SMALL CONTINUOUS FLOW RATE FLUCTUATIONS IN RAPID GRAVITY FILTRATION

By

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ABSTRACT

Rapid gravity filters used in the treatment of drinking water are subject to small continuously occurring flow rate fluctuations known as surges. Large, step changes in the rate of flow have been shown to have a detrimental effect on filtrate quality. However, less is known about the effects of surging flow on rapid filter performance. Measurements by previous researchers have found that surges from 2 to 10 % of the flow rate are common and can occur as many as one hundred times per minute. It has been suggested that surging may significantly influence rapid filter performance but the effect has yet to be confirmed under well-controlled conditions and the mechanisms critically examined. Measurements taken by this author at local water treatment plants confirmed the presence of surging flow in the rapid gravity filters of a similar nature to other researchers' findings. Evidence suggested the degree of surging present was related to the design of the filtrate piping and some design recommendations are made on this basis. Two rapid gravity filters were developed in the laboratory to investigate the influence of surging flow on filter performance. The filters were constructed from Perspex pipe and comprised 600 mm of 0.5 to 1.0 mm filter sand. The filters were operated at 30°C at an approach velocity of 8.0 metres per hour with a test suspension of PVC particles. Reproducible performance was established before applying surges to one filter only. A range of surging characteristics similar to those observed at full-scale plants was applied during the test programme. Measurements of head loss and turbidity were taken at a range of depths within the filter media periodically during each test. Samples were collected for particle size distribution analysis from selected tests. The surging flow was found to inhibit the performance of the laboratory filters. The fluctuations in flow rate were found to reduce the removal efficiency of turbidity and retard the rate of head loss development. The surges were found to inhibit the removal of all particle sizes present in the test suspension. The magnitude of the effect on filter performance was found to be dependent on the magnitude and frequency of occurrence of the surges applied. The experimental results obtained suggest that surging does have an effect on full-scale rapid filter performance and has implications for drinking water quality.

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Last, and by no means least, I thank my long-suffering girl friend, Diane. For putting up with over three years listening to me prattle on about filters, enduring the lows and sharing the highs. I think she should get an honorary doctorate at least for her contribution towards keeping me sane.

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LIST OF ABBREVIATIONS & SYMBOLS

F1	Filter One
F2	Filter Two
S1	Siphon One
S2	Siphon Two
SSF	Slow sand filtration
RSF	Rapid sand filtration
DRF	Declining rate filtration
CRF	Constant rate filtration
GAC	Granular activated carbon
PSD	Particle size distribution
SCADA	Supervisory control & data acquisition
PVC	Polyvinyl chloride
RHL	Rate of head loss development
C	Conductivity (uS/cm)
RE	% removal of turbidity
ζ	Zeta potential (mV)
ε	Porosity
u_I	Interstitial velocity (mm/s)
u_a	Approach velocity (mm/s) or (m/hr)
τ	Shear stress (N/m^2)
Q	Flow rate (m^3/s)
μ	Dynamic viscosity (kg/m/s)
A	Filter surface area (m^2)
d	Pore diameter (m)
mm	Millimetres
cm	Centimetres
m	Metres
m^2	Square metres
m/hr	Metres per hour
mm/s	Millimetres per second

l/s	Litres per second
m^3/s	Cubic metres per second
gpm/sq. ft.	Gallons per minute per square foot
Ml/d	Mega litres per day
NTU	Nephelometric turbidity units
ppm	Parts per million
ppb	Parts per billion
mg/l	Milligrams per litre
um	Microns
kg/m/s	Kilograms per metre per second
mbar	Millibar
mV	MilliVolts
uS/cm	Micro Siemens per centimetre
N/m^2	Newtons per square metre

1.0 INTRODUCTION

Rapid gravity filtration is widely used in the treatment of surface water resources such as lakes, rivers and reservoirs to provide drinking water supplies to the population. Of course, other solid/liquid separation techniques using the principles of filtration are used such as slow sand filtration and pressure filtration but it is true to say that rapid gravity filtration is the most common in the western world. Solid/liquid separation by coagulation, flocculation and subsequent settlement or flotation is widely practised in drinking water treatment but it is fair to say that these processes are used as pre-treatment steps to enable effective purification by the filtration stage. The first recorded use of filtration in the United Kingdom was at Paisley, Scotland by John Gibb in 1804. He built and operated slow sand filters to provide drinking water and being an enterprising fellow he sold the excess capacity to the public. Slow sand filtration as a technique for the purification of water spread when it was recognised that such treatment reduced the transmission of waterborne diseases. Rapid gravity filtration, developed in the United States, subsequently superseded slow filtration for the provision of potable water to large communities on the grounds of its ability to operate at greater surface loading rates and its smaller land requirements. Today, most major towns and cities in the western world are supplied using rapid gravity filtration. Slow sand filtration is still used but mostly for the supply of rural communities. It is clear therefore that the performance of rapid gravity filtration is an important factor in the maintenance of public health.

The purpose of this thesis is to describe the research programme undertaken by the author in the field of drinking water treatment by rapid gravity filtration. The subject matter for the research was to investigate the effect of small but continuously occurring rapid fluctuations in the flow rates through rapid gravity filters known as surges and to assess the implications of such surges for the supply of safe drinking water to the population.

Chapter Two presents a review of previously published material related to surging in rapid filtration to set the scene for the author's research. Past observations and measurements of surging flow in experimental and full scale rapid gravity filters are described but it is shown that the effects of surging flow on filtered water quality are not confidently known. The effects of discrete changes in the flow rate on filter performance are reviewed and concerns over the presence of pathogens such as *Giardia sp.* and *Cryptosporidium sp.* in drinking water supplies and their ability to penetrate rapid filters and survive post filtration disinfection are discussed. Particle size distribution and turbidity are reviewed as techniques for the monitoring of filter performance and the chapter closes with a summary and a statement of the objectives of this research programme.

Chapter Three presents the results of surging measurements taken at local water treatment plants by the author. Evidence gathered using the available plant instrumentation is discussed and compared with previous researchers' findings. The characteristics of the observed surging flow are analysed and used to determine the nature of the laboratory investigation described later in this thesis. A possible relationship between the degree of surging found in these full scale filters and the design of the filtrate piping is discussed.

Chapter Four describes the design and development of the laboratory apparatus used to investigate the effect of surging on rapid filter performance. The test suspension was stored in a series of mixed and temperature controlled tanks and was delivered to the experimental filters by siphons from a small constant head tank. A detailed description of the two experimental filters is presented including the filter media specifications and the design of the pressure and suspension sampling ports connected to the filter columns. A machine was developed to generate surging flow in one of the experimental filters. The machine was designed to generate surges similar to those observed at full scale filter plant as described in Chapters Two and Three. A system for backwashing the experimental filters after each test run was developed and is described here.

Chapter Five details the development of the experimental procedure and the methods of measurement and analysis used in each of the test runs conducted during the laboratory study. The preparation and storage of a reproducible test suspension is presented followed by a review of the preparations necessary for the operation of the filter columns. One test run is described in full and the methods used to measure the changing pressure drop and removal of suspended matter within the filter columns are reviewed here. Particle size distribution analysis, pH, conductivity, dissolved solids concentration and Zeta potential measurement were used and the methods of sample handling and analysis are given.

Chapter Six presents and discusses the results obtained by the laboratory study. The chapter begins by describing in chronological order the test programme conducted and the key events as they occurred. Analyses of the results of the control and surging test runs are presented and the effects of the different surging characteristics on filter performance are discussed. Comparisons and contrasts are made with the findings of previous related filtration research reviewed in Chapter Two and with the field measurements of surging presented in Chapter Three. The significance of the experimental findings is discussed in this light. Finally, additional factors, unrelated to the key focus of the research that were encountered during the study but were none the less interesting are reviewed and discussed.

Chapter Seven presents the conclusions of the study and makes recommendations for future research in this field. Appendices and the References are attached at the end of the thesis.

A first paper summarising this experimental investigation was published in the journal Water Science and Technology. The full citation is given below for reference.

"The effect of surges on the performance of rapid gravity filtration"
by Graeme D. E. Glasgow & Andrew D. Wheatley
Water Science & Technology
Volume 37, No. 2 pp. 75-81 1998

2.0 REVIEW OF RELATED RESEARCH

It is the purpose of this chapter to review previously published material relevant to the research program described in later chapters of this thesis. This will hopefully clarify in the mind of the reader the reasoning behind the laboratory investigation and explain the direction taken by the author in the pursuit of the objectives laid out at the end of this chapter.

The review begins with the findings of previous researchers' work into small, continuously occurring fluctuations in the flow rate through rapid gravity filters known as surging. A definition of surging as applicable to the field of filtration is given and the earliest observations in full scale filtration plant are presented. Subsequent assessments of surging in experimental and full scale filters are examined and some findings regarding the effect of surges on filter performance are discussed. Since little published research on surges exists, the material is covered here in some depth.

Related to but distinct from surges, are discrete changes in the flow rate through rapid filters. These changes in the flow rate can be caused by changes in plant loading and filter backwashing in full scale filter banks. The results of past investigations and observations of the effects of these discrete flow rate changes in both full scale and experimental filters are discussed.

The presence of *Giardia sp.* and in recent years *Cryptosporidium sp.* in the water environment has been a cause for concern for public health engineers. Contamination of surface waters with these organisms has led to a number of outbreaks of illness transmitted by the treated public water supply. An important factor in the control of these pathogens is the performance of the filtration stage of water treatment. The occurrence and cause of the more significant outbreaks are reviewed and recommendations regarding filter operation are discussed. As a result of these cases, surveys of the presence and concentration of *Giardia sp.* and *Cryptosporidium sp.* in the water environment were conducted. The results and implications of these surveys for water treatment are presented. Investigations into the performance of gravity filtration in the removal of *Giardia sp.* and *Cryptosporidium sp.* were carried out by a number of authors and their findings are reviewed.

The use of particle analysis has led to an increasing understanding of the complex processes found to occur within rapid gravity filters. Experimental observations with particle counters have found that the removal efficiency and head loss development are dependent on both the size of the incoming suspended particle and the distribution and concentration of adjacent particle sizes. Filtration modelling has developed to incorporate these phenomena. However, particle analysis is not yet suitable for continuous full scale filter monitoring. Turbidity measurement is the most widely used method of monitoring filtrate quality. The findings from recent particle research are reviewed and discussed and the limitations of turbidity measurement are presented in this light.

The chapter closes with a summary of the findings of the research reviewed and unanswered questions and concerns in the light of the discussion are made with respect to surging in gravity filters. A statement of objectives intended to answer these questions is made and forms the basis for the research program carried out by the author and described in subsequent chapters.

2.1 Surges in the Rate of Flow through Filters

Surging has been recognised in rapid gravity filters but little attention has been given to them in recent times. Indeed, there is little published material on this subject and none from recent years. This section will review the published research conducted on surges in some detail from the earliest observations in full scale and experimental filters to the believed effects on filtrate quality.

2.1.1 Definition of Surging and Early Observations

Baylis (1958) defined the term surging as applied to drinking water filters to be the "pressure variations resulting from momentary erratic fluctuations in the rate of flow of water through filters". These surges were analysed in a gravity filter by attaching a piezometer tube to the filter effluent piping or through the filter wall into the filter bed beneath the sand surface. The water level in the piezometer tube was seen to rise and fall in a random, erratic manner at frequent intervals. This fluctuation in the piezometer water level indicated that there were pressure fluctuations beneath the filter surface. These pressure fluctuations indicated that there were momentary flow rate fluctuations occurring within the filter of sufficient magnitude to cause pressure fluctuations. These pressure fluctuations were observed to increase with increasing loss of head through the filter.

Surging first came to Baylis' attention at the Chicago experimental filtration plant in 1928. While operating a small pilot filter, Baylis noticed that the mercury levels in the head loss and rate of flow gauges attached to the filter rose and fell in an erratic manner in the space of a few seconds. These fluctuations in the flow rate and head loss measurements were not considered important at the time and were not investigated further. However, in 1935 the surges were noticed again while measuring head losses within a larger filter at the experimental plant. Copper tubes were inserted into the filter bed at various depths below the surface of the sand and connected to mercury gauges. Each tube had a flared end covered with a brass mesh screen to prevent sand entering the tube. This apparatus was used to monitor the change in head loss with increasing depth as the filter accumulated deposit. During the filter runs, it was observed that the mercury in the tubes was moving up and down constantly in each tube. The oscillating movement took place at 2 to 4 second intervals. The observed oscillation was small at the beginning of the run when there was little head loss across the filter but as the head loss increased the amplitude of the surges was observed to rise. The filter was running at an approach velocity of four metres per hour. While operating the experimental filters at higher than normal rates Baylis noted anomalous results. He observed that coagulated solids were carried through certain of the filters at unexpectedly low flow rates. Baylis thought that the surging was the most probable reason for the poor performance at lower flows and decided to conduct a more thorough investigation into the phenomenon.

2.1.2 Surge Measurements in Experimental Filters

In 1940 Baylis conducted a series of tests on a range of pilot filters ranging from 0.015 to 9.29 square metres (0.166 to 100 square feet) in surface area variously equipped with venturi tube flow rate controllers (Baylis, 1958). 3 mm internal diameter glass piezometer tubes 300 to 400 mm long were connected to various points in the filter systems to measure the pressure fluctuations present. The tubes were connected to the filters using identical length and internal diameter rubber hosing. The open end of the piezometer tube was held above the water level and care was taken to ensure no air bubbles were trapped in the tubes. Baylis constructed a hand operated chart recorder shown in figure 2.1. The piezometer tube was attached to this mechanism. The surge profile was recorded by slowly winding the chart along behind the piezometer tube at a fixed rate while an observer marked the water level on the moving chart with a pencil. The profile was typically recorded in this manner over a 60 second period. This provided detail of the surging characteristics and allowed the surge amplitude to be estimated. The surge amplitude was taken to be the vertical distance between lines drawn through 3 average peaks and troughs occurring during the one minute of recorded data. Baylis made over 1000 surge measurements on filters of various sizes in this manner. Table 2.1 presents some characteristics of 3 experimental filters, numbers 2, 5 and 16 at the Chicago plant used by Baylis in his surging observations.

Filter Number	*Bed Shape & Size (metres)*	*Surface Area (sq. metres)*	*Venturi Dimensions (mm)*
2	3x3, rectangle	9	152x76
5	1.09 diameter	0.93	63x22
16	1.83x1.52, rectangle	2.78	102x51

Table 2.1 Surging Filter Characteristics (Baylis, 1958)

Filter number 5 at the experimental plant was made from a circular steel drum 1.09 metres in diameter. The filter was fitted with perforated pipe underdrains buried in 508 mm of graded gravel. 610 mm of sand were placed on top of the gravel. Figure 2.3 illustrates the effluent piping arrangement for this filter. The effluent pipe was 76 mm in diameter up to the venturi tube. The venturi tube had dimensions of 63 mm at the main section and 22 mm at the throat. Surges were measured at a sample port ahead of the venturi tube. Flow was controlled by a hand operated valve downstream of the venturi tube. A series of tests were conducted on filter 5 to determine how the surges responded to head loss and flow rate. After backwashing the filter a series of surge recordings were made at flow rates of 0.126, 0.189, 0.252, 0.315 and 0.378 litres per second (2, 3, 4, 5 and 6 US gallons per minute). Once completed a second set of surge recordings were made since the filter had captured solids and the head loss had increased since the first set were taken. This procedure was continued until the filter reached terminal head loss. A potential problem exists with this experimental procedure. By increasing the flow rate from 0.126 to 0.378 litres per second in steps and returning to 0.126 litres per second several times during the filter run, the filter has been subjected to a large number of flow rate increases of significant magnitude. Later work (See section 2.2) found that such step increases in the flow scoured deposits from the bed and modified the head loss. It is therefore possible that the head loss measurements made by Baylis were distorted. However, an important factor which determines how significant an effect the rate increase has is the rate at which the flow is changed. Baylis makes no mention of how rapidly or slowly the rate increases were made and it is impossible to say whether the results have been significantly affected. Nevertheless, the surge amplitudes were determined from the chart recordings and the results plotted as shown in figure 2.4. This chart illustrated how the surge amplitude appeared to change with increasing loss of head at a given flow rate. At all flow rates the surge amplitude was observed to increase with increasing head loss. From these curves values of surge amplitude were interpolated at head losses of 508, 1016 and 2032 mm (20, 40 and 80 inches) for each flow rate. These results were plotted as shown in figure 2.5. This chart illustrated how the surge amplitudes apparently changed with increasing flow rate at any given head loss. From these results for filter 5 the surge amplitude appeared to increase proportional to the square of the flow rate at a given loss of head. The described experimental procedure was repeated for filters 2 and 16.

Filter number 2 comprised a 3 by 3 metre rectangular concrete box filled with sand over an Aloxite porous plate bottom. A sketch of the filter piping arrangement is shown in figure 2.2. The filtrate passed through a 152 mm diameter effluent pipe to the clear well via a venturi tube and flow rate controller. The venturi had internal diameters of 152 mm at the main section and 76 mm at the throat. The rate controller comprised a balanced valve actuated by the venturi differential pressure. Surge measurements were taken from this filter at several different flow rates in the manner described above for filter 5. The piezometer was connected to the sample port shown

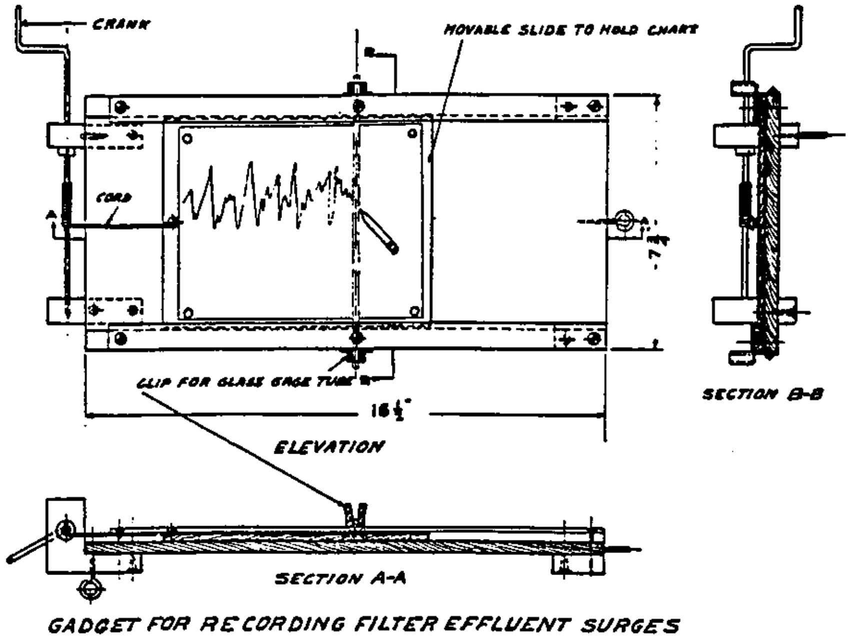


Figure 2.1: Gadget for recording filter effluent surges (Baylis, 1958)

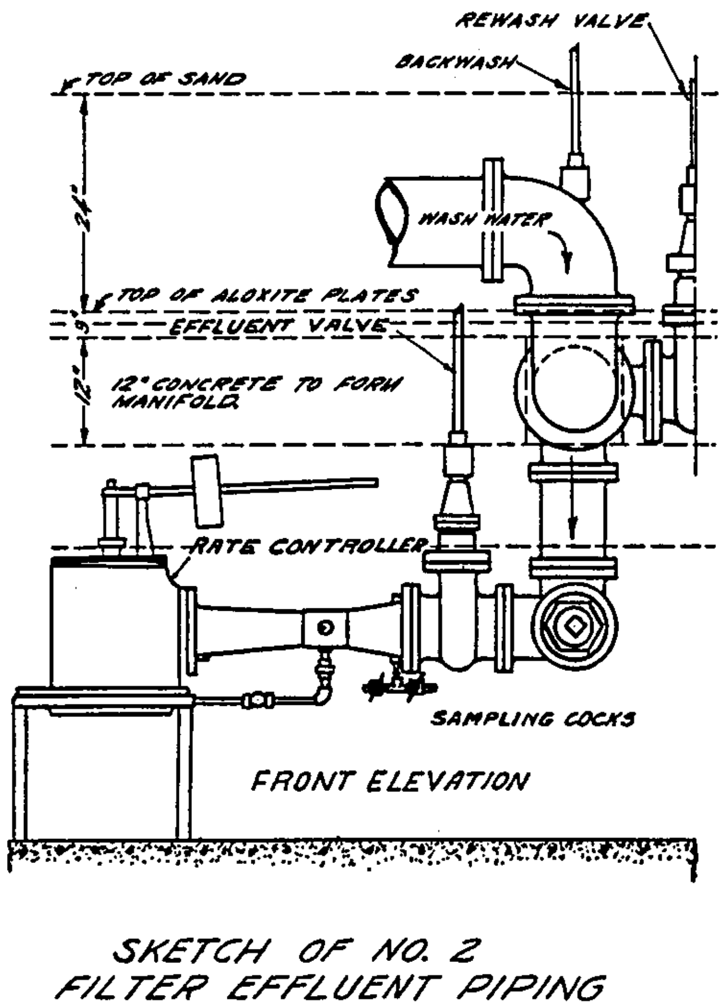


Figure 2.2: Sketch of number 2 filter effluent piping (Baylis, 1958)

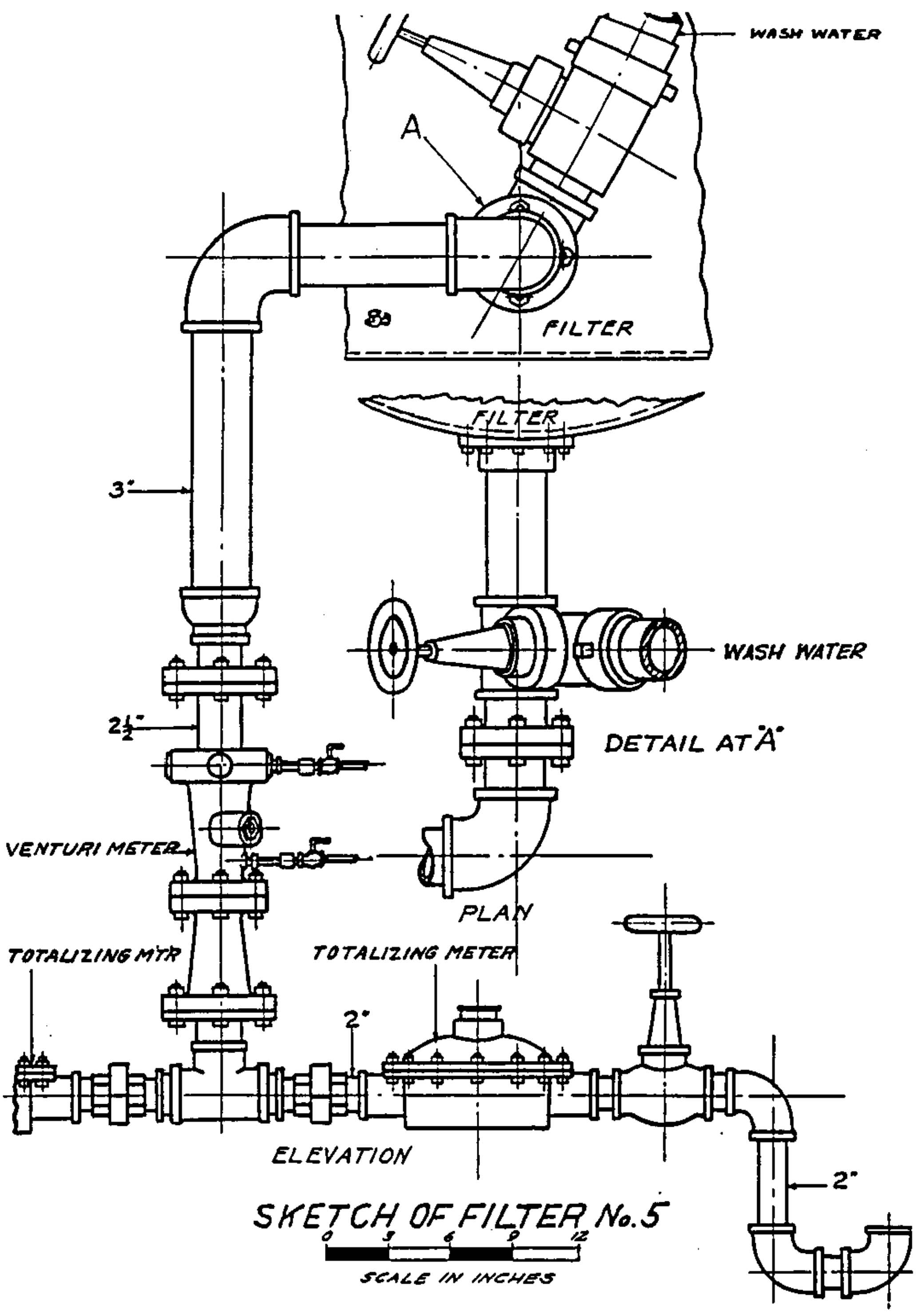


Figure 2.3: Sketch of filter number 5 (Baylis, 1958)

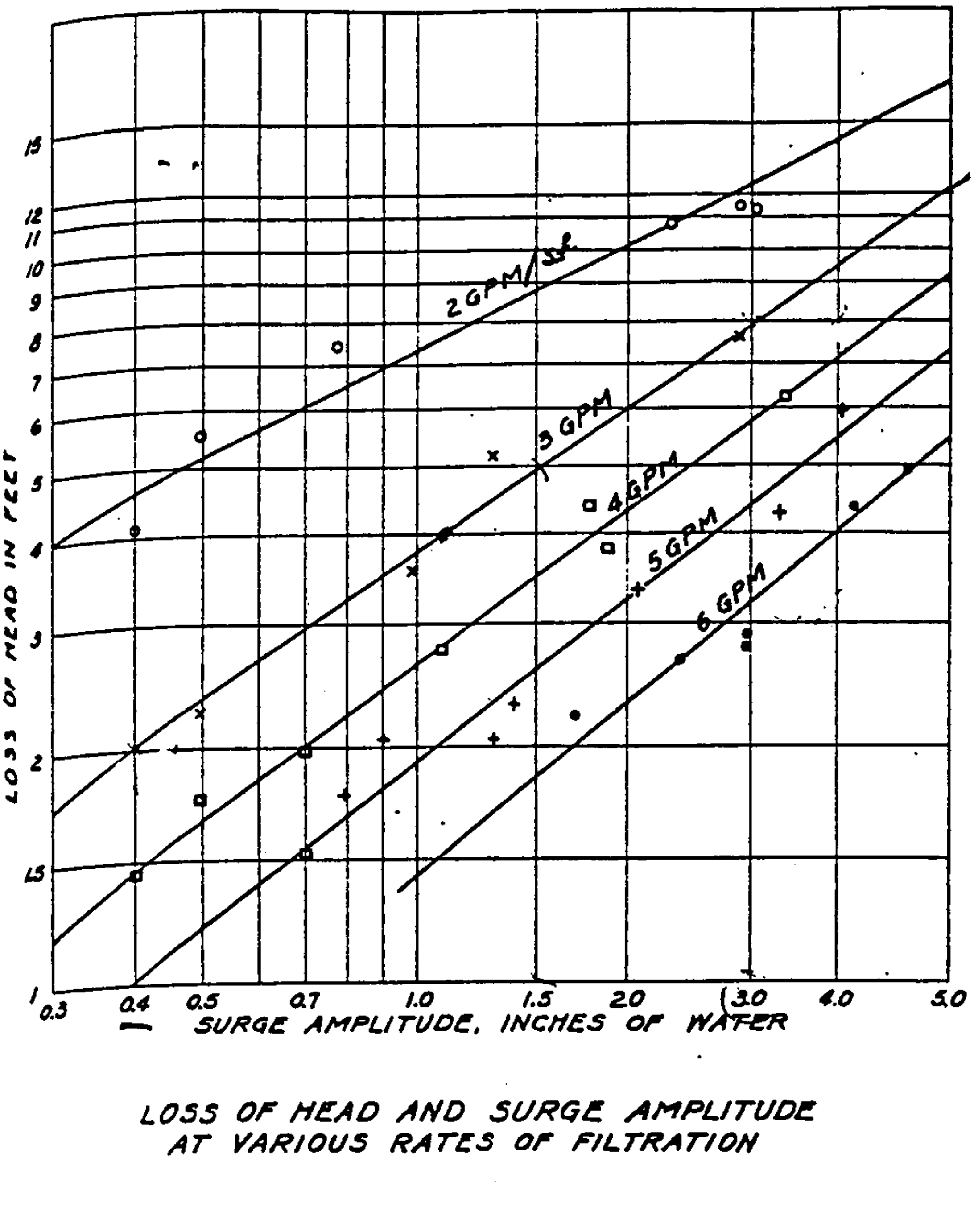


Figure 2.4: Loss of head and surge amplitude at various rates of filtration (Baylis, 1958)

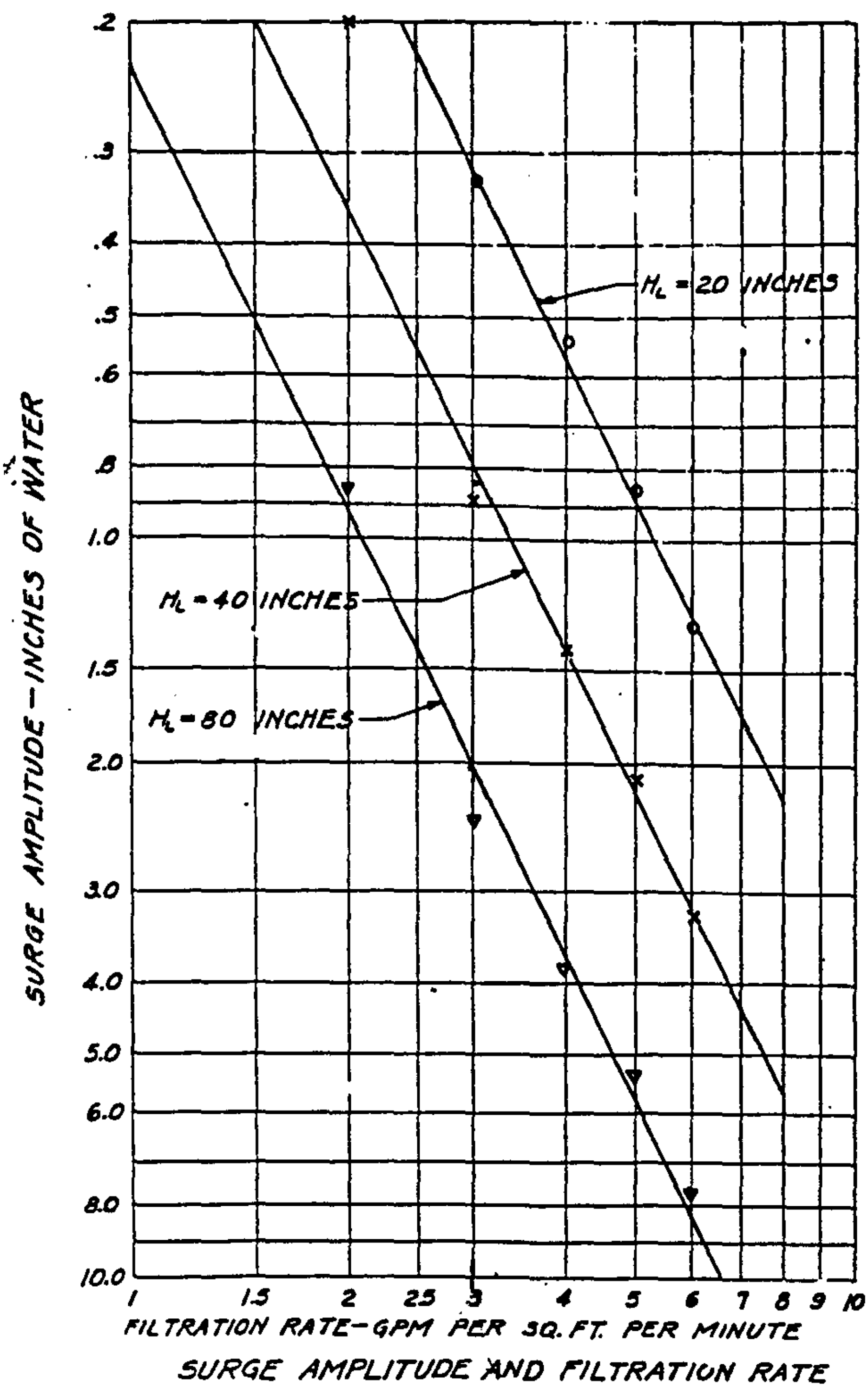


Figure 2.5: Surge amplitude and filtration rate (Baylis, 1958)

ahead of the venturi and the profile recorded using the chart recorder method described above. Figure 2.6 illustrates the surge profiles recorded at flow rates of 0.1, 0.12 and 0.21 litres per second (1.62, 1.96 and 3.29 US gallons per minute). The filter head losses and the venturi throat velocities are also shown. These recordings show that at similar filter head losses the surge amplitude increased from 31 to 102 mm (1.2 to 4 inches) as the flow rate was increased from 0.1 to 0.21 litres per second (1.62 to 3.29 US gallons per minute). Once more the surge amplitude appeared to be proportional to the square of the flow rate. It can also be seen that the surging was random and erratic in nature and occurred many times per minute.

Filter 16 comprised a steel rectangular tank 1.83 by 1.52 metres. The filter bed had 914 mm of sand placed on top of 343 mm of graded gravel with a perforated pipe collector system. The effluent piping arrangement contained a venturi tube followed by a balanced control valve. The venturi main section diameter was 102 mm with a throat diameter of 51 mm. The control valve was governed by the venturi differential pressure. An example of the recorded surges from filter 16 is shown in figure 2.7.

To try and differentiate the effects of head loss and flow rate on the surge amplitudes, Baylis normalised the surge amplitudes from the experimental work conducted on filter 5. Since the surge amplitude appeared to be proportional to the square of the flow rate, the surge amplitudes at each approach velocity of 4.89, 7.33, 9.78, 12.22 and 14.67 metres per hour (2, 3, 4, 5 and 6 US gallons per minute per square foot) were adjusted to an approach velocity of 4.89 metres per hour (2 US gallons per minute per square foot) on the basis of this relationship. For filter number 5 this approach velocity corresponded to a venturi throat velocity of 3.26 metres per second (10.7 feet per second). The corrected results are shown in figure 2.8. The correction procedure was then applied to the results from filter 2 and 16. Firstly, the surge amplitudes were adjusted to an approach velocity of 4.89 metres per hour (2 US gallons per minute per square foot) by assuming a square relationship with the flow rate. However at this rate the three filters had slightly different venturi throat velocities. As such the results were then corrected to correspond to a throat velocity of 3.26 metres per second (10.7 feet per second) as in filter number 5. These corrected results from filter 2 and 16 were also plotted in figure 2.8. It can be seen that all three filters give good correlation between loss of head and surge magnitude at the same velocity. From this chart Baylis thought that the surge amplitude for the three filters investigated increased in proportion to the loss of head raised to some power greater

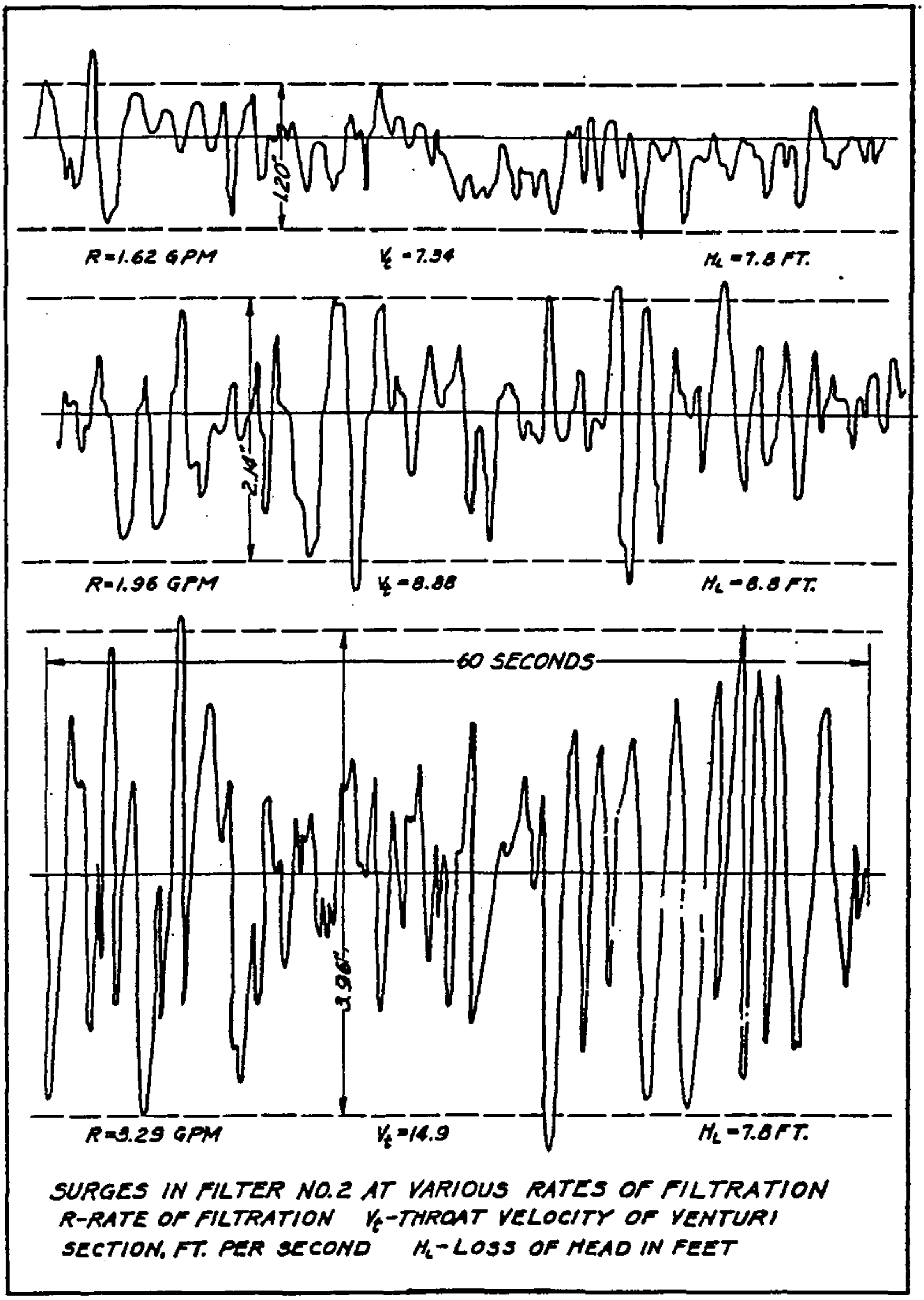


Figure 2.6: Surges in filter 2 at various filtration rates (Baylis, 1958)

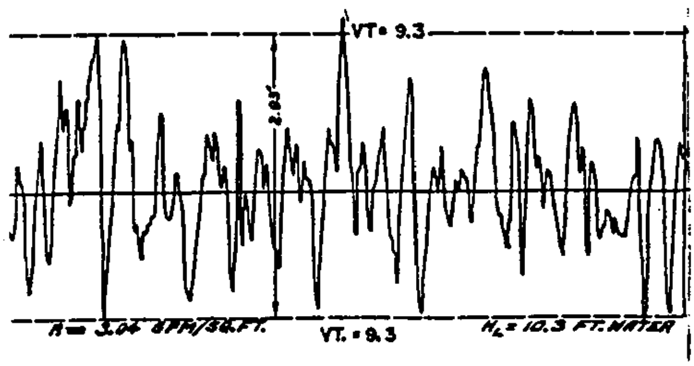


Figure 2.7: Surge recording from filter 16 (Baylis, 1958)

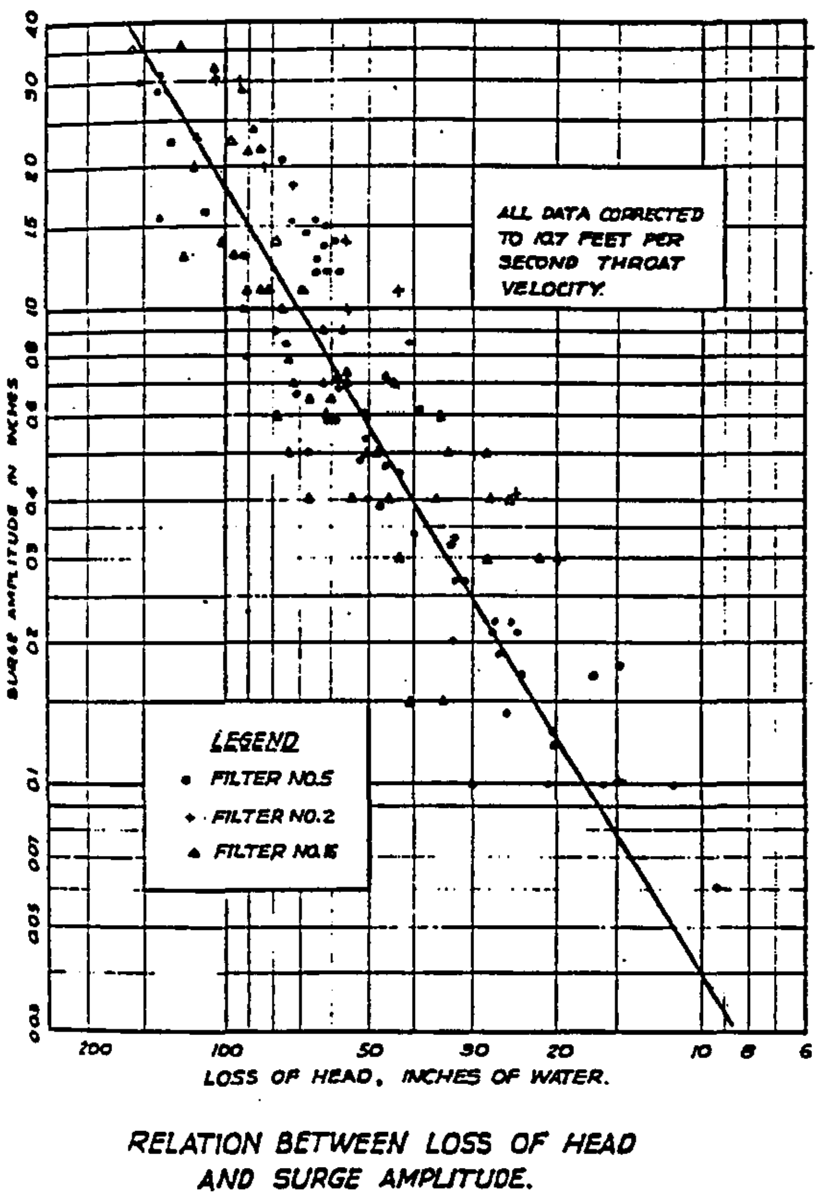


Figure 2.8: Relation between loss of head and surge amplitude (Baylis, 1958)

than one. And since the correlation appeared good he concluded that the surge amplitudes varied in proportion to the square of the venturi throat velocity. It seemed to Baylis that the surges in the effluent pressure were caused by separation of the fluid streamlines from the conduit boundary in the recovery cone of the venturi tubes. He thought that the surges were the result of the variable conversion of velocity head at the flow constriction into pressure head and turbulence once past the constriction. Baylis also noted the apparent effect of the venturi tube size on the frequency of surging present. In comparing the results of filters 2, 5 and 16, he noted that the largest venturi appeared to produce the least frequent pressure fluctuations even though each venturi had similar throat velocities. Baylis suggested that a possible explanation for this was the shorter time a turbulent eddy would spend in the recovery cone of the smaller venturi tubes. Baylis speculated that "sudden, jerky" surges may have a greater significance in filter performance than lower frequency, smoother fluctuations.

Baylis decided to investigate the effect of the venturi tube design on the degree of surging produced. For the experiments described above, filters 2, 5 and 16 were fitted with venturi tubes with an angle of divergence of 15° in the recovery cone. This is illustrated in figure 2.9 showing a cross section of the venturi tube fitted to filter 5. Baylis suspected that the sharp angle of divergence in the recovery cone was the cause of the surges observed. The venturi tube fitted to filter 5 was replaced with a long tube design with a 5° angle of divergence in the recovery cone. A cross section of this tube is also shown in figure 2.9. Surge recordings were made at an approach velocity of 4.89 metres per hour (2 US gallons per minute per square foot) and compared with the results from the 15° angle venturi tube. These results are illustrated in figure 2.10. It can be seen that the shallow angle recovery cone has reduced the magnitude of surges observed in filter 5. Baylis concluded from this that the source of the surges was flow separation in the recovery cone of the venturi tube. A reduction in the angle of divergence reduced the surge magnitudes by reducing the separation of flow.

Baylis then conducted further tests to determine if other effluent pipe flow constrictions could produced appreciable surging. The venturi tube was removed from filter 4, a steel drum filter similar to filter 5, and a 63.5 mm gate valve was fitted. Surge recordings were made ahead of the valve while using the valve to regulate the flow rate. The surges present were observed to increase in magnitude as the gate valve

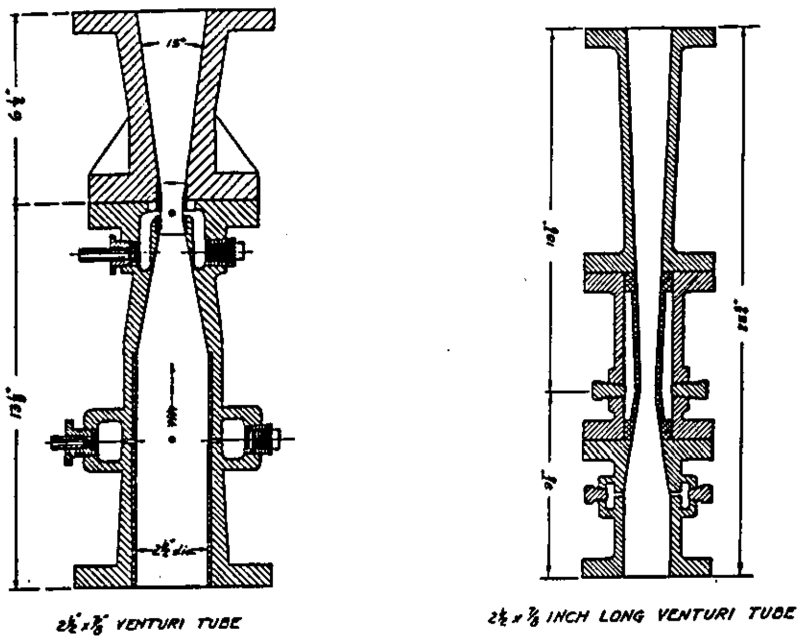


Figure 2.9: Short and long venturi tubes (Baylis, 1958)

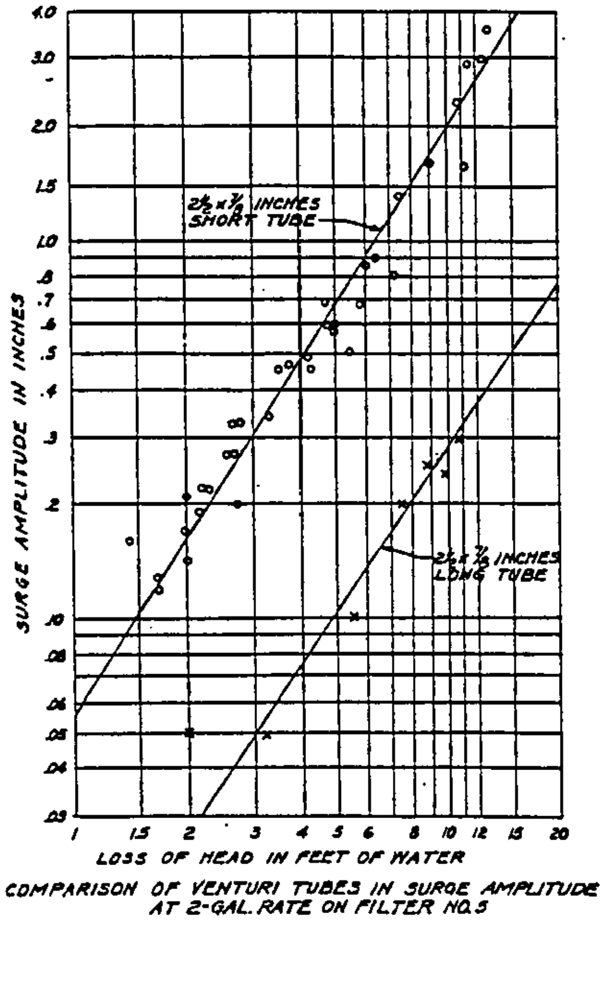


Figure 2.10 Comparison of venturi tubes and surge amplitude (Baylis, 1958)

was opened. However, the magnitude peaked and then declined as the valve was opened fully. Baylis explained that the surge magnitude increased as the valve was opened until the area of flow reached a point where the velocity through the valve then began to diminish. Beyond this point the surge magnitude fell as the flow became less constricted. From these results it then seemed that any constriction in flow which produced a region of high velocity such as the throat of a venturi tube, a partly closed control valve or a sharp bend could produce appreciable surging.

2.1.3 Surge Measurements in Full Scale Filters

Baylis (1958) gathered information on surge characteristics present in full scale filters at local water treatment plants for comparison to his experimental filter results. Some of his measurements were made using the chart recorder described in section 2.1.2. Others were made by simply observing the oscillations in a piezometer tube over a one minute interval and recording the maximum and minimum levels. Table 2.2 below presents some examples of his findings. The pressure measurements were made upstream of the venturi tubes.

Water Treatment Plant	*Filtration Rate* (Ml/d)	*Venturi Dimensions (mm)*	*Venturi Throat Velocity (m/s)*	*Loss of Head (mm)*	*Surge Amplitude (mm)*
A	1.23	203x102	1.76	457	38
	1.89		2.7	610	64
	2.84		4.1	762	102
B	7.57	356x216	2.24	1067	38
C	3.03	254x127	2.77	975	25
D	5.68	-	-	1219	44
E	6.62	356x178	3.2	1707	97

Table 2.2 Surges at Full Scale Plant (Baylis, 1958)

Surges ranging from 25 to 102 mm in magnitude were evident in these full scale filters. An example of the surge profile recorded at Plant E is shown in figure 2.11.

The horizontal scale represents one minute of recorded data. Once more it seems apparent that the surges were random and erratic in nature occurring many times per minute similar to the experimental filter results as would be expected from turbulent velocity fluctuations.

Carrol (1986) conducted a short study at a local water treatment plant in the UK and reported brief observations of erratic fluctuations in the flow rates through the rapid filters. No means of recording the oscillating flows was available. However, the author estimated the amplitude and frequency of the surging by visual observation of the flow gauges over one minute periods. The author found surges from 10 to 40% of the flow rate occurred at rates from 5 to 25 per minute or more. Figure 2.12 illustrates the nature of the surges seen at the treatment plant. Similar to Baylis, the surges were random and erratic and occurred many times per minute.

2.1.4 Effect of Surges on Filtrate Quality

Baylis (1958) compared the filtrate quality from two similar but not identical filters at the Chicago experimental filtration plant. Some details of the filter designs are given in table 2.3.

Filter Number	*Bed Shape & Size (metres)*	*Surface Area (sq. metres)*	*Venturi Dimensions (mm)*
11	1.09, diameter	0.93	64x22
13	0.127, diameter	0.013	None

Table 2.3: Characteristics of filters 11 & 13 compared by water quality (Baylis, 1958).

Filter 11 comprised a 1.09 metre diameter steel tank fitted with a perforated pipe underdrain buried in graded gravel with filter sand on top. The effluent piping was similar to filter 5 shown in figure 2.3 with a 64 by 22 mm venturi tube fitted. The filter was operated at an approach velocity of approximately 8.5 metres per hour (3.5 US gallons per minute per square foot). Filter 11 exhibited considerable surging although no recordings or details were given. Filter 13 was a glass tube filter approximately 127 mm in internal diameter operated at a rate of 9.8 metres per hour

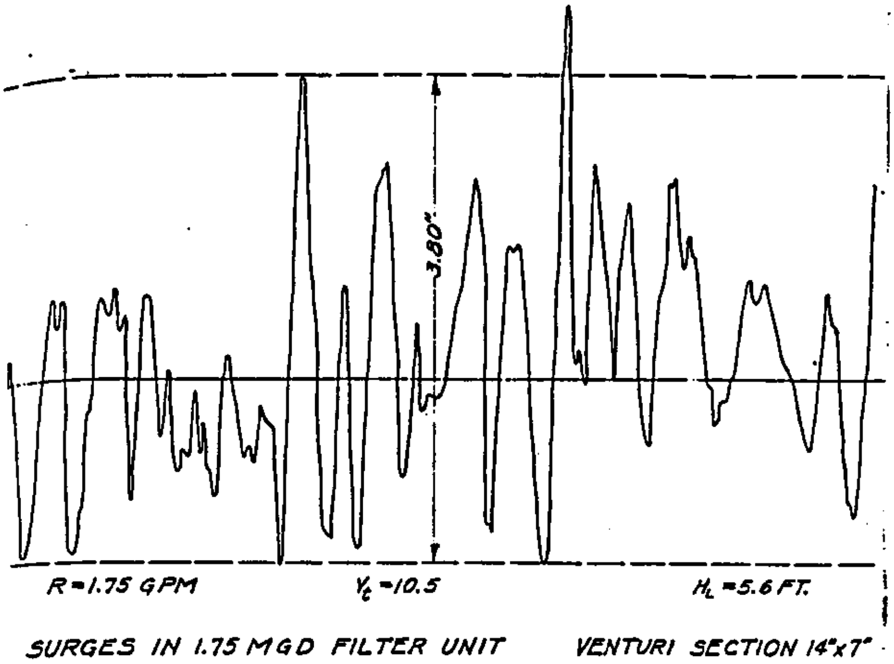


Figure 2.11: Surges in full scale filter (Baylis, 1958)

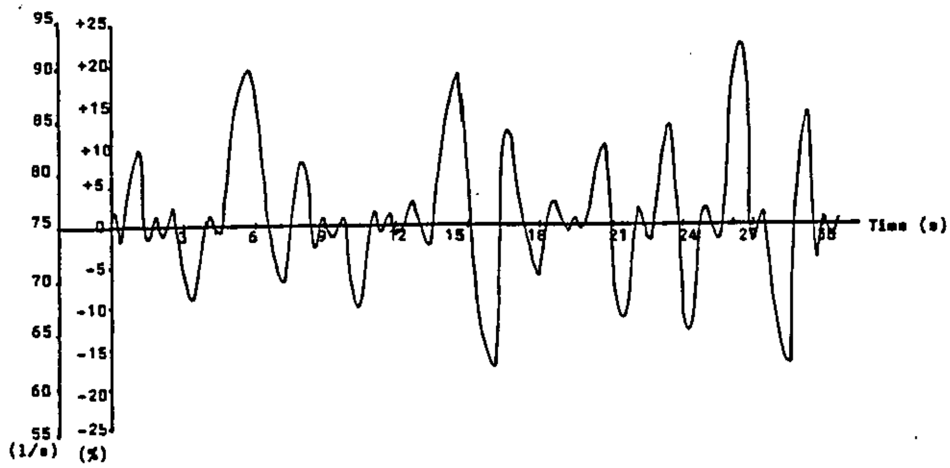


Figure 2.12: Surges in full scale filter (Carrol, 1986)

(4 US gallons per minute per square foot). No further details of the filter design were given except to say that the filter had approximately the same sand and gravel sizes and depths as filter 11. No details of the backwash procedure for each filter were given. Filter 13 was controlled by a float valve and did not exhibit any surging. Both filters were operated filtering aluminium sulphate coagulated raw water. Baylis monitored the hours of service, loss of head and filtrate quality from both filters over a period of five months. Quality was measured by turbidity and by filtering the effluent through cotton plug filters and determining the weight of material captured after combustion. Figure 2.13 and 2.14 were prepared from Baylis' data and illustrate the concentration of solids present in the filtrate from both filters during operation over the winter and summer months. It can clearly be seen that filter 13 outperforms filter 11. Note the poorer performance from both filters during the winter months, probably the effect of low temperature. Baylis concluded that the differing performance was probably the result of the surging present in filter 11. However, he could not state this with certainty since the filters were not identical and had not been shown to have similar performance in the absence of any surging in filter 11. Baylis drew the following key conclusions from all his work on surging.

- Momentary fluctuations in the flow rate through gravity filters take place
- The pressure fluctuations resulting from these rate changes were termed surges
- Surges are random and erratic in nature and can occur at high frequency
- Surges had been noted in the past but were thought to be insignificant
- Similar types of surging were identified in both experimental and full scale filters
- Surges in filters may result from the variable conversion of kinetic head to pressure head and turbulence downstream of flow constrictions
- Surge amplitude was found to be proportional to the square of the flow rate
- Surge amplitude was found to increase as the loss of head rose
- Partly closed valves and flow conditions in bends may cause appreciable surging
- Surges appeared to contribute to the passage of coagulated matter through the filter

Baylis could not quantify the effects of surging but believed that surges of 1 to 2% of the loss of head may affect filter performance where surges greater than 4% would have a considerable effect.

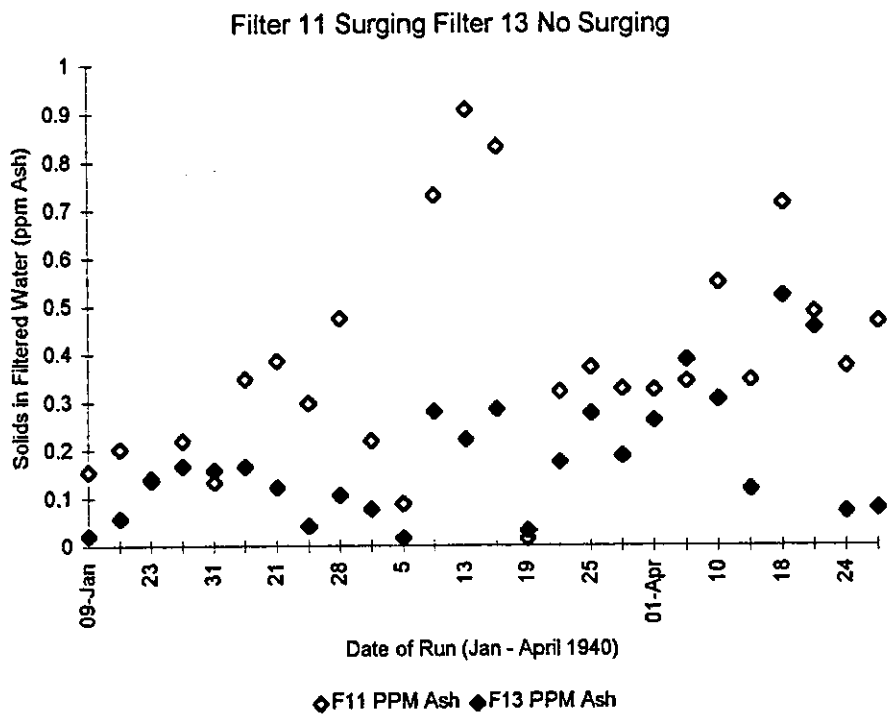


Figure 2.13: Comparison of filter 11 and 13 water quality (winter)
(After Baylis, 1958)

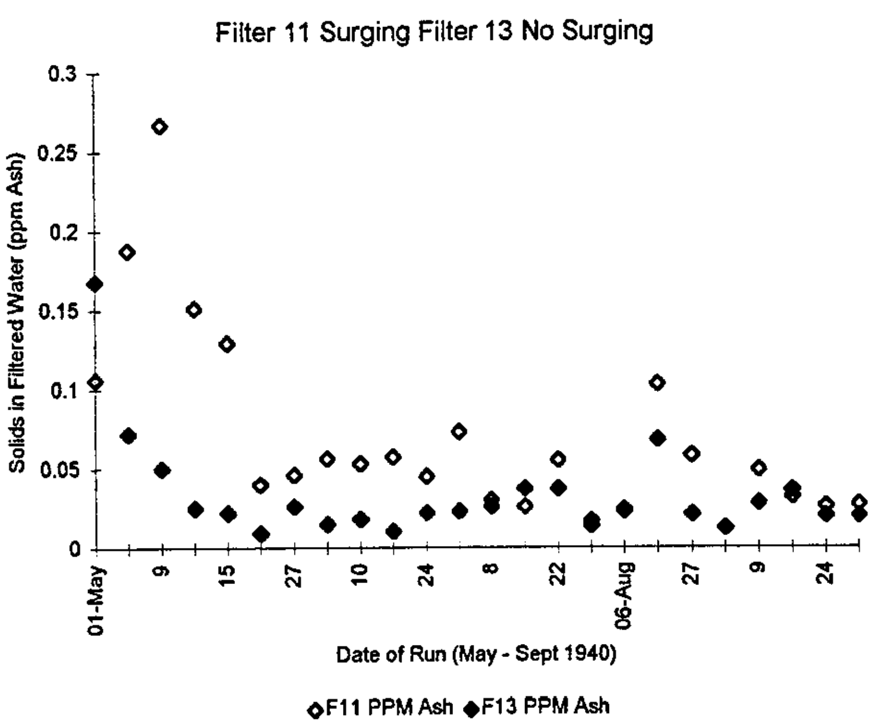


Figure 2.14: Comparison of filter 11 and 13 water quality (summer)
(After Baylis, 1958)

Hudson (1959) compared the performance of constant rate filtration (CRF) and declining rate filtration (DRF) at the Wyandotte filtration plant in Michigan to determine which was the superior method of operation. Two 3785 cubic metres per day (1 megagallon per day) filters were operated at an approach velocity of approximately 5.9 metres per hour (2.4 US gallons per minute per square foot). One filter was operated under CRF conditions using its conventional constant flow rate controller (venturi tube and control valve). The second filter was operated under DRF conditions. This filter had its flow rate controller replaced with a fixed orifice. The orifice was designed to limit the initial flow rate through the filter and allow the flow to decline as the filter accumulated solids. The orifice was sized such that the average flow rate would equal that of the CRF filter. At the beginning of the run the DRF filter rate was 6.6 metres per hour (2.7 US gallons per minute per square foot) and had slowed to 5.2 metres per hour (2.1 US gallons per minute per square foot) when the head loss reached 1524 mm (5 feet). The filters were given a standard backwash at this head loss. The filter sand beds were of similar depth and effective size (0.7-0.8 mm). The filters were compared under these conditions for a period of one year. Hudson monitored the filtrate quality, loss of head and the flow rates for each filter. Filtrate quality was determined by turbidimeters, cotton plug filters and a recording microphotometer. Hudson also determined the Breakthrough Index K, for each filter. The Breakthrough Index (Hudson, 1956), used to describe the filter conditions under which solids breakthrough occurs, is defined in the following equation

$$K = \frac{Vd^3H}{L}$$

where V is the approach velocity (US gallons per minute per square foot), d is the effective size of the sand grains in millimetres, H is the loss of head in feet at the time of the solids breakthrough and L is the thickness of the filter bed in feet. In Hudson's study, the time of breakthrough was defined as the time at which the filtrate turbidity exceeded 0.2 units. K typically exceeded 3 but under periods of weak floc strength fell as low as 0.5. Since the flow rate and thus V declines under DRF, Hudson expected better performance from the DRF filter. Figure 2.15 (a) & (b) shows typical filter run results for the CRF and DRF filters. From part (a), it can be seen that the approach velocity remains constant at 5.9 metres per hour in the CRF filter throughout the run. The head loss rose from 396 mm (1.3 feet) to 1006 mm (3.3 feet) in 19 hours. The filtrate turbidity remained below 0.2 units until breakthrough began at around 14

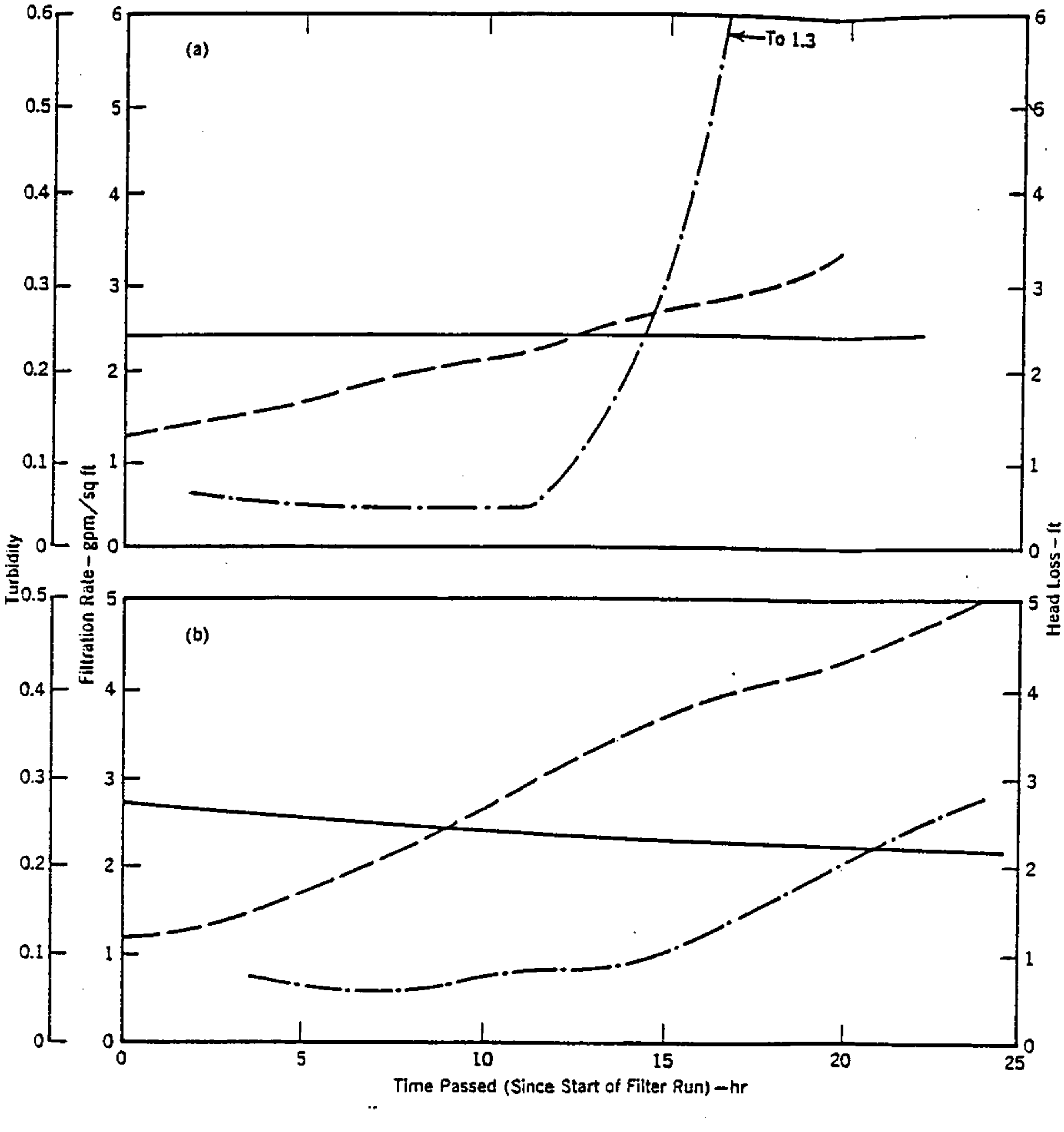


Figure 2.15 (a) & (b): Comparison of constant and declining rate filtration (Hudson, 1959) ______ Flow rate _ _ _ _ Head loss _._._._. Turbidity

hours. The breakthrough index at this point was calculated to be 1.6. The filtrate turbidity continued to rise after this time until it reached 1.3 at the end of the run. From part (b), it can be seen that the approach velocity declined from 6.6 to 5.2 metres per hour in the DRF filter. The head loss rose from 396 to 1524 mm at around 24 hours. The filtrate turbidity remained below 0.2 units until breakthrough began at around 20 hours. The breakthrough index was about 1.6 at this point. The data from these filter runs was used to generate the plot shown in figure 2.16. Here the filtrate turbidity has been plotted against increasing breakthrough index. It can be seen that although the DRF system did not prevent breakthrough from occurring it did slow the rate of rise of the filtrate turbidity with respect to the CRF system performance. From these results it appeared to Hudson that DRF was superior to CRF. However, it should be noted that there were differences in the quality of the raw water applied to the filters but Hudson felt that the results were "nearly comparable". Hudson was aware of Baylis' work on surges in rapid filters. Hudson assumed that the head loss through a filter bed was directly proportional to the rate of flow since the flow in the bed is laminar. As such, surges reflected momentary fluctuations in the filtration rate and the magnitude of the rate fluctuation was equal to the ratio of the surge amplitude to the loss of head. However, it should be noted that this will only hold if the surges are measured within the filter media where the flow is laminar. Hudson recognised that surges could originate at any high velocity zone such as a bend in the effluent piping or a partly closed control valve. Surge amplitudes were determined at Wyandotte for the CRF and DRF filters for a range of head losses although it is not clear where the surge measurements were taken from. It is assumed that the measurements were taken across the filter bed where the flow is laminar. The surge amplitudes recorded are shown in figure 2.17. In the CRF system, the surges were observed to rise with increasing head loss similar to Baylis' findings. Surge amplitudes of 0.9% of the head loss were found at head losses greater than 1524 mm (5 feet) for the CRF filter. However, the amplitudes in the DRF system initially exceeded the CRF filter. The surges in the DRF filter were greater than 0.9% at head losses less than 915 mm (3 feet). This was probably the result of the initially greater flow rate in the DRF system since Baylis found the surge amplitude to be proportional to the square of the flow. However, the surges in the DRF filter fell to 0.67% at head losses greater than 1524 mm (5 feet) as the flow rate declined. Note that the surge amplitudes in both filters

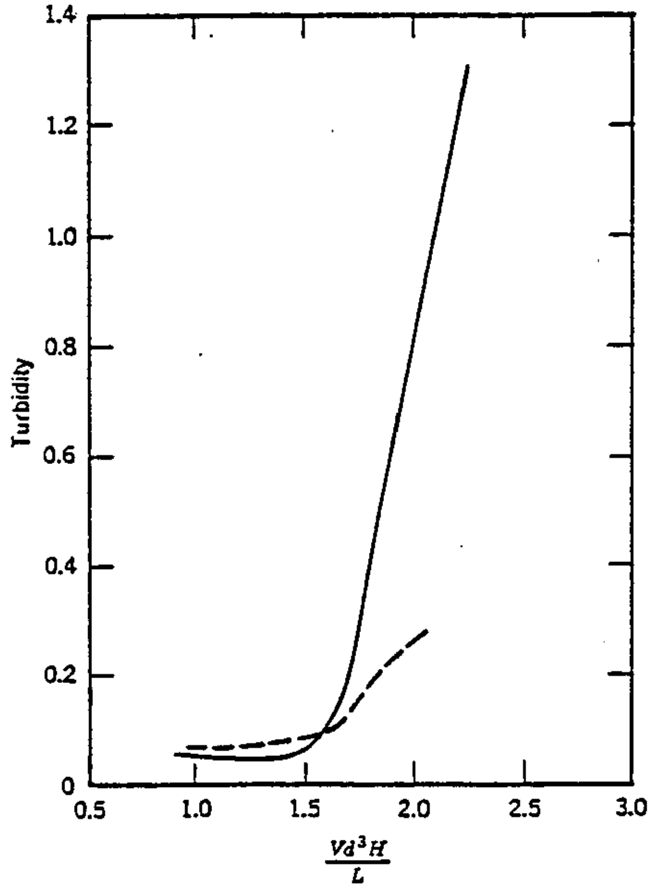


Figure 2.16: Turbidity v breakthrough index (Hudson, 1959)
___ Constant rate filtration _ _ _ Declining rate filtration

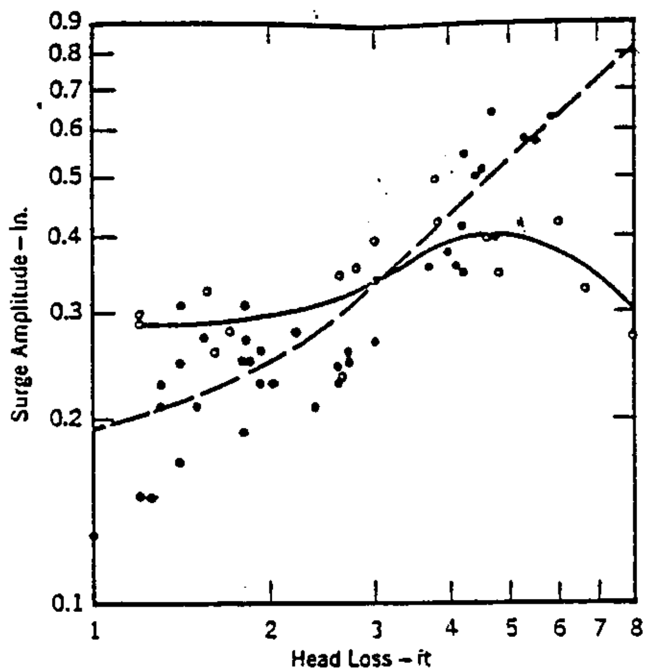


Figure 2.17: Surges in constant and declining rate filtration (Hudson, 1959)
_ _ _ Constant rate filtration ___ Declining rate filtration

were smaller than those typically observed by Baylis. This is probably due to the point where the surges were measured by Hudson. Baylis recorded the surge amplitudes in the effluent piping close to the source of the surges whereas Hudson probably recorded the surges within the filter bed. Hudson believed that the source of the surges in the DRF system was the high velocity zone at the orifice coupled with its close proximity to upstream and downstream bends. The reduction in surge amplitude towards the end of the filter run was due to the drop in the velocity through this region. Hudson believed that the reduction in surge amplitude due to the reduction in filtration rate towards the end of the filter run for the DRF system was responsible for the less rapid rise observed in filtrate turbidity during breakthrough. Hudson believed that an observed improvement of 20% in water quality from the DRF system over the course of the year long comparison with CRF was the result of the smaller surge amplitudes in the DRF system. He also thought that an additional 7% improvement during periods of weak flocculation was brought about by the reduction in breakthroughs in the DRF system. However, he could not state with certainty that the differences observed were due solely to the surging. Hudson noted that the exact relationship between surge amplitude and filtrate quality had not been established. However, it seemed clear to him from Baylis' results and the Wyandotte tests that quality improved when surges decreased.

Hudson (1963) later discussed the criteria used in the design of rapid sand filters. He presented empirical design equations which related various filter characteristics to the filtered water quality and the length of filter run. Included in these equations was a factor for the surge amplitude present. Hudson's equations included a directly proportional relationship between surge amplitude and filtered water solids concentration and filter run length. No data was presented to justify these relationships. No empirical constants were given. He again described surges to be transient fluctuations in the mean rate of filtration caused by boundary flow reversals in the filter piping system. He noted in this paper that surge amplitudes may reach 6% of the mean flow rate and could be increased by hunting of the rate control equipment. It was stated that the effect of surges was not known precisely but it was thought that filtered water quality would be impaired by their presence. Since the surges reduced the retention of solids within the filter it was thought that these fluctuations in flow would lengthen the filter run.

Hudson at a later date (1981) noted that surge amplitudes of 2 to 10% in the filtration rate were common in full scale rapid filters and that certain siphon control systems could cause surges of up to 25% amplitude. In addition, the fluctuations may be further amplified by erratic behaviour of the control system such as hunting of the effluent control valve. However, he made no mention of the frequency of occurrence of such fluctuations.

In this section we have reviewed the evidence that rapid gravity filters are subject to small, continuously occurring fluctuations in the flow rate. Observations and measurements from both experimental and full scale filters have shown erratic surges in flow up to 10% in magnitude at highly frequent intervals. The cause of these erratic flow changes has been identified as the result of turbulent flow conditions in the filtrate piping related to the piping design and appurtenances. There is some evidence that these flow rate fluctuations have a detrimental effect on the quality of water produced by rapid filtration. However, this evidence is ambiguous and has not been confirmed and shown to be reproducible in well controlled experiments. It is not known how great an influence these rate fluctuations may have on filtrate quality and thus public health. It is not clear if filtered water quality and pressure drop vary during the filter cycle when subject to these surges. The effects of different surge characteristics such as the magnitude and rate of occurrence on quality and filter run length have yet to be quantified and the mechanisms critically examined.

2.2 Discrete Flow Rate Changes in Filters

The previous section reviewed the published research into small, continuously occurring fluctuations in the flow rate through filters which have been called surges. It is the purpose of this section to review published research into discrete changes in the flow rate. Discrete flow rate changes are defined to be single step changes in the flow applied at some point during the filter run. These are similar but distinct from surges in that surges are random, erratic flow changes which occur continuously from initiation until termination of the filter run.

Cleasby, in 1960, used the then recently developed turbidimeter to continuously monitor the performance of full scale rapid filters at the Ames treatment plant, Iowa (Cleasby, 1960). During the course of his research he noted the effect of discrete flow rate increases. His observations indicated that any operational practice that resulted in an increase in the rate of flow through a partially clogged filter caused considerable amounts of previously retained solids to be flushed through to the filtrate. He noted that the filter performance recovered after such an increase in flow but previously such turbidity peaks would not have been detected since filtrate quality was only sampled periodically not continuously. Cleasby concluded that continuous turbidity monitoring was a welcome addition to filter management.

Munson, quoted by Cleasby et al (1963), noted that when filter rates were increased or when filters were physically disturbed the filtrate quality deteriorated.

Cleasby and Baumann (1962) compared the performance of three pilot plant rapid gravity filters The filters were operated at different flow rates to determine the effect of higher than normal flow on performance. Each filter was supplied with an on-line turbidimeter and a float valve flow rate controller. During the course of the experimentation the float valves occasionally stuck momentarily. When the valves suddenly released the filters were subjected to an increase in flow. The authors noted that this increase in flow rate flushed previously captured sediment through the filter and resulted in a drop in the loss of head.

Cleasby *et al* (1963) decided to conduct an experimental investigation into the effects of discrete flow rate changes on rapid filter performance. The authors cited the observations of previous researchers on discrete flow changes described above including Baylis' work on surging. The authors decided that discrete flow changes could be an important issue in filter plant operation. Plant practices and control equipment such as filter backwash and automatic rate controllers could cause discrete changes of 50 to 100% in the flow rate through filters. Observations by Denholm (1956), referred to by Cleasby *et al,* suggested that the practise of "Stop-start" filter operation could also result in a deterioration in filtrate quality. Restarting a dirty filter is essentially the same as the application of an increase in flow during a filter run. Cleasby *et al* set out with the following objectives.

- To determine the influence of the magnitude of the rate increase on the quality of the filtrate
- To determine the influence of the duration of the rate increase on quality
- And to determine the influence of the rate of change in flow rate or rapidity on quality

The investigation was conducted on both pilot scale and full scale filter plant. In both cases the filters were operated at a constant rate until a head loss of 1067 to 1524 mm (3.5 to 5 feet) was achieved. At this point the desired flow rate increase was applied and the filtrate quality was monitored

The pilot plant comprised three 152 mm (6 inch) internal diameter sand filters. Each filter was fitted with a flow meter and a turbidimeter. 0.59 to 0.84 mm sand was used in the pilot filters. Two floc suspensions were used in the pilot plant experiments. The first suspension was prepared by mixing ferrous sulphate with tap water followed by aeration and slow mixing. The second suspension was prepared in a similar manner except that 0.5 ppm copper was added in the form of copper sulphate solution. The copper acted as a catalyst to ensure complete precipitation of the iron added. In all runs the influent iron concentration was 10 ppm. 12 test runs were conducted with the three filters monitoring both filtrate turbidity and iron concentration. The iron concentration was determined colorimetrically by the bipyridine method. Where substantial solids were flushed through the filters the total iron passed was determined in addition to the peak concentration achieved. Runs were conducted at approach velocities of 4.9 and 9.8 metres per hour (2 and 4 US gallons per minute per square foot). Flow rate increases from 10 to 100% were applied. The flow rate changes were applied at rates of change or rapidity's from 1 % per minute through to instantaneous. Details of some of the test run conditions are given in Table 2.4.

Run/Filter	*Flow Rate (m/hour.)*	*Flow Rate Increase (%)*	*Rapidity (%/min)*	*Suspension*
2a	4.9	100	Instantaneous	Fe
2b	4.9	100	20	Fe
2c	4.9	100	10	Fe
7a	4.9	100	Instantaneous	Fe,Cu
7b	4.9	100	20	Fe,Cu
7c	4.9	100	10	Fe,Cu
9a	4.9	25	Instantaneous	Fe,Cu
9b	4.9	25	5	Fe,Cu
9c	4.9	25	2.5	Fe,Cu

Table 2.4: Pilot Plant Test Runs (Cleasby *et al,* 1963)

The Ames Municipal Water Treatment Plant was used for the full scale tests. The process train comprised aeration, chemical addition, mixing, settling, recarbonation, filtration, chlorination, fluoridation and metaphosphate stabilisation. 0.63 mm effective size sand was used in the filters. Eleven test runs were conducted with a range of flow rate increases applied with continuous effluent turbidity monitoring. Table 2.5 details some of the tests conducted.

Run	*Filtration Rate (m/hour)*	*Filtration Rate Increase (%)*	*Rate of Increase or Rapidity*
A3	3.13	50	inst.
A5	3.13	50	5%/min
A8	3.13	25	inst.
A9	3.25	50	inst.
A10	2.98	10	inst.
A11	3.25	50	10%/min

Table 2.5: Ames Treatment Plant Test Runs (Cleasby *et al,* 1963)

The authors found that the effluent iron concentration built up rapidly after the application of a flow rate increase in the pilot filters. The iron concentration then peaked and fell to almost the same concentration that existed before the disturbance was applied. This is illustrated in figure 2.18 for pilot plant run 9a (25% increase applied instantaneously). The filter had almost recovered 15 minutes after the rate disturbance was initiated. Similar results were observed in the full scale plant tests. The nature of the effluent iron concentration profile was found to depend on the amplitude of the flow rate increase. Larger rate increases caused greater amounts of material to pass through the filters with a higher peak solids concentration. For example, run 7a passed 5.8% of the total iron applied for a 100% flow increase applied instantaneously with a peak effluent iron concentration of 135 ppm occurring 5 minutes after the disturbance. Whereas run 9a passed only 0.3% of the total iron applied for a 25% flow increase applied instantaneously with a peak effluent iron concentration of 8.5 ppm occurring 8 minutes after the disturbance. Both filters were operated at an initial rate of 2 gpm/sq.ft. with the copper catalysed iron floc suspension in these runs. The effect of the percentage increase in flow rate on effluent peak iron concentration is illustrated in figure 2.19 for 3 different rapiditys (rate increases occurring instantaneously, over 5 minutes and over 10 minutes). It can be seen that at each rate of change the peak iron concentration in the filter effluent is approximately proportional to the cube of the magnitude of change applied.

The amount of material flushed through the filter beds was also found to be dependent on the rate of change or rapidity of the flow rate increase. More rapidly occurring flow rate increases caused more material to penetrate through the filters. For example, run 7c passed only 0.81% of the total iron applied for a 100% flow increase applied at a rate of change of 10% per minute (i.e. 10 minutes for the 100% increase to occur). This gave a peak concentration of 10 ppm occurring 10 minutes after the disturbance was applied. Both the total and peak iron values for run 7c were less than the values from run 7a described above where the same change was applied instantaneously.

The effect of the rapidity of the flow rate increase is illustrated in figure 2.20. This chart shows the peak effluent iron concentration produced by a given flow rate increase (100, 50 or 25%) occurring instantaneously, over 5 minutes and over 10 minutes. They found that for each magnitude of change the resultant peak concentration fell logarithmically with decreasing rapidity of change.

The amount of solids passed through the filter was found to be essentially independent of the duration of the flow increase. That is to say test runs where the rate increase was maintained for a long period of time did not pass significantly more material that runs where the shock change was short.

The authors finally observed that the amount of solids passed through the filters by a given magnitude and rapidity of flow rate change was dependent on the nature of the material being filtered. For example, run 7b a 100% flow increase applied at a rate of change of 20% per minute passed 1.7% of the total iron applied using the copper catalysed iron floc suspension. Whereas run 2b with the same rate disturbance passed only 0.06% of the total iron applied with the iron only floc suspension. Figure 2.21 compares the filter response to a given rapidity of flow rate change for the two suspension types. It can be seen that the copper catalysed iron floc results in a greater effluent concentration than the iron only floc for each rate of change applied during the experimental investigation. It would appear that for the copper catalysed iron floc suspension the effluent iron concentration was approximately proportional to the square of the rapidity of change.

The authors hypothesised from their findings that the dominant force involved in the dislodgement of previously retained material was hydraulic shear. They argued that a balance exists between the forces leading to particle deposition and those forces tending to dislodge attached particles and prevent the deposition of new material. If the deposition forces were greater than the dislodgement forces then the suspended particle would be deposited. If however the shear forces are later increased such that they overcome the attachment mechanism the particle will be forced back into suspension. Once back in suspension the particle may be redeposited deeper within the filter bed once the deposition forces become dominant once more. The authors argued that an increase in the rate of flow of suspension through the filter would result in an increase in the shear forces acting on the previously deposited material causing some to be dislodged and resuspended. In addition the increased shear forces would prevent the attachment of new material arriving in the filter bed. It was thought that the rate of increase in shear forces was dependent on the rate of increase in flow rate. It was thus argued that the rate of dislodgement and build up of material in the filtrate were also dependent on the rate of increase in flow. Secondly, it was thought that the peak shear force acting on the deposited material was dependent on the amplitude of the flow rate increase and that this governed the magnitude of the peak solids concentration appearing in the effluent. The authors discussed the performance differences observed between the two suspension types used in the investigation. It was though that the iron floc suspension created in the absence of the copper catalyst formed a floc with stronger internal bonding. As such the floc may have formed a

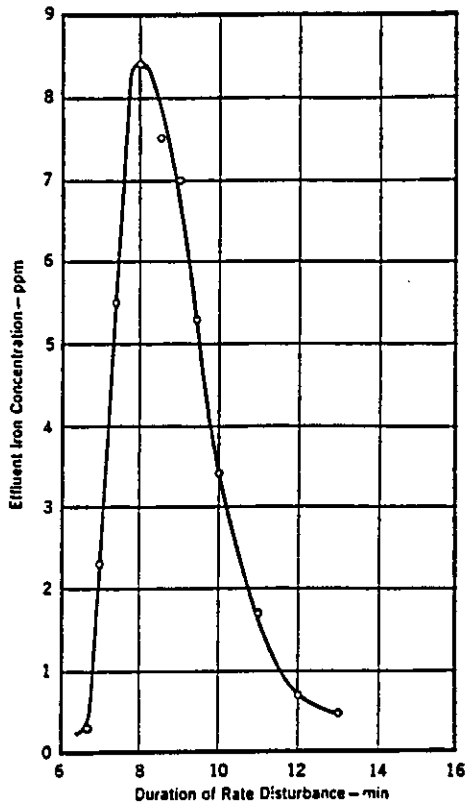
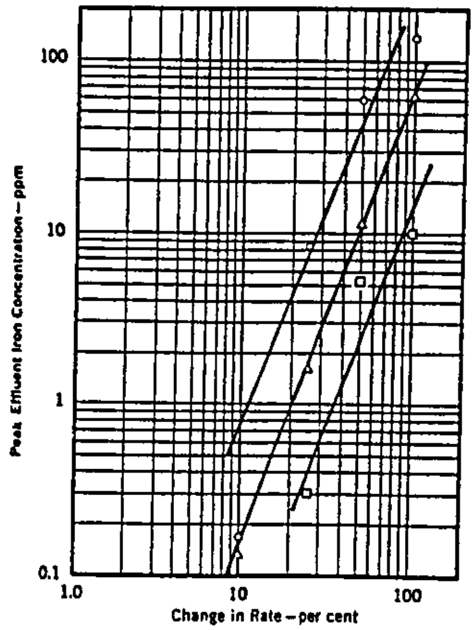
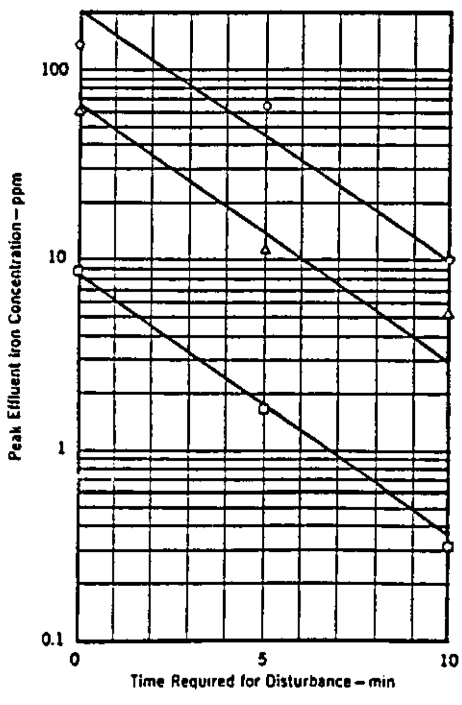


Figure 2.18: Effluent iron concentration spike (Cleasby *et al*, 1963)



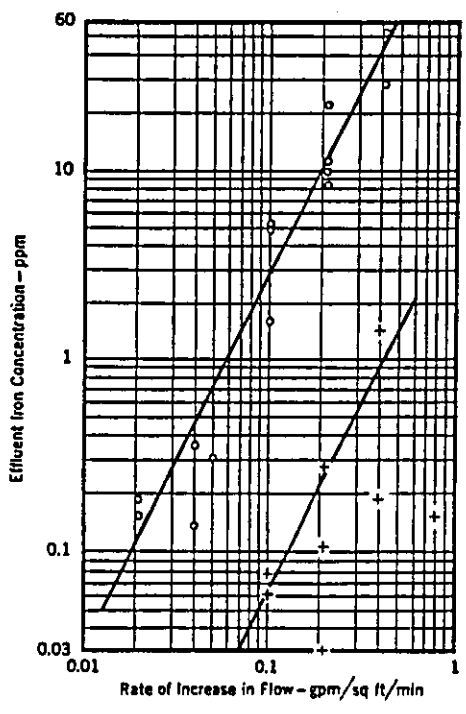
o Instant Δ 5 minutes □ 10 minutes

Figure 2.19: Iron concentration as a function of change in rate (Cleasby *et al*, 1963)



o 100% Δ 50% □ 25%

Figure 2.20:Iron concentration as a function of rapidity of change in flow (Cleasby *et al*, 1963)



o Fe & Cu + Fe only

Figure 2.21:Iron concentration for 2 types of suspension (Cleasby *et al*, 1963)

more robust deposit held together by greater electrokinetic forces with subsequently greater resistance to increases in the shear forces resulting from a flow rate increase. The mechanism of filter recovery after a flow rate increase was discussed. It was thought that filter recovery was the result of two mechanisms. Firstly, dislodged material would be redeposited deeper within the filter bed. Secondly, and perhaps more importantly, was the re-establishment of equilibrium between the shear forces and the attachment forces. It was suggested that the flushing of the interstitial pores by the flow rate increase would then reduce the shear forces by widening the channels. The reduction in shear would then allow redeposition to take place within the pores. It was thought that recovery would take place even if the flow rate were not returned to its previously lower magnitude but with perhaps a marginally poorer quality effluent.

