Development of a Design Tool for Rapid Gravity Media Filtration in Water Treatment


Dissertation for partial fulfilment of the requirement for the degree of Master of Engineering in Water Quality Engineering

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List of Acronyms and Symbols

Acronym/Symbol	Description	Additional comment
A	Filter area	
A_1	Dimensionless porosity function for fluidized beds	With reference to bed expansion
AWWA	American Water Works Association	
B	Trough width	With reference to backwash troughs
C	Average filtrate quality	
c	Coefficient for calculating permeability coefficient	With reference to Darcy flow head loss calculations
C_0	Influent quality	
C_d	Weir coefficient	
cm	Centimetre	
d	Diameter	Typically related to media size
d_{10}	Effective size; the size of sieve opening that permits 10% of the grains (by dry weight) to pass, equating to the 10% passing point on the sieve analysis curve	Related to filter media grain size
d_{60}	size of sieve opening that will pass 60% of the grains (by dry weight)	Related to filter media grain size
d_{90}	size of sieve opening through which 90% of the grains will pass	Related to filter media grain size
d_c	Critical depth	With reference to backwash troughs
d_{eq}	Equivalent spherical diameter	Related to filter media
D_{inlet}	Diameter of backwash water inlet pipework	
d_m	Media size	
D_0	Water depth at upstream end of trough	With reference to backwash troughs
d_p	Particle size	
EBCT	Empty bed contact time	With reference to adsorption
Eq.	Equation	For numbering of equations in this dissertation
ES	Effective size (see also d_{10})	Related to filter media grain size
FI	Filterability Index	
g	Gravitational acceleration	
Ga	Galileo number	With reference to fluidization velocity calculations
GAC	Granular activated carbon	
H	Head Loss	
h	Hour	
H_{AP}	Height of air pipework within flume	With reference to flume depth
HDPE	High density polyethylene	
H_f	Flow normalized head loss	Related to head loss monitoring calculations
$H_{fittings}$	Head loss through fittings	
H_H	Height requirement for hydraulic purposes	With reference to flume depth
H_L	Head loss across media bed	
H_n	Flow and temperature normalized head loss	Related to head loss monitoring calculations

Acronym/Symbol	Description	Additional comment
$H_{nozzles}$	Head loss through filter nozzles	
h_{nozzle}	Head loss per nozzle	
$h_{o/f}$	Overflow depth	For weirs and troughs
H_{QT}	measured head loss at flow rate (Q_m) and temperature (T)	Related to head loss monitoring calculations
H_T	height of trough overflow above rested filter bed	With reference to backwash troughs
H_w	Height requirement for watertight joints	With reference to flume depth
in.	Inch	
k	Head loss coefficient	
kg	Kilogram	
k_I	Head loss coefficient due to inertial forces (turbulent flow)	With reference to the Ergun equation for head loss calculations
k_k	Kozeny coefficient	With reference to Darcy flow head loss calculations
K_n	Nozzle coefficient	
k_p	Permeability coefficient	With reference to Darcy flow head loss calculations
k_v	Head loss coefficient due to viscous forces (laminar flow)	With reference to the Ergun equation for head loss calculations
L	Media bed depth	
L_e	Expanded bed depth	
L/ES	Filter bed depth to effective size ratio	
L_{weir}	Weir length	
M	Media mass	
m	Metre	
m^2	Square metre	
m^3	Cubic metre	
m/h	Metres per hour	
Mℓ/d	Megalitre per day	
mm	Millimetre	
m/s	Metre per second	
N	Number of filters	
n	Nozzle density	
n_{ec}	Number of end contractions	Weirs and troughs
ntu	Nephelometric turbidity unit	
P	Trough depth	With reference to backwash troughs
p	Pressure	
Q	Water treatment plant design flow rate	
Q_a	Air flow rate	
Q_m	Measured flow rate	Related to head loss monitoring calculations
Q_n	Flow rate used for standard for normalization	Related to head loss monitoring calculations
q	Design flow rate	e.g. Total rate of discharge per trough with reference to backwash troughs; filter design flow rate with reference to filtration
q_{known}	Known flow rate	For filter nozzle head loss calculations
Re	Reynolds number	
Re_1	Modified Reynolds number	
$r_{l/w}$	Length-to-width ratio	For filter arrangement
S	Shape factor	With reference to filter media
s	Second	
SANS	South African National Standards	
SCADA	Supervisory Control and Data Acquisition	
SG_m	Specific gravity of the media	

Acronym/Symbol	Description	Additional comment
S_T	Trough centre-to-centre spacing	With reference to backwash troughs
S_V		
T	Temperature	
t	Time	
t_{bw}	Duration of backwash utilizing backwash water	
t_f	Duration of filter run	
t_{ftw}	Duration of filter-to-waste	
UBWV	Unit backwash volume	
UC	Uniformity coefficient	Related to filter media
UFRV	Unit filter run volume	the total amount of water through one square metre of filter media between backwashes (m^3/m^2/run)
UFWV	Unit filter-to-waste volume	
v	Velocity	
V_0	Velocity (rate) through filter	
V_b	Volume of water required for one filter backwash	
v_{bw}	Backwash rate	
$v_{bw}(t)$	Backwash rate at temperature (t)	
$v_{bw}(20°)$	Backwash rate at 20°Cx	
V_E	Flume entrance velocity	
V_{EQUAL}	Equalized velocity	
V_f	Volume of water filtered during a filter run	
v_f	Filtration rate	
V_{ftw}	Volume of water discharged during filter-to-waste	
$V_{L/E}$	Lateral entrance velocity	
$V_{L/E}$ / V_M	Cross-velocity ratio	
V_M	Media volume	
v_m	Cross velocity	
V_{mf}	Minimum fluidization velocity	
V_T	Total volume of media bed	
V_V	Void volume in media bed	
WHO	World Health Organisation	
ΔH	Change in head loss	Particular reference to clogging filter bed
ΔH_0	Head loss through clean filter	
Δp	Change in pressure	
°	Degree	Related to temperature unit
ε_D	Deposit porosity	
ε	Expanded bed porosity	
ε_0	Media porosity	
η_f	Filtration efficiency	
γ_i	Empirical constants in units of volume/mass	Related to clogging filter head
#	Number	
μ	Fluid dynamic viscosity	
µg/ℓ	Micrograms per litre	
μ_T	absolute viscosity of water at the normalized/standardized temperature	
$\mu(t)$	water dynamic viscosity in centipoise at a specific temperature (t)	
%	Percentage	
π	Mathematical constant pi; i.e ~3.14	
ψ	Sphericity	With reference to filter media

Acronym/Symbol	Description	Additional comment
ρ_m	Media density	
ρ_s	Particle density	
ρ_w	Water density	
σ	mass specific deposit (mass/volume) or quantity of solids per volume of filter bed	Related to clogging filter head
σ^*	volume-specific deposit (volume solids-volume filter)	Related to clogging filter head

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CHAPTER 1 : INTRODUCTION

1.1 Study Objective, Scope and Goals

1.1.1 Background to the Study

Drinking water has been treated for many years for health and aesthetic reasons. Water treatment processes, in combination, are selected to produce water to a specific drinking standard – see Chapter 2 for further details. One such process is filtration.

Filtration is the most commonly used water treatment process (Gray, 2010) and can be found at every water treatment plant in South Africa (Van Duuren, F. A., South Africa Water Research Commission., 1997). Therefore, the design, evaluation and improvement of filtration systems is (and should be) ongoing. Currently there is seemingly a lack of consolidated information to enable filter designers to quickly and easily design and evaluate various filtration systems, as it is not always possible to conduct thorough pilot-testing at design stage. A more detailed filter design tool will enhance the plant-wide water treatment design tool that incorporates a high-level filter design spreadsheet previously developed by Morrison (2019).

1.1.2 Objectives of the Study

The aim of this study is to help filter designers by presenting guidelines to the whole filtration process.

The objectives of this study are as follows:

- to conduct a literature review to consolidate key aspects and design parameters. This includes for media selection and characterisation, filtration rate selection, operation and control, backwash rates, head losses, filter components, configuration and geometry.
- to document the findings of the literature review herein.
- to develop a design tool in Microsoft Excel that incorporates various findings, thus ensuring that the designer obtains relevant insights to the various parameters and their effects.

1.1.3 Scope of the Study

Rapid filtration is the most common granular filtration technology (Crittenden *et al.*, 2012), utilizing higher filtration rates with a resultant smaller footprint that provides a more economical solution in water treatment (Morrison, 2019). The scope of this study is therefore limited to the development of a design tool for rapid gravity media filtration in water treatment.

The review, this dissertation and tool focus on the design of the filter itself, with the following aspects considered beyond the scope of this dissertation:

- Overall treatment process selection, including selection of filtration type (e.g. slow sand, precoat, upflow, pressure, gravity media and membrane)
- Chemical dosing pre- and post-filtration
- Biological treatment - in water treatment, this is considered to be a separate process requirement to typical filtration alone whereby the chemicals/contaminants are oxidized and reduced by bacteria, converting the chemicals/contaminants into harmless products (AWWA and American Society of Civil Engineers, 2012).
- Filtration models and pilot-studies, as this dissertation is focussing on design without pilot studies

- Treatment facility/building design and layout that incorporates the filter design, despite some aspects mentioned within this dissertation that would link to this
- Filter-related equipment design and performance monitoring thereof (e.g. backwash pumps, blowers and valves)

1.2 Dissertation Outline

This dissertation consists of 16 Chapters, References and an Appendix, the outline which developed based on interconnected topics through the literature review as shown in the mind map in (see overleaf).

Chapter 1 details the dissertation aims and objectives, as well as the dissertation structure.

Chapters 2 and 3 provide an introduction to water treatment and rapid gravity filtration in water treatment. Chapter 2 discusses the requirements for potable water treatment and the role of filtration within water treatment processes, and touches on pre-treatment systems. Chapter 3 introduces rapid gravity filtration in water treatment with details of the transport and attachment mechanisms that are at play, as well as an introduction to the various design considerations.

Chapters 4 to 15 detail the key aspects of filters and various design parameters identified during the literature review. Where applicable, the chapter concludes with incorporation of the details into the design tool.

Chapter 4 is dedicated to filter media, which includes the various types, properties and configurations.

Chapter 5 discusses filtration rates, followed by the hydraulics of flow through filters in Chapter 6. Herein, head losses in filter beds are discussed with a brief review of filtration process optimization.

Chapter 7 describes the various types of filter operation; namely options for equal rate and declining rate filtration. Although this is relevant to treatment facility/building design and operation and control, both of which are outside the scope of this dissertation, the details are included herein for the designer's awareness.

Chapter 8 details the backwashing of filters.

Chapter 9 focuses on plenum and flume hydraulics, including configurations and sizing. The details are included herein for the designer's awareness despite them being more useful for the treatment facility/building design that is outside the scope of this dissertation.

Chapter 10 presents various underdrain systems, followed by media retention options in Chapter 11.

Chapter 12 discusses filter arrangement, which includes considerations for configuration, number, size and depth of filters.

Chapter 13 reviews various filter performance monitoring parameters and equipment, included herein for the designer's awareness despite these details not inherently impacting on the filter design addressed by this dissertation.

Chapter 14 describes filter design aspects such as inlet channels, waste backwash water removal, channels and pipework for hydraulic connections, gates, valves and actuators, backwash pumps and blowers.

Chapter 15 provides information on common filter problems for the designer's awareness, despite the details being more relevant to investigation of existing filter designs, which is not the purpose of this dissertation.

Chapter 16 concludes this dissertation with the development of the design tool, conclusions and recommendations.

The Appendix provides guidance to the designer on the use of the filter design tool.

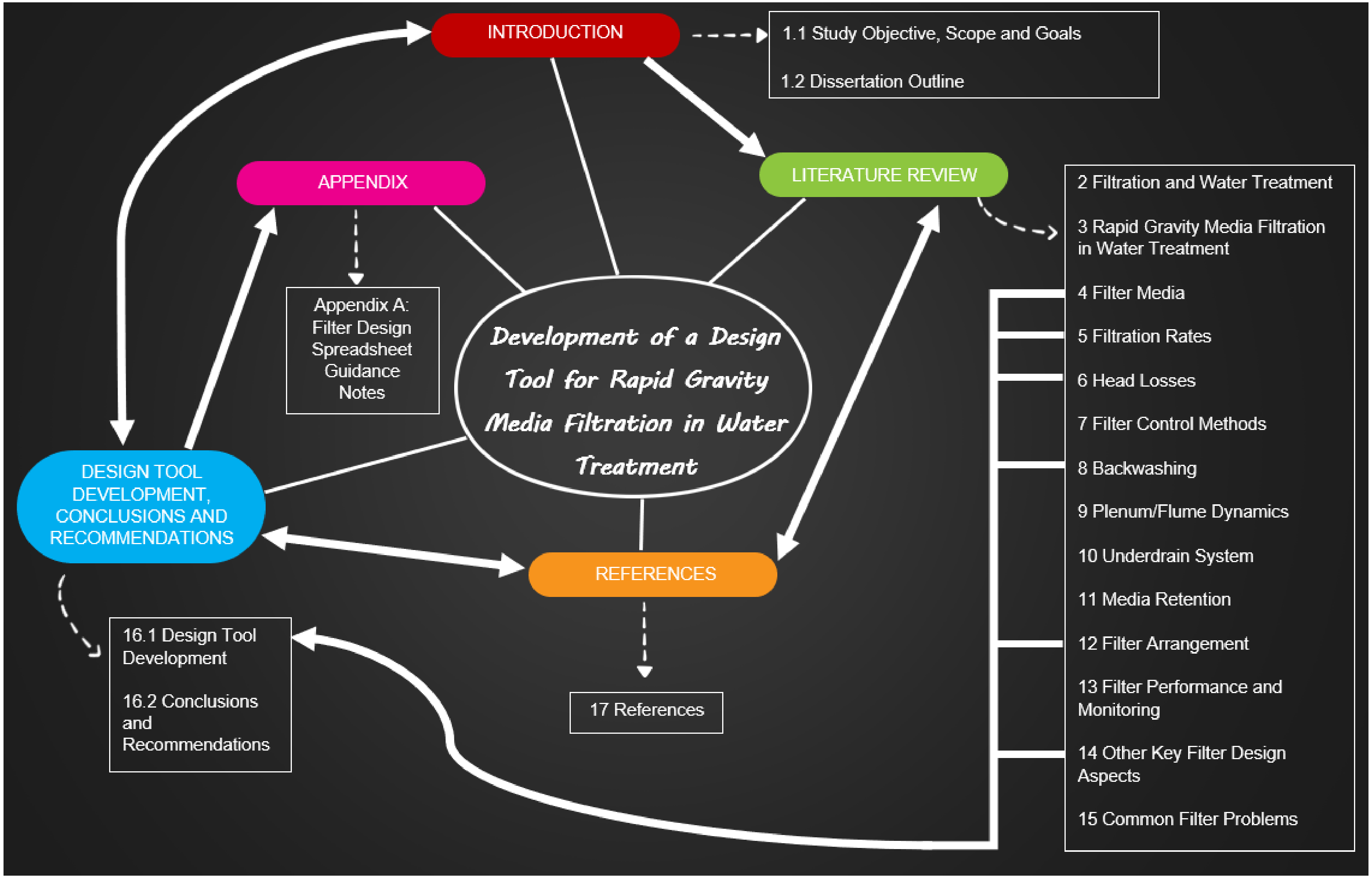


Figure 1 Mind Map (Chalkboard style) of Dissertation Outline

CHAPTER 2 : FILTRATION AND WATER TREATMENT

2.1 Introduction

Drinking water in South Africa must adhere to the quality standard limits contained in SANS 241-1:2015: *The South African National Standard for Drinking Water Part 1: Microbiological, Physical, Aesthetic and Chemical Determinands* (SANS, 2015). Furthermore, the World Health Organization (World Health Organisation, 2017) continually updates guidelines for drinking water quality in terms of physical and chemical quality parameters.

Turbidity represents the particles in the water. Although these particles may not be harmful themselves, they tend to shield pathogens from the disinfection process (Edzwald, 2011). This increases the possibility of resistant pathogens such as the Cryptosporidium species and Giardia species to pass through the disinfection process and infect consumers (Edzwald, 2011).

The turbidity of the water can be reduced by dosing coagulating chemicals (charge neutralization of negative charged colloids causing them to stick together) and providing mechanisms for particles to make contact and aggregate (flocculation). These larger particles can then be removed by phase separation processes such as settling (sedimentation/clarification) and filtration.

Filtration is the most commonly used water treatment process (Gray, 2010) and can be found at every water treatment plant in South Africa (Van Duuren, F. A., South Africa Water Research Commission., 1997). Filtration is a physical process that separates solids from liquids. The filtration process removes particles from the water by passing the water through a porous medium that retains the solid particles either on the surface or within the medium while the filtered water passes through.

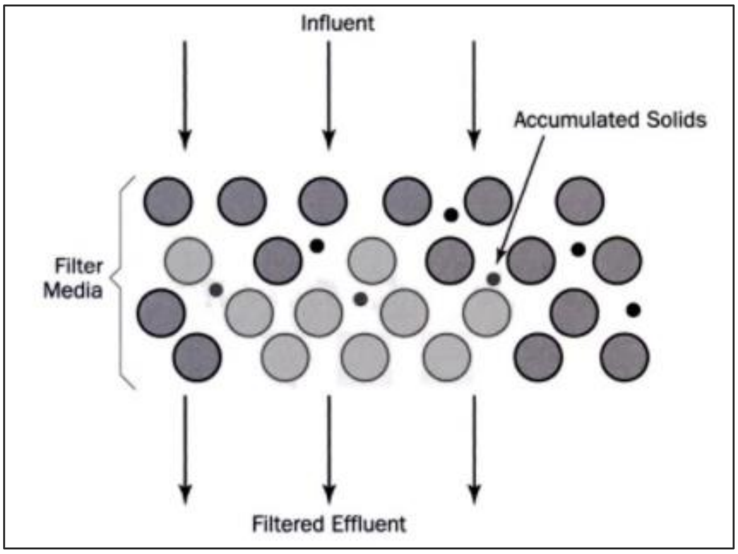


Figure 2 Filtration Principle (Beverly, 2011)

Particles to be removed can include algae, sediment, clay, other organic and inorganic particles as well as pathogens (Crittenden *et al.*, 2012), with sizes ranging from 0.1 to 1000 micrometres with variable shapes and densities (Montgomery, James M. Consulting Engineers, 1985). Of particular concern are the protozoa Giardia of 5 – 15 microns and Cryptosporidium of 4 – 7 microns (AWWA and American Society of Civil Engineers, 2012).

2.2 Pre-Filtration Water Treatment Processes

Filter pre-treatment is essential to ensure that particles are destabilised for effective removal in the filtration process (as discussed in Chapter 3.2).

The overall treatment process for particle removal can include coagulation, flocculation and sedimentation for effective filtration. Although the selection of the treatment process is beyond the scope of this dissertation, the filtration types based on treatment processes are listed in Table 1 for reference.

Table 1 Filtration Types Defined According to Upstream Treatment Processes

Filtration Type	Coagulation	Flocculation	Sedimentation/ Clarification
Conventional filtration	√	√	√
Direct Filtration	√	√	X
In-line/contact Filtration	√	X	X

Hendricks (2010) and Edzwald (2011) indicate that conventional treatment is the default filtration type, complimenting a multiple barrier approach to water treatment. The chemicals involved is beyond the scope of this dissertation. Direct and in-line filtration can still be considered for source waters with low turbidity (Edzwald, 2011), and World Health Organisation (2017) gives the impression that direct filtration is still practiced.

AWWA and American Society of Civil Engineers (2012) provides a general guideline for pre-treatment selection as per Table 2; although, pilot studies are still recommended for site-specific conditions because there are many variables in addition to turbidity that influence the process (e.g. climate, raw water quality, particle characteristics and the filter design parameters itself) .

Table 2 General Guideline for Granular Media Filtration Pre-treatment Selection (AWWA and American Society of Civil Engineers, 2012)

Filtration Type	Raw Water Turbidity (ntu)
In-line	<20
Direct	<80
Conventional	Up to 3000

Despite these guidelines, Hunce, Soyer and Akgiray (2019) suggests that in-line and direct filtration only be implemented for raw water of better quality (regularly up to 10 ntu with the odd increases up to 40 ntu).

Additionally, the selection diagram shown in Figure 3 may be of use to the design engineer during selection of treatment processes, linking various processes to raw water turbidity and concentration of chlorophyll a. It is shown that single or double stage direct filtration is suitable for turbidities less than 10 and 50 ntu, respectively, when the concentration of chlorophyll a is below 10 μg/ℓ. For higher turbidities and concentrations of chlorophyll a, dissolved air flotation or settling (i.e. sedimentation) is required as shown in the diagram.

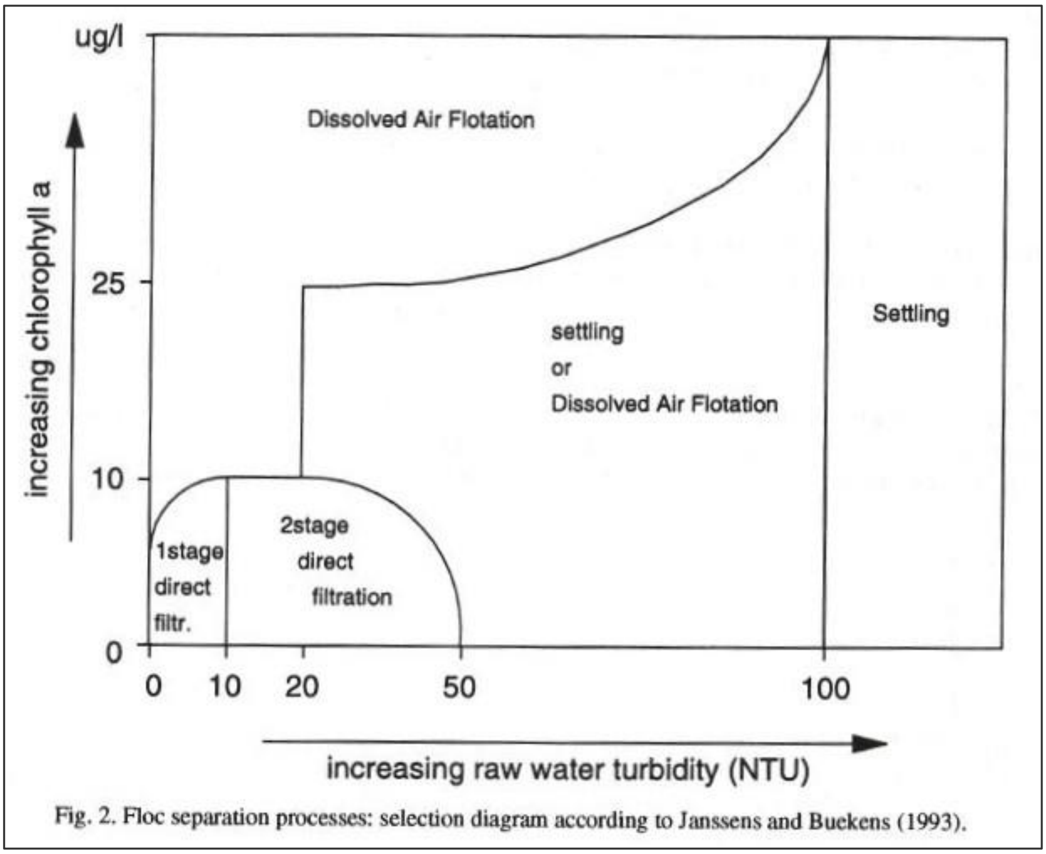


Figure 3 Filtration Type Selection Diagram (Janssens and Buekens (1993) as cited in van Puffelen *et al.* (1995))

It has long been indicated that pilot studies are the preferred method for process selection when compared with published literature, analytical models and the designer's previous experience (Montgomery, James M. Consulting Engineers, 1985).

2.3 Water Treatment Standards and Filtration Capabilities

A summary of various standards and limits for water treatment is given in Table 3.

Table 3 Water Treatment Standards and Limits

Agency	Standards		Cryptosporidium	Giardia	Virus	Bacteria	Turbidity*	Reference
South African National Standards (SANS)	SANS 241-1: 2015		Zero count per 10 L sample	Zero count per 10 L sample	For most viruses – not present in sample batch (e.g. Somatic coliphages zero count per 10 ml sample)	For most bacteria – not detectable in sample batch (e.g. E.coli zero count per 100 ml sample; total coliforms ≤ 10 per 100 ml sample)	Maximum 1.0 ntu (final water)	(SANS, 2015)
United States Environmental Protection Agency (USEPA)	Surface Water Treatment and Enhanced Surface Water Treatment Rules	Primary Drinking Water Standards	2-log removal (99 percent), With additional 0- to 3-log removal based on "bin classifications" in accordance with the Long Term 2 Enhanced Surface Water Treatment Rule	3-log removal (99.9 percent)	4-log removal (99.99 percent)		≤ 0.3 ntu – direct, conventional Combined filter effluent (95% samples in a month) ≤1.0 ntu – direct, conventional Individual filter effluent	(AWWA and American Society of Civil Engineers, 2012) (Protection Agency EPA Environmental, 2006; Environmental, 2002)
European Commission	Water Framework Directive (WFD 2000/60/EC) – Drinking Water Directive (98/83/EC)		Unclear	Unclear	unclear	0/100 ml	Normally < 1.0 ntu (final water)	(Gray, 2010) (EU., 1998)
National Health and Medical Research Council (NHMRC)	Australian Drinking Water Guidelines (ADWG)		No guideline value set	No guideline value set	For most viruses – not present in sample batch (e.g. Coliphages should have zero count per 100 mL sample of drinking water)	For most bacteria – not detectable in sample batch (e.g. E.coli zero count per 100 ml sample)	< 0.2 ntu for filtration of Giardia and Cryptosporidium < 1 ntu for effective disinfection	(NHMRC, 2011)

Agency	Standards	Cryptosporidium	Giardia	Virus	Bacteria	Turbidity*	Reference
Ministry of Health, New Zealand	Drinking-Water Standards for New Zealand	Less than 1 per 100 litre sample	Less than 1 per 100 litre sample	No guideline set	For E.coli, less than 1 in 100 ml sample	< 1.0 ntu in 95% of required sampling period and up to 2.0 ntu for any 3 minute period (Final water)	(New Zealand Ministry of Health, 2008)
*	Turbidity is the common filter performance measurement as discussed in Chapter 13.2.2, stated in nephelometric turbidity units (ntu).						

Typical removal estimates achieved by filtration are presented in Table 4. Additionally, Table 4 also includes the typical removal estimates for rapid gravity media filtration defined by the WHO. Disinfection, therefore, also plays a role in water treatment for compliance with the requirements of Table 3.

Table 4 Typical log-Removal Estimates Achieved by Filtration Type Based on Combined Filtrate Turbidity

Filtration Type	Cryptosporidium	Giardia	Bacteria	Viruses	Turbidity (individual filter effluent)	References
Conventional	2.0 – 3.0	2.5	0.5 - 1.0	2.0	≤0.3 ntu in 95% of samples each month; never >5 ntu	(Edzwald, 2011) (NHMRC, 2011) (New Zealand Ministry of Health, 2008)
Direct	2.0 – 2.5	2.0		1.0		
Rapid Gravity	0.4 – 3.3, typically 3.0 for the defined turbidity measure herein	0.4 – 3.3	0.2 – 4.4	0 – 3.5, typically 1.0 – 2.0 for the defined turbidity measure herein	≤0.3 ntu in 95% of Samples; never >1 ntu	(World Health Organisation, 2017) (New Zealand Ministry of Health, 2008)

CHAPTER 3 : RAPID GRAVITY MEDIA FILTRATION IN WATER TREATMENT PLANTS

3.1 Introduction

Granular media filtration is the most common type of filtration process used for removal of particles in water treatment (Crittenden *et al.*, 2012; Montgomery, James M. Consulting Engineers, 1985). Other filter types exist, such as slow sand, diatomaceous earth (precoat) and membrane filtration (Edzwald, 2011).

For gravity media filtration, water flow is by gravity through a porous granular medium in a downward direction. The filter bed is contained in a filter bay and supported by an underdrain system that allows filtered water to pass through whilst maintaining the position of the media bed. A clean filter bed (i.e. at the start of a filter run) has large pores prior to capturing particles and will therefore have poor filtrate quality (i.e. high turbidity). The filter undergoes “ripening” as particles are captured and the pores reduce and capture more particles (see Chapter 8.10 for filter ripening).

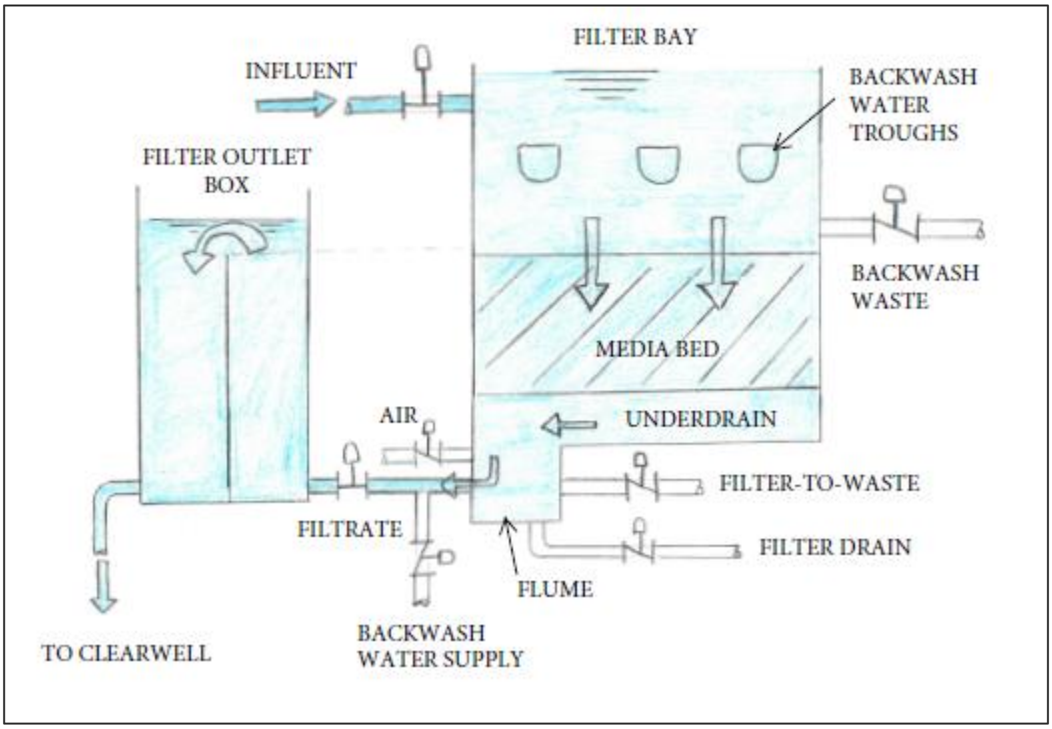


Figure 4 Typical Filtration Flow Path through a Rapid Gravity Media Filter (Adapted from Beverly (2011))

As particles are captured, pore size reduces further and velocities through the bed increase with particles being captured deeper and deeper within the filter bed (see Chapter 3.2). Once the media bed becomes clogged with particles, particles can escape into the filtrate, known as “breakthrough”. At this point, the filtration process is stopped, and backwashing is initiated to clean the filter (by reversing flow through the filter). A filter run is defined as the time between backwashes. Backwashing and other aspects that may lead to backwash initiation are discussed further in Chapter 8 : .

Crittenden *et al.* (2012) claims that rapid filtration is the most common granular filtration technology. Increased filtration rates allow for smaller surface area and therefore reduces capital costs, which has led to the ever-increasing filtration rates when the process requirements are still being met.

Binnie, Kimber and Smethurst (2002) indicates that rapid gravity filtration can address water quality issues such as turbidity, colour, iron and manganese removal, cryptosporidium and algae removal. Coagulation pre-treatment is essential for rapid gravity filtration to ensure effective removal of viruses, bacteria and protozoa (Gray, 2010) and small particles for adequate filtrate quality (Binnie, Kimber and Smethurst, 2002).

Gravity media filters can also be converted for biological treatment by means of biological rapid granular-media filtration, called biofilters. The primary difference between filters and biofilters is the encouragement of bacteria accumulation on the surface of filter media particles; i.e. biological treatment by biofilm process. (AWWA and American Society of Civil Engineers, 2012).

3.2 Particle Removal Processes

Particles in the influent are removed either by straining on the surface of the filter bed or by attachment of the particles within the filter bed. Straining should be kept to a minimum to prevent a build-up of matter on the filter bed surface, which would result in a rapid increase of the head loss through the filter (termed blinding of the filter) and reduced filter runs (Binnie, Kimber and Smethurst, 2002; Montgomery, James M. Consulting Engineers, 1985).

Herzig *et al.*, (1970) and Boller (1980) as cited in Binnie, Kimber and Smethurst (2002) and in Montgomery, James M. Consulting Engineers (1985) suggest that straining becomes probable when the particle size is greater than 20% of the media size, and Crittenden *et al.* (2012) suggests straining occurs when particles are 15% greater in size compared to the media. Such particles should then be removed by implementing conventional filtration that includes the upstream clarification , and if direct filtration is implemented then the floc formation must be limited in size to limit the straining effect (Binnie, Kimber and Smethurst, 2002).

Rapid filtration requires a more uniform media as discussed in Chapter 4 : , to encourage capturing of particles throughout the depth of the filter rather than straining particles at the surface (Binnie, Kimber and Smethurst, 2002).

Particle removal is a two-step process. Firstly, the particle is transported to the media surface (or the surface of the already accumulated particles) and secondly, the particles attach to these surfaces (AWWA and American Society of Civil Engineers, 2012; Van Duuren, F. A., South Africa Water Research Commission., 1997).

Montgomery, James M. Consulting Engineers (1985) and Binnie, Kimber and Smethurst (2002) identifies the main transport mechanisms for suspended particles in the influent to possibly make contact with the media surface, as follows:

- Interception – when a particle passes a grain of filter media within a small distance and collides with the media grain surface (Van Duuren, F. A., South Africa Water Research Commission., 1997); interception efficiency is proportional to the ratio of particle diameter to media diameter (Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012)

- Sedimentation – when the particle is drawn by gravity towards the media grain surface (Van Duuren, F. A., South Africa Water Research Commission., 1997) with the particle being large and having a much greater density than water, sedimentation efficiency is proportional to the ratio of particle's settling velocity to fluid velocity (i.e. filtration rate), with settling velocity being proportional to density difference and particle diameter squared (Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012)

- Diffusion for colloids is defined by Brownian motion; i.e. random motion (Van Duuren, F. A., South Africa Water Research Commission., 1997). This mechanism is most significant in rapid filtration for particles less than 1 micron in size. Diffusion probability increases with temperature (T) and with smaller particle and media sizes (d_p and d_m, respectively) (Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012)

- Hydrodynamic forces at high velocities and turbulence can supplement particle removal by moving particles towards the media due to particle rotation and resultant pressure difference across the particle, although not substantially (Binnie, Kimber and Smethurst, 2002).

These transport mechanisms are shown in Figure 5.

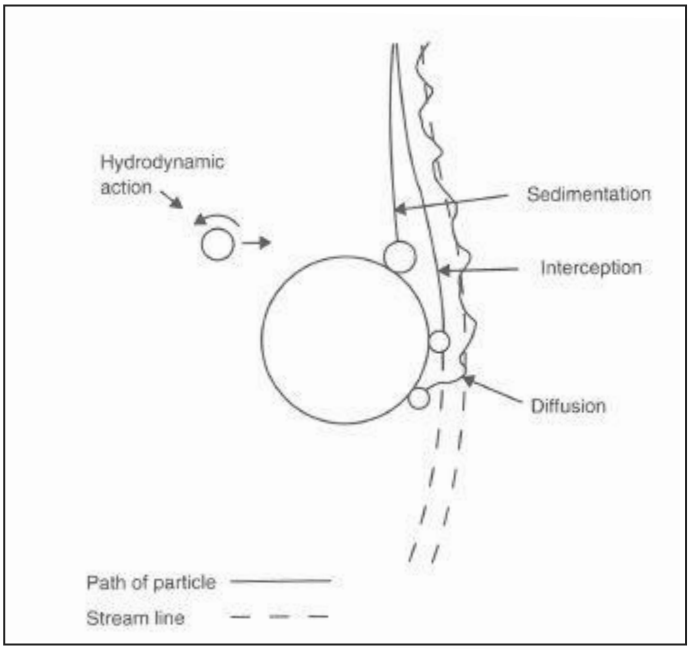


Figure 5 Transport Mechanisms for Particles in Granular Filtration (Binnie, Kimber and Smethurst, 2002)

A fifth transport mechanism identified by Montgomery, James M. Consulting Engineers (1985) and Binnie, Kimber and Smethurst (2002) is impaction, which refers to when the inertia of a particle being filtered is greater than the hydrodynamic force of the particle passing the media and hence collides with the media. This mechanism plays a minor role in water treatment due to typically low velocities and Reynolds numbers.

Attachment mechanisms forming the second part of the particle removal process may involve "electrostatic interactions, chemical bridging, or specific adsorption", which are all influenced by the chemical properties of the water, filter media and any pre-treatment chemicals (Cleasby, 1969).

Montgomery, James M. Consulting Engineers (1985) and Binnie, Kimber and Smethurst (2002) continue to explain that particle attachment to the media occurs only if they are oppositely charged. A media surface covered in particles can only capture more particles effectively if the particles are sufficiently destabilized. Therefore, coagulation is essential for effective filtration.

Cleasby (1969) mentions that the main transport and attachment mechanisms may change as filtering progresses due to a change in the various parameters such as clean filter media versus media with attached particles, and flow velocities for example, all of which will influence the filtrate quality and head loss development patterns.

Furthermore, detachment of particles from the surface may occur if the shearing force becomes larger than the forces holding the particle to the media. This is possible because as particles collect on the media surface, the gaps between the media become smaller reducing the area between the media and increasing the velocity through the filter bed, thereby increasing the hydrodynamic shearing forces. (Montgomery, James M. Consulting Engineers, 1985). Rapid filtration therefore depends on "proper destabilization of particles by coagulation" to ensure a "high attachment efficiency" (Crittenden *et al.*, 2012). An increase in pH or decrease in ionic strength for constant flow can also lead to detachment (Crittenden *et al.*, 2012), with filters performing optimally under constant operating conditions with any changes being gradual (Crittenden *et al.*, 2012).

The efficiency of particle removal based on particle size is depicted in Figure 6. Removal efficiency is lowest for particle sizes in the region of 1 micron (Binnie, Kimber and Smethurst, 2002; Edzwald, 2011), with better removal efficiencies of the larger particles by interception and the very small particles by diffusion (Edzwald, 2011).

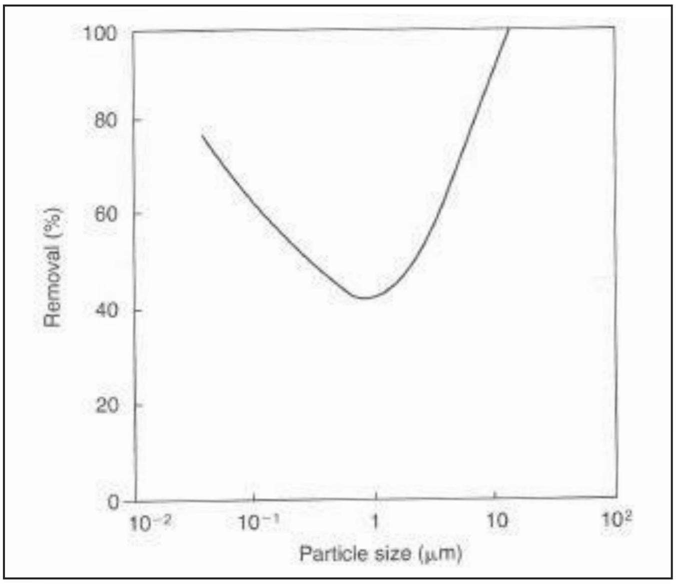


Figure 6 Typical Particle Removal Efficiency in Granular Filtration (Binnie, Kimber and Smethurst, 2002)

3.3 Filter Design Considerations

Various interrelated aspects are involved in the overall design of rapid gravity media filtration.

In terms of process design, the designer must consider the following:

- Treatment selection guidelines
- Type of pre-treatment system
- Type and size of filter media
- Depth of media
- Filtration rate
- Backwash system and backwash rates

Furthermore, various mechanical considerations are required, which include the following:

- Type of filter control system
- Type of underdrain system
- Filter arrangement such as configuration, number of filter units and size of filter units.
- Filter performance monitoring equipment
- Other key filter design aspects such as filter inlet design, washwater removal systems, piping and valves

Hendricks (2010) points out that modern filtration practice involves pilot plants for design and operation.

Site-specific conditions will impact filter performance including for aspects such as source water quality, particle properties and climate. Pilot studies are generally recommended for all new applications of media filtration units to provide an optimum process design. Pilot filters and jar testing are particularly valuable for determining optimum chemical dosing upstream of the filters. (AWWA and American Society of Civil Engineers, 2012). However, it is not always possible to conduct pilot studies due to timeframes and costs and the designer is required to utilise experience in previously designed and operating filter plants. Crittenden *et al.* (2012) explains that experience in operating rapid filters can be sufficient for design purposes; however, pilot testing would still be necessary to assess increasing rates and the effects of other treatment processes coupled with filtration.

This dissertation particularly provides the designer with insights to these filter aspects, and the tool as a design guideline for filters that are not pilot-tested.

