carried out under laboratory conditions on full scale overflow models by Ruff & Saul (1992). Typical sewage solids were used and the results show a variability of > $\pm 2^1/_2$ standard deviations about the mean when less than 250 particles are used. This variability has been reduced in all previous studies by the insertion of near spherical particles into the flow. Such a large number needed for repeatability with sewage solids (the visible solids defined in section 2.2.5) calls into question the applicability of results of several model studies which form the basis of current design practice for overflows. It also poses questions of fieldwork programmes where only a limited number of events are captured.

The review in this chapter has shown that a range of overflow and storage arrangements are in use, the behaviour of which have invariably been determined from model tests. Such tests have enabled the devices to be developed and their performance compared under controlled conditions which are particularly appropriate for hydraulic testing. However, only very limited knowledge is available on the performance of prototype overflows with storage over a range of different types of events. Information on the movement of solids within sewer systems has been obtained in several studies, but questions remain regarding the prototype pollution performance of CSO devices. These include the quantity of pollutants removed at overflows, the actual numbers of discharge events and the loads continuing to treatment. All these questions have been the subject of speculation and have yet to be properly addressed.

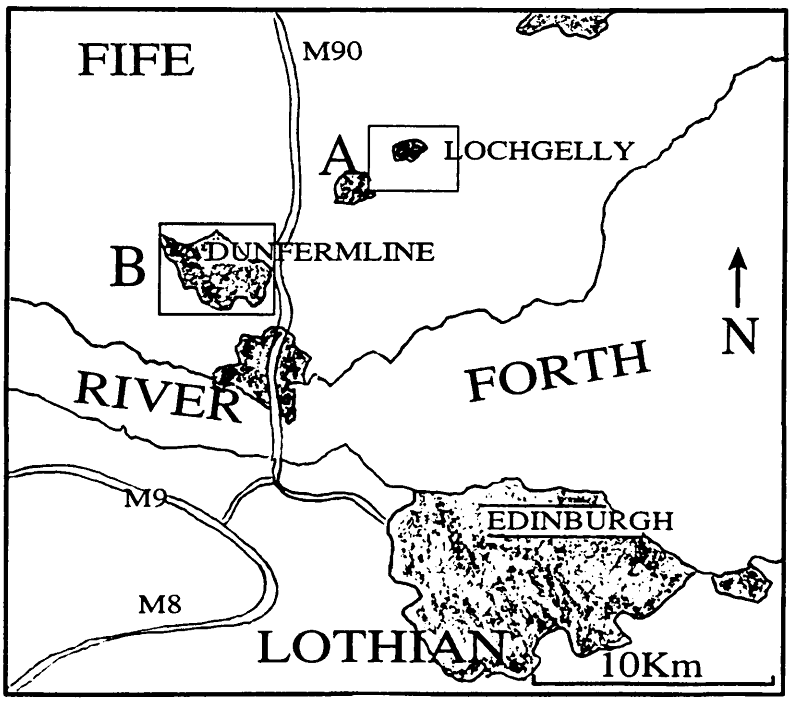
CHAPTER 3 STUDY CATCHMENTS AND FIELD SITES

I shall stay him no longer than to wish him a rainy evening to read this following discourse
Izaak Walton Compleat Angler.

3.1 INTRODUCTION

In a field study a balance has to be drawn between the general, leaving insufficient time or energy for specific analyses, and the specific, divorcing the details from the context of the study. This chapter places the study sites on the map, referring to relevant drainage history and giving contextual data relating to the overflows studied. Detailed descriptions of the installations at each site are included in Chapter 4.

The study sites were all located among the small towns of south Fife as indicated on Figure 3.1. Coal mining was run down in the 1960s and ceased to function altogether in the early 1980s, leaving the areas virtually bereft of any traditional industry. The result on the surface water system was a strongly modified drainage pattern and a sewerage system without industrial flow and very little from commercial premises.



Areas of Detail

A Figure 3.2

B Figure 3.3

Figure 3.1 General Location Plan

The chief impact on surface streams derives from the construction of mine drainage in the 17th and 18th centuries. These comprise nearly horizontal adits, known as day levels as they led eventually to daylight, and are described by Wilson (source unknown). They were driven from the coal seams to the lower reaches of the incised valleys to the south. The day level system totalled some 43km over the whole coalfield, the longest in the Dunfermline area being 8.7km. Their construction was sufficiently robust that many still act as drainage channels even though they have been out of use for over a century. Into the 1990s, and for the foreseeable future, stream flows will remain much reduced due to water capture by the day levels, their impact being so great that many streams dry up completely while the day levels themselves are perennial.

Combined sewer overflows continued with the closure of the mines and became more noticeable when many of the workings were rehabilitated to farming or recreation areas. The nuisance of these discharges became less and less acceptable with increased public access, particularly since maintenance was only on an emergency basis. The solution adopted has been to store excess storm flows (Jefferies & Stevens 1989) and only discharge occasionally to the watercourses which would otherwise be dry for significant periods.

Storage tanks on two systems were studied;

a) i) Main Dunfermline system. Discharge occurs from a multitude of overflows. The receiving conduit is a culverted storm relief outfall discharging to the Lyne Burn near to its effluence to the Firth of Forth. Pressure for improvement has arisen from pollution in the Lyne Burn and the high pollution and low dissolved oxygen levels which occur at times in the estuary (FRPB 1985)

ii) Broomhead. A peripheral catchment of Dunfermline. Discharge is to the Broomhead Burn, a minor ephemeral watercourse close to high amenity housing.

b) Lochgelly/Lumphinans. Two small towns at the head of the Levenmouth sewer system. The Lochgelly Burn also dries up in late Spring of each year.

3.2 THE DUNFERMLINE SEWER SYSTEM

3.2.1 Description of system

Dunfermline, a town of some 52,000 population is located in a highly advantageous position in central Scotland, having attracted considerable industrial and commercial growth in the past decades (Jefferies & Stevens 1989). Sewerage for the burgh is based on the conveyance of foul flows to the primary treatment works at St. Margarets Bay on the Firth of Forth and storm flows which are conveyed via the storm relief sewer which discharges to the Lyne Burn at Waulkmill as indicated in Figure 3.2. As with the sewer system for any long established town, improvements have taken place over the years. These have been described by Ashley et al. (1986).

Particular pollution problems emanated from the 1950s duplication of the principal sewers and the associated construction of the storm relief drain to Waulkmill. These improvements removed local pollution and virtually eliminated sewage derived flooding in all but localised areas. Unfortunately, although many housing developments utilise the principles of the separate system, the local authority at the time had a policy of routing both foul and storm flows in the one manhole. The dual manholes caused major pollution problems due to the inevitable blockages, locally known as chokes, which occurred.

The principal consequence of the dual manhole policy was that the storm sewer, draining approximately two thirds of the catchment, regularly carried foul discharges, the

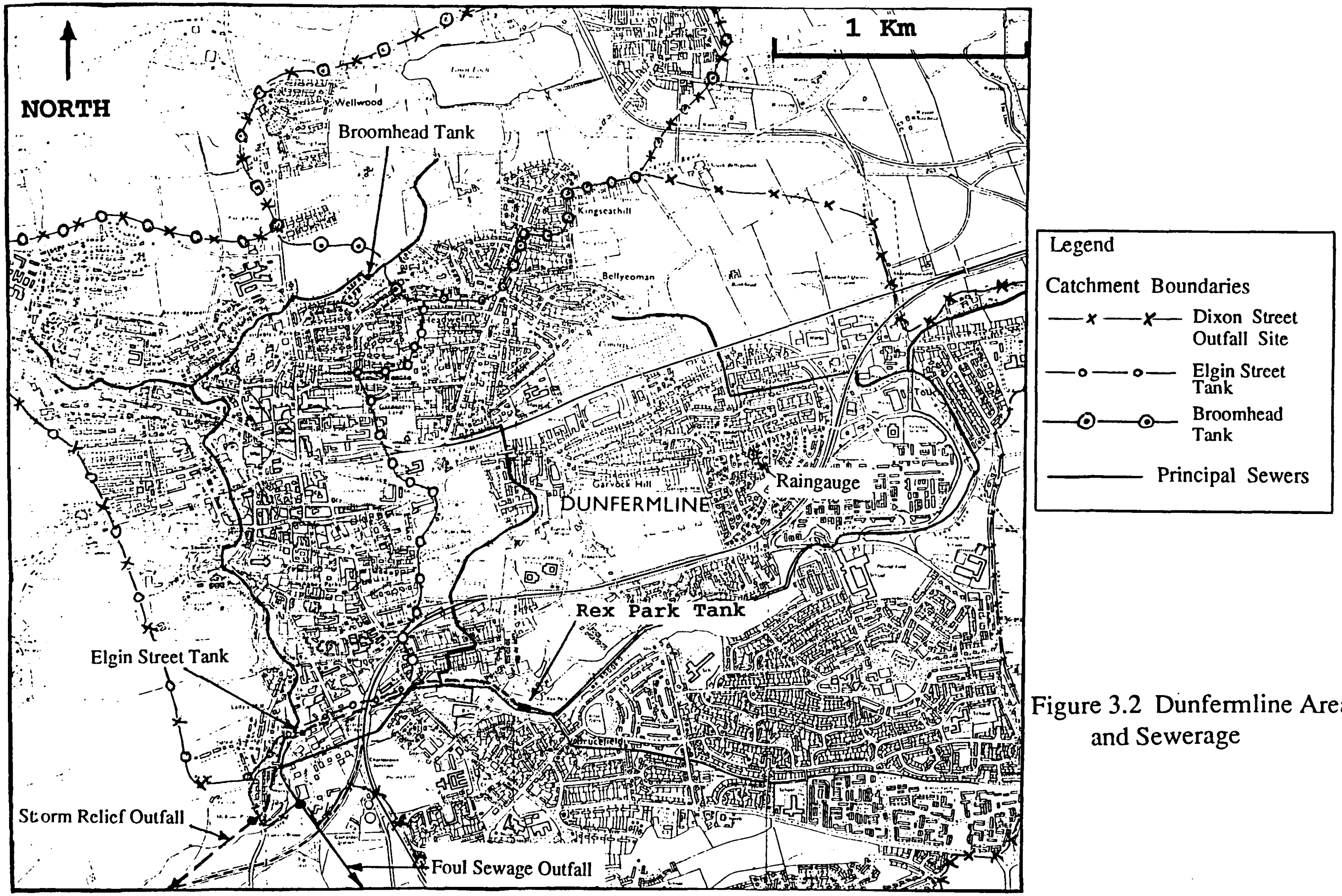


Figure 3.2 Dunfermline Area and Sewerage

precise sources of which were very difficult to locate. Additionally, a large number of low-side weir overflows were installed creating a parallel pipe system with multiple cross connections. This system, known as the "Lyne Burn Sewer" terminates at a major overflow at Bothwell Street and was described and modelled by Au Yeung (1990). The remainder of the system on the Tower Burn Sewer had no cross connections but was chronically overloaded as far downstream as a similar low-side weir overflow at Lady's Mill.

3.2.2 The Tank Construction programme

The degree of pollution caused by this badly conceived system was tolerated for a number of years. However, considerable concern was created in both District and Regional Councils when the Forth River Purification Board (FRPB) reported that the outfall at Waulkmill was "..continually discharging raw sewage.." (FRPB 1985/86).

Improvements were required and, since the storm drain conveyed discharges from such a variety of locations, the solution adopted was to intercept both foul and storm flows on the Lyne Burn system. The flows thus collected would be directed through a combined sewer overflow with an off-line tank at the same site. A similar structure was needed on the Tower Burn branch, although this was not complicated by the parallel pipes upstream. The sites were at Elgin Street for the Tower Burn and Rex Park for the Lyne Burn branch (locations are shown in Figure 3.2). It is appropriate also to mention the smaller tank at Broomhead and the projected improvements for the Bothwell Street overflow. Details of the tanks are given in Table 3.1 and relevant catchment details in Table 3.2. The Rex Park tank is included for completeness only, as its performance was not monitored.

Site	**Pop'n**	**Vol (m^3)**	**Volume (l/hd)**	**Overflow Type**	**Tank Type**	**Design Method**
B'Head	3,800	400	105	Stilling Pond	Blind +2	SDD
Elgin St	16,000	2,500	156	High-Side Weir	Blind +3	TSR
Rex Park	26,000	3,000	115	High-Side Weir	Blind +3	TSR
Lochgelly /L'nans	4,800	113	27	Storm King	N/A	Max Flow

Notes Blind+2 =Blind tank with two extra compartments
SDD - SDD (1977) See References
TSR - Time Series Rainfall

Table 3.1 - Study Tank and Overflow Details

Site	**Sewered Area (ha)**	**Percent Imp (%)**	**Mean Slope (%)**	**Mean Altitude (m)**	**SAAR (mm)**
B'Head	50.6	45.0	1.5	130	750
Elgin St	143.2	45.4	2.2	100	750
Dixon St/ M^{c}Kane Pk	550	43.1	1.3	90	750
L'gelly/ L'nans	54	46.0	2.2	133	780

Table 3.2 Catchment Details

Off-line structures were chosen for the three locations due to the limited slope available at all sites. High-side weir overflows were specified for the two larger tanks and a stilling pond at Broomhead where the site was restricted and a short overflow structure was necessary.

3.2.3 Dunfermline Outfall Sites and their use for the development of methodologies

Two sites on the outfall sewers from the Dunfermline system were selected for installation of monitoring equipment. These sites did not lie immediately adjacent to a combined sewer overflow and thus did not have a direct use in the determination of the efficiency of an overflow structure. The sites were, however, in use prior to the commencement of monitoring at the three structures reported herein, and most field monitoring techniques employed were developed at the Dunfermline outfall sites. It is considered appropriate to describe them and to include an evaluation of the data derived, due to their importance in the development of the study methodologies.

Furthermore, a preliminary conclusion from the results of testing small-bore samples from these sites was that relationships apparently existed between soluble pollutant determinands and suspended solids concentrations. As a result, only limited testing was carried out later and only the suspended solids concentration was measured for all samples. Later results from all sites showed, in common with other studies, that no such relationships existed. Comparisons of the different determinands are included in section 4.5.

The sites are also included for the intrinsic value of the data obtained, which principally related to sewage qualities. The Dixon Street site on the foul sewage outfall and the McKane Park site on the storm relief outfall receive foul and combined storm flows from the full Dunfermline area, including two of the remaining study sites. Consequently it was considered that the contributing area was relatively homogeneous with few industrial discharges and valid comparisons could be made with the other sites on the catchment. Consequently it was decided to utilise the significant amount of data gathered in order to enhance the general value of the results gained.

3.3 PERIPHERAL SEWER SYSTEMS

Described in this section are the catchments upstream from the Broomhead and Lochgelly/Lumphinans overflows. Pertinent details are included in Tables 3.1 and 3.2

3.3.1 Broomhead

This small catchment which is combined, apart from 20% of its area shown in Figure 3.2, includes the Townhill, Wellwood and Headwell areas of Dunfermline. 95% of the total area is domestic housing, the balance being shops and some garages. A small power station is located on the catchment, however this was mothballed several years before the study commenced and was assumed not to contribute to the drainage system. The storm overflow and tank were completed in 1985 and sewers were relaid upstream during the following two years eliminating upstream overflows and reducing infiltration. It is probable that sediment remained in the system following this construction work which may have had a bearing on the sewage qualities obtained.

3.3.2 Lochgelly/Lumphinans

Some 60% of the burgh of Lochgelly and all of the village of Lumphinans drain to the overflow and the sewers from each area meet a few metres upstream from its inlet. The catchment is shown in Figure 3.3 from which it can be seen that the overflow is approximately 1km from both villages. The catchment land use is very similar to that at Broomhead the only material difference being that the drainage systems of Lochgelly/ Lumphinans are both entirely combined. The overflow at the site is a Storm King hydrodynamic separator and its justification, design and construction have been described by the author (Jefferies & Dickson 1991) and some performance information was presented therein.

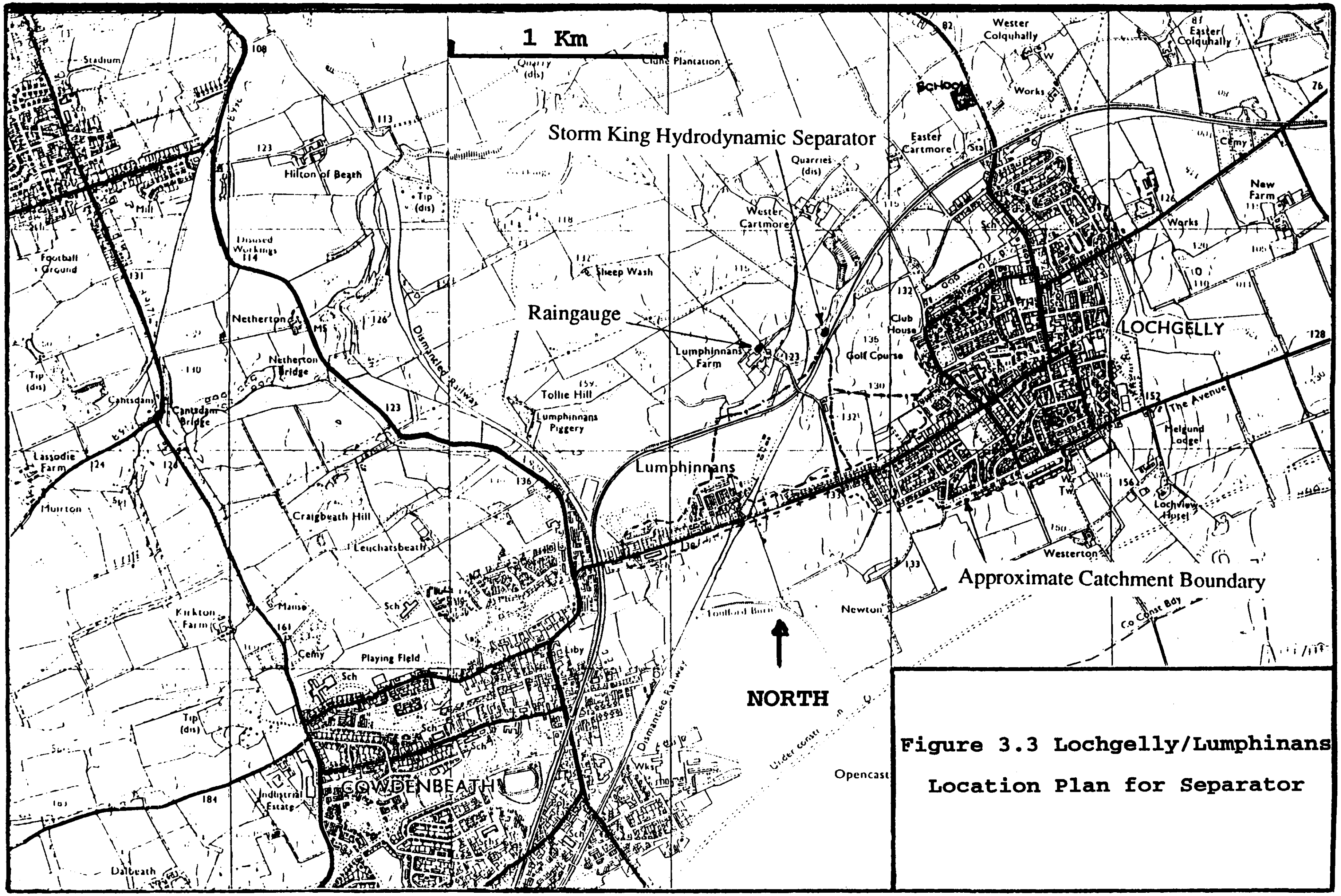
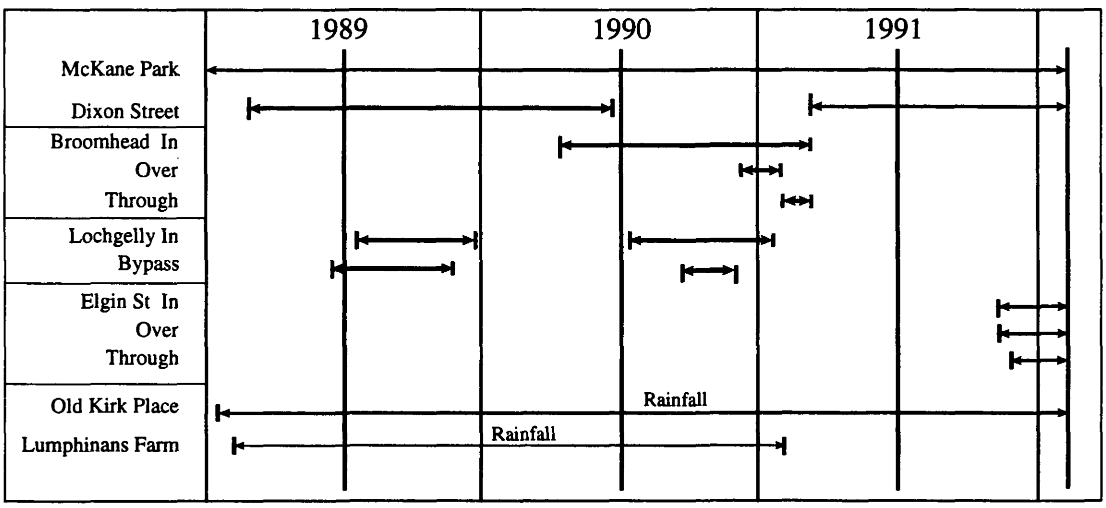


Figure 3.3 Lochgelly/Lumphinans Location Plan for Separator

3.4 FIELD EQUIPMENT INSTALLATION STRATEGY

Off-the-shelf equipment was deployed for the study where appropriate and available. The principal standard items were Flow Survey Monitors and small-bore samplers. In addition specific sampling equipment - Trash Traps and the WRc Gross Solids Sampler - are described.

Three combined sewer overflow tanks and two main outfall locations were monitored over a period of nearly three years as shown schematically in Table 3.3. A range of samplers and monitors were used at each location measuring flows, and sampling to assess pollutant loads as indicated in Table 3.4. The study was equipment-intensive and, whilst sufficient items were available to monitor the various flowpaths at an individual overflow, no attempt was made to obtain data concurrently at different sites. It is considered that the value of the work was not lessened as a result.



Study Terminated in February 1992

Table 3.3 Durations of Monitoring Programmes

Measurement and sampling locations were never ideal and nearly all had at least one hydraulic problem. For example, most flow recording was carried out at, or just upstream of overflows where instruments were prone to being covered by silt, preventing measurement of velocity. Many

Site	Flows	Trigger: Swingo (See Figure 4.4)	Trigger: Clock	Trigger: ESR	Sampler: Epic 1011	Sampler: Gross Solids	Trash Trap
Dixon Street	Yes	Early	-	-	Yes	-	-
McKane Park	Yes	Yes	-	-	Yes	-	-
Broomhead In	Yes	-	Early	Late	Yes	Yes	-
Through	Yes	-	-	-	-	-	-
Over	Yes	-	Early	Late	Yes	Yes	Yes
Spill	-	-	-	-	Yes	-	-
Lochgelly In	Yes	Early	Late	-	Yes	-	-
Bypass	Yes	-	-	-	-	-	-
Over/Spill	-	-	Yes	-	Yes	-	Yes
Elgin St In	Yes	-	-	Yes	Yes	Yes	-
Through	Yes	-	-	-	-	-	-
Over	Yes	-	-	Yes	Yes	Yes	Yes
Spill	-	-	Yes	-	Yes	-	-

Notes:- Durations of installations vary in all cases
Early/Late varies between sites and reflects the acquisition of improved equipment.

Table 3.4 Durations of Equipment Installation

velocity measurements were also made where transverse flow was obvious, but unavoidable. In addition to level and velocity measurements, obtaining samples was also problematic due to variations of concentration within the flow.

Laboratory calibration of flow survey monitors was regularly carried out and is evaluated. Field checking of measurements was also carried out, but less frequently than desirable due mainly to access problems. A number of problems were encountered with the operation of equipment due to the inability of attendance to the equipment during a number of operations. The automatic equipment enabled many problems to be circumvented, however, particular difficulties were experienced with calibration of the flow measuring equipment.

3.5 FLOW MEASURING EQUIPMENT

3.5.1 Equipment Used

Three level measuring devices were available for the study, one of which, the Detectronic flow survey monitor, also measured velocity and (by computation) flow. The flow monitor proved reliable in use (apart from calibration problems) and is considered in detail in later sections.

The flow measuring equipment was extensively calibrated both in the laboratory and at the field installations. The results of the calibrations are presented in Appendix G. The flow monitoring equipment will be reviewed here.

i) The Detectronic Swingo-Logger (discussed in more detail in 3.6.2) was used successfully by Saul & Marsh (1990a) who reported good comparisons of level with the flow survey monitor. This equipment may have been useful at two sewer sites (Dixon Street and M^{c}Kane Park) but access was difficult as was calibration, rendering it impossible to use on its own. In addition it did not measure levels to a high degree of accuracy, the degree of rotation also being a function of the length of the rotating arm and was not to be relied upon at overflows where the equipment had to be very sensitive (Saul & Marsh 1990b). As a result the three Swingo Loggers available were only used for triggering samplers at locations where flow survey monitors were also employed to measure depth continuously.

ii) One ARX Water Level Monitor manufactured by Scan Technologies Ltd was installed for a short period. This equipment had an ideal paper specification for tank monitoring as it had a high quoted accuracy on depth measurement and could directly trigger a number of samplers. The ARX measured depth ultrasonically from a sensor set horizontally below the water surface.

Unfortunately, the first unit supplied suffered from software problems and the replacement failed to give reasonable measurements. The ARX units were also complicated and slow to operate in the field and the data were incompatible with the Detectronic equipment. In consequence, the use of the ARX was abandoned after some two months.

iii) The Detectronic Flow Survey Monitor was developed in collaboration with WRc (WRc 1987) and is now the most popular sewer flow measurement system in the United Kingdom. A housing, or mouse, incorporating a pressure sensor for depth and two crystals for measuring velocity ultrasonically using doppler shift, was strapped to the sewer invert. Standard installation procedure was followed in the study (WRc 1987) and it was found that the sensor remained free from ragging in all but a few locations where sewer solids accumulated.

Very little detailed work on instrument accuracy using this method of flow measurement has been published (Wotherspoon 1990). The WRc/WAA Guide to Short Term Flow Surveys (WRc 1987) includes a nomograph showing regions of validity for flow survey monitors, but no information is included on the amount of testing carried out in its derivation. Weekly site checks and scattergraph analysis of the field data to identify instrument error are recommended. Field calibration checks can be incorporated in the data handling procedure although they are normally only made over a limited range of conditions.

Previous laboratory work by the author aimed at determining limits of accuracy for measurement (Jefferies & Ashley 1985) suggested that errors in flow of $\pm$ 20% might be expected under normal circumstances.
The results published then agreed with the WRc guidance (WRc 1987) and were;

a) the depth of flow should be greater than 100mm;and,
b) velocities should be between 0.3 and 2.5m/s.

Burrows et al (1989), using data from towing tank tests produced results suggesting that, while the depth might be measured to within 25mm, velocity is measured 25% too low. This work, although useful, was flawed by poor assumptions of the manufacturer's calibration procedure (Wotherspoon 1990) and by carrying out the tests in clear water. No other known calibration work has been reported in spite of much commercial flow survey work being carried out using this equipment both nationally and internationally.

3.5.2 Data Acquisition and Transfer

Two flow, level and rain data packages were employed and are described here in brief, since the method of data handling had implications for assessment of data accuracy. Hydromaster was developed by the author (Jefferies et al 1987) to convert and print flow data and run on Apricot PCs when no appropriate commercial software was available. It could handle any pipe cross section and zero offset, but was unable to incorporate drift of level or velocity measurements. It had the further disadvantage that electronic transfer of part data sets was extremely tedious and only selected data were actually transferred for spreadsheet analysis. Accordingly, the use of Hydromaster unfortunately affected the accuracy of the result obtained in some cases. Although this inaccuracy did occur it is considered that this had little effect on conclusions made from analysing the data. However some analysis was limited due to the time taken manually keying in values.

Improved data handling software, FLOAT from Detectronic Ltd was utilised from November 1990. This ran on IBM compatible PCs; allowed the incorporation of calibration checks; and, most importantly, enabled electronic transfer of data as ASCII files. The prime advantages of this software were its better accuracy, together with the speed and efficiency of printing out selected ranges of flow data. However it also suffered from intermittent bugs which occasionally resulted in delay due to data gaps which were relatively easily identified.

3.5.3 Accuracy of Flow Measurement

The accuracy of level and flow measurements are addressed in this section. Extensive laboratory and field calibrations were carried out and these are described in Appendix G. Chapter 4 includes a detailed description of the data from each site where the principal tool for the identification of data discrepancies was the scatter diagram. The following factors are identified as being the most common agencies of variations to the accurate measurement of flowrate;

- **i)** Uniformity or otherwise of the flow pattern;
- **ii)** Full or partial blockage of the sensor by sediment or rags;
- **iii)** The nature of the particulates in the flow;
- **iv)** Lack of penetration of the ultrasound cone;and,
- **v)** Operation outside the limits detailed in section 3.5.1.

In the velocity-area method of measurement, flowrate is the product of the area and velocity. When point measurements are made, flowrate varies linearly with the velocity and the variation of area is dependent on the channel section shape. For circular or near-circular sections, the area is approximately dependent on the square of the depth and as a result the accuracy can be taken as the sum of the velocity and twice the depth accuracy. Measurement of level suffered from drift to a greater extent than velocity although the number of laboratory velocity calibrations was limited. Errors in depth measurement approached 500% as discussed above, but such extremes were always attributable to equipment malfunction and events where such errors occurred were not analysed.

The velocity calibration has shown that the error varied from 26% to 7% as the velocity varied from 0.18 to 0.5m/s. The laboratory tests suffered from the criticism that they were carried out in water which was clearer than sewage and at low water depths (<230mm). This led to underestimation of velocities when Doppler-shift measurement was used. It was invariably found that the accuracy of velocity measurement was in the range quoted above. Figure G.2 (b) (in Appendix G) suggests that the velocity accuracy is $\pm$5%, provided it is >0.5m/s.

The laboratory and field calibrations resulted in a range of flow and level accuracies depending on the type of site and the data processing method used. The conclusions of the exercise described in Appendix G may be summarised as follows:

i) At sites where a good range of field calibration depths were measured, using FLOAT, errors in depth were reduced to 1 or 2%; and,

ii) when Hydromaster was in use this increased by some 5%. The increased error was unfortunate, however the amount of data produced using Hydromaster was prodigious and time was not available for reprocessing.

Consequently, the following accuracies are estimated. The figures are in agreement with the WRc (1987) conclusion that "Flow measurement can be $\pm$10% accurate":

i)	level only at overflows	**Accurate**
ii)	flows [No deposition, Hydromaster]	**10%**
iii)	flows [No deposition, FLOAT]	**7%**

3.6 SAMPLING EQUIPMENT

3.6.1 Small Bore Sampling and Sample Testing Strategy

The small-bore sampling programme relied on four standard Epic 1011 microprocessor controlled samplers. The suction comprised a length of rigid tubing of 18mm internal, 22mm external diameter set into the flow approximately 100mm above invert. This gave a fixed sampling location which was at a variable proportional depth within the flow during high flow events. When installed in sewers, this tube was angled forward ensuring that the flow swept rags and other solids past, thus preventing accumulation and damage. At overflows the rigid tube was angled upstream into the flow. A flexible hose of 10mm internal diameter connected the rigid tube with the 24-bottle sampler. In operation a short period of pressure to clear the hose preceded the main suction phase of the sampler, in which a sight glass filled and subsequently drained excess sample via an overflow tube to give the required volume in the sight glass.

The intake velocity at the rigid pipe was determined by measurement of the time taken to deliver 500ml and was found to be 0.31m/s. At the same time the velocity was 1.02m/s in the flexible hose. Considerable agitation occurred in the sight glass and the samples in general were considered to be representative of the sewage flow.

The Epic samplers used were not programmable for variable time intervals as has been recommended (Saul & Ellis 1990). The advantage of a variable interval is that short time intervals at the start of sampling ensure changes during the first foul flush are monitored while also ensuring that the best use is made of the 24 bottles in the sampler. This problem was circumvented by selecting a short, constant, time interval (normally 5 minutes) and visiting the site within two hours of the samplers triggering. While the Gross Solids Sampler (section 3.6.3) was in operation this presented few problems as it was equipped with a telemetry system. On arrival at site a second bottle set was normally installed with a ten-minute sample interval, and when, on the few occasions that overflow or tank spill lasted longer, a third set at thirty minute intervals was initiated.

Inevitably, during prolonged storms the time intervals were occasionally too short and samples were bulked giving a composite sample with an intermediate time interval. This procedure allowed a high degree of flexibility in the collection of samples and a typical set would be as follows;

5 min intervals	10 samples
10 min intervals	10 samples
30 min intervals	Remainder

At the start of the study for a three month period samples were taken for testing to the Fife Regional Council laboratories at Glenrothes. Unfortunately the programme of normal work within their laboratory could not cope with the uncertain arrival of samples and all later testing was carried out in the laboratories of Dundee Institute of Technology. To economise on testing to be carried out, all samples were tested at the minimum for total suspended solids (TSS).

Testing for more determinands than suspended solids was carried out on the following basis:

DWF	Biochemical Oxygen Demand Chemical Oxygen Demand Ammoniacal Nitrogen pH	All Samples

Storm Determinands as above on selected samples (Approximately every fourth). Samples from certain selected events were tested for all determinands in addition to TSS on all samples.

3.6.2 Triggering Small-Bore Samplers

All sampling sites were fitted with automatic triggering devices and associated timers. All methods relied on a rise of water level to initiate operation of the Epic samplers. Samplers were also triggered manually at the start of some storms. Three different devices were in use, each having a different mode of operation;

Float switch - with timer
Pivoted paddle - attached to a Swingo-Logger
Pressure transducer - in a Flow Survey Monitor

Descriptions of the different arrangements follow, each being illustrated in Figure 3.4;

i) Timer Relay designed and manufactured by WRc principally to trigger three samplers in rapid succession. A simple float switch closure activated a relay closing the sampler contact and starting a stop watch to record the time elapsed from sampler initiation. The float switch tended to stick due to a build up of sludge, and offered only crude adjustment of level, although at overflows the rapid rise which normally occurred avoided the need for fine adjustment.

ii) Detectronic Swingo Logger This device was based on a potentiometer which caused a relay to close when a preset angle of turn was reached. The 'angle of dangle' could be calibrated to give equivalent water level as has been described by Saul & Marsh (1990b). A significant advantage was that normally no mechanical or electronic parts had contact with the sewage flow. It was found, however, that

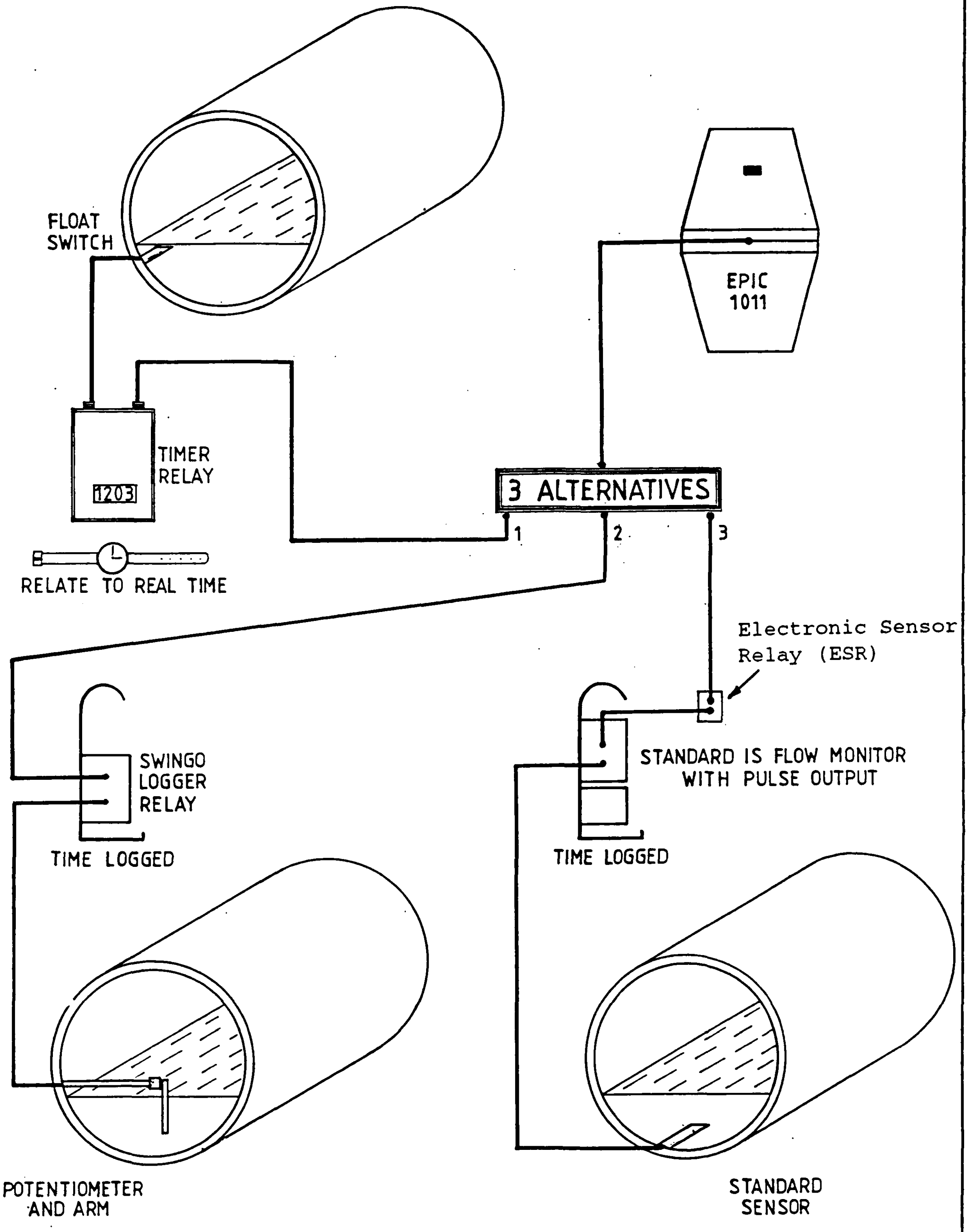


FIGURE 3·4 SAMPLER TRIGGER METHODS

the response of the floating arm was very dependent on location and buoyancy and also that the trigger level was difficult to set accurately. Consequently, and partly since accurate records of level alone were not required, a paddle of very low inertia was attached. This responded very rapidly to being struck by a stream of water and gave a sudden change in output making accurate calibration unnecessary.

iii) External Sampler Relay The ESR unit was attached to a standard Detectronic pulse output logger. This battery powered relay closed upon exceedence of a software controlled trigger level. The great advantage of this unit was that it utilised otherwise proven equipment and the data were fully compatible with the flow monitors. Complete reliability was achieved.

3.6.3 Gross Solids Sampler

The Gross Solids Sampler (GSS) was developed by WRc following the identification of a need to gather data on the behaviour of gross solids at combined sewer overflows (O'Sullivan 1990). The prototype sampler was used in this study, being commissioned in October 1990 at the Broomhead site. The GSS was developed from the Gross Solids Monitor (Cootes et al 1989) and a detailed description of the sampler has been set out by Walsh (1990).

The GSS was constructed inside a standard ISO container as shown in Plate 3.1 and Figure 3.5. At its core was a peristaltic pump with two 100mm diameter suction and delivery hoses. An ultrasonic sensor above the overflow initiated pumping when the water level rose. Two sets of hydraulic valves automatically alternated flows between the two inlet hoses and the corresponding outlets. Discharge was in two bins, each enclosing a COPA sack, to intercept the particulate matter.

Plate 3.1 External View of Gross Solids Sampler at the Broomhead Site

It is only possible to discuss sample volumes and intake velocities in the context of specific installations, as the variable resistance of different hose lengths together with differences of elevation had a marked effect on the characteristics of the peristaltic pump. This contrasts with the Epic samplers in which the overflow tube ensured a constant sample volume. Table 3.5 gives the data measured at the two sites where the GSS was installed. Part of these data have been given in previous reports (Jefferies & Walsh 1991, Walsh & Jefferies 1992) and are included here for clarity.

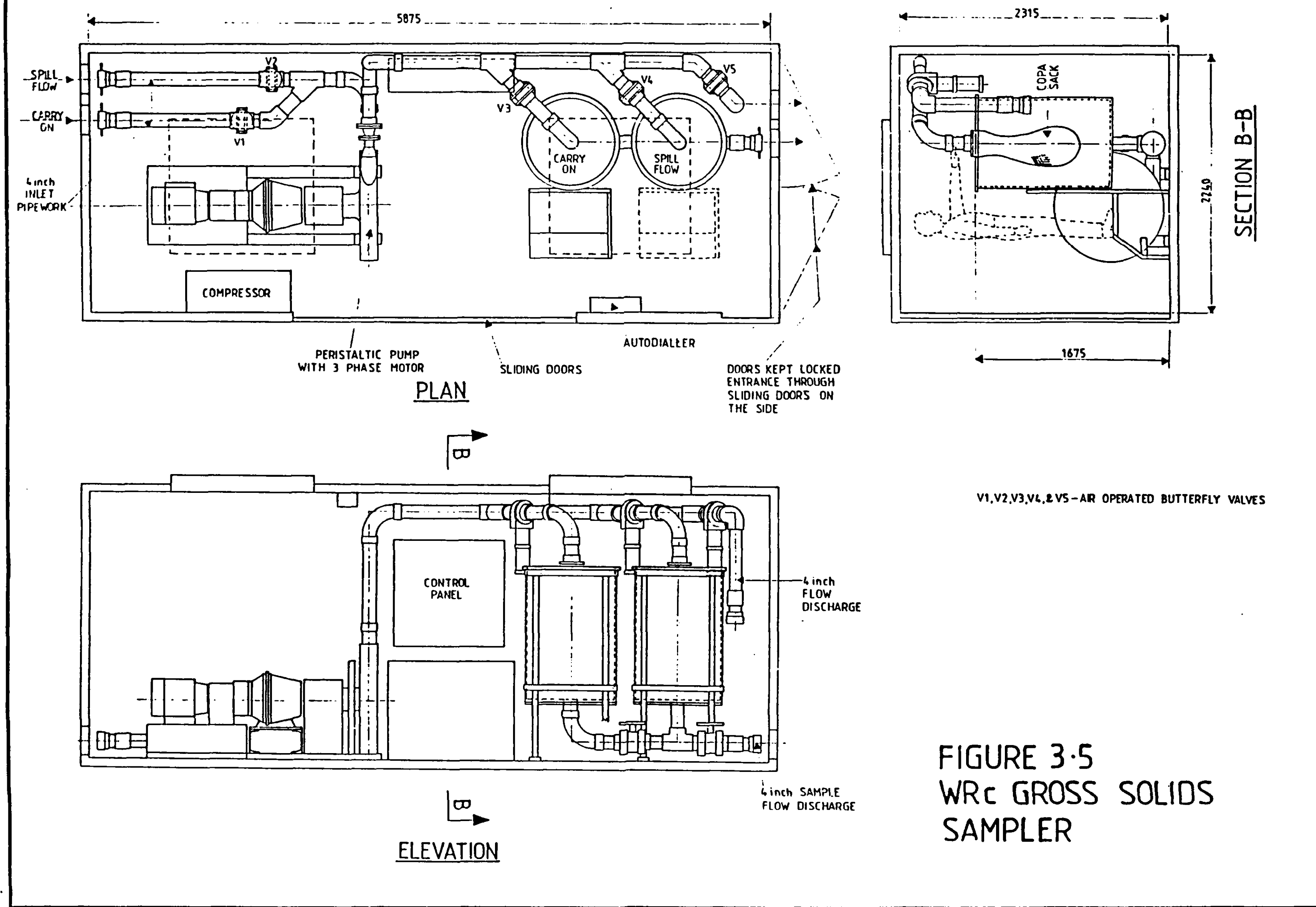


FIGURE 3·5
WRc GROSS SOLIDS
SAMPLER

Pipe Vel refers to the velocity in the intake pipe.	INFLOW			SPILL FLOW		
	Vol (l)	Rate (l/s)	Pipe Vel (m/s)	Vol (l)	Rate (l/s)	Pipe Vel (m/s)
Broomhead						
Test 1	58	3.8	0.48	32	2.2	0.28
Tests 2 - 4	58	3.8	0.48	64	2.2	0.28
Tests 5 - 12	115	3.8	0.48	88	2.9	0.37
Tests 17 - 30	230	3.8	0.48	176	2.9	0.37
Elgin St Tank						
Tests 1 - 16	168	2.8	0.36	168	2.8	0.36

Dates for testing are given in Appendix C

Table 3.5 Flows to GSS at Tank Sites

The GSS collected a single bulked sample during each operating cycle which consisted of a charge period followed by up to 20 samples to each COPA sack. Each sample was preceded by a period of charge from the relevant inlet, by-passing the COPA sack thus ensuring that the correct source was sampled. After 10 samples the charge time increased. Operational details are included in sections 6.4.1 and 6.4.2.

3.6.4 Trash Trap Description

The Trash Trap was devised by the author to be a passive method of trapping visible solids discharged from storm water overflows, thus obtaining data on rates of discharge of such material. It is composed of one or more screens set horizontally just below the discharge from an overflow weir. The trap used is illustrated in Figure 3.6.

The Trap is assembled in sections, since it is required to be passed through manholes. Where open access is possible, longer single unit traps may be used. In this study the trap was set a few centimetres below the weir, low enough not to affect the weir's hydraulic performance, yet sufficiently high to prevent the spill discharge falling vertically. A height some 50-100mm below the weir level was found to be appropriate.

The trap intercepted two types of solids from the flow. Gross solids comprising faecal matter, sanitary towels, condoms etc were all retained

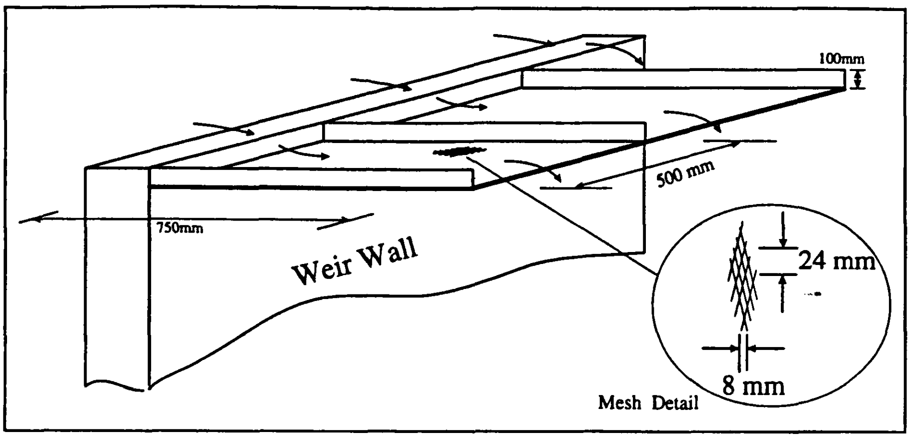


Figure 3.6 Double Trash Trap on Conventional Overflow

provided the flow did not pass straight across. Much smaller particles including shredded paper, foodstuffs and fat particles were also retained. These, together with toilet paper, caused a degree of blinding of the diamond mesh openings. Where blinding was considerable the flow would pass over the trap carrying the gross solids with it. This behaviour was confirmed by the testing discussed in section 3.6.5. In such cases results were ignored.

Two types of observations were made after an event;

i) Visible solids were lifted from the trap, taken to a laboratory and weighed after fan drying for two hours. The amount collected was expressed as a damp weight. This procedure was used to ensure that the plastic material did not retain pockets of water and that the bias introduced by the small amount of very absorbent material was minimised.

ii) The degree of blinding expressed as a percentage of the full trap area after removal of visible solids and following a visual estimation of the blinded area. Zero percent blinding might have a significant amount of visible solids present, but no blinding since all of the mesh diamonds would be visible after removal of the solids. Conversely, a trap which was 100% blinded may have had no visible solids present, either because they were absent from the flow, or because they had been swept off.

3.6.5 Trash Trap Testing for Visible Solids

Full-scale laboratory tests were carried out on the Trash Trap using typical visible solids, since controlled field testing was not possible. The tests showed that, provided the degree of blinding was less than 33%, virtually all visible solids were retained. When the vertical drop from the weir crest to the trap was 100mm or less, the energy of the flow was insufficient to wash off trapped material. The tests also showed that, provided the flowrate per unit width was less than 75 l/s per metre, no material was carried over. The conclusions drawn from the lab testing were:

i) - With blinding less than 33% all visibles were retained provided the flowrate was less than 75 l/s/m;and,

ii) - With blinding exceeding 33% some visibles were not retained on the trap.

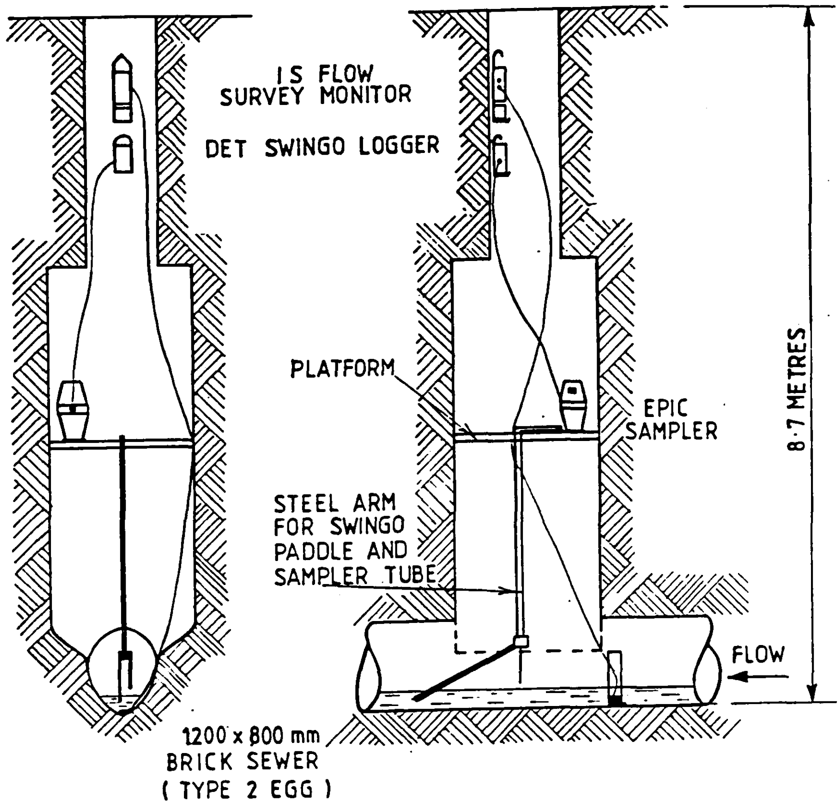
Trash Traps were installed on the overflows at all study sites.

3.7 STUDY SITES

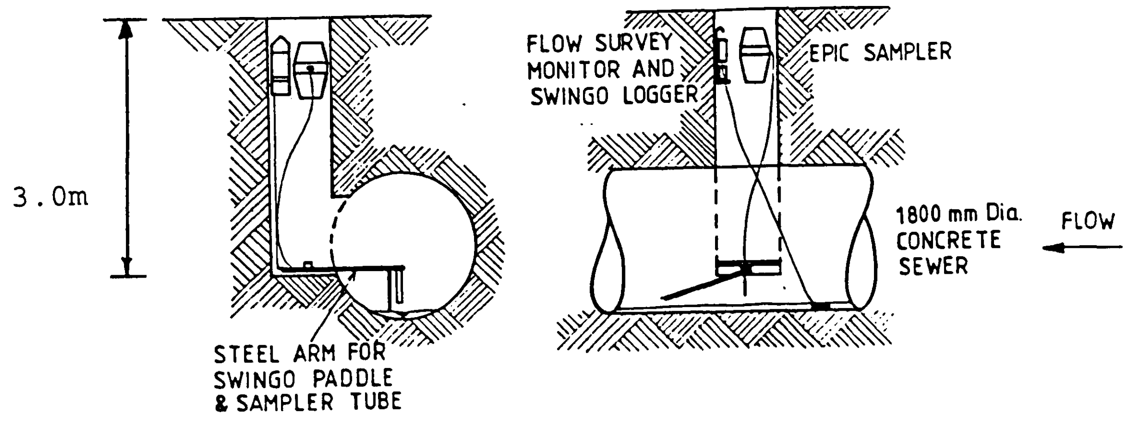
3.7.1 Dixon Street and M^{C}Kane Park

Although these sites were some 300m apart, they represented the through and spill flow from the series of overflows on the Dunfermline sewer system. The sewer at the Dixon Street site was egg shaped, 1200mm high and 800mm wide and the M^{C}Kane Park site was on the storm relief outfall sewer where the concrete pipe had a diameter of 1800mm. In contrast to the spacious Dixon Street manhole, that at M^{C}Kane Park was shallow and cramped. Figure 3.7 shows details of the equipment installation.

i) Dixon Street was a two stage manhole some 8.7m deep with an intermediate platform on which the sampler was placed. Sampler triggering used Swingo-loggers between May 1989 and June 1990 but their performance was not successful with frequent premature sampler operation. The suspected cause



a) DIXON STREET SAMPLING SITE



b) McKANE PARK SAMPLING SITE

Figure 3.7 DUNFERMLINE OUTFALLS SAMPLING SITES was vibration of the vertical arm, set off by the sampler tube when it became submerged in the high velocity flow. For the sampling period in 1991-92 (see table 3.3) an ESR unit was installed.

Equipment utilised at the site was:

IS flow survey monitor	All sampling periods
Swingo Logger	May 1989 - June 1990
ESR Unit	June - Oct 1991
One Epic 1011 sampler	All sampling periods

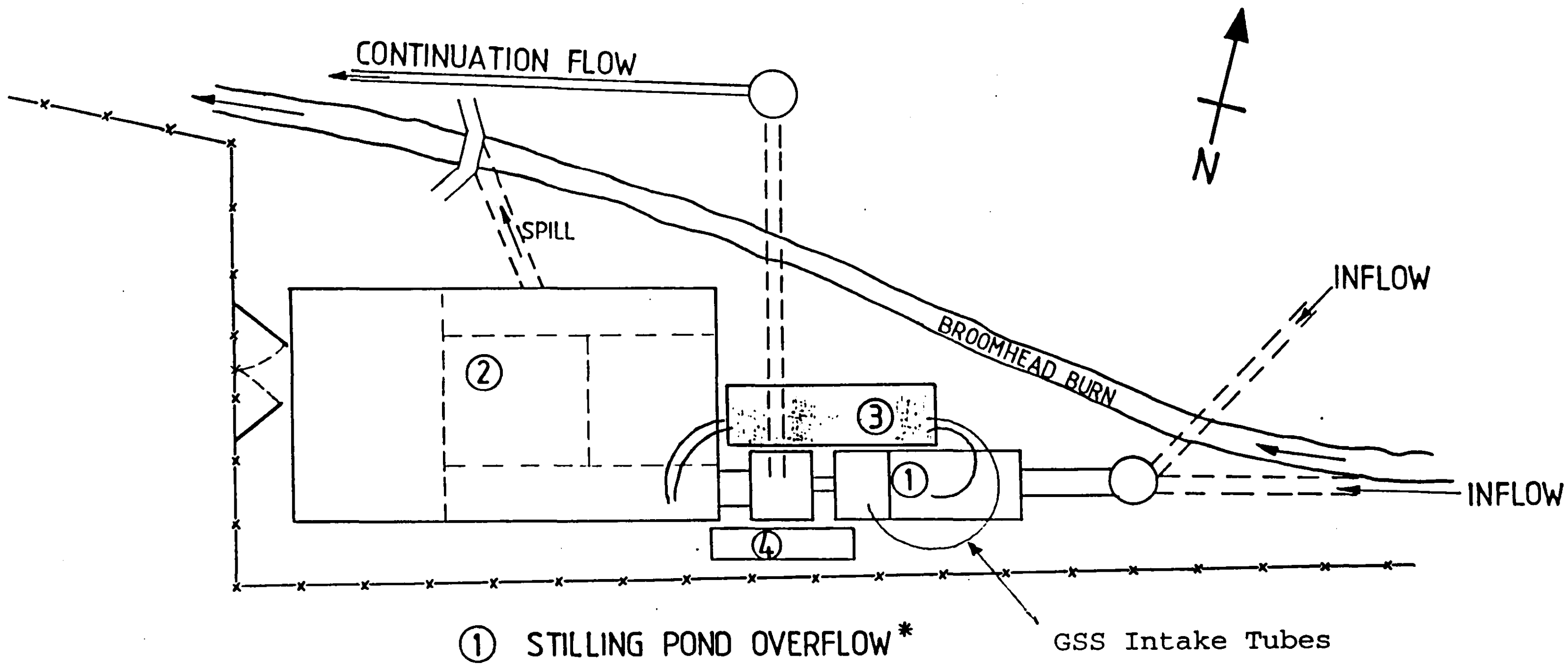
An accumulation of coarse sediment on the sewer invert periodically occurred, obscuring the ultrasonic velocity crystals.

ii) M^{C}Kane Park This storm relief sewer was normally dry apart from a small infiltration flow and afforded easy access to equipment. Sampler tube and Swingo paddle were mounted on a horizontal steel arm projecting from the manhole benching. The arrangement is illustrated in Figure 3.7 (b) and operated successfully due mainly to the equipment being rarely submerged by the flow. The access manhole at M^{C}Kane Park was very cramped and the following equipment was installed:

IS flow survey monitor	All sampling periods
Swingo Logger	May 1989 - June 1990
and	June - Oct 1991
One Epic 1011 sampler	All sampling periods

3.7.2 Broomhead Overflow and Tank

The installation consisted of a stilling pond overflow with 750mm inlet and 300mm diameter throttle pipes. The overflow discharges into a partitioned tank with a blind section and two extra compartments (See Figure 2.4(c)). The total volume of the tank was 400m^3. The tank spill weir was some 100mm higher than the overflow weir which was thus drowned during spill to the Broomhead Burn.



① STILLING POND OVERFLOW*
② OFFLINE TANK
③ GROSS SOLIDS SAMPLER
④ EQUIPMENT CABINET

* See Figure 3.9 for details of overflow and equipment installation.

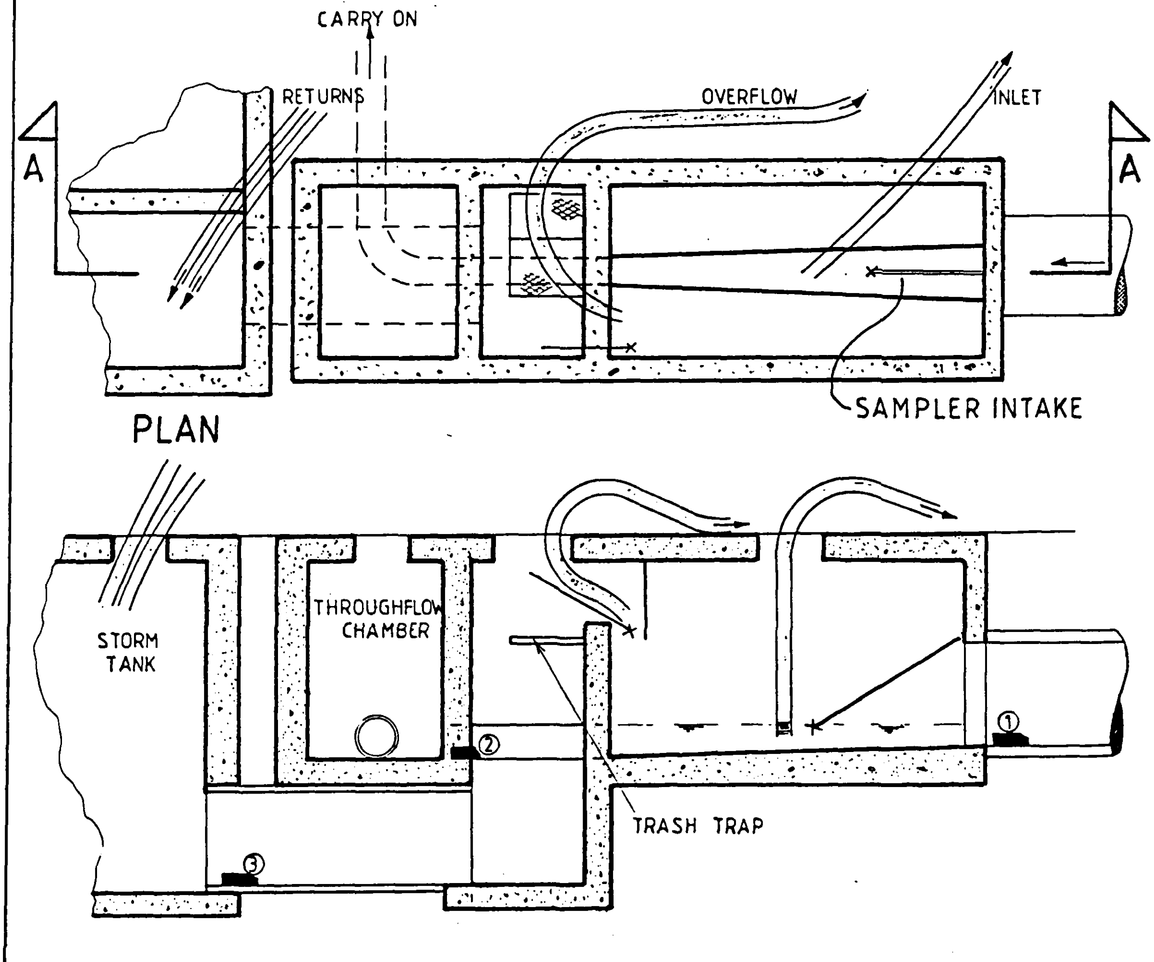
FIGURE 3·8 BROOMHEAD SITE PLAN

DATES	OVERFLOW	DATES	INFLOW
10/10/90 TO 26/10/90	BASE OF OVERFLOW CHAMBER	10/10/90 TO 6/12/90	150mm ABOVE DWF CHANNEL INVERT
28/10/90 TO 6/12/90	75mm BELOW WEIR	6/12/90 TO 18/3/91	LEVEL WITH BENCHING
6/12/90 TO 18/3/90	LEVEL WITH WEIR		

LOCATION OF GSS INTAKES

LOGGER LOCATION	EPIC INLET LOCATION
① – INLET PRINCIPAL MEASUREMENT ② – THROUGHFLOW ③ – OVERFLOW	INLET — 100mm ABOVE DWF INVERT 1.5m INTO CHAMBER OVERFLOW — 30mm BELOW OVERFLOW WEIR AT ONE SIDE SPILL — 30mm BELOW SPILL WEIR AT EAST SIDE

SAMPLER AND LOGGER LOCATIONS



SECTION A-A

FIGURE 3.9 BROOMHEAD STILLING OVERFLOW DETAILS

Figure 3.8 shows the site layout alongside the Burn and Figure 3.9 gives details of the equipment installed which are summarised as follows;

Flow survey monitors	3 No - See Figure 3.9
Timer switch	Until Sept 1990
ESR Unit	After Sept 1990
Epic 1011 samplers	3 No - See Figure 3.9
Trash Trap on overflow	Two panels
Gross Solids Sampler	Inlet & Overflow Oct 90 : March 91

The inlet and overflow samplers were normally programmed on a 5min interval with simultaneous triggering. On occasions, dependent on prevailing and antecedent conditions, a 10min interval would be used to allow extended sampling periods without attendance. The spill sampler was triggered manually on all successful occasions. The locations of the sampler inlets are shown in Figure 3.9. Samples were combined before testing in accordance with the principles set out in section 3.6.1. The Trash Trap at Broomhead was located in the overflow chamber 50mm below the weir level. Trash Trap data from events when spill occurred were rejected as the traps became surcharged, although no field evidence was obtained showing that material previously caught on the trap was subsequently removed by submergence.