The authors concluded that plant operating procedures should be reviewed to reduce the occurrence, rate of change and magnitude of filter rate fluctuations. It was thought that multiple flow rate changes would probably cause more material to be flushed through the filter bed and that better rate control might help produce better quality water at no additional cost. Cleasby *et al* suggested that the potential for flow rate fluctuations could be addressed in the design of new filtration plant with a view towards their reduction.

Hudson, in a discussion attached to the paper, reviewed and commented on the work of Cleasby *et al* on discrete flow rate changes. He pointed out that the surges described by Baylis although related were a separate matter and quite distinct from the discrete flow rate changes investigated by Cleasby *et al.* Hudson went on to restate that surges consisted of erratic, momentary fluctuations in the flow rate of up to 10% magnitude at frequencies of occurrence from 5 to 100 times per minute. These flow rate changes were present continuously throughout the filter run. Hudson concluded that the quantitative effects of surging on filter performance had yet to be evaluated but were thought to be substantial. In the light of the work of Cleasby *et al* on the effect of discrete flow increases and the comments of Hudson an investigation of the influence of surges on filter performance could prove to be worthwhile.

Hartung and Tuepker (1963), in an assessment of the performance of full scale submerged filters, noted that there were no adverse effects from frequent flow rate changes caused by changes in the outlet pumping rate provided the flow rate changes were made smoothly and slowly. This would appear to be in agreement with the work of Cleasby *et al* discussed above which illustrated the importance of the rapidity of the change in flow. However, surges which seem to occur rapidly in a random and erratic manner may prove to be significant.

Hudson (1963), in his discussion of filter design criteria, highlighted the importance of the rate of change in the filtration rate on filter start up. It was observed that initial breakthrough occurred on "rapid" start up of rapid sand filters. Similar breakthrough was observed when filters were restarted after a temporary halt during a filter run and that this could be eliminated by adopting a "slow" start up procedure where the filter was brought up to full flow over a period of 15 to 60 minutes. Again it seems apparent that the rapidity of a fluctuation in the flow is an important parameter. It was suggested by Hudson that filtration rates should not be changed more rapidly than 3% per minute to minimise any disturbance to the filter deposits. Baylis and Hudson's observations of surging suggest that these flow rate fluctuations occur much more rapidly than 3% per minute.

Conley, quoted by Barnett *et al* (1992), thought that flow disturbances would detach previously captured solids from single and dual media filters and that this material would appear in the filtered water.

A study of full scale filter performance under a variety of operating conditions including flow rate changes was conducted at the St. Louis Filtration Plant (Tuepker & Buescher, 1968). The filters used in the study consisted of 686 mm (27 inches) of 0.62 mm effective size sand upon 305 mm (12 inches) of graded gravel with a Wheeler bottom. Each filter had a surface area of 98 square metres (1056 square feet). The filters were operated at an approach velocity of 4.9 metres per hour (2 US gallons per minute per square foot) with ferric sulphate coagulated raw water with an average inlet solids concentration of 2 ppm. Two parallel filters in the filter bank were compared. One of the filters had polyelectrolyte added to the inlet stream at a concentration of 3 ppb to determine the usefulness of these conditioners. Both filters were subjected to a number of flow rate changes at two different rates of change while monitoring the filtrate turbidity. Figure 2.22 compares the performance of the two filters where several flow changes have been applied during the test run. These flow changes were made over a 10 second period. At point (a) in the chart the approach velocity was increased from 4.9 to 6.125 metres per hour (2 to 2.5 US gallons per minute per square foot). At point (b) the rate was increased further to 8.58 metres per hour (3.5 US gallons per minute per square foot). At point (c) the rate was reduced to 6.125 metres per hour and at (d) was again increased to 8.58 metres per hour. At (e) the flow was returned to 6.125 metres per hour. The dashed line shows the performance of the polyelectrolyte dosed filter. The test run was repeated with identical flow rate changes applied to the two filters at the same points in time except this time the rate changes were applied over a 10 minute period. Figure 2.23 illustrates the filtrate turbidity performance under these conditions. From figure 2.22, it can be seen that the rapid rate changes can induce turbidity breakthrough in the non-polyelectrolyte dosed filter similar to Cleasby *et al.* The filter seems to recover to an extent after the rate disturbance but does not return to its previously lower turbidity if the flow rate is maintained at the higher rate. If the rate is then reduced to the flow before the disturbance was applied the filtrate turbidity recovers further but still does not reach its previous low value. With the polyelectrolyte added to the inlet stream the filter still exhibits some breakthrough in response to the rapid rate increases but to a much smaller extent. Cleasby *et al* also found that the filter response was affected by the nature of the suspension. From figure 2.23 where the rate changes had taken place over a longer period of time, the non-polyelectrolyte dosed filter still exhibited turbidity breakthrough but to a much lesser extent than for the rapidly occurring rate changes. With the addition of polyelectrolyte no breakthrough was evident in response to the slowly changing flow rates. The authors concluded that when flow rate increases are necessary they should be made gradually over a 10 minute period especially at higher flow rates and when the filter has accumulated appreciable deposits. Perhaps the presence of appreciable rapidly occurring surging would have a measurable influence on filtrate quality even with the addition of floc strengthening polyelectrolytes.

Hartley and Vowels (1979) reanalysed the data of Cleasby *et al* regarding the effect of flow rate increases on filtrate quality. From their analysis the authors concluded that the maximum increase in the solids concentration of the filtered water was dependent on the combination of both the magnitude of the flow rate increase and the rate of change of the flow rate increase. The authors found that

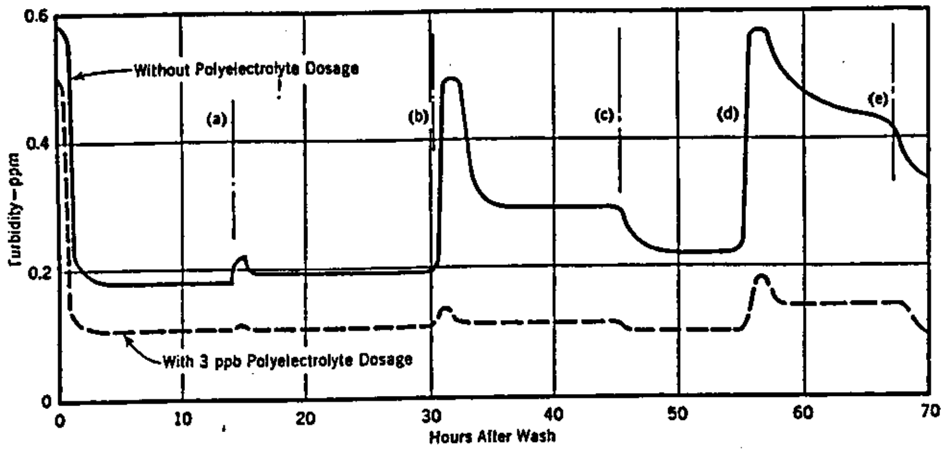


Figure 2.22: Effect of rapid flow changes on filter performance
(Tuepker & Buescher, 1968)

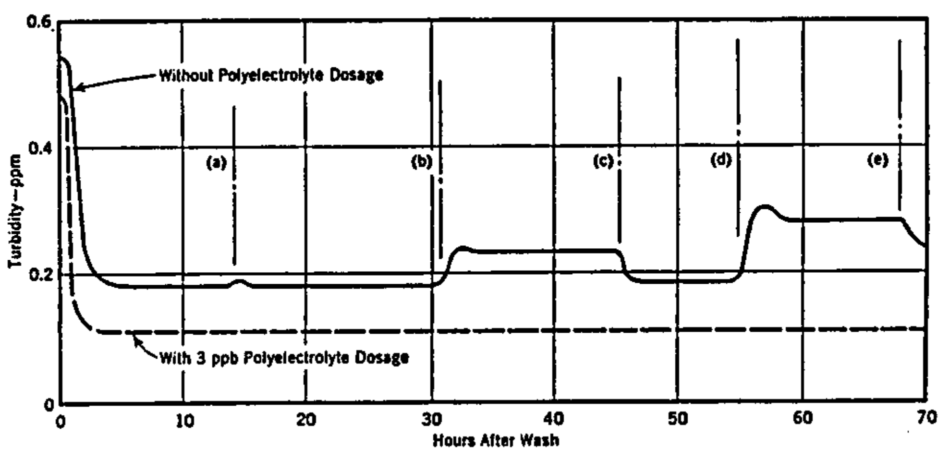


Figure 2.23: Effect of slow flow changes on filter performance
(Tuepker & Buescher, 1968)

the initial flow rate before application of the rate increase apparently had no effect on the maximum increase in solids concentration in the filtrate. Finally, Hartley and Vowels stated that reattachment of dislodged material was significant and increased as the rate of change in flow rate decreased. In this light, the presence of surging since it occurs continuously and rapidly may have a more significant effect than a discrete change in the flow by preventing reattachment of dislodged matter. The authors presented their own results of tests conducted on pilot and full scale filters. Pilot scale dual media filters were operated at 14.7 metres per hour and then subjected to 12.5 and 25 % instantaneous flow rate increases at varying times into the run. The filters were operated with an alum coagulated river water. The filtered water turbidity was monitored and the results are illustrated in figure 2.24. It was found that as the filter run progressed and the filter bed accumulated solids the effect of an instantaneous rate increase was more pronounced. Similar to Cleasby *et al* the larger magnitude rate change resulted in a greater increase in maximum filtrate turbidity. Rate changes applied to full scale plant did not show any significant change in filtered water quality. The authors explained that this was due to the relatively small amount of deposits captured by the full scale filters when the rate increases were applied. A design chart was created by Hartley and Vowels from their own data and the results of Cleasby *et al* and is shown in Figure 2.25. The authors intended this chart to illustrate the importance of the age of the filter when a rate increase was applied in addition to the magnitude and rate of change of the flow increase. It may be the case then that the influence of surging on filter performance is dependent on the volume of deposits captured too.

Bernardo and Cleasby (1980) reviewed the results of previous work on the effects of rate changes and compared the performance of constant and declining rate filtration to determine which produced the better quality filtrate. Previous research into declining rate filtration had shown uncertain results. Baylis (1959) found that the DRF system produced neither better nor worse quality filtrate than CRF but found that water production was higher. Hudson (1959) however found both better quality filtrate and longer run lengths from the DRF system which he attributed to smaller surging. The authors set out to confirm the result by a pilot plant comparison of CRF and DRF under well controlled conditions. The CRF system comprised a single 101 mm diameter filter filled with 381 mm (15 inches) of 0.95 mm effective size anthracite on

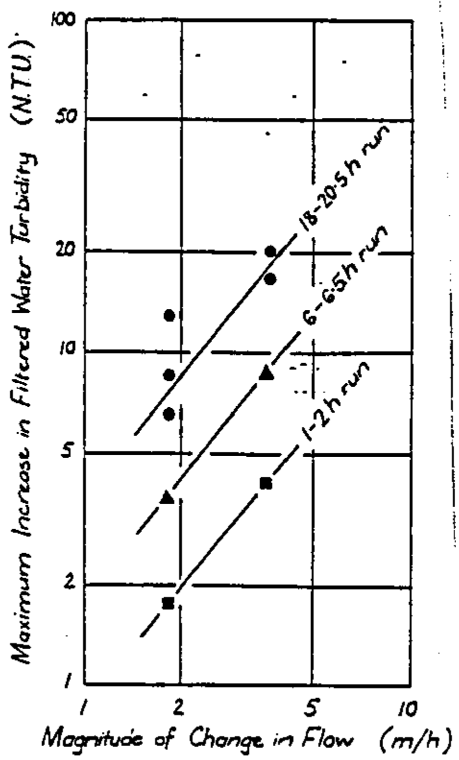


Figure 2.24: Filtrate turbidity v change in flow (Hartley & Vowels, 1979)

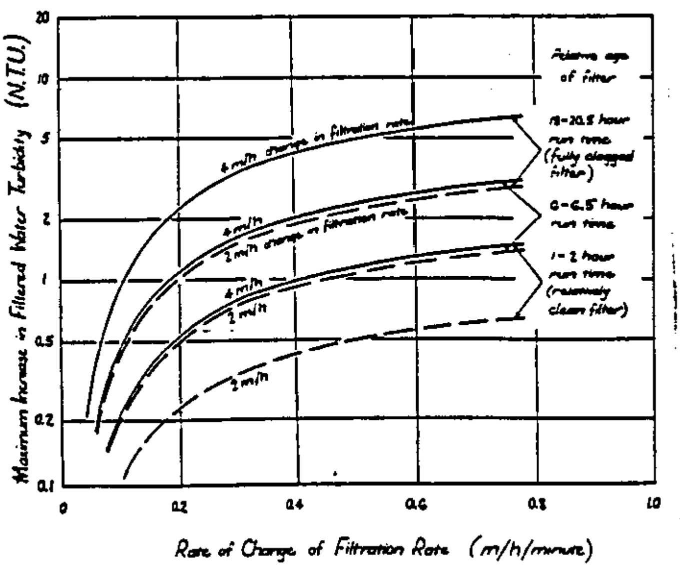


Figure 2.25: Effect of rate changes at different filter ages (Hartley & Vowels, 1979)

top of 229 mm (9 inches) of 0.5 mm effective size sand supported on a perforated underdrain. The flow rate was controlled by a float valve rate controller fitted to the effluent piping. The head loss and filtrate turbidity were monitored continuously. The DRF system used comprised four 153 mm diameter filters again filled with anthracite and sand plumbed to operate under the DRF principle without the need for effluent rate controllers. Again the head loss and turbidity were recorded continuously. The authors found that the DRF system consistently produced a better quality filtrate than the CRF system at a range of different flow rates. On average the DRF system produced 30 to 60 % lower filtrate turbidity seeming to confirm the findings of Hudson. The authors also noted the improved response of the DRF system when subjected to a flow rate increase when a filter was taken out for backwash. It was found that the gradually changing flow rate in the DRF filters had a less detrimental effect on filtrate turbidity than the sudden rate increases applied to the CRF filter. In a later study, Hilmoe and Cleasby (1986) compared CRF and DRF again. Similarly a single constant rate filter was compared with a bank of four declining rate filters. The constant rate filter comprised a 102 mm diameter column filled with 356 mm (14 inches) of 1.4 mm effective size anthracite on top of 254 mm (10 inches) of 0.52 mm effective size sand. Unlike the previous study (Bernardo & Cleasby, 1980) the CRF filter was not fitted with a float valve rate controller in the effluent piping. CRF was achieved by delivering a constant inlet flow from a splitter box. Both systems were monitored for loss of head and filtrate turbidity continuously. The authors found that there was no apparent difference in filtrate quality between the two systems contradicting the findings of the previous study. The authors offered as a possible explanation for this result the fact that in the previous study the CRF filter was fitted with a float valve effluent flow rate controller. The authors suggested that this controller may have subjected the CRF filter to "small but continuous fluctuations of back pressure" which may have resulted in the poorer filtrate quality observed. The authors cited the comments of Hudson (1981) on surging to support this claim.

Trussel *et al* (1980) noted the effect of stop-start filter operation on filtrate quality. A pilot plant filter was operated for a period of 200 minutes to achieve equilibrium before being shut down. The filter was allowed to rest for 5 minutes before restarting without backwashing the partially clogged media. The effluent particle count was observed to increase 70 fold as illustrated in figure 2.26. The elevated particle count quickly recovered to its previous value 10 minutes after the restart. These observations correlate with previous researchers' findings regarding discrete flow rate increases. The authors concluded that restarted filters should either be filtered to waste for a few minutes or that the number of restarts should be limited to 5 or 6 in a given day. It would be preferable not to operate filters in this manner at all.

Logsdon *et al* (1981) conducted an assessment of the removal efficiency of cysts of *Giardia sp.* by a rapid dual media filter. *Giardia sp.* is a water borne protozoan parasite which causes diarrhoea in humans and has been the cause of a number of drinking waterborne outbreaks (Lin, 1985). The filter used in the study comprised a 38 mm internal diameter column filled with 460 mm of 1.27 mm effective size coal on top of 150 mm of 0.36 mm effective size sand. An alum flocculated suspension, either alone or with cationic polymer, was spiked with *Giardia muris* cysts and passed through this filter. Head loss and filtrate turbidity were monitored and samples were collected for cyst enumeration. The effect of flow rate increases was assessed as part of the study. Rate increases of 50, 100 and 150 % were applied in less than 10 seconds to simulate sudden flow increases at treatment plants. In one such test the approach velocity was increased from 11 to 27 metres per hour for a period of 2 minutes and then returned to 11 metres per hour once more. The effect on filtrate turbidity and cyst count is illustrated in figure 2.27. The filtrate turbidity rose sharply from 0.25 to 1.0 nephelometric turbidity units (NTU) and rapidly declined after the flow had been reduced similar to previous investigations discussed earlier. Cyst concentration similarly rose in response to the rate increase from around 40 to almost 1000 cysts/litre before returning to normal. Thus a four fold increase in turbidity was mirrored by a 25 fold rise in cyst concentration. Logsdon *et al* concluded that rate increases can adversely affect effluent quality and that *Giardia sp.* cyst concentrations increase when turbidity rises. However cyst counts were observed to rise more than the turbidity. The authors finished by stating that turbidity increases as low as 0.05 to 0.1 NTU were associated with large increases in cyst concentrations. As such filtration alone cannot be considered an effective barrier to *Giardia sp.* cysts. As such it may be the case that the presence of small, continuously occurring fluctuations

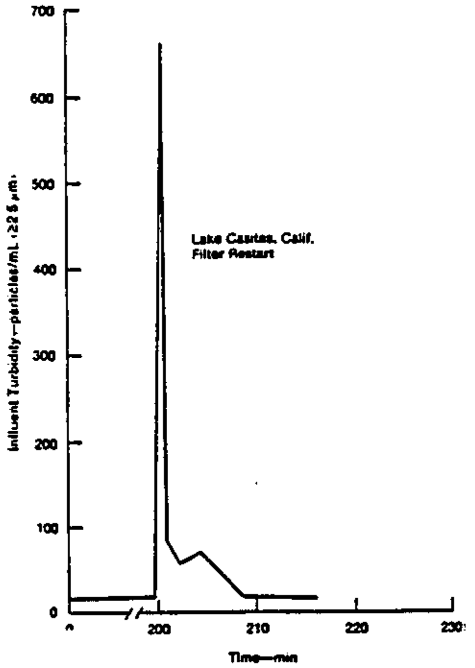


Figure 2.26:Effect of restarting a dirty filter (Trussel *et al*, 1980)

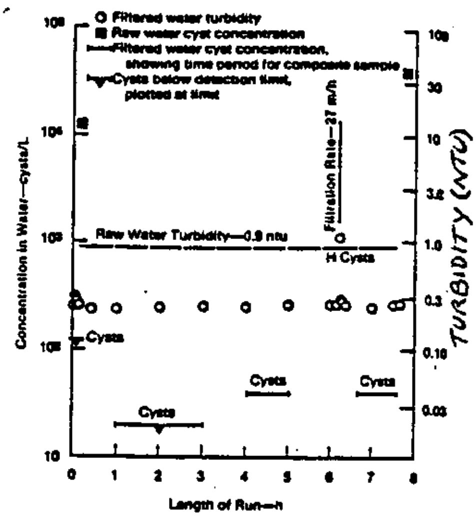


Figure 2.27:Effect of rate change on turbidity & cysts (Logsdon *et al*, 1981)

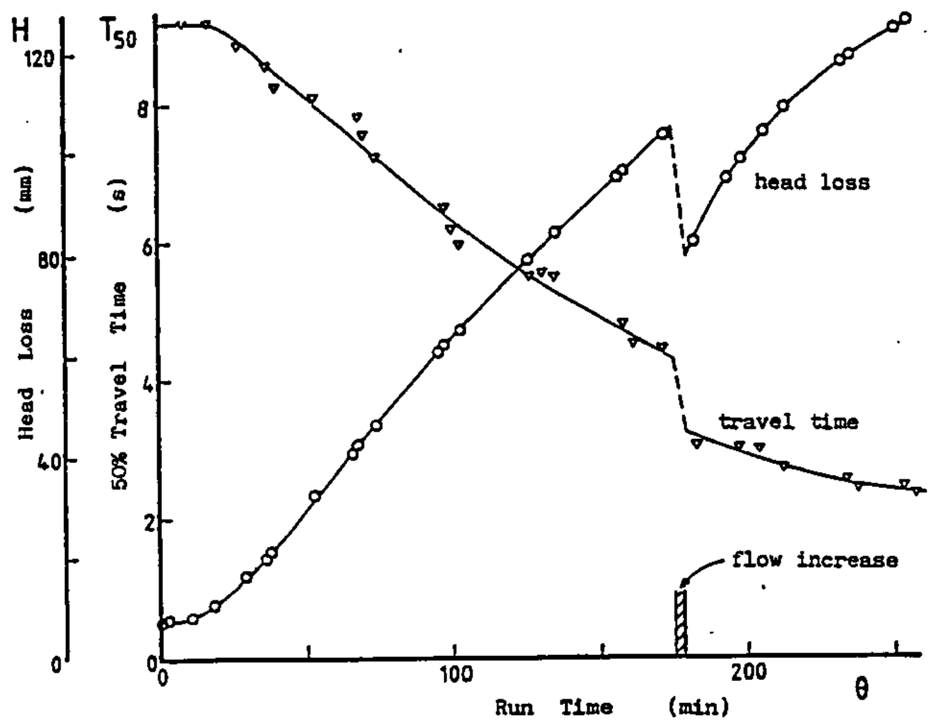


Figure 2.28: Effect of 45% flow increase on head loss & travel time (Coad, 1982)

or surges in flow may have a similar effect on the removal performance of pathogens such as *Giardia sp.*.

A tracer technique was developed to calculate the specific deposit captured in a clogging filter bed (Coad, 1982). Conductance electrodes were placed immediately above and below a 20 mm deep laboratory sand filter. The sand filter was operated a constant flow rate controlled by a float valve fitted to the effluent piping. A saline tracer was injected into the turbid suspension above the sand at frequent intervals during a test run and the conductance curves were recorded at each electrode. The 50% travel time and dispersion number were then calculated from the recorded conductance curves. The specific deposit was calculated from the measured travel time. Coad used this technique to investigate the effect of sudden flow rate increases on the filter. Figure 2.28 illustrates the response of the filter to a sudden flow increase of 45% applied midway through the run. The rapidity of the rate change was not given. It can be seen that as the filter accumulated solids the head loss rose and the travel time fell before the rate increase was applied. Application of the flow rate increase resulted in a drop in both the head loss and travel time. Coad expected the head loss to fall since it was thought that the flow rate increase would scour previously retained deposits from the bed returning the bed to a higher porosity and thus a lower head loss. However, the drop in travel time was unexpected. It was originally thought that the scouring of deposits from the bed would result in a drop in the interstitial velocity and thus produce an increase in the travel time. Coad suggested that the drop in travel time was the result of modifications to the flow pathways through the clogging media. He hypothesised that the sudden increase in flow had indeed scoured deposits from the bed but had done so mostly from the larger flow channels. These larger modified flow channels would then carry a greater fraction of the flow through the clogging media at a greater velocity which would result in a reduced travel time for the tracer. Coad argued that if this hypothesis were true then the dispersion number for the flow through the filter bed would show an increase as a result of the scour indicating that the flow pathways had become more dissimilar. Coad calculated the dispersion numbers for a test run where several flow rate increases were applied to the filter and compared them with a control run with no rate increases. Figure 2.29 illustrates the calculated dispersion numbers against head loss for the two runs. As the filter accumulated deposits and the head loss increased the dispersion number was found to rise indicating that the flow pathways through the clogging sand were becoming more dissimilar. However, with the application of the flow rate increases it was found that this process was accelerated. The dispersion number rose more rapidly indicating that the flow pathways became more dissimilar more rapidly as a result of the scouring from the rate increases. This result appeared to Coad to confirm his hypothesis. The author concluded that sudden increases in the flow rate reduced the effectiveness of the filter by diverting more of the flow through a smaller number of larger channels. These scoured larger channels effectively provide a short circuit path for the influent suspension through the clogging layer of the filter bed. Coad noted that the flow rate fluctuations in full scale filters were much smaller than the rate changes he investigated but occurred much more frequently. It may be the case that these smaller, frequently occurring fluctuations or surges have an influence on the development of flow pathways through clogging media in a manner similar to the discrete rate increases assessed by Coad.

It was noted that an abrupt increase in the flow rate can degrade filtrate quality (Logsdon & Fox, 1982). These authors stated that the shear forces acting on deposited floc within a filter bed were proportional to the velocity within the filter pores. An increase in the flow rate causes an increase in the interstitial velocity and shear stress acting on the retained floc. This is similar to the hypothesis presented by Cleasby *et al* in 1963. Floc deposited at a lower flow and shear stress may be washed out of the media at the higher flow rate. It is probable that the momentary higher shear stresses resulting from the surging described by Baylis and Hudson would have a similar effect or would inhibit attachment in the first place. The authors presented the result of the application of a flow rate increase to a rapid dual media filter operated at 9.8 metres per hour (4 US gallons per minute per square foot). The filter was operated for 4.5 hours before doubling both the flow rate and coagulant dose. The turbidity and coliform count of the raw and filtered water were monitored throughout the filter run and are shown in figure 2.30. The filtered water turbidity rose from 0.2 to 0.5 NTU after the rate increase. The filtered water coliform count increased from 200 to 2000 coliforms per 100 ml after the flow rate change. Similar to Logsdon *et al's* (1981) work with *Giardia sp.* cysts, the coliform count increased by a factor much greater than the turbidity. The authors noted that filters can be subjected to flow increases when one filter in a bank is removed for backwashing. The authors suggest

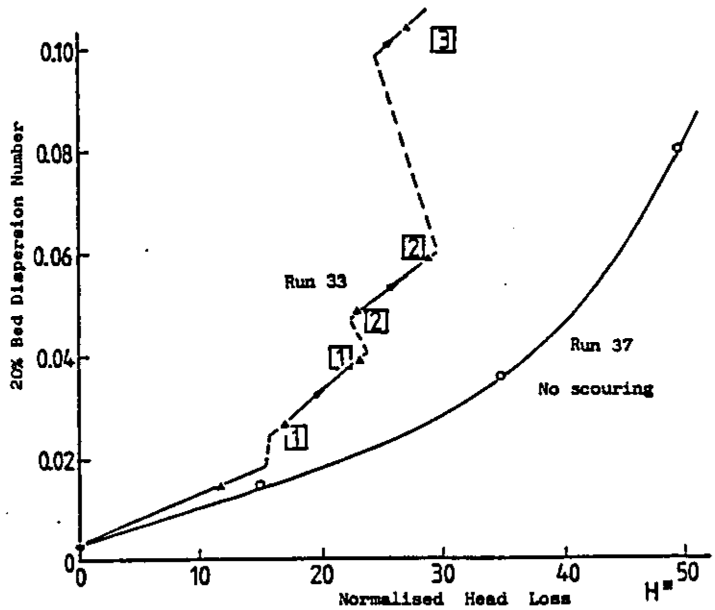


Figure 2.29: Effect of rate changes on dispersion number (Coad, 1982)

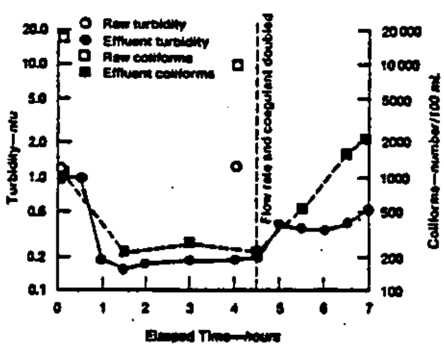


Figure 2.30:Effect of flow doubling on turbidity & coliforms (Logsdon & Fox, 1982)

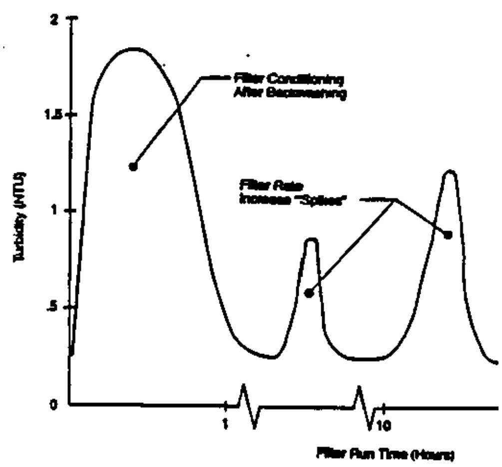


Figure 2.31: Filter rate increase turbidity spikes (Mains, 1992)

that these rate increases should be made slowly to avoid a deterioration in quality. The authors also noted that slow sand filters do not respond well to rapid flow rate changes.

Schleppenbach (1984) assessed the effect of flow rate changes on the performance of the rapid filters at the Duluth Plant, Minnesota. The filters were subjected to stop-start operation and flow changes of around 30 to 35 % in 60 seconds. Changes in effluent quality were observed but were found to be short term similar to previous research. No data was presented.

Logsdon (1987) presented a guide to the evaluation of particulate removal performance at filtration plants. He stated that filters should be run until terminal head loss is reached without stopping the filter mid run and restarting without backwash. He suggested that close attention should be paid to rate changes since rapid, "jerky" step changes in flow would likely be detrimental to water quality. It has been noted previously that surges are random and erratic or jerky in nature. The author recommended that the condition of flow rate controllers should be checked periodically to ensure smooth operation. Logsdon *et al* (1990) noted the role of poor filter operation in the occurrence of an outbreak of Cryptosporidiosis from drinking water which will be discussed later in section 2.3.

Amirtharajah (1988) discussed fundamental concepts of rapid filtration theory. He reviewed the findings of Cleasby *et al* on the effect of flow rate changes and suggested that the absence of effluent flow rate controllers in declining rate filtration was one reason for the believed superior performance of declining rate over constant rate filtration. Amirtharajah closed by noting that filtrate quality is governed by transient behaviour such as rate changes when the influent water is effectively pre-treated.

Ongerth (1990) studied the performance of three small treatment plants including the effects of flow rate changes on the removal of *Giardia sp.* cysts. Ongerth identified several sources of rate changes in the treatment plants. These included stop-start events in response to demand (50 to 75 events between backwashes), flow modifications imposed by modulation of the effluent control valves and variations in the filtered water reservoir levels. Ongerth found little change in the filtered water quality as a result of the flow fluctuations applied in contrast to the findings of Cleasby *et al* and others discussed previously. This result was attributed to the low turbidity of the raw water and relatively small amount of deposits accumulated by the filters when the rate changes were applied.

Barnett *et al* (1992), as part of a wider study, compared the performance of a dual and mixed media filter when subjected to flow rate increases. Both filters were operated at a constant rate before increasing the flow by 33 % to simulate the effect of one filter in a bank of four being taken out of service for backwash. The filtrate quality was recorded continuously by on-line turbidimeters. No significant change was observed in the filtrate turbidity from either filter as a result of the increase in flow. However, it is not clear from the authors' results how rapidly or slowly the flow rate was changed. Previous work reviewed has indicated that this is important.

Mains (1992) discussed the likely impact of regulatory changes on the operation of filtration plant in Canada. He predicted that individual filter effluent quality will be required to meet regulatory standards and not just the average blended water quality from the filter bank. As such, greater attention will be paid to individual filter performance in the future. Mains recognised that effluent turbidity from an individual filter can rise momentarily as a result of a flow rate increase. Figure 2.31 illustrates the response of a conventional rapid filter supplied with alum coagulated raw water to flow rate changes. Flow increases can be caused by an increase in the overall plant loading or by filter backwashing. Mains suggested that flow increases should be planned and controlled and thus applied in stages over a period of time to minimise the rise in filtrate turbidity.

Recent outbreaks of Cryptosporidiosis associated with drinking water (see section 2.3) led to the creation of a checklist to be used to assess and optimise water treatment plant performance (Bellamy *et al,* 1993). With regards to the filtration stage the authors made the following recommendations.

- Avoid sudden flow rate changes
- Establish as low a turbidity goal as possible (0.1 NTU say)

- Use continuously monitoring turbidimeters for each filter
- Slow start filters after backwash
- Avoid restarting a dirty filter

The authors concluded that treatment plants should be optimised to produce as high a quality finished product as possible to minimise the risk of waterborne disease to the public.

Stevenson (1997) developed a mathematical model of deep bed filtration. The filter bed was considered to be composed of a matrix of five different grain sizes unlike previous models which typically considered the bed to be composed of single sized media. In addition, the suspension was modelled as a mix of five different particle sizes instead of a single particle size. The author created a simulation program of a simplified filter bed from these theoretical principles. The model was used to simulate the effect of flow rate changes on filter performance. 33% flow rate increases were applied in the model every 6 hours to simulate the result of filter backwashing in a bank of four filters. Figure 2.32 illustrates the model output under these conditions. The increase in flow resulted in a spike in the filtrate quality and an increase in the head loss. On returning the flow to the previously lower rate the head loss fell. It can be seen that the head loss fell to less than it's previous value suggesting that deposit scour had occurred. However the head loss quickly recovered to its previous level. As the simulation progressed and the filter bed acquired greater deposits the effect of the flow rate increase became more pronounced on both the filtrate quality and the head loss. The author noted that the simulation seemed to predict this feature of real filter behaviour spontaneously from the theoretical principles inherent in the model. This suggests that inclusion of heterodisperse media size and suspension size as factors in filtration modelling is a valid and useful approach. It would be interesting to make use of such a model to try and predict the response of a rapid filter to surging of the nature observed by Baylis and Hudson discussed previously.

In this section we have seen that rapid filters can be subjected to discrete changes in the flow rate of 33 % or more. These step changes in flow are caused by changes in the plant loading, modulation of the flow control valves and the effects of filter

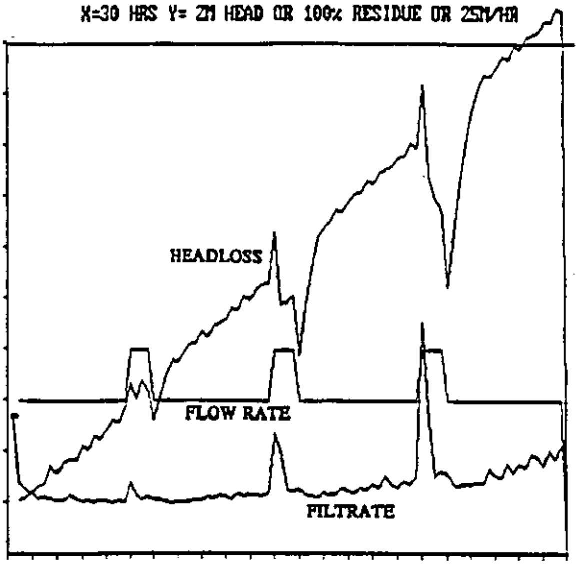


Figure 2.32: Simulated effect of flow rate increases (Stevenson, 1997)

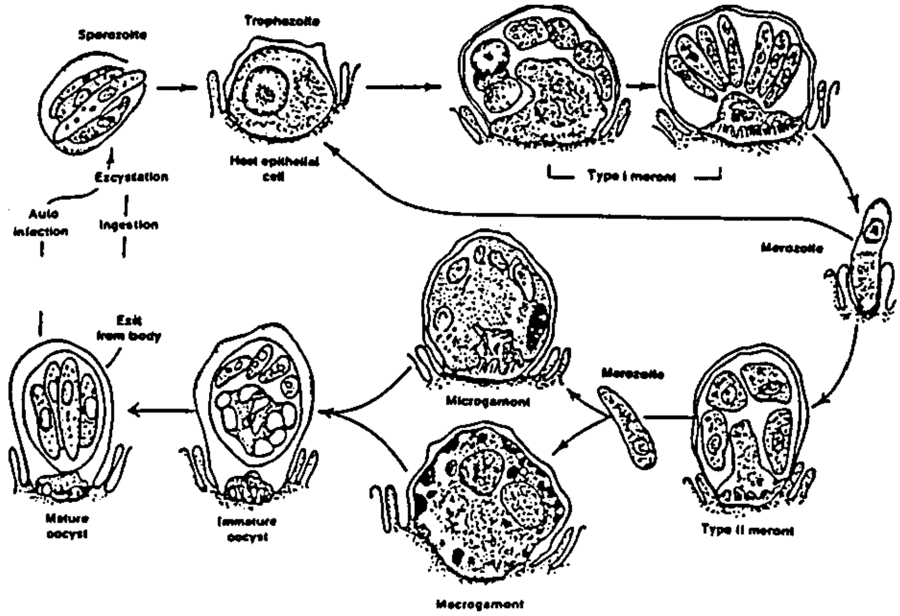


Figure 2.33: Cryptosporidium life cycle (Rose, 1988)

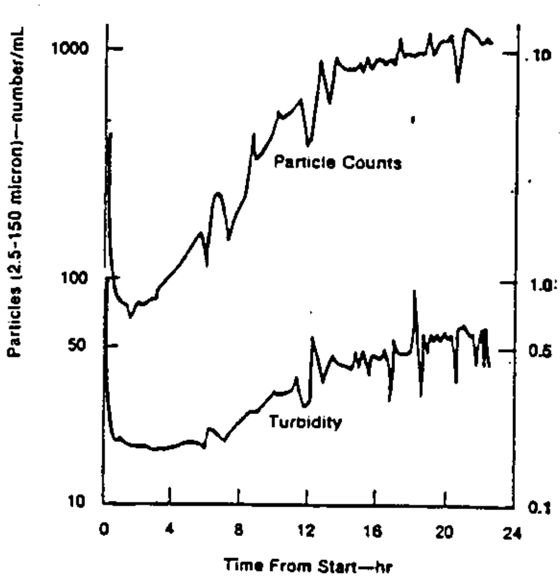


Figure 2.34: Comparison of turbidity & particle counts >2.5um (Beard & Tanaka, 1977)

backwashing. It has been found that such increases in flow rate caused short term spikes in the solids concentration of the filtered water and short term decline in the head loss. Tests have also shown selective modification to flow pathways through the filter bed. The magnitude of the effect has been found to be dependent on the amplitude of the rate increase, the rapidity of the rate increase and the volume of captured solids present in the bed but independent of the duration of the increase in flow. It would appear that the rise in flow rate caused an increase in the interstitial velocity within the filter pores and this created greater shear stress on the previously captured deposits, some of which broke off and penetrated further into the filter bed. It is not known whether surges would influence filter performance in some manner similar to these effects from discrete flow changes. Surges are generally smaller but occur continuously and very rapidly. It seems likely that surges will cause continuously occurring fluctuations in interstitial velocity and shear stress on the deposits. It is not known whether these fluctuations are sufficient to alter filter performance. They may prevent attachment of incoming suspended particles and detachment of previously captured material, modifying the pressure drop and possibly the flow pathways. Since surges occur continuously they may cause long term changes in filter performance which are less obvious than the short term effects of discrete changes in flow.

2.3 *Giardia sp.* and *Cryptosporidium sp.* in Drinking Water

Giardia sp. and *Cryptosporidium sp.* are disease causing pathogens widely present in the water environment. They have both been associated with a number of outbreaks of illness originating from the public water supply over the last 20 years or so. It is the purpose of this section to review the nature and occurrence of these organisms in water, the outbreaks which have been documented and assessments of their removal by water treatment processes. Where appropriate comments are made with respect to the issue of surging in rapid filtration.

Logsdon *et al* (1981) reported that there had been no research on water filtration for *Giardia sp.* cyst removal before 1981. It was around this time that the *Giardia sp.* organism was identified as a source of drinking waterborne disease. Previous studies of water filtration had been carried out on another pathogenic protozoan; *Entamoeba*

Histolytica and Logsdon discussed this research with respect to *Giardia sp.* removal. This research demonstrated that *Entamoeba sp.* cysts could be removed by filtration and suggested that filtration should also be capable of removing *Giardia sp.* cysts effectively. Logsdon conducted a study of *Giardia sp.* cyst removal by dual media filtration the results of which have been discussed and noted previously in section 2.2 with respect to flow rate increases. Logsdon concluded that properly operated filtration should remove a substantial proportion of cysts but not all.

Lin (1985) presented a review of current knowledge of the *Giardia sp.* organism including its life cycle, outbreaks associated with drinking water and the effectiveness of water treatment processes in removing and deactivating it. *Giardia Lamblia* was identified some 300 years ago but was not considered pathogenic. Only in the 1960's and 70's has it been recognised as a cause of diarrhoea (Giardiasis) in humans. *Giardia Lamblia* is a flagellated protozoan with two basic forms; the trophozoite and the robust cyst. The trophozoite parasitises the intestines of the host organism. In unfavourable conditions the trophozoite encysts and it is in this form that the illness is transmitted between hosts. Cysts of *Giardia sp.* are robust ellipsoids 8 to 14 microns long and 7 to 10 microns wide. Cysts are passed in large numbers by an infected host. As many as 14 billion cysts can occur in the stool but typically numbers around 300 million are found. Infection of a new host occurs by ingestion of cysts which subsequently hatch out trophozoites in the intestines. Infection is not fatal in a healthy host but can be extremely uncomfortable. Transmission of the cysts can occur by ingestion of contaminated drinking water. Lin noted that the organism is not host specific. That is to say that cysts from humans can infect dogs, cats and beaver and it is assumed vice versa. Experiments with volunteers have found that only a small number of cysts present in water can cause infection of a new host. Prevention of infection is by interruption of the transmission route and this should include adequate and properly operated water treatment facilities. A large number of outbreaks have been reported in the USA, many associated with treated drinking water. For example, in 1974, an outbreak of waterborne Giardiasis occurred in the city of Rome, New York. An estimated 5000 people were affected and *Giardia Lamblia* cysts were identified in the treated water. Water treatment of the city's supply was limited to chlorination only. A residual chlorine concentration of 0.8 mg/l was used. The source of the contamination of the surface supply was thought to be from human and animal waste. In 1976, a Giardiasis outbreak occurred in Camas, Washington. Some 10% of the population of 6000 people were infected. *Giardia Lamblia* cysts were again detected in the raw surface water and in the treated water distribution system. The original source was believed to be the beaver population present in the watershed. The water treatment facilities involved included pressure filters and chlorination. Cysts were believed to have passed through the filters because of media loss, media disruption and poor pre-treatment. However, the treated water was found to meet the standards for turbidity and coliform counts. An outbreak occurred in Berlin, New Hampshire in 1977 again associated with poor filtration performance. 7000 people out of a population of 15000 were affected. Two surface water sources were used and *Giardia sp.* cysts were recovered from both and from the treated water. Both plants had filtration; one pressure filtration and the other rapid gravity filtration. Deficiencies in the pressure filters could have allowed cysts to pass through and leakage allowed unfiltered water to bypass the rapid filters in the second plant. 5% of the population of Bradford, Pennsylvania were affected by an outbreak of Giardiasis in 1979. Cysts were isolated from the raw and treated waters. The treatment facilities used chlorination only. From these examples it seems clear that Giardiasis outbreaks associated with drinking water can occur due to defects in equipment and lapses in operation. Lin stressed the need for adequate filtration and chlorination to protect drinking water supplies. Normal concentrations of chlorine did not appear to be effective against *Giardia sp.* cysts. Higher doses and longer contact times were needed to be effective against *Giardia sp.* especially in the absence of an effective filtration barrier. Lin reviewed the work of Logsdon, discussed above, on the effectiveness of filtration in removing *Giardia sp.* cysts and noted that rate increases caused elevated turbidity and cyst counts. In this light, it is possible that the presence of surging in filtration plant may contribute to the passage of pathogens into the treated water system.

An outbreak of Giardiasis occurred in Colorado in 1981 (Braidech & Karlin, 1985). 12% of the population served by a small treatment plant were affected by the outbreak. Examination of the treatment facilities identified several problems including poor maintenance of the rapid filters. Perhaps of most concern was the fact that neither the coliform counts nor the turbidity sampling of the finished water indicated any problem with the treatment facility. The authors concluded that operation and maintenance programs in small plants should be upgraded to prevent a recurrence.

An outbreak of Giardiasis occurred in the winter of 1983-84 in McKeesport, Pa (Logsdon *et al,* 1985). Some 342 members of the local population were confirmed with the illness. It was found that the filters at the local water treatment plant had been run for several days without backwashing and a large-scale breakthrough of turbidity had occurred. Cysts of *Giardia sp.* were subsequently detected in the raw and treated waters. Alarmingly, the coliform counts were not found to be particularly high. Logsdon *et al* compared the performance of the anthracite filters used at the plant with other media in terms of turbidity and cyst removal. The authors found that the anthracite filters performed the poorest of the media assessed but noted that cysts were usually detected in the filtered water irrespective of media used. The authors were concerned that although cyst concentrations detected in the filtered water were only 3-10 cysts per litre when the filters were operated properly, this could be sufficient to cause infection. It could be the case that the presence of unobserved surging in a rapid filter apparently operating properly assists the penetration of low numbers of particles and pathogens into the filtered water. The authors recommended that care should be taken to ensure effective disinfection by using a sufficient free chlorine residual and a sufficient contact time thus maintaining an effective multiple barrier to *Giardia sp.*. The multiple barrier approach to water treatment was further stressed at a conference discussion on water quality held at the time (AWWA, 1985).

Bellamy *et al* (1985a) assessed the performance of slow sand filtration in removing *Giardia sp.* cysts . The authors found that slow filters operating at rates of 0.04 to 0.4 metres per hour using 0.28 mm effective size sand achieved virtually 100% removal of *Giardia sp.* cysts once biologically mature. A second study, investigating the effect of low temperatures and different sand sizes, found again virtually 100% removal of *Giardia sp.* cysts for each operating condition from biologically mature slow sand filters (Bellamy *et al,* 1985b). The authors concluded that slow sand filtration is an effective barrier to *Giardia sp.* and should be considered during process selection. Seelaus *et al* (1986) came to a similar conclusion. From this, it would appear that slow sand filters using smaller size sand are a more effective barrier to pathogens than rapid filters.

Mosher and Hendricks (1986) assessed the performance of pilot scale rapid filters in removing turbidity and *Giardia sp.* cysts. They found that rapid filtration can effectively remove cysts of *Giardia sp.* provided operated with "proper" coagulation and rapid mixing. When operated with poor coagulation large numbers of particles including *Giardia sp.* cysts penetrated the filters and were present in the treated water. Similar conclusions were reached by Horn *et al* (1988) in an assessment of dual stage filtration for *Giardia sp.* removal. It is likely that even when operated with effective coagulation small numbers of pathogens if present in the influent will appear in the filtrate.

Rose (1988) reviewed the nature and occurrence of *Cryptosporidium sp.* in water. *Cryptosporidium sp.* is a pathogenic protozoan with a complex life cycle (see figure 2.33) which causes diarrhoeal disease in humans and animals. Infection occurs by ingestion of the infectious stage - the oocyst. The oocyst releases sporozoites after excystation in the intestines of the host. The sporozoites infect epithelial cells of the gastrointestinal tract. Asexual reproduction takes place with further parasitisation of the tract cells. Sexual reproduction also takes place producing the oocysts which are passed out in large numbers in the faeces. The oocysts are round to oval spheres 4-6 microns in diameter (Garcia *et al,* 1987). Rose highlighted several aspects of the disease which enhance its capacity for waterborne transmission. Firstly, a single species of the organism may be responsible for illness in both humans and animals including cattle, sheep, swine, goats, dogs, cats, deer, foxes, beaver, rabbits and more. Evidence has shown cross transmission from humans to animals and vice versa (Fayer & Ungar, 1986). Secondly the excreted oocyst is robust and environmentally stable outside the host organism. Another factor important in the transmission of the disease is the minimum dose required to produce infection in a new host. Data has been presented showing that infection of primates could occur with as little as 10 oocysts (Miller *et al,* 1986). These three factors enhance the possibility of waterborne transmission of the illness. Several outbreaks of Cryptosporidiosis have occurred and are reviewed later in this section. Rose reported on a survey of waters for *Cryptosporidium sp.* from the western United States. 107 samples were taken from various sources including raw sewage, treated effluent, rivers, lakes and reservoirs. Of the samples analysed, 77 % were positive for *Cryptosporidium sp.* oocysts with a range of concentrations from 0.04 to 1732 oocysts/litre. In addition, samples of drinking water were analysed for the presence of *Cryptosporidium sp.* oocysts with several positive low concentration results. The results of the survey suggested that *Cryptosporidium sp.* was widespread in the environment and that drinking water treatment facilities could be presented with varying concentrations of the parasite in raw waters. Rose stated that the efficiency of water treatment processes in removing or deactivating oocysts of *Cryptosporidium sp.* was unknown at this time (1988) but investigations were underway. One survey of a drinking water treatment plant reported that 2906 oocysts/litre were detected in the filter backwash water (Rose *et al,* 1987). This indicated that oocysts were concentrated in the filter deposits. Rose also noted that *Cryptosporidium sp.* may be extremely resistant to routine disinfection practise but had yet to be evaluated. Rose concluded that *Cryptosporidium sp.* is a waterborne pathogen but that current limited knowledge and inefficient detection techniques (1988) make it difficult to determine the true extent of the problem. It may be that turbidity and coliforms are inadequate indicators for *Cryptosporidium sp.*. Since it has been shown previously that discrete rate changes can cause an increase in the concentration of cysts of *Giardia sp.* in filtered waters (see section 2.2), it is likely that discrete rate changes will have a similar effect on *Cryptosporidium sp.* oocysts perhaps more so since they are smaller in size than cysts of *Giardia sp.*. It could also be the case that if surging is present in a rapid filter the surges could contribute to the penetration of low concentrations of both *Giardia sp.* and *Cryptosporidium sp.* oocysts into the filtrate. Even a small increase in the average filtered water turbidity resulting from surging could be associated with a disproportionately larger increase in the concentration of these pathogens if present in the influent.

Amirtharajah, (1988) in a discussion of phenomenological and trajectory theories of filtration, pointed out that the combination of diffusion and sedimentation transport mechanisms resulted in a minimum transport efficiency for suspended particles approximately 1 micron in size. This was first identified in water filtration in 1971 (Yao *et al,* 1971). Amirtharajah drew attention to the fact that *Cryptosporidium sp.* oocysts of 3-5 micron size are close to this minimum transport efficiency. This will be discussed further in section 2.4.

In January 1987, an outbreak of Cryptosporidiosis occurred in Carrollton, Georgia (Hayes *et al,* 1989). Some 13,000 people were affected and *Cryptosporidium sp.* oocysts were identified in the stools of the sufferers and in the treated drinking water. The performance of the local water treatment plant before and during the outbreak was reviewed (Logsdon *et al,* 1990). The treatment plant comprised alum flocculation, sedimentation followed by rapid gravity filtration through sand and anthracite at 4.9 metres per hour (2 US gallons per minute per square foot). The review of the facilities and operating procedures identified a number of problems that may have contributed to passage of oocysts of *Cryptosporidium sp.* into the filtered water supply. It appeared that common practise was to operate the filters on a stop-start basis depending on demand. Before and during the outbreak, many of the filters were stopped and restarted on a number of occasions without being backwashed before returning to service. This practise was observed during the plant assessment and it was noted that restarted filters produced poorer quality filtrate similar to the findings of others discussed in section 2.2. In one case the filtered water turbidity was 1.6 NTU, more than 10 times the quality standard of the time. The investigators were reminded of the work of Logsdon *et al,* (1981) and Cleasby *et al,* (1963) on flow rate changes discussed earlier in section 2.2 where filter disturbances caused turbidity breakthrough and elevated cyst counts in the filtrate. The stop-start operation of the filters probably caused solids breakthrough in the weeks before and during the outbreak but was not detected since filtered water turbidity was sampled only periodically and not continuously. The disinfection stage would not have proven effective against any oocysts present in the filtered water since it had been shown that oocysts of *Cryptosporidium sp.* are highly resistant to chlorine (Campbell *et al,* 1982). From the investigation, a list of recommendations was produced. It was advised that dirty filters were not returned to service. They should be backwashed first. On restart, the filters should be brought on line slowly. It was recommended that any flow rate changes be made gradually to minimise deposit disturbance and that a filtered water turbidity goal of 0.1 NTU be adopted. It was recommended that continuous flow turbidimeters be fitted to each filter. Monitoring of the plant by Hayes *et al,* (1989) after the adoption of some of the investigators recommendations showed a large improvement in water quality and no recurrence of oocysts in the water supply.

An outbreak of Cryptosporidiosis occurred in the Swindon and Oxford areas of England in February 1989 (Richardson *et al,* 1991). Some 516 cases were identified and oocysts were detected in the stools of the sufferers. The geographical distribution of infection led suspicion to fall on the water supply. Samples were collected from the local water treatment works at Farmoor and tested for the presence of *Cryptosporidium sp.* oocysts. The treated water from the plant was found to contain 0.66 oocysts per litre and 34 % of samples taken from customers' taps in the area supplied by Farmoor were tested positive with concentrations from 0.002 to 24 oocysts per litre. As a result of the Oxford/Swindon outbreak, an expert group chaired by Sir John Badenoch was established to gather information on the organism. An interim report was delivered in July 1989 (Anonymous, 1989). The authors of the report recognised that discrete changes in the rate of flow through filters should be avoided since they may dislodge deposits including oocysts of *Cryptosporidium sp.*. No mention was made of the possible influence of surges on filter performance. The interim report concluded that contamination of the water supply has the potential to infect large numbers of the population and that existing treatment technology cannot prevent the passage of oocysts entirely. The full report was delivered in July 1990 (Badenoch, 1990). The report presented a comprehensive review of knowledge of *Cryptosporidium sp.* to date, much of which is discussed in this section. The well documented outbreaks of Cryptosporidiosis associated with drinking water supply in the UK and USA were reviewed. Additional information regarding the Oxford/Swindon outbreak was presented. It was reported that although only 500 cases were confirmed it was believed that up to 5000 people were affected. Treatment at Farmoor comprised conventional rapid sand filtration followed by chlorine disinfection. The final water had an acceptable turbidity of 0.2-0.4 NTU with a residual chlorine concentration of 0.5 mg/l. Investigations at the plant found 0.002 -14 oocysts/litre in the reservoir water but 10000 oocysts/litre in the backwash water from the sand filters. Backwash water was treated by settlement but was found to achieve only 83% removal of oocysts present. The highest concentration of oocysts found in the investigation was 10^6 oocysts/litre in the settled backwash water. Settled backwash water was recycled to the head of the works. The authors stated that this practise in conjunction with the influent oocyst load would have resulted in an average daily load of some 10^9 oocysts on each of the sand filters. The authors reported that although the filters achieved over 2 log removal (greater than 99%) of oocysts this would not have prevented penetration of substantial numbers through the filters to the treated water. It was noted that monitoring of the filtered water turbidity, chlorine demand and plate counts before and during the outbreak did not indicate any problems with the treatment process. It could be the case that if surging were present in a rapid filter, it could produce a small increase in the average filtered water turbidity of say 0.1 to 0.2 NTU without causing any turbidity alarm. However, even a small increase in turbidity has been shown to be equivalent to a large increase in the number of particles present in the filtrate. Under such conditions a filter challenged with a large number of oocysts of *Cryptosporidium sp.* in the influent would pass substantial numbers into the filtrate. In this light surges may be more significant than has previously been thought. The report drew a number of recommendations with regard to the operation of treatment plant. As mentioned in the interim report, the authors reiterated the need to avoid sudden increases in the flow rate through filters since the increased shear can wash deposits through to the filtrate. As such, it was recommended that rapid filters are not restarted after shutdown without backwashing. On-line turbidity monitoring should be installed on each filter to provide early warning of breakthrough and the use of coagulant aids should be considered to assist floc strength and thus oocyst retention. The authors concluded that current treatment practise appears able to prevent contamination of the water supply by *Cryptosporidium sp.* oocysts when operated optimally and assuming that the level of contamination of the raw water is low. However, current treatment practise cannot guarantee removal of all oocysts and since it is believed that the infective dose in humans is small there is cause for concern. Packham (1990) presented a summary review of the Badenoch report and Ives, at an IWEM seminar on Cryptosporidia, suggested a target of less than 100 oocysts per 100 litres to prevent outbreaks. Ives went on to say that filtration was essential since settlement and chlorine disinfection alone were ineffective (IWEM, 1991).

It was reported that there are doubts about the ability of conventional water treatment including filtration to deal with heavy contamination of surface water resources with *Cryptosporidium sp.* from agricultural and sewage effluent sources (Pendrous, 1990). The author noted that rapid filtration has widely replaced slow sand filtration in water treatment but was recognised to be less effective at removing *Cryptosporidium sp.* oocysts. Concerns over the ability of the oocysts to resist conventional chlorine disinfection had led to increasing interest in alternative disinfection technology such as ozone and ultraviolet treatment. These may prove more effective in the protection of public water supply from pathogens. However, chlorine is currently the most widely used method of large scale water disinfection and will continue to be used for the foreseeable future in drinking water supply.

Hill (1991) reported on *Cryptosporidium sp.* research at University College London (UCL). Tests of oocyst removal by slow sand filtration achieved only 93% before the formation of a Schmutzdecke layer. This stressed the importance of biological maturity in slow sand filtration for effective protozoan removal. Filtrate turbidity measurements showed that low turbidity did not necessarily mean no oocysts were present in the filtrate.

An outbreak of Cryptosporidiosis occurred in the Isle of Thanet in January 1991 (Joseph *et al,* 1991). No oocysts were identified in the untreated raw water or treated drinking water. However, statistical analysis of gathered data from the affected population showed a strong correlation with consumption of tap water. The source of infection was believed to be contamination of abstracted river water by runoff from local farmland coupled with poor turbidity removal performance at the treatment facilities. The authors speculated that land disposal of human and animal excreta by slurry spreading may be a source of surface water contamination leading to elevated oocyst loads entering treatment plants. It was noted that conventional water treatment may not be able to cope with high concentrations of oocysts in the raw water. It was also noted that low turbidity was not necessarily indicative of an absence of oocysts since turbidimeters were less sensitive to particles in the 1 to 10 micron range (see section 2.4).

LeChevallier *et al* (1991) assessed the occurrence of *Giardia sp.* and *Cryptosporidium sp.* oocysts in filtered drinking water in the US and Canada. Samples were collected and analysed from 66 water treatment plants using an immunofluorescence technique. *Giardia sp.* cysts were detected in 17% of the samples with concentrations ranging from 0.29 to 64 cysts per 100 litres. *Cryptosporidium sp.* oocysts were detected in 27% of the samples with concentrations ranging from 0.13 to 48 oocysts per 100 litres. However, microscopic observation of the cysts and oocysts suggested that most were not viable and therefore not capable of causing infection. This suggested to the authors that the treatment processes in some way inactivated the organisms but they could not say how. The treatment plants found to be positive for cysts in the filtered water had an average filtrate turbidity of only 0.19 NTU, less than the 0.5 NTU prescribed by the Surface Water Treatment Rule in the US (USEPA, 1989). Thus low turbidity water does not necessarily mean an absence of parasites. The authors stressed that the treatment plants evaluated were well run and well maintained. An update to the survey was presented in 1995 and reported lower rates of contamination of filtered water (LeChevallier & Norton, 1995). The authors believed this was due to lower raw water concentrations of *Giardia sp.* and *Cryptosporidium sp.* and improvements in turbidity removal at the treatment plants.

Rose *et al* (1991) conducted a survey of water supplies for the presence of *Cryptosporidium sp.* and *Giardia sp.* cysts. 188 samples from rivers, lakes and springs across the USA were evaluated. 55 % of surface water samples were tested positive for *Cryptosporidium sp.* with an average concentration of 43 oocysts per 100 litres. 16 % of the samples were positive for *Giardia sp.* cysts with an average concentration of 3 cysts per 100 litre. The authors noted that the highest counts were detected in waters receiving agricultural and sewage discharges and that no correlation was observed with other indicator organisms. The highest counts were found to occur in the Autumn. 36 drinking water samples were collected and analysed. 17 % were tested positive for *Cryptosporidium sp.* oocysts with concentrations of 0.5 to 1.7 oocysts per 100 litres. The authors concluded that there was a risk of waterborne transmission of *Cryptosporidium sp.* and *Giardia sp.* infections since the organisms were common in surface waters used for supply and had been detected in treated waters.

Smith (1992) presented a review of *Cryptosporidium sp.* in water. It was noted that *Cryptosporidium sp.* produced self limiting gastro-enteritis in immunocompetent patients but in the immunocompromised such as AIDS patients and the malnourished could prove fatal. Smith stated that concentrations of 13,700 oocysts per litre had been detected in crude sewage and up to 149,000 oocysts per litre in untreated effluent from a Scottish slaughterhouse. In addition, oocysts had been found to survive for over 12 months at low temperatures in laboratory experiments. Smith stressed the need for a multiple barrier approach to protect against *Cryptosporidium* *sp.* in water since recent outbreaks had shown the organism to be chlorine resistant and able to survive up to 16,000 mg/l free chlorine in the laboratory. Smith highlighted the limitations of coliforms as indicator organisms since oocysts had been detected in final waters free from coliforms. Smith thought that since the infective dose was believed to be small, low level contamination of drinking water supplies with *Cryptosporidium sp.* could result in large scale infection of consumers. It was concluded that current limitations in techniques of *Cryptosporidium sp.* recovery and enumeration would lead to an underestimation of oocyst contamination. Secondly, since no validated method of viability determination existed for small numbers of oocysts, each detected organism in treated water should be considered potentially infectious.

An outbreak of Cryptosporidiosis occurred in Jackson County, Oregon in 1992 (Leland *et al,* 1993). It was reported that 43 cases were identified and there appeared to be an association with the drinking water supply. Concern centred upon a local water treatment plant. Treatment comprised coagulation, dual media filtration followed by granular activated carbon (GAC) filtration and post chlorination. Investigations at the plant revealed that the filtered water quality was only just able to meet the Surface Water Treatment Rule limit of 0.5 NTU. Samples taken from the plant failed to yield any oocysts of *Cryptosporidium sp.* but did show excessive amounts of debris in the filtered water. The poor plant performance was attributed to poor coagulation and flocculation which allowed considerable turbidity to penetrate the dual media filters to be removed in the GAC filters. Improvements were made to the coagulation/flocculation process which improved the filtered water quality to 0.1 NTU. It was subsequently found that the incidence of illness in the local community declined to normal within two weeks. So it seems that to effectively guard against pathogen outbreaks turbidity should be maintained below 0.1 NTU.

Smith *et al* (1993) conducted a survey of Scottish waters for the presence of *Giardia sp.* cysts. Samples of surface and tap waters were collected from across Scotland. In addition, samples of the raw and final waters were taken from 21 water treatment plants. Cysts were detected in 49 % of the surface water samples with concentrations up to 1.05 cysts per litre. 19 % of tap water samples were tested positive for *Giardia sp.* cysts with concentrations up to 1.67 cysts per litre. 43 % of raw treatment plant samples and 19 % of final water samples contained cysts of *Giardia sp.*. Cyst viability was assessed by microscopic observations. Viable cysts were found in both raw and final waters. Smith *et al* concluded that viable cysts of *Giardia sp.* may be ingested by the public with drinking water. The authors made interesting speculation from one set of results. It was found at one treatment plant that cyst concentrations in the final water was more than 10 times the raw water concentration. The authors recognised that this result may have been due to limitations in the technique. However, if true, the authors suggested that the higher concentration in the final water could be due to turbidity breakthrough from the filters releasing previously captured cysts into the final water.

An outbreak of Cryptosporidiosis occurred in Milwaukee in 1993 (MacKenzie *et al*, 1994). An estimated 400,000 people were infected with the organism resulting in over 100 deaths. The source of the infection was traced to a local water treatment plant. It was believed that oocysts present in the raw surface water source entered the plant and were ineffectively removed by the treatment train. Possible sources for the *Cryptosporidium sp.* contamination of the surface water were cattle, slaughterhouses and human sewage. Examination of the plant records identified periods of poor turbidity removal by the flocculation and filtration processes. Prior to the outbreak, turbidity from the filters was maintained below 0.25 NTU. However, periods of increasing and high turbidity up to 1.7 NTU occurred in the weeks leading up to the outbreak. The plant was closed once the outbreak was confirmed. Investigations at the treatment facility did not identify a reason for the poor removal performance. There was no evident mechanical breakdown of the flocculators or filters. A possible factor may have been poor targeting of the coagulant dose. Actions were taken at the plant to improve turbidity removal. Modifications to the coagulant dosing system were made to improve floc strength. Recycling of filter backwash, a source of concentrated oocysts, was discontinued and continuous turbidity monitoring was installed for each filter. A limit of 0.3 NTU was set for the turbidimeters resulting in automatic shut down if exceeded. In addition, frequent particle counts were initiated for the raw and treated water. The authors recommended that a turbidity goal of less than 0.1 NTU be adopted to guard against a recurrence of such a massive outbreak.

Ongerth and Pecoraro (1995) evaluated the removal performance of a pilot scale multimedia filter for *Cryptosporidium sp.* and *Giardia sp.* cysts. They found around 3 log removal of both organisms with good flocculation of the raw water using alum. This corresponded with a filtered water turbidity of less than 0.1 NTU. However, when operated with a less than optimal dose of coagulant the removal of cysts and oocysts fell to approximately 1.5 log with a filtered water turbidity of 0.36 NTU. The authors concluded that effective and consistent coagulation was necessary to achieve 3 log removal of protozoans with a corresponding turbidity of less than 0.1 NTU. The presence of surges or discrete rate changes may make it difficult to maintain such a low turbidity filtrate.

The occurrence of *Giardia sp.* and *Cryptosporidium sp.* in South African waters was determined (Kfir *et al,* 1995). Samples of raw sewage, treated effluent, surface water and drinking water were collected and analysed for the presence of cysts and oocysts using a fluorescence technique. 30 % of sewage samples were positive for both *Giardia sp.* and *Cryptosporidium sp.*. Some 25 % of treated effluents tested positive for both organisms. 13 % of drinking water samples contained parasites.

Edzwald and Kelly (1998) stressed the importance of an integrated multiple barrier approach to potable water supply to guard against *Cryptosporidium sp.* transmission. The authors presented data which illustrated lower *Cryptosporidium sp.* levels in protected surface water sources such as pristine lakes and protected reservoirs. Unprotected sources such as polluted lakes and rivers presented treatment facilities with higher raw water concentrations of pathogens. Thus the first barrier to control *Cryptosporidium sp.* should be source protection. The authors assessed the performance of various treatment technologies including clarification and filtration. It was reported that proper coagulation is critical for effective *Cryptosporidium sp.* removal by clarification and filtration. The combination of coagulation with dissolved air flotation followed by filtration achieved up to 5 log removal of *Cryptosporidium sp.*. The authors concluded that filtered water turbidity of less than 0.1 NTU would suggest good control of *Cryptosporidium sp.*. As stated earlier, surges and discrete rate changes may make it difficult to achieve and maintain this target even with effective coagulation.

A survey of 6 conventional water treatment plants in Germany was conducted over a four year period (Karanis *et al,* 1998). The raw water, final water and samples from intermediate steps in the processing were assessed for the presence of *Giardia sp.* and *Cryptosporidium sp.* oocysts. The authors found *Giardia sp.* and *Cryptosporidium sp.* in 76 % of the raw water sources. *Giardia sp.* cyst concentrations ranged from 0.1 to 1314 per 100 litres and *Cryptosporidium sp.* oocyst concentrations ranged from 0.2 to 1081 per 100 litres. 15 % of final water samples were positive for *Giardia sp.* and 30 % for *Cryptosporidium sp.*. *Giardia sp.* concentrations in the final waters were from 0.2 to 17 cysts per 100 litres. *Cryptosporidium sp.* concentrations in the final waters were from 0.13 to 21 oocysts per 100 litres. Samples of the filter backwash waters were tested for parasites. 84 % were positive for *Giardia sp.* at average concentrations of 33 cysts per 100 litres. 82 % were positive for *Cryptosporidium sp.* with average concentrations of 22 oocysts per 100 litres. From their analyses the authors concluded that the occurrence of these parasites was similar to published studies in the UK and USA but reported that no outbreaks from the water supply had yet been recognised in Germany. Their results showed that *Giardia sp.* and *Cryptosporidium sp.* were capable of penetrating conventional water treatment plants including rapid filtration with no indication of poor final water quality. The authors stressed the need to adopt and maintain a multiple barrier approach to water supply to protect against these organisms. The authors recommended that surface water sources be protected to prevent contamination. Treatment plants should aim to produce a filtered water with a turbidity of less than 0.1 NTU and should avoid recycling filter backwash water. The efficiency of backwash water treatment techniques should be evaluated. From this survey and others reviewed earlier in this section it seems that *Giardia sp.* and *Cryptosporidium sp.* are wide spread in the water environment world-wide and can be frequently detected in treated drinking waters with no indication of poor treatment performance.

In this section we have reviewed the evidence that *Giardia sp.* and *Cryptosporidium sp.* are common water borne pathogens. They have been detected frequently in both surface waters and in treated drinking waters world-wide. A number of outbreaks caused by these organisms have been reported associated with the drinking water supply and it has been seen that these pathogens can penetrate conventional water treatment processes and are capable of resisting typical chlorine disinfection doses. In some cases there was no indication of plant failure and the treated water quality had acceptable turbidity and indicator organism levels. Since the infective dose of these pathogens is believed to be small it is possible that sufficient numbers to cause infection could enter the water supply without triggering any indication of poor quality. The detrimental effects of flow rate changes, restarting of dirty filters and recycling of backwash water have been noted and it has been recommended that such practises are avoided. Several researchers have recommended a 0.1 NTU turbidity limit to safeguard the public water supply from these organisms. The presence of surging in rapid filters could make the attainment of such a quality goal difficult if these small, rapid flow changes do have a long term influence on filter performance. It is not known whether the presence of surges in rapid filters could assist the penetration of low levels of such pathogens without any obvious indication of poor filter performance.

2.4 Particle Size Distribution Analysis and Turbidity Measurement

Turbidity measurement has become the most common measure used to monitor filtrate quality continuously at full scale plant. However, in recent years, the importance of particle size and size distribution analysis has been recognised and this has highlighted limitations of turbidity measurement alone as an indicator of treated water quality. It is the purpose of this section to review the key research in this field with respect to the issue of surging in filtration.

Beard and Tanaka (1977) compared the performance of particle counting and nephelometric turbidity measurement of filtered water from a full scale plant. The plant comprised coagulation with alum and a cationic polymer followed by sedimentation and dual media filtration. The turbidity and particle counts were monitored continuously throughout the filter runs. Figure 2.34 compares the filtered water turbidity and count of particles greater than 2.5 microns for a typical run. It can be seen that the effluent turbidity and particle count mirror each other. However, it would appear that a 3 fold increase in turbidity corresponded with a greater than 10 fold increase in particles greater than 2.5 microns in size. This suggested that particles greater than 2.5 micron contribute less to nephelometric turbidity measurement than smaller particle sizes. The authors concluded that particle counting is a much more sensitive technique in measuring water quality.

O'Melia and Ali (1978) recognised the importance of the size of particles suspended in the influent to deep bed filters from previous experimental observations. They recognised that the removal efficiency for a given particle size was influenced by its size and that a minimum removal efficiency occurred for particles around 1 micron. A filtration model was developed from this concept and the idea that previously captured particles could act as additional collectors for new particles, modifying the filter removal performance. The verified model was used to simulate filter head loss development and removal efficiency under different operating conditions in deep bed filters. From their simulated results, the authors drew the following key conclusions.

- Initial filter removal efficiency is dependent on the particle size. Sub-micron particles are effectively transported to the media surface by diffusion processes and larger particles by sedimentation. A minimum removal occurs around 1 micron.
- The minimum removal for 1 micron particles continues during the ripening period.
- Filter head loss is dependent on the suspended particle size. Smaller particle sizes produced proportionally greater head losses than larger particle sizes.
- Removal efficiency improves or ripens with time since previously captured particles act as additional collectors.
- Chains of captured particles or dendrites can form, extending into the pore fluid flow.

The authors stated that particle sizes were not often measured in water treatment at that time since the available techniques were limited and expensive.

The experiments of Logsdon *et al* (1981), discussed earlier in this chapter, found that a small increase in filtered water turbidity could be accompanied by a large increase in the concentration of *Giardia sp.* cysts of 8 to 12 microns in size. This is similar to the findings of Beard and Tanaka reviewed above. An increase of 0.1 NTU was associated with an increase in *Giardia sp.* cysts of 10 to 50 times. This could be important with respect to surges and rate changes. Even if surges are responsible for only a 0.1 NTU deterioration in filtrate quality this could correspond to a large number of cysts of *Giardia sp.* if present in the influent water.

Hutchinson (1985) stated that particle counts are much more sensitive indicators of potential filter breakthrough than turbidimeters since they determine the actual number and size of particles in the water sample. Turbidimeters work on the basis of light scatter from the suspended material and are influenced by the particle shape, size, refractive index and concentration. The author reported that particulate breakthrough has preceded turbidity breakthrough by as much as 2 hours. That is to say particles can begin to breakthrough rapid filters without triggering a significant turbidity increase.

Mackie *et al* (1987) developed a mathematical model of deep bed filtration performance. The important aspect of their model was that it took account of the polydispersity of the influent suspension. Previous modelling of filtration assumed that the influent suspension was monodisperse or used the mean particle size in the calculations. However, Mackie *et al* recognised that filter performance is affected by the size of the suspended particles. It was noted that as a filter pore constricted with deposits the interstitial velocity rose. The authors recognised that this increase in velocity would affect larger suspended particles more than smaller particles and incorporated this factor into the model calculations. It could therefore be the case that the momentary increases in interstitial velocity caused by surging may influence the removal of larger particles more than smaller particles. The model results were compared with experimental observations and the authors found good qualitative and fair quantitative agreement.

Darby and Lawler (1990) noted that particle size of the influent suspension is an important factor affecting filter performance. The authors conducted a series of well controlled laboratory experiments filtering suspensions of monodisperse, bimodal and trimodal latex particles to discover more about the influence of the particle size and size distribution on filtration. The head loss and particle size distribution with time and depth were monitored from sample ports fitted through the filter column walls. The results showed that the particle size distribution of the suspension changed with time and with depth. The authors main observations are summarised below.

- Particle removal increased with depth as expected. Ripening or increasing removal of particles was observed with time for different particle sizes.

- Changes in particle size distribution occurred with increasing depth and time as a result of preferential removal of particular particle sizes and floc formation and subsequent breakoff.
- The greatest increase in hydraulic gradient occurred in the top sections of the bed since more particles per unit depth were removed in the upper sections.
- The removal of a given particle size appeared to be influenced by the presence of other particle sizes. It seemed that a given particle size was captured most effectively by a previously captured particle of the same size and hindered by the presence of other particle sizes.

The authors concluded that particle size, size distribution and filter ripening were important factors governing filter performance that had largely been ignored by previous studies. The use of turbidity or suspended solids measurement could not identify the effects of particle size on ripening and head loss development. The experimental results were later used to calibrate and test a new filtration model (Darby *et al,* 1992) developed from an earlier version (O'Melia & Ali, 1978) that accounted for heterodisperse suspensions with improved predictive performance. From these results the use of particle analysis in addition to turbidity measurement would be a useful tool in an assessment of the effect of surges on filtration performance. Particle analysis may well identify effects that turbidity measurement alone may miss.

Laboratory scale experiments were conducted using pollen grain suspensions to determine the effect of suspended particle size and size distribution on the performance of deep bed filtration (Vigneswaran *et al,* 1990). The authors found that particles of different sizes had different removal efficiencies. Removal efficiency was found to increase with increasing particle size from 22 to 85 microns. The removal efficiency was found to improve or ripen with time and then decrease due to particle detachment. The removal efficiency of a given particle size was found to be influenced by the presence and concentration of adjacent particle sizes. The authors found that as the concentration of coarser particles was increased the removal efficiency of the finer particles was improved. The authors concluded that the use of the mean particle size of the influent suspension was inadequate in the characterisation of filter performance.

Mackie and Bai (1992 & 1993) reviewed the results of recent filtration investigations using particle analysis (Vigneswaran *et al,* 1990; Darby and Lawler, 1990; Tobiason *et al,* 1990). It was noted that Vigneswaran and colleagues (discussed above) found that the removal efficiency of smaller pollen grain particles was enhanced by the presence of larger particles. However, it was also noted that Darby and Lawler (discussed above) found latex particles of a given size preferred to be captured by similar sized particles and were hindered by the presence of other size particles. In addition, Tobiason and co-researchers found the removal of smaller latex particles was inhibited by the presence of larger particles and that larger particle removal was enhanced by the presence of smaller particles. The evidence was somewhat contradictory. Mackie and Bai set out to clarify the influence of particle size distribution on filtration. They performed filtration experiments with suspensions of PVC powder from 0.5 to 15 microns in size. The authors found that the removal of smaller particles was enhanced by the presence of larger particles in the range 0.5 to 15 microns similar to Vigneswaran *et al's* work in the range 22 to 85 microns. Two reasons for this effect were offered. Firstly, the presence of larger particles which are more easily removed would result in more rapid filter ripening. Secondly, the larger particles could alter the deposit morphology in some manner improving the removal of smaller size particles. They concluded that the size distribution of a suspension changes with depth and time and can affect the filter in terms of both removal efficiency and head loss development. The experiments with polydisperse suspensions illustrated that filtration is a complex process and models must include the influence of the suspension particle size distribution to accurately predict filter performance. It would seem from these findings that particle and head loss sampling with time and depth are useful techniques in a laboratory filtration research program and can yield important insights.

Moran *et al* (1993a & 1993b) conducted laboratory scale filtration experiments with pre-treated water from a full scale water treatment plant. The authors wished to observe the ripening and breakthrough characteristics of deep bed filtration under a variety of operating conditions using a natural heterodisperse suspension unlike previous studies discussed above. Batches of coagulated, settled water with a turbidity of 1.6 to 3.2 NTU were obtained from a local treatment plant and transported to the laboratory for each test run. Test runs were of 24 to 50 hours duration to observe the filter performance across the full filter cycle. The suspension was filtered at rates of 1.8 to 5.5 mm/s approach velocity through a 76 mm internal diameter filter column. The laboratory filter used spherical glass beads as the filter media for improved experimental control. Bead sizes of 0.78 and 1.85 mm were used in the experiments to determine the effect of different media sizes. Most of the experiments used a media depth of 946 mm. Pressure and suspension sampling ports were fitted to the filter column above the media surface and at depths of 193, 565 and 746 mm. Pressure measurements were recorded throughout the filter cycle automatically using transducers fitted to the pressure ports and a data acquisition system. Suspension samples were collected periodically and analysed using a Coulter counter. The turbidity and suspended solids were also determined to compare with the particle results. Figure 2.35 illustrates the total turbidity and suspended solids removal performance of one test run conducted at 5.5 mm/s approach velocity with the 1.85 mm glass beads. Ripening of the removal of both suspended solids and turbidity can be seen over the first 600 minutes of the test run. The suspended solids removal was greater than the turbidity removal at this time suggesting that removal of larger size particles was more effective. After 600 minutes the removal declined somewhat and became more erratic suggesting incipient breakthrough. The scatter in the results suggested to the authors that breakthrough was not a smooth process like ripening but was the result of discrete avalanche events within the filter media. Figure 2.36 illustrates the particle removal performance early in a test run conducted at 5.5 mm/s approach velocity using the 0.78 mm glass beads. The horizontal axis shows the suspended particle size from less than one micron to greater than 25 microns expressed as a log. Thus one micron particles are shown at 0.0 and 10 micron particles at 1.0. The vertical axis shows the particle size distribution function. This illustrates the number concentration present in each size channel measured by the counter. Since the channel sizes recorded by the counter are not all the same width the concentration has been expressed as the number count per unit volume in a given size channel normalised to the width of the channel. The figure shows the particle size distribution function for the influent and samples at depths of 193 and 746 mm 16 minutes into the run. It can be seen that the number concentration for all particle sizes decreased with increasing depth. However, the decrease in number concentration increased with increasing particle size. There appeared to be a minimum removal of

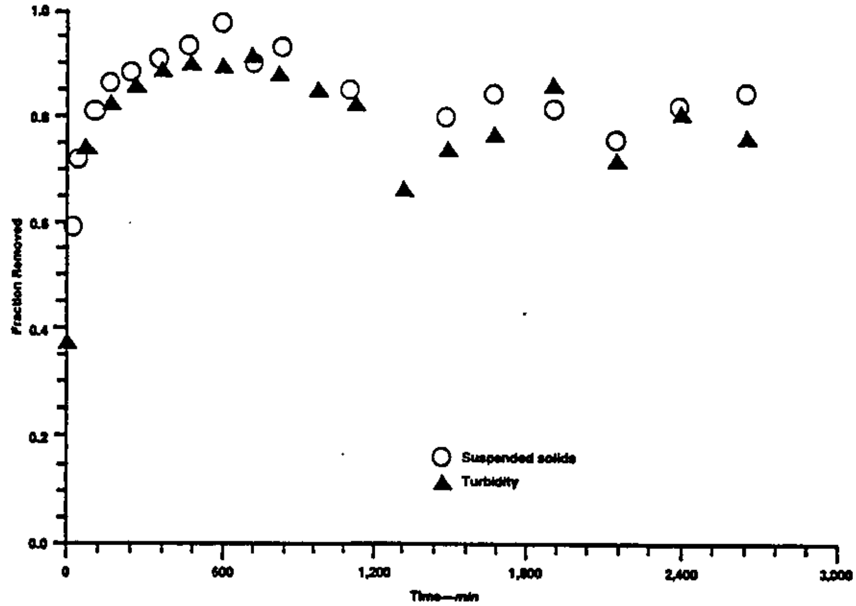


Figure 2.35: Turbidity & suspended solids ripening (Moran *et al,* 1993a)

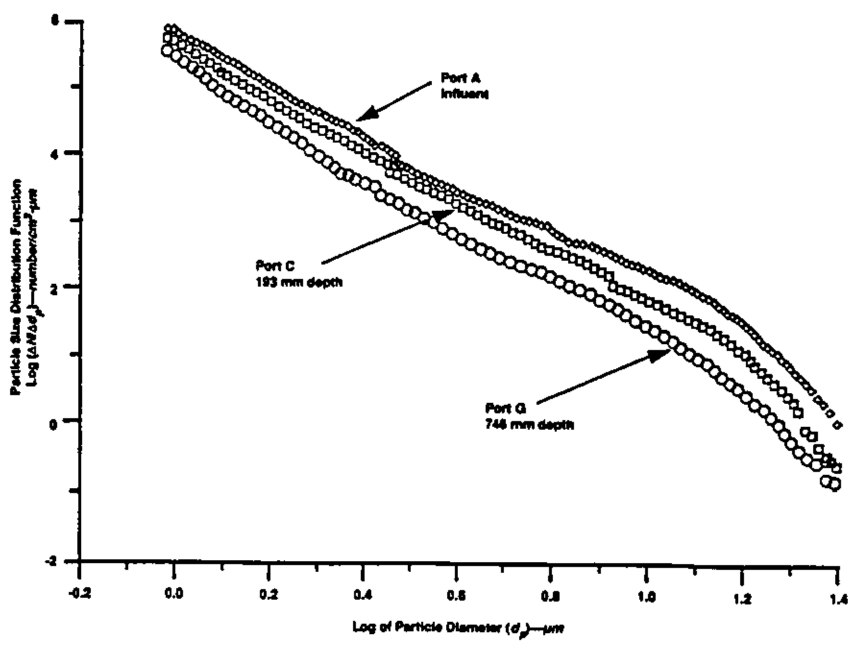


Figure 2.36: Particle size distribution function (Moran *et al,* 1993a)

particles around one micron in size across 746 mm of depth. Figure 2.37(A) shows the effect of ripening on particle removal and size distribution at a given depth. This chart shows the particle size distribution function of samples taken from 746 mm depth at 16 and 61 minutes into the run compared with the influent. Clearly, removal efficiency across all particle sizes had improved with time at this depth. The poorest removal was still evident for the smallest particle sizes measured. With increasing time, breakthrough became evident as shown in figure 2.37(B). Here the removal at 746 mm depth is compared at 61 and 1500 minutes run time. The removal efficiency had declined somewhat for particles greater than 1.8 micron from 61 to 1500 minutes. However, the removal efficiency was still effective with respect to the influent concentration of these particle sizes. No breakthrough was evident for particles less than 1.8 micron size. The authors presented evidence for detachment of previously captured particles. The influent suspension to the filter was switched to a reduced particle suspension at the end of one filter run. Figure 2.38 illustrates the particle size distribution of the reduced particle influent and a sample taken at a depth of 193 mm 15 minutes after the switch. It appeared that the concentration at 193 mm depth was greater than the influent suspension for particles greater than 1.4 micron. The difference between the two distributions can be taken as a measure of the breakoff of previously captured particles. Note that particles less than 1.4 micron were still effectively removed. Evidence of breakoff of previously captured particles was found during standard filter runs using the settled coagulated water too. Figure 2.39 (A), (B) & (C) presents particle size distribution results from samples taken 16, 732 and 1500 minutes into the run at depths of 193 and 565 mm. The vertical axis here represents the fraction of particles removed for each particle size. After 16 minutes, increasing removal with depth was evident across all particle sizes. At 732 minutes, the removal efficiency can be seen to have improved or ripened at both depths for all particle sizes, especially so for the smaller sizes. However, it appeared that the removal efficiency in the top section of the filter experienced a dip in removal for intermediate size particles compared to smaller and larger sizes. This pattern became more obvious after 1500 minutes run time. Here the top section of the filter experienced negative removal of particles from 3.2 to 6.3 microns. This suggested that particles or agglomerates of particles of this size range were detached from the upper section of the filter. Note, however, that the removal of smaller particle sizes was still very effective and of larger sizes was still good although poorer than at 732

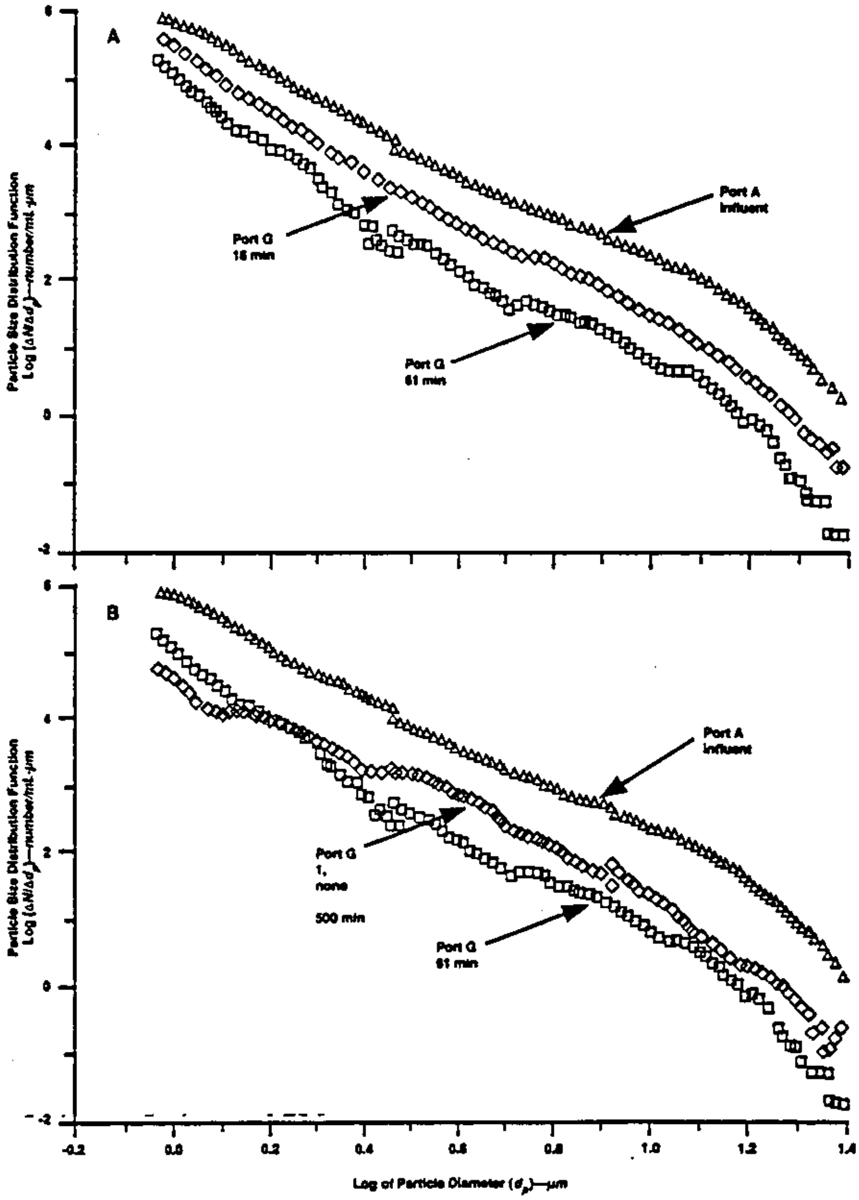


Figure 2.37: Particle ripening & breakthrough (Moran *et al,* 1993a)

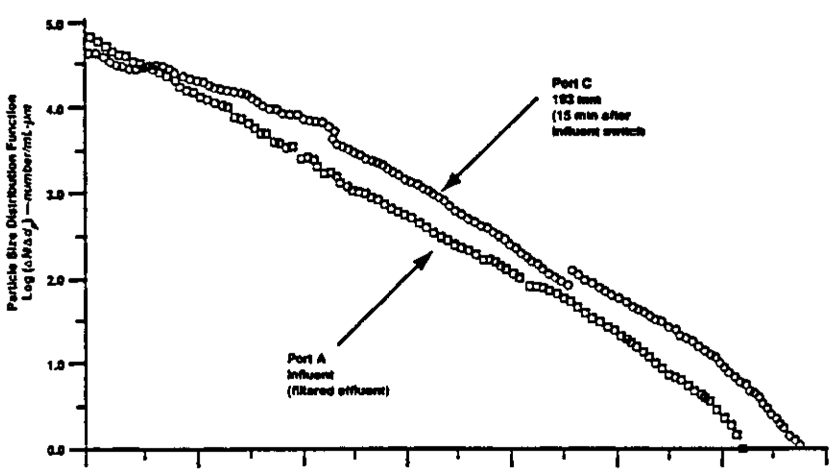


Figure 2.38: Particle detachment (Moran *et al,* 1993b)

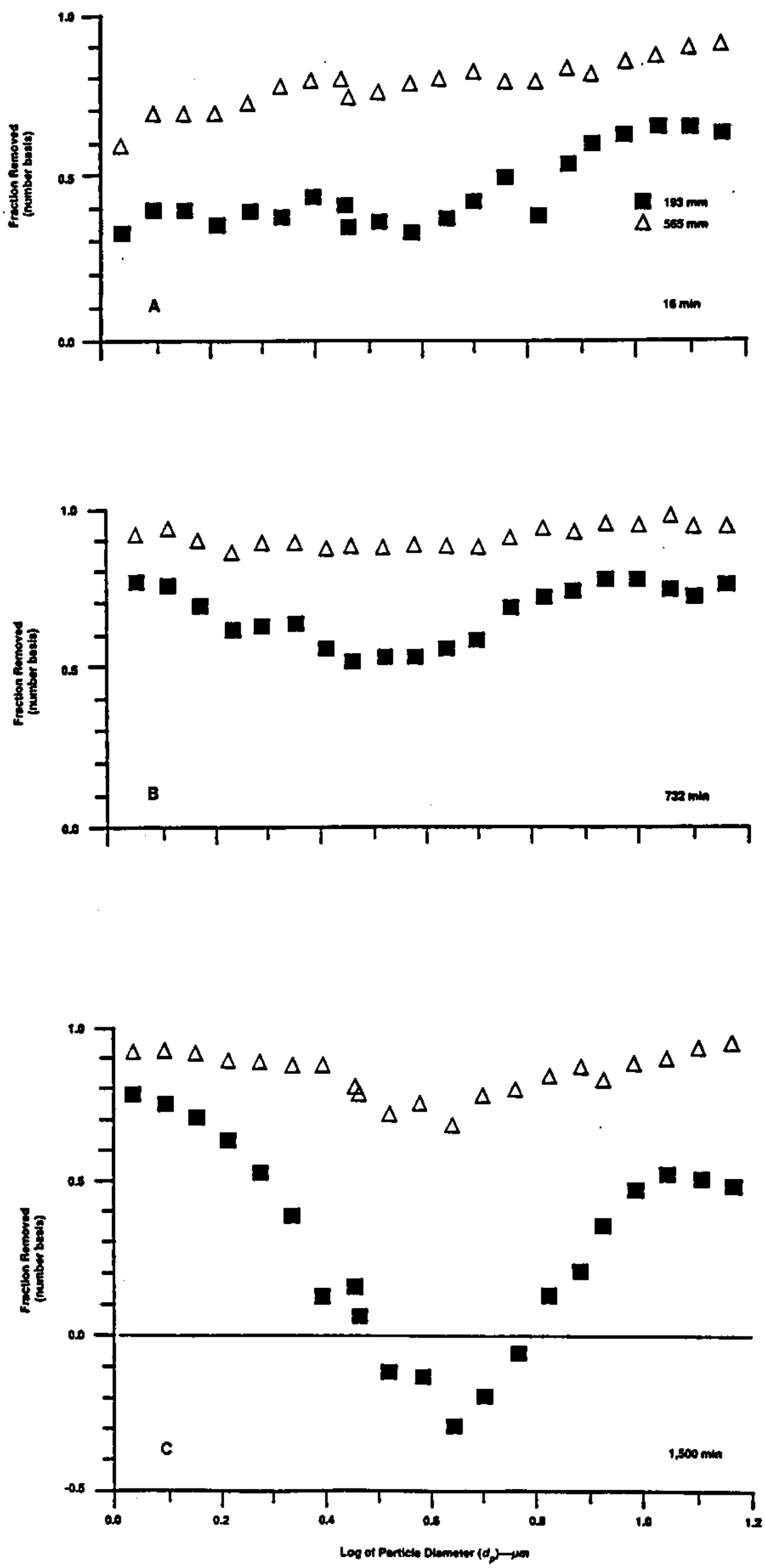


Figure 2.39 (A), (B) & (C): 3 to 6 um particle breakthrough (Moran *et al,* 1993b)

minutes. With increasing depth the removal of the intermediate sizes recovered but was still poorer than for smaller and larger particles. A summary of the key conclusions the authors drew from their results is given below.