3.4 Filterability

Due to the complex nature of granular filtration, it would be advantageous to predict whether a suspension is filterable, and by what filter media. Cleasby (1969) proposed a "simple" filterability index with associated test procedure for predicting filter performance (as well as a means for plant optimization and comparisons of filterability of suspensions across the world), and proposed that the American Water Works Association (AWWA) Task Group adopt this approach and do further data collection and assessment of the usefulness of the filterability index. Ives (1979) explained that Cleasby's Filterability Index was complex with difficult-to-measure parameters ("filter coefficient" and "specific deposit") and proposed a similar Filterability Index (FI) using compact, simple laboratory apparatus to determine the parameters.

$$FI = \frac{HC}{v(C_0 - C)t} \quad (Eq. 3.1)$$

Where H = head loss during filter run (measured by manometer)

C = average filtrate quality

C_0 = influent quality

v = approach velocity

t = time of filter run

This method of determining the FI could be utilised as a pre-screening step ahead of pilot studies for design of filters, and for operational checks of filters at water treatment plants. Hunce, Soyer and Akgriay (2019) indicates that this method has been used by many researchers between 1986 and 2017 to assess filter performance; however, further studies are required to clarify whether the FI can be used to establish filtration effectiveness.

Logsdon *et al.* (2002) refers to an alternate plant-specific filterability index as an assessment of the pre-treatment, whereby the Chester Water Authority's conventional treatment plant determines this index as the ratio of the "time to filter settled water" to "time to filter filtered water". No further details on this method were found during this study.

CHAPTER 4 : FILTER MEDIA

4.1 Overview

A porous medium is utilized in filtration.

Filter media is the prime component of gravity media filtration. Larger media sizes will have lower clean bed head loss, but the larger pore sizes for this media make it less effective at capturing particles. (Logsdon *et al.*, 2002; Sincero and Sincero, 2002).

Beverly (2011) describes the perfect filter media as capable of “high solids storage, low head loss, consistently high removal efficiency, and very high operating rates”. Manufacturing processes and physical constraints are filter media restrictions.

A wide range of media for filtration exists. Granular materials include sand, anthracite, garnet and ilmenite (AWWA and American Society of Civil Engineers, 2012; Crittenden *et al.*, 2012; Edzwald, 2011). Granular activated carbon (GAC) is often used when taste-and-odour control and organics removal is necessary (Beverly, 2011; Edzwald, 2011) and sometimes when filtration and biological treatment are combined; i.e. in biofilters (Crittenden *et al.*, 2012; Edzwald, 2011).

The designer has control over various aspects such as “bed composition, bed depth, grain size distribution, and, to a lesser extent, specific gravity”. Various other properties include “hardness or abrasion resistance, grain shape, acid solubility, impurities, moisture, adsorptive capacity, manner of shipment, and other such factors”. (AWWA and American Society of Civil Engineers, 2012).

Table 5 lists the typical characteristics for media used in rapid filters.

Table 5 Typical Characteristics for Media Used in Rapid Filters (Crittenden *et al.*, 2012)

Property	Unit	Sand	Anthracite	GAC	Garnet	Ilmenite
Effective Size, ES	Mm	0.4 – 0.8	0.8 – 2.0	0.8 – 2.0*	0.2 – 0.4	0.2 – 0.4
Uniformity Coefficient, UC	-	1.3 – 1.7	1.3 – 1.7	1.3 – 2.4*	1.3 – 1.7	1.3 – 1.7
Specific gravity	-	2.65	1.4 – 1.8	1.3 – 1.7	3.6 – 4.2	4.5 – 5.0
Porosity, ε_0	-	0.40 – 0.43	0.47 – 0.52	Not available	0.45 – 0.58	Not available
Hardness	Mohs	7	2 – 3	Low	6.5 – 7.5	5 – 6
* Typical GAC properties for adsorption applications are ES of 0.5 – 3 (Crittenden *et al.*, 2012) and UC of 1.9 – 2.1 (Beverly, 2011). See also Chapter 4.4.						

The higher UC for GAC compared to other media relates to more fines being present at the surface after backwash, which is advantageous for adsorption filters because the surface area is increased. For combined filtration and adsorption applications, this could shorten filter runs if the influent water quality is low; i.e. clarification may be necessary to ensure high quality influent water (Beverly, 2011).

It is recommended that anthracite specifications must limit the number of production sources to one source, to prevent a wide range of properties within the media (Beverly, 2011).

4.2 Media Properties

4.2.1 General

There is no specific standard for filter media in South Africa and designers must rely on international standards, previous experience and local media availability (Van Duuren, F. A., South Africa Water Research Commission., 1997).

The AWWA Standards B100 (Granular Filter Material) and B604 (Granular Activated Carbon) exist to ensure good quality, providing a standard for purchasing and installation of the media; and these standards clearly state that it is not for filter design purposes.

Note that a range is generally specified for filter media properties because it is not possible to produce media precisely (Beverly, 2011).

4.2.2 Size and Uniformity

Media size affects particle removal, head loss and backwashing requirements (Edzwald, 2011).

Media size is commonly specified by the effective size (ES) and the uniformity of the filter media is specified by the uniformity coefficient (UC).

ES is determined by sieve analysis and is defined as the 10th percentile size, which is the size of sieve opening that permits 10% of the grains (by dry weight) to pass, equating to the 10% passing point on the sieve analysis curve. ES is often abbreviated by d_{10} as shown in the example sieve analysis in Figure 7.

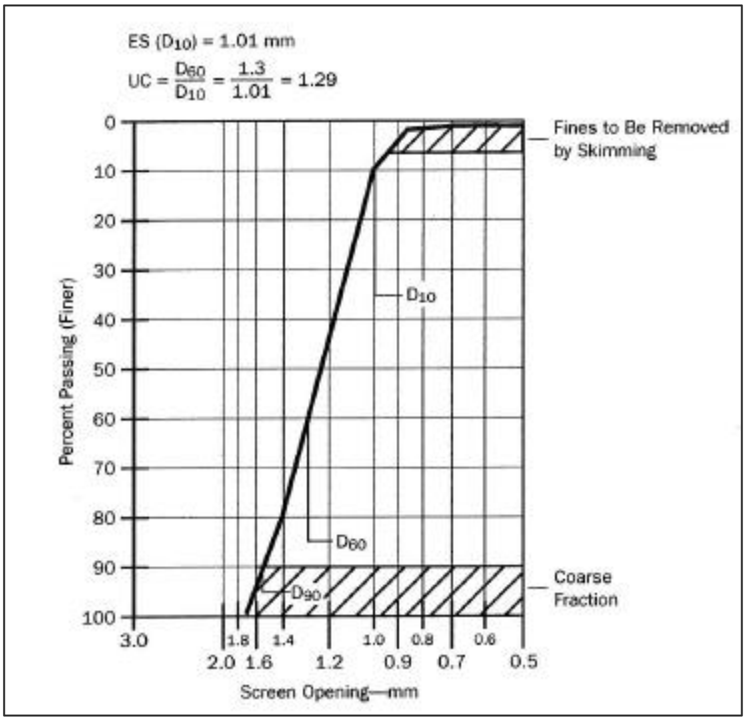


Figure 7 Example Sieve Analysis for Anthracite (Beverly, 2011)

Logsdon *et al.* (2002) points out that regular sieve analyses can uncover whether media is getting smaller (by being worn down by abrasion) or increasing in size (due to calcium carbonate precipitation accumulation, for example).

Selection of ES influences filtration rates, filtrate quality and backwash rates. Smaller filter grains will capture particles more effectively and produce better filtrate quality (i.e. better turbidity removal), but the length of filter run will decrease. Mismatch of ES and backwash rates can lead to media loss (rate too high) or inadequate cleaning (rate too low). Non-fluidizing coarse media can be forgiving in this regard because it requires very low backwash rates (Beverly, 2011).

ES also affects the design of media retention mechanisms – see Chapter 11 : .

Media can range in size from 0.1 to 10 mm (Montgomery, James M. Consulting Engineers, 1985), but more commonly tends to range between 0.4 and 1.5 mm (Binnie, Kimber and Smethurst, 2002). Effective sizes for different media are typically 0.45 to 0.7 mm for sand, 0.9 to 1.7 mm for anthracite and 0.2 to 0.3 mm for garnet or ilmenite (Logsdon, Gary S., 2008). Crittenden *et al.* (2012) indicates typical ES range for rapid filtration as 0.5 – 1.2 mm. Refer also to Table 5 in Chapter 4.1.

The designer's specification for ES should allow a tolerance due to sieving and sampling difficulties as well as the tolerances allowed for sieve manufacturing. Edzwald (2011) suggests a tolerance of ±10%, whereas De Lathouder (1973) as cited in Van Duuren, South Africa Water Research Commission (1997) suggests ±5% to be considered acceptable. For example, an ES of 0.45 – 0.55 mm should be specified for a chosen ES of 0.5 mm (Edzwald, 2011).

Uniformity coefficient (UC) is defined as the ratio d_{60}/d_{10}, where d_{60} equates to the size of sieve opening that will pass 60% of the grains (by dry weight). A higher UC is indicative of greater nonuniformity and should therefore be as low as practically possible.

Rapid filtration requires a more uniform media for lower head loss at the higher rates (Binnie, Kimber and Smethurst, 2002) and a uniform media (UC of 1.3 – 1.7) prevents the undesirable media classification of fine to coarse grains downward through the filter bed during backwash (Hofkes *et al.*, 1981). A low UC ensures that the larger media grains (say d_{90}) can be fluidized at a rate that is not too high for smaller media grains (d_{10}) (Logsdon, 2008). Despite Hendricks (2010) indicating a typical UC of 1.5 (referenced as a maximum in the South African context by Van Duuren, South Africa Water Research Commission (1997), Crittenden *et al.* (2012) indicates that the UC should be less than 1.4 for rapid filtration. According to Kawamura (1999), the design of any filter bed should specify a low UC of less than 1.4 and preferably 1.3. This might be difficult or expensive to achieve in South Africa (Ceronio (1993) as cited in Van Duuren, South Africa Water Research Commission (1997).

The size of sieve opening through which 90% of the grains will pass (d_{90}) is important for effective backwashing because the larger media grains must also be fluidized (not only the smaller (d10-size) media grains (Logsdon, 2008) – see Chapter 8 : . Logsdon *et al.* (2002) notes that this size is often not measured and can be calculated using the following equation:

$$d_{90} = d_{10} \times 10^{1.67 \times logUC} \quad (Eq. 4.1)$$

(Cleasby and Logsdon (1999) as cited in Logsdon *et al.* (2002); (Edzwald, 2011; Logsdon, 2008))

4.2.3 Acid Solubility

Beverly (2011) brings attention to aggressive filter influent water, particularly that of low pH. If this is anticipated, the design should include filter media (and support media, if applicable) with low acid solubility to lessen degradation.

4.2.4 Hardness, Toughness and Friability

Beverly (2011) defines friability as the ease of crumbling, which is important for anthracite. A high friability crumbles easily and leads to fines and loss of the media. Friability less than 4% is recommended.

Hardness is important for resistance to abrasion and breakdown of filter media during backwash (Crittenden *et al.*, 2012).

Hardness is ranked on the Moh table, which is a relative ranking ranging from 1 for talc hardness to 10 for diamond hardness (Crittenden *et al.*, 2012). Sand (typically 7 Moh (Logsdon, 2008)), garnet and ilmenite (typically minimum 5 Moh (Logsdon, 2008) are hard enough to withstand abrasion and need not be a concern. However, anthracite and GAC are generally not hard enough and should generally be specified with a minimum hardness of 2.7 Moh to reduce abrasion effects. GAC requires special care and hardness is decided in accordance with AWWA B604 Standard for Granular Activated Carbon

(Crittenden *et al.*, 2012). GAC durability depends on its resistance to abrasion that occurs during transportation, installation, backwashing and reactivation. A low abrasion resistance will result in crushed GAC and increased fines that can lead to loss of GAC, increased head loss development or filtrate quality deterioration. (AWWA and American Society of Civil Engineers, 2012).

Very hard material can be brittle and therefore toughness is more desirable (Beverly, 2011).

4.2.5 Shape, Sphericity and Porosity

Mathematical models often assume a spherical media grain (Crittenden *et al.*, 2012), but with grain shape affecting the sieve analysis, the "equivalent-volume" sphere is typically 5 – 10 % larger for sand and anthracite and 2% larger for garnet than the grain diameters determined by the sieve analysis, as discovered by Cleasby and Woods (1975) as cited in Crittenden *et al.* (2012).

Sincero and Sincero (2002) expresses equivalent spherical diameter d_{eq} as:

$$d_{eq} = \left(\frac{6}{\pi}\right)^{\frac{1}{3}} \varphi^{\frac{1}{3}} ES \quad (Eq. 4.2)$$

Apart from affecting the size determined by the sieve analysis, shape also influences how the media grains are packed in the filter bed (and hence porosity) and the hydraulics of flow through the filter bed; i.e. head loss through a clean filter bed and the backwash rate required for bed fluidization (Crittenden *et al.*, 2012; Edzwald, 2011).

Media grain shapes are often characterised by sphericity (ψ) or shape factor (S), where sphericity is the ratio of equivalent-volume sphere surface area to actual grain surface area and is linked to shape factor as shown in the following equations.

$$\psi = \frac{surface\ area\ of\ equivalent - volume\ sphere}{actual\ surface\ area\ of\ grain} \quad (Eq. 4.3)\ (\text{Crittenden } et\ al., 2012)$$

$$\mathrm{S} = \frac{6}{\psi} \quad (Eq. 4.4)\ (\text{Crittenden } et\ al., 2012)$$

So ψ = 1 and S = 6 for spherical grains.

Filter bed porosity (ε_0) is the percentage of filter bed volume not occupied by the media, calculated as follows:

$$\varepsilon_0 = \frac{V_V}{V_T} = \frac{V_T - V_M}{V_T} \quad (Eq\ 4.5)\ (\text{Crittenden } et\ al., 2012)$$

or equivalently

$$\varepsilon_0 = 1 - \frac{M}{\rho_m V_T} \quad (Eq. 4.6)\ (\text{Van Duuren, F.A., South Africa Water Research Commission., 1997})$$

Where

V_V = void volume in media bed

V_T = total volume of media bed

V_M = media volume

M = media mass

ρ_m = media density

Table 6 indicates the various media grain shapes and their associated typical shape factors and porosities.

Table 7 lists typical sphericities for different media.

Table 6 Typical Sphericity, Shape Factor and Porosity for Different Media Grain Shapes (Montgomery, James M. Consulting Engineers, 1985)

Media Grain Shape	Sphericity, ψ	Shape Factor, S	Porosity, ε_0
Spherical	1.00	6.0	0.38

Media Grain Shape	Sphericity, ψ	Shape Factor, S	Porosity,ε_0
Rounded	0.98	6.1	0.38
Worn	0.94	6.4	0.39
Sharp	0.81	7.4	0.40
Angular	0.78	7.7	0.43
Crushed	0.70	8.5	0.48

Table 7 Typical Sphericity for Different Media Types (Edzwald, 2011)

Media	Sphericity, ψ
Sand	0.7 – 0.8*
Anthracite	0.46 – 0.6*
GAC	0.75
Garnet	0.6
Ilmenite	Not available
* (Logsdon, 2008) too	

The silica sands produced in South Africa typically have sphericities ranging from 0.5 to 0.7 and from 0.7 to 0.8 depending on the source (Ceronio (1993) as cited in Van Durren, South Africa Water Research Commission (1997)).

Porosity strongly impacts head loss in a filter bed as discussed in Chapter 6 : . The bed porosity is typically 0.38 for spherical media grains, but can range from 0.4 – 0.6 (Crittenden *et al.*, 2012). The average porosity is 0.4, which is consistent with Binnie, Kimber and Smethurst (2002) indicating this as the typical value in water treatment and Logsdon *et al.* (2002) indicating this as approximate for typical rounded sands. Beverly (2011) indicates typical porosity values of 0.4 for sand and 0.5 for anthracite. Kawamura (2000) records bed porosities of 0.43 to 0.45 for sand and 0.48 to 0.50 for anthracite, whereas Logsdon (2008) provides a range of 0.42 to 0.47 for sand and 0.56 to 0.60 for anthracite.

Despite the effects of grain shape on filter design and performance, Crittenden *et al.* (2012) explains that shape factor and sphericity have limited value in practice and cannot easily be accounted for in filter design. This is mainly due to the monotonous task of determining the equivalent-volume sphere surface area and industry practice of specifying media size based on sieve analysis for commercially available filter media (Crittenden *et al.*, 2012).

The Kozeny or Ergun equation (discussed in Chapter 6 :) can be used to calculate sphericity using a measured head loss (Edzwald, 2011). However, Crittenden *et al.* (2012) points out that the sphericity values published in literature have mostly been calculated indirectly through various head loss experiments with head loss coefficients assumed to be independent of grain shape. For this reason, published sphericity values are “really just empirical fitting parameters for head loss”.

4.2.6 Surface Area

The **specific** surface area of a filter bed, S, is defined as the total surface area of the filter media divided by the volume of the filter bed (Crittenden *et al.*, 2012) and can be written as:

$$S_v = \frac{S(1-\varepsilon_0)}{d_{eq}} \quad (Eq. 4.7)\ (\text{Crittenden } et\ al., 2012;\ \text{Montgomery, James M. Consulting Engineers, } 1985)$$

Note that shape factor (S) is defined in Chapter 4.2.5.

But this equation is useful only if d_{eq} is known (Crittenden *et al.*, 2012).

4.2.7 Density/Specific Gravity

Media density affects the backwash flow requirements, with lower densities being easier to fluidize (Edzwald, 2011). Typical densities for media are listed in Table 8. Sand produced in South Africa has a typical specific gravity of 2.63 (Van Duuren, F. A., South Africa Water Research Commission., 1997).

Table 8 Common Density Ranges for Different Media (AWWA and American Society of Civil Engineers, 2012; Edzwald, 2011)

Media	Specific gravity
Sand	2.55 – 2.65
Anthracite	1.45 – 1.75
Garnet	3.6 – 4.3
Ilmenite	4.2 – 4.6
GAC	1.3 – 1.5 (increasing with adsorption)

4.3 Media Bed Types for Filter Classification

4.3.1 Introduction

Crittenden *et al.* (2012) classifies rapid filtration according to media bed type configurations as follows, as depicted in Figure 8:

- Monomedia filter – single layer of filter medium, typically sand. This is an old design that is not commonly used.
- Deep-bed monomedia filter – single layer of filter medium, typically anthracite or GAC.
- Dual-media filter – Two layers of filter media, which is more robust than monomedia filters. Binnie, Kimber and Smethurst (2002) explains that the dual-media filter is commonly used to ensure that the top layer remains the coarser layer after backwashing, which is made possible by using media of differing densities.
- Mixed-media filter – Three layers of filter media.

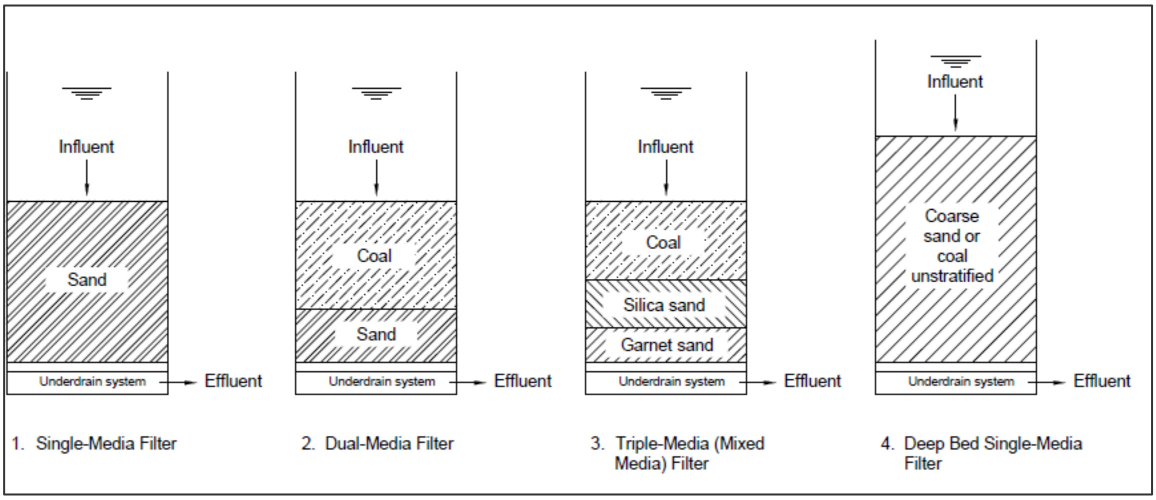


Figure 8 Typical Filter Media Bed Type Configurations (adapted from Edzwald (2011))

4.3.2 Selection

4.3.2.1 General

If pilot testing is not possible, media selection is typically based on previous experiences with the treatment of similar waters (Montgomery, James M. Consulting Engineers, 1985), but will typically be dual media in this instance (Monk (1987) as cited in Hendricks (2010)). As noted in Chapter 4.3.1, the original monomedia filter bed type configuration is generally not used. Kawamura (1999) indicates that high-rate filters are generally dual-media or coarse deep-bed monomedia. Crittenden *et al.* (2012) suggests that "dual-media filters are more robust than monomedia filters" when pre-treatment/coagulation is periodically inadequate.

Nevertheless, the selection is generally the designer's decision, either arbitrarily or based on tradition or a standard approach (Hendricks, 2010). It is further noted that the designer must be aware of the various media parameters that significantly impact on the filter design, such as smaller media sizes increasing particle capture efficiency, but rapidly increasing head losses, lower densities that are easier to fluidize and expand (i.e. a denser media of equal diameter requires higher backwash flow rates (Crittenden *et al.*, 2012; Logsdon *et al.*, 2002) and porosity effect on head loss.

Beverly (2011) describes the perfect bed as one with smooth transition of grain size from coarse on top to fine on bottom.

Coarse deep-bed monomedia filters are popular due to increased filter run time and their use at high filtration rates (Hendricks, 2010). Direct and in-line filtration require beds that are capable of holding a lot of flocs, which is possible with reverse-graded dual-media or a deep bed of coarse uniform media (AWWA and American Society of Civil Engineers, 2012). Multimedia filters are effective for rapid filters, but with the disadvantage that the media must be properly selected, and this will also require a high backwash rate for restratification (Kawamura, 2000). Binnie, Kimber and Smethurst (2002) comments that it is generally accepted that the additional complexity and expense for multimedia filters outweighs any benefits. See also Chapter 5.1 for consideration of filtration rates.

Edzwald (2011) explains the evolution of media bed type for rapid filters as follows:

a) 0.5 – 0.9 m deep fine sand monomedia filters (ES of 0.5 to 0.9 mm) with majority of particles being captured only in the top section of the filter bed

b) 0.5 – 0.9 m deep dual media filters to promote deeper penetration of particle capture in the filter bed and reduce the rate of head loss development thereby increasing the filter run time

c) 0.5 – 0.9 m deep mixed media filters to hopefully further improve filtrate quality, however, this is not guaranteed (and furthermore the introduction of a third, finer layer to the filter bed could reduce the filter run time due to increased head loss development if implemented in an existing filter configuration)

d) 1.2 – 1.8 m deep deep-bed monomedia filters due to various studies demonstrating a further increase in filter run time compared to the dual- and mixed-media filters

Crittenden *et al.* (2012) indicates a typical bed depth range of 0.6 – 1.8 m for rapid filtration. Van Durren, South Africa Water Research Commission (1997) states that bed depths of 0.6 – 1.0 m are practical in South Africa.

Typical bed details from literature are provided in Chapter 4.3.2.3 for further reference.

4.3.2.2 Backwashing Considerations – Fluidization, layers and intermixing

Hendricks (2010) defines fluidization as the upward flow of water through the filter bed that causes the media to be suspended in the fluid during the backwash process. See Chapter 8 : for backwashing aspects.

Fluidization of d_{90} grains in monomedia beds would result in fine-to-coarse media gradation from top to bottom layers; i.e. the opposite of the perfect bed (Beverly, 2011), rapidly increasing head loss during filtration (AWWA and American Society of Civil Engineers, 2012). Coarse-to-fine media allows for slower head loss development and increased storage capacity in the bed with the finer material providing more contact zones for particle removal and ensuring good filtrate quality (AWWA and American Society of Civil Engineers, 2012). Thus, the dual- and mixed- media filter beds better approximate the perfect bed (Beverly, 2011).

Furthermore, if deep-bed coarse monomedia filters were to be fluidized, impractically high backwash rates would be required (Edzwald, 2011). Therefore, air is used in combination with a low-rate backwash (non-fluidizing) to clean the media. The media does not classify and hence the grains are very nearly equal within the bed. Few fines are present after manufacture for sand, whereas anthracite will have fines. However, not many of the fines will normally surface because of the non-fluidizing backwash (Beverly, 2011). The other media bed types are typically backwashed with full fluidization (Edzwald, 2011).

Layers from top to bottom are therefore provided with decreasing size and increasing densities. Note that the media for various layers should also be chosen for compliance with the AWWA B100 standard 2:1 size gradation recommendation (Beverly, 2011). Furthermore, Logsdon *et al.* (2002) explains that the media types used in dual- and mixed-media filter beds must have similar fluidization velocities.

Kawamura (2000) suggests the following relationship between two media sizes in a filter bed to ensure the same settling velocity:

$$\frac{d_1}{d_2} = \left(\frac{\rho_2 - \rho_w}{\rho_1 - \rho_w}\right)^{0.667} \quad (Eq. 4.8)$$

Beverly (2011) indicates the importance of specific gravity in dual or mixed-media filters to both promote intermixing at layer interfaces and limit the intermixing. AWWA and American Society of Civil Engineers (2012) reports that there are conflicting views on whether separate layers or intermixed layers are preferred. Completely mixed layers would essentially be a single media, whereas no mixing would result in the fine-to-coarse media configuration with the associated probable rapid head loss development. Intermixing of differing media could result in smaller grains being trapped in the pores between larger grains, thereby reducing the porosity and increasing clean-bed head loss (Logsdon, 2008).

Intermixing at the interfaces is provided for the smooth transition of grain size to reduce sudden head loss increases between layers. An intermixing zone of 100 – 150 mm (Beverly, 2011) is common, but garnet can be mixed with the sand above by as much as 50%. Pilot studies would typically be used to choose the intermixing zone (Beverly, 2011).

Intermixing of silica sand and garnet sand in mixed media beds happens more easily than intermixing of the anthracite and silica sand (AWWA and American Society of Civil Engineers, 2012).

Cleasby and Woods (1975) as cited in AWWA and American Society of Civil Engineers (2012) propose a coarse-to-fine ratio of 1.5 or less for silica sand to garnet to promote intermixing but also to ensure that some of the finer material remains at the bottom of the filter bed. Brosman and Malina (1972) as cited by Cleasby and Woods (1975) as cited in AWWA and American Society of Civil Engineers (2012) found that an anthracite to silica sand ratio of less than 3 reduces intermixing and a ratio greater than 3 linearly increases the intermixing zone.

4.3.2.3 Typical Bed Details

A summary of typical media bed type characteristics and associated media types from literature is given in Table 9 for reference.

Table 9 Common Media Characteristics for Different Media Bed Types

Filter Bed	Monomedia	Deep-Bed Monomedia	Dual Media		Mixed Media			References
Layers	1	1	1 (Top)	2 (Bottom)	1 (Top)	2	3 (Bottom)	
Typical Media	Sand	Anthracite (or coarse sand)	Anthracite	Sand	Anthracite	Sand	Garnet/Ilmenite	
Bed/Layer Depth (m) **Associated ES (mm)**	0.6 – 0.76	1.5 – 1.8	Older: 0.45 – 0.6 Newer: 1.5 – 1.8	Older: 0.23 – 0.3 Newer: 0.23 – 0.3	0.45 – 0.6	0.23 – 0.3	0.1 – 0.15	(Crittenden *et al.*, 2012)
	0.6 – 1.0 (ES 0.5 - 1.0)							(Gray, 2010)
	0.6 – 2.0 (ES 0.5 -1 .0)		0.2 (ES 1.5)	0.6				(World Health Organisation, 2017)
	0.6 – 0.9 (ES 0.45 - 0.55)	1.2 – 18 (ES 0.5 - 6.0)	0.46 – 0.76 (ES 0.8 - 1.2)	0.15 – 0.36 (ES 0.45 - 0.55)	0.46 – 0.61 (ES 0.8 - 1.2)	0.15 – 0.23 (ES 0.35 - 0.5)	0.075 – 0.1 (ES 0.15 - 0.35)	(AWWA and American Society of Civil Engineers, 2012)
	0.6 – 0.7 [1] 0.6 – 0.9 [2]	0.9 – 1.2 [1] 1.5 – 3.0 [2] 1.0 – 2.0 [3]	0.3 – 0.6	0.15 – 0.3	0.3 – 0.6	0.15 – 0.3	0.1	(Edzwald, 2011)
		1.2 – 1.8 (ES 1.3 - 1.5)	0.45 – 0.6 (ES 0.9 - 1.5)	0.25 – 0.4 (ES 0.5 - 0.8)	0.4 – 0.45 (ES ~ 10)	0.2 – 0.25 (ES ~5)	0.075 – 0.115 (ES 0.25)	(Beverly, 2011)
		0.8 – 2.0 (ES 0.8 - 2.0)			0.45 (ES 0.9 – 1.4)	0.3 (0.45 - 0.65)	0.0075 (ES 0.25 - 0.3)	(Kawamura, 2000)
			0.61 (ES 0.9)	0.254 (ES 0.45)				(Hendricks, 2010)

Notes:

1. Common in the United States for rapid gravity, conventional filtration filters
2. Common in the United States for rapid gravity, biological filters for removal of iron and manganese
3. Common in the United States for rapid gravity, direct filtration filters

4.4 Use of GAC in Rapid Filtration

4.4.1 Introduction

Granular activated carbon, being an adsorbent material, is particularly effective at removal of algal toxins, synthetic organic compounds, endocrine-disrupting compounds, pharmaceutical and personal care products, disinfection by-products and their precursors and taste- and odour-causing compounds (AWWA and American Society of Civil Engineers, 2012). As this study relates to rapid gravity media filtration, gravity GAC media filters are also discussed hereunder. However, pressure and upflow GAC media filtration and process selection is beyond the scope of this dissertation.

With reference to filtration in water treatment, GAC media filters can be used as combined filters (i.e. a layer in the filter bed to encourage adsorption) or for pre-filtration adsorption (i.e. upstream of the filters), or for post-filtration adsorption (i.e. downstream of the filters). Pre-filtration is the least common option with limited applications and benefits. Post-filtration can possibly meet the secondary filtration requirements under USEPA's Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) for 0.5 log Cryptosporidium removal credit. (AWWA and American Society of Civil Engineers, 2012).

Granular activated carbon can be as effective as sand (monomedia filters) or anthracite (dual-media filters) for filtration purpose, but is not as effective in adsorption as a postfilter GAC contactor due to the reduced contact time when backwashing the filter (Edzwald, 2011). A disadvantage of dual-media filters utilising GAC is the added effort to ensure that no sand is present in the GAC for regeneration, which would pose challenges for the regeneration furnace (Edzwald, 2011).

4.4.2 Further Media Property Considerations

Granular activated carbon size ranges are typically defined in terms of the minimum and maximum mesh sizes as mesh "A x B", where A refers to the mesh through which granules of carbon will fall through and B refers to the mesh that will capture the carbon granules (Crittenden *et al.*, 2012). Generally, the larger the mesh number, the smaller the granules. (Crittenden *et al.*, 2012) indicates typical particle diameters as 8 x 30 mesh and 12 x 40 mesh.

The typical filter bed arrangement for retrofitted filters is given in Table 10.

Table 10 Typical Filter Bed Properties for GAC Retrofitted Filters (Edzwald, 2011)

Media Bed Type	Media	Bed Depth (m)	GAC Mesh Size	ES (mm)	UC	Note
Dual-media	GAC (top layer)	0.38 – 0.76	12 x 40	0.55 – 0.65	High but <2.4	Shorter filter run time but better adsorption function
			8 x 30	0.8 – 1.0		
	Sand (bottom layer)	0.15 – 0.3	-	0.35 – 0.6		
Deep bed monomedia	GAC	0.9 – 1.8	8 x 20 or 8 x 16	≥ 1.3	Lower ~ 1.4	Longer filter run time and increased time to replace/regenerate GAC

Granular activated carbon has similar media loss characteristics as anthracite; however, the lower density of GAC impacts on losses during backwashing, which could be a concern (Edzwald, 2011).

4.4.3 Empty Bed Contact Time and Breakthrough

The amount of time the influent is in contact with the GAC is important for adsorption. This is generally described by the Empty Bed Contact Time (EBCT), defined as the total volume of GAC (including porosity according to Crittenden *et al.* (2012)) divided by the influent flow rate. Pilot studies are recommended for determining the optimal EBCT and bed depth.

The L/ES ratio discussed above is also applicable as a design indicator for GAC filter beds (Kawamura, 1999). Longer EBCTs reduces the rate of carbon usage and delays the time to breakthrough, while shorter EBCTs reduces head loss but increases the time to breakthrough (AWWA and American Society of Civil Engineers, 2012). It is noted that rapid filtration reduces the contact time, thereby limiting the effectiveness of GAC in filters (Binnie, Kimber and Smethurst, 2002).

Carbon usage and EBCT are the most important aspects for costing analysis. A deeper bed; i.e. deeper than the optimal bed depth, reduces reactivation frequency (operating costs) but increases the adsorber volume (capital costs) and should be compared with more frequent reactivation and shallower bed depth to establish an optimum design (AWWA and American Society of Civil Engineers, 2012).

The GAC must be replaced or reactivated/regenerated once it has exhausted its adsorptive capacity; i.e. it cannot adsorb any further contaminant molecules. This is detected when the concentration of the specific contaminant in the filtrate exceeds the treatment requirement; i.e. breakthrough occurs. Figure 9 depicts the typical breakthrough curve for a GAC filter (AWWA and American Society of Civil Engineers, 2012).

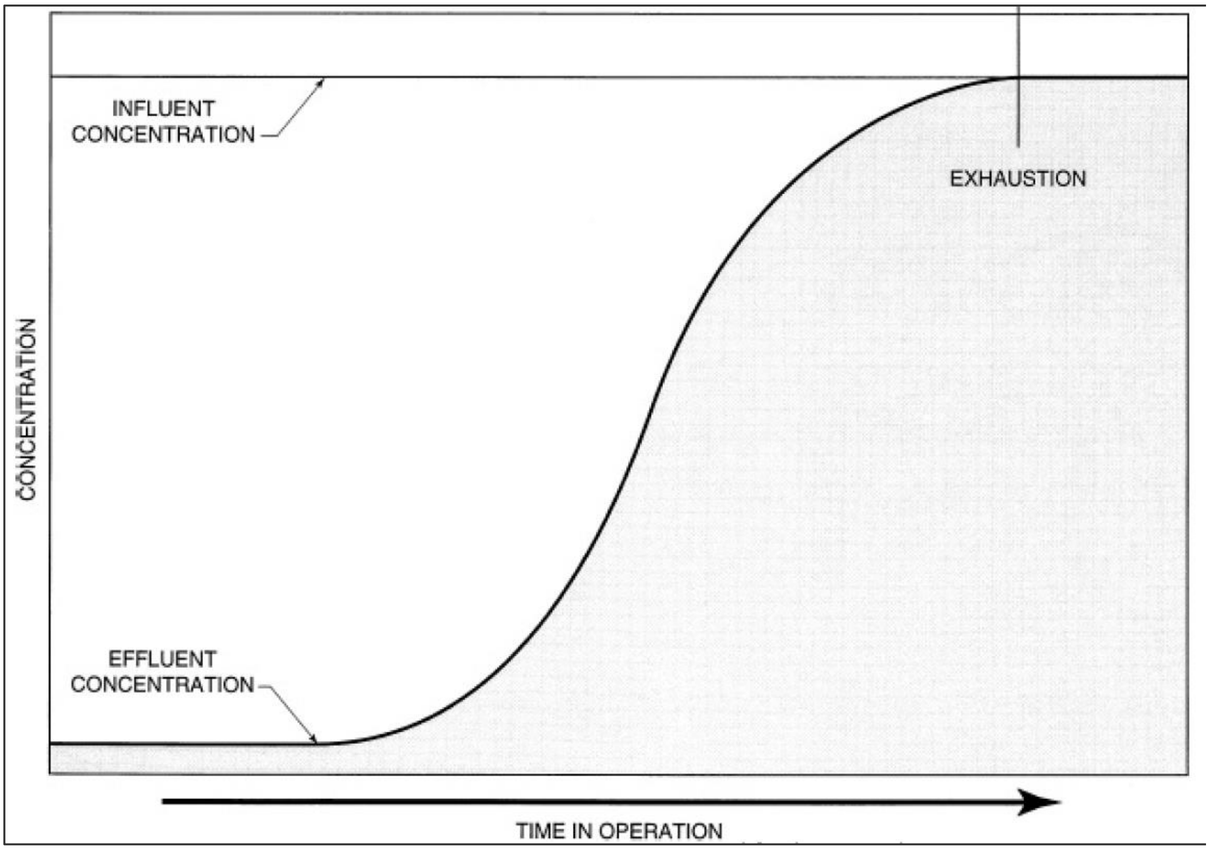


Figure 9 Typical Breakthrough Curve for GAC Filter (AWWA and American Society of Civil Engineers, 2012)

The typical EBCT is 10 – 15 minutes (Kawamura, 2000). Once EBCT and breakthrough curves have been established (through pilot studies), the GAC volume and bed depth can be ascertained. A steep breakthrough curve indicates that a shallower bed can be installed. Bed depth can range from 0.75 to 5 m. For combined filters, the bed depth and flow rates are limited to that for filtration purposes; i.e. for turbidity removal. (AWWA and American Society of Civil Engineers, 2012).

4.5 Bed Depth and Media Size Relationship

The filter bed depth to effective size (L/ES) ratio conceptualises the number of filter media grains that a particle should pass along the way through the filter bed to the filtrate; where higher values indicate more grains to pass through (Logsdon, 2008). Figure 10 shows a constant relationship between media depth and effective size. The graph illustrates that a coarser media requires a deeper bed whilst a finer media requires a shallower bed.

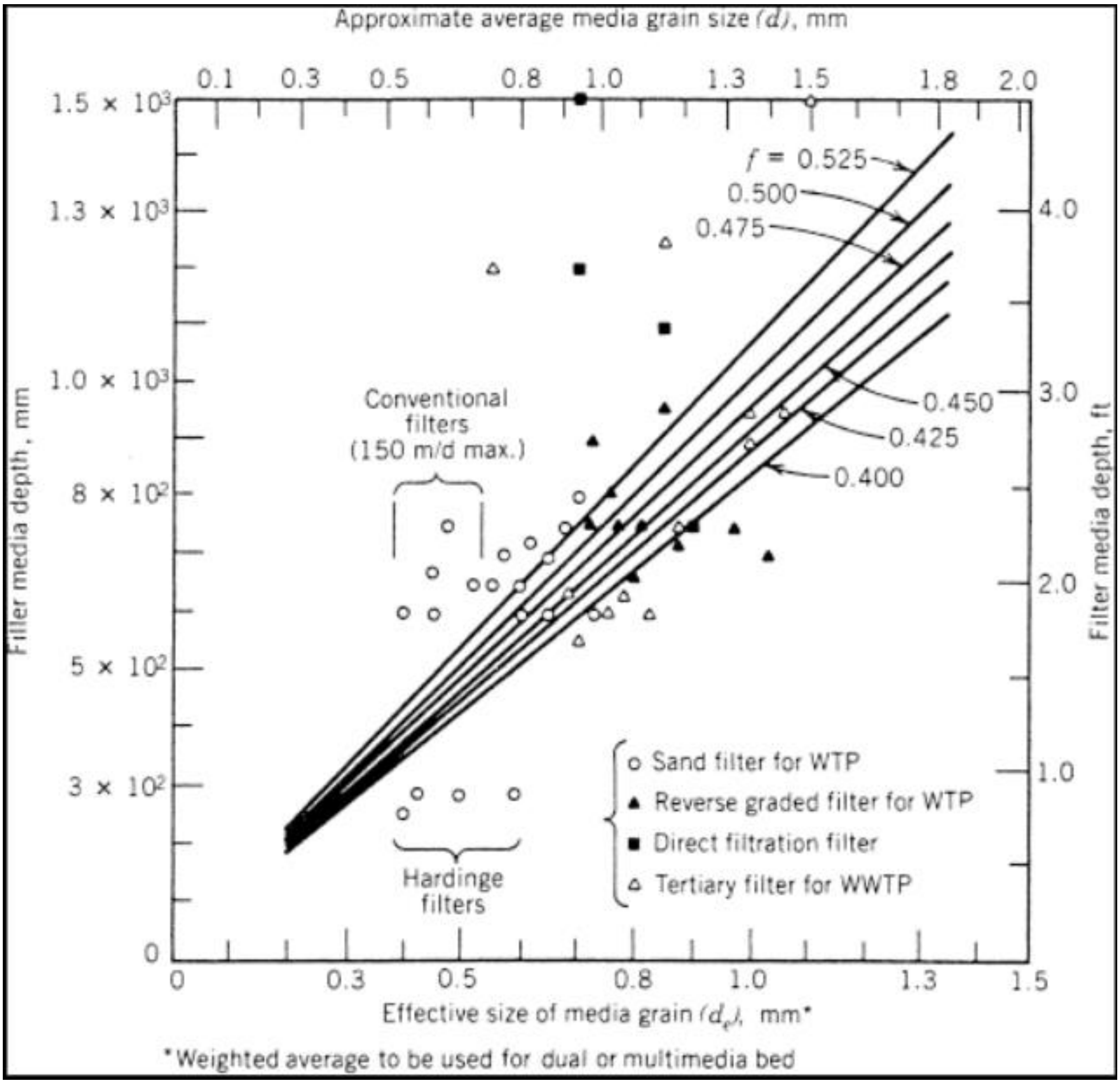


Figure 10 Relationship between Filter Bed Depth and Media Effective Size (Montgomery, James M. Consulting Engineers, 1985)

Kawamura (2000) recommends the following design guideline for selection of bed depth and media effective size:

- L/ES ≥ 1000 in rapid monomedia sand filters and dual media filters
- L/ES ≥ 1250 in mixed media filters
- L/ES between 1250 and 1500 in most coarse deep beds : monomedia filters with ES of 1.2 to 1.4 mm and bed depth of 1.8 to 2.0 m
- L/ES between 1500 and 2000 in most coarse deep beds : monomedia filters with ES of 1.5 to 2.0 mm (but only used as an estimate L/ES when ES > 1.5 mm)

Edzwald (2011) notes that L/ES is accumulative for the different media types used in a filter bed.