The GSS was positioned alongside the overflow structure as shown in Figure 3.8. The site was level causing the suction pipes to have low points. Although this created no problems during normal operation since the peristaltic pump was able to draw both air and water, during freezing weather in January and February 1991 the trapped water became frozen preventing suction and invalidating some data.

3.7.3 Elgin Street Overflow and Tank

The installation consisted of a high-side weir overflow with three separate inlet pipes. The principal inlet had a diameter of 900mm with a slope of 1 in 10 immediately at the inlet, while the secondary inlets had slack gradients and diameters of 450mm and 675mm respectively. The configuration of the structure was such that the inflow turned through 90° prior to approaching the overflow section. The overflow weir length was 12m either side and the throughflow was controlled by a standard venturi flume with 260mm throat width. As at Broomhead, discharge from the overflow was into a partitioned tank, the total volume of $2500m^3$ being divided into one blind tank and three storage compartments. The tank spill weir was some 100mm lower than the overflow weir, which in consequence had free surface flow under all flow conditions. Figure 3.10 shows the site layout and Figure 3.11 gives details of the equipment installed, which are summarised as follows:

Flow survey monitors	3 No - See Figure 3.11
ESR Unit	Throughout study
Epic 1011 samplers	2 No - See Figure 3.11
Trash Trap on overflow	Three panels
Gross Solids Sampler	Inlet & Overflow Nov 91-Feb 92

The inlet and overflow samplers were programmed on a 5min interval for six events and 10min thereafter and spill samples were obtained manually, only one event being sampled. Samples were combined before testing in accordance with the principles set out in section 3.6.1.

The Trash Trap at Elgin Street was located to the right of the overflow and 50mm below the weir level. The GSS was positioned alongside the overflow structure as shown in Figure 3.10. The site was level causing the suction pipes to have low points and, as at Broomhead, the static sewage in the pipes was liable to freeze.

3.7.4 Lochgelly/Lumphinans Overflow

The background and first stage of monitoring at this site during 1989-90 have been described previously (Jefferies & Dickson 1991). Further monitoring was carried out in 1990-91 and the results of the full study are reported. A site plan is included in Figure 3.12 for completeness, along with details of the Trash Trap installation.

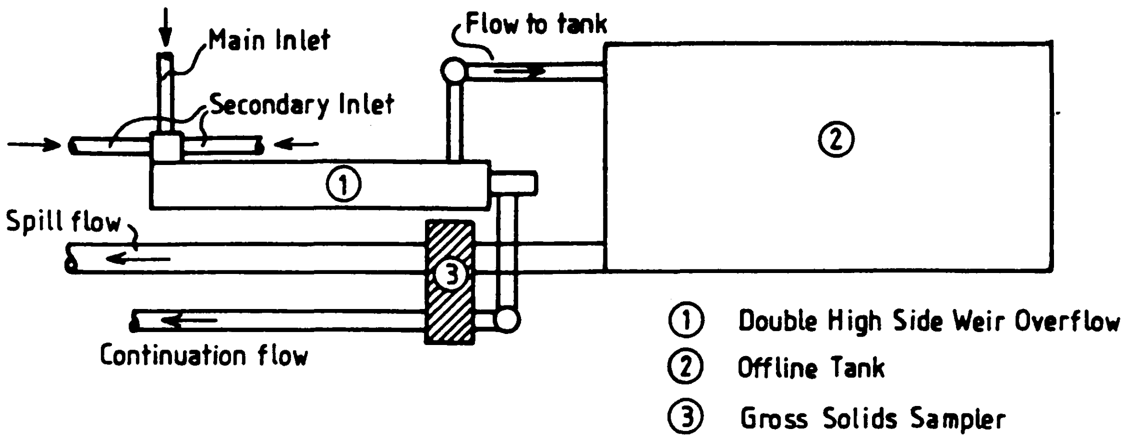


FIGURE 3.10 ELGIN STREET SITE PLAN

18/11/91 to 3/2/92	Inlet Start of weirs Middle of DWF channel	Overflow Mid point of right weir Behind scumboard 25mm below crest

Location of GSS Intakes

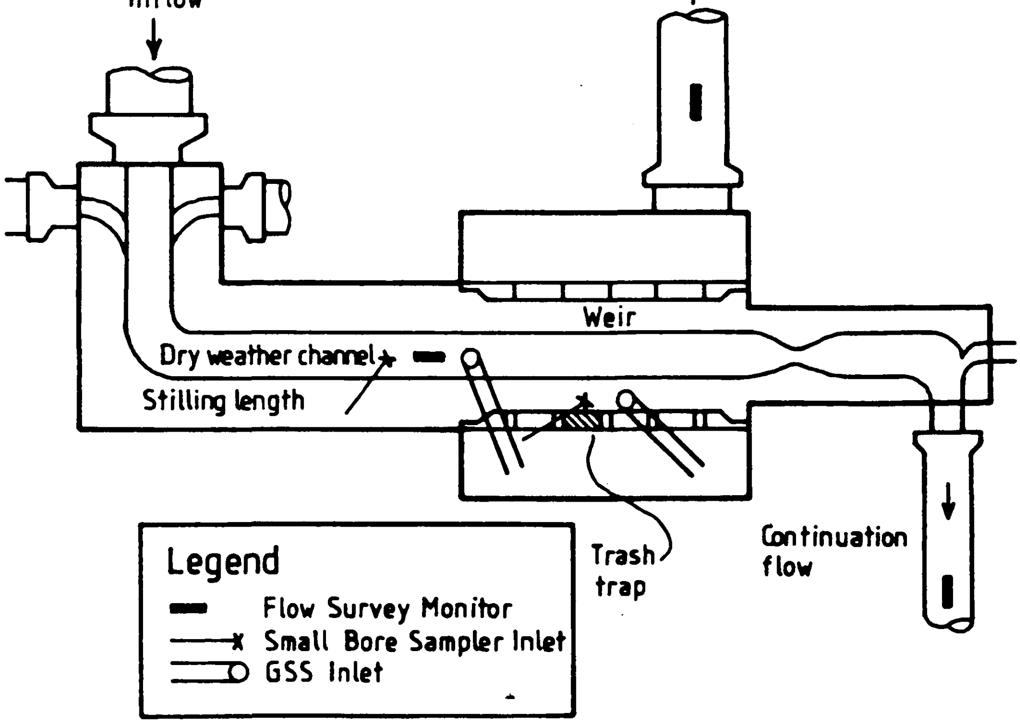


FIGURE 3.11 DETAILS OF DOUBLE HIGH SIDE WEIR OVERFLOW AT ELGIN STREET

The overflow incorporated two Storm King hydrodynamic separators which, in addition to the claimed treatment of the overflowing storm sewage (section F1.4 in Appendix F) have approximately 110m^3 of off-line storage. Tank B only was chosen for sampling even though it was discovered to be carrying approximately 2/3 of the total overflow (See section 4.2.3). When interpreting data, discharge rates and masses have been scaled to account for the observed differences between the tanks. Equipment installed at the Lochgelly/Lumphinans site was:

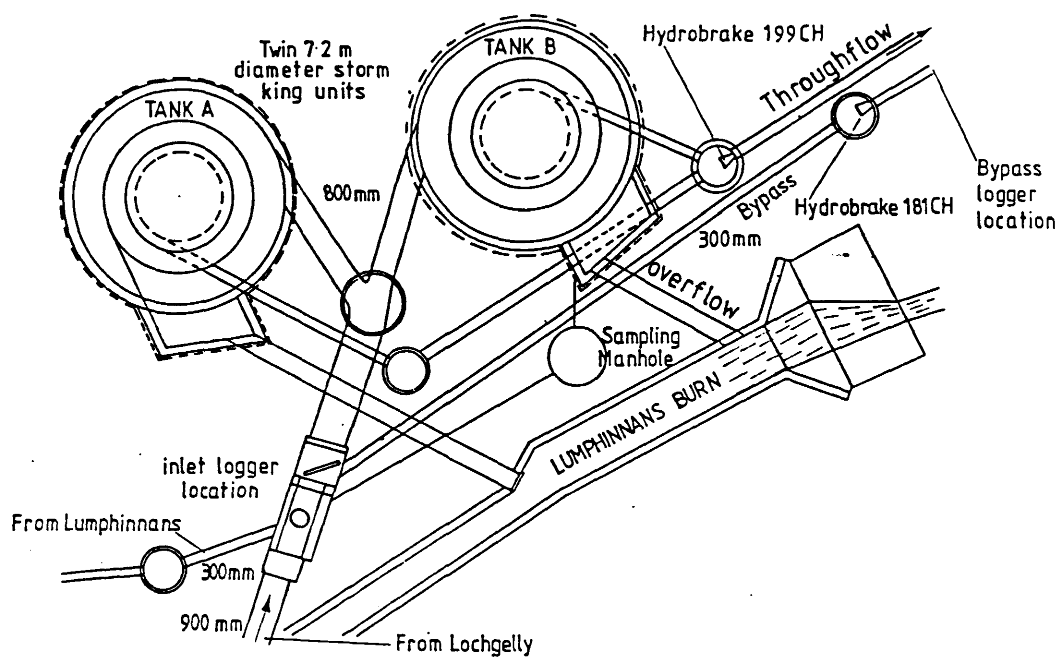
Flow Survey Monitor	1 for full study, 1 intermittently
Timer switch	2 on inlet and spill
Swingo Logger	On inlet until Nov 1989
Epic 1011 Sampler	Inlet & Overflow
Trash Trap on Overflow	3 Panels

The samplers and other equipment were placed in a purpose built manhole shown in Figure 3.12(a). This was ideally suited for the spill flows, but was below the inlet manhole water level which led to syphoning of flows, frequently invalidating inlet sample sets. The intake positions were as follows:

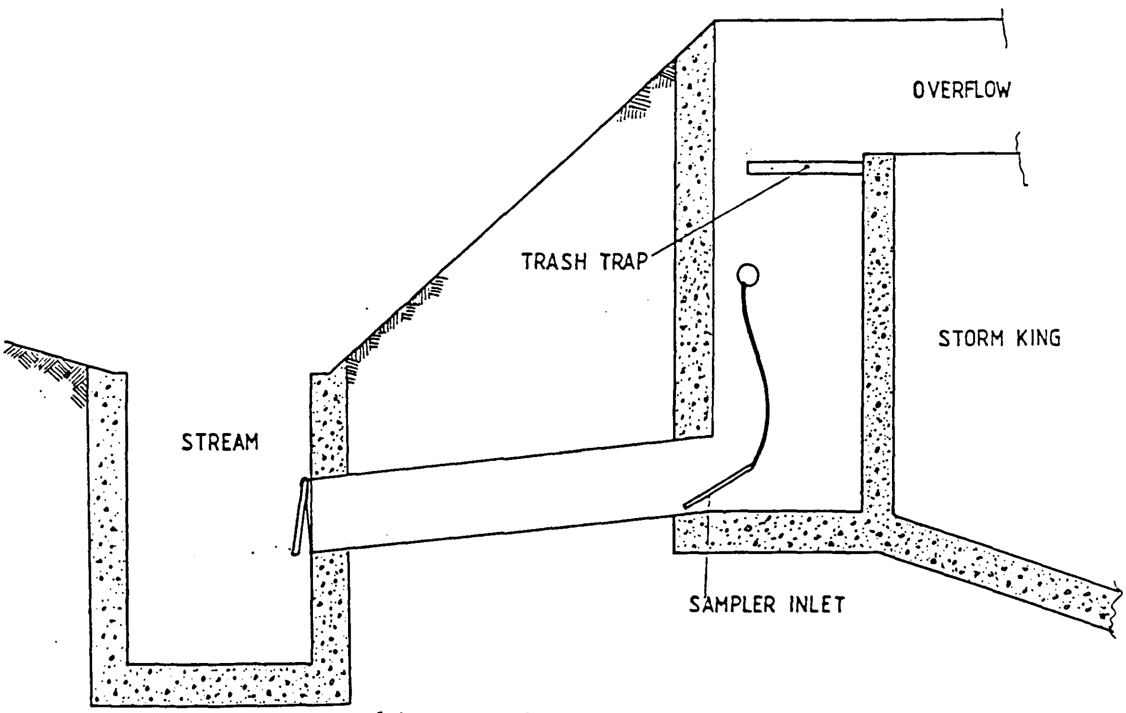
Inlet - 75mm above invert of bypass pipe (300mm dia)

Overflow - At invert of discharge pipe (This pipe was always observed to be free from sediment and, during spill was partly full due to backing up by flap valve)

The Trash Trap was installed 50mm below the spill weir. The Trap covered 90% of the weir and was straight rather than curved to suit the Storm King wall. Some very slight loss of flow past the Trash Traps was observed to occur.



a) Site Plan



b) Sampling Arrangements

Figure 3.12 Lochgelly/Lumphinans Site Details

3.8 RAINFALL MEASUREMENT

Rainfall measurement was not given a high priority in the study, as detailed modelling of catchment responses was not required. It was used to indicate the general magnitude of events and to assist in the determination of antecedent dry weather periods. Each area was assigned only one raingauge, a Cassella tipping bucket with a Technolog logger, compatible with the Detectronic data retrieval software. The bucket size was 0.2mm. The locations are shown on the relevant plans, Figures 3.2 and 3.3.

The Dunfermline catchments were served by the gauge in a back garden at 11 Old Kirk Place. It has been shown (Au Yeung 1990) that this gauge representatively measures the catchment average rainfall.

Lumphinans Farm was the site of the gauge for the second catchment. Although it was not located within the catchment area, it was approximately equidistant from the villages and, significant in this area, not prone to vandalism.

3.9 CONCLUSION

The sampling, as indicated in Table 3.3 extended over a three-year period in 1989-92. A wide range of flow and quality events were monitored. Table 3.6 is included as a summary of the events. The event data is listed in Appendices C and D, and Table 3.6 is included as a summary of the events.

Numbers of Events Monitored in Each Category					
Site	Flow and Quality Data				Flow Data Only
	Inlet	Over	Spill	Concurrent In/Over/Spill	
Dixon & McKane	21	24	N/A	15	60
Broomhead	24	28	4	4	53
Elgin Street	9	5	1	1	20
Lochgelly	4	N/A	14	4	40

Table 3.6 Summary of Event Data Gathered

CHAPTER 4 INTERPRETATION OF FIELD DATA

But facts are chiels that winna ding
An' downa be disputed
Robbie Burns A Dream

4.1 INTRODUCTION

Data for the study were obtained in two basic formats, electronically gathered flow and level information, and quality data, following testing of discrete samples. Flow data transfer and initial analysis utilised two programs, Hydromaster, developed by the author (Jefferies et al 1987) and FLOAT from Detectronic Ltd. Data management and analysis utilised QUATTRO-PRO from Borland International, 4585 Scotts Valley Drive, Scotts Valley, CA 95066 USA. Flow data from each site were examined for accuracy in the light of laboratory and field data (see section 3.5), utilising scatter graphs, and by comparison with other equipment at the same site. On acceptance of the data, flow relationships were developed where appropriate for each site as described in section 4.2.

The fieldwork programme required intensive effort and the reliability of the results can only be assessed in the light of the methods used and observations obtained at each site. Section 4.3 has been included to clarify both the extent to which practical difficulties influenced the data gathering process and to assist in the interpretation of the flow and pollutant behaviour at the different sites. Due to the local circumstances, slightly different rules were applied to define events at the various locations. The procedures are explained through a description of the analysis of one characteristic event at each site.

The catchments studied were predominantly residential, and similarities between them were important in supporting comparisons of behaviour at overflows. Events where quality data were obtained were characterised in terms of their first foul flush, mean concentration and pollutant load behaviour in section 4.4. It was concluded from these analyses that the results for suspended solids were found to compare well with other published data. Data for other pollutant determinands were more limited requiring the pooling of all information to allow comparisons to be made.

Early in the study, positive relationships were developed between BOD, COD and suspended solids concentrations from data obtained at the Dixon Street and McKane Park sites. Although previous studies (eg Pearson et al 1986) have shown no relationships between TSS and NH_3 and BOD were likely with pollutants derived from domestic sewage, regression straight line fits were sufficiently good to support the view that relationships would remain once the full data set for this study was gathered. In consequence and partly also due to resource limitations much higher priority was placed on testing for TSS, and only selected samples were tested for the remaining determinands as discussed in 3.6.1.

The different pollutant determinands gathered throughout all sites during the study are compared in section 4.5 and are shown to have no correlation in spite of the inclusion of the McKane Park and Dixon Street data which produced the good fits. As a result, use of the data derived for BOD, COD, NH_3 and pH has been limited to demonstrating the similarities between the different sites and the commonalities with data from other locations.

Similarities between the catchments are also justified on the basis of their dry weather flow behaviour. It is demonstrated in section 4.6 that the variation of flows, together with pollutant concentrations and loads, were similar between the different catchments and with data from elsewhere, as do pollutant concentrations and loads.

It is contended following the data interpretation presented in chapter 4 that the catchments were typical of United Kingdom conditions, and that comparisons between the different overflow sites are valid.

4.2 HYDRAULIC CALIBRATIONS

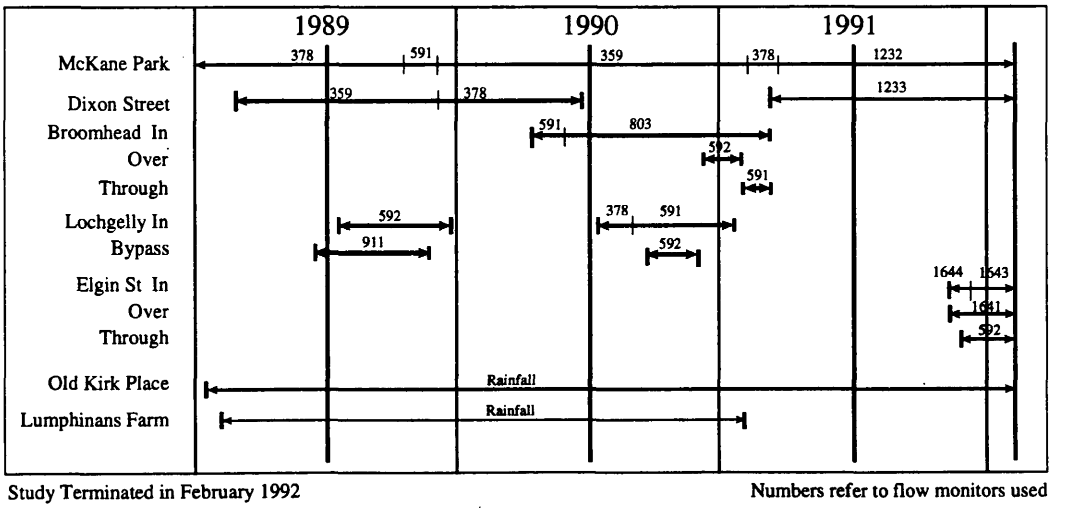
Each site has been considered in turn for the accuracy and reliability of the flow data obtained. This examination had the following objectives;

- **i)** Determination of the accuracy of measurements.

- **ii)** Identification of malfunctioning equipment.

- **iii)** Determination of level-flow relationships for over- and through-flows at overflows for use in interpretation of quality measurements. Such equations were used only where flow data were not gathered or were judged inaccurate.

A number of flow monitors were employed at each site and periodic removal for calibration checks and repairs was necessary. Table 4.1 shows details of the equipment utilised for the duration of the study. The M^c^Kane Park and Dixon Street sites are not included in this hydraulic assessment as their data have been used principally to understand the quality relationships addressed in sections 4.4 to 4.6.



Study Terminated in February 1992

Numbers refer to flow monitors used

Table 4.1 Flow Monitor Deployment

4.2.1 Broomhead Overflow and Tank

Relationships were developed for through-, over- and spill-flows at the overflow and off-line tank. Such relationships were required as there were insufficient flow monitors to allow continuous monitoring of these three flows in addition to measurements at the inlet. Flow monitors deployed at the site are itemised in Table 4.1 from which it will be noted that level and velocity (when not ragged) were monitored at the inlet for the duration of the study.

It was impossible to carry out in-situ calibration checks of velocity at any of the three monitor locations due to access difficulties. Level calibration measurements were, however, made regularly at the inlet. Laboratory and field calibrations of the equipment are included in Figures G.1 to G.3 in Appendix G, from which the following was determined;

i) Inlet monitor (ID 803) showed a zero drift of 40mm but no span error. This was considered to be acceptable as all relationships developed relate directly to the level measurements which - as highlighted by Figure G.3(c) - showed great consistency.

ii) The continuation flow monitor was located on the throttle pipe which always ran full during events. The flow accuracy was assumed to be $\pm$7% as discussed in section 3.5.3.

iii) The overflow monitor located on the pipe at entry to the tank was only partly successful. Suspected errors were attributed to flow discontinuities caused by high velocity flow from the overflow. In a number of events unreasonably high flows were deduced and flows from selected events only at this site were used to develop an overflow rating curve. A small number of depth checks were made while the tank was spilling and there was no reason to suspect depth measurement errors.

A number of hydraulic controls operated at the overflow as follows;

i) The 225mm diameter throttle pipe controlled throughflow.

ii) The overflow weir operated with free discharge to the off-line tank until it filled and drowned out the weir.

iii) The spill weir discharged freely to the watercourse. Before spill occurred the level in the overflow chamber backed up and hence the flow depth could be used to monitor spill flows.

The three flows listed above could all be determined from continuous level monitoring within the overflow chamber, utilising the flow relationships which were developed and which are included in graphical form in Figure 4.1. These plots show relatively little scatter, demonstrating that unique relationships apply in each case and the flows may be predicted from level measurements taken at the inlet.

The throughflow relationship has a change point and two equations have been developed as illustrated in Figure 4.1(a), the change in the relationship corresponding to the transition to full-bore flow within the inlet pipe. Figure 4.1(b) shows the data pertaining to the surcharged inlet conditions. This figure also illustrates a hysteresis loop, higher flows being recorded after the peak of the event due to draining of the system which would allow higher hydraulic gradients.

The relationships presented in Figure 4.1(a&b) show relatively low r^2 values in spite of the modest scatter of points. This is however considered to be acceptable as all values lie within $\pm 7\%$ of the fitted line and the throughflow is relatively low in comparison to the over- and spill-flows.

The free discharge overflow relationship developed from the data set illustrated in Figure 4.1 c) is;

$$Q = 0.0618 \times H^{3/2} \qquad \textbf{4.1}$$

Where H = Observed Level - 810 (dimensions - mm)

This data-derived equation compares with the theoretical weir equation which for a 1.7m long weir is;

$$Q = 0.092 \times C_d \times H^{3/2} \qquad \textbf{4.2}$$

The resultant value of coefficient of discharge, C_d is 0.674. Although not unreasonable, this is higher than standard values, for example Subramanya (1982) suggests C_d = 0.528. The higher value is probably due to suppression of the flow by the confining weir end walls.

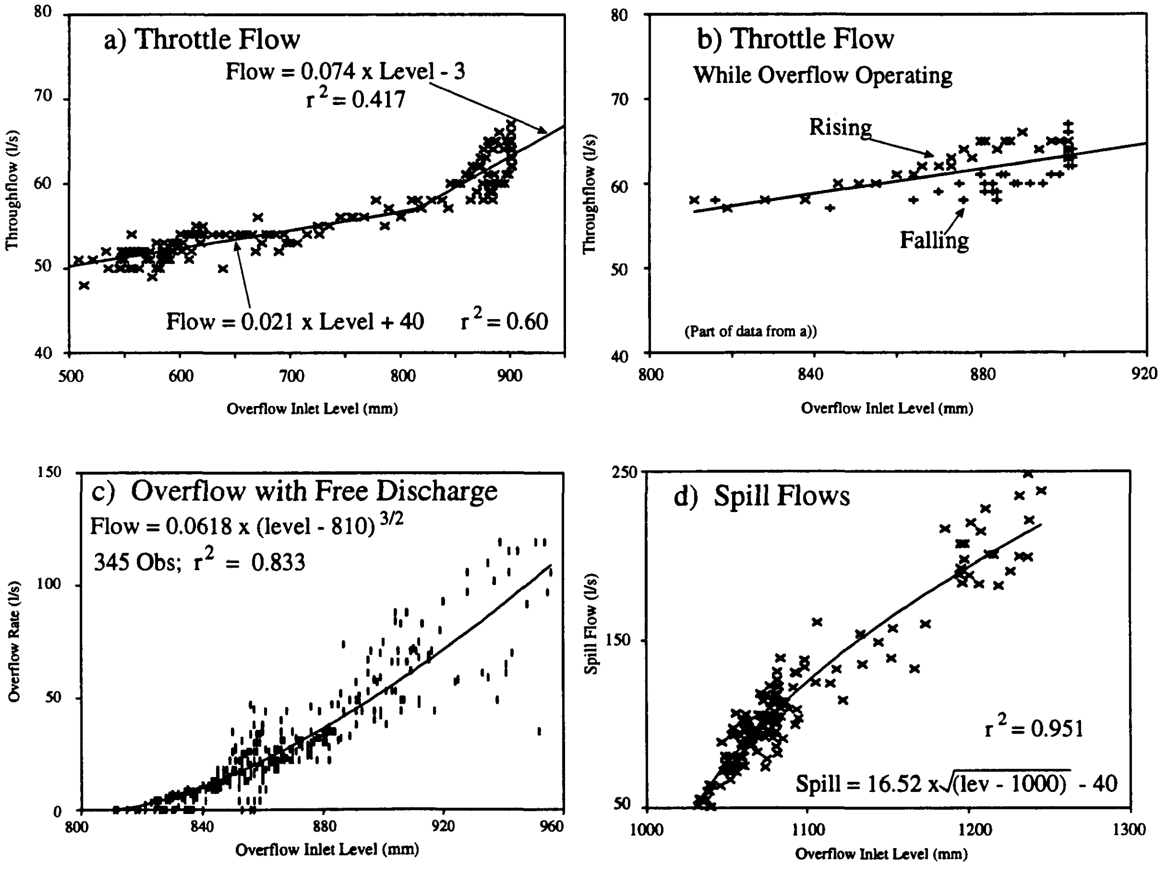


Figure 4.1 Broomhead Overflow - Hydraulic Relationships

Hydraulic control of spill flows was at the final spill weir. A weir type relationship might have been expected to apply to the data, however, with level measurements made at the overflow inlet, the level and flow data, plotted as Figure 4-1 (d) clearly did not follow this form of relationship. A series of regression analyses on different types of expression were tested. The best was found to be;

$$Q = 16.52 \times H^{1/2} - 40 \qquad \textbf{4.3}$$

Where H = Observed Level - 1000 (dimensions - mm)

This relationship resembles that of a submerged orifice and it is suggested that this was caused by the connecting pipe between overflow and tank behaving as an orifice. Equation 4.3 was selected as it had the highest r^2 (0.951) of the relationships tested.

4.2.2 Elgin Street Overflow and Tank

Three flow survey monitors were deployed for the duration of the study at the Elgin Street installation and consequently there was less need to develop relationships between levels and flows. The inflow monitor suffered from frequent covering of the sensor by gravel thus preventing velocity measurements. This location also had a compound cross-section, comprising a dry weather flow channel with wide benching which caused flow discontinuities, and the inlet monitor was only used to record levels and trigger samplers. The inflow was taken as the sum of the throughflow and overflow measurements but could only be checked independently at low flows. Such checks were not carried out as they were considered to be of little relevance.

The throughflow was monitored continuously during the study period. The variation of throughflow with inlet level is shown in Figure 4.2, the peak flowrate observed being 440l/s. The lack of scatter evident in Figure 4.2 was used to check for misreading of the inlet logger which was found to have malfunctioned for a period. The calibration of the flume could not be checked directly, as the inlet logger was located at the upstream end of the overflow weir, and the head loss between this location and the flume entry could not be determined.

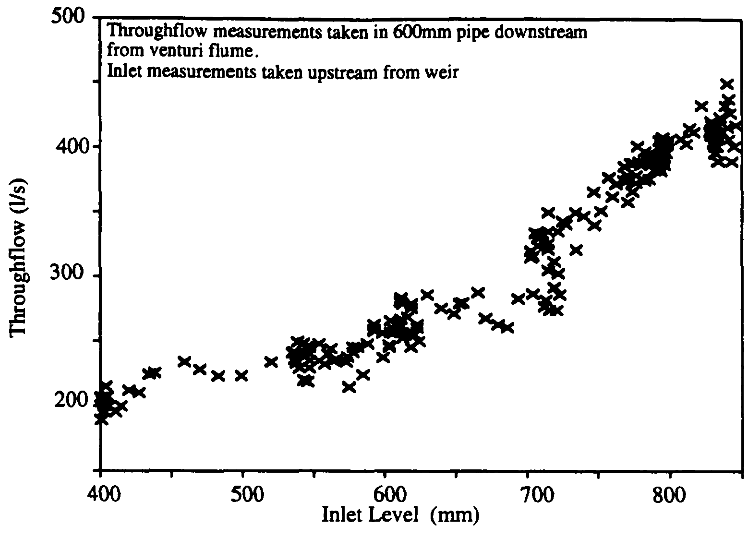


Figure 4.2 Elgin Street Overflow - Comparison of Inlet Level and Throughflow

Low flow depths at the overflow monitor occurred at the start of events due to the large diameter (1200mm) of the pipe connecting the overflow and storage tank. Velocities were also low when the tank was full or part full due to backing up of the connecting pipe, both factors contributing to errors which required correction. Checks were possible using volumes determined from the rise in tank level which was measured at the overflow monitor as discussed in section 4.3.4. Spill occurred twice during the study, however spill flow was not independently monitored. Its commencement was determined from the overflow monitor level and once spill started, all overflow was assumed to spill.

4.2.3 Lochgelly Twin Hydrodynamic Separators

Two flow monitors were employed, at the 900 mm inlet and on the 300mm bypass pipes. Neither location proved entirely satisfactory due firstly to a connection just upstream from the inlet sensor location and because in the bypass low depths of flow occurred. Ragging also covered the inlet sensor due to low velocities when the installation was overflowing.

The inlet site was used to monitor the tank levels and, by association, the head on the two hydrobrakes and the overflow weirs. It also enabled dry weather flow monitoring to take place, although with a pipe as large as 900mm diameter this was occasionally unsatisfactory due to shallow depths. Flow not discharged through the bypass pipe backed up to a low diversion weir and entered the Storm Kings. When full, spill was to the local watercourse, although the rates of discharge were different for each of the two units. Visual evidence on a number of occasions showed that approximately twice as much flow discharged from tank B (Figure 3.12) as tank A. This observation had implications for the Trash Trap monitoring and general operation but unfortunately could not be avoided, nor backed up by measurements.

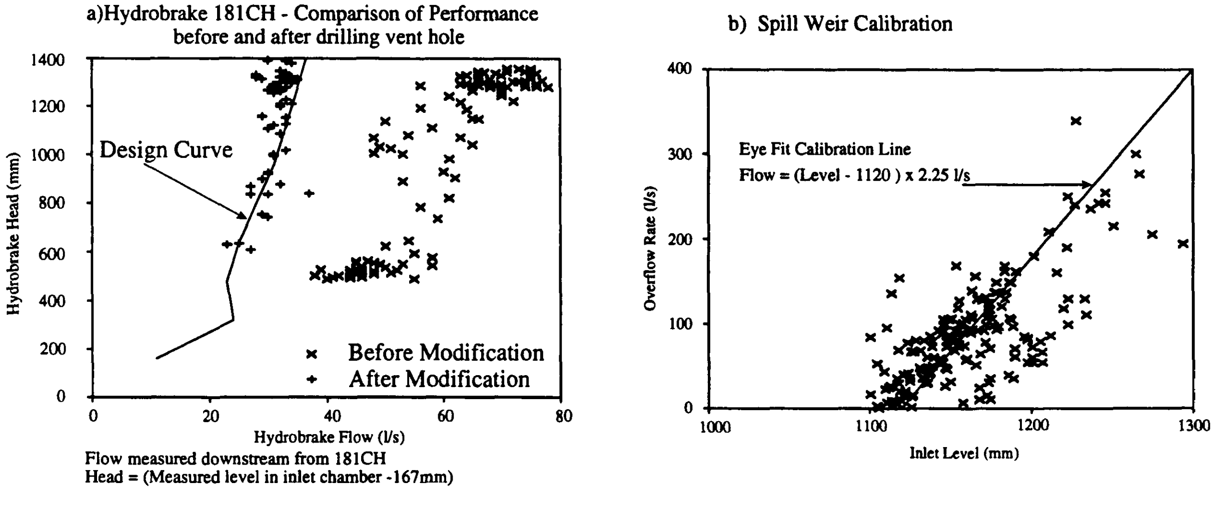


Figure 4.3 Lochgelly Storm King Installation - Hydraulic Relationships

The two Hydrobrakes in use (coded 181CH and 199CH) were located on the bypass and underflow respectively. Two flow monitors gathered information relating to the 181CH unit on the bypass. The initial stage of monitoring in 1989 as described by Jefferies & Dickson (1991) showed that the flow passing was approximately 100% greater than specified by the manufacturer. At that time it was speculated that excess air was entering the vortex which was consequently suppressed, resulting in orifice flow persisting even though the head should have been sufficient to make the vortex form.

Figure 4.3 (a) shows the data published previously (Jefferies & Dickson 1989) together with the results from further monitoring in 1990. In August of 1990 a 12mm diameter hole was drilled in the top of the Hydrobrake unit in an experiment to determine the effect of venting excess air. It was believed that a pocket of air became trapped thus preventing the vortex initiation.

The results of monitoring three events after drilling showed that the performance was closely in line with the manufacturer's performance curve and supported strongly the hypothesis that lack of air venting was indeed forcing the Hydrobrake to operate as an orifice and not to specification. It can be concluded that, since the 181 CH unit installed was standard, similar venting problems must occur elsewhere although the manufacturer was unable to report other cases (private communication). It may also be noted that the simple expedient of including a small hole in a Hydrobrake which could be plugged as necessary permitted two characteristic curves with the one device.

Due to the difference in elevation of the instruments, the Hydrobrake head was 167mm greater than the level measured at the inlet. A range of inflows were observed while the Storm Kings were discharging (Hydrobrake heads > 1287mm) during which only a small variation of level was recorded due to the length of spill weirs of the Storm Kings. Consequently, the discharge through the bypass while spill was occurring has been assumed, on the basis of Figure 4.3 (a) to be as follows;

1989 and to 23/8/90	70l/s
23/8/90 onwards	30l/s

The underflow hydrobrake operated under higher heads and the manufacturer's data for its performance were accepted on the basis that there was less chance of air entrapment.

It was necessary to develop a relationship between inlet level and spillflow since direct flow measurements could not be carried out. Spillflow was determined using inflow rates measured during events when the sensor was free from debris, with the bypass discharges identified above, and with the manufacturers data for the underflow Hydrobrake 199CH. During steady state spillflow, the following relationship applied;

Spillflow = Inflow - Bypass flow - Hydrobrake 199CH flow **4.4**

These data are plotted in Figure 4.3 (b) but unfortunately are confused by the volume contained in the Storm Kings. It is believed that the storage lag produced an apparent reduction in the spill flows during some events. Consequently a regression analysis was not carried out and a line was fitted by eye as shown. It was furthermore considered illogical to use a relationship based on any minimum level other than that of the spill weirs. The resultant equation was;

Spill Flow = 2.25 x (Level - 1120) **4.5**

It is accepted that inaccuracies in the order of $\pm$25 - 30% of individual values will arise, but this may be expected with this form of analysis. The large amount of data gathered and the use of the flows as volumes, or combined with concentrations to give loads following integration, means that overall errors will be lower.

4.3 METHODS USED TO INTERPRET EVENT MONITORING

4.3.1 A Review of Fieldwork Methods

The fieldwork programme necessitated quick attendance whenever high flow occurred at the overflow site(s) where equipment was installed. The Gross Solids Sampler was installed for most of the time, providing a telemetry link and samples were normally collected within two hours of the start of an event. A period of intense activity would follow arrival at site during which time the sampling equipment would be cleaned and reset. The flow monitors would also be checked, data retrieved as appropriate and calibration readings taken. There were three items of sampling equipment - Epic and Gross Solids Samplers and Trash Traps - and this procedure inevitably took time. Apart from making relevant physical observations, no attempt was made to interpret the data gathered at the time of the site visits. Additionally, during prolonged events, the Gross Solids Sampler would be restarted, necessitating a further period of waiting in damp, generally cold, conditions which were not conducive to precise working.

Saul & Marsh (1990b) have recommended that dry weather flow monitoring should be carried out prior to an event. Resources for the study described here were insufficient for such extensive sampling since attention was primarily on the overflows. The work clearly suffers from this deficiency which could not be avoided.

4.3.2 Definition of Events

Definition of events varied according to the method of sampling since only for the suspended and dissolved loads was the 'total event' required.

Epic Sampler(s) - Particular problems were encountered in defining when the flow rose above the prevailing dry weather conditions.
Gross Solids Sampler - Started just before the water rose to the overflow weir level and ceased at a slightly lower setting. The start and end times were easily recorded, however, they did not normally relate directly to the 'total event'.
Trash Traps - Responded passively to imposed flow which could relatively easily be found from level records.

Events were defined after all data were assessed. The catchments were diverse in nature and the amount of baseflow derived from infiltration and additional to dry weather flow would vary depending on the amount of preceding rainfall.

The catchments at two overflows had rapid responses, flow returning quickly to dry weather conditions, however, at two sites (Elgin Street and Dixon Street) the baseflow increased significantly during wet weather. For the rapidly responding catchments, the start of an event could easily be defined as the time when the level rose above the unchanging dry weather flow and presented no problem of definition. In contrast, for the sites where the baseflow varied significantly, each event had to be considered separately and on occasions an arbitrary start had to be defined. No overall rule could be made to apply to all events at a site.

Definition was a particular problem for minor events which did not fill the available storage to any extent. At Broomhead and Lochgelly an event was considered to have occurred provided there was flow to the tank. Storage was limited in the overflow section at both these off-line locations and only trivial events were excluded using this definition. Termination was defined as the time when overflow ceased. In general, the cessation of overflow also marked the time when the flowrate returned to the capacity of the downstream sewer system at the location.

More subjectivity had to be applied at Elgin Street since the continuation flow was higher, when expressed as a multiple of dry weather flow. Additionally event definition was difficult when multiple peaks occurred. This posed problems when the storage at each site had been at least partially filled and significant drawdown was deemed to have been necessary for the event to be considered as separate.

Location	**Date**	**ADWP (h)**	**Rain Total (mm)**	**Mean Intensity (mm/h)**	**Peak Intensity (mm/h)**
Broomhead	4-5/1/91	7	12.2	1.4	3
Elgin Street (continuous)	7/1/92 8/1/92	8 -	3.0 18.0	1.0 3.0	3 12
Lochgelly	16/10/90	19	3.6	3.5	18
Dixon/ McKane	7/10/91	54	9.0	1.6	12

Table 4.2 Basic Rainfall Data for Example Events

To highlight the application of these general principles, an assessment of each site follows. This assessment has been made via an examination of a characteristic event at each site. Table 4.2 is an abbreviated statement of the rainfall statistics for each event which, for the overflow sites were chosen from those when spill occurred.

4.3.3 Broomhead - Event of 4-5th January 1991

This event was caused by moderate rainfall on a snow-covered, saturated catchment. Figure 4.4 shows appropriate levels, flows and concentrations measured. The throttle pipe had a capacity of some 60l/s during overflow (figure 4.1(b)) and the storage in the chamber was small, consequently overflow occurred even with small rainfall events. The inlet level plotted in Figure 4.4(a) shows initially free discharge at the overflow weir where the crest level corresponded to an inlet level of 810mm. Some 35min after the peak inflow, the tank level rose to drown out the overflow weir and hydraulic control transferred to the spill weir (Equation 4.3). During the periods when transfer of control occurred, no clear relationship was available and interpolation of values for short durations was required.

A similar approach was necessary during periods when the tank was full but not spilling and, as occasionally occurred, the inlet logger sensor became blocked. At such times, inflow approximated to throughflow which was either measured or determined using equation 4.2. This effect was very apparent during the event under discussion (eg 16:00 - 18:30) and a significant amount of interpolation was necessary.

Inlet and overflow samples were taken simultaneously at the Broomhead site, although, during prolonged events, only the inlet sampler was used.

This was the case in this event after 18:00. Spill samplers were always started manually as telemetry system of the GSS was found to give sufficient warning for arrival at site before the tank became full. Suspended solids concentrations at inlet and overflow, plotted in Figure 4.4(b), reveal a clear first foul flush during the first peak, a characteristic reduction of concentrations until approximately 16:00, and a rise thereafter.

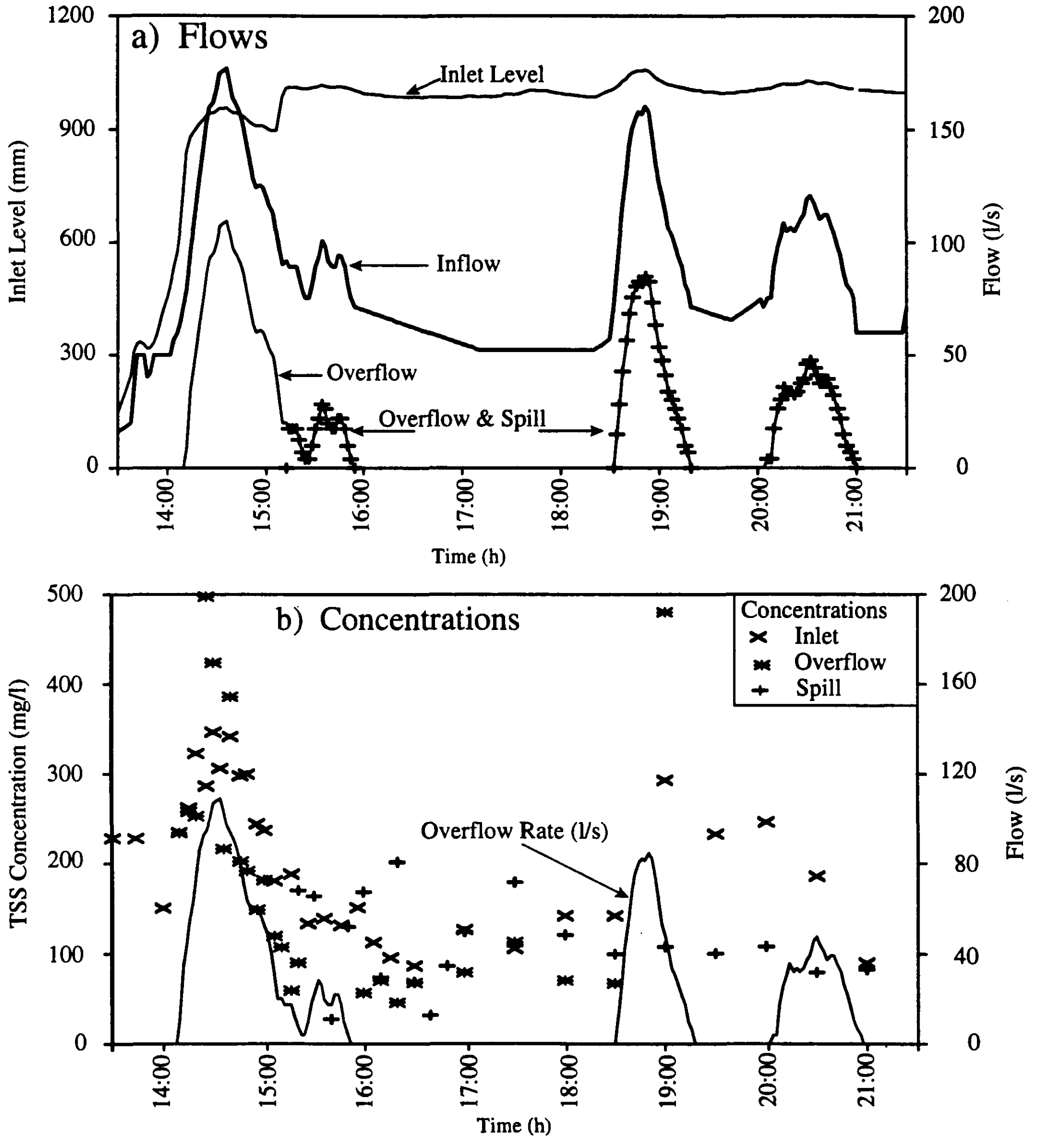


Figure 4.4 Broomhead Overflow - Event of 4-5th January 1991

Concentrations at the overflow were frequently high at the start of events, consistent with the visual observation during several events that the overflow caused an accumulation of material in the chamber close to the weir, requiring high flows to wash it over the weir. After the peak flow, inlet concentrations were normally greater than overflow suggesting that some settlement of material was occurring in the overflow chamber. This is considered further in section 7.2. Spill concentration progressively reduced during the course of the event but not appreciably below 100mg/l. Indeed they rarely did so during any event, even when the spill rates were small.

4.3.4 Elgin Street - Event of 7-8th January 1992

Problems of defining the start of events were most severe at the Elgin Street Site. During summer conditions, dry weather flow was estimated to be some 103l/s. However the nature of the sewered and natural catchments were such that the DWF increased to above 200l/s following wet periods in winter. This phenomenon was evident for the event under consideration and its start was taken as the minimum flow following the previous, smaller, event of 7th January.

Data from the event are plotted in Figure 4.5. It will be observed that some three hours of gradually increasing flow occurred at the start of the overflow period. The full rainfall event consisted of some 21 hours of rainfall commencing at 21:00 on 7/1/92 and had a mean intensity of 1.6mm/h. Between 21:00 and 02:30 the intensity was lower but after 02:30 it increased and for nine hours varied from 4mm/h to 6mm/h. This caused problems with definition of both the start, assumed to be 00:00, and the end which was perhaps even more arbitrary as it was assumed to coincide with the end of sampling - at 06:00 - after all night sampling. Although not entirely satisfactory, the event thus defined did coincide with the most severe meteorological conditions which occurred. Curtailment of the event for analysis could not in any case be avoided as the local watercourse, the Lyne Burn, flooded its banks during mid-morning of the event, drowning out the tanks and partially inundating the Gross Solids Sampler for the subsequent 24-hour period.

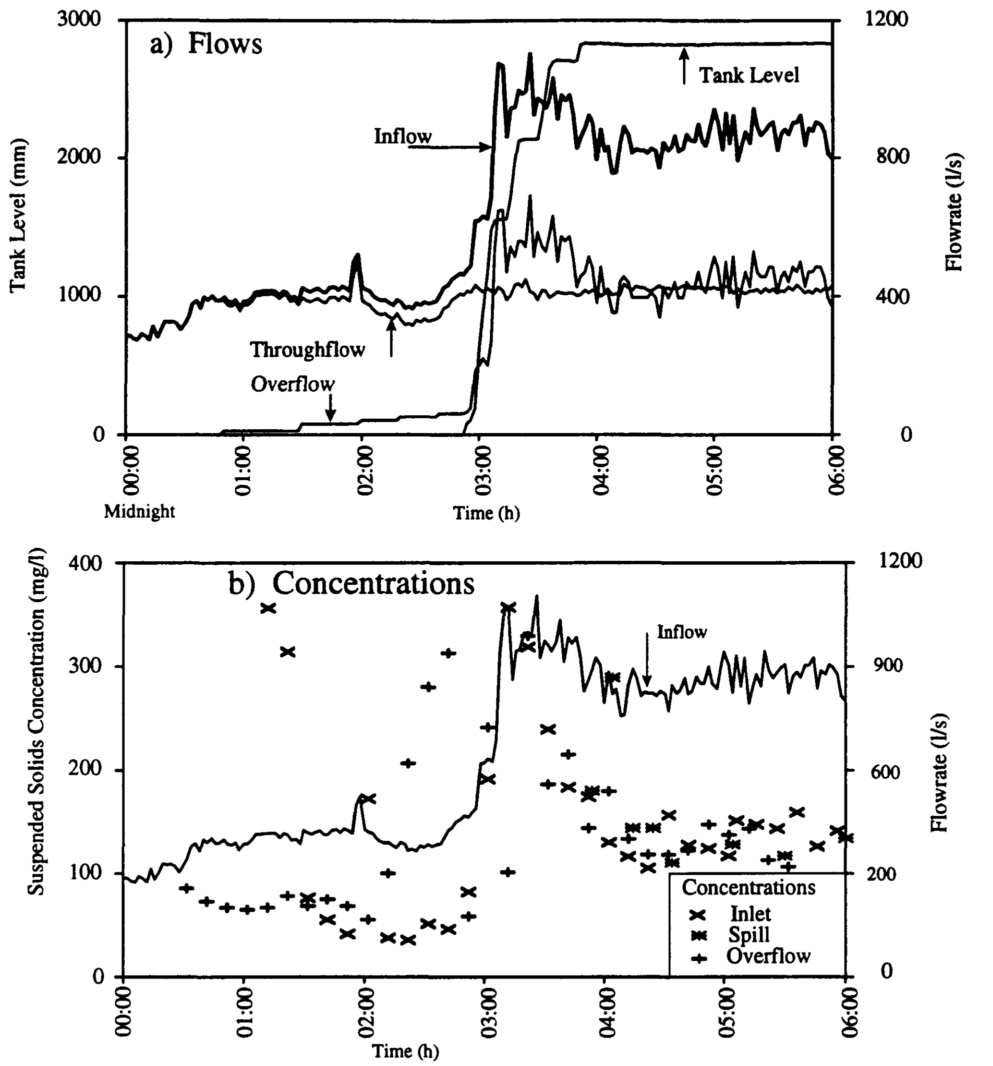


Figure 4.5 Elgin Street Overflow Event of 7-8th January 1992

The inlet monitor malfunctioned for a period and in view of the site difficulties, little reliance was placed on information gained. Inflow was computed by adding the through- and over-flows. The latter were checked by reference to the filling of the overflow tank to the sequential weir levels, as can be seen from the tank level plot in Figure 4.5 (a). It was found that the overflow rates were undermeasured by 30% and all overflow rates were adjusted accordingly.

Inlet and overflow samplers were triggered by inlet level (somewhat erratically) at the start of events and manually during extended sampling. A time interval of 10 minutes was used throughout. Spill samples during this event were all taken by passing a bucket under that cascading flow as site difficulties prevented the installation of a spill sampler. Inlet and overflow Suspended Solids concentrations were similar at the start of the event as can be seen in Figure 4.5 (b). However sufficient overflow concentrations were high to suggest that some aggregation of solids close to the weir was occurring and this effect should be compared with Broomhead as illustrated in Figure 4.4 (b). The increased flow after 03:00 gave rise to increased concentrations which can only be described as a first flush, although hardly rapid, and thereafter the concentrations reduced.

It will be noted that the mean concentration for all samplers between 4:00 and 6:00 am was 130mg/l, significantly higher than the average of 70mg/l prior to 3:00am. It is suggested that the increased flowrate was carrying a significant load of material which was not foul sewage in origin, particularly as the time was 4:00 - 6:00 in the morning. The colour of the combined sewage would suggest that soil was being washed into the sewer system from permeable areas.

4.3.5 Lochgelly - Event of 16th October 1990

The baseflow for this nigh-time event was extremely low at approximately 12l/s and as a result of this, together with the intensity of the rainfall, definition was simple. The flow hydrograph consisted of a minor peak followed by a clear main peak. Data from the event are plotted in Figure 4.6 (a). The low bypass weir approximately 300mm high in the inlet chamber caused flow to back up enabling all events to be considered to commence when this level was reached. The logger sensor at this location remained free from sediment during most events and, where data were not available, inlet, bypass and underflow flowrates were computed as described in section 4.2.3. Additionally, the inlet logger was used to monitor levels within the Storm King units. There was an estimated 25mm difference in water surface levels between the inlet and the spill weir at maximum flowrates and this was considered minor in context. Equation 4.5 was used to determine spill flowrates.

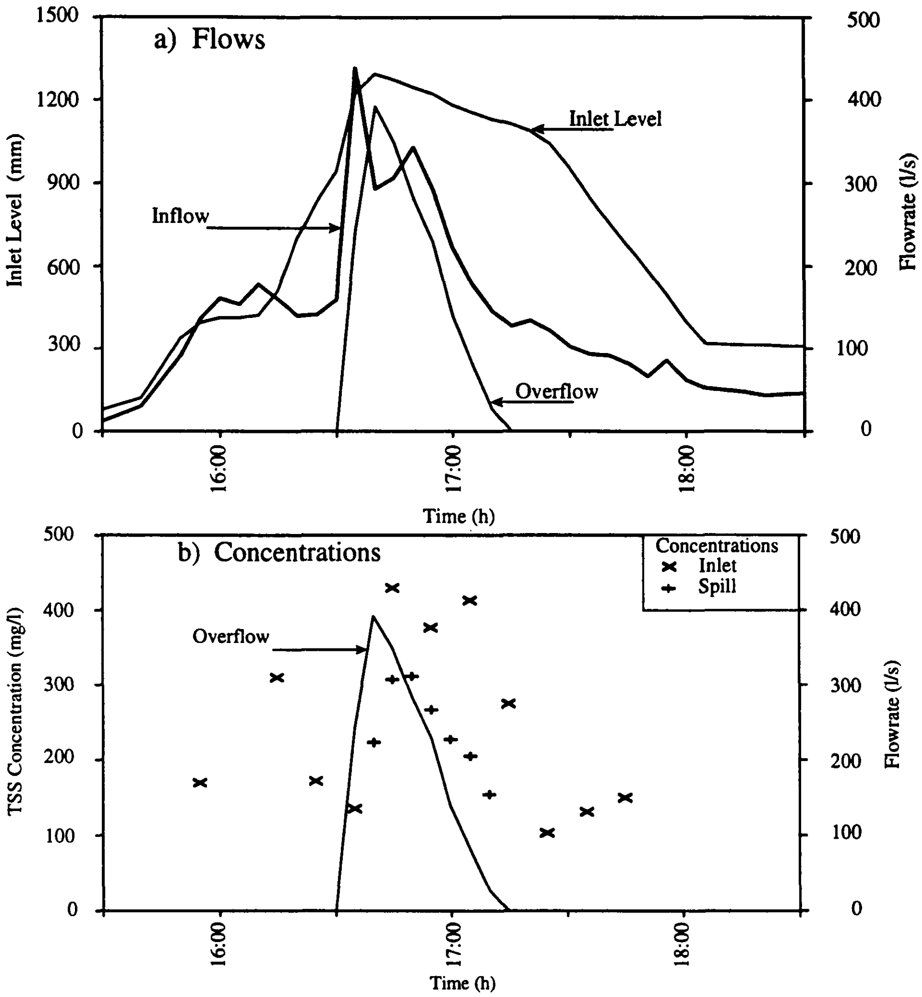


Figure 4.6 Lochgelly Overflow - Event of 16th October 1990

The inlet sample tube was located slightly downstream from the logger sensor at the entry to the bypass pipe in an area of high turbulence, allowing the samples taken to be considered to be representative of the inflow. Unfortunately, the sampling manhole was lower than the water level at the inlet allowing syphoning from the inlet and invalidating many sample sets. This explains the paucity of event-based samples from the inlet at this otherwise good site. The overflow sampler was triggered by a float switch some 50mm below spill weir level with the tube intake located in the splash zone at the base of the spill weir. The operation record of this sampler was excellent with, invariably, the first sample bottle empty due to the short time period of final filling of the storage. Figure 4.6 (b) shows some evidence of a first foul flush occurring during the first rise in flow with a strongly marked concentration peak mirroring the inflow. The concentrations of the spill flow reflect this effect although to a reduced degree.

4.3.6 Dixon Street & McKane Park - Event of 7th October 1991

The McKane Park location responded to small events rapidly due to the large number of overflows located upstream, and this rapid rise was used to define events for both sites. During dry weather, a small infiltration flow was always present, and events were considered to have occurred whenever the flow exceeded 100l/s irrespective of the flow at Dixon Street. Flows during the event under consideration are plotted in Figure 4.7 and show a typical response with a rapid rise to peak flow reflecting the steep catchment. The peak flow at Dixon Street was consistently of the order of 420l/s over a wide range of events. In spite of the site difficulties described in section 3.7.1, sets of samples were regularly obtained and the data from the event under consideration are considered to be typical.

The quality parameters observed during this event are plotted in Figure 4.8 and exhibit a strong first flush effect, a drop to low levels as the flow receded and a rise to values more typical of DWF as the event passed. The return to dry weather concentrations was particularly pronounced on this occasion as the recession of the event coincided with peak DWF conditions (DWF flows and qualities are discussed in section 4.5).

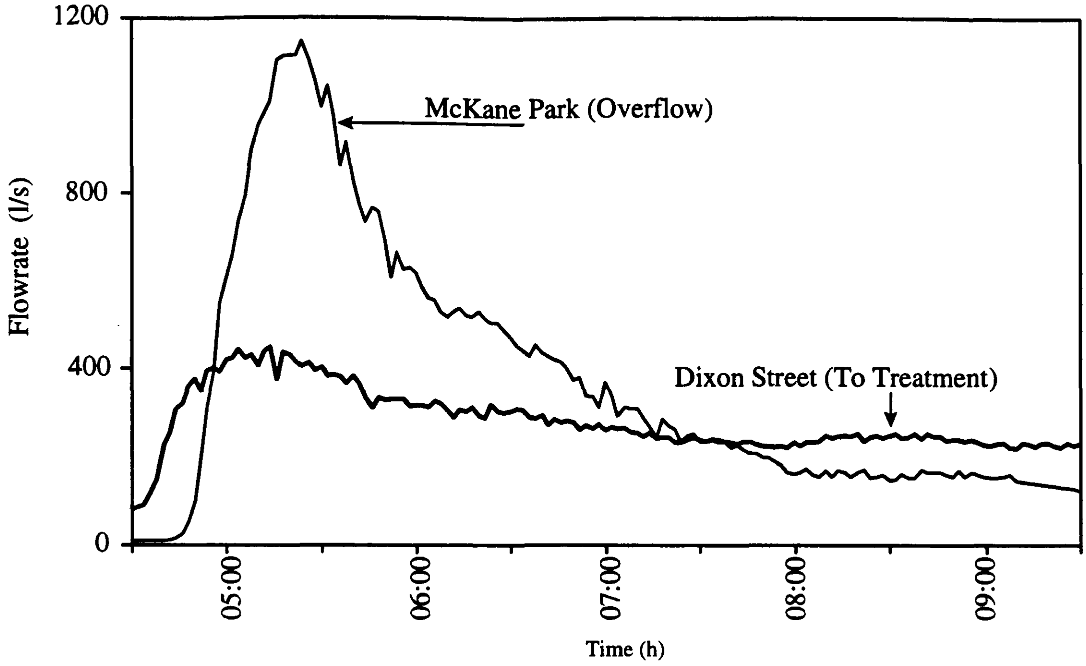


Figure 4.7 Dunfermline Outfalls - Flows during Event of 7th October 1991

All pollutant parameters selected for the study were measured during this event and their behaviour was similar to that observed elsewhere, such as that reported by Geiger (1986) and Pearson et al (1986). In the current study and those selected from the literature, all determinands showed their peak values at the start of the events and declined as the high flow continued. TSS lagged slightly behind the other determinands, peaking closer to the peak flow. One difference was that concentrations during both events selected from the literature continued to decline, presumably due to the occurrence of a subsequent event whereas those of 7/10/1991 showed a characteristic rise of concentrations to prevailing DWF values. The ammonia values for Dixon Street apparently also show an initial peak contrary to the expected dilution of dry weather flows (Pearson et al 1986). However only one value was higher than DWF values and is not considered to be exceptional. Since the data set was the most complete, particularly for determinands other than TSS, it has been included in the subsequent analysis of pollutant concentrations.