- smaller particles ripen for the longest duration and exhibit the greatest increase in removal by ripening
- intermediate particles ripen quickly but breakthrough earlier than other sizes
- the largest particles have the most consistent removal but breakthrough at high deposit levels.
- detachment of previously captured particles or agglomerates of particles contributes to filter breakthrough and could be the dominant mechanism. This occurs primarily for intermediate and larger sizes
- Turbidity and suspended solids cannot detect the differing removal of different particle sizes.

The authors expressed their concern over the breakthrough of particles in the range 3 to 7 microns. The breakthrough of these particles was not picked up by the turbidity or suspended solids measurements; the most common parameters used in monitoring full scale filter performance. The authors speculated that these intermediate size particles could pass through a full scale filter without triggering any indication of poor performance. It was noted that oocysts of *Cryptosporidium sp.* are 4 to 6 microns in diameter and the poorer removal of particles of this size could have implications for drinking water quality and public health. It was, however, also noted that the intermediate particles of 3 to 7 micron size could be composed of smaller particles which entered the bed, were captured and broke off as intermediate size flocs. Nevertheless the authors noted that the different ripening and detachment rates for different particle sizes were important factors in the theoretical understanding and practical operation of deep bed filters. In a later study, similar trends in ripening and breakthrough were found using sand as the filter medium (Kau & Lawler, 1995). Surging in the flow rate through filters could have an influence on the ripening and breakthrough of different particle sizes with time and depth. The erratic fluctuations in interstitial velocity resulting from the surging could in some manner affect the ripening and early breakthrough of intermediate sized particles which includes *Cryptosporidium sp.* oocyst sizes.

Gregory (1994) discussed the performance of turbidity measurement and particle counting with respect to suspended particle size. Turbidity measurement is the most common technique used in the monitoring of water quality from treatment works. The technique works on the principle of light scatter from particles suspended in the sample known as nephelometry. The magnitude of the scattered light is measured by a detector typically set at a 90° angle to the incident light. The detected signal is converted to a measure of turbidity and is reported in nephelometric turbidity units (NTU). Obviously, the magnitude of the detected signal will increase as the concentration of particles in the sample increases. However, the degree of scatter from a suspended particle is influenced by such particle properties as its size, shape and refractive index. As such the technique does not provide an absolute measure of the concentration of suspended material nor does it provide detail of the suspended particle size or size distribution. The effect of suspended particle size on turbidity measurement is particularly important. To illustrate this point, Gregory measured the turbidity of a series of prepared standards using two widely used turbidimeters; Hach XR ratio turbidimeter and the Hach 2100A turbidimeter. The XR ratio turbidimeter measures forward scattered light (i.e. detector angle positioned at greater than 90° to the incident light) and the 2100A measures 90° scattered light. The prepared suspensions were manufactured from uniform polystyrene latex particles of different sizes. Each suspension was produced from one particle size to a concentration of 1 mg/l. That is to say each suspension contained the same mass of suspended solids. The only difference was the size of the particles. Figure 2.40 illustrates the turbidity results for each prepared suspension from the two turbidimeters. It can be seen that from a particle size of 0.25 to 6.0 microns, the turbidity fell from 6-8 NTU to 0.5-1.0 NTU. This result has implications regarding the monitoring of *Giardia sp.* and *Cryptosporidium sp.* oocysts in water. Gregory noted that oocysts of *Cryptosporidium sp.* are 4 to 6 microns in diameter and that cysts of *Giardia sp.* are about 10 to 12 microns in size and that turbidimeters responded less well to these sizes. Gregory also noted the effect of refractive index on turbidity measurement. As the refractive index of a suspended particle approaches that of the water the particle becomes transparent to the incident light and the degree of light scatter becomes very low. Since the major component of *Cryptosporidium sp.* oocysts is water this creates greater difficulty in

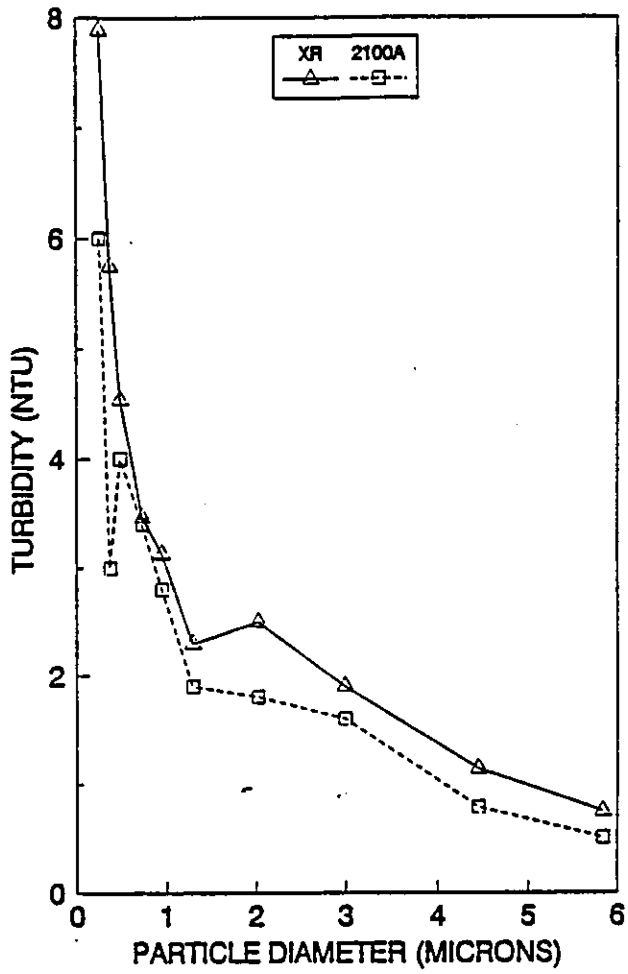


Figure 2.40: Influence of particle size on turbidity measurement (Gregory, 1994)

the detection of these particles by nephelometric turbidity measurement. Gregory stated that particle counting is a much more sensitive technique for monitoring filtered water quality since a filtered water of less than 0.1 NTU turbidity may still contain thousands of particles per millilitre. The author concluded that maintaining a low level of particles in the filtered water should prove effective against the presence of *Cryptosporidium sp.* and *Giardia sp.*. Monitoring the filtered water particle count should provide a more effective method of quality control than turbidity measurement.

In this section we have seen how particle analysis has become an important tool in filtration operation and research. Experiments have shown that filter performance is influenced by particle size and size distribution and that these factors change with both time and depth. Intermediate particle sizes corresponding to *Cryptosporidium sp.* oocyst sizes have been shown to ripen and breakthrough while removal of smaller sizes continues to improve. Turbidity measurement alone cannot identify these processes. Particle breakthrough can occur unobserved by turbidity measurement and it has been shown that a small change in turbidity can correspond to a disproportionately large change in particle count. Turbidity measurement has been found to be less sensitive to particle sizes greater than one micron. It seems clear that particle analysis has become an essential tool in the assessment and understanding of filtration performance. It is not known what influence the presence of surges will have on the removal of particles of different size.

2.5 Chapter Summary & Statement of Objectives

Previously published research has identified that rapid gravity filters are subject to small, frequent flow rate fluctuations known as surges. Surges have been found to be caused by turbulent flow conditions in the filtrate piping produced by such flow constrictions as bends, orifices and control valves. Assessments of pilot and full scale filters have shown that these flow changes are common and studies suggest but have not confirmed a detrimental effect on filtrate quality.

Discrete changes in the flow rate through rapid filters occur. These can be caused by filter backwashing, plant loading changes and modulation of flow control valves. Laboratory and full scale studies have shown that these flow changes can cause short term turbidity breakthrough and a drop in head loss as a result of scour. There is evidence that the flow pathways through the clogging bed are modified by such changes. It has been found that the magnitude of the effect of a discrete rate change is dependent on the amplitude and rate of change of the flow rate and on the type of suspension filtered. Increased shear forces acting on the retained deposit is the mechanism believed to produce the scour and breakthrough from a flow rate increase. It has been recommended that filter operation and design should be modified to minimise the occurrence of such changes in the flow rate.

Giardia sp. and *Cryptosporidium sp.* have been identified as disease causing protozoans present in the water environment. The infectious stages are present in large numbers in the faeces of infected humans and animals and are environmentally robust. There is evidence of cross species infection and surveys have found the organisms to be common in waste waters and surface waters. Several studies have identified these organisms in drinking water supplies. There have been a number of well documented outbreaks of *Giardia sp.*sis and Cryptosporidiosis associated with drinking water. Deficiencies in treatment were identified in a number of cases including poor filtration performance. However, several outbreaks occurred with no apparent failure or poor quality in the drinking water supply. The organisms have been shown to be resistant to chlorine disinfection and are believed to have a low infectious dose. Studies have shown that discrete rate changes can cause an increase in *Giardia sp.* penetration of rapid gravity filtration. The same is likely true for *Cryptosporidium sp.*. Small increases in filtrate turbidity were found to correspond to disproportionately larger increases in *Giardia sp.* count. It has been recommended that rate changes be avoided to minimise the possibility of pathogen penetration of gravity filters. Research has concluded that conventional water treatment including rapid filtration can remove *Giardia sp.* and *Cryptosporidium sp.* if operated optimally but cannot ensure complete removal. A multiple barrier approach was recommended to safe guard the public and a standard of 0.1 NTU was suggested.

Particle counting and size distribution analysis have been recognised as important tools in filtration research and operation. Removal performance has been found to be dependent on the influent suspended particle size and size distribution. These parameters are not static. They vary with both time and depth in the filter media as the filter ripens and begins to breakthrough. Particle analysis in experimental studies have found that intermediate size particles from 3 to 7 micron ripen early and begin to breakthrough while smaller particles continue to be effectively removed. It has been shown that previously captured particles break off and penetrate further into the filter. Turbidity and suspended solids measurement cannot perceive these changes and particle breakthrough can occur unobserved by turbidity measurement for some time. It has been noted that a small change in turbidity can be equivalent to a disproportionately large change in particles above 2.5 micron. A change of even 0.1 NTU can be equivalent to thousands of particles per millilitre. It has been found by particle analysis that there is a minimum removal efficiency for particles around 1 to 2 micron. Several researchers have noted that oocysts of *Cryptosporidium sp.* are in the intermediate particle size band, close to this minimum. Studies of turbidimeters have identified that turbidity measurement is less sensitive to particles greater than 1 micron. In addition, if the suspended particle refractive index is close to that of water then the particle becomes more difficult to detect. Oocysts of *Cryptosporidium sp.* have these characteristics.

From the above brief summary of previous research described in detail in this chapter, it is hopefully clear to the reader that there are unanswered questions regarding surging in rapid filters and concerns from a 90's perspective which were not apparent in the past. The influence of surges on the performance of rapid gravity filtration has not been unambiguously confirmed. The past research conducted in this field is 30 to 50 years old and in the authors opinion is limited by the experimental techniques used. The mechanisms in which surging influences filter performance during a filter run have not been identified and have yet to be critically examined in the laboratory under well controlled and reproducible conditions. The effect of surging has yet to be quantified. Since discrete rate changes have been shown to clearly influence filtrate quality, head loss development and possibly the flow pathways through the media, it would seem worthwhile to pursue the effects of surging on these parameters from a modern perspective. Since a discrete flow increase produces a short term turbidity spike it may well be the case that surging, present continuously from start to finish, may cause say a 0.1 to 0.2 NTU poorer average quality filtrate throughout the filter cycle. This could amount to a substantial volume of solids passing into the filtered water supply which since it would not be an obvious deterioration in quality could pass unnoticed. *Giardia sp.* and *Cryptosporidium sp.* can penetrate rapid filters. They are common in the water environment and have been shown to be resistant to chlorine disinfection. The infective dose of these organisms is believed to be small. They have been identified as the cause of a number of outbreaks of illness from the water supply. Discrete flow increases have been shown to cause increased *Giardia sp.* counts. In this light, surges may well be an important factor in filter performance and could contribute to the undetected penetration of low concentrations of pathogens continuously into the filtered water supply if present in the source water. Particle analysis has shown that intermediate size particles including *Cryptosporidium sp.* oocyst sizes ripen and breakthrough rapid filters earlier than smaller sizes and as such may not cause obviously poorer quality filtrate. The presence of surges may enhance or accelerate these ripening and breakthrough processes for different particle sizes in some manner and may therefore have an important bearing on filtration performance.

There follows a statement of research objectives intended to address the above discussed issues which forms the heart of the laboratory investigation into surging described in subsequent chapters of this thesis.

Research Objectives:

- To identify under well controlled and reproducible laboratory conditions whether surges influence rapid gravity filter performance or not. This has yet to be unambiguously determined.
- To critically examine the mechanisms, if any, in which surges influence rapid filter development during the full filter cycle. That is how do the surges affect the head loss development and removal efficiency of turbidity and particles with both time and depth during a filter run.
- To quantify the effect of surges on rapid filter performance. That is what magnitude, rapidity of change and rate of occurrence of surges are necessary to produce a given effect on filtrate quality and head loss development.
- To draw conclusions from the results regarding the impact of surges on full scale filter performance and make what recommendations are possible to improve filter design and filtrate quality in this respect.
- To make recommendations of potentially fruitful avenues of future filtration research in this field.

3.0 FIELD MEASUREMENTS AT LOCAL WATER WORKS

It was decided to begin this research programme into the effects of surging on rapid filtration by conducting a brief assessment of the extent of surges in rapid filters at several local water treatment plants. The purpose of this small survey was to gather additional information where possible about the nature and occurrence of surges in different rapid gravity filter designs. Comparison of the filtration plant design and then linking this information with any surging found may yield some useful insights. It was intended to use this information to provide data on the range and characteristics of the surges to be applied to the laboratory scale experimental filters. This surge survey was intended as a brief investigation of the range of surging likely to be found and was not intended to be comprehensive. A comprehensive survey would have required more extensive measurements at many filtration plants which was beyond the scope of this investigation.

3.1 Cropston Water Treatment Works

Cropston water works in Leicestershire was designed to a maximum 32 Ml/d (mega litres per day) capacity which historically supplied the city of Leicester. In practise, flow rates were usually around 20 Ml/d. Raw water abstracted from the River Lin was stored in Cropston reservoir. Treatment comprised pH adjustment and alum addition followed by flash mixing and floc formation with polyelectrolyte addition. Clarification was achieved by dissolved air flotation before rapid filtration. Chlorine disinfection was practised before final pH adjust. The plant operated 4 rapid gravity filters each of 50 square metres surface area. These were operated at a low approach velocity of only 2.5 metres per hour on the day of the visit. The flow rate was controlled by an effluent butterfly valve governed by the water level in the filter box. The filter media comprised 1 metre of granular activated carbon. The design of the filter gallery including the filtrate piping is shown in figure 3.1. Filtered water was removed from the underdrain by a 400 mm internal diameter pipe. The flow passed into a 'T' junction as shown. To one side was the closed backwash valve; a dead end during normal filter operation. The flow executed a 90° turn in this junction before entering a reducing taper. This reduced the pipe diameter from 400 mm to 250 mm

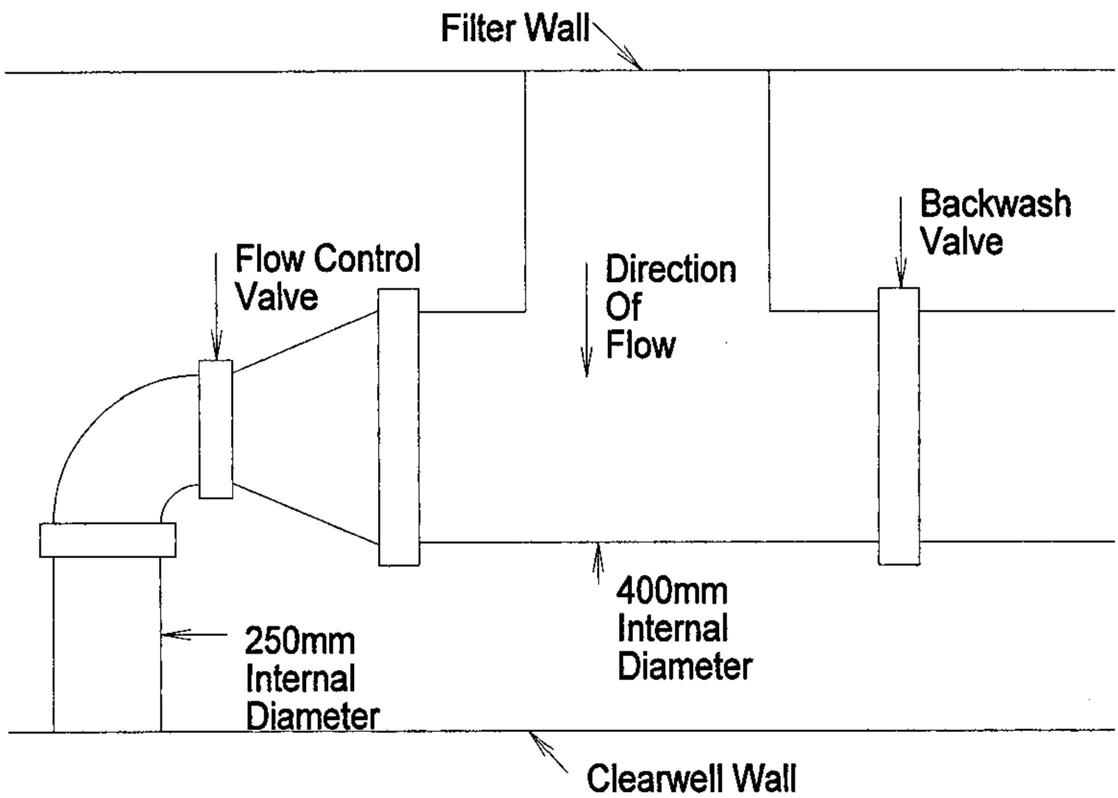


Figure 3.1 Cropston Filter Gallery

where the flow entered the rate controlling butterfly valve. Immediately after the valve the flow executed a second 90° turn before passing into the common outlet pipe. This effluent pipe design seemed to present ample opportunity for turbulent flow rate fluctuations.

To assess filters for the presence of surging required some method of monitoring either the flow rate or the pressure drop across the filters continuously or at as high a resolution possible. Flow rate from the filter bank was monitored as part of the plant's typical operation and control practise. However, no flow meters were fitted to the filters individually so no information on flow rate fluctuations could be gathered. However, the head loss across each filter was monitored by differential pressure transducers. These devices measured the water pressure above the filter bed and in the effluent piping upstream of the control butterfly valves. Surges in the flow can be detected by fluctuations in the head loss measurement as described in section 2.1. It was thought that if surging were present it would appear as fluctuations in the head loss readings from these transducers. It was hoped that inspection of the head loss trends might have revealed useful information about surging. The plant SCADA (Supervisory Control And Data Acquisition) system recorded and stored the head loss trends from each of the filters by sampling the differential pressure reading from the transducers fitted to each filter. This information could be retrieved and inspected. Close inspection of these trends revealed a problem. The SCADA system was designed to sample the differential pressure transducer output once every minute. 15 consecutive readings were taken and the average of these 15 measurements was recorded in the head loss trend. Since surging is rapid with frequencies up to 100 oscillations per minute the result of this averaging procedure would be to mask the pressure fluctuations present. The system was incapable of recording the pressure fluctuations continuously. One option available in the system was to record not only the average of the 15 consecutive one minute readings but also the maximum and minimum of the 15 readings. This would not provide the level of resolution ideally required to evaluate the frequency and amplitude of the pressure fluctuations present but would at least give an indication of the magnitude of pressure fluctuations present in the filter. The system programming was modified to record the maximum and minimum pressure measurements in addition to the average trend. Several weeks worth of information was recorded from the filters in this manner. Close inspection of the results revealed that significant pressure fluctuations did occur in the rapid filters. A typical example of the gathered results is illustrated in figure 3.2. This chart shows the head loss trend recorded from filter two over a full filter cycle. The average head loss trend showed that the filter developed from less than 100 mm initial head loss to over 1200 mm in approximately 30 hours run time. Disturbances in the average trend were apparent at 10, 15 and 20 hours. These correspond to the increase in flow resulting from backwash of adjacent filters in the bank (backwash times are shown as arrows in the chart). As would be expected by an increase in flow the head loss increased during the backwash procedure. The minimum and maximum head loss trends are also shown in this chart. These give an estimate of the magnitude of the pressure fluctuations that occurred in the filter during the run. It can be seen that at the start of the filter run the head loss fluctuations around the average head loss were approximately 50 mm in amplitude. Since the sampling frequency here is only once a minute no comment can be made regarding the frequency of occurrence of these fluctuations. With increasing head loss the fluctuations can be seen to increase in magnitude similar to the findings of Baylis and Hudson discussed in chapter two. By the end of the filter run the pressure fluctuations increased to 250 mm at an average head loss of 1300 mm. Figure 3.3 plots the maximum surge or pressure fluctuation against the average head loss as an absolute amplitude and as a percentage of the head loss. From this chart it seemed that the maximum surge amplitude at the beginning of the filter cycle was approximately 40 % of the head loss. With increasing head loss the percentage surge amplitude declined to less than 20 % of the head loss. So it seemed that although the absolute surge amplitude rose during the run it declined relative to the average head loss. Baylis, discussed previously in section 2.1.2, also found that the surge amplitude rose as the head loss increased but his results suggested that the surge amplitude rose more rapidly than the head loss. Hudson, discussed previously in section 2.1.4, also found that the surge amplitude in a constant rate filter rose as the head loss increased. However, he apparently found that the surge amplitude rose more slowly than the head loss. These measurements from Cropston works confirm that pressure fluctuations similar to those found by previous researchers occur in rapid filters.

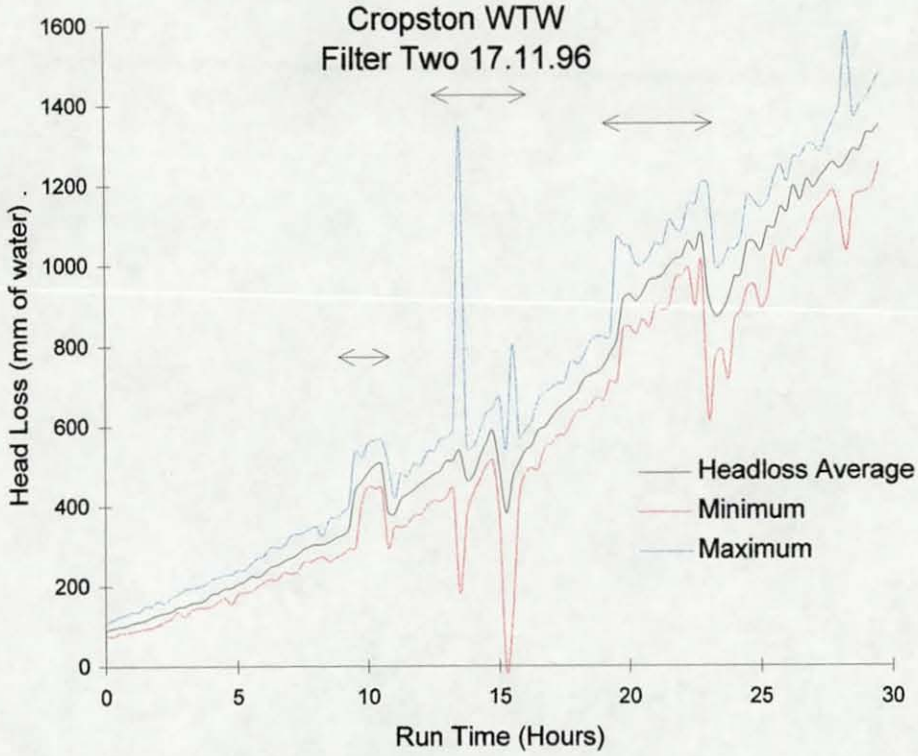


Figure 3.2: Cropston head loss data

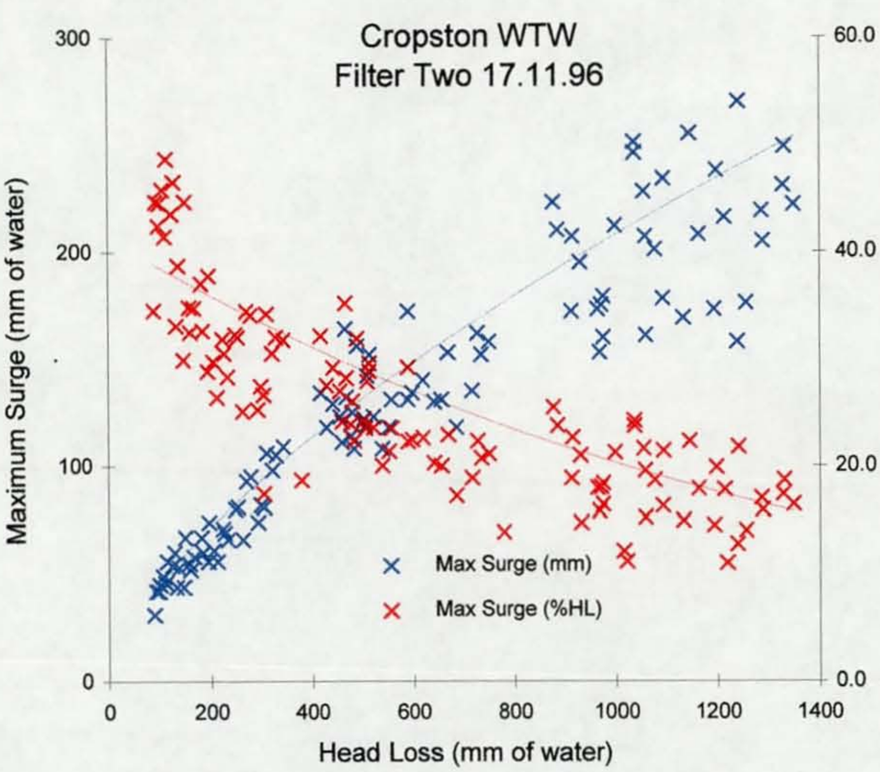


Figure 3.3: Cropston surge amplitudes

3.2 Church Wilne Water Treatment Works

Church Wilne water works in Nottinghamshire was designed as a 140 Ml/d maximum capacity plant which supplied a population of around 250,000 in west Nottingham and northern Leicestershire. Raw water was abstracted from the River Derwent at Draycott and Little Eaton. The treatment comprised coagulation and flocculation followed by either clarification in hopper bottomed clarifiers or by dissolved air flotation. The clarified water was passed on to rapid filtration followed by granular activated carbon treatment. Disinfection was achieved by chlorine dosing with a residual of 0.5 mg/l free chlorine leaving the contact tank. The plant had 12 dual media filters each of 67 square metres surface area. These were operated at approach velocities of 4.0 to 7.25 metres per hour depending on demand. Filter flow rate was controlled by effluent butterfly valves linked to water level sensors in the filter boxes. The filter media comprised 250 mm anthracite and 460 mm sand supported on 260 mm gravel. The settled water applied to the filters had a turbidity of 1.0 to 1.5 NTU and filtered water turbidity was typically 0.25 to 0.5 NTU. The design of the filtrate piping is shown in figure 3.4. Filtered water was drawn from the underdrain by a 380 mm internal diameter pipe. The flow passed into a T piece fitting. On one arm of the T piece was the closed backwash control valve. The flow executed a 90° turn before entering a 380 to 250 mm taper. The 250 mm effluent control valve was fitted at the end of this taper. The flow then executed a 90° turn before it discharged into the filtered water clearwell. It can be seen from this design that there are several flow constrictions in the filtrate piping. Baylis and Hudson, discussed in chapter two, agreed that valves and bends as well as reductions in the pipe diameter were capable of creating turbulent flow fluctuations. It seems likely that the combination of T pieces, tapers, butterfly valves and 90° bends would generate turbulent conditions in the filtrate piping and produce surges.

Similar to Cropston water works, Church Wilne monitored the flow rate from the filter banks but did not provide flow metering from each individual filter. However, once again, the head loss development was monitored by pressure transducers in each filter. Pressure measurement was taken from above the filter media and in the filtrate piping upstream of the butterfly control valve. Unlike Cropston, the signals from these

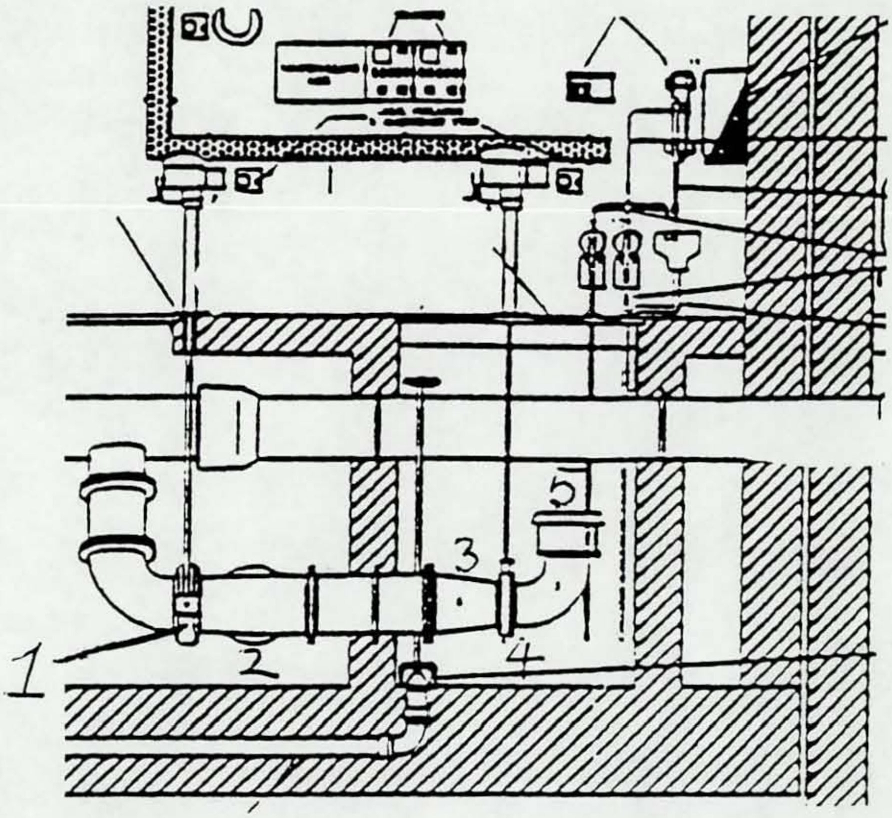
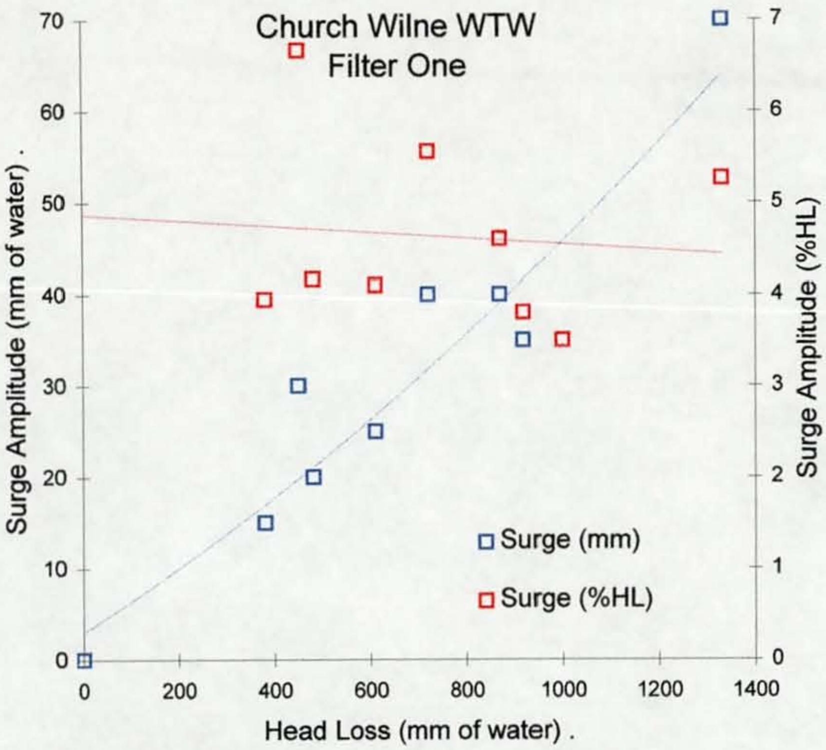


Figure 3.4: Church Wilne Filter Gallery

1 Backwash control valve
2 T Piece with flow coming out of filter underdrain
3 Taper
4 Flow control valve
5 Bell mouth discharge into clear well

devices were displayed continuously on analogue gauges in the plant control room in addition to electronic sampling. This allowed the surges in flow through the filters to be monitored continuously by visual observation of these dials similar to a technique used by Baylis described in section 2.1.2. Indeed, close inspection of the gauges revealed random and erratic high frequency surging similar to previous researchers' observations. Ideally, these signals should be recorded by some electronic means for analysis. However, this wasn't possible in the circumstances. Instead, estimates of the average surge amplitude and frequency of occurrence were made by observing the gauge oscillations over several one minute periods. These readings were taken periodically during the filter cycle to determine how the surges changed with increasing head loss. Examples of the recorded amplitude data are presented in figures 3.5 (a) - (d). These plots illustrate the absolute surge amplitude and surge amplitude as a percentage of the head loss as a function of increasing head loss. In each case the absolute surge amplitude increased with increasing loss of head similar to the findings of Baylis and Hudson and similar to the observations at Cropston water works. Surge amplitudes of 10 to 30 mm were evident early in the filter cycles at head losses of 200 to 400 mm. Amplitudes of 35 to 100 mm were evident towards the end of the filter cycles with head losses of 1000 to 1400 mm. These amplitudes are similar to Baylis' findings but smaller than those found at Cropston. Cropston surge amplitudes were probably an over estimate of the average surge amplitude due to the sampling technique used. In relative terms the amplitude results at Church Wilne were more ambiguous. As a percentage of the head loss the surge amplitudes do not appear to have any clear pattern. The relative surge amplitudes in the examples shown both increased, decreased and remained constant with respect to the increasing loss of head. Baylis, discussed in section 2.1.2, found that the relative surge amplitudes increased with rising head loss. This may however reflect his experimental method. Examples of the recorded surge frequency data from Church Wilne are given in table 3.1.



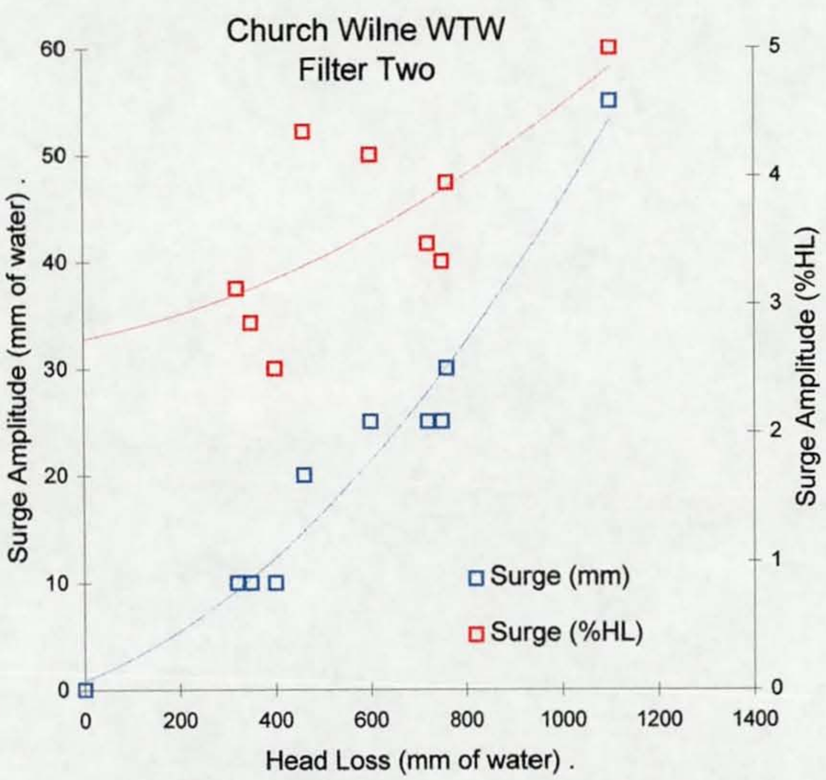
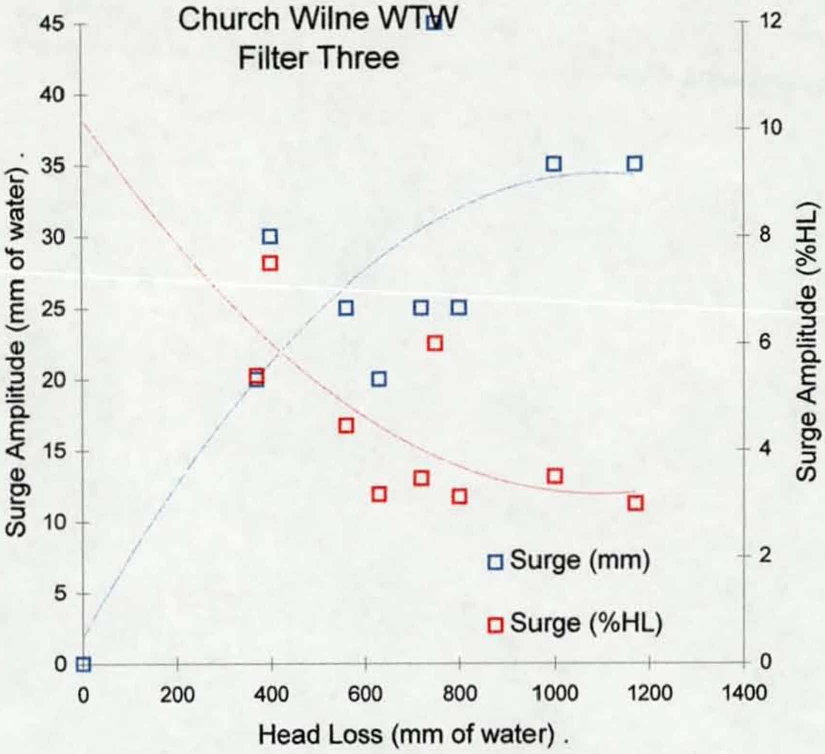


Figure 3.5 (a) & (b): Church Wilne surge amplitudes



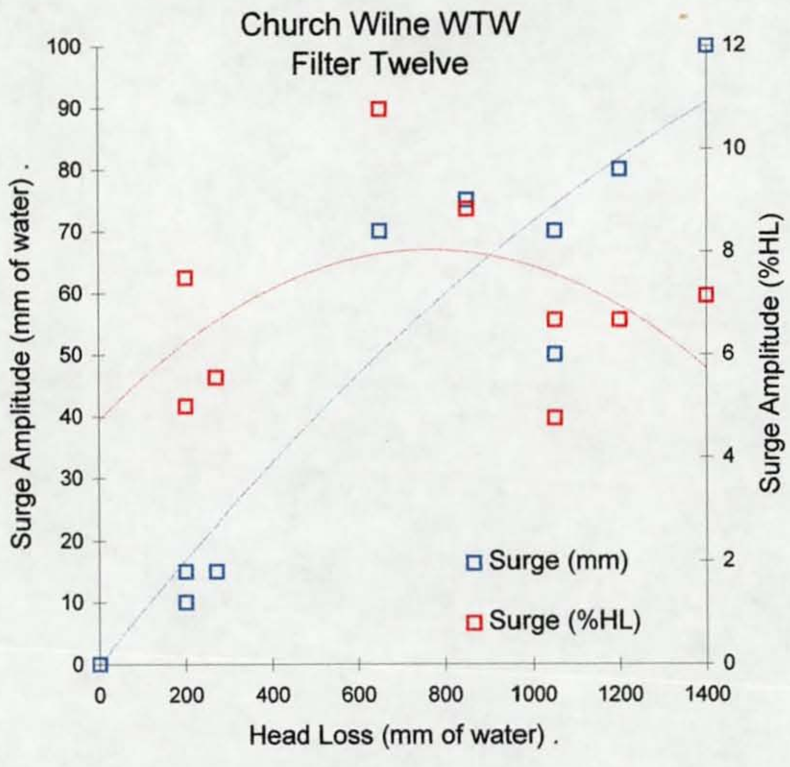


Figure 3.5 (c) & (d): Church Wilne surge amplitudes

Time (Hours)	*Filter One (No./minute)*	*Filter Two (No./minute)*	*Filter Seven (No./minute)*	*Filter Twelve (No./minute)*
0	40	30	80	55
2	30	35	70	45
4	45	30	75	80
6	40	35	75	60
8	60	30	100	-
16	50	35	80	-
18	40	30	75	50
20	40	25	60	60
22	40	30	75	55

Table 3.1: Surge Frequency Data From Church Wilne Works

From these observations, surges ranging from 25 to 100 oscillations per minute were apparent in these filters similar to the observations of Baylis. However, the frequency of occurrence in any one filter seemed to remain constant with increasing loss of head. This seems logical. Since the average flow rate is held constant by the control system, the average amplitude and frequency of the flow fluctuations in the filtrate piping would presumably remain essentially constant.

3.3 Heigham Water Treatment Works

Heigham water works in Norfolk was a 60 Ml/d maximum capacity plant which supplied approximately 250,000 customers in and around Norwich. Raw water was abstracted from the River Wensum and received settlement to remove larger particulate matter. Ferric sulphate coagulation was used and clarification was achieved by dissolved air flotation. Anthracite, sand and garnet (ASG) filters were used followed by ozone oxidation, carbon filters for pesticide removal and chlorination for final disinfection. The plant had eight rapid ASG filters each of 66 square metres surface area. The filters were operated at approach velocities up to 5 metres per hour. Flow rate was controlled by effluent butterfly valves. The filter bed comprised 125 mm of anthracite, 180 mm sand, 120 mm fine garnet and 100 mm coarse garnet. These were supported on 50 mm fine gravel over 75 mm coarse gravel.

The filtered water piping is shown in Plate 3.1. Filtered water was removed from the underdrain by a 300 mm internal diameter pipe. An ultrasonic flow meter was fitted to this pipe to record the flow from each filter. Note that the meter was fitted in the centre of a long straight section of pipe. The flow passed through the butterfly flow control valve at the end of this section before discharge into the clearwell. Compared to the other plants visited, the filtrate piping at Heigham had few constrictions in the flow. There were no changes in pipe diameter, no 90° bends or dead ends. Indeed, the only constriction was the butterfly control valve and this was positioned at the end of a long straight section of pipe far from the filter underdrain. It was expected that any surges present would be less significant than those observed at the three other plants in the survey. Flow measurements were taken every few seconds from the ultrasonic meter. This information was captured by the plant SCADA system and could be inspected for evidence of flow rate fluctuations. Figure 3.6 illustrates a typical set of flow data from filter one at Heigham. From this chart it can be seen that filter one was operated at 55 to 60 litres per second (3.3 metres per hour) on the day of the visit. This is quite a low approach velocity for rapid filtration. Close inspection of the flow profile revealed small fluctuations in the flow rate. These fluctuations appeared to be around 2 % amplitude and occurred perhaps 5 times per minute. These are small and infrequent compared with the findings of Baylis and the results obtained at Cropston and Church Wilne discussed above and Melbourne discussed below. No head loss information was available from the filters.

3.4 Melbourne Water Treatment Works

Melbourne water works in Derbyshire was a 240 Ml/d maximum capacity plant which supplied a population of 750,000. The plant was supplied from two storage reservoirs, Staunton Harold and Foremark. The process train comprises pH adjustment at the intake using sulphuric acid and caustic soda. Coagulation was achieved by the addition of ferric sulphate and subsequent clarification was produced by dissolved air flotation. This treatment produced a clarified water with a turbidity of around 1.0 NTU. Melbourne water works had 34 rapid gravity filters each of 42.8 square metres surface area which further reduced the turbidity to 0.15 to 0.2 NTU. Taste and odour problems were controlled by 14 granular activated carbon filters and

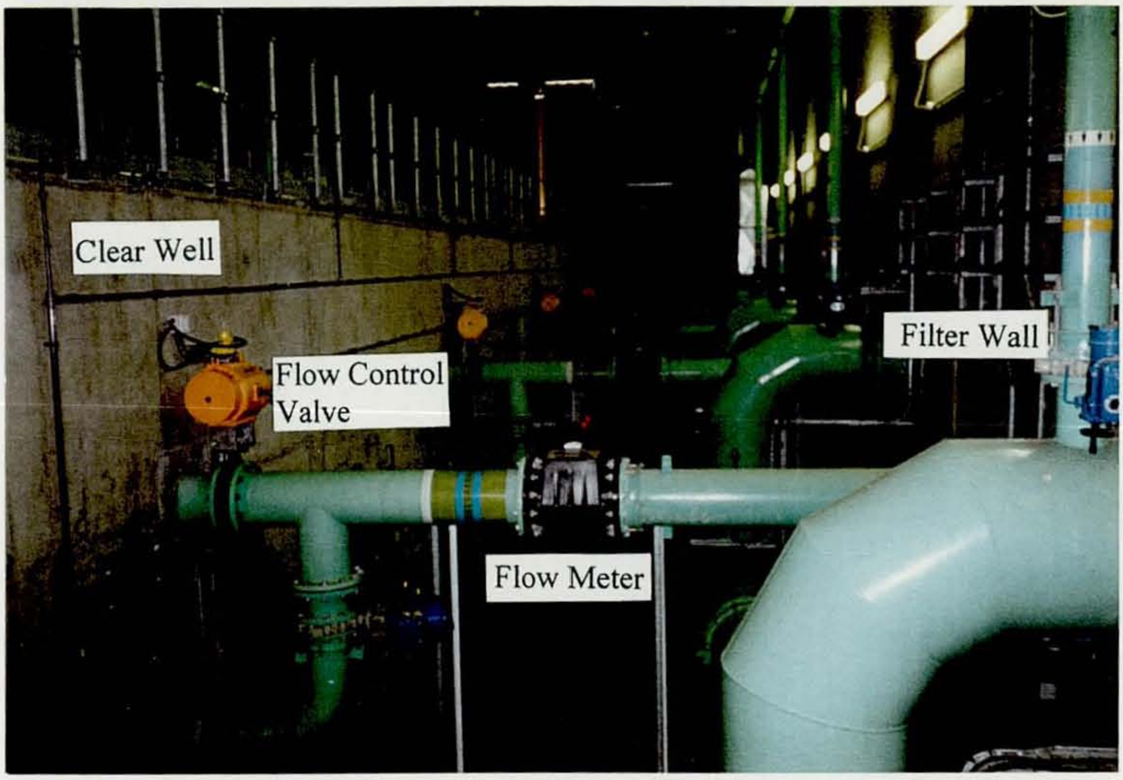


Plate 3.1: Heigham water works filter gallery

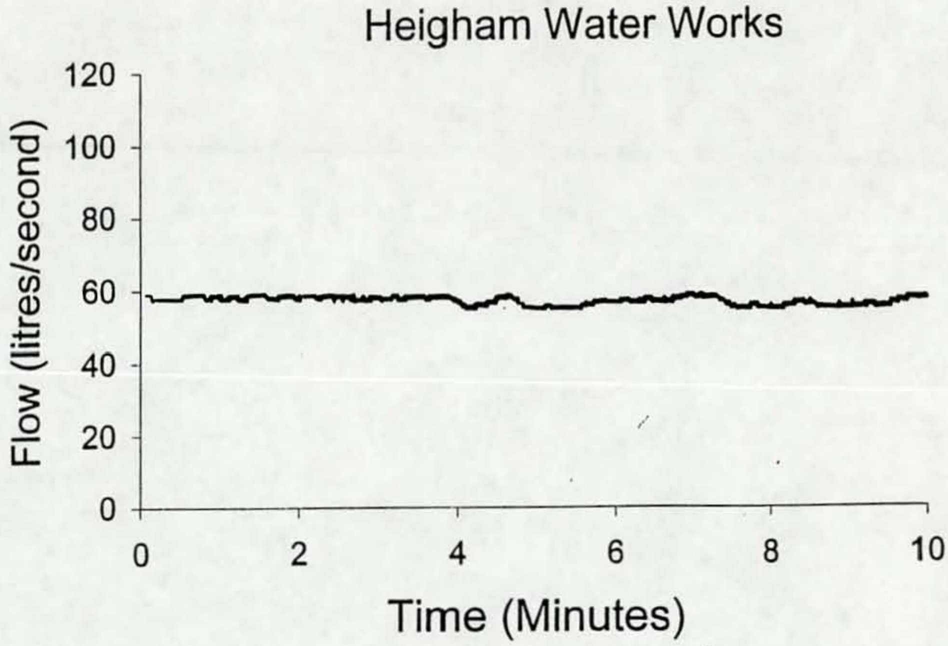


Figure 3.6: Heigham flow rate fluctuations

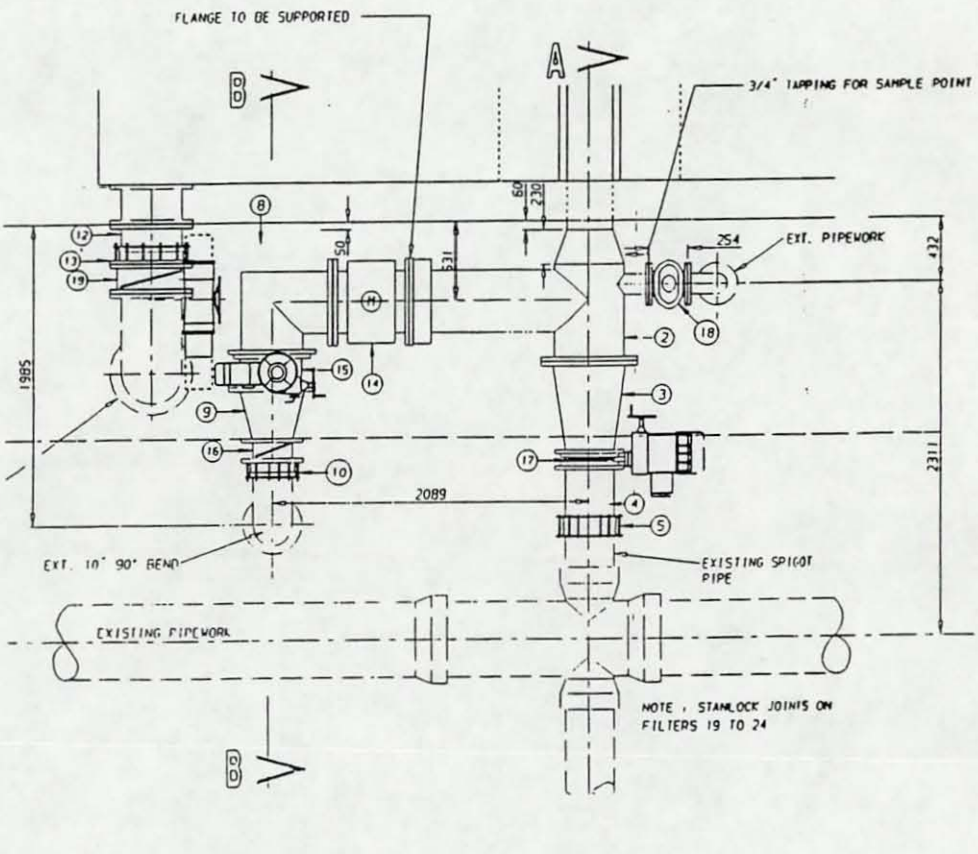
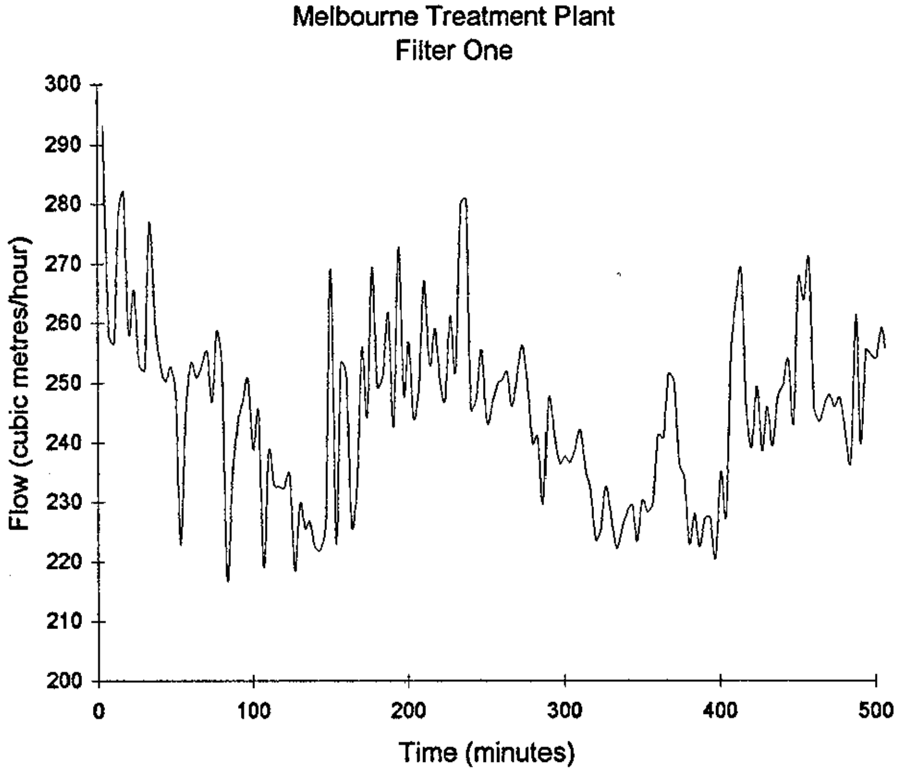


Figure 3.7: Melbourne filter gallery

disinfection was achieved by chlorination leaving a residual of 0.6 mg/l free chlorine. The rapid filters were operated at an approach velocity of 5.7 metres per hour. The filter beds comprised 360 mm of 1.2 to 2.5 mm anthracite, 240 mm of 0.5 to 1.0 mm sand, 120 mm of 0.3 to 0.6 mm garnet over supporting gravel layers. The design of the filtrate piping is shown in figure 3.7. Filtered water was drawn from the underdrain by a 300 mm internal diameter pipe. The flow passed from this pipe to a 450 mm internal diameter T piece by a short taper. Ahead of the flow was a second taper with the closed backwash valve fitted at the end; effectively a dead end. The filtered water executed a 90° turn into a 400 mm internal diameter pipe fitted with an electromagnetic flow meter. Immediately after the meter the flow executed a sharp 90° turn into a 400 mm internal diameter butterfly flow control valve followed by a 400 to 250 mm taper. A 250 mm internal diameter butterfly isolating valve and 90° bend passed the filtrate into the filtered water channel. There appeared to be ample opportunity for surges to occur in this filtrate piping design. Any fluctuations in the flow should be obvious from inspection of the flow records stored by the plant SCADA system. Figure 3.8 (a) and (b) illustrates typical flow rate information from filters one and six. From these charts, it can be seen that the filters operated at average flow rates of 240 to 250 cubic metres per hour. However, it is evident from the plots that the flow rates fluctuate by as much as 25 % peak to peak in a random erratic manner similar to the findings of Baylis and Hudson and the observations at Church Wilne Water Works described above. However, it should be noted that the SCADA system only sampled the signals from the flow meters once every 200 seconds. Ideally, the flow should be sampled every second. However, greater resolution wasn't possible in the circumstances. In addition to the flow rates, the plant SCADA system recorded the filter head losses via pressure measurements above the filter bed and within the filtrate piping. Figure 3.9 (a) and (b) illustrates the head loss data for filters one and six corresponding to the flow data shown in figure 3.8. Filter one head loss can be seen to fluctuate by 50 to 100 mm at a head loss over 1000 mm. Filter six head loss can be seen to fluctuate by 20 to 50 mm at a head loss of 200 mm. Once again, it appeared that the head loss fluctuation increased as the loss of head rose. Note however that the measured fluctuation in flow rate was of similar amplitude in both filters. This suggested that the fluctuation in head loss rose as the head loss increased for an essentially constant fluctuation in flow rate. The flow



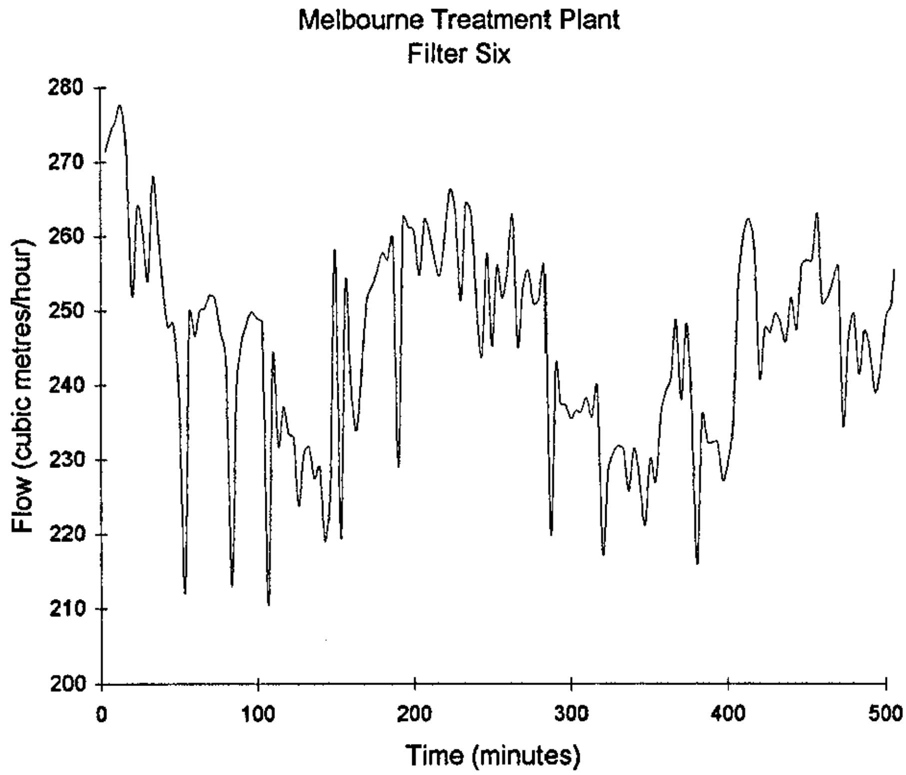
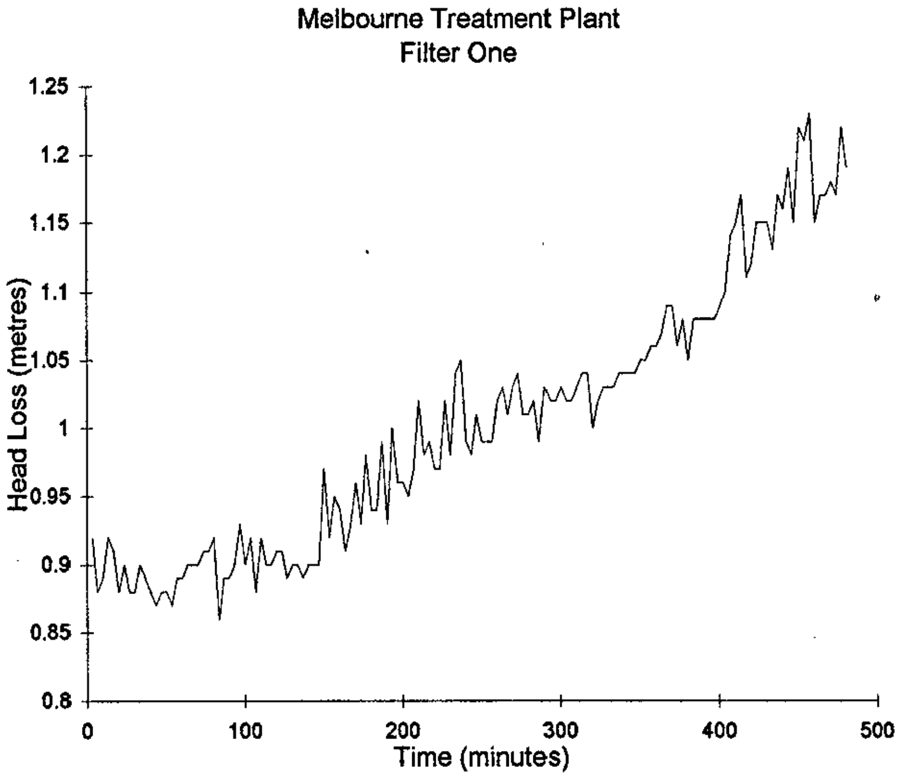


Figure 3.8 (a) & (b): Melbourne flow rate fluctuations



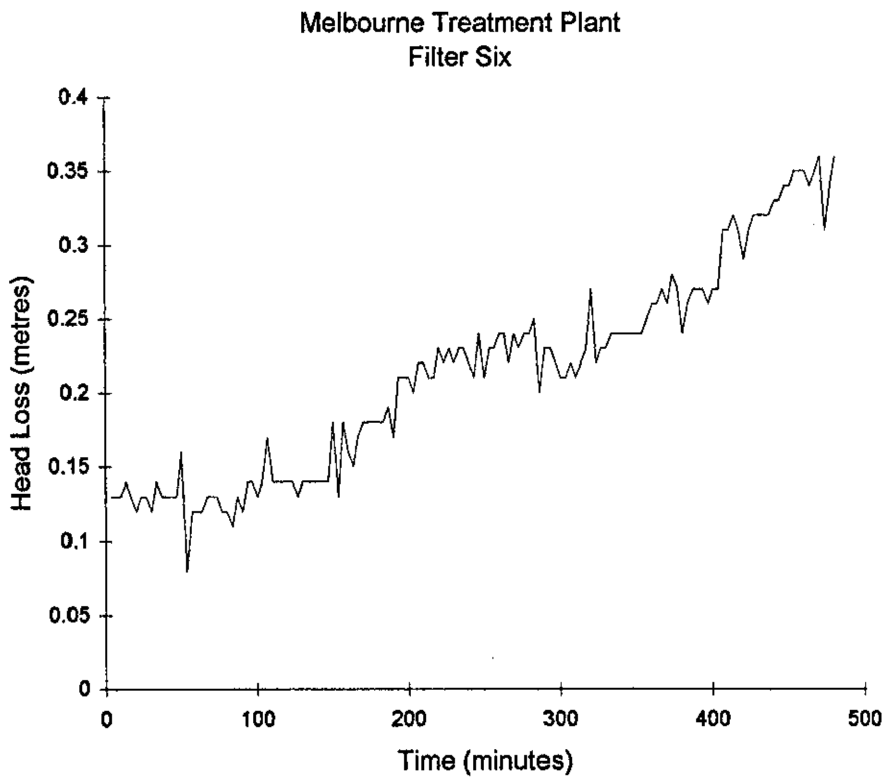


Figure 3.9 (a) & (b): Melbourne head loss fluctuations

fluctuation was reflected in the head loss fluctuation irrespective of the head loss at the time. This suggested that the flow rate fluctuation remained constant during the run since the average flow rate was held constant by the control system. The fluctuation in head loss increased because the loss of head increased not due to any change in the flow fluctuations. Similar to the flow rate, the SCADA system sampled the pressure measurement only once every 200 seconds. As such, little can be stated about the amplitude and frequency of surging at any greater resolution. Never the less there is still evidence that the flow rates in rapid gravity filters do fluctuate in the manner described by Baylis and Hudson.

Baylis, discussed in section 2.1.2, found that the surge amplitude appeared to be related to the square of the velocity of the flow in the filtrate piping. He measured this velocity at the throat section of the venturi meters used in his study. A comparison of the filtrate pipe velocities at each of the treatment plants visited in this study is shown in table 3.2. Pipe velocities were calculated at the butterfly control valves at each plant for comparison.

Plant	***Pipe Velocity (m/s)***	***Pipe Diameter at Control Valve (mm)***	***Reynolds Number***	***Surge Amplitude***	***Surge Frequency***
Cropston	0.71	250	177,000	20-40 %HL	1/15min
Church Wilne	1.6	250	398,000	2-10 % HL	25-100/min
Heigham	0.85	300	254,000	1-2 %Q	5/min
Melbourne	0.55	400	220,000	Max 25 %Q	<1/3min

Table 3.2: Comparison of pipe characteristics and observed surging (HL - Head Loss, Q - Flow)

The velocity of the flow at the butterfly control valves ranged from 0.55 to 1.6 metres per second. Taking the water temperature as 15°C and calculating the Reynolds numbers from these velocities and the valve diameters gave figures from 177,000 to 398,000. These numbers clearly show that the flow in the filtrate piping of modern rapid filters can be turbulent and thus can produce surges. Evidence of surging was found at each plant to a greater or lesser extent. However, the smallest and least frequent surging was found at Heigham Water Works. The pipe velocity of 0.85 metres per second gave a Reynolds number of 254,000, similar to that found at Melbourne. However, surge amplitudes were found to be only 1 to 2 % of the flow at rates of only 5 oscillations per minute. Comparison of the filtrate piping design and appurtenances at each of the plants found that Heigham had the least flow constrictions and bends. It therefore seems likely that the smaller surging at Heigham was the result of the simpler filtered water piping design. Even though the high Reynolds number could produce turbulent flow conditions and thus surges in the filtrate piping, the long straight section of pipe housing the flow meter and control valve seemed to reduce the incidence of turbulent flow. It is recognised that this speculation is based on a small sample of data and to be confirmed would require a more thorough survey of rapid filter designs. Nevertheless, these results suggest that convoluted filtrate piping design contributes to the occurrence of surges in modern rapid gravity filters. It may be the case that adopting more simplified filtered water piping designs similar to Heigham Water Works would help minimise the occurrence of small frequent flow rate fluctuations in rapid filters.

3.5 Chapter Summary

A brief survey of modern rapid gravity filters was conducted locally to provide additional information on the nature of surging. The nature of the measurement systems in use limited the usefulness of the results but nevertheless evidence of surging was found at each plant visited either in the head loss trends or the flow metering data. Surges ranging from 1 % of the flow to 40 % of the measured head loss were found at frequencies of occurrence up to 100 oscillations per minute. The absolute amplitude of the pressure fluctuations observed was found to increase with rising head loss during the filter cycle. These findings are similar to the results published by Baylis and Hudson discussed in chapter two. Comparison of the filter plant designs suggested that the occurrence of surging was affected by the type and number of flow constrictions present in the filtered water piping. Minimising these flow constrictions in future designs may well reduce the occurrence of surging in rapid filters. It was decided to pursue the objectives stated in section 2.5 by applying surges to laboratory scale rapid filters at similar amplitudes and frequencies to those found at local water works and by previous researchers.

4.0 EXPERIMENTAL APPARATUS

In this chapter the experimental apparatus used in this investigation of surging is described in detail. The apparatus was developed over 36 months and the final design and layout is presented here. The details of and the reasons for changes to the design of individual components in the system are described within the discussion of its operation in this and the following chapter describing the experimental procedure. This chapter begins with a brief overview of the whole plant and subsequent sections describe the design of each sub-system in depth.

The approach taken in this study was, firstly, to construct two identical laboratory scale filters and the associated equipment needed to run them. Secondly, the apparatus and experimental methods were developed until similar and reproducible performance from both filters was established. The performance of these two filters was then compared applying surges to one filter only. The second non-surging filter was used as a control for comparison. Test runs with no surging in either filter were conducted periodically in between sets of surging test runs to ensure on-going good control over the filters' performance. Filter performance was assessed by measurement of head loss development and removal efficiency of suspended matter during the filter cycle using the techniques described in this chapter. A series of surging test runs were conducted in this manner using the apparatus to characterise the effects of different surging amplitudes and rates of occurrence. The apparatus used is presented in detail in the following sections.

4.1 Apparatus Overview

The layout of the experimental rig is illustrated in figure 4.1. A photograph of the apparatus is presented in Plate 4.1. The prepared raw water suspension was held in the stirred storage tanks shown. Preparation of the test suspension will be discussed in section 5.1 The test suspension was pumped to a small tank fitted to a gantry at a height several metres above the storage tanks which provided an elevated constant head. Suspension overflow was returned from this constant head tank to the storage tanks to provide mixing. The test suspension was drawn from the constant head tank by plastic siphons. These delivered flow to the two filter columns via plastic tubing.

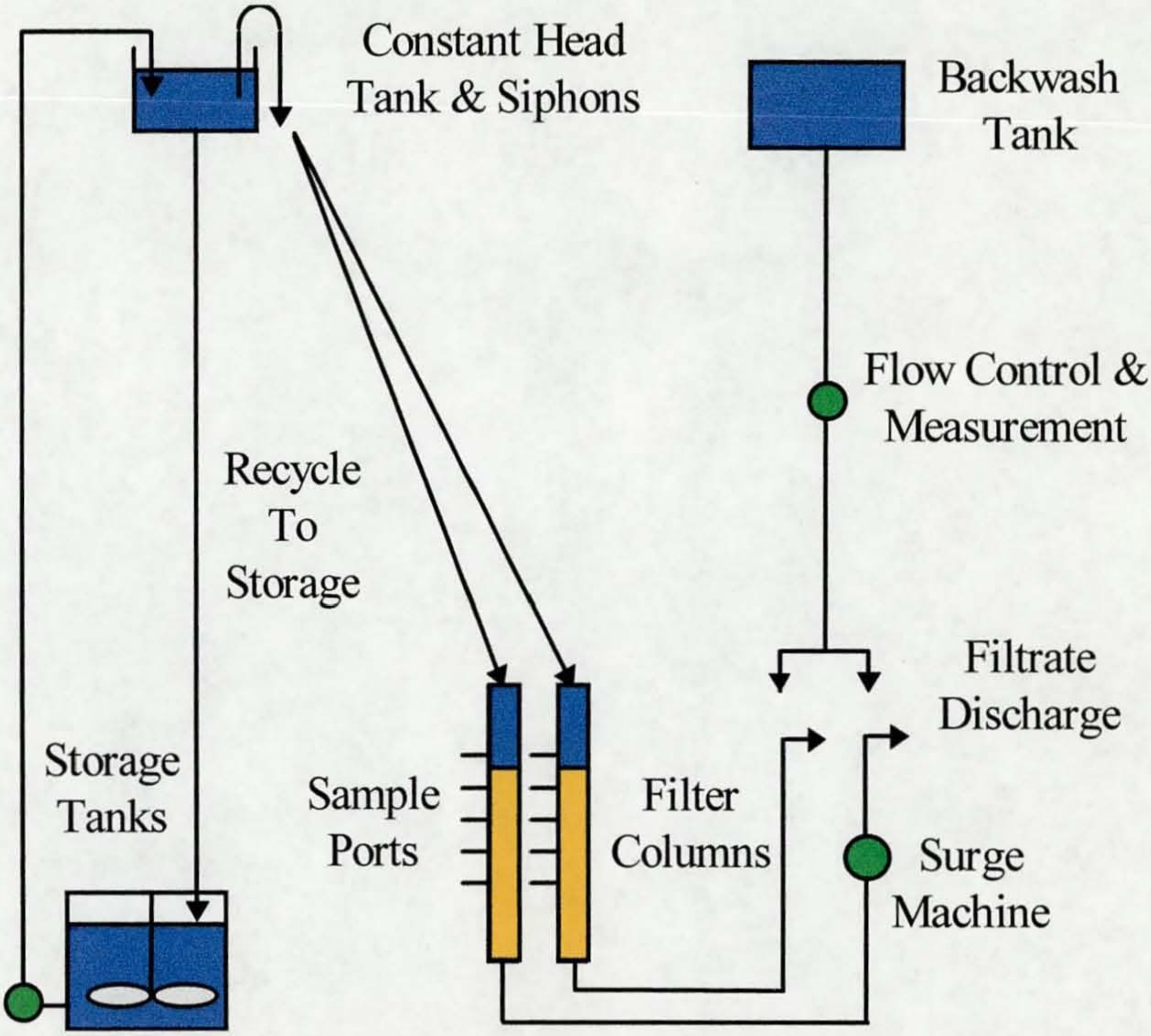


Figure 4.1: Schematic layout of the experimental apparatus

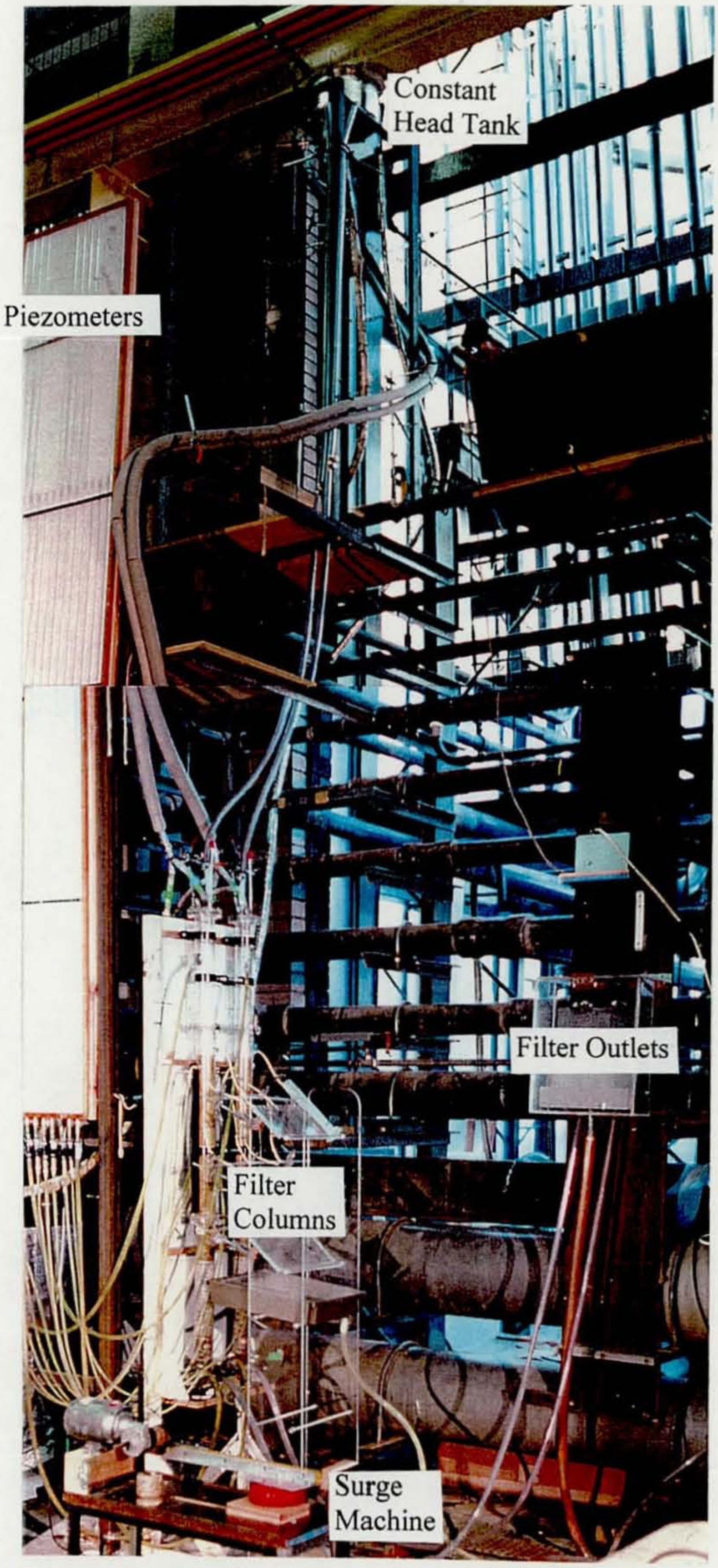


Plate 4.1: Experimental Apparatus

Flow passed down the filter columns through the filter media and support layers and out of the base of the filter columns. Fitted to the filter columns were sample ports for pressure measurement and suspension sampling. Pressure was measured using piezometer tubes. Suspension samples were gathered in measuring cylinders in the sample stand shown. Flow passed from the base of the filter columns to the discharge ports by plastic piping. Fitted to the effluent water piping of one filter was a machine designed to produce flow rate fluctuations. The discharged water was collected in a trough and routed to a waste water sump for disposal. A backwash water storage tank was located at an elevated height. Backwash water was drawn from this tank and passed to the filter columns by detachable piping fitted to the discharge ports. Backwash water was passed up through the filter columns and out at the top by overflow ports. The backwash water was collected and routed to the waste water sump for disposal.

4.2 Suspension Storage & Delivery

The raw water suspension was stored in three 500 litre capacity tanks in the arrangement shown in figure 4.2. Each tank was fitted with an electric motor driven paddle operated at 20 revolutions per minute to keep the suspension thoroughly mixed and prevent settlement. Storage tanks two and three were raised on platforms to provide gravity feed into tank one controlled by a ball float valve. This arrangement maintained a constant water level in tank one, necessary to maintain a constant pumping rate to the elevated head tank. The suspension was recycled in tank one by a centrifugal pump. The rate of flow of suspension to the elevated constant head tank was controlled by a gate valve fitted to this recycle loop. Recycling the suspension in this manner aided mixing and prevented settlement. To maintain a constant suspension temperature, tank one was insulated with silvered bubble insulation and fitted with two 1 kilowatt immersion heaters controlled by a thermostat and temperature sensor. The temperature sensor was inserted into tank one through the tank wall. The plastic piping used to connect tank one and the constant head tank was wrapped in insulation to reduce heat loss and prevent algal growth. The constant head tank was placed at the top of the gantry at an elevation three metres above the filter columns to provide sufficient head to run the filters to a high head loss. The design of the constant head tank is illustrated in figure 4.3. Plate 4.2 shows the constant head

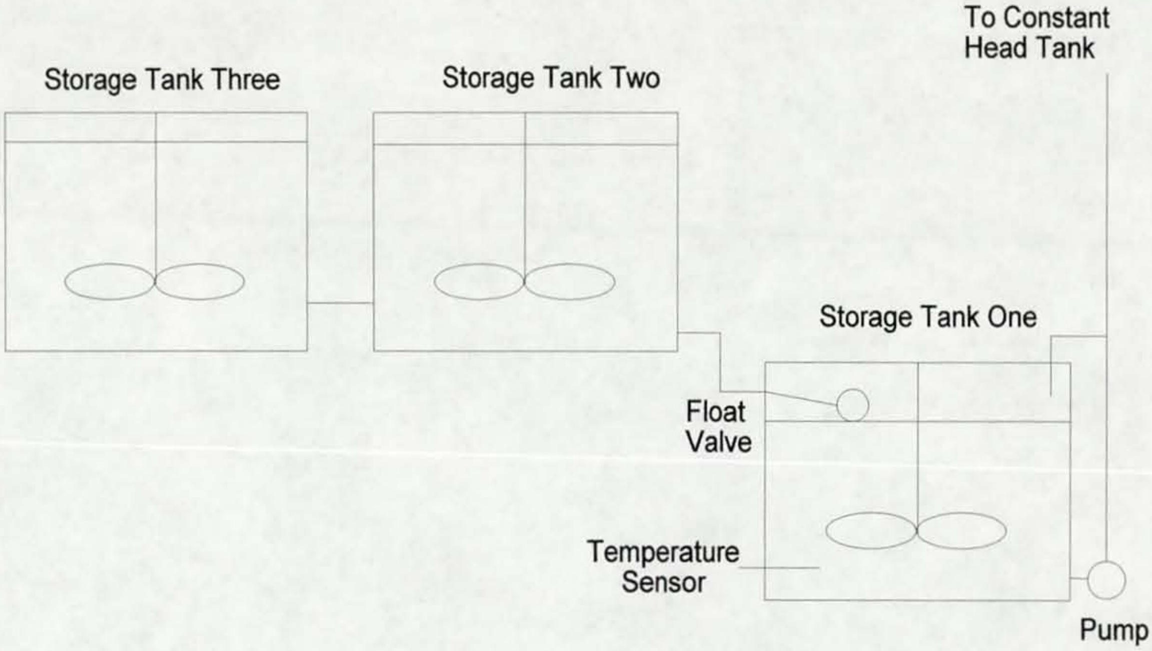


Figure 4.2: Suspension storage tank arrangement

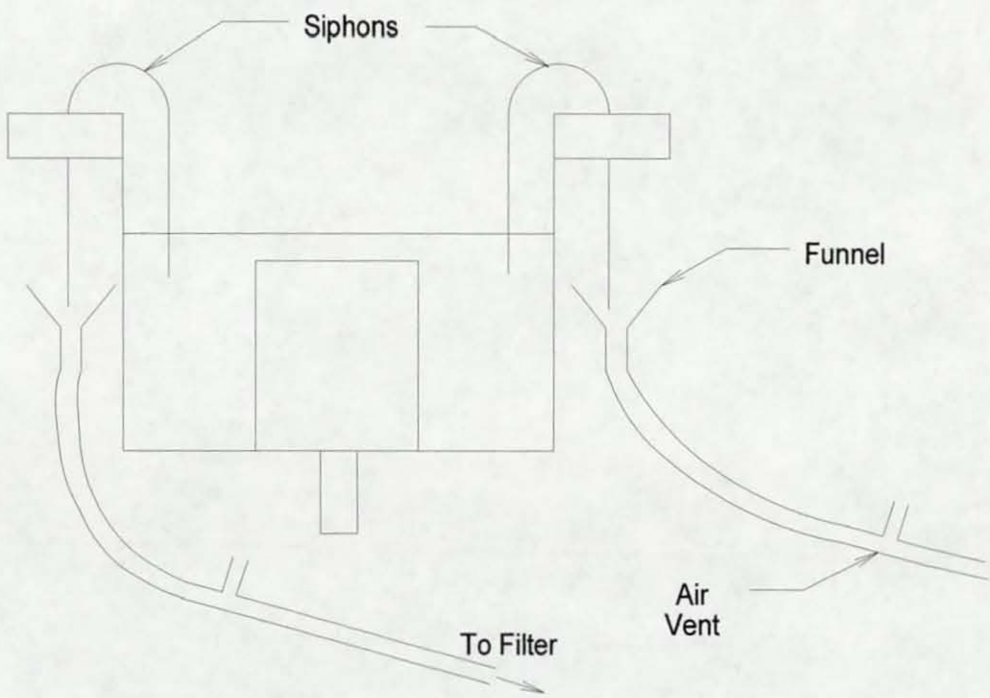


Figure 4.3: Constant Head Tank, Siphons and Inlet Piping

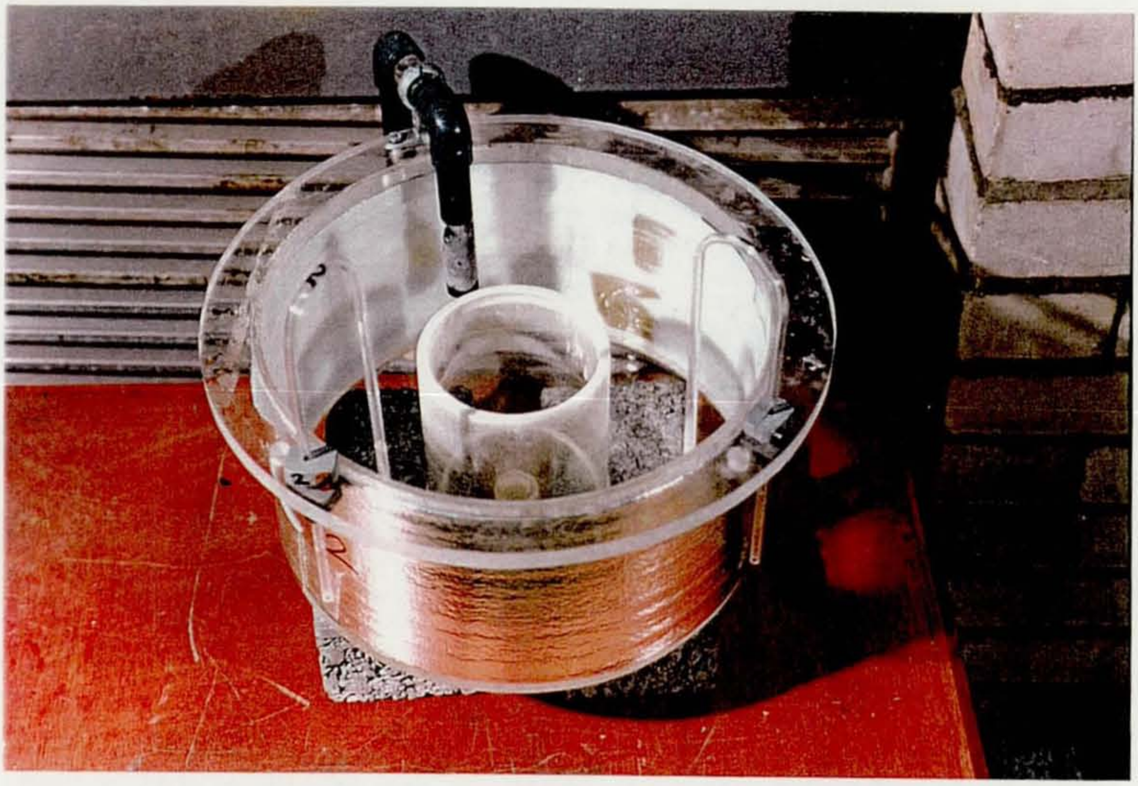


Plate 4.2: Constant head tank fitted with siphons

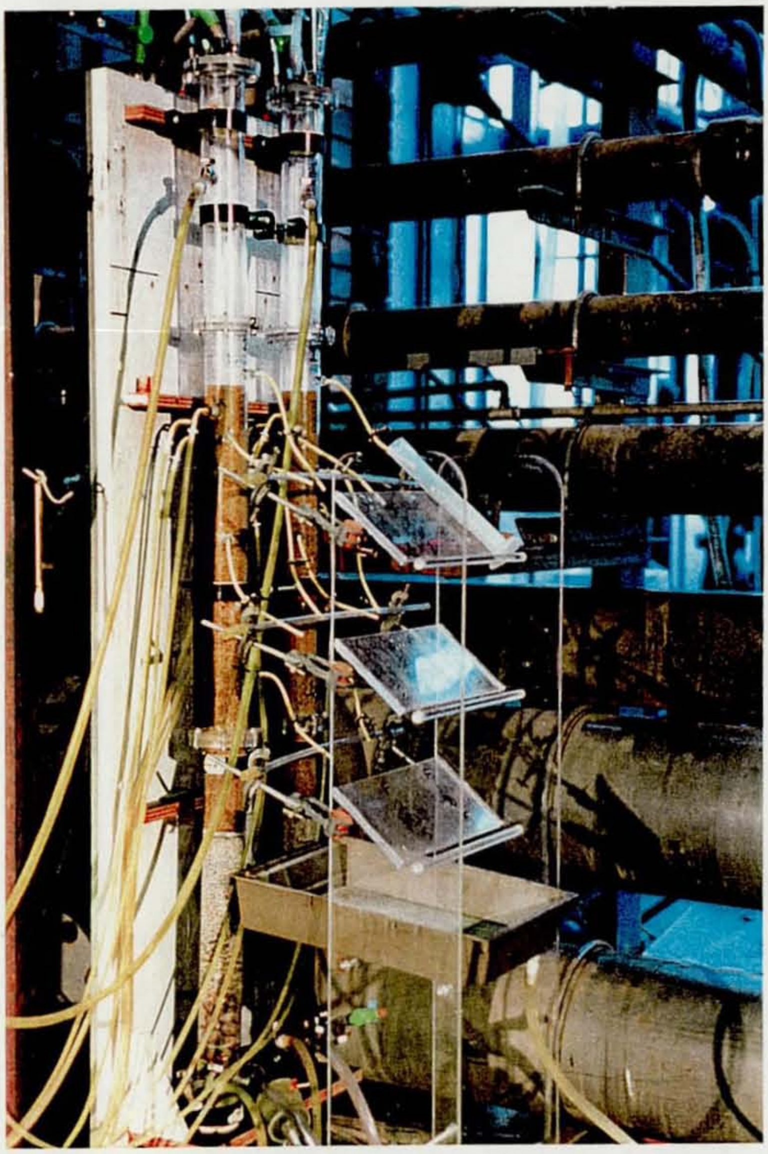


Plate 4.3: Filter columns on stand

tank fitted with the plastic delivery siphons. The tank was manufactured from Perspex to ease observation of the suspension flow and level from below. The constant head tank was 200 mm tall with an internal diameter of 350 mm. The top of the tank was fitted with a flange to clamp the siphons in place. Suspension overflow was provided by an internal circular weir 100 mm in diameter. Suspension entered the tank from the PVC pipe bolted to the flange. The flow stabilised in the head tank before flowing over the internal weir. The head tank was levelled in place to evenly distribute the flow over the weir. This resulted in a constant stable water level for the siphons to draw from. The overflow was collected in the centre cylinder and returned to the storage tank by the discharge port in the base of the tank. The tank was fitted with a lid to reduce heat loss and prevent air borne contamination. The constant head tank was covered with silvered bubble insulation to reduce heat loss and algal growth. Two delivery siphons were manufactured from 4 mm internal diameter clear plastic piping and are shown in figure 4.3 and Plate 4.2. The inlets were moulded into bell mouths to smooth the flow streamlines into the siphons and improve flow rate stability. The siphons were fitted with two plastic screw clamps. These were used to clamp the siphons in place on the head tank flange. The head across the siphons was set to deliver identical flow rates of 0.35 litres per minute to the filter columns. The siphon flow was collected by plastic funnels and fed into flexible plastic tubing as shown in figure 4.3. Downstream from the collector funnels, plastic 'T' piece fittings were installed. These were clamped in place at an angle of 30° to the horizontal with the free connection held upright. This allowed the suspension to flow along the bottom of the inverted 'T' leaving the upright section unsubmerged. This device provided pressure release from the feed piping to prevent air trapping and suspension overflow. The feed piping was routed to the filter column inlets at an incline of 45° to slow the suspension velocity and prevent air bubble and foam generation and entrapment in the flow into the filters. The test suspension could be passed into the filter columns or recycled into tank one by cut-off valves fitted to the top of the filter columns.