Kawamura (1999) notes that L/ES must increase by at least 20% if a filter aid is not used, but this results in higher construction cost, higher initial head losses and longer backwashing time without a significant increase in filter performance.

4.6 Installation Considerations

This dissertation does not discuss the methods of installation. However, it is noteworthy to mention the installation aspects that could affect filter performance.

Filter fines need to be removed from each filter media layer/type during installation. If not removed, the backwash procedure would result in the fines being forced to the media surface, which would restrict the flow through the filter during filtration and capture the suspended solids mostly at the surface and, thereby, cause shorter filter runs (Beverly, 2011).

Fines removal is achieved by firstly backwashing the media to bring the fines to the surface (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011; Montgomery, James M. Consulting Engineers, 1985)– see Figure 11. Montgomery, James M. Consulting Engineers (1985) proposes that this backwash process for rapid gravity filters be at the maximum design backwash rate for approximately 15 minutes, after which the filter water level is drained to some distance below the top of the media.

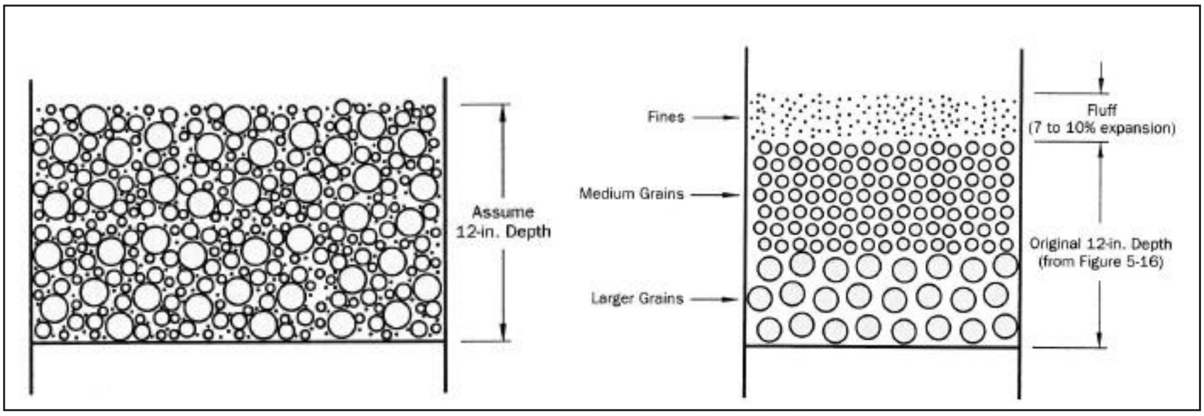


Figure 11 Fines per Media Layer at Installation (left) and After Backwashing (right) (Beverly, 2011)

Beverly (2011) advises full fluidization for an efficient skimming process.

The surfaced fines are then "skimmed" by manually removing them with a flat shovel to a depth determined by visual inspection, up to approximately 250 mm (Montgomery, James M. Consulting Engineers, 1985). Beverly (2011) suggests 5% of the media layer/type be removed.

This fines removal process is repeated until the amount of fines seen after the backwash process seems insignificant.

For anthracite, future skimming may be required as it tends to break down over time. Beverly (2011) mentions the unpredictability of how often this is required. Future skimming may also be required for media that could not be subjected to the full fluidization backwash, which could result in trapped fines surfacing from time to time (Beverly, 2011). A fines check is recommended every 3 to 6 months until a routine inspection period is developed for the specific filter.

When adding media to an existing filter, it is recommended to skim the old media before installing the new media, which would thereafter also need to be skimmed (Beverly, 2011).

For new GAC filters, the filter bay needs to be disinfected before GAC installation, followed by a day (minimum) of submerging and soaking the media bed. Thereafter, a low rate backwash is done to remove air from the bed and then fines are washed out at a slightly higher backwash rate. (Edzwald, 2011).

4.7 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates two sheets related to filter media; namely "Filter Media – Design" and "Filter Media – Example Analysis" as per the screenshots below.

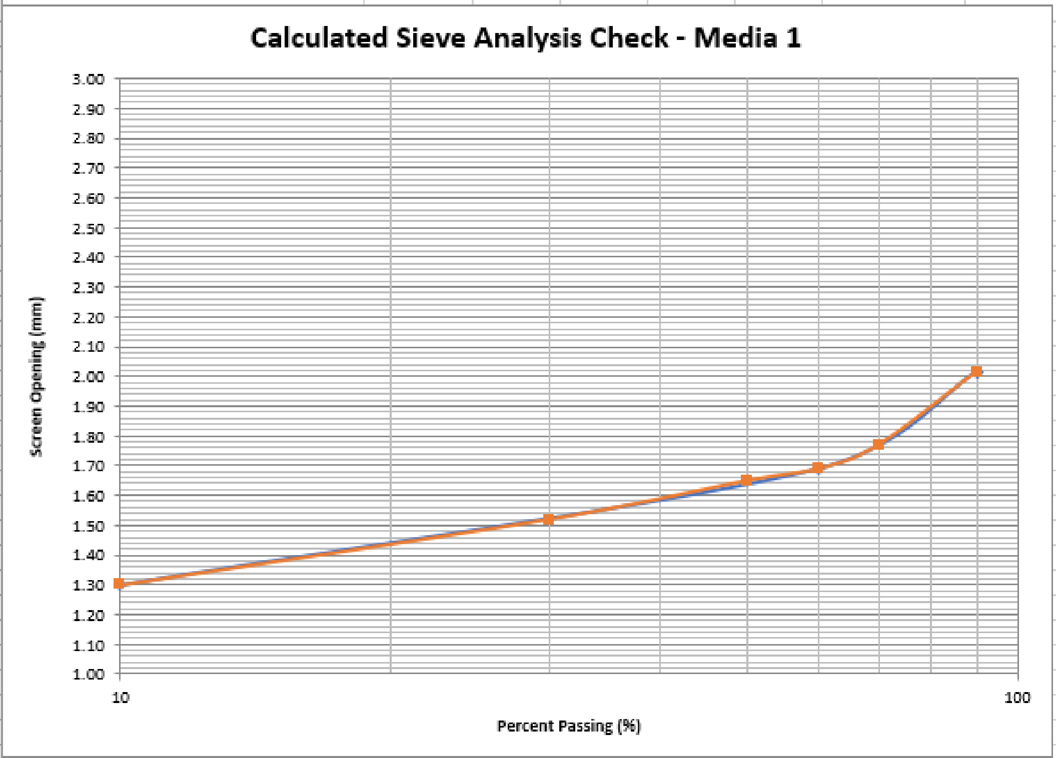
	A	B	C
1	**Filter Configuration**		Section Reference
2	Media Bed Type	Monomedia	4.3
6	Number of Media	1	
7			

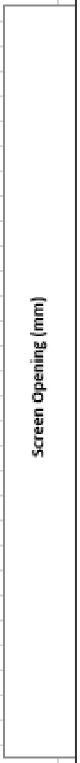
	A	B	C	D	E	F	G
8	**Filter Media Parameters**						Section Reference
9	Parameter	Symbol	Media 1	Media 2	Media 3	Unit	
10	Medium	-	Anthracite	Sand	Garnet	-	
11	Included in Design for Filter Configuration	-	no	yes	no	-	
12	Effective Size	ES or d10	1.3	1.5	0.25	mm	4.2.2 ; 4.3
13			0.0013	0.0015	0.00025	m	
14	Uniformity Coefficient	UC	1.3	1.3	1.3	-	4.2.2
15	Loose/initial bed porosity	ε_0	0.58	0.4	0.58	-	4.2.5
16	Sphericity	ψ	0.53	0.75	0.6	-	4.2.5
17	Medium media grain size	d_{60}	1.7	2.0	0.3	mm	4.2.2
18			0.0017	0.0020	0.0003	m	
19	Large media grain size	d_{90}	2.0	2.3	0.4	mm	4.2.2
20			0.0020	0.0023	0.0004	m	
21	Equivalent Spherical Diameter	deq	1.31	1.69	0.26	mm	4.2.5
22			0.0013	0.0017	0.0003	m	
23	Media Density	ρ_m	1600	2650	3950	kg/m^3	4.2.7
24	Selected bed depth per medium	L medium	450	1500	75	mm	4.3
25	Actual bed depth per medium		0	1500	0	mm	
26			0	1.5	0	m	
27	Bed depth to size ratio per medium	L/ES medium	0	1000	0	-	4.5
28	Total Bed Depth	L		1500		mm	
29				1.5		m	
30	Accumulative Bed Depth to Size Ratio	L/ES		1000		-	4.5
31							

$$d_{60} = UC \times d_{10}$$

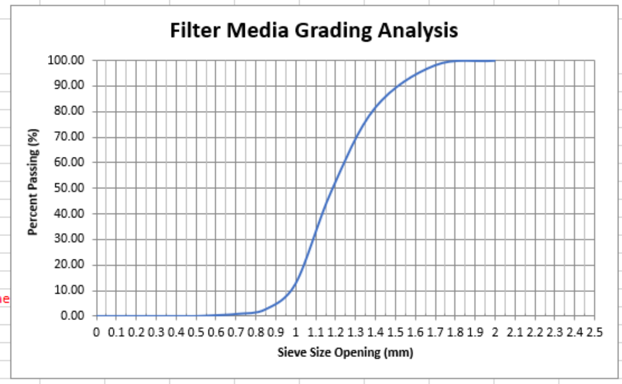
$$d_{90} = d_{10} \times 10^{1.67 \times log UC}$$

Five Media Sizes (Coarse-to-Fine Layers in Bed) for Head Loss and Expansion Calculations								Section Reference
Media 1								
Layer No.	Layer Depth	deq for layer	Screen opening	Percent passing by weight	Sphericity (ψ)	Porosity (ε_0)	Density (ρ	6.3; 8.2
	mm	-	mm	%	-	-	kg/m^3	
1	0	d_{10}	1.30	10	0.53	0.58	1600	
2	0	d_{30}	1.52	30	0.53	0.58	1600	
3	0	d_{50}	1.65	50	0.53	0.58	1600	
[redacted]		d_{60}	1.69	60	0.53	0.58	1600	
4	0	d_{70}	1.77	70	0.53	0.58	1600	
5	0	d_{90}	2.01	90	0.53	0.58	1600	





Filter Media Grading Analysis				Section Reference
Sieve Size Opening (mm)	Mass Retained	% Retained at size	% Passed at size	4.2.2
2	0	0.00	100.00	
1.7	2.7	1.85	98.15	
1.4	24.2	16.61	81.54	
1.18	47.6	32.67	48.87	
1.00	52.8	36.24	12.63	
0.85	14.5	9.95	2.68	
0.71	2.6	1.78	0.89	
0.60	0.8	0.55	0.34	
0.50	0.5	0.34	0.00	
0.43	0	0.00	0.00	
0.300	0	0.00	0.00	
0.00	0	0.00	0.00	
Total	145.7	100.00		should be close to 100% total retaine



Filter Media Parameters Evaluation					Section Reference	
Parameter	Symbol	Design	Actual from graph	Unit		
Effective Size	ES or d_{10}	1.20	0.97	mm	4.2.2	might not be acceptable - look into this
Bed Depth	L	1450	1450	mm	4.3	
Medium Grain Size	d_{60}	1.44	1.25	mm	4.2.2	
Uniformity Coefficient	UC	1.2	1.3	-	4.2.2	
Bed depth to size ratio	L/ES	1208	1495	-	4.5	generally acceptable, but look at acceptable ranges for specific filter configuration

Further guidelines to the designer on how to use these sections of the tool being provided in the Appendix sections A4 and A5, respectively.

CHAPTER 5 : FILTRATION RATES

5.1 Filtration Rate Selection

The filtration rate is the superficial water velocity through the filter, defined by the influent flow rate divided by the cross-sectional area of the filter bed, commonly expressed in units of metres per hour [m/h]. It is conventional to use superficial velocity and not the interstitial velocity (that would take porosity into account). (Hendricks, 2010).

Rapid sand filters replaced slow sand filters with associated hydraulic rates of 0.1 – 0.2 m/h increasing to 2.5 – 5 m/h (AWWA and American Society of Civil Engineers, 2012). Hendricks (2010) summarises filtration rates as traditionally being ~ 5m/h from 1900 to 1950 and a norm of ~ 12 m/h by 1970, with at least one WTP in America operating filters at 32.3 m/h in 1987.

Numerous investigations have proven increased filtration rates of 7 – 20 m/h in the past few decades (AWWA and American Society of Civil Engineers, 2012).

Rapid and high-rate terminology is commonly used interchangeably in literature and generally refers to the same thing. Few references distinguish between rapid filters with rates of 5 to 7.5 m/h and high-rate filters with rates of 10 to 25 m/h (AWWA and American Society of Civil Engineers, 2012; Kawamura, 2000). Gray (2010) mentions that 5 – 10 m/h is common for rapid filters. AWWA and American Society of Civil Engineers (2012) explains that the “rapid filters” require large filter areas and experience short run times, whereas the “high-rate filters” are more economical. For the purposes of this dissertation, rapid filter design includes for high-rate filters due to the growing interest of utilising these higher rates.

Kawamura (2000) states the average filtration rate in the United States as 13 – 15 m/h. Although experienced large cities may be operating filters at rates up to 20 – 33 m/h, it has been shown that particle increase in the filtrate occurs at rates above 20 – 25 m/h, and therefore the filtration rate is typically limited to a maximum of 15 m/h.

Binnie, Kimber and Smethurst (2002) mentions rates of up to 12 m/h with dual media and up to 15 m/h using coarser media. Hofkes *et al.* (1981) indicated rapid filtration of 5 – 15 m/h with coarse sand. Binnie, Kimber and Smethurst (2002) also associates coarser media with rates of 15 m/h for manganese removal; i.e. biological filtration. Crittenden *et al.* (2012) indicates typical ranges for rapid filtration rates as 5 – 15 m/h with an ES of 0.5 – 1.2 mm.

Kawamura (1999) discusses the restricted filtration rates in the United States (with one filter out of operation) as follows:

- Regular rapid sand filters at 7.5 m/h
- Dual-media filters at 10 – 12.5 m/h
- Deep bed monomedia filters at 15 m/h

Kawamura (1999) also notes that pilot studies are required for rates greater than those stated above; where dual-media filters have operated at 20 m/h and deep bed monomedia filters at 32.5 m/h (1800 mm bed with ES of 1.4 mm).

AWWA and American Society of Civil Engineers (2012) suggests reverse-graded coarse-to-fine media or a deeper coarser monomedia bed (1.0 to 1.4 mm ES) generally operates at 10 to 25 m/h. Kawamura (2000) indicates that it is typical and effective to have deep-bed monomedia filters with filtration rates of 12.5 to 30 m/h, whereas Beverly (2011) suggests that 20 to 30 m/h is advantageous for deep-bed monomedia filters.

Logsdon *et al.* (1993) as cited in AWWA and American Society of Civil Engineers (2012) describes effective filtration of high-turbidity source waters up to 60 ntu with pre-treatment and filter aid at 22 m/h through a deep bed monomedia filter.

Reliable operation at 24 – 37 m/h has been confirmed for deep-bed anthracite filters with a filter aid (AWWA and American Society of Civil Engineers, 2012).

For combined filters (with GAC), the flow rates are limited to the filtration rates for turbidity removal (AWWA and American Society of Civil Engineers, 2012), with typical filtration rates of 5 – 24 m/h (AWWA and American Society of Civil Engineers, 2012).

Filtration rates greater than 25 m/h are unfeasible for normal dual-media filters with the head loss developing at an exponential rate and reducing the filter run time. Nevertheless, coarse deep bed filters can generally accommodate this high filtration rate with a filter aid. (Kawamura, 2000).

Logsdon *et al.* (2002) explains that filtration may be less effective in cold water conditions due to higher viscosity that causes higher shear stresses within the filter bed making it more difficult for attachment of particles to the media grains but also making it easier for particles to become detached. Temperatures below 8°C can reduce filter run time and poor filtrate quality in high-rate filters due to the affected chemical pre-treatment. In cold conditions, conventional filters operating at high rates should be limited to a safe filtration rate of 15 m/h (Kawamura, 1999).

According to Van Duuren, South Africa Water Research Commission (1997), conservative average filtration rates of below 10 m/h (often 5 m/h) are commonly used in South Africa so as not to rely on optimal pre-treatment.

5.2 Changes in Filtration Rates

Filtration rates can change during operation, such as when one filter is backwashed or taken offline for maintenance or when water treatment demand increases, or even when starting a filter run after backwashing (Logsdon *et al.*, 2002).

Sudden changes in filtration rate (rather than gradual) negatively impacts on filtrate quality (AWWA and American Society of Civil Engineers, 2012; Binnie, Kimber and Smethurst, 2002; Cleasby, Williamson and Baumann, 1963; Edzwald, 2011). Cleasby, Williamson and Baumann (1963) further explains that a larger negative impact on quality is experienced when the percentage change is larger; however, the impact is somewhat independent on the duration of a disturbance. An increased filtration rate can dislodge captured particles (especially if there is weak floc strength or cold water), which makes it riskier for a relatively clogged filter to experience the increased rate (Logsdon *et al.*, 2002). Hence, if an increased rate is required, it would be better for this to take place at the start of a filter run and therefore backwashing of the filters should be planned upfront to be ready for higher filtration rates when water treatment demand increases are anticipated.

The increase in filtration rate should preferably be limited to 15-20% to the remaining filters, but not more than 33% (Kawamura, 2000).

Edzwald (2011), as well as Cleasby *et al.* (1992) as cited in Logsdon *et al.* (2002), refer to 10 minutes gradual change in filtration rate. Logsdon *et al.* (2002) further describes various recommendations for the gradual change ranging from 5% per minute to 1% per minute depending on floc strength. However, this does not guarantee that filtrate quality will not be affected. An increase in the time for the change as well as polymer addition to strengthen floc may reduce the impact on the filtrate quality (Logsdon *et al.*, 2002). Crittenden *et al.* (2012) indicates that the United Kingdom requires a design with maximum 1.5% per minute for soft waters and 5% per minute for hard waters when removing Cryptosporidium.

Furthermore, there is evidence that the media bed type also plays a role in the effects of increased filtration rate on filtrate quality (Logsdon *et al.*, 2002). Dual-media filters may shed particles in the top layer which can be recaptured in the bottom layer, whereas deep-bed monomedia filters would shed these particles to the filtrate.

Beverly (2011) explains the options of reducing the incoming flow or having an entire spare filter available to come online when backwashing occurs to reduce the impacts of filtration rate changes.

5.3 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates the sheet "Filter Number, Size and Rate", as per the screenshots below.

	A	B	C	D	E	F	G	H
1	Filter Number, Size & Filtration Rate Summary - Design Selection							
2	Parameters	Symbol	Value	Unit				
3	Operating Hours per Day	-	24	h				
4	Water Treatment Plant Design Flow Rate	Q	380	Mℓ/d				
5	Number of Filters	N	12	-				
6	Number of Offline Filters for Design	-	1	-				
7	Filter Width (filtration)	w	7.00	m				
8	Filter Length (filtration)	l	12.00	m				
9	Filter Area (filtration)	A	84.00	m^2				
10		v	15.7	m/h				
11	Filtration Rate (i.e. all filters online)	Q_f	0.367	m^3/s				
12		v_{max}	17.1	m/h				
13	Maximum Filtration Rate (i.e. x filters offline)	$Q_{f,max}$	0.400	m^3/s				
14	Filtration Rate Increase	Δv	9%	%				
15	Backwash channel position	-	Side	-				
16	Backwash channel wall width	BCW_{width}	0.250	m				
17	Filter Width (construction)	$w_{construct}$	8.050	m				
18	Filter Length (construction)	$l_{construct}$	12.000	m				
19	Filter Areaa (construction)	$A_{construct}$	96.600	m^2				
20	Filter Depth	H_{filter}	5.800	m	only defined after head losses are defined			
21								

	A	B	C	D	E	F–K
23	Filter Number, Size & Filtration Rate - Design Calculation				Section Reference	
24	Parameters	Symbol	Value	Unit		
25	Water Treatment Plant Design Flow Rate	Q	380	Mℓ/d		
26	Estimated Number of Filters	$N_{estimate}$	12.1	-	12.2	$N \approx 0.62\,(Q^{0.5})$
27	Number of Filters	N	12	-	12.2	
28	Operating Hours per Day	-	24	h		
29	Number of Offline Filters for Design	-	1	-	12.2	1 is common
30	Flow Rate per Filter (all online)	q	1319.4	m^3/h		
31	Design Flow Rate per Filter (x offline)	q_{design}	1439.4	m^3/h		
32	Desired Filtration Rate	$v_{desired}$	15	m/h	5	15 m/h is generally recommended
33	Estimated Filter Area	$A_{estimate}$	96.0	m^2	12.3	practical max = $100m^2$
34	Desired Filter Length-to-Width Ratio	$r_{l/w\ desired}$	2	-	12.3	2-4 is typical $w = \sqrt{\frac{A}{r_{l/w}}}$ $l = w \times r_{l/w}$
35	Estimated Filter Width	$w_{estimate}$	6.9	m		
36	Estimated Filter Length	$l_{estimate}$	13.9	m		
37	Underdrain Panel/Lateral Width	w_{panel}	0.305	m		check suppliers; typical values: panel = 0.61; lateral = 0.305
38	Underdrain Panel/Lateral Length	l_{panel}	1.220	m		check suppliers; typical values: panel = 0.61; lateral = 1.22
39	Number of Panels along Filter Width	$n_{p,w}$	23	-		$n_{p,w} = \frac{w}{w_{panel}}, (rounded)$ $n_{p,l} = \frac{l}{l_{panel}}, (rounded)$
40	Number of Panels along Filter Length	$n_{p,l}$	11	-		
41	Resultant Filter Width	w_{res}	7.02	m		
42	Resultant Filter Length	l_{res}	13.42	m		
43	Proposed Filter Width	w	7.00	m		
44	Proposed Filter Length	l	12.00	m		
45	Resultant Filter Length-to-Width Ratio	$r_{l/w}$	1.71	-	12.3	2-4 is typical
46	Resultant Filter Area	A	84.00	m^2	12.3	practical max = $100m^2$
47		v	15.7	m/h		compare to desired filtration rate
48	Actual Filtration Rate (i.e. all filters online)	Q_f	0.367	m^3/s		
49		v_{max}	17.1	m/h		compare to desired filtration rate
50	Actual Maximum Filtration Rate (i.e. x filters offline)	$Q_{f,max}$	0.400	m^3/s		
51	Filtration Rate Increase	Δv	9.1%	%	5	limit of 15 - 20% is recommended, maximum = 33%

23	Filter Number, Size & Filtration Rate - Design Calculation				Section Reference	
52	Backwash channel position	-	Side	-	14.3	select from drop-down list
53	Backwash channel width	BC_{width}	0.800	m		reference Backwash Channel Width - ref Sheet
54	Backwash channel wall width	BCW_{width}	0.250	m		
55	Filter Width (construction)	$w_{construct}$	8.050	m		
56	Filter Length (construction)	$l_{construct}$	12.000	m		
57	Filter Area (construction)	$A_{construct}$	96.600	m^2		
58	Filter Underdrain Height	h_{ud}	0.500	m	10	include for media retention mechanism and plenum, if applicable
59	Total Bed Depth	L	1.500	m		
60	Media head loss during maximum filtration	H_{media}	0.368	m		uses original Ergun equation (more conservative)
61	Underdrain head loss during maximum filtration	$H_{underdrain}$	0.125	m		
62	Filter pipework head loss during maximum filtration	$H_{pipework}$	0.000	m		
63	Weir (in-filter) overflow height during maximum filtration	H_{weir}	0.364	m		
64	Weir (trough) overflow height during maximum filtration	H_{trough}	0.049	m		$clean\ bed\ headloss = media\ losses + underdrain\ losses + filter\ pipework\ losses + weir\ overflow\ height + trough\ overflow\ height$
65	Clean bed head loss (max filtration)	H_{clean}	0.907	m	6.3	
66	Clogging head allowed	H_{clog}	2.000	m	6.1	2-3 is typical
67	Freeboard in filter	$H_{freeboard}$	0.500	m		$filter\ depth = filter\ underdrain\ height + total\ bed\ depth + clean\ bed\ headloss + clogging\ head\ allowed + freeboard$
68	Calculated Filter Depth	$H_{filtercalc}$	5.407	m		
69	Proposed Filter Depth	H_{filter}	5.800	m	12.4	4.5-7.6 is typical

The filtration rate is interlinked with filter quantity and size considerations. Further guidelines to the designer on how to use this section of the tool is provided in the Appendix section A6.

CHAPTER 6 : HEAD LOSSES

6.1 Introduction

Driving head is the force required to push water through the media bed and overcome the resistance of the bed with any accumulated solids. This resistance is the pressure difference across the media and is termed "head loss" (Beverly, 2011)

Head losses through filter bed are divided into two categories: clean bed head loss and operational head loss (through a clogging filter bed). Other head losses through the filter include the underdrain and fittings and any weirs and/or troughs to the delivery water level. The total driving head for operating a gravity filter is based on the distance from the filter influent level to the level of the filter discharge control point. This can be a control weir, the top of reverse bend piping, the water level in the clearwell (if filter delivery is below the water level in the clearwell) or the level of a downward bend if freely discharging from the filter delivery as shown in Figure 12 (Beverly, 2011; Edzwald, 2011).

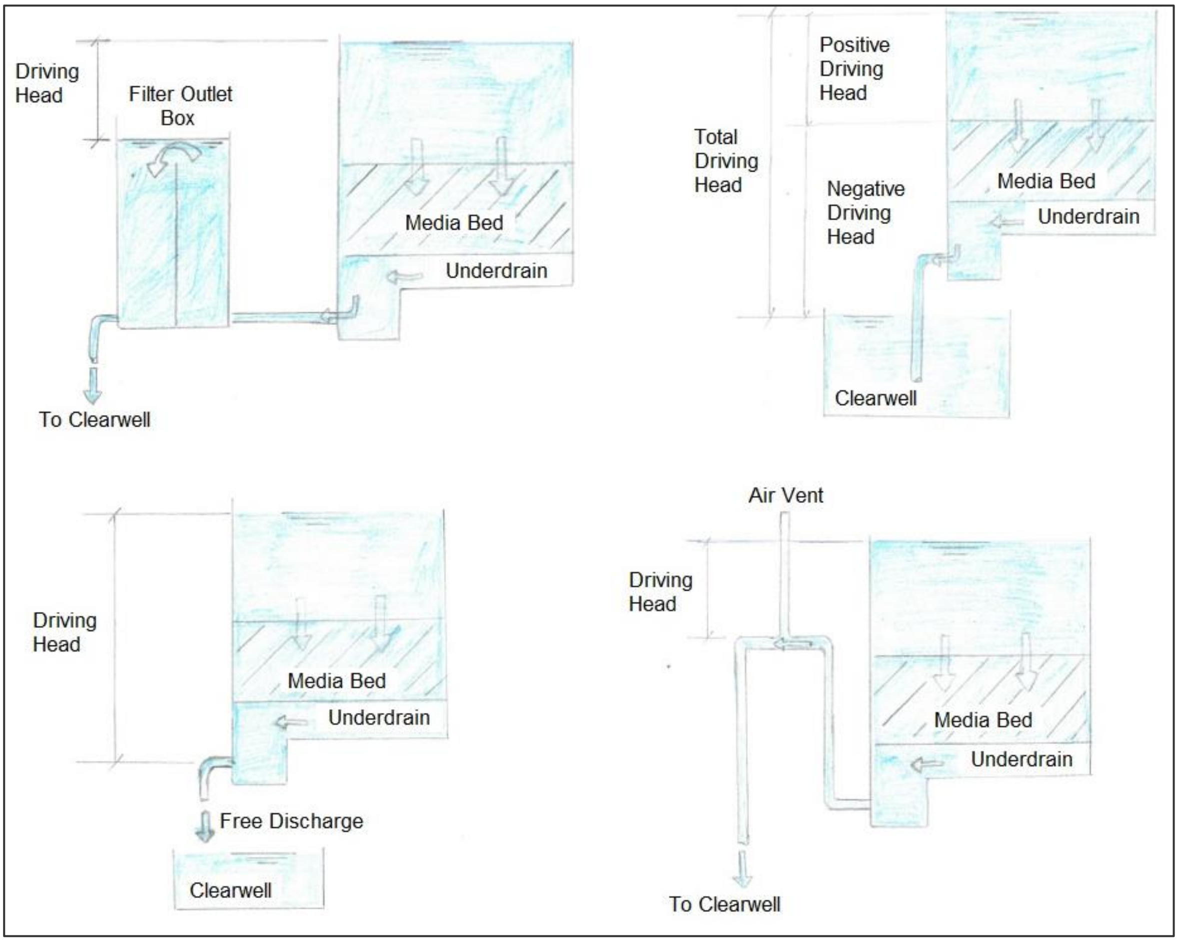


Figure 12 Total Driving Head Configurations (adapted from Beverly (2011))

If the clearwell level can fluctuate, driving head would fluctuate too and should be designed at maximum clearwell level; ie. the available driving head will increase as the clearwell level lowers.

If filter outlet pipework discharges below the clearwell level, an upturned elbow should be installed at the end of the delivery pipe to prevent air from disrupting the driving head (similar to a p-trap function) (Beverly, 2011).

Filter media location influences the design of the driving head. Positive driving head is the distance between the filter water level and the top of the media bed. Negative driving head is the distance between the top of the media and the discharge level. Depending on how much positive driving head is available, the filter can experience negative pressures during the filter run as depicted in Figure 13 – note that any media retention mechanism and underdrain head losses are assumed negligible (Beverly, 2011).

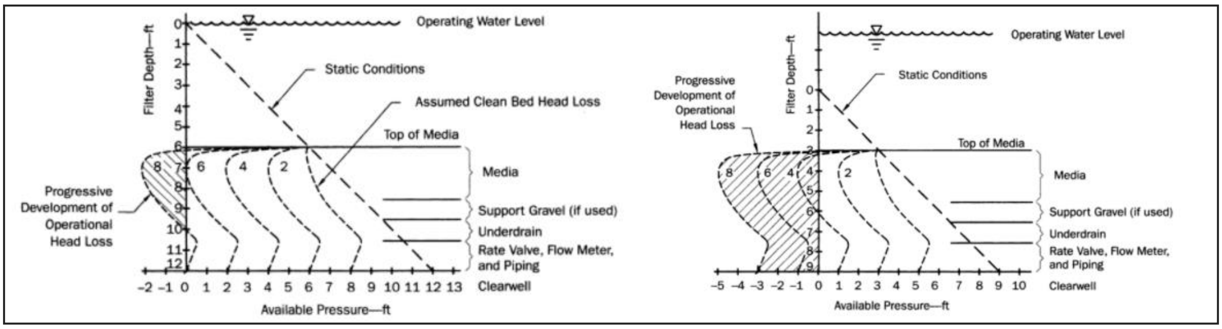


Figure 13 Development of Negative Pressure within a Filter – 6ft Positive Driving Head on Left; 3ft Positive Driving Head on Right (Beverly, 2011)

Negative pressures are undesirable as dissolved gasses can be released and accumulate in the media (called air binding – see Chapter 15.6), which results in increased velocities around the accumulated gas that increases the rate of head loss development and increased filtrate turbidity. Negative pressures are avoided by ensuring that the filter downstream control point is above the top of the media or by stopping the filter run before the total head loss reaches the media's submergence depth. (Edzwald, 2011).

A relatively high discharge control point is preferred to ensure that the filter's water level does not drop to below the top of the media, preventing negative head or partial dewatering of the filter bed. This can result in an abnormally deep filter, typically consisting of an additional 1.5 – 1.8 m depth according to AWWA and American Society of Civil Engineers (2012) that also notes a design guideline to maintain a water level above the filter media that exceeds the clogging head loss for a dirty filter, whereas Gray (2010) suggests a height equivalent to the depth of media, but at least 1.0 m. According to AWWA and American Society of Civil Engineers (2012) the minimum water depth above the filter bed surface should be 0.9 m with deeper filters up to 1.8 m typically chosen for filters operating according to constant rate with constant level (i.e. proportional level equal rate filter operation), for high filtration rates or as an additional measure to prevent air binding. Kawamura (2000) also recommends a minimum of 2.0 m water depth above the filter bed to minimize air binding. The recommendation to prevent negative pressures from occurring should apply to the complete range of filter operation (including for head losses and backwashing process).

Beverly (2011) suggests that the total driving head should then be greater than 3.5 m with a positive driving head of at least 1.8 m.; however, Crittenden *et al.* (2012) suggests a typical design range for available head, i.e. required for clean-bed head loss and operational head loss, to be 2.0 – 3.0 m due to construction costs for gravity filters where the available head is dependent on the filters position relative to up- and downstream structures. AWWA and American Society of Civil Engineers (2012) limits the difference in water level above the media and the water in the underdrain or downstream pipework to 2.4 – 3.0 m.

Van Duuren, South Africa Water Research Commission (1997) indicates that the clogging head allowance guideline in South Africa is 1.5 to 2.5 m and typically 2.0 m. Cleasby,and Haarhoff (1996) notes that this can be reduced for declining rate filters (see Chapter 7 : for a description of this type of filter control method) due to reallocation of turbulent head losses to clogging head loss during the filter run, and the most clogged filter "ends its cycle below the design filtration rate". A simplistic method for estimating the clogging head for declining rate filters is to assume that the lowest rate filter operates at 70% of the design filtration rate and, because head loss is proportional to filtration rate, the allowance can be reduced by 70% (i.e. 1.05 to 1.75 m but typically 1.4 m).

Terminal head loss is the maximum head loss a filter can handle typically indicating the end of a filter run, which typically ranges from 2.4 to 3 m (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011). Further head losses may cause overflows or flooding (AWWA and American Society of Civil Engineers, 2012). Decreasing the operational head loss could reduce the filter run unless pre-treatment is very efficient (Beverly, 2011). Kawamura (2000) suggests that filtrate turbidity starts to increase at 1.8 m head loss and informing a backwash to be initiated. Binnie, Kimber and Smethurst (2002) recommends that backwashing be initiated when the head loss across the filter reaches approximately 1.5 – 2.0 m, whereas Gray (2010) suggests that backwash is normally required when head loss across the filter approaches 2.5 m. It is therefore noted that the decision on the terminal head loss depends on the allowances in determining the driving head and filter depth.

6.2 Filtration Models

Models are not commonly used for design purposes due to their complexities; however, they can be used to evaluate results from pilot studies (Montgomery, James M. Consulting Engineers, 1985; Van Duuren, F. A., South Africa Water Research Commission., 1997). Models are beyond the scope of this study with the contents herein being used for design purposes particularly when pilot studies have not been undertaken.

6.3 Clean Filter Bed

6.3.1 Introduction

The head loss at the start of a filter run is called the "clean-bed head loss". The typical clean filter bed head loss is approximately 0.3 m under design conditions (Binnie, Kimber and Smethurst, 2002; Hendricks, 2010). Clean bed head losses through the media and underdrain system are typically between 0.3 to 0.6 m (AWWA and American Society of Civil Engineers, 2012). Van Durren, Soutth Africa Water Research Commission (1997) also mentions 0.4 m.

The flow patterns through a filter bed are complicated and head loss calculations are not straightforward with the number of variables involved, such as media porosity, particle shape, roughness, media size and size distribution and fluid flow regime; i.e. laminar, transitional or turbulent flow (Montgomery, James M. Consulting Engineers, 1985).

Various empirical models for determining head losses through clean media have been developed. According to Montgomery, James M. Consulting Engineers (1985), the models discussed in this chapter are reasonably accurate at predicting head losses during the initial stages of filtration through clean media.

The flow regime in a filter bed (i.e. porous media) is characterized by Reynolds number as:

$$Re = \frac{\rho_w d_m V_0}{\mu} \quad (Eq\ 6.1)$$

where

Re = Reynolds number

ρ_w = density of water

d_m = media diameter, typically d_{10} (Hendricks, 2010)

V_0 = rate through filter (velocity)

μ = fluid dynamic viscosity

The following flow regimes are possible (Crittenden *et al.*, 2012; Hendricks, 2010):

- Laminar:
  - Darcy / creeping flow : Re < 1
  - Forchheimer flow : 1 ≤ Re > 100
- Transitional : 100 ≤ Re > 600-800
- Turbulent : Re ≥ 800

Laminar flow results for the typical parameters of media filtration in water treatment (Binnie, Kimber and Smethurst, 2002; Hendricks, 2010).

6.3.2 Darcy flow head loss

In 1856, Henry Darcy observed that the hydraulic gradient for laminar flow through media can be represented by:

$$\frac{H_L}{L} = \frac{V_0}{k_p} \quad (Eq.6.2)$$

(Crittenden *et al.*, 2012; Kawamura, 2000)

H_L= head loss across media bed

L = media bed depth

k_p = permeability coefficient, where

$$k_p = c(0.7 + 0.03T)ES^2 \quad (Eq.6.3)$$

(Kawamura, 2000)*

*Note that k_p is determined with ES expressed in centimetres and T (water temperature) in degrees Celsius.

c = coefficient ranging from 123 to 125 for media with ES of 0.3 – 2.0 mm; 124 is commonly used (Kawamura, 2000).

The approximate head loss for typical filter beds at 15°C is listed in Table 11.

Table 11 Typical Approximate Initial Head Loss for Typical Filter Beds at 15°C (Kawamura, 2000)

Filtration Rate (m/h)	Head loss (m)	Type of Filter Bed
5	0.3	Rapid sand
7.5	0.45	Rapid sand
10	0.3	Dual-media
12.5	0.45	Dual-media
20	0.58	Dual-media
25	0.75	Dual-media

The Kozeny equation was later developed to relate the porosity of media to Darcy's law:

$$\frac{H_L}{L} = \frac{k_k \mu {S_v}^2 V_0}{\rho_w g {\varepsilon_0}^3} \quad (Eq.6.3)$$

(Crittenden *et al.*, 2012)

Where

g = gravitational acceleration at 9.81 m/s^2

k_k = Kozeny coefficient, commonly approximated as 5 for spherical media (Fair and Hatch (1933) as cited in (Crittenden *et al.*, 2012; Edzwald, 2011; Logsdon *et al.*, 2002; Montgomery, James M. Consulting Engineers, 1985)).

Substituting the equation for specific surface area (see Chapter 4.2.6) into the Kozeny equation above, and the equation for the relationship between shape factor and sphericity (see Chapter 4.2.5), then the head loss through the filter media per unit depth for laminar flow is therefore:

$$\frac{H_L}{L} = \frac{k_k \times 6^2(1-\varepsilon_0)^2 \mu V_0}{\psi^2 \varepsilon_0{}^3 d_m{}^2 \rho_w g} \quad (Eq. 6.4)\ (\text{Logsdon } et\ al., 2002)$$

6.3.3 Forchheimer flow head loss

For Reynolds number greater than 1, (Crittenden *et al.*, 2012) proposes the Ergun equation as:

$$H_L = k_v \frac{\mu L V_0}{\rho_w d_m{}^2 g} \frac{(1-\varepsilon_0)^2}{\varepsilon_0{}^3} + k_I \frac{L V_0{}^2}{d_m g} \frac{1-\varepsilon_0}{\varepsilon_0{}^3} \quad (Eq. 6.5)$$

Where k_v = head loss coefficient due to viscous forces (laminar flow)

k_I = head loss coefficient due to inertial forces (turbulent flow)

The Ergun equation is the recommended head loss equation for rapid filtration (Crittenden *et al.*, 2012).

It is noted that the Ergun equation can also be utilized if Reynolds number is less than 1 due to the negligible effect of the $V_0{}^2$ parameter in the second term of the equation at these low Reynolds numbers (Hendricks, 2010).

The original Ergun equation (Edzwald, 2011):

$$H_L = k_v \frac{\mu L V_0}{\psi^2 d_m{}^2 \rho_w g} \frac{(1-\varepsilon_0)^2}{\varepsilon_0{}^3} + k_I \frac{L V_0{}^2}{\psi d_m g} \frac{1-\varepsilon_0}{\varepsilon_0{}^3} \quad (Eq. 6.6)$$

utilised head loss coefficients k_v = 150 and k_I = 1.75 (Ergun (1952) as cited in Crittenden *et al.* (2012), Sincero and Sincero (2002), van Duuren, South Africa Water Research Commission (1997)), and later k_I = 2.88 (Ergun (1952) as cited in Edzwald (2011)), but with the use of effective diameters based on specific surface that are not easily measured (Crittenden *et al.*, 2012).

Trussel and Chang (1999) as cited in Crittenden *et al.* (2012) and Hendricks (2010) further researched head loss through granular media to develop a range of head loss coefficients that incorporate media shape and allow for the use of effective size in this Ergun equation. The parameter ranges are provided in Table 12, and Crittenden *et al.* (2012) recommends that the average values are used when site-specific or pilot study information is not available.

Table 12 Recommended Ergun Equation Parameters for Various Media (Crittenden *et al.*, 2012)

Media	k_v Range	Average k_v	k_I Range	Average k_I	Porosity, ε_0, Range	Average Porosity, ε_0
Sand	110 – 115	112	2.0 – 2.5	2.3	0.40 – 0.43	0.41
Anthracite	210 - 245	228	3.5 – 5.3	4.4	0.47 – 0.52	0.50

6.3.4 Transient and turbulent flow head loss

For Reynolds numbers greater than 1000, Montgomery, James M. Consulting Engineers (1985) proposes the use of the Burke-Plumber equation

$$\frac{\Delta p}{L} = \frac{1.75}{d_m} \rho_w V_0{}^2 \frac{1-\varepsilon_0}{\varepsilon_0{}^3} \quad (Eq. 6.7)$$

However, the Ergun equation in Chapter 6.3.3 can be used for turbulent flow conditions (Crittenden *et al.*, 2012; Hendricks, 2010)

6.3.5 Parameter effect on head loss

The effect of various parameters on head loss is illustrated in Figure 14.