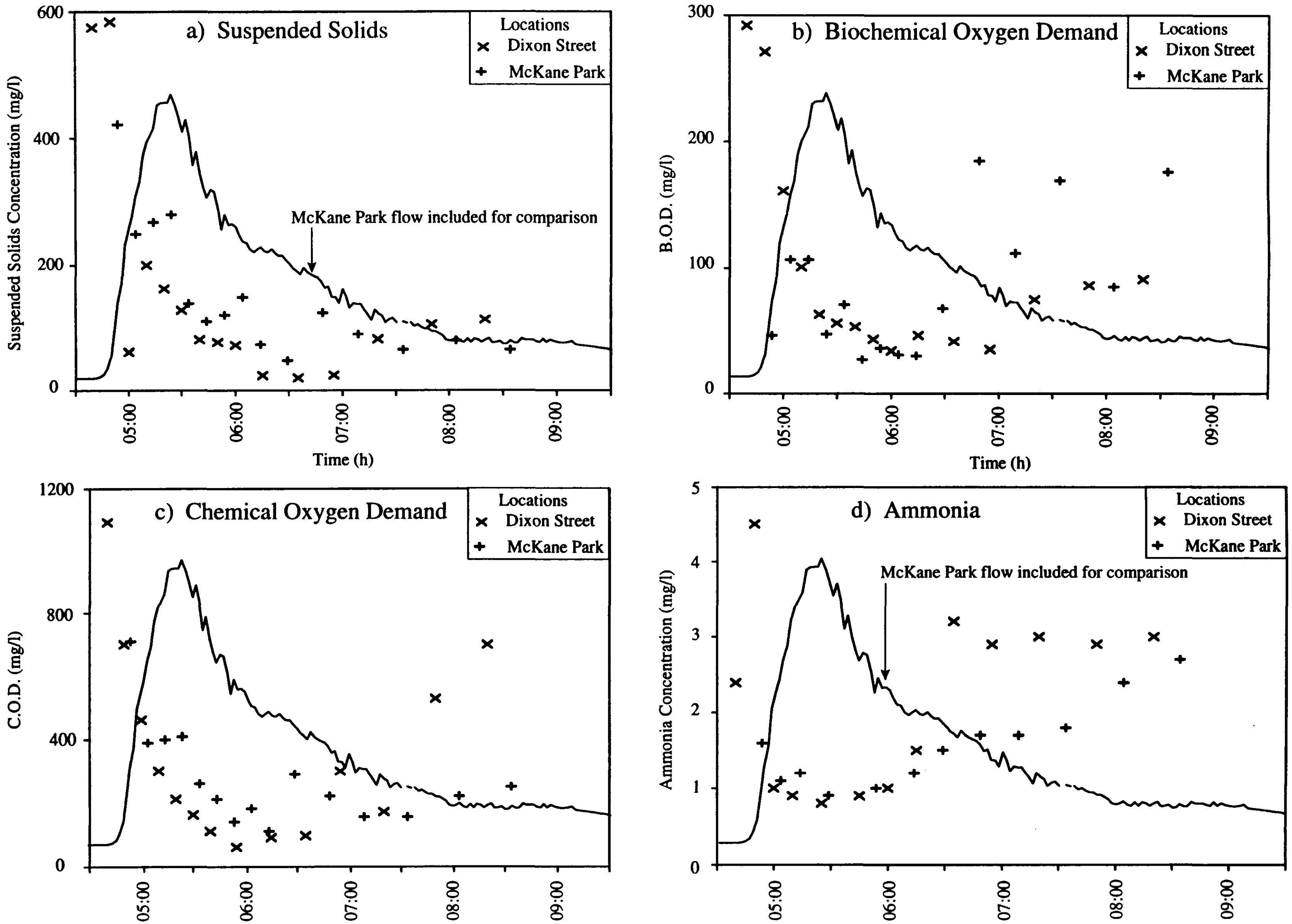


Figure 4.8 Dunfermline Outfalls - Qualities During Event of 7th October 1991

4.4 POLLUTANT MOVEMENT DURING STORMS

4.4.1 First Foul Flush

It is recognised (Tucker & Mortimer 1978, Saul & Thornton 1989) that the first foul flush is the most frequently quoted indicator of the behaviour of pollutants, particularly suspended solids, during combined sewer events. There are many approaches to the analysis of the first foul flush. The general principles have been discussed in section 2.2.3 and the definitions proposed by Geiger (1984) and Pearson et al (1986) have been applied to the data. Cumulative suspended solids load curves were plotted for all available event-based data (including some not used in the main analysis) at the three overflow sites. These curves were interpreted to determine the type of flush, if any, which occurred.

Following the method described by Geiger (1984) a strong flush, negative or positive, was defined as having a cumulative load more than 20% from the diagonal and an indifferent flush being less than 5%. Moderate flushes were deemed to lie between these limits.

Location	Number of Events	Positive		Equil-	Negative		Percentage
		Strong	Moderate	ibrium	Moderate	Strong	of Type B Flushes*
Broomhead							
Inlet	22	13 (60)	3 (15)	1 (5)	2 (10)	3 (15)	(73)
Overflow	22	10 (44)	2 (10)	5 (23)	0	5 (23)	
Elgin Street							
Inlet	8	5	0	1	1	1	(75)
Overflow	5	0	0	3	0	2	
Lochgelly							
Inlet	7	3	0	2	1	1	(86)
Spill	13	4	1	6	1	1	
Geiger (1984)	125	32(26)	39 (31)	29 (23)	16 (13)	9 (7)	

Numbers in brackets represent percentages.
*As defined by Pearson et al (1986)

Table 4.3 First Foul Flush Events

The results of this classification procedure are presented in Table 4.3. The catchments may be compared using inlet data at each location. The data from the large catchment in Munich described by Geiger show that the percentage of flushes was smaller for positive and similar for negative flushes in comparison with the present study. This might be expected as the current catchments were all on the periphery of urban areas and subject to greater variability.

Pearson et al (1986) defined Types A & B flushes as having concentrations less than and greater than the prevailing DWF values respectively. The 102 events analysed from Great Harwood in Lancashire had 56% of the total as type B storms. This percentage contrasts with the data from this study in which the percentages of Type B were;

Broomhead	73%
Lochgelly	86%
Elgin Street	75%

The number of inlet events amenable to analysis was smaller than at Great Harwood. However each site in the study described here showed a consistently higher proportion of type B flushes. Once again it is proposed that this was due to the more variable nature of the peripheral catchments under study here. This analysis has shown that pronounced positive flushes predominated at all three overflow inlets and it may be deduced (Geiger 1984, 1986) that sediment deposits must have existed prior to many storm events.

Monitoring of the overflows at the three sites showed that less pronounced flushes occurred than at the inlets. It is suggested that this was due to settlement within the overflow structure, a mechanism proposed by Ellis (1986) as being as important as dilution in reducing peak concentration levels. It was further noted that, although many overflow events showed high initial concentrations, the accompanying loads and volumes were frequently extremely small. Thus the concentrations were insufficient to produce an identifiable flush at the overflow.

4.4.2 Event Mean Concentrations

Mean concentrations were computed for each event using equation 4.6 expressed as mg/l;

$$\mathbf{EMC} = \frac{\textbf{Total Load of Pollutant}}{\textbf{Total Volume of Flow}} \qquad \mathbf{4.6}$$

Table 4.4 shows statistical information relating to EMC values. One event (25/11/90 at Broomhead Overflow) has been excluded from the data as an outlier since, although the data were valid, the event had an EMC of 2231mg/l but with a discharge volume of only 1m^3. The average of the individual concentrations has been compared with the overall average, the latter figure being biased towards larger events with a high volume of relatively less polluted discharge.

Location	Number of Events	Average of EMCs (mg/l)	Average of All Observations (mg/l)	Log-Normal Distribution r^2
Broomhead				
Inlet	23	328	298	0.965
Overflow	25	289	316	0.954
Elgin Street				
Inlet	8	212	151	*
Overflow	5	320	146	*
Lochgelly				
Inlet	4	250	232	*
Spill	13	181	139	0.967

* Insufficient Data

Table 4.4 Event Mean Suspended Solids Concentrations at Overflow Locations

Normal distribution fits were examined for three of the data sets. The ranked data were plotted against normal distribution scores and best fit straight lines were determined using linear regression. The data were found to fit the normal distribution with acceptable r^2 values, but with poor fits at extreme values. In contrast very good fits were obtained using the log-normal distribution and the values are presented in Table 4.4. Figure 4.9 shows the resulting normal distribution plots.

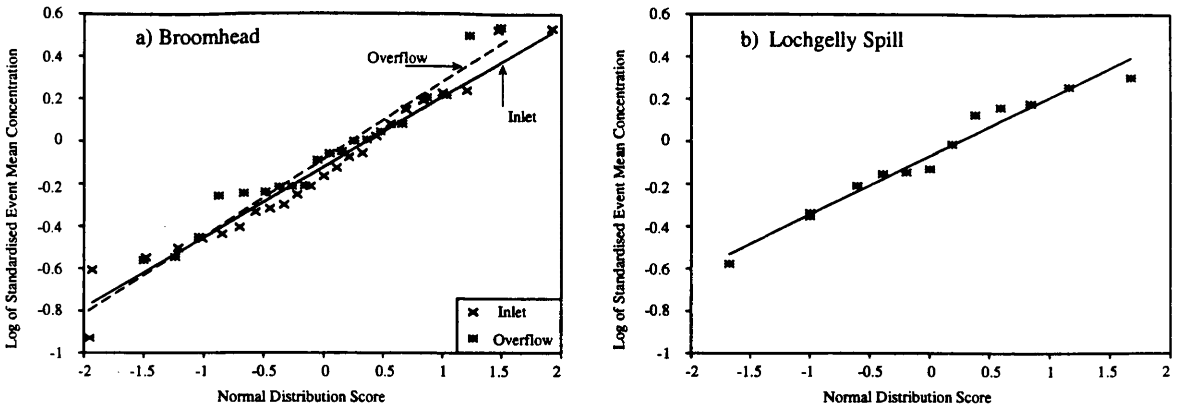


Figure 4.9 Log-Normal Distribution Plots For Event TSS Mean Concentrations

Discussion of EMC distributions was included in section 2.2.4. Geiger (1987) and Ellis (1986) both found that a log-normal distribution could be fitted to EMC data. This supports the contention that the data sets conform in their behaviour with others for which sufficient data are available to be statistically valid.

4.4.3 Event Loads

A similar statistical manipulation to that for EMC values was carried out for event suspended solids loads. The resultant averages found by averaging all event loads for each site in the same manner as for EMCs in section 4.4.2 are included as Table 4.5. Loads were also found to fit log-normal distributions and plots are included in Figure 4.10. High r^2 values were obtained indicating excellent adherence to the log-normal distribution.

Location	Number of Events	Average Event Load kg	Log-Normal Distribution r^2
Broomhead			
Inlet	23	184	0.985
Overflow	25	75	0.984
Elgin Street			
Inlet	8	771	*
Overflow	5	186	*
Lochgelly			
Inlet	4	491	*
Spill	13	128	0.961

* Insufficient Data

Table 4.5 Event Suspended Solids Loads at Overflow Locations

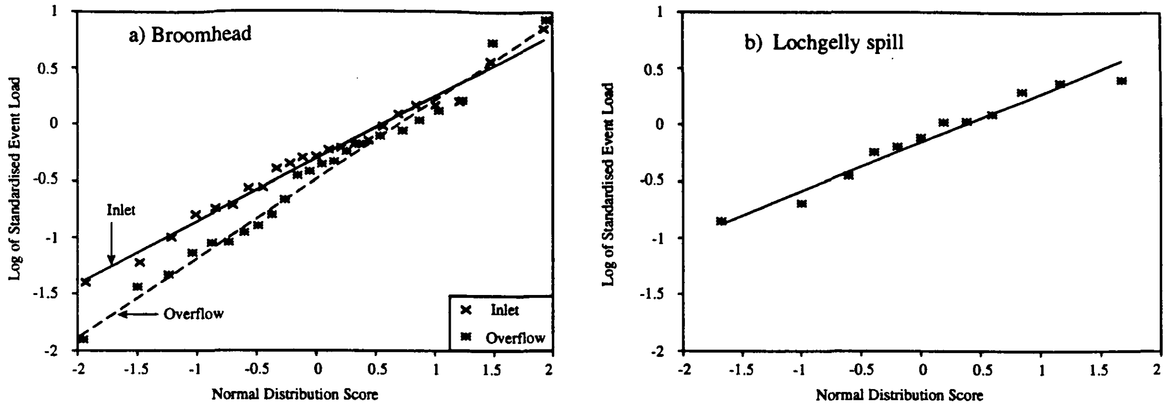


Figure 4.10 Log-Normal Distribution Plots For Event TSS Loads

4.5 COMPARISON OF POLLUTANT DETERMINANDS FROM EVENT BASED DATA

Budget restrictions during the study limited the range of possible testing to suspended solids concentrations on every sample with only a proportion of samples being tested for further determinands. This reduction in determinands was partially supported by comparisons made early in the study (also section 3.2.3) in which good correlations were obtained with COD and BOD against TSS. Regression coefficient values of r^2 = 0.83 and 0.77 were obtained for BOD against TSS and BOD against COD respectively for storm data gathered at the McKane Park site.

Data Set	Independent Variable X	Dependent Variable Y	No of Data Points	Y = mX + C		r^2
				C	m	
All	TSS	BOD	180	22	0.280	0.620
FFF	TSS	BOD	149	23	0.281	0.675
All	TSS	COD	347	111	0.927	0.728
FFF	TSS	COD	245	116	0.899	0.752
All	COD	BOD	130	30.8	0.175	0.494
All	BOD	NH3	120	60.4	6.14	0.106

All = Data from all events at all locations
FFF = Data from positive Flush Events at all locations

Table 4.6 Correlation Information for Event Based Quality Data

It was believed at the time that the behaviour of BOD and COD in particular could be satisfactorily predicted on the basis of the TSS results. Upon analysis of the full data set from all sites at the end of the study, reliance on such relationships proved to be optimistic and only marginal use could be made of non-TSS data.

Statistical comparisons were made between the determinands tested and these are summarised in Table 4.6. Linear regression was applied to the data but the resulting r^2 values were generally low. For determinands such as NH_3, no correlation against TSS would be expected (Pearson et al 1986) as ammonia is characteristic of dissolved pollutant load, whose behaviour is well catalogued as being different from that of suspended particulate matter. In an effort to obtain enhanced relationships which might be applied to specific ranges of events, data from events with a Type B flush were selected and when tested, contrary to Pearson et al (1986), produced a marginally better fit, although still of little value.

Selected plots showing the variability of the data are included as Appendix B, Figure B1. It must be concluded from this analysis that the relationships produced were not satisfactory for predictive purposes.

4.6 DRY WEATHER FLOW QUALITY DATA

Pollutant concentrations for all dry weather flow periods monitored at all sites are presented in Appendix B, Figures B2 - B5. The data exhibit significant scatter which is greatest for BOD and COD. Data from sites on smaller catchments show less scatter than for larger catchments. Mean concentrations were computed for each hour and these figures are also plotted. The variation of the means for each location are shown in Figure B6.

4.6.1 Dry Weather Flows

Dry weather flow figures were abstracted from the full flow data sets and represent days which follow extended dry periods. Flows are shown in Figure 4.11(a) and averages are listed in Table 4.7. They may be seen to reflect the contributing populations although the Elgin Street figure is high, probably due to infiltration. To compare the diurnal variation, each flow has been normalised by dividing by its average and the resultant data are plotted in figure 4.11 b).

Location	Average DWF (l/s)	Population	DWF per Capita (l/person/day)
Lochgelly	14	4800	252
Broomhead	8	3800	182
Elgin Street	58	16000	313
Dixon Street	105	52000	174

Table 4.7 Dry Weather Flow Averages

To allow comparison with other data, DWF data used to develop the MOSQITO flow simulation model (Henderson (1988) have been included in Figure 4.11 b). Generally most flows were within ± 25% of the MOSQITO values except at night time for Broomhead and Lochgelly when low flows exaggerated percentage differences. The morning peak at Dixon Street was earlier than at the other sites, presumably due to commercial activity. The data from Broomhead showed greatest variation, but this was expected as it was the smallest catchment studied and significant lengths of the trunk sewers in the area were recently replaced. It is concluded from Figure 4.11 b) that the diurnal variations of flows were typical for the types of catchments.

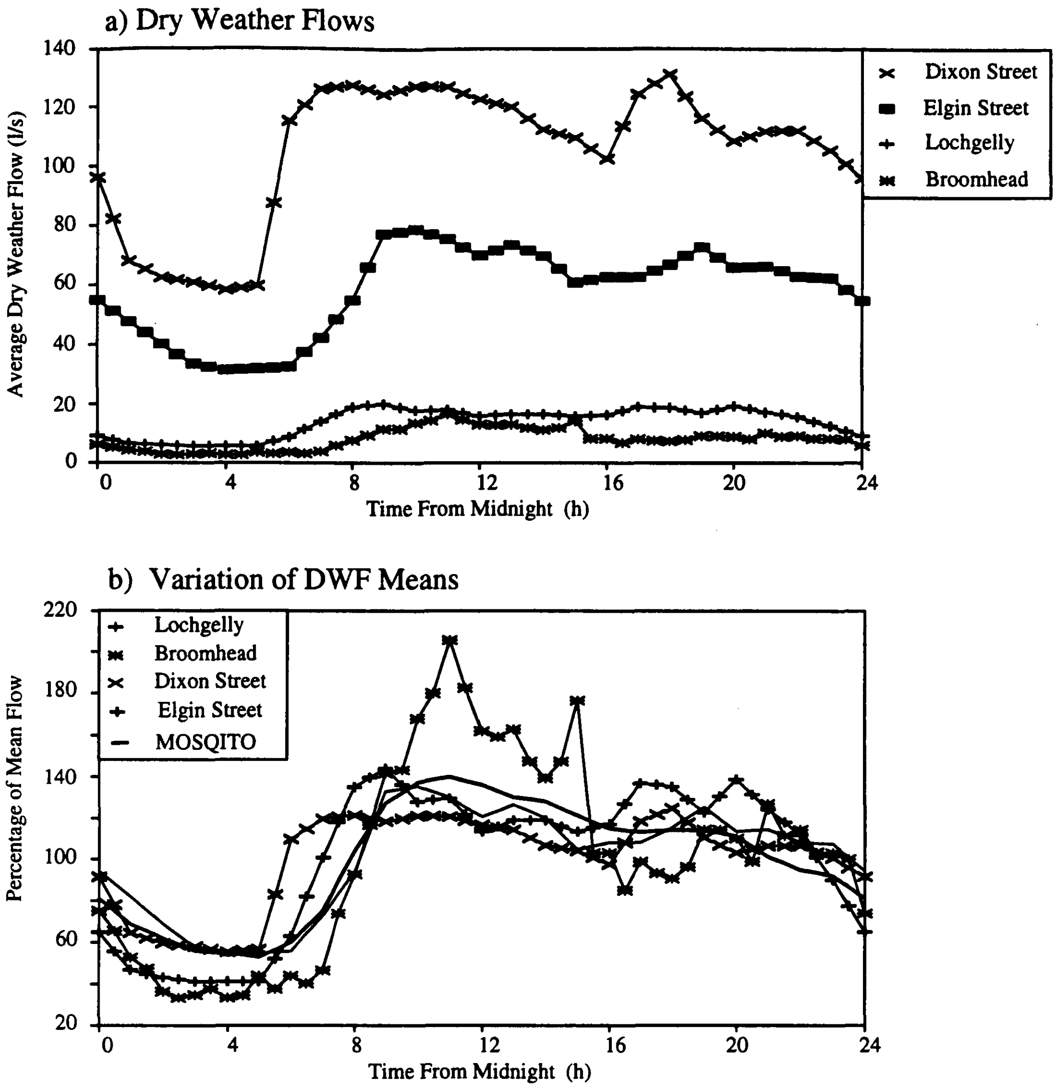


Figure 4.11 Dry Weather Flow Comparisons at Study Sites

4.6.2 Dry Weather Concentrations

All determinands exhibited significant scatter at each site. Such variation was expected and has been reported frequently (Crabtree et al 1991). The values for BOD showed particularly large variations and this may in fact reflect the testing technique. It is conventional to express the variations of concentrations at each time of day as a percentage of the means which are listed for the four study sites in Table 4.8.

Location	TSS (mg/l)	BOD (mg/l)	COD (mg/l)	NH_3 (mg/l)
Lochgelly	137	115	461	23.6
Broomhead	182	116	689	36.6
Elgin Street	188	106	535	14.6
Dixon Street	193	106	366	18.1

Table 4.8 Dry Weather Flow Average Concentrations

In order to determine whether the variations are abnormal, the normalised mean TSS concentrations have been plotted in Figure 4.12 along with the comparable MOSQITO figures (Henderson 1988) Broad agreement is noted, with predominantly random variation, although all sites show earlier and higher morning peak values. It is suspected that the data collated by Henderson had less pronounced peaks due to their larger catchment areas and population.

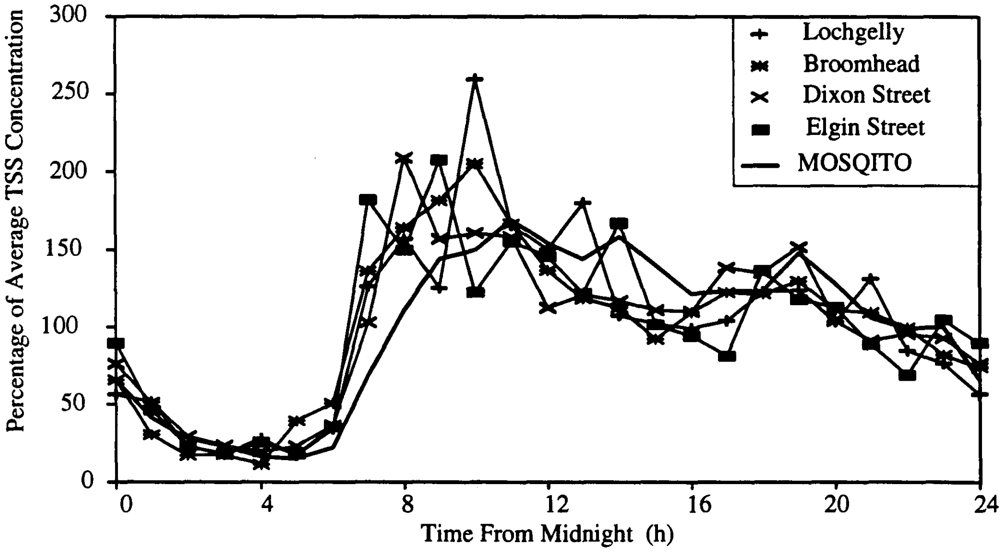


Figure 4.12 Variation of DWF Hourly Mean TSS Concentrations

4.6.3 Dry Weather Loadings

The cumulative load curve has been used effectively to present dry weather flow variability (SDD 1977). Ashley et al (1990) have used the method of presentation to show marked difference in pollutant transport between summer and winter dry weather flow days, when road salting is in operation. Comparison has been made with data from Scotland published in SDD (1977) and the results are plotted in Figure 4.13. DWF data from the four sites in this study have been compared with data from two of the SDD sites, the small suburban catchment of Westhill and the larger Persley catchment. All points lie within ±10% of the Westhill curve and vary from the Persley by a slightly greater margin.. The conclusion must be drawn from Figure 4.13 that the DWF behaviours of the catchments under study are both similar to each other and to other comparable catchments.

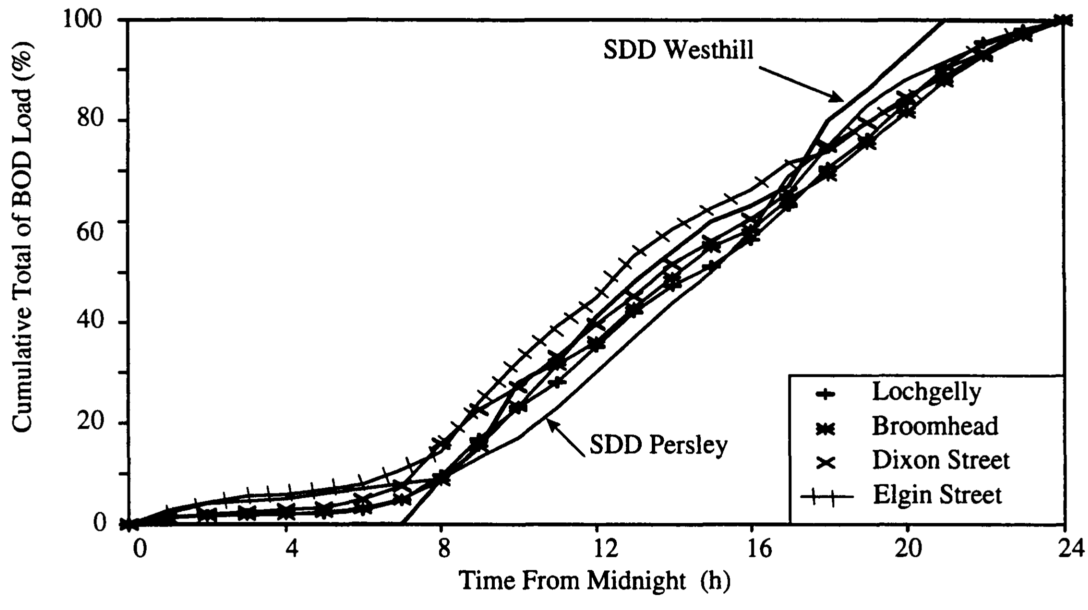


Figure 4.13 Cumulative BOD Loadings

4.6.4 Conclusion

The methodologies used in the interpretation of the field data have been explained and compared with other studies in chapter 4. The methods applied have followed standard procedures and the measured flows and concentrations from small-bore samplers have been used to make comparisons with other catchments in the United Kingdom. It is contended that the results show sufficient common characteristics to allow the conclusion to be drawn that the catchments are both similar and typical.

Differences of detail are always to be found in a study of this nature, and these were seen in the point readings of the DWF data. DWF variations of four physico-chemical determinands (TSS, BOD, COD & NH_3) were presented and the values, although showing scatter, provided the basis for comparison. The averaged dry weather behaviour has been shown to be typical when presented either as flows and concentrations separately, or as cumulative loads.

This variability of the DWF data was exaggerated by the response of the different catchments to rainfall. Two showed increases of baseflow in wet weather while two did not. It was found that more positive first foul flushes occurred than in other studies. This was interpreted as being an indicator of greater variability between catchments. Where sufficient data were available for meaningful statistics, however, event loads and mean concentrations were shown to have log-normal distributions in common with other studies. In spite of the variablilities shown, it is concluded that the catchments were sufficiently similar to provide a basis for further analysis.

CHAPTER 5 ASSESSMENT OF THE TRASH TRAPS

What the Eye does not see
the heart does not grieve
Anon

5.1 Introduction

No widely accepted methods of measuring or interpreting gross and visible solids were available at the start of the study. The two devices deployed for this purpose were Trash Traps and the Gross Solids Sampler (GSS). The sampling characteristics of both devices had to be evaluated prior to interpretation of the results. The Trash Traps are considered in chapter 5 and the GSS in chapter 6.

Trash Traps permitted the direct estimation of visible, sewage-related solids. Visible solids are defined in section 2.2.5 and a principal aim of this research was to determine whether the performance of various CSO structures might be differentiated on the basis of these solids. A comparison of different designs of CSO structure on this basis would allow the principal cause of complaints relating to CSO discharges to be addressed directly.

Trash Traps were installed on the overflow weirs at three sites in the study. At Elgin Street and Broomhead the Traps sampled the flow to the off-line tanks (q_t in Figure 2.3) but at Lochgelly only the final spill flow could be sampled. At the start of the study, since discharge at Lochgelly was to a small watercourse, counts were made of the visible solids discharged from the CSO and retained on the bed and by the bankside vegetation. While this technique, which is evaluated in Section 5.2, had the advantage of measuring directly the material, it could not be applied to the remaining sites due to their spill arrangements. Interpretation of the results of this exercise was also inconclusive and the technique was discontinued in favour of the use of Trash Traps.

Methods of expressing the Trash Trap data are developed in Section 5.3. The aim of this work was to develop tools applying to the measurement and estimation of visible solids which might later be applied to different CSOs. Various presentations were assessed using the numbers and masses of the retained material. These data were related to the volume of flow and the mass of suspended solids passing through the Trap. Relationships between the variables are presented and compared in terms of their utility for application to the separate CSOs.

Comparisons between the sites using the Trash Trap measurements are developed in Section 5.4. Clear differences between the nature of the material retained on the Traps installed at Lochgelly from those at the other two sites are indicated in Figures 5.5 and 5.6.

It is concluded in Section 5.5 that, assuming inputs to all the CSOs studied to be similar, the novel Trash Trap method described and developed in this thesis has shown the Storm King installation to be more efficient at removing solids than either the stilling pond or the high side weir overflows studied.

5.2 Stream Sampling for Visible Solids

Assessments of the amounts of visible solids discharged were made by hand counting during the summer of 1989 at Lochgelly/Lumphinans. This stream was dry for most of the relevant study period apart from during spill, and visible matter could be collected from both bankside vegetation and the bed. A 25m length of stream was cleaned eleven times during the period with at least one event between collections. The stream bed was composed of a short section of boulders followed by gravel and stones in which material was easily trapped. It is contended that the counting method, while being site specific, did ensure that representative material was collected. The visible material was virtually all paper and plastic strips, faecal solids representing 2% and fatty lumps a further 2% by number of the visible material collected.

The flow loggers at the site enabled flow rates and volumes to be determined together with antecedent dry weather periods (ADWP). Evaluation of an appropriate measure for ADWP was complicated by smaller rainfall events which part-filled the overflow without causing spill. On occasions there was also more than one event between spillage. ADWP for this purpose was taken as the greatest time between periods of filling, even though this may not necessarily have been complete, and spill occurring. This definition was used to ensure that the spillage most likely to have produced the largest numbers of solids was related to the duration of its preceding dry period.

Strong correlation (r^2 = 0.901) was found between the volume spilled and the causative rainfall events. However, in agreement with Mutzner (1987) who carried out a similar exercise but without flow measurement, no relationship was found between the numbers of visible solids collected in the stream and either spill volume or peak spill flowrate. Indeed a negative correlation may have existed as illustrated in Figure 5.1 a). This may possibly be explained by the larger events washing the material past the observation length.

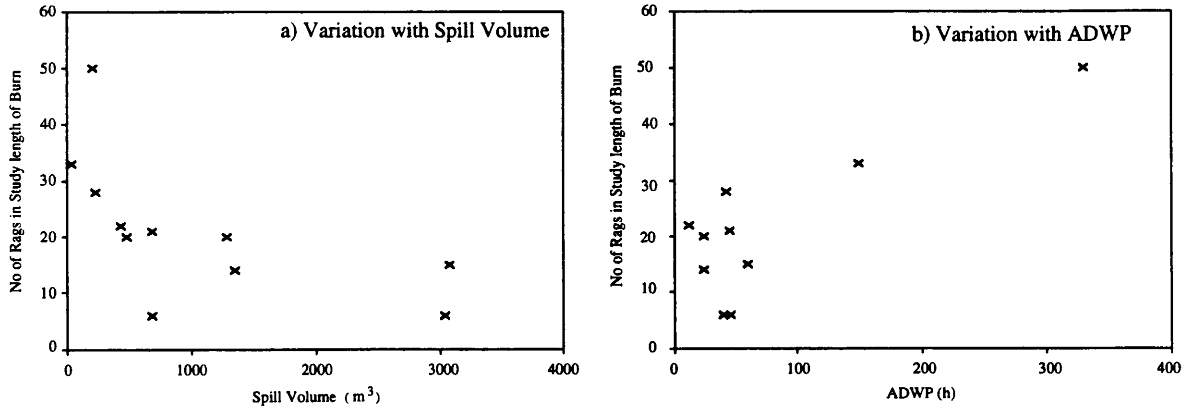


Figure 5.1 Visible Solids in Lochgelly Burn

In contrast with Mutzner, a weak correlation (r^2 = 0.666) was found with ADWP as illustrated in Figure 5.1 b) from which the amount of material discharged can be seen to increase with ADWP. The correlation was very dependent on two data points and no relationship was developed. It is suggested that the different definition of ADWP made possible with flow data being available may have made a clearer relationship than found by Mutzner. Although this result is interesting, the counting method used is highly dependent on local conditions, and for later studies including those at the Lochgelly site, visible solids were counted using the Trash Trap.

5.3 TRASH TRAP TESTING

In this section, estimating tools are developed for further CSO comparisons. Relationships which should apply to measurements taken at any CSO are developed prior to using the Trash Trap based data at the three study sites.

5.3.1 Use of Trash Traps

A discussion of the method of operation and the testing carried out to establish the validity of the Trash Trap results is included in sections 3.6.4 & 3.6.5. Trash Traps were installed at the Lochgelly/Lumphinans, Broomhead and Elgin Street sites. Field data were obtained by picking off, counting and weighing all visible solids material from the trap and estimating the proportion of blinding.

The methods employed to determine the flow volumes and Suspended Solids masses passing the traps are described in section 4.3 and all Trash Trap results are presented in Appendix C, Tables C1-C3.

5.3.2 Solids Interception by Trash Traps

The mass of material collected from the Trash Traps was weighed and the total number of separate items having two dimensions >6mm were counted following each event. The proportions of different types of material collected were determined for a limited number of events. It was found on average that plastic and paper strips comprised from 76% to 89% of the total numbers, averaging 82%, the remainder being made up of almost equal proportions of faecal matter, plastic sticks and condoms. Floatable particles comprised less than 5% of the total numbers of material collected on all traps. The visible solids from all events were weighed after two hours drying at 100°C. After this period some absorbent material was still damp, however, this procedure ensured that the very thin plastic material remained intact.

Blinding of the Trash Trap mesh was estimated after the removal of visible solids as discussed in section 3.6.4. Blinding was caused by small particles in the flow such as shreds of toilet paper and threads of cotton which became embedded in the mesh. Plates 5.1 and 5.2 illustrate the different degrees of blinding which varied from 0% where there was no interruption of flow through the trap to 100%, when all flow passed straight over the trap. The trash traps at Elgin Street and Broomhead became submerged during tank spill and observations for events when spill occurred were rejected.

From visual inspections it is suggested that the blinding material retained on the Trash Traps comprises type C sewage solids (Crabtree 1988). Comparison of plates 2.1 and 5.2 support this contention.

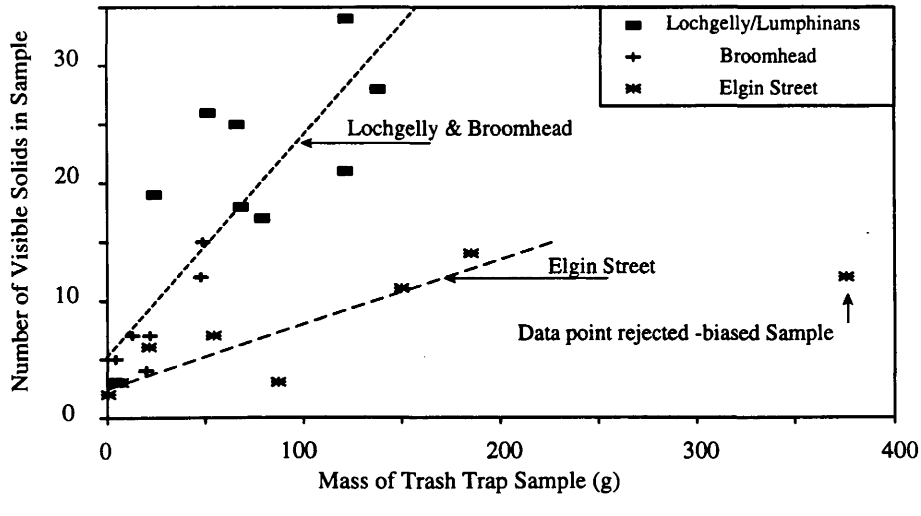


Figure 5.2 Relative Mass of Trash Trap Samples

Plate 5.1 Trash Trap Event 19 on 16/11/90 Lochgelly

Blinding 15% Trash Mass 66.5g

Number of Visible Solids = 25

Plate 5.2 Trash Trap Event 8 on 25/11/90 Broomhead

Blinding 50% Mass Collected 47.9g

Number of Visible Solids = 12

Site	No of Points	Number = C_1 x Mass + C_2		
		C_1	C_2	r^2
Lochgelly	9	0.189	5.3	0.69
Broomhead	7	-	-	-
Elgin St	7	0.055	2.6	0.77

Data relate to Figure 5.2

Table 5.1 Numbers of Visible Solids for a given Mass

A range of simple correlations between the measured variables were investigated. No relationship which incorporated the time of day of the overflow event could be developed. The mass of visible solids collected after each event also showed poor correlation with the volume discharged. However, for the three sites linear relationships were obtained between the number of visible solids and their mass, results being plotted in figure 5.2 with relevant statistics in Table 5.1.

Linear relationships would be expected if all particles from a site were of similar mass, with the scatter, as represented by their r^2 values, reflecting the variation in the type of particle. The scatter is large for all sites indicating high variability. The data also suggest that the Lochgelly and Broomhead solids were similar, while the masses of those from Elgin Street were approximately three times greater. It is suggested that the greater turbulence at the Elgin Street inlet, together with the non-submerged inlet pipes are contributing factors to the collection of heavier solids at that site.

Relationships involving the degree of blinding were also investigated. It would appear logical that the blinding should increase with both the numbers and masses of visible solids collected. The numbers collected when plotted against percentage blinding produced parabolic relationships with high r^2 values and are shown plotted in Figure 5.3 with relevant statistics in Table 5.2. There was considerable scatter among the limited number of data points available for the Lochgelly site, and no relationship could be developed. In contrast, the data for Broomhead and Elgin Street appear to follow similar curvilinear relationships. Differences of behaviour are apparent from inspection of Figure 5.3, the Lochgelly data showing significantly greater numbers of visible solids for a given degree of blinding than the remaining sites. These differences are considered to be a function of the performance of the CSO structures and are discussed further in section 5.4.

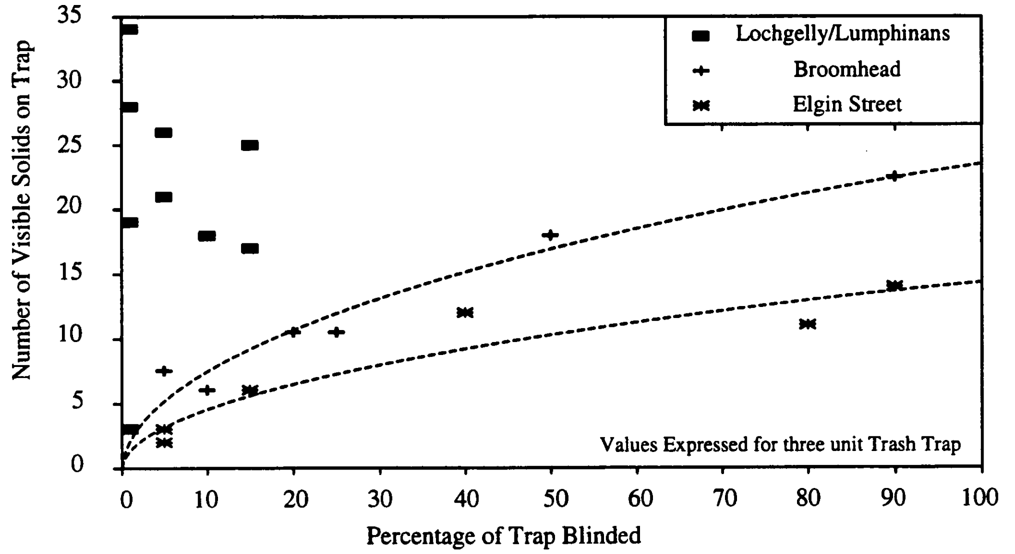


Figure 5.3 Trap Loads and blinding

Site	No of Points	Number=C_1 x $Blind^{1/2}$ +C_2		
		C_1	C_2	r^2
Lochgelly	9	*No Relationship		
Broomhead	6	2.35	0.2	0.95
Elgin St	6	1.46	0.08	0.89

Data relate to Figure 5.3

Table 5.2 Numbers of Visible Solids for a given Percentage Blinding

Figures 5.2 and 5.3 are interpreted as follows;

For a given mass of sample, higher numbers of visible solids were trapped at Lochgelly and their average weights were approximately one third of those at Elgin Street. The Broomhead data have a smaller range and it was unclear whether they were similar to the data from the Lochgelly or the Elgin Street sites. Ignoring the Broomhead data, it may be implied from Figure 5.2 that heavier, more readily settleable material was prevented from spilling at Lochgelly than at Elgin Street CSO. This view is supported by Plates 5.1 and 5.2 which illustrate the relative preponderance of light, neutrally buoyant, plastic strips retained on the Lochgelly Trash Traps.

The blinding also varied between the sites, as illustrated in Figure 5.3. The maximum degree of blinding at Lochgelly was 15%, while on six separate events at Broomhead the Trash Trap was blinded and the maximum at Elgin Street was 90%. A clear and consistent behaviour was observed at both the Elgin Street and Broomhead locations, with good correlations for the fitted relationships. Scatter of the Lochgelly data was too great to allow any relationship to be developed, however, the behaviour shown in Figure 5.3 clearly indicates that significantly less blinding matter was discharged than from the remaining sites.

It is concluded from these data that the principal material passing the Lochgelly installation was plastic strip material which had close to neutral buoyancy. The lack of the blinding-type material at Lochgelly indicates that particles in the size range 1-6mm were removed at this CSO. Greater numbers of visible solids including tampons and nappy liners were discharged from the Broomhead and Elgin Street locations but were not observed at Lochgelly.

5.3.3 Prediction of the Discharge of Visible Solids Using Trash Traps

In Section 2.2.1 it was suggested that an appropriate categorisation of solids particles was to use their physical characteristics, these being dissolved & colloidal, fine particulate and gross/visible solids. The boundaries between such categories are ill-defined and in any sewage flow, a continuous gradation of size is likely to occur between each principally due to the gradual degradation of some of the gross solids into finer particles. In a highly mixed sewage, such as those monitored in this study, it is reasonable to presume that an increase of gross/visible solids would be accompanied by an increase of fine particles, as expressed by TSS. Similar relationships have been reviewed for other solids-related pollutants in Table 4.6. It is also likely that a greater flow volume might result in greater numbers of visible solids.

This section is concerned with the development of any such relationships, derived from Trash Trap measurements, which might apply to visible solids. In all relationships and presentations, loads or numbers of visible solids as appropriate have been used. This contrasts to other studies where concentrations were measured and is due to the fact that the Trash Trap (and the Gross Solids Sampler) integrated the amount of material collected over each event. To produce values for concentration from these data would have required to be divided by either flow volume or flowrate. In either case the disaggregation of the information would have been inappropriate, resulting in indirectly computed data in place of better quality, directly obtained information.

The numbers of visible solids have been plotted against mass of TSS and flow volume respectively in Figure 5.4. Figure 5.4(a) shows less scatter than Figure 5.4(b) and this is taken as evidence that a relationship between visibles numbers and TSS mass has a greater reliability of prediction than with flow volume.

In both Figures 5.4(a & b) the numbers of visible solids increase rapidly with discharge volume when the latter is low. This is taken to correspond with the washing out of material previously deposited in the pipe system or overflow structure. Following the exhaustion of this

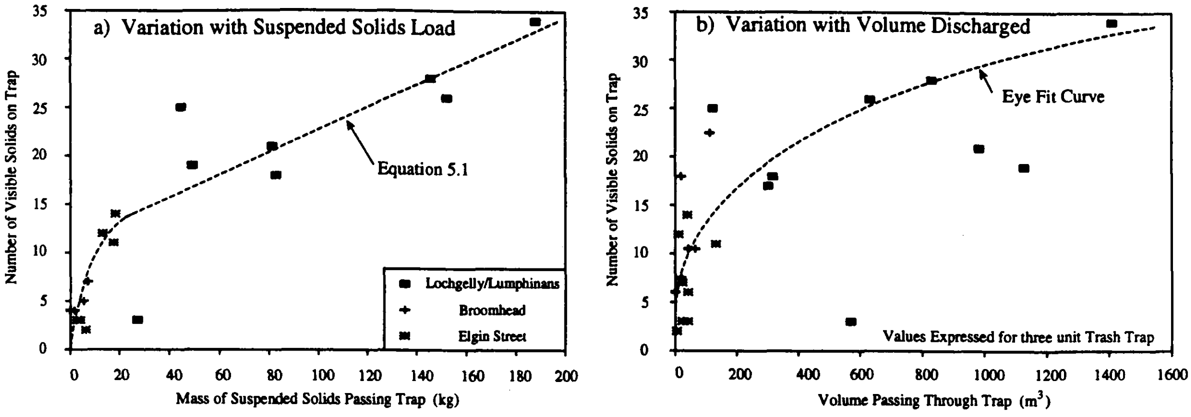


Figure 5.4 Relationships for Visible Solids Based on Trash Trap Method

source, the numbers discharged would have been principally reliant on the material within the foul sewage flow, and consequently would be reduced in concentration. Figure 5.4(a) shows a linear relationship following the initial flushing reflecting the lower availability of solids. A linear regression using data points with greater than 7 visible solids gave equation 5.1

No of Visible Solids = 0.115 x Mass of TSS (kg) + 11 **5.1**

r^2 for equation 5.1 is 0.960 confirming that, for the data used, the relationship is reliable. An eye-fit line was drawn on Figure 5.4(b) reflecting nothing more than a possible upper bound of data and no relationship could be developed using the discharge volume.

Equation 5.1 incorporates data from all three sites and should be compared with Equation 5.2 which was developed previously by the author (Jefferies 1992) using only Broomhead data. The method of analysis has also been simplified from the earlier work.

No of Visible Solids = 0.15 x Mass of TSS (kg) + 11 **5.2**

The result now presented as equation 5.1 is close to that produced previously and, since the principal additional data were from the Lochgelly site, it is considered that this equation has wider applicability and may be used for all sites in the study. It is further suggested that this equation confirms that a relationship exists between visible solids and TSS. Equation 5.1 may have a wider applicability but this would require to be tested using data from further studies.

5.4 RESULTS FROM THE TRASH TRAP STUDY

In section 5.4 the results from the Trash Trap analysis are applied to the total flows from the three CSOs under study. The data are interpreted as valid measurements of CSO performance from which conclusions may be drawn. Performance comparisons were made difficult by the paucity of inflow information gathered. Reliance had to be placed on the established similarity of the contributing catchments.

5.4.1 CSO Comparison Based on Numbers of Solids Collected on Trash Traps

The masses of visible solids collected on the Trash Traps have been factored up to represent the full discharge at each overflow. The factors used at each site are included in Appendix C, Tables C1 to C3. It is necessary here to caution the reader against making direct comparisons between the tables of Appendices C & D. This is because the various samplers had different event durations, consequently loads and volumes cannot be compared directly between tables. Reference should be made to Section 4.3.2 for the different event definitions used.

The masses have been plotted in Figure 5.5 against the total masses of TSS discharged at the same time. The data fall into zones within the plot, the results for Broomhead and Elgin Street being differentiable from those for Lochgelly by showing a higher rate of discharge of visible solids. The data shown in this format allow a degree of generalisation to be made and the plot enables the suggestion to be made that guidelines for visible solids performance may be possible.

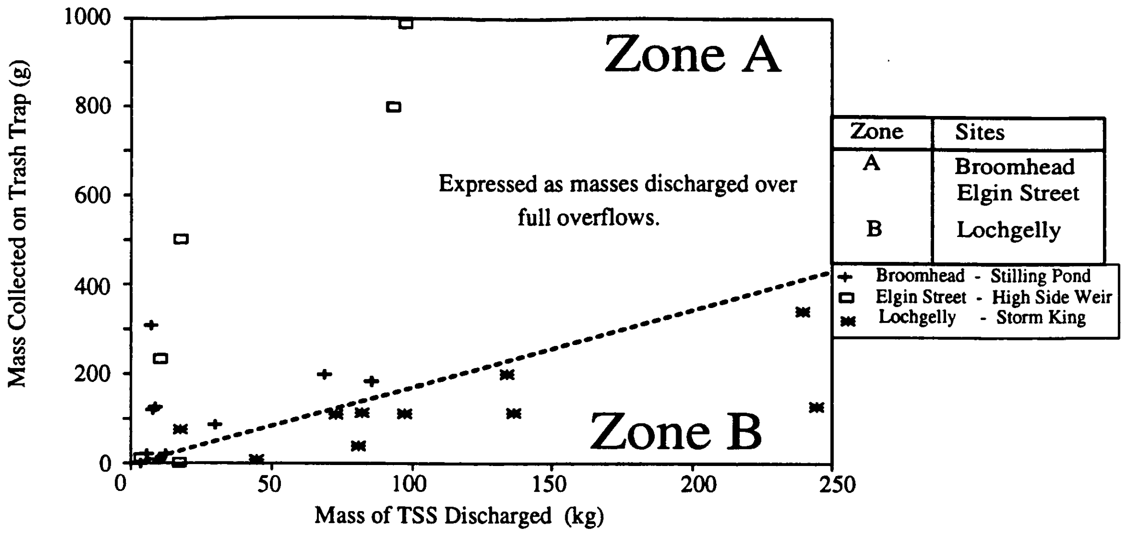


Figure 5.5 Comparison of Performance Using Mass of Solids On Trash Trap

The dividing line between the zones has been drawn specifically to separate the groups of data points. With this separation, only two points from each group lie on opposing zones, a combination which has less than 0.5% probability of occurring by chance using Fisher's exact test for a 2x2 table (Seigel 1956). The overlapping points also have low values of discharge, in an area of the graph where poor definition is to be expected.

5.4.2 CSO Comparison Based on Trash Trap Blinding

A complementary but different method of presentation of the Trash Trap data is included as Figure 5.6. The percentage blinding has been plotted against the volume passing over each trap, here expressed in terms of the amount per trap. The data fall into the same groups, but to a more exaggerated extent than in Figure 5.5 with overlapping of the data being minimal. Straight line regression fits have been applied to the data and, although the scatter is wide and the low r^2 values suggest the lines have little meaning, they do have completely different slopes and clearly represent very different data populations.

The best fit lines of Figure 5.6 are expressed as equations 5.3 and 5.4.

For Broomhead & Elgin Street;

$$\text{Blinding (\%)} = 0.479 \times \text{Volume Passing Trap } (m^3) + 22.3 \quad (r^2 = 0.24) \qquad \textbf{5.3}$$

For Lochgelly;

$$\text{Blinding (\%)} = -0.0057 \times \text{Volume Passing Trap } (m^3) + 8.3 \quad (r^2 = 0.21) \qquad \textbf{5.4}$$

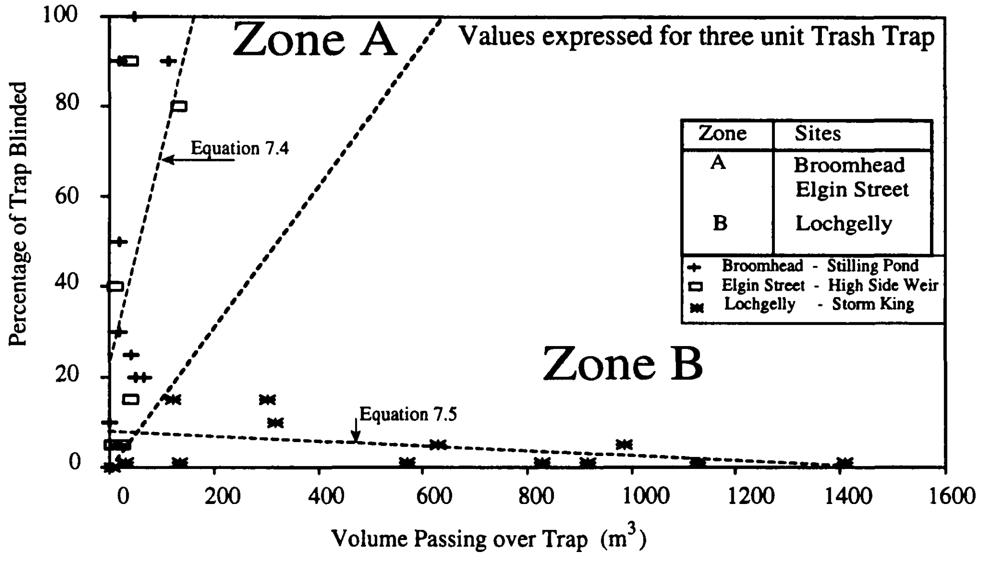


Figure 5.6 Comparison of Performance Using Blinding of Trash Traps

The data suggest that the performance of the stilling pond and high-side weir are similar as might be expected due to the lack of storage volume. In neither presentation of the data can their behaviour be differentiated, however, they are clearly different from the hydrodynamic separator.

5.4.3 Trash Trap comparison of Discharge Quality

Figures 5.5 and 5.6 have been developed to characterise the CSO discharge quality in terms of the indicators measured by the Trash Traps. Figure 5.5 shows that, for a given mass of TSS, significantly less mass of visible solids passed the overflow of the separator than at the other locations. Figure 5.6 shows that, for a given discharge volume, significantly less blinding material was also discharged. Together these figures show that the combined sewage discharged at both Elgin Street and Broomhead contained significantly greater amounts of blinding material **and** visible solids than at Lochgelly. This joint effect suggests that the following conclusions may be drawn:-

i) The blinding material and visible solids were subject to the same hydraulic influences;and,

ii) Both types of material were removed preferentially at Lochgelly in comparison with the other sites.

The first conclusion above may be developed further with reference to section 5.3.2 in which it was observed that the blinding material was visually similar to type C sewage sediment. Such similarity was also deduced from the Gross Solids Sampler results, and this argument is developed further in section 6.5.3.

It may be argued that data are absent relating to inlet loads, and that the second conclusion, which relates to the removal of solids, cannot be based on direct measurements at the CSO inlets. Such information was not gathered as it is extremely difficult to obtain during storm conditions due to practical problems of blinding causing hydraulic blockages. However, it was contented in section 4.6.4 that there are sufficient other data to support the claim of little material difference between the sites. Both storm and dry weather data from the small-bore samplers show differences between sites but these cannot be described as being significant. Consequently it is believed that sufficient evidence has been presented to conclude that Figures 5.5 and 5.6 do indeed indicate differences in the performance of the CSO structures.

5.5 CONCLUSIONS

The Trash Traps provided information on the numbers of visible solids intercepted and on the percentage of the Trap blinded. This information has been interpreted to provide an expression for the discharge of visible solids and to compare the performance of the combined sewer overflows studied.

5.5.1 Rate of Discharge of Visible Solids

It may be noted from Figure 5.3 that the Elgin Street and Broomhead Traps collected approximately twice as many visibles as those at Lochgelly for a given percentage blinding. Large visible solids such as tampons and nappy liners were found on the Traps at the former sites, but not at the latter, these observations being made in spite of the loadings of visible solids per Trap being the highest at Lochgelly.

The Trash Traps have provided in Equation 5.1, a novel basis for estimating the numbers of visible solids likely to be discharged at an overflow. This equation relies on the estimation of the TSS load discharged during an event and is in an appropriate form to be incorporated in a sewage quality model such as MOSQITO (Moys 1987). Equation 5.1 does not apply to small events with a strong positive first flush.

5.5.2 Performance Evaluation using Trash Traps

It is proposed that Trash Trap results, when presented in the form of Figure 5.5 or 5.6 may be used as a method of differentiating between overflow devices. The Figures, while presenting information in terms of mass of visible solids and blinding matter respectively, show consistent hydraulic behaviour as it is contended that both types of particle are subject to the same hydraulic influences. With additional information from more sites in future, further subdivision, particularly of zone A may be possible.

Scatter in these figures is too great to allow linear relationships to be established. Figure 5.6 shows that the discharge of visible solids at Lochgelly was approximately one third of that at Elgin Street. The data from Broomhead suggest that the performance of this installation lies in an intermediate position between the others.

The Trash Trap method is put forward as a novel performance indicator for combined sewer overflows. It has the merit that the Traps are cheap to install and simple to operate. The principal information relating to assessment is contained in Figures 5.5 and 5.6. These two diagrams show that the Hydrodynamic Separator as installed at Lochgelly removed more visible solids and considerably more blinding solids than the two conventional overflows studied.

It is contended that conclusions relating to the actual performance of the CSOs may be made in the absence of direct information on whether the inputs to the three installations are similar. The case has been made that the inputs were at least comparable and that the differences observed at the overflows using the Trash Traps resulted from the operation of the overflows themselves.

Application of the Trash Trap method requires the following:-

i) Installation of Traps over a length of the CSO weir. Installation and measurement should be as described in section 3.6.4. The masses of visible solids collected and the percentage blinding should be recorded.

ii) Flow monitoring is essential during the Trash Trap installation period. Flows and volumes during events should be determined in accordance with the principles set out in section 4.3.2.

iii) Small-bore sampling during CSO discharge is desirable and operation should be as described in section 3.6.1. Samples should be tested for total suspended solids.

iv) The information should be plotted in accordance with Figures 5.5 and/or 5.6 depending on the data available. Data lying in zone B indicates improved performance over zone A.

5.5.3 Further Work for the Trash Traps

Maximum flowrates which could be accomodated were limited by the design of the Traps used. An improved design has been laboratory tested and should be used in further studies. The improved design is included in Appendix H.

Continuing studies are required to provide more detail for zones A and B. With more detail, linear or curvilinear relationships may be developed to define acceptable performance of different CSO installations.

CHAPTER 6 THE WRc GROSS SOLIDS SAMPLER

To Observations which ourselves we make
We grow more partial for th'observer's sake
Alexander Pope   Moral essay to Lord Cobham

6.1 Introduction

In this chapter, the performance of the Gross Solids Sampler (GSS) is reviewed and the results interpreted. The GSS was the second item of equipment used in the study for sampling gross and visible solids. The Trash Traps, used for the direct interception of visible solids at the CSO weirs were detailed in chapter 5.

The GSS was first installed at the Broomhead site where it operated for five months, entirely during wet weather flow periods. This site was the first field installation of the GSS and some time was spent initially obtaining settings which were appropriate for the particular site conditions. Following this initial period, standard operating settings were used. At the second site, Elgin Street, the GSS operated during both dry and wet weather flow periods. As part of the GSS evaluation exercise, and in common with the Trash Traps, methods of presenting the data had to be developed prior to further analysis.

From the dry weather flow data there was found to be close correspondence between the variation of gross and suspended solids in the sewage. A relationship was developed between the load of gross solids in dry weather and that of suspended solids at the observation point. It is contended that this relationship is an addition to knowledge of the behaviour of this type of material and it is presented for use with other quality predictive methods.

The principal aim of installing the GSS was to determine whether the performance of a CSO could be evaluated in terms of gross solids. Interpretation of the data showed that this aim could not be achieved as the GSS was unable to differentiate between the inlet and overflow at the two sites. Wide variations were noted between the samples retrieved from the inlet and overflow intakes and it was found that the equipment was of greater value in comparing the influent gross solids at each site rather than the overflow types.

The information from the inlets at each GSS site was related to the contributing catchments and a chart is presented which differentiates with a high degree of reliability the rate of gross solids production of the two different types of catchments. It is contended that one site was representative of a collector sewer catchment, the other of a trunk. A consistent and further division of the data is presented on the basis of antecedent dry period. Smaller antecedent dry periods allowed considerably smaller accumulations of gross solids than those longer than 24 hours, and the evidence suggests that there was little accumulation thereafter. It was further concluded that the gross solids were subject to the same hydraulic influences as type C sewer sediment material.

An appraisal of the performance of the GSS is included in section 6.2 together with an evaluation of the variables used in its operation. Section 6.3 details the results from its operation in dry weather flows and the prediction method for the number of visible solids in the flow, based on the total suspended solids load is presented as equation 6.4. The inconclusive analysis of the results from the operation of the GSS during CSO events are contained in section 6.4.

Section 6.5 deals with the principal claim to an advancement of knowledge in chapter 6. This is in the chart (Figures 6.8 & 6.9) which enables the GSS production of the catchments to be differentiated. The importance of the 24 hour ADWP and the commonality of gross solids behaviour with type C sediments are also deduced in section 6.5. Section 6.6 contains interpretation of the results gained and recommendations for further work which might be undertaken using the Gross Solids Sampler.

Details of all relevant results from the Gross Solids Sampler are included as appendix C. It should be noted that no attempt has been made in chapter 6 to compare the GSS results with those from other sampling methods, particularly the Trash Traps. Such comparisons are included in chapter 8.

6.2 GROSS SOLIDS SAMPLER TESTING

6.2.1 Overview of Sampler Operation

Installation details for the Gross Solids Sampler (GSS) are included in section 3.6.3. During combined sewer overflow events the GSS operated successfully a total of 27 times during 22 separate events on 18 days at the Broomhead Site, and 16 times during 14 separate events on 10 days at Elgin Street. Appendix C, Tables C4-C7 give details of the operations and the control settings used, together with the basic data derived from the GSS and accompanying equipment. The GSS and an associated test, named the Ring Bag test were also operated at the Elgin Street site during dry weather flow for which the data are presented in Appendix C, Tables C8 & C9.

Three normal modes of operation were used as follows;

Automatic for storm events when sampling was triggered by a rise in level within the overflow

Manual Several storm events were prolonged and the GSS terminated its run before cessation of the event. The samplers were restarted to obtain more data from the end of the storm.

Dry Weather Operation was almost continuous for selected periods to sample dry weather conditions. The Ring Bag test was used concurrently with the GSS to determine the numbers of visible solids in the flow.

The charge, wait and sample times were varied at the start of sampling at Broomhead. Adjustment of the measured values was necessary to account for these differences. The COPA sack size was also varied during events 22-26 at Broomhead.

In an initial trial of twelve minutes duration, fine (2-3mm) and medium (4-6mm) COPA sacks were switched every minute thus collecting 6 samples in each sack, the samples collected being effectively from the same flow. Almost no difference was noted between the masses collected in the different sack types. A similar test was carried out using the same equipment in Swansea (Walsh 1992) and this led to the same conclusion that the size of COPA sack opening had little effect on the amount of material collected. Measurements during events 22-26 at Broomhead when 2-3mm sacks were used were included in the analysis unaltered.

The GSS inlet was raised for a trial run to be as close to the sewage surface as possible while excluding air. Buoyant particles were noted to pass the inlet and it was concluded that the GSS was not effective at sampling floating particles.

6.2.2 Volumes Sampled

During event-based sampling, different sample volumes were drawn by the GSS and are shown Tables C.5 and C7. The differences were due to varying intake velocities and sample times used. To compare the actual volumes of water passing through the COPA sacks during each test, the volume sampled was multiplied by the number of samples. To indicate the relative magnitude of the volumes sampled, during the event of 6/12/90 (Test 13 at Broomhead), 3.2 and 2.5 m^3 passed through the inflow and spill flow sacks respectively. This in turn represented 1.2% and 2.9% of the inflow and spill flow. Although these figures may appear to be small, they should be compared with the Epic samplers operating at the same time which sampled 0.0008% and 0.0005% of the flows respectively.

During dry weather flow sampling at Elgin Street, the sampler operated for twenty minutes in each hour. The net sack weights were scaled up to represent the full flow in this time period since only a portion of the variable dry weather flow was sampled. Equation 6.1 was used to scale up the values.

$$\text{Full Flow } \mathbf{NSW} = \mathbf{NSW} \times \frac{\text{Flow Volume in 20 min } (m^3)}{\text{Volume Through sack } (m^3)} \qquad \mathbf{6.1}$$

where **NSW** = Net Sack Weight

6.2.3 Missed Events

A number of storm overflow events were not sampled at each site. The causes for missing the events have a bearing on the interpretation of the results and are listed below:

(a) In showery weather, each overflow event may be of a short duration. The GSS had to be manually reset and, unless there was immediate attendance following events, data were lost when the inter-event time was short. This was particularly noticeable at Broomhead.