4.3 Filter Columns & Sampling Ports

The filter columns were manufactured from 58 mm internal diameter Perspex piping to allow observation of the media and suspension flow through the sides of the columns. Each filter was manufactured in four modules; the top cap, the top section, middle section and bottom section to allow easy assembly and disassembly, cleaning and packing. These sections were bolted together and mounted on an independent filter stand to eliminate unwelcome vibrations and physical disturbances. The assembled filter columns and filter stand are shown in Plate 4.3. Fitted at various depths and orientations on the filter columns are the pressure and suspension sampling ports. The pressure sampling ports were connected to water piezometers. The suspension sampling ports fed into measuring cylinders mounted on a collector stand. The filter columns were packed with filter sand supported on layers of graded gravel. The design of each of these major components of the filter columns and sample ports and the specifications of the filter beds will be described in detail in the following sub-sections.

4.3.1 Filter Column Top Cap & Top Section

The top cap of the filter columns is illustrated in figure 4.4 and Plate 4.4. 115 mm diameter disks of Perspex were cut from 10 mm thick sheeting. Ports were drilled in the centre of the disks and fitted with Perspex tubes. The suspension piping from the constant head tank was attached to these inlet tubes via cut off valves in the arrangement shown in Plate 4.4. These cut-off valves were used to route the suspension flow from the siphons either into the filter columns or to recycle the suspension back into storage tank one as required. Ports were drilled adjacent to the suspension inlet tubes. Plastic tubing, fitted to these ports, was attached to the head tank gantry and opened to the atmosphere several meters above the filter columns. These vents allowed air within the filter columns to escape as the water levels rose during a filter run. The underside of the top caps were tapered at a shallow angle to prevent trapping of air bubbles. A 'T' piece was fitted to these air vent ports as shown in Plate 4.4. The outlet from this 'T ' piece was used as the wastewater overflow during backwashing. Waste backwash water was routed to the wastewater channel for disposal. During normal filter operation these backwash overflows were shut off.

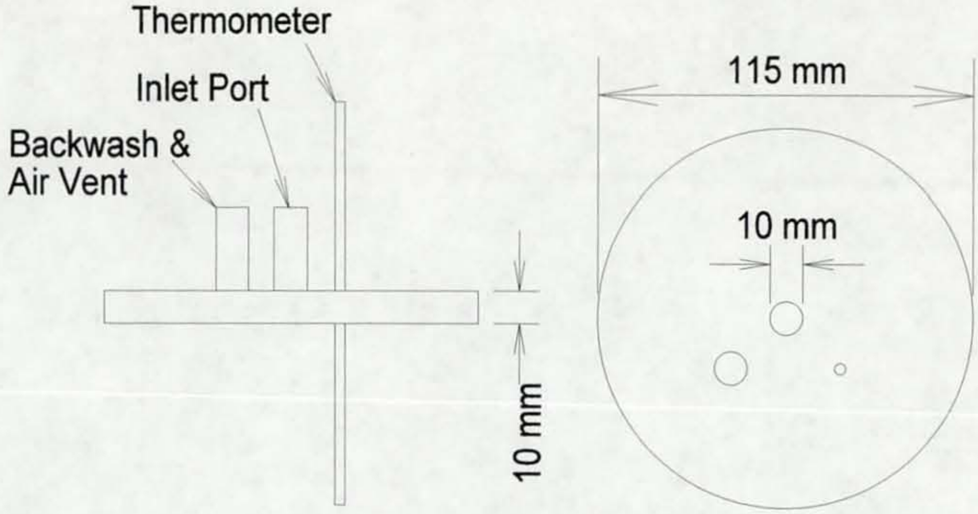


Figure 4.4: Filter Top Cap

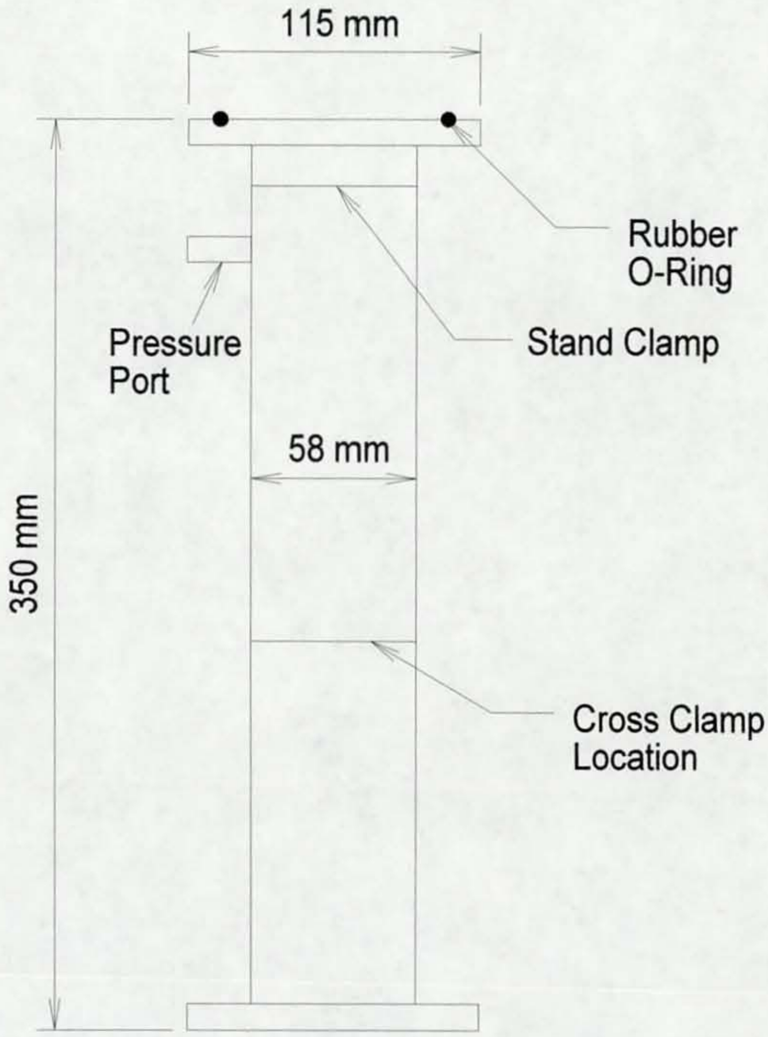


Figure 4.5: Filter Column Top Section

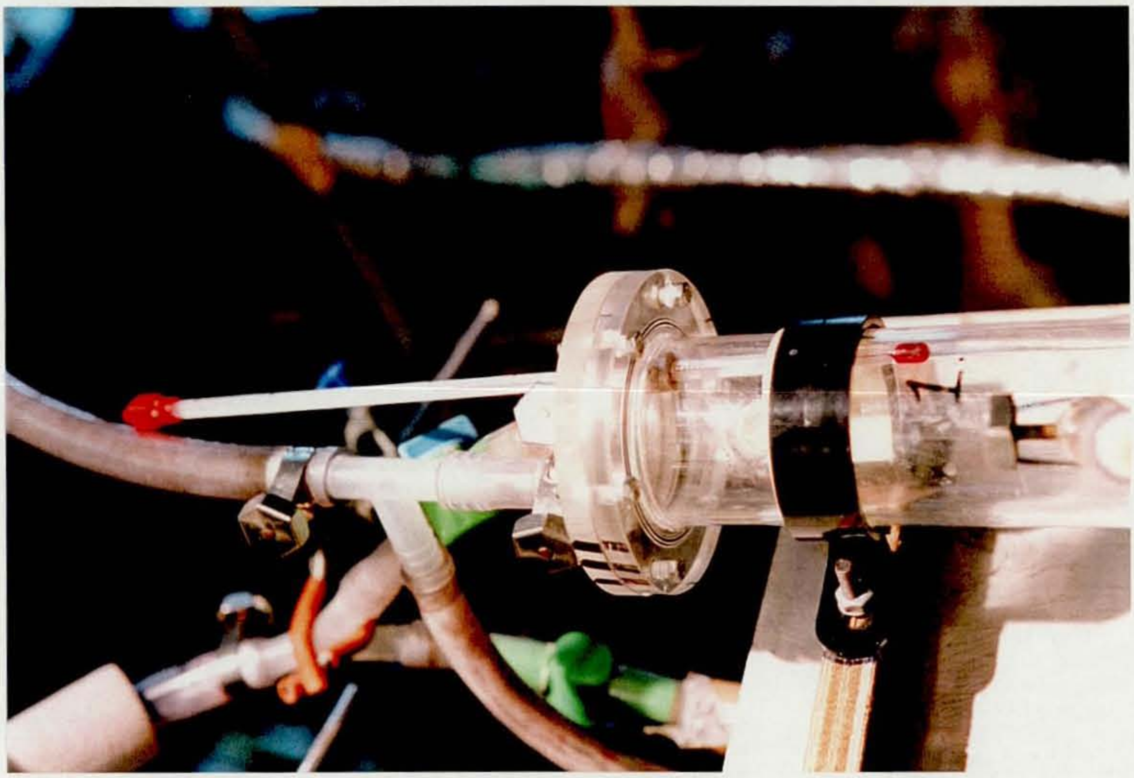


Plate 4.4: Filter top cap

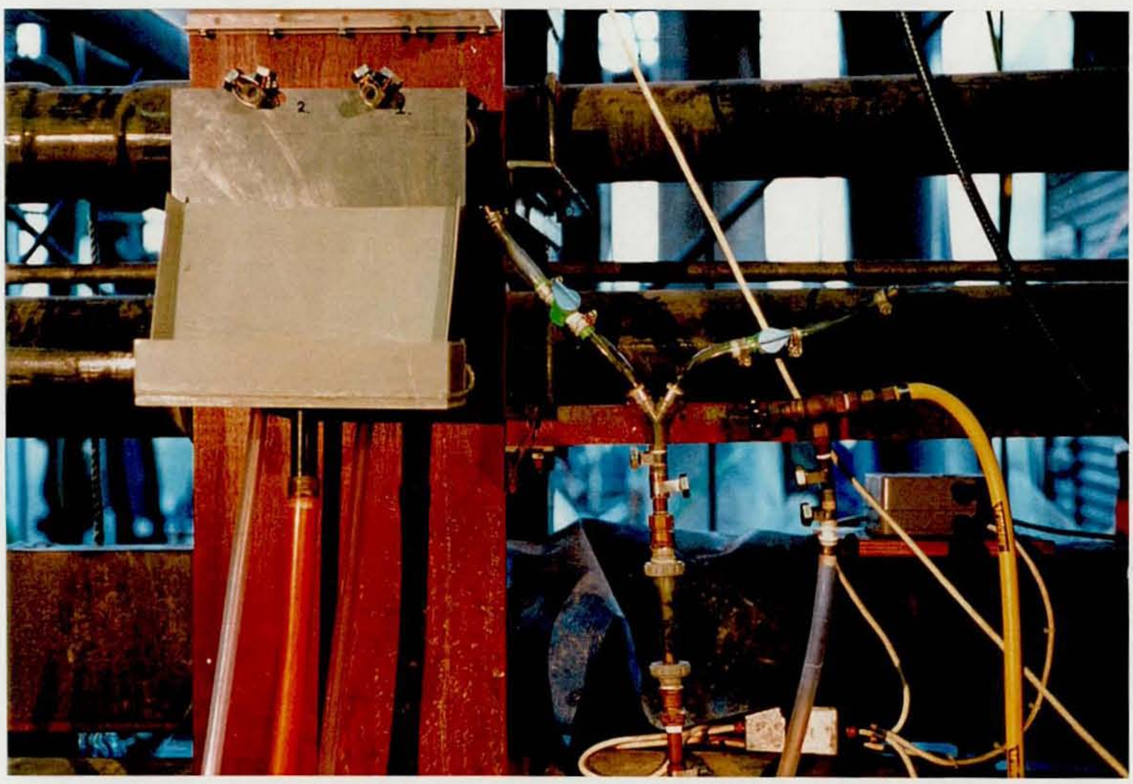


Plate 4.5: Outlet ports and backwash piping

Glass thermometers were glued into holes drilled in the top caps. The thermometers extended down into the top section of the filter columns to monitor the water temperatures in each column. The top caps were bolted to the filter top sections as shown. A watertight seal was ensured by the use of rubber O-rings set into the flanges of the top sections. The top section of the filters is shown in figure 4.5 and can be seen in Plate 4.3. The top section comprised a 350 mm long Perspex tube 58 mm in internal diameter. 115 mm diameter flanges were glued on at each end flush with the end of the pipe. The top flange had a circular groove counter sunk to take the rubber O-ring seal. Each flange was drilled to take the nuts and bolts used to secure the top cap in place and to connect to the middle section of the filter column. Pressure tappings were fitted to the top sections in the position shown in figure 4.5. These fittings were connected to the water piezometers and allowed measurement of the rising supernatant water levels over the filter beds. The top sections were designed to allow sufficient head room to allow expansion of the filter media during backwashing without media overflow. The top sections were also designed to provide stabilisation of the flow of incoming suspension above the filter media during normal filter operation without disturbing the surface of the sand. The top sections were fitted with thermal insulation to minimise temperature changes and to reduce algal growth in the columns. The filters were clamped to the filter stand at the top and bottom of the filter columns. The top clamp position is illustrated in figure 4.5. In addition, the filters were clamped to each other at three positions along the columns. The first of these cross clamps is also illustrated in the figure. These cross clamps were necessary to achieve reproducible bed settlement after backwashing and are discussed fully in section 4.4.2.

4.3.2 Filter Column Middle Section

The filter column middle sections are illustrated in figure 4.6 and can be seen in Plate 4.3. These middle sections comprised 600 mm long tubes of 58 mm internal diameter Perspex with flanges glued at each end. A circular groove counter sunk in the top flange accommodated the O-ring rubber seal between this section and the top section of the column. Fitted to the front of the middle sections were millimetre scales used to determine the filter sand bed levels after backwashing and to monitor for bed movement during filter operation. Pressure tappings and suspension sampling ports

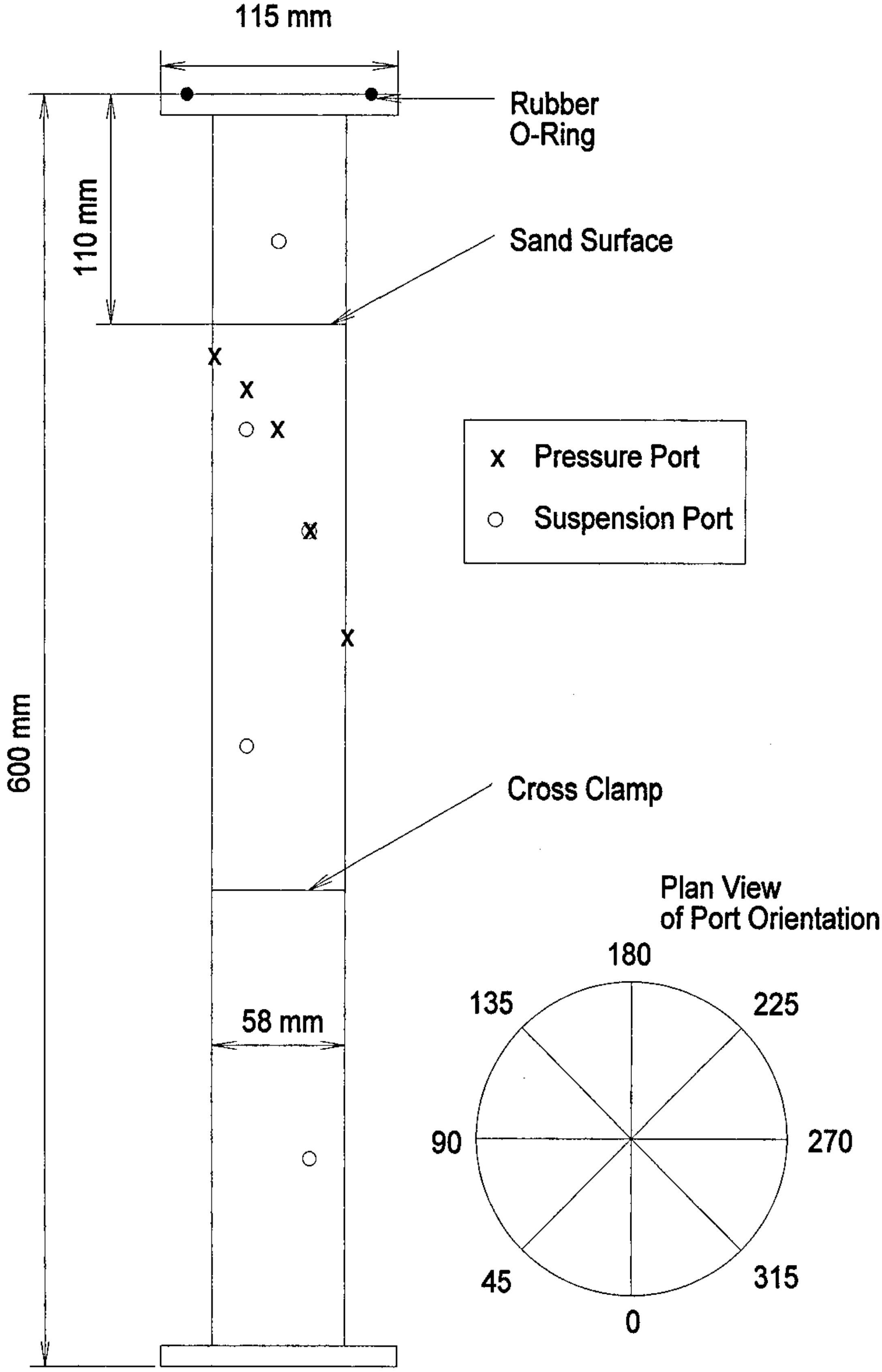


Figure 4.6: Filter Column Middle Section

were fitted at various depths and orientations along the middle sections of the filter columns. Figure 4.6 and table 4.1 illustrate the position and identification of each sample port.

Port Function	*Depth below surface (mm)*	*Orientation (Front 0°, Clockwise)*
Pressure	0	90°
	15	90°
	30	135°
	50	180°
	100	225°
	150	270°
	600	90°
Suspension	0	0°
	50	45°
	100	315°
	200	45°
	400	315°

Table 4.1: Pressure & Suspension Sampling Port Positions

The ports were arranged in this manner such that consecutive ports through the depth did not line up. This was necessary to avoid creating a line of weakness through the filter bed. The second of the three cross clamps between the two filter columns was located at the position shown in figure 4.6. The middle sections were fitted with thermal insulation to minimise temperature changes and prevent algal growth.

4.3.3 Filter Column Bottom Section

The filter column bottom sections are illustrated in figure 4.7 and can be seen in Plate 4.3. The filter bottom section comprised a 550 mm long tube of Perspex 58 mm in internal diameter. These were closed at the bottom and had plastic flanges glued at the top end. These flanges included O-ring seals in counter sunk grooves to provide water tight seals with the middle sections. At the base of the bottom section three ports were fitted 50 mm above the base. 10 mm diameter holes were drilled in the front and sides of the filter columns with Perspex tubes glued in place. The front ports provided the

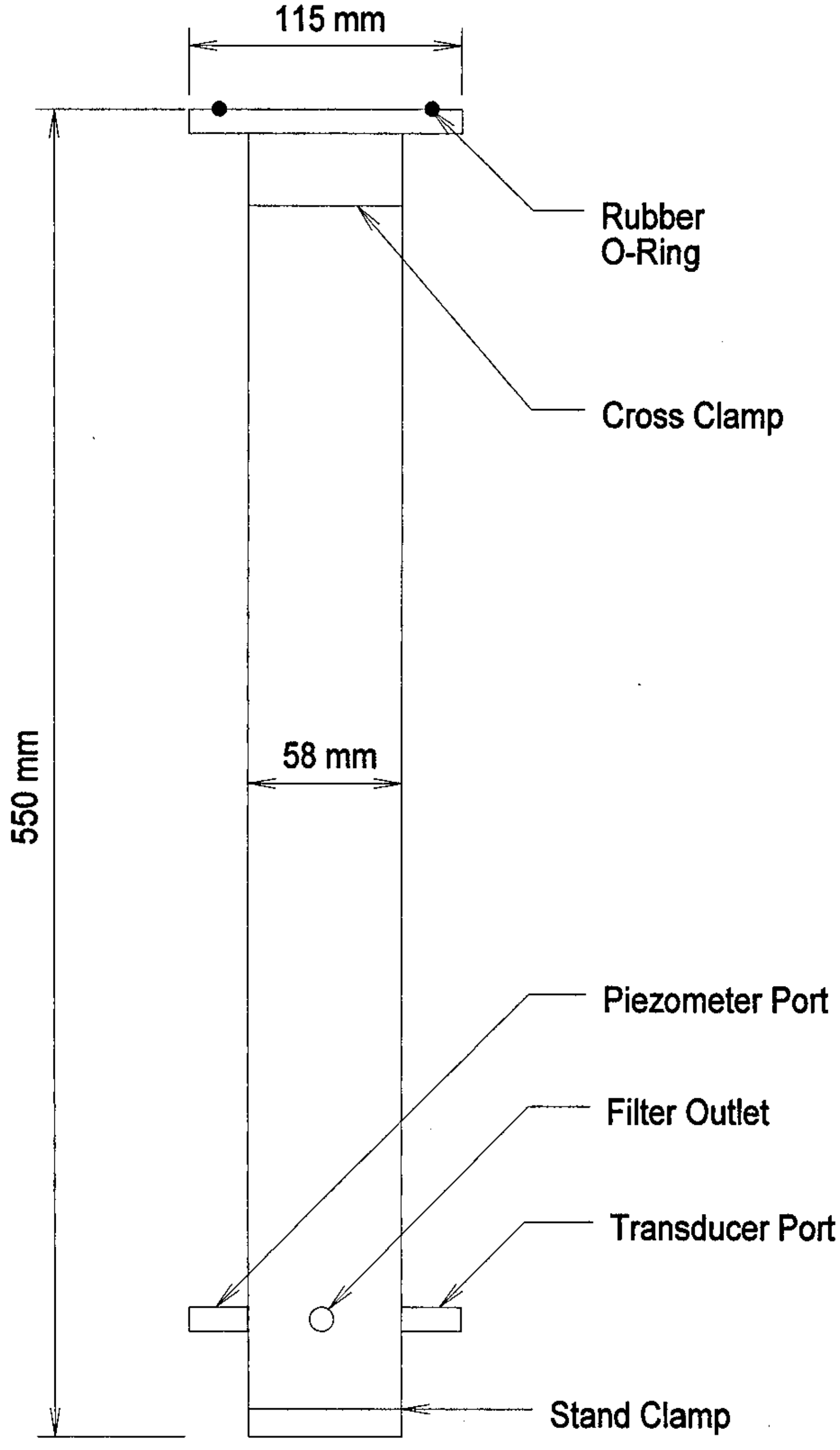


Figure 4.7: Filter Column Bottom Section

filtered water outlet into the effluent piping. The side ports were used for pressure measurement and filter draining for repairs. One port was connected to the water piezometers to measure the pressure beneath the filter media. The second port was used to attach a pressure transducer to record the profile of the surges applied during the experimental investigation. The transducer used was a 0-350 mbar vented gauge device (Model PT213-157) supplied by Kulite Sensors, Basingstoke. The signal from this device was captured by an analog to digital converter and stored by a simple computer. The filter columns were clamped to the filter stand at the base of the bottom section as shown in the figure. The third of the cross clamps connecting the two filter columns is also shown in figure 4.7. The bottom sections were fitted with thermal insulation to minimise temperature fluctuations and reduce algal growth.

4.3.4 Pressure Sampling Ports & Piezometers

The pressure ports were connected to the piezometers by identical lengths of 5 mm internal diameter flexible tubing. The piezometers were constructed from 4 mm internal diameter rigid plastic tubing three metres long. These tubes were fitted vertically to a piezometer board backed with millimetre scale graph paper to measure the free surface levels in the tubes. Each tube was fitted with a cut-off valve to isolate the piezometers during filter backwashing and to aid in cleaning. The design of the pressure ports is illustrated in figure 4.8. These ports had a number of design requirements. Firstly, the ports had to allow water to enter into the piezometer without allowing sand to penetrate into the tube. Secondly, the ports had to be designed in such a manner to minimise the opportunity for localised scouring of deposits in the filter bed near the entrance to the port. Finally, the port design should measure the water pressure within the filter column away from the filter wall to minimise the effects of the region of higher porosity at the wall. This wall effect extends some 4 or 5 sand grain diameters into the filter bed (Leclerc, 1975). To avoid this effect pressure should be measured beyond this point in the bed. The design adopted was similar to that used by Coad, (1982). Two mm diameter holes were drilled through brass bolts. 2 mm diameter brass tubing was inserted through these holes and soldered in place. 0.5 mm width slots were milled in the underside of the brass tubing half way through the tubing as shown to allow water but not sand to enter the port. The slots were positioned on the underside of the port to reduce deposition within the slots. The end to the tubing was sealed with solder. Perspex cylinders were glued to the filter columns at the positions described in table 4.1 and were drilled and tapped to accommodate the prepared brass ports. The brass ports were screwed into the filter columns with PTFE tape to provide water tight seals. The ports projected 15 mm into the filter bed with the first 0.5 mm slot positioned 5 mm from the filter wall. In this manner pressure readings were taken from within the bed away from the filter wall. To minimise scouring of deposits around the port entrances, the velocity of the flow of water into the pressure port should be less than or equal to the interstitial velocity of the flow in the filter pore spaces. It was intended to operate the experimental filters at an approach velocity of 8.0 metres per hour. The filter material was sand with a porosity of 0.4 (see section 4.3.6). From this the clean bed interstitial velocity can be estimated from the following relationship (Ives, 1975a):

$$u_i = u_a/\varepsilon$$

where u_i is the interstitial velocity, u_a is the approach velocity and ε is the media porosity. The interstitial velocity under these conditions is estimated to be 5.6 millimetres per second. The rate of head loss development across the filter beds was expected to be 2 to 3 mm per minute with the intended operating conditions. At this rate of development the rate of flow of suspension into the 4 mm internal diameter piezometer tubes can be calculated.

Area of Piezometer Tube	: 1.26×10^{-5} m^2
Rate of Flow in Tube	: 3 mm/min
Flow into Tube	: 2.3 ml/hour

From this flow rate into the pressure ports the velocity at the entrance to the 0.5 mm slots in the brass ports can be estimated.

Slot Width	: 0.5 mm
Slot Length	: 3.14 mm
Slot Area	: 1.57 mm^2
Number of Slots	: 5
Velocity into Slot	: 0.08 mm/s

The velocity of the flow into the slots of the pressure ports is thus less than 2 % of the interstitial velocity which would prevent any localised scour around the ports. The pressure ports were positioned at depths of 15, 30, 50, 100, 150 and 600 mm below the sand surface. These ports were positioned closer together nearer the sand surface since the bulk of the solids removal was expected in this region.

4.3.5 Suspension Sampling Ports & Collection Stand

Sampling the suspension from within the filter bed required some method of withdrawing a representative sample of fluid without disturbing the bed deposits or significantly altering the flow rate within the filter. Ideally, the point of withdrawal should be away from the filter walls to avoid the effects of the proximity of the wall on the medium porosity. Secondly, the rate of sampling should be isokinetic to avoid scouring the deposits near the entrance to the sampling port. That is to say the velocity of the fluid entering the body of the sample port should be similar to the interstitial velocity within the filter pore spaces. In addition, maintaining the velocity of the flow inside the port similar to the interstitial velocity within the filter pores would reduce the likelihood of break-up of flocculated particles in the suspension. Thirdly, the dimensions of the sample port should prevent the ingress of the filter media. Finally, the rate of sampling should amount to only a small fraction of the total flow to minimise the effect of sampling on the flow rate within the filter but be sufficient to provide a useful sample volume in a reasonable time interval. The design of the suspension sampling ports used is illustrated in figure 4.9. 10 mm diameter re-enforcing rods of Perspex were glued to the filter columns in the positions shown in table 4.1 and figure 4.6. One mm diameter holes were drilled through these rods into the filter bed. Stainless steel 0.6 mm internal diameter needles were inserted into the filter beds through septa glued in place on the ends of the re-enforcing rods to form a water tight seal. This narrow gauge was necessary to prevent the filter media entering the sample port. The steel needles extended 15 mm beyond the filter column wall into the filter media to avoid the effect of the walls. Identical lengths of rubber tubing were attached to the ends of the steel needles to carry the fluid sample to the collection measuring cylinders. The tubing was arranged such that the sample flowed into the measuring cylinder at a shallow angle. The measuring cylinders were held at a

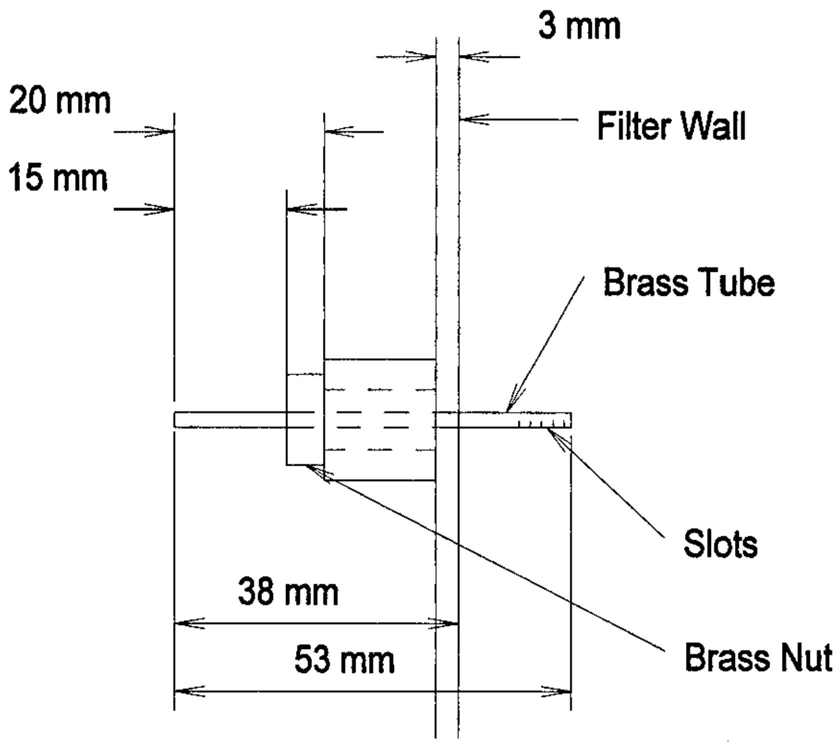


Figure 4.8: Pressure Port

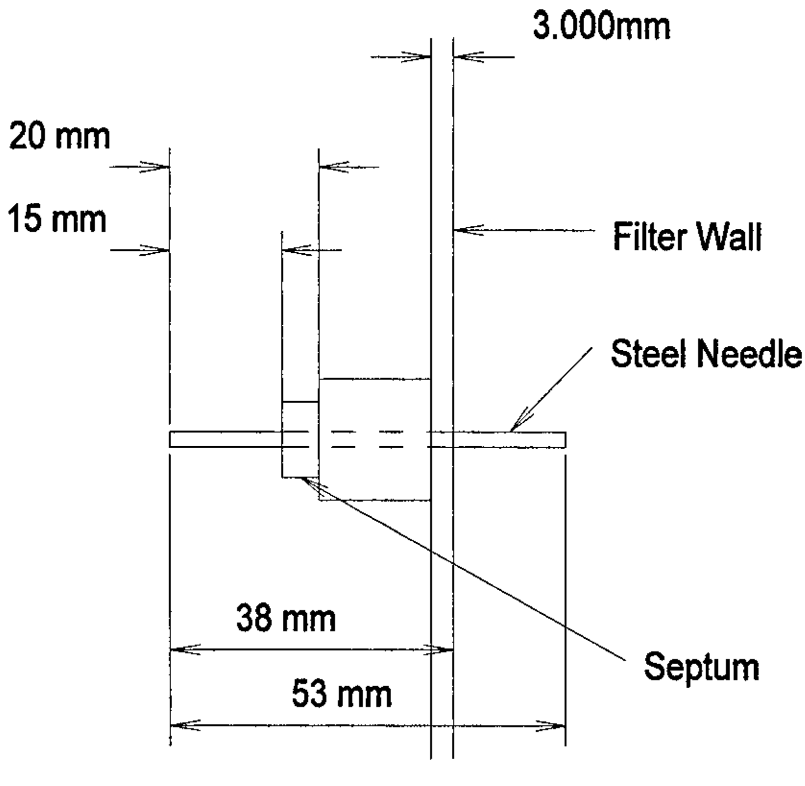


Figure 4.9: Suspension Port

shallow angle by a custom built sample stand (shown in Plate 4.3). The flow through the ports was regulated by screw clamps fitted to the rubber tubing. To achieve isokinetic sampling the flow rates through these sample ports would need to be very low. The interstitial velocity within the filter beds was calculated to be 5.6 mm/s under clean conditions. To limit the velocity to this value within the steel needles would require a flow rate of only 5.7 millilitres per hour. This is too low to provide a useful sample volume in a reasonable time interval. Secondly, it was thought that at this low flow sedimentation of suspended particles within the needles and tubing would very quickly block the ports. A sample volume of 50 to 100 millilitres was required for turbidity and particle analysis. It was decided to compromise on the isokinetic principle and collect a maximum sample volume of 100 millilitres in 30 minutes. This gave a velocity of 196 mm/s inside the steel needles. Other researchers have recognised that isokinetic sampling is an ideal impossible to achieve in filtration research and that a compromise is necessary (Darby & Lawler, 1990; Lawler, 1996). The work of Moran *et al,* discussed in section 2.4, used 0.69 mm internal diameter stainless steel needles inserted into 0.78 mm size media in laboratory filter columns for suspension sampling (Moran *et al,* 1993). Sample collection rates used by Moran *et al* were 150 millilitres in 15 to 20 minutes which gave a velocity of 446 mm/s inside the sample needles. The authors accepted this compromise since it was found that the sample was not distorted by the elevated velocity in the needle. This was due to the use of a non-flocculated suspension in their study. Since a non-flocculated suspension was used for this laboratory study it was felt that the issue of floc break-up in the ports could be ignored (Lawler, 1996). In addition, since in this study the surging filter performance was compared with a control filter sampled in an identical manner it was felt that the compromise was acceptable in this case.

Lawler (1996) recommended that the maximum amount of suspension removed from experimental filters by sampling within the filter bed should not exceed 5 % of the total flow through the column. Sampling at a rate of 100 millilitres in 30 minutes from each of the four ports within the filter beds would amount to less than 4 % of the total flow through the filters at an approach velocity of 8.0 metres per hour.

4.3.6 Filter Bed Specifications

To support and prevent the filter media entering the filtrate piping, the bottom sections of the filter columns were packed with layers of graded gravel. The size and depth of each gravel support layer were similar to that recommended by Kawamura (1991) and are given below.

75 mm of 2.0 to 3.35 mm gravel over
75 mm of 3.35 to 6.3 mm gravel over
75 mm of 6.3 to 10 mm gravel over
75 mm of 10 to 20 mm gravel over
120 mm of 20 to 40 mm gravel

During the development of the backwash system, a degree of intermixing of the sand and top gravel layers was evident. This led to problems in achieving similar bed settlement in both filter columns after washing. Similar bed settlement after backwash was found to be essential to achieve reproducible performance in both experimental filters. To eliminate this intermixing, a piece of stainless steel mesh was inserted between the top gravel layer and the sand. Improvements in control over the bed settlement after backwash were found which aided the achievement of reproducibility (design of the backwash system will be discussed in section 4.4.2).

For simplicity it was decided to use a mono-medium filter bed of Leighton Buzzard sand with a specific gravity of 2.65 (supplied by Garside Industrial Sands) typical for conventional rapid filtration. The manufacturer's given porosity was 0.41. The actual porosity of the filter beds after backwashing and compaction was calculated to be 0.40. For rapid filtration a sand depth of 600 to 750 mm is recommended for mono-medium filters (Kawamura, 1991). A bed depth of 600 mm was chosen for these laboratory filters to reflect full scale filter design. To achieve similar results in both filters, it was essential that the filter beds were identical in terms of mass and size distribution. To acquire identical bodies of sand, the filter beds were derived from a single fifty kilogram bag of sand. The sand was initially washed and dried to remove dust and fine particles. The prepared fifty kilogram mass was then thoroughly mixed and divided in half using a riffle separator. Half was discarded and the remainder was divided again. This process of division and disposal was continued until the sand was reduced to a mass of 5.43 kilograms. This body of sand was split into equal halves using the riffle box producing two identical masses of 2.715 kilograms. To confirm that these bodies of sand were identical in size distribution, each was analysed using a standard set of brass sieves (British Standard 410) from 0.5 to 1.18 mm aperture. The brass sieves were weighed before the samples were sieved on a mechanical shaker for a period of 20 minutes to separate the sizes. The sieves were then weighed again to determine the mass of each separated size. The sieve analysis results are given in table 4.2 and illustrated in figure 4.10.

Standard Sieve Size	*Weight Retained (g) Sample One*	*Weight Retained (g) Sample Two*
0.5	109.7	109.7
0.6	846.9	846.9
0.71	1241.4	1241.4
0.85	480.2	480.2
1.0	36.8	36.8

Table 4.2: Sieve Analysis of Filter Sand

From figure 4.10, it was clear that the prepared sand bodies were identical in mass and size distribution. The sieve analysis results gave an effective size of 0.62 mm and a uniformity coefficient of 1.26. Stevenson (1994) in a review of filter media specification pointed out that the effective size and uniformity coefficient were introduced to describe slow sand filtration media and that they have no sound basis in rapid filtration. He recommended the abandonment of these parameters and suggested their replacement with the hydraulic size. The hydraulic size is calculated from sieve analysis and therefore incorporates the size distribution of the media. The hydraulic size of the sand used in this study was calculated to be 0.75 mm.

An important consideration in laboratory and pilot plant filtration studies is the effect of the filter walls on the filter media. It has been noted in section 4.3.4 that the filter column wall causes a region of higher porosity extending to some 4 or 5 grain diameters into the bed (Leclerc, 1975). This region of higher porosity presents less resistance to the fluid flow and can thus distort filter performance by "channelling" a portion of the flow down the filter walls. This could lead to a reduction in the rate of

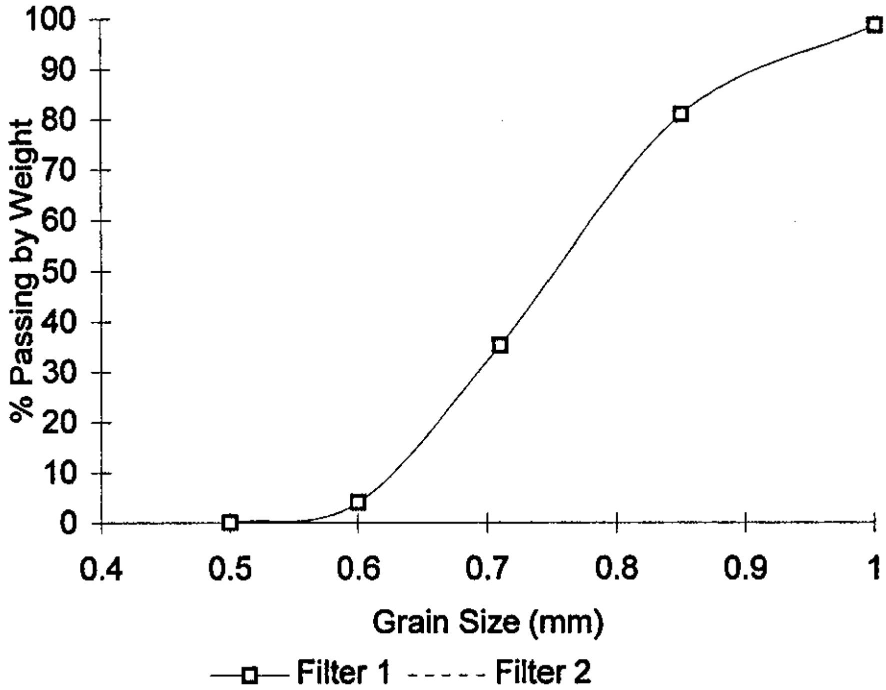


Figure 4.10: Sieve analysis of filter sand

head loss development and an increase in the amount of solids passing through the filter. This "wall effect" can be rendered negligible provided the filter column diameter is sufficiently greater than the filter media grain diameter. This is known as the aspect ratio. Chellam and Wiesner, (1993), reported that "rules of thumb" in filter pilot plant design based on the work of Rose, (1950), typically required an aspect ratio of greater than 50 times to minimise the effect of the filter walls. A study conducted by Lang *et al* (1993) compared the performance of filter columns with aspect ratios ranging from 26 to 6000. The authors did find that filter performance became more variable at low aspect ratios. The authors recommended an aspect ratio greater than 50 for filtration studies. In this study the filter columns have an internal diameter of 58 mm. The sand media used has a maximum size of 1 mm. This results in a minimum aspect ratio of 58. As such, wall effects should have a negligible impact on the filter performance.

The prepared sand samples were added to the filter columns. Sample one was added to filter column one and sample two was added to filter column two. After several backwashes it was noted that the surface of the sand beds contained a thin layer of very fine dark particles. This small quantity of fine material was believed to be present in the sand to begin with and was not completely removed by washing and sieving. It was thought that these fine particles of less than 200 to 500 micron in size would block the surface pores. This would result in surface removal instead of true depth filtration. Ives (1975b) noted the effect of surface removal on head loss development. The development of a surface mat would result in exponential head loss development. Experiments did indeed produce a surface mat and exponential head loss growth in the laboratory filters with the fine particles present on the surface. Ives and Pienvichitr (1965) noted surface mat development in laboratory rapid filters using a similar suspension of PVC particles to this study. Hudson (1981) stated that whenever filter media was placed in a bed it should be backwashed several times and the surface inspected for the presence of fine particles. He also noted that these fine particles may result in a sharp rise in the rate of head loss development. He recommended that these fine particles should be removed by scraping off the top layers. As such, after backwashing the filters several times, the filter columns were opened and identical small masses of sand were removed from the top of each bed including the fine dark particles. The amounts removed were less than 2 % of the total bed mass and would not have altered the bed size distribution. Further experiments confirmed depth penetration of the solids with no further surface mat development.

4.4 Filtered Water Discharge & Filter Backwashing

The flow of filtered water passes out of the filters from ports fitted to the bottom sections of the columns as described in section 4.3.3. The filtered water is discharged from the filtrate piping from the outlet ports shown in Plate 4.5. A potential problem in rapid filtration is the formation of negative head (AWWA, 1990). These outlet ports were positioned at an elevation 100 mm above the filter bed surfaces to prevent negative head formation within the filter media.

4.4.1 Surge Machine

To pursue the objectives stated at the end of chapter two some means of producing surging in the laboratory filters was required. A method of generating flow rate fluctuations in a manner similar to those described by Baylis in section 2.1.2 and those observed at local water treatment plants described in chapter three was needed. Random and erratic flow rate fluctuations could perhaps be produced by the inclusion of some flow constriction to generate turbulence in the filtrate piping. However, this would generate fluctuations in an uncontrolled manner which would create difficulty in quantification of the effects on the filter performance. It was decided to manufacture a machine to generate artificial flow fluctuations in a controlled manner. To determine the effect of different surge amplitudes and rates of occurrence, the developed machine would require some means of controlling the characteristics of the surges generated. A machine designed to generate surges at a range of amplitudes and rates of occurrence was developed and is shown in Plate 4.6. This machine was attached to the filtrate piping of filter two. A rubber pump was filled with water and connected to the filtrate piping of filter two. The pump was mounted on a stand and fitted with a threaded piston. A motor driven lever was used to compress and inflate the rubber pump by acting on the piston. The threaded piston fitted through a slot cut in the lever and could be locked in place. The lever was clamped at one end and attached to a return spring suspended at the other. The lever/pump arrangement was driven by a variable speed electric motor. The drive shaft of the motor was fitted with an offset cam used to operate the lever. Different sizes and shapes of cam could be fitted to the drive shaft to vary the surge characteristics. By sliding the pump and piston back and forth along the lever the surge amplitude could be modified as required without stopping the motor. By varying the motor speed, the surge frequency of occurrence could be manipulated. Different combinations of pump position and motor speed could be set to produce a range of surging characteristics. This machine was used to apply surges to one of the filters for comparison with the performance of the non-surging filter. The characteristics and effects of the surging applied are discussed in chapter six.

4.4.2 Backwash System

After each experimental test run the filters needed washing to remove the captured solids from the filter media. This was achieved by reversing the flow in the filter columns to fluidise and agitate the filter beds. The disturbed solids were washed out of the filter beds and carried away in the supernatant waste wash water via an overflow at the top of the columns described in section 4.3.1. This backwashing procedure had several important requirements in this laboratory study. Firstly, the backwash system needed to be able to fluidise and expand the filter sand in both columns in a similar manner. To achieve reproducible filter performance it was necessary to clean the filter sand in both columns to the same extent repeatedly such that the filters would be returned to the same initial clean state. Secondly, the filter beds needed to be settled down after backwashing to identical levels in each column and identical levels in each run. It was found by experiment that the performance of the two filters was dissimilar when the bed levels were not identical in each column. This would be expected since if the bed levels were not the same then the sand packing must be dissimilar. It was also found that filter performance from run to run exhibited poor reproducibility when the filter beds were not settled to the same level for each run. Finally, insufficient settlement of the filter beds after backwashing resulted in significant bed movement during the filter run as the hydraulic gradient increased. Bed movement during the filter run was undesirable since it disturbed the captured deposits and altered the filter's development.

The final version of the developed backwash system which met these needs is shown in Plate 4.5. A 250 litre storage tank supplied with mains tap water was elevated 10 metres above the filter columns to provide sufficient head to expand the filter beds by 20 % during backwash. A combined backwash flow rate of 3.6 litres/minute split evenly between the two columns was required to produce this expansion.

Difficulty was encountered in achieving reproducible bed expansion and settlement after backwash from run to run due to fluctuations in the temperature of the backwash water dependent on the time of day, weather conditions and season. Changes in the water viscosity with temperature resulted in different degrees of bed expansion and settlement at any given backwash flow rate. As such, the storage tank was fitted with a one kilowatt heater to raise the water temperature to a steady 20 °C during the winter months. During the summer months, fresh tap water from the mains was used to reduce the water temperature to 20°C as necessary. The temperature controlled backwash water was delivered to the filter columns by hose pipe which could be attached to the outlet ports shown in Plate 4.5 as required. The backwash flow rate was governed by a needle control valve fitted to the hose pipe. The flow rate was measured using a 0 to 8 litres/minute rotameter fitted after the control valve. The backwash flow was split between the filter columns by a "Y" piece fitting and two shutoff valves.

Problems were still encountered in achieving reproducible settlement of the filter beds after washing. It was found that the magnitude and reproducibility of bed settlement after backwashing was dependent on the manner in which the backwash flow was cut off. Poor settlement reproducibility after backwash was found to produce dissimilar head loss and removal efficiency performance in the filter columns during trial test runs. This dissimilar performance would mask the effects of surging on the filter performance. Further development of the backwash system was required to improve this performance The results of these developments are summarised in table 4.3 below.

Flow Cut-off	*Vibrations*	*Difference in Bed Levels*	*Movement During Run*	*Comments*
Instantaneous	None	40-60 mm	20-30 mm	Poor reproducibility & stability
Manual Slow	None	20-40 mm	20-30 mm	Poor reproducibility & stability
Manual Slow	Manual	10-20 mm	1-3 mm	Poor reproducibility & Improved stability
Standard Slow	Shaker	5-10 mm	1 mm	Better reproducibility & Improved stability
Standard Slow	Shaker with Cross Clamps	< 2 mm	0-1 mm	Improved reproducibility & stability

Table 4.3: Backwash & Settlement Performance

Instantaneous cut-off of the backwash flow from 3.6 to zero litres per minute caused the filter beds to collapse under their own weight. This method of backwash cut-off produced widely different sand bed levels each time and was impossible to control. Slowly closing the backwash needle valve produced better performance but it still proved difficult to achieve similar bed levels in each column. In addition, the filter beds were found to shift unevenly by several centimetres during trial runs as the hydraulic gradient increased. Difficulty in manually controlling the rate of cut-off contributed to the poor reproducibility. This was overcome by adopting a standard rate of cut-off. The backwash flow was reduced from 3.6 to zero litres per minute in steps of 0.4 litres per minute every 30 seconds timed with a stopwatch. Manually tapping the filter columns after slow backwash cut-off settled the beds further and reduced movement during the filter run. However, this method too failed to produce similar and reproducible bed levels and filter performance in each of the columns. This was probably the result of dissimilar packing in each of the filter beds from differences in the manual tapping applied. It was decided to fit a mechanical shaker to the filter stand to settle the filter beds during backwash cut-off. It was hoped that this device would produce similar vibrations in each filter column during slow cut-off of the backwash flow. This method would hopefully settle the filter beds to similar levels in each column reproducibly each time and produce similar filter performance. The mechanical shaker consisted of a small electric motor fitted with a flywheel. The motor speed could be varied by a control box. The shaker can be seen fitted to the rear of the filter stand in Plate 4.7. The flywheel was drilled and tapped at a range of radii. A small steel weight could be fastened to the flywheel at any of these points to imbalance the flywheel. By shifting the location of the steel weight and altering the motor speed a range of vibration amplitudes and frequencies could be produced in the filter columns. The optimum speed and amplitude was determined by trial and error until the desired magnitude of bed settlement was achieved. From the table, it can be seen that the combination of the standard slow backwash cut-off and mechanical shaker improved the settlement performance such that there was little or no bed movement during trial runs. However, the difference in the sand levels in each column still varied around 5 to 10 mm each time. This difference still resulted in dissimilar and poorly reproducible performance from the filters. It was noticed that the vibrations generated in each column by the electric shaker attached to the filter stand were not identical along the lengths of the filter columns. It was felt that these differences created the remaining variation in the sand levels after settling. Observations by touch and with point displacement gauges were used to assess the amplitude of the vibrations in each column. These differences in the amplitude of the vibrations were eliminated by clamping the filter columns together at three points along their lengths. These cross clamps can be seen in Plate 4.3. From the table, it can be seen that this modification virtually eliminated the differences in the sand levels after settling with the electric shaker. Trial runs comparing the filter performance showed improved similarity and reproducibility. Control test runs were conducted periodically during the investigation to ensure continuing good performance of the backwash procedure. These results will be discussed further in chapter six.

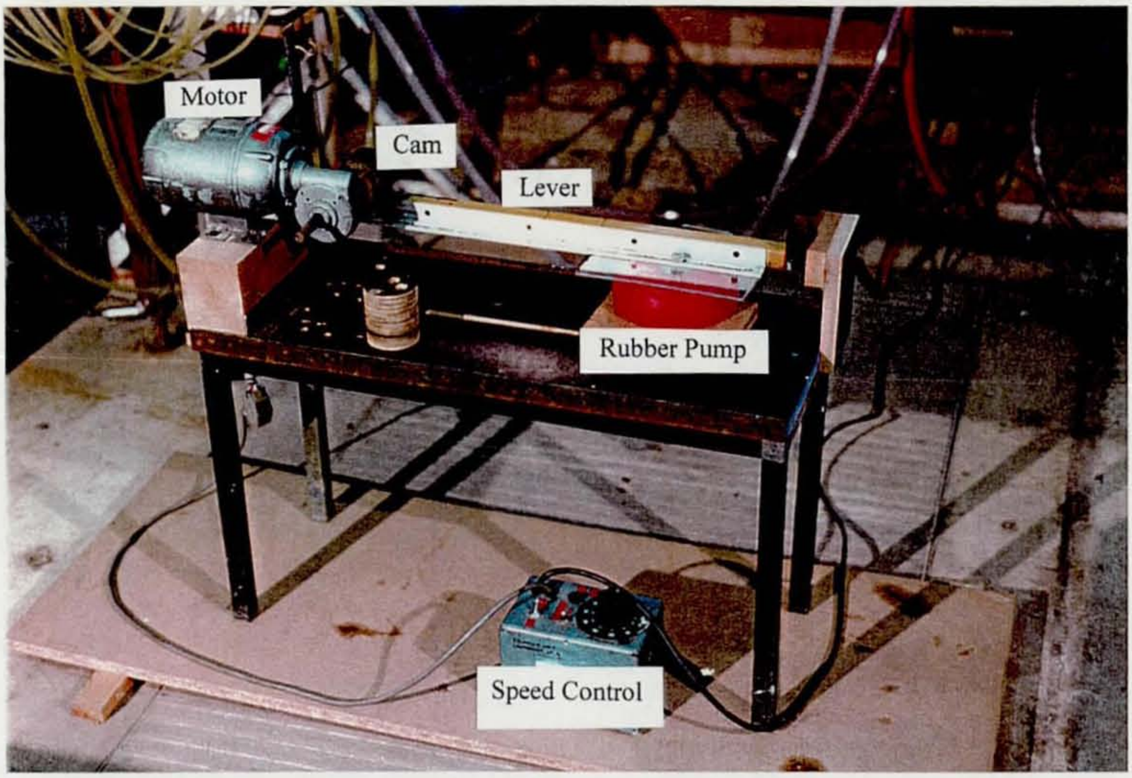


Plate 4.6: Surge machine

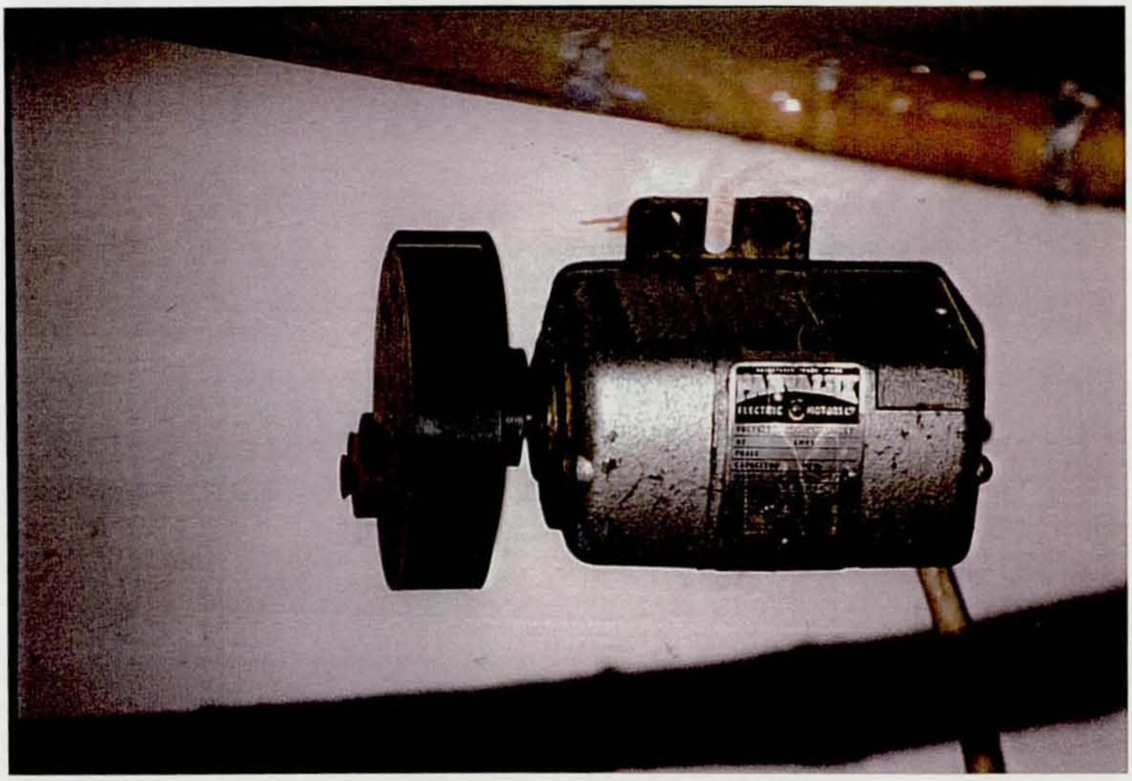


Plate 4.7: Filter shaker

4.5 Chapter Summary

Key points developed to generate reproducible results from the test apparatus were storage of the test suspension, control of the flow rates and operating temperature, design and sampling of the filter columns and vibration settlement of the filter beds after backwash. The test suspension was stored in a series of stirred temperature controlled tanks totalling 1500 litres volume. It was shown that the test suspension had to be delivered to the filter columns at a controlled flow rate. The best way to do this was by siphons fitted to an elevated constant head tank. The design of the filter stand and the filter columns was reviewed in detail. Two modular filter columns were constructed from Perspex pipe and mounted on an independent stand to eliminate vibrations. The methods of sampling both the pressure and the suspension within the filter columns were important. Pressure was measured by a series of specially designed ports fitted at various depths and connected to water piezometers. The suspension was sampled by a series of specially designed needle ports fitted at various depths and collected in measuring cylinders. The preparation and specification of the filtering media also affected performance. Beds of Leighton Buzzard filter sand were supported on layers of graded gravel in each filter column. A surging machine was developed to produce controlled artificial flow rate fluctuations in one filter. Backwashing and subsequent settlement techniques were developed to achieve similar performance in each filter and reproducible performance from one run to the next. Temperature controlled backwash water was necessary from an elevated tank fitted with control valves and flow measurement. Settlement was achieved by controlled backwash cut-off and a specially designed electric shaker fitted to the filter stand. The experimental procedure developed in parallel with the apparatus and used to measure the impact of the flow surges is detailed in the next chapter.

5.0 EXPERIMENTAL PROCEDURE & METHODS

The design and development of the experimental apparatus used was discussed in the previous chapter. In this chapter, the experimental test procedure and methods of sample analysis used in the operation of the apparatus will be presented. The experimental procedure and methods used in this surging investigation evolved in conjunction with the experimental apparatus over thirty six months. The final test procedure and methods of sample analysis used to gather the results presented in chapter six will be reviewed here.

5.1 Test Suspension Preparations

During trial filter runs, it was found that suspension deposits tended to form on the inner walls of the system piping. Continuous pumping of the test suspension around the constant head tank and recycle loops caused a drop in the solids concentration and turbidity of the test suspension. In subsequent test runs, these deposits often detached from the piping as large particles and flakes which caused blocking of the surface pores of the sand media. This was undesirable since it caused the formation of surface cake instead of true depth filtration. In addition the presence of deposits in the piping at the beginning of a new test run accelerated the rate of deposition and rate of decline of suspension concentration. Changes in the suspension concentration during a test run and from run to run were undesirable interferences which affected the performance reproducibility. As such it was necessary to clean the storage tanks and system piping after each test to prevent these interferences accumulating with each successive filter run. Deposits were occasionally found to form in the neck of the delivery siphons. If allowed to build up these deposits caused a drop in the rate of flow delivered to the filter columns. Since a constant flow rate was desired it was necessary to clean the delivery siphons every test run to prevent deposit accumulation.

Having thoroughly cleaned the system tanks, piping and siphons the storage tanks were filled with fresh clean mains water to the desired levels. Samples were taken from storage tank one for pH, conductivity and dissolved solid concentrations analysis. As will be discussed in chapter six, these parameters influenced filter performance, the methods of analysis are discussed in section 5.4.3.

5.1.1 Flow Rate & Temperature Control

It was noted by a number of researchers that filter head loss and removal were affected by different flow rates (Hudson, 1956; Cleasby & Baumann, 1962). At higher flows, solids tended to penetrate further into the filter media resulting in poorer quality filtrate. In addition, as discussed in section 2.2, flow rate changes during the filter cycle can cause poorer performance by disturbing previously captured solids. As such, for this laboratory investigation of the effect of surging on filter performance it was necessary to develop a method of stable and identical flow delivery to each filter column to eliminate these sources of interference. As described in section 4.2, siphons were used to draw the test suspension from an elevated constant head tank. By setting similar heads across the siphons it was expected to produce similar flow rates from each. The flow rate from each siphon was measured by timing the flow into a two litre graduated flask. However, consecutive flow measurements from each siphon showed poor reproducibility. It was noted that the water level in the constant head tank fluctuated by several millimetres rapidly. This fluctuation in head across the siphons was thought to be causing some of the poor flow results. A larger volume tank was fitted to reduce these fluctuations. In addition, it was found to be necessary to run the siphons for a period of time to generate reproducible flow readings. The siphon levels could then be adjusted to produce similar flows from each. In practise, the most convenient approach was to run the siphons overnight to allow both the flow rates and the water temperature to stabilise. In this manner, similar and stable flows were delivered to each filter column during the test runs. Figure 5.1 illustrates the performance of the siphons achieved expressed as filter approach velocity.

Another factor which can influence filter performance is the suspension temperature. Ives and Fitzpatrick, (1989), presented the following validated equation describing the shear stresses generated in the filter pore spaces by fluid flow.

$$\tau(\max) = \frac{8\mu Q}{A\varepsilon d}$$

where $\tau(\max)$ is the maximum shear stress within the filter pore space, μ is the dynamic viscosity, Q is the fluid flow rate, A is the filter surface area, ε is the media porosity and d is the pore "diameter". From this equation, it is apparent that the shear stresses within the pore spaces of a filter bed are influenced by the temperature of the suspension. Higher temperatures will produce lower shear stresses within the pores as a result of lowered fluid viscosity. Shear stresses play an important role in filter performance. The attachment of suspended particles to the surface of the sand grains will be influenced by the shear stress acting on them. Previously captured particles may be detached as a result of changes in the shear stresses within the filter pores. The electrochemical characteristics of suspended particles are also influenced by suspension temperature. Gregory (1975), described the energy barrier which exists between charged particles in suspension and how higher temperatures result in greater proportions of attachment from particle collisions. Ives (1975a), described the transport mechanisms in rapid filtration and his discussion notes that the key transport mechanisms of sedimentation and diffusion will be influenced by changes in the fluid viscosity. The settlement of a suspended particle within the filter pores can be described using Stoke's Law where the particle settling velocity is inversely proportional to the fluid viscosity. Diffusion transportation of sub-micron particles is governed by the Brownian motion of the water molecules. Higher suspension temperatures cause greater diffusion transport because of the greater thermal energy of the water molecules. It was clear therefore that the suspension temperature had to be controlled to achieve reproducible filter performance. Storage tank one was fitted with two 1kW immersion heaters as described previously in section 4.2. Initially, it was intended to conduct test runs at a range of operating temperatures to reflect the changing conditions at full scale plant. Figure 5.2 presents seasonal water temperature data collected at Cropston water works described earlier in section 3.1. Water temperatures entering the plant can be as low as 5°C over the winter months and reach 22°C in the summer. However, to operate at these lower temperatures would have required complex cooling equipment which was unavailable. It was decided to operate the laboratory filters at a fixed temperature of 20°C to reflect summer time conditions. However, it proved impossible to keep the temperature down due to high air temperatures in the laboratory and heat generated from the centrifugal pump. Trial runs were conducted at 25°C instead. However, problems were still encountered due to ambient temperatures in the laboratory in summer. The laboratory air temperature sometimes rose to 28°C depending on the weather conditions. As such, it was necessary to operate the filters at a suspension temperature of 30°C in order to

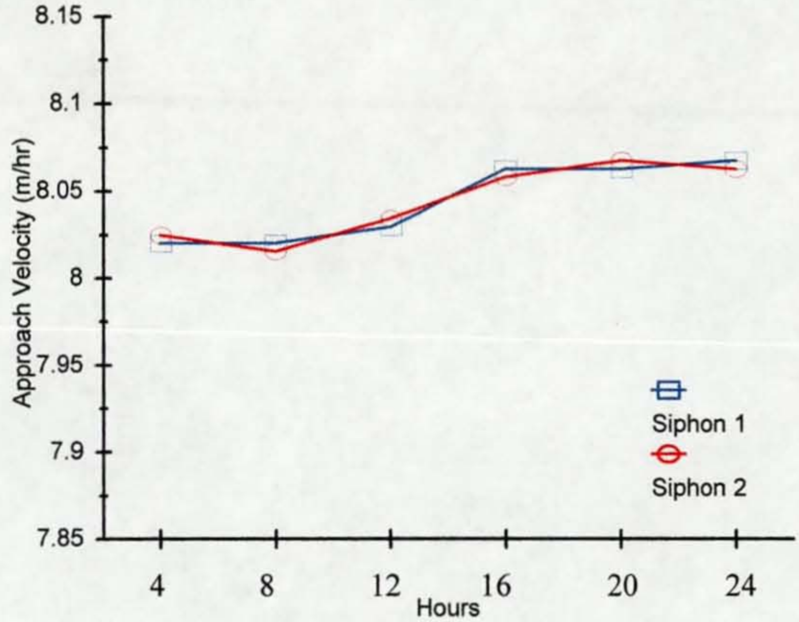


Figure 5.1: Flow control performance

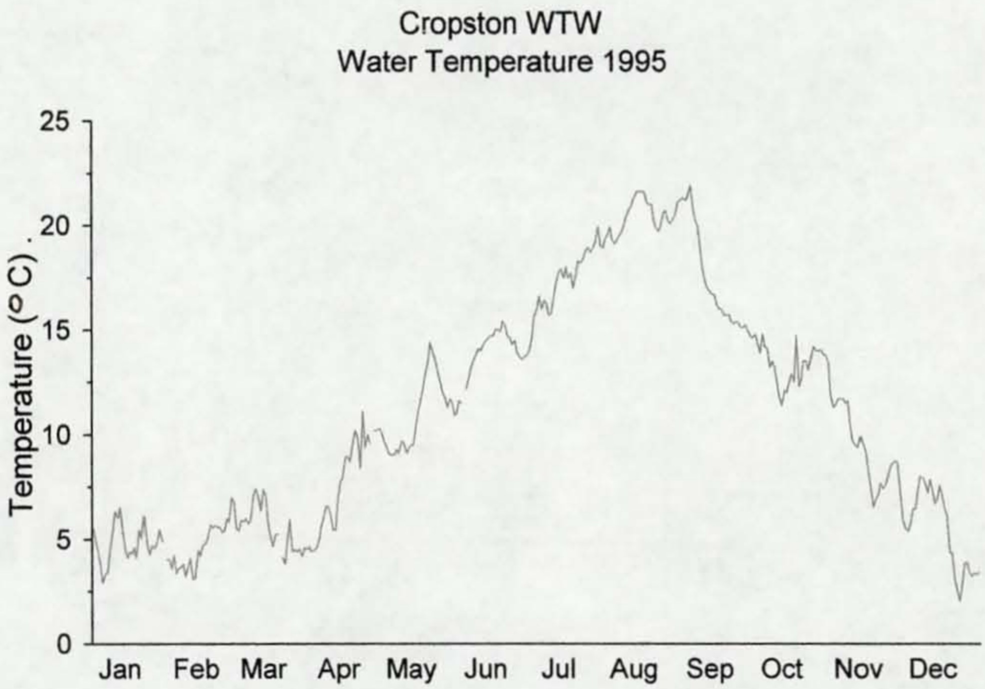


Figure 5.2: Cropston water works water temperature

achieve a constant and reproducible operating temperature in each test run during the summer months. As described earlier, at 30°C shear stresses in the filter pores will be reduced and transport and attachment mechanisms enhanced compared to lower temperature performance. This is likely to reduce the effects of any surges applied since the resultant fluctuations in shear stress will be reduced by the lowered suspension viscosity. This will be discussed in more detail in chapter six. It was hoped to conduct lower temperature tests during the winter months. The suspension temperature was monitored in each filter during each test to ensure good control. The temperature controller was found to be able to maintain 30°C ± 0.5°C.

5.1.2 PVC Concentrate Preparation

A turbid suspension of particles was required to operate the laboratory filters. Certain suspension characteristics were desirable in this study of the influence of surges on performance. Firstly, the generated artificial suspension should have a similar size range to that found in the natural waters applied to rapid filters. Flocculated settled natural waters have a size range from 0.01 to 10 or 15 microns (Amirtharajah, 1988). Secondly, the material should be easily handled and dispersed in water for simplicity of use. Finally, the material selected should be able to generate a suspension of known size and concentration reproducibly that can be stored for the duration of the test. From the discussion in section 2.4, variations in the size, size distribution and concentration of the suspension during the test cycle and from test to test are undesirable since these factors influence filter performance. Uncontrolled, these factors would mask any effects of the flow rate fluctuations on the filter performance.

It was decided to use a concentrated suspension of polyvinyl chloride (PVC) particles to produce a turbid test suspension. PVC powders have been used by other researchers for fundamental filtration studies (Ives & Pienvichitr, 1965; Coad, 1982; Mackie *et al*, 1987; Mackie & Bai, 1992). Their attraction is that the PVC powder is easily handled and dispersed in water and can be used to create stable test suspensions of known concentration and particle size distribution reproducibly. The PVC powder used was Evipol MP7057 supplied by European Vinyls Corporation. Under a microscope the PVC particles comprised discrete particles and agglomerates of 3 or 4 particles with sizes ranging from sub-micron sizes to 40 or 50 microns or more. It was necessary to reduce the size range of the PVC powder to reflect natural waters. 100 gram batches of PVC powder were added to one litre volumes of tap water. The suspension was dispersed in a high shear blender to thoroughly wet and mix the particles. The mixed suspension was placed in a beaker and allowed to settle for a known time interval. The supernatant suspension was carefully siphoned off and the size distribution analysed using a Coulter laser sizer (Model LS 130). By experiment, it was found that settling the mixed suspension for a total of eight hours produced a concentrated suspension of particles with a size range from sub-micron sizes to 15 microns. The volumetric dose of this concentrate required to produce a suspension of 50 nephelometric turbidity units (NTU) in the storage tanks each test was also experimentally determined. A turbidity of 50 NTU was arrived at by further experiment to produce 1 to 2 metres of head loss development in the laboratory filters in 15 to 20 hours of operation (see section 6.1). The storage tank mixers were operated at 20 rpm to prevent suspension settlement. Samples of the prepared suspension were taken from tank one shortly after dosing and 24 hours later for particle size distribution analysis by the Coulter laser sizer. The size distributions are illustrated in figure 5.3. From this chart, it can be seen that the prepared test suspension had a size distribution from 0.1 to 15 micron as desired and remained stable over 24 hours in the stirred storage tanks without generation of large particle agglomerates by flocculation.

Batches of PVC powder were mixed, settled and siphoned in the manner described to produce sufficient concentrate volume to dose the clean tap water in the storage tanks to the required concentration for each test run. The prepared concentrate was held in suspension on a magnetic mixer until required. In this manner, a stable suspension of particles of known turbidity and size distribution was prepared for each experimental run to investigate the effects of surging on filter performance.

5.2 Filter Column Preparations

After each test run, the filter columns were backwashed at 20°C for 5 minutes to an expansion of 20 % to remove the accumulated PVC deposits. However, during trial

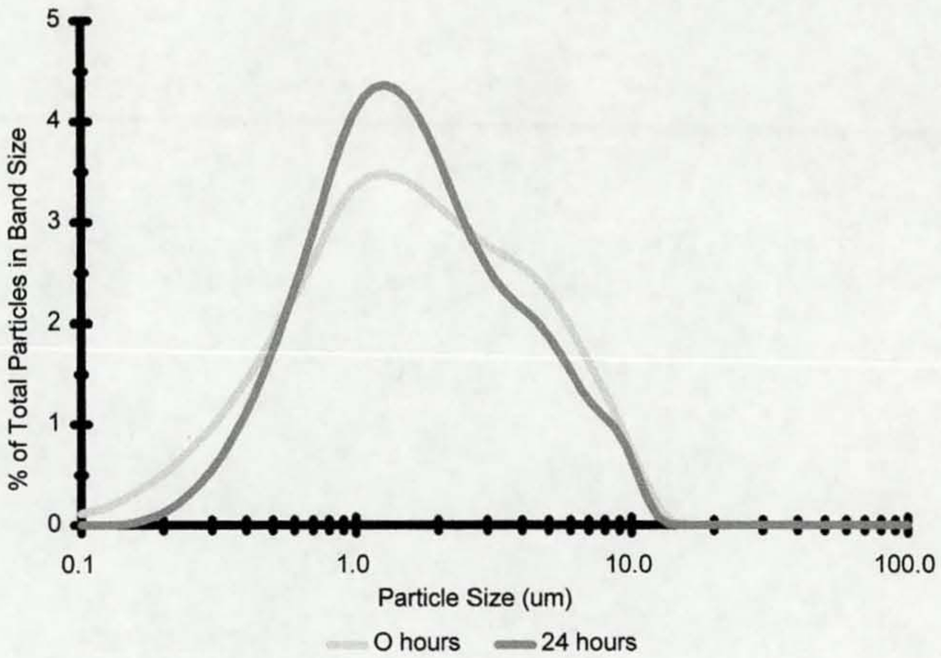


Figure 5.3: PVC suspension particle size distribution

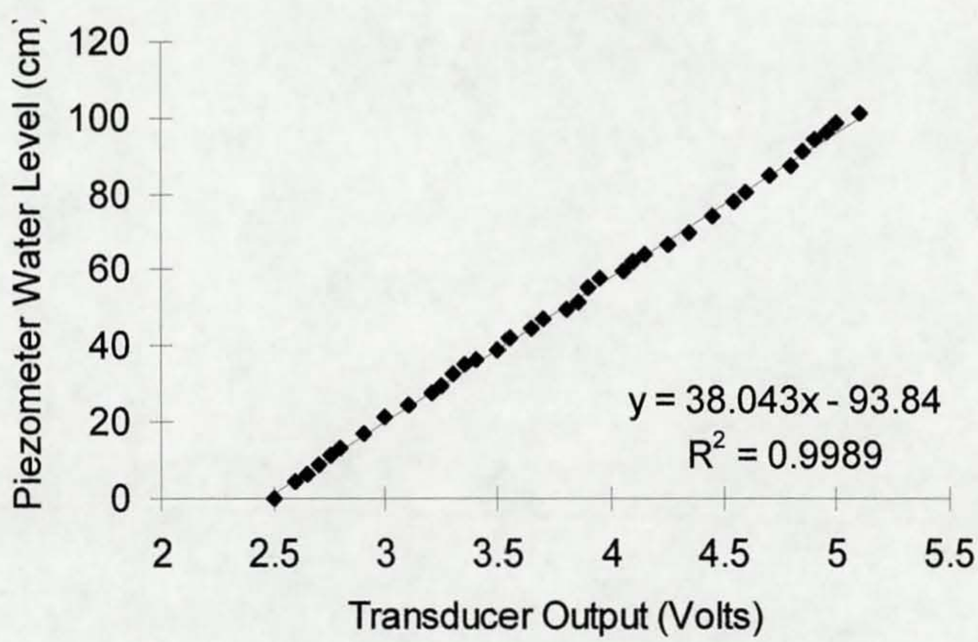


Figure 5.4: Pressure transducer calibration curve

runs it was noticed that after backwashing some PVC deposits remained on the surface of the sand beds. These deposits were agglomerates of PVC several millimetres in size and were not carried away by the backwash flow. Additional sand cleaning was necessary to break up and remove the remaining solids to prevent blocking of the surface pores in subsequent test runs. The filter sand was siphoned out of each column and separately stirred to break up the large particles of PVC deposit. The sand was then returned to its respective column. The filters were backwashed for a further 5 minutes to remove the remaining particles of PVC. The columns were then settled using the backwash and settlement procedure described in section 4.4.2 to identical levels ready for use. During trial runs, the pressure and suspension sampling ports occasionally blocked with PVC. It was necessary to clean the pressure and suspension ports after each test run while the sand was removed to prevent this. The pressure and suspension sample ports were flushed with air and water to remove any deposits. Little port blocking occurred during subsequent tests. The pressure ports and suspension ports were primed and run with clear water for a few minutes to remove any trapped air. To ensure bed stability and induce any further settlement before the test run commenced, the siphon flow was switched to the filter columns. The filters were operated at 8.0 metres per hour at 30°C using the clean water from the storage tanks for a period of one hour. This usually resulted in further bed settlement of 1 millimetre. In addition, the clean bed head losses were recorded at the end of this hour for comparison. The siphon flow was switched to bypass the filters and the filters were allowed to drain down to the discharge port levels 100mm above the sand surface, ready for the test run to commence.

5.3 Test Run Procedure

Samples of the clean water were taken from the storage tanks for pH, conductivity and dissolved solids analysis. The storage tanks were then dosed with the prepared PVC concentrate to a turbidity of 50 NTU in each tank. The suspension was allowed to thoroughly mix before a sample was taken for Zeta potential measurement. The flow rates from the siphons were measured and checked before commencing the test run. The siphon flow was switched to the filter columns and the filters allowed to come up to full flow. The filter surfaces were checked to ensure no disturbances occurred during start up. The suspension sample ports were opened and adjusted to the required flow rates and the surge machine was started and adjusted to the desired frequency and amplitude. Pressure measurements were then taken from the water piezometers every 90 minutes. Samples were collected from the suspension ports every 90 minutes for turbidity measurement. Samples were also collected from the final discharge ports for turbidity measurement. Samples from the final and suspension ports for particle size distribution analysis were collected independently at the desired time. The collection times reported for the suspension samples were from the middle of the 30 minute collection period. Water temperature was recorded in each filter column every 90 minutes to monitor for any fluctuations. The filter bed levels were noted every 90 minutes to monitor for bed movement during the test run. The surge amplitude and frequency were recorded from the piezometers and adjusted every 90 minutes. The surge profile was also recorded from the pressure transducer. The filters were operated in this manner for 15 to 20 hours or until 1.5 to 2 metres of head loss developed. The flow rates from the siphons were rechecked at the end of the filter run for any drifting. The filter columns were allowed to drain down to the outlet levels before being backwashed to remove the bulk of the captured PVC deposits. The remaining test suspension in the storage tanks was pumped to waste. The system was then cleaned and prepared for the next test as described in sections 5.1 and 5.2.

5.4 Pressure Measurement & Suspension Sample Analysis

The test suspension preparations, filter column preparations and the test procedure have been described. This section will discuss the detail methods of pressure measurement and describe the analytical methods for the water samples.

5.4.1 Pressure Measurement

Piezometer and pressure transducer readings were taken every 90 minutes during a test run. The water levels in each of the 14 piezometer tubes were recorded by sighting on to the millimetre scale attached to the piezometer board behind the tubes. However, it took one to two minutes to take all 14 readings in this manner. During this time, it was possible that the levels in the tubes may have changed, introducing an error into the recorded levels. As such, on completion of the 14 readings the first reading was retaken to discover how much it had changed since it was first taken. It was found that the change in level in this first piezometer tube was never greater than 1 or 2 millimetres. This was deemed to be an acceptable degree of error in the piezometer measurements.

When a surging test run was conducted, surge amplitudes and frequencies were noted from the surging filter piezometer tubes by recording the maximum and minimum water levels and timing the oscillations with a stopwatch. In this manner, the surge characteristics introduced at the base of the filter column and therefore the resulting surges within the filter bed could be observed. In addition, the surge characteristics introduced at the base of the filter column were recorded by measuring the voltage output from the pressure transducer plumbed into the base of the filter column. Every 90 minutes, the output signal from the transducer was recorded over a 30 second period at a sampling rate of 20 hertz using simple data logging equipment. To convert the voltage output from the transducer to a piezometer water level, a calibration curve was used. The calibration curve was created by slowly filling the filter column with water and recording both the transducer voltage output and the water level in the piezometer connected to the base of the filter column. The calibration curve produced is shown in figure 5.4.

5.4.2 Turbidity Measurement

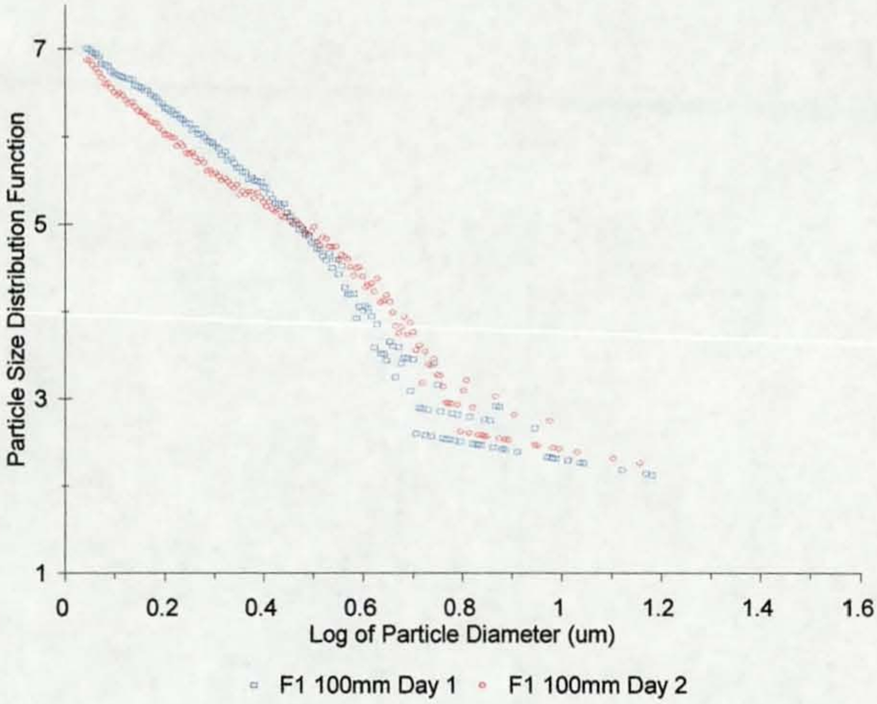
Samples were taken from the suspension sampling ports and the discharge ports every 90 minutes for turbidity analysis. The collected samples were decanted into sample cells for use with a Hach XR Ratio turbidimeter. The turbidimeter was calibrated using the standard cells before every test run. It was noticed that the turbidity reading from the turbidimeter was unstable when the decanted sample was immediately placed in the cell holder. The reading was found to oscillate up and down around a mean value. This phenomenon made it difficult to resolve small differences between samples. By experiment, it was found that the signal stabilised if the decanted sample was allowed to stand for 15 minutes before inserting into the cell holder. The oscillating reading was thought to be the result of the swirling of the sample within the sample cell immediately after decanting. Standing for 15 minutes before reading allowed the fluid motion to die away resulting in a more stable turbidity reading from the meter. Since the samples comprised colloidal particles, no settlement of the sample would occur in such a short time period. This technique was used during the test runs described in chapter six.

5.4.3 Particle Size Distribution Analysis

Samples for particle analysis were gathered as described previously, from the sample ports at different times during the test runs. These samples were collected in 100 millilitre measuring cylinders and analysed using a Coulter Multisizer II according to the manufacturer's recommendations. The Coulter sample jar was filled with 200 millilitres of filtered electrolyte solution and a blank particle count taken. The collected samples were gently mixed by slowly inverting the measuring cylinders a few times. Sample volumes of 5, 10 or 15 millilitres depending on the sample concentration were then slowly drawn from the measuring cylinders using a wide mouthed pipette. These samples were dispersed in the 200 millilitres of filtered electrolyte solution in the sample jar and placed in the Coulter sample stand for analysis. Multiple particle counts were taken using a 50 micron orifice tube for each sample and the counts averaged. The blank particle count was subtracted from the average sample count and the results stored. This method gave the particle number counts in each sample in each of 256 channels from 0 to 50 microns. The number count in each particle size band was modified by the dilution factor to give the count present in the original sample. Figure 5.5 (a) and (b) give examples of the particle size distribution analysis results. The results are presented as the particle size distribution function plotted against log of the particle diameter. The definition of particle size distribution function and calculation was described in detail previously in section 2.4. Briefly, the particle size distribution function calculation normalised the particle count in each Coulter counter size band to the width of the size band which is then expressed as a logarithm. This process was necessary since the 256 size channels collated by the Multisizer were of different widths. This method allowed easy presentation of the whole particle size distribution on a single chart.

During initial experiments it was found that the particle size distribution of the collected samples shifted with time. Figure 5.5 (a) illustrates a sample taken from 100 mm depth in filter one during run 29. The sample was analysed on the Multisizer in the manner described above on the day it was taken and after storage overnight. It was

(a)



(b)

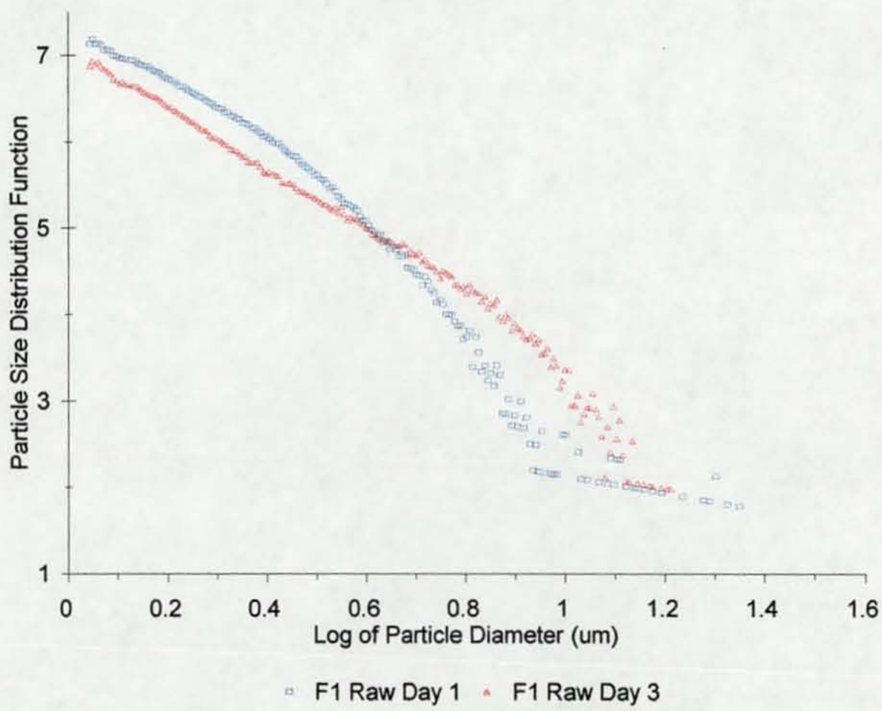


Figure 5.5 (a) & (b): Example particle size distributions from runs 29 & 25

noted that the multiple particle counts used to generate an average count became more erratic after storage overnight. It can be seen from the chart that the averaged number count had fallen for particle sizes from 1 to 3 microns and increased for sizes greater than 3 micron. Similarly, figure 5.5 (b) illustrates the particle analysis of the raw water applied to filter one in run 25. The sample was analysed on the day it was collected and two days later after storage. Once more, the size distribution has been altered by storage reducing the smaller particle counts and increasing the larger particle counts. It would appear that storage overnight in the measuring cylinders caused smaller particles to flocculate generating larger particle sizes. Efforts were made to develop a method of sample storage which did not alter the particle results but these were not successful. Subsequently all analysis was carried out on the day they were collected to prevent distortion of the results. The particle results presented in chapter six were analysed on the day the samples were collected unless otherwise stated.

5.4.4 pH, Conductivity & Dissolved Solids Concentrations

As mentioned in section 5.1, water samples were taken from storage tank one before dosing with the PVC concentrate for pH, conductivity and dissolved solids concentration analysis for each test run. During trial runs, the tap water pH and conductivity were monitored to identify any fluctuations in the water chemistry and subsequent effects on the filters' performance. pH and conductivity were measured using electrical probes (pH: Mettler Model Delta 340, conductivity: Ele Model 4070). The probe calibrations were checked with buffer solutions, made up and replaced according to the manufacturer's instructions, before each measurement. It was identified that the tap water conductivity could vary from 300 to 700 uS/cm from run to run and did influence the rate of removal in the filters. These values are equivalent to dissolved solids concentrations of approximately 200 to 500 mg/l. Attempts were made to control the water conductivity by mixing the tap water with distilled water. However, the quantities of distilled water required to alter some 1500 litres of tap water to a desired conductivity for each run proved to be prohibitive. The effects of the tap water conductivity on filter performance and the surging investigation will be discussed in detail in chapter six. In addition to the conductivity analysis, measurements on the dissolved solids concentrations of a range of elements and ions were also measured to provide greater information on the fluctuating mains water quality. The impact of these changes in water chemistry on the performance of the laboratory filters was monitored. Samples of the clean mains water from storage tank one were analysed using an Atomscan inductively coupled plasma (ICP) spectrometer and a Dionex ion chromatograph. Each machine was calibrated before sample analysis and yielded measurements for the elements and ions shown in table 5.1 below.

Machine	*Elements/Ions Analysed*
ICP spectrometer	Mg, Al, Ca, Cr, Mn, Fe, Co, Ni, Cu, Zn, As, Se, Sr, Ag, Cd, Ba, Hg, Pb, Bi, Li, Be, B, Na, Si, P, K, Mo, W
Ion chromatograph	Fluoride, Chloride, Nitrate, Phosphate, Sulphate, Sodium, Ammonia, Potassium, Magnesium, Calcium

Table 5.1: Summary of elements and ions determined in each test

The spectrometer and chromatograph results are discussed in detail in chapter six.