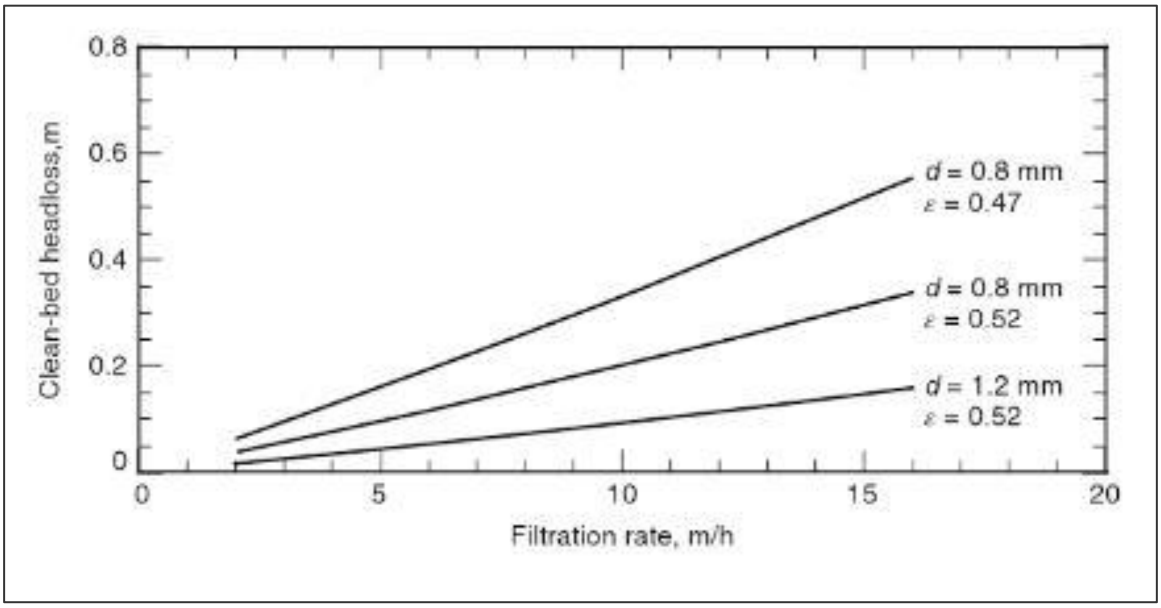


Figure 14 Effect of Various Parameters on Head Loss (for L = 1m; T = 15°C; kv = 228; kl = 4.40) (Crittenden *et al.*, 2012)

Montgomery, James M. Consulting Engineers (1985) and Crittenden *et al.* (2012) point out that head loss is also affected by temperature due to fluid viscosity being dependant on temperature. A reduced temperature increases fluid viscosity and will therefore increase the head loss in accordance with the Kozeny and Ergun equations. The rate of head loss accumulation could also be higher (Logsdon *et al.*, 2002). Crittenden *et al.* (2012) notes that the effect of increased head loss might not be realised if a reduced filtration rate is implemented for lower demands during the winter months (with cold water).

6.3.6 Head loss through non-uniform filter bed

Montgomery, James M. Consulting Engineers (1985) and Crittenden *et al.* (2012) explain that the head loss through a multimedia filter can be determined by summing the head loss through each media layer using the effective size of the media.

Van Duuren, South Africa Water Research Commission (1997) rather calculates head loss for the entire bed by summing the head loss of 10 layers within the bed and using the equivalent diameter of each layer being approximated by an average of the smallest and largest media grains in the specific layer.

Edzwald (2011) rather calculates head loss for the entire bed by summing the head loss of at least five media sizes within the monomedia filter using the original Ergun equation, and by summing the head loss of at least five media sizes within each of the different layers in dual- and mixed-media filters. It is assumed for the calculation that each layer has invariant sphericity, initial porosity and media particle density.

6.4 Clogging Filter Bed

The pore space in the filter bed decreases as the media captures particles; i.e. the effective porosity decreases. At the same time, the media size changes due to the accumulation of particles on the media surfaces. The resultant increasing head loss in the filter bed over time until backwashing occurs is shown in Figure 15. The intermittent head loss peaks are due to other filters being backwashed with an associated increase in the other filter's filtration rate.

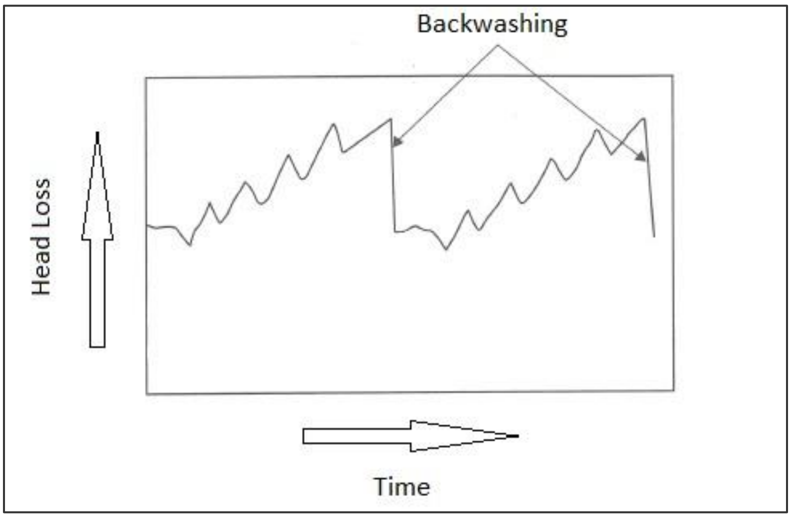


Figure 15 Typical filter head loss development over time (adapted from Binnie, Kimber and Smethurst (2002))

The increase in head loss caused by accumulated particles can be approximated by the infinite series:

$$\frac{\Delta H}{\Delta H_0} = 1 + \gamma_1\sigma + \gamma_1\sigma^2 + \cdots \quad (Eq. 6.8)$$

(Herzig *et al.* (1970) and Ives (1982) as cited in Montogmery, James M. Consulting Engineers (1985))

ΔH_0 = head loss through clean filter

σ = "mass specific deposit (mass/volume) or quantity of solids per volume of filter bed"

γ_i = "empirical constants in units of volume/mass"

Higher order terms than the first-order term can be ignored for low specific deposits and pore volume occupied by solids is less than 10% (Montgomery, James M. Consulting Engineers, 1985).

The effect of the accumulated solids on head loss depends on the type of solid, its density and the degree of compaction within the pore space (Montgomery, James M. Consulting Engineers, 1985). The constant, γ_1, in the above equation can be interpreted as a compaction coefficient (the ratio of volume-specific deposit to mass-specific deposit) given by

$$\frac{\sigma^*}{\sigma} = \frac{\frac{1}{\rho_w} - \frac{1}{\rho_s}}{\left(\frac{\rho_s}{\rho_w} - 1\right)(1 - \varepsilon_D)} \quad (Eq. 6.9)$$

σ^* = volume-specific deposit (volume solids-volume filter)

ρ_s = density of particle

ε_D = deposit porosity

However, measurements of deposit density are not easy, and it is difficult to theoretically determine this compaction coefficient. Table 13 shows the compaction coefficients for various filtration systems.

Table 13 Compaction Coefficients for Various Filtration Systems (Montgomery, James M. Consulting Engineers, 1985)

Type of Solids	Approximate Density, ρs (kg/m³)	Media Size, d_m (mm)	Superficial Velocity, v (m/h)	Compaction Coefficient, σ*/σ (L/g)
Discrete,TiO_2	3.9	0.27	5	0.26
		0.54	5	0.11
		0.94	5	0.05
Discrete, Latex	1.02	0.94	10	0.36

Type of Solids	Approximate Density, ρs (kg/m³)	Media Size, d_m (mm)	Superficial Velocity, v (m/h)	Compaction Coefficient, σ*/σ (L/g)
		1.52	10	0.10
Discrete, Kaolin	2.2	1.8	11	0.04-0.28
Flocculant	1.01	2.2	10	0.5
$Fe(OH)_3$-$FePO_4$		1.9	10	0.7

However, more recently, Van Duuren, South Africa Water Research Commission (1997) notes that there is "no commonly accepted relationship between specific deposit and head loss".

6.5 Underdrain and Fittings

The head loss through filter nozzles can be calculated as follows:

$$H_{nozzles} = \left(\frac{1}{n \times K_n}\right)^2 \times v^2 \quad (Eq.6.10) \ \text{(Cleasby and Haarhoff, 1996)}$$

Where v = filtration rate (m/s)

n = nozzle density (#/m²)

K_n = nozzle coefficient, obtained from nozzle suppliers, but typically ranging from 2.5 x 10^{-4} to 4.5 x 10^{-4} $m^{5/2}s^{-1}$ for nozzles installed in South Africa (Cleasby and Haarhoff, 1996). For a known head loss per nozzle (h_{nozzle}) for a specific flow rate (q_{known}), K_n can be calculated as follows:

$$K_n = \frac{q_{known}}{\sqrt{h_{nozzle}}} \quad (Eq.6.11) \ \text{(Cleasby and Haarhoff, 1996)}$$

Head losses through fittings are calculated and summed using the general form of head loss equation:

$$H_{fittings} = \frac{kv^2}{2g} \quad (Eq.6.12) \ \text{(Cleasby and Haarhoff, 1996)}$$

Where

k=head loss coefficient

Montgomery, James M. Consulting Engineers (1985) indicates that the head loss through underdrain systems during backwash is typically 0.1 to 3 m.

6.6 Weirs and Troughs

The overflow heights for weirs and troughs can be calculated from the following formula (assuming a rectangular overflow weir):

$$q = \frac{2}{3}\sqrt{2g}C_d\left(L_{weir} - \frac{n_{ec}h_{o/f}}{20}\right)h_{o/f}{}^{3/2} \quad (Eq.6.13) \ \text{(Cleasby and Haarhoff, 1996)}$$

or alternately written as

$$0 = -q + \frac{2}{3}\sqrt{2g}C_d\left(L_{weir} - \frac{n_{ec}h_{\frac{o}{f}}}{20}\right)h_{\frac{o}{f}}^{\frac{3}{2}} \quad (Eq.6.14)$$

Where

q = design flow rate (m³/s)

$h_{o/f}$ = overflow depth (m)

C_d = weir coefficient = 0.616

L_{weir} = weir length (m)

$n_{ec.}$ = number of end contractions (0,1 or 2)

6.7 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates the following related sheets (presenting screenshots from the tool):

- "Media Head Losses" for head losses through the filter bed

	A	B	C	D
1	Media Head Loss Summary			
2	Parameters	Value using Modified Ergun Equation	Value using Original Ergun Equation	Unit
3	Head Loss During Actual Filtration	15.7		m/h
4	Media 1 : Anthracite	0.000	0.000	m
5	Media 2 : Sand	0.209	0.332	m
6	Media 3 : Garnet	0.000	0.000	m
7	Total Head Loss	0.209	0.332	m
8	Head Loss During Maximum Filtration	17.14		m/h
9	Media 1 : Anthracite	0.000	0.000	m
10	Media 2 : Sand	0.232	0.368	m
11	Media 3 : Garnet	0.000	0.000	m
12	Total Head Loss	0.232	0.368	m
13	Head Loss during Backwash Water Rate 1	45		m/h
14	Media 1 : Anthracite	0.000	0.000	m
15	Media 2 : Sand	0.821	1.259	m
16	Media 3 : Garnet	0.000	0.000	m
17	Total Head Loss	0.821	1.259	m
18	Head Loss during Backwash Water Rate 2	50		m/h
19	Media 1 : Anthracite	0.000	0.000	m
20	Media 2 : Sand	0.955	1.457	m
21	Media 3 : Garnet	0.000	0.000	m
22	Total Head Loss	0.955	1.457	m

	A	B	C	D	E	F	G	H	I	
24	Media Head Loss Using Effective Size (Modified Ergun Equation)								Section Refere	
25									6.3	
26	Parameter	Symbol	Value						Units	
27	Water Density	ρ_w	997.048						kg/m³	
28	Dynamic Viscosity	μ	0.00089						N.s/m²	
29	Gravitational Acceleration	g	9.81						m/s²	
30	Actual Filtration Rate	v	15.7						m/h	
31			0.004						m/s	
32	Maximum Filtration Rate	v_{max}	17.14						m/h	
33			0.005						m/s	
34	Backwash Water Rate 1	v_{bw1}	45						m/h	
35			0.013						m/s	
36	Backwash Water Rate 2	v_{bw2}	50						m/h	
37			0.014						m/s	
70	Media 2 : Sand									
71			Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Total		
72		L medium	300	300	300	300	300	1500	mm	
73	Bed Depth		0.300	0.300	0.300	0.300	0.300	1.500	m	
74	Laminar coefficient for head loss	k_v	112						-	
75	Turbulent coefficient for head lo	k_l	2.3						-	
76	Effective Size	ES or d10	0.0015						m	
77	Bed depth	L	1.5						m	
78	Loose/initial bed porosity	ε_0	0.4						-	
79	Media Density	ρ_m	2650						kg/m³	
80	Expanded bed porosity	ε	0.40	0.38	0.36	0.34	0.32	-	-	
81	Expanded Bed Depth	L_e	0.300	0.300	0.300	0.300	0.300	1.500	m	
82	Head Loss During Actual Filtration									
83	Laminar head loss	(1st	0.167						m	$H_L = k_v \frac{\mu L V_0}{\rho_w d_m^2 g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
84	Turbulent head loss	(2nd	0.042						m	
85	Total head loss	H_L	0.209						m	
86	Head Loss During Maximum Filtration									
87	Laminar head loss	(1st	0.182						m	$H_L = k_v \frac{\mu L V_0}{\rho_w d_m^2 g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
88	Turbulent head loss	(2nd	0.050						m	
89	Total head loss	H_L	0.232						m	
90	Head Loss during Backwash Water Rate 1 - No Fluidization									
91	Laminar head loss	(1st	0.478						m	$H_L = k_v \frac{\mu L V_0}{\rho_w d_m^2 g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
92	Turbulent head loss	(2nd	0.343						m	
93	Total head loss	H_L	0.821						m	
94	Head Loss during Backwash Water Rate 2 - No Fluidization									
95	Laminar head loss	(1st	0.531						m	$H_L = k_v \frac{\mu L V_0}{\rho_w d_m^2 g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
96	Turbulent head loss	(2nd	0.424						m	
97	Total head loss	H_L	0.955						m	
98	Head Loss during Backwash Water Rate 2 - Full Fluidization									
99	Total head loss	H_L	0.300	0.306	0.319	0.330	0.339	1.594	m	$H_L = L_e(SG_m - 1)(1-\varepsilon)$
100	Head Loss during Backwash Water Rate 2 - Actual									
101	Total head loss	H_L	0.955						m	

	A	B	C	D	E	F	G	H	I	
135	Media Head Loss Using Sphericity (Original Ergun Equation)								Section Reference	
136									6.3	
137	Parameter	Symbol	Value						Units	
171	Media 2 : Sand									
172			Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Total		
173		L medium	300	300	300	300	300	1500	mm	
174	Bed Depth		0.300	0.300	0.300	0.300	0.300	1.500	m	
175	Laminar coefficient for head loss	k_v	150						-	
176	Turbulent coefficient for head loss	k_l	2.88						-	
177	Sphericity	ψ	0.75	0.75	0.75	0.75	0.75	-	-	
178		deq	1.50	1.60	1.85	2.10	2.32	-	mm	
179	Equivalent Diameter		0.00150	0.00160	0.00185	0.00210	0.00232	-	m	
180	Loose/initial bed porosity	ε_0	0.40	0.40	0.40	0.40	0.40	-	-	
181	Media Density	ρ_m	2650	2650	2650	2650	2650	-	kg/m³	
182	Expanded bed porosity	ε	0.40	0.38	0.36	0.34	0.32	-	-	
183	Expanded Bed Depth	L_e	0.300	0.300	0.300	0.300	0.300	1.500	m	
184	Head Loss During Actual Filtration									
185	Laminar head loss	(1st term)	0.079	0.070	0.052	0.041	0.033	0.275	m	$H_L = k_v \frac{\mu L V_0}{\psi^2 d_m^2 \rho_w g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{\psi d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
186	Turbulent head loss	(2nd term)	0.014	0.013	0.011	0.010	0.009	0.057	m	
187	Total head loss	H_L	0.093	0.083	0.064	0.050	0.042	0.332	m	
188	Head Loss During Maximum Filtration									
189	Laminar head loss	(1st term)	0.087	0.076	0.057	0.044	0.036	0.300	m	$H_L = k_v \frac{\mu L V_0}{\psi^2 d_m^2 \rho_w g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{\psi d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
190	Turbulent head loss	(2nd term)	0.017	0.016	0.013	0.012	0.011	0.068	m	
191	Total head loss	H_L	0.103	0.092	0.070	0.056	0.047	0.368	m	
192	Head Loss during Backwash Water Rate 1 - No Fluidization									
193	Laminar head loss	(1st term)	0.227	0.200	0.150	0.116	0.095	0.788	m	$H_L = k_v \frac{\mu L V_0}{\psi^2 d_m^2 \rho_w g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{\psi d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
194	Turbulent head loss	(2nd term)	0.115	0.108	0.093	0.082	0.074	0.471	m	
195	Total head loss	H_L	0.342	0.307	0.243	0.198	0.169	1.259	m	
196	Head Loss during Backwash Water Rate 2 - No Fluidization									
197	Laminar head loss	(1st term)	0.253	0.222	0.166	0.129	0.105	0.875	m	$H_L = k_v \frac{\mu L V_0}{\psi^2 d_m^2 \rho_w g}\frac{(1-\varepsilon_0)^2}{\varepsilon_0^3} + k_l \frac{L V_0^2}{\psi d_m g}\frac{1-\varepsilon_0}{\varepsilon_0^3}$
198	Turbulent head loss	(2nd term)	0.142	0.133	0.115	0.101	0.091	0.582	m	
199	Total head loss	H_L	0.394	0.355	0.281	0.230	0.197	1.457	m	
200	Head Loss during Backwash Water Rate 2 - Full Fluidization									
201	Total head loss	H_L	0.300	0.306	0.319	0.330	0.339	1.594	m	$H_L = L_e(SG_m - 1)(1-\varepsilon)$
202	Head Loss during Backwash Water Rate 2 - Actual									
203	Total head loss	H_L	1.457						m	

■ "Other Head Losses"

	A	B	C
1	Other Head Losses Summary		
2	Parameters	Value	Unit
3	Head Loss During Actual Filtration	15.7	m/h
4	Underdrain losses	0.109	m
5	Pipework losses	0.000	m
6	Weir Overflow Height	0.344	m
7	Trough Overflow Height	0.047	m
8	Total Other Head Losses	0.499	m
9	Head Loss During Maximum Filtration	17.14	m/h
10	Underdrain losses	0.125	m
11	Pipework losses	0.000	m
12	Weir Overflow Height	0.364	m
13	Trough Overflow Height	0.049	m
14	Total Other Head Losses	0.539	m
15	Head Loss during Backwash Water Rate 1	45	m/h
16	Underdrain losses	0.790	m
17	Pipework losses	0.816	m
18	Weir Overflow Height	0.149	m
19	Trough Overflow Height	0.075	m
20	Total Other Head Losses	1.830	m
21	Head Loss during Backwash Water Rate 2	50	m/h
22	Underdrain losses	0.952	m
23	Pipework losses	1.005	m
24	Weir Overflow Height	0.160	m
25	Trough Overflow Height	0.081	m
26	Total Other Head Losses	2.198	m

	A	B	C	D	E	F	G	H	I	J	K
31–32	Other Head Losses										
33		Actual Filtration Rate		Maximum Filtration Rate		Backwash Water Rate 1		Backwash Water Rate 2			
34		v		v_{max}		v_{bw1}		v_{bw2}			
35		m/h	m/s	m/h	m/s	m/h	m/s	m/h	m/s		
36		15.7	0.004	17.1	0.005	45.0	0.013	50.0	0.014		
37	Underdrain										Section Reference
38	Underdrain Type	Nozzle False Bottom									10
39	Media Retention Mechanism	None									11
40	Parameter	Values								Units	
41	Nozzle Density (n)	43								#/m^2	
42	Nozzle Coefficient (K_n)	0.00035								$m^{(5/2)}/s$	6.5
43	Nozzle head loss ($H_{nozzles}$)	0.084		0.100		0.690		0.852		m	6.5
44	Media Retention Cap head loss (H_{retcap})	not applicable		not applicable		not applicable		not applicable		m	
45	Laterals head loss ($H_{laterals}$)	not applicable		not applicable		not applicable		not applicable		m	10.3
46	Other related head losses ($H_{underdrain,other}$)	0.025		0.025		0.100		0.100		m	
47	Underdrain head loss ($H_{underdrain}$)	0.109		0.125		0.790		0.952		m	

Row	A	Actual Filtration Flow Rate	Maximum Filtration Flow Rate	Backwash Water Flow Rate 1	Backwash Water Flow Rate 2	Unit	Notes
31–32	**Other Head Losses**						
48	**Pipework**						
49		Actual Filtration Flow Rate	Maximum Filtration Flow Rate	Backwash Water Flow Rate 1	Backwash Water Flow Rate 2		
50		Q_f	$Q_{f,max}$	Q_{bw1}	Q_{bw2}		
51		m^3/s	m^3/s	m^3/s	m^3/s		
52		0.367	0.400	1.050	1.167		
53	**Pipe No 1**						
54	Pipe Material	Stainless Steel		Stainless Steel		-	
55	Pipe Length	5		30		m	
56		600		900		mm	selected in Filter Conduit & Valve Sizing Sheet
57	Pipe Diameter	0.6		0.9		m	
58	Pipe Area	0.283		0.636		m^2	
59	Pipe Velocity	1.30	1.41	1.65	1.83	m/s	$Re = \frac{\rho_w d v}{\mu}$
60	Reynold's Number	871312	950522	1664114	1849015	-	
61	Pipe Absolute Roughness (e)	0.015		0.015		mm	reference Pipework & Fittings - reference Sheet
62	Pipe friction factor (f)	0.01247	0.01232	0.01126	0.01111	-	$f = \left(\frac{1}{-2\log\left(\frac{e}{3.7D} + \frac{5.1286}{N_R^{0.89}}\right)}\right)^2$ $h_f = f\,\frac{L}{D}\,\frac{V^2}{2g}$
63	Pipe Length Head Loss (H_{pipe1})	0.00890	0.01046	0.05211	0.06350	m	
64	Fittings friction coefficient sum (∑K)	3		10		-	reference Pipework & Fittings - reference Sheet
65	Pipe Fittings Head Loss ($H_{fittings1}$)	0.257	0.306	1.388	1.714	m	$h_L = K\,\frac{v^2}{2g}$ $H_{pipework} = H_{pipe} + H_{fittings}$
66	Pipework Losses ($H_{pipework1}$)	0.27	0.32	1.44	1.78	m	
67	**Pipe No 2**						
68	Pipe Material	HDPE		HDPE		-	
69	Pipe Length	0		70		m	
70		600		900		mm	estimated at nominal diameter of pipework
71	Pipe Diameter	0.6		0.9		m	
72	Pipe Area	0.283		0.636		m^2	
73	Pipe Velocity	1.30	1.41	1.65	1.83	m/s	$Re = \frac{\rho_w d v}{\mu}$
74	Reynold's Number	871312	950522	1664114	1849015	-	
75	Pipe Absolute Roughness (e)	0.015		0.015		mm	reference Pipework & Fittings - reference Sheet
76	Pipe friction factor (f)	0.01247	0.01232	0.01126	0.01111	-	$f = \left(\frac{1}{-2\log\left(\frac{e}{3.7D} + \frac{5.1286}{N_R^{0.89}}\right)}\right)^2$ $h_f = f\,\frac{L}{D}\,\frac{V^2}{2g}$
77	Pipe Length Head Loss (H_{pipe2})	0.00000	0.00000	0.12159	0.14816	m	
78	Fittings friction coefficient sum (∑K)	0		5		-	reference Pipework & Fittings - reference Sheet
79	Pipe Fittings Head Loss ($H_{fittings2}$)	0.000	0.000	0.694	0.857	m	$h_L = K\,\frac{v^2}{2g}$ $H_{pipework} = H_{pipe} + H_{fittings}$
80	Pipework Losses ($H_{pipework2}$)	0.00	0.00	0.82	1.01	m	
81	Total Pipework Losses ($H_{pipework}$)	0.27	0.32	2.26	2.78	m	

Row	A	Actual Filtration Flow Rate	Maximum Filtration Flow Rate	Backwash Water Flow Rate 1	Backwash Water Flow Rate 2	Unit	K
31–32	**Other Head Losses**						
82	**Weir (in-filter)**						6.6
83		Actual Filtration Flow Rate	Maximum Filtration Flow Rate	Backwash Water Flow Rate 1	Backwash Water Flow Rate 2		
84		Q_f	$Q_{f,max}$	Q_{bw1}	Q_{bw2}		
85		m^3/s	m^3/s	m^3/s	m^3/s		
86		0.367	0.400	1.050	1.167		
87	Gravitational Acceleration (g)	9.81				m/s^2	
88	Weir Coefficient (C_d)	0.616				-	
89	Weir Length (L_{weir})	1.000	1.000	10.000	10.000	m	
90	Number of End Contractions (n)	0	0	0	0	-	
91	Overflow Height (H_{weir}) - calculated	0.344	0.364	0.149	0.160	m	
92	Overflow Height (H_{weir}) - design	0.344	0.364	0.149	0.160	m	
93	Weir Formula	0.000	0.000	0.000	0.000	-	
94	**Backwash Water Troughs**						6.6
95		Actual Filtration Flow Rate	Maximum Filtration Flow Rate	Backwash Water Flow Rate 1	Backwash Water Flow Rate 2		
96		Q_f	$Q_{f,max}$	Q_{bw1}	Q_{bw2}		
97		m^3/s	m^3/s	m^3/s	m^3/s		
98		0.367	0.400	1.050	1.167		
99	Gravitational Acceleration (g)	9.81				m/s^2	
100	Weir Coefficient (C_d)	0.616				-	
101	Number of Troughs	2				-	
102	Trough Length	7.000				m	
103	Weir Length (L_{weir})	28.000	28.000	28.000	28.000	m	
104	Number of End Contractions (n)	0	0	0	0	-	
105	Overflow Height (H_{trough}) - calculated	0.047	0.049	0.075	0.081	m	
106	Overflow Height (H_{trough}) - design	0.047	0.049	0.075	0.081		
107	Weir Formula	0.147	0.160	0.000	0.000	-	

Further guidelines to the designer on how to use these sections of the tool is provided in the Appendix sections A10 and A12, respectively.

CHAPTER 7 : FILTER CONTROL METHODS

7.1 Introduction

Filter control has an impact on the treatment facility/building design and operation and control, both of which are outside the scope of this dissertation, but the details are included herein for the designer's awareness.

Filter control has three objectives (Crittenden *et al.*, 2012):

- "Control the filtration rate of individual filters
- Distribute flow among individual filters
- Accommodate increasing head loss"

Additionally, the control system must provide acceptable filtrate quality whilst dealing with the following (Cleasby and Haarhoff, 1996):

- Flow variations during filter cycles
- Different clogging properties of filter influent
- Differing stages of clogging of individual filters
- Filters being backwashed/out of service

Filter control by gravity can be classified according to rate control as either declining rate or equal rate.

The term "constant rate" is often used in industry instead of equal rate; however, Edzwald (2011) explains that true constant rate filtration would require a constant plant flow rate with constant number of online filters, which is not commonly possible.

Declining rate and equal rate control can be further categorised as per Table 14.

Table 14 Filter Operational Control Methods (Edzwald, 2011)

Control Mode	Control Methods
Equal Rate	Variable level influent flow splitting
	Proportional level influent flow splitting
	Proportional level equal rate
Declining Rate	Variable level
	Proportional level

7.2 Variable level influent flow splitting (equal rate)

Flow splitting control includes a flow-splitting device upstream of the filters, typically weirs or orifices in the influent channel. The weir design is important for the hydraulics to ensure an equal portion of the flow to each filter without introducing unnecessary head loss through the water treatment plant.

Variable level influent flow splitting control can be described as follows and as illustrated in Figure 16 (Edzwald, 2011):

- Non-mechanical
- Inlet to filters above filter maximum water level
- Filtrate to clear well at a level above the filter bed
- Water level in each filter differs, increasing as the filter clogs to provide the head necessary for flow through the filter bed. The water level may drop briefly if the flow rate through the water treatment plant is reduced, after which the water level will increase again as the filter continues to clog (Cleasby and Haarhoff, 1996).
- Initiate backwash for filter whose level rises to a preset high level
- No instrumentation required; simplest operation whereby operators monitor the filter levels – making this method simple and effective (Crittenden *et al.*, 2012)
- If the water level in the filter rises to above the inlet, i.e. the inlet becomes submerged, the filter control will become declining rate control

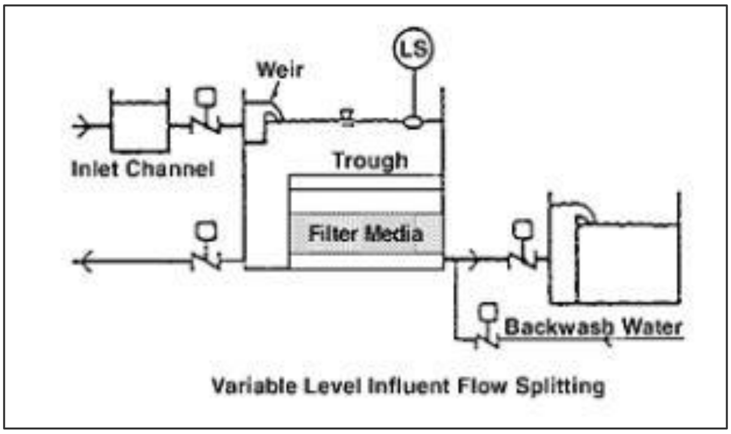


Figure 16 Variable Level Influent Flow Splitting (Equal Rate) Control (Edzwald, 2011)

7.3 Proportional level influent flow splitting (equal rate)

Proportional level influent flow splitting control is similar to variable level influent flow splitting control, but with added instrumentation and filter outlet control valves.

Proportional level influent flow splitting control can be described as follows and as illustrated in Figure 17 (Edzwald, 2011):

- Mechanical
- Level instrument per filter
- Head loss instrument per filter
- <u>Modulating control valve</u> on each filter outlet to control the <u>associated filter level</u>; i.e. the modulating valve is partially closed at the start of the filter run to dissipate extra head and gradually opens as the respective filter media clogs, providing adjustable resistance to the flow, to maintain a constant water level and flow for the filter. (AWWA and American Society of Civil Engineers, 2012).
- Initiate backwash based on head loss

- No flow measurement instrumentation required

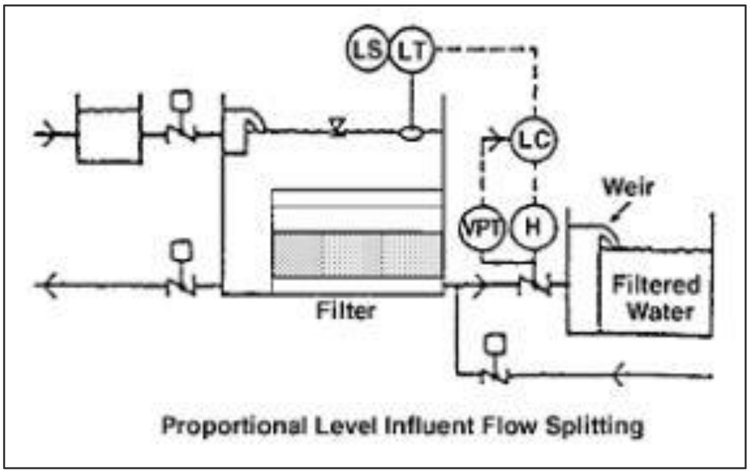


Figure 17 Proportional Level Influent Flow Splitting (Equal Rate) Control (Edzwald, 2011)

7.4 Proportional level equal rate

Proportional level equal rate control is the most common filter control method (AWWA and American Society of Civil Engineers, 2012; Edzwald, 2011). It can be described as follows and as illustrated in Figure 18 (Edzwald, 2011):

- Mechanical
- Submerged inlet to filters
- Level instrument in filter influent channel serving all filters – Beverly (2011) notes that this is suitable for fewer number of filters, whereas a lot of filters could result in large variances from one side of the channel to the other end and then an instrument per filter would be better.
- Flow measurement per filter (e.g. venturi or magnetic flow meter) with associated outlet modulating control valve to ensure equal distribution of flow between the filters (proportional to filter influent level)
- Head loss instrument per filter
- Initiate backwash based on head loss
- Note that unequal flow can easily take place undetected

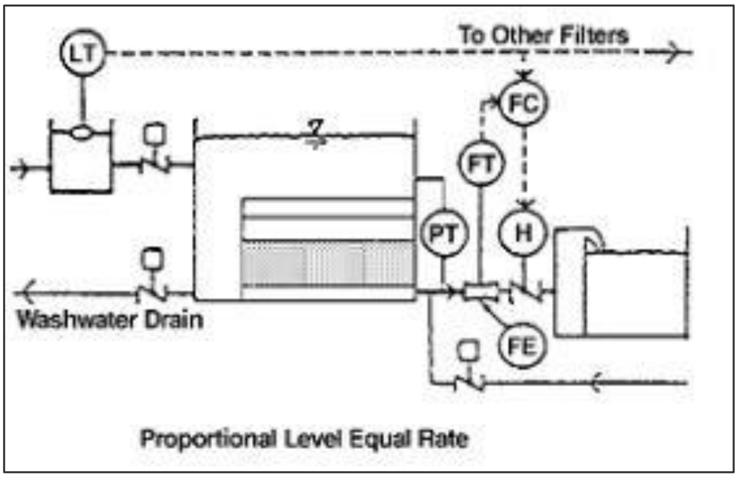


Figure 18 Proportional Level Equal Rate Control (Edzwald, 2011)

During backwash or when a filter is taken out of service, the remaining filters must accommodate the additional flow or the inlet flow must be reduced to maintain the filtration rate.

Filter media clogging, backwashing, filters being taken out of service for maintenance and influent flow rate changes affect the filter influent channel water level. The operational influent channel water level variation is generally less than 150 mm (AWWA and American Society of Civil Engineers, 2012).

7.5 Variable Level Declining Rate

Declining rate control allows for the inlet flow to distribute to the filters in accordance with their bed condition; i.e. flow is controlled by head loss through each filter.

Variable level declining rate control can be described as follows and as illustrated in Figure 19 (Edzwald, 2011):

- Non-mechanical
- Deep, submerged inlet to filters (below normal water level)
- Filtrate to clear well at a level above the filter bed
- All filters operating at approximately same water level with same total head loss available to filtrate weir level
- Higher filtration rate through cleanest filter and lowest filtration rate through dirtiest filter
- Level instrument in filter influent channel serving all filters
- Initiate backwash based on influent channel water level reaching a preset high level (or on a timer or low filtrate flow rate) – the filter that has been operating for the longest period is backwashed (Cleasby and Haarhoff, 1996).
- "Filtration rate of each filter declines in a stepwise fashion after each backwash" with the cleanest filter operating at the highest filtration rate
- Fixed flow-limiting device required on the filtrate line of each filter to limit the maximum filtration rate through a clean filter (e.g. orifice plate or valve in a fixed position); typically limited to 130% of design average flow (Binnie, Kimber and Smethurst, 2002)
- No flow measurement or head loss instrumentation required; however, operators find flow measurement beneficial for understanding the system

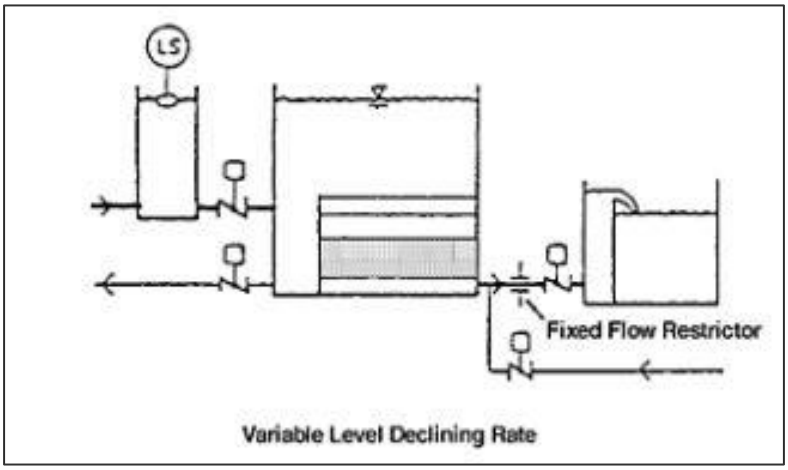


Figure 19 Variable Level Declining Rate Control (Edzwald, 2011)

7.6 Proportional Level Declining Rate

Proportional level declining rate control is similar to variable level declining rate control, but with added filter outlet control valves and can be described as follows and as illustrated in Figure 20 (Edzwald, 2011):

- Mechanical
- Level in the filter influent channel controls all the modulating control valves on each filter outlet within a filter level band, opening as the water level increases

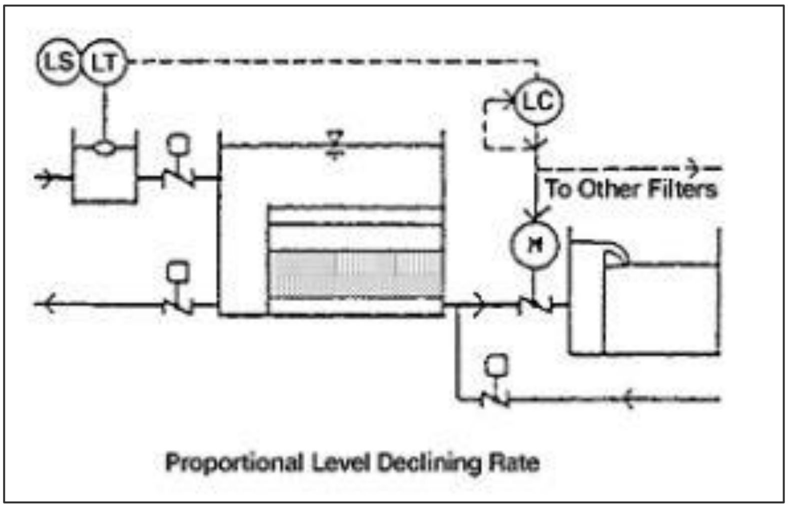


Figure 20 Proportional Level Declining Rate Control (Edzwald, 2011)

7.7 General control elements

Irrespective of the filter control method, "smooth transition during changes in filtration rate is highly desirable" (AWWA and American Society of Civil Engineers, 2012) as discussed in Chapter 5 : . The non-mechanical filter control methods have the advantage that changes in filtration rate are based on the inherent slow and smooth change in water level (Edzwald, 2011). The mechanical systems must be configured such that the control valves have a slow rate of change (Edzwald, 2011) for slow start and stop processes to minimize abrupt changes to the filters filtration rate that would negatively impact the filtrate quality (see Chapter 5 :).

Also, the higher filtration rate immediately experienced after backwashing can cause high filtrate turbidity for a period, which is disadvantageous (AWWA and American Society of Civil Engineers, 2012). Therefore, a filter-to-waste control function, as discussed in Chapter 8.10, is recommended for declining rate control.

7.8 Choice of control method

AWWA and American Society of Civil Engineers (2012) suggests filter control methods be selected early in the design process as it will affect the filter configuration, treatment plant hydraulics and filter performance. Beverly (2011) relates the control choice to influent quality, hydraulic loading and filtrate quality requirements, whereas Crittenden *et al.* (2012) states that the choice of control method is typically made according to "designer and owner preferences" with "cost, complexity and reliability" factors in mind. Kawamura (2000) suggests that the complexities of the control system correlate with the qualifications and capabilities of the operational and maintenance personnel.

Edzwald (2011) recommends the control methods for the listed conditions as per Table 15, although other conditions may exist.

Table 15 Conditions and Control Method Choices (Edzwald, 2011)

Condition	Control Methods
Full automation	▪ Proportional level equal rate ▪ Proportional level declining rate
Minimizing equipment	▪ Variable level influent flow splitting ▪ Variable level declining rate
Small plants (≤ 4 filters); non-automated; unskilled operators	Variable level influent flow splitting
Large non-automated plants	Variable level declining rate

Beverly (2011) mentions that the variable level equal rate control method needs at least four filters, but works best with six to eight or more.

The disadvantages of variable level influent flow splitting control include deeper filters, which can be expensive to construct, and possible air entrainment or floc break up in the filter due to free falling influent water, which will degrade filter performance (Beverly, 2011).

Flow control methods that utilise modulating control valves offer maximum flexibility and control, but are the most complex requiring maintenance and valves possibly hunting to find the correct position which could impact on filtration rates and breakthrough (Crittenden *et al.*, 2012).