(b) When prolonged wet weather occurred the storm event continued beyond the total sample cycle. Days of prolonged low intensity rainfall resulted in a number of sets of samples being collected. The GSS was then switched off as further data were considered to be of little value.

(c) Operator error in forgetting to set or arm the sampler.

(d) Equipment malfunction resulting in some events being missed.

(e) Severe frost for a prolonged period froze the water in the section of sample tubes outside the container. Since the sites were relatively flat it was impossible to avoid low spots in these tubes which remained full. Rapid thaw during heavy rain triggered the sampler, however, the results were invalid due to the blockage. Severe frost occurred in January and February 1991 when some data were missed due to the frozen tubes.

6.3 DRY WEATHER GROSS SOLIDS TESTING

6.3.1 Dry Weather Flow Operation

A total of thirteen tests were carried out in dry weather flow in which all the solids were intercepted in the total flow. A brief description of this test, for convenience termed the **Ring Bag Test** is presented together with results in Appendix C, Table C.8. Counting the material collected from the full flow showed that the visible proportion of the solids were 15, 55 and 30 percent plastic, paper and faecal matter respectively. Table C.9 shows the results from all GSS runs on DWF days, these being presented separately, since the ring bag tests were only carried out on a proportion of the GSS runs.

6.3.2 Daily Variation of Gross Solids in Dry Weather Flow

The results from the ring test have been plotted together with the COPA sack weights in Figure 6.1. Visibles have been expressed as the number in the 20 mins concurrent with the GSS samples.

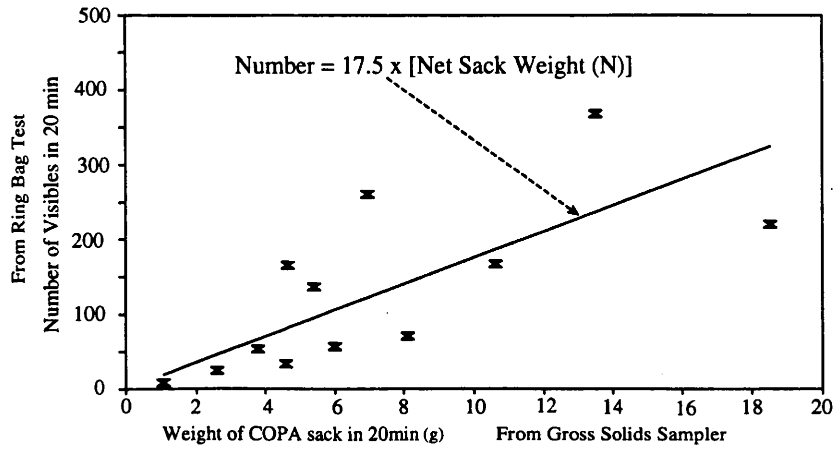


Figure 6.1 COPA Sack Calibration for DWF Elgin Street

Equation 6.2 resulted from a least squares regression fit;

$$\text{No of Visibles} = 17.5 \times [\text{Net sack weight (N)}] \qquad \textbf{6.2}$$

It is believed that the scatter in Figure 6.1 results from the use of instantaneous measurements with no averaging and as a consequence a low r^2 value of 0.503 resulted. The scatter could not be attributed to the time of day at which the readings were taken. In view of the scatter of individual points, averaged values at concurrent times, where available, have been used in further analysis.

The results from a total of 26 runs using the GSS, each lasting twenty minutes are presented in Table C.9. The resulting averaged GSS sack weights and averaged TSS concentrations are plotted against time in Figure 6.2 which shows a degree of correspondence between TSS and sack weight.

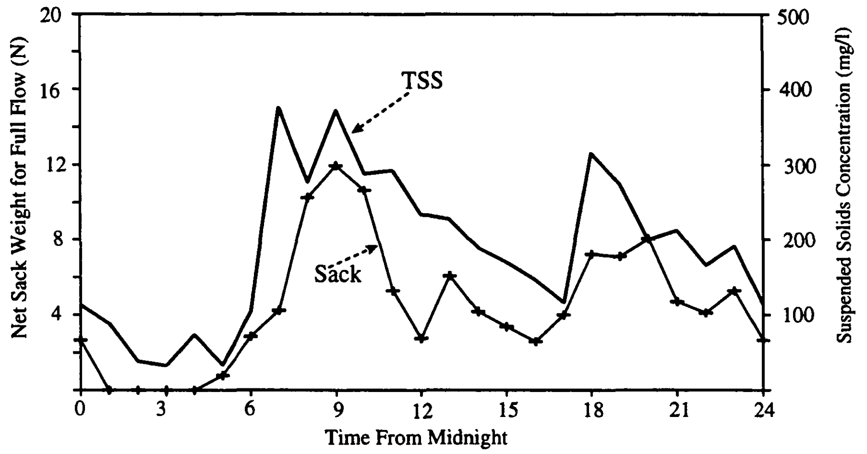


Figure 6.2 Daily Sack Weight Variation for DWF Elgin Street

Equation 6.2 and Figure 6.2 have been combined with the flowrate to obtain concentrations of TSS and visible gross solids. The results of this exercise are plotted in Figure 6.3, a moving average of 3 values having been used as a smoothing function. The visibles concentration reached peaks at corresponding times to the TSS values and it is contended that, by visual comparison of the suspended and gross solids concentrations, the diurnal variation of gross solids is reasonable. The data have a correlation coefficient of 0.61, showing that a fairly strong relationship exists, however, due to the derivation technique used for Figure 6.3, a relationship (Equation 6.4) was developed as described in section 6.3.3.

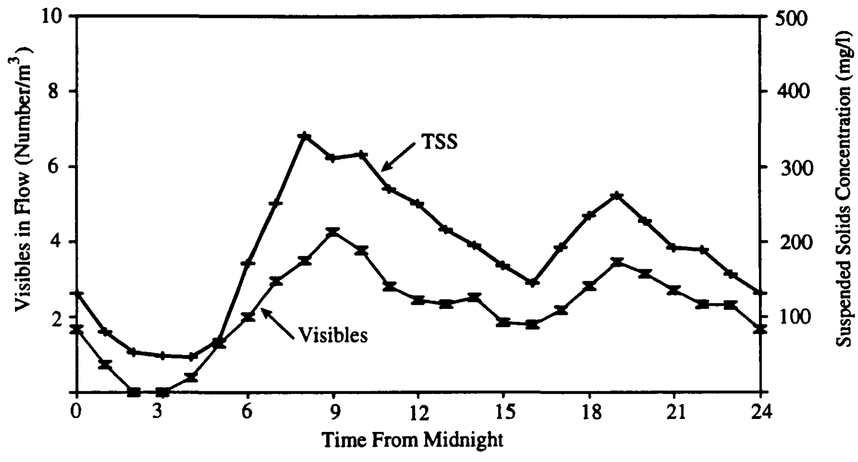


Figure 6.3 Daily Total Visible Solids Variation for DWF Elgin Street

6.3.3 Prediction of Discharge of Gross Solids During DWF.

Two relationships have been developed using average values for concentration and flows. These are illustrated in Figures 6.8 and 6.9 in which the GSS weights and Visibles numbers have been plotted against the TSS load. The following equations result from these figures;

GSS Sack Weight (N) = 0.721 x TSS Load (kg) - 0.18 **6.3**

Number of visibles = 0.61 x TSS Load (kg) + 20 **6.4**

Equation 6.3 implies that a small TSS load may occur at the observation point without any measurement by the Gross Solids Sampler, suggesting that the sampling by the GSS was incomplete, particularly at low flowrates.

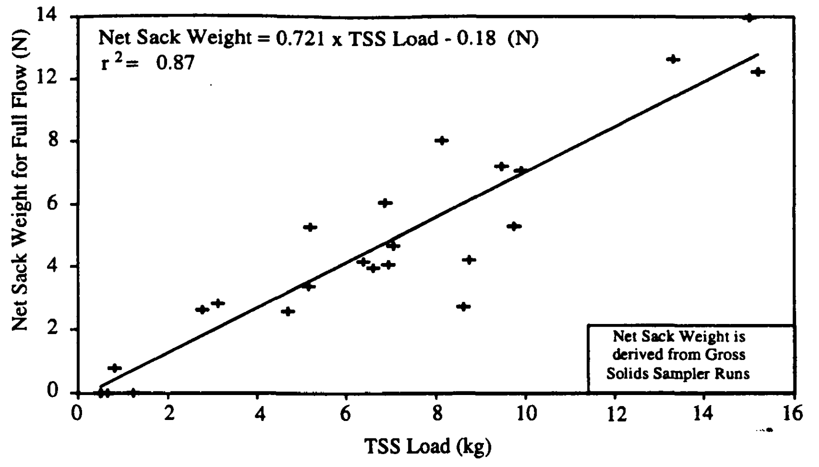


Figure 6.4 Sack Weight v TSS Relationship for DWF Elgin Street

In contrast, Equation 6.4 suggests that visible solids may have been present in the flow of sewage even when the TSS load reduced to zero, implying that the tractive force required to maintain visible solids in suspension was less than for suspended solids particles. In combination the two equations point to problems of sampling by the GSS at low flows. This was unlikely to have been due to low suction velocities as most of the material under consideration was almost neutrally buoyant, but was probably caused by the wrapping of solids around the suction tube. An alternative explanation of Equation 6.3, is that visible solids particles passed below the suction tube, thereby not being sampled.

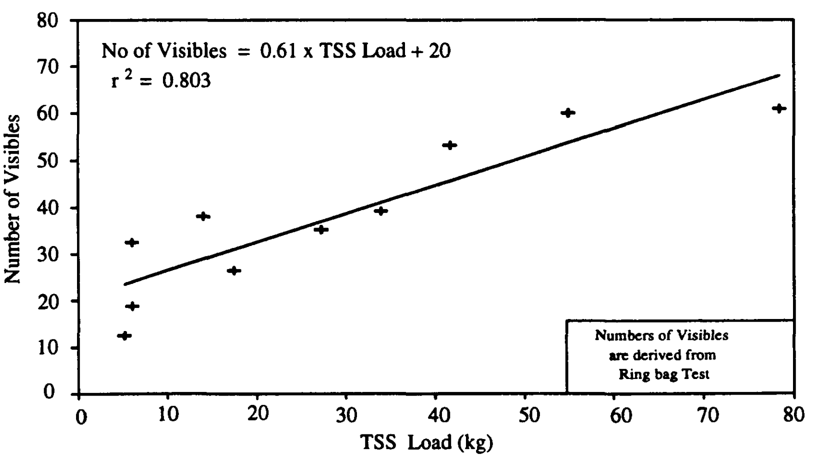


Figure 6.5 Visible Solids Calibration for DWF Elgin Street

6.4 GROSS SOLIDS SAMPLER OPERATION AT COMBINED SEWER OVERFLOWS

6.4.1 Observations on samples collected

Following sampler operation during storm events the COPA sacks were taken from their drums and hung for at least two hours before their weight was recorded. The samples were normally damp upon weighing and only the small samples, composed primarily of paper, were completely dry. The net sample weights given in Appendix C, Tables C.5 and C.7 were recorded after the drying period and exclude the weight of the sack. These tables also give flow and suspended solids information derived from the other equipment at the site. Plates 6.1 to 6.4 show typical sack samples for a range of event types.

Three principal observations were made from visual examinations of the samples collected;

> The overflow sacks were notable for having very little trapped material. On 14 events at Broomhead and 4 at Elgin Street, no measurable weight of material was collected in the overflow sack.
>
> The majority of events produced small amounts of material in the inlet sack and on all but three occasions paper and plastic strips were predominant.
>
> Only on three events (all at Broomhead inlet) was the mass trapped greater than 500g.

The contents of the largest sample (event 13 on 6th December 1990 - see plate 6.1) were examined visually. The material was found numerically to be 50% faecal matter and 50% tampons and associated plastic material. Almost no condoms or plastic strips were recovered from the COPA sacks.

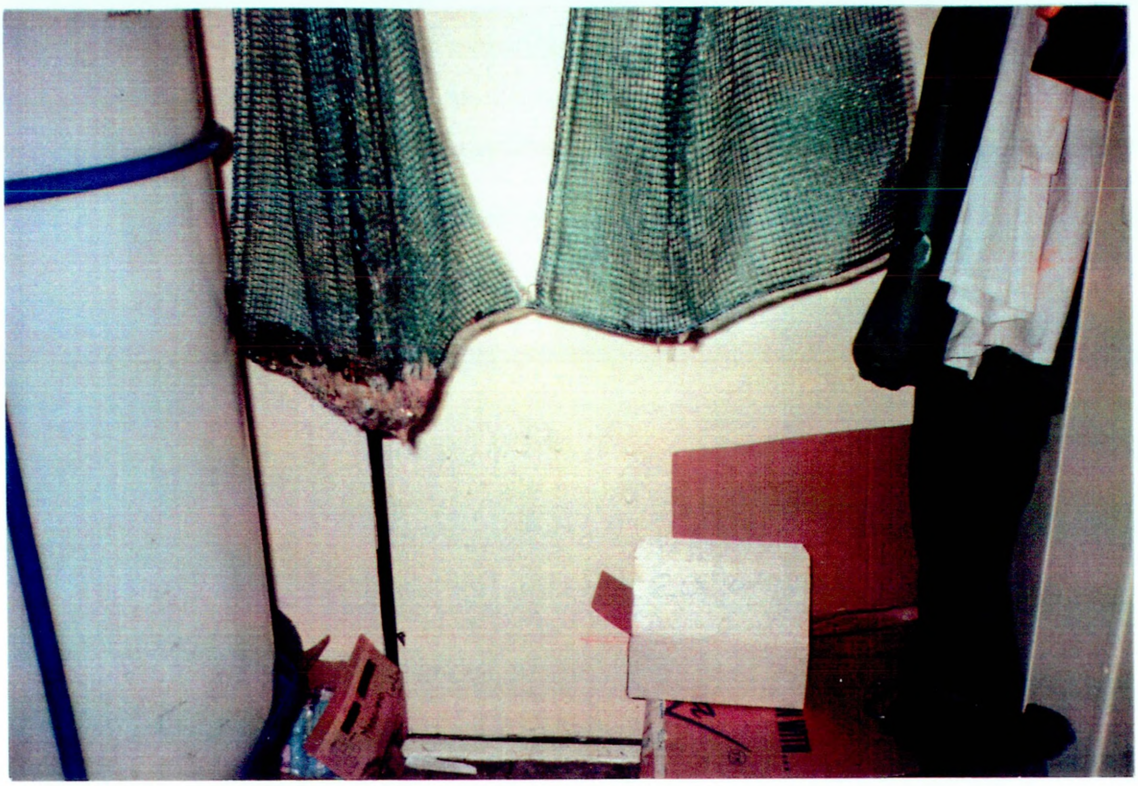


Plate 6.1 Test 4 20/10/90 at Broomhead

Almost no material in Overflow Sack

Plate 6.2 Test 10 on 16/11/90 at Broomhead

Inlet Sack Mass 1.70kg

Plate 6.3 Test 13 on 6/12/90 at Broomhead

5.58kg in Inlet Sack - the largest recorded

Little in Overflow Sack - principally leaves

Plate 6.4 Test 27 on 17/3/91 at Broomhead

Very little material collected

6.4.2 Event Based Expressions for Gross Solids

In considering the results from the GSS it should be noted that the single bulked GSS samples did not relate directly to the volume discharged as they were not flow proportioned. In contrast the Epic samplers took discrete samples from which the Total Suspended Solids (TSS) concentrations and other determinands could be found. Concentrations were related to the flowrate allowing the load in each time interval to be deduced using the relationships presented in section 2.5. Notwithstanding these comments, GSS concentrations have indeed been calculated using the method described below and are considered to be an appropriate method of expression of the results.

In expressing the behaviour of the gross solids, a number of different terms have been employed, these being the Event Mean Concentration (EMC), Gross Solids Load (LGSS), GSS Load Rate and GSS Ratio.

Event mean concentrations of GSS and TSS were determined using equations 4.6 and 6.5.;

$$\text{Mean GSS Concentration} = \frac{\text{Net Sample Load}}{\text{No of samples x Volume}} \qquad \textbf{6.5}$$

The GSS Load (LGSS) and GSS Load Rate have been used to express the discharge of GSS during an event and were determined using equations 6.6 and 6.7.;

$$\text{LGSS} = \text{Mean GSS Concentration x Flow Volume} \qquad \textbf{6.6}$$

LGSS is expressed in kg

$$\text{GSS Load Rate} = \frac{\text{LGSS}}{\text{Sampler Run Time}} \qquad \textbf{6.7}$$

To make comparisons between different locations and in particular between the inlet and spill of overflows, the term GSS Ratio was used. This relates the Gross Solids Load to the TSS load (derived from the Epic Sampling) as follows;

$$\text{GSS Ratio} = \frac{\text{LGSS (kg)}}{\text{TSS Load (kg)}} \qquad \textbf{6.8}$$

The term TSS Load is equivalent to PSL and PIL in considering CSO efficiencey (equations 2.11 and 2.12) and in the following sections, where both loads are used the term LTSS is used.

6.4.3 Spill Flow Separation.

The basic sample weights of the COPA sacks were reduced by the sack weight and converted to grammes, giving the net sample weights. All data were factored to represent sample times of one minute, thus the net weights were quadrupled for the first test at Broomhead and doubled for tests 2-12. The inlet and overflow weights are presented in this raw form in Figure 6.6 (a&b).

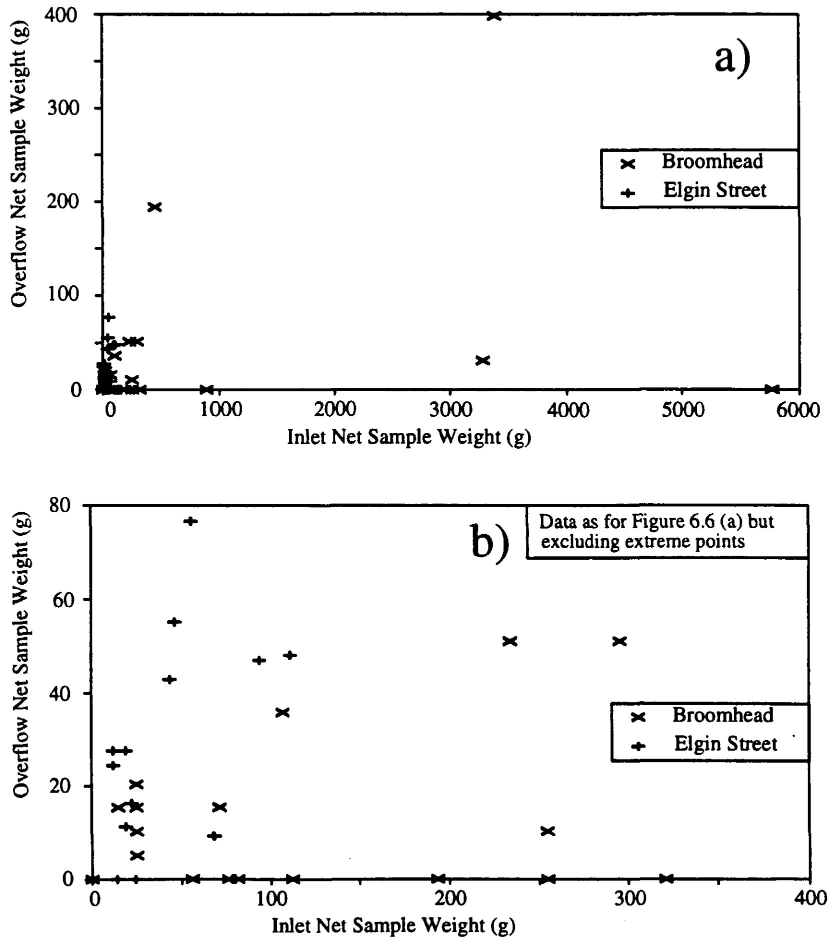


Figure 6.6 Gross Solids Sampler Results. Sample Loads - Broomhead & Elgin Street

Figure 6.6(a&b) is presented in a basic format to illustrate the differences between inlet and overflow sack net sample weights. Figure 6.6(a) shows all weights for both CSOs. It will be observed that Broomhead was characterised by having the occasional large mass of solids in the inflow COPA sacks, with more modest quantities from the spill flow. These high amounts were absent from the Elgin Street data where all weights were less than 150g. Figure 6.6(b) gives a more detailed representation of the separation from inflow to spill flow.

Interpretation of the information was made difficult by the very low amounts of solids obtained from the spill flows at each site. Of particular note was event 13 at Broomhead when nearly 6kg was collected during one event at the inlet and almost nothing from the spill flow. Some of the events showed a small excess of spill flow sample weight when compared with the inflow, particularly at Elgin Street, suggesting that the structure in certain circumstances caused a concentration of material close to the weir.

This observation concurs with that for suspended solids concentrations as described in section 4.3.3. The effect was only observed on minor spill flow events and it is suspected that this was due to an accumulation of material close to the weir wall before the level rose above the base of the scumboard. While this effect must be seen as being significant during small spill flow events, there is no evidence that it occurred during more severe events.

The data in figure 6.6 (a&b) are widely scattered. Some of the Elgin Street data lies above the 45° line suggesting a higher rate of gross solids overflow than inflow, however, this is likely to be a random effect caused by uneven sampling of material. When presented in this manner, the data show no clear separation of gross solids at the overflow.

6.5 GROSS SOLIDS AND SEDIMENT MOVEMENT

It was shown in section 6.4 that the data from the GSS could not be interpreted to enable a comparison to be made of the gross solids separating efficiency of the Broomhead and Elgin Street CSOs. Consequently this prime aim of installing the GSS could not be achieved, the cause being principally the small amounts of material retained by the overflow sacks. Greater amounts were gained from the inlet sacks and in section 6.5, data from the inlet sacks only are considered, along with information from the other equipment deployed. The duration of the ADWP was found to have an important influence on the rate of gross solids collected, and is used as a basis for differentiating between the gross solids behaviour at the two sites.

Figures 6.8 and 6.9 are proposed as charts which give a method of differentiating between the behaviour of the catchments. The technique may easily be related to similar Gross Solids Sampler data from other catchments since only flow data are required in addition to the GSS results. It is suggested that catchments with slack gradients allowing greater numbers of deposition zones (Collectors) will correspond to zone 1, while steeper, larger catchments (Trunks) will correspond to zone 2.

6.5.1 ADWP and Gross Solids

The study precluded a direct comparison of loads with rainfall data or with peak concentrations due to the method of sampling by the Gross Solids Sampler. ADWP could however be related to event mean concentrations and data for the inlets to each overflow are presented in Figure 6.7. TSS mean event concentrations have been included to allow comparisons to be made between determinands.

A number of studies (Pearson et al 1986, Stotz & Krauth 1984, Tucker & Mortimer 1978) where suspended loads were measured have shown correlation between the type of first flush and the antecedent dry period, whereas in

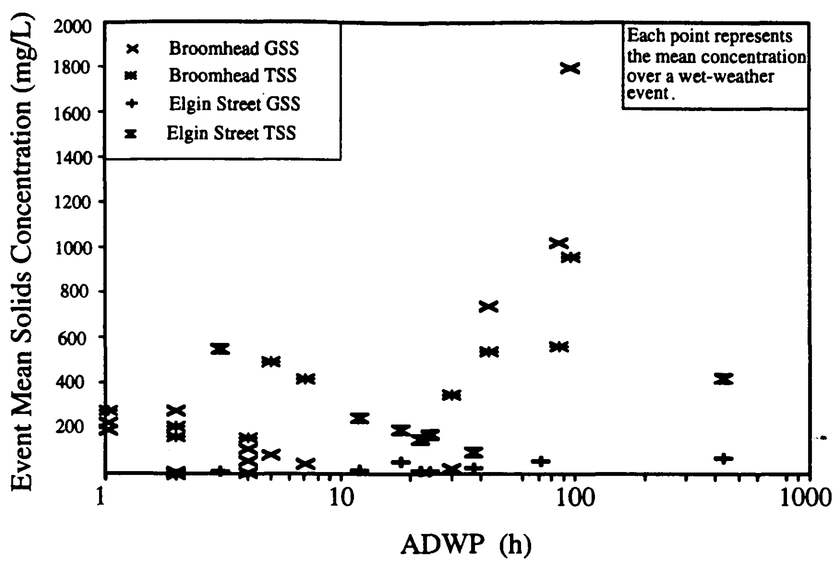


Figure 6.7 Gross Solids Sampler Results Dependence upon Preceding Dry Period

others, particularly on larger systems (eg Geiger 1986), the flush has been found to be dependent on the dry weather flow levels. Further studies (for example Ellis 1986) have tended to suggest that there is no relationship. In the present study, some correlation between ADWP and amount of gross solids during CSO events was anticipated particularly as there was a predominance of type B flushes at both sites.

It is suggested that considerable significance may be attached to antecedent durations, ie; whether it is greater or less than 24h, in the interpretation of the rate of production of gross solids from the two catchments. A number of different arguments are employed to support the belief that the 24h dry period is critical for the accumulation of gross solids sediments.

Figure 6.7 shows the event mean solids concentrations found from the Gross Solids Sampler (GSS) and small-bore sampler (TSS). A wide range of TSS concentrations from almost zero up to nearly 600mg/l was observed to occur at both sites when ADWP was short, and least variation was observed with ADWP in the region of 24 hours. It is contended that this lack of variation is a reflection of the regular daily deposition and erosion of sediments. following longer antecedent dry periods, there was an increase in the observed concentrations, this effect being most apparent at Broomhead and only to a lesser degree at Elgin Street.

At Broomhead the gross solids concentrations were always higher than suspended solids when ADWP > 24h, whereas they were always lower for shorter antecedent dry periods. This suggests that greater proportions of gross to suspended material were released following a period when the time available for accumulation was greater. In contrast, at Elgin Street, with only one exception, the gross solids concentration was always less than that for suspended solids. The inference is that the depositional characteristics in the Broomhead catchment produced a greater accumulation of gross solids (as compared with TSS) than at Elgin Street.

6.5.2 GSS Load Rate and Average Event Flow

Various methods of presentation of the gross solids loads with flow volumes for each event were evaluated to determine whether any form of relationship existed. No form of linear relationship could be found between load and volume, however when these data were expressed as rates by dividing the loads by the duration of the Gross Solids Sampler runs, the results were found to be consistent and are presented in Figures 6.8 & 6.9. Figure 6.8 includes all data for each site, whereas Figure 6.9 shows only data with a GSS load rate $\leq$10kg/min. Figure 6.9 has been termed a chart to allow more ready differentiation from other figures in this thesis. This form of presentation is justified since the Gross Solids Sampler produced a single bulk sample for each run and the GSS rate is in effect an averaged rate of accumulation over the duration of the sampler run. The basic data for these figures is to be found in Appendix C Tables C.4 to C7. It will be observed in Figure 6.8 & 6.9 that there is a strong association between ADWP and GSS production.

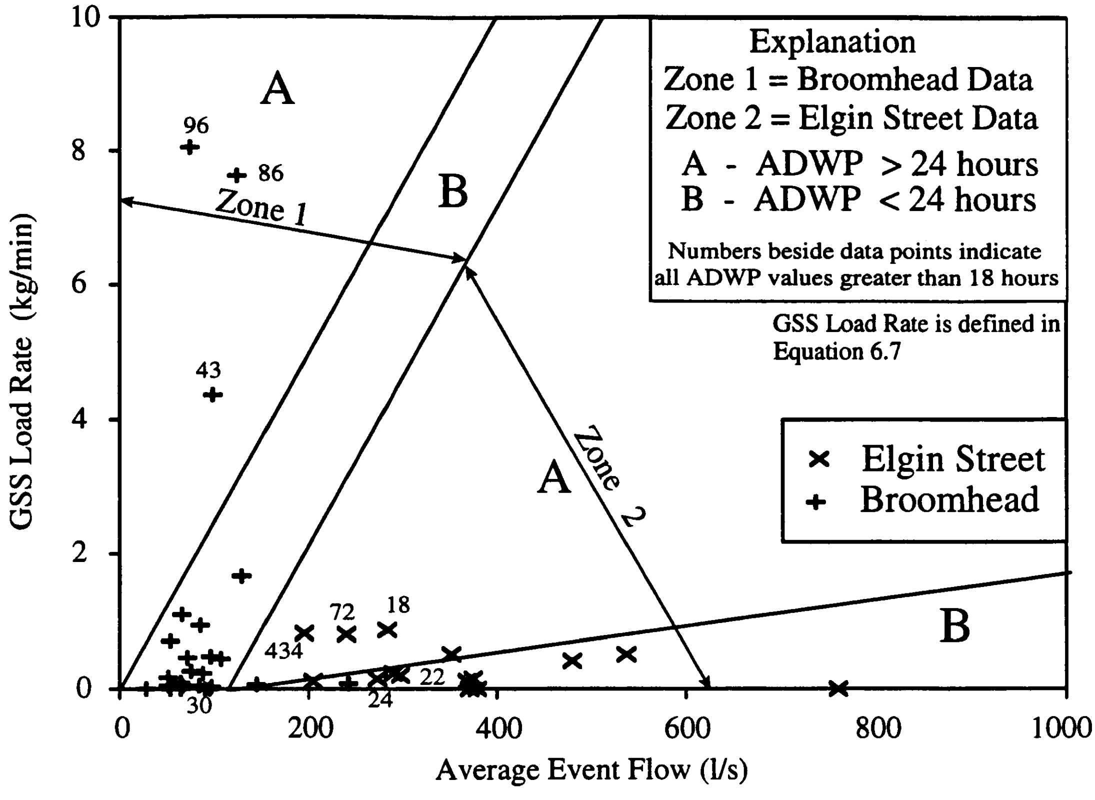


Figure 6.8 Chart for Comparison of Sites by GSS Load Rate - All Data

Data from the inlets only at the two sites have been used. Consequently the data relate to the catchments and are independent of the operation of the overflows, although the amount of backing up caused by the overflow weirs may have had an influence on the presence or otherwise of sediment deposits. The Broomhead site was characterised by low flows and high GSS rates, whereas at Elgin Street there were higher flows reflecting the size and population of the catchments. The gross solids concentrations at Elgin Street were significantly lower as were the Suspended Solids concentrations.

The antecedent dry period is seen to have had a significant effect on the GSS load rates at the sites. It may be noted, in agreement with the presentation in Figures 6.8 & 6.9 that any ADWP greater than one day caused higher GSS load values to be obtained, indeed one value of ADWP = 18h leads to the suspicion that the required period may be defined as a sufficiently long period when the dry weather flow was at or below average. This form of definition presumes that gross solids are deposited in the sewer on a daily basis at times of low flow and are transported during diurnal peaks.

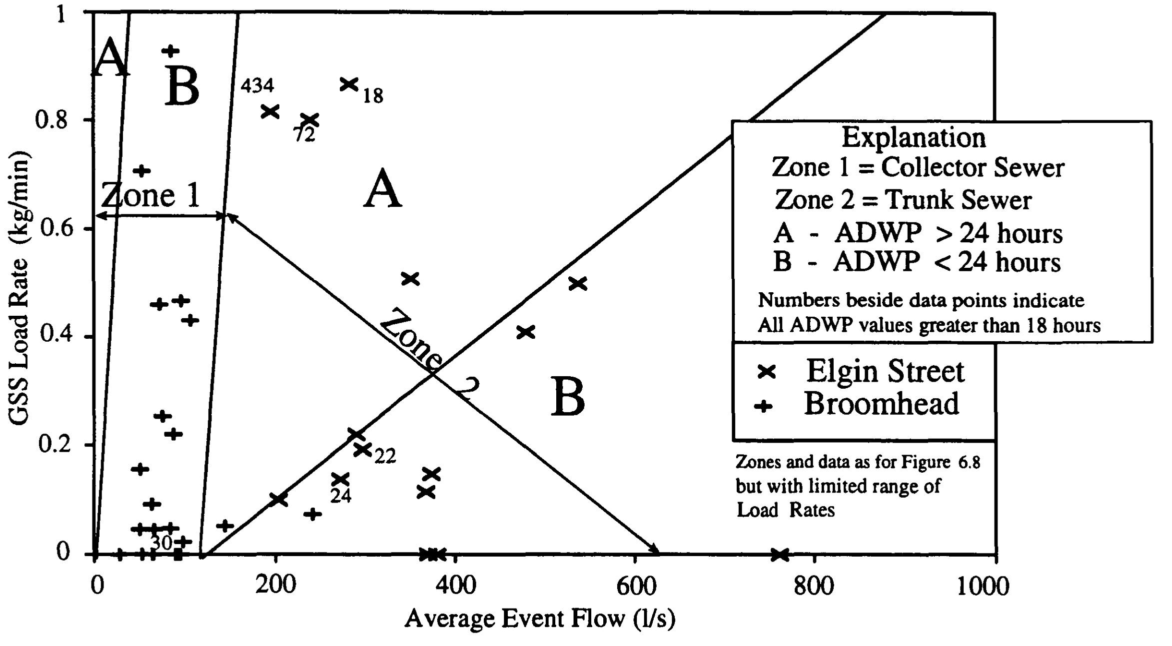


Figure 6.9 Chart for Comparison of Sites by GSS Load Rate - Low Data Range

This rationale has long been known in United Kingdom sewerage engineering practice where traditional design methods call for self-cleansing velocity for separate sanitary sewers to be based upon 2 x DWF or approximately daily peak flow. Other studies have also noted the effect, Crabtree et al (1991) suggested that some 15% of the dry weather solids may potentially exist as deposits at any one time, and the author (Jefferies et al 1990) reported on a study which suggested that diurnal deposition and erosion of sediments could be deduced from observations of bacterial changes.

6.5.3 A method for Differentiating Catchment Behaviour based on GSS Load Rate.

The deposition of sediments within sewer systems has been related to the nature both of the sewer system and catchment (Ashley et al 1992a). It has been suggested that smaller collector sewers require storm inputs to clear the sediment deposits which occur during dry weather, whereas trunk sewers tend to have steeper gradients with less deposition. It has been implied from many studies (Fletcher et al 1982, Hogland et al 1984, Pearson et al 1986, Aalderink et al 1990) that first foul flush effects, when they occur are derived from these deposits which have been shown to have extreme pollutant potential (Ashley, Wotherspoon et al 1992). Much current research is being directed to studies of their deposition and entrainment (Crabtree et al 1991). In the systems monitored in this study, comprising mainly collectors, a predominance of type B flushes were observed implying the existence of deposits and it was likely that the material accumulated at slack gradients, pipe junctions and at the overflow structures.

Of the five sediment types proposed by Crabtree (1989b) the most mobile deposits, type C, were considered to be the most likely to be entrained during storm events. No in-situ studies of the material sampled by the Gross Solids Sampler have been reported (O'Sullivan 1990) and direct evidence is lacking on the locations of deposition of gross and visible solids, however, it is presumed that they are associated with the type C deposits as illustrated in Plate 2.1. This contention is supported by the results of the dry weather flow study (Section 6.3.3) which implied that the tractive force required to move the visible solids was very small in line with that required for Type C material (Ashley et al 1992a). These authors have hypothesised that a weak surficial layer overlies a stronger layer of consolidating sediments and it is believed that the cumulative evidence presented shows that both visible solids and Type C material are subject to the same hydraulic influences and respond to flow in a similar fashion.

Figures 6.8 & 6.9 have been divided into two zones reflecting the sites, each with two further subzones divided by ADWP = 24h. Zone 1 includes all data from Broomhead apart from two small events while all data from Elgin Street falls into zone 2. Each zone has been subdivided into sections A and B using ADWP as interpreted above. One event at Broomhead does lie in zone 1B rather than 1A, however, the continuation pipe at the overflow was partly blocked at the time, allowing extra settlement of solids within the overflow chamber, causing this data point to be unreliable. The same rule was applied to the Elgin Street data and, apart from two borderline events, zone 2 was divided satisfactorily.

The division of Figure 6.9 into zones was tested for randomness using Fisher's exact test for a 2x2 table (Siegel 1956). It was found that there was less than a 1% chance of zone 1 (Broomhead) being differentiated from zone 2 (Elgin Street) on the basis of random chance. Following the subdivision by site, the rule for Broomhead was formulated that a dividing line could be drawn on the basis of the 24h ADWP. When the Elgin Street data were tested on the same basis it was found to be significant at the 5% level. Thus the statistics were found to support the hypothesis that a subdivision of the data points on the basis of 24h ADWP was justified.

It was concluded from Figure 6.3 that very low numbers of gross solids were to be observed at times of low flowrates, and the difference between night and day of gross solids production rates confirms that night time deposition was occurring and that gross solids were not reaching the sampling point. Ashley et al (1992b) have suggested that, depending on the particular sewer system, many solids are eroded during the daytime peak and the evidence of Figure 6.3 is that this effect is magnified when gross solids are considered. Thus it is clear that the 24h dry weather period is significant. It is further clear that, where deposition of visible solids occurs, then the greatest build up will be during the night prior to the morning peak, at which time the reservoir of gross solids available for erosion would be greatest.

The corollary of this argument is that, should an event occur during or shortly after the diurnal peak, the reservoir of gross solids would have been depleted and the event concentrations would be unexceptional or lower than would be the case had the event occurred before the diurnal peak. An analysis of the data in Figure 6.9 to include the time of day of the start of the events did not support this view since it produced almost random scatter. It is likely that other factors such as ADWP itself and the volume discharged masked the effect of the time of day, however a gross solids load rate of any given magnitude was just as likely to occur at night time as during the day, thus providing no evidence to disprove this hypothesis.

The zones of Figure 6.9 reflect the different characteristics of the sewer systems at the sites where the GSS was installed. Hydraulic backing up caused by the weir at Broomhead produced low inflow velocities even at maximum flowrates. The average of the maximum inflow velocities for all GSS events at Broomhead was 0.26m/s, while for Elgin Street the comparable figure was 0.67m/s. The Broomhead catchment was small and sediment was regularly deposited, whereas at Elgin Street the area was medium sized and the contributing sewer steep. It is suggested that the former installation was on a collector sewer (as proposed by Ashley & Crabtree 1992) whereas the latter was on a trunk. Notwithstanding these classifications, there was clear evidence of diurnal movement of sediment at both sites and, by inference, deposition must have occurred, although at different rates.

A discussion of the application of these results is included in section 6.6.3.

6.5.4 A Possible Gross Solids Load Rate Relationship

Further analysis of the performance of the overflows was carried out on the basis that their performance might be reflected in the behaviour of the GSS Rate. The justification of this concept is that although there may be little separation of TSS at the overflows (Walsh & Jefferies 1992), there may be preferential removal of Gross Solids. A significant number of overflow events produced no measurable mass in the COPA sack resulting in zero values for the ratios. Such values are meaningless and reflect the inadequacy of the GSS sampling method.

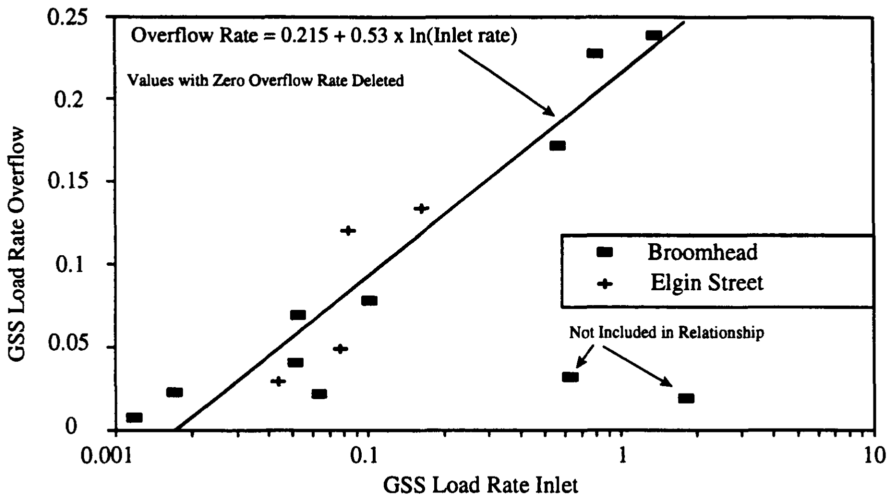


Figure 6.10 Separation at Broomhead & Elgin Street Overflows Using GSS/TSS Rates

The GSS Load Rates for the two overflows at each site are shown against the rate for the inflow without the zero values in Figure 6.10. No differentiation may be detected between the two sites and equation 6.9 results from a regression analysis of the data. Two data points had abnormally high inlet load rates and were excluded from this analysis.

Overflow GSS Rate = 0.215 + 0.530 x ln(Inflow GSS Rate) **6.9**

Equation 6.9 has $r^2 = 0.894$ and it is proposed that it may be used as a characteristic equation by which the performance of the Elgin Street and Broomhead overflows may be compared with other sites. Limited and inconclusive evidence that separation did occur at the overflows is provided by Equation 6.9. In spite of these criticisms, Equation 6.9 is included for comparison with future studies where a different relationship may apply.

6.5.5 The Relationship Between Gross and Suspended Solids

In an attempt to develop better methods of prediction of Gross Solids quantities, a further analysis of the GSS and TSS information was carried out. The justification for carrying out this analysis was the assumption that when an event had an increased TSS load, it would also have a greater GSS load. In view of the inability of the Gross Solids Sampler to produce time dependent data, event concentrations for GSS against TSS for both sites were calculated and are presented in Figure 6.11.

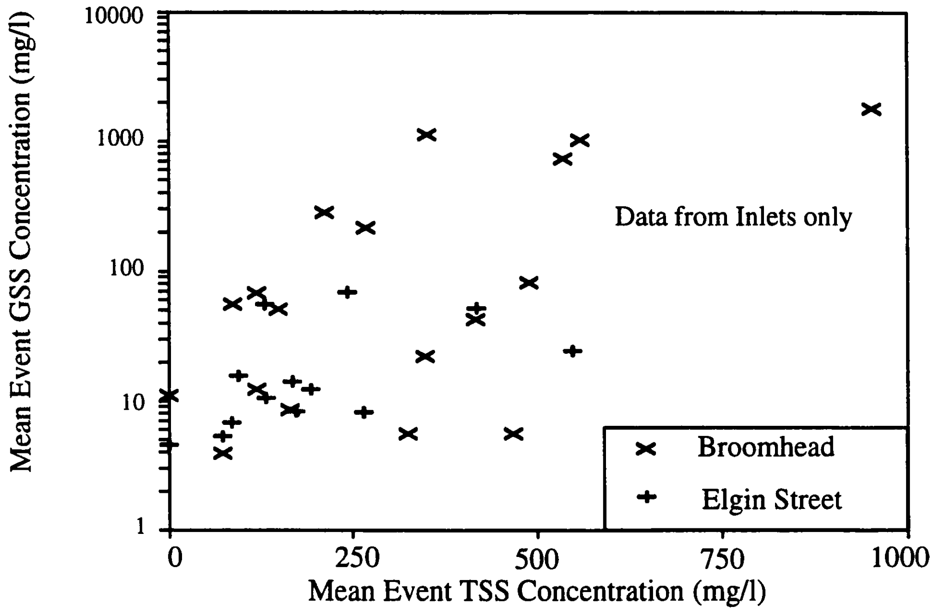


Figure 6.11 Gross Solids Sampler Results
Comparison of GSS and TSS Event Mean Concentrations

A presentation of the Broomhead data has been given by the author previously (Jefferies 1992) and a predictive equation for GSS based on TSS mean concentrations proposed. Some correlation would appear to exist and the Elgin Street data is probably from the same data population, however the scatter is extreme and no statistical analysis was carried out. No zones, of the form of figures 6.8 and 6.9, could be drawn for figure 6.11.

6.6 CONCLUSIONS

The results from the Gross Solids Sampler sampling programme at two CSO sites have been analysed in chapter 6. This analysis had two successful outcomes and a number which were less positive. A valid relationship (equation 6.4) for the prediction of visible solids in dry weather flow has been developed. In addition, the analysis of wet weather event data has produced figures 6.8 & 6.9. These figures form a novel chart which enables a differentiation of catchment types to be made on the basis of the event GSS load rate.

In contrast, a principal aim of the GSS evaluation could not be achieved. The data obtained could not be interpreted as showing whether gross solids separation occurred between the inlets and overflows at either CSO studied. Also, no relationship could be found which might allow the prediction of gross solids loads or rates during CSO discharge.

The outcomes of the GSS sampling are reviewed in this section, particularly the use of figures 6.8 & 6.9.

6.6.1 Gross Solids in Dry Weather Flow

A relationship has been presented which enables the prediction of numbers of visible solids likely to be present in dry weather flow. This equation (6.4) may be used in conjunction with any sewage quality model such as MOSQITO (Moys 1987) which predicts the total mass of suspended solids discharged in a dry weather period. It is believed that equation 6.4 is an original contribution to the knowledge of the movement of sewer solids. It was further found that the numbers of solids at the observation point during dry weather flow were small and dropped almost to zero at night time. This information has been interpreted as confirming previous work (Ashley et al 1992) which suggested that deposition of gross solids occurs principally during the night. It also has significance for the interpretation presented in section 6.6.3. of figures 6.8 & 6.9.

6.6.2 Gross Solids in combined sewer overflow events

The GSS results were notable for the small amounts of solids retained from the overflow intake. The discussions and analysis presented in section 6.4 showed that the separation performance of the CSOs, or lack of it, could not be explained, particularly when the relative flows at inlet and overflow were taken into account. No satisfactory explanation could be found for the inconsistent results which may have been due to;

i) Lack of gross solids at the overflow weirs - This was unlikely to have been the case, since the Trash Traps collected measurable amounts during all overflow events,

ii) Poor sampling of the flow by the GSS intake,

iii) Small volumes of overflow leading to settlement of the solids. A number of events did have low volumes, however, the volumes were large for a similar number.

None of the above arguments are convincing and the study left unresolved doubts over the ability of the GSS to sample from the overflows. It is consequently concluded that the overflow monitoring at the CSOs was unsuccessful. However, further operation of the GSS is recommended in section 6.6.4 due to the failure to account for its lack of success.

A further disappointment of the study was the failure to develop a relationship for the numbers of gross solids in wet weather flows similar to equation 6.4 which applies to dry weather flows.

6.6.3 The Gross Solids Rate Chart

It is contended that a real contribution to the knowledge of the behaviour of sewage solids has been made with the development of the Gross Solids Rate Chart (Figures 6.8 & 6.9), together with the inferences which have been drawn from it. The chart has been derived from data gathered at the inlets to each of the CSO sites, and the method of presentation has been chosen so that only flow measurements are required in addition to the GSS observations. The chart shows that the two sites may be differentiated clearly on the basis of their rate of gross solids production during high flow events. The data making up the chart have apparently wide scatter, however, it has been shown that the zones drawn have less than a 1% chance of random occurrence. This demonstrates that a clear difference of behaviour between the sites was observed.

It is suggested that zones 1 & 2 may be representative of broader categories of sewer systems with zone 1 (Broomhead) representing a collector and zone 2 (Elgin Street) a trunk sewer. Further data must be collected from other catchments representing a range of solids production and deposition conditions to support this suggestion. Whether or not such categories of sewer systems are justifiable, there remains a significant difference between zones 1 and 2, and this is likely to be due to the depositional characteristics of the sites and their contributing catchments.

The chart does not itself present a method of prediction of gross solids rates, the data scatter being too wide. It is suggested that further data from other sites should at least lie within the same zones and, should this be the case, then further data may permit forms of relationships to be developed. The goal would be a predictive tool which will enable the rate of gross solids production to be estimated from a consideration of the catchment type.

A further contribution to knowledge is that the Gross Solids Rates during wet weather events at both sites were clearly differentiated on the basis of a 24 hour ADWP. Greater rates were observed for each site when the ADWP was greater than 24 hours than for periods less than this value. The corollary of this is that the rate did not increase with ADWP values greater than 24 hours. The implication of this result is that the gross solids accumulated within a 24 hour period but not significantly thereafter. It is presumed that deposits of gross solids which accumulate will remain restricted in size by the diurnal variation in flowrate. Thus the apparent dichotomy may be explained that, although at night time the rate of gross solids production is small, it is also at night that deposition occurs due to the low flows which prevail.

A final result of note is the inference that gross solids have been found to be subject to the same hydraulic influences as the highly mobile Class C sediments (Crabtree 1989b) and they respond to flow in a similar fashion. The two types must however be differentiated due to their different pollution potential. Consequently it is contended that gross solids must be included as an additional sediment class which would be deposited and eroded along with type C sediment but are differentiable due to the different pollution effects.

6.6.4 Further Work for the Gross Solids Sampler

Many of the conclusions above suffer from the limitation of the GSS having been installed at only two sites. The following are recommended for further work;

i) Installation of the GSS at two further CSO structures to establish whether or not the lack of CSO performance data was due to local factors inherent in this study.

ii) Instalaltion of the GSS at one each of a catchment where deposition is a problem and where it is not. This is required to refine the Gross Solids Rate Chart. The high deposition catchment should correspond to zone 1, and that with low deposition to zone 2.

iii) With further information available from these studies, information may be available to enable the development of a gross solids prediction equation for wet-weather flows.

CHAPTER 7 RESULTS OF THE SMALL-BORE SAMPLING PROGRAMME

What is Written without effort is in general read without pleasure
Samuel Johnson   Miscellanies

7.1 INTRODUCTION

Small-bore samplers are the most widely used equipment for obtaining sewage quality data. They may be installed in a variety of locations and have a range of applications due to their small size and automatic operation. Questions may be raised over the volume of sample taken and the location of the intake within the flow. However, the research presented in chapter 7 relies on the the chief advantage of the small-bore sampler, knowledge of the time at which samples were taken. Pollutant concentrations may thus be related to the flow, making this the only source of quality data from which the efficiency determinands of chapter 2 may be properly evaluated.

In chapters 5 and 6, two alternative samplers, Trash Traps and the Gross Solids Sampler, were considered. The volumetric problems of the small-bore samplers were illustrated using a comparison with the GSS in section 6.2.2. The Trash Trap avoided sampling location problems, particularly at low overflow rates, but this equipment could not be installed on two of the spill weirs due to site difficulties. These points illustrate the problems of the various devices, none of which could be considered to be without faults.

Chapter 7 contains an interpretation of all results obtained from the small-bore samplers. It should be recalled that, due to resource restrictions, reliance was placed in this study, on measurement of TSS concentrations and that a more complete data collection programme might have included analysis for BOD, COD and ammonia levels. An analysis of the changes across the overflows, where there was little storage, is presented in section 7.2. Simultaneous overflow concentrations are compared with those at the inlets. This approach suffers from the criticism that no account is taken of the flowrate nor of the retention time. However, it is considered that the results are of merit as they permit a direct comparison to be made of the three overflows.

The methods of determination of overflow efficiency detailed in Chapter 2 are applied at each installation to compute efficiencies. The methods have been applied to the overflows on their own, and the combinations of overflow and tank, where present. It is claimed that a contribution to knowledge of the operation of combined sewer overflows including storage is made in sections 2.3 and 2.4. Total Efficiency and Flow Split, measures of the retention of pollutants and flow within sewer systems are determined in section 7.3. The efficiency of an overflow structure to separate solids throughout a complete storm, as expressed by the Treatment Factor, considered in section 7.4. Also determined for each installation in section 7.4 is Pollution Separation Efficiency which relates only to the period when the structure is discharging. These concepts were established during the course of the study and their application to three installations in one study had not previously been made.

While the performance of the structure is critical in a determination of the most effective arrangement, the volume, concentration and total loads discharged are essential for the monitoring of CSO events. Loads and concentrations discharged, also necessary for the determination of stream impacts, are presented in section 7.5.

It is concluded that the amount of effort required in monitoring fully a CSO installation is prodigious and would be unlikely to be carried out on a routine basis. It is further shown that most appropriate measure for assessment of installation performance is Flow Split

7.2 A COMPARISON OF POLLUTANT CONCENTRATIONS

It is frequently wise in studying the performance of a system to follow a relatively simple method of analysis initially to determine whether any broad indication of behaviour may be revealed. A simplified approach is rarely the best, rather, additional measurements necessary in a rigorous examination are avoided thereby limiting the number of variables which may be in error. For this reason it was considered desirable to attempt to compare concurrent concentrations of the inlet, overflow and spill flows at each installation. A comparison of this nature completely ignores factors such as variations of flowrate and the retention time of the storage. The retention time was very small in all of the installations studied, improving the validity of this analysis. In the case of the Elgin Street and Broomhead overflows it was negligible, and approximately fifteen minutes at overflow rates typical for Lochgelly.

The comparison, included as Figure 7.1 shows both the variability of all data and the probability that an averaged relationship does exist for each site, the relevant statistics being included in Table 7.1. Best fit linear regression lines have been plotted in Figure 7.1 and the data for Broomhead and Elgin Street are almost indistinguishable. Data scatter is extreme and the r^2 value for the latter is 0.640 negative, rendering any prediction invalid. The scatter of the data and the slope of the regression lines are such that the conclusion must be drawn that no change of quality can be observed across these overflows. The data for Broomhead show much less scatter and the r^2 value of 0.69 does allow some confidence to be attached to the best fit line for these data. However the conclusion from the data derived from both sites is that there was in general no change of suspended solids concentrations across either overflows.

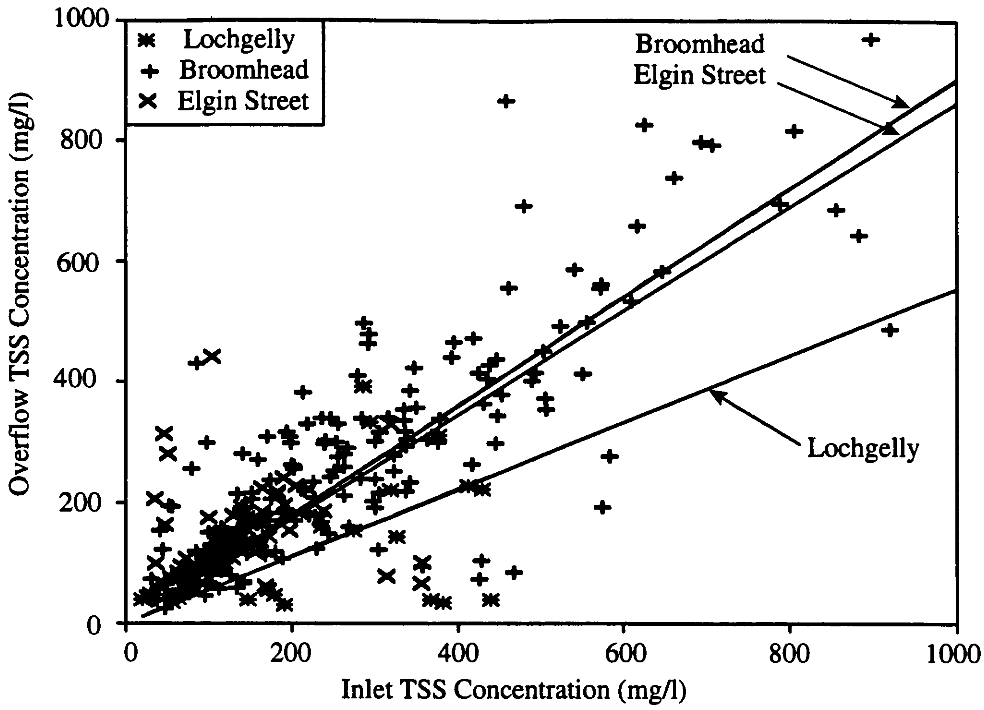


Figure 7.1 Comparison of TSS Concentrations Across Three Overflows

Scatter for the Lochgelly data is less than at Elgin Street, although the r^2 value of 0.124 is still extremely low. The slope of the best fit line at 0.556 does suggest improvement in quality across the device.

Site	Number of Samples	Mean Concentrations (mg/l)		Best fit Line	
		Inlet	Over/Spill	r^2	Slope
Overflows					
Elgin St.	61	135.1	139.5	-0.64	0.863
Broomhead	197	301.2	291.1	0.69	0.900
Lochgelly	34	189.6	122.6	0.12	0.556
Spills					
Elgin St.	5	138.2	165.8	-	-
Broomhead	28	207.2	179.9	0.69	0.835

Data Relate to Figure 7.1

Table 7.1 Comparison of Inflow and Overflow/Spill Concentrations

The t Distribution test, with the line constrained to pass through the origin, indicated that there was less than a 1% chance that the data population actually had a slope of unity, the true slope being less than one. This statistic suggests that there actually was an improvement in quality between the inlet and spill at Lochgelly.

The spill data from Elgin Street and Broomhead (also included in Table 7.1) merely serve to illustrate the limitations of this form of simple comparison. The mean concentration for the 197 inlet samples at Broomhead when overflow was concurrently sampled was 301mg/l, while that for the 28 values in the spill set was 207mg/l. This difference reflects principally the reductions of concentration during prolonged events which were necessary to cause spill. Any changes in quality across the overflows which are suggested by this analysis are thus masked by the variation of the inlet quality. It is also notable that the average spill concentration during the one event sampled at Elgin Street was significantly higher than at the inlet. This was due to all but one of the concurrent spill samples having higher concentrations than at the inlet. This is a reflection both of the event and catchment response and of the deficiency of concurrent sample analysis.

In summary this analysis suggests no quality change occurred across the two standard overflows, while an improvement did occur at Lochgelly.

7.3 RETENTION OF POLLUTANTS WITHIN THE SEWER SYSTEMS

In chapter 2 definitions were given for the various efficiencies which apply to CSO installations. The terms Flow Split and Total Efficiency, are used to describe the volumetric and pollution performance of an overall installation. Implicit in their derivation are the factors which contribute to the discharge or retention of polluted flows within the sewer system. These terms have been derived for individual events at all sites in this study, and for all events together. It was found that Flow Split was the only performance criterion which could be compared with data from other studies.

7.3.1 Volumetric Performance

Comparison of performance between the different sites has been carried out using parameters which are, as far as possible, common between locations. Flows and suspended solids concentrations present few problems, however, true comparisons are complicated by the multitude of site factors including flow rates, concentrations and volumes, overflow setting and the storage installed, in addition to any pollutant separation which may have occurred.

The ability of an overflow with any associated storage to retain flow within the sewer system is expressed by the Flow Split (Equation 2.1) which has been determined using the event definitions outlined in section 4.3. Total Efficiency (Equation 2.5) is generally recognised as being a more relevant measure of efficiency than flow split since it considers pollutant retention in addition to flow. However, the values of Treatment Factor derived in section 7.4.2, and the limited numbers of events where loads could be computed makes a consideration of Flow Split highly relevant. Both measures have been determined, separately where appropriate, for overflow and full installations including storage, the spill discharge being used to monitor the overall installation efficiency.

Basic data on the operation of the three sites is included in Table 7.2. The Lochgelly and Broomhead installations were monitored for approximately one year including summer and winter periods. In contrast, monitoring at Elgin Street was shorter. Flow Split and Total Efficiency were determined for all events during which sufficient data were gathered and are expressed as percentages in Appendix D, Tables D1-D3.

Location & Period	Time Months	Inflow Events				Overflow Events			Spill Events		
		No. in Summer	No. in Winter	Total	Volume m^3	No.	Volume m^3	Flow Split	No.	Volume m^3	Flow Split
Lochgelly **26/4/89 - 5/12/89** **20/7/90 - 14/1/91**	7 6	16	17	33*	61,500	N/A	N/A	N/A	33*	20,984	65.9
Broomhead **2/4/90 - 9/4/91**	12	23	29	52	47,560	52	22,727	52.2	14	16,104	66.1
Elgin Street + **9/11/91 - 11/2/92**	3	0	16	16	101,716	9	13,384	86.8	1	9,174	91.0

* Only Events causing spill are listed in Appendix D

+ One event of approx 2 year return period occurred

Table 7.2 Volumetric Data for Monitored Events

7.3.2 Flow Split and Total Efficiency

Flow Split and Total Efficiency are plotted in Figures 7.2 and 7.3 against the total volume of flow entering the overflow during the event. In these figures, the term **overflow** refers to the flow separation device, and **installation** to the complete structure between inlet and spill to the watercourse. Inflow volume has been selected for the abscissa in Figure 7.2 as data were available for all events. Additionally, although not including quality parameters, the volume does reflect the total mass of pollutants discharged together with (implicitly) the greatest flowrate during the event.

Differences between the sites dominate the form of expression of the results used in Figures 7.2 and 7.3, principally since the catchments were of differing areas.

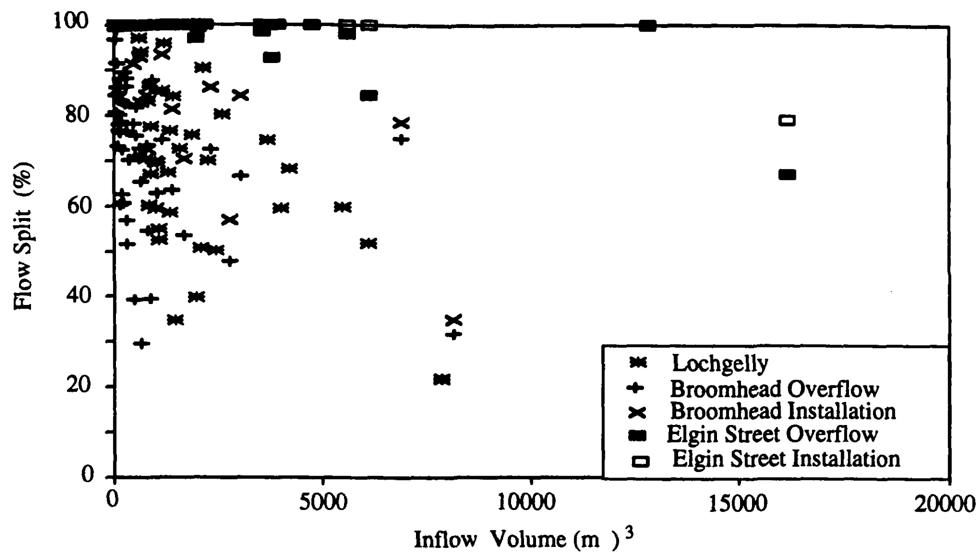


Figure 7.2 Variation of Flow Split with Inflow Volume

Flow Split and Total Efficiency were higher for the complete installations than for the overflows. This must be expected due to the additional volumes contained in the tanks. The relative ease of obtaining data for Flow Split is reflected in the greater number of points (107) in Figure 7.2, whereas there were only 40 data points for Total Efficiency in Figure 7.3, the smaller number caused by the need to carry out sampling in addition to obtaining flow data.