5.4.5 Zeta Potential Measurement

As mentioned in section 5.3, a sample was taken from storage tank one after dosing with the PVC concentrate for Zeta potential measurement. The Zeta potential was measured using a Malvern Zetamaster. Ten consecutive Zeta potential measurements were taken and the average value recorded for each test run. In each case the spread of the ten readings was small indicating good reproducibility. The prepared PVC suspension was found to have a Zeta potential ranging from -15 to -25 mV during the experimental investigation. The Zeta potential results are discussed in detail in chapter six.

5.5 Chapter Summary

In this chapter the preparation of the test suspension and essential maintenance of the experimental apparatus necessary for well controlled performance have been described. The need for good flow and temperature control was found to be important and the methods used to achieve this were reviewed. Flow control was provided by identical siphons with characterised performance. Temperature control was maintained by immersion heaters and insulation. The development and preparation of the PVC dosed synthetic raw water used as the test suspension were presented. This was chosen to give a known concentration and particle size distribution. The concentrate was prepared by mixing quantities of PVC powder in water and siphoning off the supernatant after a period of settlement. The preparation of the filter columns for a typical test run was discussed including the issues of backwashing and bed settlement. The procedure for a typical experimental test run and the sampling regime have been described. The methods and equipment used in the analysis of the collected samples have also been described.

6.0 EXPERIMENTAL RESULTS AND DISCUSSION

In this chapter, the experimental programme on surging and the results obtained will be reviewed and discussed. This will begin with a description of the test runs carried out presented in chronological order to place the results in context. Key events in the findings and evolution of the equipment and methodology will be highlighted where appropriate. The chapter goes on to present analyses of various aspects of the test results obtained. These aspects include the characteristics of the development of head loss and the removal efficiency of both turbidity and particles when subjected to surging flow and the influence of the suspension chemistry on laboratory filter performance. The significance of the test results will be discussed in comparison with the results of previous research reviewed in chapter two and the findings of the survey of local water treatment plants discussed in chapter three.

6.1 Chronology & Key Events of the Test Programme

As discussed in the introduction to chapter four, the approach taken in this surging experimental investigation was to develop the apparatus and experimental methods until similar and reproducible performance was established in both the laboratory filters before the application of surging flow to filter two only. The purpose of this chronology is to describe briefly the evolution of the experimental programme and its findings before conducting detailed analysis and discussion of the results. Key developments of the apparatus described in chapter four and the experimental procedure and methods described in chapter five will be introduced as they occurred in the laboratory test programme.

6.1.1 Test Runs One to Ten

Initial test runs were conducted to determine suitable operating conditions for the investigation into the effects of surging on filter performance and to test and identify weaknesses in the equipment design and method of operation. Table 6.1 presents a summary of the first ten test runs' operating conditions and results.

Run	*Influent Turbidity (NTU)*	*Temp (°C)*	*Run Length (Hours)*	*Filter One Head Loss (mm)*	*Filter One % Removal*	*Backwash & Settle*	*Surges Applied*
1	161	20	6	440	63	Slow cut-off	None
2	152	20	5	626	79.7	Pulsed	None
3	131	23	4	626	75.3	Shock	None
4	Abandoned	-	-	-	-	-	-
5	193.5	32	2	1116	97.1	Vibrator (V)	None
6	45	30.5	4.5	326	84.9	V	None
7	110.2	32	5	699	93.1	V	None
8	117.5	31	5	622	85.3	V	None
9	116.1	30.5	5	435	79.8	V	5-8% 60/min
10	121.1	30.5	5	608	82.4	V	20% 10/min

Table 6.1: Operating Conditions Runs One to Ten

The table includes the turbidity of the influent PVC suspension used, the suspension temperature, the length of the filter run, the head loss and turbidity removal efficiency achieved by filter one at the end of the run, the method of backwash and settlement of the filter beds used and the nature of the surges applied to filter two in each test run. Tables of all the head, turbidity and flow readings taken are presented in Appendix A for all the test runs conducted in this experimental investigation. All the tests were conducted at an approach velocity of 8.0 metres per hour.

As discussed in section 5.1.1, the laboratory apparatus was originally operated at a suspension temperature of 20°C (Runs 1 to 3). An influent turbidity of 131 to 161 NTU produced a head loss of 440 to 626 mm of water in 4 to 6 hours with a turbidity removal efficiency of 63 to 80% in filter one. These results and the trends are illustrated in figures 6.1 to 6.3. It can be seen that filters one and two show near similar performance in both head loss and turbidity removal with time but with poor consistency. During these runs the problems with the backwash cut-off and manual settlement of the filter beds described in section 4.4.2 were encountered.

Test run four was abandoned due to PVC deposits forming in the system piping and siphons. The influent suspension turbidity fell by over 50 NTU and the siphon flow rates dropped by 50% overnight. The system piping and the siphons were thoroughly cleaned after each test run from this point onward.

The use of the electric vibrator described in section 4.4.2 to settle the filter beds during backwash cut-off was introduced in run five. The suspension operating temperature was by this time raised to 30 °C as discussed in section 5.1.1. Test runs five to eight were concerned with identifying a suitable suspension concentration at this temperature to develop initially 0.5 to 1.0 metres of head loss in 5 hours and improving the consistency and similarity of performance in both filters in order to be able to identify the effects of surging. Figures 6.4 to 6.7 illustrate the head loss in cm of water and turbidity removal efficiency trends for these runs. By runs seven and eight the filters' performance in both head loss and removal can be seen to have improved in consistency and similarity. These runs, conducted with a suspension turbidity of 110 to 120 NTU, developed 600 to 700 mm head loss in 5 hours with a removal efficiency of 85 to 93%.

Having achieved this performance, surges were applied to filter two in runs nine and ten as a first attempt to identify their impact on performance. As discussed in chapters two and three, surges can occur in rapid gravity filters with amplitudes of 2 to 10 % or more of the filter head loss at rates of up to 100 oscillations per minute. As a starting point, surges 5 to 8 % of the head loss were applied to filter two at a rate of 60 oscillations per minute in run nine using the surge machine described in section 4.4.1. Surges 10 to 20% of the head loss were applied to filter two at a rate of 10 oscillations per minute in run ten. The surges applied in run ten were recorded at the end of the filter run by the pressure transducer attached to the base of filter two. The recorded profile is illustrated in figure 6.31. From this chart it can be seen that the fluctuations in pressure at the base of the filter column as measured by the pressure transducer were calculated to be 85 mm in magnitude from the transducer calibration and were similar to the water level fluctuations observed in the corresponding piezometer.

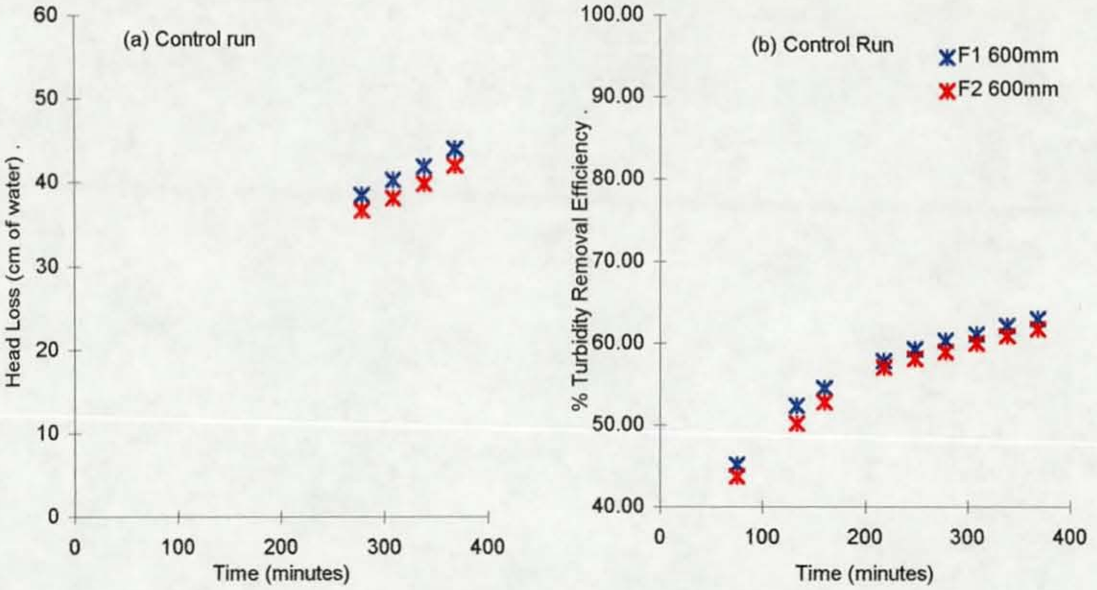


Figure 6.1 (a) & (b): Run One Head loss & Turbidity Removal Efficiency

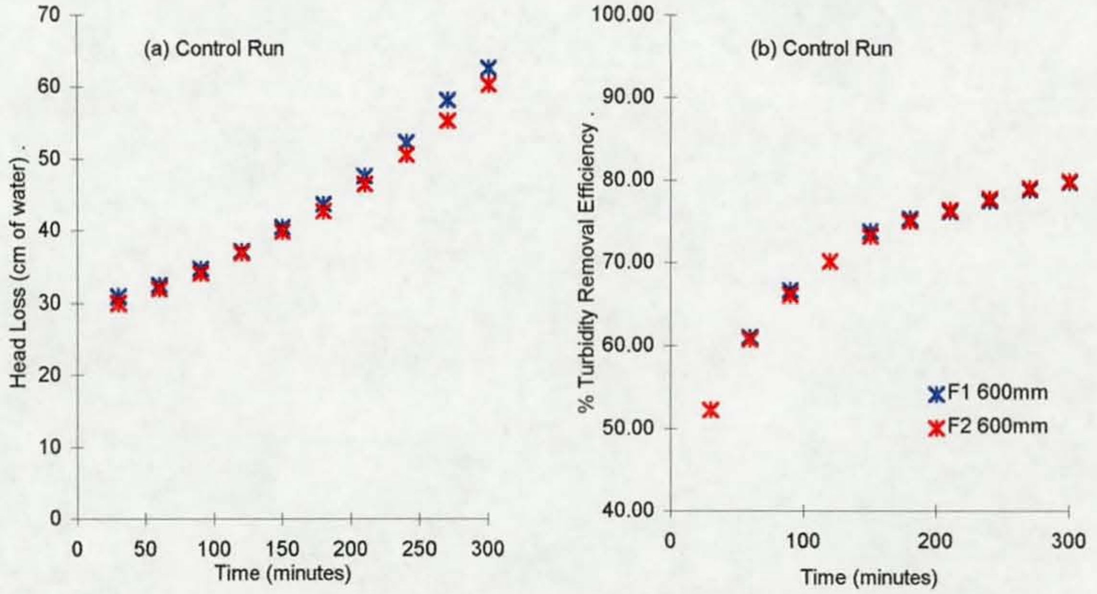


Figure 6.2 (a) & (b): Run Two Head loss & Turbidity Removal Efficiency

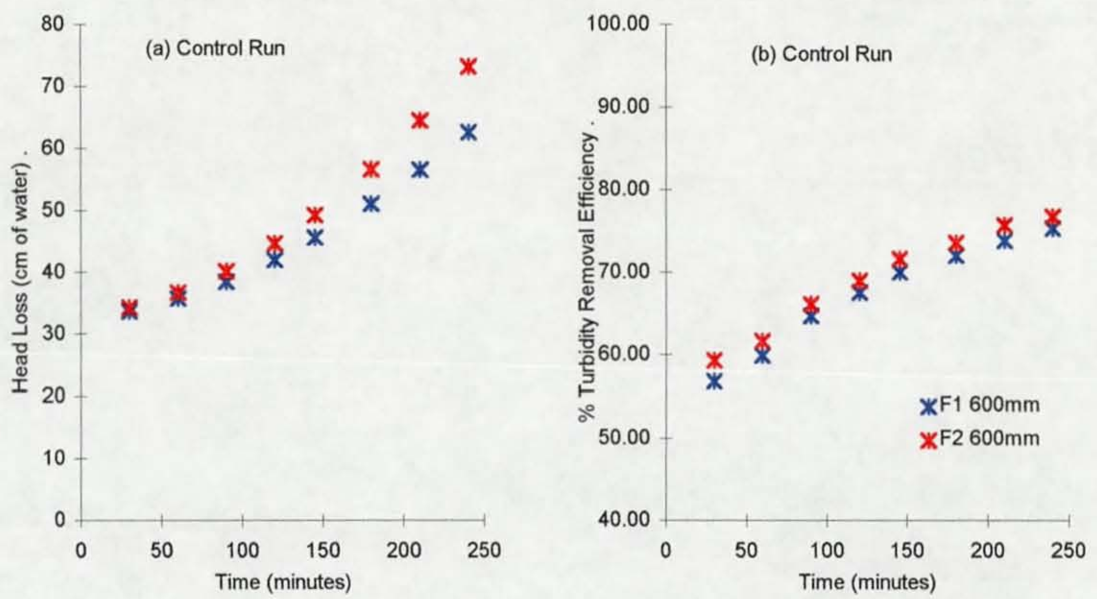


Figure 6.3 (a) & (b): Run Three Head loss & Turbidity Removal Efficiency

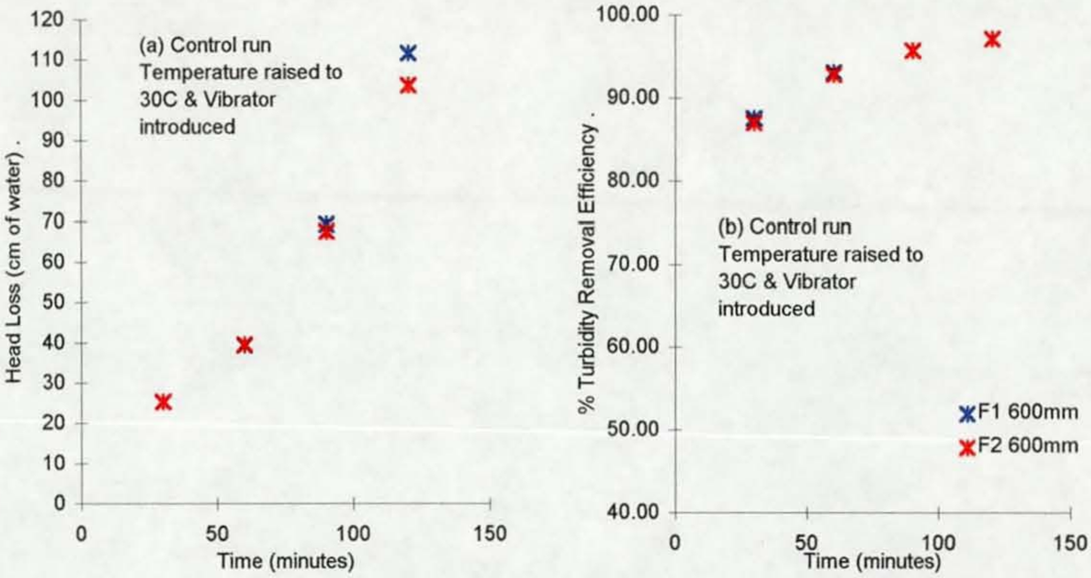


Figure 6.4 (a) & (b): Run Five Head loss & Turbidity Removal Efficiency

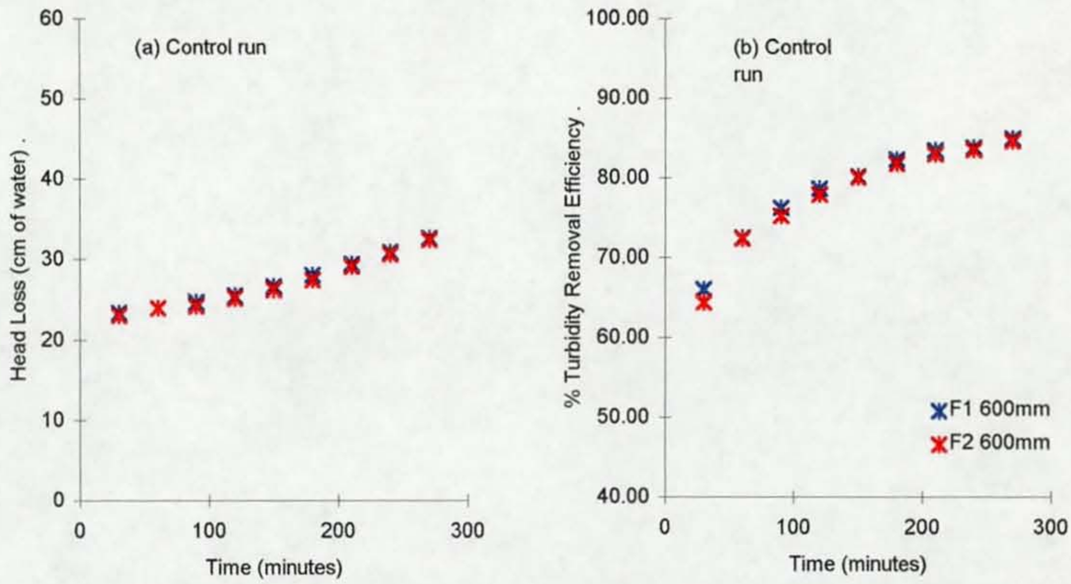


Figure 6.5 (a) & (b): Run Six Head loss & Turbidity Removal Efficiency

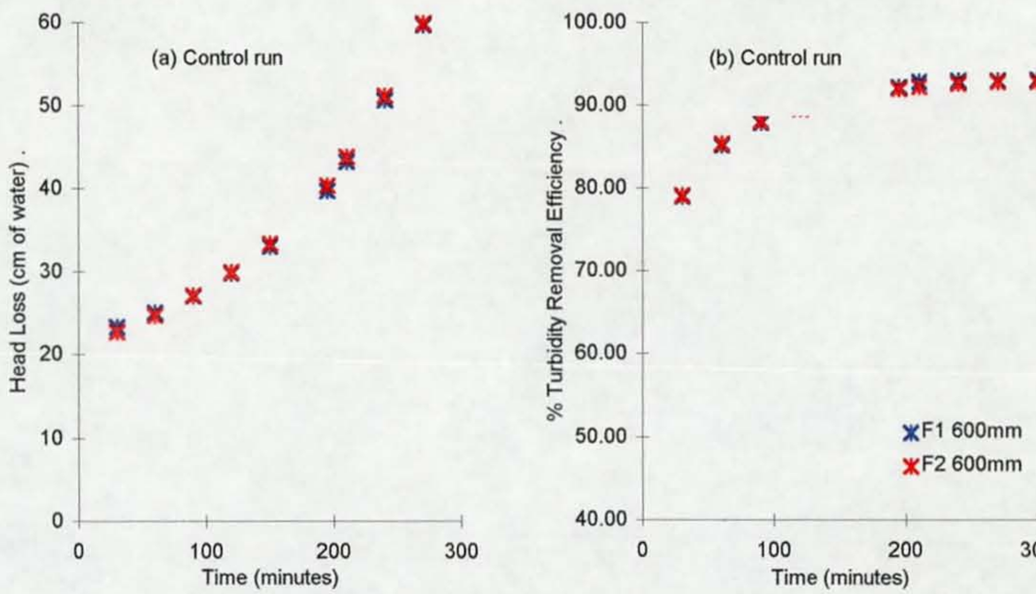


Figure 6.6 (a) & (b): Run Seven Head loss & Turbidity Removal Efficiency

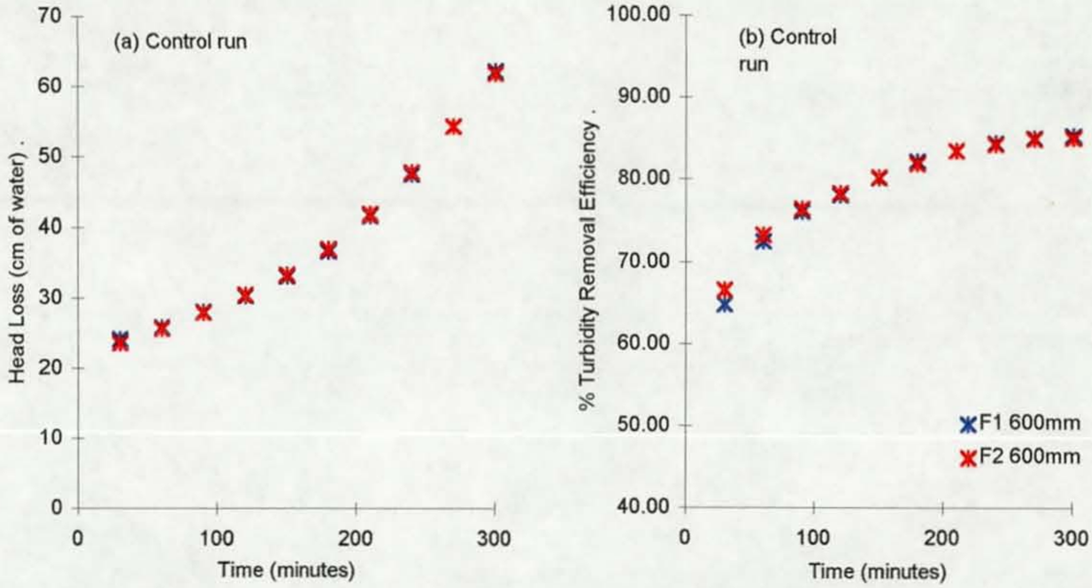


Figure 6.7 (a) & (b): Run Eight Head loss & Turbidity Removal Efficiency

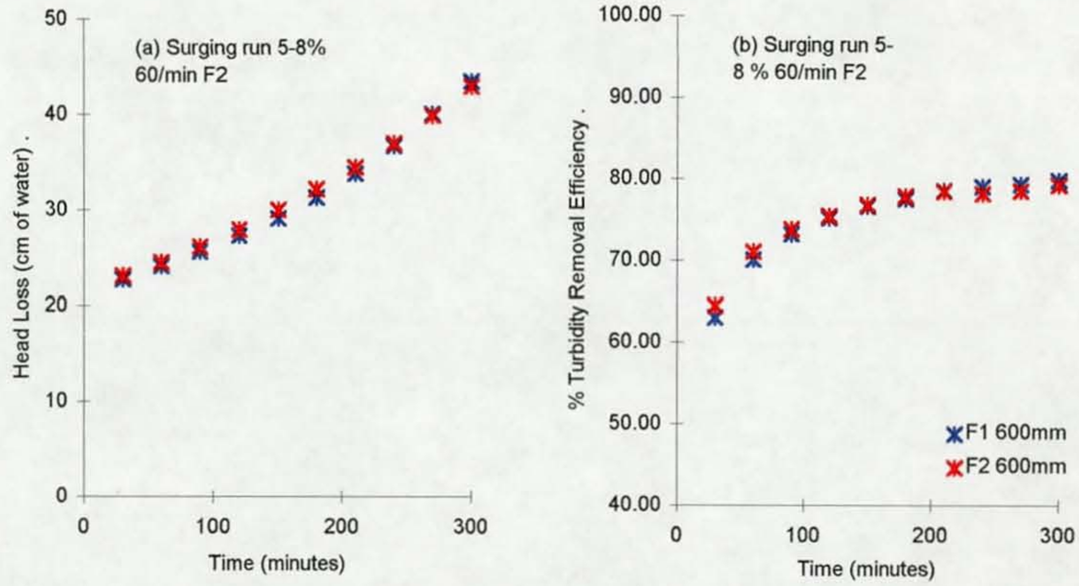


Figure 6.8 (a) & (b): Run Nine Head loss & Turbidity Removal Efficiency

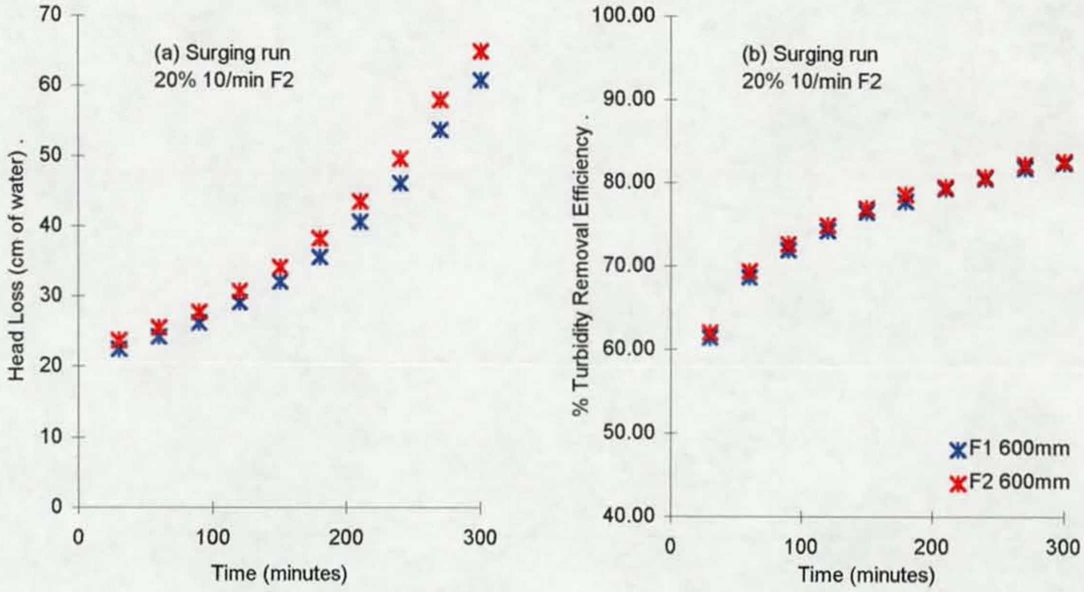


Figure 6.9 (a) & (b): Run Ten Head loss & Turbidity Removal Efficiency

The head loss in cm of water and turbidity removal efficiency curves from runs nine and ten are shown in figures 6.8 and 6.9. No apparent difference in the head loss development was observed in filter two compared with filter one in run 9 with respect to the previously established performance of filters one and two in the non surging test runs seven and eight. Similarly, there was no obvious change in the turbidity removal efficiency of filter two compared with filter one and with the previous two test run results. The surges applied in run nine appeared to have no identifiable impact on filter development. In run ten, filter two appeared to develop head loss more rapidly than filter one such that by the end of the test there was a difference of 40 to 50 mm. However, it was felt that this was not due to the presence the surging flow in filter two since there was no apparent effect on the removal efficiency. The difference was more likely to be due to variation in filter performance from run to run. However, if the observed difference was in fact due to the surging flow this would be confirmed by later runs.

6.1.2 Test Runs Eleven to Twenty

From the first ten test runs, it was concluded that the filters needed to be operated to greater head losses and run lengths to bring out any effect of the surges more clearly. Head losses of 500 or 600 mm and run lengths of 5 hours did not appear to be sufficient to clearly show any influence on filter performance from the flow rate fluctuations applied. As such, additional storage capacity was added to the system to enable longer filter runs of ten hours. In addition, the constant head tank was moved to provide additional available head to operate the filters to over a metre of head loss. Table 6.2 presents a summary of the operating conditions and results of test runs eleven to twenty.

Run	*Influent Turbidity (NTU)*	*Temp (°C)*	*Run Length (Hours)*	*Filter One Head Loss (mm)*	*Filter One % Removal*	*Surges Applied*	*Conductivity (uS/cm)*
11	108.1	30	10	1397	78.4	20% 10/min	753
12	107.4	30	10	1152	82.0	None	744
13	110.8	30.5	11	1340	89.1	10% 50/min	754
14	-	-	-	-	-	-	-
15	97.5	29.0	7.5	1463	88.0	20% 10/min	765
16	95.7	30	7.5	1529	88.8	None	-
17	93	30	7.3	1523	90.2	20% 10/min	-
18	44.8	30	20	614	70.1	None	-
19	64.9	29.5	9	2279	90.8	None	700
20	47.7	30	18	2397	93.2	None	710

Table 6.2: Operating Conditions Runs Eleven to Twenty

Run eleven was conducted at an influent turbidity of 108 NTU and operated for ten hours. The filters developed over 1400 mm of head loss in this time.
Surges of 20% of the filter head loss were applied to filter two at a rate of 10 oscillations per minute in this run similar to run ten. The surge profile recorded by the pressure transducer is shown in figure 6.32 From this chart it can be seen that the surge magnitude was 250 mm 550 minutes into the test and was similar to the observed amplitude in the corresponding piezometer. Figure 6.10 (a) and (b) illustrate the head loss in cm of water and turbidity removal efficiency trends for filters one and two in run eleven. From the head loss chart it can be seen that filter two developed head loss more rapidly than filter one such that the difference was 230 mm by the end of the test. Run ten described earlier conducted with similar surging appeared to give a similar outcome. Close inspection of the turbidity removal efficiency chart appeared to show a marginally better removal in filter two. Run twelve was conducted at a

similar influent turbidity and run length to run eleven but with no surges applied to filter two for comparison with the results of runs ten and eleven. The head loss in cm of water and turbidity removal efficiency trends for this control test run are shown in figure 6.11 (a) and (b). It can be seen that once again filter two developed head loss more rapidly that filter one and achieved a marginally better removal efficiency. This result suggested that the difference in performance observed between the two filters in runs ten and eleven was not the result of the surging flow applied. It seemed that 20% surges applied at 10 oscillations per minute had no clear influence on the filter performance with these operating conditions.

Test run thirteen was conducted with 10% surges applied to filter two at 50 oscillations per minute to determine if smaller more rapidly occurring surges would produce a clear impact on filter development with these operating conditions. Run thirteen was conducted at an influent turbidity of 110 NTU for eleven hours similar to runs eleven and twelve for comparison. The surge amplitude in the piezometer connected to the base of filter two was noted to be 122 mm at a rate of 50 oscillations per minute at the end of the filter run. Figure 6.33 illustrates the surge profile captured by the pressure transducer attached to the base of filter two. From this chart the transducer recorded an amplitude of 250 mm at a rate of 50 oscillations per minute at the end of the filter run. This difference in the piezometer and transducer observed amplitude was recognised later in the test programme and will be discussed fully in section 6.5 of this chapter. Figure 6.12 (a) and (b) illustrates the head loss in cm of water and turbidity removal efficiency trends for run thirteen. It can be seen that filter two again developed head loss more rapidly than filter one with a marginally better removal efficiency similar to control run twelve with no surging. 10% surges at 50 oscillations per minute had no identifiable effect on filter development with these operating conditions.

Test run 14 was abandoned due to suspension overflow from the inlet piping to the filter columns. This was caused by trapped air in the piping. The inlet funnels and air vents were modified to prevent recurrence of the problem.

Test run 15 was conducted at an influent turbidity of 97.5 NTU and developed over 1400 mm of head loss in 7.5 hours. Surges 20 % of the head loss were applied to filter two at a rate of 10 oscillations per minute similar to run eleven. Figures 6.13 (a) and (b) illustrate the head loss in cm of water and turbidity removal efficiency trends. During the test run an event

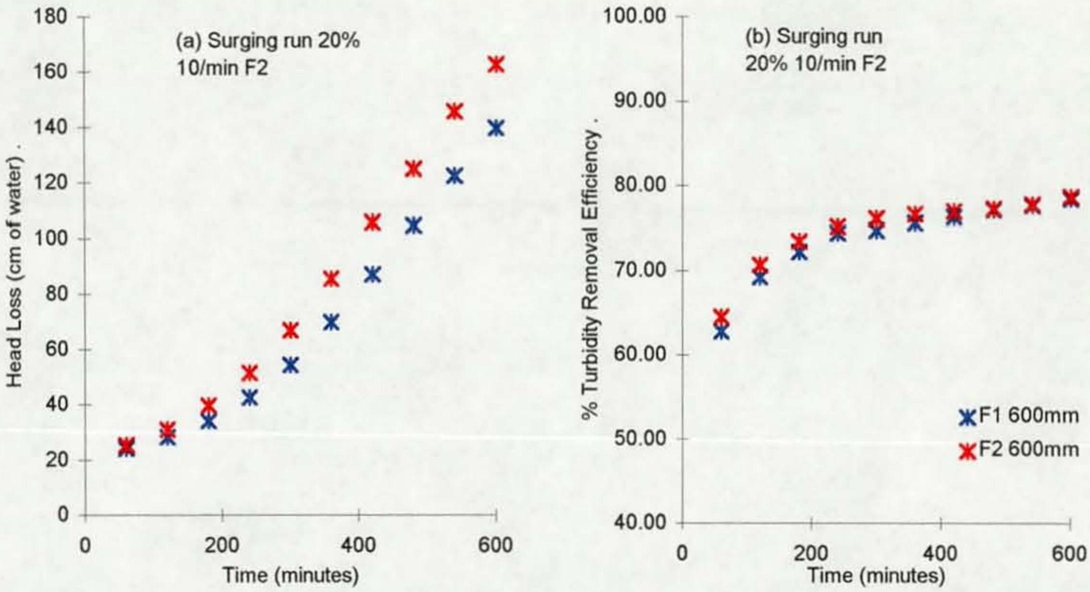


Figure 6.10 (a) & (b): Run Eleven Head loss & Turbidity Removal Efficiency

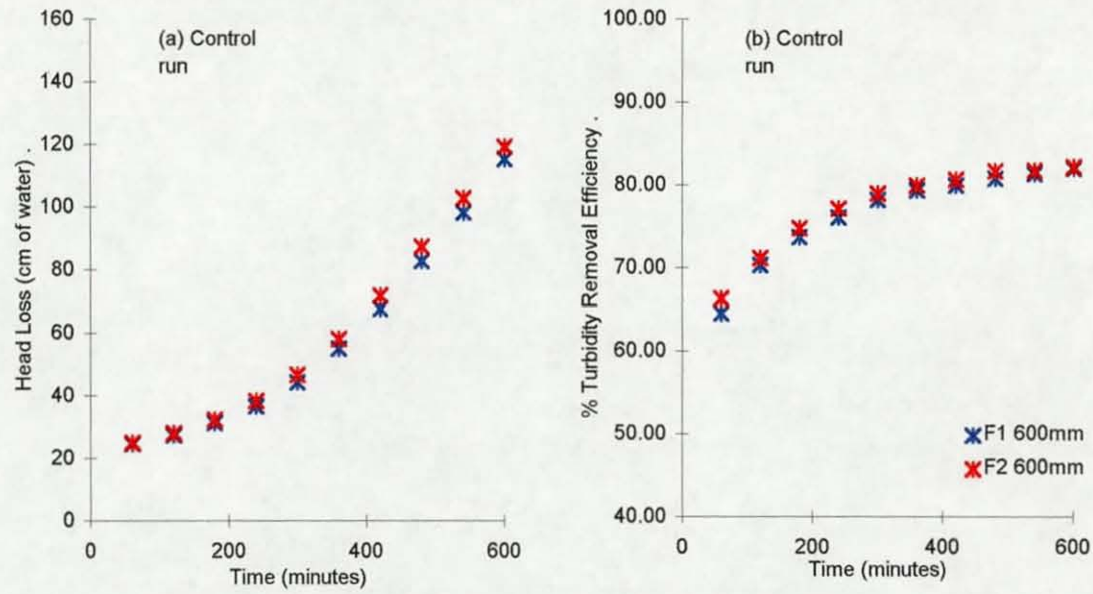


Figure 6.11 (a) & (b): Run Twelve Head loss & Turbidity Removal Efficiency

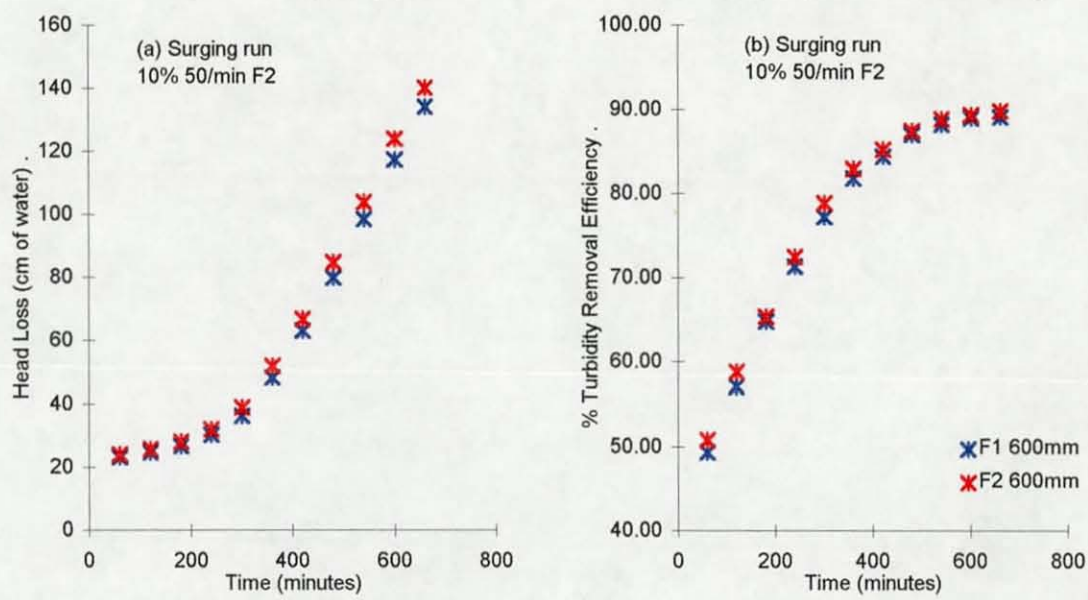


Figure 6.12 (a) & (b): Run Thirteen Head loss & Turbidity Removal Efficiency

occurred which altered the performance of filter two. Four hours into the test run the media in filter two was observed to compact rapidly down by 1 to 2 mm. Close inspection of the head loss and removal efficiency trends show that this rapid bed movement altered the filter performance by disturbing the deposits accumulated in the first 4 hours of the test. Initially the head loss in filter two led filter one by a few centimetres. However, after the bed movement the head loss trend in filter two lagged behind filter one increasingly with time. Similarly, initially the removal efficiency in filter two was marginally better than in filter one but subsequent to the bed movement dropped behind filter one and never recovered.

Inspection of previous test run bed level measurements revealed that bed movement during the filter run was common. It was found that the filter bed levels fell by a few millimetres to several centimetres in each filter and by differing amounts in each filter as the hydraulic gradient increased over the course of the test run. This movement of the filter beds was believed to have contributed towards the lack of reproducibility in previous tests and as found in run fifteen could significantly alter the filter development possibly masking the influence of the surges. To overcome this problem and prevent future bed movement the filter beds were settled down by an additional 20 mm during cut-off of the backwash flow. In addition, the filters were operated with clean tap water at 8.0 metres per hour and 30°C for a period of one hour as described in section 5.3 to induce bed movement before commencing the test run. It was felt that this additional settlement of the filter beds would produce greater bed stability and prevent significant movement during subsequent tests.

It was at this point in the test programme that the standardised cut-off of the backwash flow, the temperature control of the backwash water and fitting of the cross clamps between the filters were introduced to further improve the backwash and settlement reproducibility. These improvements and their effects were discussed in section 4.4.2 earlier. Also at this point, the 1 mm aperture steel mesh was introduced between the filter sand and the supporting gravel layers as described in section 4.3.6

Run sixteen was conducted after all these improvements had been completed. Run sixteen was conducted at an influent turbidity of 95.7 NTU and developed over 1500 mm of head loss in 7.5 hours. No surges were applied to filter two to determine how filters one and two compared after the above described improvements were made. Figure 6.14 (a) and (b) illustrates the head loss in cm of water and turbidity removal efficiency trends from this test. As can be seen, filters one and two showed similar head loss and removal efficiency trends. Run seventeen was conducted with identical conditions but with 20% surges applied to filter two at a rate of 10 oscillations per minute. Filter one developed over 1500 mm of head loss in 7.5 hours similar to run sixteen. The surge profile captured by the pressure transducer at the end of the test is shown in figure 6.34. Figure 6.15 illustrates the head loss in cm of water and turbidity removal efficiency trends for this test. Comparison of the head loss charts for runs sixteen and seventeen showed a lag in the development of filter two when subjected to the surging flow. Close inspection of the removal efficiency trends appeared to show a small change in the removal efficiency of filter two when surging but was too small to be certain. It was felt at this time that it would be necessary to run the filters to greater head loss and over longer time periods to confirm clearly and unambiguously the apparent effects of the surging flow applied in run seventeen.

A third storage tank was added to the system to provide additional capacity to run the filters for up to 20 hours; more similar to full scale filter run times. The constant head tank was elevated further to increase the available head such that the filters could be operated to greater than two metres of head loss if required. The piezometer tubes were modified to allow measurement up to these levels. In addition, at this time in the test programme, the pressure ports and suspension sampling ports within the filter media described in section 4.3.4 and 4.3.5 were added to the filter column middle sections. These ports would provide detail of the filter head loss and removal performance within the filter media as opposed to purely the overall removal and head loss from the whole filter bed. It was believed that surging flow may have an influence on the upper layers of the filter media but this effect would subsequently be masked by the performance of the lower layers of the filter bed. These effects would not be apparent from measurements from the filter outlets only.

The modifications described above involved complete breakdown and reassembly of the filter units and took several months. It was therefore expected that several test runs would be required to re-establish reproducible filter performance and to target

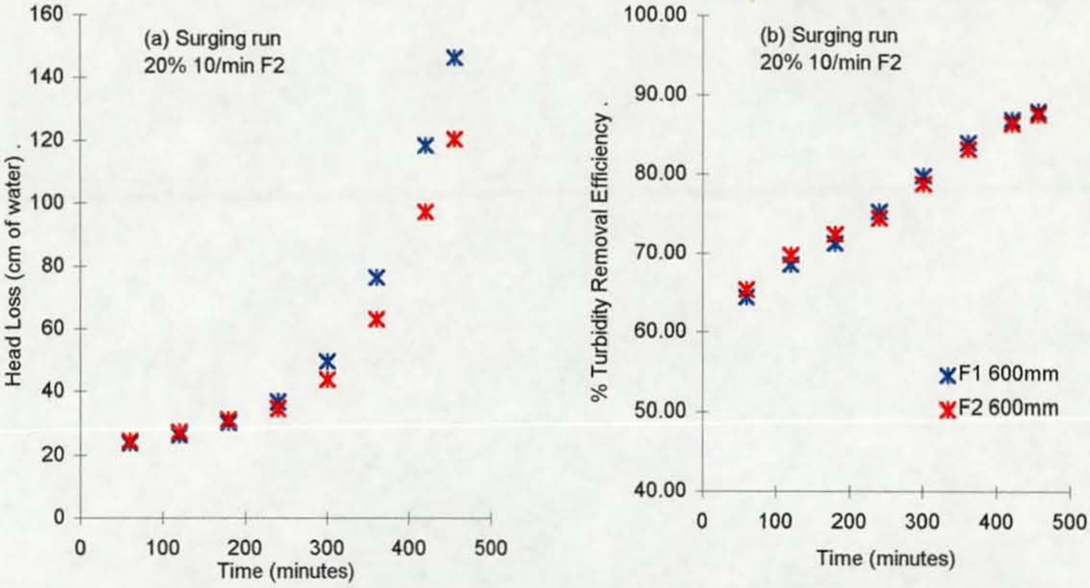


Figure 6.13 (a) & (b): Run Fifteen Head loss & Turbidity Removal Efficiency

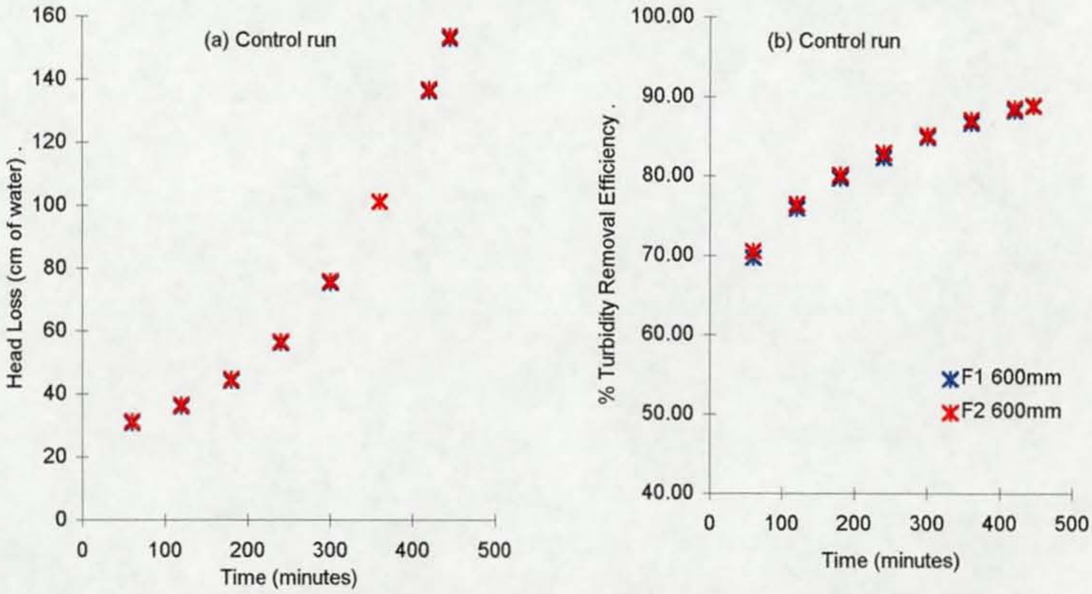


Figure 6.14 (a) & (b): Run Sixteen Head loss & Turbidity Removal Efficiency

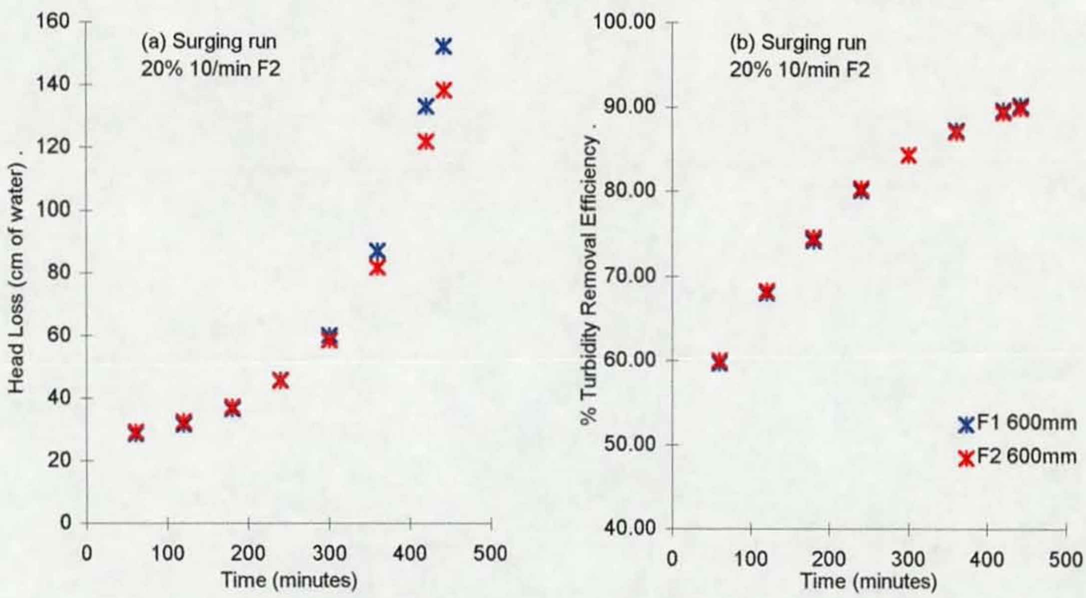
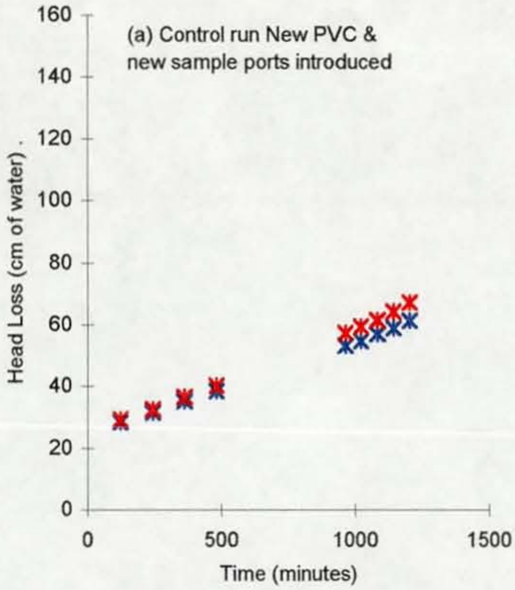


Figure 6.15 (a) & (b): Run Seventeen Head loss & Turbidity Removal Efficiency

the influent concentration to generate up to two metres of head loss in up to twenty hours run time. In run eighteen the inlet turbidity was dropped to 44.8 NTU as a first attempt to increase the run length to up to twenty hours. However the filters only developed 600 to 700 mm of head loss in this time; less than expected at this concentration from previous experience. Figures 6.16 (a) and (b) illustrate the head loss in cm of water and turbidity removal efficiency trends from this test. Note that filter two developed head loss more rapidly that filter one in this test and that the filters only achieved 70 % removal of turbidity. This poor performance may have been due to the filter downtime of 3 to 4 months while the modifications were carried out. The sand, unused for this period of time, may have required several runs to become re-established. The influent turbidity was increased to 65 NTU for test run nineteen to increase the head loss developed over run eighteen. Run nineteen developed over 2000 mm of head loss but in only nine hours; faster than expected at this concentration. Figure 6.17 (a) and (b) illustrate the head loss in cm of water and turbidity removal efficiency trends. Here the filters achieved over 90 % removal. Again, filter two developed greater head loss than filter one. The influent turbidity was reduced to 47.7 NTU once more for run twenty. Run twenty developed over 2000 mm of head loss in eighteen hours. Figures 6.18 (a) and (b) illustrate the head loss in cm of water and turbidity removal trends. The filters achieved over 90% removal with greater head loss in filter two once again.

Important factors which influenced the filters' development began to come to light during the conduct of runs eighteen to twenty. Firstly, the removal efficiency achieved fluctuated from only 70% in run eighteen to 90% in run twenty producing head losses of 600 to over 2000 mm respectively in 18 to 20 hours. In earlier test runs, it was suspected that the mains water used to generate the test suspension exhibited fluctuations in conductivity and that this variation could influence the filter removal efficiency. The suspension conductivity was monitored from runs eleven to fifteen to determine if this was so. The conductivity measurements are given in table 6.2. However, the conductivity of the mains water remained stable around 750 uS/cm with corresponding removal efficiencies of 80 to 90 %. The conductivity monitoring was discontinued after run fifteen since there was no apparent variation. However, run eighteen achieved only 70 % removal and lower head loss development than expected. This may have been the result of the downtime but it was decided to initiate conductivity monitoring once more to ensure no unobserved interference had



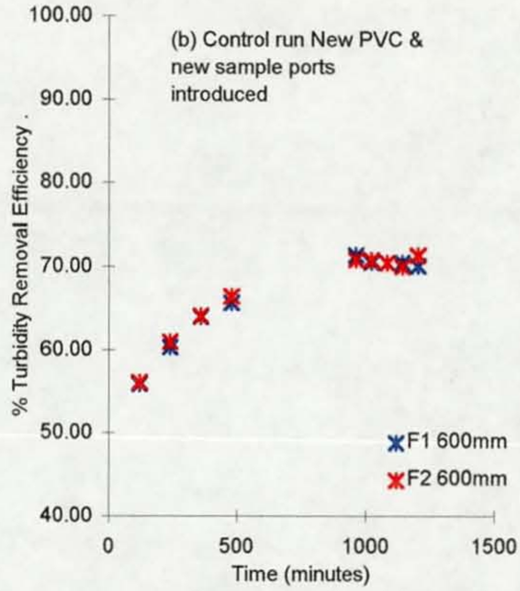
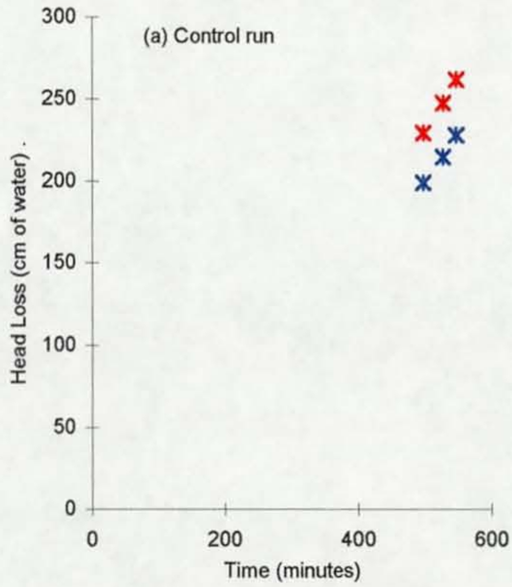


Figure 6.16 (a) & (b): Run Eighteen Head loss & Turbidity Removal Efficiency



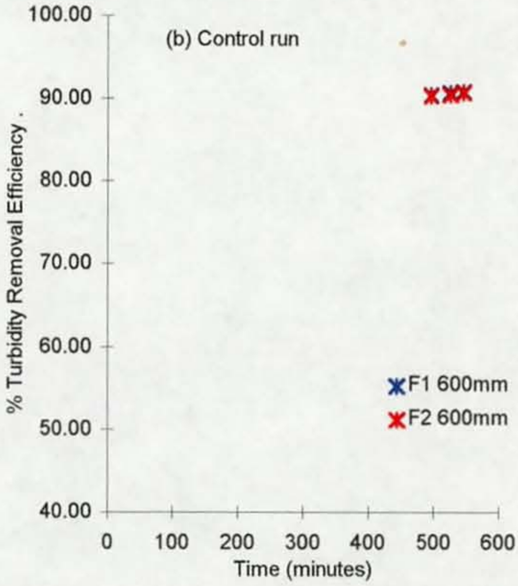
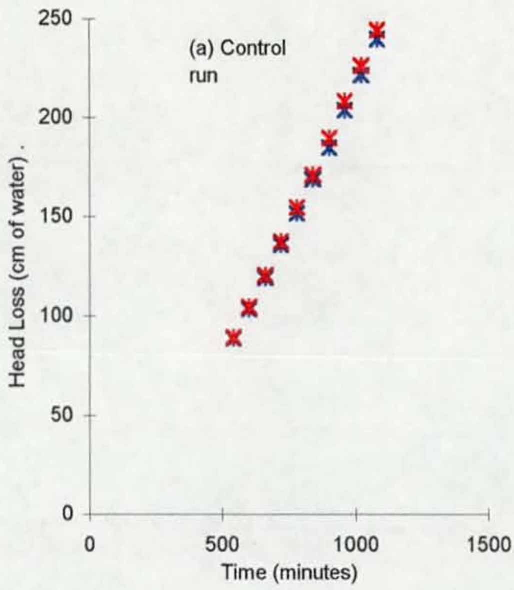


Figure 6.17 (a) & (b): Run Nineteen Head loss & Turbidity Removal Efficiency



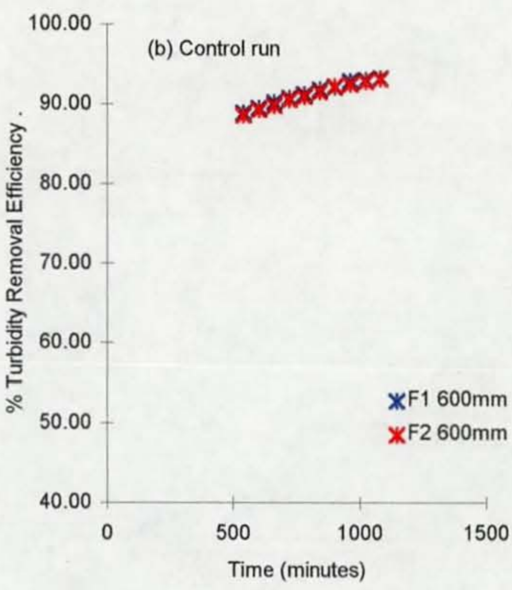


Figure 6.18 (a) & (b): Run Twenty Head loss & Turbidity Removal Efficiency

occurred. As will be seen in later runs, the combination of the mains water conductivity and the Zeta potential of the test suspension influenced the removal efficiency achieved and the head loss developed at a given influent concentration. Run nineteen measured 700 uS/cm similar to earlier runs but developed head loss more rapidly than expected. However, a second variable may have influenced this result. At this point in the test programme the original bag of PVC powder ran out and was replaced with a new bag. The new bag of PVC powder was of identical specification but was from a different batch and had been in storage for several years. Run twenty measured 710 uS/cm similar to earlier runs but took 18 hours to develop over 2000 mm of head loss; the result of the decrease in influent turbidity from 65 to 48 NTU. However, discussions with the PVC supplier revealed that the PVC powder had a tendency to stratify when stored long term. This stratification of the powder in the bag could move smaller particle sizes to the surface and it was recommended that the bag be mixed to eliminate any such stratification. It was believed that the PVC suspensions prepared for runs nineteen and twenty may have had a smaller mean particle size than previous runs resulting in the more rapid development observed in run nineteen. After run twenty, the bag of PVC powder was thoroughly mixed before taking a sample to prepare the PVC concentrate. It was therefore expected that the filter performance would change in the next test run.

6.1.3 Test Runs Twenty One to Thirty Two

In the previous ten test runs, significant improvements were made to the filter apparatus and experimental procedure until runs sixteen and seventeen appeared to show a difference resulting from the application of surging flow. Further modifications were then added to enable more detailed sampling. However, problems were encountered with the suspension chemistry and particle distribution on restarting the experimental programme. It will be seen in this section that these problems were overcome and the effect of the surges confirmed by repeated tests. Table 6.3 presents a summary of the test conditions for runs twenty one to thirty two.

Run	*Influent Turbidity (NTU)*	*Run Time (Hours)*	*Filter One Head Loss (cm)*	*Filter One % Removal*	*Surges Applied*	*Conductivity (uS/cm)*	*Zeta Potential (mV)*
21	46	19	120.8	>92% 400mm	20% 10/min	677	-
22	46.9	17.5	144.2	>82% 400mm	10% 40/min	730	-
23	52.5	18	111.1	79.1	None	780	-
24	51.7	17.5	113.2	80.3	10cm 80/min	758	-
25	51.8	18	110.4	75.6	None	708	-16.6
26	50.4	10.5	156.2	92.1	10cm 80/min	310	-18.9
27	51.8	13.5	131.1	80.2	10cm 80/min	635	-20
28	51.7	9	127.4	90	None	512	-16.1
29	54.0	13.5	128.9	68	Scaled 80/min	643	-24.1
30	50.7	10.5	130.5	87	Scaled 40/min	625	-17.5
31	51.3	12	109.9	88	None	656	-15.9
32	51.4	9	113	93	10cm 80/min	584	-16.2

Table 6.3: Operating Conditions Runs Twenty One to Thirty Two

Having thoroughly mixed the bag of PVC powder, run twenty one was conducted at an influent turbidity of 46 NTU for 19 hours similar to run twenty. However, as expected the filters' performance changed once more. Filter one developed only 1200 mm of head loss in this time. Figure 6.19 (a) & (b) illustrate the head loss in cm of water and turbidity removal efficiency trends throughout the filter depth from this test. Due to the problems encountered with runs eighteen to twenty, a different approach was tried in this test. The filters were operated with no surges for the first 800 minutes to identify any differences between the filters before the application of surging flow to filter two only. From the charts, it can be seen that filter two developed head loss more rapidly that filter one during this time with marginally greater removal efficiency throughout the depth. The surge machine was started at 800 minutes and set to produce surges of 20% of the head loss at a rate of 10 oscillations per minute for the rest of the run in filter two. Figure 6.35 illustrates the surge profile captured by the transducer at the end of the test. Close inspection of the head loss and removal trends before and after 800 minutes showed no apparent change in the performance of filter two. The test was repeated in run twenty two with 10% surges applied at a rate of 40 oscillations per minute from 810 minutes onward in filter two only. Figure 6.20 (a) & (b) illustrate the head loss and turbidity removal trends. Figure 6.36 illustrates the surge profile captured in filter two by the transducer at 1050 minutes. It can be seen that filter two developed head loss more rapidly than filter one throughout the depth with better removal efficiency in the upper layers of the media, similar to run twenty one. However, once more the application of the surges midway through the filter run did not result in any change in the performance of filter two. Since the modifications introduced after run seventeen, filter two had consistently developed greater head loss than filter one. Indeed, from the pressure and suspension ports within the filter beds, it had become clear that filter two produced greater head loss throughout the filter depth with greater removal efficiency in the upper layers of the media. It would appear that the changes made to the filters after run seventeen had introduced small but reproducible differences in the performance of filter one and two. It was felt that these small inherent differences between filters one and two were not a problem provided they were confirmed to be reproducible by control test runs conducted between surging test runs. As such, run twenty three was conducted as a control test run with no surging to confirm the differences observed in runs twenty one and twenty two. Run twenty three was conducted at an influent turbidity of 52.5 NTU over 18 hours. Figure 6.21 (a) & (b) illustrate the head loss and turbidity removal efficiency trends throughout the filter depths. Once more filter two developed greater head loss than filter one throughout the depth with better removal in the upper 200 mm, confirming the differences observed in runs twenty one and twenty two. Having re-established this reproducible performance further surging tests could be conducted. Previous surging test runs were conducted with surges applied at fixed magnitudes relative to the head loss achieved. That is to say, the absolute surge magnitude was

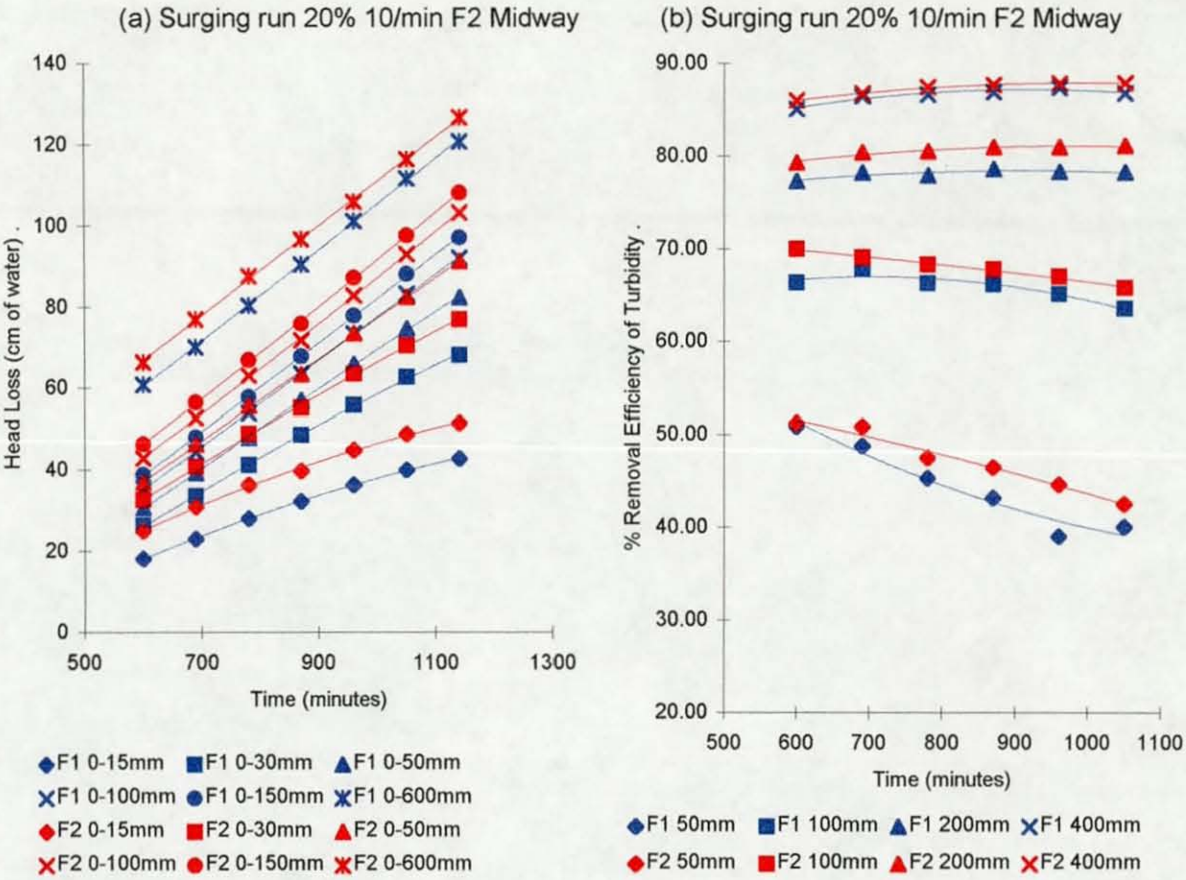


Figure 6.19 (a) & (b): Run Twenty One Head loss & Turbidity Removal Efficiency

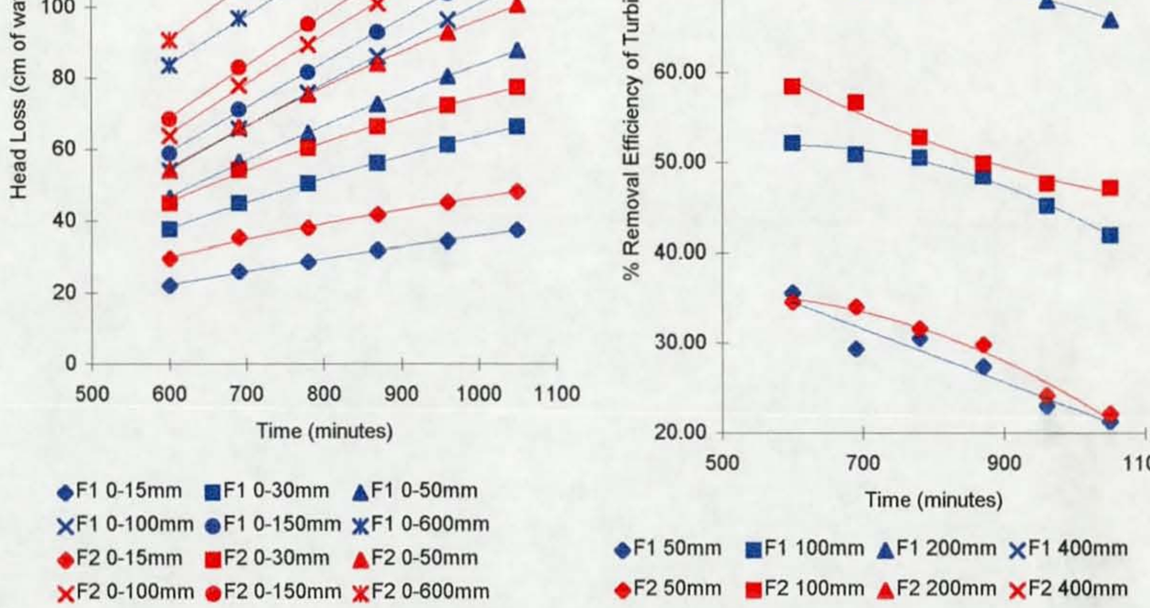


Figure 6.20 (a) & (b): Run Twenty Two Head loss & Turbidity Removal Efficiency

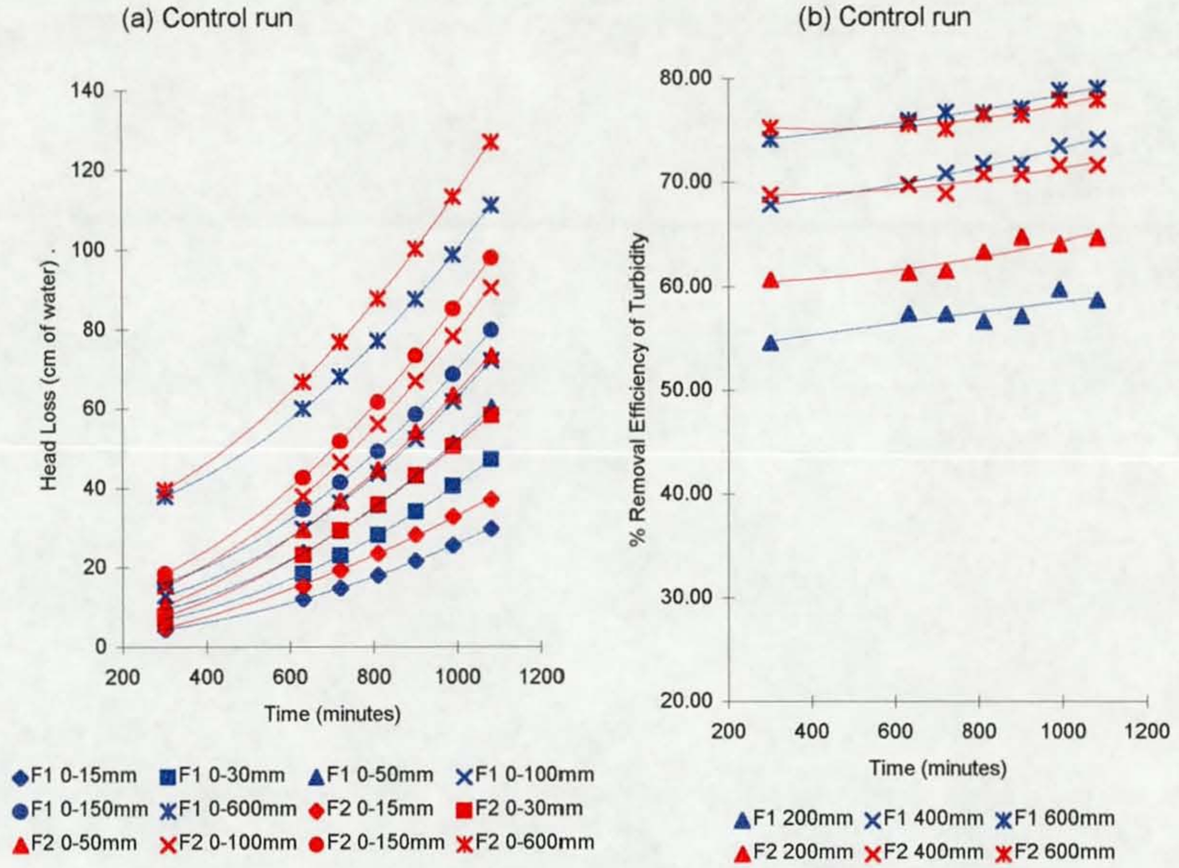


Figure 6.21 (a) & (b): Run Twenty Three Head loss & Turbidity Removal Efficiency

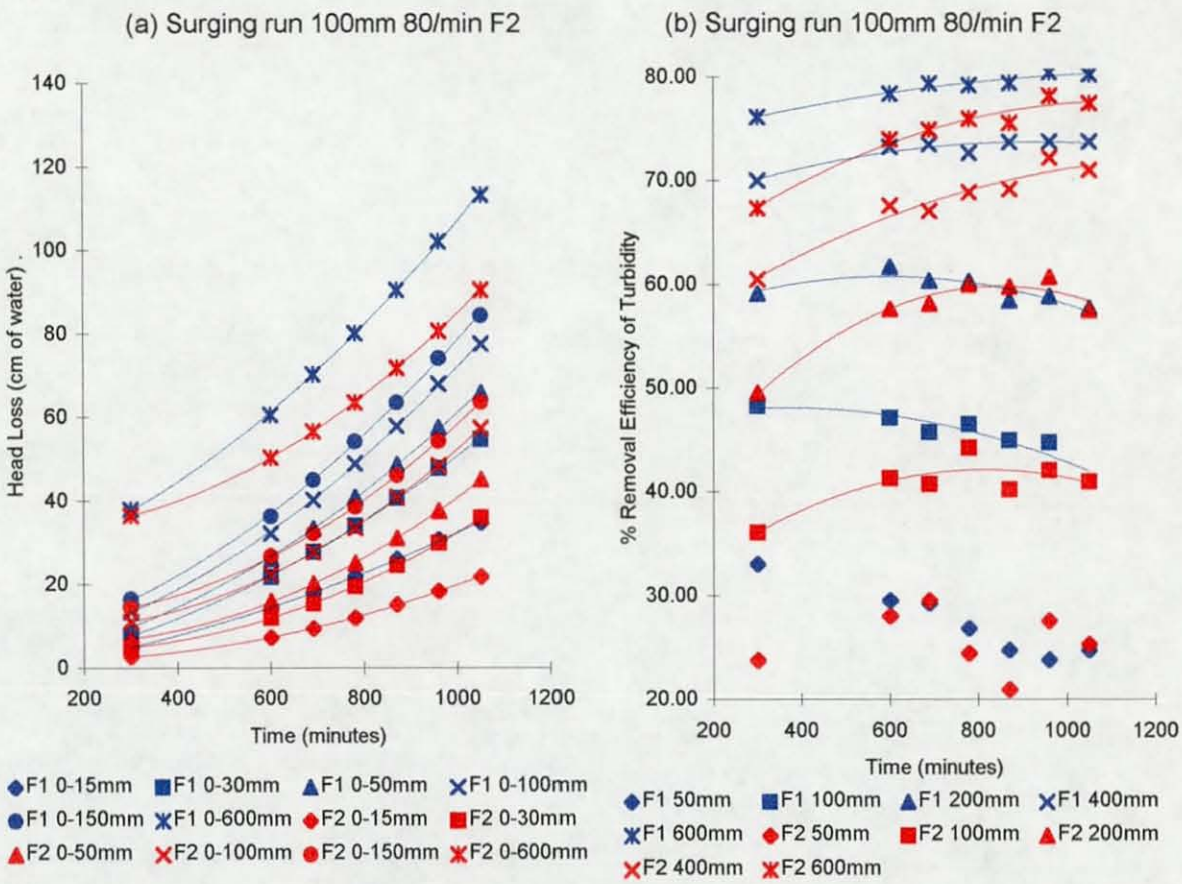


Figure 6.22 (a) & (b): Run Twenty Four Head loss & Turbidity Removal Efficiency

increased proportional to the rising head loss to maintain a fixed percentage fluctuation. These surges were applied at rates of occurrence of 10 to 40 oscillations per minute from the start of the test and applied midway through the test. However, these surges produced no clearly identifiable effect on filter performance. It was decided to adopt a different approach in run twenty four. In this test run, surges were applied to filter two from the beginning of the test at a fixed absolute amplitude of 100 mm at a rate of occurrence of 80 oscillations per minute, similar to the amplitude and frequency observed by Baylis discussed in section 2.1. Run twenty four was conducted at an influent turbidity of 51.7 NTU over 18 hours similar to run twenty three. Figure 6.22 (a) & (b) illustrate the head loss and turbidity removal efficiency trends for this test. Figure 6.37 illustrates the surge profile captured by the pressure transducer. It can be seen that the surges have significantly influenced the development of filter two with respect to control run twenty three. Filter two developed head loss more slowly than filter one throughout the depth of the media and can be seen to have lagged behind filter one in removal efficiency at each depth. To confirm this result test runs twenty three and twenty four were repeated.

Run twenty five was conducted as a repeat of control run twenty three to confirm the observed differences were reproducible. At this stage in the test programme, measurement of the dissolved solids concentrations and suspension Zeta potential were begun in addition to the conductivity and pH. Figure 6.23 (a) and (b) illustrate the head loss and turbidity removal trends from run twenty five. It can be seen that similar to control run twenty three, filter two developed head loss more rapidly than filter one throughout the filter depth with marginally greater removal in the upper layers of the media.

Run twenty six was conducted as a repeat of run twenty four to confirm that the differences observed were reproducible. Figure 6.24 (a) and (b) illustrate the head loss and turbidity removal trends from this surging test run. The filters developed more rapidly in this run than in run twenty four such that the test was terminated after 10 hours instead of 18. From table 6.3 it can be seen that the mains water conductivity was significantly lower in this run than in previous tests, confirming the suspicions mentioned earlier in this section that the mains water may be subject to fluctuations in dissolved solids content. The conductivity was measured to be 310 uS/cm, less than half the previous result. This fall in dissolved solids concentration resulted in an increase in the filter removal efficiency, shortening the test run length. The influence

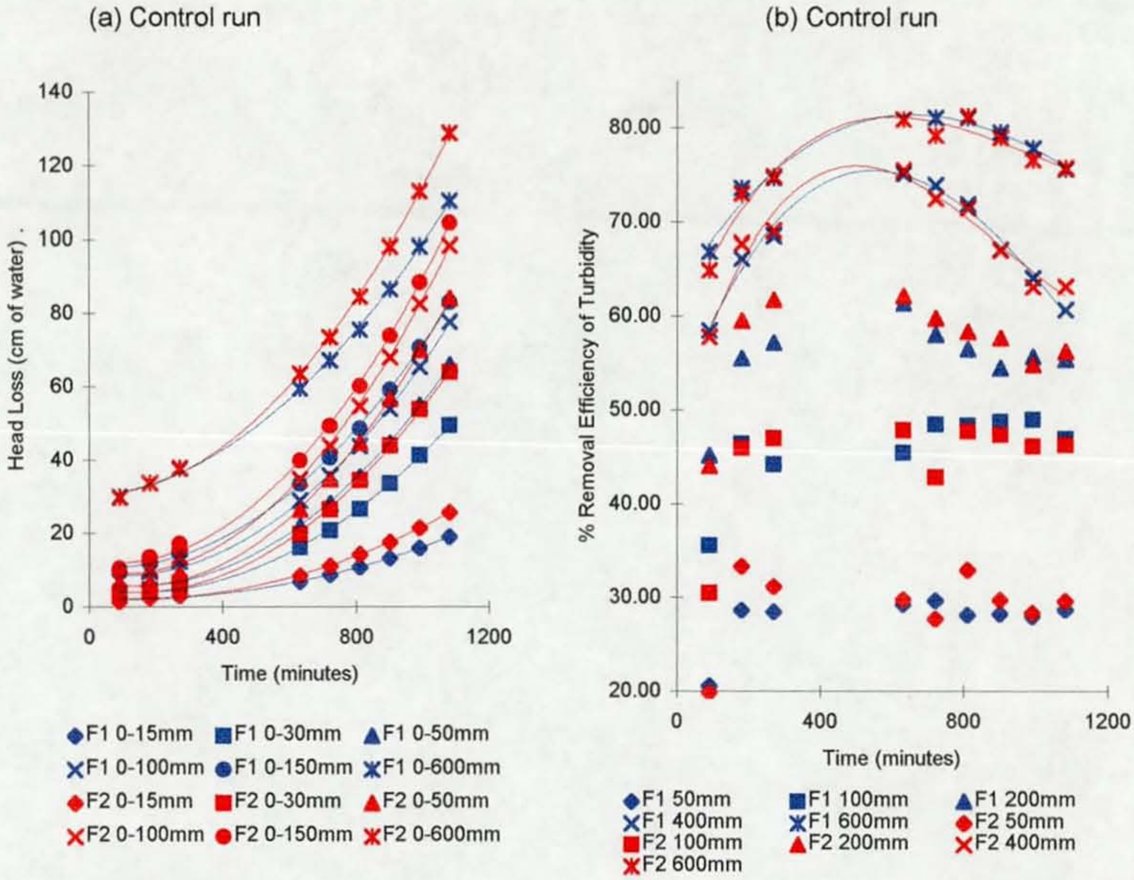


Figure 6.23 (a) & (b): Run Twenty Five Head loss & Turbidity Removal Efficiency

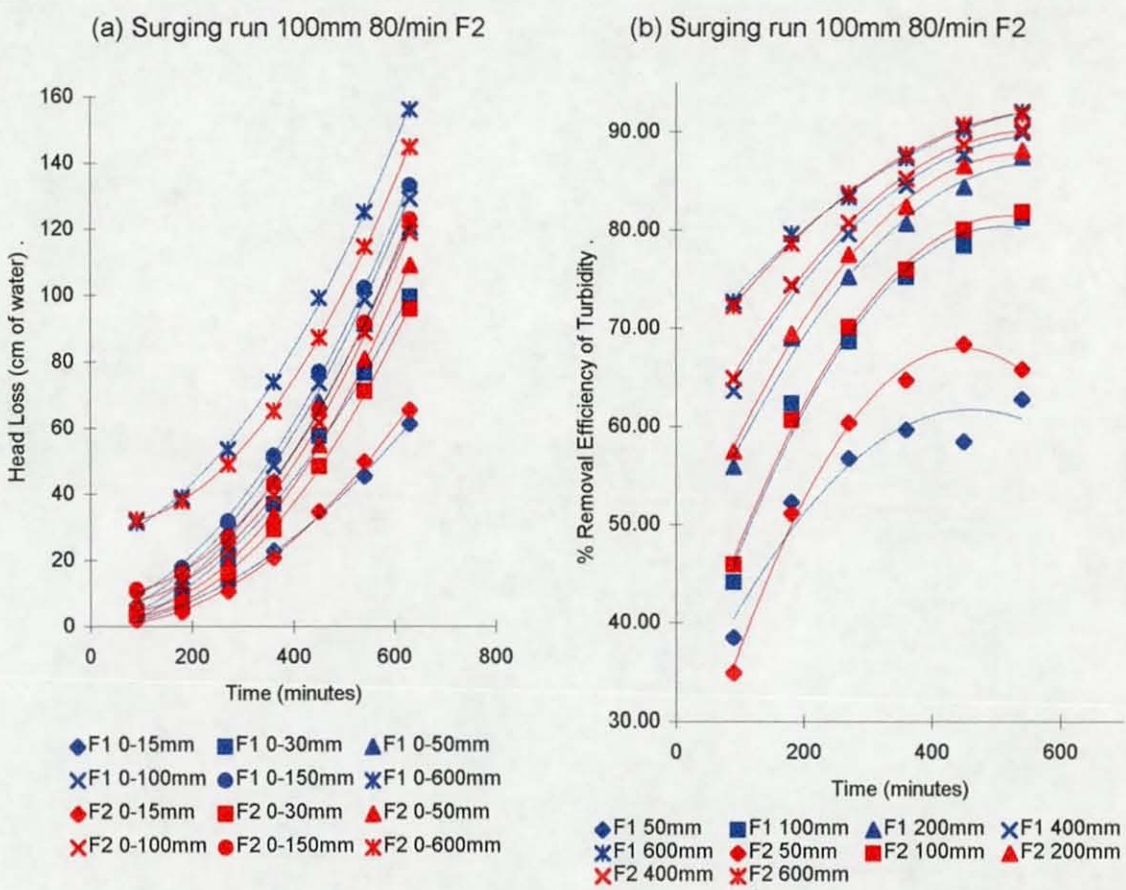
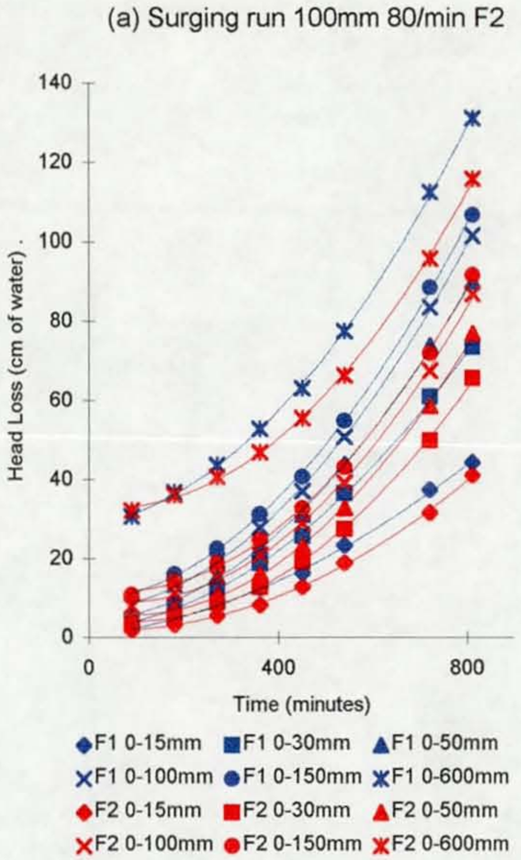


Figure 6.24 (a) & (b): Run Twenty Six Head loss & Turbidity Removal Efficiency

of the suspension chemistry will be discussed in greater depth in section 6.4 later. From the head loss and removal trends, it can be seen that filter two developed head loss more slowly than filter one similar to run twenty four but less so due to the shortened run length. From the removal trend, filter two initially achieved poorer removal efficiency than filter one in the upper 50 mm of the bed but quickly closed the gap and overtook filter two. Due to the fluctuation in the mains water conductivity, surging run twenty four was repeated again in run twenty seven. From table 6.3, it can be seen that the mains water conductivity had recovered somewhat to 635 uS/cm. Figure 6.25 (a) and (b) illustrate the head loss and turbidity removal trends from this surging test run. It can be seen that filter two developed head loss more slowly than filter one throughout the filter depth due to the surging flow, confirming the result of run twenty four. Similarly, from the removal trend, it can be seen that filter two lagged behind filter one in removal efficiency throughout the filter depth, confirming the result of run twenty four. Run twenty eight was conducted as a control test with no surges applied to filter two to ensure that the filters' performance continued to be well controlled. Figure 6.26 (a) and (b) illustrate the head loss and turbidity removal efficiency trends for this test. From table 6.3, the mains water conductivity was once again lower than normal at 512 uS/cm which resulted in a shorter filter run of 9 hours. Nevertheless, filter two developed head loss more rapidly than filter one with greater removal efficiency in the upper layers similar to previous control test runs. Having identified and confirmed clear effects from the fixed amplitude surges at 80 oscillations per minute, it was decided to apply surges proportional to the head loss once more at this rate of occurrence. From Baylis' observations discussed in section 2.1 and from this author's observations at local water works discussed in section 3.0, it was apparent that the absolute surge amplitudes did not remain fixed during full scale filter operation. From these observations, it appeared that the absolute surge magnitude increased in proportion to the rising head loss (this will be discussed further in section 6.2). Consequently, it was decided to apply surges scaled in proportion to the rising head loss in a similar manner to the observations made by Baylis and this author at a rate of occurrence of 80 oscillations per minuteRun twenty nine was conducted in this manner. Figure 6.27 (a) and (b) illustrate the head loss and turbidity removal efficiency trends from this test. Figure 6.38 (a) and (b) illustrate the surge profiles captured by the pressure transducer at the beginning and



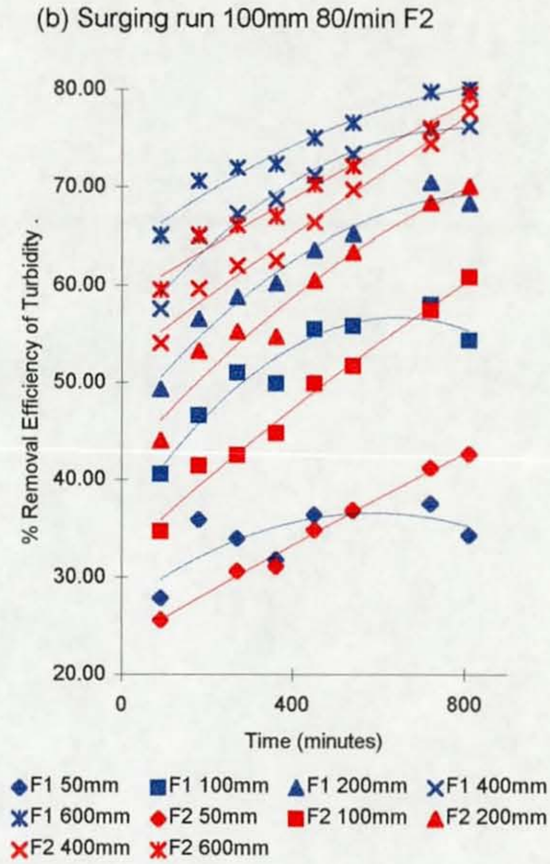
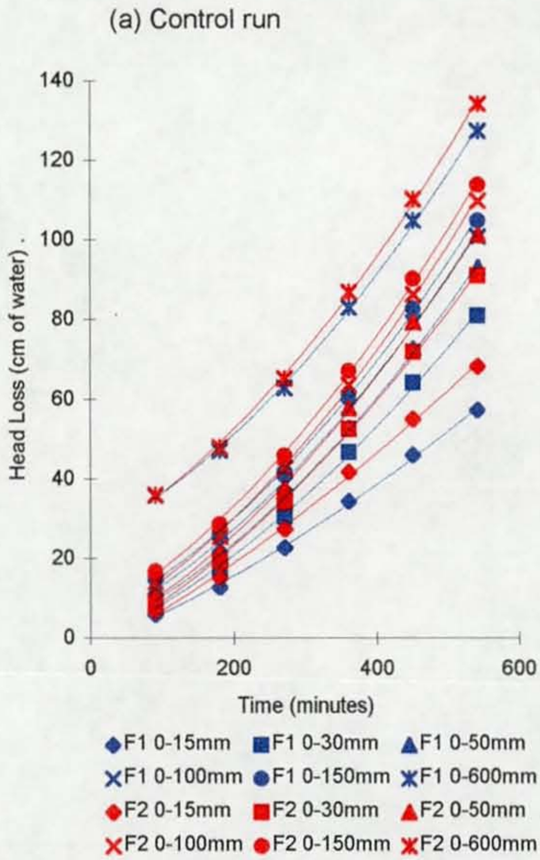


Figure 6.25 (a) & (b): Run Twenty Seven Head loss & Turbidity Removal Efficiency



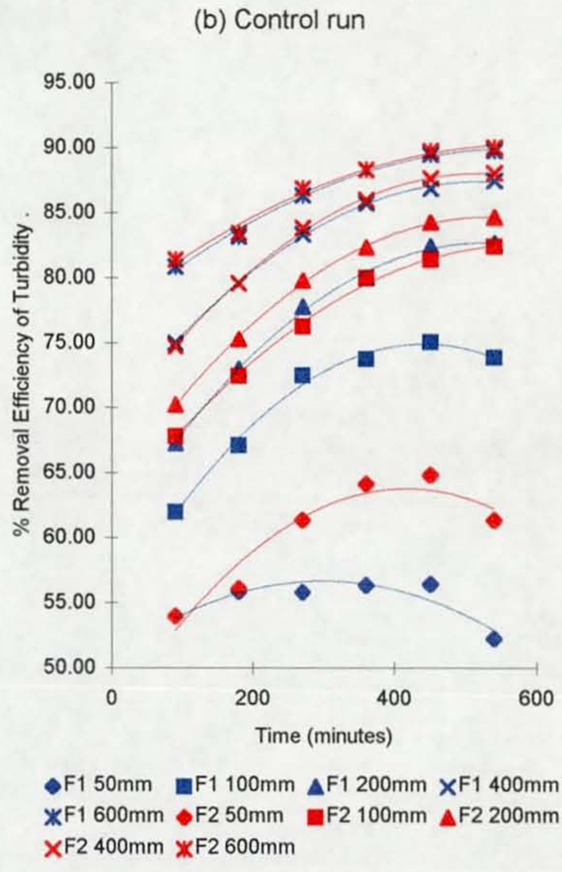


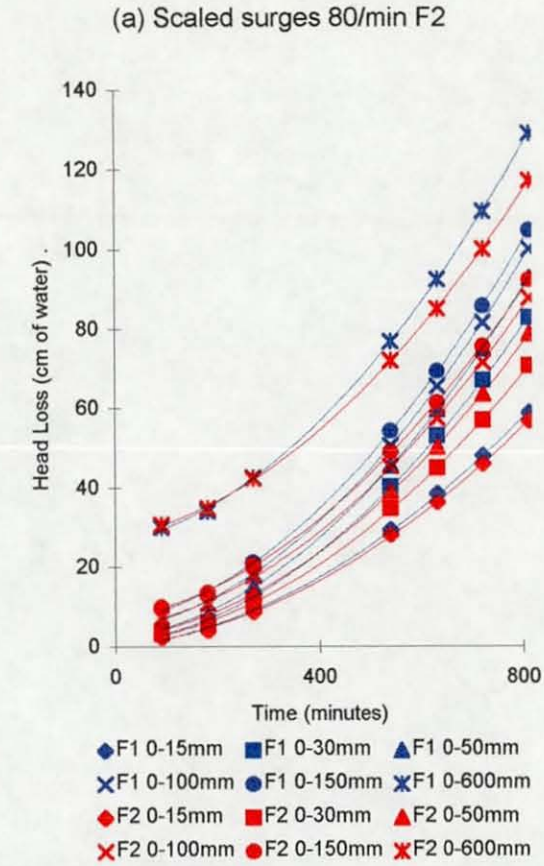
Figure 6.26 (a) & (b): Run Twenty Eight Head loss & Turbidity Removal Efficiency

end of the test. These charts illustrate the increase in absolute surge amplitude during the test. It can be seen that these scaled surges slowed the rate of head loss development and removal efficiency in filter two similar to the fixed amplitude surges of runs twenty four and twenty seven but with certain differences. These will be discussed in depth in section 6.2.