Declining rate control has the advantages of higher water production per filter run and improved filtrate quality (AWWA and American Society of Civil Engineers, 2012), is simple with low maintenance and is suited to developing countries (Kawamura, 2000). However, declining rate control is not common. Kawamura (1999) states that, despite the simple design and operation, declining rate is not popular with the minimal operational control available to operators, and the high filtration rates at the start of the filter run. Furthermore Kawamura (2000) explains that there is no clear indication for filter backwashing with head loss similar for all filters, no flow meter to indicate the filter filtration rates and filtered water turbidity not increasing with time. AWWA and American Society of Civil Engineers (2012) adds that declining rate control is uncommon due to advances in instrumentation technology. Binnie, Kimber and Smethurst (2002) provides a more detailed explanation of the reduced use of declining rate control methods that led to the discontinuation of declining rate control in the UK and discouragement in many US states. The disadvantages are associated with operating the filter system at less than the design flow or deterioration in influent quality. Deterioration in influent quality can lead to increased head loss development, which may require that many filters require backwashing to maintain the overall flow rate. Operation of the system at lower flows than the design average can result in disproportionately high flows through a clean filter bed, which can lead to inefficient removal of Cryptosporidium at these high and uncontrolled filtration rates.

Equal rate control is most common in South Africa (Van Duuren, F. A., South Africa Water Research Commission., 1997). Kawamura (2000) further indicates that equal rate control provides better filter control and is popular with operators. Proportional level equal rate control is the most common of the equal rate control methods (Edzwald, 2011). This method provides good control over the filters, but capital costs may be more expensive in construction due to space requirements for the flow meter and outlet valve. Beverly (2011) and Cleasby and Haarhoff (1996) further note that equipment malfunction is common with this control method, which can result in unequal flow distribution that goes unnoticed.

Designing a plant that can easily interchange between control methods is advantageous. Designing a filter with equal rate and a modulating valve and flow meter can easily be converted to declining rate by removing the valve's modulating function and editing the programming.

CHAPTER 8 : BACKWASHING

8.1 General

As previously mentioned, filters need to be cleaned to dislodge and remove the particles that have accumulated in the filter bed during the filter run. Effective backwashing is necessary to ensure successful filter service in the long-term. Backwashing of rapid media filters is also used during installation and commissioning of filter media as discussed in Chapter 4.6.

Backwashing is the method of restoring the filter bed to its original state to be able to filter again; i.e. restoration of voids for capturing particles and, if applicable, restratification of the media. Restratification results in larger, less dense media particles resting towards the top of the filter bed and smaller, denser media particles resting towards the bottom of the filter bed (Logsdon, 2008). The efficiency of backwashing is dependent on the backwash rate and media expansion, backwash water volume and the underdrain's ability to distribute evenly.

As particles accumulate in the filter bed, bed porosity decreases with an increase in head loss and shear on captured particles. Backwashing should be initiated before the head loss increases to an unacceptable level or turbidity breakthrough occurs. (AWWA and American Society of Civil Engineers, 2012).

It is suggested that a filter run of 24 hours is good (Beverly, 2011; Crittenden *et al.*, 2012; Montgomery, James M. Consulting Engineers, 1985); but Beverly (2011) notes that this should not be shorter than 12 hours. Filter run times are commonly 1 to 3 days (AWWA and American Society of Civil Engineers, 2012; Gray, 2010; Montgomery, James M. Consulting Engineers, 1985), even up to 4 days (Crittenden *et al.*, 2012; Edzwald, 2011), but Beverly (2011) recommends backwashing at least every 2 days to avoid media compaction that will make backwashing difficult. Van Duuren, South Africa Water Research Commission (1997) notes that timers set to initiate backwash are typically set at 48 to 72 hours filter run time.

Kawamura (2000) indicates a 24-hour filter run time for filtration rates less than 15 m/h and an average of 48 hours for reverse-graded filters operating at filtration rates of 10 – 15 m/h.

AWWA and American Society of Civil Engineers (2012) indicates that biofilters are typically backwashed at the same frequency as conventional filters.

Filter backwashing must be managed and monitored. Apart from effective backwashing, there must be sufficient filters online for water production, sufficient source water for backwashing, sufficient capacity available in the backwash residuals management system (if applicable), operator availability and capability to ensure water production without consecutive or concurrent filter backwashing.

Filters that are not backwashed properly can lead to mudball accumulation in the filter bed that results in clogging. As head loss increases, the clogged areas contract and cracks can form at the filter bed surface or shrinkage away from the filter walls. These cracks can channel the flow both during filtration and backwashing leading to poor filtrate quality and bed upset, respectively. (AWWA and American Society of Civil Engineers, 2012). See Chapter 15 : for details of filter problems.

8.2 Fluidization Velocity and Bed Expansion

Full fluidization means that the entire bed; i.e. all layers, are fluidized (AWWA and American Society of Civil Engineers, 2012). This is generally required for filter beds that require restratification (dual and mixed media filters); i.e. the media is regraded according to its density allowing for discrete layers of different media types for better filter performance (Gray, 2010; Logsdon *et al.*, 2002). However, full fluidization is not required for single medium filter beds (Logsdon *et al.*, 2002) where restratification would result in fine grains moving to the top of the filter bed, which would increase head loss development and reduce the filter run (AWWA and American Society of Civil Engineers, 2012). Additionally, full fluidization of the deep-bed filter would require excessive backwash water rates and therefore backwashing without full fluidization is possible provided that concurrent air is used to scour the media (improving the effectiveness of the backwash). The concurrent air and water wash mixes the media with minimal stratification, if any (Edzwald, 2011).

Montgomery, James M. Consulting Engineers (1985) notes that full fluidization is common in the United States, whereas Europe generally backwashes with partial fluidization and air scour. Edzwald (2011) summarises the comparative requirements for backwashing with and without fluidization as per Table 16.

Table 16 Comparison of Backwashing Media Beds With and Without Fluidization

	With Fluidization	Without Fluidization
Applications	• Fine sand • Dual media • Mixed Media	• Coarse monomedia or anthracite
Backwash Methods (see Chapter 8.3)	• Water and surface wash • Water and air scour (no air during overflow)	• Water and air scour (simultaneous during overflow)
Fluidization	Yes, during backwash water	No
Bed Expansion	15 – 30%	Negligible
Backwash Troughs (see Chapter 14.3.2)	Usually provided	Usually not provided

AWWA and American Society of Civil Engineers, 2012 defines a fluidizing velocity as a velocity of upward flow of a fluid (air or water) through solid media that suspends the solids in the fluid. The velocity at which fluidization of the particular medium occurs is called the minimum fluidization velocity, V_{mf}, i.e. when media grains start to suspend in the water.

Velocities higher than V_{mf} will cause fluidization and higher velocities will cause media expansion. Logsdon *et al.* (2002) indicates that backwash rates must be within 1.1 – 1.3 V_{mf} to achieve backwashing, utilizing 1.1 factor when support gravel is provided and 1.3 factor when support gravel is not provided. Edzwald (2011) similarly suggests this, whereas Hendricks (2010) recommends a constant 1.3 factor for uncertainty.

AWWA and American Society of Civil Engineers (2012) defines low-rate water backwash as a rate that does not fully fluidize the bed; i.e. less than 10% bed expansion, and high-rate water backwash as a rate that can fully fluidize the bed expansion.

Logsdon *et al.* (2002) and Edzwald (2011) as well as Hendricks (2010) provide the following equations for estimating V_{mf}:

(Wen and Yu equation developed by Wen and Yu (1966) as cited in Edzwald (2011))

$$V_{mf} = \frac{\mu}{\rho_W d_{eq}}(33.7^2 + 0.0408Ga)^{0.5} - \frac{33.7\mu}{\rho_W d_{eq}} \quad (Eq. 8.1)$$

And Ga (the dimensionless Galileo number)

$$Ga = \frac{{d_{eq}}^3 \rho_W(\rho_m - \rho_W)g}{\mu^2} \quad (Eq. 8.2)$$

or similarly

$$Ga = \frac{{d_{eq}}^3 {\rho_w}^2(SG_m - 1)g}{\mu^2} \quad (Eq. 8.3)$$

For fluidization of the entire bed, the calculation is done for the coarser grains in the bed. The required fluidization velocity must be adequate for the coarsest media without extreme bed expansion that would cause loss of finer media (AWWA and American Society of Civil Engineers, 2012). For this, Beverly (2011) defines the d_{90} fluidization rate to be the maximum required backwash rate. A higher rate will not increase shear or agitation and will merely increase the distances between grains. Logsdon *et al.* (2002) and Edzwald (2011) suggest that the more readily available d_{90} can be used as an approximation of d_{eq} in the above equations. Edzwald (2011) further notes that using d_{90} for media with large grain size variations shouldn't be a problem due to the uniformity coefficient (UC) typically being limited to less than 1.7 by the media specifications. It is noted that Hendricks (2010) also utilizes d_{90} for d_{eq} in the above equations.

Amirtharajah and Cleasby (1972) and Hewitt and Amirtharajah (1984) as cited in Hendricks (2010) also describes a different equation resulting in a slightly higher V_{mf} than that above, with d_{60} for d_{eq}, as follows:

$$V_{mf} = \frac{3.2193 \times 10^{-11} \times {d_{eq}}^{1.82} \times ({\rho_w}^2(SG_m - 1))^{0.94}}{\mu^{0.88}} \quad (Eq. 8.4)$$

But this equation is only valid for a Reynolds number less than or equal to 10, with the following correction factor applicable to 10<Re<300 as follows (Hendricks, 2010):

$$corrected\ V_{mf} = V_{mf}(1.775Re^{-0.272}) \quad (Eq. 8.5)$$

Alternatively, V_{mf} can be estimated by consulting test data, considering media type, grade, size and density as well as water temperature (AWWA and American Society of Civil Engineers, 2012).

A simple method to physically establish V_{mf} is to extend a long pole into the filter bay that rests on the surface of the media, and then gradually increase the backwash flow rate until the pole begins to sink into the media (Logsdon, 2008).

A different experimental method to calculate V_{mf} is by measuring the head loss across the media for different upflow velocities. Head loss will increase as upflow velocity increases until fluidization occurs (AWWA and American Society of Civil Engineers, 2012), as depicted in Figure 21.

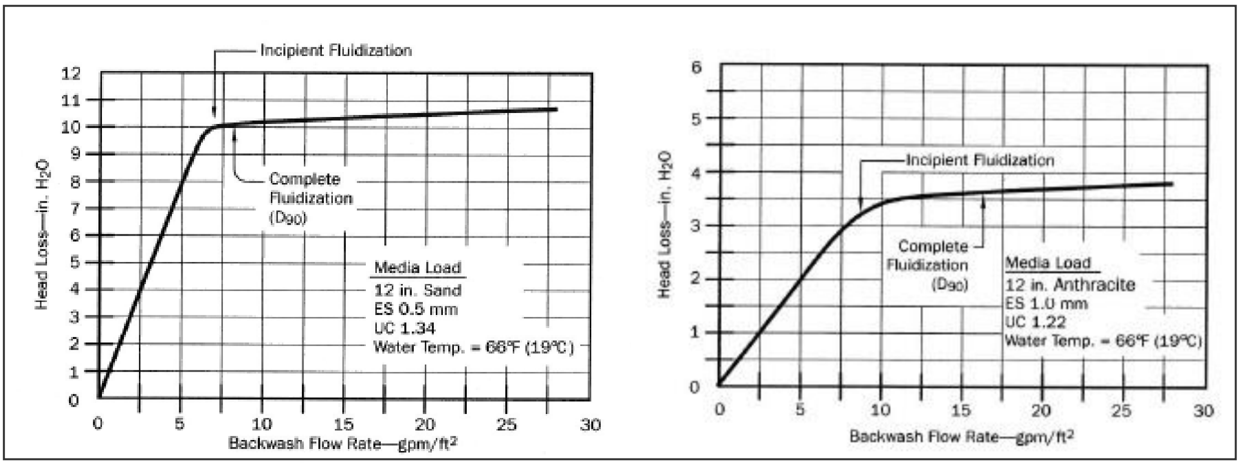


Figure 21 Example Head Loss Versus Backwash Rate Curves for Monomedia Filters (Beverly, 2011)

Note that anthracite has a V_{mf} higher than sand at almost the equivalent of a fully fluidized sand bed, and full fluidization occurring at almost double the rate of sand.

A review of an example clean dual media filter's head loss versus backwash rate is shown in Figure 22. Full fluidization occurs at approximately the same rate as anthracite alone (as shown in Figure 21 above), but fluidization starts earlier.

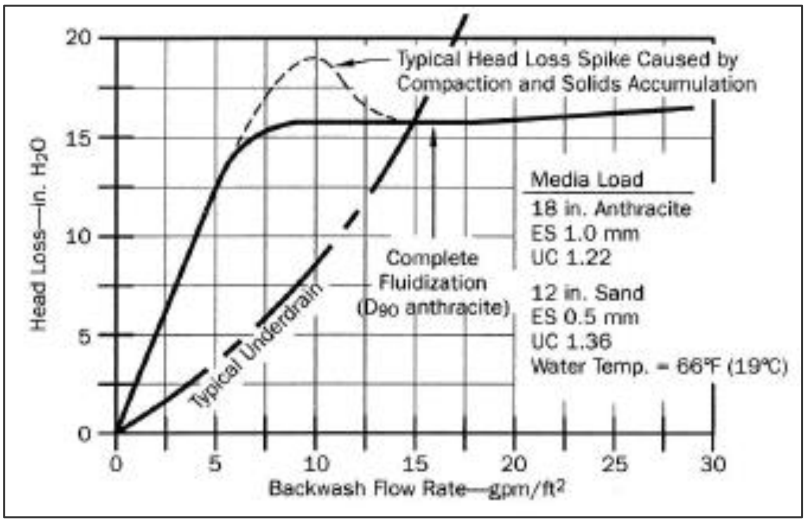


Figure 22 Example Head Loss Versus Backwash Rate Curves for Dual Media Filters (Beverly, 2011)

Figure 22 also depicts the head loss spike between the start of fluidization and full fluidization that would occur in a dirty filter. Beverly (2011) indicates that this spike is highly variable and can be several times greater than that shown depending on chemical coagulation, solids loading, run time and head loss.

Figure 22 also shows the desirable head loss curve for the underdrain system in relation to media head loss, with a higher head loss through the underdrain system than through the media at the flow rate used for backwashing, thus ensuring uniform distribution of backwash water through all parts of the filter.

Granular activated carbon, with its substantial differing properties compared to conventional sand or anthracite, requires lower fluidization rates due to lower density which could result in less effective backwashing if surface wash or air scour is not implemented. Additionally, the higher UC results in a higher percentage of bed expansion with better stratification if the bed is backwashed with fluidization. (Edzwald, 2011).

Beverly (2011) explains that filter media height in the bed will vary during filtration and backwash. The media will contract during solids accumulation and be restored to installation height after backwash (provided no media has been lost).

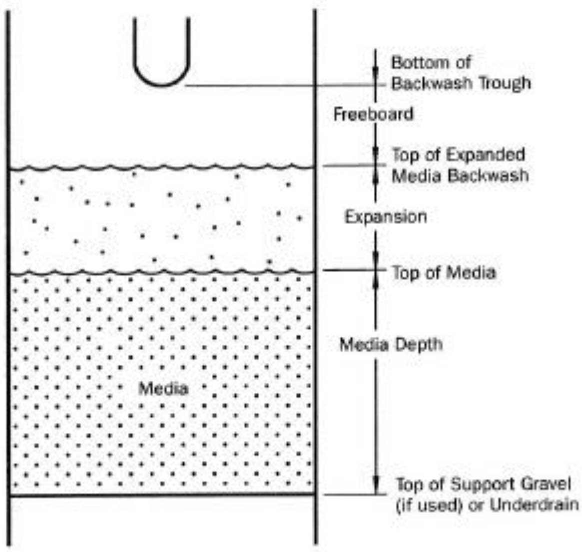


Figure 23 Picture Defining Expansion (Beverly, 2011)

The required bed expansion during backwash should be determined through pilot tests as it depends on the media ES, UC, specific gravity, grain shape and temperature (Beverly, 2011).

Media filters typically require 15 – 30% bed expansion for effective backwashing (Binnie, Kimber and Smethurst, 2002; Logsdon, 2008). Logsdon *et al.* (2002) and Edzwald (2011) refer to this same range for full fluidization. AWWA and American Society of Civil Engineers (2012) and Montgomery, James M. Consulting Engineers (1985) indicate a minimum of 20% bed expansion required for backwashing with water alone and an expansion of 20 – 50% for the high-rate backwash water following surface wash. Binnie, Kimber and Smethurst (2002) mentions that bed expansion for combined air scour and water is typically less than 5 %.

Beverly (2011) indicates 22 – 33% bed expansion for fluidizing media is common.

Kawamura (2000) targets the bed expansion rate under optimal backwash rate using:

$$Bed\ Expansion\ (optimal) = \frac{0.6 - \varepsilon_0}{0.4} \quad (Eq. 8.6)$$

Bed expansion can be determined experimentally by measuring bed depth at various upflow velocities.

Amirtharajah and Cleasby (1972) and Cleasby and Fan (1981), as cited in Hendricks (2010) provide a formula for predicting the expanded bed porosity utilising the Reynolds number and the Galileo number when velocity is greater than or equal to the minimum fluidization velocity with d_{60} used for the calculation:

$$\varepsilon^{4.7} Ga = 18Re + 2.7Re^{1.687} \quad (Eq. 8.7)$$

Edzwald (2011), however, predicts the expanded-bed porosity (ε) for any backwash rate (i.e any velocity) and any media size using the modified Reynolds number (Re_1) and a dimensionless porosity function for fluidized beds (A_1) as follows:

$$Re_1 = \frac{v_{bw}\rho_w}{\frac{6}{\varphi d_{eq}}(1-\varepsilon)\mu} \quad (Eq. 8.8)$$

$$A_1 = \frac{\varepsilon^3}{(1-\varepsilon)^2} \frac{\rho_w(\rho_m - \rho_w)g}{(\frac{6}{\varphi d_{eq}})^3 \mu^2} \quad (Eq. 8.9)$$

$$logA_1 = 0.56543 + 1.09348 logRe_1 + 0.17971(logRe_1)^2 - 0.0392(logRe_1)^4 - 1.5(log\varphi)^2 \quad (Eq. 8.10)$$

and solve for ε.

Van Duuren, South Africa Water Research Commission (1997) describes this as the most comprehensive model for bed expansion (and that illustrates good predictions for South African produced silica sand), but notes the limitations of its use for media with the following properties:

a) ε > 0.85 with Re < 100

b) ε > 0.9 with Re > 100

c) ρ_m > 4300 kg/m^3

Beverly (2011) assumes the optimum expansion when the d_{90} media grain size is agitated; therefore, the initial backwash rate for the calculation should be chosen based on the d_{90} media grain size.

Then, with a known initial bed porosity (ε_0),

$$\frac{L_e}{L} = \frac{1-\varepsilon_0}{1-\varepsilon} \quad (Eq. 8.11)$$

determines the ratio of expanded depth to the fixed bed depth and hence the depth of the expanded bed (L_e) (Cleasby and Fan (1981) as cited in Edzwald (2011), (Kawamura, 2000)).

Expansion of the entire bed is calculated by summing the expansion of at least five media sizes within the monomedia filter, and by summing the expansion of at least five media sizes within each of the different layers in dual- and mixed-media filters. It can be assumed for the calculation that each layer has invariant sphericity, initial porosity and media particle density (Edzwald, 2011).

The expanded bed depth is then used to calculate the bed expansion as follows:

$$Bed\ Expansion\ (\%) = \frac{L_E - L}{L} \times 100 \quad (Eq. 8.12)$$

or

$$\left(\frac{L_E}{L} - 1\right) \times 100 \quad (Eq. 8.13)\ (\text{Edzwald, 2011; Hendricks, 2010})$$

or alternatively written as

$$Bed\ Expansion\ (\%) = \frac{Height\ of\ Expansion}{Media\ Depth} \times 100 \quad (Eq. 8.14)\ (\text{Beverly, 2011})$$

On the other hand, Hendricks (2010) calculates bed expansion as follows:

$$Bed\ expansion = \frac{L_E - L}{L} = \frac{\varepsilon - \varepsilon_0}{1 - \varepsilon} \quad (Eq. 8.15)$$

If the bed expansion is lower than anticipated/desired, then the backwash rate should be increased to achieve this.

The head loss required to expand a bed is approximated by:

$$H_L = L_e(SG_m - 1)(1 - \varepsilon) \quad (Eq. 8.16)\ (\text{Kawamura, 2000})$$

8.3 Backwash Methods

8.3.1 General

Backwashing involves forcing water and more commonly a combination of water and air through the filter media.

Tobiason *et al.* (2011) as cited in AWWA and American Society of Civil Engineers (2012) and Logsdon (2008) describe the three backwash methods as follows:

- Method 1 – upflow water backwash only
- Method 2 – upflow water backwash with surface wash (auxiliary water scour)
- Method 3 – air scour assisted backwash
  - Air scour alone before water backwash; i.e. sequential air and water
  - Simultaneous air scour and water backwash
    - before backwash overflow
    - during overflow

Edzwald (2011) summarises various aspects related to the different backwash methods as shown in Table 17.

Table 17 Comparison of Different Backwash Methods (Edzwald, 2011)

Aspect	With Fluidization			Without Fluidization
	With surface wash	Without air scour	With air scour	With simultaneous air scour and water backwash
Wash Effectiveness	Fair	Weak	Fair	Good
Transport Particles to Overflow	Fair	Fair	Fair	Good
Fine Media Compatibility	Yes	Yes	Yes	No
Dual and Mixed Media Compatibility	Yes	Yes	Yes	No
Graded Support Gravel Compatibility	Yes	Yes	Yes	No
Media Loss to Overflow	Yes, mainly for coal	No	Yes, if not used properly	Substantial, if not used properly

The various methods are discussed in more detail hereunder.

8.3.2 Method 1 – Water Only

Backwashing with water alone, often referred to as the “American” method (Van Duuren, F. A., South Africa Water Research Commission., 1997) usually involves 3 – 15 minutes of a high backwash rate and lower backwash rate towards the end of the backwash cycle (AWWA and American Society of Civil Engineers, 2012). Van Duuren, South Africa Water Research Commission (1997) notes that this method is not commonly used in South Africa.

Media agitation was typically done with mechanical rakes which assisted washing, but this method was discontinued early in the application of rapid filtration (Logsdon, 2008).

8.3.3 Method 2 – Water Plus Surface Wash

Backwashing with water alone (method 1) is the least effective cleaning method and a surface wash was introduced to improve the efficiency and reduce mud ball formation by providing some scrubbing action (Logsdon, 2008; Montgomery, James M. Consulting Engineers, 1985). Surface wash also breaks up particles that have deposited on the media bed surface.

If surface wash is implemented, this is first operated alone for 1 – 3 minutes (AWWA and American Society of Civil Engineers, 2012; Montgomery, James M. Consulting Engineers, 1985). Simultaneous surface wash and backwash water follows for approximately 5 – 10 minutes after fluidizing the filter bed (AWWA and American Society of Civil Engineers, 2012; Crittenden *et al.*, 2012). Thereafter the higher backwash water rate continues for 2 – 3 minutes (AWWA and American Society of Civil Engineers, 2012; Edzwald, 2011). Beverly (2011) mentions 8 – 10 minutes for the higher backwash water rate. This duration of the high-rate backwash water can be determined visually or according to a low turbidity measurement (Beverly, 2011).

Careful consideration of the combined surface wash and backwash water is required to prevent media loss at excessive flows. The surface wash is stopped before backwash water is stopped to allow for the media to be hydraulically graded (Montgomery, James M. Consulting Engineers, 1985).

Surface wash can be provided using fixed or rotary systems with nozzles, suspended approximately 2.5 to 5 cm above the filter bed (AWWA and American Society of Civil Engineers, 2012; Edzwald, 2011). Kawamura (2000) suggests nozzle placement at 5 to 9 cm above the filter bed. Kawamura (1999) states that the surface wash systems are both effective to a bed depth of 1.2 m; however, Kawamura (2000) states that an additional air-scouring system should be provided for bed depths of more than 750 mm.

The advantages of surface wash systems in general are its relative simplicity and ease of operation and maintenance, with the units being easily accessible (Edzwald, 2011; Kawamura, 2000). Kawamura (1999) states that the fixed surface wash system is favoured because of no moving parts and effectiveness even once the filter bed has lost some of the surface media. Logsdon *et al.* (2002) mentions that the rotary systems may not always be effective at reaching the corners of a filter bay.

AWWA and American Society of Civil Engineers (2012) explains that surface wash may not be effective for mixed media filters where auxiliary scour for particle removal is needed predominantly at the media interfaces. Kawamura (2000) explains that surface wash agitates only the top 150 – 250 mm of the filter bed. Submerged “surface wash” systems can experience nozzle blockages by the media and are not common (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011).

Other disadvantages of surface wash systems include rotary systems getting stuck in position, not being able to reach mudballs within the filter bed, obstruction of the filter bed surface making maintenance and repair work more difficult and nozzle blockages (Edzwald, 2011).

Beverly (2011) recommends 3-monthly regular inspections of nozzles for wear and damage (e.g. vibration or nonuniform rotation); however, it may be possible to observe any issues if the water level in the filter is drawn down to the top of the filter bed during the backwash sequence (Logsdon, 2008).

8.3.4 Method 3 – Water Plus Air

In the past, backwash with air and water was considered too complex for small water treatment plants (Hofkes *et al.*, 1981); however, backwashing with water and air is common these days. The inclusion of air creates a more complex configuration with additional pipework, blowers and control requirements. Pressure and air relief must also be included for protection of the underdrain system. (AWWA and American Society of Civil Engineers, 2012).

A typical flow path for filter backwashing is depicted in Figure 24.

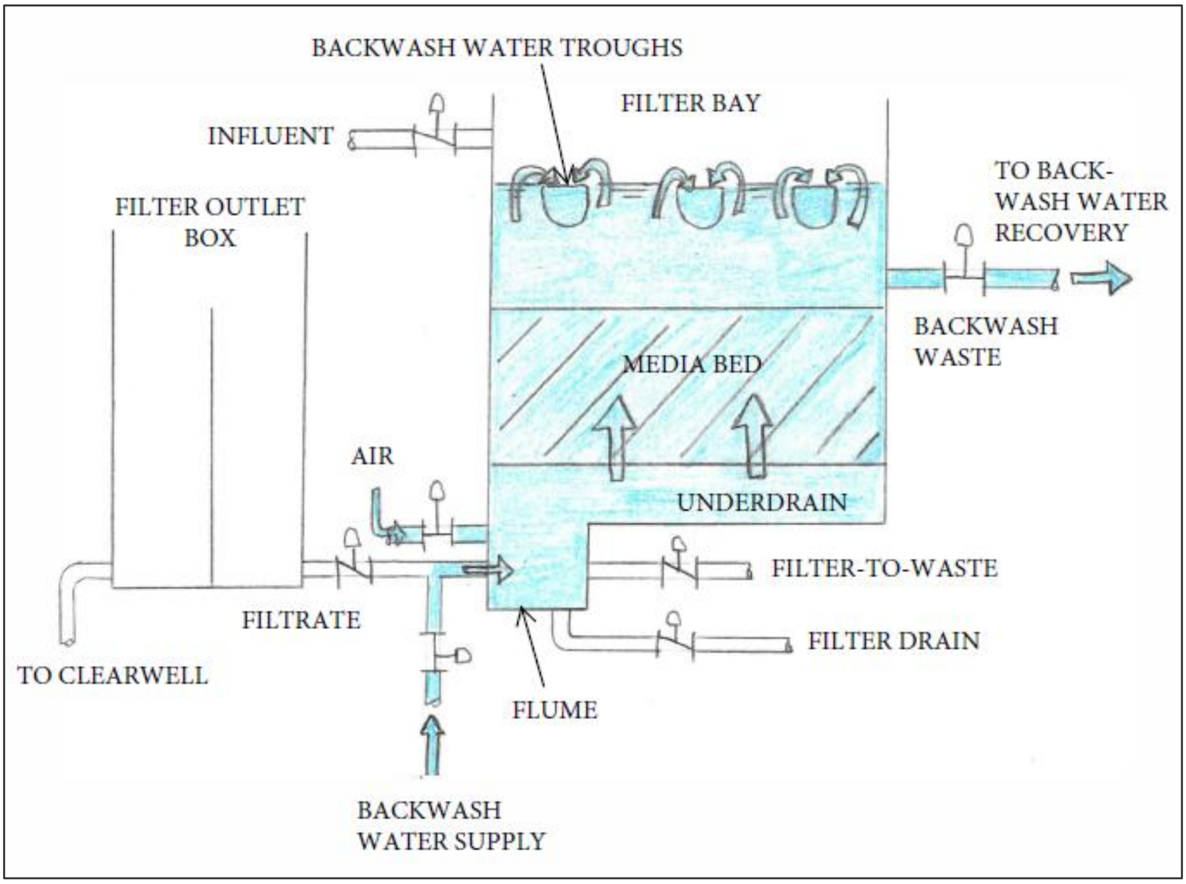


Figure 24 Typical Backwashing Flow Path (adapted from Beverly (2011))

The inclusion of air promotes abrasion; i.e. grain collisions, and high interstitial velocities resulting in effective cleaning and can significantly reduce backwash water quantities. Furthermore, AWWA and American Society of Civil Engineers (2012) explains the *collapse-pulse backwashing* phenomenon whereby backwashing is optimised by turbulence created by the growth and collapse of air bubbles pulsing through the filter bed at particular air scour and backwash water rates. Logsdon *et al.* (2002) and Amirtharajah (1993) describes collapse-pulsing as combining air scour with subfluidizing water rates. Hewitt and Amirtharajah (1984) as cited in Hendricks (2010) describes this phenomenon as air rising through the media in “air channels” that form “air pockets”. As more air flows into the pocket, further air channels and air pockets form above the original air pocket. The first air pocket collapses as more air flows into the pocket and sand is thrown up into it.

Kawamura (2000) recommends air scouring for deep bed filters with coarse media. Air scour is also highly recommended for dual adsorption filters (utilising GAC) to assist in bed cleaning (Beverly, 2011).

Air scour with water backwash should also be implemented when filter aid forms part of the pre-treatment system, to prevent mudballing (AWWA and American Society of Civil Engineers, 2012).

Air scour could occur initially on its own prior to backwash water being introduced i.e. sequential air and water commonly referred to as the “British” method (Van Duuren, F. A., South Africa Water Research Commission., 1997) or it could be simultaneous with backwash water and for the duration of the backwash cycle, commonly referred to as the “French” method (Van Duuren, F. A., South Africa Water Research Commission., 1997), both of which are currently used in South Africa (Van Duuren, F. A., South Africa Water Research Commission., 1997).

Sequential air and water backwash is common in Great Britain for monomedia sand filters with a media effective size of 0.6 – 1.2 mm (AWWA and American Society of Civil Engineers, 2012). Kawamura (2000) states that both are similarly effective, but that the sequential option is generally limited to monomedia filters with bed depths less than 1000 mm. Logsdon *et al.* (2002) notes that it has been shown that air scour prior to backwash water alone cannot prevent long-term accumulation on the media surface(s) and is considered to be no more effective than backwashing with water with a surface wash (Logsdon, 2008).

Simultaneous air scour and backwash water is the most effective backwashing method (Logsdon, 2008) and is necessary for cleaning deep bed filters (Crittenden *et al.*, 2012). Van Duuren, South Africa Water Research Commission (1997) describes a possible consecutive air and water cycling system (with the automated possibilities nowadays) for two to three times that closely approximates the efficiency of simultaneous air and water.

A sequential air scour followed by water is not recommended for dual parallel lateral underdrains when support gravel is provided due to air trying to escape the lateral and water trying to re-enter the lateral from above, which could ultimately result in peeling off top layers of support gravel (Beverly, 2011). Crittenden *et al.* (2012) notes that air should be supplied above the support gravel by means of a piped network. A media retention mechanism will not experience this problem with sequential air (Beverly, 2011).

Air operating on its own causes the suspended solids to move towards the bottom of the bed and hence this is generally allowed for a short period before introducing backwash water (AWWA and American Society of Civil Engineers, 2012).

Despite AWWA and American Society of Civil Engineers (2012) indicating air supply for 3 – 5 minutes before backwash water is applied, Beverly (2011) proposes that adequate air is provided in 1 minute. Logsdon *et al.* (2002) and Logsdon (2008) also suggests that air scour is applied for 1 – 2 minutes before backwash water is added.

Simultaneous air scour and backwash water is typically applied for 5 – 10 minutes followed by 5 – 10 minutes of backwash water alone (AWWA and American Society of Civil Engineers, 2012). Beverly (2011) proposes that one bed volume change during combined air and water backwash through the filter is sufficient to clean the filter. With the intention of using collapse-pulse backwashing, the air scour should stop with sufficient time for the higher water rate to remove air in the filter bed before the water exits the filter, to prevent media loss.

Introduction of air when the water level is just above the filter bed allows for air bubbles to grow to maximum size as they reach the surface “creating turbulent conditions and optimizing the grinding circulating action of the air scour” (AWWA and American Society of Civil Engineers, 2012).

Beverly (2011) and AWWA and American Society of Civil Engineers (2012) discuss the low-rate water wash with combined air scour to detach accumulated particles from the media, followed by a higher rate water wash (rinse) for flushing solids out and reclassifying/restratifying the media bed. Logsdon *et al.* (2002) reiterates that restratification of the media cannot be achieved during any operation of air scour or surface wash.

Air scour is generally stopped before backwash water overflow to troughs or channels to prevent media loss, unless the concurrent backwash water flow rate is low (Beverly, 2011).

8.4 Backwash water volume

Backwash water volume is a key parameter to ensure a clean filter. A high backwash rate for a short period is equivalent to a low backwash rate for a long period. (Hendricks, 2010).

Edzwald (2011) discusses the backwash water volume being dependant on the filter bed depth (typically four displacement volumes) plus the volume associated with the distance from filter bed surface to backwash overflow level. Hofkes *et al.* (1981) indicates typical volume of ~ 3 – 6 m^3 per m^2 of filter bed area, however, Edzwald (2011) notes that typical backwash volumes are 4 – 8 m^3/m^2 of filter area/wash up to 12 m^3/m^2/wash for deep filters or very large freeboards.

Hendricks (2010) recommends that backwash turbidity versus backwash water volume per unit area of filter bed is plotted for a water treatment plant, and done so for various raw water and pre-treatment conditions, to establish the maximum backwash volume after which there is no effect on the backwash water quality.

8.5 Backwash water source

Filtered water is typically used to backwash filters, and typically has a chlorine residual (Logsdon *et al.*, 2002). Development of bacteria and algal growth within the filter bed can be slowed down if a chlorine residual is maintained in the filter influent. Some control is possible by providing chlorine to the filter bed intermittently; for example, into the backwash water supply (Montgomery, James M. Consulting Engineers, 1985). AWWA and American Society of Civil Engineers (2012) notes that chlorinated backwash water supply will reduce the life of a GAC filter, making it more brittle due to chemical oxidation of some of the GAC.

Where biological treatment is incorporated into the filter design, it is common to use unchlorinated water for backwash. If chlorinated water is used, short backwash periods are preferred to maintain suitable contaminant-removal performance. (Urfer et al, 1997)

Edzwald (2011) discusses the impact of differences in water chemistry for the filter influent, filter effluent (i.e. filtrate) and backwash water, particularly where the backwash water source contains chemicals for disinfection (e.g. water containing free chlorine) and corrosion control (e.g. acids or bases for pH correction). Any negative effect would depend on the relative differences in water chemistry and volumes of backwash water compared to filter influent. The impacts could be reduced by implementing filter-to-waste as described in Chapter 8.10.

AWWA and American Society of Civil Engineers (2012) lists the following backwash water source options:

- Draw-off from a high-pressure discharge for:
  - Direct filter backwash
  - Filling an elevated water storage tank and subsequent gravity backwash
- Gravity backwash from an elevated final water storage tank
- Pumped from a sump or lower level clearwell

The advantages, disadvantages and design considerations for these backwash water sources are shown in Table 18.

Table 18 Advantages and Disadvantages of Various Backwash Water Sources (AWWA and American Society of Civil Engineers, 2012)

Backwash Water Source	Advantages	Disadvantages	Design Considerations
Direct high-pressure discharge draw-off	• Eliminates dedicated backwash pumps • Eliminates dedicated elevated tank	• Requires pressure reduction to prevent loss of media; cavitation is likely • Difficult backwash flow control	• Size backwash water pipe to restrict maximum flow to prevent excessive backwash rates and potential underdrain uplift
Elevated storage tank	• Reduces pumping capacity (smaller filling pump versus large backwash pump) • Economical for small plants (Binnie, Kimber and Smethurst, 2002)	• Complex control • Large plants require high flow rates for backwash and hence large diameter pipes (Binnie, Kimber and Smethurst, 2002) • Flow rate can decline as elevated tank level drops (Crittenden *et al.*, 2012)	• Ensure volume sufficient for maximum backwash rate for maximum time; typically for two or more filters (Binnie, Kimber and Smethurst, 2002; Kawamura, 2000). • Ensure filling rate is sufficient to fill tank during minimum time between backwashes (AWWA and American Society of Civil Engineers, 2012). Hofkes *et al.* (1981) approximates the pumped filling rate of 10 – 20% of the backwash water rate. • Top water level of tank must be sufficiently higher than pressure required for filter backwashing, typically a minimum of 9 m above the filter bed level (Binnie, Kimber and Smethurst, 2002). Hofkes *et al.* (1981) indicates typical placement ~ 4 – 6 m above the filter water level and Kawamura (2000) suggests 11-12 m between the lowest water level in the tank and the top of the backwash troughs. • Prevent formation of vortex at the backwash main pipe exit (Kawamura, 2000)
Pumped	• Better backwash flow control • Redundancy possible in terms of duty, standby pumps	• Pumps are required (Crittenden *et al.*, 2012)	• Pressure-relief vent pipe in pump delivery pipeline for underdrain protection (Kawamura, 2000)

Self-backwashing filters involving hydraulic backwash with filtrate based on filter outlet channel levels are discouraged (AWWA and American Society of Civil Engineers, 2012). Kawamura (1999) notes that although self-backwashing filters do not require backwash pumps, valves nor a storage tank, there is no filter-to waste and backwash rates cannot easily be adjusted and the filter is typically 1.2 m deeper than normal filters to accommodate the self-backwashing function. Hendricks (2010) also makes note that a reservoir between the filters and the clearwell is required for the self-backwashing function and that flow through the filters into this reservoir needs to be more than the backwash flow during backwash.

Loss of backwash water flow rate control can lead to costly failure, particularly if the filter floor is damaged. Other negative effects are loss of media or disruption of media layers.

Over-pressurization can be avoided by simply installing a suitably sized vent pipe without isolation extending from the backwash water pipe to a level above the top of the filter bay (AWWA and American Society of Civil Engineers, 2012). The vent pipe could discharge into a filter, the inlet channel or the backwash outlet channel depending on the filter configuration. Kawamura (2000) recommends a pipe of not less than 300 mm diameter extending to 1800 mm above the top of the filter bay structure or 600 – 900 mm above the hydraulic gradeline during backwashing.

Montgomery, James M. Consulting Engineers (1985) notes that most filters only require 1.2 – 2 m of static head at the filter bottom; but elevated tanks typically provide more than 10 m of static head above the backwash trough to maintain a constant backwash rate during the backwash cycle. The additional head required is dissipated through the pipes and valves and therefore backwash systems with little to no valves such as the self-backwashing filters experience significant backwash rate changes with a slight change to the available static head.

8.6 Air Scour

The media bounces at the start of an air scour, irrespective of fluidizing or nonfluidizing media (Beverly, 2011). Support gravel (if used) must therefore be very stable, or air should “be introduced through a piping system located just above the support gravel to prevent dislodging the gravel” (Crittenden *et al.*, 2012).

Thereafter, fluidizing media rolls and is mixed and expands to the water surface, and the air increasing the water surface by as much as 10%. After 30 to 45 seconds, this rolling and mixing stops and compaction of the media begins. As shown in Figure 25, some media is still fluidized to the water surface but most of the media compacts with channels of air up through the media. “Pulse-collapse mode” with air alternately pulsing upward and media collapses slightly before the next bubble passes; simultaneously slowly recirculating the media. Combined water and air will cause the rolling and mixing again. (Beverly, 2011).

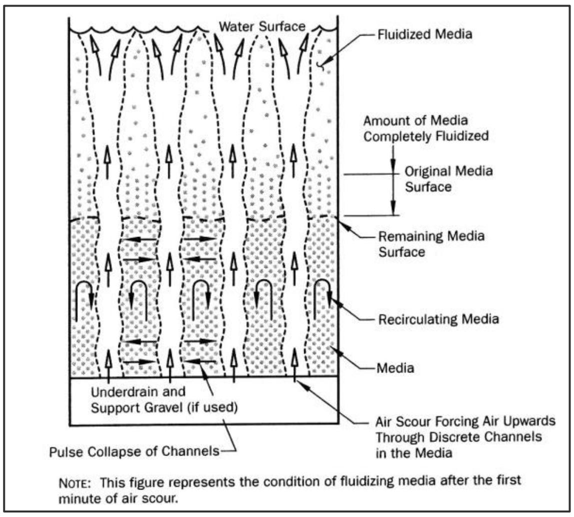


Figure 25 Pulse-collapse Mode during Air Scour (Beverly, 2011)

8.7 Backwash Initiation

Backwashing is initiated, ending a filter run, by either of the following conditions:

- turbidity breakthrough (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011; Kawamura, 2000; Van Duuren, F. A., South Africa Water Research Commission., 1997)
- head loss development to terminal head loss (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011; Kawamura, 2000; Van Duuren, F. A., South Africa Water Research Commission., 1997) – see also Chapter 6.1 for discussion on head loss allowances.
- timer to prevent exceptionally long filter runs (Beverly, 2011; Binnie, Kimber and Smethurst, 2002; Kawamura, 2000; Montgomery, James M. Consulting Engineers, 1985; Van Duuren, F. A., South Africa Water Research Commission., 1997) – see Chapter 8.1.

For backwash initiation, Logsdon (2008) suggests that filtrate turbidity is monitored for all filters and flow rate through the filters for declining rate filters and head loss across equal rate filters.