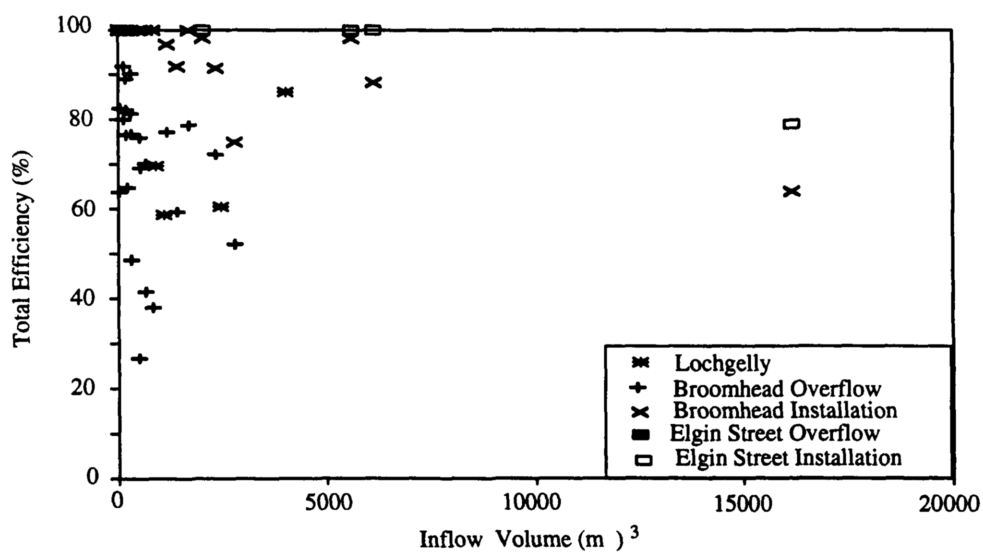
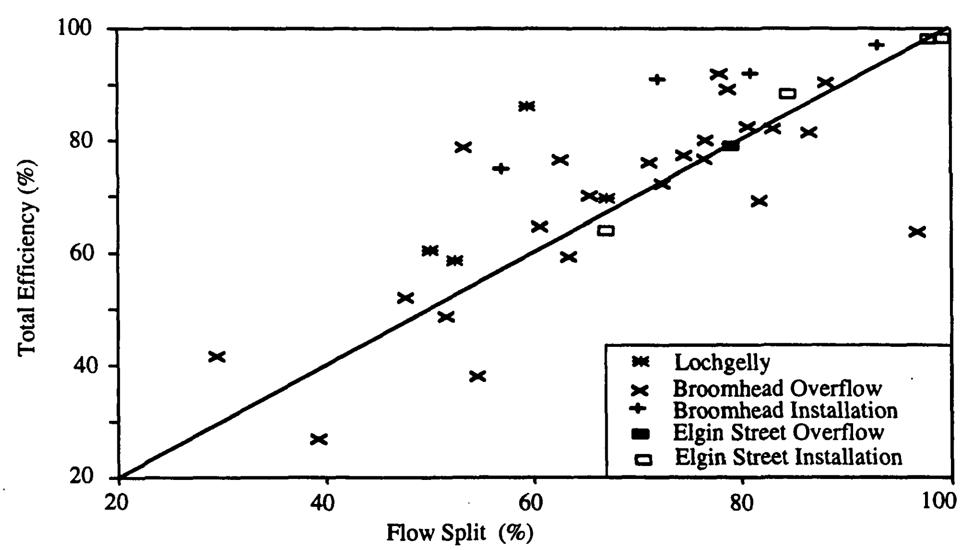


Figure 7.3 Variation of Total Efficiency with Inflow Volume

7.3.3 Comparison of Efficiencies

The range of values obtained for Flow Split and Total Efficiency is shown in Figure 7.4 in which the two measures are compared. In this diagram the 45° line represents the condition where all of the Total Efficiency is explained in terms of the flow separation. Points above the line represent events in which improvement in quality occurred, as measured by TSS, the reverse being the case with points below. Considerable scatter may be observed, and this is particularly true of the data from the Broomhead overflow. The Elgin Street data show very little scatter with all points close to the 45° line, suggesting that on average no quality change occurred and in effect only flow split occurred. In contrast, the Broomhead overflow data, showing significant scatter as they do, imply a very variable regime, some events being retained with a higher proportion of pollutant load than flow going to treatment, while others show a degradation of quality. Results are more consistent for the Lochgelly overflow and the Broomhead installation, in each case all points lie above the 45° line implying that improvement in quality normally occurs. These comments are entirely consistent with the conclusions reached for Treatment Factor discussed in section 7.4.2.



A value of 100% indicates that flow entered tank but was insufficient to cause spill

Figure 7.4 Comparison of Total Efficiency with Flow Split

7.3.4 Comparison With Previous Studies

Comparisons were made using the data presented in Table 7.2 with published information from other studies of combined sewer overflows which include storage. These data, which are included as Table 7.3, show a high degree of variability, and highlight the difficulties encountered in comparing results from different sites.

Comparison between sites is difficult except using the simplest of comparators. SDD (1977) used the dry weather flow and catchment area for comparison and these have been used by others. More recently Pisano (1990) and Tyack et al (1992), have suggested sewage grading curves should be used for design purposes. Most information available from the literature is in a summarised form similar to Table 7.3 and uses the parameters suggested by SDD (1977).

No	Location	Imp Area (ha)	Pop'n	AVE DWF (l/s)	Setting (l/s)	Setting x DWF	Storage Volume (m^3)
1	Lochgelly	24.9	4,800	14	110	7.9	113
2	Broomhead	50.6	3,800	8	62	7.8	400
3	Elgin St.	143	16900	58	410	7.1	2,500
4	Bucksburn[1]	13.5	6,440	19	88	4.6	366
5	Stoneywood[1]	9.1	4,970	13	77	5.9	282
6	Gt Harwood[2]	55.7	12,500	30	278	9.3	138
7	Tengen[3]	11.0	1,500	2	35	17.5	79
8	James Br.[4]	Data limited by site problems, 3 events only					
9	Stuttgart[5]	60.2			25		1,452
10	Rubgarten 1[5]	15.6			11		400
11	Rubgarten 2[5]	33.5			38		120
12	Ense 1[5]	30	2,110	5	13	2.5	654
13	Ense 2[5]	29	1,360	4	28	6.9	855
14	Matten[6]	32.6	4,300	17	90	5.3	250
15	Hilterfingen[6]	10.5	1,500	6	40	6.7	330

[1] SDD (1977)
[2] Saul, Thornton & Henderson (1985)
[3] Brombach et al (1992)
[4] Hedges et al (1992)
[5] Dohman et al (1986)
[6] Krejci et al (1986)

Table 7.3 (a) Comparison of CSO data from Different Studies - Basic Data

No	Location	Imp Area (ha)	Pop'n	Storage Volume (m³)	Flow Split (%)	Volume/ Imp. Area (m³/ha)	Volume/ Person (L/head)
1	Lochgelly	24.9	4,800	113	70.9	4.5	24
2	Broomhead	50.6	3,800	400	66.1	7.9	105
3	Elgin St.	143	16900	2,500	91.0	17.5	266
4	Bucksburn[1]	13.5	6,440	366	71.3	27.0	57
5	Stoneywood[1]	9.1	4,970	282	88.9	31.0	57
6	Gt Harwood[2]	55.7	12500	138	73.3	2.5	11
7	Tengen[3]	11.0	1,500	79	52.2	7.2	53
8	James Br.[4]	Data limited			73.8		
9	Stuttgart[5]	60.2		1,452	27	24.1	
10	Rubgarten 1[5]	15.6		400	75	25.6	
11	Rubgarten 2[5]	33.5		120	61	3.6	
12	Ense 1[5]	30	2,110	654	80	21.8	310
13	Ense 2[5]	29	1,360	855	83	29.5	629
14	Matten[6]	32.6	4,300	250	59.7	7.7	58
15	Hilterfingen[6]	10.5	1,500	330	85.5	31.4	10

Table 7.3 (b) Comparison of CSO data from Different Studies - Derived Data

The information in Table 7.3 originates from Germany and Switzerland in addition to the United Kingdom. The data have been presented in graphical format as Figure 7.5 to show the influence that volume and through-flow setting have on flow split.

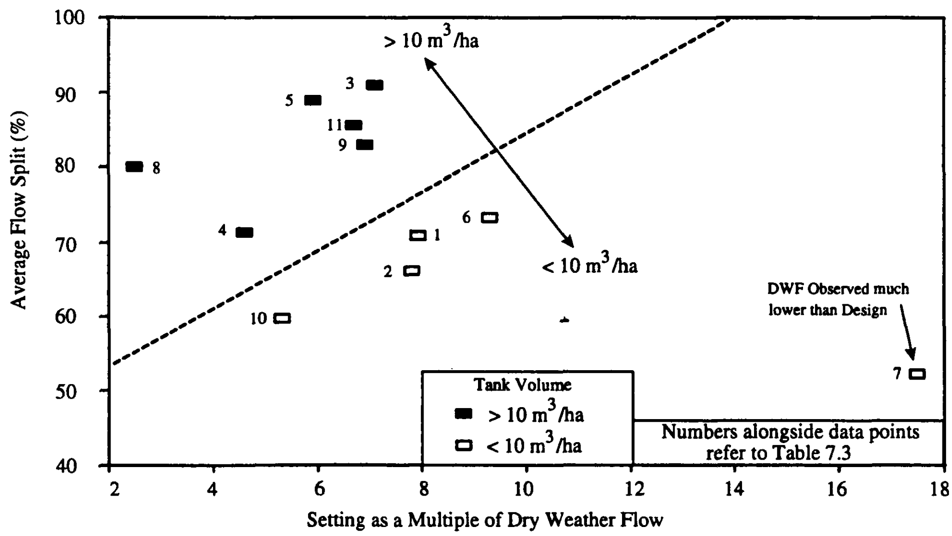


Figure 7.5 Comparison of Retention Tanks by Volume

Figure 7.5 has been drawn on a base of the continuation flow setting expressed as a multiple of dry weather flow. This suggests that bands may be drawn for tanks with small (Volume<10m^3 per impermeable hectare) and large (Volume>10m^3/ha) volumes. The bands were found to be distinct but to have no gradation within each band. The average flow split for the Elgin Street installation was found to be greater than 90%. The Broomhead and Lochgelly installations had very similar performances with each having an average flow split of approximately 70%. The Elgin Street installation incorporates approximately twice the storage volume as Broomhead when expressed as a ratio of the impermeable area as detailed in Table 7.3. It is believed that the higher flow split for the Elgin Street installation was due to the relatively larger volume installed there. These data show the long term behaviour of the overflow storage, however, they do not necessarily show a link with the detailed behaviour.

7.4 OVERALL EFFICIENCIES

It is necessary in this section to consider the operation of the overflow devices both excluding and including the tanks to determine whether one installation, as constructed, had a greater ability to separate pollutants than another. The event data which are summarised in Appendix D, Tables D1 to D3, have been used to calculate the Pollution Separation Efficiency, defined in section 2.5.6. Treatment Factor, for which values are listed in Tables D4 to D6, is defined in section 2.5.5. All relevant results are summarised in Table 7.4. It will also be recalled that Total Suspended Solids is the principal determinand by which these parameters have been compared in this study, particularly since the Gross Solids Sampler could not be used to monitor spill flows.

Location	Date	Overflow			Installation		
		FS	PSE	TF	FS	PSE	TF
Lochgelly	13/8/89	50			50	35	1.21
Lochgelly	18/9/90	67			67	49	0.93
Lochgelly	16/10/90	52			52	46	0.99
Lochgelly	28/10/90	66			66	79	1.40
Broomhead	28/10/90	75	76	1.00	93	54	1.03
Broomhead	22/12/90	72	75	1.06	86	80	1.05
Broomhead	4/1/91	63	59	0.92	81	85	1.08
Broomhead	18/3/91	48	52	1.09	57	70	1.32
Elgin Street	19/12/91	84	86	1.05			
Elgin Street	3/1/92 A	99	98	0.99			
Elgin Street	3/1/92 B		92	1.00			
Elgin Street	8/1/92	67	56	0.95	79	48	1.01
Averages							
Lochgelly		59	52	1.12	59	52	1.12
Broomhead		65	66	1.02	79	72	1.12
Elgin Street		63	56	1.00	79	48	1.01

FS = Flow Split (%) **TF** = Treatment Factor
PSE = Pollution Separation Efficiency (%)

Table 7.4 Treatment Factor and Pollution Separation Efficiency

7.4.1 Pollution Separation Efficiencies

Pollution Separation Efficiency (PSE) expresses the ability of a structure to retain pollutants within the sewer system while it is actually discharging, and is comparable with Total Efficiency, which expresses the same concept over the complete storm.

The values obtained for PSE are contradictory in showing some values higher for the overflows than for the full installations. It would be expected that, with the additional settlement provided by the off-line storage tanks, a higher percentage of pollutant would be separated by an overall installation (overflow and tank combined) than by the overflow alone. Table 7.4 shows that for the two relevant sites, Broomhead and Elgin Street, each had one event during which the pollution separation was higher for the overflow. At Elgin Street this was the only fully monitored event.

It is believed that this contradiction is explained by the variation of concentrations throughout the events. Although both tanks incorporated blind compartments to contain the first foul flush, mixing probably occurred within the remaining sections, thus retaining particulates in suspension. Spill occurred late in each event and well past the peaks of concentration. In consequence, the PSE values are based on the spillage of the relatively poorer quality streams together with influent flows which had reduced suspended solids concentrations.

The time lag of pollutant streams caused by the storage also affected spill quality calculations. Retention times of 13.9 and 12.0 hours of DWF (Table 7.3) in the storage at Broomhead and Elgin Street respectively were large, and consequently the duration of spill was relatively short except during prolonged events. In contrast, the retention time for the Lochgelly installation was small at 2.3 hours of DWF. In comparison, and as would be expected, the retention time during storm flows was relatively small, although the ratios of the retention times between foul and storm conditions was little changed. The average peak inflow for all events was determined for the Broomhead and Lochgelly sites and retention times of 90 and 15 min at the average peak inflow rates respectively were determined.

It is concluded from this assessment that:

i) The off-line tanks retained pollutants within the sewer system principally by virtue of the storage incorporated in the installations;

ii) Qualities of the spill discharges were on average the same as concurrent inlet qualities, and;

iii) Pollution separation efficiency is not a valid means of comparing overflows in which storage is an integral component.

7.4.2 Treatment Factor

Treatment Factor (TF) represents more accurately than Pollution Separation Efficiency the ability of the installations to remove pollutants. By comparing the percentage of the input load retained within the system with that of the volume retained at the same time, the criticisms of the use of PSE are avoided. The author considers that TF is the field variable closest to that determined during tests on models. Contrary arguments exist to the effect that Pollution Separation Efficiency should represent steady state laboratory conditions better. The criticism against PSE has been discussed in 7.4.1. Treatment Factor is determined over the range of flows from zero up to the event maximum, however, the full range of concentrations are also incorporated. Since, as was demonstrated in section 5.4.1, most events at all of the sites exhibit type B flushes, any parameter which includes the most extreme concentrations must be seen as being advantageous.

Values for TF shown in Table 7.4 are within 5% of unity for six of the nine events captured at all sites. The data are too limited to carry out a statistical analysis, but unweighted averages of Treatment Factor support the view that a small amount of improvement in quality does in fact occur across the overflows.

Comparison of the values for Treatment Factor in Table 7.4 shows that the installations studied compare with the minima of the curves given in Figures F.4 & F.5 in Appendix F. Such minima, applying as they do to neutrally buoyant or slowly falling particles, are governed by the q/Q split used in model tests. With event-average flow split values ranging from 48% to 99% when applied to the overflows and 50% to 79% for the installations it is clear that, except for short durations during maximum flows, the observed split values did not approach the values used in model tests. Figure F.5 (c) is included to demonstrate the considerable effect that flow split values have on efficiencies. With higher flow split values observed in practice it is reasonable to presume that conditions within the installations monitored were such that the separation efficiencies predicted by the model tests should have been bettered. The results show that they were not. The Treatment Factor results also suggest that no change of quality across the overflows occurred, and only a very marginal improvement across the full installations. The observed and model results are sufficiently different to show that the model results cannot be valid.

The data in Table 7.4 were compared with those from the hydrodynamic separator at James Bridge in which three events were monitored with TF = 1.2, 1.0, 1.03 (Hedges et al 1992). Very similar results to the present study were obtained in spite of the James Bridge installation being hydraulically over-designed for the site. It is surprising that higher values of TF were not obtained in view of the low loading rates observed. Hedges offers the comment that, as was the case in the study described here, no treatment of the spill flow took place at James Bridge.

The data from the present study suggest that the treatment provided at Lochgelly is of the same order as that at Broomhead even though the storage at the latter site was significantly larger (see Table 7.3). Data for the Elgin Street overflow show similar characteristics to those at Broomhead, however, the single overflow event observed, although giving consistent results, was insufficient to allow any conclusions concerning Treatment Factor at that site to be drawn.

It is concluded from the values of Treatment Factor (TF) obtained that;

i) The average value of TF, based on suspended solids measurements at Lochgelly and Broomhead was 1.12 indicating an improvement of quality of some 12%. The minimum TF obtained was 0.92, and the maximum, 1.40;

ii) The limited data for the Elgin Street site showed that there was no comparable improvement;

iii) There was more variability in the Lochgelly data;

iv) There was no change in quality at the overflows at Broomhead and Elgin Street, and;

v) The evidence from all sites suggested that for larger events, there was a tendency for higher values of Treatment Factor.

7.5 SPILL CONCENTRATIONS AND LOADS

Discharge monitoring of CSOs relies on the measurement of both concentrations and loads spilled to the watercourse. Table 7.5 gives maximum, minimum and average values of pollutant determinands together with loadings for those events monitored.

Site	Date	TSS (mg/l)			COD (mg/l)			BOD (mg/l)			TSS Load (kg)
		Max	Min	Ave	Max	Min	Ave	Max	Min	Ave	
L	28/6/89	250	100	127							26
L	13/8/89	253	67	129							158
L	18/9/90	402	122	274							26
L	30/9/90	435	163	327							82
L	15/10/90	128	28	80							98
L	16/10/90	307	154	261							18
L	28/10/90	253	13	83							137
L	16/11/90	446	290	362							134
L	22/12/90	88	21	48							73
L	1/1/91	351	64	241							45
L	1-2/1/91	320	54	133							251
L	4/1/91	375	47	175							300
L	4-5/1/91	98	27	44							321
		Av		**151**							
B	28/10/90	107	28	55	580	280					4
B	22/12/90	98	33	50	180	90					16
B	4-5/1/91	202	27	98							26
B	18/3/91	590	42	293				34	31		353
		Av		**213**							
E	8/1/92	289	110	135							469

L = Lochgelly B = Broomhead E = Elgin Street

Table 7.5 Spill Event Concentrations

Sufficient data were available only for the Lochgelly site to provide meaningful statistics relating to the distribution of spill events. The data from the thirteen spill events at Lochgelly have been replotted in dimensional form as Figure 7.6.

The average event mean concentration was found to be 181mg/l TSS and the 95%ile value was 388mg/l.

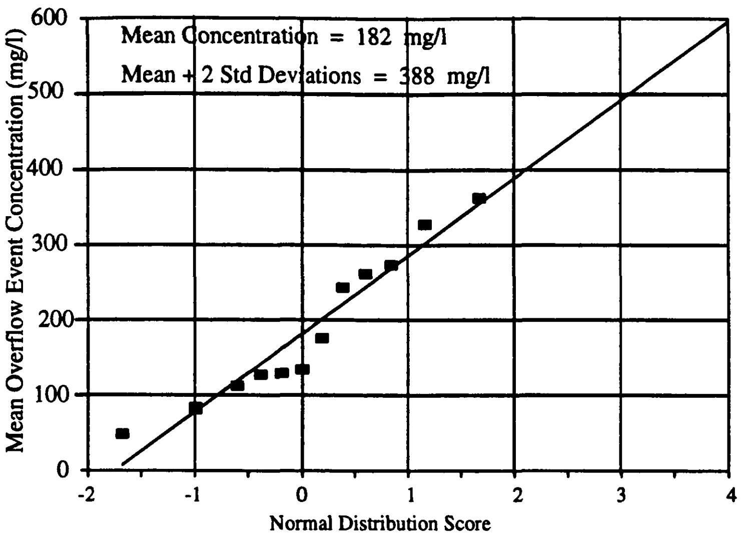


Figure 7.6 Normal Distribution of Overflow Events Lochgelly

This exercise was repeated for inflow event loads utilising the data from Figure 5.10b. The average event load was found to be 128kg TSS, and the 95%ile exceedance load was 372kg.

A knowledge of the distribution of the spill event loads would be necessary if a statistical approach to the consenting of CSO discharges were to be contemplated. Such an approach is well known and is widely applied to treatment works discharges which of course are not intermittent. The value of 95%ile presented above would require to be linked to an appraisal of stream conditions to have any real value. Little use can be made of this information in the context of the present study, however it is pertinent to reflect on the effort required to derive this single value for a single determinand, TSS. The work required for the determination of the appropriate statistics would require to be replicated at every site.

7.6 CONCLUSIONS

The results from the small-bore sampling programme have been presented and analysed in chapter 7. In contrast with the other sampling methods presented in this study, the small-bore results permitted differentiation between inlet and overflow sampling locations. Data from other studies have been reviewed and were found to be consistent with the results from this study. Tables were presented showing the values obtained for Flow Split and Treatment Factor. The information in these tables, together with the associated discussion, are considered to represent new knowledge concerning the performance of combined sewer overflow installations which incorporate storage.

7.6.1 The Resources Required to Obtain Pollutant Performance Data

An over-riding conclusion from the study was that, despite three years of sampling at the study sites, the amount of data obtained were small. This was particularly the case with spill data. Table 7.2 shows that some 48 events which caused spill were monitored at the three sites. When the data were analysed and summarised, sufficient flow and quality data to permit a full analysis were found to have been obtained from only nine events.

Practical problems caused this apparently very poor success rate. All equipment had to be installed and operating for full performance evaluation, and laboratory personnel required to be on hand, frequently at inconvenient times. It has been concluded that the resources required for routine quality performance monitoring are likely to be greater than could be envisaged during routine monitoring programmes.

Sufficient spill quality data were obtained from one site, Lochgelly, to carry out a statistical analysis of the spill Event Mean Concentrations. It was found that the data followed the normal distribution. This result, expressed as

Figure 7.6 required thirteen months of fieldwork followed by three of data analysis. A study of similar duration would be required at any CSO to provide a statistical basis for permission to discharge. The effort required is considered to be too great to be carried out on a routine basis.

7.6.2 Pollutant Separation

Pollutant separation at installations is expressed by the Treatment Factor (TF). The values presented in section 7.4 are believed to be an advancement in knowledge of the operation of CSO installations. The values are summarised in Table 7.4 and show a range for each installation on an event-by-event basis. Pollution Separation Efficiency values are included, but as they were found to represent conditions for a small part of each event, are not considered to be of value.

Pollutant separation at the three overflow structures was also investigated by examining concurrent inlet and overflow TSS values. This analysis showed that, at the two CSOs where storage was negligible, there was no change of pollutant concentration. At the remaining CSO, Lochgelly, the overflow concentration averaged 56% of that at the inlet over a range of events and flow conditions. The data showed wide scatter and this result could not be shown to be statistically significant. The improvement in quality may have been due to the retention time which was 15 minutes at typical overflow rates, or, as claimed by the manufacturers, due to the treatment ability of the Storm King units installed. It is suggested that it was due to a combination of both reasons.

The Treatment Factor values obtained showed that only a marginal improvement of quality occurred across the full installations. It is suggested that this was because the larger rainfall events which were necessary to cause spill from the off-line tanks, were dominated by persistent high flows with moderate suspended solids concentrations. In contrast, the smaller events which caused spill at Lochgelly would in general have had higher concentrations, with better defined first foul flushes. Thus, at least partially, the values for Treatment Factor may be biased towards higher values for smaller installations. This problem requires investigation in further studies.

7.6.3 Retention of Pollutants

In 7.6.1 arguments against CSO quality monitoring were presented on the pragmatic basis that resources would be likely to preclude frequent studies. In 7.6.2 the low values and likely bias of Treatment Factor in favour of smaller installations were highlighted. These arguments suggest that there is little value in considering CSO monitoring for pollution performance, and that a simpler measure of the retention of pollutants within the sewer system is of more value.

From the discussion presented on Treatment Factor it was concluded that, although there was a paucity of events recorded, on average, the measured improvement in TSS quality was small across the installations studied. There was also no clear evidence that the larger installations produced higher Treatment Factor values as should have been the case with the larger retention times to allow settlement of particles. To conclude that a larger storage volume at a particular installation is of no value would be invalid, as smaller pollutant loads would be discharged. This raises doubts over the validity of Treatment Factor as a useful measure of installation performance.

The overall ability of an installation to retain pollutants is expressed by the term Total Efficiency. The presentation given in Figure 7.4 showed that the installations produced higher Total Efficiencies than Flow Splits, but the differences were small. It was also concluded in section 7.4.1 that the installations studied retained pollutant loads principally by virtue of the storage incorporated. Consequently it is concluded that Flow Split is the most appropriate term which may be used when measuring the performance of combined sewer overflow installations with storage.

CHAPTER 8 IMPLICATIONS FOR CSO DESIGN, AND FURTHER RESEARCH

An Engineer - one who can make for a penny what any fool can make for a pound.
Anon.

8.1 INTRODUCTION

The results from the research set out in this thesis have a number of benefits to practitioners concerned with CSO design and their improvement. The principal applications of the results are highlighted in this chapter, together with guidance as to how those results might be incorporated into current practice. Recommendations for further work, which will enhance both the applicability and research needs, are also proposed.

The implications of the results on current design practice are laid out in section 8.2, and values are proposed for the various performance indicators. The application of the two principal monitoring methods are detailed in sections 8.3 & 8.4, together with appropriate flowcharts for their use. The final section contains recommendations for further research work.

8.2 A SUMMARY OF THE PRACTICAL APPLICATIONS

The principal benefits of the research to those involved with CSO design and monitoring lie in the values of the performance indicators gained using flow monitors and small-bore samplers. Data from the Trash Traps produced a proposed methodology for CSO comparison and some valuable results, but further evaluation is needed before firm recommendations may be made.

It was found that off-line tanks retain pollutants within a sewer system by virtue of the continuation flow setting and storage installed rather than by the treatment provided within that storage. The off-line storage volumes and the flow settings of the installations studied are given in tables 7.3 (a) & (b). The average Treatment Factor, based on TSS, was 1.12 for the two sites in spite of the differences detailed in table 7.3 (b). Treatment Factor was found to be unity for the Elgin Street site. Detailed information on the values found is given in table 7.4. Events with higher total rainfall tended to produce higher Treatment Factors, although the effect was small. The values found confirm that, at best, only marginal improvement of TSS quality occurred, even with the storage volumes indicated.

The volumetric performance of a CSO installation is measured by Flow Split. Data were gathered from a total of 99 events, from which the volumetric performance was deduced. Peak inflow rates were such that instantaneous Flow Split values never dropped below 19% as shown in table 8.1. This table is included to give guidance to the actual values of flow split likely to be encountered. The instantaneous values relate to the peak flow rates during events. Naturally, these values will only apply for the duration of the peak flow and, if percentages of total event durations were determined, the values would be much reduced. The values given

	INSTANTANEOUS FLOW SPLIT				**EVENT-INTEGRATED Flow Split (%)**	
	Minimum Observed Flow Split (%)	**% of events where given Flow Split was exceded**				
		20%	**33%**	**50%**	**CSO**	**Installation**
Lochgelly	19	3	15	61	-	66
Broomhead	19	0	23	56	52	66
Elgin St.	35	0	0	13	87	91

CSO refers to the flow separation structure only.
Installation refers to the separation structure and storage combined.

Table 8.1 Observed Flow Split Information

might typically be used to select laboratory-based efficiency values if required in preference to the field values derived in this study. Results from the Trash Trap study were used to develop a relationship for the numbers of visible solids to be expected during storm flows at a CSO. This relationship is given in equation 5.1 and shows that the numbers of visible solids during storm flows might be expected to be approximately one fifth of those in the same TSS load in dry weather flow. The Trash Trap can be used for the development of performance indicators as outlined in the following section. It may also be used as a screen for the final removal of visible solids at CSO discharges.

8.3 CSO ASSESSMENT USING TRASH TRAPS

The Trash Trap was developed by the author and has been shown to have considerable merit as a means of assessing the performance of CSOs. Results are presented in figures 5.5 and 5.6 which may be used to assess the likely discharge of visible solids at a CSO. Figure 5.5 shows that the mass of visible solids discharged during an event may approach 1% of the mass of TSS discharged at a CSO where no solids separation occurs. In contrast, the visible solids may be as little as 0.1% for a CSO which separates the gross solids efficiently.

The flow chart in figure 8.1 shows the installations required and the interpretation of the Trash Trap data needed to enable CSOs to be compared. It was stated in chapter 5 that results from further installations will be required to confirm the zones in figures 5.5 and 5.6. On the assumption that the zones are indeed reliable, then a general assessment of relative performance may be made, using the logic of figure 8.1. Zone B indicates a more satisfactory performance, in terms of visible solids discharged from the CSO, than would be indicated by a value in zone A.

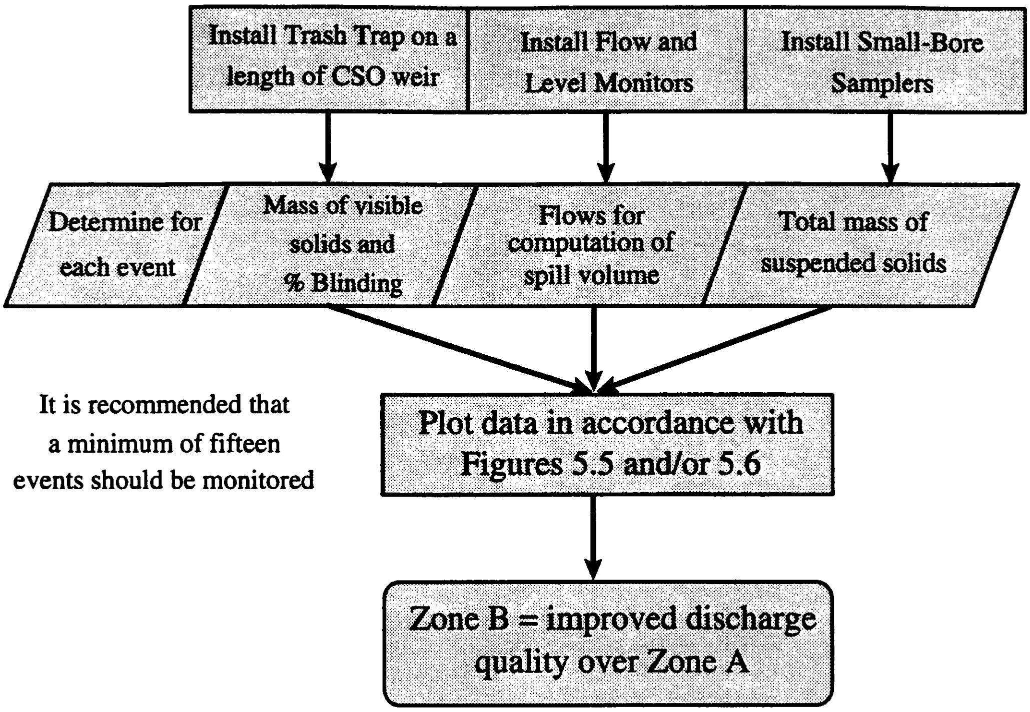


Figure 8.1 Flow Chart for CSO assessment using Trash Traps

8.4 INTERPRETATION OF GROSS SOLIDS SAMPLER RESULTS

The novel charts for the interpretation of the Gross Solids Sampler data which were developed by the author are included as figures 6.8 and 6.9. These figures allow the gross solids load rate during an event to be estimated. For a catchment where there is no significant deposition of solids, the rate of gross solids passing the observation point was found to be of the order of 1kg/min, and for a catchment where there was significant deposition, the rate rose to 8kg/min.

A GSS Load Rate chart, such as given in figures 6.8 and 6.9 may be developed using the flow chart included as figure 8.2. Figure 8.2 may also be used for estimation of gross solids Load Rates, although uncertainties remain due to the limited data used in its development. As with the Trash Trap data,

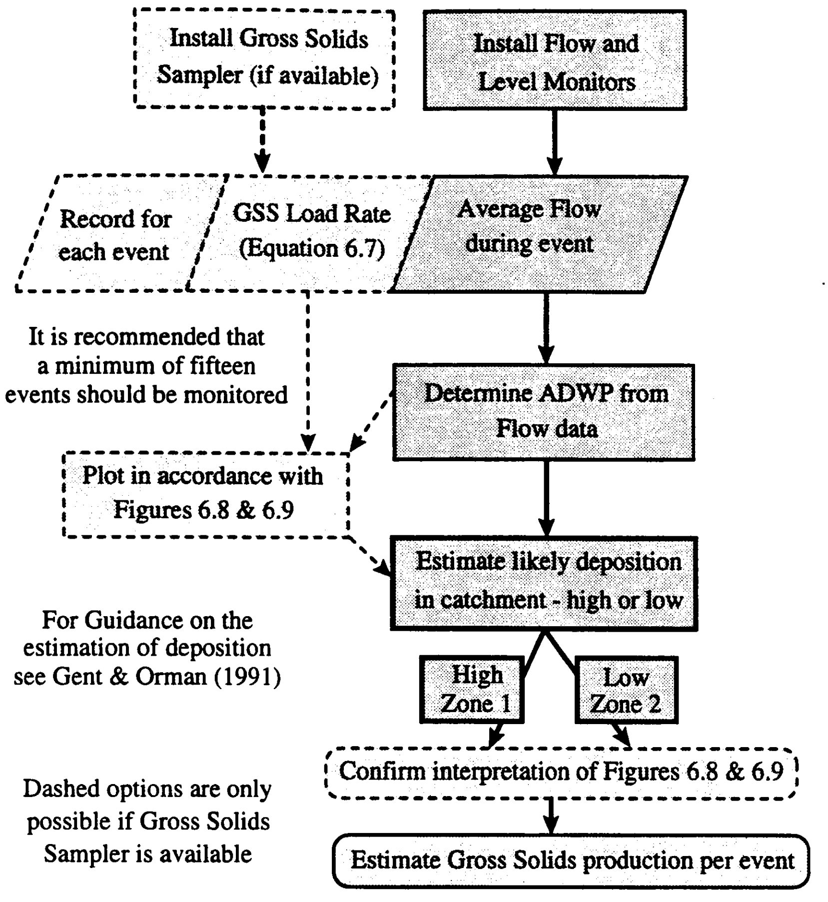


Figure 8.2 Flow Chart for Interpretation of Gross Solids Sampler results

further information from different sites is required to confirm the zones on the Rate Chart. However, assuming the zones are correct, then the procedure illustrated in figure 8.2 may be followed in order to determine the likely quantities of gross solids which may be discharged during an event.

8.5 RECOMMENDATIONS FOR FUTURE WORK

Chapters 5 & 6 contain recommendations for future research using the Trash Traps and Gross Solids Sampler respectively.

It is recommended that use of the Trash Traps be extended to a range of different CSO sites with varying configurations and volumes in order that their use as a method of assessing overflow performance may be further evaluated. At the time of completion of this thesis, three overflows in Strathclyde Region were under study using Trash Traps.

It is recommended that two further overflows be studied using the GSS to confirm that the inability to differentiate between inlet and overflow was not a function of the particular installations. This recommendation is made in spite of the inability of the GSS to determine the performance of the overflow structures monitored in this research programme. Part of the future work should be directed to determining the ability of the GSS to draw sewage solids from the flow. It is considered essential that small bore sampling is carried out at the same time.

It is also recommended that the apparent common conditions which cause deposition and movement of both type C sediments and gross solids be further investigated. This study would require a field site where type C sediment is known to be deposited and the Gross Solids Sampler should be utilised to obtain samples of suitably large volumes.

The suggestion has been made in this thesis that the classification of sewer types into collectors, trunks and interceptors proposed by others may be justified. Further studies should be carried out at sites, including those in this study, to determine the measures by which this classification system might be formalised. This research would involve surveys of the systems to identify locations of sediment accumulation. Locations and quantities of accumulation would be correlated with catchment parameters such as size, slope and population. It is probable that sewer system models would also be employed to predict the hydraulic characteristics of the catchment.

Flow Split has been identified as the only practicable method of monitoring the performance of CSO installations where storage is incorporated. A programme of monitoring should be instigated to gather flow and level data from a range of installations so as to better compare a range of installations.

CHAPTER 9 CONCLUSIONS

Oh that a man might know
The end of this day's business, ere it come;
But it sufficeth that the day will end,
And then the end is known.
Shakespeare Julius Caesar

9.1 PRINCIPAL CONCLUSIONS

The results of a field study using three different types of equipment for sampling sewage solids are presented in this thesis. The equipment was installed at three combined sewer overflow (CSO) installations and conclusions have been drawn in three specific areas, these being:-

i) The efficacy of the equipment in sampling sewage solids;
ii) The performance of the CSOs both with and without storage, and;
iii) The nature and hydraulic influences on the different types of material sampled.

In gathering, interpreting and evaluating the data, all aims stated in the introduction were achieved. The conclusions listed in section 1.2 are discussed and justified in chapters 5, 6 & 7. Some amplification is included in the following sections since the discussion in those chapters focussed on each sampling method in turn without presenting a comparison of the different results.

9.1.1 Sampling for Gross Solids

The Trash Traps, developed by the author as part of this study, and the Gross Solids Sampler (GSS), a WRc device, produced measures of visible and gross solids respectively. The Trash Traps proved to be simpler to operate, and normally collected larger masses of solids from the flow than the GSS which was prone to breakdowns and pipe freezing in winter. The GSS was expensive both to construct and operate In contrast, the Trash Traps were simple and cheap to operate.

Both items of equipment had failings in their sampling technique. Some material was washed off and/or over the Trash Traps at high flows and their location was problematic due to submergence when the storage tanks became full. Sampling by the GSS may have been unrepresentative. Paper and rags were caught on the suction pipes and probably were obstructed from entering the intake tubes.

Considerable manipulation of the data obtained using each device was necessary prior to final interpretation of the information gained. The principal methods of data presentation were, from the GSS, an average rate of solids passing during each event, and, from the Trash Trap, the load or number of solids per event. Both methods of presentation were found to be valid in view of the sampling procedure in each case. The information from the Trash Traps was found in general to be more readily understood and clearer than that from the GSS. In view of their low cost and simplicity of operation, the Trash Traps are considered to be the most appropriate of the devices evaluated for monitoring CSO performance. Evaluation of the gross solids performance of full installations is only likely to be possible using Trash Traps in normal monitoring programmes by the NRA or River Purification Boards. This is due to the lower costs incurred in operating Trash Traps.

9.1.2 Performance of the Combined Sewer Overflows Studied

The Trash Traps provided the only method by which the gross solids performance of the CSOs studied could be assessed. This method may be limited, since there were unanswered questions on inlet loads. Performance of the CSOs could not be compared using the Gross Solids Sampler.

No difference in performance could be identified between the stilling pond at Broomhead and the high-side weir at Elgin Street. This conclusion is based both on a comparison of inlet and overflow concentrations, and on the Trash Trap results. The Storm King at Lochgelly produced a 56% reduction of TSS concentrations between inlet and overflow, although this was found not to be statistically significant due to data scatter. The Trash Trap results could not be presented in linear form, however, the Storm King clearly showed improved separation of visible solids in comparison with the remaining devices studied. It was not possible to determine whether this was due to the **treatment** provided by the Storm King, or by the volume of storage. The retention time of the Storm King installation was fifteen minutes at typical overflow rates in comparison to negligible values at the remaining CSOs.

The performances of the overall installations were determined using the results from the small-bore samplers. Values for Total Efficiency, Flow Split and Treatment Factor have been presented for all the installations studied. The Treatment Factors, on the basis of the suspended solids results, were all found to be close to unity, and it was concluded that the only viable method of comparison was by using Flow Split.

9.1.3 An Additional Sediment Class

The research has highlighted the lack of previous understanding of the behaviour of gross solids in sewer systems. This material has been ignored in classification methods, and this study has shown that visible solids should be added as a sediment type to the classes proposed by Crabtree (1989b). The additional sediment class may be defined in terms of size (greater than 6mm in two dimensions), material (paper or plastic), and intrusiveness (visible on bankside vegetation). It was concluded separately from both the Trash Trap and GSS studies that visible solids are subject to the same hydraulic influences as type C sediment.

A gross solids rate chart was developed from the GSS data. This chart shows that sewer sites may be differentiated on the basis of their rate of gross solids production during high flow events. From the chart it was also concluded that a 24 hour antecedent dry weather period was critical in gross solids deposition in the sewer system. The interpretation has been made that gross solids would accumulate over a 24 hour period, but that the deposit would not significantly increase thereafter.

9.2 ON THE RELIABILITY OF THE RESULTS

Many factors affect the reliance which may be placed on the results from a field study and only to a certain extent may a numerical accuracy be attached to that reliance. Flows and levels were shown in this research to have accuracies in the range from zero to $\pm10\%$. Comparable figures for the physico-chemical determinands were approaching $\pm2\%$ for COD and up to $\pm10\%$ for BOD and TSS. No accuracy could be applied to other measurements such as the Trash Trap and Gross Solids Sampler results.

The levels of accuracy of the individual measurements must be considered in conjunction with the validity of application of a particular result to the flow field in which the measurement was taken. No effort was made in this research to evaluate in detail the extent to which site factors affected the validity of measurements taken. As far as possible the hydraulic conditions at all flow measurement points were evaluated and only validated readings were considered to have the accuracies quoted above. Sampling both by the small-bore and the gross solids samplers suffered from the uncertainty of knowing whether or not the mean concentration within the flow was being sampled.

Concentration gradients have been shown by other research to occur in sewer flows. At two of the sites, turbulence and cross flows at the inlets were significant, thereby aiding mixing of the flow, however, low velocities were a feature of the Broomhead inlet and the impression is gained that some stratification of the concentrations would have occurred. It is highly likely that the samples obtained would have been affected by the concentration gradients at this site. However, it was impossible to quantify the effect. Consequently it has been assumed that the samplers obtained representative samples from all sites.

The inaccuracies and variabilities must be addressed by consideration of both the generality of the catchments and the sufficiency of the data. In chapter Five the case was made that the catchments were typical of United Kingdom conditions. It is contended that general conclusions may be drawn as a result and that catchment variability, while still present, was not significant, and this allowed valid comparisons to be made.

The second plank upon which comparisons may be made is that of the sufficiency of the data. A full analysis of rainfall data was not carried out since the focus of the study was on the structures rather than the catchments, however, the rainfall was believed to have been typical of the areas with one event only, on 7th January 1992 having a return period of greater than two years. Any general applicability of the results has relied on sufficient amounts of data being obtained to allow statistical comparisons to be made. Where sufficient overflow and spill data were gathered to allow probabilities to be computed the data was found to fit the log-normal distribution and it is contended that, at least for two sites, sufficient data were gathered.

In summary, the catchments varied but were typical. The events showed variation but sufficient were sampled to allow general conclusions to be drawn. A sufficient range of inflow conditions were monitored to support the contention that the performance of each overflow could be determined.

9.3 POSTSCRIPT

This thesis started life as a sort of journey through a technical land. The achievements en route have been set out as best as could be, and the work has to stand or fall on their stature. It is difficult not to feel a sense of disappointment that they could not be better, more time could have been spent on measurement, or some other form of presentation tried to allow a clearer picture to emerge. The work has progressed knowledge a few steps further, it has suggested some new directions where research should be directed. For Stevenson travelling on his donkey, perhaps some mud has been cleared from the signpost.

APPENDIX A REFERENCES

Aalderink R.H, Van Duln E.H.S, Peels C.E, & Scholten M.J.M. (1990) Some Characteristics of Run-off Quality from a separated Sewer System in Lelystad, the Netherlands. Proc 5th ICUSD, Osaka, Japan May 1990.

Ackers P., Harrison A.J.M & Brewer A.J. (1967) Laboratory Studies of Storm Overflows with Unsteady Flow. Proc. Symp. on Storm Sewage Overflows ICE 1967.

Ashley R.M, Coghlan B.P. & Jefferies C. (1990) The Quality of Sewage Flows and Sediment in Dundee. Wat Sci Tech Vol. 22 No. 10/11 1990.

Ashley R.M, & Crabtree R.W. (1992) Sediment Origins, Deposition and Build up in Combined Sewer Systems. Wat Sci Tech Vol. 25 No. 8 1992.

Ashley R.M, Jefferies C. & Stevens G.S.W. (1986) Overloading of the Dunfermline Sewerage System due to Peripheral Urban Expansion. Proc Conf Infrastructure Renewal & Waste Control Univ Manchester April 1986.

Ashley R.M, Wotherspoon D.J.J, Coghlan B.P. & McGregor I. (1992a) The Erosion and Movement of Sediment and Associated Pollutants in Combined Sewers. Wat Sci Tech Vol. 26 No. 5-6 1992.

Ashley R.M, Wotherspoon D.J.J, Goodison M.J, McGregor I. & Coghlan B.P. (1992b) The Deposition and Erosion of Sediments in Sewers. Wat Sci Tech Vol. 26 No. 5/6 1992.

Au-Yeung W.K. (1990) Flow Routing in the Parallel Pipes of a Combined Sewer System. PhD Thesis, Dundee Institute of Technology, Dundee 1990.

Balmforth D.J. (1982) Improving the Performance of Stilling Pond Storm Sewage Overflows. Urban Drainage Systems. Ed Featherstone & James. Pitman 1982.

Balmforth D.J. (1986) Effectiveness of storm sewage overflow structures in handling gross polluting solids. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Balmforth D.J. (1990) The Pollution Aspects of Storm Sewage Overflows. Proc IWEM. Vol4. No3. June 1990.

Balmforth D.J & Henderson R.J (1988) A Guide to the Design of Storm overflow Structures. Report ER 304E. Water Research Centre, Swindon 1988.

Balmforth D.J., Lea S.J. & Sarginson E.J. (1987) Development of a Vortex Storm Sewage Overflow with Peripheral Spill. Proc 4th ICUSD Lausanne, June 1987.

Balmforth D.J. & Sarginson E.J. (1983) An Experimental investigation of the Discharge Capacity of Side Wiers. Proc. Int. Conf. on Hydraulic Aspects of Floods and Flood Control. BHRA 1983.

Bennett N.J, & Rossbrook R.B. (1992) Combined Sewer Overflows - The Bexhill Solution. Proc Seminar on Combined Sewer Overflows - Practice and Performance Hull Oct 1992 Hydro Research & Development.

Brechenmacher M, Wirth H. & Gaeffen K. (1992) Investigations on Large Stormwater Retention Tanks in Munich. Wat Sci Tech Vol. 26 No. 5/6 1992.

Brombach H. (1989) Equipment and Instrumentation for CSO control. Urban Storm Quality Enhancement: Source Control, Retrofitting and Combined Sewer Technology. H.C. Torno Ed. ASCE 1989.

Brombach H. (1990) Liquid-Solid Separation at Vortex Storm Overflows. Proc 5th ICUSD, Osaka, Japan, May 1990.

Brombach H, Xanthopoulos C, Hahn H.H. & Pisano WC. (1992) Experience with Vortex Separators for Combined Sewer Overflow Control. Proc AQUATEC 92 RAI Amsterdam 1992.

Burrows R. & Ali K.H.M. (1982) Stormwater Overflows: Interpretation of Particle Test Results. Urban Drainage Systems. Proc 1st Int Seminar. Southampton UK 1982.

Burrows R, Crowder G. & Lowther J. (1989) Sewer Flow Surveys: An Appraisal of Combined Velocity/Depth Logger Performance by Laboratory Studies. Proc Conf on Drainage and Waste Management into the 1990's Dundee Institute of Technology, Dundee May 1989.

Cant J.A. (1990) Detention tank Design and Maintenance. Report UM 1051. Water Research Centre, Swindon 1990.

Chebbo G. & Bachoc A. (1992) Characterisation of Suspended Solids in Urban Wet Weather Discharges. Wat. Sci. Tech. Vol. 25 No 8.

Cherry-Garrard (1929) The Worst Journey in the World. (an Account of Scott's last Antarctic Expedition 1910-1913) Chatto & Windus 1965.

Clifforde I.T, Fiddes D, & Tyson J.M. (1990) Urban Pollution Control in the United Kingdom. Proc 5th ICUSD, Osaka, Japan, May 1990.

CIRIA (1987) Sediment Movement in Combined Sewerage and Storm-Water Drainage Systems. Project Report 1, CIRIA, London UK.

Cootes T, Balmforth D.J, Russel S. & Doughty GM. (1989) Monitoring a Vortex Storm Sewage Overflow with Peripheral Spill. Proc 2nd Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1989.

Crabtree R.W. (1989a) A Classification of Sewer Sediment Types and Characteristics. Foundation for Water Research Report ER 324E Sept 1989.

Crabtree R.W. (1989b) Sediments in Sewers JIWEM Vol.3 No.6 Dec 1989.

Crabtree R.W, Ashley R.M, & Saul A.J. (1991) A Review of Research into Sediments in Sewers and Ancilliary Structures. Foundation for Water Research Report FR 0205 June 1991.

Davis P.S, & Parkinson A.D. (1990) Littleborough Combined Sewer Overflow - a New Approach to Planning and Design. Proc 5th ICUSD, Osaka, Japan May 1990.

Delo E.A. & Saul A.J (1989) Charts for the hydraulic Design of High Side Weirs in Storm Sewage Overflows. Proc ICE Part 2 1989 June.

Dohman H, Tänkler J, & Weyand M. (1986) Effects of Detention Basins (DISC) on the composition of Stormwater and Combined Sewer Overflow. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Ellis J.B. (1986) Pollutional Aspects of Urban Runoff. Urban Runoff Pollution, Nato ASI Series Vol. G10. Springer-Verlag.

Ellis J.B. (1982) Blights and Benifits of Urban Storm Water Quality Control. Urban Drainage Systems. Proc 1st Int Seminar. Southampton UK 1982.

Field R. (1974) Design of a combined Sewer Overflow Regulator/Concentrator. Jnl. Water Poll. Control Federation 1974 Vol. 46 Pt.7.

Fletcher I.J, Pratt C.J. & Elliot G. (1982) An Assessment of the Role of Gulley Pots in Determining the Quality of Urban Storm Runoff. Proc First Int. Seminar Southampton UK. 1982.

FRPB (1985, 1986 & 1990) Forth River Purification Board Annual Reports

Frederick M.R. & Markland E. (1967) The performance of Stilling Ponds in handling solids. Proc Symp on Storm Sewage Overflows ICE 1967.

Geiger W. (1984) Characteristics of Combined Sewer Runoff Proc 3rd ICUSD Goteburg, June 1984.

Geiger W. (1986) Variation of Combined Runoff Quality and Combined Runoff Pollutant Retention Strategies. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Geiger W. (1987) Flushing Effects in Combined Sewer Systems Proc 4th ICUSD Lausanne, June 1987.

Green M. (1991) Assessment of Sewer Overflow Performance. Water Research Centre Report UM 1193, Swindon, March 1991.

Gujer W. & Krejci V. (1987) Urban Storm Drainage and Receiving Waters Ecology. Proc 4th ICUSD Lausanne, June 1987.

Hall M.J, Ellis J.B. & Brizio M.G. (1990) On the Statistical Distribution of Event Mean Concentration of Pollutants in Stormwater Runoff. Proc 5th ICUSD, Osaka, Japan May 1990.

Halliwell A.R & Saul A.J (1980) The use of hydraulic models to examine the performance of storm-sewage overflows. Proc Inst Civil Engrs. Part 2 1980 69 June.

Hedges P.D & Lockley P.E. (1992) A Comparison of the Performance of an Hydrodynamic Separator with contemporary CSOs. Proc. 65th Conf. WPCF. New Orleans Sept 1992.

Hedges P.D, Lockley P.E. & Martin J.R. (1992) A Field Study of an Hydrodynamic Separator Combined Sewer Overflow. Procedings NOVATECH 92 Lyon Nov. 1992.

Henderson R.J. (1988) Prediction of Foul Flow Quality in Sewers. Report (unpublished) WRc 1988.

Hogland W., Berndtsson & Larson M. (1984) Estimation of Quality and Pollution Load of Combined Sewer Overflow Discharge. Proc 3rd ICUSD Goteburg, June 1984.

House M.A., (1989) A Water Quality Index for River Management (1989) JIWEM 1989 3 Aug.
Hydro Research & Development Ltd. (1990) Publicity Material

Jefferies C. (1992) Methods of Estimating the Discharge of Gross Solids from Combined Sewer Systems. Wat Sci Tech Vol. 26 No. 5/6 1992.

Jefferies C & Ashley R.M. (1985) The Use of a microcomputer for the Simulation of Flows in the Dunfermline Sewerage System. Proc 2nd Int Conf on Civil and Structural Engineering Computing. Vol 2 Sept. 1985.

Jefferies C. & Dickson R.A. (1991) The Design, Construction and Performance Assessment of a Storm King Storm Water Overflow. JIWEM Vol 5 No 2. April 1991.

Jefferies C. & Stevens G.S.W (1989) Sewerage Rehabilitation for Pollution abatement in Fife - A Review and Case Study. Proc Conf on Drainage and Waste Management into the 1990's. Dundee Institute of Technology, Dundee May 1989.

Jefferies C, Wakelin A.W. & Ashley R.M. (1987) Keeping Track of Hydrological Data. Proc 3rd Int. Conf. on the use of Computers in Civil Engineering. ICE, London, Sept 1987.

Jefferies C. & Walsh A.M. (1991) Interim Evaluation of a Stilling Pond Overflow using the WRc Gross Solids Sampler (GSS). Water Research Centre Report UM 1212, Swindon, March 1991.

Jefferies C, Young H.K. & McGregor I (1990) Microbial Aspects of Sewage and Sewage Sludge in Dundee, Scotland. Wat. Sci. Tech Vol.22 No. 10/11 1990.

Knott G.E & Taylor A.J (1985) The use of Detention Tanks for Flow Attenuation - a Review of Current Practice. WRc Report ER 171E (1985).

Krejci V. & Baer E. (1990) Screening Structures for Combined Sewage Overflow Treatment in Switzerland. Proc 5th ICUSD, Osaka, Japan May 1990.

Krejci V, Munz W. & Gujer W. (1986) Efficiency of Stormwater Overflow Tanks. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Lansing A. (1959) Endurance Shackleton's Incredible Voyage Hodder & Stoughton.

Lindholm O.G. (1984) Dry Weather Deposition in Combined Sewers Proc 3rd ICUSD Goteburg, June 1984.

Mance G. (1981) The Quality of Urban Storm Discharges: A Review. Technical Report 192-M, Water Research Centre, Stevenage UK.

M.H.L.G (1970) Taken for Granted, Report of the Working Party on Sewage Disposal. Ministry of Health and Local Government. HMSO 1970.

Moffa P.E. (1990) The Combined Sewer Overflow Problem: An Overview. In Control and Treatment of Combined Sewer Overflows Ed P.E. Moffa Van Nostrand 1990.

Moys G.D. (1987) Modelling of Stormwater Quality Including Tanks and Overflows (MOSQITO) - design specification Hydraulics Research Report SR 127 March 1987.

Mutzner H. (1987) Visible Pollution from Combined Sewer Overflows. Proc 4th ICUSD Lausanne Sept. 1987.

Nakamura E. (1990) Characteristics of Wet Weather Combined Sewage Information to the Design and Operation of Improved Systems. Proc 5th ICUSD, Osaka, Japan. May 1990.

National Water Council (1977) Final Report of the Working Party on Consent Conditions for Effluent Discharges to Freshwater Streams. NWC London 1977.

Nicoll E. & McGillivray (1977) Report of the Working Party on Storm Sewage Scotland - a Review. Proc. Annual Conf. Inst. Wat. Poll. Cont. 1977.

O'Sullivan P. (1990) A Review of the Performance of Storm Sewage Overflow Structures with Respect to Aesthetic Criteria. Report UM 1037 Water Research Centre, Swindon. March 1990.

Page S.J (1986) Quantifying the Screenings Problem at Sewage Works. Proc IWPC 1986. pp420-426

Pearson L.G, Thornton R.C, Saul A.J. & Howard K. (1986) An Introductory Analysis of Factors affecting the Concentration of Pollutants in the First Foul Flush of a Combined Sewer System. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Pisano W.C. (1988) Swirl Concentrators Revisited. The Design of Urban Runoff Quality Controls. ASCE 1988.

Pisano W.C. (1990) Recent US experience with Designs of German Vortex Solids Separator for CSO Control. Proc 5th ICUSD, Osaka, Japan June 1990.

Reddy Y.R & Pickford J. (1972) Stilling Pond Overflow Studies. Sanitary Division ASCE. SA4 August 1972.

Ruff S. & Saul A.J. (1992) Private Communication on Preliminary Results of Large Scale Model Overflow Testing Programme. Sheffield University March 1992.

Saul A.J, Brownbill V.R, Skipworth P, Roulston M. & Lafond J.M. (1992) Laboratory Investigations of Shaft Tank Efficiency. Proc NOVATEC 92. Lyon.

Saul A.J. & Delo E.A. (1982) Laboratory tests on a storm sewer overflow chamber with unsteady flow. Urban Drainage Systems. Proc 1st Int Seminar. Southampton UK 1982.

Saul A.J. & Ellis D.R. (1990) Optimum Design of Storage Tanks for Self-cleansing operation with Cohesive Sediments. University of Sheffield Department of Civil and Structural Engineering. 1990

Saul A.J. & Marsh P.M. (1990a) Mosqito Verification Study, Draft Final Report to Water Research Centre. University of Sheffield, UK 1990.

Saul A.J. & Marsh P.M. (1990b) A Strategy for the Monitoring of Pollutants in Sewers, Overflows & Tanks. Proc 5th ICUSD, Osaka, Japan June 1990.

Saul A.J. & Thornton R.C. (1989) Storage Tanks as a Future Strategy for the Retention of First Flush Load. Proc Conf on Drainage and Waste Management into the 1990's Dundee Institute of Technology, Dundee May 1989.

Saul A.J, Thornton R.C. & Henderson R.J. (1985) The Flood Control Characteristics of a Storage Volume in a Sewer System based on Field Measurements and Computer Simulation Proc. 2nd Int. Conf. on the Hydraulics of Floods & Flood Control BHRA 1985.

SDD (1977) Scottish Development Department - Working Party on Storm Sewage (Scotland). Storm Sewage Separation and Disposal. HMSO 1977.

Seager J. & Milne I. (1990) Ecological Impact of Combined Sewer Overflow Discharges. Proc 5th ICUSD, Osaka, Japan June 1990.

Sharpe D.E. & Kirkbride T.W. (1959) Storm Water Overflows; the Operation & Design of a Stilling Pond. Proc. ICE Vol 13 Aug 1956.

Sidwick J.M. (1984) Screenings and Grit Removal in Sewage: Removal, Treatment and Disposal. Technical Note 119 CIRIA, London 1988.

Sidwick J.M. (1988) Screenings and Grit Removal in Sewage: Removal, Treatment and Disposal Phase 3: Storm Water Overflows and Pumping Stations. Technical Note 132 CIRIA, London 1988.

Siegel (1956) Non Parametric Statistics for the Behavioural Sciences. M^{c}Graw Hill.

Smisson B. (1967) Design, Construction and Performance of Vortex Overflows. Proc Symp on Storm Sewage Overflows ICE 1967.

Sorensen M.S. & Larsen T. (1990) Comparison of Suspended Solids Separation in Advanced Storm Overflow Structures. Proc 5th ICUSD, Osaka, Japan May 1990.

Stotz G. & Krauth K. (1984) Factors Affecting First Flushes in Combined Sewers. Proc 3rd ICUSD Goteburg, June 1984.

Stotz G. & Krauth K. (1986) Detention Basins and their capabilities for the Retention of Pollutants. Proc 1st Int. Conf. on Urban Storm Water and Effects on Receiving Waters. Wageningen, Netherlands. Sept 1986.

Subramanya K. (1986) Flow in open channels. Tata McGraw Hill.

Thornton R.C. & Saul A.J. (1986) Some Quality Characteristics of Combined Sewer Overflows. The Public Health Engineer Vol 14 (3) July 1986.

Tucker C.G.J. & Mortimer G.H. (1978) The Generation of Suspended Solids Loads in Urban Stormwater. Urban Storm Drainage Pentach Press. 1978.

Tyack J.N, Hedges P.D. & Smisson R.P.M. (1992) The Use of Settling Velocity Grading in Combined Sewer Overflow design. Proc NOVATEC 92. Lyon.

Veenhuis J.E, Parrish J.H. & Jennings M.E. (1988) Monitoring and Design of Stormwater Control Basins. The Design of Urban Runoff Controls ASCE 1988.

Walsh A.M. (1990) The Gross Solids Sampler - Development of a Sampler for Combined Sewer Overflows. Report UM 1172, Water Research Centre, Swindon, UK. November 1990.

Walsh A.M. (1992) Private Communication on Results of Gross Solids Sampling in Swansea.

Walsh A.M. & Jefferies C. (1992) Final Report on the Evaluation of Combined Sewer Overflows in Dunfermline, Scotland using the WRc Gross Solids Sampler.Report UM 1320, Water Research Centre, Swindon, UK. November 1992.

White J.B. (1978) Wastewater Engineering. Edward Arnold.

Wilson G. Day Level Drainage in Eighteenth Century Fife Coal Mines. Source & date unknown. Dunfermline Central Library.

WRc (1987) A Guide to Short Term Flow Surveys of Sewer Systems. Water Research Centre, Swindon, 1987.

WRc (1988) Interim Water Quality Planning Procedures for Controlling Intermittent Pollution from Storm Sewage Overflows. WRc Engineering 1988

WRc/WAA Sewerage Rehabilitation Manual 2nd Edition. (1986) Water Research Centre, Swindon, UK.

Wotherspoon D.J.J. (1990) Cohesive Sediment Transport in Large Diameter Combined Sewers. Report Prepared for Transfer to PhD. Dundee Institute of Technology Nov. 1990.

Wotherspoon D.J.J & Ashley R.M. (1992) Rheological Measurements of the Yield Strength of Combined Sewer Sediment Deposits. Wat Sci Tech Vol. 25 No. 8 1992.