Run thirty was conducted with similar scaled surges but at a lower rate of occurrence of 40 oscillations per minute to determine what influence these would have on filter performance. Figure 6.28 (a) and (b) illustrate the head loss in cm of water and turbidity removal trends from this test. Figure 6.39 (a) and (b) illustrate the surge profiles captured by transducer in filter two at the beginning and end of the test. Here, the surges did not appear to slow the rate of head loss development like previous runs but did still inhibit the filter removal efficiency in the upper 50 mm of filter two early in the test to some degree. However, at 570 minutes, filter two bed level dropped by 1 mm which caused a distortion in the head loss results similar to run fifteen (illustrated by the dotted line). This bed movement caused filter two removal to subsequently lag behind filter one in the top 50 mm after this time. These findings are discussed in detail in section 6.2.

Run thirty one was conducted as a control test run with no surges to ensure ongoing good experimental control. Figure 6.29 (a) and (b) illustrate the head loss in cm of water and turbidity removal trends for this test. As in previous control runs, filter two developed head loss more rapidly that filter one throughout the filter depth with greater removal in the upper layers of the media.

Run thirty two was conducted with fixed amplitude surges of 100 mm at 80 oscillations per minute in filter two; a repeat of runs twenty four and twenty seven for final confirmation of the effect of these surges on the filter performance. In this run, the pressure ports connected to both the filter columns within the filter media were isolated from the piezometer tubes by closing the stopcocks. This test was conducted to ensure that the oscillating water levels in the piezometers in filter two during surging tests was not a source of interference. Figures 6.30 (a) and (b) illustrate the head loss in cm of water and turbidity removal efficiency trends from this test. It can be seen that filter two developed head loss more slowly than filter one with poorer removal efficiency in the upper layers of the media as in the previous fixed amplitude surging tests confirming that the pressure ports were not a source of interference. At this point, the test programme was terminated.



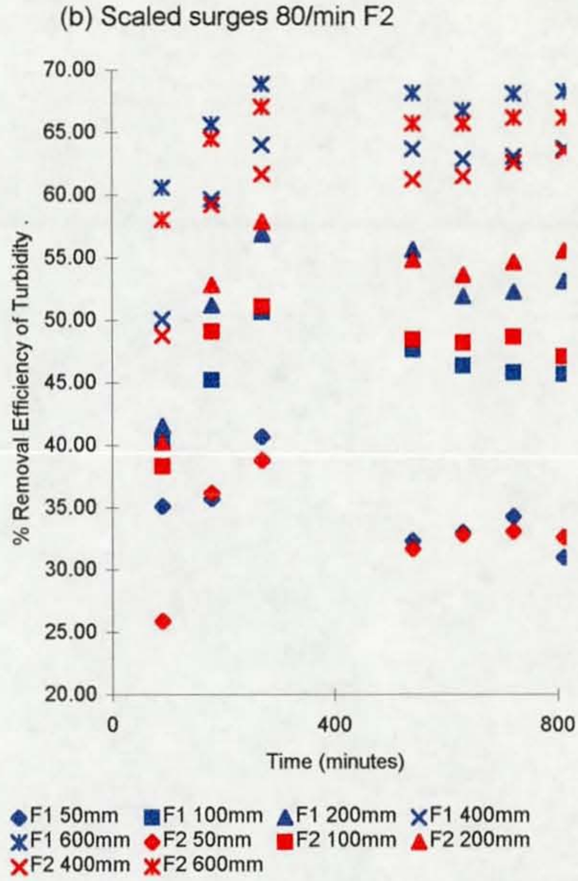
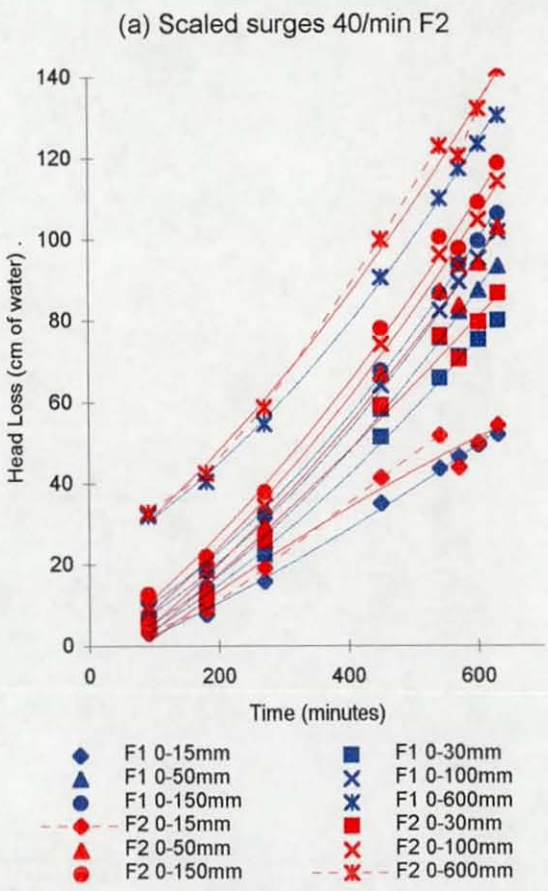


Figure 6.27 (a) & (b): Run Twenty Nine Head loss & Turbidity Removal Efficiency



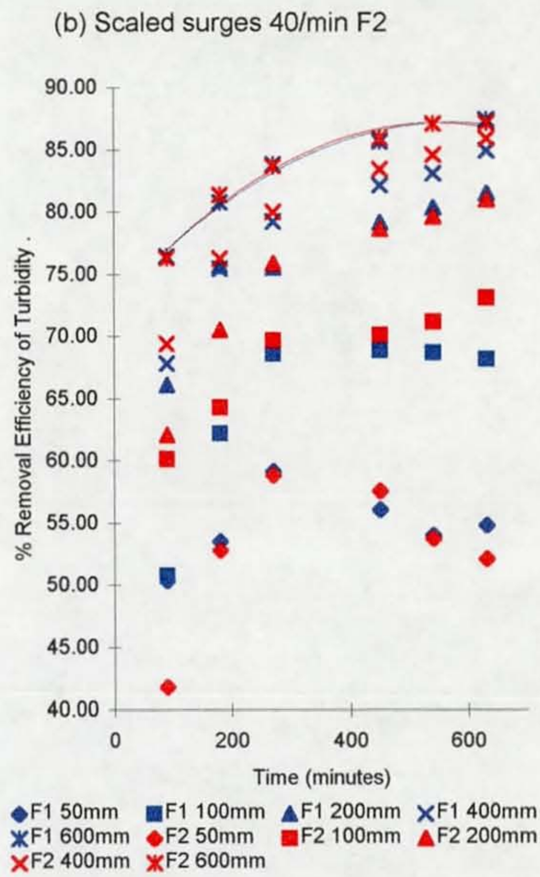


Figure 6.28 (a) & (b): Run Thirty Head loss & Turbidity Removal Efficiency

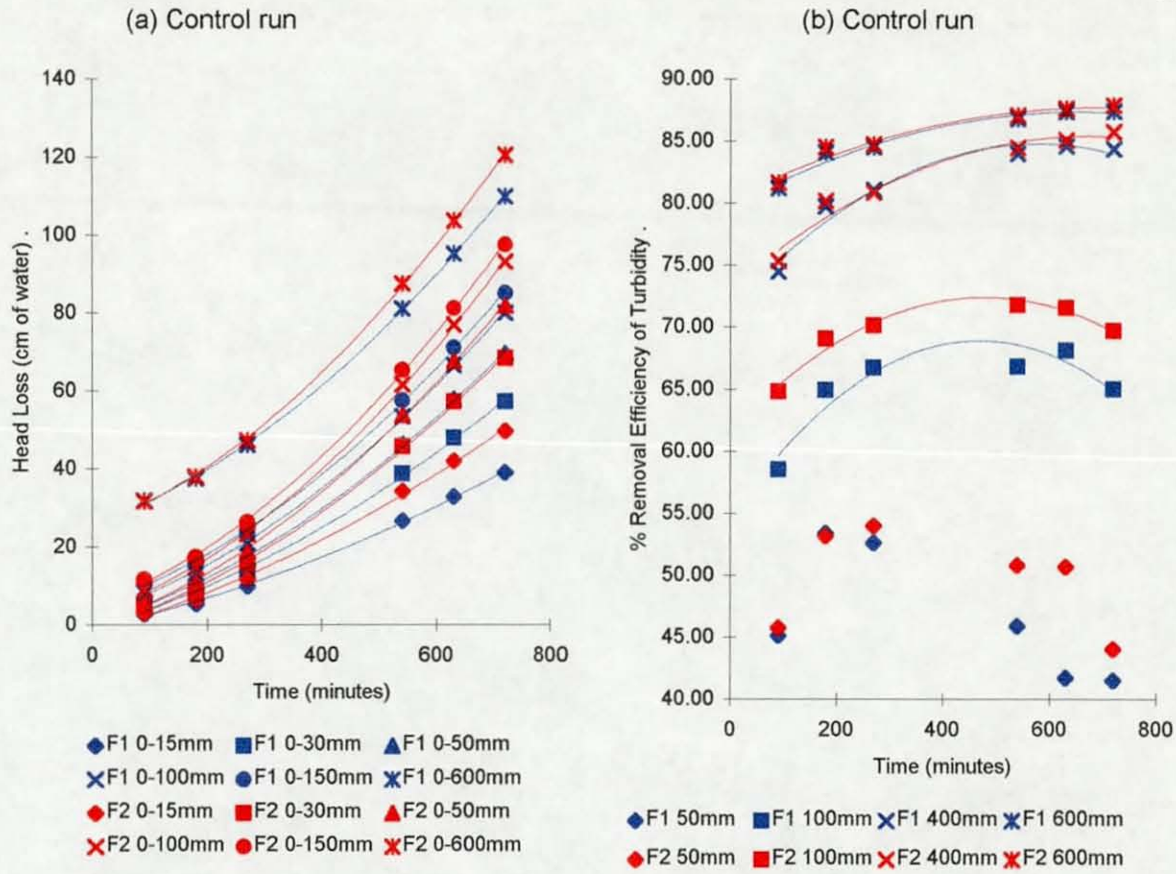


Figure 6.29 (a) & (b): Run Thirty One Head loss & Turbidity Removal Efficiency

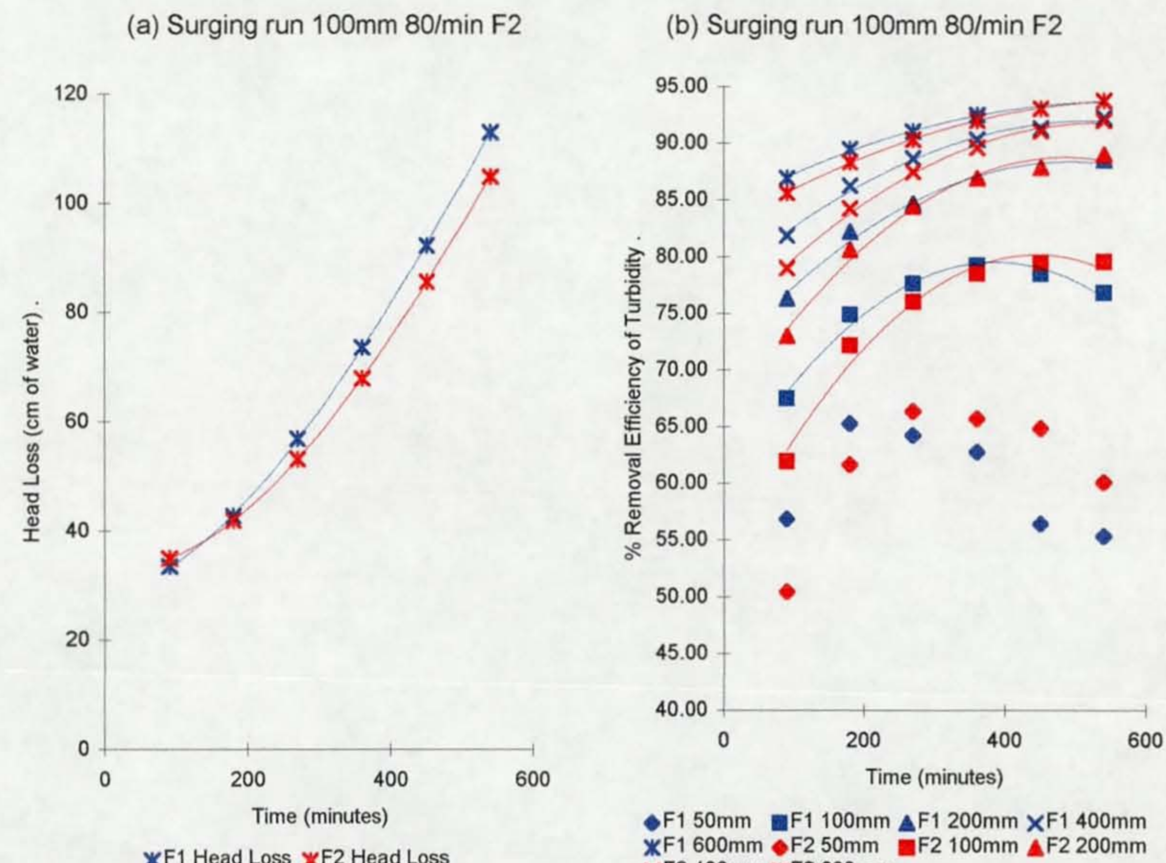


Figure 6.30 (a) & (b): Run Thirty Two Head loss & Turbidity Removal Efficiency

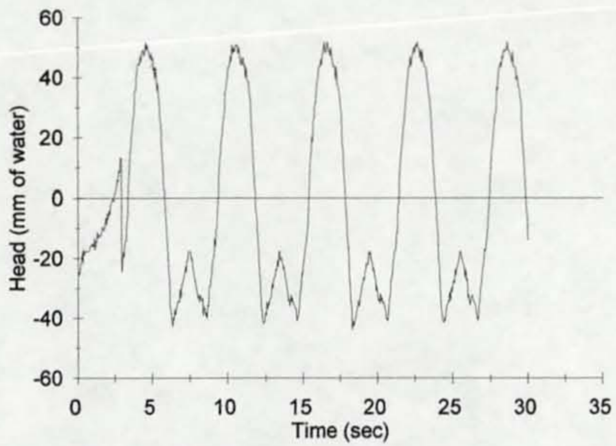


Figure 6.31: Run 10 Surges

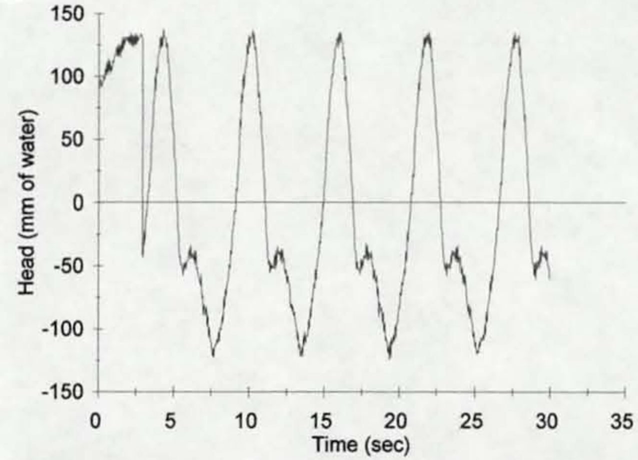


Figure 6.32: Run 11 Surges 550 minutes

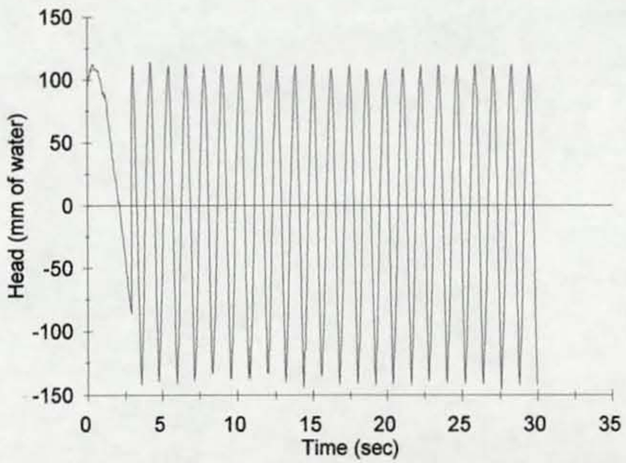


Figure 6.33: Run 13 Surges 600 Minutes

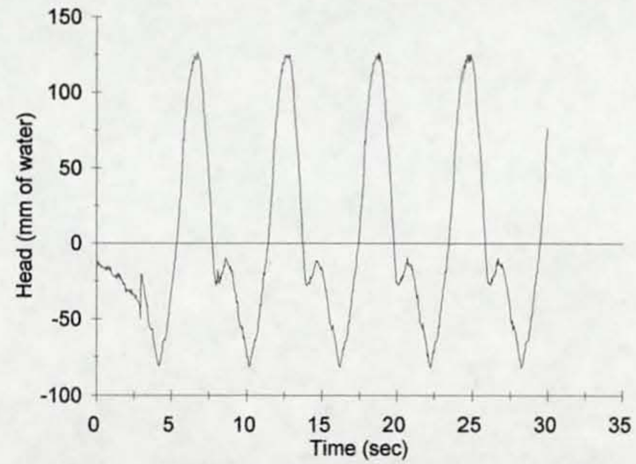


Figure 6.34: Run 17 Surges 435 Minutes

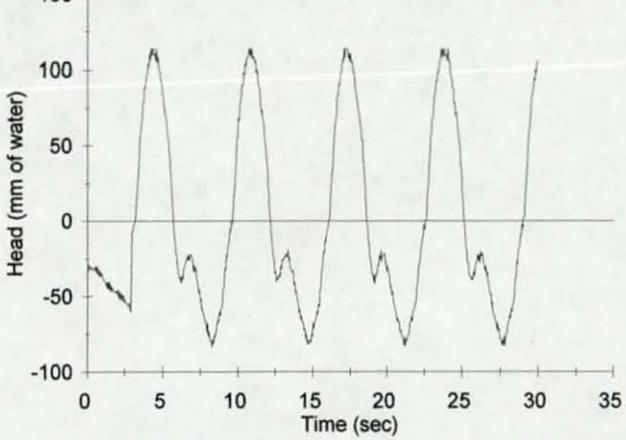


Figure 6.35: Run 21 Surges 1130 minutes

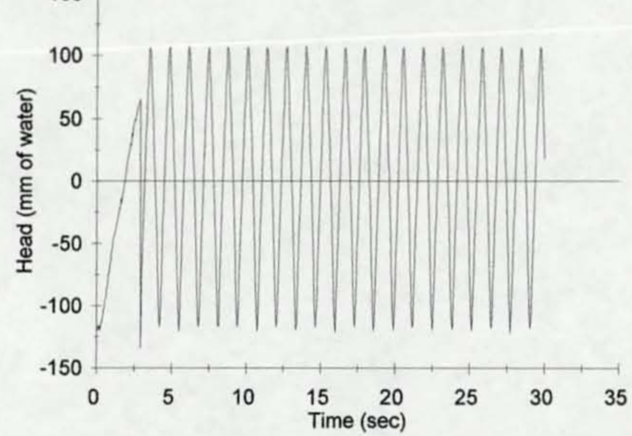


Figure 6.36: Surges Run 22 1050 Minutes

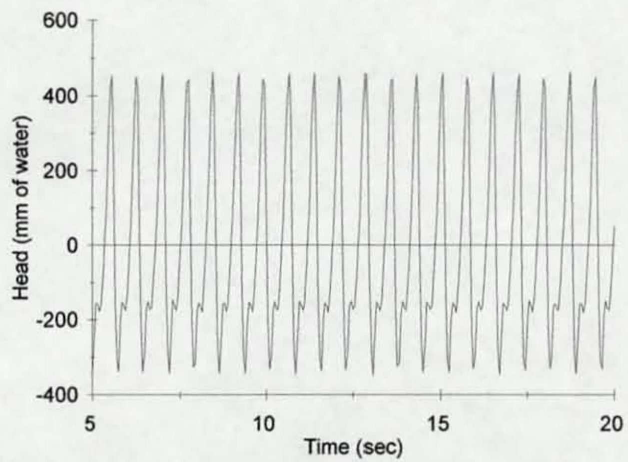


Figure 6.37: Surges Run 24

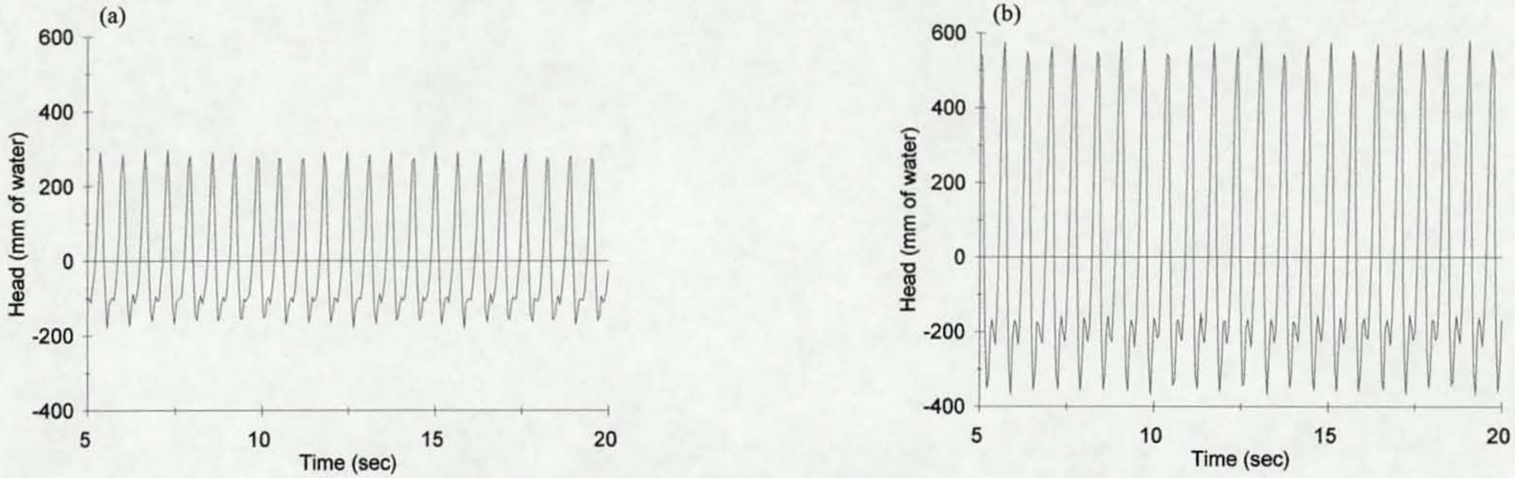


Figure 6.38 (a) & (b): Surges Run 29 Beginning and End

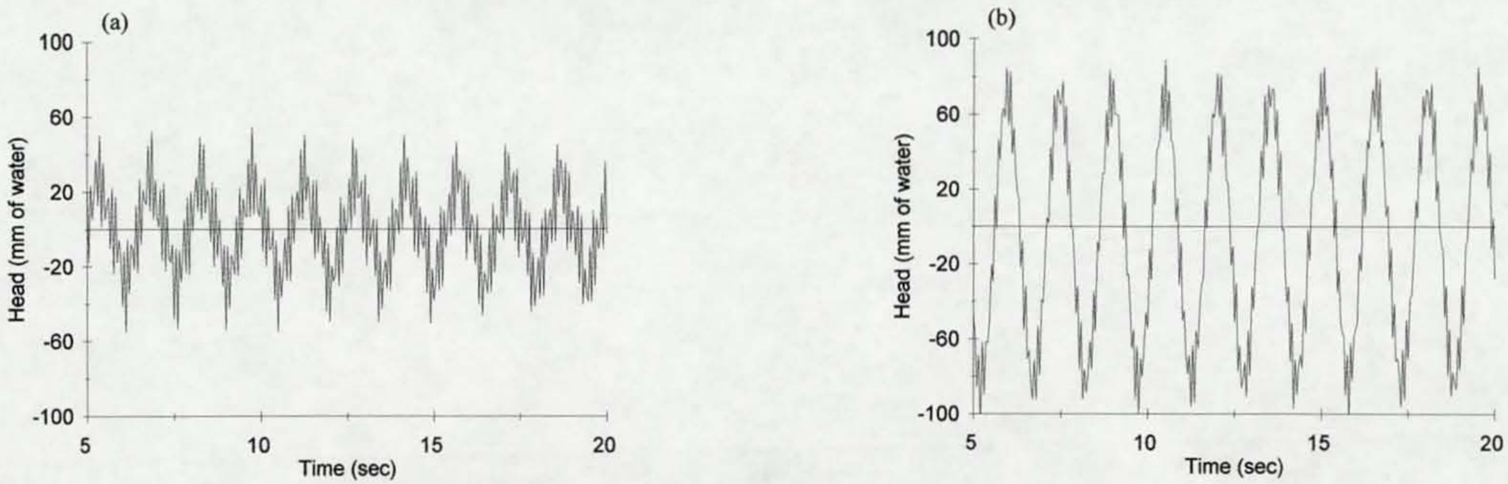


Figure 6.39 (a) & (b): Surges Run 30 Beginning and End

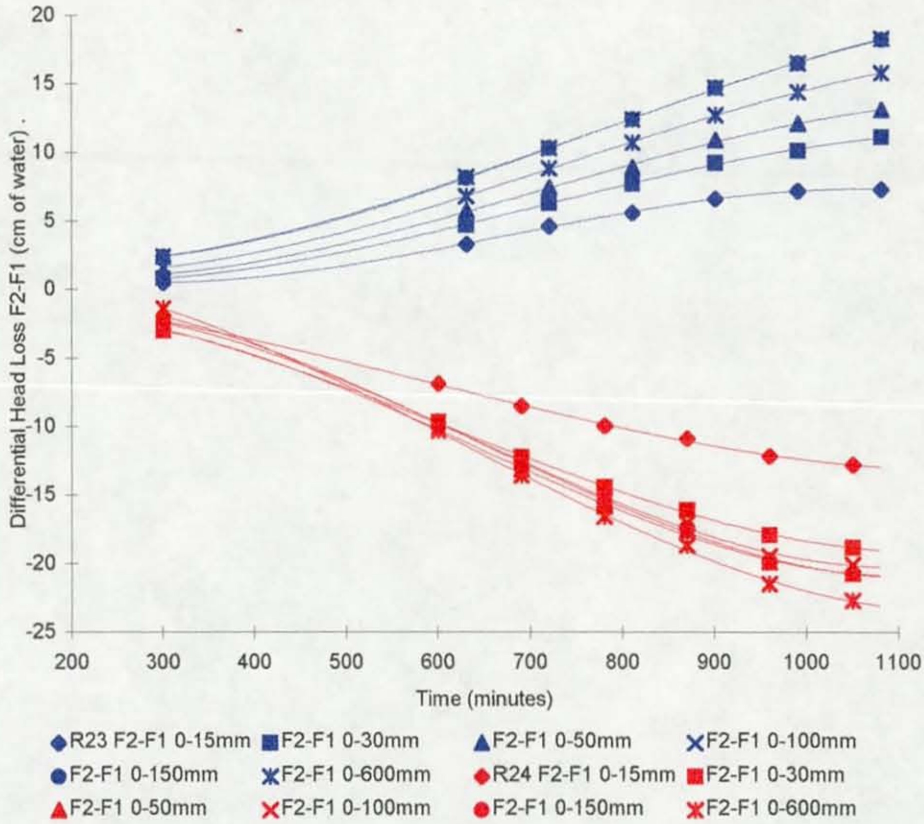
6.2 Effect of Surges on Filter Performance

In section 6.1, the experimental test programme was presented in chronological order with key events and findings highlighted to place the results in context. In this section the effect of the surges on filter performance will be analysed in greater depth and discussed with respect to the findings of previous research reviewed in chapter 2 and this author's observations at local water works discussed in chapter 3. The discussion will focus upon the results of tests twenty three onward. The previous test runs were used to establish the operating conditions and the design of the apparatus necessary to achieve detailed and reproducible results. Surging runs conducted during this phase of the test programme did not produce any clear and unambiguous effect on filter performance due firstly to the poorer test control at that time and secondly due to the nature of the surges applied. Surging applied in these earlier runs was 10 to 20 % of the head loss in amplitude at rates of 10 to 50 oscillations per minute, applied from the start of the test and midway through the test. These characteristics did not have any clear influence on the filters. Clear and reproducible effects were identified from runs twenty three onwards and it is proposed to analyse and discuss these results in detail.

6.2.1 Fixed Amplitude Surging Effects

The experimental work of Baylis (1958) and Hudson (1959), discussed in section 2.1.2, found that surge amplitudes increased in rapid gravity filters with rising loss of head and noted that these surges occurred up to 100 oscillations per minute (Cleasby *et al*, 1963). Subsequently, surges were applied in proportion to the rising head loss in early tests in this experimental programme in a similar manner. Surges at amplitudes of 10 to 20 % of the head loss were applied at rates of 10 to 50 oscillations per minute with no apparent effect on the performance of filter two with respect to filter one. Since these variable amplitude surges at lower frequencies did not produce any clear result, it was decided to apply surges at a fixed absolute amplitude of 100 mm throughout the test at a higher rate of occurrence of 80 oscillations per minute. An amplitude 100 mm was chosen since the observations of Baylis, discussed in section 2.1.3, found surges up to this magnitude at full scale plant. In addition, observations conducted at local water plants by this author, discussed in chapter 3, found surge amplitudes and rates of occurrence of this nature. It should also be noted that the experimental test runs in this study were conducted at 30 °C; higher than found at full scale plant. As discussed in section 5.1.1, settled water temperatures applied to full scale filters can fluctuate from 20 down to 5 °C from summer to winter at full scale works. At these lower temperatures the effect of a given magnitude of surging was expected to be more significant as a result of the increased shear stresses acting on retained deposits within the filter pores. In this light, fixed surges of 100 mm magnitude at 80 oscillations per minute at an operating temperature of 30 °C were considered representative to begin with. 100 mm amplitude surges were applied to filter two at a rate of occurrence of 80 oscillations per minute in test runs twenty four, twenty seven and thirty two. These results were compared with control runs twenty three, twenty five and thirty one and are discussed below.Figure 6.40 (a) and (b) and 6.41 (a) and (b) compare the head loss and turbidity removal efficiency results respectively for control run twenty three and surging run twenty four. In these charts the results have been presented as the differential head loss and removal efficiency to highlight the surge effects more clearly than the absolute plots presented in section 6.1. Differential head loss and removal efficiency were calculated at each time step and sample point within the filter depths by simply subtracting the results of filter one from filter two. These charts therefore illustrate the relative performance of filters one and two in the control and surging tests. Figure 6.40 (a) and 6.41 (a) illustrate the differential head loss and turbidity removal efficiency respectively against time at each sample port within the filter depths. Figure 6.40 (b) and 6.41 (b) illustrate the differential head loss and turbidity removal efficiency respectively against depth from the surface of the sand at each sample time during the filter test run. For additional clarity, the differential removal efficiency plots against time and against depth have been generated individually for each sample depth and selected sample times in figures 6.42 (a) to (c) and 6.43 (a) to (d) respectively.

From figure 6.40 (a) and (b), it can be seen that filter two developed head loss more rapidly that filter one in control run twenty three at each depth from 0 to 100 mm from the sand surface with increasing time. From 150 to 600 mm depth filter one reduced the deficit by a small margin compared with the upper 150 mm but still lagged increasingly behind with time. By the end of the test filter two led filter one by 150 mm of head loss at 600 mm depth. However, in surging run twenty four, filter



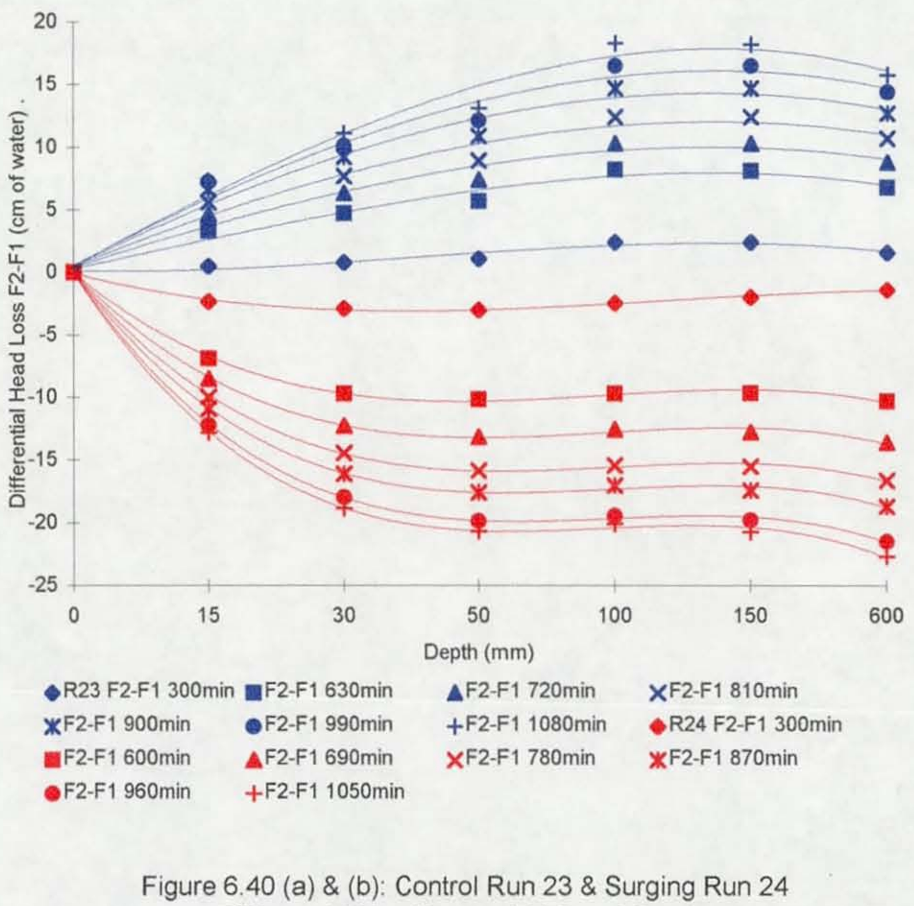
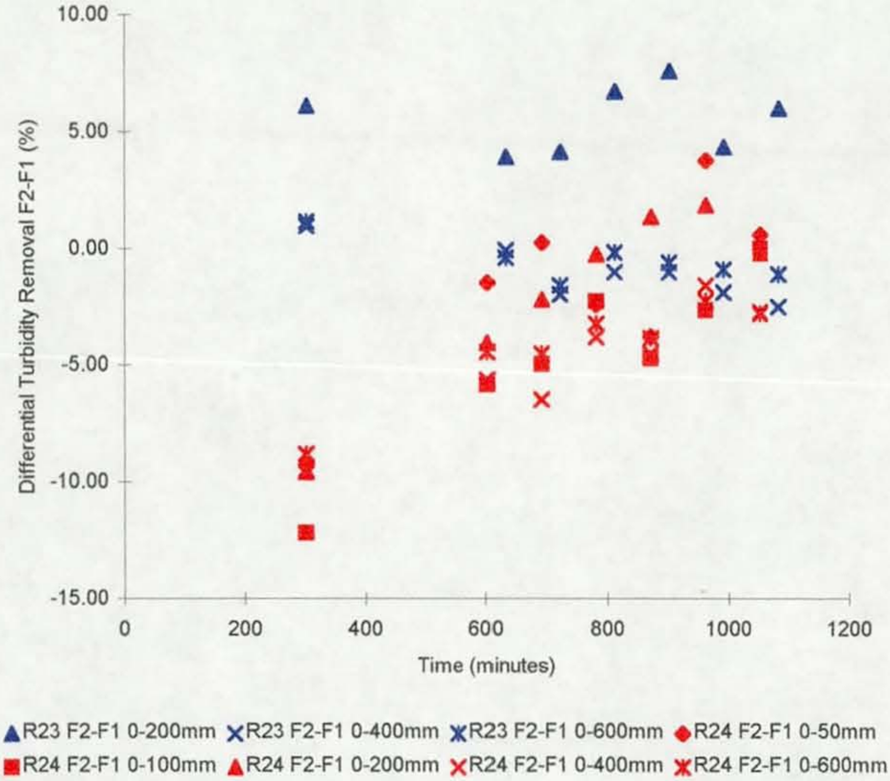


Figure 6.40 (a) & (b): Control Run 23 & Surging Run 24
Differential Head Loss with Time & Depth



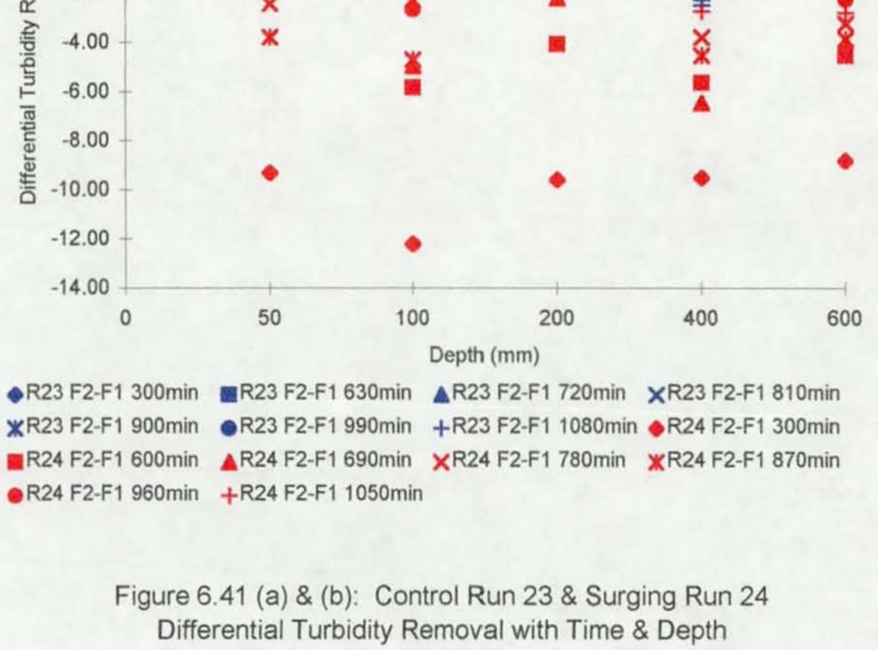


Figure 6.41 (a) & (b): Control Run 23 & Surging Run 24
Differential Turbidity Removal with Time & Depth

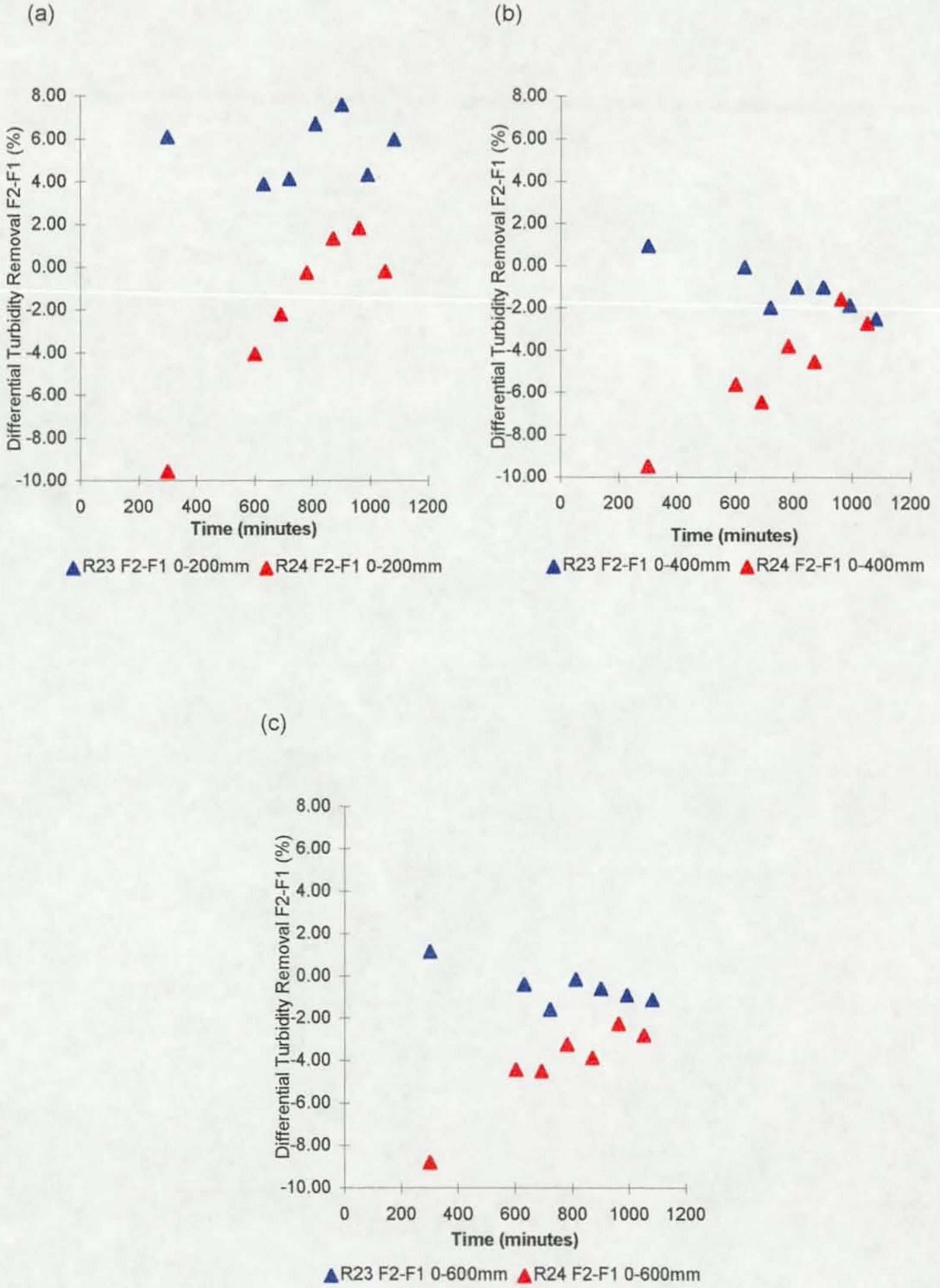


Figure 6.42 (a) to (c): Control Run 23 & Surging Run 24
Differential Turbidity Removal Charts at each Depth

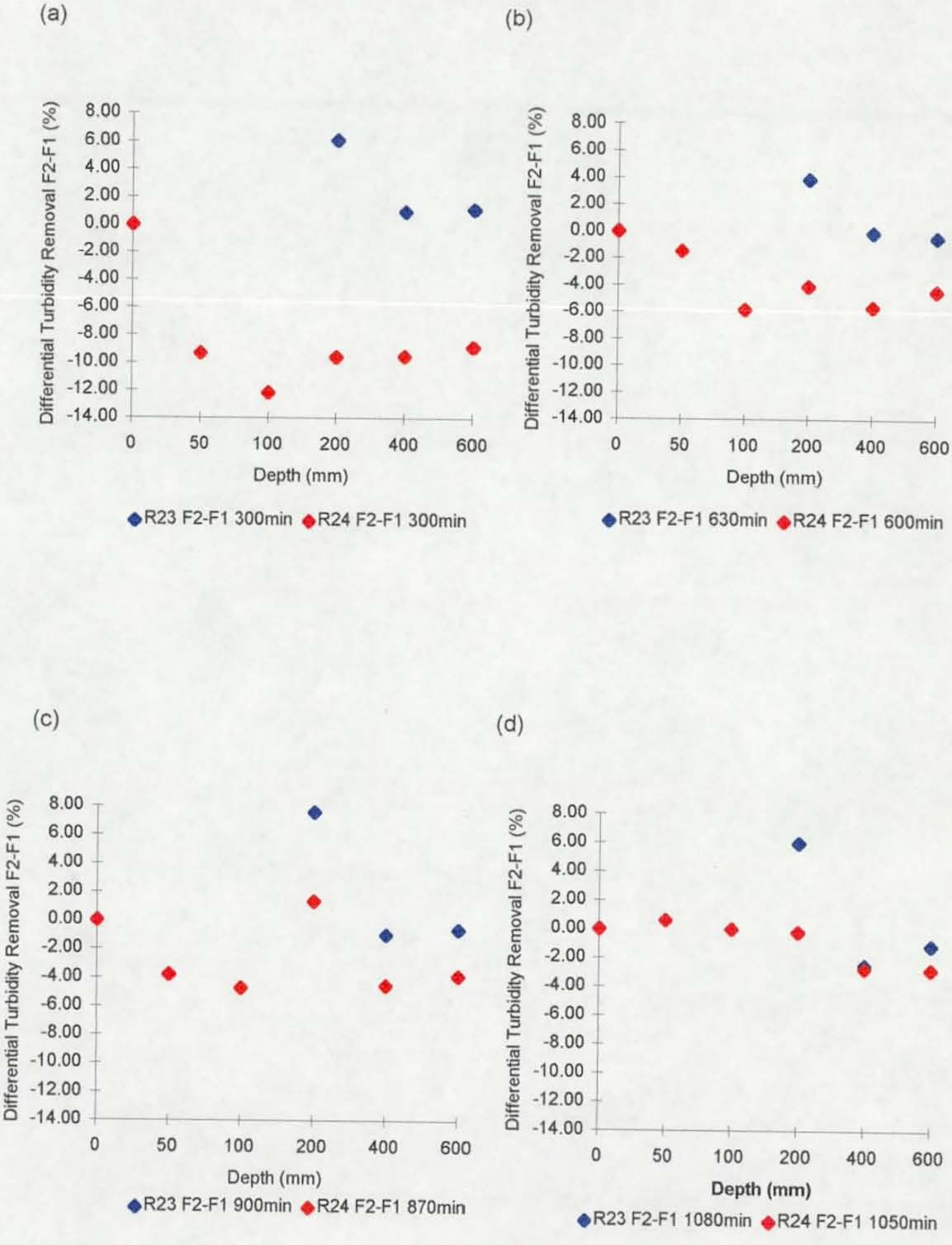


Figure 6.43 (a) to (d): Control Run 23 & Surging Run 24
Differential Turbidity Removal Charts at Selected Times

two lagged behind filter one in development of head loss from 0 to 50 mm from the surface and did so increasingly with time. From 50 to 150 mm the deficit remained constant but increased marginally from 150 to 600 mm. However, the deficit continued to increase with time from 50 to 600 mm. By the end of run twenty four, filter two lagged behind filter one by some 230 mm at 600 mm depth. Compared to filter two performance in control run twenty three, the 100 mm surges at 80 oscillations per minute caused a lag of almost 400 mm in head loss development over the 600 mm of filter sand in filter two.

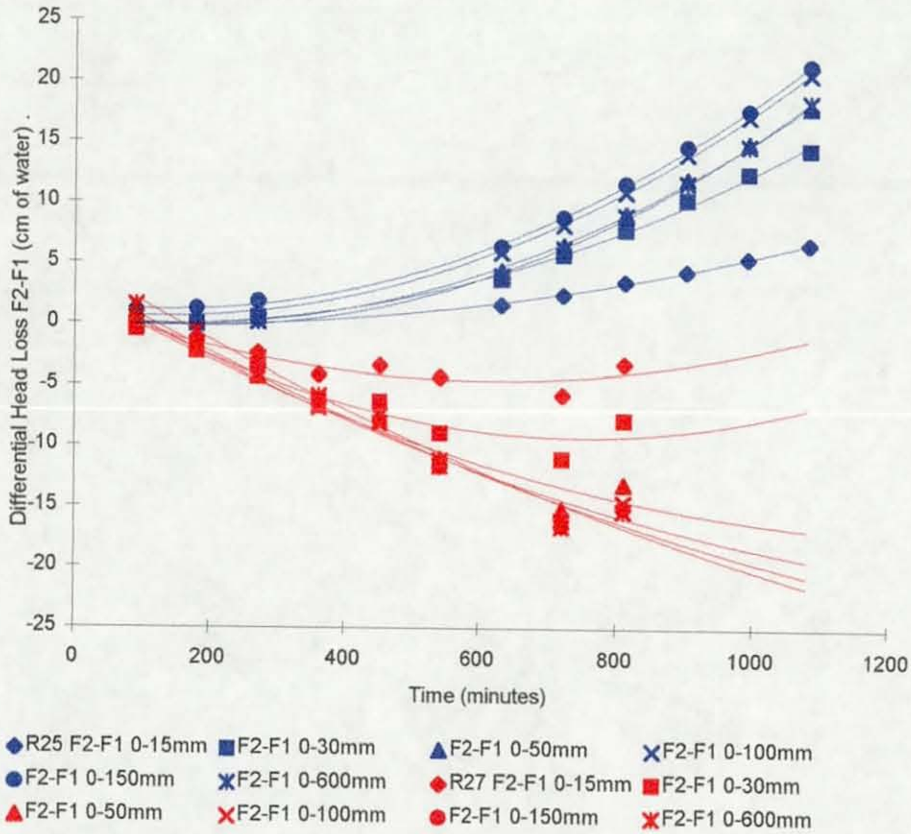
From figure 6.41 (a) and (b), filter two removed turbidity more effectively than filter one by some 5% in the top 200 mm of sand throughout control run twenty three. This removal correlated with the more rapid head loss development in the upper layers of filter two as described above. From 200 to 600 mm depth, filter one recovered the deficit such that the final filtrate quality was of similar turbidity from each filter. This pattern was apparent throughout test run twenty three. In surging run twenty four, filter two produced poorer quality filtrate at each depth 300 minutes into the test. The difference in removal was 10 to 15 % throughout the filter depth at this time compared with control run twenty three. However, with increasing time filter two removal efficiency improved such that at the end of the test the deficit had been reduced to almost zero at 600 mm depth. However, the deficit remained around 5% at 200 mm depth at the end of the test compared with the previous run. The presence of the fixed amplitude surges of 100 mm at 80 oscillations per minute caused poorer removal efficiency in filter two, especially in the first 5 to 10 hours of the test. The difference in head loss occurred across the top 50 mm of the filter which correlated with the large deficit in removal efficiency which remained throughout the test in the upper regions of the filter. The deficit in removal efficiency was largest in the first 5 to 10 hours of the test and declined towards the end of the run. The deficit in head loss increased initially due to the poorer removal but tended towards a constant lag as the deficit in removal efficiency reduced. The reason for this early large deficit in removal efficiency and subsequent recovery is explained by table 6.4 below.

Time (minutes)	*Amplitude at 50 mm (mm)*	*(% HL)*	*Amplitude at 150 mm (mm)*	*(% HL)*	*Amplitude at 600 mm (mm)*	*(% HL)*
300	2.0	2.8	4.0	2.8	100	27.0
600	3.0	1.9	5.0	1.8	100	19.9
780	3.0	1.2	6.0	1.6	100	15.7
1050	4.0	0.9	7.0	1.1	100	10.5

Table 6.4: Absolute & Relative Surge Amplitude in Run 24 Filter Two
(HL - Head Loss)

Here, the surge amplitudes measured in filter two during run twenty four are presented both in absolute terms and as a proportion of the head loss at selected depths throughout the test. Since the surge amplitude was held fixed at 100 mm the relative surge amplitude expressed as a fraction of the head loss declined as the head loss rose with time. At 600 mm depth, the surge amplitude declined from 27 to 10% of the loss of head. The relative surge amplitudes are also presented at depths of 50 and 150 mm. At 50 mm, the absolute surge amplitude rose from 2 to 4 mm as the run progressed. In relative terms, the surge declined from 2.8 to 0.9 %. Similarly, at 150 mm depth, the absolute surge amplitude rose from 4 to 7 mm but declined from 2.8 to 1.1 % of the head loss with time. It can be seen that for fixed absolute amplitude surging applied to the base of filter two, the absolute surge amplitude within the filter bed slowly rose as the loss of head increased but at a rate less than the rising head loss. Since the flow within the filter bed is laminar, the fluctuation in head loss is taken to reflect directly a fluctuation in the flow rate (Hudson, 1959 discussed in section 2.1.4). In this case, the fluctuation in flow rate within the filter media at depths of 50 and 150 mm was observed to decline with time. This explains the early large deficit in removal efficiency in the upper sections of the filter bed. The solids removal had been inhibited by the fluctuating flow rate but the effect declined with time since the observed fluctuation declined with rising loss of head.

Runs twenty five and twenty seven were conducted as control and fixed surge amplitude test runs to confirm the findings of runs twenty three and twenty four discussed above. No surges were applied in run twenty five to confirm the control performance of filters one and two. 100 mm surges were applied to filter two at a rate of 80 oscillations per minute in run twenty seven to confirm the reproducibility of the outcome of run twenty four. Since the effect of the fixed amplitude surging was greater in the earlier stages of the test, measurements of head loss and turbidity were taken at 90 minute intervals from the beginning of the tests in these runs for greater detail. Figure 6.44 (a) and (b) present the differential head loss results against time and depth from these two runs. In control run twenty five, filter two developed head loss more rapidly that filter one throughout the depth in a similar manner to run twenty three. In surging run twenty seven, filter two developed head loss more slowly than filter one in a similar manner to run twenty four. Figure 6.45 (a) and (b) illustrate the differential turbidity removal efficiency results of these two runs at each depth in the filter media with time and at each time step with depth. For additional clarity, these curves have been plotted individually in figures 6.46 (a) to (e) and figures 6.47 (a) to (d). Similar to runs twenty three and twenty four, the effect of the surges applied was to inhibit the removal efficiency of turbidity throughout the filter depth early in the test run. The deficit in removal was some 6 to 10% in filter two compared with the control performance throughout the filter depth 270 minutes into the test. With increasing time, the deficit was seen to shrink at each depth as filter two recovered. At 50 mm depth, the deficit in removal was reduced to zero by 600 minutes and subsequently appeared to improve on the control performance. A similar pattern was observed at greater depths but with a longer time interval before filter two caught up with the control performance. By the end of the tests, filter two produced similar quality filtrate as the control performance at 600 mm depth and indeed overtook the control quality at depths of 50 to 100 mm. Table 6.5 below illustrates the absolute and relative surge amplitudes in run twenty seven at selected depths during the test.



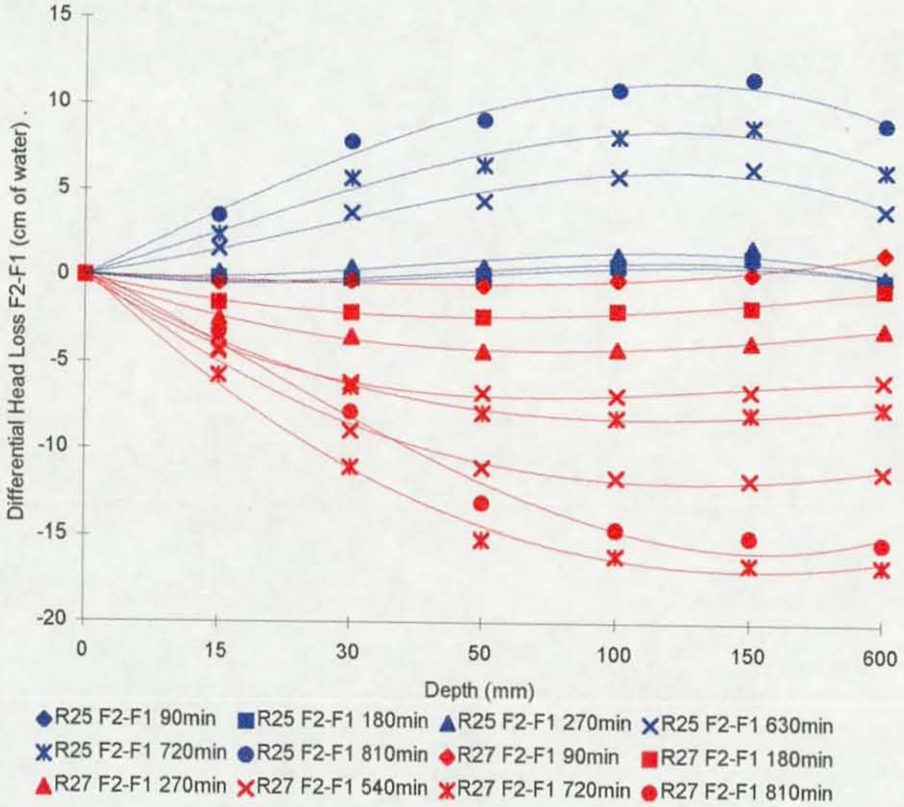
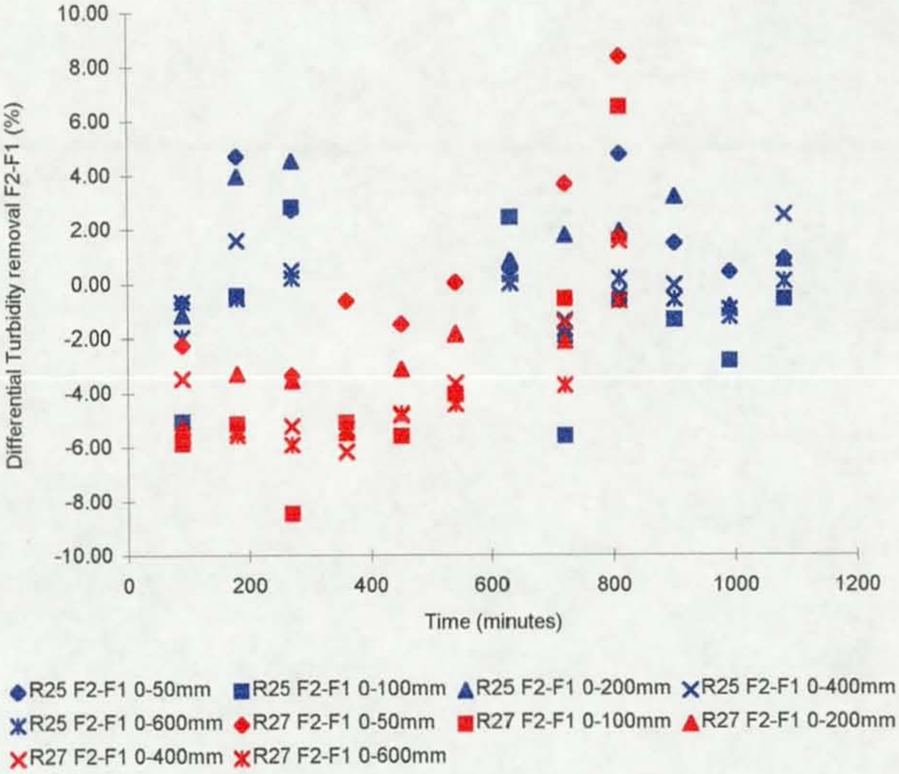


Figure 6.44 (a) & (b): Control Run 25 & Surging Run 27
Differential Head Loss with Time & Depth



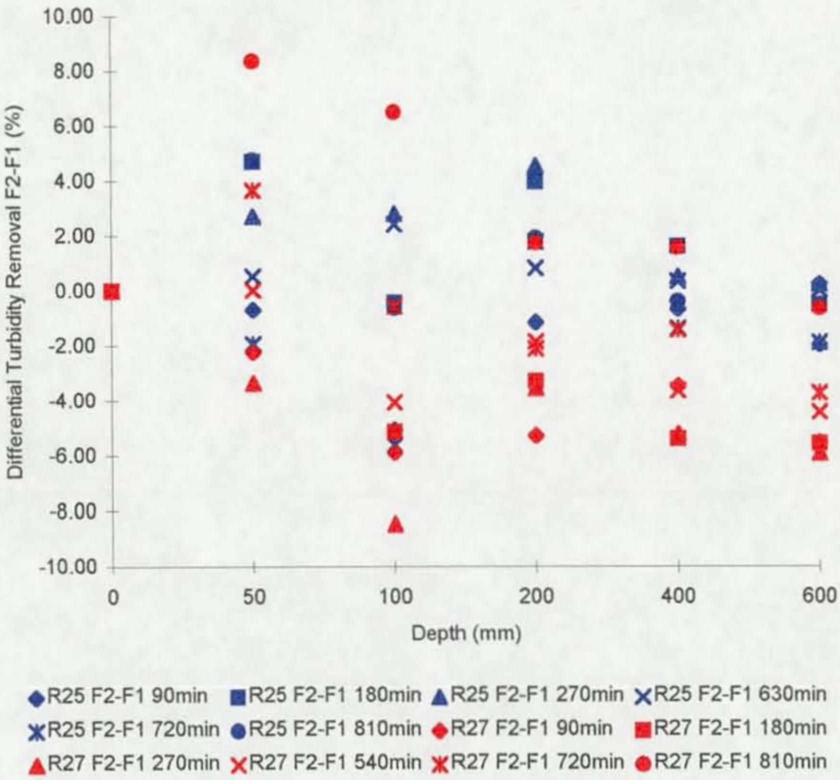


Figure 6.45 (a) & (b): Control Run 25 & Surging Run 27
Differential Turbidity Removal with Time & Depth

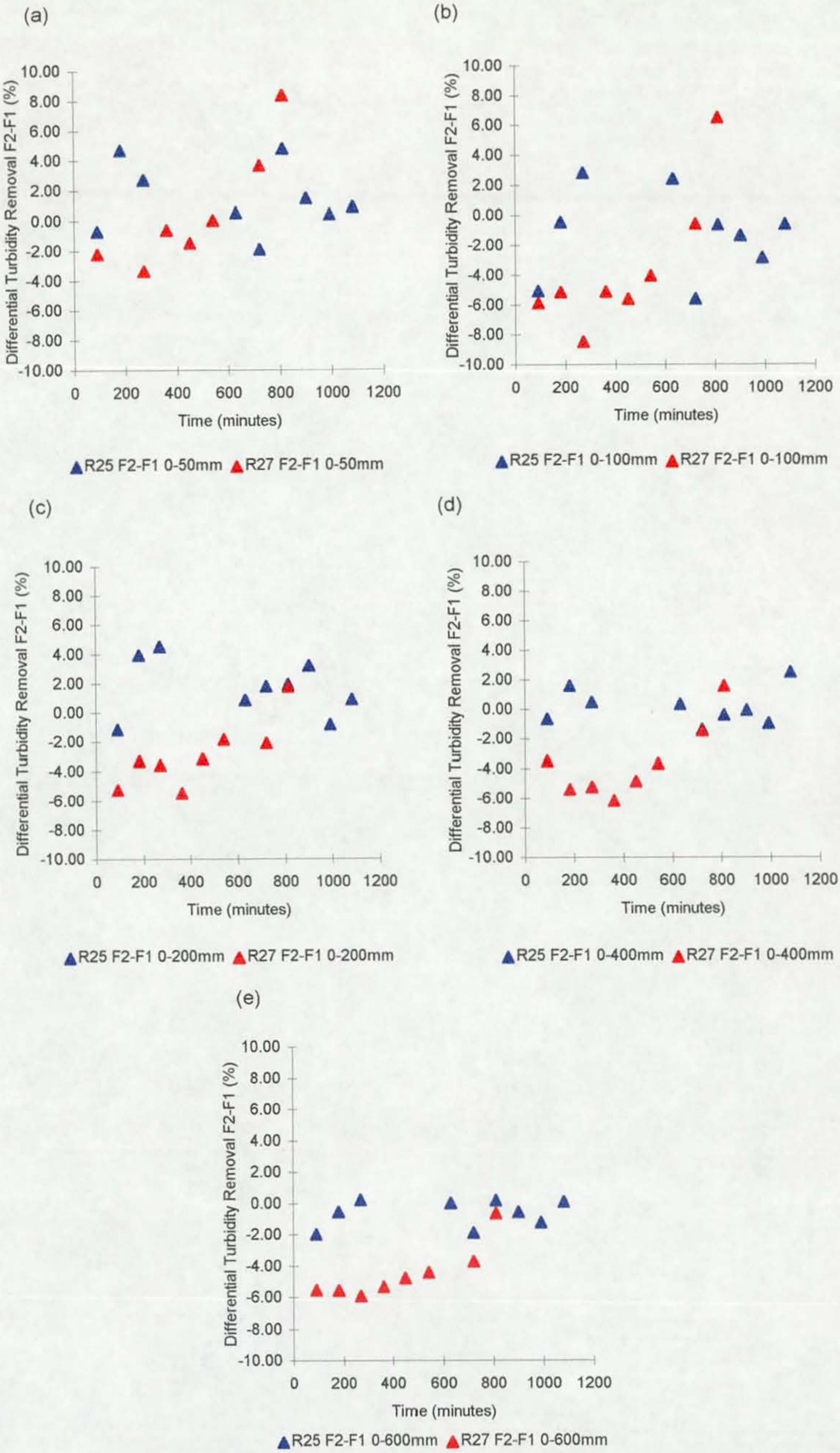


Figure 6.46 (a) to (e): Control Run 25 & Surging Run 27
Differential Turbidity Removal Charts at each Depth

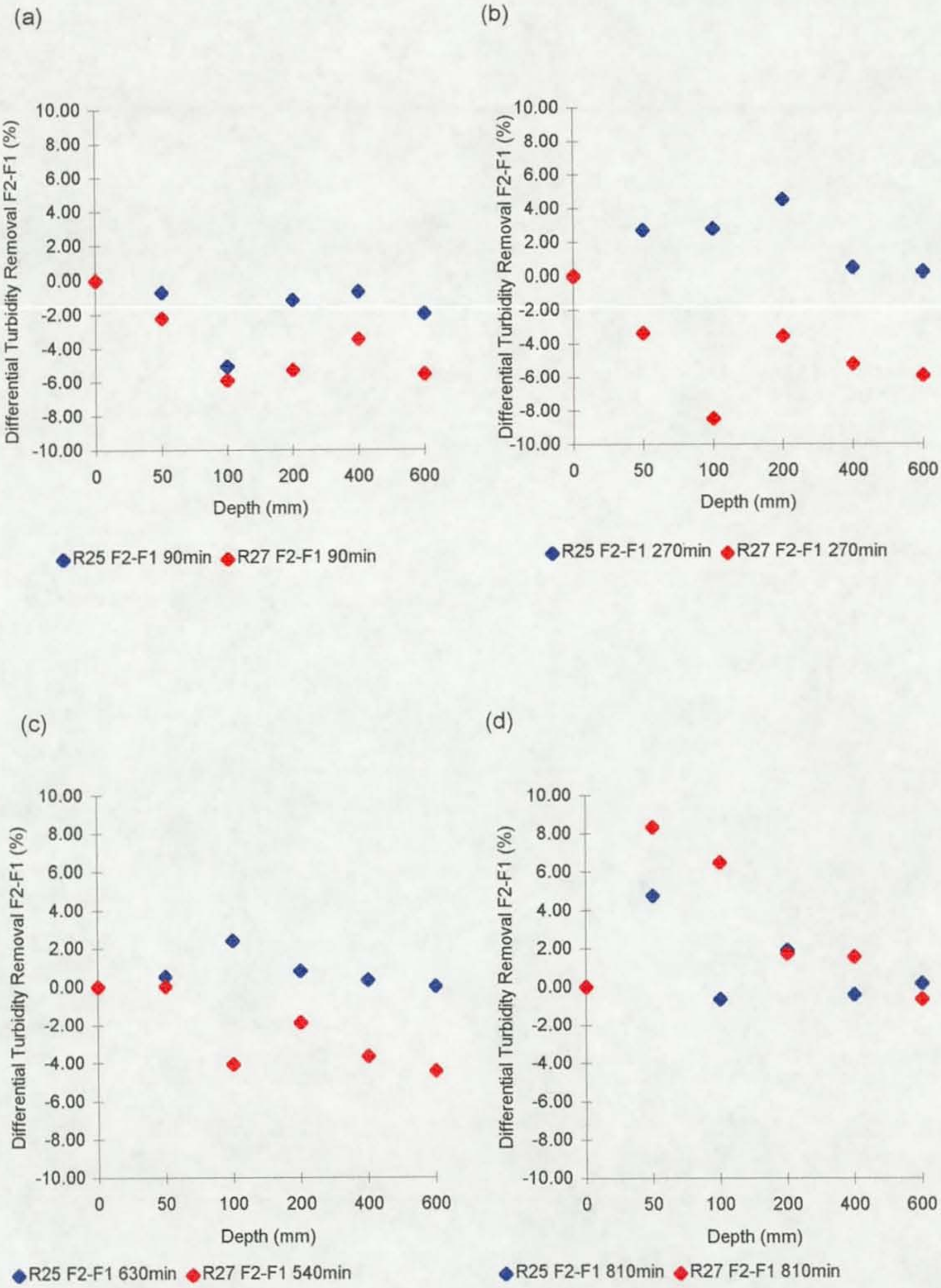


Figure 6.47 (a) to (d):Control Run 25 & Surging Run 27
Differential Turbidity Removal Charts at Selected Times

Time (minutes)	*Amplitude at 50 mm (mm)*	*(% HL)*	*Amplitude at 150 mm (mm)*	*(% HL)*	*Amplitude at 600 mm (mm)*	*(% HL)*
90	2.0	4.3	3.0	2.8	100	31.0
270	2.0	1.8	4.0	2.1	100	24.4
540	5.0	1.5	6.0	1.4	100	15.0
810	6.0	0.8	6.0	0.6	100	8.6

Table 6.5: Absolute & Relative Surge Amplitude in Run 27 Filter Two

(HL - Head Loss)

Similar to run twenty four, the absolute surge amplitude was observed to increase with rising loss of head at 50 and 150 mm depth for a fixed amplitude of 100 mm applied at the base of the filter column. In relative terms, the surge amplitude declined from 3 to 4 % to 0.6 to 0.8 % of the head loss at these depths. Similar to run twenty four, the surges appeared to inhibit the solids removal more in the early stages of the test since the magnitude of the flow fluctuations was greatest at this time.

Later in the test programme, runs thirty one and thirty two were compared. Run thirty one was conducted as a non-surging run to ensure good experimental control and reproducibility were maintained. As mentioned previously, run thirty two was conducted as a repeat of run twenty four and twenty seven with fixed amplitude surging of 100 mm applied to filter two at 80 oscillations per minute. In this test, the pressure ports within the filter media were isolated to ensure no interference resulted from the oscillating water levels in the piezometers in filter two. No such interference occurred and run thirty two produced similar results to runs twenty four and twenty seven.

6.2.2 Variable Amplitude Surging Effects & Comparison with Fixed Amplitude Surging Tests

In the previous section, the effects of fixed amplitude surging flow were described. In chapter two, the surging observations of previous researchers were reviewed (Baylis, 1958; Hudson, 1959). Baylis, discussed in section 2.1.2, observed that the surge magnitudes did not remain fixed during a filter run. His measurements and analysis found that the surge magnitudes increased with rising loss of head. Analysis of his surge recordings from several filters led Baylis to conclude that the surge amplitudes increased in proportion to the head loss raised to some power greater than one. Similarly, Hudson, discussed in section 2.1.4, observed the surge amplitudes to increase with rising loss of head during the filter cycle. However, Hudson found the surge amplitudes rose in proportion to the head loss raised to some power less than one. This author's surging measurements at local water treatment plants were discussed in chapter three. At each of the plants visited the measured pressure fluctuations were observed to increase with rising loss of head. However, some measurements found the surge magnitudes to increase at a rate greater than the rising head loss and others found the surge magnitudes to increase at a rate less than the rising head loss. From these and previous researchers' findings it is clear that the pressure fluctuations or surges increase during the filter cycle as the filter loss of head increases but it is not yet clear whether the surge magnitudes increase in proportion to the head loss raised to a power of greater than or less than one.

Test run twenty nine was conducted with increasing amplitude surges to determine the impact of these surge characteristics on filter performance and for comparison with the fixed amplitude surging run results discussed in the previous section. The surging in filter two was increased from 50 mm amplitude at a head loss of 270 mm to 130 mm amplitude at a head loss of 1170 mm from the start to the finish of the test, maintaining 80 oscillations per minute throughout. These surging characteristics were chosen to reflect the findings of previous researchers discussed in chapter two and this author's findings discussed in chapter three. Table 6.6 below illustrates the surge absolute and relative amplitudes at depths of 50, 150 and 600 mm periodically during the test.

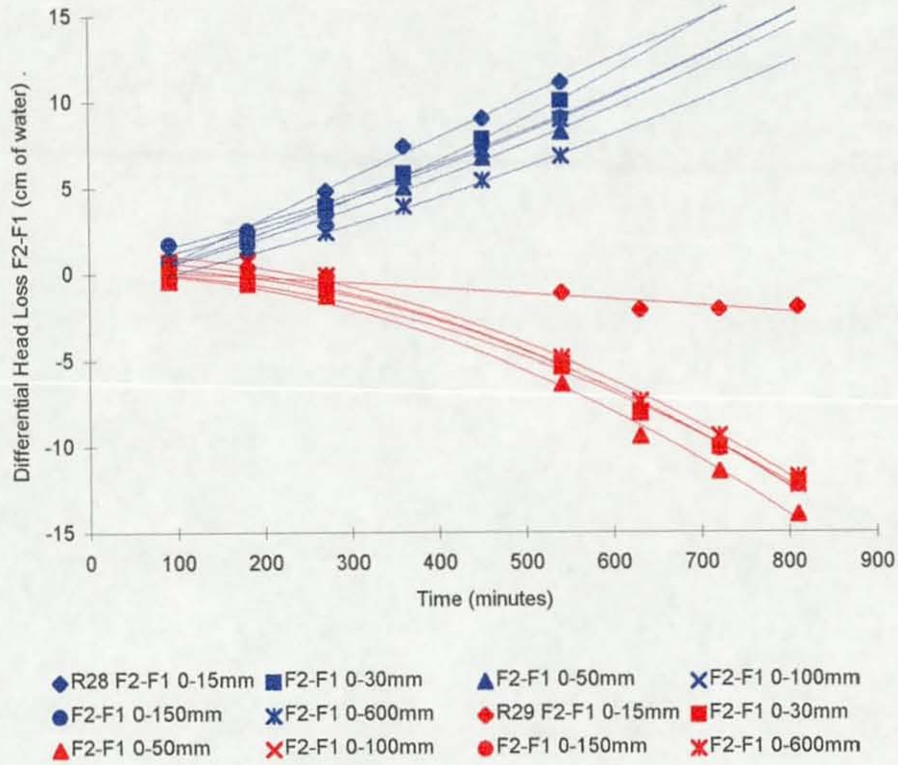
Time (minutes)	*Amplitude at 50 mm (mm)*	*(% HL)*	*Amplitude at 150 mm (mm)*	*(% HL)*	*Amplitude at 600 mm (mm)*	*(% HL)*
90	1.0	2.2	2.0	2.0	53	17.1
270	2.0	1.5	3.0	1.5	69	16.1
540	4.0	1.0	5.0	1.0	90	12.4
810	7.0	0.9	8.0	0.9	130	11.0

Table 6.6: Absolute & Relative Surge Amplitude in Run 29 Filter Two

Comparing this table with table 6.5 for run twenty seven with fixed amplitude surging, it can be seen that the surge amplitude at each depth in the filter column began this test smaller than for the fixed amplitude surging test run. However, with rising head loss the surge amplitude at each depth ended the test larger than for the fixed amplitude run. For example, at 50 mm depth in run twenty seven, the surge amplitude rose from 2 to 6 mm whereas in run twenty nine the surge amplitude rose from 1 to 7 mm. Note however that in both tests the relative surge amplitude declined but declined more rapidly in run twenty seven from a greater initial amplitude to a smaller final amplitude.

Figures 6.48 (a) and (b) compare the differential head loss trends with time and depth from control run twenty eight and surging run twenty nine. It can be seen that the control run performance was similar to previous control runs. Once more filter two developed head loss more rapidly than filter one with time with the bulk of the difference in the top 50 mm of the sand. Similar to the fixed amplitude surging runs, filter two developed head loss more slowly than filter one in run twenty nine, the variable surging test, with the bulk of the deficit in performance in the top 50 mm of the sand. However, the deficit in head loss development in this variable surging run did not appear to trend towards a constant deficit with time. In earlier fixed amplitude surge runs, the deficit in filter two appeared to trend towards a constant lag in head loss as the removal efficiency recovered.

Figure 6.49 (a) and (b) illustrate the differential turbidity removal efficiency results for runs twenty eight and twenty nine with time and depth. Figures 6.50 (a) to (e) and 6.51 (a) to (d) illustrate these trends individually for additional clarity. Comparing these trends with the figures for the fixed amplitude surging runs (runs twenty five and twenty seven), it can be seen that the effect of the variable amplitude surging was to inhibit the initial removal efficiency of filter two similar to the fixed amplitude surge results. However, unlike the fixed surging performance, filter two removal efficiency did not recover and close the deficit on filter one. It would seem that the variable amplitude of the surges in run twenty nine produced a smaller initial deficit in removal since the surge amplitude was initially smaller than for the fixed surge runs. However, as the surge amplitude was increased with time the deficit remained present throughout the filter run and did not recover. This explains the head loss performance described above where the lag in head loss did not tend towards a constant value.



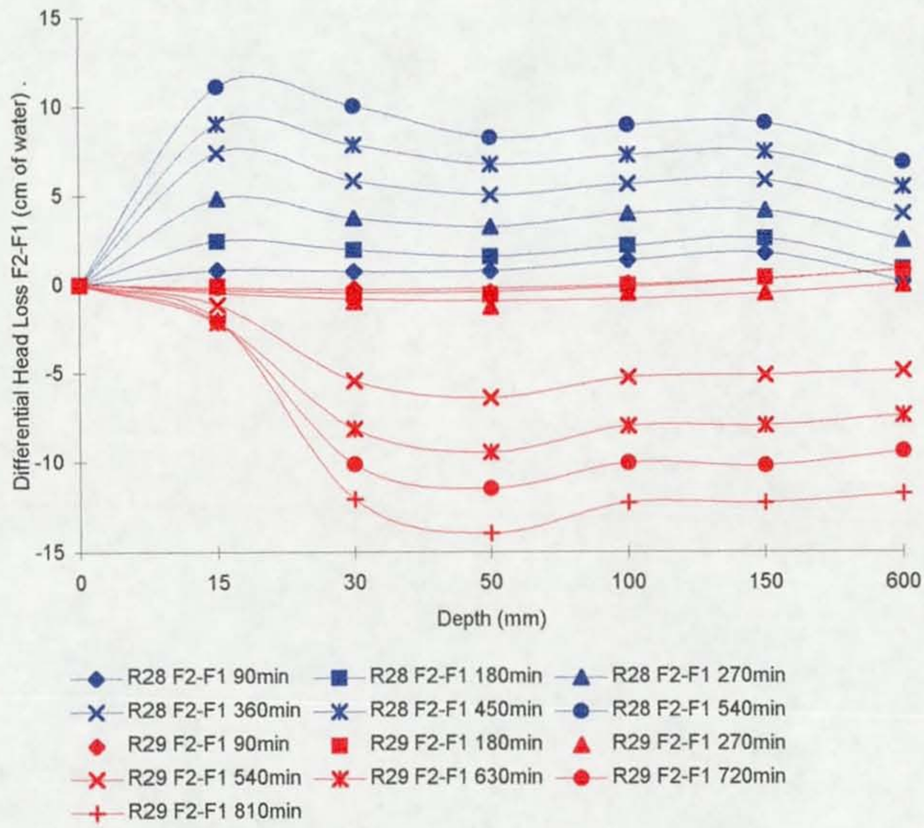
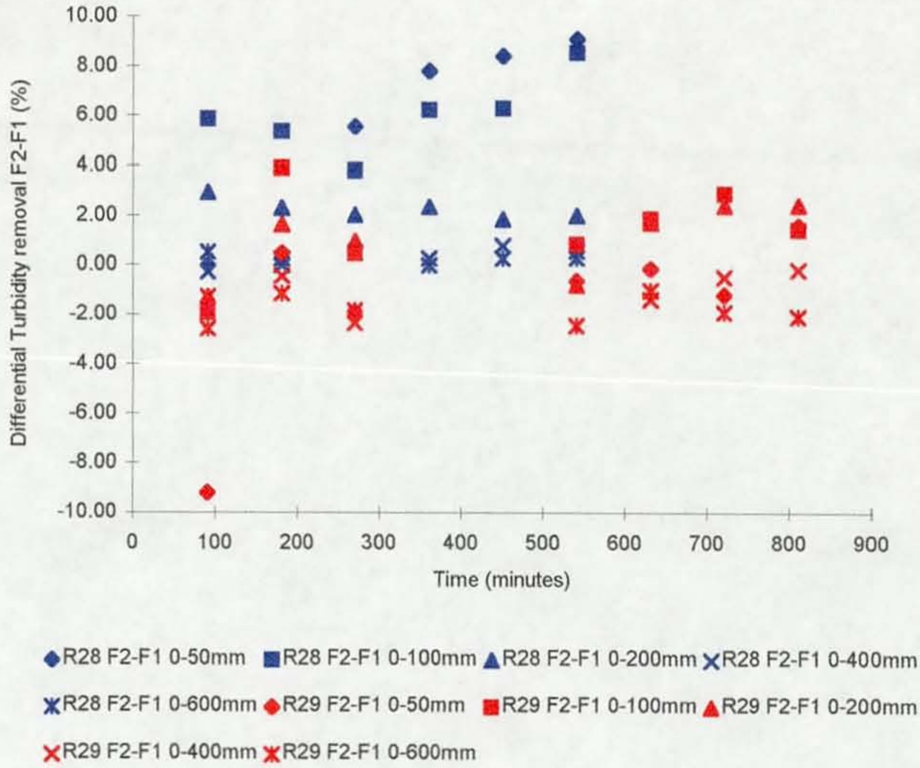


Figure 6.48 (a) & (b): Control Run 28 & Surging Run 29
Differential Head Loss with Time & Depth



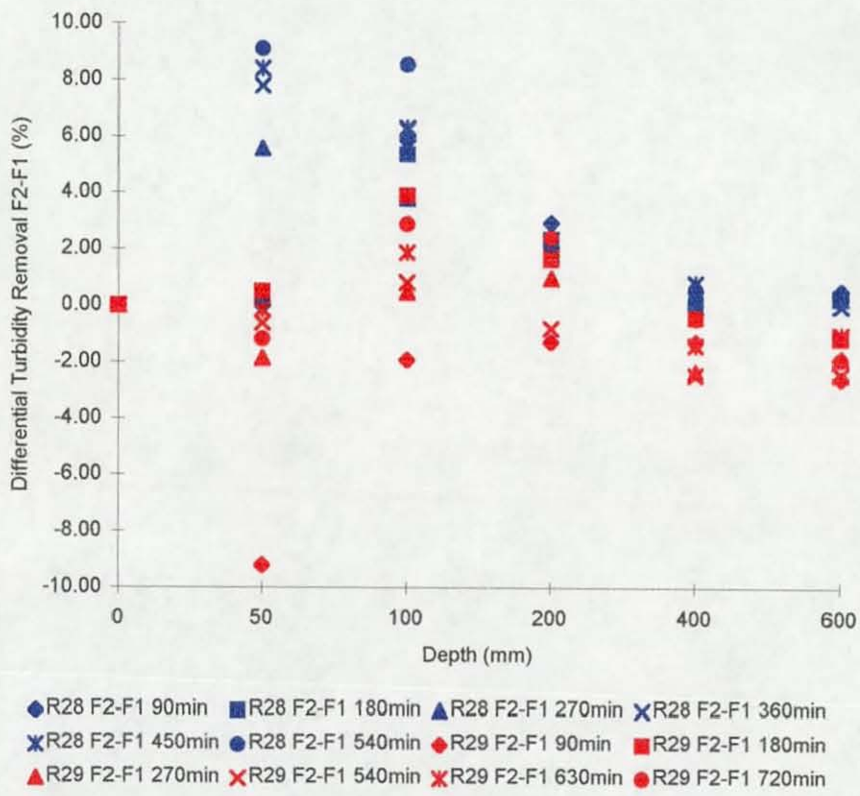


Figure 6.49 (a) & (b): Control Run 28 & Surging Run 29
Differential Turbidity Removal with Time & Depth

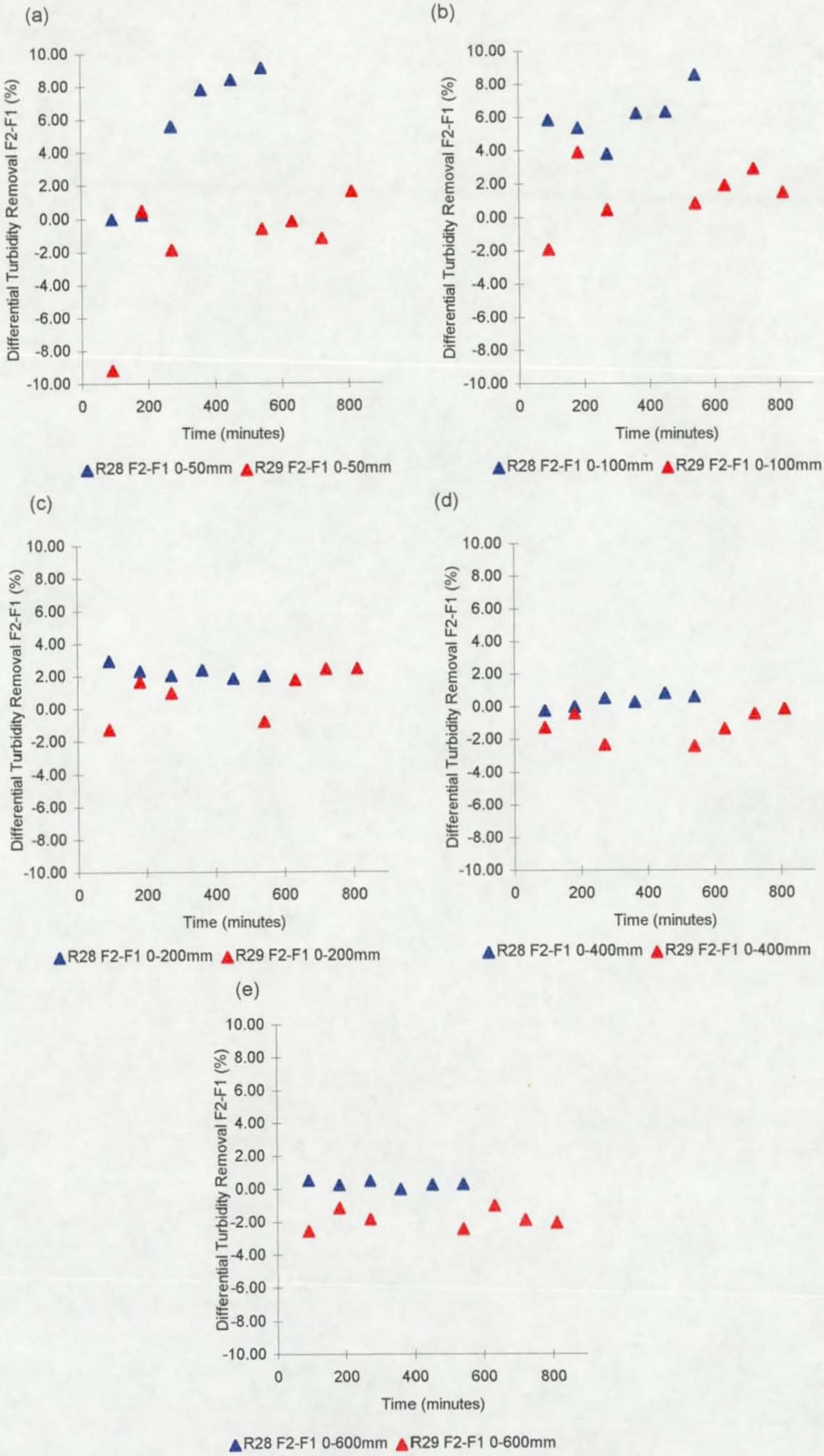


Figure 6.50 (a) to (e): Control Run 28 & Surging Run 29
Differential Turbidity Removal Charts at each Depth

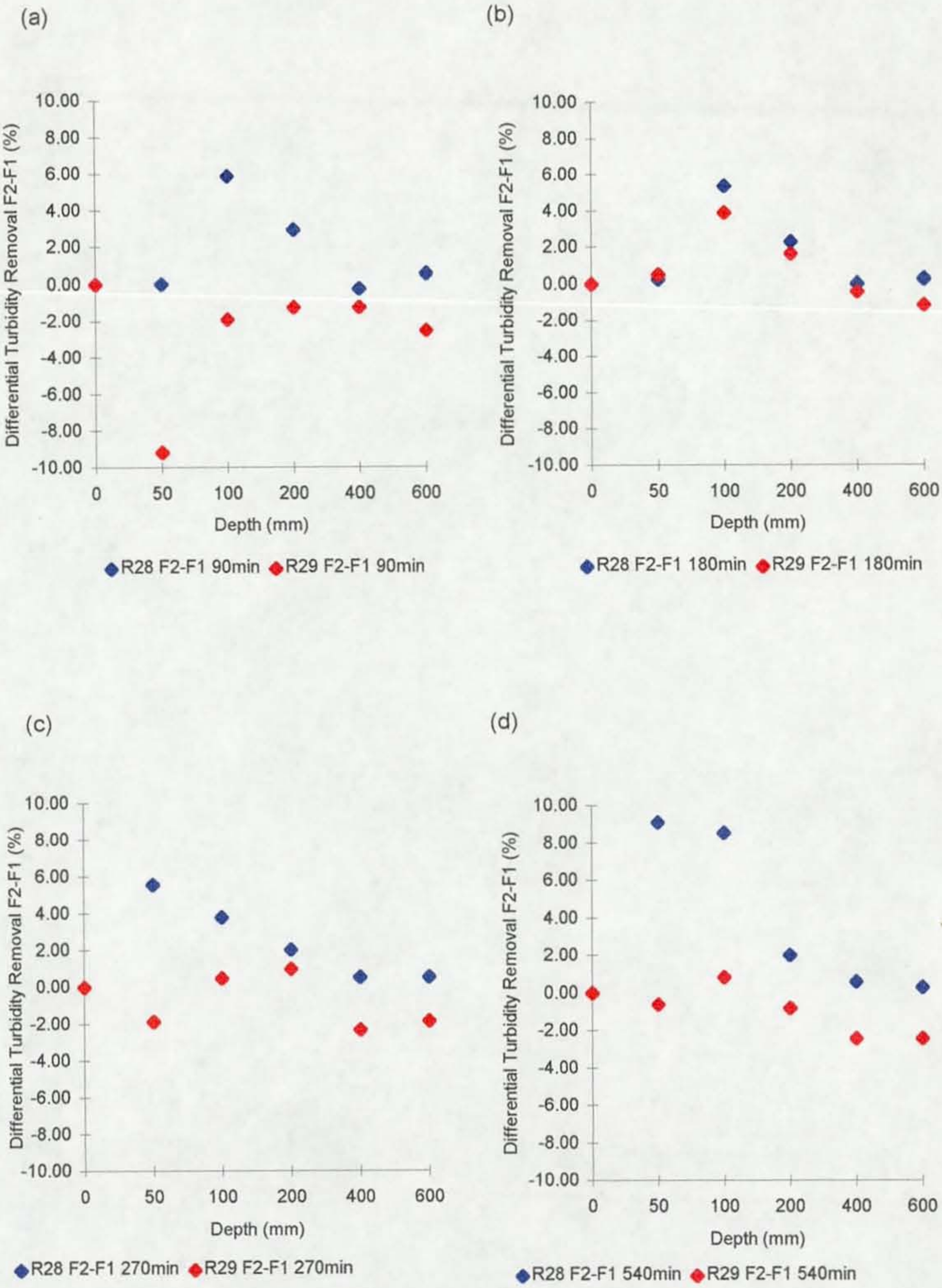


Figure 6.51 (a) to (d): Control Run 28 & Surging Run 29
Differential Turbidity Removal Charts at Selected Times

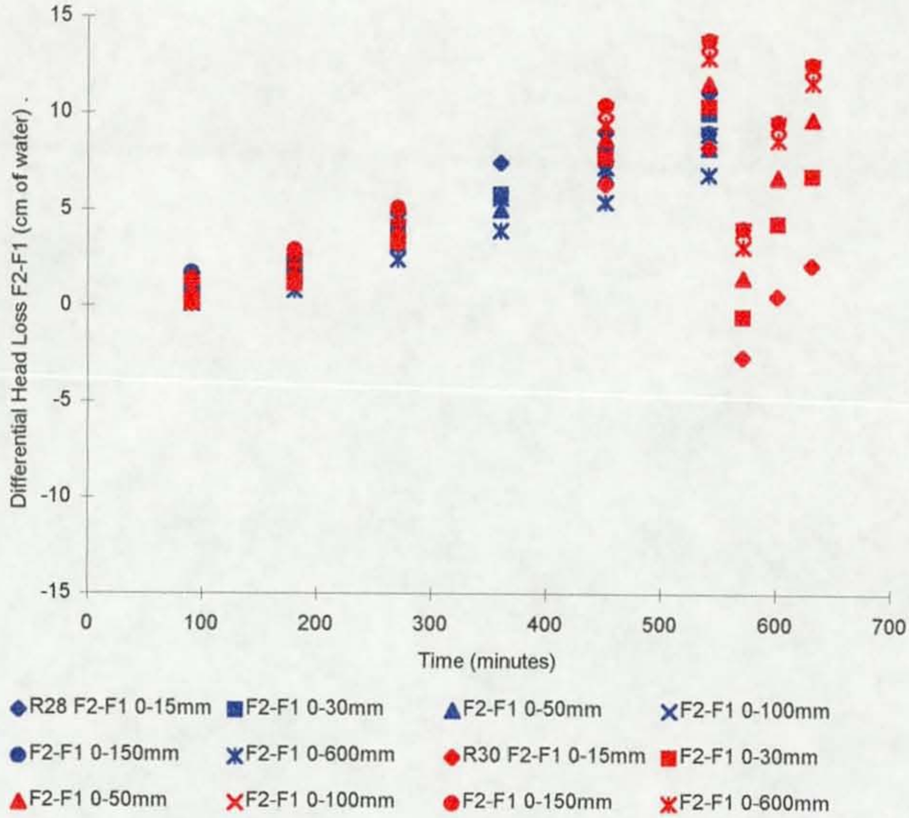
To determine the importance of the rate of occurrence of the surges on filter performance, test run thirty was conducted with increasing amplitude surging similar to run twenty nine but at a lower frequency of occurrence of 40 oscillations per minute. The surging in filter two was increased from an initial amplitude of 50 mm at a loss of head of 280 mm to an amplitude of 165 mm at a loss of head of 1420 mm, maintaining a frequency of occurrence of 40 oscillations per minute throughout. Table 6.7 below presents the absolute and relative surge amplitudes at depths of 50, 150 and 600 mm periodically during this test.