For downstream filter control methods, backwashing is typically initiated when the filter outlet valve is fully open. For upstream filter control methods, backwashing is necessary when the water level in the filter rises to the distribution channel/point to the filter. For the declining rate control method, a set high water level in the filter typically indicates the need for backwashing (Van Duuren, F. A., South Africa Water Research Commission., 1997).

In the past, backwash equipment was manually operated at the equipment to facilitate the backwash sequence. This could be difficult if an idle filter(s) is to be started up simultaneously when another filter is backwashed (Logsdon *et al.*, 2002). Today, local filter control consoles; i.e. positioned at each filter, allow the operator to operate the equipment whilst viewing the backwash sequence. Remote operation is also possible from a central filter control console (AWWA and American Society of Civil Engineers, 2012). The frequency of backwashes could inform the number of operators required for the water treatment plant (Crittenden *et al.*, 2012).

Also, today, semiautomatic and automatic backwashing is possible. Semiautomatic operation requires the operator to initiate the backwash cycle, after which the backwash sequence occurs without operator input. Automatic backwashing does not require any operator input. Nevertheless, easy modification to the backwash sequence such as the rates of backwash water and air scour should be possible. Automatic backwashing can be initiated by measuring turbidity and head loss development or by a timer for a set maximum filter run time. (AWWA and American Society of Civil Engineers, 2012).

It is essential that local control is provided irrespective of the backwash operational modes discussed above so that maintenance and troubleshooting is possible. Also, filters should be observed during the backwash sequence to identify any problems, which may include mudballing, cracking, jetting, a broken underdrain system or incomplete backwash cycles. Remote viewing using closed-circuit television with appropriate lighting is possible. (AWWA and American Society of Civil Engineers, 2012).

8.8 Backwash Sequence

A typical backwash sequence adapted from AWWA and American Society of Civil Engineers (2012) is described below, as follows:

1) Filter inlet closes (valve or gate)
2) Filter outlet valve closes when the level in filter has dropped to a preset level above the top of the filter bed. This could be just below the level of the backwash outlet weir or trough (AWWA and American Society of Civil Engineers, 2012; Binnie, Kimber and Smethurst, 2002; Edzwald, 2011) or lower still to maximise filtrate production. Beverly (2011) proposes draining to a minimum of 150 mm below the top of the backwash outlet weir or trough to reduce media loss but to drain to 150 mm above the media surface when air scour is used (Crittenden *et al.*, 2012; Logsdon, 2008). AWWA and American Society of Civil Engineers (2012) also notes to drain the water to just above the top of the filter bed when backwashing with air. Logsdon (2008) explains this drawdown to above the media surface is to accelerate the removal of any air trapped within the media, which would

otherwise pose a risk of the media being carried to the surface by the air bubbles and possibly removed from the filter.

3) Waste backwash water outlet opens (valve or gate)

4) Depending on the backwash method:

   a) Surface wash water inlet valve opens; or

   b) Air inlet valve opens and backwash blower starts. Note that if purging is required prior to air scour (i.e. if the air pipework and filter configuration is such that water is present in the air pipework), then the following procedure follows (Beverly, 2011):

      i) Open air inlet valve

      ii) (note that the vent valve should be open) Close/"modulate" vent valve to purge position

      iii) Start blower

      iv) Wait for air to appear in the filter

      v) Modulate vent valve to position (if not to the closed position) to allow required air scour rate through filter media

5) After a set period, backwash water is added with the backwash water inlet valve opening slowly to avoid disturbing the support gravel and underdrain and to purge air out of the bed at the beginning of the washing (Kawamura, 2000). For the set period, Beverly (2011) suggests backwash water is added after 1 minute of surface wash to prevent media loss. For backwashes incorporating air scour, Logsdon (2008) and Edzwald (2011) suggest that backwash water is added after 1-2 minutes of air scour, whereas Van Duuren, South Africa Water Research Commission (1997) suggests after 2-3 minutes of consecutive air and water.
   Starting a backwash pump against a closed valve is recommended by Beverly (2011) to eliminate air as discussed in Chapter 15.4. Logsdon (2008) states that a slow start is safer than an abrupt high rate of upward flow applied to the filter. Logsdon *et al.* (2002) and Logsdon (2008) and Edzwald (2011) mention that gradual backwash water addition should occur over at least 30 seconds to protect the underdrain and support gravel and prevent media loss.

6) After a set period, the respective surface wash or air scour is stopped and the respective surface wash water inlet valve or air inlet valve closes. Logsdon (2008) suggests turning surface wash off at 2-3 minutes before fluidization ends to ensure restratification can take place. Rather than stopping air scour in accordance with a timer, Logsdon *et al.* (2002) and Logsdon (2008) suggest that air scour is stopped when the water level in the filter reaches a distance 150 mm below the backwash outlet weir or trough. However, for coarse uniform monomedia with ES ≥ 1.0 mm, Logsdon *et al.* (2002) and Edzwald (2011) suggest that air is only stopped after 10 minutes of combined washing, irrespective of overflowing. Note that if air pipework purging forms part of the process (see paragraph 4b above), then the vent valve should be opened before stopping air scour (Beverly, 2011).

7) Backwash water rate is increased (backwash water inlet valve opens further, or backwash pump speed increases). Beverly (2011) additionally proposes that the backwash rate increase occurs when the water level is below the backwash outlet weir or trough to allow any further air in the filter system to escape without losing media; Hendricks (2010) suggests at least 150 mm below the trough/weir.

8) Waste backwash water outlet closes (valve or gate)

9) Backwash water inlet valve closes and backwash pump stopped, if used. Edzwald (2011) indicates that the water is stopped when the backwash waste water is sufficiently clear at approximately 10 ntu. Edzwald (2011) also notes that the rate of stopping the backwash water is less important than starting because there is "no danger" to the underdrain system or support gravel; however, a slower stop will increase the level of restratification (considered to be less important for monomedia filters). Hendricks (2010) suggests a 30 – 60 seconds stopping period.

10) Filter inlet slowly opens (valve or gate) – note that Beverly (2011) recommends that the filter influent be reinstated with a relatively high filter water level present, particularly to reduce the negative effects of media surface disturbances discussed in Chapter 15.8. Hendricks (2010) suggests that this water level is 450 mm above the media.
11) Filter-to-waste valve slowly opens
12) After a set period, or when filtrate quality is acceptable, filter-to-waste valve closes
13) Filter outlet valve slowly opens

8.9 Backwash Rates

8.9.1 General

Backwash rates depend on the filter media, backwash method as well as the backwash water temperature (AWWA and American Society of Civil Engineers, 2012).

As is shown in Figure 26, a higher backwash water temperature requires a higher backwash rate; i.e. fluidization velocity, to achieve the required bed expansion. This is due to a decrease in water viscosity with an increase in temperature, and the resultant reduced drag forces on the media grains. (AWWA and American Society of Civil Engineers, 2012).

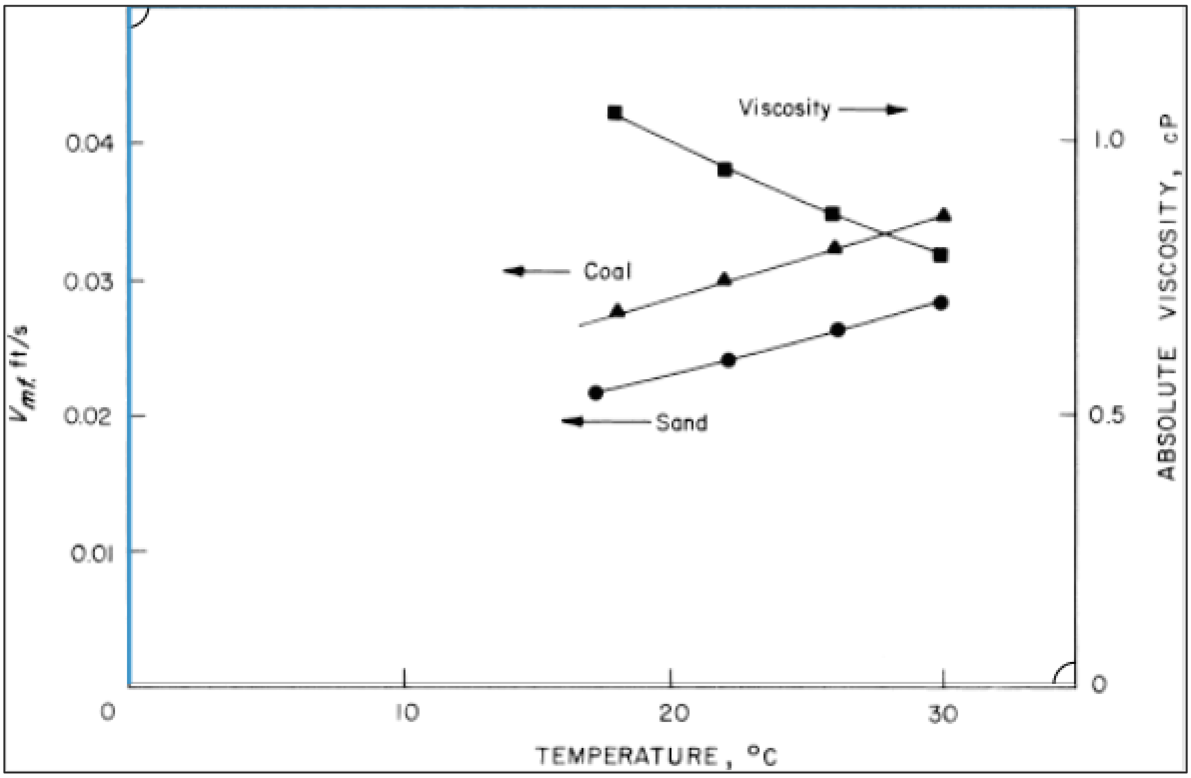


Figure 26 Effect of Backwash Water Temperature on Backwash Water Viscosity and Minimum Fluidization Velocity, V_{mf} (AWWA and American Society of Civil Engineers, 2012)

Kawamura (1999) provides the following equation for adjustment of the backwash rate for the backwash water temperature:

$$v_{bw}(t) = v_{bw}(20°) \times \mu(t) \times t^{\frac{-t}{3}} \quad (Eq. 8.17)$$

Where

$v_{bw}(t)$ = backwash rate at temperature (t in degree Celsius)

$v_{bw}(20°)$ = backwash rate at 20°C

μ(t) = water dynamic viscosity in centipoise at temperature (t)

Media loss could occur if the system is designed for the maximum temperature and the same rates are used for colder temperatures. Conversely, mudballing could occur if the system is designed for a low temperature and the same rates are used for higher temperatures. Therefore, it is common for the rates to vary seasonally and sometimes during the backwash sequence. (AWWA and American Society of Civil Engineers, 2012). It is strongly recommended that it be possible to easily adjust the backwash rates (Beverly, 2011; Binnie, Kimber and Smethurst, 2002).

Backwash rates also need to increase with increased media grain size and density (AWWA and American Society of Civil Engineers, 2012; Logsdon *et al.*, 2002).

Media suppliers can provide backwash rate curves for different media (AWWA and American Society of Civil Engineers, 2012). The rates are typically based on the following:

- experience with the various backwash methods, similar source waters and installations
- regulatory guidelines
- media fluidization and bed expansion calculations
- cost analyses
- pilot studies
- media suppliers and underdrain manufacturers

Figure 27 provides a guideline for the backwash rate for specific filter media for a water temperature of 20°C, and is applicable to filter beds that are fluidized for restratification (i.e. dual and mixed media filters), but is not applicable to filters implementing sequential air scour and backwash water (Kawamura, 2000).

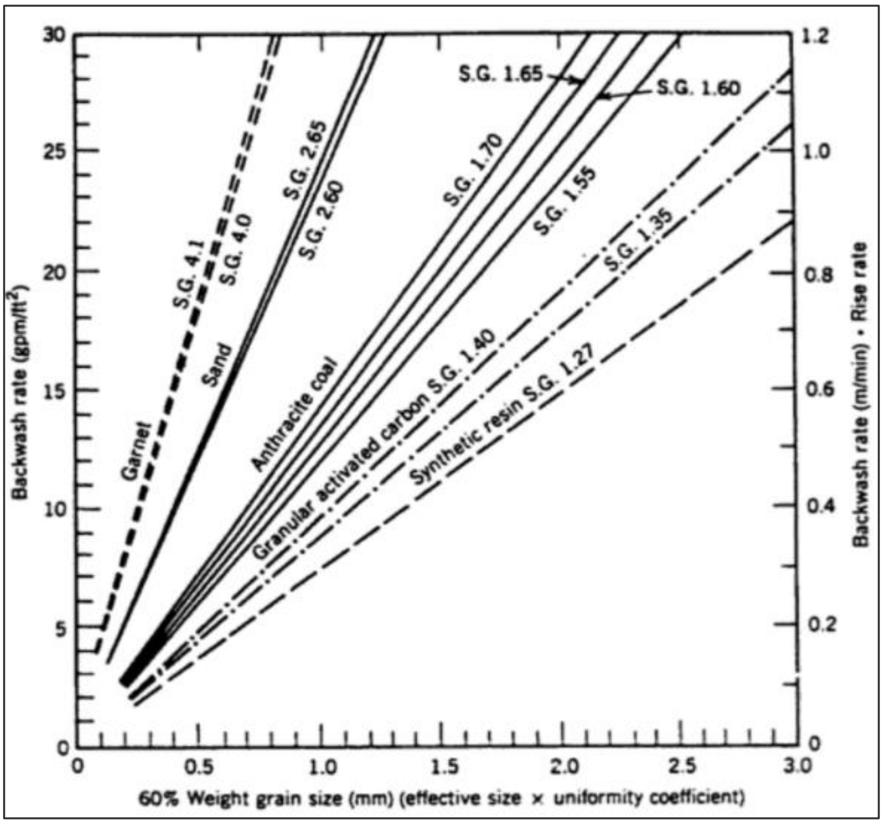


Figure 27 Selection of Backwash Rates for various Media at Water Temperature of 20°C (Kawamura, 2000)

Even if full fluidization is not required, the backwash water rate must be capable of washing particles out of the bed and filter bay (Logsdon *et al.*, 2002).

Typical backwash rates range from 37 to 56 m/h (AWWA and American Society of Civil Engineers, 2012) with lower rates required at lower temperatures and if GAC is used, which typically consists of a ramped backwash rate from 8 – 10 m/h to 34 – 37 m/h. AWWA and American Society of Civil Engineers (2012) and Beverly (2011) mention that backwash rates of 24 – 34 m/h are typical for GAC. Beverly (2011) mentions a typical 12 m/h for fluidizing media during concurrent air scour and 15 – 30 m/h for coarser media that does not receive a higher wash/rinse rate.

Further details on backwash rates for the various backwash methods are discussed hereunder.

8.9.2 Method 1 – Water Only

Backwashing with water alone requires the highest backwash rates, typically ranging from 37 to 56 m/h, followed by a lower backwash rate towards the end of the backwash cycle to flush any particles still within the filter bed (AWWA and American Society of Civil Engineers, 2012).

8.9.3 Method 2 – Water Plus Surface Wash

Logsdon (2008) indicates a surface wash of 1.2 – 2.4 m/h for rotary systems. AWWA and American Society of Civil Engineers (2012) describes single-arm rotary systems operating at 1.2 – 1.7 m/h (even up to 2.4 m/h (Edzwald, 2011)). Kawamura (2000) suggests 1.25 – 1.75 m/h for single arm and 3.25 – 3.75 m/h for dual arm rotary systems.

AWWA and American Society of Civil Engineers (2012) and Edzwald (2011) describe fixed systems delivering 5 – 10 m/h, whereas Kawamura (2000) suggests 7.5 to 10 m/h for these systems. Montgomery, James M. Consulting Engineers (1985) prescribes the optimum surface wash to include surface wash jets directed approximately 64 degrees from vertical at approximately 5 – 7 m/h.

The backwash water rate is usually 37 – 56 m/h (AWWA and American Society of Civil Engineers, 2012). Kawamura (2000) suggests 37.5 to 45 m/h for rapid monomedia, 45 to 55 m/h for dual- and mixed media filters and 30 to 40 m/h for GAC filter beds

8.9.4 Method 3 – Water Plus Air

8.9.4.1 General

Binnie, Kimber and Smethurst (2002) indicates that typical rates for air scour ranges from 60 – 90 m/h and backwash water rates of 10 – 30 m/h.

Air rates for sequential air and water backwashing is typically 18 – 36 m/h (AWWA and American Society of Civil Engineers, 2012). For monomedia filters, the subsequent application of backwash water is at a low rate where bed expansion and stratification are not achieved, typically of 12 – 18 m/h (AWWA and American Society of Civil Engineers, 2012), but could be up to 30 m/h just above the point that fluidization would start (Van Duuren, F. A., South Africa Water Research Commission., 1997).

Sequential air and water backwashing of multimedia filters has been used in the United States with air scour at 36 – 90 m/h followed by a higher backwash water rate to ensure restratification, typically of 37 – 56 m/h (AWWA and American Society of Civil Engineers, 2012).

Simultaneous air scour and water backwash is generally limited to deep, coarse monomedia filters, with air scour and backwash rates for various media sizes typically as shown in Table 19. The backwash water rate operating after air scour has stopped (i.e. rinse rate) is typically one to two times that applied during air scour (AWWA and American Society of Civil Engineers, 2012). Kawmura (1999) mentions that a rinse rate of ~45 m/h is necessary to purge air in coarse media beds and to restratify dual-media beds.

Table 19 Typical Backwash Rates for Simultaneous Air Scour and Backwash Water (AWWA and American Society of Civil Engineers, 2012)

Backwash Method	Media Type	Effective Size (mm)	Air Scour Rate (m/h)	Simultaneous Backwash Water Rate (m/h)	Backwash Water Rinse Rate (m/h)	Reference
Simultaneous air scour and backwash water followed by backwash water alone	Monomedia	1 – 2	36 – 72	15.4	~ 15.4 – 30.8	(AWWA and American Society of Civil Engineers, 2012)
		2 – 6	108 – 144	15.4 – 18.3	~ 15.4 – 36.6	(AWWA and American Society of Civil Engineers, 2012)
	-	-	55 - 73	20 – 25	40 – 60	(Kawamura, 2000)
	-	>0.9	50 – 60	10 – 15	Not indicated	(Van Duuren, F. A., South Africa Water Research Commission., 1997)

8.9.4.2 Collapse-pulse Backwashing

Logsdon *et al.* (2002) reports that collapse-pulse backwashing should incorporate air scour at 30 – 135 m/h. Backwash water rates should be 40 – 60% and 25 – 45% of V_{mf} for the d_{90} media grain size at the lower and upper ranges of air flow rates, respectively.

Logsdon *et al.* (2002) indicates the low-rate water wash (during simultaneous air scour) should be applied at half of the minimum fluidization velocity and the full rise rate is applied to rinse the bed (Logsdon, 2008). Edzwald (2011) also indicates that for low backwash rate and for backwashing without fluidization, the backwash water rate is typically below half the minimum fluidization velocity.

For the higher rinse backwash rate it is normally increased to fluidization velocity (Crittenden *et al.*, 2012; Edzwald, 2011). Edzwald (2011) notes that an increased rate would still typically be utilized when backwashing without fluidization to accelerate the backwash process, but to a rate below fluidization velocity.

Figure 28 depicts the recommended air flow rates and backwash rates (as percentage V/V_{mf}) for single and dual media filters to ensure collapse-pulsing.

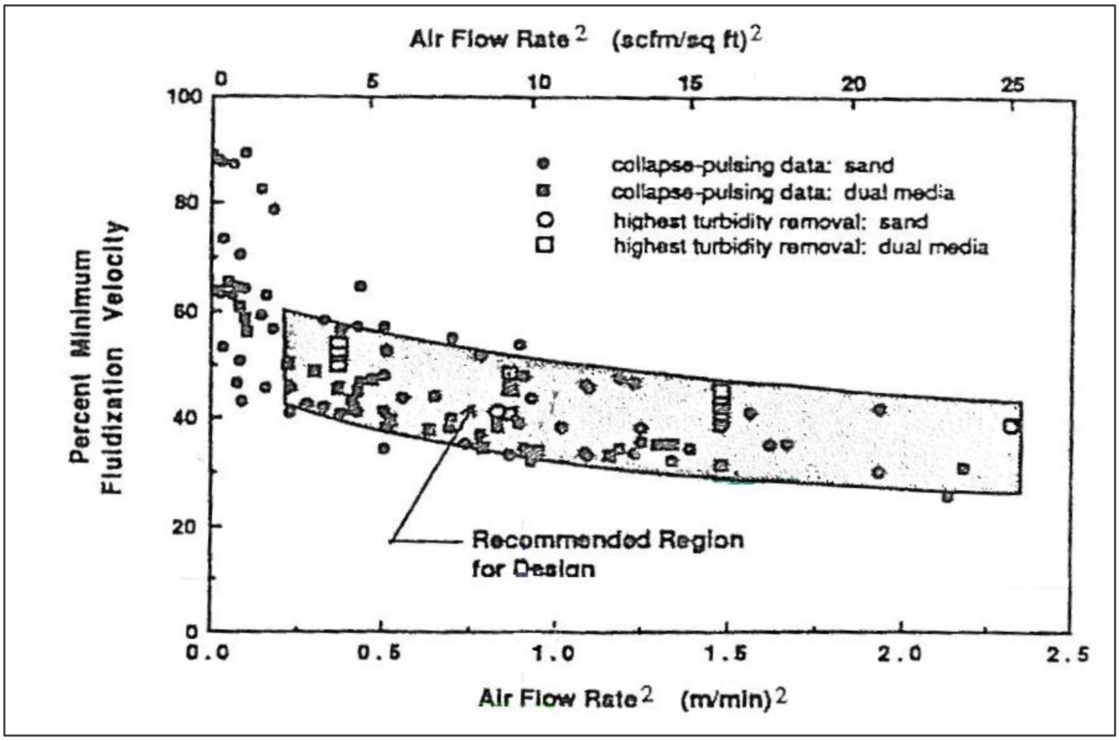


Figure 28 Recommended Combination of Air Flow Rates and Backwash Rates for Sand and Dual Media Filters to Ensure Collapse-Pulse Backwashing (Amirtharajah, 1993)

Table 20 provides equations for collapse-pulsing in the form $aQ_a^2 + (\%V/V_{mf}) = b$, which can be utilized for filter beds with media of similar size and density (Amirtharajah, 1993).

Table 20 Example Equations for Collapse-Pulsing (Amirtharajah, 1993)

Equation	Applicable Range of Q_a (m/min)	(m/h)	Media Type	ES (mm)	d_{90} (mm)	UC	Density (kg/m^3)
$8.5\ Q_a^2 + (\%V/V_{mf}) = 43.5$ (*Eq.* 8.18)	0.5 – 1.4	30 – 84	Sand	0.38	0.69	1.53	2650
$17.8\ Q_a^2 + (\%V/Vmf) = 43.0$ (*Eq.* 8.19)	0.4 – 1.3	24 – 78	Anthracite	1.10	1.99	1.55	1700
$18.5\ Q_a^2 + (\%V/V_{mf}) = 39.5$ (*Eq.* 8.20)	0.2 – 0.7	12 – 42	Sand and Anthracite	As above	As above	As above	As above
$35.2\ Q_a^2 + (\%V/Vmf) = 26.6$ (*Eq.* 8.21)	<0.8	<48	GAC	0.68	1.48	1.7	1840

Tobisaon *et al.* (2011) as cited in AWWA and American Society of Civil Engineers (2012) indicated that the equation predicted higher water rates than experienced at operating plants and advised on using the values as per Table 21 until further research suggests otherwise.

Table 21 Suggested Backwash Water and Air Scour Rates for Method 3 – Backwash With Air Scour (AWWA and American Society of Civil Engineers, 2012; Edzwald, 2011; Logsdon, 2008)

Media	Backwash mode	Air scour rate (m/h)	Backwash water rate (m/h)
Fine sand ES ~ 0.5 mm	Air	37 – 55	
	Backwash water		37
Coarse sand ES ~ 1.0 mm	Air and backwash water	55 – 73	15 – 17
	Rinse backwash water		15 – 17 or 30 – 34

Media	Backwash mode	Air scour rate (m/h)	Backwash water rate (m/h)
Coarse sand ES ~ 2.0 mm	Air and backwash water	110 – 146	24 – 29
	Rinse backwash water		24 – 29 or 48 – 58
Coarse Anthracite ES ~ 1.5 mm	Air and backwash water	55 – 91	20 – 24
	Rinse backwash water		20 – 24 or 40 – 48
Fine anthracite (dual-media and mixed-media top layer) ES ~ 1.0 mm	Air	55 – 73	
	Backwash water		37 – 49
Coarse anthracite (dual-media and mixed-media top layer) ES ~ 1.5 mm	Air	73 – 91	
	Air and backwash water	73 – 91	24
	Rinse backwash water		61

8.10 Filter Ripening and Filter-to-Waste

Filter ripening is the phenomenon occurring after backwashing whereby the filter media is conditioned for particle capturing, becoming more efficient with time. This can be monitored with filtrate turbidity measurements. Figure 29 depicts a typical filtrate turbidity profile.

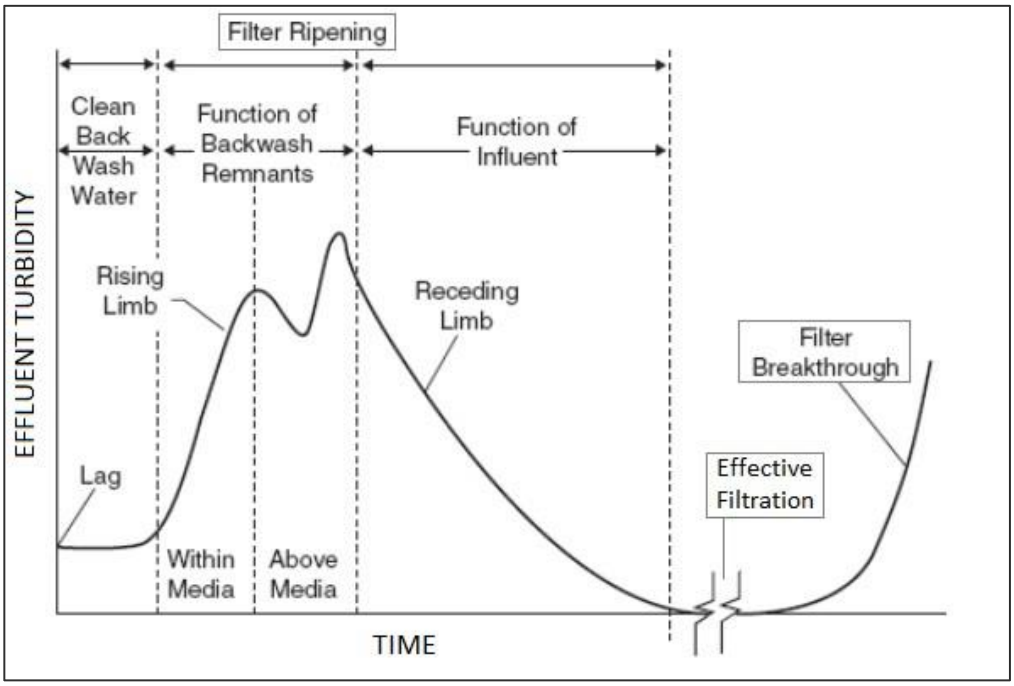


Figure 29 Filtrate Turbidity Profile (adapted from Amirtharajah and Wetstein (1980) as cited in AWWA and American Society of Civil Engineers (2012) and Amburgey (2005)

The lag phase is due to filtered water in the underdrain system at the end of backwashing. Turbidity then increases once filtration starts and typically contains the two distinct peaks as shown in Figure 29. The first peak relates to backwash water that is still within the bed and the second relates to particles in the backwash water above the filter media after backwash, both of which are then removed through the filtration process. Hendricks (2010) indicates that the two peaks are usually experienced within 5 to 15 minutes.

As filtration continues, the filter media slowly resettles with captured particles becoming collectors of other particles, improving filtration (Amburgey, 2005). During this filter ripening process, various particles and pathogens could escape through the bed into the filtrate.

After ripening, turbidity settles at a constant low turbidity until breakthrough occurs with a spike in turbidity indicating that a backwash is required (unless terminal head loss or a timer already initiates a backwash) (AWWA and American Society of Civil Engineers, 2012). Hendricks (2010) indicates that design should allow for breakthrough to preferably occur first, which may be achieved by including additional head loss in the design in setting the overall filter depth.

AWWA and American Society of Civil Engineers (2012) notes that filter ripening can occur for 5 to 30 minutes, whereas Binnie, Kimber and Smethurst (2002) suggests this to occur for approximately 90 minutes and Crittenden *et al.* (2012) notes that ripening can last for as long as 2 hours.

Various methods are available to reduce the spikes in turbidity after backwash, such as:

- diverting the filtered water to waste (filter-to-waste as shown in) for a short period after backwash is recommended (AWWA and American Society of Civil Engineers, 2012; Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012; Edzwald, 2011; Kawamura, 2000; Logsdon *et al.*, 2002); typically diverted to the inlet to the water treatment plant (Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012; Kawamura, 2000) to blend with incoming water and re-coagulate; i.e. not to other filters or inlet to the filters; and is also considered wasteful to blend with waste backwash water, diluting it but not guaranteeing that it will be recycled to the inlet to the water treatment plant (Logsdon *et al.*, 2002). Beverly (2011) indicates that 3 – 5 minutes is typical for filter-to-waste, but could be 30 – 60 minutes (Gray, 2010). Amburgey (2005) explains this to be effective, although wasteful, and could take several hours for the turbidity to reduce to an acceptable value.

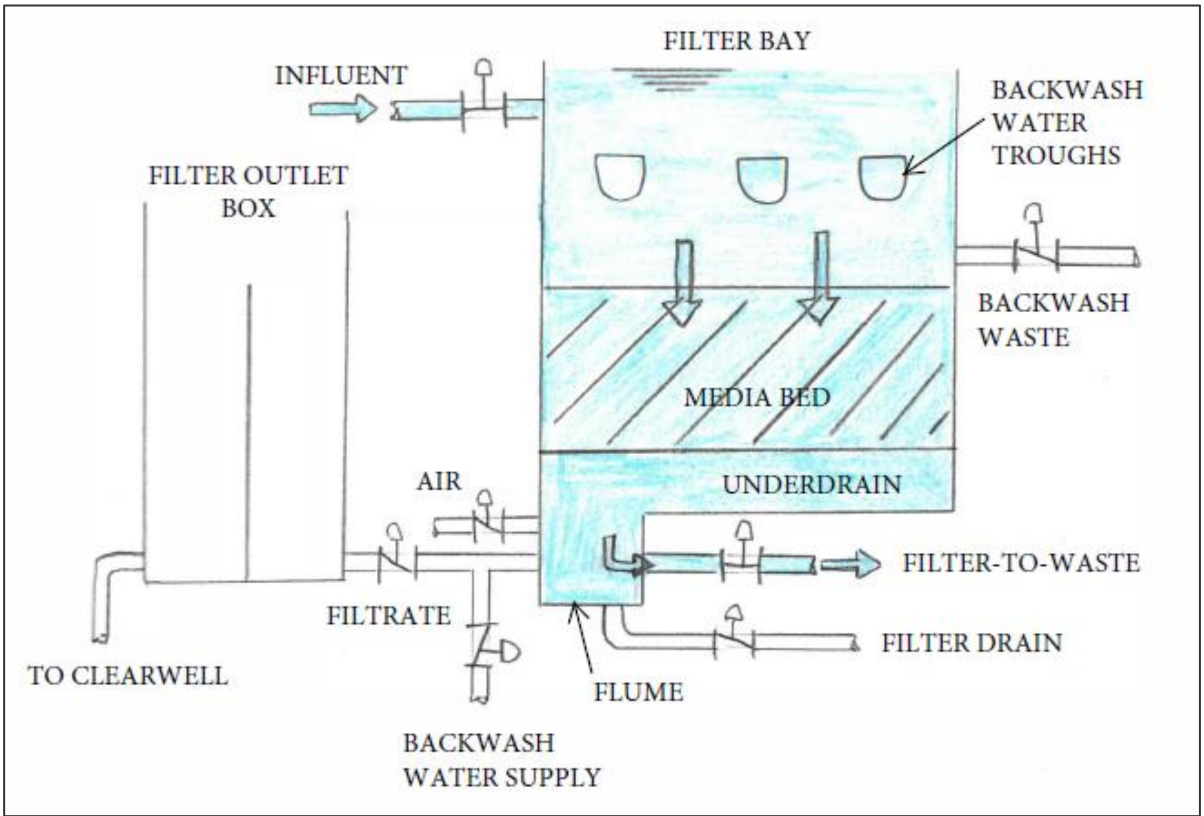


Figure 30 Typical Filter-to-Waste Flow Path (adapted from Beverly (2011))

- slowly increasing the filtration rate after backwash (Amburgey, 2005; AWWA and American Society of Civil Engineers, 2012; Binnie, Kimber and Smethurst, 2002; Kawamura, 2000) to allow filter media to settle into position and floc to settle and/or reattach to filter media (Logsdon *et al.*, 2002). This can be achieved by filling the filter with water before slowly starting to open the filter outlet valve.

- delayed filter start/keeping the filter out of service for a short period (Amburgey, 2005; AWWA and American Society of Civil Engineers, 2012) for 20 minutes to 24 hours to 48 hours (Logsdon *et al.*, 2002).
- adding a coagulant/filter aid polymer to the backwash water supply (Amburgey, 2005; AWWA and American Society of Civil Engineers, 2012; Crittenden *et al.*, 2012; Logsdon *et al.*, 2002), but Kawamura (2000) notes that this is not a robust method and not always successful. Amburgey (2005) adds that this is a complicated process that requires an accurate chemical dosing system with the necessary dosing equipment and capability of dosing for varying water quality parameters. It could also lead to floc formation in the underdrain that could be discharged into the filtrate during the filter run. Addition of coagulants to the influent water has similar results and disadvantages (Logsdon *et al.*, 2002).
- extended terminal subfluidization wash (ETSW), whereby the backwashing procedure is extended to include a subfluidizing backwash rate for a period equivalent to one filter-volume of water being removed (up to the backwash overflow level). The optimal rate depends on various parameters such as "water temperature, floc strength, particle density and the size of the detached particles". Although this method can be effective, it requires tailoring to the specific water treatment plant conditions starting with the various filter media sizes and various water temperatures experienced (Amburgey, 2005).

Filter-to-waste is recommended to achieve the required turbidity limits. Edzwald (2011) explains that filter-to-waste was largely abandoned in the early twentieth century based on the assumption that disinfection would control any pathogens that escaped the filter. However, Giardia and Cryptosporidium are very resistant to chlorine disinfection and therefore filter-to-waste was reinstated towards the end of the twentieth century. The turbidity profile should be analysed to determine the time and volume of wastage for optimising the effect of the filter-to-waste process. Logsdon *et al.* (2002) explains that the rate of filter-to-waste should be equivalent to the filtration rate expected at the start of the filter run so as not to prolong the ripening period and to limit exposure to hydraulic shocks when returning to filtration, which could result in poorer filtrate quality.

A combination of methods above can further reduce the turbidity spike(s) and ripening period, as investigated by Logsdon *et al.* (2002) whereby numerous water treatment plants limited the initial turbidity spike to 0.3 ntu and targeted 0.1 ntu with filter ripening durations typically of 10 to 20 minutes. Utilities may want to investigate combinations of the methods, but as a minimum, filter-to-waste and gradual rate changes are included in the backwash sequence described in Chapter 8.8.

8.11 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates the following related sheets (presenting screenshots from the tool):

- "Fluidization"

	A	B	C	D	E	F	G
1–2	Minimum Fluidization Velocities						Section Reference 8.2
3	Parameter	Symbol	Minimum Temperature	Mean Temperature	Maximum Temperature	Design Temperature	Units
4			tmin	tmean	tmax	tdesign	
5			0	20	25	25	
6			°C	°C	°C	°C	
7	Water Density	ρw	998.8426	998.2063	997.048	997.048	kg/m³
8	Dynamic Viscosity	μ	0.001787	0.001002	0.00089	0.00089	N.s/m²
9	Gravitational Acceleration	g	9.81				m/s²
10	Media 1 : Anthracite						
11	Large media grain size	d_{90}	0.0020				m
12	Media Density	ρ_m	1600				kg/m³
13	Galileo Number	Ga	15086.60113	48005.14212	60893.93689	60893.93689	-
14	Safety Factor	-	1.3				-
15	Minimum fluidization velocity	v_{mf}	0.009405305	0.014201361	0.015244254	0.015244254	m/s
16			**33.86**	**51.12**	**54.88**	**54.88**	m/h
17	Media 2 : Sand						
18	Large media grain size	d_{90}	0.0023				m
19	Media Density	ρ_m	2650				kg/m³
20	Galileo Number	Ga	63655.42269	202413.5811	256445.6137	256445.6137	-
21	Safety Factor	-	1.3				-
22	Minimum fluidization velocity	v_{mf}	0.027409153	0.035488907	0.036936479	0.036936479	m/s
23			**98.67**	**127.76**	**132.97**	**132.97**	m/h
24	Media 3 : Garnet						
25	Large media grain size	d_{90}	0.0004				m
26	Media Density	ρ_m	3950				kg/m³
27	Galileo Number	Ga	526.7269664	1674.619305	2120.985353	2120.985353	-
28	Safety Factor	-	1.3				-
29	Minimum fluidization velocity	v_{mf}	0.001904983	0.003364282	0.003774711	0.003774711	m/s
30			**6.86**	**12.11**	**13.59**	**13.59**	m/h

$$Ga = \frac{{d_{90}}^3 \rho_W (\rho_m - \rho_W) g}{\mu^2}$$

$$V_{mf} = \frac{\mu}{\rho_W d_{90}} (33.7^2 + 0.0408 Ga)^{0.5} - \frac{33.7\mu}{\rho_W d_{90}}$$

- “CP Backwash & Design Rates”

Row	A	B	C	D	E	F	G	H	I	J
1	Collapse Pulse Backwash Rates Estimates									Section Reference 8.9
3			Media 1 : Anthracite		Media 2 : Sand		Media 3 : Garnet		Units	
4	Minimum Fluidization Velocity	v_{mf}	54.88		132.97		13.59		m/h	
5	Air Flow Rate		Backwash water rate-minimum fluidization velocity ratio	Backwash Water Rate	Backwash water rate-minimum fluidization velocity ratio	Backwash Water Rate	Backwash water rate-minimum fluidization velocity ratio	Backwash Water Rate		
6	Q_a		$\%(v_{bw}/v_{mf})$	v_{bw}	$\%(v_{bw}/v_{mf})$	v_{bw}	$\%(v_{bw}/v_{mf})$	v_{bw}		
7	m/min	m/h	%	m/h	%	m/h	%	m/h		
8	0.5	30	38.6	21.2	41.4	55.02				
9	0.75	45	33.0	18.1	38.7	51.48				
10	1	60	25.2	13.8	35.0	46.54				
11	1.25	75	15.2	8.3	30.2	40.18				
12	1.5	90	3.0	1.6	24.4	32.41				
13	1.75	105	-11.5	-6.3	17.5	23.23				
14	2	120	-28.2	-15.5	9.5	12.63				
15	2.25	135	-47.1	-25.9	0.5	0.62				
16			For anthracite		For sand		No equation for garnet	For GAC		
17			$17.8\ Q_a^2 + (\%V/V_{mf}) = 43.0$		$8.5\ Q_a^2 + (\%V/V_{mf}) = 43.5$			35.2 Qa2 + (%V/Vmf) = 26.6		
18			for Q_a 24 - 78 m/h		for Q_a 30 - 84 m/h			for $Q_a < 48$ m/h		
19			ES ~ 1.1 mm		ES ~ 0.38 mm			ES ~ 0.68 mm		
20			UC ~ 1.55		UC ~ 1.53			UC ~ 1.7		
21			d_{90} ~ 1.99 mm		d_{90} ~ 0.69 mm			d_{90} ~ 1.48 mm		
22			ρ_m ~ 1700 kg/m^3		ρ_m ~ 2650 kg/m^3			ρ_m ~ 1840 kg/m^3		
23–24	Backwash Rates Design Selection								Section Reference 8.9	
25	Parameter	Symbol	Value	Units	Value	Units	Value	Units		
26	Air Flow Rate	Q_a	60	m/h	5040.0	m^3/h	1.40	m^3/s		
27	Backwash Water Rate 1	v_{bw1}	45	m/h	3780.0	m^3/h	1.05	m^3/s		for combined air scour
28	Backwash Water Rate 2	v_{bw2}	50	m/h	4200.0	m^3/h	1.17	m^3/s		for rinse

- “Bed Expansion”

Row	A	B	C	D	E	F	G	H	I	J
1–2	Bed Expansion								Section Reference 8.2	
3	Parameter	Symbol	Value						Units	
4	Water Density	ρ_w	997.048						kg/m^3	
5	Dynamic Viscosity	μ	0.00089						N.s/m^2	
6	Gravitational Acceleration	g	9.81						m/s^2	
7	Backwash rate	v_{bw}	50						m/h	
8			0.014						m/s	
26	Media 2 : Sand									
27			Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Total		
28		L medium	300	300	300	300	300	1500	mm	
29	Bed Depth		0.300	0.300	0.300	0.300	0.300	1.500	m	
30	Sphericity	ψ	0.75	0.75	0.75	0.75	0.75	-	-	
31		deq	1.50	1.60	1.85	2.10	2.32	-	mm	
32	Equivalent Diameter		0.00150	0.00160	0.00185	0.00210	0.00232	-	m	
33	Loose/initial bed porosity	ε_0	0.40	0.40	0.40	0.40	0.40	-	-	
34	Media Density	ρ_m	2650	2650	2650	2650	2650	-	kg/m^3	
35	Expanded bed porosity	ε	0.40	0.38	0.36	0.34	0.32	-	-	guess 0.5 (reasonable value) to start, then solve expansion correlation below
36	Modified Reynold's Number	Re_1	4.84	5.06	5.60	6.15	6.64	-	-	$Re_1 = \frac{v_{bw}\rho_w}{\frac{6}{\varphi d_{eq}}(1-\varepsilon)\mu}$ $A_1 = \frac{\varepsilon^3}{(1-\varepsilon)^2}\frac{\rho_w(\rho_m-\rho_w)g}{(\frac{6}{\varphi d_{eq}})^3\mu^2}$
37	Porosity Function for Fluidized Beds	A_1	23.26	24.62	28.11	31.71	35.02	-	-	
38	LHS of equation	-	1.367	1.391	1.449	1.501	1.544	-	-	
39	RHS of equation	-	1.367	1.391	1.449	1.501	1.544	-	-	$logA_1 = 0.56543 + 1.09348logRe_1 + 0.17971(logRe_1)^2 - 0.0392(logRe_1)^4 - 1.5(log\varphi)^2$
40	Expansion Correlation	-	0.0	0.0	0.0	0.0	0.0	-	-	solve for zero to determine expanded bed porosity; Solve Expansion Correlation for Media 2
41	Bed Expansion	-	0	0	0	0	0	0	%	$Bed\ expansion = \frac{L_E - L}{L} = \frac{\varepsilon - \varepsilon_0}{1-\varepsilon}$
42	Expanded Bed Depth	L_e	0.300	0.300	0.300	0.300	0.300	1.500	m	

Row	A	B	C	D	E	F	G	H	I	J
1–2	Bed Expansion								Section Reference 8.2	
3	Parameter	Symbol	Value						Units	
4	Water Density	ρ_w	997.048						kg/m^3	
5	Dynamic Viscosity	μ	0.00089						N.s/m^2	
6	Gravitational Acceleration	g	9.81						m/s^2	
7	Backwash rate	v_{bw}	50						m/h	
8			0.014						m/s	
43	Media 3 : Garnet									
44			Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Total		
45		L medium	0	0	0	0	0	0	mm	
46	Bed Depth		0.000	0.000	0.000	0.000	0.000	0.000	m	
47	Sphericity	ψ	0.6	0.6	0.6	0.6	0.6	-	-	
48		deq	0.25	0.29	0.31	0.34	0.39	-	mm	
49	Equivalent Diameter		0.00025	0.00029	0.00031	0.00034	0.00039	-	m	
50	Loose/initial bed porosity	ε_0	0.58	0.58	0.58	0.58	0.58	-	-	
51	Media Density	ρ_m	3950	3950	3950	3950	3950	-	kg/m^3	
52	Expanded bed porosity	ε	0.79	0.75	0.74	0.71	0.68	-	-	guess 0.9 (reasonable value) to start, then solve expansion correlation below
53	Modified Reynold's Number	Re_1	1.83	1.82	1.82	1.83	1.87	-	-	$Re_1 = \frac{v_{bw}\rho_w}{\frac{6}{\varphi d_{eq}}(1-\varepsilon)\mu}$ $A_1 = \frac{\varepsilon^3}{(1-\varepsilon)^2}\frac{\rho_w(\rho_m-\rho_w)g}{(\frac{6}{\varphi d_{eq}})^3\mu^2}$
54	Porosity Function for Fluidized Beds	A_1	6.19	6.15	6.15	6.19	6.32	-	-	
55	LHS of equation	-	0.792	0.789	0.789	0.792	0.801	-	-	
56	RHS of equation	-	0.792	0.788	0.788	0.792	0.801	-	-	$logA_1 = 0.56543 + 1.09348logRe_1 + 0.17971(logRe_1)^2 - 0.0392(logRe_1)^4 - 1.5(log\varphi)^2$
57	Expansion Correlation	-	0.0	0.0	0.0	0.0	0.0	-	-	solve for zero to determine expanded bed porosity; Solve Expansion Correlation for Media 3
58	Bed Expansion	-	98	69	59	45	30	60	%	$Bed\ expansion = \frac{L_E - L}{L} = \frac{\varepsilon - \varepsilon_0}{1-\varepsilon}$
59	Expanded Bed Depth	L_e	0.000	0.000	0.000	0.000	0.000	0.000	m	
60	Combined Filter Bed									
61		L	1500						mm	
62	Bed Depth		1.500						m	
63	Expanded Bed Depth	L_e	1.500						m	
64	Bed Expansion	-	0.0						%	check if ok with this expansion %, otherwise change backwash rate

Further guidelines to the designer on how to use these sections of the tool is provided in the Appendix sections A7,A8 and A9, respectively.