Appendix B

Pollutant Concentration Data

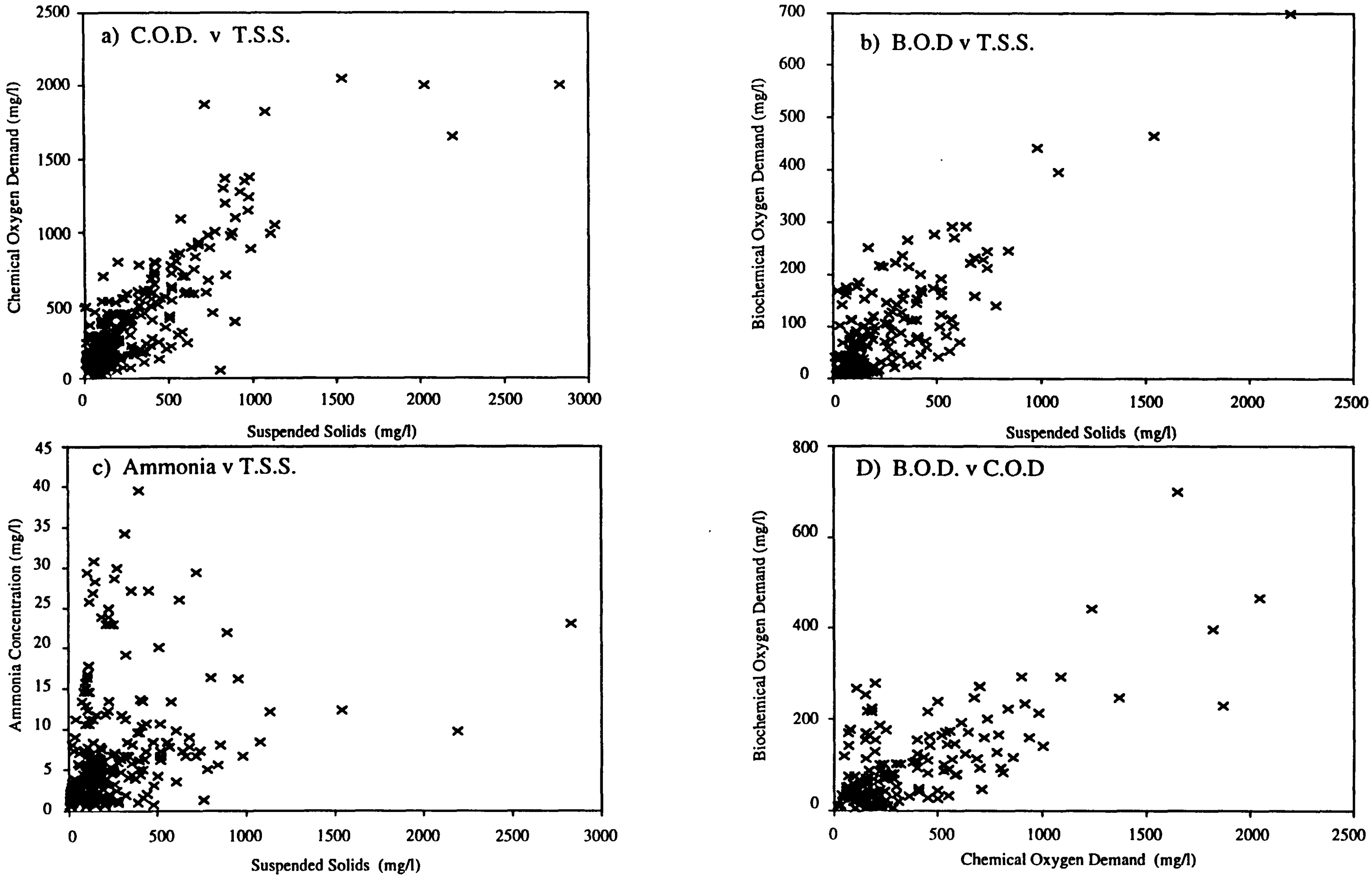


Figure B1 Pollutant Concentration Scatter Plots For Combined Sewer Overflow Events

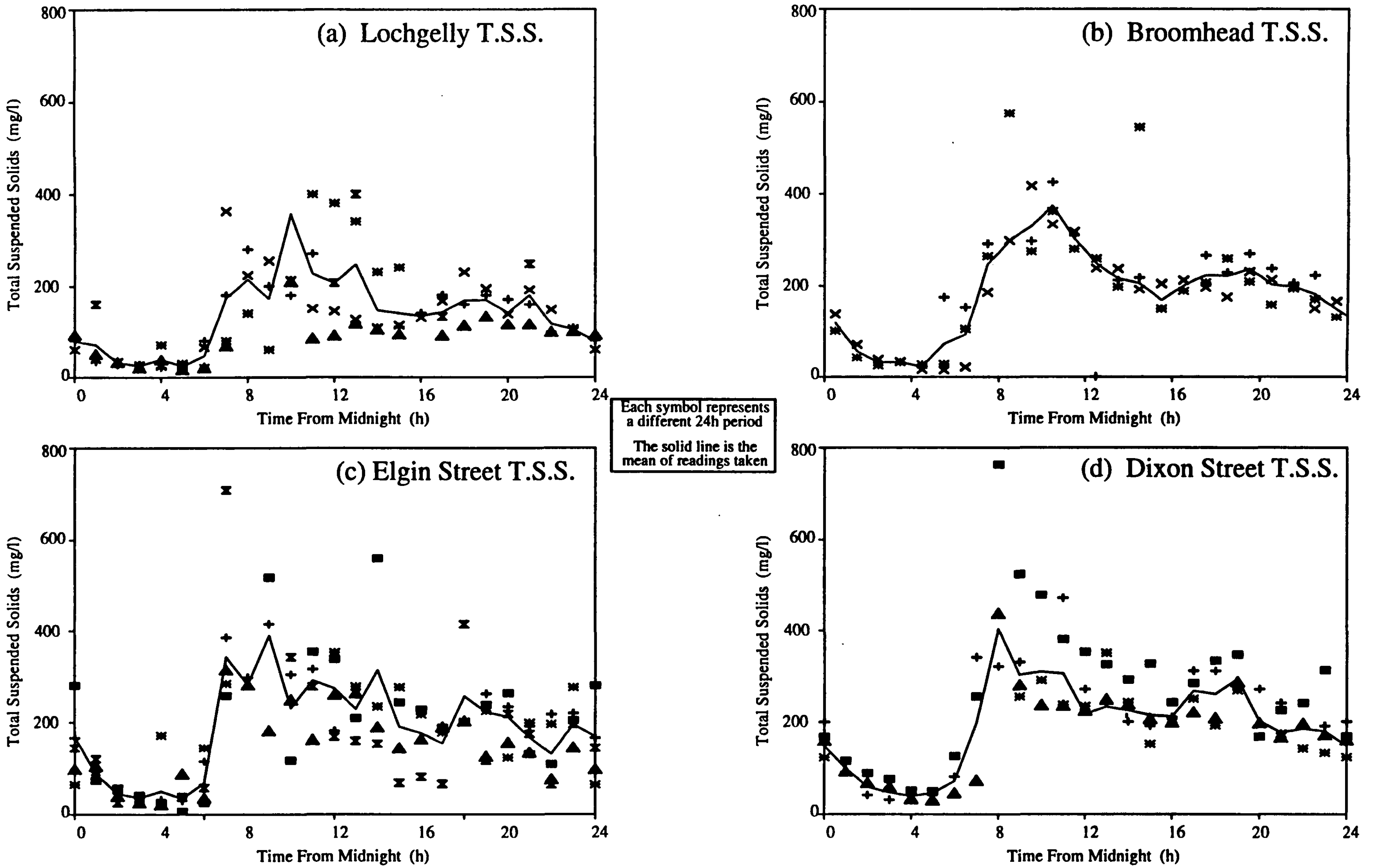


Figure B2 DWF Suspended Solids Concentrations

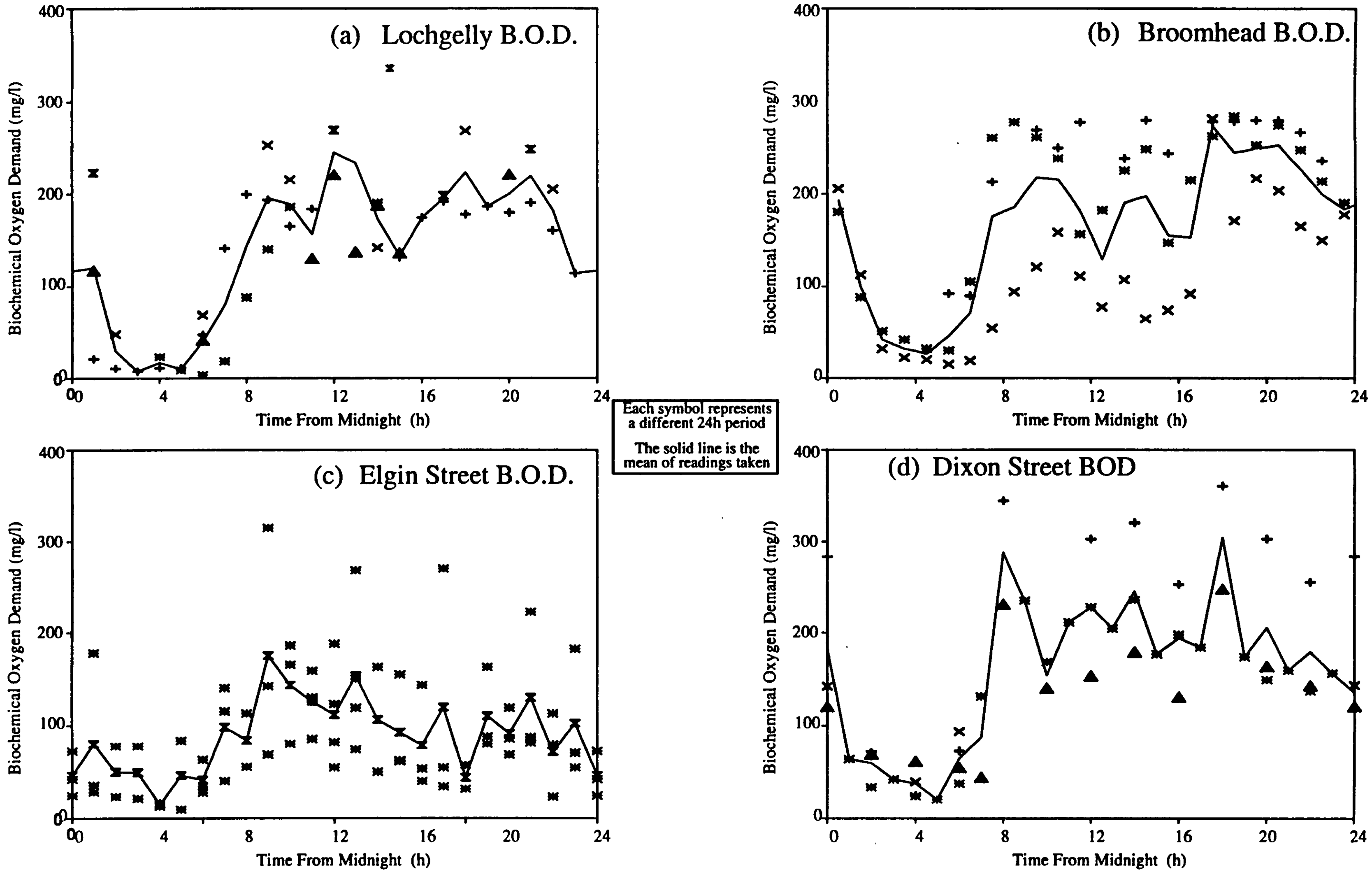


Figure B3 DWF Biochemical Oxygen Demand Concentrations

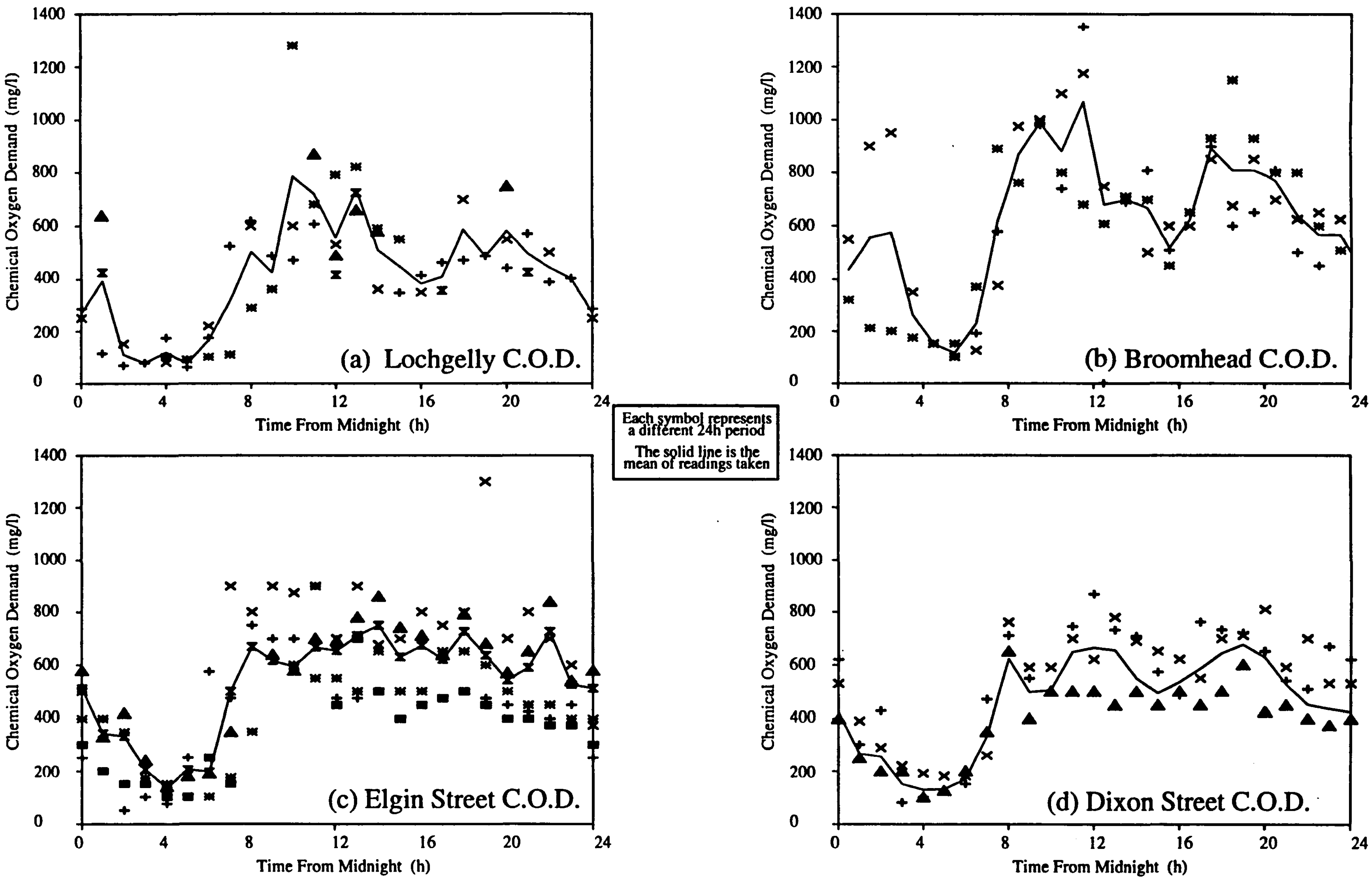


Figure B4 DWF Chemical Oxygen Demand Concentrations

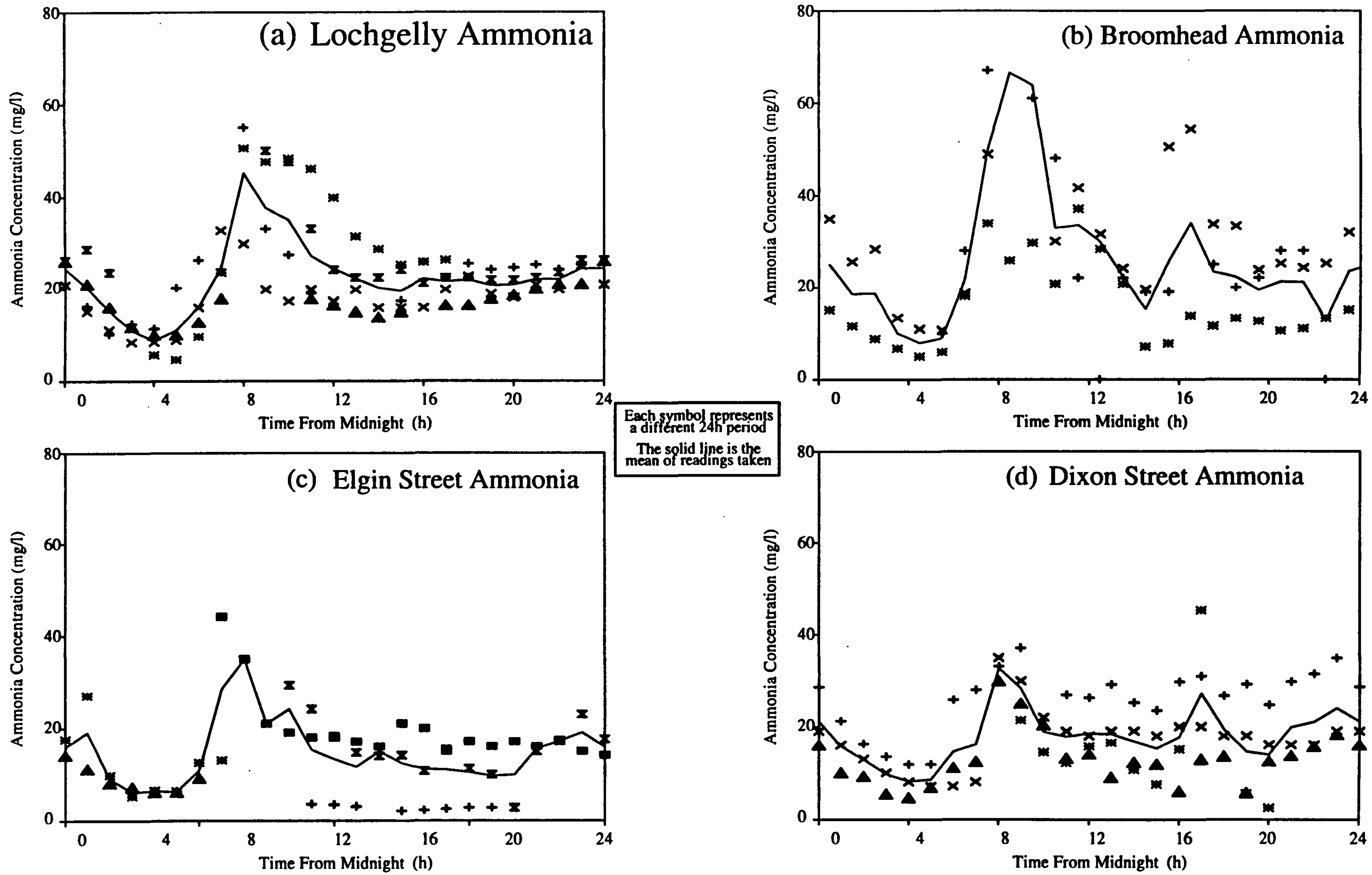


Figure B5 DWF Ammonia Concentrations

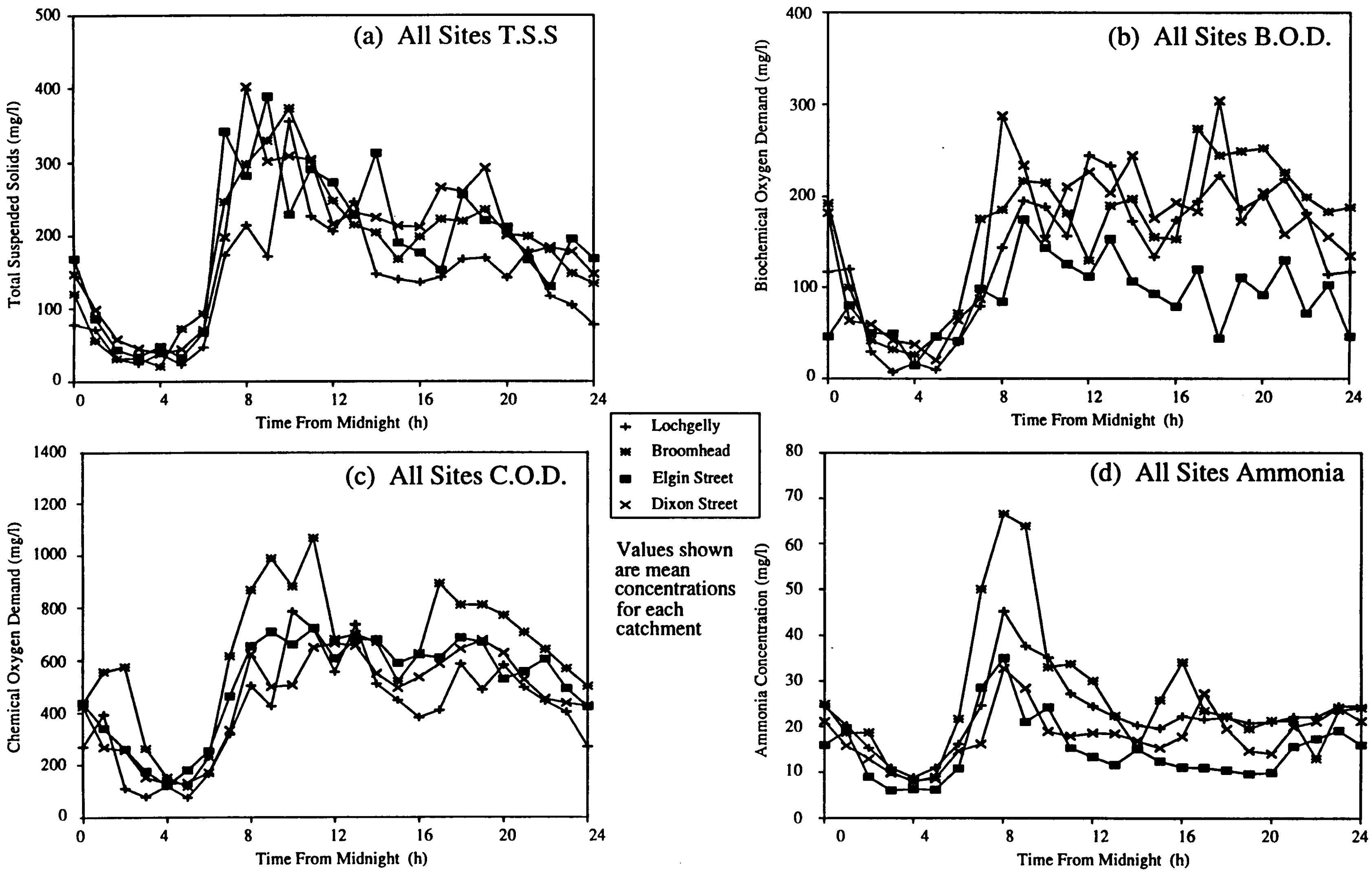


Figure B6 Resume of DWF Pollutant Concentrations

Appendix C - Trash Trap and Gross Solids Sampler Data

						INLET				SPILL					
No	Date	Start Time	End Time	Rain Tot mm	ADWP h	Max Lev mm	Max Flow l/s	Epic Mass Inlet Kg	Vol m^3	Max Flow l/s	Over Vol m^3	Epic Mass Spilled g	Trash Wt g	Trash No	Blind 1 = Full 0 = Nil
13	18/9/90	20:00	23:00	4.2	372	1199	261	270	1183	178	300	82.1	68.5	*	
14	30/9/90	04:55	05:20	3.4	89	1246	356	*	986	284	298	97.5	67.3	*	
15	15/10/90	07:00	09:15	3.8	59	1164	267	*	*	99	224	17.9	45.0	*	0.00
16	16/10/90	03:50	05:15	3.6	21	1294	439	331	1287	392	523	136.7	68.2	18	0.10
17	28/10/90	07:20	12:15	7.8	29	1211	315	964	4111	205	1624	134.1	121.4	21	0.05
18	30/10/90	*	*	4.2	21	1202	275	*	*	185	1512	*	47.8	*	0.00
19	16/11/90	09:11	09:46	*	71	1165	254	*	1069	101	202	73.1	66.5	25	0.15
20	22/12/90	04:40	15:00	5.0	44	1147	204	*	2133	61	52	*	2.6	*	0.00
21	22/12/90	21:00	04:00	2.4	7	1191	274	*	4379	160	941	44.8	4.7	3	0.00
22	28/12/90	13:30	18:00	1.6	5	1259	423	*	1120	313	497	*	78.9	17	0.15
23	1/1/91	15:55	20:00	3.0	5	1234	235	*	1492	256	1041	251.0	51.6	26	0.05
24	1-2/1/91	21:00	14:45	2.4	0	1280	470	*	7070	360	2324	310.0	122.0	34	0.00
25	4/1/91	13:55	17:30	Snow	23	1259	423	*	3014	313	1370	240.0	138.1	28	0.00
26	4-5/1/91	17:30	10:00	Snow	0	1243	387	*	7400	167	1860	81.0	23.8	18	0.00

To convert Trash Trap readings to full flow the observed values should be multiplied by 1.65

* No Data

Table C.1 Trash Trap Data - Lochgelly

No	Date	Overflow Start Time	Overflow End Time	ADWP h	Max Lev mm	Max Flow l/s	Over Vol m^3	Epic Mass Over kg	Sample Wt g	Trap Total Solids Number	Blind 1=full 0=Nil
1	18/9/90	21:15	22:05	7	912	64	112	*	*	*	1
2	20/9/90	18:30	18:45	1	855	19	7	*	16.9	*	*
3	30/9/90	04:30	05:00	43	859	21	18	*	31.1	*	*
4	2-3/10/90	23:45	00:25	2	900	53	40	7.1	174.6	*	1
5	9/10/90	13:20	13:30	66	871	29	11	*	*	*	1
6	28/10/90	11:57	13:32	1	1013	93	104	30.2	48.7	*	Surch
7	16/11/90	08:33	10:08	43	916	66	365	69.1	*	*	1
8	25/11/90	14:52	15:33	0	900	52	36	*	47.9	12	0.50
9	25/11/90	19:10	19:20	2	829	5	1	2.9	20.0	4	*
10	6/12/90	20:15	21:14	96	918	69	86	85.8	103.8	*	1
11	9/12/90	06:45	07:06	4	834	5	2	0.4	20.0	4	0.10
12	20/12/90	07:33	08:14	7	886	34	37	8.7	71.2	*	0.90
13	22/12/90	12:47	13:40	4	876	24	35	5.8	11.2	*	0.30
14	22/12/90	18:45	20:37	0	853	35	91	8.0	67.0	*	0.20
15	28/12/90	13:00	18:00	11	942	115	201	*	48.7	15	0.90
16	28-29/12	18:00	15:00	5	850	16	117	*	21.9	7	0.20
17	8/3/91	13:00	14:22	86	934	85	240	150	*	*	1
18	15/3/91	15:40	16:30	30	864	25	35	9.9	4.5	5	0.05
19	17/3/91	01:44	02:12	2	853	17	21	3.7	*	*	*
20	17/3/91	06:18	07:22	2	888	43	77	12.6	11.6	7	0.25

To convert Trash Trap readings to full flow the observed values should be multilied by 1.77.

* No data

Table C.2 Trash Trap Data - Broomhead

No	Date	Time	ADWP h	Net Weight g	Max Flow l/s	Over Vol m^3	Epic Mass Over Kg	Paper	Plast	Sanit	Other	Trash Trap Solids Total	% Blind	Flow Factor
1	10/11/91	19:18-19:50	66	54.2	78	60	*	4	0	1	2	7	*	2.66
2	19/12/91	01:18-03:22	13	149.9	226	709	93.5	3	5	2	1	11	80	5.33
3	1/1/92	04:34-05:24	12	185.2	183	216	97.6	10	1	1	2	14	90	5.33
4	3/1/92	03:58-04:12	44	376.1	64	16	17.6	7	1	2	2	12	40	2.66
5	3/1/92	09:04-11:04	0	7.4	5	35	4	2	0	0	1	3	5	1.33
6	3/1/92	20:38-21:04	6	87.5	110	110	10.7	2	0	1	0	3	*	2.66
7	7/1/92	11:50-12:10	12	21.2	43	55	*	2	3	0	1	6	15	1.33
8	8/1/92	00:36-02:06	4	0.6	7	16	17.3	2	0	0	0	2	5	1.33

To convert Trash Trap readings to full flow the observed values should be multilied by the Factors shown in the right hand column

* No Data

Table C.3 Trash Trap Data - Elgin Street

Test No	Date	ADWP (h)	Start Time	Stop Time	COPA SACK Type (mm)	Auto	Chge Time (min)	Wait Time (min)	Samp. Time (s)	No of Cycles
1	10/10/90	4	15:00	15:20	4-6	Y	2	5	15	3
2	15/10/90	1	07:23	07:57	4-6	Y	2	5	30	5
3	15/10/90	5	17:15	18:00	4-6	Y	2	5	30	6
4	20/10/90	1	09:09	09:54	4-6	Y	2	5	30	6
5	28/10/90	1	08:10	09:45	4-6	Y	1	1.5-4	30	20
6	28/10/90	0	09:53	10:58	4-6	N	1	1.5-4	30	15
7	28/10/90	0	11:57	13:32	4-6	N	1	1.5-4	30	20
8	30/10/90	0	09:30	10:47	4-6	N	1	1.5-4	30	17
9	30/10/90	0	13:30	14:23	4-6	Y	1	1.5-4	30	13
10	16/11/90	43	08:33	10:08	4-6	Y	1	1.5-4	30	20
11	25/11/90	1	14:52	15:33	4-6	Y	1	1.5-4	30	11
12	25/11/90	2	19:10	19:20	4-6	Y	1	1.5-4	30	3
13	6/12/90	96	20:15	21:14	4-6	Y	1	1.5-4	60	14
14	9/12/90	4	06:45	07:06	4-6	Y	1	1.5-4	60	6
15	20/12/90	7	07:33	08:14	4-6	Y	1	1.5-4	60	11
16	22/12/90	4	12:47	13:40	4-6	N	1	1.5-4	60	13
17	22/12/90	0	18:45	20:37	4-6	Y	1	1.5-4	60	20
18	22/12/90	0	21:07	22:37	4-6	N	1	1.5-4	60	17
19	15/2/91		00:19		Bad Data)					
20	19/2/91		15:20		Bad Data >			Pipes Frozen		
21	19/2/91		16:37		Bad Data)					
22	27/2/91	4	23:58	00:59	2-3	Y	1	1.5-4	60	13
23	28/2/91	0	01:06	01:51	2-3	N	1	1.5-4	60	7
24	28/2/91	0	01:56	03:51	2-3	N	1	1.5-4	60	20
25	8/3/91	86	12:59	14:12	2-3	Y	1	1.5-4	60	14
26	15/3/91	30	15:39	16:55	2-3	Y	1	1.5-4	60	14
27	17/3/91	2	01:46	02:31	4-6	Y	1	1.5-4	60	10
28	17/3/91	2	06:20	07:25	4-6	Y	1	1.5-4	60	13
29	18/3/91	0	15:22	17:17	4-6	Y	1	1.5-4	60	20
30	18/3/91	0	17:36	19:31	4-6	N	1	1.5-4	60	20

Table C.4 Gross Solids Sampler Operation and Control - Broomhead

Test No	Stilling Pond Inflow: Net Sample Weight (g)	Comments	Max Lev (mm)	Max Flow (l/s)	In Vol (m^3)	Epic Mass In (Kg)	Stilling Pond Overflow: Net Sample Weight (g)	Comments	Max Flow (l/s)	Over Vol (m^3)	Epic Mass Over (Kg)
1	224.3	Fatty	889	106	57	20.0	0.0	Empty	43	11.4	3.5
2	127.4		906	70	110	*	0.0	Empty	6	2.0	*
3	56.1		941	159	259	127.0	0.0	Leaves	131	80.0	27.7
4	147.8		976	201	350	94.0	25.5		132	155.0	45.2
5	127.4		853	80	432	38.0	5.1	3 Fecal	17	38.0	3.4
6	117.2		914	131	419	50.3	25.5	1 Paper	64	145.0	16.3
7	0.0		1013	162	518	39.2	0.0	Empty	93	104.0	9.5
8	0.0		900	115	438	*	0.0	1 Leaf	5	387.0	*
9	0.0		832	62	185	*	0.0	1 Fecal	16	9.0	*
10	1697.2		916	159	563	302.0	198.8	Mainl Fecal	67	146.0	69.1
11	229.4	Mainly Paper	900	141	210	*	96.8	Fecal + Leaves	52	36.0	*
12	96.8	Mainly Paper	829	77	40	8.5	0.0	Empty	5	1.3	2.9
13	5774.7		918	150	265	253.0	0.0	Leaves + bean	69	86.0	85.7
14	76.5	Paper	834	84		*	0.0	3Fecal + San	5	2.0	0.4
15	107.0	Paper	886	119	217	90.3	35.7	Almost empty	34	37.0	8.7
16	0.0	Nil	876	85	207	32.5	0.0	Empty	24	33.0	5.8
17	56.1	Paper	853	95	451	54.2	0.0	Empty	35	67.0	8.0
18	15.3	Dry	1002	124	530	39.2	15.3	1 Plas.strip	62	192.0	14.1
19											
20											
21											
22		Bag Blocked	1024	89	265	*		Mainly Tampons	62	170.0	
23	81.5	Mainly Food	1023	66	139	21.0	0.0	Empty	39	70.0	11.0
24	0.0	Empty	1016	51	199	11.0	0.0	Empty	26	43.0	3.0
25	265.0	Mainly Food	934	150	546	306.0	30.6	Paper	85	240.0	155.0
26	71.4		864	85	296	103.0	15.3		25	35.0	9.9
27	35.7	Nearly Empty	853	84	140	*	30.6	Empty	17	21.0	3.7
28	56.1		888	105	328	54.2	40.8	Nearly Empty	43	77.0	12.6
29	56.1		1033	142	798	373.0	35.7	Gum paper	71	331.0	127.0
30	35.7		1095	208	1000	325.0	45.9	Condom	124	498.0	127.0

* No Data

Table C.5 Gross Solids Sampler Observations - Broomhead

Test No	Date	ADWP (h)	Start Time	Stop Time	Chge Time (min)	Wait Time (min)	Samp Time (min)	No of Cycles
1	18/11/91	72	16:45	17:45	1	1.5-4	60	12
2	18/11/91	0	19:08	20:07	1	1.5-4	60	12
3	29/11/91	18	07:32	08:24	1	1.5-4	60	11
4	17/12/91	434	09:50	10:17	1	1.5-4	60	6
5	19/12/91	12	00:55	02:50	1	1.5-4	60	20
6	22/12/91	0	00:08	01:25	1	1.5-4	60	16
7	22/12/91	0	12:10	13:44	1	1.5-4	60	17
8	3/1/92	37	03:58	05:11	1	1.5-4	60	14
9	3/1/92	3	08:33	10:28	1	1.5-4	60	20
10	3/1/92	0	18:33	19:13	1	1.5-4	60	9
11	3/1/92	0	20:37	21:22	1	1.5-4	60	10
12	7/1/92	22	05:00	05:52	1	1.5-4	60	11
13	7/1/92	0	23:58	01:25	1	1.5-4	60	16
14	8/1/92	0	01:29	02:48	1	1.5-4	60	15
15	8/1/92	0	02:52	04:32	1	1.5-4	60	18
16	3/2/92	24	10:26	11:39	1	1.5-4	60	14

Table C.6 Gross Solids Sampler Operation and Control - Elgin Street

Test No	High Side Weir Inflow				High Side Weir Overflow			
	Net Sample Weight (g)	Max Flow (l/s)	In Vol (m^3)	Epic Mass In (Kg)	Net Sample Weight (g)	Max Flow (l/s)	Over Vol (m^3)	Epic Mass Over (Kg)
1	111.0	280	862	113	48.0	*	*	*
2	24.5	383	1026	199	0.0	*	5	1.0
3	93.8	331	883	369	46.9	*	*	*
4	68.3	274	316	77	9.2	*	*	*
5	46.9	659	3310	560	55.0	226	536	74.0
6	12.2	338	1585	-	24.5	*	*	*
7	43.8	576	3029	288	42.8	181	446	50.5
8	56.1	452	1540	845	76.5	64	16	18.0
9	22.4	420	2580	224	16.3	20	16	1.6
10	12.2	270	490	130	27.5	*	*	*
11	0.0	494	1000	151	0.0	110	110	10.0
12	19.4	297	927	124	27.5	*	*	*
13	14.3	415	1920	143	0.0	8	11	1.0
14	0.0	415	1800	250	0.0	7	26	2.4
15	0.0	945	4570	772	0.0	532	2030	350.0
16	19.4	302	1194	208	11.2	*	*	*

* No Data

Table C.7 Gross Solids Sampler Observations - Elgin Street

Date	Time GMT	Flow l/s	TSS mg/l	Ring Time	Numbers in Ring Bag				Ring No Rate	Ring Conc No/m^3
					Plastic	Paper	Faecal	Total		
14/8/91	6.00	15	104	3	1	3	1	5	8.3	1.8
14/8/91	7.00	29	343	4	9	9	15	33	41.3	4.8
14/8/91	8.00	37	324	2.5	2	35	9	46	92.0	8.3
13/8/91	16.00	27	142	5	1	1	4	6	6.0	0.7
13/8/91	18.00	34	332	4	2	5	7	14	17.5	1.7
13/8/91	19.00	31	496	5	6	4	4	14	14.0	1.5
13/8/91	20.00	30	226	5	2	2	2	6	6.0	0.7
17/8/91	6.00	16	252	5	1	1	0	2	2.0	0.4
17/8/91	7.00	24	209	3	3	5	0	8	13.3	3.0
17/8/91	8.00	33	334	2	0	22	4	26	65.0	16.4
17/8/91	9.00	40	284	2	2	15	5	22	55.0	11.5
17/8/91	10.00	41	293	3	0	15	10	25	41.7	5.6
17/8/91	11.00	43	229	2.5	5	4	8	17	34.0	5.3

Table C.8 Results from Direct Gross Solids Sampling Elgin Street

Description of Ring Bag Test

The ring bag comprised a 6mm COPAsack attached firmly to a steel logger band allowing flow to pass but not solids greater than 6mm. The bag was held into the flow in the dry weather flow channel at Elgin Street for five minutes or, during times of high flow, until the bag became blocked. The Gross Solids Sampler was started immediately upon removing the sack from the flow and operated continuously for twenty minutes. In this manner, the mass retained by the **GSS** could be compared directly with the total number of solids in the flow.

Date	GMT Time	Sewer Flow l/s	TSS mg/l	GSS Flow l/s	% of total Flow	GSS Wt (N)	GSS Net Sack Wt (N)	Full Flow Net Sack Wt (N)
14/8/91	5.00	10	33	2.61	26.6	0.6	0.2	0.8
14/8/91	6.00	15	104	2.61	17.2	1.2	0.8	4.6
14/8/91	7.00	29	342	2.61	9.0	0.8	0.4	4.7
14/8/91	8.00	37	324	2.61	7.0	1.4	1.0	13.5
14/8/91	9.00	40	329	2.61	6.5	0.8	0.4	5.4
13/8/91	11.00	30	282	2.61	8.7	0.9	0.5	5.2
13/8/91	12.00	29	234	2.61	9.1	0.7	0.3	2.7
13/8/91	13.00	26	205	2.61	9.9	1.0	0.6	6.1
13/8/91	14.00	27	189	2.61	9.6	0.8	0.4	4.2
13/8/91	16.00	27	142	2.61	9.6	0.7	0.3	2.6
13/8/91	17.00	29	107	2.61	9.1	1.0	0.6	7.0
13/8/91	18.00	34	332	2.61	7.8	1.0	0.6	8.1
13/8/91	19.00	31	496	2.61	8.3	0.9	0.5	6.0
13/8/91	21.00	31	282	2.61	8.5	0.8	0.4	4.7
13/8/91	22.00	31	188	2.61	8.6	0.8	0.4	4.1
13/8/91	23.00	28	74	2.61	9.5	0.9	0.5	5.3
17/8/91	6.00	16	252	2.7	16.9	0.6	0.2	1.1
17/8/91	7.00	24	209	2.7	11.1	0.8	0.4	3.8
17/8/91	8.00	33	334	2.7	8.2	1.0	0.6	7.0
17/8/91	9.00	40	284	2.7	6.8	1.7	1.3	18.5
17/8/91	10.00	41	293	2.7	6.6	1.1	0.7	10.6
17/8/91	11.00	43	229	2.7	6.3	0.7	0.3	5.4
21/8/91	17.00	32		2.7	8.4	0.4	0.1	0.9
21/8/91	18.00	36		2.7	7.6	0.6	0.5	6.3
21/8/91	19.00	33		2.7	8.2	0.7	0.7	8.1
21/8/91	20.00	31		2.7	8.7	0.8	0.7	8.0

Table C.9 Gross Solids in Dry Weather Flow
Elgin Street

Appendix D

Overflow Efficiency Data

START DATE	RAINFALL START TIME	RAIN (mm)	INLET PEAK Lev (mm)	PEAK Flow l/s	IN Vol (m^3)	TSS MASS (kg)	EVENT MEAN CONC mg/l	OVERFLOW PEAK Flow l/s	OVER Vol (m^3)	TSS Mass (kg)	EVENT MEAN Conc mg/l	FLOW SPLIT %	TOTAL EFFY %
28/06/89	11:03	4.8	1187	292	2150			151	206	26	127	90	
30/06/89	16:12	2.8	1147	229	660			61	40			94	
10/08/89	19:45	5.8	1182	229	1022			140	414			59	
13/08/89	03:13	5.6	1280	420	2456	399	162	378	1227	158	129	50	60 *
13/08/89	09:51	4.0	1253	204				105	1520				*
15/08/89	14:55	3.2	1237	350	843			263	336			60	*
16/08/89	15:15	0.8	1120	175				0					*
20/08/89	09:21	5.8	1326	570	1990			464	1200			40	*
20/08/89	16:25	1.8	1164		1169			99	170			85	*
26/08/89	08:46	5.8	1198	216	2270			176	683			70	*
30/08/89	11:35	4.4	1276	439	1360			331	565			58	
30/08/89	17:45	3.8	1164	197	869			99	118			86	
15/09/89	11:53	3.8	1176		780			126	230	26	112	71	
22/09/89	09:45	1.2	1139	218	602			43	17			97	
20/10/89	21:05	5.4	1177	229	1880			128	460			76	
27/10/89	08:25	5.2	1205	305	4235			191	1350			68	
09/11/89	09:18	3.4	1229		1484			286	970			35	
10/11/89	17:05	2.2	1161		1034			92	316			69	
12/11/89	14:16	5.2	1163		1585			160	433			73	
15/8/90	08:20	3.8	1176	267	1300			124	425			67	
15/8/90	16:40	3.4	1204	280	830			189	140			83	
15/8/90	20:05	6.0	1187	280	2590			155	513			80	
18/9/90	20:00	4.2	1199	261	910	270	297	178	300	82	273	67	70
30/9/90	04:13	3.4	1246	356	986			284	298	98	327	70	
15/10/90	05:58	3.8	1164	267	1416			99	224	18	80	84	
16/10/90	03:40	3.6	1294	439	1100	331	301	392	523	137	261	52	59
28/10/90	05:30	8.6	1211	315	1191	152	240	128	408	16.7	41	66	89
28/10/90	16:15	3.6	1202					184	1196				
30/10/90	08:30	4.2	1187		1352			151	317			77	
16/11/90	09:11		1165	254	900			101	202	73	362	78	
22/12/90	23:25	5.0	1147	204	1220			61	52			96	
22/12/90	18:50	2.4	1191	274	3707			160	942	45	48	75	
28/12/90	12:40	1.6	1259		1100			313	497			55	
1/1/91	15:14	3.0	1234	235	2110			156	1042	251	242	51	
1-2/1/91	22:20	2.4	1280		5470			360	2200	300	134	60	
4/1/91	13:55		1259		6110			313	2950	321	175	52	

* = Bypass hydrobrake partially blocked with an umbrella
Only Events Causing Overflow Included

Table D1 Review of Overflow Events Lochgelly

		RAINFALL		INFLOW					OVERFLOW				SPILL				OVERFLOW		SPILL	
No	Date	ADWP h	Total mm	Max Lev mm	Max Flow l/s	Total Vol m^3	Epic Mass kg	Event Mean Conc mg/l	Max Flow l/s	Over Vol m^3	Epic Mass kg	Event Mean Conc mg/l	Max Flow l/s	Spill Vol m^3	Epic Mass kg	Event Mean Conc mg/l	Over Flow Split	Over Total Effy	Spill Flow Split	Spill Total Effy
1	1/6/90	2	7.0	914	130	671	123	183	66	232	37	159					65	70	100	100
2	3/6/90	27	5.4	894	111	132	12	93	48	29	1	34					78	92	100	100
3	6/6/90	70	12.0	1064	167	1687	292	152	95	787	62	79	164	502			53	79	70	*
4	6-7/6/90	3	8.2	1074	179	834	100	120	102	379	62	164		230			55	38	72	*
5	8/6/90	6	2.4	949	169	315			101	136							57		100	100
6	20/6/90	2	3.6	1043	294	495	142	287	220	301	104	346					39	27	100	100
7	22/6/90	44	6.6	1177	519	664	725	1092	434	468	424	906		113			30	42	83	*
8	29/6/90	46	3.6	899	115	112			52	30							73		100	100
9	30/6/90	17	2.0	875	94	126			32	25							80		100	100
10	30/6/90	3	5.2	1045	297	813			71	216			71	126			73		85	*
11	30/6/90	4	2.6	1056	314	460			100	100			56	40			78		91	*
12	30/6-1/7	7	31.0	1618	279	7850			193	6140			193	6140			22		22	*
13	9/8/90	2	2.6	937	155	146			88	58							60		100	100
14	11/8/90	50	1.2	930	186	210			81	58							72		100	100
15	15/8/90	1	9.8	911	127	1054			63	392							63		100	100
16	15-16/8/9	0	22.6	1043	190	3063			120	1026			68	483			67		84	*
17	16/8/90	3	2.8	862	83	204			23	34							83		100	100
18	16/8/90	3	2.2	821	67	70			2	1							99		100	100
19	28/8/90	81	1.4	854	78	80			18	11							86		100	100
20	16/9/90			885	103	260			40	27							90		100	100
21	18/9/90	7		912	128	375			64	112							70		100	100
22	20/9/90	1	1.2	855	80	82			19	7							91		100	100
23	30/9/90	43	2.0	859	110	150			21	18							88		100	100
24	2-3/10/90	2	4.2	900	116	296	38	129	53	40	7	177					86	81	100	100
25	5/10/90	54	7.0	915	131	532	55	103	66	97	17	175					82	69	100	100
26	6-7/10/90	18	43.6	1368	236	8134			155	5560			150	5300			32		35	*
27	9/10/90	66	1.4	871	91	71			29	11							85		100	100
28	10/10/90	2	0.2	889	106	57	20	344	43	11	4	318					81	83	100	100
29	15/10/10	1	7.4	906	110	540			58	132							76		100	100
30	15/10/90	5	2.4	941	159	214	118	551	93	80	28	346					63	77	100	100

* Insufficient small-bore sample results to determine effieiencies

Table D2A Review of Overflow Events Broomhead

		RAINFALL		INFLOW					OVERFLOW				SPILL				OVERFLOW		SPILL	
No	Date	ADWP h	Total mm	Max Lev mm	Max Flow l/s	Total Vol m^3	Inlet TSS Mass kg	Event Mean Conc mg/l	Max Flow l/s	Total Vol m^3	Over TSS Mass kg	Event Mean Conc mg/l	Max Flow l/s	Spill Vol m^3	Inlet TSS Mass Kg	Event Mean Conc mg/l	Over Flow Split %	Over Total Effy %	Spill Flow Split %	Spill Total Effy %
31	20/10/90	84	4.0	976	200	320	88	275	132	155	45	292					52	49	100	100
32	28/10/90	1	13.2	1013	162	1159	132	114	93	295	30	102	93	77	4	55	75	77	93	97
33	30/10/90	0	6.4	899	114	887			51	537							39		100	100
34	16/11/90	43	4.8	916	155	506	287	567	66	146	69	473					71	76	100	100
35	25/11/90	0	2.4	900	141	151			52	36							76		100	100
36	25/11/90	2	0.0	829	80	40	8	200	5	1	3	2231					97	64	100	100
37	6/12/90	96	4.2	918	150	219	243	1110	69	86	86	994					61	65	100	100
38	9/12/90	4	2.8	834	78				5	2		180							100	100
39	20/12/90	7	2.6	886	124	174	80	460	34	37	9	235					79	89	100	100
40	21/12/90	36	2.4	872	87	120	36	244	26	28	7	257					77	80	100	100
41	22/12/90	3	2.8	878	91	266			32	46							83		100	100
42	22/12/90	4	2.4	876	84	207	32	157	24	35	6	166					83	82	100	100
43	22/12/90	0	7.2	1038	135	2342	189	81	62	645	53	82	62	327	16	50	72	72	86	91
44	28/12/90	11	5.8	872	195	1300		224	115	201							85		100	100
45	28/12/90	5	2.0	850	75	928			16	113							88		100	100
46	29/12/90	5	5.0	827					4	4							*		*	*
47	1-2/1/91	50	14.6	1089	258	6900			208	1730			208	1480			75		79	*
48	3/1/91	Snowmelt		990	157	646			95	176			24	45			73		93	*
49	4-5/1/91	9	12.2	1057	177	1411	317		109	517	129	250	85	264	26	98	63	59	81	92
50	15/3/91	40	6.8	864	85	296	103	393	25	35	10	286					88	90	100	100
51	17/3/91	25	2.4	853	77	140			17	21	4	176					85		100	*
52	17/3/91	2	2.8	888	105	328	54	164	43	77	13	164					77	77	100	100
53	18-19/3/91	0	22.2	1245	327	2788	1409	505	218	1460	675	462	218	1204	353	293	48	52	57	75

* Insufficient small-bore sample results to determine effieiencies

Table D2B Review of Overflow Events Broomhead

No	Date	RAIN Total mm	INLET Max Flow l/s	INLET Total Vol m^3	INLET TSS Mass In kg	INLET Event Mean Conc mg/l	OVERFLOW Max Flow l/s	OVERFLOW Over Vol m^3	OVERFLOW Epic Mass Over kg	OVERFLOW Over Mean TSS Conc	SPILL Max Flow l/s	SPILL Spill Vol m^3	SPILL Epic Mass Spill kg	SPILL Epic Ave Conc mg/l	OVERFLOW Over Flow Split %	OVERFLOW Over Total Effy %	SPILL Spill Flow Split %	SPILL Spill Total Effy %
1	10/11/91	12.8	465	1985			80	60				0			97	*	100	100
2	12/11/91	9.8	350	1800								0			100	100	100	100
3	18/11/91	7.5	383	4750	584	123						0			100	100	100	100
4	29/11/91	4.2	244	2200	546	248						0			100	100	100	100
5	17/12/91	3.8	274	1943								0			100	100	100	100
6	17-18/12/9	4.4	287	3607								0			100	100	100	100
7	18/12/91	3.6	268	780								0			100	100	100	100
8	19/12/91	11.6	727	6120	813	133	294	950	95	100		0			84	88	100	100
9	21-23/12/9	34.6	890	49160			460	6600			460	5700			87	*	88	100
10	1/1/92	6.0	705	3800			238	280				0			93	*	100	100
11	3/1/92 A	5.4	452	2030	1010	498	64	16	19	1156		0			99	98	100	100
12	3/1/92 B	5.6	404	2522	235	93	5	25	2.5	100		0			100	100	100	100
13	3/1/92 C	1.6		1330	207	156						0			100	100	100	100
14	3/1/92 D	1.8	494	5606	545	97	104	115	11	93		0			98	98	100	100
15	7/1/92	2.4	297	1400				0				0			100	100	100	100
16	7/1/92	3.2	410	3570			43	55				0			98	*	100	100
17	8/1/92	32.0	1145	16170	2228	138	692	5273	803	152	692	3474	469	135	67	64	79	79
18	3/2/92	6.0	724	3950				0				0			100	*	100	100

* Insufficient small-bore sample results to determine effieiencies

Event of 8/1/92 extremely long and measurements ceased prior to event termination

Table D3 Review of Overflow Events Elgin Street

		INFLOW				OVERFLOW	
Event Date	Determinand	Inflow Over Event (m^3)	Inflow During Spill (m^3)	Load Over Event (kg)	Load During Spill (kg)	Overflow During Spill (m^3)	Load During Spill (kg)
13/8/89	TSS	2456	1786	402	243	1227	158
27/10/89	COD	4134	3232	532	334	1258	42
	BOD			135	79		27
18/9/90	TSS	1183	716	270	162	300	82
16/10/90	TSS	1287	760	331	254	523	137
28/10/90	TSS	1191	1191	152	152	408	17

		EFFICIENCIES				
		Total %	Pollution Separation %	Volume Ratio %	Treatment Factor	Treatment Factor TSS Only
13/8/89	TSS	60.7	35.0	50.0	1.2	1.2
20/8/89	TSS				1.1	1.1
27/10/89	COD	92.1	87.4	69.6	1.3	
	BOD	80.0	65.8	69.6	1.1	
18/9/90	TSS	69.6	49.4	74.6	0.9	0.9
16/10/90	TSS	58.6	46.1	59.4	1.0	1.0
28/10/90	TSS	89.0	89.0	65.7	1.4	1.4
Averages		74.5	60.5	63.9	1.16	1.12

Table D4 Principal Event Efficiencies Lochgelly

	INFLOW						OVERFLOW				SPILL	
Date	Inflow Over Event (m^3)	Inflow During Overflow (m^3)	Inflow During Spill (m^3)	Load Over Event (kg)	Load During Overflow (kg)	Load During Spill (kg)	Overflow During Overflow (m^3)	Overflow During Spill (m^3)	Load During Overflow (kg)	Load During Spill (kg)	Spill During Spill (m^3)	Load During Spill (kg)
28/10/90	1277	1006	194	131	126	10	295	77	30	4	77	5
2-23/12/90	3862	1762	1207	481	266	81	725	315	66	25	327	16
4/1/91	2127	1415	902	421	317	172	517	272	129	60	264	26
18/3/91	2801	2765	2268	1417	1403	1179	1453	1202	672	588	1204	353

	CONCENTRATIONS			INSTALLATION EFFICIENCIES				OVERFLOW EFFICIENCIES			
	Inlet (mg/l)	Overflow (mg/l)	Spill	Total %	Pollution Separation %	Volume Ratio %	Treatment Factor	Total %	Pollution Separation %	Volume Ratio %	Treatment Factor
28/10/90	103	102	58	96.6	54.1	94.0	1.03	77.1	76.1	76.9	1.00
21-23/12/90	124	91	50	96.6	79.8	91.8	1.05	86.3	75.3	81.2	1.06
4/1/91	198	250	98	93.9	85.0	87.2	1.08	69.3	59.2	75.7	0.92
18/3/91	506	462	293	75.1	70.1	57.1	1.32	52.6	52.1	48.1	1.09
Averages	243 *	226 *	125 *	90.5	72.3	82.5	1.12	71.3	65.7	70.5	1.02

* = Value is weighted average

Table D5 Principal Event Efficiencies Broomhead

Date	INFLOW						OVERFLOW				SPILL	
	Inflow Over Event (m^3)	Inflow During Overflow (m^3)	Inflow During Spill (m^3)	Load Over Event (kg)	Load During Overflow (kg)	Load During Spill (kg)	Overflow During Overflow (m^3)	Overflow During Spill (m^3)	Load During Overflow (kg)	Load During Spill (kg)	Spill During Spill (m^3)	Load During Spill (kg)
19/12/91	6120	4876	0	813	700	0	950	0	95	0	0	0
3/1/92A	2030	1502	0	1010	794	0	16	0	19	0	0	0
3/1/92D	5606	921	0	545	140	0	115	0	11	0	0	0
8/1/92	16170	12790	6790	2228	1825	898	5273	3474	803	460	6608	469

	CONCENTRATIONS			INSTALLATION EFFICIENCIES				OVERFLOW EFFICIENCIES			
	Inlet (mg/l)	Overflow (mg/l)	Spill (mg/l)	Total %	Pollution Separation %	Volume Ratio %	Treatment Factor	Total %	Pollution Separation %	Volume Ratio %	Treatment Factor
19/12/91	133	100						88.3	86.4	84.5	1.05
3/1/92A	498	1188						98.1	97.6	99.2	0.99
3/1/92D	97	96						98.0	92.1	97.9	1.00
8/1/92	138	152	71	78.9	47.8	78.5	1.01	64.0	56.0	67.4	0.95
Averages	154 *	146 *	71 *	78.9	47.8	78.5	1.01	87.1	83.0	87.3	1.00

* = Value is weighted average

Table D6 Principal Event Efficiencies Elgin Street

Appendix E - Papers and Reports Puplished during Registration Period

Jefferies C. (1990) **Quality Objectives for Stormwater Overflows, Practical Guidelines - Are They Possible?** Procedings of Seminar on Stormwater Management an Pollution Control, South Wales Accociation of Municipal Engineers, Swansea, February 1990.

Jefferies C. (1992) **Methods of estimating the discharge of Gross Solids from Combined Sewer Systems.** Wat. Sci. Tech Vol.26 No. 5/6 1992.

Jefferies C. & Dickson R.A. (1991) **The Design, Construction and Performance Assessment of a Storm King Storm Water Overflow.** JIWEM Vol 5 No 2. April 1991.

Jefferies C. & Stevens G.S.W (1989) **Sewerage Rehabilitation for Pollution abatement in Fife - A Review and Case Study.** Proc Conf on Drainage and Waste Management into the 1990's. Dundee Institute of Technology, Dundee May 1989.

* Jefferies C. & Walsh A.M. (1991) **Interim Evaluation of a Stilling Pond Overflow using the WRc Gross Solids Sampler (GSS).** Water Research Centre Report UM 1212, Swindon, March 1991.

Jefferies C, Young H.K. & M^{C}Gregor I (1990) **Microbial Aspects of Sewage and Sewage Sludge in Dundee, Scotland.** Wat. Sci. Tech Vol.22 No. 10/11 1990.

* Walsh A.M. & Jefferies C. (1992) **Final Report on the Evaluation of Combined Sewer Overflows in Dunfermline, Scotland using the WRc Gross Solids Sampler (GSS).** Water Research Centre Report UM 1320, Swindon, March 1992.

* Not Bound with Thesis

Quality Objectives for Storm Water Overflows - Practical Guidelines - Are they Possible?

Chris Jefferies Dundee Institute of Technology

INTRODUCTION

The problems of pollution emanating from storm and combined sewer overflows are well known and have been widely reported (1,2). Storm drains undoubtedly contribute significant levels of pollutants into watercourses, particularly where surface water catchments are extensively urbanised or heavily trafficked. It is however from combined sewer overflows that the most severe stream quality problems arise and the most visually offensive material is discharged. The Working Party on Storm Sewerage (Scotland) (3) produced recommendations for the design of overflows and the sizing of storm tanks which are widely used in spite of the paucity of data used in their development. A recent report (4) has drawn together the results of recent investigations into overflow design details and improved overflow design parameters have been set out. However, little is made of the capabilities of the overflows to remove pollution and no mention is made of environmental effects.

Studies of watercourse impacts have been carried out on a national basis by various bodies including the Rivers Authority, the Water Authorities and academic institutions. Much of this work has taken the form of regular routing sampling generally on a monthly basis, while in a few instances detailed impact assessments have been made. Unfortunately most of these studies of necessity have concentrated on specific locations where one or more overflows occur. Detailed investigations of this type are required due to the number of parameters which influence performance at each overflow site. There exists however, in addition to the results of detailed studies, a large body of impact data resulting from regular stream quality sampling as well as the circumstantial reports from local residents. This variability of sources of data and the number of factors governing the amount and effects of pollution from the different types of overflows have been key impediments preventing more widespread investigations and a more general comparison of the effectiveness of the devices from being made. It is significant also that no clear control standards for intermittent discharges exist at present providing little incentive for concerted efforts to be made in the comparison of performance. Already, the arbitrary 6mm screen equivalent required in some areas has caused alarm bells sound and as soon as standards are defined realistic means of comparing overflow types will be essential.

At present however, no form of general guidance is available to allow the sewerage engineer to make the all important bridge or link between the sewer structure to be installed and its likely benefit. Nor is there any means available for comparing the performance of one type of overflow from another on anything other than the crudest basis. Clearly a set of practical guidelines to form this bridge would be valuable. Unfortunately to gain acceptability it would nave to be based on a sufficiently wide data set to encompass the majority of overflow types and a very wide range of what may loosely be described as site conditions. The need for guidelines is clear and large amounts of operational data potentially suitable for their formulation exists. The question must be raised as to whether it is possible to formulate guidelines which are sufficiently broad and practical to gain widespread acceptance.

THE NEED FOR GUIDELINES

In developing rehabilitation works for the improvement of the pollution performance of combined sewer systems the sewerage engineer must almost always include one or more storm overflows. A number of different designs including stilling pond, high side weir and vortex are normally possible. However the capabilities of the different types of overflow are at present extremely uncertain. Additionally the overflow may include significant storage or may be claimed to "treat" the discharge to a certain standard. The impact of overflow discharges on receiving watercourses has been the subject of a number of studies but at present this work has tended to be very specific and local. The designer of the overflow is currently forced to produce designs on the basis of very sparse information concerning how well the various overflow types operate.

In the case of new or radically changed discharges it is also now necessary (as has been the case in Scotland with the River Purification Boards for a number of years) to gain consent for discharge with the real threat of refusal. The only obvious course at present is the construction of large on or off line storage to reduce the number of spills thus manifestly obviously reducing the degree of pollution discharged from the sewer system. It is frequently suggested that overflow discharges should have some form of screen performance to be set against, yet screens themselves at overflows ".....should be avoided if possible" (5). Yet again no replacement is recommended. There exist at present no practical guidelines which effectively allow the overflow to be engineered either on the basis of its ability to separate pollutants or on the likely end effect on the receiving water body.

In addition to providing an aid to the design process such guidelines could form the basis for the assessment of existing overflow performance. Currently this is either on the basis of the effect of the overflow on stream category or resulting from public complaints. It should be possible to identify a relatively concise set of field measurements to be made which could be related directly to the overflow. There is also the probability that WALLRUS will play a part in the development of peak flows at the ovrflow, and, with time series rainfall, spill volumes may be evaluated. When it arrives, MOSQITO (6) will provide an input to the process of evaluation process as a predictor of pollutant loads. Thus, without too great a degree of effort, a statement could be prepared as to whether the overflow discharge rate and volume reaches the required quality standard.