Time (minutes)	*Amplitude at 50 mm (mm)*	*(% HL)*	*Amplitude at 150 mm (mm)*	*(% HL)*	*Amplitude at 600 mm (mm)*	*(% HL)*
90	1.0	1.5	2.0	1.6	50	15.2
270	2.0	0.7	3.0	0.8	92	15.6
450	3.0	0.5	4.0	0.5	140	14.0
630	4.0	0.4	5.0	0.4	165	11.6

Table 6.7: Absolute & Relative Surge Amplitude in Run 30 Filter Two
(HL - Head Loss)

Similar to run twenty nine, the surge amplitudes were initially greater than for the fixed amplitude tests but declined less rapidly as the loss of head increased. Figure 6.52 (a) and (b) illustrate the differential head loss trends with time and depth for control run twenty eight and variable surging run thirty. It is clear from these trends that the variable amplitude surging at 40 oscillations per minute did not cause any significant lag in the development of head loss compared with the control run performance. Figure 6.53 (a) and (b) illustrate the differential turbidity removal efficiency trends with time and depth for control run twenty eight and surge run thirty. These have been plotted individually for additional clarity in figures 6.54 (a) to (e) and 6.55 (a) to (e).

Similar to run twenty nine, the increasing amplitude surges appear to have inhibited the turbidity removal efficiency within the bed of filter two. However, the filtrate quality produced at 600 mm depth was similar to the control performance. It seemed that the lower frequency surging inhibited removal within the filter column but less so than the higher frequency oscillations. Interestingly, the deficit in removal efficiency



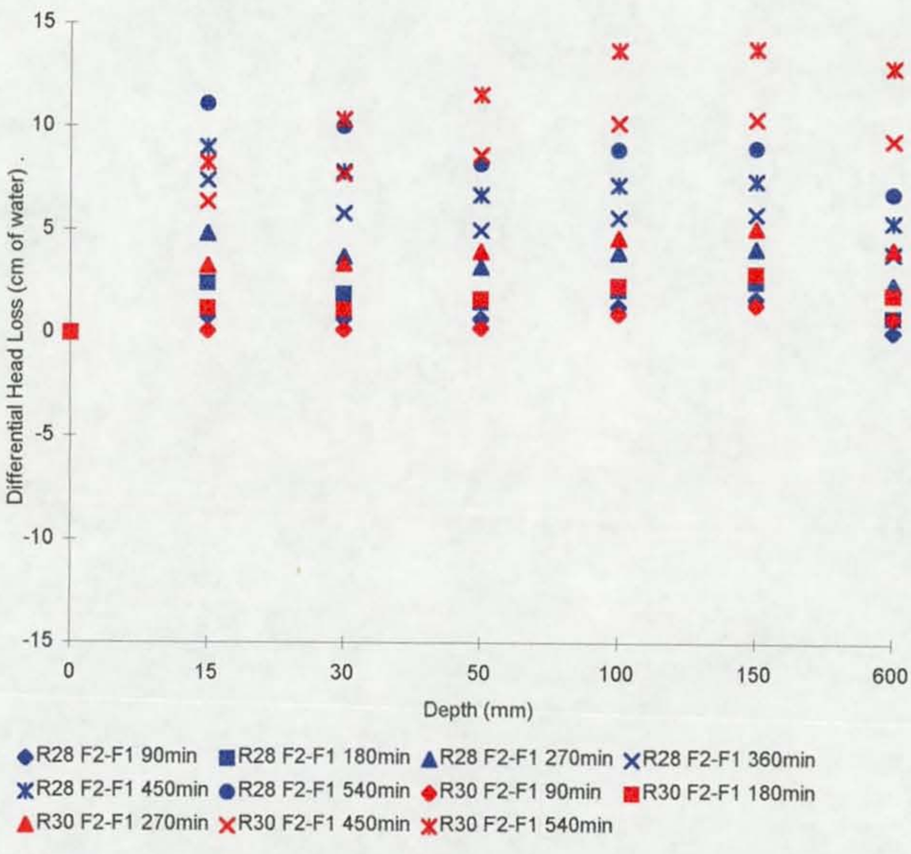
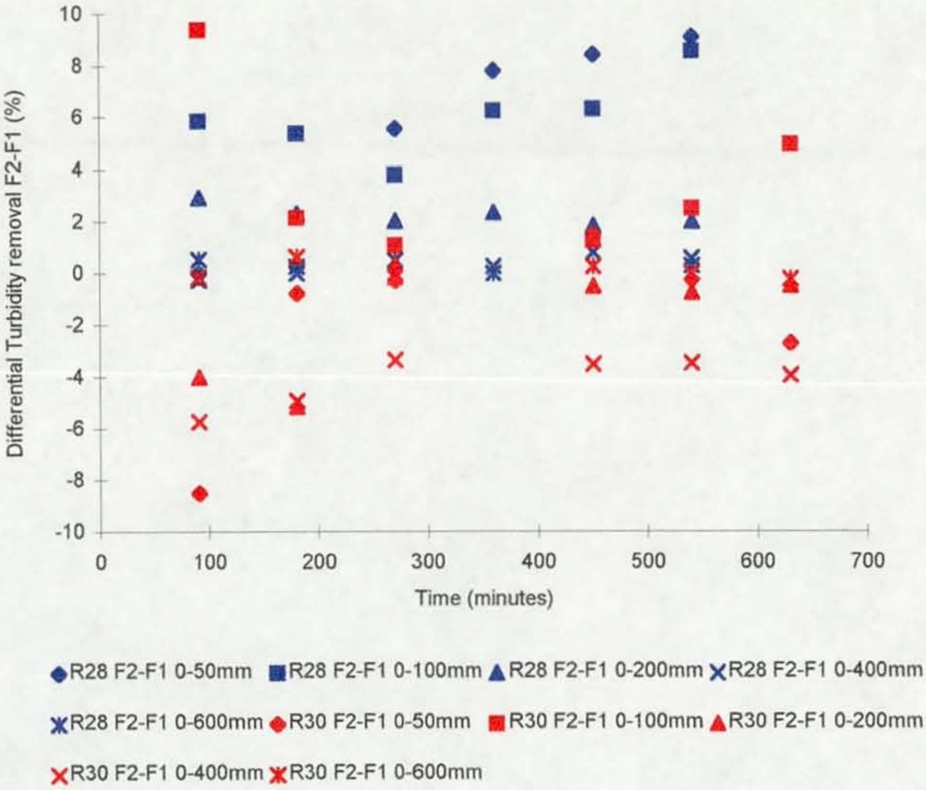


Figure 6.52 (a) & (b): Control Run 28 & Surging Run 30
Differential Head Loss with Time & Depth



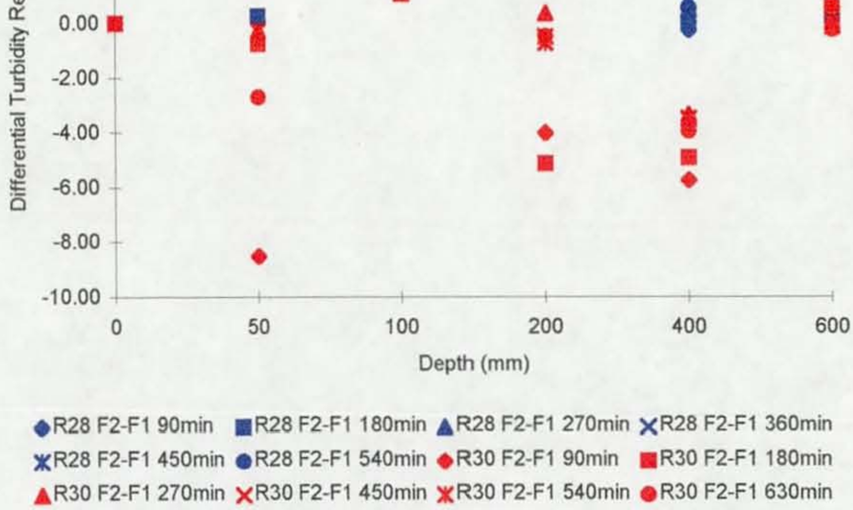


Figure 6.53 (a) & (b): Control Run 28 & Surging Run 30
Differential Turbidity Removal with Time & Depth

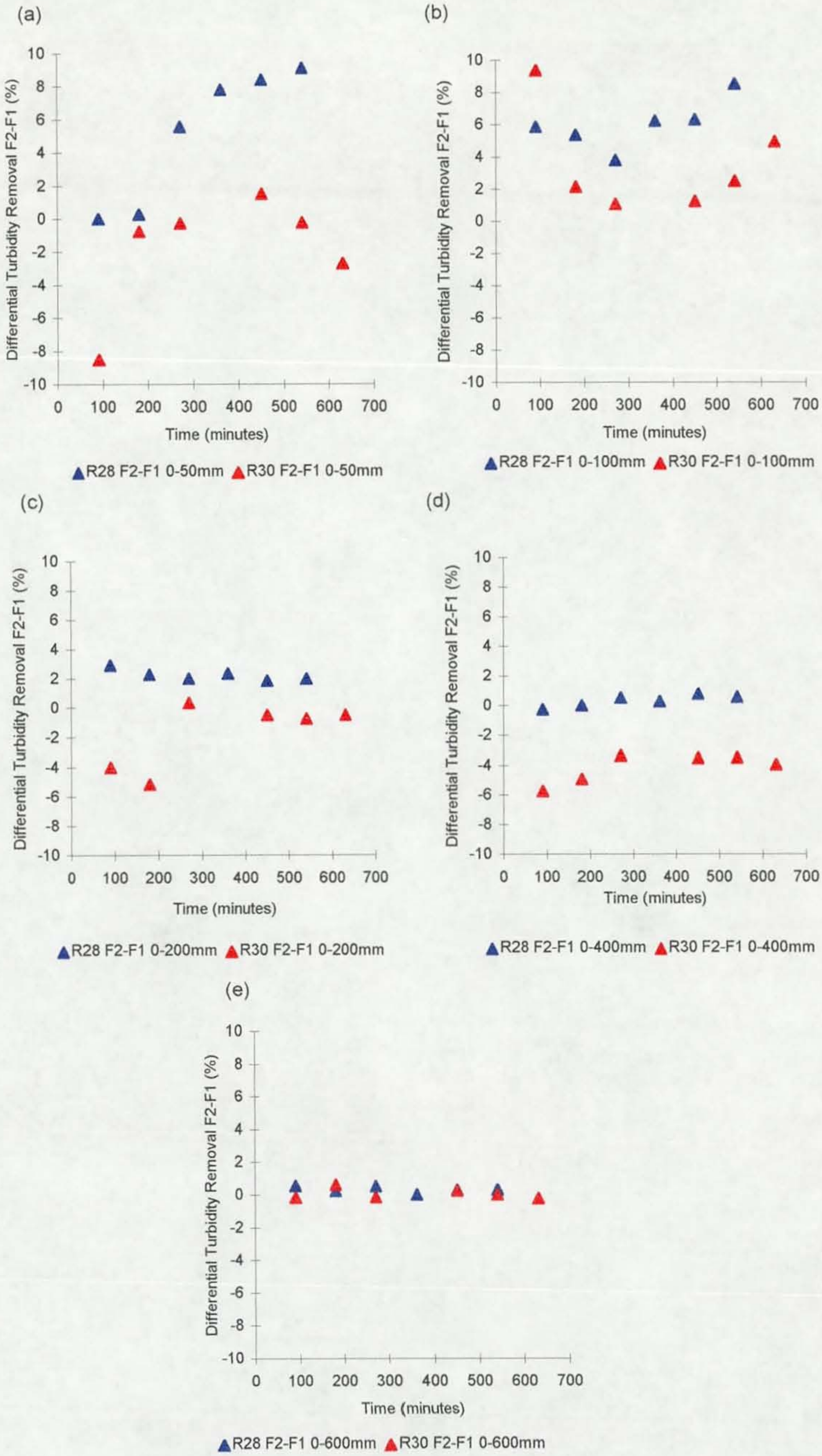


Figure 6.54 (a) to (e): Control Run 28 & Surging Run 30
Differential Turbidity Removal Charts at each Depth

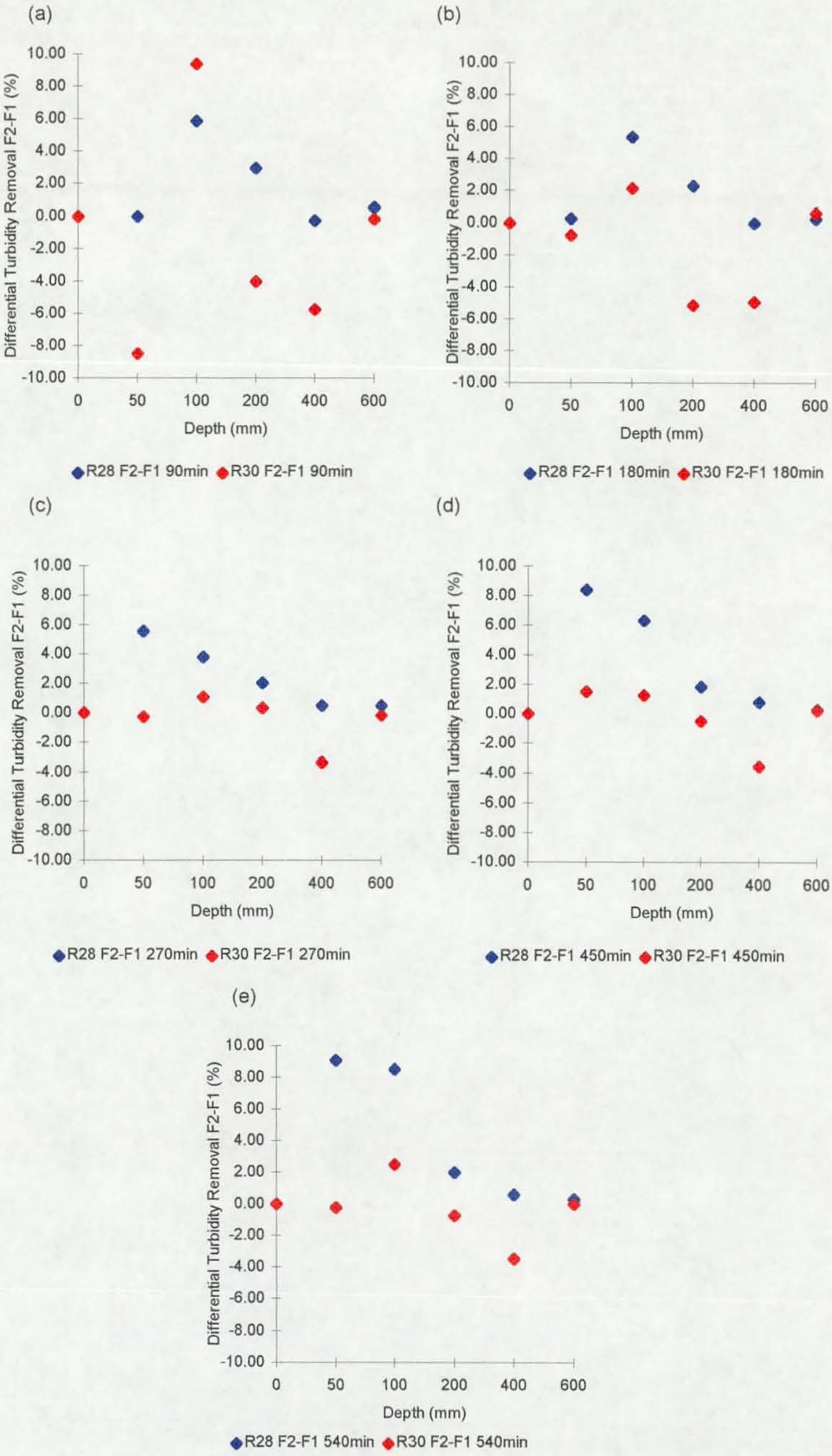


Figure 6.55 (a) to (e): Control Run 28 & Surging Run 30
Differential Turbidity Removal Charts at Selected Times

observed in run thirty was not accompanied by a deficit in the head loss development. It is possible the difference in the surging characteristics applied in runs twenty nine and thirty affected the effective volume occupied by the absolute volume of particles captured and in this manner modified the head loss development. It is accepted that retained deposits are self porous and occupy a greater effective volume than the absolute volume of the captured particles (Ives, 1975b). Since the deficit in removal efficiency did not generate a deficit in the rate of head loss development in run thirty it could be argued that the deficit in head loss development in run twenty nine was at least partly the result of a change in the structure of the deposits formed from the captured particles and not solely the result of the poorer removal efficiency. The more rapidly occurring fluctuations applied in run twenty nine may have resulted in the formation of a more compact deposit structure occupying less effective volume and thus generating a smaller loss of head. The less rapidly occurring pressure fluctuations in run thirty may have resulted in a more open structure deposit generating a greater loss of head. This could be why the bed in filter two moved late in the test during run thirty. The more open deposit structure may have failed under the rising hydraulic gradient. However, these explanations will require confirmation by further research.

The differential head loss and differential turbidity removal efficiency analysis presented above illustrated clearly the qualitative impact of the surging flow on filter performance. The surges reduced the removal efficiency of turbidity and slowed the rate of head loss development by a magnitude determined by the characteristics of the surging applied. Analysis of the differential head loss and removal efficiency with time and depth highlighted the changing influence of the surges on performance as the test progressed. However, quantitative comparison of the differential head loss and differential removal efficiency at any given time during the control and surging test runs was of limited use. It was found that the magnitude of the differential head loss or removal efficiency at any given instant in time was influenced not only by the time from the start of the test but by the magnitude of the absolute head loss and removal efficiency achieved by the filters. Fluctuations in the suspension conductivity and Zeta potential mentioned earlier in this chapter influenced the rate of development of head loss and the removal efficiency achieved at any given time into the tests.

To overcome this difficulty, the results of test runs twenty three to thirty two were analysed further to compare the performance of the control and surging tests at similar loss of head and turbidity removal as opposed to similar time from the start of the test. Figure 6.56 illustrates the differential head loss in cm of water from the control and surging tests plotted against the absolute head loss achieved in cm of water in filter one at a depth of 600 mm. Similar to the analysis described earlier for the control runs, filter two developed head loss more rapidly than filter one with rising loss of head in filter one in all the control tests. However, the magnitude of the differential varied from control test to test at any given loss of head. Again, similar to the analysis described earlier for the surging runs, filter two developed head loss more slowly than filter one with rising loss of head in filter one in all the surging test runs. However, the magnitude of the differential head loss varied from surging run to run at any given loss of head. The fluctuation in the differential head losses at any given loss of head in filter one was caused by the differing rates of development of head loss from test run to test run. This variation was caused by the changes in suspension conductivity mentioned earlier and the findings are summarised in table 6.8 below.

Run	Conductivity (uS/cm)	Time (hours)	Head Loss in F1 (mm)	Differential Head Loss (mm)
23	780	18	1111	+158
24	758	17.5	1132	-227
25	708	18	1104	+184
27	635	13.5	1311	-153
28	512	9	1274	+68
29	643	13.5	1289	-118
31	656	12	1099	+107
32	584	9	1130	-81

Table 6.8: Differential Head Loss as a function of Head Loss, Time and Conductivity

From figure 6.56 and the table above, it can be seen that control runs twenty three and twenty five with similar suspension conductivity of 708 to 780 uS/cm developed some 1100 mm of head loss in filter one in 18 hours with a similar differential head loss of +158 to +184 mm. However, control run twenty eight with a lower conductivity of

512 uS/cm developed some 1274 mm of head loss in filter one in only 9 hours. The differential in control run twenty eight was only +68 mm. Control run thirty one with a conductivity of 656 uS/cm developed 1099 mm of head loss in filter one in 12 hours with a differential of +107 mm. Clearly, the differential head loss at any given head loss in filter one has been influenced by the rate of head loss development which itself was determined by the suspension conductivity. From figure 6.56 and Table 6.8, a similar pattern can be seen in the surging test results. Fixed amplitude surging run twenty four with a conductivity of 758 uS/cm developed 1132 mm of head loss in filter one in 17.5 hours with a differential head loss of -227 mm. However, fixed amplitude surging run thirty two with a conductivity of 584 uS/cm developed 1130 mm of head loss in filter one in only 9 hours. The differential head loss was only -81 mm in this test. Fixed surging run twenty seven with a conductivity of 635 uS/cm, between runs twenty four and thirty two, as expected developed 1311 mm of head loss in filter one in 13.5 hours with a differential head loss of -153 mm. Once more, the magnitude of the differential head loss in the fixed amplitude surging runs at any given loss of head in filter one was influenced by the rate of head loss development which was determined by the suspension conductivity.

Variable amplitude surging run twenty nine was conducted at a conductivity of 643 uS/cm, similar to fixed amplitude surging run twenty seven. As such, any differences between the results of these two tests were though to be solely due to the different surging characteristics applied. From table 6.8, it can be seen that filter one developed 1289 mm of head loss in 13.5 hours in run twenty nine, similar to filter one in run twenty seven as expected from the similar conductivity measurements. However, from figure 6.56, filter two did not lag behind filter one by as large a margin in run twenty nine during the test. It seemed that the smaller initial surge amplitudes in run twenty nine produced a smaller lag in the rate of development of head loss than for the fixed amplitude surges in run twenty seven. The lag in performance in filter two in fixed surging run twenty seven appeared to tend towards a constant magnitude around -150 mm with rising loss of head. A similar pattern can be seen in the trends for fixed surging runs twenty four and thirty two. However, the lag in head loss in filter two in variable surging run twenty nine appeared to continue to increase with rising loss of head and presumably would have lagged behind the result of run twenty seven by a head loss of 1500 mm. This increasing deficit in the head loss development was the result of the increasing amplitude of the surges applied to filter two as the loss of head rose. Figure 6.57 illustrates the differential turbidity removal efficiency results of runs twenty three to thirty two plotted against the absolute turbidity removal efficiency achieved by filter one during the runs at a depth of 600 mm. From this chart, it is clear that the removal efficiency achieved changed from test to test. The turbidity removal efficiency achieved by each test was determined by the suspension conductivity and Zeta potential as mentioned previously in section 6.1.2 and 6.1.3. These factors are discussed further in section 6.3. Filter one in control test runs twenty three and twenty five ripened from an initial removal efficiency of 74 % to around 80 % in 18 hours with similar suspension conductivity measurements. Filter one in control test runs twenty eight and thirty one ripened from an initial removal efficiency of 81 % to 90 and 88 % in 9 and 12 hours respectively with similar suspension Zeta potentials of -16.1 and -15.9 mV and conductivity measurements of 512 and 656 uS/cm respectively. However, the differential removal efficiency of all these control tests was approximately zero throughout the test irrespective of the removal achieved and the rate of ripening of removal efficiency. Filter one in fixed amplitude surging runs twenty four and twenty seven ripened from initial removal efficiency measurements of 76 % and 65 % to 80 % in 17.5 and 13.5 hours respectively with conductivity measurements of 758 and 635 uS/cm respectively. Filter one in fixed amplitude surging run thirty two ripened from 87 % to 93 % in 9 hours with a suspension conductivity of 584 and a Zeta potential of -16.2 mV. However, in each fixed amplitude surging run the differential removal efficiency was negative and tended towards zero with rising removal efficiency in filter one irrespective of the removal efficiency achieved during each test. In run twenty four, the differential removal efficiency was - 9 % at a removal of 76 % in filter one. The deficit in removal declined from this value to - 2 % at a removal efficiency of 80 %. In run twenty seven, the deficit in removal efficiency in filter two declined from - 5.5 % at a removal efficiency of 65 % in filter one to less than - 1.0 % at a removal of 80 % in filter one. The differences in the deficits in removal efficiency in filter two in these identical fixed amplitude surging runs were the result of the differing rates of development of removal efficiency determined by the suspension conductivity and Zeta potential. Run twenty four had a greater deficit than run twenty seven at equal removal efficiency in filter one due to the slower rate of ripening in run twenty four. Fixed amplitude surging run thirty two had a deficit in

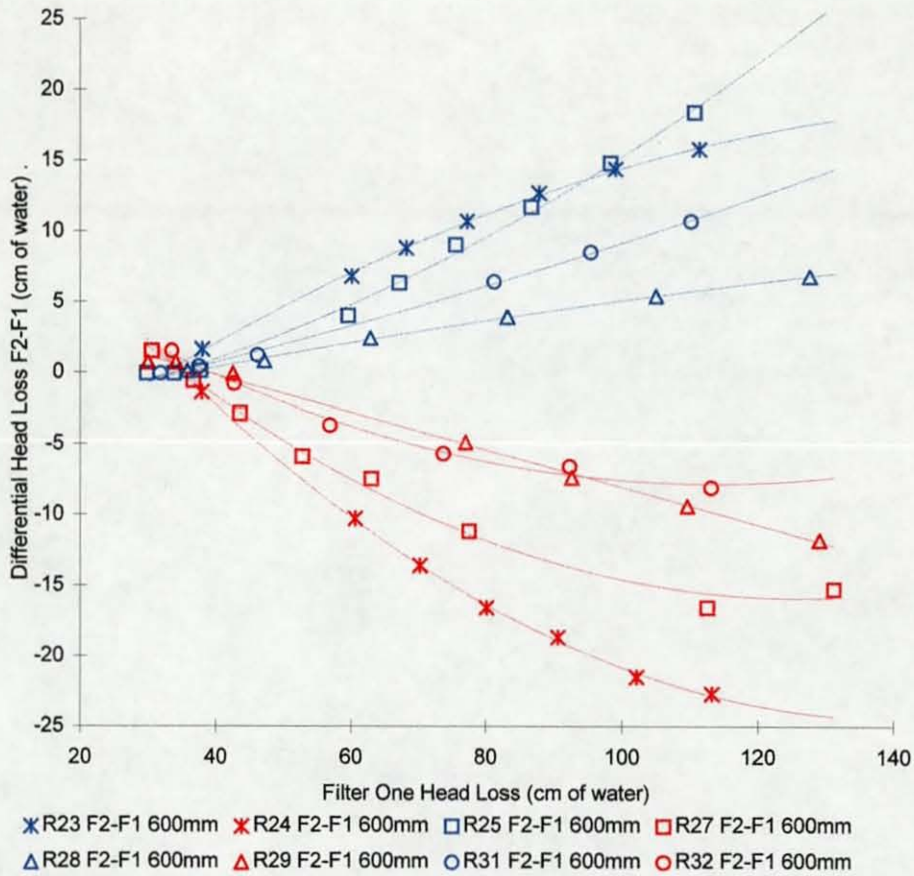


Figure 6.56: Comparison of Differential Head Loss with Loss of Head in Filter One

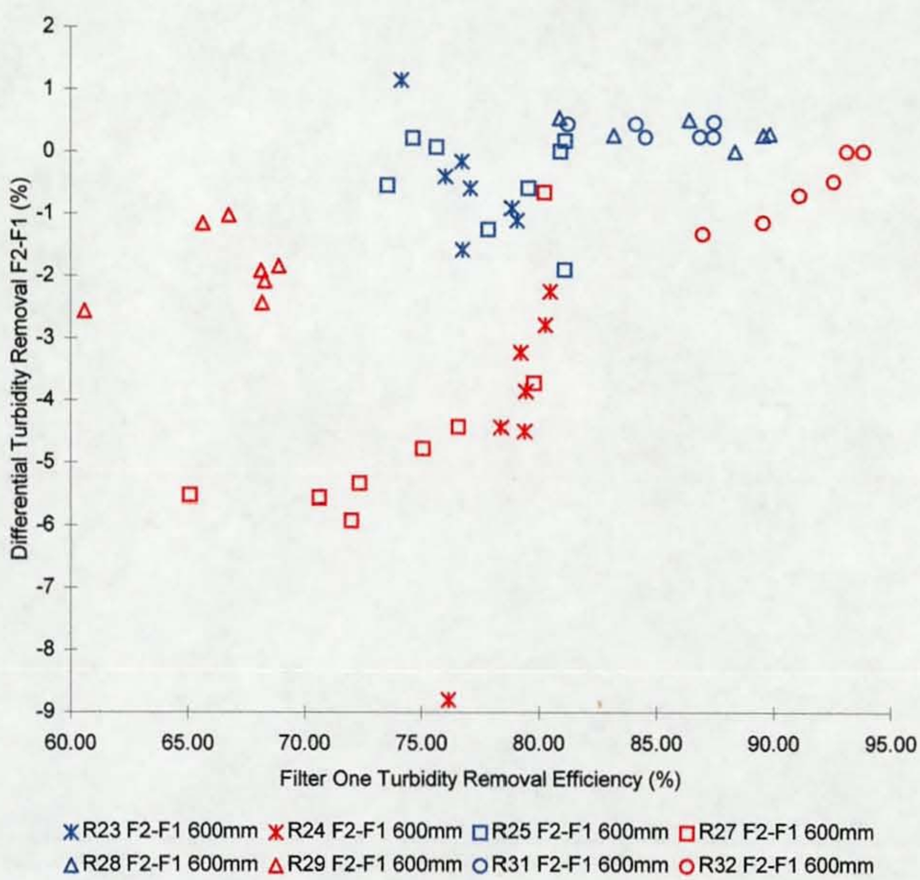


Figure 6.57: Comparison of Differential Turbidity Removal with Removal in Filter One

removal efficiency in filter two of - 1.0 % at a removal efficiency of 87 % in filter one and declined to 0.0 % at a removal of 93 % in filter one. The deficit in removal in this fixed amplitude surging run was less than run twenty four and twenty seven due to the rapid filter ripening and greater removal efficiency achieved as a result of the lower suspension conductivity and smaller magnitude Zeta potential.

Test run twenty nine, included in figure 6.57, was conducted with variable amplitude surging at 80 oscillations per minute for comparison with the fixed amplitude surging tests discussed above. Filter one ripened from 60 to 68 % in 13.5 hours with a suspension conductivity of 643 uS/cm and a Zeta potential of -24.1 mV. Although the suspension conductivity was similar to run twenty seven, the removal efficiency achieved was suppressed by some 10 % due to the larger magnitude Zeta potential. Nonetheless, once more filter two had a deficit in removal as a result of the surging flow. The deficit in removal in filter two was initially 2.5 %, smaller than for fixed amplitude surging runs twenty four and twenty seven but larger than for run thirty two. However, unlike the fixed amplitude surging runs, the deficit in removal efficiency in filter two did not decline towards zero. The deficit in removal in filter two in this variable amplitude surging run instead remained essentially constant at 2 % with increasing removal efficiency. It would appear that the initially smaller amplitude surges in run twenty nine produced a smaller initial deficit but since the surging amplitude was increased as the loss of head rose the surges continued to inhibit the removal efficiency throughout the test. This result correlated with the differential head loss results discussed previously and illustrated in figure 6.56.

Comparison of the differential head loss with head loss achieved and the differential turbidity removal efficiency with removal efficiency achieved in filter one discussed above has shed additional light on the effect of the surges on filter development. However, it should be noted that these analyses are limited too in that the differential at a given loss of head or removal was dependent not only on the loss of head or removal achieved but by the length of time taken to reach that head loss or removal . However, the combination of the differential head loss and turbidity removal efficiency trends with time and depth discussed earlier in this chapter and these trends with absolute head loss and removal achieved yield the whole picture. The results from runs twenty three to thirty two are summarised in table 6.9 below to illustrate this point more clearly.

Test Run	Final Head Loss F1 (mm)	Final Removal F1 (%)	Time (hours)	Final Diff. Head Loss (mm)	Initial Diff. Removal (%)	Final Diff. Removal (%)
23	1111	79.1	18	+158	+1.1	-1.1
24	1132	80.3	17.5	-227	-8.8	-2.8
25	1104	75.6	18	+184	-0.6	+0.1
27	1311	80.2	13.5	-153	-5.5	-0.7
28	1274	90	9	+68	+0.5	+0.3
29	1289	68	13.5	-118	-2.6	-2.1
30	1305	87	10.5	+116	-0.2	-0.2
31	1099	88	12	+107	+0.4	+0.5
32	1130	93	9	-81	-1.3	0.0

Table 6.9: Summary of results at 600 mm depth from Runs 23 to 32

From this table, it can be seen that the difference in head loss development and turbidity removal efficiency in the control test runs was influenced by a combination of the head loss and turbidity removal efficiency achieved and the time it took to reach this stage of development. As such, these factors also influenced the measured effect of the surging flow on the filter development. It seems clear from the previous discussion and the summary table above that the more rapidly the filters developed the smaller was the observed effects from a given magnitude and frequency of occurrence of surging. Efforts were made to eliminate the effects of the fluctuations in suspension conductivity and Zeta potential from the experimental results. Section 6.3 later in this chapter discusses the influence of the suspension chemistry on the experimental results further and describes the results of statistical analysis of the data. Nevertheless, it is clear that surging of a similar magnitude and frequency to those observed by previous researchers and by this author at local water treatment facilities had a clear and detrimental effect on filtrate quality in this laboratory investigation.

6.2.3 Filter Particle Removal: Effect of Surges & Other Observations

In addition to the head loss and turbidity measurements taken from each test samples were taken from the suspension sample ports located above the filter media and at depths of 50, 100, 200 and 400 mm and from the filter outlet ports for particle size distribution (PSD) analysis to determine the influence of the surging flow on removal of different particle sizes. The sampling ports were described in chapter 4 and the methodology for the particle sampling and analysis was discussed in chapter 5. Particle size distribution analysis was introduced after run twenty and a summary of the sampling conducted in subsequent test runs and the key findings are presented in table 6.10 below.

Test Run	Surges Applied	PSD Sampling Regime	Comments on PSD Results
21	Midway: 20% 10/min	All ports at end of test	No effect on PSD
22	Midway: 10% 40/min	All ports at end of test	No effect on PSD
23	Control	Influent & 100 mm at 5, 10.5 & 18 Hours	Evidence of detachment
24	Fixed surges 80/minute	All ports at end of test	No effect on PSD
25	Control	All ports at 5 Hours	F2 greater removal than F1
27	Fixed surges 80/min	All ports at 5 Hours	F2 poorer removal than F1
29	Variable surges 80/min	All ports at 5 Hours	Discarded
30	Variable surges 40/min	All ports at 5 Hours	Discarded

Table 6.10: Summary of Particle Size Distribution Analysis Sampling & Findings

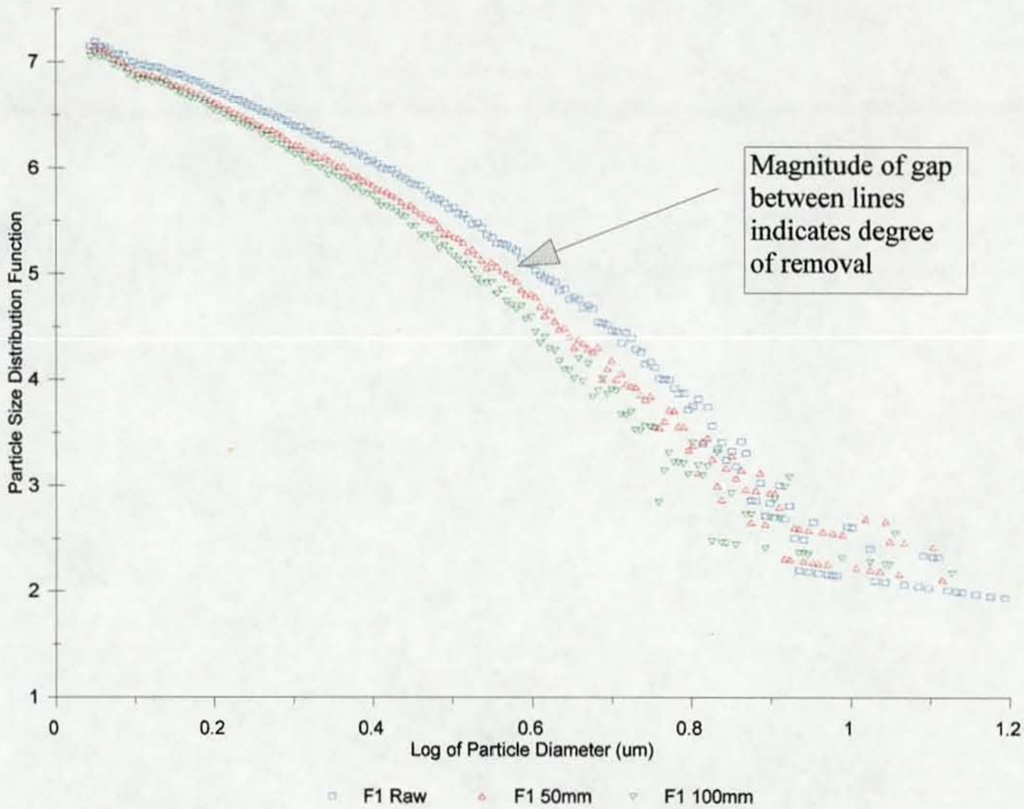
Initially, the sampling regime adopted was varied from run to run to determine the most useful approach to take. Ideally, particle samples should be collected from each sample port periodically during the tests similar to the turbidity samples to determine the changing particle size distribution with time and depth. However, due to problems associated with the limited shelf life of collected samples (discussed in chapter 5) and limits on the number of samples that could be practically handled during the conduct of a test run, this approach could not be undertaken. A more limited particle sampling regime needed to be followed. It was decided to collect samples from all the suspension ports through the depth of the filter columns at a chosen time into the test and if necessary to change the collection time from test to test to acquire the greatest amount of information possible.

As discussed in section 6.1 earlier the surges were initiated in filter two midway through the test in test runs twenty one and twenty two. Therefore, samples for particle size distribution analysis were collected from each of the suspension sampling ports at the end of the test from both filters to determine if the applied surges had influenced the removal efficiency of different particle sizes with depth. Similar to the head loss and turbidity removal efficiency measurements described earlier in this chapter, the surges had no apparent effect on the particle size distribution analysis of the samples from filter two when compared with filter one at the end of the tests.

In surging run twenty four, the fixed amplitude surges of 100 mm magnitude were applied from the start of the test to filter two only. The effects of the surges on the head loss and removal of turbidity were described earlier in this chapter. In addition, samples were collected from all the ports at the end of the test for particle size distribution analysis. As discussed previously, the effect of the fixed amplitude surges on the turbidity removal was most significant during the early stages of the test. By the end of the test run, the effect of the fixed amplitude surges had declined such that there was no significant difference in turbidity removal between the filters. Similarly, comparison of the particle size distribution analysis results of the samples taken from both filters at the end of the test did not show any significant difference in concentration or size distribution. Subsequently, it was decided to collect the samples for particle size distribution analysis earlier in the test where the surges had a clear impact on the turbidity removal efficiency.

Samples were collected for particle size distribution analysis from all of the suspension ports in filters one and two 5 hours into the test in control run twenty five. Figure 6.58 (a) and (b) illustrate the particle size distribution functions against log of particle diameter for filters one and two for the influent suspension collected above the filter media and the suspension samples collected at depths of 50 and 100 mm.

(a)



(b)

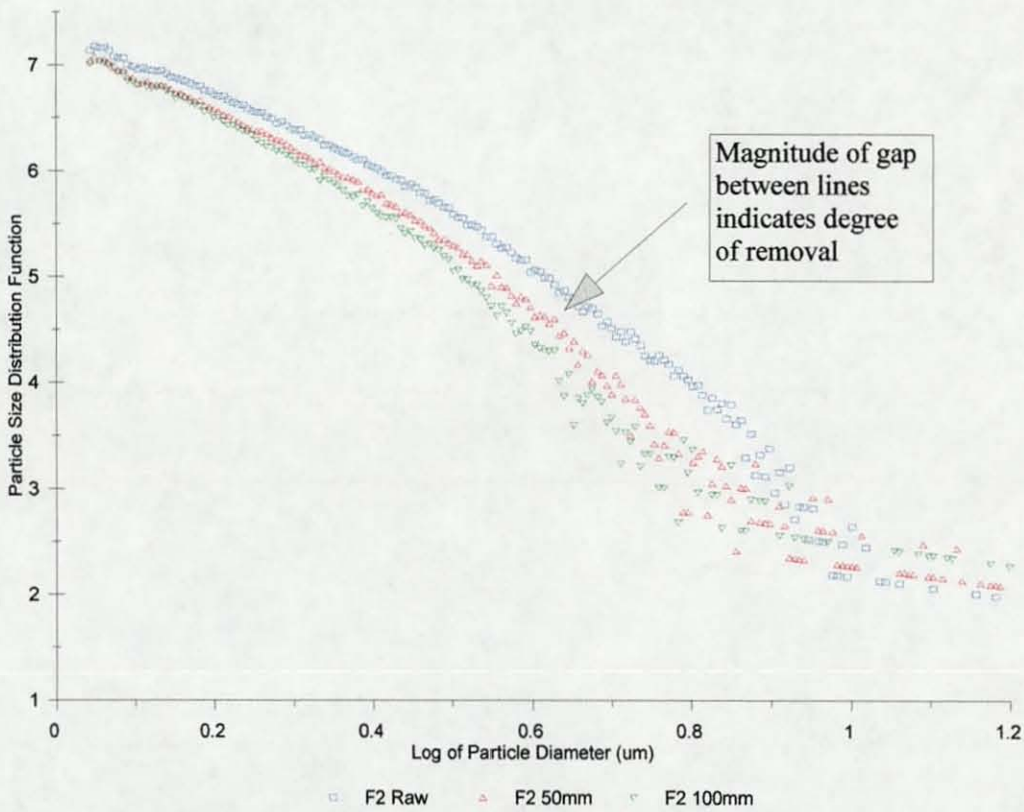
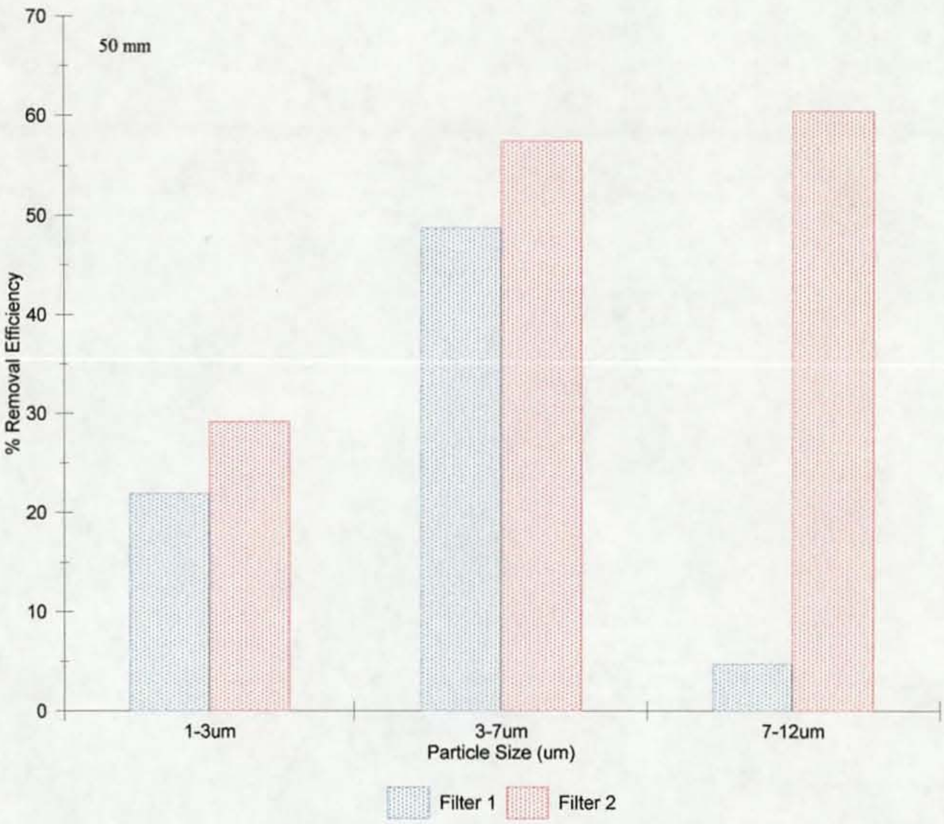


Figure 6.58 (a) & (b): PSD Functions for Filter 1 & 2 Control Run 25

The results from 200, 400 and 600 mm depths were discarded since they were not analysed on the day of sampling. It can be seen that with increasing depth particles from 1 to 10 micron size are progressively captured by the filter media. Above 10 micron, the trends become noisy as the particle counts decline. Close inspection of the particle size distribution functions at 50 and 100 mm for filters one and two revealed that the gap between the raw water function and the functions at 50 and 100 mm depth was marginally larger for filter two. It seemed that filter two removed a marginally greater proportion of all particle sizes up to 10 micron compared with filter one. To illustrate this more clearly, figure 6.59 (a) and (b) illustrate the removal efficiency of particles in 3 particle bands of 1-3, 3-7 and 7-12 microns at 50 and 100 mm depth. In all 3 band sizes, filter two clearly removed a greater proportion of particles that filter one at both 50 and 100 mm. As discussed previously, these findings correlated with the turbidity removal performance observed in filters one and two at 50 and 100 mm depth for control run twenty five where filter two achieved marginally greater turbidity removal than filter one at 5 hours.

Samples were collected from all of the suspension ports in both filters 5 hours into the test in fixed amplitude surging run twenty seven for comparison with the results discussed above for control run twenty five. Figure 6.60 (a) and (b) illustrate the particle size distribution functions for filter one at depths of 0, 50 and 200 mm and for depths of 0 and 100 mm respectively. As expected, it can be seen that with increasing depth the particle counts for each particle size from 1 to 10 microns were progressively reduced. Figure 6.61 (a) and (b) illustrate the same results from surging filter two for comparison. Close inspection of these trends revealed that the gaps between the raw water particle function and the functions at depths of 50, 100 and 200 mm were smaller in the surging filter. These trends showed that filter two removed less particles at each particle size from 1 to 10 microns compared with control filter one and compared with filter two performance in the control test run twenty five 5 hours into the test. For additional clarity and comparison with the results of run twenty five above, figure 6.62 (a), (b) and (c) illustrates the removal efficiency of particles in the 3 band sizes described previously for filters one and two at depths of 50, 100 and 200 mm for surging run twenty seven 5 hours into the test. It can clearly be seen that filter two removed less particles than filter one at depths of 50, 100 and 200 mm for each band size of 1-3, 3-7 and 7-12 microns. Comparison with the turbidity removal trends discussed earlier in this chapter for surging run twenty seven

(a)



(b)

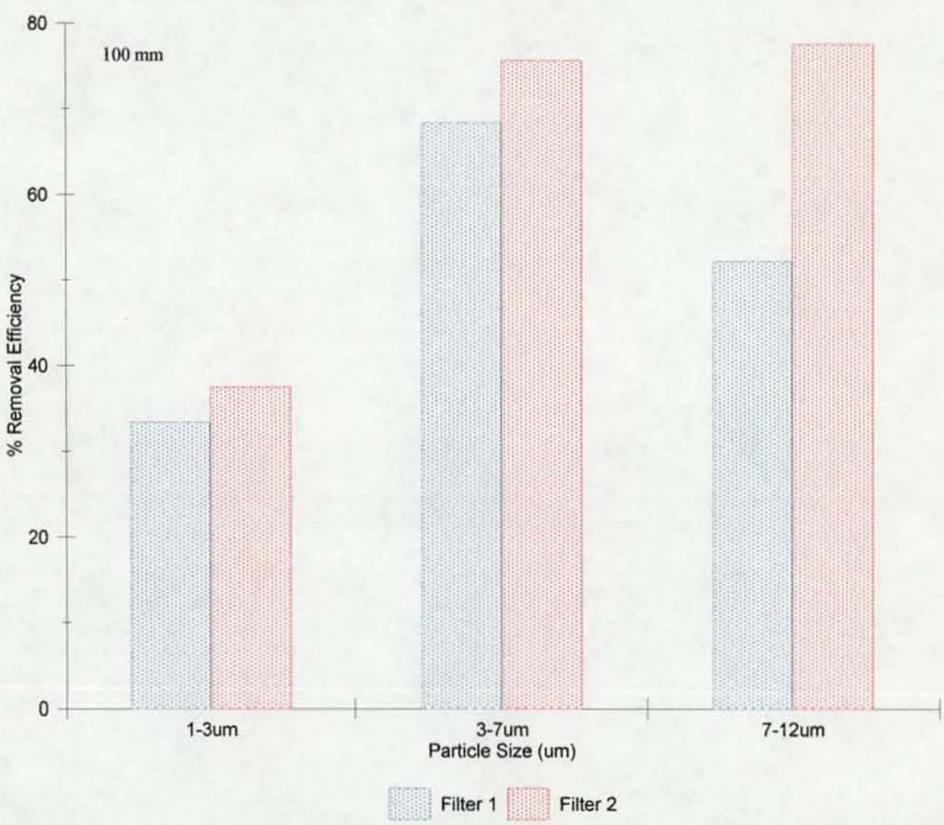
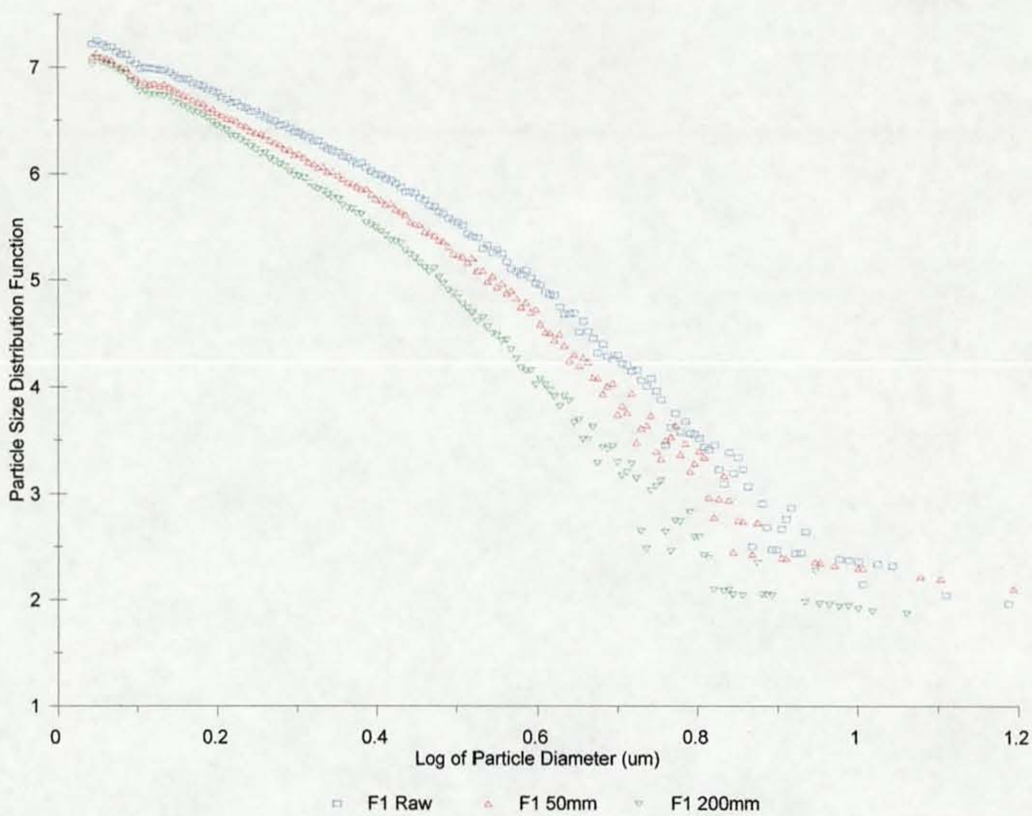


Figure 6.59 (a) & (b): PSD Histograms at 50 and 100mm for Run 25

(a)



(b)

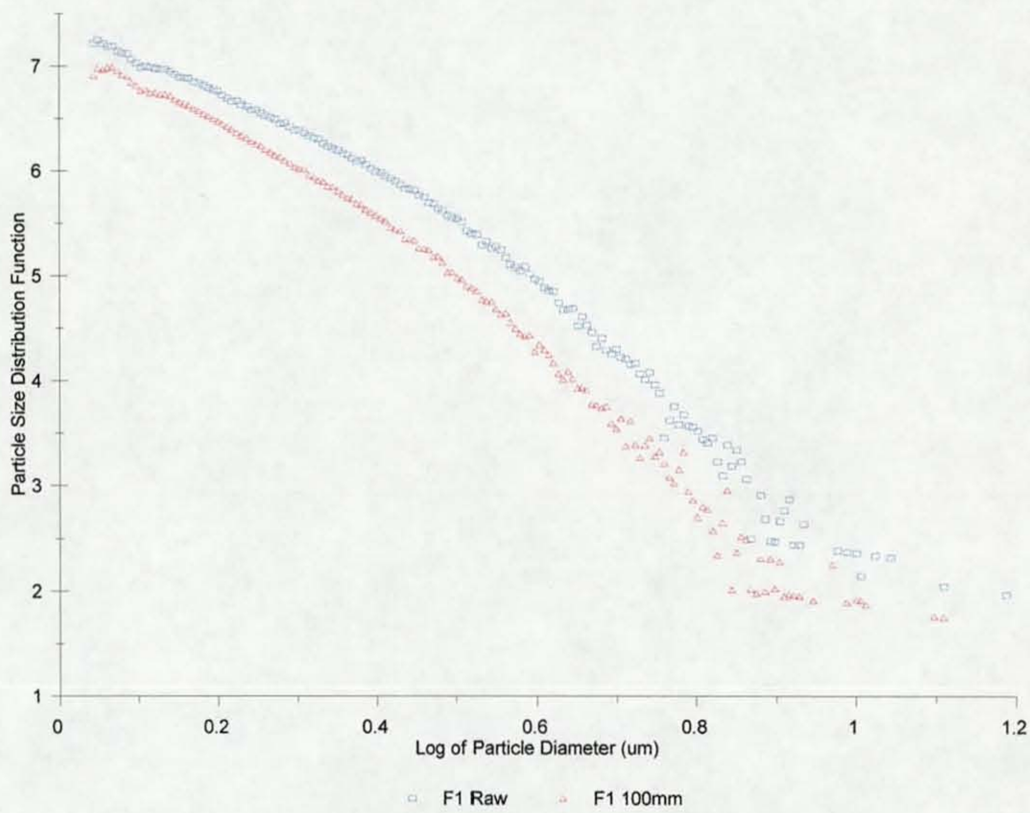
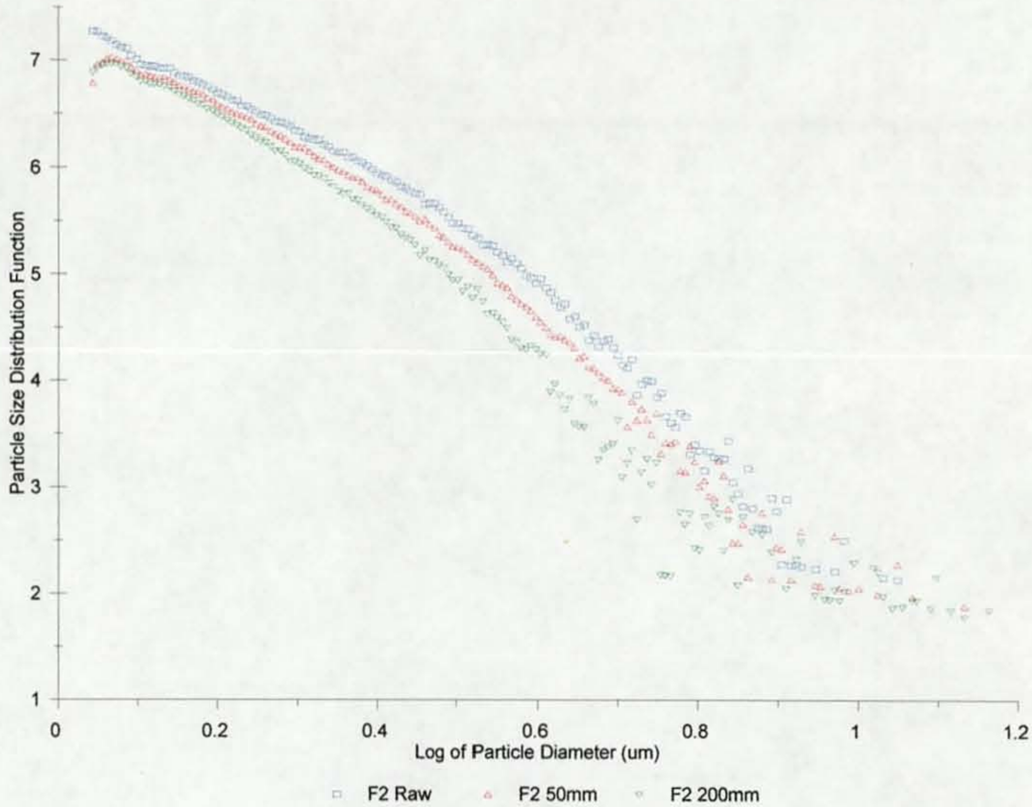


Figure 6.60 (a) & (b): PSD Functions for Filter 1 Run 27

(a)



(b)

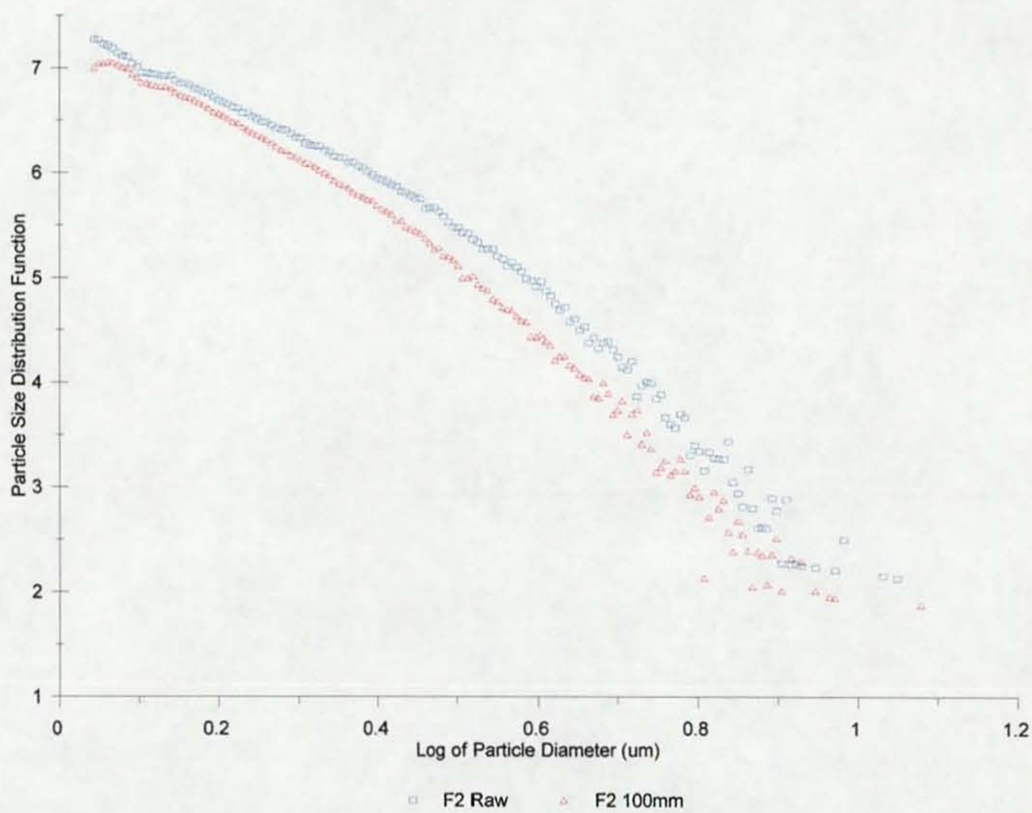
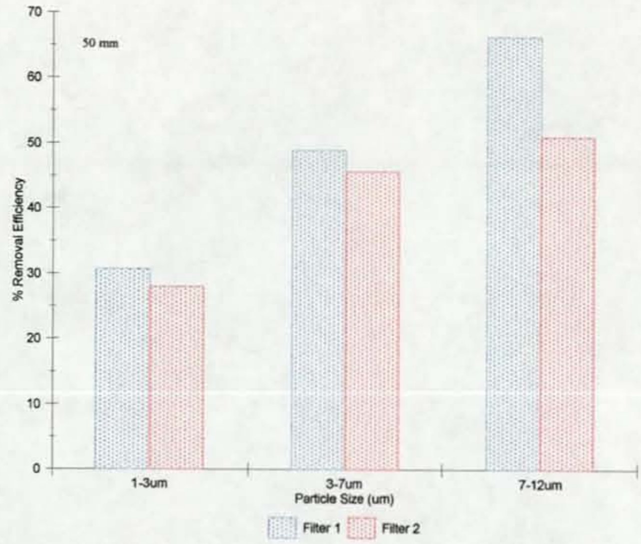
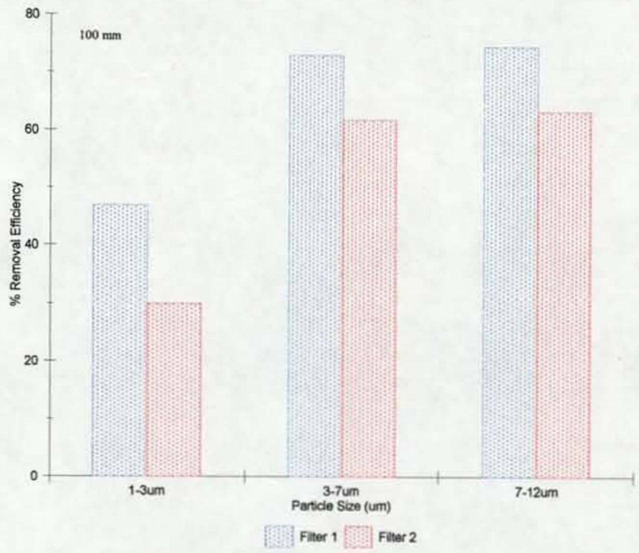


Figure 6.61 (a) & (b): PSD Functions for Filter 2 Run 27

(a)



(b)



(c)

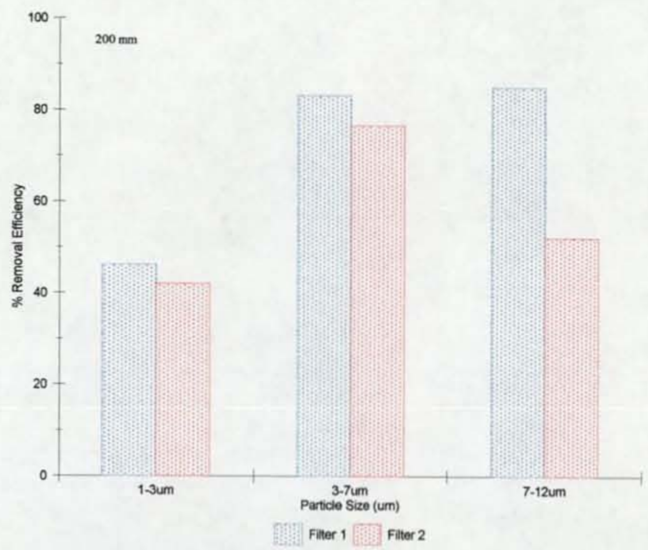


Figure 6.62 (a), (b) & (c): PSD Histograms at 50, 100 & 200mm for Run 27

show that these particle findings correlate with the poorer turbidity removal observed in filter two at depths of 50, 100 and 200 mm at 5 hours run time. Clearly, the presence of the fixed amplitude surges of 100 mm amplitude at a rate of occurrence of 80 oscillations per minute inhibited the capture of suspended particles. There was, however, no evidence that the surging flow influenced the removal performance of any one particle size more or less than any other size.

During the course of the experimental programme, other observations were made of filter particle removal performance that were of interest but were unrelated to the effects of surging. In control run twenty three, particle samples were collected from above the filter media and at a depth of 100 mm at 5, 10.5 and 18 hours from the start of the test. Figure 6.63 illustrates the particle size distribution functions for these sample analyses from filter one. In these trends there is evidence of filter ripening and particle breakthrough similar to the findings of Moran *et al* (1993a) and evidence of a minimum particle removal efficiency around 1 micron similar to the findings of Yao *et al* (1971), both discussed in section 2.4. It can be seen in figure 6.63 that similar processes occurred in filter one at 100 mm depth. Here, it was evident that particle removal improved with time from 5 hours to 10.5 hours for intermediate particle sizes of 3 to 7 micron but from 10.5 to 18 hours the removal of these particles then declined. 1 to 3 micron particles did not exhibit ripening or breakthrough during this time. It would appear that detachment of these particles in the top 100 mm and perhaps breakthrough of influent particles in this size range occurred during this test similar to the observations of Moran *et al.* As discussed earlier, this breakthrough of intermediate particle sizes has implications for *Cryptosporidium sp.* removal by rapid filtration. It also appeared that there was a greater number of larger size particles of 7-12 micron present at a depth of 100 mm at 5 hours than was present in the raw suspension at 18 hours. However, this could be the result of the drop in influent concentration that occurred during test runs caused by attachment of the PVC particles to the system piping as discussed in section 5.1. Yao stated that sub-micron particles were effectively transported to the surface of the sand grains by diffusion processes and greater than one micron size particles were transported across the streamlines by sedimentation processes. The result was an observed minimum removal efficiency for particles near to 1 micron in diameter. From figure 6.63, there appeared to be a similar removal minimum for particles near to one micron in size in

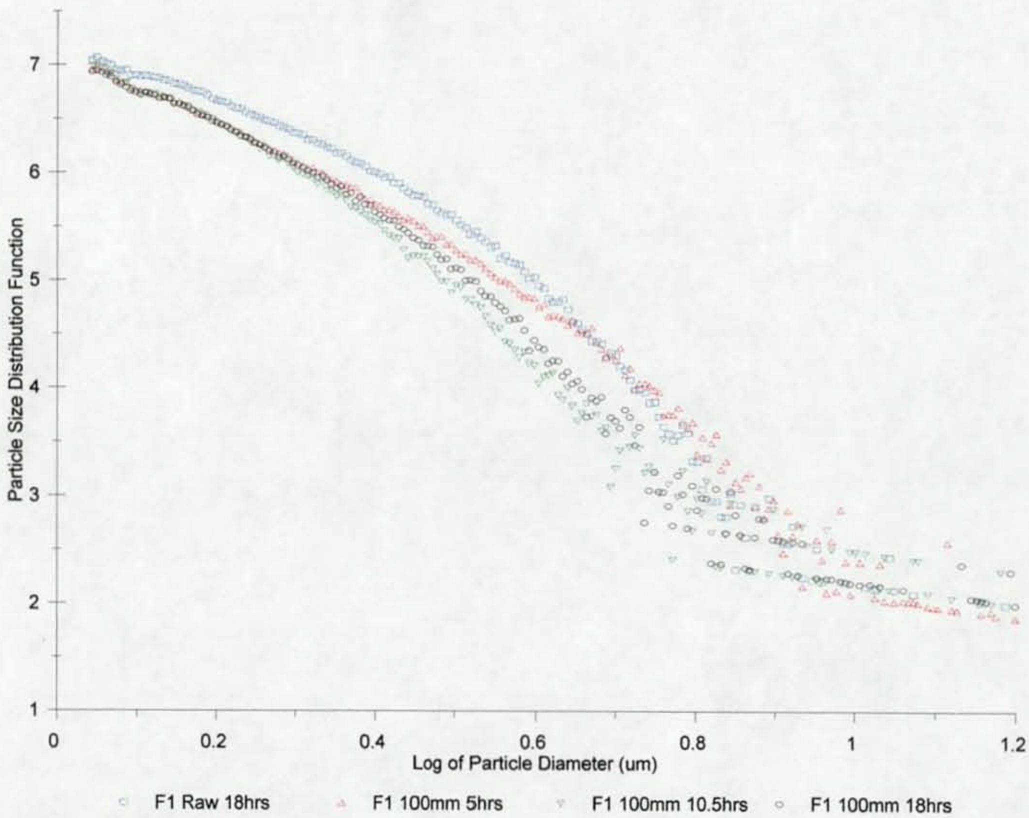


Figure 6.63: PSD functions at 5, 10.5 & 18 Hours F1 100 mm Run 23

run twenty three. This pattern was also noted in the results from other test runs described above.

6.2.4 Discussion of Results

As discussed in section 2.1.4, Baylis (1958) compared the performance of two similar but not identical pilot scale filters with one subject to surging flow. He found that over a period of nine months the surging filter produced poorer quality filtrate. No details of the effect of the surges during the filter cycle were given to compare with. No details of the surging characteristics present were given to compare with but the difference in performance was of the order of 0.1 ppm suspended solids in the filtrate. Baylis believed that the surges were the cause but could not conclude so with certainty since the filters were not identical and were not operated under well controlled conditions. Hudson (1959), also discussed in section 2.1.4, observed increasing amplitude surges with rising head loss in a constant rate filter and noted that the amplitudes were less in a comparable declining rate filter. Hudson believed that the smaller amplitude surges in the declining rate filter resulted in less tendency to breakthrough towards the end of the filter cycle. He also believed that an observed 20% improvement in water quality from the declining rate system over a year long comparison with the constant rate system was due to the smaller surges present. However, Hudson could not state these findings with any certainty. Hudson, (1963) later stated his belief that the presence of surges would lengthen filter runs by reducing the retention of solids within the filter (see section 2.1.4). A reduction in solids retention would slow the rate of head loss development and produce longer run times before terminal head loss was reached. In this laboratory investigation, the effect of surging flow has been confirmed under well controlled conditions and the mechanisms examined. The presence of small, rapidly occurring fluctuations in the flow rate did indeed produce a poorer quality filtrate as measured by both turbidity and particle counts confirming the beliefs of both Baylis and Hudson described above. It was found that the magnitude of the deficit in turbidity was influenced by the amplitude and frequency of occurrence of the fluctuations in the flow rate. The greater the magnitude and frequency of occurrence of the surges the more significant the deficit in filtrate quality. This adds weight to Hudson's argument that constant rate filtration can generate poorer quality filtrate due to greater surges in the flow rate than in declining rate filtration. The deficit in filtrate turbidity was present throughout the filter cycle and generated head loss at a slower rate than the non-surging filter. This confirms Hudson's belief that surges produce longer filter runs. However, this is at the expense of poorer quality filtrate throughout the run.

The observations of a number of researchers on the effects of discrete changes in the flow rate through rapid filters were reviewed in section 2.2. Cleasby (1960) first noted that step changes in the flow caused previously retained solids to penetrate through filters. Similar observations were made by Munson (1963). Cleasby and Baumann (1962) found that flow changes caused by sticky float controllers produced solids breakthrough and a drop in head loss. Similar poor filter performance was attributed to flow fluctuations caused by float controllers by Hilmoe and Cleasby in 1986 (see section 2.2). Surges are different in nature from these discrete flow changes in that they are much smaller but occur continuously throughout the filter cycle. Similar to the discrete changes described above, the surges applied to the experimental filters in this study produced higher turbidity and particle counts and lower head losses. However, unlike the short term, transient turbidity breakthrough and drop in head loss resulting from step increases in the flow rate the effect of the surging flow on turbidity and head loss was present continuously from start to finish. From the particle results presented in section 6.2.3, it is clear that the surges inhibited particle attachment throughout the filter depth similar to the effects on turbidity removal. However, there were no clear indications that the surging flow influenced one particle size more or less than any other particle size. The surges produced a continuous deficit in turbidity removal during the filter cycle. Even a small deficit in turbidity removal can be significant since several researchers have reported that small changes in turbidity were matched with larger changes in particle concentration. Beard and Tanaka (1977) found that a 3 fold increase in turbidity corresponded to a 10 fold increase in the number of particles of 2.5 micron and above. Similarly, Logsdon *et al* (1981) found an increase of 0.1 NTU correlated with a 10 to 50 times increase in the number of *Giardia sp.* cysts of 8 to 12 micron size. Although the particle samples were collected throughout the filter depth at one sample time only it is certain that the surges have inhibited particle removal continuously with time too. It is not known whether the surging flows applied influenced the ripening and breakthrough of different particle sizes with time and depth due to the limited sample data acquired.

However, it seems likely that the continuously fluctuating interstitial velocities will influence ripening and breakthrough of small and large particles in a different manner.

Cleasby *et al* (1963), reviewed in section 2.2, conducted a pilot scale study of the effects of discrete flow changes on rapid filters to characterise the mechanisms. They found that the application of flow rate increases from 10 to 100% at different rates of change caused short term solids breakthrough similar to previous observations. Additionally, it was found that the amount of solids breakthrough and the peak solids concentration in the filtrate was related to the magnitude of the change in flow and the rate of change. It was found that larger flow increases resulted in greater solids penetration with a greater peak concentration. Discrete rate increases were part of an investigation by Tuepker and Buescher (1968) on full scale filters, Logsdon *et al* (1981) on laboratory scale filters and Logsdon and Fox (1982) on dual media filters (see section 2.2). They found similar effects to Cleasby *et al.* Schleppenbach (1984) also found detrimental effects on filtrate quality from discrete flow increases. Ongerth (1990) found that flow rate increases had little effect on filtrate quality. However, this was attributed to the low turbidity raw water and small amounts of captured deposit. Barnett *et al* (1992) found little effect on filtered water quality from 33% flow increases but it wasn't clear how rapidly the changes were applied. From all these studies of discrete flow changes it was clear the amount of material which was disturbed from the bed and appeared in the filtrate was proportional to both the magnitude of the flow increase and the rate at which it was applied. Hartley and Vowels (1979) further analysis of Cleasby *et al's* results was reviewed earlier in section 2.2 and suggested that the initial flow rate before the application of an increase had little effect on the amount of solids which penetrated the filters. Pilot and full scale experiments by Hartley and Vowels gave similar conclusions to Cleasby *et al* and Tuepker and Buescher. In addition, Hartley and Vowels found that the impact of a rate increase was more significant later in the filter run when the filter had accumulated more attached deposits. Stevenson (1997), reviewed in section 2.2, simulated the effects of flow rate increases on rapid filtration and like previous experimental studies found that the discrete changes caused short term turbidity breakthrough and a drop in the head loss. Similar to the findings of Hartley and Vowels (1979), Stevenson found that the rate increase was more significant later in the simulated filter run when the filter had accumulated greater volumes of deposit.

In this study of surging it was found that the greater the magnitude of the surges applied the greater the deficit in the removal efficiency of turbidity in the surging filter similar to the findings of Cleasby *et al,* Tuepker and Buescher, Hartley and Vowels and Logsdon *et al* discussed previously. However, unlike these investigations the continuously occurring surges caused a continuous deficit in turbidity removal throughout the filter cycle and not just a short term breakthrough. Cleasby *et al* (1963) and Tuepker and Buescher (1968) also found that the more rapidly the flow changes were applied for any given magnitude of change the greater the amount of solids that penetrated the filter and the greater the peak solids concentration. Similarly, in this study of surging flow the more rapidly occurring pressure fluctuations produced a greater deficit in turbidity removal but once more continuously throughout the filter cycle and not just in the short term. Where Hartley and Vowels concluded that the effect of a discrete flow rate increase was determined by the filter age, in this study it was found that the continuously occurring surges impeded the filter removal of solids from the very beginnings of deposition. Cleasby *et al* (1963) found that the effect of an increase in the flow rate was independent of the duration of the higher flow. The amount of solids disturbed and washed through the filter was similar whether the higher flow was maintained for a long or short interval. Equilibrium of the attachment and detachment of deposits was re-established once the change in flow was over. It appeared that the disturbance of the solids occurred while the flow rate was actually changing. From this result it seems likely that if the surging flow were stopped during the filter cycle the deficit in removal efficiency would quickly be recovered. Cleasby *et al* (1963) finally noted that the impact of the flow rate increases was influenced by the type of material filtered. They found that copper catalysed iron floc passed more solids than the non catalysed floc for any given magnitude and rate of change in the flow rate. Tuepker and Buescher (1968) found that the addition of polyelectrolytes strengthened the floc and reduced the impact of the rate increases re-enforcing the results of Cleasby *et al*. In this laboratory study of surging, a suspension of non flocculated PVC particles was used. Use of artificial suspensions makes it difficult to apply quantitatively laboratory results to the field. However, it has been noted (Ives & Fitzpatrick, 1989) that non flocculated suspensions produce more robust deposits than flocculated suspensions. Deposits formed from flocculated suspensions have a more open deposit structure held together by weak hydroxide bonds and are more susceptible to fluid shear. Even with floc strengthening polyelectrolytes it seems likely the flocculated nature of the deposit will be more susceptible to rapid changes in hydraulic shear. The effect of surging flow on flocculated suspensions found in the field will probably be more significant than the effects observed in this laboratory study using a non flocculated suspension.

Cleasby *et al* (1963) suggested that the detachment of previously captured deposits must be caused by an increase in hydraulic shear resulting from the increase in the flow rate and interstitial velocity. A similar hypothesis was proposed by Logsdon and Fox (1982) discussed in section 2.2. It was argued that the greater the increase in flow rate and the more rapidly the increase was applied the greater the increase in the shear stresses acting on the captured solids within the filter pores and the more rapidly the shear stresses changed. Both the greater amplitude and rapidity of the change produced greater solids penetration of the filter. The constantly fluctuating flow rates applied to the filters in this experimental study will have a similar effect on hydraulic shear. Momentary increases in the flow rate will generate momentary increases in the interstitial velocity. The momentary greater interstitial velocity will produce greater hydraulic shear forces acting on both attached and suspended particles within the filter pores. If the increased shearing forces exceed the attachment force of the captured particles they will be re-entrained into the fluid flow and will be carried further into the filter bed. The momentary drop in shear forces resulting from a momentary drop in the flow rate may allow these detached particles to reattach but when the hydraulic shear rises once more they may be detached and driven further into bed. These fluctuations in the hydraulic shear forces will also tend to prevent the attachment of incoming new particles. Where a discrete increase in the flow rate caused the detachment of particles which attached at the lower hydraulic shear before the rate change occurred, the continuously occurring fluctuations in the flow or surges will prevent the attachment of a proportion of suspended particles in the first instance. It seems probable that these continuous fluctuations in the hydraulic shear will influence different particle sizes in a different manner. Mackie *et al* (1987) recognised that changes in the interstitial velocity would affect larger particles more than smaller ones. However, no evidence that the surging flow inhibited attachment of different particle sizes to differing extents was found in this study. O'Melia and Ali (1978) and Vigneswaran *et al* (1990) reported that filter removal efficiency ripened with time since captured particles act as additional collectors leading to an increase in the rate of capture of new particles. It seems clear from this study that the continuously fluctuating interstitial velocities resulting from the surging flow reduce the initial rate of particle attachment and slow subsequent ripening.

Cleasby *et al* (1963) recommended that plant changes be made to prevent the occurrence of flow rate changes. Logsdon and Fox (1982) recommended that flow changes should be made slowly. Logsdon (1987) later advised against restarting dirty filters and believed that jerky flow changes should be avoided. Mains (1992) recommended that flow rate increases should be planned and staged over a long period where possible. Bellamy *et al* (1993) suggested that sudden flow increases be avoided where possible and that dirty filters should not be restarted without first backwashing. A filtered water turbidity goal of 0.1 NTU was recommended to minimise the risk of waterborne disease. From the results of this study, similar recommendations should be made to minimise the occurrence of surges in the flow rate through rapid gravity filters. Hartung and Tuepker (1963), discussed in section 2.2, found that flow rate increases did not have a significant effect on filtered water quality provided such changes occurred smoothly and slowly. In a similar vein, Hudson (1963) and Trussel *et al*, discussed in section 2.2, pointed out that rapid filter start up after backwash or restart after a temporary halt caused initial breakthrough of solids. This effect could be eliminated by bringing the filter on line slowly over a period of 15 to 60 minutes. Hudson recommended that flow rates should not be changed more rapidly than 3% per minute while Trussel *et al* recommended a period of filtering to waste until the filtrate quality improved. The surges applied to the experimental filters in this study were rapidly occurring similar to those observed in full scale plants (see section 2.1.3 and chapter 3). The rates of change of the flow rates were certainly greater than 3% per minute and were found to significantly influence filter performance. Additionally, surging in full scale filters is likely to be more significant since they are both rapidly occurring and erratic as opposed to slowly changing or smooth.

Coad (1982), discussed in section 2.2, found that the application of a rapid 45% increase in the flow rate through a laboratory filter caused a short term drop in the head loss, scouring of previously captured particles from the bed and modifications to the flow pathways through the clogging sand. In this study, the presence of much smaller but continuously occurring fluctuations in the flow throughout the filter cycle caused a continuous lag in the head loss and inhibited particle attachment but it is not known whether the surges influenced the flow pathways through the filter media in any way. It seems likely that the continuously occurring fluctuations in interstitial velocity will influence the development of flow pathways through the saturated upper layers of the filter bed but this will require confirmation by further study.

From the laboratory findings it is believed that the presence of surging flow in full scale filters operating under field conditions would generate poorer quality filtrate of the order of 0.1 to 0.2 NTU throughout the filter cycle. A hypothetical illustration of this performance is shown in figure 6.64 and 6.65. Here, non-surging flow at an approach velocity of 8 metres per hour is shown to generate a filtered water quality of 0.2 NTU. Periodically, the flow rate exhibits step changes from 8 to 12 metres per hour as can occur from filter backwashing. The effect on the filtered water quality is shown as short term turbidity spikes up to 0.6 NTU in magnitude. Surging flow is shown in figure 6.58 as small continuously occurring flow rate fluctuations superimposed on the steady state flow of 8 metres per hour. The effect of this fluctuating flow is shown in figure 6.59 as a deterioration in the filtered water quality of 0.1 NTU throughout the filter cycle. The filter still generates spikes in the water quality from the step changes in the flow but it is believed that the marginally poorer quality filtrate resulting from the surging flow could be more substantial in terms of total solids penetration of the filter integrated across the whole filter cycle.
The effects of surging described above could have significant implications for the penetration of pathogens such as *Giardia sp.* and *Cryptosporidium sp.*. These organisms and outbreaks associated with drinking water supply were reviewed in section 2.3. Logsdon *et al* (1981), discussed in section 2.2 found that rapid flow rate increases of 50 to 150 % caused turbidity breakthrough and an increase in the concentration of *Giardia sp.* cysts. The turbidity rose by a factor of 4 as a result of the flow increases. However, the corresponding cyst concentration was found to increase more than 20 times. A similar result was obtained using coliforms (Logsdon & Fox, 1982), discussed in section 2.2. The authors concluded that turbidity changes of as little as 0.05 NTU were associated with a large increase in cyst concentrations where present in the raw water. The surges applied in this study caused a deterioration in filtered

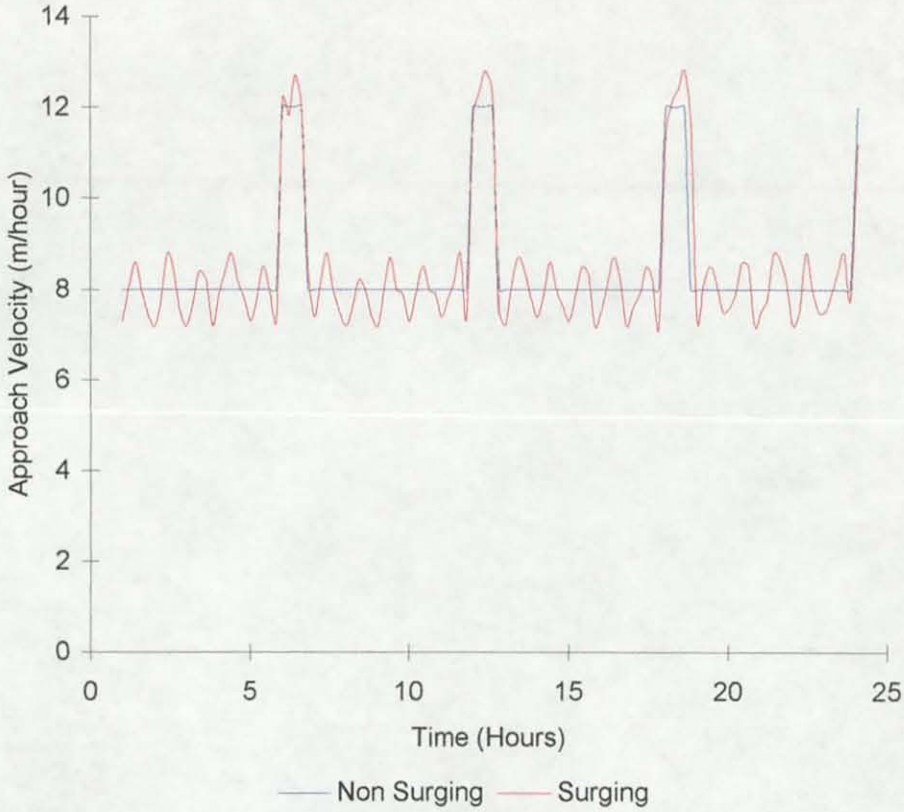


Figure 6.64: Hypothetical Non-Surging and Surging Flow in Full Scale Filters

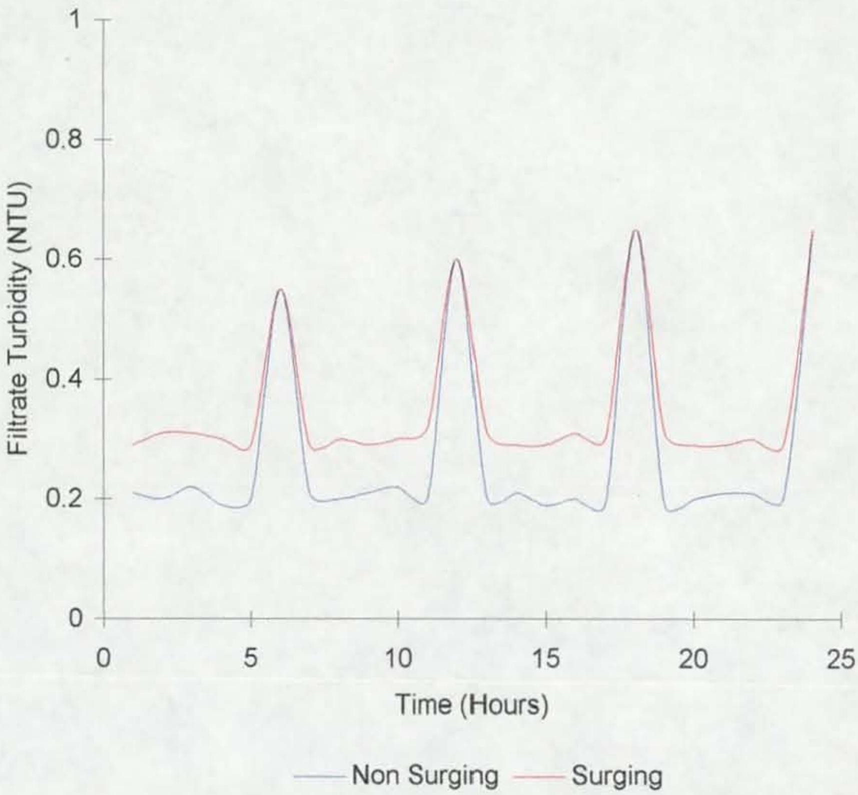


Figure 6.65: Hypothetical Effect of Non-Surging and Surging Flow on Full Scale Filters

water turbidity. As discussed earlier, the effect of surging on full scale filter performance is believed to be a continuous deficit in turbidity removal of around 0.1 NTU depending on the nature of the surges present. Such a deficit in turbidity removal throughout the filter cycle is equivalent to a large number of particles entering the filtered water supply which will include pathogen cysts if present in the raw water. Lin (1985), discussed in section 2.3, reviewed the nature of *Giardia sp.* and described a number of outbreaks associated with the water supply. In one such outbreak, the filtered water met the turbidity and coliform standards. Similar cases were described by Braidech & Karlin (1985) and Logsdon *et al* (1985). The need for effective filter performance was stressed to protect the public. By minimising the occurrence of surging flow, the efficiency of filter performance can be further enhanced to safeguard against pathogens. Bellamy *et al* (1985a) and Seelaus *et al* (1986), discussed in section 2.3 reported that slow sand filtration removed virtually 100 % of *Giardia sp.* cysts. It is probably the case that slow sand filtration is less sensitive to surging flow than rapid filtration. The lower approach velocities used in slow filtration will result in smaller interstitial velocities and thus smaller shear stresses acting on the captured solids. Fluctuations in interstitial velocity in slow sand filtration will be much smaller and have a less significant effect on filtrate quality than in rapid filtration and this may have contributed to the better pathogen removal performance found. Mosher and Hendricks (1986) reported that rapid filtration can effectively remove *Giardia sp.* cysts with proper pre-treatment. Poor pre-treatment and thus weak floc led to a larger particle count in the filtered water. However, Horn *et al* (1988) noted that even effective pre-treatment can allow small numbers of pathogens to reach the filtered water supply. A similar conclusion was reached in the Badenoch report (1990). This correlates with the findings of Cleasby *et al* (1963) and others discussed earlier where the effect of rate changes was influenced by the nature of the flocculated matter entering the filter. Nevertheless, whether the floc is weak or strong the presence of fluctuating hydraulic shear will reduce the rate of attachment and produce poorer quality filtrate. If pathogens are present, surging will certainly contribute to their penetration into the filtered water.