CHAPTER 9 : PLENUM/FLUME HYDRAULICS

9.1 Introduction

Despite Beverly (2011) being the only reference during the literature review to detail plenum/flume hydraulic guidelines, the details are included herein for the designer's awareness. These details are useful for the treatment facility/building design (outside the scope of this dissertation) that will incorporate the filter design.

A plenum is a large area below an underdrain, whereas a flume is a smaller channel serving the same function. It is common for the false bottom filters to be constructed with a plenum as shown in Figure 39. Flume terminology is shown in Figure 31. For plenum configurations, the lateral entry velocity is replaced by the velocity of the nozzle.

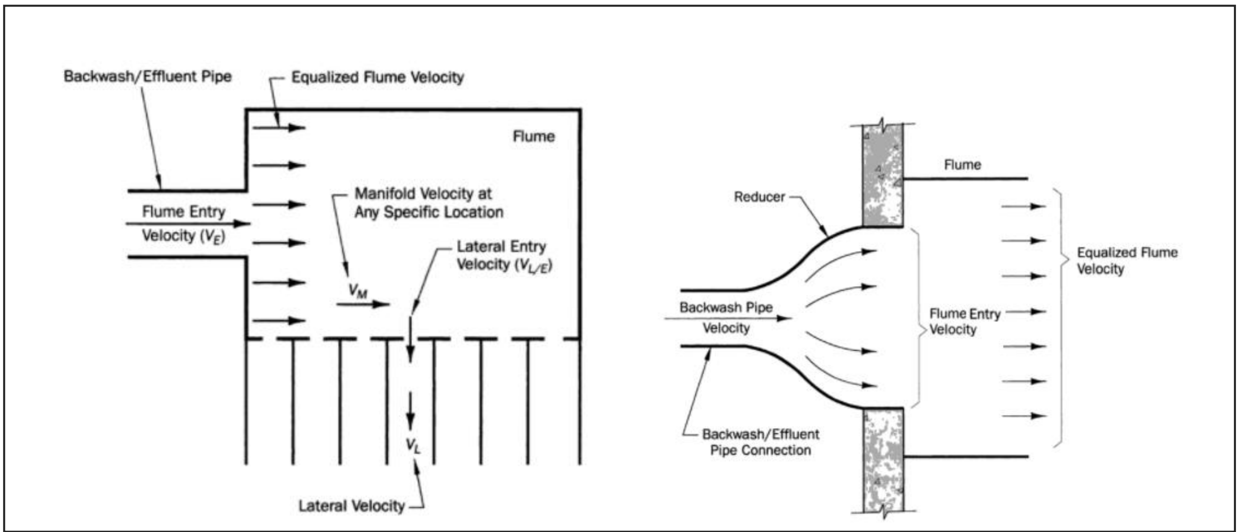


Figure 31 Flume Terminology (Beverly, 2011)

As with the underdrain system, plenum/flume design is governed by backwashing due to the higher flow rates compared to filtration. Uniform distribution of backwash water and air is critical to filter performance. The distribution efficiency of the backwash water across the filter is measured as percent maldistribution between two points in the filter, with a goal of less than 5 % maldistribution. The underdrain and the plenum/flume contribute to maldistribution efficiency and therefore the distribution of water equally to the various nozzles/laterals is important. Some underdrains distribute with less than 3 % maldistribution and therefore the flume/plenum distribution would need to be less than 2%. Larger flumes will be required if they are to reduce the total maldistribution for underdrain systems with a relatively high maldistribution (Beverly, 2011).

9.2 Maldistribution Calculations and Flume Sizing

Beverly (2011) suggests that the cross-velocity ratio defined by $V_{L/E}$ / v_M should be greater than 2.3 for acceptable maldistribution. An increase in lateral entrance velocities results in higher lateral head losses and a higher cross velocity ratio with better distribution control by the laterals. On the other hand, high flume velocities reduce the cross-velocity ratio and increase the maldistribution in the flume.

The flume entrance is typically a pipe serving two functions; namely filtrate exit and backwash water inlet. The flume entrance velocity (V_E) also impacts on distribution if it is too high (despite a large flume), as shown in Figure 32.

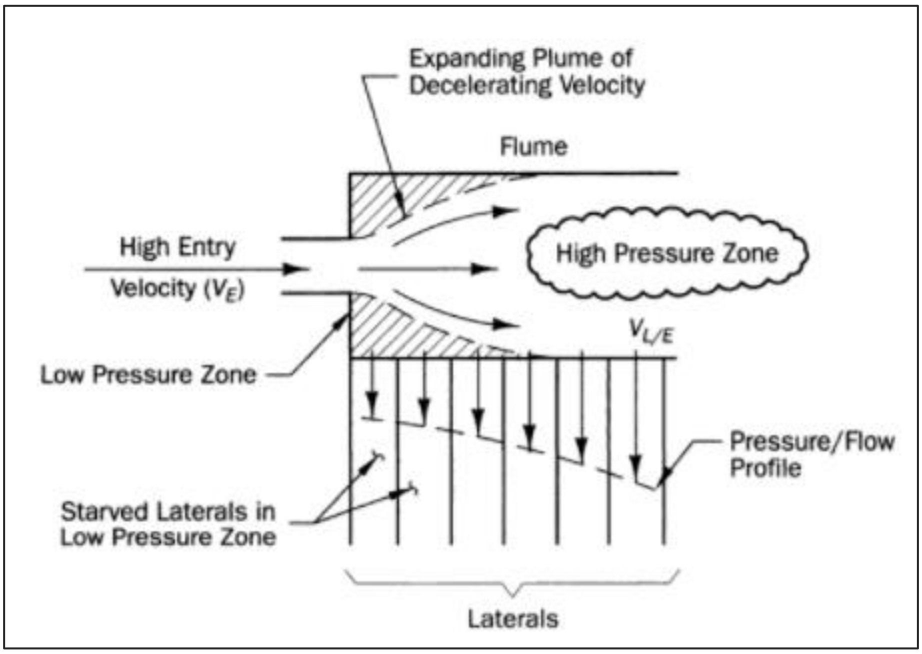


Figure 32 High Flume Entrance Velocity (Beverly, 2011)

Distribution is practically controlled by head loss (Beverly, 2011). For even distribution of flow across the filter, the underdrain should have a higher head loss than other filter components. This is not the case for coarse non-fluidizing monomedia due to the media having a higher head loss than the underdrain based on the low backwash rates. However, for fluidizing media Beverly (2011) recommends that the underdrain head loss exceeds the flume entry head loss by a factor of five to six or more for even distribution.

$$Rule\ of\ thumb\ for\ fluidizing\ media: V_E \leq \sqrt{\frac{2g \times underdrain\ head\ loss}{5\ (or\ 6)}} \quad (Eq. 9.1)$$

The pipework entering the flume should then be sized for this required low flume entrance velocity, which can be achieved either by utilizing large pipe diameters or alternately including a reducer (as shown in Figure 31) at the entrance to the flume to enlarge the delivery diameter (and thereby reduce the flume entrance velocity).

There are practical limits to further increasing the lateral entrance velocity and therefore a reduction in the flume velocity is necessary to satisfy the recommended cross velocity ratio of greater than 2.3. Although the backwash water entering the flume does not immediately flow equally across the flume, it is reasonable to assume a constant equalized velocity (V_{EQUAL}) in the design if the rule of thumb applies and the flume velocity head is very low/negligible. Therefore, it is recommended that $V_{EQUAL} \leq 0.6$ m/s at the front end of the flume. For a cross velocity ratio to be greater than 2.3, the lateral entrance velocity ($V_{L/E}$) must then be greater than 1.4 m/s. This high lateral entrance velocity may not be the case, which will require orifices at the entry to each lateral to produce the necessary velocity and associated head loss to ensure equal distribution across the underdrain. (Beverly, 2011).

Special consideration is required for the position of any air pipework in the flume. For filters that require air pipework, these pipes must not be installed such that it causes turbulence of the inlet water, which could affect distribution efficiency. A clearance of 400 to 450 mm between the top of backwash inlet pipe and the bottom of the underdrain is common for the installation of air pipework with risers. (Beverly, 2011).

Flume depth is based on the requirement for air pipework within the flume (H_{AP}), the diameter of the backwash water inlet (D_{inlet}) and any requirement for watertight joints (H_W), as well as any additional height required for hydraulic purposes (H_H) in order to achieve the required V_{EQUAL}; i.e. Flume depth = $H_{AP} + D_{inlet} + H_W + H_H$. (Beverly, 2011).

A minimum flume width of 600 mm is recommended for accessibility. An access hatch can be provided, or access via dismantling backwash pipework may also be possible.

9.3 Flume Configurations and Impact on Calculations and Sizing

9.3.1 Centre-Fed Front Flume

Note that a centre fed front flume will have the manifold velocity halved for the calculations – see Figure 33. Therefore, smaller flumes may be possible.

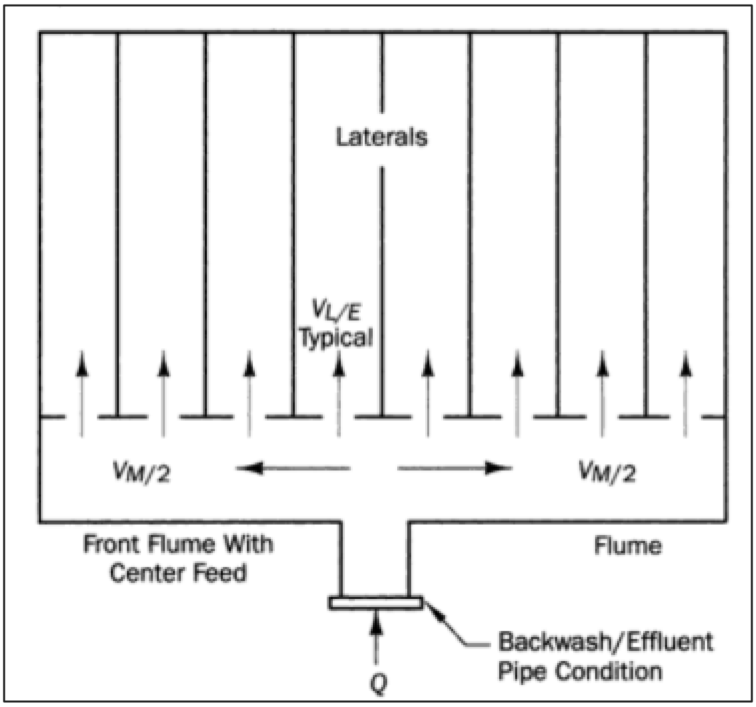


Figure 33 Centre Fed Front Flume Configuration (Beverly, 2011)

9.3.2 Centre-Fed Middle Flume

Middle entry is common for larger filters with long laterals that could result in maldistribution within the lateral with very high lateral entrance velocities. Centre-fed middle flumes will also have the manifold velocity halved for the calculations and the lateral entrance velocity will be half that of a front flume configuration due to feeding in two directions. This configuration also assists in uniformly distributing air when laterals are longer than 11-12 m.

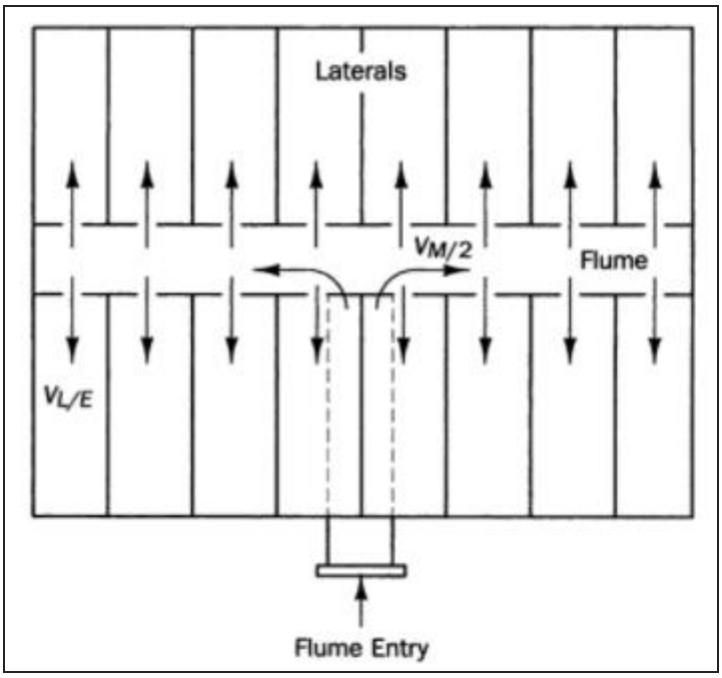


Figure 34 Centre-Fed Middle Flume Configuration (Beverly, 2011)

9.3.3 End-Fed Middle Flume

End-fed middle flumes require larger flumes to satisfy the cross-velocity ratio.

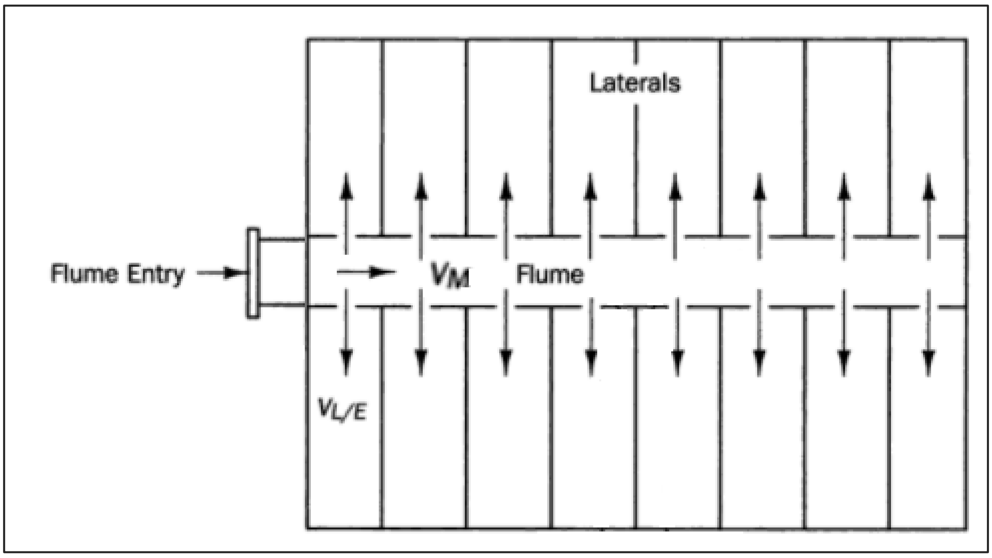


Figure 35 End-Fed Middle Flume Configuration (Beverly, 2011)

9.3.4 End-Fed Front or Side Flume

End-fed front or side flumes require larger flumes. End-fed front flumes are not common. The end-fed side flumes are common for larger filters that are more square than rectangular.

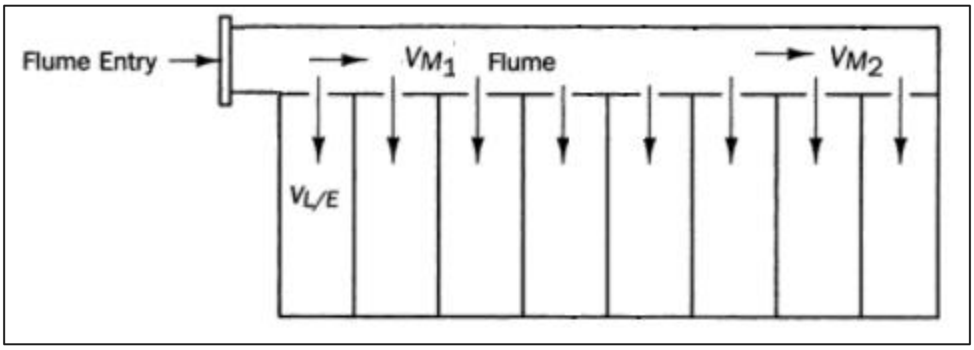


Figure 36 End-Fed Side Flume Configuration

The area above the flume is often used for filter inlet and backwash waste pipework.

Additionally, the flume is normally tapered to keep a nearly constant cross velocity ratio ($v_{M2} \sim v_{M1}$) . An air vent is required for the tapered flume as shown in Figure 37.

Each lateral requires penetrations for the water and air as shown in Figure 37. The backwash entry pipe is turned downwards to prevent whirlpooling that could otherwise result in maldistribution. The recessed flume should also allow for the water pipe to enter below the air/water interface which could also otherwise result in maldistribution.

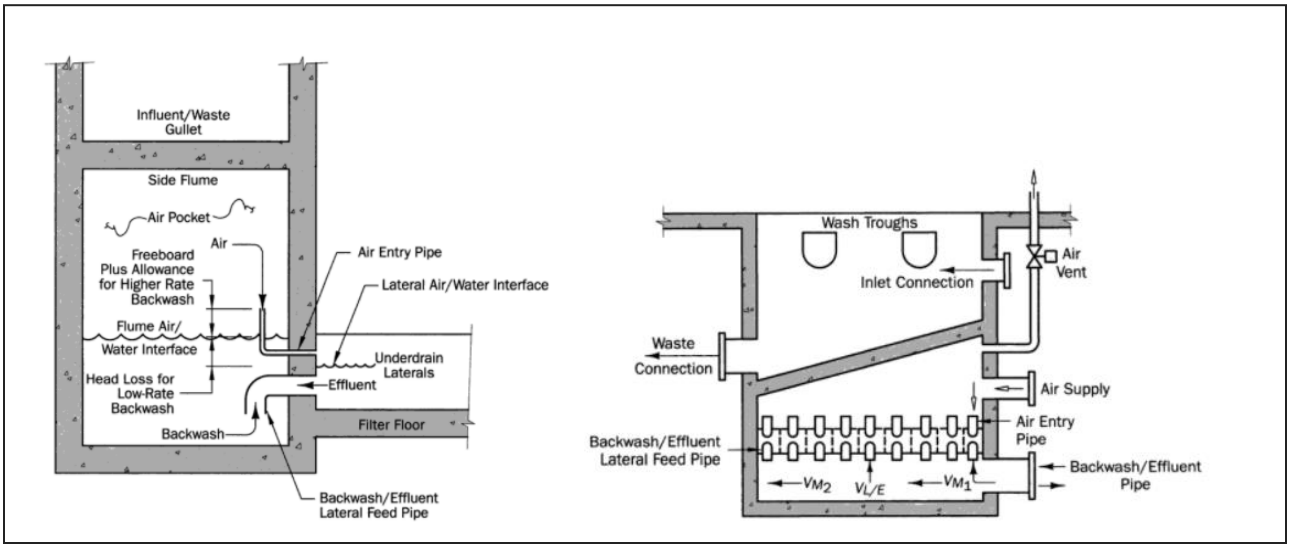


Figure 37 Side Flume Configuration Cross-Sections (Beverly, 2011)

CHAPTER 10 : UNDERDRAIN SYSTEMS

10.1 General

Underdrain systems are installed below the filter bed to uniformly collect filtered water across the filter bed and uniformly distribute water (and air, if applicable) across the filter bed during backwash, whilst preventing media from passing through (Binnie, Kimber and Smethurst, 2002; Crittenden *et al.*, 2012; Hendricks, 2010). The design is normally governed by the higher backwash flow rates (compared to filtration rates).

Gray (2010) suggests a depth allowance of 0.5 m for underdrains.

Underdrain systems are often proprietary and vary in detail. Kawamura (1999) lists the most important characteristics of underdrain systems as follows:

- Proven satisfactory
- Sufficiently anchored
- Good quality control by manufacturer
- Good supervision during construction/installation

There are two basic types of underdrain systems (Beverly, 2011):

- False bottoms
- Lateral, which can be further classified as follows:
  - Dual parallel lateral configuration in clay tile or plastic
  - Folded plate
  - Pipe laterals
  - Wedge wire

The main selection criteria should be the uniform distribution of flow, which can be achieved by selecting an underdrain system with small orifice openings that promotes a controlling loss of head and with uniformly low velocities in the underdrain system pipe or channel for the entire filter area (Montgomery, James M. Consulting Engineers, 1985). Montgomery, James M. Consulting Engineers (1985) describes orifice opening area, i.e. orifice area to bed area ratio, of 0.2 – 1.5 %. A media retention mechanism is necessary if the filter media directly above the underdrain system is smaller than the openings therein. This is discussed further in Chapter 11 : .

Other selection criteria include simplicity, durability and cost-effectiveness.

10.2 False Bottoms

False bottoms are precast or cast-in-situ concrete sections.

The Wheeler type false bottom underdrain shown in Figure 38 is generally no longer implemented (AWWA and American Society of Civil Engineers, 2012).

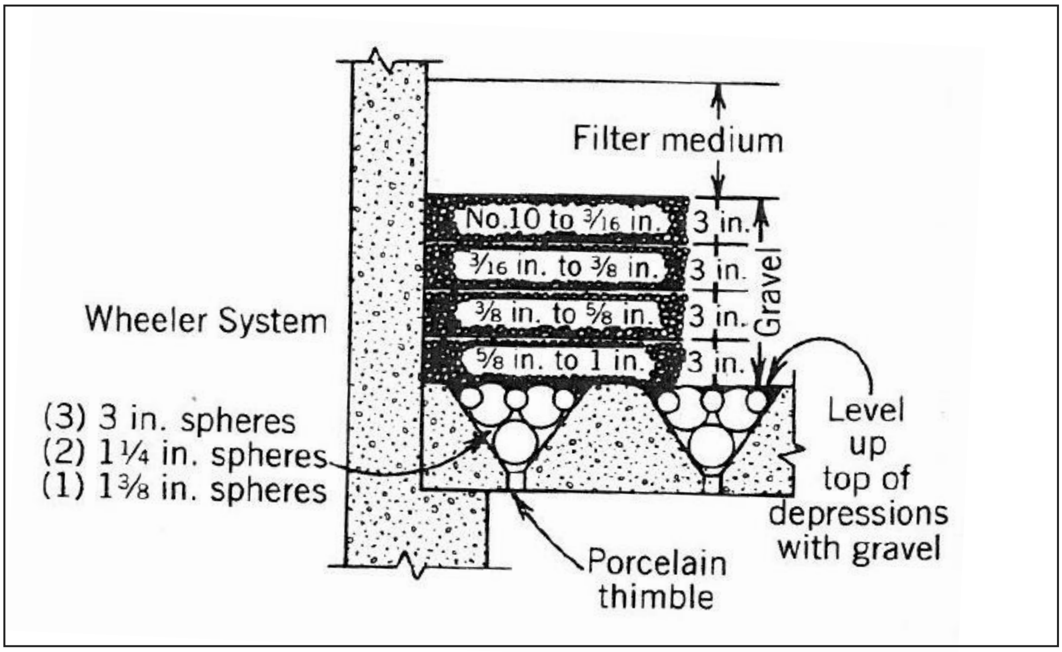


Figure 38 Wheeler Type False Bottom Underdrain (Montgomery, James M. Consulting Engineers, 1985)

The nozzle-type false bottom underdrain systems as shown in Figure 39 are more common, particularly for combined air and water backwash. Nozzle design is critical to proper filter operation. (AWWA and American Society of Civil Engineers, 2012).

Montgomery, James M. Consulting Engineers (1985) indicates false-bottom underdrains with nozzles to be the most popular choice of underdrain for filters with a backwash method that incorporates air scour; with this underdrain system common for uniform monomedia filters (AWWA and American Society of Civil Engineers, 2012).

It is essential for the false bottom and nozzles to be level (Binnie, Kimber and Smethurst, 2002).

False bottoms generally have an orifice opening area of over 1 % (AWWA and American Society of Civil Engineers, 2012) with an opening per nozzle of approximately 0.5 mm (Binnie, Kimber and Smethurst, 2002). Kawamura (2000) describes the nozzle slit size to be half that of the media effective size.

Edzwald (2011) describes nozzles spaced at 130 – 200 mm centres with either fine or coarse openings (up to 6 mm) and stems extending 150 – 230 mm into the plenum and comprising slots or holes for distributing air.

The false floor is typically located 300 – 600 mm above the filter bottom, creating an underdrain plenum below the false floor. Kawamura (2000) indicates a minimum plenum height of 600 mm.

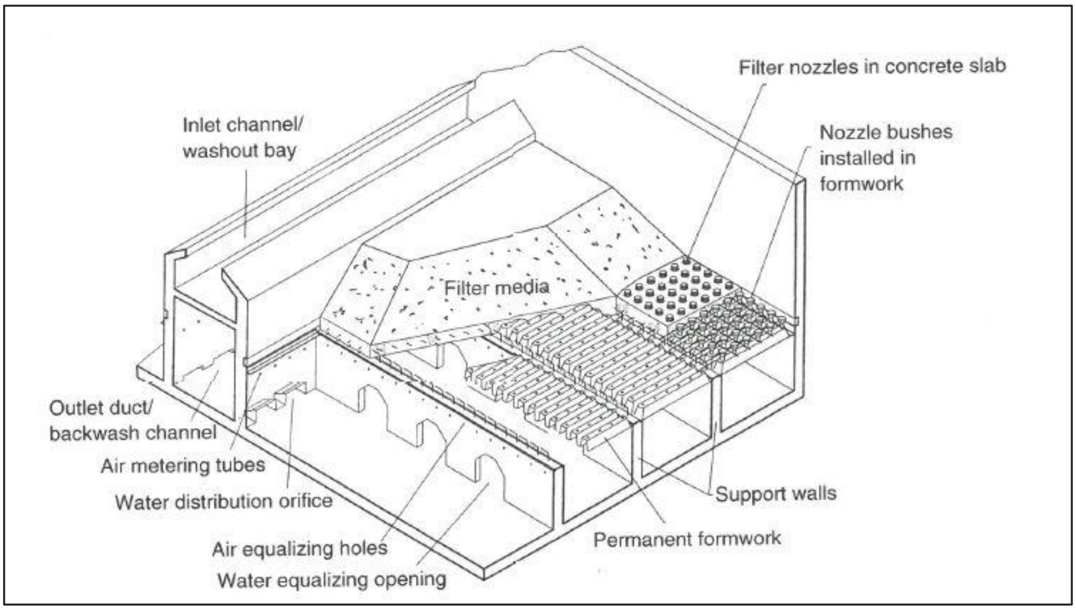


Figure 39 Typical False Bottom Underdrain (Binnie, Kimber and Smethurst, 2002)

Another example of a false bottom type underdrain is the porous plate underdrain as shown in Figure 40.

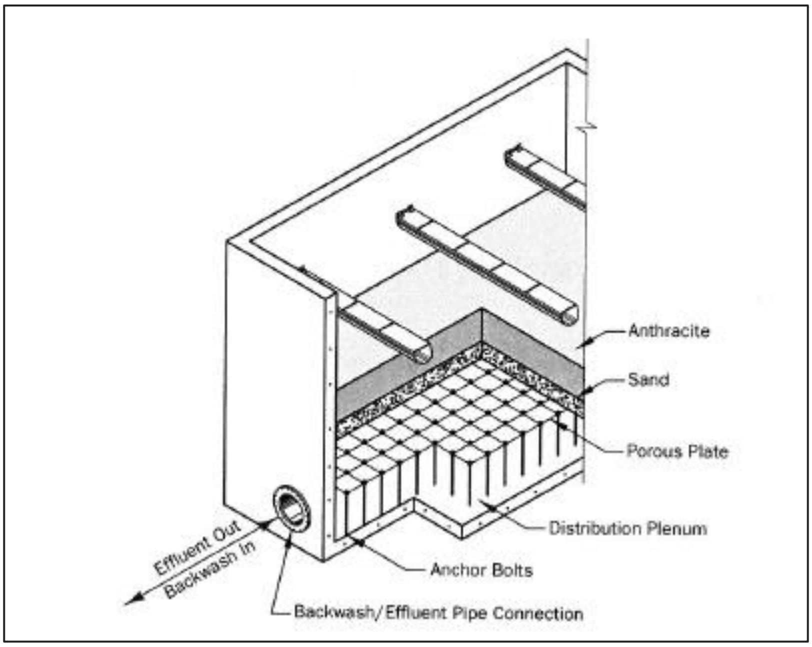


Figure 40 Typical Porous Plate Underdrain (Beverly, 2011)

AWWA and American Society of Civil Engineers (2012) explains that false bottoms are not recommended for adsorbers due to build-up of carbon fines and possible nozzle blockages that may lead to structural failure of the false bottom.

10.3 Dual Parallel Laterals

Dual parallel laterals are very popular (Beverly, 2011). A typical dual parallel lateral underdrain system is depicted in Figure 41.

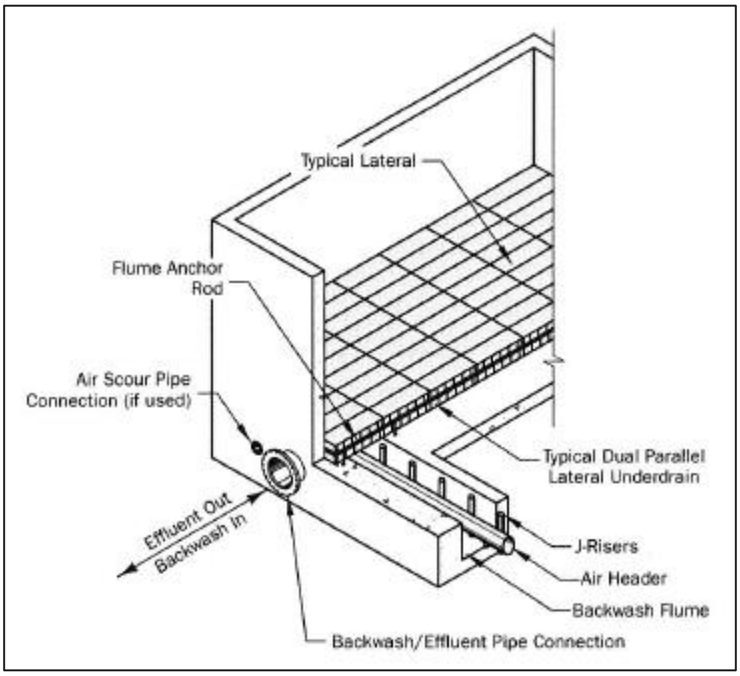


Figure 41 Typical Dual Parallel Lateral Underdrain Configuration (Beverly, 2011)

Vitrified clay blocks have been used in the past but are only suitable for backwash with water only. A separate air piping system at the sand-gravel interface is required for air scour. Each block has a lower primary feeder lateral and an upper secondary compensating lateral with small control orifices opening from the feeder to the compensating lateral. Dispersion orifices (typically 6 mm) are spaced evenly on the top of the block. (AWWA and American Society of Civil Engineers, 2012).

Plastic underdrains, typically of high density polyethylene (HDPE), are designed for simultaneous air and water backwash. The feeder lateral is triangular with upper and lower sets of holes for air and water to flow into the compensating lateral, as shown in Figure 42.

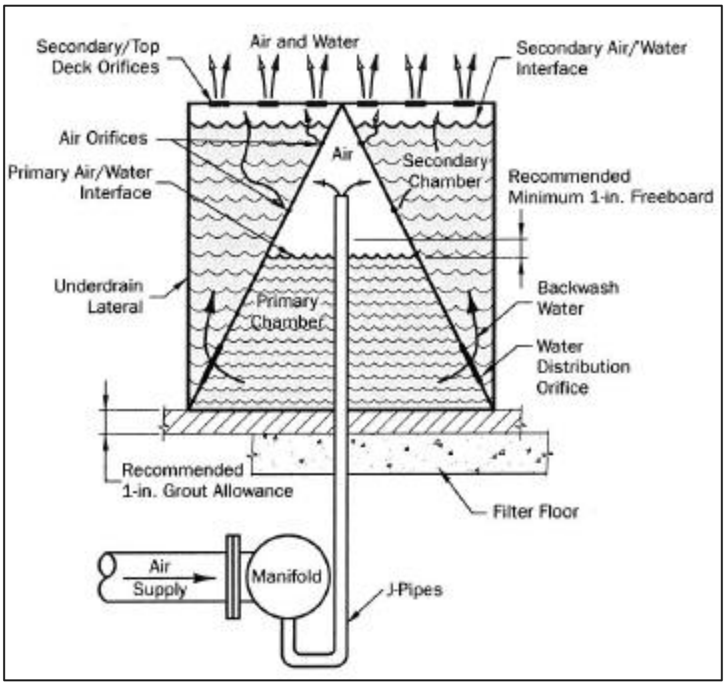


Figure 42 Typical Detail of Dual Parallel Lateral Underdrain (Showing Backwash Flow Path) (Beverly, 2011)

The j-risers in air pipework must be designed for the range of filter water levels during backwash (including water purging) so that more head loss is generated than the j-riser height, which would otherwise cause fluctuation or uneven air distribution (Beverly, 2011). J-risers must be designed to generate adequate head loss so that all the water is purged from the system. Otherwise, water in the air header could move back and forth in a wave-like motion and cause laterals to be online and offline as the water wave passes by (Beverly, 2011).

Beverly (2011) suggests a maximum of 11 m of lateral length from the air supply, after which air distribution may be compromised. During backwash, the starting of concurrent water within 30 to 45 seconds is recommended when long filters are used to limit poor air distribution along the underdrain length. Middle-fed flumes can assist but careful consideration of the flume design and lateral velocities is required, as discussed in Chapter 9 : .

Kawamura (2000) suggests a maximum lateral length of 15.3 m with approximately 248 dispersion orifices (of 6 mm) per m^2 of bed area.

Kawamura (2000) indicates a typical head loss across this underdrain during backwash as 0.6 – 1.8 m.

10.4 Folded plate

A typical folded plate underdrain system is shown in Figure 43.

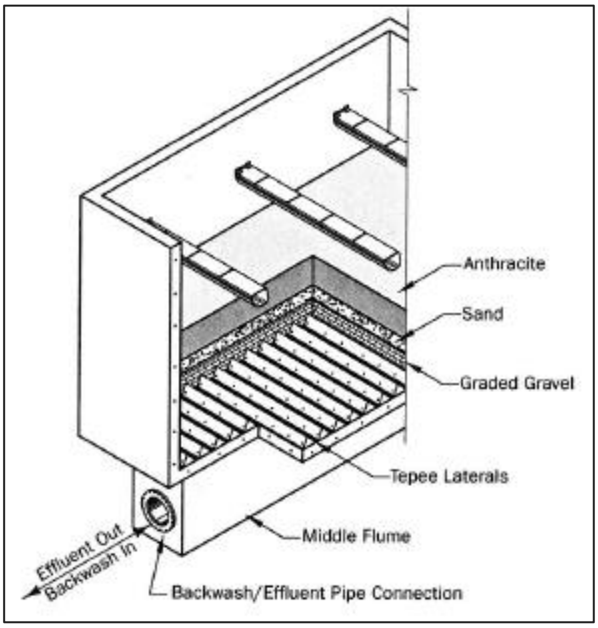


Figure 43 Example Detail of Folded Plate Lateral Underdrain (Beverly, 2011)

10.5 Pipe laterals

The pipe lateral underdrain system typically comprises a central manifold per filter bed with smaller laterals extending from the manifold with 6 – 19 mm holes on their undersides and sometimes incorporates nozzles too, as shown in Figure 44. The holes are normally spaced at 8 – 30 cm (AWWA and American Society of Civil Engineers, 2012).

AWWA and American Society of Civil Engineers (2012) presents the following design guidelines:

- Total orifice area to bed surface area of 0.0015 to 0.005:1
- Lateral cross-sectional area to total area of orifices served of 2 to 4:1
- Manifold cross-sectional area to total area of laterals served of 1.5 to 3:1

Montgomery, James M. Consulting Engineers (1985) also notes the general orifice opening area of less than 0.5 %.

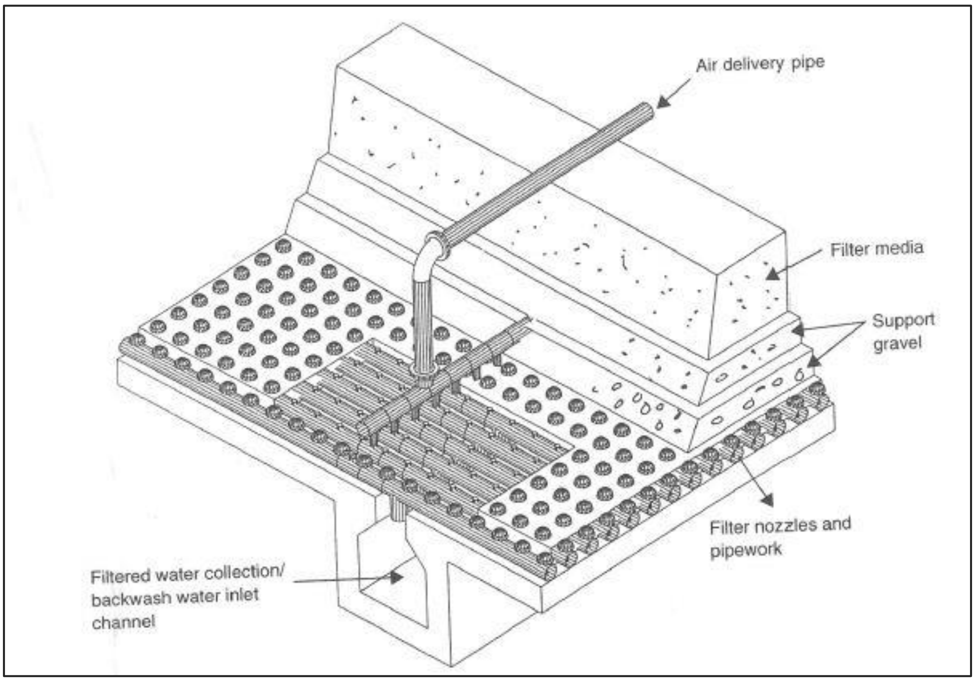


Figure 44 Example Detail of Gravity Media Filtration with Pipe Lateral Underdrain System (Binnie, Kimber and Smethurst, 2002)

Kawamura (2000) indicates a typical head loss across this underdrain system during backwash as 0.9 – 1.5 m.