Many types of standard are possible and it is not the purpose of this paper to look into the value of each. It is however helpful to consider some requirements and consider the practicalities of taking measurements to aid decision making:

* The overflow discharge should be equivalent to that having passed through a screen of some set spacing.

* There should be no more than a set number of discharges per year.

* The total volume of discharge in a given time should not exceed a set amount.

* The first foul flush should be retained.

* The storm overflow should remove a proportion - say 90% of settlable solids in the flow.

* Acceptable numbers of flora and fauna in general counts should survive in the receiving watercourse.

* A specific organism could be selected for a specific ecotoxological standard.

It is clear that while such criteria relate to the watercourse which is of course the root of the requirement for standards, most are in fact directed towards the operation of the overflow itself. If guidelines are to become available they must relate both to the overflow operation and to the watercourse.

THE VARIABILITY OF OVERFLOW OPERATION

The diversity of factors which produce a discharge from a combined sewer overflow is very wide. These include factors related to the nature of the catchment, rainfall factors, the likely pollution load and the type and capacity of the overflow structure itself. Some data from a storm relief sewer are included to illustrate both the variability of discharges and the possibility that, even with such a wide range of data some commonalities can be seen.

The sewer in question drains a catchment in Eastern Scotland where land use is primarily residential and the population is in the order of 35,000. The foul sewer has been duplicated by the construction of the storm relief sewer in the past to relieve flooding and a series of cross connections were installed where the flow is controlled by low side weir overflows. Very little rainfall is required for flow to pass into the relief sewer, figure 1 showing the diversity of "events" in 1989 in which the peak flows ranged from 50 to 1550l/s with the median peak being approximately 250l/s. The sewer discharges into a relatively small watercourse which becomes grossly polluted for approximately a kilometre to the tidal limit. Trout inhabit the watercourse upstream from the discharge point whereas downstream it is a grossly polluted class 4 stream. The environmental damage caused by the overflow discharges is underscored by the abstraction of industrial process water downstream. Complaints of sewage derived solids in the pumped supply are regularly received!

Eleven of the 56 events included in figure 1 were sampled using a portable sampler and tests were carried out for a number of physical/chemical determinands. Suspended solids results for six of the events are plotted in figure 2. Ccumulative plots have been utilised both to illustrate the variety of total pollutant loads by event and the occurrence of a first flush (steeper initial section) in all but one of the events. The cumulative plots show that the events included are, as might be expected, very diverse in their production of suspended solids.

Although the data are highly variable, it is possible to draw comparisons between several of the determinands measured. For example figure 3 shows the observed values of BOD plotted against suspended solids for the storm data. It can be seen that a reasonably linear relationship exists between these parameters and it can be concluded at the very least from this relationship that the amount of testing for BOD may be able to be limited, with resultant time and cost savings. With further study, and the inclusion of other variables, more general conclusions should become more apparent. A further example of the ability to express commonalities in the data is given in figure 4 where the cumulative loads of figure 2 are expressed as percentages of the total load passing the observation point. Again one event is exceptional, however the remaining five show a high degree of commonality with between 50 and 70% of the total load passing in the first hour of the event. These plots give rise to optimism that a typical percentage load curve could be obtained for this overflow. It is believed that the data presented here as examples point to the possibility that categorisation of the performance will be possible.

DATA FOR THE FORMULATION OF GUIDELINES

It is clear that a very large data set relating to the operation of existing overflows must be collated for the formulation of a usable set of guidelines. It is only by this means that a realistic separation can be made between those overflows which operate satisfactorily and those which do not. It is only by amassing a very large data base that the variabilities in the types and quality of the data can be categorised and drawn into more general rules. At present it is not clear what such categories should be, however there is little doubt that they would have to become evident from the evailable data.

Threshold values of certain parameters will most probably provide a sound basis for categorisation. The device under consideration would be considered to be within a certain category provided the key measured parameters were above the threshold with the performance of the overflow still being acceptable. Figure 5 illustrates the formulation of a possible set of rules relating to a particular type of overflow. Each overflow would be considered to be acceptable if the relevant values of the decision parameters were all above the thresholds. Provided this was the case the categories of overflow should result from consideration of the size, type and location factors.

Data providing the basis of this set of rules must be highly diverse and variable both in nature and reliability. Some will result from rigorous studies of the type described with detailed measurement of flow and quality parameters, while, at the other extreme, information which is almost completely circumstantial should be able to be incorporated. Typical examples of the data which could be incorporated in order of increasing complexity are:

Flow and field measurements:-

Complaints from local residents;

Numbers of fish affected or killed (from anglers);

Stream quality as measured regularly by purification board;

Number and duration of overflow events;

Flow logging to determine overflow frequency and quantity;

Full flow, quality and environmental sampling;

Catchment related parameters:-

Size of sewer system and contributing population;

Catchment area and flow characteristics of watercourse;

Land use and industrial parameters for natural catchment;

Land use and industrial parameters for sewer catchment;

Overflow factors:-

Overflow Type;

Volume;

A factor related to the ability to separate solids;

The presence of screens;

This list does not purport to be exhaustive but it does however reinforce the amount and variability of the data required for any categorisation exercise.

Currently data on overflow performance are being collected at a number of locations in the UK. In all instances the studies have been commissioned by the Water Authorities with WRc acting in a coordinating function. In particular, Welsh, Severn Trent, North West and Yorkshire have investigations in progress, frequently with academic institutions and occasionally with private companies. Manchester and Aston Universities, Sheffield and Middlesex polytechnics, and Dundee Institute of Technology all have active field studies in progress.

New instrumentation under development for overflow studies include the gross solids monitor at Sheffield (7) and a large volume event triggered sampler at WRc. A rugged but simple device has also been developed in Germany (8) to monitor and statistically analyse the number and durations of overflow events. The data being amassed is both broad reaching and detailed, but specific to each location and must be considered in conjunction with the river quality information gathered regularly for most watercourses.

A HYPOTHETICAL GUIDELINE

As an exercise to develop ideas on how a set of guidelines might be presented, figure 6 is included as a flight of fancy. The curves are entirely speculative and the axes wholly unmarked, but the logic is very useful to show what the result might be. Entry to the nomograph is on axis 1 with the physical size of the contributing area, possibly modified for region, possibly expressed as output from WALLRUS or similar. Sediment and catchment land use factors clearly play an important part in the ability of the catchment to produce pollutant loading and the sloping lines of **Section A** represent these factors. This section will, it is hoped be replicated by MOSQITO.

Section B deals with the requirement of removal of visible solids from the overflow stream. Currently there is a lot of pressure for installations to have an equivalent screen size. The smaller the screen spacing, the better the removal of these solids, however screens are only likely to be considered "...only in situations of extreme environmental sensitivity" (5). It is clear that section B holds the key to the usefulness of the guidelines for comparisons of efficiency of solids separation. Different devices are claimed to provide a form of treatment of the discharged flow and such variations would be represented by varying positions of the sloping lines.

Section C represents the quality that the overflow is required to maintain in the watercourse. Probably this must be on the basis that this is the first overflow on the watercourse, or where more are present, the combined discharge of a number. The ability of the different overflow arrangements to separate solids, and the volume included to retain first foul flush and reduce the numbers of discharges would again be represented by the different curves.

CONCLUSION

The case has been put forward that a set of guidelines for combined sewer overflow design and evaluation based on feasibly collectable data are desirable. The purpose of the guidelines would be to allow estimation of the effect that a particular overflow arrangement would have on the watercourse quality. The assembly of a data base upon which the guidelines could be based is currently being mooted by WRc and it would appear that this would form a useful starting point for their development. To be of real value this data set must include what might loosely be described as circumstantial evidence on performance as well as the results of a series of detailed studies. It must also contain sufficient information to enable a realistic comparison of the different overflow types - with or without storage - to be made. Most importantly, by drawing on a wide set of information from both river and sewer, it is possible to foresee a bridge being made between the overflow type and the impact on the watercourse.

There are a number of potential uses of such guidelines. For new installations a baseline of performance details would be available for comparisons to be made at individual locations. The rivers inspector would have common ground with the drainage engineer when agreeing to the arrangement to be used for the new works. At present both the selection of the type and its performance criteria are the subject of much debate. For existing overflows the criteria, since they must inevitably bridge the knowledge between the sewer structure and the stream quality, would provide the basis for assessment of acceptability.

The variability of the data has been demonstrated, as have been some basic means of systemisation for the eventual formulation of categories and the basis of a possible guideline has been postulated. It is increasingly evident that, with the privatisation of the Water Authorities, the imposition of discharge quality standards is merely a matter of time. While work is proceding on the formulation of such standards utilising river quality models, there is a clear gap in the knowledge of the comparative behaviour of existing overflows to enable solutions to be found. A large number of individual studies of overflow performance are underway, and sampling of stream quality is both regular and routine, so the basic data must be being gathered. It is to be hoped that there is sufficient far sightedness on the part of the Water Authorities to part with what may be considered to be sensitive information and that finance can be found for the assembly of the data on a national basis to enable practical guidelines to be formulated. The need is clear, practical guidelines are possible.

REFERENCES

1. Department of the Environment. River quality in England and Wales 1985. HMSO London 1986.

2. Cliffordе I T et al. Urban pollution of rivers - the UK Water Industry research programme. Proc Int Conf on Water Quality Modelling in the Inland Natural Environment, BHRA 1986.

3. Report of the working party on storm sewage (Scotland). Storm sewage: Separation and disposal. HMSO 1977.

4. Balmforth D J and Henderson R J. A guide to the design of storm overflow structures WRc External report ER 304E 1988

5. Sidwick J M Screenings and grit in sewage : Removal, treatment and disposal phase 3 : Storm water overflows and pumping stations. CIRIA Technical note 132 1988.

6. Movs G D Modelling of stormwater quality including tanks and overflows (MOSQITO) - design specification. Hydraulics Research Ltd report SR 127 March 1987.

7. Cootes T, Balmforth D J et al. Monitoring a vortex stotm sewage overflow with peripheral spill. Procedings of the 2nd Wageningen conference on Urban storm water quality and ecological effects upon receiving waters. CHO/IAC Sept 1989.

8. Kleinschrot J and Brombach H. Monitoring of combined sewer overflows. Proceedings of the conference on drainage and waste management into the 1990s. Dundee Institute of Technology May 1989.

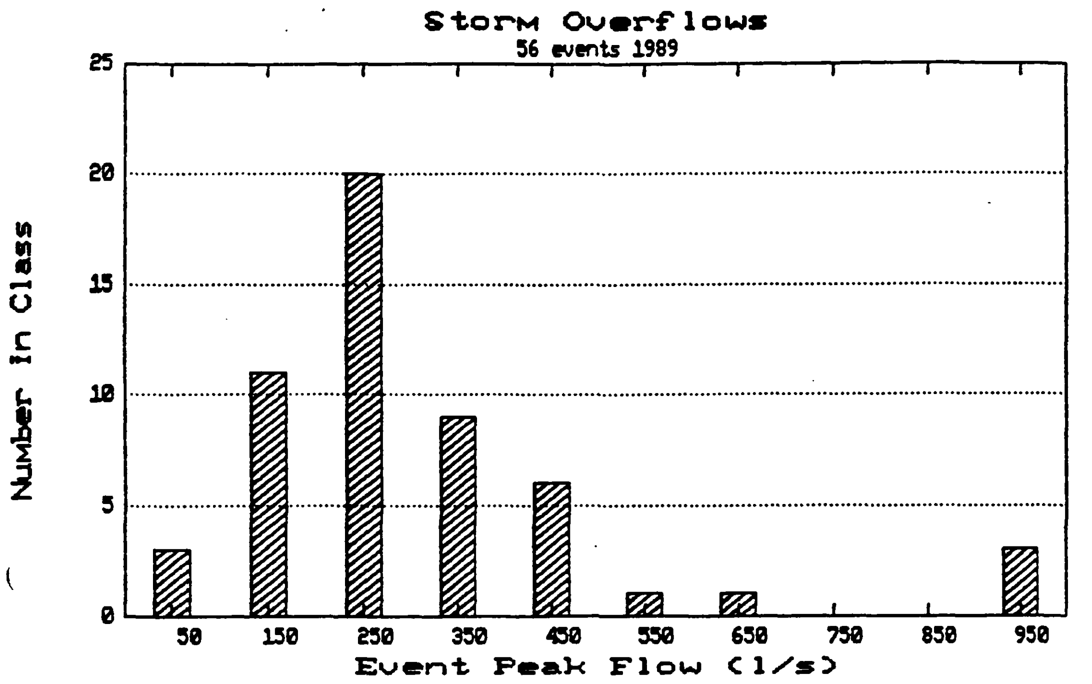


Fig 1. Variation of Peak overflow rates measured in a storm relief sewer.

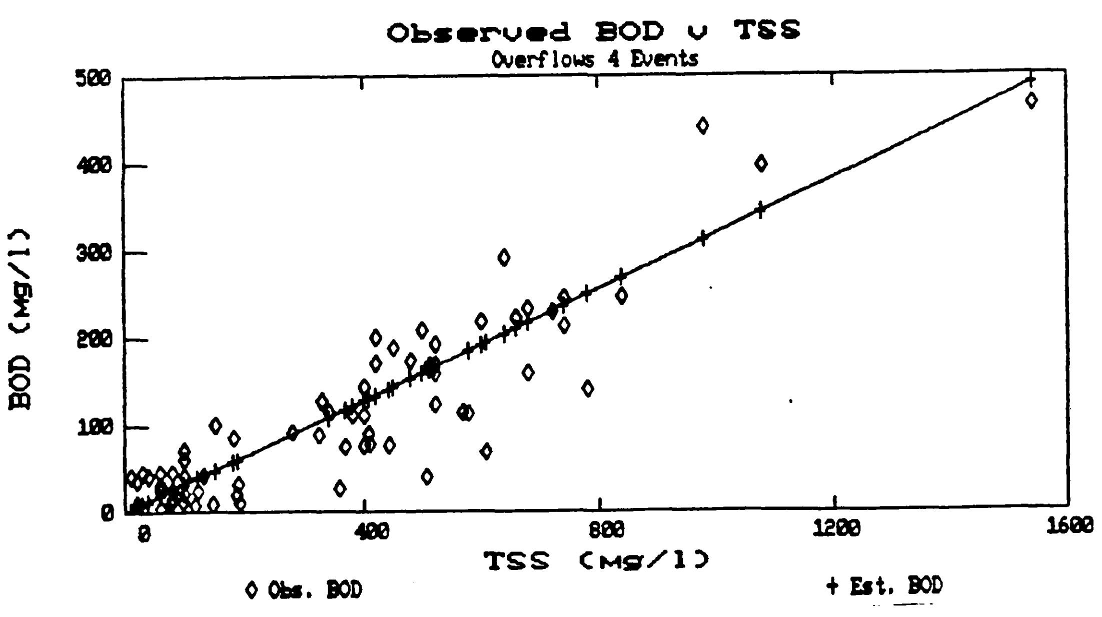


Fig 2. Comparison of measured BOD and TSS for four overflow events.

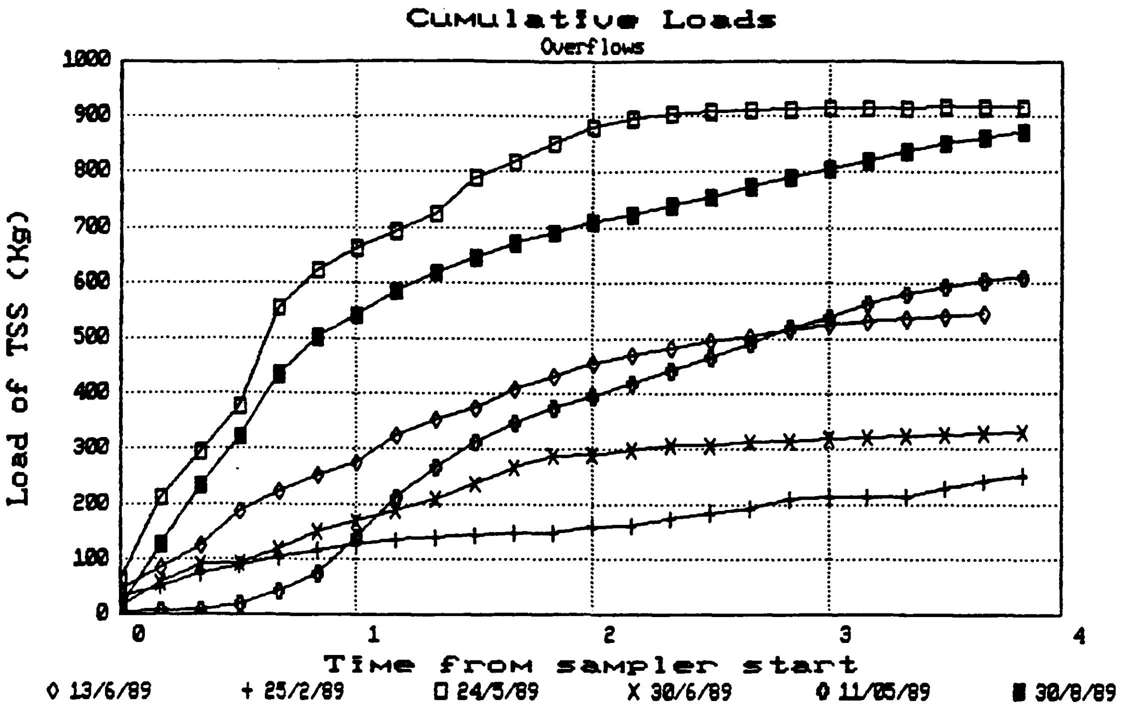


Fig 3. Cumulative pollutant loads in six overflow events
Expressed as load of TSS.

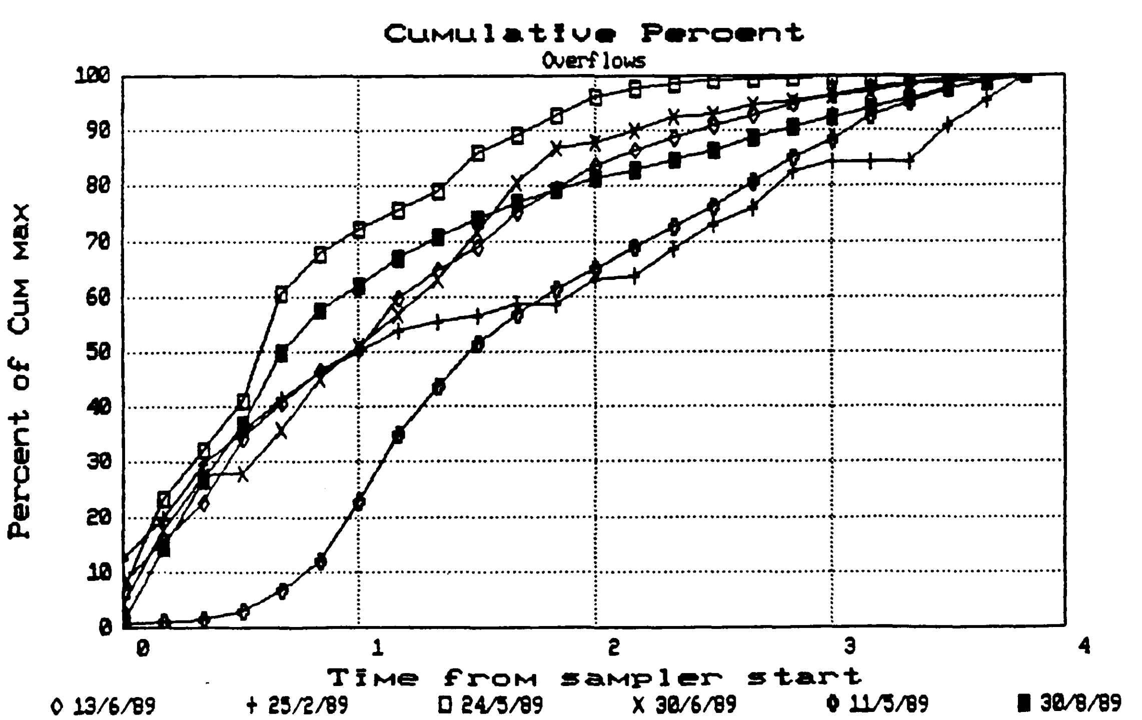


Fig 4. Cumulative pollutant loads in six overflow events
Expressed as percentages of total load.

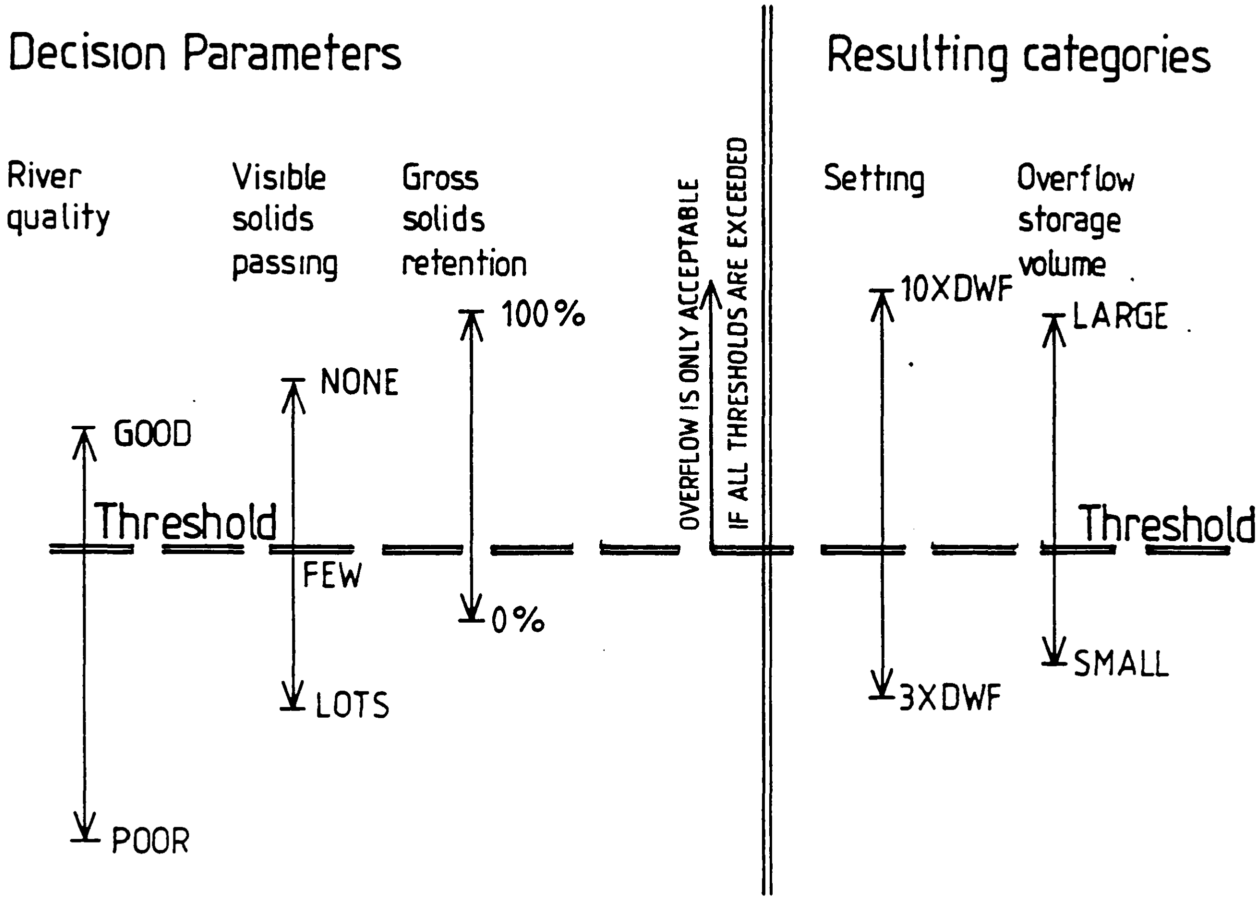


Fig.5 Performance rules for acceptability of overflows

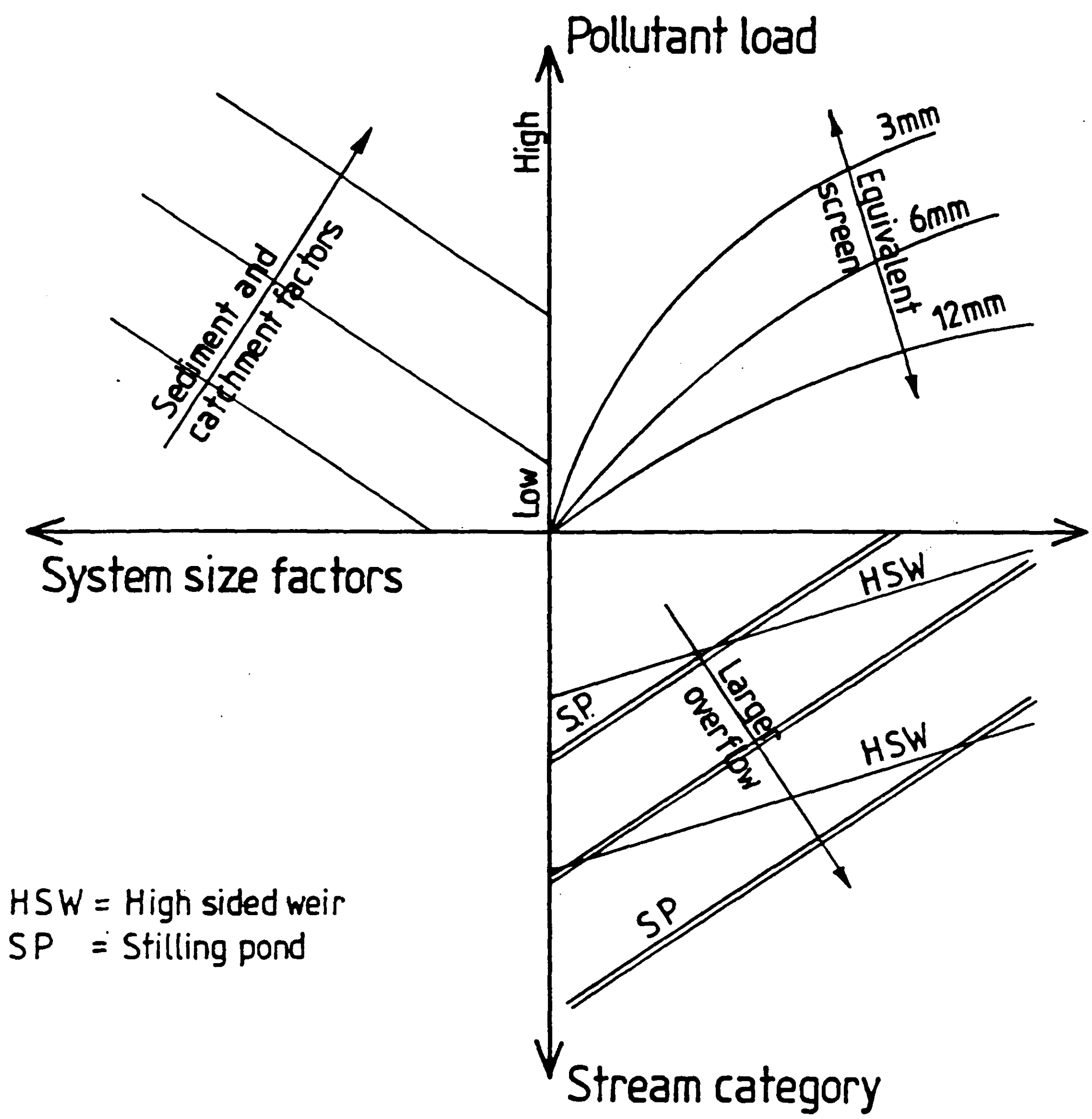


Fig 6. Hypothetical Storm overflow Selection Nomograph

The published papers cited below have been removed from the e-thesis due to copyright restrictions:

Jefferies, C. (1992) Methods of estimating the discharge of gross solids from combined sewer systems. In: *Water Science & Technology*, 26(5-6), pp.1295-1304.

Jefferies, C, Dickson, R.A. (1991) The design, construction and performance assessment of a Storm King storm-sewage overflow. In: *Journal IWEM*, 5, April, pp.151-154.

Jefferies, C, Young, H.K. and McGregor, I. (1990) Microbial aspects of sewage and sludge in Dundee, Scotland. In: *Water Science & Technology,* 22(10-11), pp.47-52.

SEWERAGE REHABILITATION FOR POLLUTION ABATEMENT IN FIFE - A REVIEW AND CASE STUDY

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Title: SEWERAGE REHABILITATION FOR POLLUTION ABATEMENT IN FIFE - A REVIEW AND CASE STUDY

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1.0 INTRODUCTION

Fife Regional Council are currently undertaking a number of sewer rehabilitation schemes to reduce pollution in streams and coastal waters. Receiving waters range from the high amenity coast of St. Andrews Bay to the mining affected coalfield streams of south Fife. A number of techniques are being used within the Region to achieve appropriate solutions to the problems encountered including storage tanks, vortex separators and trunk sewer renewal. The rehabilitation programme underway in the Burgh of Dunfermline is discussed in depth. The primary purpose of this scheme is the reduction of the gross and offensive pollution occurring in the local watercourses. New works include the upgrading of a complex system of parallel interconnected foul and storm relief pipes and the construction of three storage tank overflows at strategic locations designed to retain the first foul flush. In addition the new works have to account for projected urban developments in the burgh. It is appropriate to examine the current techniques for the formulation of rehabilitation options by taking this system as a case study. The method has included WASSP modelling and the use of time series rainfall tank sizing. An innovative feature of the work has been the development of a rainfall time series for Central Scotland and the application of this series to the Dunfermline area.

2.0 STORM OVERFLOWS IN FIFE

2.1 POLICIES

In Fife, as in most of the country the practice of constructing combined sewerage systems has long been abandoned in favour of the completely separate system. The inheritance of combined sewers from the 1890's in the centres of the conurbations of the region has given rise to pollution problems particularly where peripheral development has been fully separated. Overflows have of course been provided to limit flows to treatment or outfalls and not only has the increased proportions of foul flow overflowing caused an increase of the overflow strength, but also over the years it has become more visible due to the increased content of paper and plastics in the sewage. This visible material is normally the prime reason for public complaint about overflows, in its own way reflecting the paradox of modern society, on the one hand expecting a cleaner world in which to live while on the other producing more waste than ever before.

The policy in Fife has until comparatively recently been to concentrate drainage capital resources of some £3 million per annum on providing treatment for up to Formula A (1) flows on all discharges. At the time of writing there are still some major untreated discharges to tidal waters for which resources have been inadequate and which are causing concern. Levenmouth is a major example where some £25 million is required to treat sewage from an equivalent population of some 250,000. However provision has also been made in the Region's capital bidget for increased spending on sewer improvements, particularly in the Dunfermline area. Over the next few years the environmental improvements will become increasingly noticeable.

All discharges, including those from stormwater overflows and surface water sewers must meet the quality standards set by the Forth and Tay River Purification Boards (RPB's) under the terms of the Control of Pollution Act 1974. The RPB's do not set blanket standards for each category of discharge because of the widely varying nature of the receiving waters in Fife. It is normal for minimum acceptable standards to be set in formal consents after consultation. In practise, to make the best use of the available sewers as economically as possible, the application of Formula A has been to overflow excess combined sewage to watercourses as far up the system as possible.

2.2 COASTAL SEWERAGE

The geography of Fife, "the Kingdom betwixt Forth and Tay" may be summmed up as being densely populated along the Forth shore, less so along the Tay, and sparsely in the centre. The exception to this generality are the dense concentrations of population in the mining and industrial heartland of Central and West Fife. In the coastal burghs the drainage systems are relatively short, consisting typically of several discrete catchments each culminating in its own outfall. Storm overflows in these systems are few and when constructing intercepting sewers and pumping stations the practise has been to retain the existing outfalls as overflows. In these cases, automatically raked screens together with stilling ponds and suitable baffles are generally provided. At two such pumping stations, Kincardine and Tayport Links, where the outfalls cannot be extended below the low tide mark, detention tanks have been provided. These tanks have been sized according to the recommendations of the Working Party on Storm Sewage (Scotland) in order to delay the onset of overflow and to retain the first foul flush.

St. Andrews is the exception to the norm as far as the coastal burghs are concerned because the sewage from practically the whole town either gravitates or is pumped into one main sewer following the course of the Kinness Burn, along which there is a proliferation of unsatisfactory overflows. These result from over extension of the system to serve development in the hinterland. The whole area alongside the Kinness Burn is of extremely high amenity as is the coastline where there are two very attractive beaches suitable for a range of watersports. However, only the west beach is registered as a bathing beach, the east beach, while being the more popular and attractive for swimming, cannot meet the bacteriological standards laid down in the register. Although the Burn supports trout, the visible signs of pollution have proved unacceptable and there is obviously a need to prevent any faecal contamination. Extensive further development is in the offing and a storm relief sewer scheme is being implemented to remove the overflows and to provide a detention tank instead.

2.3 STORAGE OVERFLOWS IN FIFE

In the landward area there are countless storm overflows, many causing little offence, but some of which require major improvement. The main focus of attention by the Region lately has been the communities of Central and West Fife from Lochgelly to Dunfermline where there are extensive and related problems of surcharged sewers, unsatisfactory overflows and mining settlement. Pollution is particularly severe where the headwaters of the receiving burns disappear into old mines as at Dunfermline (2). A provision of some £1.6 million per annum has been made in the drainage capital programme both to deal with the problems and to provide capacity for the further development planned for the area.

Several schemes are in progress involving improved overflows which incorporate storage as shown in Table 1. Storage overflows have the great advantage that, providing they are of sufficient capacity, most overflow occurences can be kept within the system and the first foul flush is always retained. Different configurations have been chosen on the various schemes and their relative merits are to be evaluated in a study by Dundee Institute of Technology. In Dunfermline, computer modelling techniques have been used to develop rehabilitation options (3), the type of arrangement favoured has been a conventional overflow structure feeding a compartmented detention tank. Figure 1 shows the main sewerage system in Dunfermline, while Figure 2 is the general arrangement of a typical compartmented tank. The first compartment of each is 'blind' to contain the first foul flush and has no overflow weir. The remaining compartments fill up in turn via stepped weirs, and when full, act as parallel settlement tanks with outlet weirs at the same level, overflowing to the receiving burn. Three detention tanks are proposed in the rehabilitation programme for this system. The capacity of the first, at Broomhead, was chosen from Table 5 of the Working Party Report (1), however a rainfall time series has been developed for the area and this was used as input to a WASSP model in order to determine the Towerburn tank capacity (see Section 4).

The techniques used in the development of the Dunfermline rehabilitation options are discussed in later sections, however it is appropriate to highlight some different aspects of other schemes. No screening has been installed at Dunfermline in conformity with the latest recommendations (4), there being sufficient baffle and scum boards to prevent the discharge of floating solids. At Oakley on the Bluther Burn a tank overflow incorporates automatically raked storm screens mounted on the crest of the weirs feeding the detention tanks, the screenings being returned to the foul flow. At Lochgelly, two proprietry hydrodynamic separators have been installed. The rotating flow in these devices provides some degree of treatment which allows much less storage volume to be installed in comparison with conventional tanks. The monitoring programme referred to earlier includes these separators and the results of the study will be available in late 1989.

3.0 A LOCAL RAINFALL TIME SERIES

Rainfall time series, suitable for use with the current sewer simulation techniques such as WASSP and WALLRUS have been developed for a number of UK regions (5). These series have received fairly widespread use in drainage area studies over the last few years for investigations into storm overflow characteristics and upgrading. Other applications have included, amongst others the design of detention tanks and headworks storage for coastal sewerage.

However, a major criticism of the existing rainfall time series developed for use with WASSP has been that the data from which the series were derived was only available for a few locations.

To provide more applicable data for eastern Scotland a series was developed using data from a gauge located at Falkirk. Charts from a syphon recording raingauge at this location had been converted into one minute interval rainfall data similar to that used in earlier studies. Data from other sites were considered, the nearest being at Bishop Auckland and Newcastle. However, it was felt that, although the rainfall variation would probably be similar for all three sites, they were so distant that the resulting series would be little better than the original north east region series applied unmodified to Scotland.

The basic data from Falkirk was filtered, giving a statistically reliable 'average' year from from which rainfall events can be selected. This series is in effect an east of Scotland series which can form the basis for series at other locations within this area. It was transferred to Dunfermline (approx. 30km away) by using a simple ratio of SAAR values for the two sites.

The full 'average' year contains innumerable trivial quantities of rainfall of which only a relatively limited number form significant events. The depth and intensity criteria for separation of events will be determined by the use to which the series is to be put. The particular requirement in this case was for storm pollutant retention tanks and events were extracted by applying the following rules:

Minimum intensity	1.50mm/h
Minimum depth	2.00mm
Time between events	25 min.

These criteria resulted in identification of 22 events for Dunfermline. For micro-WASSP however, this number of events would result in undue run times and a PCD file was prepared using the 10 greatest rainfalls (see Table 2). Some modification of the data was necessary to reduce the duration of the events to 480 minutes, a restriction of the version of micro-WASSP used in this study. Standard equations for API5 and UCWI were applied, with the series data being used for the former and monthly average Dunfermline SMD values for the latter.

The 'average' concept year described above has gained wide acceptance in the UK and an example of its use at Dunfermline is described in 4.2. Further work is also continuing, particularly at WRc Engineering. The series developed for Falkirk is much more appropriate locally than other series, but it still suffers from requiring highly accurate minutely rainfall from one gauge which may or may not represent local conditions. To circumvent this problem, a statistical technique is under development at WRc for the production of minutely sereis from hourly data. The availability of hourly data nationally is good, allowing much more local rainfall data to be used. A second development if the derivation of historic storms which have reliable return periods from long rainfall records. These storms are appropriate for storage studies where, for example, overflow might only be tolerated once every five years. Currently this type of analysis is being applied to storm overflows discharging to the sea on the south coast of England, but would be appropriate wherever there are bathing beaches such as at St. Andrews.

4.0 DUNFERMLINE - A CASE STUDY

4.1 SEWERAGE

The Burgh of Dunfermline is in a highly advantageous location in Central Scotland and has attracted considerable industrial and commercial growth in the last decade. Expansion has been rapid, with a wide variety of development in progress or planned within the catchment area. The implications that this expansion has had on the adequacy of the drainage systems to cope have been described elsewhere (6). Major rehabilitation works are currently underway primarily for pollution control. Since the rehabilitation methods being used are dictated by the nature of the existing system, a description is called for.

Figure 1 shows an outline of the system which is composed of three major branches each named after the watercourse they follow, the Lyne, Calais and Tower Burn Sewers. A schematic diagram of the main branches with the rehabilitation options is given in Figure 3. The main Lyne Burn system consists of two parallel, interconnected pipes nominally for foul and storm flows. The original single combined sewer was duplicated in the 1960's to relieve flooding and there are approximately 20 haphazardly operating cross connection allowing flow from foul to relief and on occasions in the reverse direction. The cross connections are predominantly low side weir overflows and in at least one location, presumably to reduce costs at the time of duplication, both pipes combine for a short length. The duplication was carried out purely for the solution of a hydraulic problem with flow being very effectively diverted to the relief sewer thus avoiding surcharging of the foul pipe and the discharge of sewage above ground following rain. Flow records downstream from two connections in the Rex Park area reveal an almost steady flow during a wide variety of rainfall events confirming that the concept of flow diversion is working very well. The effect however has been to cause gross and offensive pollution of the receiving watercourse, the Lyne Burn, at Waulkmill. The Purification Board has described (7) this outfall as 'discharging foul sewage almost continuously'. Further indication that the system does not operate to current standards comes from flow records which show that the maximum inflow to the foul side of the main overflow at Bothwell Street rarely exceeds 6 x DWF - on the inlet to the overflow!

The Calais burn branch of the system has one of the smaller contributing areas, but is the source of considerable pollution due to the existence of dual manholes. These manholes originate from the time of the first separate sewrage systems. It was the policy of Dunfermline Burgh at that time to construct single manholes for both foul and surface water through which access could be gained to either pipe. The surface water pipe was supposed to have a sealed access cover. A brick wall was built between each pipe in the manholes both on properties and in the public sewers. These walls separate the two systems and will perform this function properly if the sewerage is well maintained or if storms of only lower intensity occur. However, when either pipe is blocked or if flow is sufficient to surcharge the pipes then there is regular cross flow from foul to storm systems and vice versa leading both to offensive material being discharged to the watercourses and to extra storm flow in the nominally foul system. The Calais Burn sewer does have one main low side weir overflow on its length, but this also connects with the Lyne Burn storm relief pipe.

The Tower Burn branch is in contrast relatively conventional when compared with the other main branches of the Dunfermline system. The catchment is however extremely steep over a considerable part of its length giving rise to high in-sewer velocities and very little attenuation of flood peaks. In spite of the steepness, pipe capacities are low due to small diameters and surcharging has been a regular feature of storm flows. This has led to severe pollution of the Tower Burn by discharges from manholes, a highly unsatisfactory condition as a considerable length is through public park. Furthermore the Burn dries up regularly in summer due to seepage into mineworkings and the sewage pollution can be most offensive. As a result an overflow, poorly located at the bottom of this surcharging section, rarely overflows at present due to the flow having been lost from the system upstream. Discharge from the overflow is to the Lyne Burn relief sewer, as for the other branches of this system.

4.2 REHABILITATION FOR POLLUTION ABATEMENT

Rehabilitation for this sewerage system has had three major aspects, surcharging of certain sections, the sheer complexities of the duplicated main branch which makes the identification of solutions difficult, and stream pollution. Where pipes are of insufficient capacity event to carry the flow at present, the priority must be their replacement with larger diameters. This has been done over a variety of pipe lengths, sizes being fixed following WASSP runs. Figure 4 shows the results of this type of exercise for a length of the Tower Burn branch. A WASSP model including the surcharging section had previously been verified and the model was then progressively modified by increasing the diameters of the relevant section to eliminate the surcharging problem at that location. Relaying of such overloaded sections must be carried out before the storage tanks can operate effectively.

The duplication of the main branch undoubtedly solved the surcharging at the time of construction, however current and anticipated discharge requirements dictate either that the cross connections should be abandoned or that each low side weir should be modified. An intensive programme of flow monitoring has allowed an understanding of the system's operation to be built up - Figure 5 illustrates this by showing observed and estimated flows for the peak of one storm. It is clear from this information that the cross connections are restricting the flow to the foul side of the main overflow at Bothwell Street and that several links must be abandoned. Abandonment of course can only take place if at the same time the foul sewer is relaid at a larger size, otherwise surcharging would again become a problem. At the time of writing this size had not been fixed. Essential components of this aspect of the project will be the construction of an upstream tank and the improvement or reconstruction of the existing overflow at Bothwell Street thereby effecting the reduction of pollution reaching the relief sewer.

The primary problem of pollution in a number of the local watercourses, particularly the Lyne Burn has been exacerbated by the ephemeral nature of some of the streams as a result of the flow disappearing at times into old mineworking. The solution to controlling pollution problems of this nature can only be by the use of storage. In systems where gradients are slack the unused capacity of existing pipes can frequently be sufficient, however such a relatively cheap solution has not been possible at Dunfermline where slopes are steep. Consequently storage tanks have been specified to introduce the required extra volume.

4.3 STORAGE TANK SIZING

The tanks at Tower Burn and Blacklaw Road have been sized using the 'average' year rainfall time series developed for the area.

In view of the complexities of the system, a rather blunt approach has been adopted for tank sizing. WASSP models for parts of the system above each tank have been developed and verified in the normal manner. For the Tower Burn the WASSP model was used with design storms to evaluate new pipe sizes for the surcharging section referred to above and the model for the renovated system was run with the TSR.

The 10 most severe storms in the series were used for simulation, unfortunately a tedious process on micro-WASSP. The output from WASSP was run through a short routine to determine the volume above the overflow setting for each event. This approach is very simplistic and will overestimate the volume for an online tank, but it is reasonably appropriate for offline tanks. A plot of a typical TSR output hydrograph with the tank volume required for that event is shown in Figure 6. The results of this exercise were used directly to determine the volume for the offline Tower Burn tank. Figure 7 shows the ranked volumes necessary for each for both the Tower Burn and Blacklaw tanks event and enables the influence of a variation of tank volume to be evaluated. The volume selected for the Tower Burn tank from this information was 2,500 cubic metres giving an anticipated four spills per year.

5.0 CONCLUSIONS

A variety of storm overflow types which incorporate storage are being installed in Fife. A programme of evaluation is underway in conjunction with Dundee Insitute of Technology and involving both flow and quality sampling. A rainfall time series appropriate to the east of Scotland has been developed and used for the sizing of overflow tanks on the complex Dunfermline sewerage system.

6.0 REFERENCES

6.1 Scottish Development Department; Storm Sewage: Separation and Disposal. Report of the Working Party on Storm Sewage (Scotland) HMSO 1977.

6.2 Jefferies C., Stevens G.S.W. & Ashley R.M.; A Scottish Burn. Journal of the Institution of Water Engineers & Scientists. Vol. 40 No. 6 December 1986.

6.3 Jefferies C. & Ashley R.M. The use of microcomputers for the simulation of flows in the Dunfermline Sewerage system. Proceedings of the 2nd International Conference on Civil & Structural Computing. December 1985.

6.4 Sidwick J.M. Screenings and grit in sewage: Removal, Treatment and Disposal Phase 3: Storm Water Overflows and Pumping Stations. Technical note No. 132 CIRIA 1988.

6.5 Henderson R.J. Rainfall time series for sewer system modelling. External report ER 195E WRc June 1986.

6.6 Ashley R.M., Jefferies C. & Stevens G.S.W. Overloading of the Dunfermline sewerage system due to peripheral urban expansion. Proceedings of the Centenary Conference on Infrastructure Renovation and Waste Control. Institution of Civil Engineers North Western Association April 1986.

6.7 Forth River Purification Board Annual Report 1985.

SCHEME	DATE	POPULATION	STORM FLOW (l/s)	THROUGH FLOW (l/s)	TYPE OF STRUCTURE	VOLUME (m^3)	REPLACING	RECEIVING WATERCOURSE CLASS
<u>DUNFERMLINE</u>								
Broomhead	85 - 88	5,000	700	120	Stilling Pond/ offline	400	Surcharging sewers & poor overflow	2
Tower Burn	89 - 91	16,000	2,300	450	High side weir/ offline	2,500	*Surcharging sewers & relief sewer overflow	2 above 4 below
Lyneburn	90 - 92	26,000	4,800	Not fixed	Online	Not fixed	*Complex lowside weirs to relief sewer	2 above 4 below
<u>OAKLEY</u>								
Bluther Burn	86 - 87	5,000	612	73	Low side weir/ raked screens	200	Surcharging sewers	2
Lochgelly/ Lumphinnans	88 - 89	4,800	2,500	110	Hydrodynamic separator	150	Very poor bar screen overflow	2 above 3 below

*The Dunfermline tanks overflow to a relief sewer which intercepts flow from the two tanks shown and other overflows. Stream quality given is for the Lyne Burn at Waulkmill, the discharge point of the relief sewer.

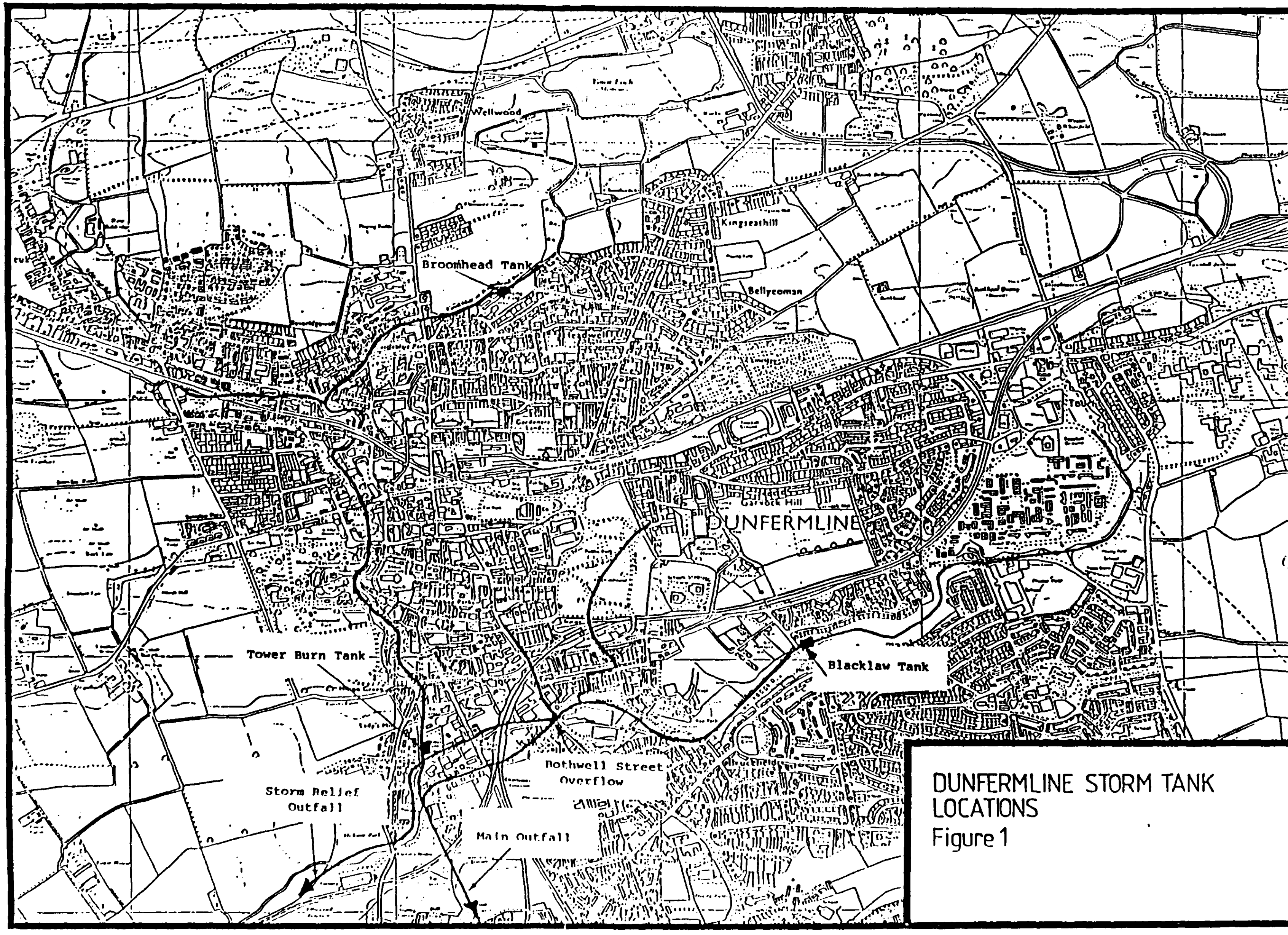
TABLE 1

TABLE 2 - TIME SERIES RAINFALL SUMMARY
DUNFERMLINE

Event No.	START DATE	START TIME (hhmm)	END DATE	END TIME (hhmm)	Duration (min)	Depth (mm)	Average Intensity (mm/h)
1	31 Mar	1 - 54	31 Mar	9 - 53	480	17.65	2.21
2	11 Jun	8 - 2	11 Jun	16 - 1	480	12.63	1.58
3	24 Aug	17 - 53	24 Aug	20 - 45	172	6.56	2.29
4	19 Sep	22 - 10	20 Sep	6 - 13	479	34.48	4.29
5	23 Sep	11 - 30	23 Sep	19 - 30	480	21.40	2.68
6	26 Sep	14 - 49	26 Sep	22 - 43	473	20.84	2.64
7	26 Sep	23 - 14	27 Sep	6 - 12	412	11.08	1.59
8	3 Oct	17 - 30	3 Oct	21 - 21	230	6.85	1.79
9	24 Oct	0 - 0	25 Oct	8 - 8	479	14.76	1.82
10	3 Nov	14 - 25	3 Nov	18 - 45	261	7.30	1.68

Events have been selected on the basis of:

Minimum intensity (mm/h) : 1.50
Minimum depth (mm) : 6.50
Time between events (min) : 25



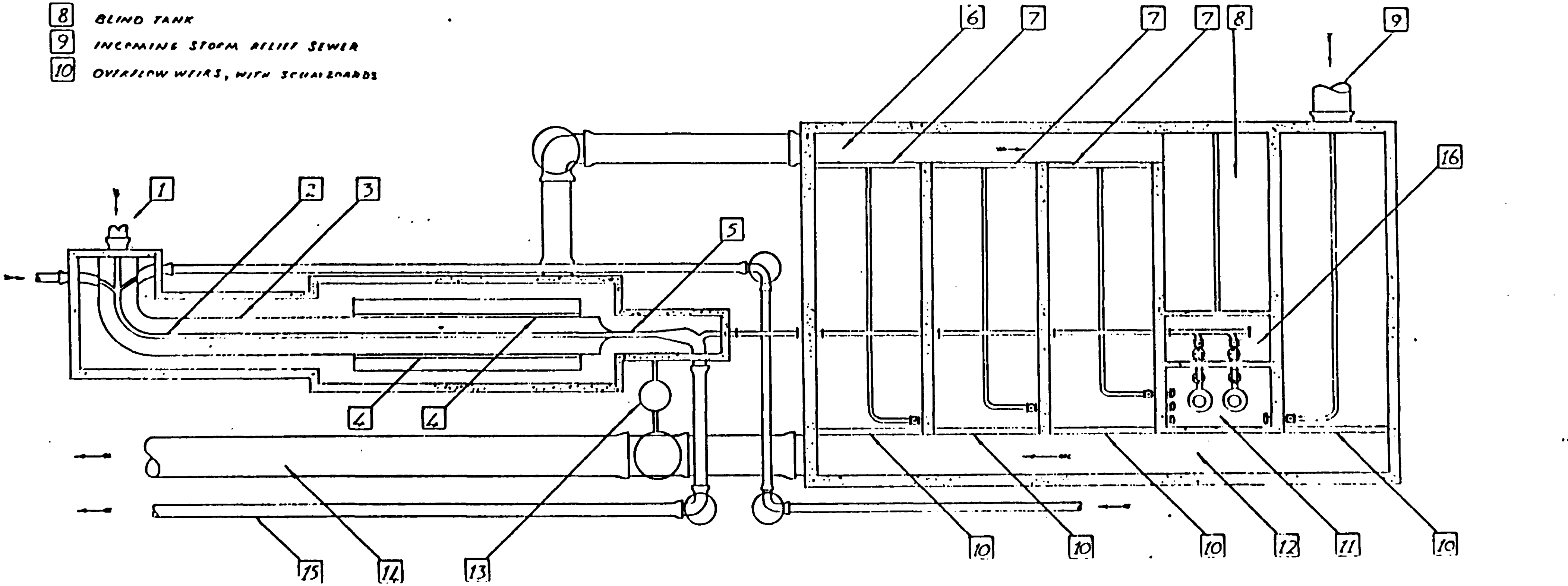
DUNFERMLINE STORM TANK LOCATIONS
Figure 1

DUNFERMLINE TRUNK DRAINAGE IMPROVEMENT SCHEME

KEY

1. INCOMING COMBINED SEWERS
2. DRY WEATHER CHANNEL
3. STORM CHANNEL
4. OVERFLOW WEIRS, "FORMULA A" SETTING
5. CONTROL FLUME
6. STORM INLET CHANNEL
7. STEPPED INLET WEIRS
8. BLIND TANK
9. INCOMING STORM RELIEF SEWER
10. OVERFLOW WEIRS, WITH SCUMBOARDS
11. PUMPS, FLAP VALVES IN SUMP IN BLIND TANK, ALL TANKS DRAIN TO SUMP VIA FLAP VALVES BUT MAY BE ISOLATED BY CLOSING SLUICE VALVES INDICATED THUS
12. STORM OUTLET CHANNEL
13. CHAMBER HOUSING AUTOMATIC SAMPLER
14. OUTGOING STORM RELIEF SEWER
15. OUTGOING FOUL SEWER
16. VALVE CHAMBER SUSPENDED ABOVE BLIND TANK.

NORTH



0 5 10 15
SCALE IN METRES

TOT. VOLUME OF DETENTION TANK = 2,500 m³

FEB. 1989

Figure 2. Storage tank overflow at T.A. centre.

G.S.W.S.

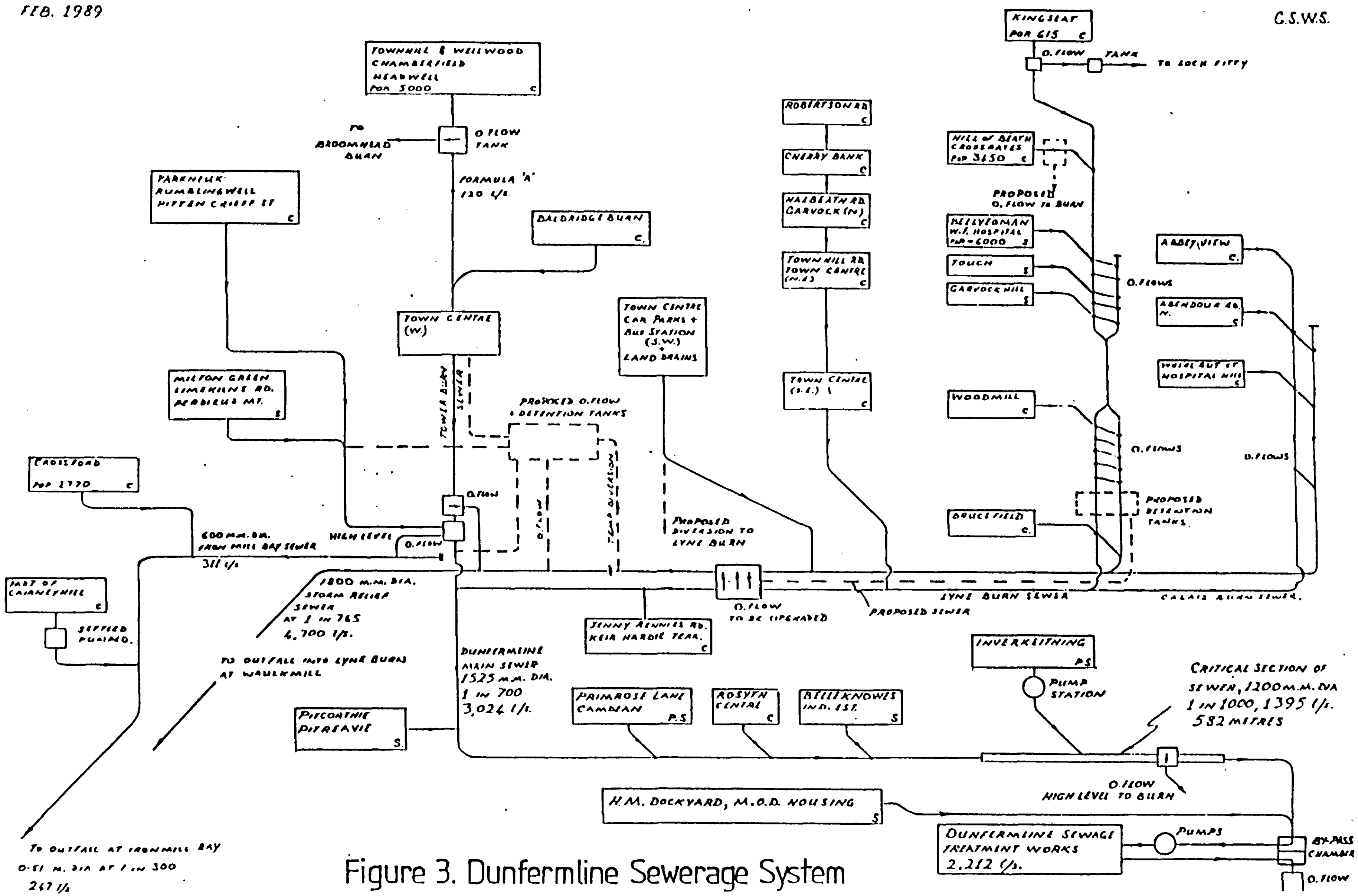


Figure 3. Dunfermline Sewerage System

Note - Diameters refer to relaid section in Tower Burn

M5 - 2hr. rainfall
Total rainfall (mm): 20.35

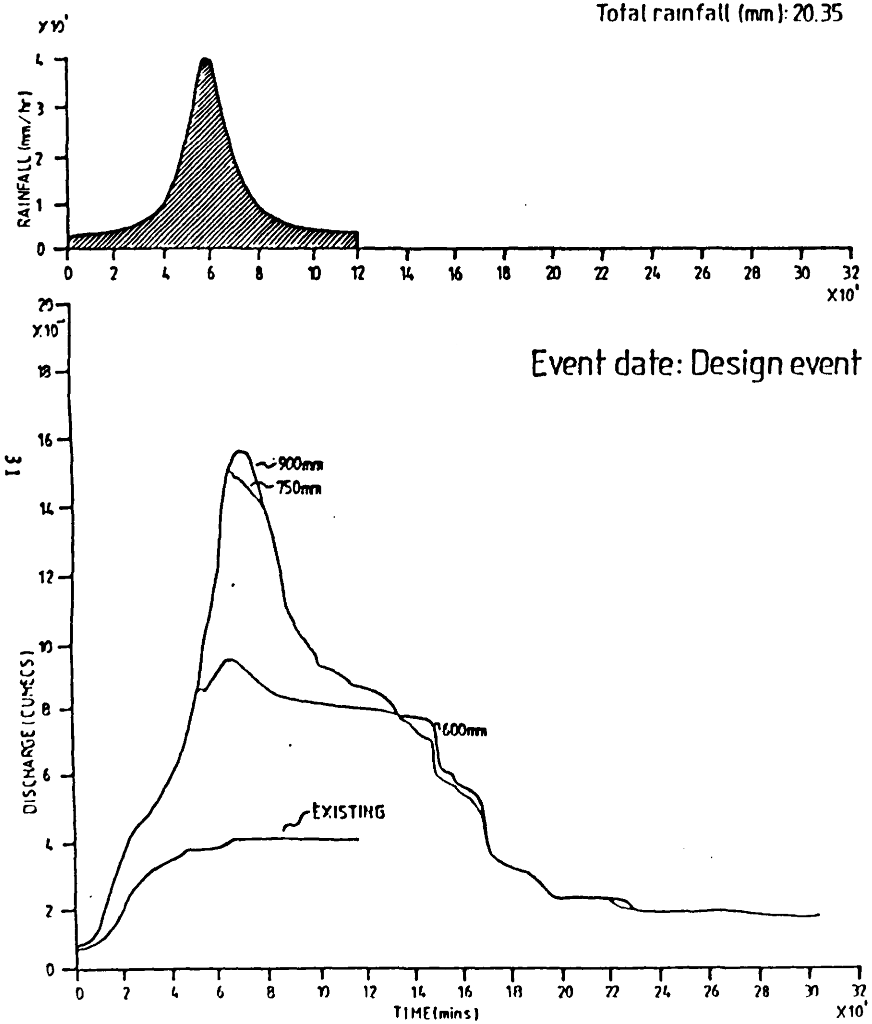


Figure 4 M5120 Design event.