Rose (1988), discussed in section 2.3 earlier, reviewed the nature of a second pathogen; *Cryptosporidium sp.*. It was reported that oocysts of the organism are common in surface waters and have been found in treated drinking waters. A number of similar surveys were reviewed in section 2.3 and came to similar conclusions concerning pathogenic protozoans (Lechevallier *et al*, 1991; Rose *et al*, 1991; Smith *et al*, 1993; Kfir *et al*, 1995; Edzwald and Kelly, 1998; Karanis *et al,* 1998). Major concerns with *Cryptosporidium sp.* are that the infective dose is believed to be small and the oocyst is resistant to conventional chlorine disinfection. The presence of even small numbers of this organism in drinking water supplies has the ability to infect large numbers of the population (Smith, 1992). Numerous outbreaks of Cryptosporidiosis associated with filtered drinking water were reviewed in section 2.3 (Hayes *et al*, 1989; Logsdon *et al,* 1990; Richardson *et al,* 1991; Joseph *et al,* 1991; Leland *et al,* 1993; Mackenzie *et al,* 1994). Hayes *et al* and Logsdon *et al* concluded that restarting dirty filters without backwashing was a major factor in the cause of the Carrollton outbreak. They recommended that filters should be washed and restarted slowly. They also recommended that flow changes should be minimised where possible and a target of 0.1 NTU be adopted for filtered water quality. The findings from the Jackson County outbreak of Cryptosporidiosis support this target of 0.1 NTU (Leland *et al,* 1993). The filtered water quality barely met the USEPA standard of 0.5 NTU during the outbreak. However, modifications to the plant improved the quality to 0.1 NTU and the incidence of illness drop off rapidly. Periods of poor filtered water turbidity were believed to have contributed to the Milwaukee outbreak of Cryptosporidiosis reported by Mackenzie *et al* (1994). Similarly, the investigation recommended a target of 0.1 NTU for filtered water. The pilot scale study of Ongerth and Pecoraro (1995) found that a filtered water turbidity of less than 0.1 NTU was equivalent to 3 log removal of pathogen cysts whereas a filtered water turbidity of 0.36 NTU was equivalent to only 1.5 log removal. Edzwald and Kelly (1998) and Karanis *et al* (1998) too supported a 0.1 NTU filtered water target. From the results of this study, the presence of surging should be minimised where possible to minimise the effects on filtered water quality and achieve a target turbidity of 0.1 NTU to guard against pathogens. A possible relationship between filter gallery design and the degree of surging present was described in chapter 3 for local full scale plants. The degree of surging present appeared to be related to the complexity of the filtered water flow path from the underdrain to the clearwell and it is recommended that dead ends, T fittings and bends be avoided where possible in future to help minimise the occurrence of surges. Where possible the Reynolds number should be minimised to help reduce surging flow further. It is believed that the presence of surging in full scale filters will make the achievement of a 0.1 NTU filtered water turbidity difficult.

The Oxford/Swindon outbreak of Cryptosporidiosis described in section 2.3 by Richardson *et al* (1991) led to the Badenoch report (1990). In this report, it was concluded that current conventional treatment cannot guarantee complete oocyst removal and this is a cause for concern since *Cryptosporidium sp.* has a low infective dose. In the report it was again recommended that flow changes be avoided since they contribute to the penetration of pathogens. This recommendation should be extended to the surging flow rate changes investigated in this study.

Rose (1988) noted that oocysts present in the raw water were concentrated in the deposits captured in rapid filtration. In this study, surging flow was found to reduce the removal of incoming suspended particles. As such, the presence of surging in full scale filters has the ability to assist the penetration of oocysts of *Cryptosporidium sp.* where present in the raw water. Rose also noted that turbidity and coliform counts may be inadequate indicators of *Cryptosporidium sp.* removal. A similar conclusion was reached by Smith, 1992. The occurrence of outbreaks where the filtered water met the turbidity and coliform standards seems to confirm this. Indeed, the turbidity and plate counts from the Oxford/Swindon outbreak of Cryptosporidiosis did not show a problem (Badenoch, 1990) and Joseph *et al* (1991) concluded that low turbidity did not necessarily imply an absence of pathogens. The results of a survey by Lechevallier *et al* (1991) found *Giardia sp.* and *Cryptosporidium sp.* oocysts frequently in filtered waters with turbidity less than the USEPA standard of 0.5 NTU. Gregory (1994) noted that waters with a turbidity of less than 0.1 NTU still contained thousands of particles per millilitre. From the experimental results of this study, surging can cause a continuous small degradation in filtered water quality throughout the filter cycle. This effect could contribute to low level contamination of the filtered water supply with pathogen cysts if present in the source water without triggering high turbidity or coliform alarms. The detection of even small numbers of oocysts should be cause for concern since current recovery and detection techniques underestimate oocyst counts and the presence of even one oocyst should be considered infectious (Smith, 1992). Gregory (1994) reported that turbidity measurement was particle size dependent and was less sensitive to the presence of larger sized particles above one micron. He noted that the presence of cysts of *Giardia sp.* and *Cryptosporidium sp.* in drinking water supplies were of concern and were greater than one micron in size. Darby and Lawler (1990) and Moran *et al* (1993a & b) reported that the removal efficiency of particles was size dependent and that intermediate particle sizes corresponding to *Cryptosporidium sp.* oocyst sizes broke through filters earlier than smaller particle sizes. These authors pointed out that turbidity measurement did not identify these factors. The reasons for this were explained by Gregory's work (1994). Concern was expressed that intermediate size particle breakthrough could occur without significant change in the measured turbidity. Hutchinson (1985) noted that particle breakthrough could precede turbidity rise by several hours. Gregory concluded that particle analysis was a much more sensitive tool than turbidity measurement. In this experimental study of surging flow, the presence of surges caused a continuous deterioration in filtered water turbidity and poorer particle removal efficiency. No evidence was found that the surging flow influenced particle removal efficiency on the basis of particle size. However, the particle size distribution sampling used in this study was limited in its extent. It is possible that surges influence the ripening and removal of different particle sizes in a different manner during the filter cycle. The presence of continuous fluctuations in the interstitial velocity may accelerate the early breakthrough of intermediate sized particles with implications for the effective removal of pathogens such as *Cryptosporidium sp.* and *Giardia sp.*. A more comprehensive particle sampling regime will be required to identify such effects.

6.3 Effect of Suspension Chemistry on Filter Performance

During the course of this study of surging in rapid filters it was noticed that filter performance was influenced by the test suspension dissolved solids concentration and Zeta potential. Table 6.11 below summarises the effects of these parameters on the rate of head loss development and removal efficiency of turbidity in filter one at the end of tests twenty five to thirty two.

	Run 25	*Run 26*	*Run 27*	*Run 28*	*Run 29*	*Run 30*	*Run 31*	*Run 32*
HL Growth Rate (cm/min)	0.14	0.34	0.21	0.25	0.21	0.23	0.16	0.23
% Removal	75	92	80	90	68	87	88	93
pH	8.2	7.7	7.8	7.0	7.5	7.4	7.2	8.0
Cond. (uS/cm)	708	310	635	512	643	625	656	584
Zeta Pot. (mV)	-16.6	-18.9	-20.0	-16.1	-24.0	-17.5	-15.9	-16.2

Table 6.11: Comparison of Rate of Head Loss (HL) Growth & % Removal of Turbidity in Filter 1 and Suspension pH, Conductivity & Zeta Potential

Close inspection of this data clearly showed an inversely proportional relationship between the rate of head loss development and the suspension conductivity. Similarly, there appeared to be an inversely proportional relationship between the removal efficiency achieved and the suspension conductivity but complicated by the magnitude of the Zeta potential. There did not appear to be any relationship between the filters' performance and the suspension pH. Samples of the mains water used to generate the test suspension in test runs twenty five to thirty two were submitted to inductively coupled plasma (ICP) spectroscopy and ion chromatography to determine the dissolved solids present in the test suspension. The methods used for these analyses were reviewed in chapter 5. It was intended that these analyses coupled with the conductivity and Zeta potential measurements would provide characterisation of the composition of the suspension used in each of the test runs. It was intended to use this composition data to determine relationships between the chemical nature of the suspension and the experimental test results. These relationships could then perhaps be used to normalise the experimental results and eliminate the variation in test performance produced by the fluctuations in the test suspension chemistry. Elimination of the influence of the suspension dissolved solids and Zeta potential by normalisation of the data would enable the effects of the surges to be quantified more easily. Table 6.12 below details the concentration of a range of dissolved elements detected in the mains water from each test run by ICP spectroscopy.

(ppm)	*Run 25*	*Run 26*	*Run 27*	*Run 28*	*Run 29*	*Run 30*	*Run 31*	*Run 32*
Mg	12.9	8.4	12.3	10.5	11.8	13.1	15	13.1
Ca	77.2	33.5	48.2	53	60	58.8	63.3	58.2
Zn	0	0.06	0.05	0.15	0.06	0.06	0.05	0
Sr	0.29	0.1	0.14	0.14	0.2	0.22	0.21	0.2
B	0.1	0	0.13	0.17	0.15	0.13	0.13	0.12
Na	28	9.8	39.6	25	40	37.4	38.1	35.9
Si	2.7	2.4	2.7	2.6	1.3	1.8	2.2	2.3
P	0.75	0.4	0.67	0.45	0.76	0.82	0.83	0.7

Table 6.12: ICP Spectroscopy Analysis of Mains Water used for each Test

From this table, it can be seen that the major elements present in the mains water from each test were Magnesium, Calcium and Sodium with trace quantities of Zinc, Strontium, Boron, Silicon and Phosphorous. Magnesium concentrations ranged from 8.4 to 15.0 ppm. Calcium concentrations ranged from 33.5 to 77.2 ppm. Sodium concentrations ranged from 9.8 to 40.0 ppm. Table 6.13 below details the concentration of the major anions and cations determined by ion chromatography in the mains water from each test run.

(ppm)	Run 25	Run 26	Run 27	Run 28	Run 29	Run 30	Run 31	Run 32
Fluoride	0.2	0.16	0.47	0.05	0	0.363	0	0.36
Chloride	53.3	18	53.6	40.4	51	51.3	56.8	47.5
Nitrate	19.3	5.6	20.2	9	20.1	19.8	20.5	17.4
Phosphate	2.19	1	1.77	0.26	2.11	2.11	2.16	2.02
Sulphate	111.7	44.9	79.8	45	90.2	98.6	100.8	84.2
Sodium	44.1	11.1	45.4	31.6	43.2	37.9	45.1	38.7
Potassium	6.07	0.65	2.74	2.44	3.16	3.49	4.07	3.7
Magnesium	14.8	8	13	9.9	14.2	13.2	15.3	13.6
Calcium	83.8	37.1	67.7	48.9	61.7	61.4	69.5	61.9

Table 6.13: Ion Chromatography Analysis of Mains Water for each Test

It can be seen that the major anions present in the mains water during the test programme were chloride, nitrate and sulphate with traces of fluoride and phosphate. Chloride concentrations ranged from 18 to 56.8 ppm. Nitrate concentrations ranged from 5.6 to 20.5 ppm. Sulphate concentrations ranged from 44.9 to 111.7 ppm. The major cations present were sodium, potassium, magnesium and calcium. Sodium concentrations ranged from 11.1 to 45.4 ppm. Potassium concentrations ranged from 0.6 to 6.1 ppm. Magnesium concentrations ranged from 8 to 15.3 ppm. Calcium concentrations ranged from 37.1 to 83.8 ppm. Comparison of these Calcium and Magnesium concentrations with the above ICP results showed good correlation.
A range of regression analyses were conducted with this data to identify any statistical correlation between the suspension characteristics and filter performance. Simple and multiple linear regression was performed with the rate of head loss growth and removal efficiency as the dependent variables and the suspension conductivity and Zeta potential as the independent variables. Table 6.14 summarises the correlation coefficients found from each analysis.

Correlation Coefficients	*Conductivity*	*Zeta Potential*	*Conductivity & Zeta Potential*
Rate of HL Growth	**0.95**	0.18	0.96
% Removal of Turbidity	0.63	0.67	**0.92**

Table 6.14: Correlation coefficients for simple & multiple regression analyses

Strong statistical correlation was found between the rate of head loss development and the suspension conductivity. Simple linear regression gave a correlation coefficient of 0.95 with a critical value of 0.87 for 99% confidence. The regression equation is given below and the regression line is illustrated in figure 6.66 (a).

$$RHL = 0.49 - 0.00046(C)$$

where RHL is the rate of head loss development (cm/minute)

C is the suspension conductivity (uS/cm)

Strong correlation was found between the rate of head loss growth and the combination of suspension conductivity and Zeta potential but the relationship failed the t test for the Zeta potential regression coefficient and was therefore discarded (the t test determines the usefulness of the regression coefficient for each independent variable in predicting performance).

Strong correlation was found between the removal efficiency and the suspension conductivity and Zeta potential. Multiple linear regression analysis gave a correlation coefficient of 0.92 with a critical value of 0.88 for a 95% confidence and passed the t tests for each regression coefficient. The regression equation is given below and is illustrated in figure 6.66 (b).

$$RE = 144.7 - 0.04(C) + 2.05(\varsigma)$$

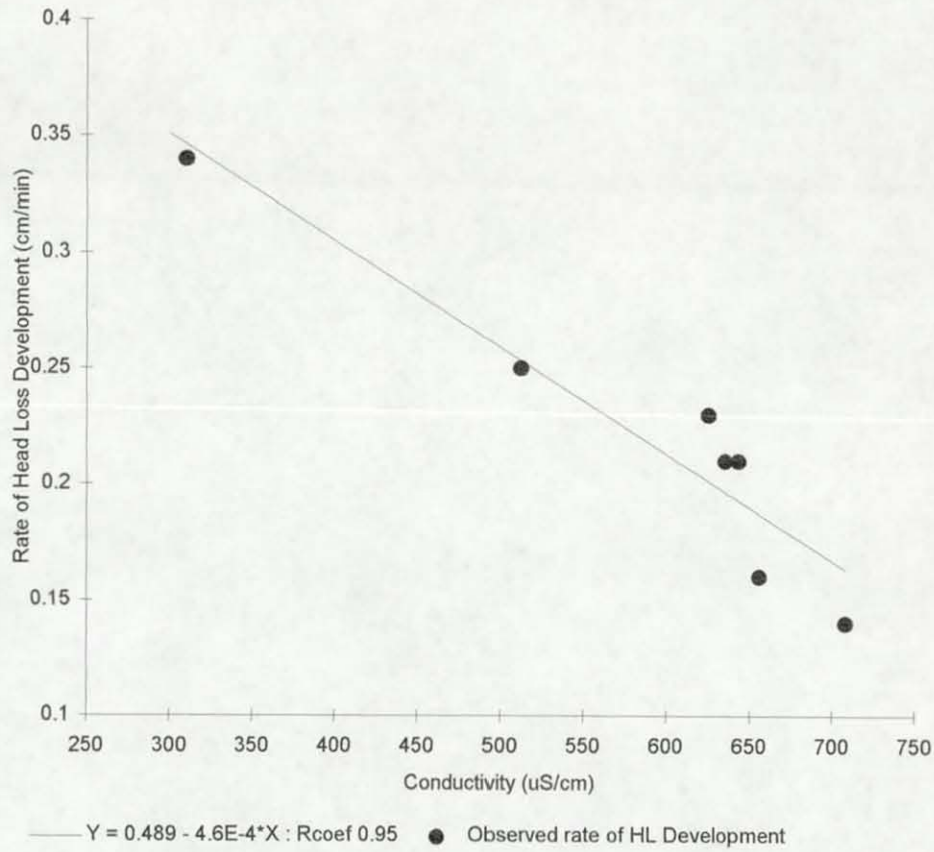
where RE is the % removal efficiency of turbidity

C is the suspension conductivity (uS/cm)

ζ is the suspension Zeta potential (mV)

Regression analyses were conducted with the spectroscopy and chromatography data described earlier to determine the most significant components of the dissolved solids present in the mains water used to generate the test suspension. Strong statistical correlation was found with the Calcium, Magnesium and Sodium cations and the Chloride and Sulphate anions in the analyses with the rate of head loss development and removal efficiency. This was to be expected since these ions were present in the greatest concentrations in the mains water. However, the most statistically significant relationships were found with the suspension conductivity as a measure of the total

(a)



(b)

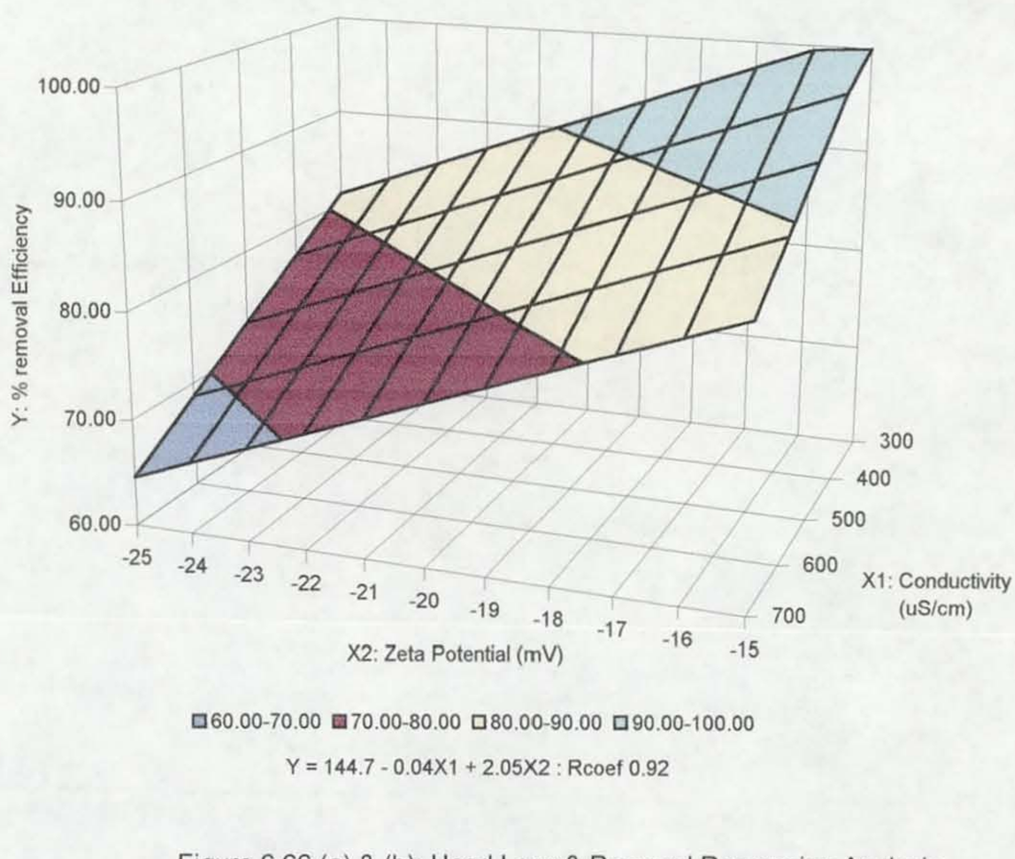


Figure 6.66 (a) & (b): Head Loss & Removal Regression Analysis

dissolved solids present as opposed to the concentration of any individual element or ion. The regression equations given above for the rate of head loss development and removal efficiency as functions of the suspension conductivity and Zeta potential were used in a normalisation procedure to modify the test results to a standardised suspension conductivity and Zeta potential. This process was undertaken to eliminate the suspension conductivity and Zeta potential as sources of variation in the filter performance to facilitate further quantification of the effects of the differing surge characteristics on the performance of rapid filtration. The details of the normalisation undertaken are presented in Appendix B. However, it was concluded that normalisation of the experimental data on the basis of the statistically determined relationships presented above was in some instances impossible and where possible was inappropriate. It was found that the process of normalisation introduced statistical errors into the experimental data of a magnitude that prevented useful quantification of the effects of the surging flow on filter performance.

6.4 Piezometer & Transducer Surging Pressure Measurement

In section 6.1 and 6.2, the surge magnitudes and rates of occurrence applied during each test run were presented both in terms of the piezometer measurements and the recorded signals from the pressure transducer. The reader may have observed that the magnitude of the surges recorded by the pressure transducer appeared to be greater than the magnitude observed in the piezometer plumbed into the base of filter two at certain surging frequencies. These observations are summarised in table 6.15 below for test runs twenty four, twenty nine and thirty.

Run	*Surge Frequency (No./minute)*	*Surge Amplitude: Piezometer (mm)*	*Surge Amplitude: Transducer (mm)**
24	80	100	780
29	80	130	900
30	40	165	169

* Amplitude calculated from output voltage and static calibration curve shown in figure 5.4

Table 6.15: Comparison of Surge Amplitude measured by Piezometer & Pressure Transducer at the Base of Filter Two at the End of Selected Tests.

As described in section 5.4, the output voltage signal from the pressure transducer connected to the base of filter two was recorded by an analogue to digital converter over a 30 second time interval periodically during each surging test run in addition to the piezometer measurements of the surge amplitude and frequency . This recorded voltage data was then converted to a pressure measurement expressed in millimetres of water by the static calibration curve presented in section 5.4. From the above table, it can be seen that the piezometer and calculated transducer surge amplitude correlated closely in test run thirty. Test run thirty was conducted with surges applied at a rate of 40 oscillations per minute. However, the surge amplitudes calculated from the transducer output voltages in tests twenty four and twenty nine were some 7 or 8 times the magnitudes observed in the piezometer tube connected to the base of filter two. These tests were conducted with surges applied at a rate of 80 oscillations per minute. In both cases, the piezometer and transducer frequencies correlated well.

A dynamic comparison of the performance of the pressure transducer and the water piezometer connected to the base of filter two was conducted to investigate this discrepancy further. Surges were applied to filter two at a range of frequencies from 30 to 80 oscillations per minute. In each case, the surge machine was adjusted to generate 100 mm surges in the water piezometer. The output voltage from the pressure transducer was recorded at each frequency for comparison with the observed 100 mm amplitude surges in the water piezometer. The output voltage from the pressure transducer was converted to pressure expressed in mm of water using the calibration chart shown in figure 5.4 previously. Figure 6.67 (a) to (f) illustrates the surge characteristics recorded by the pressure transducer at each frequency applied from 30 to 80 oscillations per minute. At 30 to 40 oscillations per minute, the magnitude derived from the transducer correlated well with the 100 mm oscillations in the water piezometer. However, it can be seen that above 50 oscillations per minute the correlation progressively broke down with increasing amplitudes recorded by the transducer. At 80 oscillations per minute the transducer surge magnitude was around 700 mm. One possibility was that the transducer device was unable to accurately track rapidly fluctuating pressures without distortion of the magnitude. However, consultation with the transducer manufacturer confirmed that the device was capable of tracking pressure fluctuations at these magnitudes and rates of change without distortion. It therefore seemed that the water piezometer was unable to respond accurately to rapidly fluctuating pressures without damping the amplitude of the observed surges. It would seem that at these amplitudes at higher surging rates the frictional and inertial damping effect of the water in the piezometer tubes became significant and the observed amplitudes were smaller than recorded by the pressure transducer. In this experimental study, the characteristics of the surges applied to the filters were chosen to reflect those observed by previous authors described in chapter 2 and the results of measurements at local water works described in chapter three. The surges applied were of similar magnitude to those observed by previous authors using the same measurement technique; the water piezometer. From this study, it seems clear that water piezometers have their limitations. They are unable to accurately track the magnitude of the pressure fluctuations applied when those fluctuations occur rapidly. The magnitude of the oscillation in the water piezometer is damped by frictional loses with the side of the tube and the inertia of the volume of water within the tube. This damping effect must also apply to the measurements taken by previous

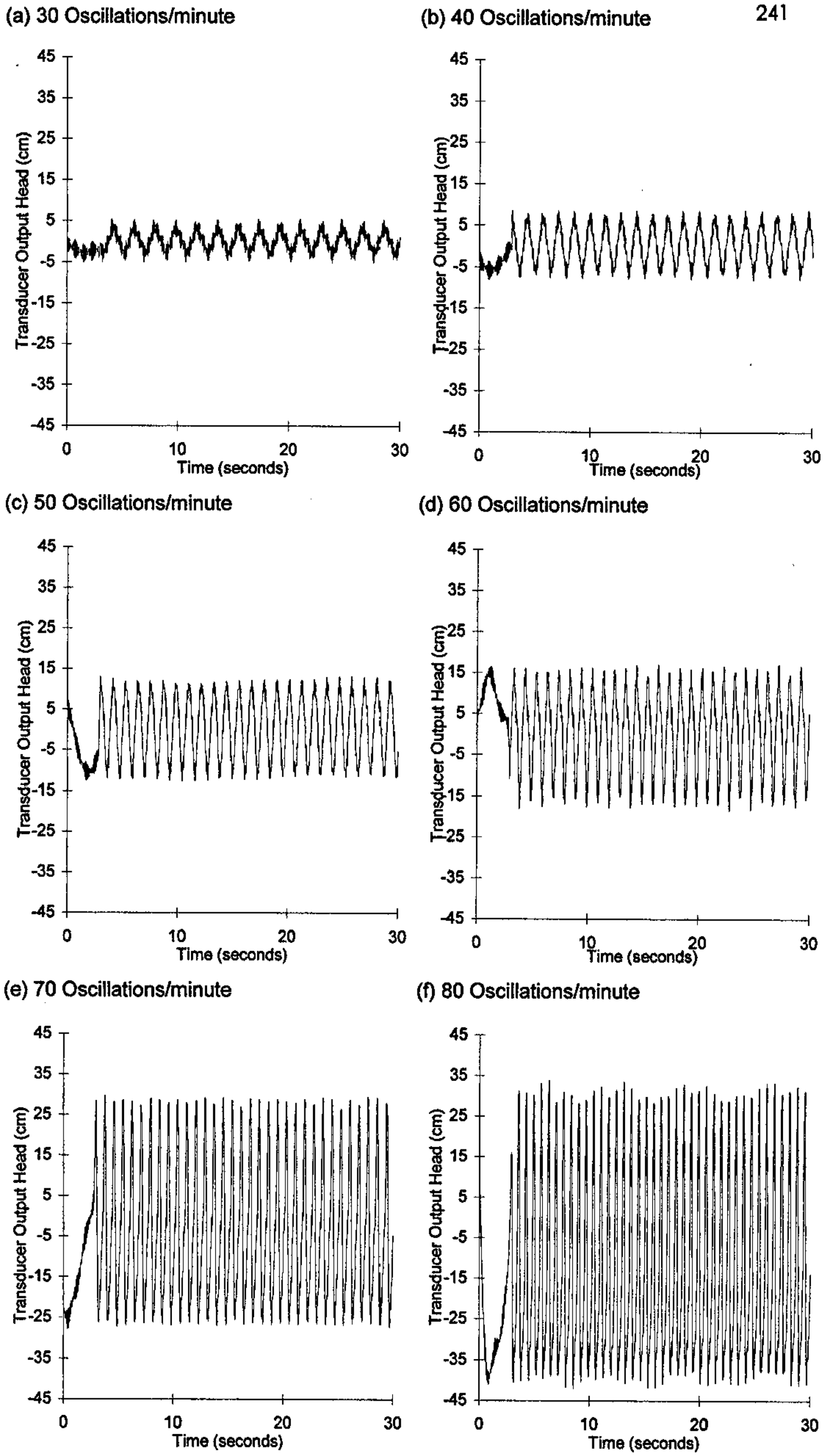


Figure 6.67 (a) to (f): Transducer Response to 100 mm Piezometer Surges at Different Rates of Occurrence

authors using piezometers. It is believed that the measurements made by previous authors described in chapter two underestimated the magnitude of the pressure fluctuations present for this reason. Nevertheless, the pressure fluctuations applied to the laboratory filters in this study were of a similar nature and amplitude to the surges found in full scale filters. In addition, it should be pointed out that these laboratory tests were conducted at elevated temperatures such that the surges applied were equivalent to much smaller flow rate fluctuations at the lower operating temperatures found at full scale plant. A useful avenue of future research would be a more comprehensive survey of surges in modern rapid gravity filters using pressure transducers in addition to water piezometers for comparison. It seems probable that the actual pressure fluctuations present in rapid filters are much larger than was previously observed due to the limitations of the measurement technique. These refined surge measurements could then be used as the basis of further experimental research into the effect of surging flow.

6.5 Chapter Summary

In this chapter the results of the experimental investigation into the effect of surges on rapid filter performance have been presented. The 32 test runs completed were described in chronological order with key events in the test program and equipment and methods development highlighted to place the results in context. Analysis of the test results was conducted and a detailed description of the effects of the surges on filter head loss, turbidity and particle removal with time and depth were given. The fixed and variable surges applied were found to inhibit turbidity and particle removal and slow the rate of head loss development within the filters. This resulted in a poorer quality filtrate throughout the test cycle. The implications of these test results were discussed with respect to previously published research in this field and the conclusions of the study and recommendations for further work are presented in the next and final chapter.

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions regarding Field Measurements

Evidence of surges in the rate of flow through rapid gravity filters was found at a number of local water treatment plants. Recordings taken from pressure transducers and flow meters illustrated that random and erratic fluctuations in the flow rate through the filters occurred continuously throughout the filter cycle. The frequency of the fluctuations in head loss was found to range from 5 to 80 oscillations per minute. The amplitudes of the head loss fluctuations were found to increase as the head loss rose with accumulating filter deposit. Insufficient data was available to be clear whether the head loss fluctuations rose at a rate greater, similar to or less than the rising loss of head. The magnitude of the fluctuations in head loss and the observed fluctuations in the rate of flow suggest that rapid surges of 5 to 10 % are common in rapid gravity filtration. These findings confirm the observations of filter surges made by previous researchers identified in the literature review.

From the limited body of data collected there seemed to be a relationship between the surging found and the design of the filtrate piping. It appeared that the magnitude and frequency of occurrence of the pressure and flow rate fluctuations were related to the complexity of the filtrate piping design. The presence of flow constrictions such as dead ends, sharp bends and butterfly control valves appeared to generate greater surging magnitudes and rates of occurrence. Previous researchers made similar observations. These constrictions in the flow produce localised high velocity zones that can generate flow separation and surging flow conditions in the filtrate piping.

To reduce the occurrence of surging flow it is recommended that constrictions in the flow be avoided where possible in the design of future plants and the upgrade of existing filters. It is recommended that the filtrate piping be designed with as few joints, bends and valves as practically possible and that larger pipe diameters be used to reduce the velocity of the flow to a minimum. Where flow constrictions are necessary they should be positioned as far from the filter box as possible. This will serve to minimise the magnitude of the surges generated within the filter media.

7.2 Conclusions regarding Experimental Techniques

Reproducible rapid gravity filter performance can be achieved in the laboratory using the techniques described in this thesis. Polyvinyl chloride (PVC) powder dispersed in mains water can be used to generate a stable suspension of reproducible turbidity and particle size distribution. This suspension can be maintained at the desired concentration and particle size distribution in continuously stirred storage tanks. The suspension temperature can be controlled to a high degree using thermostatically controlled immersion heaters and thermal insulation. The suspension flow rate applied to laboratory filters can be governed by the use of an elevated constant head tank and delivered at similar flows by identical plastic siphons to a high precision and accuracy. The concentration of the suspension delivered to the laboratory filters did decline due to deposition within the system piping. However, since the inlet concentration declined in both filters in a similar manner this was not considered to be a problem.

Variation in the mains water dissolved solids concentration and the prepared suspension Zeta potential did influence the removal efficiency performance of the laboratory filters from test to test. However, comparison with an identical control filter operated in parallel to the surging filter in each test overcame this difficulty and allowed the effect of surging flow to be clearly identified.

The research showed that identical laboratory scale filters can be constructed from Perspex pipe. A sufficiently large filter diameter was used in this study to ensure that wall effects were minimised. Careful preparation of the filter media was shown to be necessary to achieve reproducible performance in the filters. The filter media should be sieved and weighed to ensure that each filter column receives identical material in terms of both grain size distribution and mass. After the first backwashing, the filter surface may need skimming to remove fine particles that can cause surface removal instead of true depth filtration. An important factor in attaining reproducible filter performance is the packing of the media after backwash. Insufficient packing in these experiments led to filter bed movement under high hydraulic gradients. Dissimilar packing can create widely differing filter performance in laboratory filters. A mechanical shaker was used in this study in conjunction with standardised flow cut-off and temperature controlled backwash water to successfully achieve sufficient packing to prevent movement and similar media packing within the filter columns in each test. This was necessary to generate reproducible performance.

Periodic turbidity measurement and particle size distribution analysis were successfully used to monitor filter removal efficiency in this study. Suspension samples were collected using specially designed needle sample ports inserted into the filter columns at selected depths. Regular cleaning was necessary to prevent the suspension ports from blocking. Cleaning was done by backflushing with air and clean water after each test. Problems were encountered with distortion of the particle size distribution of samples stored overnight. It was necessary to analysis particle samples on the day of sampling since no effective method of storage could be developed.

Periodic pressure measurement was successfully used to monitor head loss development in this study. Pressure drop was measured using specially designed pressure ports inserted into the filter columns at selected depths, which were connected to water piezometers. Water piezometers are a simple, reliable and low cost method for pressure drop measurement. However, it has been found that piezometers were not suitable for the measurement of rapidly changing pressure due to inertial and frictional damping. Similar to the suspension sample ports, the pressure ports required regular cleaning to prevent blocking. Pressure transducers were used to record the profile of the pressure fluctuations applied to the laboratory filters. These devices were found to be reliable with no maintenance required. The transducers were not subject to damping and recorded the true pressure fluctuation magnitudes.

7.3 Conclusions regarding the Effect of Surges

Surges similar to those observed in local water treatment plants during this work and pilot and full scale filter data produced by previous researchers were found to have a detrimental effect on the laboratory scale filters used in this study. Surges of a similar magnitude and frequency of occurrence to the field observations were found to inhibit the performance of the laboratory filter compared with the control performance. The pressure fluctuations applied inhibited the initial rate of attachment of suspended polyvinyl chloride particles in the surging filter compared with the control filter. The mechanism of inhibition was believed to be hydraulic shear. Suspended particles approaching the surface of the collector sand grains will experience momentary increases in hydraulic shear resulting from the increases in interstitial flow rate caused by the surges. This short term higher shearing force reduced the rate of attachment of suspended particles and caused greater penetration of solids into the filter media. The presence of the surges also slowed the rate of improvement or ripening of particle removal in the laboratory filters. The mechanism was again believed to be the continuous momentary increases in hydraulic shear caused by the continuous shock increases in interstitial velocity. The result of the continuous fluctuations in the flow rate was the production of poorer filtered water quality throughout the filter cycle. The smaller initial removal of particles and slower rate of ripening of particle removal produced by the presence of the surges generated head loss at a slower rate than the control. This will lead to longer filter runs before terminal head loss is reached but at the expense of poorer filtered water quality. The magnitude of the initial deficit in particle removal and slowing of the subsequent ripening of particle removal was influenced by the characteristics of the surges applied. Larger and more rapidly occurring pressure fluctuations generated a greater deficit in removal efficiency and in rate of head loss development. The larger and more rapidly occurring surges generated larger and more rapidly changing fluctuations in interstitial velocity within the filter media. These larger and more rapidly occurring fluctuations in interstitial velocity will create greater and more rapidly changing fluctuations in hydraulic shear acting on suspended particles within the filter pores resulting in a greater reduction in the rate of particle attachment.

From the results of this laboratory study, the effect of surges similar to those observed at local water plants and by previous researchers is believed to be of the order of a 0.1 NTU deterioration in the filtered water quality from full-scale plant. However, this will require confirmation by scale up studies operated with natural raw waters and at lower water temperatures than used in these well controlled laboratory studies. Nevertheless, as a precaution efforts should be made to minimise the occurrence of surging flow in full-scale filters where possible. A small improvement in filtered water turbidity can be equivalent to a reduction of many thousands of particles per millilitre. Chlorine resistant pathogens such as *Cryptosporidium sp.* and *Giardia sp.*

are common in surface water sources and have been found in low turbidity filtered drinking water. From the results of this experimental study, the presence of surges in full-scale filters will contribute to the penetration of all suspended particles including pathogenic protozoans and viruses into the filtered drinking water supply. Current operational practise at water treatment plants will not highlight any problems with filtered water quality from surging flow since the effect is small but present continuously and is therefore not immediately obvious to on-line turbidity measurement.

7.4 Recommendations for Further Research

The survey of local water treatment plants conducted in this experimental investigation was limited to four plants. The measurements of the fluctuations in the pressure drop and flow rate were limited by the capabilities of the plant supervisory control and data acquisition system (SCADA) and the location of the measurement devices within the filter gallery. The information gathered confirmed that surging similar to that observed by previous research does exist in modern rapid gravity filters. A more comprehensive survey and assessment of the occurrence of surges in modern rapid filters would help measure the extent of the risk. Surge recordings collected at various points in the filter system using modern pressure transducers operating at high sampling rates would yield greater detail of the characteristics of the surging flow within the filtrate piping and more importantly the resultant fluctuations within the filter media itself. Parallel measurements using water piezometers would allow comparison of the pressure transducer data with the measurements conducted much earlier by previous researchers using only piezometers. The analysis of such data would also provide greater information regarding the limitations of piezometers to record rapidly changing pressures. Pressure measurement at the same location as the flow meters would allow further characterisation of the relationship between the pressure and flow fluctuations. A survey of a sufficiently large number of plants would allow statistical analysis of the relationship between filtered water quality and the characteristics of the surges found within the filter media. This would serve to further quantify the effect of surging flow under full scale operating conditions. The suggested wider survey of the occurrence of surges may confirm the presence of a relationship between the complexity of the filtrate piping design and the magnitude and frequency of surging present.

The effect of surging flow on the removal of particles in rapid gravity filtration has been confirmed in this laboratory scale study using a suspension of non-flocculated polyvinyl chloride particles applied at an approach velocity of 8 metres per hour to mono-medium sand filters but at an elevated temperature of 30 °C. These operating conditions are not extreme. Higher approach velocities up to 25 metres per hour are to be found in full-scale plant. Raw water temperatures are invariably much lower than those used in this laboratory study resulting in higher fluid viscosity. Winter temperatures can be as low as 4 °C with summer peaks of 20 to 25 °C. Deposits formed from flocculated suspensions found at full-scale works have been shown by previous workers to be more susceptible to fluid shear than deposits formed from the non-flocculated particles used in this study. The mechanisms of the effect of the surges have been examined and efforts were made to quantify the phenomenon using a range of surging profiles. A pilot plant study sited at a full-scale water treatment plant would be worthwhile and useful to further investigate the mechanisms. The study could aim to quantify the effects on filter head loss development and particle removal with time and depth under field conditions. The natural raw water could be used with a range of coagulants and floc aids to characterise the effect of fluctuating flow rates on the attachment, ripening and subsequent detachment of flocculated particles. These mechanisms could be investigated with a number of different filter bed designs including dual and triple media beds. The pilot plant could be operated at the range of flow rates and operating temperatures to be found at full-scale rapid gravity filter plants but under more control than the standard full-scale units. These experiments would serve to quantify the effect of surges on full-scale filter performance and would identify the optimal operating conditions and filter design to minimise the effect of the surges on filtered water quality.

The literature review showed that techniques for the recovery and enumeration of *Cryptosporidium sp.* and *Giardia sp.* cysts are improving continuously. Techniques to rapidly detect viruses in water have also been developed. The raw water applied to the proposed pilot plant could be dosed with such pathogens to determine the influence of flow rate fluctuations on the penetration of these organisms through rapid filters.

Comparison with turbidity measurement would further characterise the limitations of on-line turbidimeters in the detection of low level contamination of filtered drinking water. Comparison with particle size distribution data may lend further weight to the adoption of a 0.1 NTU filtered water turbidity standard and introduction of particle monitoring in place of turbidity measurement.

The suspension dissolved solids concentration was observed to influence filter performance during the course of this surging study. From these observations, there is scope for further investigation of the effect of the dissolved solids concentrations in raw waters on the performance of rapid filters. Blending of pure water with mains water and the addition of various chemicals could be used to generate a range of suspensions of different dissolved solids concentrations. The proportions of each of the dissolved compounds present could be varied. These suspensions could be used to further characterise the effect of dissolved solids on the initial attachment of particles to sand grains and the subsequent rate of ripening of particle removal with time and depth. The effect of surges could be assessed with these different suspensions to determine the field conditions most vulnerable to rapidly occurring flow fluctuations. Raw water conductivity measurements are taken at water treatment plants. A survey of this data may yield useful insights into the variations in raw water chemistry from site to site and the variation with time at individual sites. It may be the case that the raw water dissolved solids concentration fluctuates at particular sites with subsequent effects on the performance of the rapid filters. There may be scope for using conductivity measurement as a control parameter for the addition of coagulants and flocculant aids to improve particle attachment during known periods of fluctuating raw water chemistry.

For both future pilot and laboratory scale investigations in filtration research it is suggested that an automatic approach to monitoring should be followed. This would allow longer filter runs and the collection of more data than in this study. For example, the use of pressure transducers connected to data logging equipment for all pressure connections would allow automatic continuous head loss profiling. Water piezometers are a low cost option but cannot be automated and all pressure readings must be made by eye and so do not provide continuous information. Similarly, continuous on-line turbidity measurement would provide greater depth of information and reduce laborious sampling procedures. The use of such techniques would allow the investigator more time to conduct a more comprehensive particle size distribution sampling programme and to record the changing particle size distribution and number counts with time and depth. The ideal would be on-line particle analysis using a flow through particle counter such as available in flow cytometry and switching equipment to sample each port in turn.

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APPENDIX A

HEAD, TURBIDITY, SURGE, BED LEVEL AND TEMPERATURE MEASUREMENTS FROM TEST RUNS 1 TO 32

F1	Filter One
F2	Filter Two
Turb	Turbidity
NTU,ntu	Nephelometric turbidity units
mins	Minutes
Raw	Raw water
%HL	Percentage of head loss
F1T, F2T	Filter one/two pressure port above filter media
F1P1, F2P1	Filter one/two pressure port at 15mm depth
F1P2, F2P2	Filter one/two pressure port at 30mm depth
F1P3, F2P3	Filter one/two pressure port at 50mm depth
F1P4, F2P4	Filter one/two pressure port at 100mm depth
F1P5, F2P5	Filter one/two pressure port at 150mm depth
F1B, F2B	Filter one/two pressure port at filter base
F1D1, F2D1	Filter one/two suspension port above filter media
F1D2, F2D2	Filter one/two suspension port at 50mm depth
F1D3, F2D3	Filter one/two suspension port at 100mm depth
F1D4, F2D4	Filter one/two suspension port at 200mm depth
F1D5, F2D5	Filter one/two suspension port at 400mm depth
F1F, F2F	Filter one/two outlet port
cm OD	centimetres over datum

Run 1: Water Temperature 20°C Raw turbidity 160 NTU

Time (mins)	Head loss (m)		Turbidity (ntu)	
	F1	F2	F1F	F2F
75			87.7	90
133			76.2	79.7
160			72.8	75.5
218			67.4	68.7
248			65.1	67
278	0.385	0.366	63.5	65.7
308	0.403	0.381	62.2	64
338	0.419	0.398	60.6	62.5
368	0.44	0.42	59.2	61.2

Run 2: Water Temperature 20°C Raw turbidity 152 NTU

Time (mins)	Head loss (m)		Turbidity (ntu)	
	F1	F2	F1F	F2F
30	0.309	0.3	72.6	72.6
60	0.324	0.319	59.2	59.6
90	0.347	0.341	50.7	51.5
120	0.371	0.369	45.4	45.4
150	0.405	0.399	39.9	40.8
180	0.437	0.427	37.5	38
210	0.476	0.465	36.2	35.9
240	0.523	0.506	34.3	33.9
270	0.582	0.553	32.2	31.9
300	0.626	0.603	30.8	30.6

Run 3: Water Temperature 20°C Raw turbidity 131 NTU

Time (mins)	Head loss (m)		Turbidity (ntu)	
	F1	F2	F1F	F2F
30	0.337	0.343	56.6	53.3
60	0.357	0.367	52.6	50.3
90	0.384	0.401	46.3	44.4
120	0.419	0.446	42.6	40.7
145	0.456	0.492	39.3	37.2
180	0.51	0.566	36.7	34.7
210	0.565	0.645	34.3	31.9
240	0.626	0.732	32.4	30.5

Run 5: Water Temperature 32°C

Time (mins)	Head loss (m)		Turbidity (ntu)		
	F1	F2	F1F	F2F	Raw
30	0.254	0.254	23.1	24	185.3
60	0.394	0.395	12.3	12.7	177.5
90	0.693	0.676	7.2	7.2	170
120	1.116	1.037	4.6	4.6	160.7

Run 6: Water Temperature 31°C

Time (mins)	Head loss (m)		Turbidity (ntu)		
	F1	F2	F1F	F2F	Raw
30	0.234	0.231	15.3	16	45
60	0.24	0.24	12.22	12.25	44.4
90	0.247	0.243	10.56	11	44.4
120	0.255	0.252	9.33	9.65	43.7
150	0.266	0.263	8.51	8.56	42.9
180	0.28	0.274	7.61	7.82	43
210	0.294	0.291	7.06	7.25	42.7
240	0.309	0.306	6.66	6.76	41.1
270	0.326	0.324	6.37	6.51	42.3

Run 7: Water Temperature 32-33°C

Time (mins)	Head loss (m)		Turbidity (ntu)		
	F1	F2	F1F	F2F	Raw
30	0.233	0.227	23.2	23	110.2
60	0.25	0.247	16.1	15.9	108.5
90	0.27	0.271	13.2	13	109
120	0.298	0.299	11.3	11.3	108.1
150	0.33	0.333	9.7	9.75	106
195	0.397	0.403	8.2	8.4	105
210	0.432	0.438	7.4	8	103.5
240	0.506	0.511	7.3	7.6	103.5
270	0.597	0.599	7.15	7.27	101.8
300	0.699	0.698	6.9	7.22	100.4

Run 8: Water Temperature 31-33°C

Time (mins)	Head loss (m)		Turbidity (ntu)		
	F1	F2	F1F	F2F	Raw
30	0.241	0.236	41.1	39.1	116.9
60	0.258	0.256	31.8	30.9	115.8
90	0.28	0.279	27.4	27	114.3
120	0.303	0.304	24.9	24.7	113.5
150	0.331	0.333	22.3	22.2	112.4
180	0.367	0.37	19.7	20	110.7
210	0.417	0.419	18.15	18.17	110
240	0.475	0.478	16.97	17.08	108.7
270	0.544	0.544	16.14	16.24	107.5
300	0.622	0.619	15.56	15.85	105.8

Run 9: Water Temperature 30.5°C F2 Surging 4-8% of head loss
60 oscillations/minute

Time (mins)	Head loss (m)		Turbidity (ntu)			Surge (%HL)
	F1	F2	F1F	F2F	Raw	F2
30	0.227	0.231	41.8	40.2	113.1	4.33
60	0.241	0.245	33.5	32.4	111.9	4.08
90	0.256	0.261	29.6	29	110.5	3.83
120	0.273	0.279	27.2	26.9	109.5	5.73
150	0.291	0.3	25.2	25	107.7	5.33
180	0.313	0.322	24	23.6	106.6	4.97
210	0.338	0.345	22.6	22.7	105.1	4.64
240	0.367	0.37	21.7	22.6	103.5	7.84
270	0.401	0.399	21	21.8	101.5	7.52
300	0.435	0.43	20.4	21	100.9	6.98

Run 10: Water Temperature 30.5°C F2 Surging 20% of head loss
10 oscillations/minute

Time (mins)	Head loss (m)		Turbidity (ntu)			Surge (m)
	F1	F2	F1F	F2F	Raw	F2
30	0.225	0.237	45.9	45.2	118.8	0.04
60	0.242	0.255	36.9	36.1	117.7	0.04
90	0.262	0.277	32.4	31.6	115.5	0.056
120	0.291	0.307	29.4	28.6	114	0.056
150	0.321	0.341	26.5	26	112.8	0.069
180	0.356	0.382	24.7	23.8	111.2	0.069
210	0.406	0.435	22.4	22.2	108.2	0.09
240	0.461	0.496	20.8	20.6	106.8	0.092
270	0.537	0.58	19.15	18.71	105	0.11
300	0.608	0.649	18.05	17.82	102.4	0.122

Run 11: Water Temperature 30°C F2 Surging 20% of head loss
10 oscillations/minute

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*			*Surge (m)*
	F1	F2	F1F	F2F	Raw	F2
60	0.242	0.253	38.8	37	104.2	0.05
120	0.283	0.309	31.8	30.3	103.3	0.063
180	0.341	0.396	28.5	27.2	102.4	0.082
240	0.425	0.514	26	25.2	101.6	0.097
300	0.543	0.669	25.2	23.8	99.7	0.136
360	0.699	0.855	24	23	98.4	0.171
420	0.871	1.059	23	22.4	97.2	0.211
480	1.047	1.25	21.8	21.7	95.5	0.252
540	1.226	1.457	20.8	20.7	93.1	0.291
600	1.397	1.627	20.5	20.3	95.1	

Run 12: Water Temperature 30°C

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*		
	F1	F2	F1F	F2F	Raw
60	0.246	0.25	38.1	36.2	107.4
120	0.273	0.28	31.7	30.8	106.6
180	0.312	0.322	27.7	26.6	105.2
240	0.365	0.382	24.7	23.6	103.1
300	0.442	0.466	22.4	21.6	102.6
360	0.55	0.581	21	20.4	101.6
420	0.676	0.717	19.9	19.2	99.1
480	0.827	0.873	18.9	18	98.3
540	0.982	1.028	18.2	17.8	97.4
600	1.152	1.192	17.7	17.5	98.1

Run 13: Water Temperature 30.5°C F2 Surging 10% of head loss
50 oscillations/minute

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*			*Surge (m)*
	F1	F2	F1F	F2F	Raw	F2
60	0.233	0.24	55.6	54	109.7	0.034
120	0.248	0.257	45.3	43.5	105.6	0.035
180	0.268	0.281	37.4	36.8	106.1	0.038
240	0.304	0.32	30.2	29	105.3	0.037
300	0.362	0.389	23.9	22.2	104.9	0.039
360	0.482	0.519	18.52	17.4	101.7	0.053
420	0.63	0.667	15.48	14.69	99.2	0.072
480	0.799	0.847	12.62	12.2	97.1	0.09
540	0.984	1.039	11.19	10.63	95	0.102
600	1.173	1.239	10.29	9.92	92.9	0.122
660	1.34	1.402	9.79	9.17	89.7	0.134

Run 15: Water Temperature 29°C F2 Surging 20% of head loss
10 oscillations/minute

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*			*Surge (m)*
	F1	F2	F1F	F2F	Raw	F2
60	0.239	0.246	34.7	33.8	97.5	0.05
120	0.264	0.274	30.7	29.5	97.6	0.056
180	0.304	0.314	28	26.9	97.5	0.064
240	0.368	0.347	24.3	25.1	98.2	0.072
300	0.496	0.438	19.7	20.7	97.4	0.085
360	0.763	0.63	15.4	16.2	96.2	0.128
420	1.185	0.973	12.5	12.9	95.1	0.202
455	1.463	1.205	11.3	11.7	94.1	0.202

Run 16: Water Temperature 30°C

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*		
	F1	F2	F1F	F2F	Raw
60	0.308	0.314	29	28.2	95.7
120	0.361	0.367	23.3	22.8	96.7
180	0.443	0.448	19.41	19.01	95.8
240	0.564	0.567	17.02	16.48	96.2
300	0.753	0.758	14.51	14.24	95.6
360	1.011	1.012	12.82	12.45	96.1
420	1.363	1.367	11.36	11.05	96.4
445	1.529	1.535	10.77	10.65	96.1

Run 17: Water Temperature 30°C F2 Surging 20% of head loss
10 oscillations/minute

Time (mins)	*Head loss (m)*		*Turbidity (ntu)*			*Surge (m)*
	F1	F2	F1F	F2F	Raw	F2
60	0.286	0.293	37.5	37.2	93	0.059
120	0.317	0.324	29.8	29.6	93.2	0.067
180	0.365	0.373	24	23.6	93	0.08
240	0.458	0.457	18.5	18.3	92.9	0.1
300	0.602	0.587	14.43	14.41	92	0.116
360	0.872	0.819	11.63	11.8	91.2	0.177
420	1.332	1.219	9.3	9.55	90.1	0.25
442	1.523	1.383	8.79	9.03	89.7	0.25

Run 18: Water Temperature 30°C

Time	Head Level (cm)													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	60.7	59.6	58.5	57.5	55.4	53.2	34.1	61.1	60	59	57.8	55.4	53.1	34
120	62.5	61.3	59.9	58.9	56.5	54.1	34	63.2	61.7	60.4	59	56.1	53.5	33.9
240	65.6	63.5	61.6	60.2	57.4	54.7	34.1	66.5	64.1	62.2	60.4	57	54.1	33.9
360	69.2	66.1	63.7	61.8	58.6	55.6	34	70.5	67	64.3	62	57.9	54.7	33.9
480	72.4	68.7	65.6	63.4	59.6	56.2	34	74.3	70	66.4	63.5	58.8	55.4	34
960	87.2	80.7	73.8	69.2	63.1	58.7	34	91.4	84	75.5	69.7	61.9	57.4	34
1020	88.9	82.1	74.6	69.8	63.5	58.9	34.1	93.5	85.7	76.5	70.3	62.1	57.5	34
1080	91.1	83.9	75.7	70.4	64	59.1	34.1	95.7	87.6	77.6	70.9	62.4	57.6	34.1
1140	93.1	85.6	76.8	71	64.2	59.3	34.2	98.4	89.8	78.8	71.6	62.6	57.8	34.1
1200	95.6	87.5	78	71.8	64.5	59.5	34.2	101.3	92	80.1	72.4	62.9	58	34

Time	Turbidity (NTU)		
mins	F1F	F2F	Raw
120	18.85	18.75	42.7
240	16.31	16.04	41.1
360	14.81	14.77	41.1
480	14.03	13.71	40.8
960	11.55	11.75	40.3
1020	11.9	11.8	40.4
1080	11.9	11.9	40.3
1140	11.9	12.1	40.3
1200	12	11.5	40.1

Run 19: Water Temperature 29.5°C

Time	Head Level (cm)													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	60.8	59.6	58.7	57.6	55.6	53.4	34	60.3	59	57.9	56.9	54.4	52.1	34
495	232.9	147.6	97.2	73.5	62.4	57.6	34	263.5	132.7	89	70.8	59.2	55.1	34.1
525	248.7	158.1	102.2	75	62.6	57.7	34	281.6	142.1	93.1	72.1	59.5	55.2	34.1
545	261.9	165.9	106.7	76.4	63.1	57.9	34	295.9	149.7	96.3	73.2	59.7	55.3	34.1

Time	Turbidity (NTU)		
mins	F1F	F2F	Raw
495	4.85	4.9	50.4
525	4.7	4.8	50.3
545	4.63	4.68	50.1

Run 20: Raw Data

Time	*Head*	*Level*	*(cm)*											
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	60.9	59.7	58.8	57.8	55.7	53.6	34.1	60.5	59.4	58.2	57.3	54.8	52.6	34.1
540	122.5	89.4	72.7	66.5	60.9	57.1	34	123.2	84.4	71.4	65	58.6	55	34.1
600	137.5	97	75.5	67.8	61.4	57.4	34.1	138.6	91	74.2	66.4	59	55.3	34
660	153.2	105.9	78.9	69.4	62	57.8	34.1	154.4	98.7	77.4	67.7	59.5	55.6	34
720	169.8	116.1	82.7	70.9	62.6	58.1	34.1	171.4	107.4	81.1	69.2	59.9	55.8	34
780	185.9	125.7	87.3	72.7	63.2	58.5	34.1	188.8	117.1	85.7	70.9	60.3	56.1	34
840	203.2	136.5	92.4	74.6	63.8	58.7	34.1	205.3	126.8	90.5	72.6	60.7	56.3	34
900	219	148.5	98.2	76.9	64.5	59	34.1	223.9	138.5	96.4	74.8	61.2	56.5	34
960	237.8	159.7	104.2	79.4	65.2	59.4	34.1	242.5	149.5	102.7	77.1	61.6	56.7	34
1020	255.8	172.3	111.2	82.2	65.8	59.6	34.1	260.6	160.8	109.6	79.9	62.1	57	34
1080	273.8	185	118.3	85.2	66.5	59.9	34.1	278.2	173	116.8	82.6	62.5	57.2	34

Time (min)	*Turbidity (NTU)*			*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp.*
	F1F	F2F	Raw	F1	F2	F1	F2
540	4.29	4.4	38.5	-1.1	-1	29.75	29.5
600	4.01	4.07	37.9	-1.1	-1	29.5	29.5
660	3.74	3.88	38	-1.1	-1	29.5	29.5
720	3.49	3.57	37.7	-1.1	-1	29.5	29.5
780	3.29	3.39	37.4	-1.1	-1	29.5	29.25
840	3.07	3.16	37.4	-1.1	-1	29.5	29.25
900	2.92	2.9	37.2	-1.1	-1	29.5	29.25
960	2.66	2.79	37.2	-1.1	-1	29.5	29.25
1020	2.59	2.66	37.1	-1.1	-1	29.5	29.25
1080	2.47	2.5	36.4	-1.1	-1	29.5	29.25

Run 21: F2 Surging 20% of Head loss
10 oscillations/minute applied at 800 minutes

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61	59.7	58.7	57.7	55.6	53.5	34.1	60.5	59.3	58.3	57.3	54.8	52.6	34
600	94.7	76.5	68.4	64	59.2	55.8	33.9	100.5	75.5	67.9	63.5	57.6	54.2	34.1
690	104	80.9	70.4	64.8	59.6	55.9	33.9	110.9	80	70.1	64.6	58.1	54.4	34
780	114.2	86.2	73	66.2	60.2	56.4	33.9	121.6	85.2	72.7	65.9	58.5	54.7	34
870	124.5	92.2	75.9	67.5	60.9	56.7	34	130.7	90.9	75.5	67.3	58.9	54.8	34
960	135.1	98.7	79.3	69.2	61.6	57.2	34	139.9	95	76.4	66.4	57.0	52.7	34
1050	145.7	105.7	83	71	62.4	57.6	34	150.4	101.6	79.9	68	57.4	52.8	34
1140	154.8	112	86.7	72.5	62.8	57.8	34	160.7	109.2	83.7	69.4	57.4	52.5	34

Time	*Turb (NTU)*									
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F2D1	F2D2	F2D3	F2D4	F2D5
600	29.1	14.3	9.8	6.6	4.33	29.3	14.3	8.8	6.05	4.1
690	28.9	14.8	9.3	6.3	3.91	29.1	14.3	9	5.7	3.85
780	28.8	15.75	9.71	6.36	3.84	29	15.21	9.2	5.65	3.64
870	29	16.47	9.81	6.2	3.78	29.5	15.78	9.5	5.6	3.63
960	29.8	18.17	10.4	6.45	3.76	29.7	16.45	9.8	5.65	3.61
1050	29.7	17.8	10.82	6.45	3.91	29.7	17.06	10.15	5.6	3.58

Time	*Bed Level (cm OD)*		*Water Temp (°C)*		*Surge (cm)*
mins	F1	F2	F1	F2	F2
600	-1	-1	29.5	29.25	0
690	-1	-1	29.5	29.25	0
780	-1	-1	29.5	29.25	0
870	-1	-1	29.5	29.25	20
960	-1	-1	29.5	29.25	21
1050	-1	-1	29.5	29.25	23.3
1140	-1	-1	29.5	29.25	23.3

Run 22: F2 Surging 10% of Head loss
40 oscillations/minute applied at 810 minutes

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.6	60.5	59.5	58.5	56.4	54.3	34.1	61	59.9	58.9	57.9	55.4	53.1	34
600	117.6	95.6	80.1	71.2	63.5	58.7	34	124.6	95.1	79.7	70.4	60.8	56.1	34
690	130.8	104.8	85.9	74.3	65	59.6	34	139.9	104.7	85.7	73.7	62	56.9	34
780	142.1	113.4	91.6	77.5	66.3	60.4	34	152.7	114.8	92.4	77.3	63.2	57.6	34
870	154.4	122.6	98.3	81.7	68.2	61.5	34	166.2	124.4	99.8	82.1	65.2	59.2	34
960	166.3	131.9	105	85.8	69.9	62.4	34	179.6	134.4	107.3	86.9	66.9	60.1	34
1050	178.2	140.8	111.8	90.4	71.8	63.4	34	192.5	144.3	115	91.95	68.4	60.85	34

Time	*Turb (NTU)*									
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F2D1	F2D2	F2D3	F2D4	F2D5
600	44.7	28.8	21.4	13.1	8.44	44.5	29.1	18.5	12.09	8.4
690	44.6	31.5	21.9	13.6	8.45	45.2	29.8	19.6	12.1	8.15
780	44.5	30.9	22	13.2	7.9	44.3	30.3	20.9	11.9	7.85
870	44.6	32.4	23	13.2	8.1	44.3	31.1	22.2	12.9	8.09
960	44.7	34.4	24.5	14.3	8.3	44.7	33.9	23.4	13.3	8.08
1050	44.6	35.1	25.9	15.2	8.2	44.7	34.8	23.6	13.4	7.97

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
600	-1.2	-1.1	29.5	29.5	0
690	-1.2	-1.1	29.5	29.5	0
780	-1.2	-1.1	29.5	29.5	0
870	-1.2	-1.1	29.5	29.25	13.5
960	-1.2	-1.1	29.5	29.25	14.2
1050	-1.2	-1.1	30	29.5	15.2

Run 23: Raw Data

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61	59.8	58.9	57.8	55.7	53.6	34.1	61	59.9	58.8	57.8	55.3	53	34.1
300	72	67.6	65	62.5	59.1	55.9	34	73.6	68.7	65.8	63	58.3	55.1	34
630	94	81.9	75.5	70.2	64.2	59.4	34	100.9	85.5	77.7	71.4	62.9	58.2	34.1
720	102.1	87.3	79.1	72.7	65.8	60.6	34	110.9	91.5	81.6	74.1	64.3	59.1	34
810	111.1	93.1	83	75.3	67.4	61.8	34	121.8	98.2	86	77.1	65.7	60.1	34
900	121.6	99.9	87.6	78.4	69.2	63	34	134.3	106	91.1	80.2	67.2	61	34
990	132.8	107.2	92.3	81.5	71	64.1	34	147.2	114.4	96.6	83.8	68.9	62	34
1080	145.1	115.3	97.9	84.8	72.9	65.3	34	161	123.9	102.7	87.6	70.5	63	34.1

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
300	48	32.6		21.8	15.43	12.42	49			19.27	15.29	12.12
630	47	32.6		20	14.2	11.28	46.3			17.9	14.02	11.3
720	47.4	33	25.8	20.2	13.8	11.02	44.7		26	17.2	13.9	11.1
810	46.8	32.8	24.9	20.3	13.2	10.9	46.9		25.9	17.2	13.7	11
900	46.7	32	24	20	13.2	10.7	46.8		26.8	16.5	13.7	11
990	48.7	32.8	24.3	19.6	12.9	10.3	47.6		26.3	17.1	13.5	10.5
1080		33.3		20.1	12.6	10.2				16.8	13.5	10.5

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
300	-0.9	-1	30.25	30	0
630	-0.9	-1	30	29.75	0
720	-0.9	-1	30	29.5	0
810	-0.9	-1	30	29.5	0
900	-0.9	-1	29.75	29.5	0
990	-0.9	-1	29.75	29.5	0
1080	-0.9	-1	30	29.5	0

Run 24: F2 Surging 10cm Amplitude
80 oscillations/minute

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.1	59.8	58.9	57.9	55.8	53.6	34.1	61.4	60.4	59.4	58.3	55.6	52.9	34.1
300	71.9	66.8	64	61.9	58.3	55.4	34	70.5	67.75	65.5	63.5	59.35	56	34
600	94.5	80.3	72.8	68.4	62.3	58.2	33.9	84.3	77	72.3	68.35	61.8	57.65	34
690	104.1	86.1	76.4	70.8	63.8	59.1	33.9	90.6	81.1	75.15	70.45	62.85	58.35	34
780	114	92	80	73.1	65.1	59.9	33.9	97.5	85.45	77.95	72.45	64	58.9	34
870	124.4	98.3	83.8	75.7	66.5	60.9	33.9	105.8	90.6	81.3	74.7	64.95	59.7	34
960	136	105.4	88.2	78.5	68.1	61.9	33.9	114.6	96.2	84.75	77	66.15	60.3	34
1050	147.1	112.4	92.4	81.3	69.6	62.8	33.9	124.5	102.6	88.65	79.4	67.1	60.95	34

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
300	46	30.8	23.8	18.8	13.8	10.97	43.8	33.4	28	22.1	17.3	14.3
600	45.4	32	24	17.4	12.15	9.8	46	33.1	27	19.5	14.9	11.97
690	44.4	31.4	24.1	17.6	11.75	9.15	45.4	32	26.9	19	14.95	11.4
780	44.3	32.4	23.7	17.6	12.1	9.2	45	34	25.1	18	14	10.8
870	44.5	33.5	24.5	18.5	11.7	9.15	43	34	25.7	17.3	13.25	10.5
960	45.8	34.9	25.3	18.85	12	8.95	46.8	33.9	27.1	18.4	13	10.2
1050	46.1	34.7	27.2	19.5	12.1	9.1	46.6	34.8	27.5	19.8	13.5	10.5

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
300	-1	-1.2	30	29.75	10.3
600	-1	-1.2	29.5	29.25	10.6
690	-1	-1.2	29.25	29	11.2
780	-1	-1.2	29.25	29	10.6
870	-1	-1.2	29.25	29	10.8
960	-1	-1.2	29	29	11.4
1050	-1	-1.2	29.25	29	11.3

Run 25: Raw Data

Time	Head Level (cm)													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	62.2	61.1	60	58.9	56.7	54.6	34	62.2	61.2	60.3	59.3	56.4	53.7	34.4
90	63.9	62.1	60.9	59.4	56.9	54.5	34	64.1	62.6	61.2	59.8	56.7	53.7	34.3
180	67.8	65.3	63.2	61.4	58.2	55.5	34	68	65.6	63.5	61.5	57.7	54.5	34.3
270	71.7	68.7	65.7	63.2	59.3	56.3	34	72.1	68.9	65.6	63	58.4	54.9	34.3
630	93.5	86.6	77.4	71.4	64.6	60	34	97.8	89.3	78	71.3	63	57.9	34.3
720	101.1	92.4	80.3	72.9	65.4	60.6	34	107.7	96.6	81.2	73	63.8	58.4	34.3
810	109.4	98.7	82.8	73.9	65.6	60.8	34	118.7	104.5	84.3	74.1	64	58.5	34.3
900	120.4	107.2	86.8	75.7	66.4	61.3	33.9	132.5	114.9	88.6	75.8	64.4	58.7	34.3
990	132	116	90.7	77.1	66.6	61.3	33.9	147.2	125.7	93.4	77.3	64.6	58.8	34.3
1080	144.3	125.2	94.8	78.3	66.6	61.3	33.9	163.1	137.3	99.1	79.1	64.7	58.7	34.3

Time	Turb NTU											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	46.7	37.1	30.1	25.6	19.4	15.47	44.3	35.5	30.8	24.8	18.7	15.55
180	46.1	32.9	24.7	20.5	15.6	12.2	45.9	30.6	24.8	18.6	14.8	12.4
270	43.9	31.4	24.5	18.8	13.8	11.14	44.9	30.9	23.8	17.2	13.9	11.3
630	45.8	32.4	25	17.7	11.4	8.75	46	32.3	24	17.4	11.3	8.79
720	44.8	31.5	23.1	18.8	11.7	8.47	41.8	30.2	23.9	16.8	11.5	8.7
810	45.5	32.7	23.5	19.8	12.8	8.6	45.9	30.8	24	19.1	13.1	8.6
900	43.5	31.2	22.3	19.8	14.3	8.9	43.7	30.7	23	18.5	14.4	9.2
990	42.9	30.9	21.9	19	15.4	9.5	42.3	30.3	22.8	19.1	15.6	9.9
1080	43.5	31	23.1	19.4	17.1	10.6	43.2	30.4	23.2	18.9	15.9	10.5

Time (mins)	Bed (cm OD)	Level	Water (°C)	Temp	Surge (cm)
	F1	F2	F1	F2	F2
90	-1.3	-1.3	29	29	0
180	-1.3	-1.3	29	29	0
270	-1.3	-1.3	29	29	0
630	-1.3	-1.3	29	29	0
720	-1.3	-1.3	29	29	0
810	-1.3	-1.3	29	29	0
900	-1.3	-1.3	29	29	0
990	-1.3	-1.3	29	29	0
1080	-1.3	-1.3	29	29	0

Run 26: F2 Surging 10cm Amplitude
80 oscillations/minute

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	62	60.9	59.7	58.6	56.5	54.4	34	61.5	60.5	59.5	58.5	55.7	53.1	34.3
90	65.4	63.1	61.5	59.7	57.1	54.7	34	66.5	64.55	63.05	61.5	58.35	55.4	34.3
180	73	67.5	64.1	61.2	58	55.3	34	72.2	67.9	65.3	63.1	59	55.8	34.3
270	87.5	74.8	67.1	62.7	58.7	55.8	34	83.3	72.6	67.55	64.6	59.75	56.2	34.3
360	107.8	85	70.9	64.2	59.3	56.2	34	99.4	78.6	70	65.6	59.8	56.1	34.3
450	133.3	98.7	75.8	65.6	59.8	56.4	34	121.5	86.85	73.15	66.7	59.85	55.95	34.3
540	159.2	113.8	82.6	67.8	60.5	56.9	34	149	99.2	77.9	68.35	60.25	57.25	34.3
630	190.2	129	90.7	69.8	60.9	57	34	179.2	113.8	83.4	70.15	60.35	56.2	34.3

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	37.4	23	20.9	16.5	13.6	10.2	39	25.4	21.1	16.6	13.7	10.8
180	34.8	16.6	13.1	10.8	8.9	7.1	35.6	17.4	14	10.9	9.1	7.6
270	31.9	13.8	10	7.9	6.5	5.3	33.8	13.4	10.1	7.6	6.5	5.5
360	31.5	12.7	7.8	6.1	4.86	3.98	32.9	11.6	7.9	5.8	4.85	4.04
450	30.1	12.5	6.5	4.7	3.7	2.93	31.6	10	6.3	4.25	3.55	2.93
540	29.3	10.9	5.5	3.68	2.95	2.32	28.1	9.6	5.1	3.35	2.73	2.29

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
90	-1.3	-1.2	29	29	10.3
180	-1.3	-1.2	29	29	10.2
270	-1.3	-1.2	29	29	11.6
360	-1.3	-1.2	29	29	11.3
450	-1.3	-1.2	29	29	11
540	-1.3	-1.2	29	29	11.8
630	-1.3	-1.2	29	29	11.4

Run 27: F2 Surging 10cm Amplitude
80 oscillations/minute

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.7	60.6	59.5	58.5	56.1	53.9	34	62	61	59.9	58.8	56	53.4	34.3
90	64.6	62.4	61	59.4	56.6	54	34	66.4	64.55	63.05	61.7	58.5	55.55	34.3
180	70.6	65.8	63	60.8	57.3	54.5	33.9	70.4	67.15	64.9	62.9	59.05	56	34.3
270	77.5	69.5	65.2	62.1	57.9	54.9	33.9	75	69.45	66.2	63.9	59.55	56.1	34.3
360	86.7	74.2	67.9	64	58.9	55.5	33.9	81.2	72.9	68.55	65.2	60.25	56.55	34.3
450	96.9	80.6	71.3	65.8	59.9	56.3	33.9	89.8	76.95	70.6	66.6	60.95	57.1	34.3
540	111.4	88.1	75	67.7	60.7	56.6	33.9	100.6	81.65	73.15	67.95	61.5	57.5	34.3
720	146.4	109	85.5	72.6	62.8	57.9	33.9	130.2	98.55	80.35	71.6	62.7	58.25	34.3
810	165	120.7	91.6	75	63.5	58.3	33.9	150.1	109	84.55	73.2	63.2	58.4	34.3

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	42.4	30.6	25.2	21.5	18	14.8	41.8	31.1	27.3	23.4	19.2	16.9
180	41.2	26.4	22	17.9	14.4	12.1	40.65		23.8	19	16.4	14.2
270	40	26.4	19.6	16.5	13.1	11.2	39.5	27.4	22.7	17.7	15	13.4
360	38.7	26.4	19.4	15.4	12.1	10.7	38.2	26.3	21.1	17.3	14.3	12.6
450	39.3	25	17.5	14.3	11.3	9.8	38.7	25.2	19.4	15.3	13	11.5
540	38	24	16.8	13.2	10.1	8.9	37.7	23.8	18.2	13.8	11.4	10.5
720	38.6	24.1	16.2	11.4	9.3	7.8	37.6	22.1	16	11.9	9.6	9
810	37.9	24.9	17.3	12	9	7.5	40.1	23	15.7	12	8.9	8.2

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
90	-1.2	-1.2	29	29	10.5
180	-1.1	-1.2	29	29	10.1
270	-1.1	-1.2	29	29	11
360	-1.1	-1.2	29	29	11.3
450	-1.1	-1.2	29	29	11.7
540	-1.1	-1.2	29	29	12.5
720	-1.1	-1.2	29	29	10.3
810	-1.1	-1.2	29	29	10.5

Run 28: Raw Data

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.6	60.1	59.2	58.2	56.2	54.1	34	61.7	60.2	59.3	58.3	55.7	53.1	34.2
90	69.8	63.9	61.5	59.7	57	54.7	34	70.2	63.5	61.2	59.4	56.1	53.4	34.3
180	81.1	68.3	64	61	57.7	55	34	82.2	67	63.2	60.6	56.7	53.6	34.3
270	96.8	74.2	66.4	62.3	58.1	55.1	34	99.5	72.1	65.4	61.8	56.9	53.7	34.3
360	117	82.7	70.3	64.3	59	55.7	34	121.2	79.5	68.7	63.5	57.6	54.1	34.3
450	138.8	92.8	74.7	66.1	59.6	56	34	144.5	89.5	72.6	65.1	58.1	54.3	34.3
540	161.4	104.2	80.4	68.4	60.5	56.5	34	168.5	100.2	77.5	67.3	58.7	54.6	34.3

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	37.6	17.3	14.3	12.3	9.4	7.2	37.6	17.3	12.1	11.2	9.5	7
180	39.2	17.3	12.9	10.6	8	6.6	39.2	17.2	10.8	9.7	8	6.5
270	39.6	17.5	10.9	8.8	6.6	5.4	39.6	15.3	9.4	8	6.4	5.2
360	38.5	16.8	10.1	7.7	5.5	4.5	38.5	13.8	7.7	6.8	5.4	4.5
450	38.1	16.6	9.5	6.7	5	4	38.1	13.4	7.1	6	4.7	3.9
540	35.2	16.8	9.2	6.1	4.4	3.6	35.2	13.6	6.2	5.4	4.2	3.5

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
90	-1	-1	29.5	29.5	0
180	-1	-1	29.5	29.5	0
270	-1	-1	29.5	29.5	0
360	-1	-1	29	29	0
450	-1	-1	29	29	0
540	-1	-1	29	29	0

Run 29: F2 Scaled Surging 80 oscillations/minute

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.2	59.7	58.9	57.8	55.7	54	34.1	61.5	60.4	59.5	58.4	56	53.6	34.3
90	64	61.6	60.5	59	56.6	54.4	34	65	62.75	61.75	60.45	57.6	55.1	34.3
180	68.1	64	61.9	60.1	57.2	54.8	34	69.1	65.1	63.5	61.7	58.3	55.5	34.3
270	76.5	67.7	64.2	61.6	58.2	55.4	34	76.7	68	65.4	63.1	58.95	56.15	34.3
540	110.9	81.6	70.7	65.4	60.3	56.7	34	106.3	78.15	71.55	67.2	60.95	57.25	34.3
630	126.5	88	73.4	66.8	61	57.2	34	119.4	83.05	74.45	69.15	61.9	58.05	34.3
720	143.5	95.4	76.5	68.5	61.9	57.7	34	134.4	88.4	77.5	70.9	62.85	58.8	34.3
810	162.9	104.1	80.1	70.2	62.8	58.3	34	151.4	94.6	80.7	72.7	63.6	59.1	34.3

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	46.7	30.3	27.9	27.3	23.3	18.4	46.7	34.6	28.8	27.9	23.9	19.6
180	45.1	29	24.7	22	18.2	15.5	45.6	29.1	23.2	21.5	18.6	16.2
270	45	26.7	22.2	19.4	16.2	14	44.6	27.3	21.8	18.8	17.1	14.7
540	42.4	28.7	22.2	18.8	15.4	13.5	42.3	28.9	21.8	19.1	16.4	14.5
630	41.2	27.6	22.1	19.8	15.3	13.7	42.3	28.4	21.9	19.6	16.3	14.5
720	41.7	27.4	22.6	19.9	15.4	13.3	41.7	27.9	21.4	18.9	15.6	14.1
810	41.6	28.7	22.6	19.5	15.1	13.2	41.4	27.9	21.9	18.4	15.1	14

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*	*Surge (%HL)*
	F1	F2	F1	F2	F2	F2
0	-0.9	-1			5	18.4
90	-0.9	-1	29.5	29.5	5.3	17.3
180	-0.9	-1	29.5	29.5	6.4	18.4
270	-0.9	-1	29.5	29.5	6.9	16.3
540	-1	-1.1	29.5	29.5	9	12.5
630	-1	-1.1	29.5	29.5	11	12.9
720	-1	-1.1	29.5	29.5	14	14.0
810	-1	-1.1	30	30	13	11.1

Run 30: F2 Scaled Surging 40 oscillations/minute

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	61.9	60.5	59.6	58.6	56.6	54.5	34.1	62.5	61.2	60.4	59.1	56.8	54.1	34.4
90	66.2	63	61.3	59.8	57.3	55	34.1	67.3	64	62.25	60.65	57.45	54.7	34.4
180	74.6	66.9	63.5	61.4	58.2	55.6	34.1	76.8	67.9	64.55	61.95	58.1	54.9	34.4
270	88.8	72.9	66.2	63	59	56.1	34.1	93.2	74.05	67.25	63.45	58.8	55.4	34.4
450	124.8	89.7	73.4	66.4	60.7	57.1	34.1	134.5	93.05	75.4	67.45	60.2	56.4	34.4
540	144.3	100.7	78.4	68.5	61.7	57.6	34.1	157.5	105.6	81.25	70.15	61.2	57	34.4
570	151.6	105	80.5	69.3	62	57.8	34.1	155	111.0	84.45	71.2	61.4	57.15	34.4
600	157.7	108.3	82.4	70.2	62.3	58.1	34.1	166.6	116.6	87	72.4	61.75	57.45	34.4
630	164.6	112.5	84.6	71.2	62.7	58.3	34.1	176.5	122.2	89.75	73.4	62.15	57.65	34.4

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	46.3	23	22.8	15.7	14.9	10.9	46.4	27	18.5	17.6	14.2	11
180	45.2	21	17.1	11	11.1	8.7	45.1	21.3	16.1	13.3	10.7	8.4
270	43.3	17.7	13.6	10.6	9	7	43.5	17.9	13.2	10.5	8.7	7.1
450	39.8	17.5	12.4	8.3	7.1	5.7	39.8	16.9	11.9	8.5	6.6	5.6
540	40.2	18.5	12.6	7.9	6.8	5.2	40.2	18.6	11.6	8.2	6.2	5.2
630	40.5	18.3	12.9	7.5	6.1	5.1	40.5	19.4	10.9	7.7	5.7	5.2

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*	*Surge (%HL)*
	F1	F2	F1	F2	F2	F2
0	-1	-1.2			5	17.8
90	-1	-1.2	29.5	29.5	6.5	19.8
180	-1	-1.2	29.5	29.5	7	16.5
270	-1	-1.2	29.5	29.5	9.2	15.6
360					11	
450	-1	-1.2	29.5	29.5	14	14.0
540	-1	-1.2	29.5	29.5	16.5	13.4
570	-1	-1.3	29.5	29.5		
600	-1	-1.3	29.5	29.5		
630	-1	-1.3	29.5	29.5	16.5	11.6

Run 31: Raw Data

Time	*Head Level (cm)*													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	62	60.6	60	58.9	56.8	54.6	34.1	62.1	60.7	60	58.8	56.5	54	34.4
90	65.9	63.1	61.7	60.3	57.6	55.2	34.1	66	63	61.6	60.2	57	54.4	34.3
180	71.6	66.2	63.7	61.6	58.4	55.7	34.1	72.2	65.8	63.5	61.3	57.6	54.8	34.3
270	80.2	70.3	66.1	63.3	59.3	56.3	34.1	81.6	69.8	65.9	62.8	58.3	55.3	34.3
540	115.2	88.5	76.5	68.9	61.8	57.6	34.1	121.8	87.6	76	67.9	60.2	56.4	34.3
630	129.3	96.4	81.3	71.4	62.7	58.2	34.1	138	96	80.8	70.3	61	56.8	34.3
720	144	105	86.8	74.4	64	58.9	34.1	154.9	105.2	86.5	73.1	61.8	57.3	34.3

Time	*Turb NTU*											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	46.3	25.4	19.2		11.8	8.7	46.3	25.1	16.3	14.4	11.4	8.5
180	45.9	21.4	16.1		9.3	7.3	45.9	21.5	14.2	11.3	9.1	7.1
270	43.9	20.8	14.6		8.3	6.8	43.9	20.2	13.1	10.5	8.4	6.7
540	42.5	23	14.1		6.8	5.6	42.5	20.9	12	8.7	6.6	5.5
630	43.6	25.4	13.9		6.7	5.5	43.6	21.5	12.4	8.5	6.5	5.4
720	42.9	25.1	15		6.7	5.4	42.9	24	13	8.3	6.1	5.2

Time (mins)	*Bed (cm OD)*	*Level*	*Water (°C)*	*Temp*	*Surge (cm)*
	F1	F2	F1	F2	F2
0	-1	-1	29.5	29.5	0
90	-1	-1	29.5	29.5	0
180	-1	-1	29.5	29.5	0
270	-1	-1	29.5	29.5	0
540	-1	-1	29.5	29.5	0
630	-1	-1	29.5	29.5	0
720	-1	-1	29.5	29.5	0

Run 32: F2 Surging 10cm Amplitude
80 oscillations/minute

Time	Head Level (cm)													
mins	F1T	F1P1	F1P2	F1P3	F1P4	F1P5	F1B	F2T	F2P1	F2P2	F2P3	F2P4	F2P5	F2B
0	62.3						34.2	62.8						34.4
90	67.6						34.1	69.3						34.3
180	76.8						34.1	76.2						34.3
270	91						34.1	87.5						34.3
360	107.7						34.1	102.2						34.3
450	126.3						34.1	119.9						34.3
540	147.1						34.1	139.2						34.3

Time	Turb NTU											
mins	F1D1	F1D2	F1D3	F1D4	F1D5	F1F	F2D1	F2D2	F2D3	F2D4	F2D5	F2F
90	45.2	19.5	14.7	10.7	8.2	5.9	45.2	22.4	17.2	12.2	9.5	6.5
180	43.8	15.2	11	7.8	6	4.6	43.8	16.8	12.2	8.5	6.9	5.1
270	42.5	15.2	9.5	6.5	4.8	3.8	42.5	14.3	10.2	6.6	5.3	4.1
360	41.4	15.4	8.6	5.4	4	3.1	41.4	14.2	8.9	5.4	4.3	3.3
450	40.4	17.6	8.7	4.9	3.5	2.8	40.4	14.2	8.3	4.9	3.6	2.8
540	40.1	17.9	9.3	4.6	3.1	2.5	40.1	16	8.2	4.4	3.2	2.5

Time (mins)	Bed (cm OD)	Level	Water (°C)	Temp	Surge (cm)
	F1	F2	F1	F2	F2
0	-1.1	-1.3	29.5	29.5	
90	-1.1	-1.3	29.5	29.5	11.5
180	-1.1	-1.3	29.5	29.5	11.5
270	-1.1	-1.3	29.5	29.5	13
360	-1.1	-1.3	29.5	29.5	13
450	-1.1	-1.3	29.75	29.75	14
540	-1.1	-1.3	30	30	13.5

APPENDIX B

ELIMINATION OF THE EFFECT OF SUSPENSION CHEMISTRY ON THE EXPERIMENTAL DATA BY REGRESSION ANALYSIS NORMALISATION

The influence of the variation in the PVC suspension dissolved solids concentration and Zeta potential on the performance of the laboratory filters was described in chapter six. Simple and multiple linear regression analysis yielded relationships with high correlation coefficients between the rate of development of head loss and the removal efficiency of turbidity and the suspension conductivity and Zeta potential. These relationships were presented and discussed in chapter six. The variation in the filter performance resulting from the Zeta potential and conductivity changes did not prevent the identification of the effect of the surging flow since a control filter was operated during each test. However, the presence of this variation created difficulty in the comparison of results from different tests. Efforts were made to eliminate from the experimental data the variation generated by the changing suspension Zeta potential and conductivity. Several regression analysis equations were developed and used to normalise the experimental data to a standard suspension Zeta potential and conductivity. However, it was concluded that these normalisation procedures did not improve the usefulness of the experimental data. To illustrate the limitations of the regression analysis normalisation, an example is described below.

Normalisation of Filter One Turbidity Removal Efficiency:

The following table illustrates normalisation of the removal efficiency achieved by filter one using the following regression analysis equation.

$$\% \text{ Removal Efficiency} = 144.7 - 0.04(\text{Conductivity}) + 2.05(\text{Zeta Potential})$$

This equation was presented and described in chapter six and relates the removal efficiency achieved by filter one to the test suspension conductivity and Zeta potential. This equation was determined by multiple linear regression analysis and yielded a correlation coefficient of 0.92 with a standard error of 4.3%.

Run	F1 %Rem	F2 %Rem	Diff.Rem (F2-F1)	Cond. (uS/cm)	Zeta (mV)	Run Type	Norm. F1 %Rem
23	79.1	77.9	-1.2	780		c	82.3
24	80.3	77.5	-2.8	758		sf	82.6
25	75.6	75.7	+0.1	708	-16.6	c	87.3
26	92.1	91.9	-0.2	310	-18.9	sf	82.4
27	80.2	79.6	-0.6	635	-20.0	sf	85.8
28	89.8	90.1	+0.3	512	-16.1	c	82.5
29	68.3	66.2	-2.1	643	-24.1	sv	82.6
30	87.4	87.2	-0.2	625	-17.5	sv	87.5
31	87.4	87.9	+0.5	656	-15.9	c	85.4
32	93.8	93.8	0.0	584	-16.2	sf	89.6

F1, F2 – Filter one filter two, %Rem – Percentage removal of turbidity, Diff.Rem – Differential removal, Cond. – Conductivity, Norm. F1 %Rem – Normalised filter one percentage removal, c – control run, sf – fixed surging run, sv – variable surging run.

The table presents the removal efficiency achieved by filters one and two, the difference between filters one and two, the conductivity and Zeta potential in each test, the nature of the test run (surging/non-surging) and the normalised removal efficiency in filter one from tests twenty three to thirty two for comparison. The regression analysis equation shown above was used to normalise the removal efficiency achieved by filter one in each test to a conductivity of 700 uS/cm and a Zeta potential of –16 mV. The regression equation predicts a removal efficiency of 83.9% under these conditions. It can be seen that the normalised results for filter one ranged from 82.3 to 89.6%. The reason for this variation is the magnitude of the standard error associated with the regression analysis equation. The standard error of the regression analysis was 4.3%. The normalisation procedure works in principle but is not particularly useful due to the magnitude of the statistical error compared with the difference in filter performance resulting from the surges.

A second difficulty arose regarding the elimination of the effects of changing suspension chemistry from the performance of filter two. The regression equation described above can be used to modify the results of filter one since it was derived from the results of filter one. However, it cannot be used to modify the results of filter two. Separate regression analysis equations developed from the results of filter two would be required. However, filter two was operated with a range of different conditions. In some tests, filter two was operated identical to filter one. In other tests, filter two was subjected to surging flow. Of the surging tests, some were operated with fixed amplitude surges and others were operated with variable amplitude surges. Furthermore, these surges were applied at different frequencies in different tests. The result of these different operating characteristics was that there was insufficient data of a like nature available to develop multiple regression analysis equations for the performance of filter two. Subsequently, there was no sensible method available to eliminate the variation in filter performance resulting from the changes in suspension conductivity and Zeta potential from test to test.