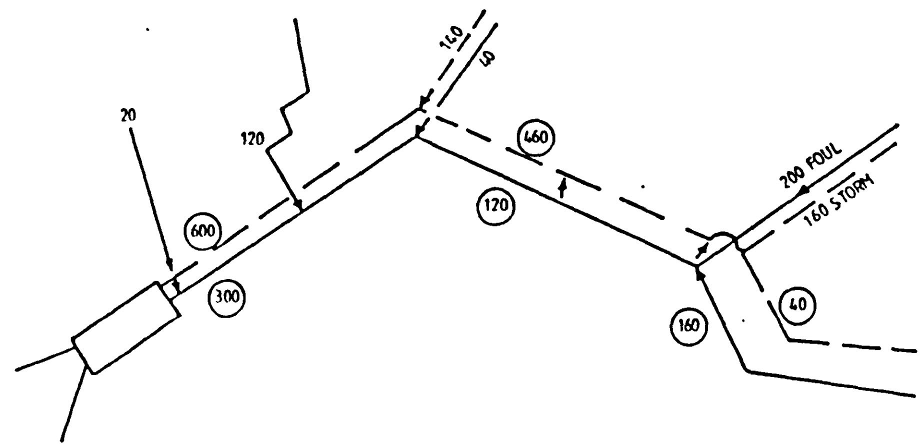


Figure 5 Recorded flows during event of 6/8/86

Note - Recorded flows are ringed.
Flows not ringed are implied from other data.
Flows are in litres / sec.

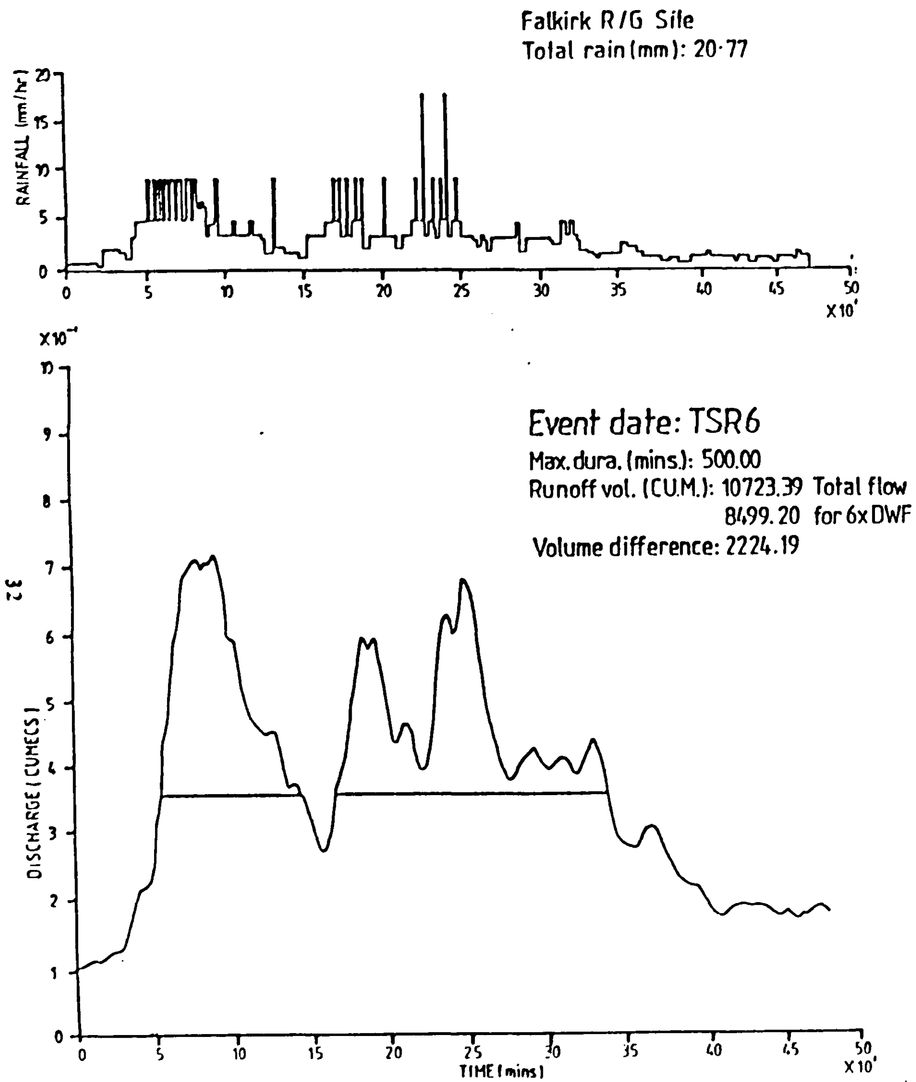


Figure 6 Typical TSR Output.

32

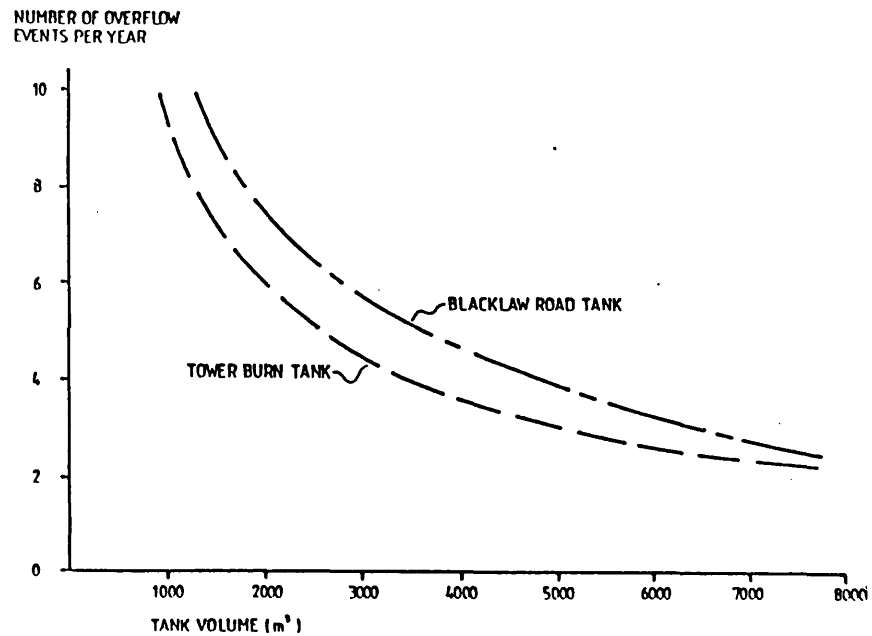


Figure 7. Tank volumes determined from time series rainfall.

Appendix F
Combined Sewer Overflow Structures
and
Performance from Model Tests

F1 COMBINED SEWER OVERFLOWS

F1.1 Overflow Setting and Control of Flows

The overflow setting is a well-established concept whereby a specific flowrate is determined such that when the inflow to the device exceeds the setting, the overflow should fill and subsequently spill. The commonest relationship used for determining the setting is known as **Formula A** as put forward by MHLG (1970). The principle embodied in **Formula A** is that of a fixed continuation flow to treatment regardless of different sewer systems and, particularly, the effect on the receiving watercourse. For smaller systems the settings which result from application of **Formula A** are close to 6xDWF and SDD (1977) reported settings ranging from 3xDWF to 6xDWF for a limited number of overflows investigated.

Procedures to assess the acceptability of intermittent discharges to a river reach are currently being developed under the aegis of the urban pollution management programme (Clifforde et al. 1990). The conclusion of the development of such procedures will be a complete rationale for the determination of overflow settings and target spill volumes thereby making **Formula A** redundant although in some studies it may be retained. Currently a simple desktop procedure is available, CARP (WRc 1988), however, all overflows studied in the research programme described here have settings based on **Formula A** and the procedure is noted for completeness.

A variety of flow control devices are used at CSOs to achieve the setting. The commonest are:

- Orifice Plate
- Venturi Flume
- Vortex Control
- Throttle Pipe
- Adjustable Penstock

The hydraulics of all of the above, apart from the vortex control are well documented in standard texts. Vortex controls are patented and discharge characteristics are supplied by the manufacturer in the form of curves as illustrated in Figure F.1.

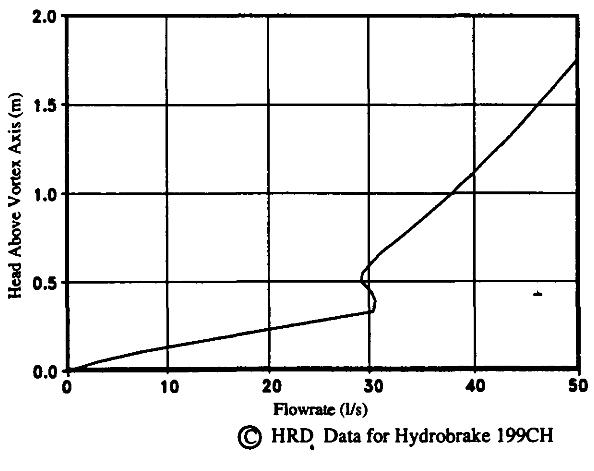
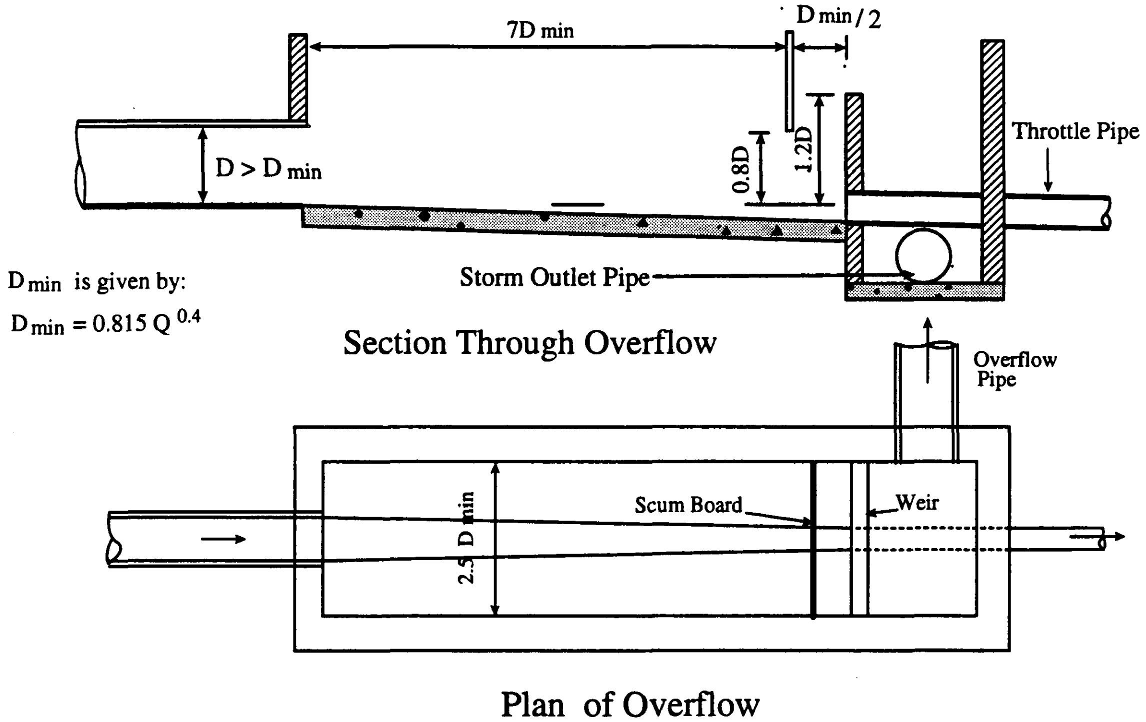


Figure F.1 Typical Hydrobrake Performance Curve

Vortex devices operate by acting as orifices at low flows. When the head increases, rotational flow occurs entrapping an air cone in the centre, thereby reducing the flow area resulting in a lower flow for a given head than with the orifice.

F.1.2 Stilling Pond Overflows

The performance of a stilling pond overflow was first examined by Sharpe & Kirkbride (1959) initially using paper, wood and sand as particulate matter. The initial tests showed that three basic flow patterns occurred. In the most efficient pattern, currents prevented floating matter from reaching the weir during overflow until the water level subsided. More rigorous testing made use of beads of diameter 9.5, 12.5 and 16mm with specific gravity (SG) varying between 0.89 and 1.21. In later experiments attention was concentrated on floating particles, since most sinkers were found to pass directly to the throttle pipe.



Dimensions shown are as recommended by Balmforth & Henderson (1988)

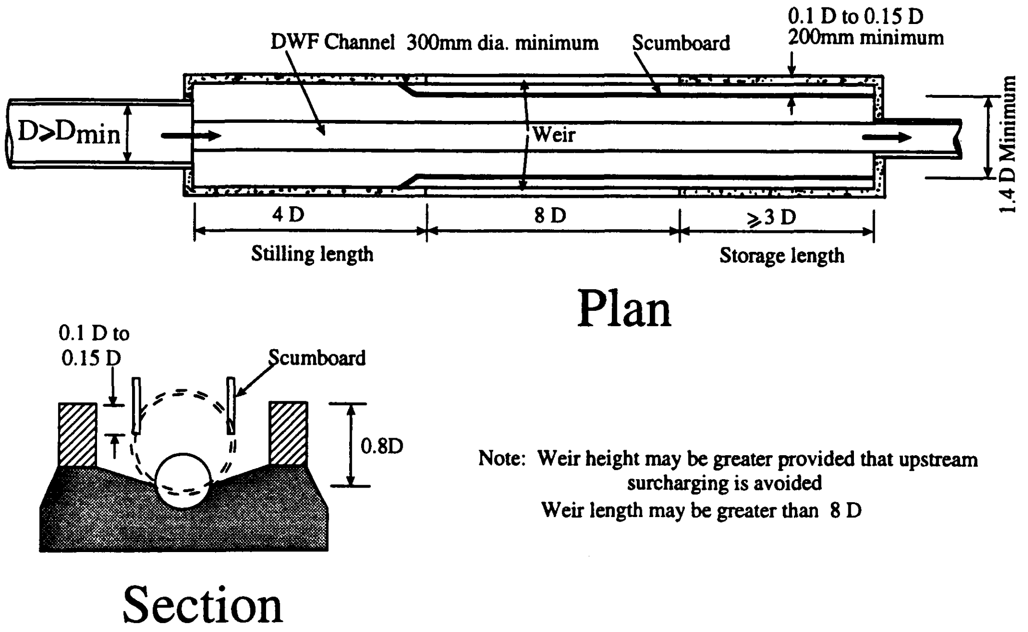
Figure F.2 Extended Stilling Pond Overflow

Sharpe & Kirkbride presented recommendations for stilling pond design and further development with different chamber geometry was carried out by Frederick & Markland (1967) and Reddy & Pickford (1972). These tests resulted in more rational dimensions, together with position of baffles for improved performance under varying flow conditions. Balmforth (1982), found that the previous work was inconclusive and presented the results of tests which allowed the location of the scumboard to be specified together with the weir height. The length of the chamber was increased from $4.5D_{min}$ to $7D_{min}$ and the name was changed to the extended stilling pond overflow. His development of the standard Sharpe & Kirkbride chamber forms the basis of the current recommendations for the design of stilling pond CSOs. In particular it was found that nothing was to be gained by raising the weir height above 1.2D as this ensured sufficient depth to submerge the inlet pipe and create the flow patterns recommended by Sharpe & Kirkbride. Figure F.2 shows recommended dimensions as published in Balmforth & Henderson (1988).

F.1.3 High-Side Weir Overflows

The high side weir overflow has not historically been well thought of. Writing in 1978, White condemned it without any supporting argument as being '..unlikely to be adopted except where it is quite impossible to arrange for a drop in sewer level through the chamber.' (White 1978). This design evolved from earlier low side weir designs which were subsequently demonstrated by Balmforth & Sarginson (1983) to have inadequate flow control and a roller action which mixes polluting solids in the flow.

Later designs have their weir set higher than the incoming half pipe diameter and, as with stilling ponds, are fitted with a flow control to ensure the correct setting is achieved. It is currently recommended (Balmforth & Henderson 1988) that double weirs are used and that first flush storage is incorporated downstream from the weirs as shown in Figure F.3.



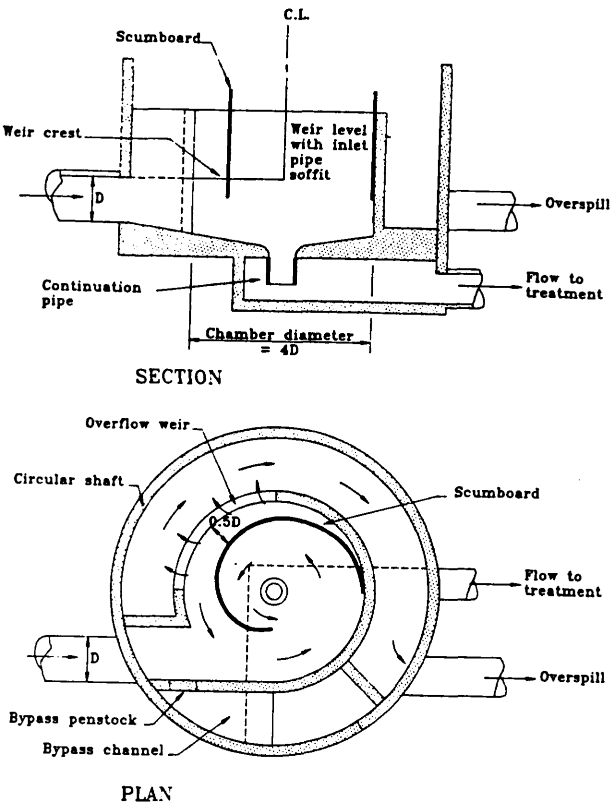
Dimensions shown are as recommended by Balmforth & Henderson (1988)

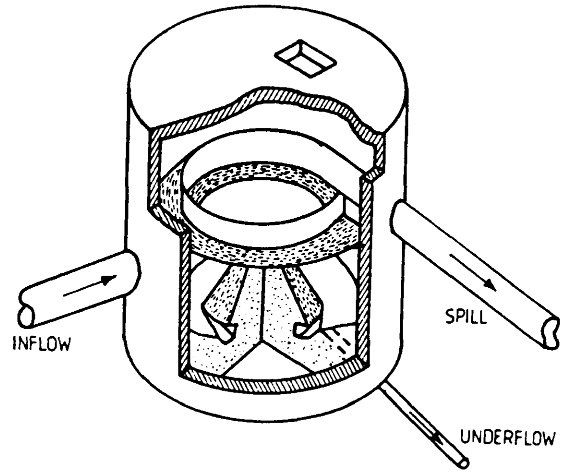
Figure F.3 High Side Weir Overflow

The length of weir has most commonly been determined by the method of De Marchi (Balmforth & Sarginson 1983) give an interpretation of the method) while, more recently, Delo & Saul (1989) have presented a graphical method of determining the required length. The weir length does not appear to be critical provided it is greater than eight times the incoming pipe diameter as indicated in Figure F.3.

F.1.4 Vortex Overflows

The vortex overflow was originated by Smisson (1967) to solve a purely practical problem where efficient hydraulic performance was required in a very small space. The original central drop shaft concept was changed by Balmforth et al (1987) to give better retention of solids by utilising peripheral spill with the flow to treatment from the centre where solids naturally congregate.





Source : Balmforth & Henderson (1988)

(a) Vortex Overflow

Source : Hydro Research and Development (1990)

(b) Hydrodynamic Separator

Figure F.4 Vortex Overflows

The basic dimensions of the vortex overflow are included as Figure 2.7(a). Higher efficiencies were claimed than for a stilling pond of equal volume, and the salient features are the spill weir located in the third quadrant and a spiral scumboard.

The vortex overflow is currently being further developed commercially both in the UK (Sorensen & Larsen 1990) and Germany (Brombach 1990). The two designs are similar, having a submerged plate to retain floating solids in place of a simple dip plate; a central cone around which secondary vortices form, giving rise to the name **hydrodynamic separator;** and a tangential bottom drain. It is claimed that the secondary vortices, created by the shaped cone, trap a greater number of suspended particles than would be the case in unmodified rotational flow.

Smisson's original vortex work was extended in the US where it was named the swirl concentrator by Field (1974) who produced a comprehensive design manual. A central overflow weir was retained, but additional internal plates were added to form a floatables trap and a spoiler. The concept of the swirl concentrator was flawed by having these internal protrusions which created turbulence within the vortex, thus retaining solids within the flow and reducing separation efficiency. An extensive programme of field testing was initiated but it was also flawed by poor instrumentation and a design giving excessive tank volumes. Reports on this testing by Pisano (1988 & 1990) show the work to have been inconclusive.

F.2 EFFICIENCY OF POLLUTANT SEPARATION FROM MODEL TESTS

Rigorous comparison of the performance of hydraulic structures requires repetitious measurements to be made using consistent methods of measurement. A large number of variables influence the operation of a CSO during storm conditions and great difficulty is normally experienced obtaining relevant data from field performance tests. It is almost impossible to express results from field studies with any degree of conciseness. Most performance studies have relied as a result on model testing to reduce the number of variables although, even with the most rigorous testing, results and techniques used can be misleading (Halliwell & Saul 1980).

The principal virtue of model testing, that of repetitive operation with each overflow configuration, is extremely attractive. In contrast, observations from field installations are sparse and rely on a limited number of monitored events. Consequently the number of long-term studies of the performance of CSOs is limited (Pisano 1988, Thornton & Saul 1986, Veenhuis et al 1988, Hedges & Lockley 1992). A variety of expressions are used when defining performance, most of which are different definitions of efficiency expressed as a ratio of, in a general sense, inputs and outputs. Published data on efficiency have until recently only been from model tests.

The efficiency of an operating overflow is affected by;

- temporal variation of flowrate;
- temporal variation of pollutant load;
- rate of continuation flow which may be near constant;
- physical dimensions including storage;
- storage volume within overflow and sewer network;and,
- the pollutant separation characteristics of the device.

The methods of expressing CSO efficiencies using model results, where steady state flows are the norm, are quite different from field installations. Consideration is given in this section to steady state testing due to the reliance placed on laboratory work in the development of most devices.

F.2.1 Particulate Material in Steady State Testing

Particulate matter has been modelled by most researchers using near spherical particles with appropriately modified buoyancies. Attempts were made by Sharpe & Kirkbride (1959) to use paper material, however they found that the results could only be used qualitatively. Froude scaling is used for overflow modelling since flows are predominantly free surface with particle size $\leq 0.07D$, where D is the diameter of the inlet pipe (Halliwell & Saul 1980), to avoid excessive scale effects. The above authors have also highlighted the importance of interpreting the results in terms of settling velocities as an alternative to relative density, terms which only relate directly for geometrically identical particles.

Various approaches have been used to determine efficiency. Frederick & Markland (1967), concentrating on floating solids, utilised mainly spherical particles of relative density between 0.94 and 1.00. Halliwell & Saul (1980) employed spherical wooden and plastic beads with relative density both greater and less than unity. The beads were inserted in groups with generally similar characteristics, and also as individual particles introduced at least 100 times. This latter method, also used by Sorensen & Larsen (1990), eliminates differences between the particulates and was found to reduce the scatter of results.

The technique used by Balmforth (1982) was essentially similar but involved ten different types of chips and spheres with terminal velocities ranging from 138.8 mm/s (rise) to 173.7mm/s (fall). More recently Balmforth (1990) reported inconclusive tests on mixtures of particles which had a distribution of settling velocities to represent storm sewage. It can generally be concluded that laboratory studies of CSO chambers have all used similar particulate matter and insertion techniques and that reliability of results increases with the number of insertions of particles.

F.2.2 Interpretation of Steady State Testing

Efficiency at steady state flows is defined as the percentage of particles which do not pass over the overflow. Most experimenters, having carried out tests using particulates with a range of settling velocities, have expressed results as a series of efficiency curves such as those in Figures F.4(a&b) and F.5(a-c). The curves show characteristically high efficiencies for material with high rise/fall velocities, while, as the terminal velocity approaches zero, the performance drops dramatically.

Most authors have expressed the abcissa in terms of $\mathbf{w/u_o}$ where:

$\mathbf{w}$ = terminal velocity of particle;and,

$\mathbf{u_o}$ = mean velocity in inlet pipe.

The variability of the size and shape of sewage particles have presented modelling problems. SDD (1977) compared different examples of storm sewage and Balmforth (1982) used discrete particles of various shapes. Burrows & Ali (1982) suggested that the different settling characteristics should be represented by using the term $\mathbf{C_d^{1/2}w/u_o}$ where $\mathbf{C_d}$ is the drag coefficient of the particle under motion at terminal velocity. Burrows & Ali suggested that $\mathbf{C_d}$ should be determined from settling velocity tests, however, this concept has not been used by other workers. Crushed olive stone has been recommended (Saul & Ellis 1990) as representing cohesive sewage solids, and, to model gross solids, full scale material is now being used (Ruff & Saul 1992).

Halliwell & Saul (1980) have demonstrated the difficulties of interpreting results, laying particular emphasis on constant entry conditions by utilising the same inlet pipe in all tests. In contrast Sorensen & Larsen (1990) attempted to achieve similarity with vortex chambers by using identical volumes. In practical modelling, such external impacts are relatively easily reduced by retaining common features, since costs are also reduced. Provided external influences are indeed minimised, modelling under Froude's criteria should produce results which are not model specific.

The flow ratio $\mathbf{q/Q}$ remains to be addressed, where:

$\mathbf{q}$ = continuation flow, and;

$\mathbf{Q}$ = inflow.

As $\mathbf{q/Q}$ increases, less flow and fewer particles reach the overflow.

The ratio **q/Q** is extremely important in the determination of prototype performance from field tests and depends upon catchment flows in addition to overflow settings. It ranges typically between unity as overflow commences and 0.25 at peak flowrate, while several authors have selected 0.16 to express results as this has been considered to be representative of UK practice, however, many efficiency curves omit mention of **q/Q**. The flow ratio is incorporated in recent hydraulic design curves for high-side weirs by Delo & Saul (1989).

F.2.3 Model Efficiencies based on Steady State Testing

Principal efficiency results from models of CSO devices currently recommended (F.1.2 to F.1.4) have been reported by Nicoll & M^c^Gillivray (1977), whose results are reproduced in Figure F.4 (a).

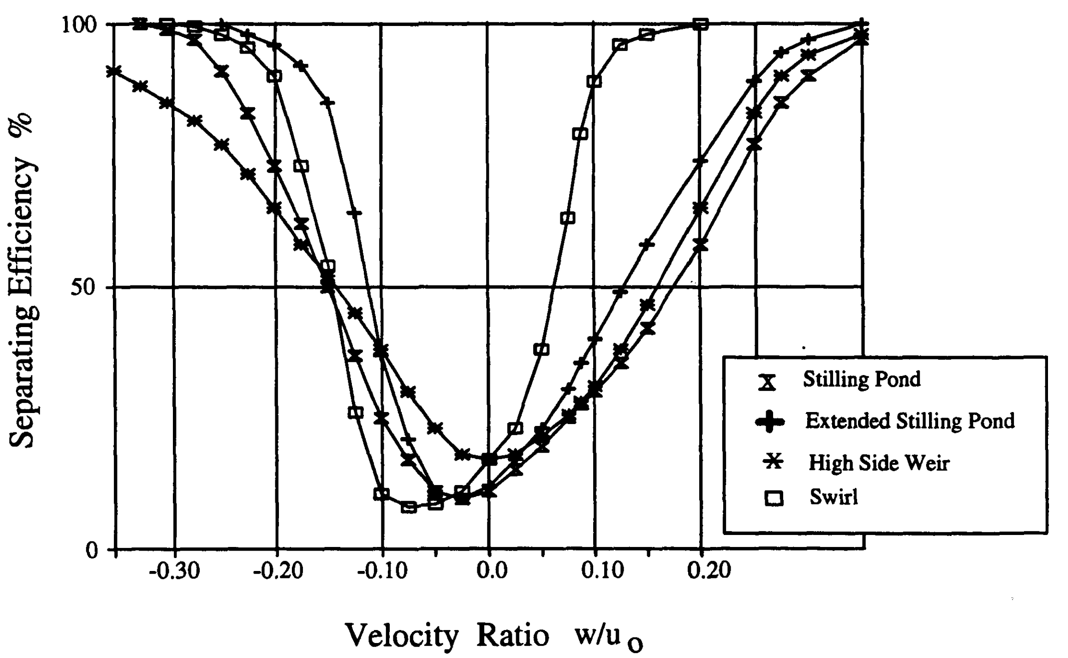


Figure F.4 (a) Combined Sewer Overflow Efficiencies From Nicoll & M^c^Gillivray (1977)

Further work was reported by Balmforth (1986) who presented the same data on the high-side weir overflow as the above authors. Balmforth's curves are reproduced in Figure F.4 (b) along with the efficiency for a high-side weir with storage reported by Crabtree et al (1991).

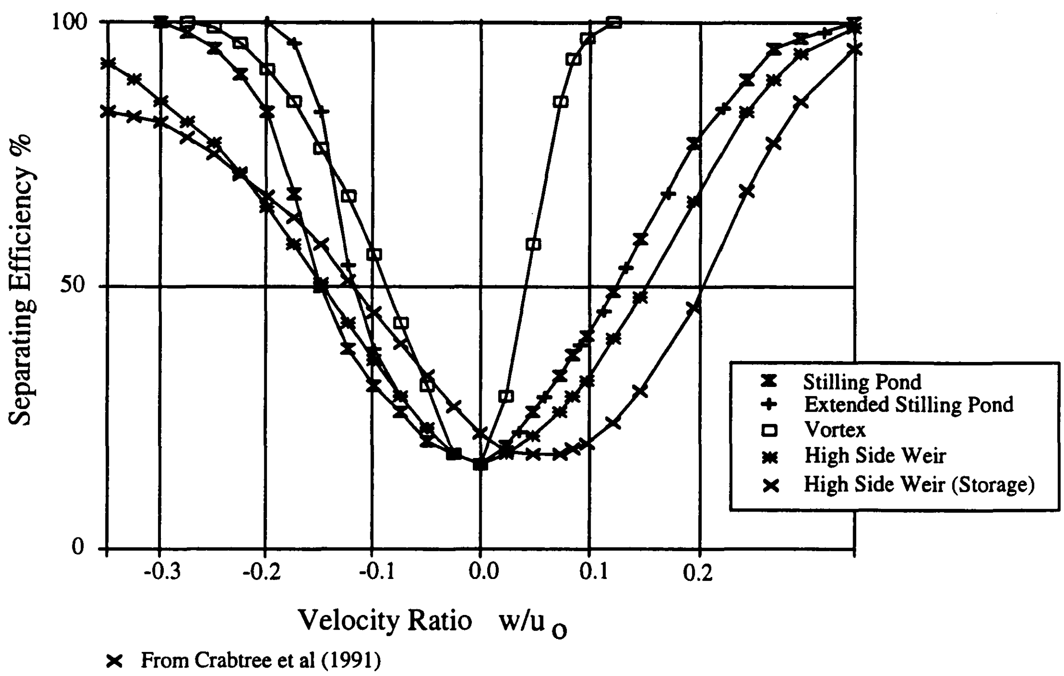


Figure F.4(b) Combined Sewer Overflow Efficiencies From Balmforth (1986)

The data in Figure F.4(b) reflect differing values of **q/Q**. Nicoll & M[c]Gillivray presented all efficiencies for the common ratio of **q/Q** = 0.156, whereas Balmforth's presentation relates partly to the same **q/Q** ratio, but also to **q/Q** = 0.2 for stilling pond and vortex. The impact of this difference must be to cause an apparent improvement of efficiency and is considered to be significant with regard to the small differences in efficiencies between the devices.

The general observation is made (Balmforth 1990) that the vortex devices are significantly more efficient than stilling ponds & high-side weirs, particularly for falling particles. The Balmforth vortex is itself an improvement on the swirl as demonstrated by Figure F.5 (a), particularly for low rise velocity material. Recent work on the Hydrodynamic Separator by Hedges & Lockley (1992) has shown very high efficiencies for settling and a relatively good performance for neutrally buoyant particles, although the performance for floating material was very poor. Their data, reproduced in Figure F.5 (a) show the separator to have the best efficiency although the storage volume is also higher than conventional overflows. Their results also suggested that removal of slow rise velocity particles was poor with the device acting as a flow split.

In contrast to Hedges & Lockley's work, tests carried out by Sorensen & Larsen (1990) showed the separator to be the poorer design, particularly for floating solids, although the model used was suspect. These authors concluded that overflow efficiency can easily be spoiled by poor construction.

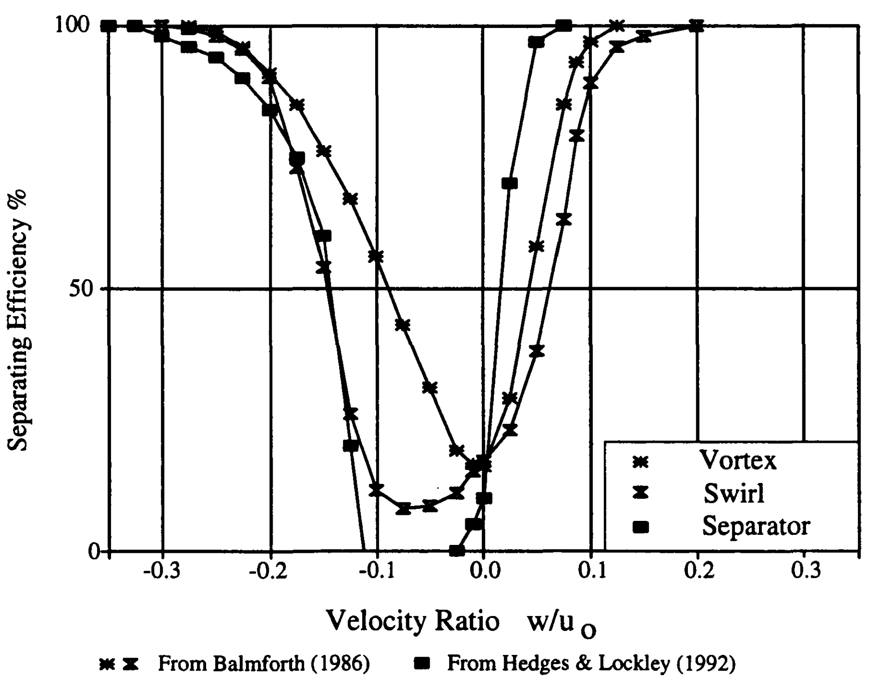


Figure F.5(a) Comparison of swirl and vortex overflows

Nicoll & McGillivray report as shown in Figure F.4(a) that the extended stilling pond (ESP) has a maximum efficiency 35% greater than the stilling pond (SP) on rise and 15% greater on fall particles. A comparison with Balmforth's data indicates a similar difference for rise, and only a 10% improvement on falling particulates. Balmforth shows the SP to be little different from the HSW for most rise particles, and identical with the ESP for fall particles. Enhancements reported by Balmforth for the ESP show a maximum improvement on efficiency for $\mathbf{w/u_o}$ between 0% and 10%. Otherwise there is no difference in the performance of the two devices and it is suspected that Balmforth has reported the same data. Similar improvements are reported for the stilling pond for slow rise and a relatively constant improvement of approximately 15% for fall particles as shown in Figure F.5 (b)

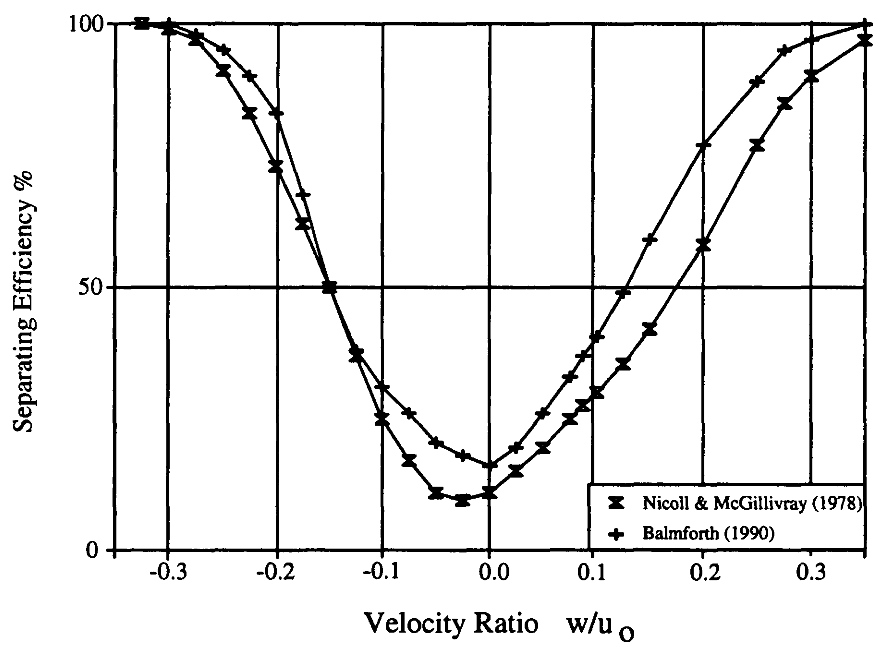


Figure F.5 (b) Comparison of stilling pond overflows

The minimum efficiencies reported occur for neutrally buoyant and slightly rising particles and would not be expected to differ significantly from the **q/Q** ratio. Nicoll & McGillivray reported minima of 18%, 10%, and 10% on high side weir, stilling pond and swirl respectively, the latter efficiencies being poorer than the **q/Q** ratio. Balmforth

(1990) reports all minima as 20%, consistent with a flow split of this amount.

The impact of variation in the **q/Q** ratio has been reported in most studies and an example is given in Figure F.5 (c) using data from SDD (1977) which is an amplification of the data reported by Nicoll & M^{c}Gillivray. The doubling of relative throughflow produces an average improvement of 15% and 40% on rise and fall particulate respectively, although the improvement is limited to 10% for low rise velocity particles. Halliwell and Saul (1980) present a detailed discussion on this subject, although based on the same research programme. It was concluded that '..it is difficult to justify the much longer and more expensive test programme needed to produce the general set of curves to define (the full problem) completely.' In view of the highly variable inflows to prototypes, the failure of research programmes to carry out complete model test programmes is a significant flaw.

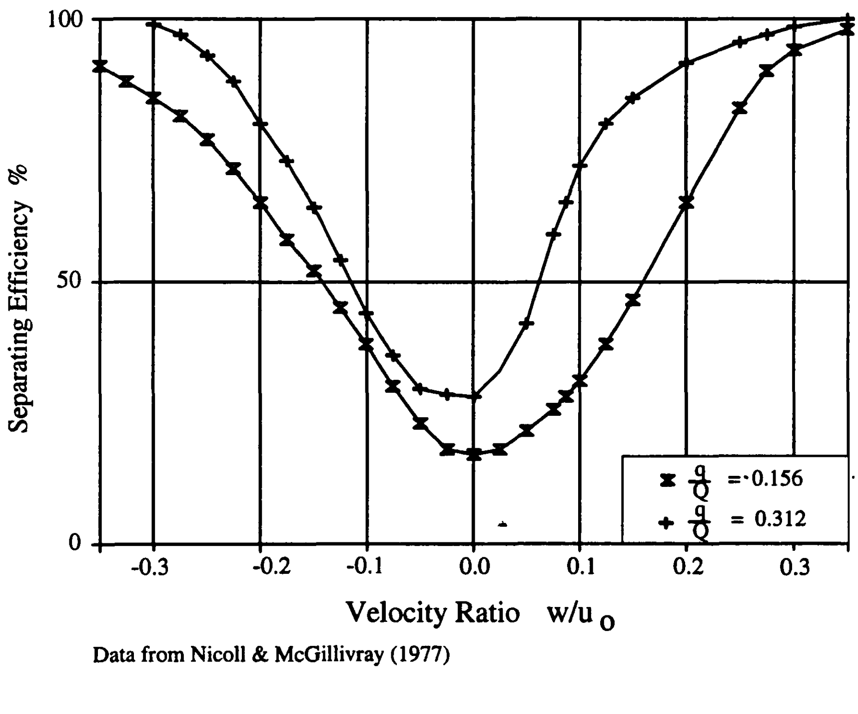


Figure F.5 (c) Variation of Inflow Ratio q/Q

The conclusions from this review of published efficiency data are;

(i) Poorer efficiencies are reported for low rates of rise than low fall particles although improved devices reported by Balmforth show smaller such differences. The hydrodynamic separator has poor performance for rise particles.

(ii) The minimum efficiency always occurs for material with low rise/fall velocity and depends upon the **q/Q** ratio chosen in each study.

(iii) There is little difference in reported efficiency between the high side weir and stilling pond overflows.

(iv) The vortex, swirl and separator overflows have greater efficiencies, particularly for falling material.

(v) The flow ratio **q/Q** has an effect on device efficiencies which exceeds all others

The importance of the flow ratio **q/Q** highlights the difficulty of expressing the performance of an intermittently operating system by steady state modelling, thus taking no account of the behaviour of the material retained within storage. As an example, a 50% change in the **q/Q** ratio produces a similar difference in efficiencies to the total spread of all the published efficiency curves. It may be concluded that prototype CSO devices with variable inflow rates must have radically different efficiencies from those published, which have been based on steady flow model tests.

F.2.4 Models with Unsteady Flows

Steady state models fail to allow representation of the temporal variation of either flows or pollutant concentrations. The variation of the flow ratio **q/Q** has been highlighted, as has the first foul flush (FFF). Additionally, since any storage volume produces settlement of particles, variations in the amount of stored pollutants must occur during unsteady conditions. The deficiency of steady state modelling in representing these variables is well known but the problems of controlling the variations of flows and concentrations have until recently been too great for satisfactory unsteady flow modelling. Ackers et al (1967) for example recognised the difficulties and produced some empirical rules for the retention of FFF, at the same time failing to substantiate their recommendations. Saul & Delo (1982) used particulate matter similar to that used in steady state testing and lower efficiencies were found, particularly for lower settling/rise velocity particles. It was concluded that steady state results were optimistic.

Some automatic controls for unsteady flow were developed to support research into the factors which affect the self cleansing characteristics of storage tanks (Saul & Ellis 1990) and the use of crushed olive stone has been confirmed as representing cohesive sediment in models. A significant recent advance has been the development of a variable flow and concentration control system for the input and monitoring of laboratory sediments at Sheffield University (Saul et al 1992). This type of system is essential if the degree of repeatability required in modelling work is to be achieved, as it allows any desired concentration to be provided automatically. Results are available only for sample variations of concentration with time and no investigations of the performance of different storage arrangements have yet been published.

APPENDIX G FLOW MONITOR CALIBRATIONS

G.1 Laboratory Depth Calibrations

Laboratory calibrations of depth and velocity were carried out each time a monitor was removed from site, provided time was available. The procedure for checking depth was to record a range of logged and measured water depths before and after workshop maintenance. Where possible the depth was adjusted to minimise both zero and span error, although the older equipment used (Monitors 591 & 592: both purchased in 1985) had no adjustment facility. The accuracy in level measurement is expressed as the difference from the exact depth in Figure G.1.

Depth measurement did not suffer unduly from drift of range with only one set of data (ID 591 on 26/2/91) exhibiting more than 10mm range drift. Zero drift was more frequent and in the case of two tests it was extreme, at greater than 50mm (ID 359 before 25/2/91 and ID 591 on 20/9/89). In practice these two tests were symptomatic of logger malfunction and all data were rejected for the relevant periods, only partly on the basis of these extreme calibration results. In consequence, the accuracies discussed later exclude data which were dependent on these two tests.

The results expressed in Figure G.1 show that the zero value drifted low on five of the nine relevant tests, while three showed less than 5mm drift in either direction. After each set was averaged the maximum drift was -34.5mm, while the average was -4.5mm. More indicative was the average of the absolute drift values, which showed a zero drift of ± 6.7mm.

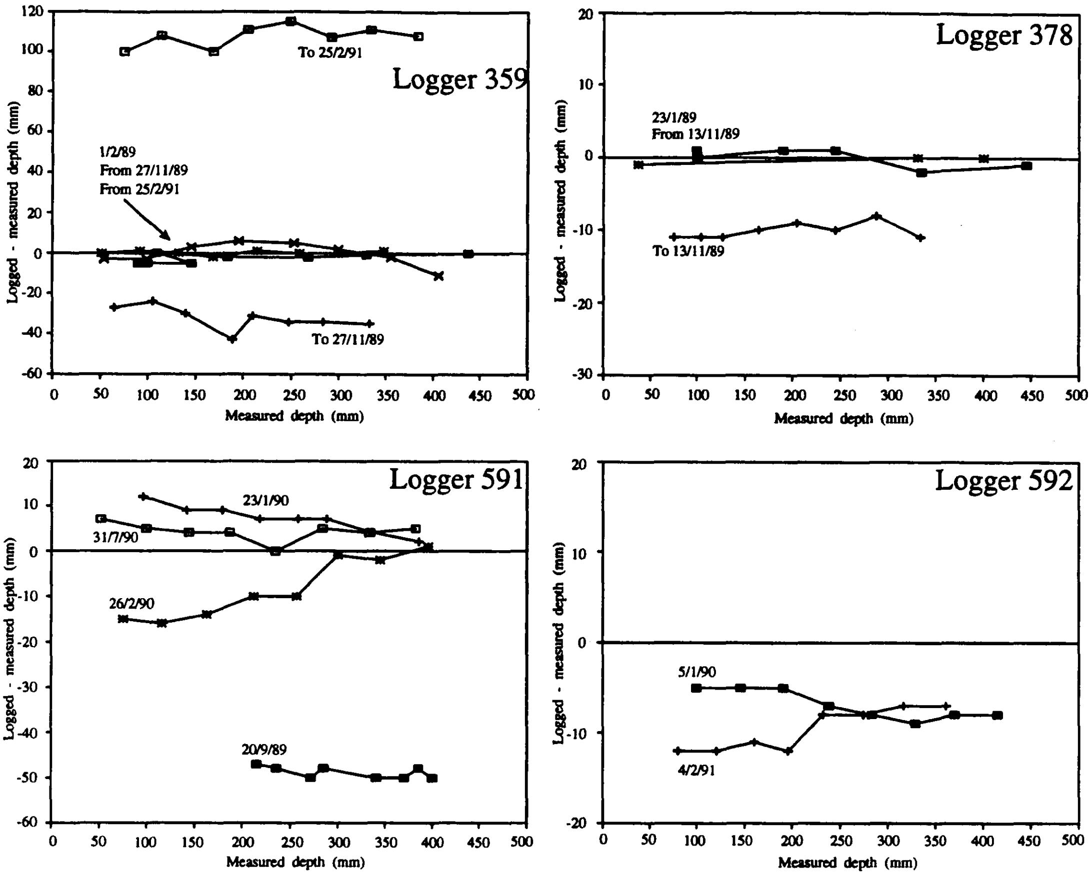


Figure G.1 Laboratory Depth Calibrations for Flow Monitors

It is also relevant to express the above zero drift errors as percentages of the exact depth. For all depths >100mm the maximum error on any calibration prior to adjustment was -15.9% with an average error of -2.0%. When only the absolute error was considered, this latter figure became ± 3.3% and when depths less than 250mm were excluded the absolute error dropped to ± 2.5%.

G.2 Laboratory Velocity Calibrations

Concurrent with the depth calibrations, spot velocities were taken in an open channel flume 0.305m wide. The range of conditions investigated was limited by the pump capacity of 21 l/s, resulting in maximum velocities of approximately 0.6m/s when the depth was >100mm. The water in the recirculating system contained both solid particles and air bubbles, essential for valid readings.

Velocities were determined in two ways: using a Nixon Miniflow meter inserted at the mouse; and by measurement of the mean velocity using the mass flowrate (as determined by a weighing tank), measured cross-sectional area and water density.

Observations from all loggers deployed in the study were grouped together in spite of the extended time lapses between tests. The data are shown in Figure G.2 (a). It is immediately apparent that, although scatter of the readings did occur, there was no evidence of drift as noted in the level calibrations. A large part of such scatter may be attributable to turbulence in the flow, leading to fluctuations in the response of the ultrasound crystals.

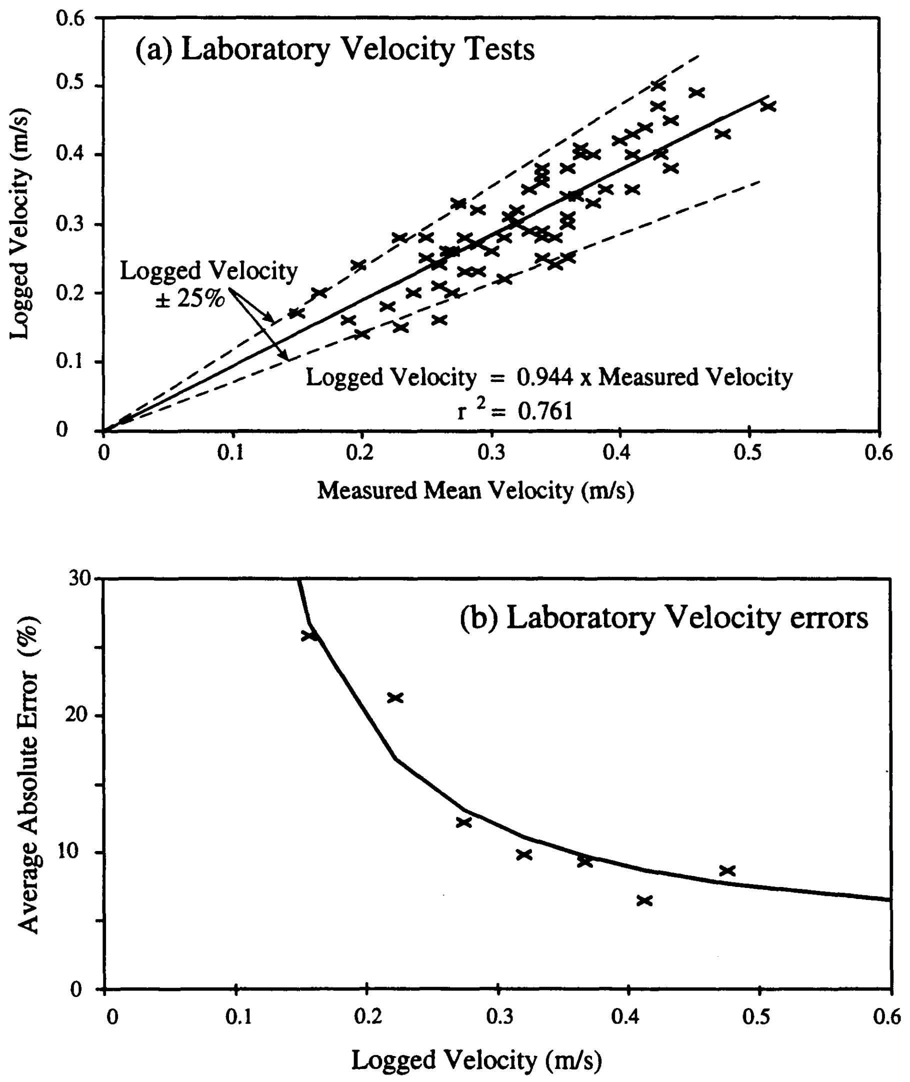


Figure F.2 Laboratory Velocity Calibrations for Flow Monitors

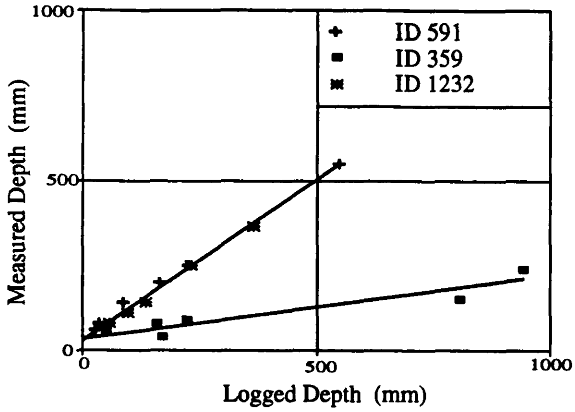
Operation of the doppler shift principle to a flow containing particulate matter results in a mean flow velocity and it was not possible with the equipment available to determine precisely where in the flow this mean applied, contributing to the scatter of the points.

A straight line regression of the data has been included in Figure G.2 (a) on the assumption that the origin should be included. The regression shows that on average the logged velocity was 5.6% lower than the measured mean velocity. Considerable scatter was observed and is illustrated in Figure F.2 (b) in which the absolute error due to scatter is expressed as the difference from the observed mean velocity, regardless of sign. Average values were plotted for velocity steps of 0.5m/s. Although the percentage error ranged from 12% to 26% at velocities <0.3m/s, the percentage error due to scatter reduced rapidly and changed only slowly at 0.5m/s. Due to the experimental limitations higher velocities could not be obtained, however extrapolation of the data would suggest that average errors no greater than $\pm$ 5% in velocity readings would be expected at mean velocities greater than 0.5m/s.

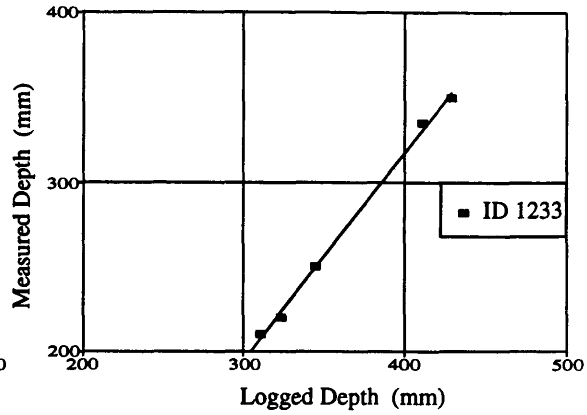
G.3 Field Calibrations

Low priority was given in the study to field calibrations until late 1990. Prior to that time, reliance was placed on scatter plots to determine gross error and laboratory calibrations to evaluate drift and other instrument errors. Gross inconsistencies in a monitor installed at M^{c}Kane Park were not identified using these techniques and during 1991 and 1992 particular efforts were made to obtain reliable field checks on depth over the full range of monitored flow depths at each site.

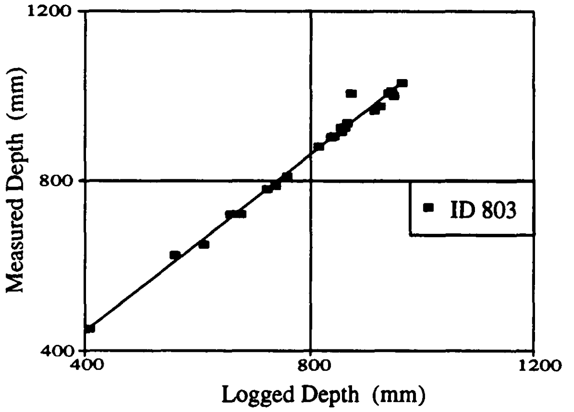
The effect of velocity in modifying pressure at the transducer can be discounted, as field calibration was made at representative velocities.



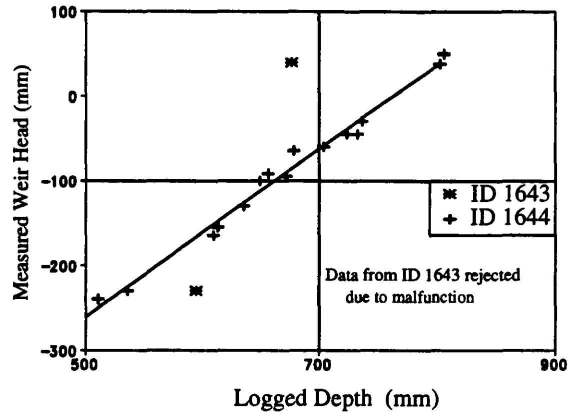
(a) McKane Park field calibration



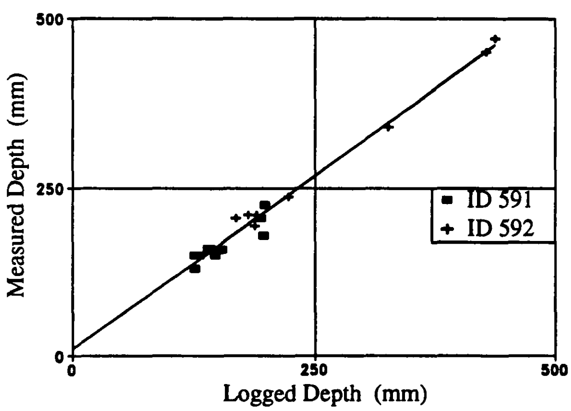
(b) Dixon Street field calibration



(c) Broomhead Inlet field calibration



(d) Elgin Street Inlet field calibration



(e) Elgin Street Throughflow field calibration

Measured = C1 x Logged + C2				
Figure	Monitor ID	C1	C2	r^2
a	591 1232	0.951	29	0.988
a	359	0.189	33	0.866
b	1233	1.233	-176	0.997
c	803	1.041	29	0.988
d	1644			0.989
e	591 592	1.032	10	0.984

(f) Depth Calibration Equatia

Figure G.3 Field Calibrations for Flow Monitors

The field data gathered from a number of sites are plotted in Figure G.3(a-e). Linear Regression was applied to the data and the resulting coefficients are presented in Figure G.3 (f). The values of r^2 show that high confidence can be placed in the reliability of the units in measuring depth.

At the M^{c}Kane Park site, two units, 591 and 1232 operated successfully throughout their installation period. Monitor ID 359 however, was suspected of giving excessively high logged measurements. Errors were not apparent during low flows and it was only by making specific visits to the site during heavy rain that the data of Figure G.3 (a) were obtained. They show the logger to be recording more than five times the actual depth. This was caused by a malfunction of the unit and resulted in the rejection of the data referred to in section G.1.

The data from monitor ID 1233 at Dixon Street show a consistent difference between logged and observed. This was due to the difficulty of taking site measurements however the interpretation of Figure G.3 (b) is that the monitor showed no drift.

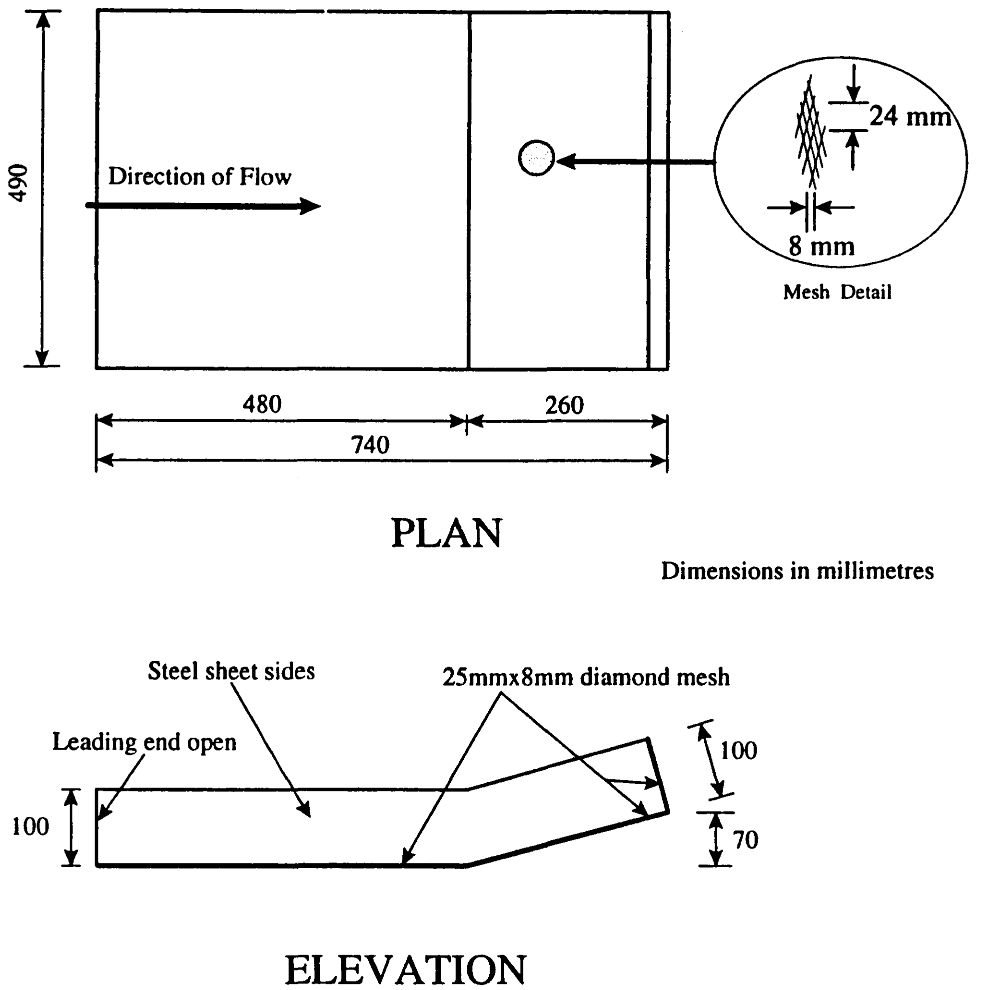
Figure G.3 (c) showed that monitor ID 803 had a zero correction of 29mm. This constant only was applied with this monitor as the slope of the fitted line in this figure was close to unity.

Two monitors were installed at Elgin Street inlet as indicated in Figure G.3 (d). Monitor ID 1643 showed severe drift and was replaced by ID 1644. The data here are expressed as weir head and show consistent readings were obtained after replacement.

Two monitors were also installed on the Elgin Street throughflow. Once again only a zero shift constant (10mm) was applied to the data.

Appendix H

Details of Improved Trash Trap



Details are similar to the Basic Trash Trap Which is Illustrated in Figure 3.6 and Plates 5.1 & 5.2