Montgomery, James M. Consulting Engineers (1985) states that pipe laterals are not commonly used today because they are perceived to be outdated. AWWA and American Society of Civil Engineers (2012) suggests that their disuse is due to poor backwash water distribution and relatively high head losses. This type of underdrain system, however, is still utilized for pressure filtering systems (AWWA and American Society of Civil Engineers, 2012).

CHAPTER 11 : MEDIA RETENTION

11.1 Introduction

Media retention is effected by support gravel or an engineered media retention cap directly on top of the underdrain, which helps create the barrier between the filter media and underdrain during filtration and backwashing at the same time as avoiding blockage of the underdrain (AWWA and American Society of Civil Engineers, 2012).

It is common for false floors to be provided with support gravel and laterals to be provided with either support gravel or with a media retention cap.

11.2 Support gravel

11.2.1 Properties

The support gravel needs to remain undamaged during both filtering and backwashing to prolong the life of the filter media (Beverly, 2011).

Support gravel must be strong enough and durable to maintain itself over time. Support gravel is a coarse aggregate composed mainly of silica and some calcium (Beverly, 2011).

Beverly (2011) indicates that support gravel is also governed by AWWA B100 standard, which stipulates the following properties:

Table 22 Recommended Support Gravel Properties (Beverly, 2011)

Property	Unit	Requirement
Acid solubility	%	<5 for gravel ≤ No.8 (2.36 mm)
		<17.5 for No.8 (2.36 mm) > gravel < 1” (25.4 mm)
		<25 for gravel ≥ 1” (25.4 mm)
Fractured surfaces	%	≤25 with more than one fractured face
Specific gravity	-	>2.5

Acid solubility should be low, which would otherwise lead to decomposition of gravel within acidic water which can lead to intermixing and weakened support capabilities.

Rounded gravel allows for free passage of water compared to crushed gravel (with fractured faces) that compacts closely.

Gravel with a higher specific gravity is often used for the barrier layer (the finest layer) for additional stability. This is usually garnet or ilmenite with a minimum specific gravity of 3.8.

Porosity of 40% is common (Beverly, 2011).

Gravel sizing is discussed below.

The demand for gravel for filters is relatively small and therefore strong quality control is required to ensure that the supplied gravel meets the design specifications (Beverly, 2011).

11.2.2 Configurations, Size and Layer Thickness

The support gravel is either layered as single taper or double taper.

Single taper layered support gravel is a support gravel bed consisting of layers of smaller and smaller grain as shown in Figure 45.

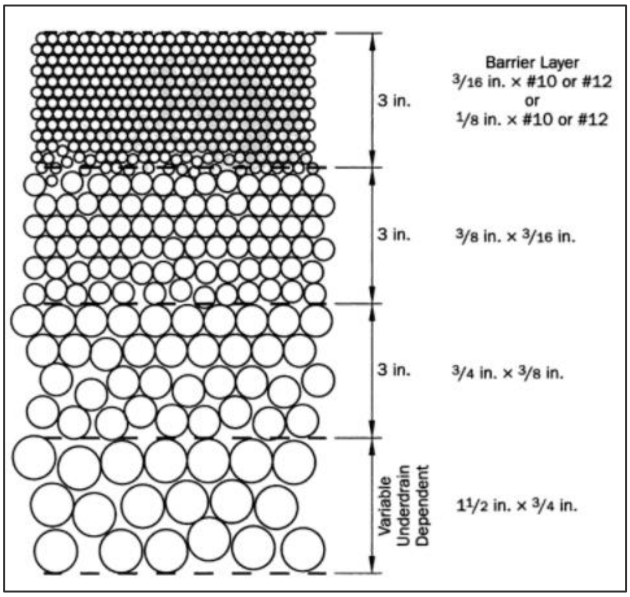


Figure 45 Typical Support Gravel – Single Taper Layers (Beverly, 2011)

The size of gravel in the lowest layer must protect the underdrain and a recommendation should come from the underdrain supplier. Edzwald (2011) prescribes a lower layer's lower grain size limit of 2 to 3 times the underdrain orifice diameter. For dual parallel lateral underdrains, Beverly (2011) and Kawamura (2000) indicate that a lower layer of ¾ in. x ½ in. would be sufficient for protection thereof.

Kawamura (2000) recommends a total support gravel depth of 300 mm for dual parallel lateral underdrains and 400 to 450 mm for other underdrains, with at least a 150 mm bottom layer for pipe lateral underdrains.

The layers above the lowest layer should be sized based on a 2:1 gradation as shown in Figure 45; the layer above should consist of grains half the size of that of the layer below as recommended by AWWA B100 standard. This ratio should never be more than 4:1 which would result in layers falling through the layer below (Beverly, 2011). Edzwald (2011) prescribes a top layer (the media/support gravel interface) lower grain size limit of 4 to 4.5 times the media's ES.

Each layer should be as uniform as possible – Edzwald (2011) makes reference to the support gravel with the sizes of d_{10} and d_{90} being not more than $\sqrt{2}$ apart. According to Beverly (2011), the AWWA B100 standard allows a specified size range tolerance of ± 8%. It is also important to confirm on which end of the range the gravel averages at – fine versus coarse end to ensure compliance with the 2:1 size gradation recommendation. Edzwald (2011) prescribes an upper grain size limit of the coarser layer to be 4 times the lower grain size limit of the adjacent finer layer.

Single taper layers are utilised for hydraulic wash only when backwash rates range from 25 – 60 m/h. Fluidization of the coarse layer typically occurs above 72 m/h. (Beverly, 2011).

However, single taper layers are susceptible to gravel upset if air scour is used. Sand and gravel is often more easily disrupted than gravel alone and fine gravel at the gravel-sand interface is most easily disturbed. (AWWA and American Society of Civil Engineers, 2012). Air tends to lift the barrier layer and push it to the side (Beverly, 2011). To assist with this problem, double taper layered support gravel is recommended (Beverly, 2011).

Double taper layered support gravel is a support gravel bed consisting of coarse-to-fine-to coarse layers, also known as the hourglass, as shown in Figure 46. The barrier layer is situated between larger layers. Three to five graded layers is typical (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011; Binnie, Kimber and Smethurst, 2002).

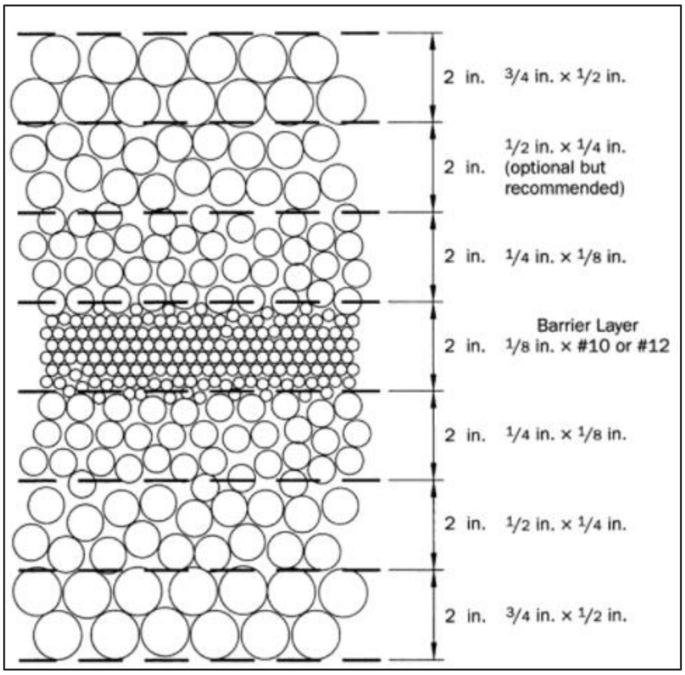


Figure 46 Typical Support Gravel – Double Taper Layers (Beverly, 2011)

During filtration, filter media could fall through the upper layers of the double-taper gravel to the barrier layer. During backwash, the media will be blown back out of the gravel without disturbing the larger gravel layer. The recommended gravel gradations for fluidizing media with air scour is shown in Figure 46. (Beverly, 2011).

For non-fluidizing filter media (normally larger than 0.5 mm), the filter media might be packed too closely into the upper layer of ¾ in. x ½ in. gravel, which could lead to localised eruptions and movement of the upper gravel layer and possibly result in loss of the gravel bed. Pilot testing for the design may be required and the size of top layer and barrier layer as shown in Figure 46 may not even be installed. (Beverly, 2011).

The size gradation between the barrier layer and filter media is typically 2:1 or 4:1 for coarse filter media (i.e. non-fluidizing media), and can be 6:1, 8:1 or even 10:1 for filter media with effective size of 0.5 mm (i.e. fluidizing media) due to the higher bridging effect of the fine sand. Pilot testing may be required to confirm compatibility between filter media and gravel support. (Beverly, 2011).

Cleasby (1972) as cited in AWWA and American Society of Civil Engineers (2012) reported that the double taper layered support gravel is stable at high backwash water rates; however, Kawamura (2000) established that solids could accumulate in the middle fine gravel layer when backwashing with only water.

Support gravel thickness depends on the media and underdrain. According to Beverly (2011), the AWWA B100 standard states that "…the thickness of each layer of gravel should be at least three times the maximum particle size of gravel in the layer, but not less than 3 in. in any case"; i.e. a minimum of approximately 76 mm per layer is required. Edzwald (2011) prescribes the thickness of each layer to be the greater of 3 times the upper grain size limit or 70 mm (3 in.). The bottom layer thickness depends on the underdrain, but is typically 25 mm above pipe laterals or above the fillet at the ends of dual parallel lateral underdrains to allow for water to spread out and around these obstacles, above which normal backwash will occur. (Beverly, 2011).

Although 2 in. layers are shown in Figure 46, Beverly (2011) recommends 3 in. layers.

Binnie, Kimber and Smethurst (2002) and AWWA and American Society of Civil Engineers (2012) indicate support gravel to typically be typically 450 mm deep, whereas Kawamura (2000) recommends 150 mm for nozzle floor systems and 300 to 380 mm for the plastic underdrains.

Gray (2010) indicates 300 mm as a typical support gravel depth with sizes of 12 – 25 mm.

11.2.3 Installations

Uniform monomedia filters; i.e. deep beds with coarse media, generally do not require support gravel because stratification is not required and air scour does not have a large impact on bed operation (AWWA and American Society of Civil Engineers, 2012).

Although AWWA and American Society of Civil Engineers (2012) suggests that nozzle-type false-bottom underdrain systems do not require support gravel due to the fine slot openings in the nozzles, a gravel support layer is generally provided because designing orifice openings to prevent media loss would result in a substantial number of nozzles to reduce head losses (Binnie, Kimber and Smethurst, 2002).

AWWA and American Society of Civil Engineers (2012) stipulates that for pipe laterals, the bottom layer of support gravel should extend to 100 mm above the above the highest backwash water outlet.

During construction, it is recommended that an additional 8% extra gravel per layer and 20% extra for the base layer gravel be supplied to allow for construction levelling issues and site wastage (Beverly, 2011). An indication at each layer’s height should be provided on the filter wall. This will assist in installation levelling too.

During operation, the filter media should not be dried out, which could otherwise result in filter media penetrating through the gravel to the underdrain (Beverly, 2011).

11.3 Media Retention Cap

An alternative to the double taper layered support gravel being used for preventing gravel upset during backwashing with air scour, is to eliminate the support gravel altogether by implementing a media retention mechanism (Edzwald, 2011).

Plastic block underdrains commonly use porous plate caps on top of the underdrain instead of the support gravel (AWWA and American Society of Civil Engineers, 2012).

Unlike support gravel, the filter media is subjected to backwash flow flowing straight up due to the filter media installed directly onto the underdrain media retention cap. Therefore, certain areas may not be properly cleaned. This inefficiency is reduced if air scour is used (Beverly, 2011) and is therefore recommended.

The components of underdrain that obstruct backwash flow through the filter media, such as end fillets and grouted sides, must be minimized.

The use of a media retention cap instead of support gravel in a filter utilizing GAC has the added benefit of facilitating easier removal and change of the media when required (Logsdon *et al.*, 2002).

CHAPTER 12 : FILTER ARRANGEMENT

12.1 Configuration

Filters are generally constructed in a concrete structure (Binnie, Kimber and Smethurst, 2002) which is typically seen in industry for gravity filtration, despite this being previously linked to water treatment plant capacities greater than 43 Mℓ/d (Montgomery, James M. Consulting Engineers, 1985).

Filters are typically configured next to each other along one or both sides of a pipe gallery depending on the capacity of the water treatment plant (AWWA and American Society of Civil Engineers, 2012; Hendricks, 2010). The common walls simplify construction and creates a compact solution. Future expansions can be accommodated by areas next to the filter row(s) and the extension of blanked off pipework. (AWWA and American Society of Civil Engineers, 2012).

Special consideration should be given to the interfacing of filtered and unfiltered water contamination risks. Designs in the United States of America avoid any possibility of contaminating filtered water with unfiltered water as documented in US state regulations (AWWA and American Society of Civil Engineers, 2012). For this reason, the clearwell would not normally be located below the pipe gallery nor would common walls be constructed between the filtered and unfiltered waters. Where compact design layouts are intended, the common walls would be two walls with a narrow drainage space between them (AWWA and American Society of Civil Engineers, 2012).

Filters should generally be enclosed in a building to prevent algae formation in warmer climates or ice formation in colder climates (AWWA and American Society of Civil Engineers, 2012).

Beverly (2011) discusses the option of being able to flush underdrain systems which is useful if media fouling is found in the underdrain when removing the filter media. A pipe lateral should be flanged for easy removal. A flushing manifold should be installed on the end of the parallel laterals to flush along the bottom of the underdrain into the flume.

12.2 Number of filters

The main factors influencing the number of filters are the plant capacity, maximum dimensions of a filter, the effect of rates through the media that change during backwashing and cost (Crittenden *et al.*, 2012).

It is common for a minimum of two filters to be provided in parallel so that filtration can still occur whilst one filter is being backwashed or is taken out of service for maintenance (AWWA and American Society of Civil Engineers, 2012; Hofkes *et al.*, 1981; Montgomery, James M. Consulting Engineers, 1985).

Kawamura (1999) and Kawamura (2000) recommend the following equation as a guideline to determining the number of filters*:

$$N \approx 0.62\,(Q^{0.5}) \quad (Eq. 12.1)$$

Where

N = number of filters

Q = water treatment plant design flow rate (megalitres per day)

*modified co-efficient for conversion from US megagallons/day to Mℓ/d

A minimum of four filters is typically recommended. Montgomery, James M. Consulting Engineers (1985) recommends this for plants greater than 19 – 38 Mℓ/d, whereas Kawamura (1999) recommends at least four filters for medium sized plants of 38 – 110 Mℓ/d and Kawamura (2000) recommends at least four filters for plants greater than ~ 7.5 Mℓ/d. Binnie, Kimber and Smethurst (2002) and Crittenden *et al.* (2012) also recommends a minimum of four filters, although six filters is considered the norm (Binnie, Kimber and Smethurst, 2002) so that one filter can be taken out of service and a second filter can be backwashed. Logsdon *et al.* (2002) also promotes that one or two additional filters be available on standby for when plant production has to be increased, particularly if treated water storage capacity is limited or if filter resting is included for filter ripening after backwashing (see Chapter 8.10). Beverly (2011) also explains the option of having a standby filter available for when backwashing occurs to reduce the effects of filtration rate changes. However, excessive periods of standby should be avoided (Logsdon *et al.*, 2002).

Beverly (2011) mentions that declining rate control methods for filters works best with numerous filters, typically six to eight or more.

Crittenden *et al.* (2012) states that the online filter(s) must be capable of filtering the plant design capacity at an acceptable filtration rate during all operations. The designer must also ensure that any increased filtration rate through the filter(s) during operation is not unreasonable. See Chapter 5 : .

12.3 Size of Filters

The following factors should be considered to determine the filter size (AWWA and American Society of Civil Engineers, 2012):

- Plant design flow rate
- Filtration rate
- Number of filters
- Effects of removing one filter for backwash
- Maximum area for even distribution of air and backwash water rates
- Maximum span of backwash water collection troughs
- Sizes for surface wash equipment, if implemented

Hofkes *et al.* (1981) also alerts the designer to consider the costs of construction, particularly for larger plants. As a starting point, the guideline is that the area of one filter (expressed in square metres) = N x 3.5.

Filters up to 420 m^2 have been reported (Cleasby *et al.* (1977) as cited in AWWA and American Society of Civil Engineers (2012)). Hendricks (2010) indicates that American standard practice is a maximum size of 93 m^2 per filter, but that these are typically much smaller at 20 – 30 m^2. Kawamura (2000) suggests that the filter area range begins at 25 m^2.

Although large sizes may be feasible for larger water treatment plants, the maximum practical size of one filter is approximately 90 m^2 (AWWA and American Society of Civil Engineers, 2012; Kawamura, 1999) to 100 m^2 (Crittenden *et al.*, 2012; Kawamura, 2000; Montgomery, James M. Consulting Engineers, 1985; Van Duuren, F. A., South Africa Water Research Commission., 1997), with an upper limit of approximately 150 m^2 (Binnie, Kimber and Smethurst, 2002). Binnie, Kimber and Smethurst (2002) explains that the practical limit is due to the high backwash rates whilst ensuring good flow distribution, reduced backwash trough lengths and practical backwash pipework diameters.

Kawamura (2000) indicates a width range of 3 – 6 m, suggesting 6 m for filter width in order to use standard off-the-shelf backwash troughs, and limiting this to 4 – 4.5 m when troughs are not used to ensure effective removal of backwash waste.

The typical length-to-width ratio ($r_{l/w}$) for filters is 2:1 (AWWA and American Society of Civil Engineers, 2012); however, Kawamura (2000) indicates a range of 2:1 to 4:1 with a 3:1 average.

12.4 Depth of Filters

The designer must consider the following aspects for filter depth determination (AWWA and American Society of Civil Engineers, 2012):

- Influent quality, which influences head loss development
- Available head
- Fixed head losses
- Filter media head losses
- Depth of water over filter bed surface
- Filtration rates
- Elevations of downstream level controls such as a filtrate control weir

The available head can be guided by site conditions and plant layout, an intention to minimize head losses, control and backwash system designs, filter run time and filter run termination/backwash initiation (AWWA and American Society of Civil Engineers, 2012).

Fixed head losses through channels, pipework and valves can be determined by hydraulic analyses and based on supplier literature (AWWA and American Society of Civil Engineers, 2012).

Head losses through the media and operational head losses, as well as aspects of negative head and dewatering, are discussed in Chapter 6 : . Additionally, bed expansion should be incorporated into the design depth – see Chapter 8 : .

Once the above aspects are known, water depth and weir elevation can be determined with the designer's attention on minimizing costs. The design criteria are essentially to reduce construction costs and to prevent the possibility of negative pressures developing in the filter (AWWA and American Society of Civil Engineers, 2012).

Filter bay depth typically ranges between 4.5 and 7.6 m (Kawamura, 2000).

12.5 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates the sheet "Filter Number, Size and Rate". Screenshots of this sheet have been presented in Chapter 5.3. The number of filters and size are interlinked with filtration rate considerations. The calculation and incorporation of aspects to determine the filter depth are also addressed in this sheet. Further guidelines to the designer on how to use this section of the tool is provided in the Appendix section A6.

CHAPTER 13 : FILTER PERFORMANCE AND MONITORING

13.1 Introduction

Filter performance should be monitored alongside other treatment processes so that operators have the necessary information to properly be in control of the water treatment plant and process. Filter operational records are useful to assess filter performance and to inform process adjustments and plant upgrades. (AWWA and American Society of Civil Engineers, 2012). Although these details do not inherently impact on the filter design addressed by this dissertation, the details are included herein for the designer's awareness.

13.2 Filter Monitoring

13.2.1 Overview

Turbidity is the most common performance measurement. Other useful measurements include filtration rate, head loss, backwash water and air scour rates, filter run time and sometimes particle count (AWWA and American Society of Civil Engineers, 2012), as well as temperature (Logsdon *et al.*, 2002). Beverly (2011) also recommends recording head loss over time, the rate of head loss development and the backwash initiation trigger (head loss versus filtrate turbidity).

Measurements can be continuously recorded and trended with modern instrumentation and electronics, including records on a computer or Supervisory Control and Data Acquisition (SCADA) system, which has been possible since the 1980s (Hendricks, 2010). Statistics can then be used to check performance and identify abnormal operation, and alarms can also be set to alert the operator to various issues (Logsdon *et al.*, 2002).

The instrumentation supplier's installation recommendations should be implemented to ensure measurement accuracy.

Filter-related equipment such as backwash pumps, blowers and valves also require monitoring to ensure performance requirements are met; however, the details of this equipment performance monitoring are beyond the scope of this thesis report.

13.2.2 Turbidity

The maximum contaminant level for drinking water specified by SANS 241-1 is 1.0 ntu.

Binnie, Kimber and Smethurst (2002) designates a good quality filtrate as crystal clear with a turbidity of less than 0.2 ntu. NHMRC (2011) indicates that filtrate turbidity of less than 0.1 ntu is normally achieved with conventional treatment. Hendricks (2010) and Kawamura (2000), too, indicates that filtrate turbidity should be less than 0.1 ntu, which should be steadily maintained after filter ripening (Crittenden *et al.*, 2012). After backwashing, the turbidity will be high as discussed in Chapter 8.10 and can be up to 0.5 – 1.0 ntu (Gray, 2010). This generally drops to approximately 0.2 ntu during the ripening period, eventually reaching 0.1 ntu (Binnie, Kimber and Smethurst, 2002).

Continuous turbidity measurements for each filter's filtrate is recommended to observe performance during a filter run, including for breakthrough anticipation and filter ripening behaviour. Continuous records will also assist the operator in understanding the entire treatment process with changes in other filter-influencing parameters such as source water quality, influent flow rates and chemical feed variations (AWWA and American Society of Civil Engineers, 2012). If intermittent measurements are taken instead of continuously, AWWA and American Society of Civil Engineers (2012) recommends that turbidity is recorded in maximum intervals of 15 minutes. USEPA's Long-Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) also prescribes turbidity measuring for individual filters at intervals of at least every 15 minutes. Logsdon *et al.* (2002) describes the CUSUM chart derived from the difference between a filter's mean daily turbidity and the bank of filters' mean daily turbidity. Figure 47 shows an example where filter 4 requires further investigation due to events around day 10 and day 15.

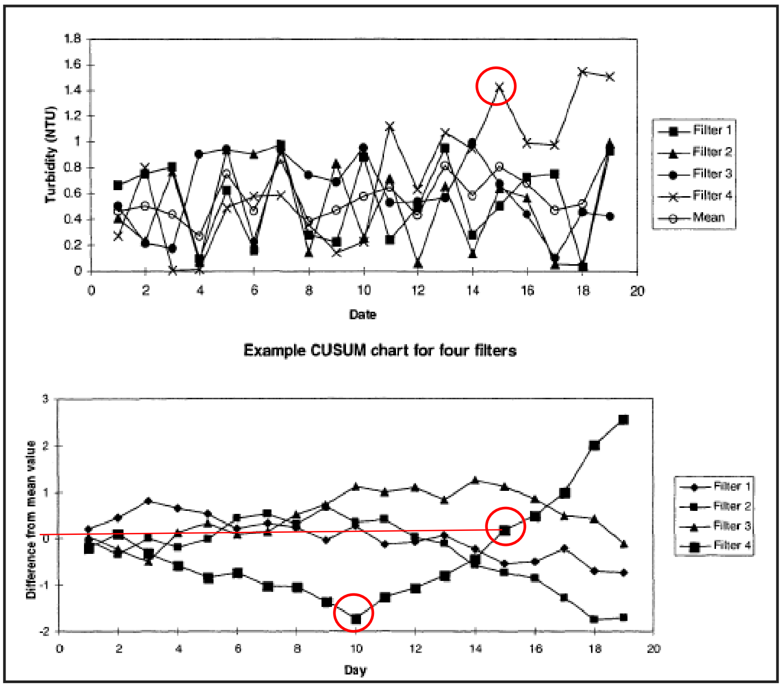


Figure 47 Example Turbidity and CUSUM charts for analysing filter turbidity (Logsdon *et al.*, 2002)

Turbidity can be used to automatically initiate a backwash cycle. As a minimum, high turbidity should alarm. Most treatment plants implement a high turbidity set point that is much lower than the regulatory requirements (AWWA and American Society of Civil Engineers, 2012).

The turbidity of the waste backwash water and filter-to-waste water can also be measured to assess backwash (monitoring quality and minimizing backwash water volume) and ripening performance, respectively (AWWA and American Society of Civil Engineers, 2012). Beverly (2011) suggests this turbidity reading as an option only for information and that operators rather visually observe backwash processes for proper operation and cleaning and setting the duration of filter-to-waste. However, Logsdon *et al.* (2002) explains the process of stopping a backwash according to waste backwash water turbidity rather than backwashing until the operator can see the surface of the bed, which increases the ripening period and contributes to post-backwash turbidity breakthrough. A target of 10 ntu is recommended (Logsdon *et al.*, 2002; Logsdon, 2008) unless site-specific investigations indicate otherwise.

Logsdon *et al.* (2002) discusses two cases where turbidity can be measured at the media-interface of dual-media filters (for review of the turbidity removal efficiency of the upper layer) and at various sample points along the depth of the filter media bed, which will assist the operator in monitoring filter performance and anticipating when breakthrough could occur.

Instrumentation should be installed in accessible positions as they require regular cleaning and calibration for accurate measurements (AWWA and American Society of Civil Engineers, 2012).

13.2.3 Particle Count

Particle counters measure how many particles are in the water and can also indicate size distribution of the particles within the water (Crittenden *et al.*, 2012), and are typically capable of quantifying and sizing particles in water over the range of approximately 1 – 500 microns (AWWA and American Society of Civil Engineers, 2012).

Logsdon *et al.* (2002) indicates that online particle count for filtrate is increasingly being monitored on water treatment plants.

Despite there being no regulations or standard methods for counting particles (Crittenden *et al.*, 2012) and results not being comparable (Logsdon *et al.*, 2002), AWWA and American Society of Civil Engineers (2012) reports that particle count is useful in determining log removal of particles, particularly Giardia of 5 – 15 microns and Cryptosporidium of 4 – 7 microns. The measurement effectiveness is negatively impacted by clean source waters (because of low count requirements); however, fluctuations can provide successful performance monitoring.

Edzwald (2011) explains that particle counters are more effective than turbidimeters at determining the onset of breakthrough for particles larger than 2 microns that may not have an associated turbidity increase. Despite being more expensive, the particle counters offer the advantage of optimizing and fine-tuning filter performance, particularly if particle breakthrough occurs whilst the turbidity is still acceptable (Logsdon *et al.*, 2002).

13.2.4 Filtration Rate

Monitoring flow rate can be used to identify step changes in flow (due to hydraulic shock or change from filter-to-waste to filtration), to calculate UFRV (for filtration efficiency calculations – see Chapter 6.5) and to evaluate slow starts and stops to filter runs (particularly when valves do not generally have a linear flow response) (Logsdon *et al.*, 2002).

Filtration rates are generally calculated from filtrate flow measurements and a known filter area. Magnetic meters are commonly installed in the filtrate piping for this purpose. The typical up- and downstream straight length requirements for flow meter accuracy is not normally a concern when the inlet flow is split equally between the filters (AWWA and American Society of Civil Engineers, 2012); however, the flow rate through each influent equal flow-splitting filter can be calculated if the total incoming or outgoing flow is measured (Logsdon *et al.*, 2002).

AWWA and American Society of Civil Engineers (2012) draws attention to overall plant production flow rate reporting and states that the summation of filtrate flow rates is not reliable and plant inlet and plant outlet flow meters should be installed for this purpose.

13.2.5 Head Loss

Head loss monitoring across the filter media is critical for proper filter operation. This is generally achieved using head loss gauges (Montgomery, James M. Consulting Engineers, 1985) with head loss measured between the water level above the filter bed and the head in the piping outside the filter (AWWA and American Society of Civil Engineers, 2012).

Logsdon *et al.* (2002) proposes the monitoring of head loss by comparing results based on dividing head loss gain over a filter run (i.e. difference between terminal head loss and clean bed head loss) by UFRV (with units of 100's of m^3/m^2). See Chapter 13.3 for further details.

Head loss monitoring across each type of media allows for even finer control (Montgomery, James M. Consulting Engineers, 1985). This can be achieved with pressure taps at various levels in the filter bed and connected to transparent tubes to create a "piezometer board" for visual monitoring, which is beneficial even if this is provided for only one filter (AWWA and American Society of Civil Engineers, 2012).

Head loss monitoring is also useful for assessing the filter bed condition when checking the clean bed head losses; i.e. after backwashing. Changes in the clean bed head loss measurement indicates that an issue has arisen; possibly media loss, inadequate backwashing or underdrain blockage (AWWA and American Society of Civil Engineers, 2012).

Logsdon et al (2002) explains that clean bed head loss measurements should only be done after the full operating filtration rate is achieved (particularly if the filter has a start that ramps up) and the measurement should be averaged for four 15-minute interval measurements in the hour after achieving the full operating filtration rate. These clean bed head loss calculations must also be normalised for varying filtration rates and temperatures, as follows:

$$H_f = H_{QT} \frac{Q_n}{Q_m} \quad (Eq. 13.1)$$

and

$$H_n = H_f \frac{\mu_T}{\mu_M} \quad (Eq. 13.2)$$

Where H_n = flow and temperature normalized head loss

H_f = flow normalized head loss

Q_n = flow rate used for standard for normalization

Q_m = measured flow rate

H_{QT} = measured head loss at flow rate (Q_m) and temperature (T)

μ_T = absolute viscosity of water at the normalized/standardized temperature

μ_M = absolute viscosity of water at the measured temperature (e.g. mean weekly water temperature)

The water level within each filter can also indicate head loss. However, the levels associated with declining rate filters can indicate only the entire filter bank's head loss and not that of each filter (Montgomery, James M. Consulting Engineers, 1985).

Head loss can be used to automatically initiate a backwash cycle.

Head loss gauges should be installed at a level approximately 10 cm above any waste backwash water to prevent contamination. Nevertheless, a sediment trap with drain is recommended to remove any sediment within the instrument pipeline. The pipeline should bend up to minimize air entrainment. A screen should be provided for the gauges at the filter media to prevent clogging the pipeline with media; however, the screen must be cleaned regularly so that it does not affect the measurement. (AWWA and American Society of Civil Engineers, 2012).

13.2.6 Backwash Water and Air Scour Rates

Backwash rates are generally calculated from flow measurements and a known filter area. Magnetic meters are commonly installed in the backwash water supply piping for this purpose. AWWA and American Society of Civil Engineers (2012) argues that recording meters are not necessary and that a totalizer is adequate for determining the volume of water utilized during backwash. But flow measurement is required to ensure the varied rate is accurately controlled (whether this be for varying rates during the backwash sequence or for seasonal changes due to temperature changes) (Logsdon *et al.*, 2002).

Montgomery, James M. Consulting Engineers (1985) suggests orifice flow meters for air flow measurements.

13.2.7 Filter Run Time

Long filter runs are desirable to limit the number of backwashes to be performed; however, if the filter runs are too long it may cause potential backwashing issues due to compaction of removed particles and/or hardening thereof or even cementation of the media, as well as bacterial growth (Crittenden *et al.*, 2012).

Monitoring filter run changes could be insightful. See also Chapter 8.1 for details on common filter run durations.

13.2.8 Media Loss

AWWA and American Society of Civil Engineers (2012) suggests that a reference mark be provided to indicate the top of the filter bed, such as a stainless steel plate, to allow for monitoring of any media loss. An alternative reference is to measure the media height from a fixed reference point, such as from the top of the filter bay. Media replacement should aim to re-establish the original filter bed media specifications. (Beverly, 2011).

Kawamura (2000) indicates that annual typical media losses are 1 – 2 % for sand and 5 – 7 % for dual- and mixed-media filters. Typically, 20% media loss will start impacting on filter performance.

AWWA and American Society of Civil Engineers (2012) indicates that 50 – 250 mm loss of the GAC bed depth per year in adsorbers is common. Pilot columns can be installed for sampling of the media at different depths and checking the remaining adsorptive capacity (Beverly, 2011).

Regular checking of media height is advised and should be done after backwashing to eliminate effects of run time and clogging. If media loss is suspected during backwash, a screening mechanism can be used to check media loss over the weir, which can identify general loss or loss from concentrated area(s). (Beverly, 2011)

Beverly (2011) mentions that media loss can still occur during backwash of non-fluidizing media if the effective size is on the smaller end of the range.

When complete removal of filter media is required, Beverly (2011) proposes generating a profile that could be used as a fault-finder and/or to propose filter improvements.

13.2.9 Temperature

Temperature monitoring is useful as it influences the evaluation of the clean bed head loss and the required backwash rates (due to changes in water viscosity with temperature), as well as various other treatment processes including coagulation, flocculation and disinfection. Logsdon *et al.* (2002) proposes periodic measurement for small water treatment plants and online (automated) measurement for larger water treatment plants.

13.2.10 Maldistribution

As discussed in Chapter 9 : , uniform distribution during backwashing is crucial for filter performance.

Water velocities are not easily measurable within the underdrain system and therefore pressure measurements can be used during the backwash and related to the flow (Beverly, 2011).

$$Maldistribution\ (\%) = \left(1 - \sqrt{\frac{p_{highest}}{p_{lowest}}}\right) \times 100 \quad (Eq. 13.3)\ (\text{Beverly, 2011})$$

Where p = pressure measurements with consistent units

The pressure measurements can determine maldistribution of the underdrain system, the flume and the total maldistribution (generally determined from a measurement near the inlet and a point farthest away). Suggested pressure tap locations for different flume configurations is presented in Figure 48. For false floor underdrains, these pressure taps should be cast or prefabricated into the underdrain.

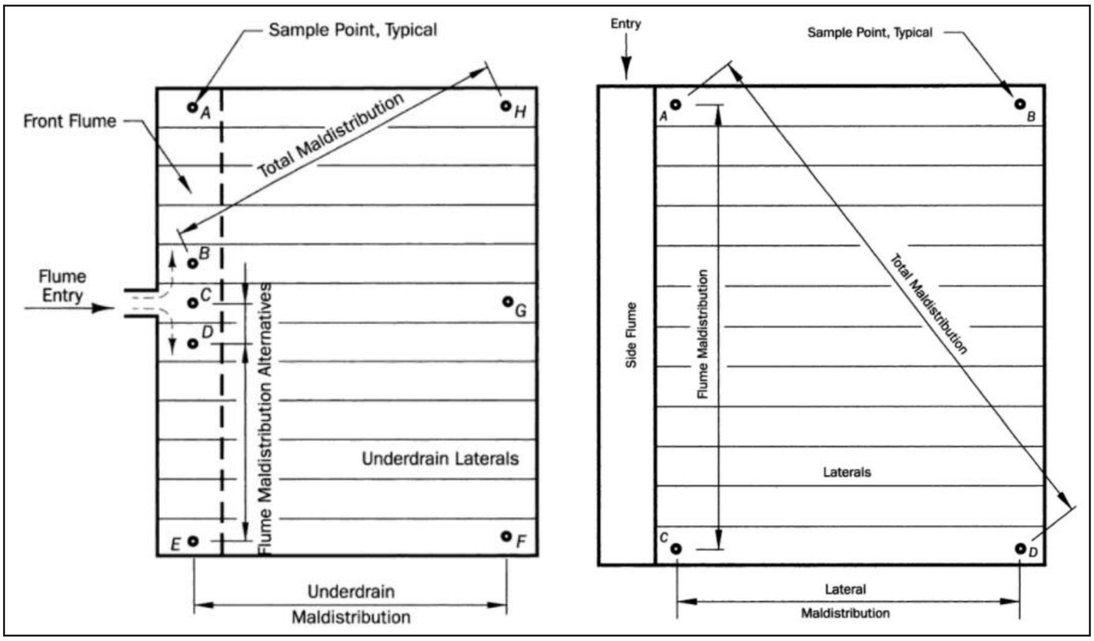


Figure 48 Proposed Pressure Tap Locations for Various Flume Configurations (Beverly, 2011)

13.3 Filtration Process Optimization

The granular media filtration process is considered optimized when the time at which the terminal head loss is reached equates the time at which breakthrough occurs (i.e. when the filtrate quality exceeds the specified standard); i.e. when $t_1 = t_2$ in Figure 49 (Crittenden *et al.*, 2012; Montgomery, James M. Consulting Engineers, 1985).

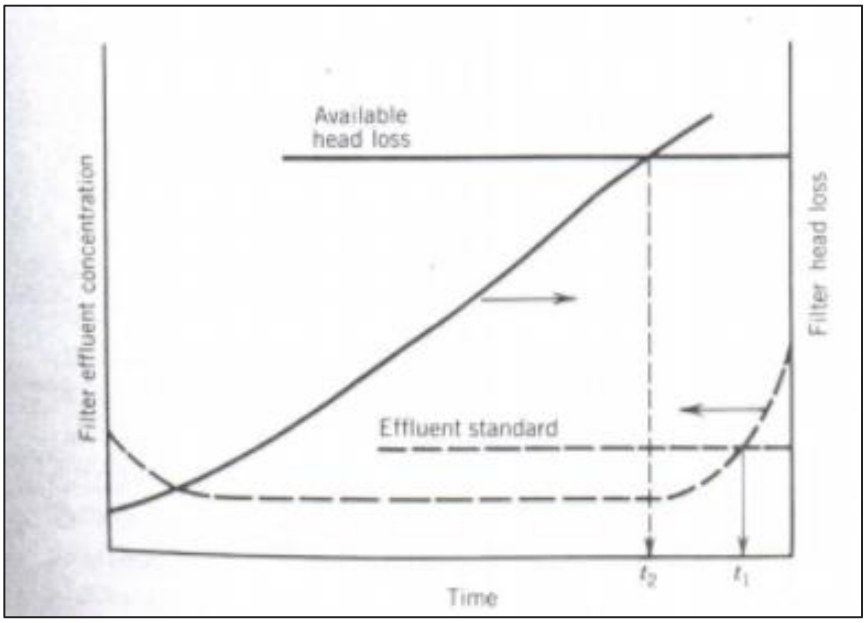


Figure 49 Diagram Representing Filtration Optimization (when t1 = t2 and still provides satisfactory filtration efficiency) (Montgomery, James M. Consulting Engineers, 1985)

Montgomery, James M. Consulting Engineers (1985) indicates that various optimization models exist, but that it is not yet possible to optimize the design without pilot studies for calibrating any given model, after which sensitivity analyses can be carried out. Note that models are beyond the scope of this dissertation. However, Montgomery, James M. Consulting Engineers (1985) does go on to indicate the effect of various parameters on filter performance as per Table 23. The designer can select media diameter, filter bed depth and filtration rate. Choosing a media size to prevent interlayer mixing will however decrease porosity and result in a rapid increase in head loss. Influent solids concentration depends on where filtration occurs in the overall water treatment process. Floc strength and deposit density are not easily controlled, although influenced by using polymers for destabilization.

Table 23 Effect of Parameter Increase on Filter Performance (Crittenden *et al.*, 2012; Montgomery, James M. Consulting Engineers, 1985)

Parameter (INCREASE)	**Effect on Time to Breakthrough (t_1)**	**Effect on Time to Terminal Head Loss (t_2)**
Media Diameter (ES)	↓	↑
Filter Bed Depth	↑	↓
Filtration Rate	↓	↓
Porosity	↓	↑
Influent Solids Concentration	↓	↓
Floc Strength	↑	↓
Deposit Density	↓	↓

Even with time to terminal head loss equating time to breakthrough, the filter must still have an acceptable filtration efficiency, defined as the ratio of net water produced by a filter to filter influent water. The net water accounts for backwash water utilised from the filtrate (AWWA and American Society of Civil Engineers, 2012) as well as filter-to-waste (Crittenden *et al.*, 2012; Logsdon *et al.*, 2002).

Therefore, filter efficiency is given by:

$$\eta_f = \frac{UFRV - UBWV - UFWV}{UFRV} \quad (Eq.13.4)\ (\text{Crittenden } et\ al., 2012)$$

Where

η_f = filtration efficiency

UFRV = unit filter run volume [m^3/m^2]

UBWV = unit backwash volume [m^3/m^2]

UFWV = unit filter-to-waste volume [m^3/m^2]

$$UFRV = \frac{V_f}{A} = v_f t_f \quad (Eq.13.5)\ (\text{Crittenden } et\ al., 2012;\ \text{Kawamura}, 1999)$$

$$UBWV = \frac{V_b}{A} = v_{bw} t_{bw} \quad (Eq.13.6)\ (\text{Crittenden } et\ al., 2012)$$

$$UFWV = \frac{V_{ftw}}{A} = v_f t_{ftw} \quad (Eq.13.7)\ (\text{Crittenden } et\ al., 2012)$$

Where

V_f = volume of water filtered during a filter run [m^3]

V_b = volume of water required for one filter backwash [m^3]

V_{ftw} = volume of water discharged during filter-to-waste [m^3]

A = filter area [m^2]

v_f = filtration rate [m/h]

v_{bw} = backwash rate [m/h]

t_f = duration of filter run [h]

t_{bw} = duration of backwash utilizing backwash water [h]

t_{ftw} = duration of filter-to-waste [h]

The backwash water volume depends on the backwash method and duration of sequence steps. Programming and control of equipment could impact on these durations for calculation purposes. (Beverly, 2011). Ives (1981) as cited in Logsdon *et al.* (2002) stated backwash volumes of 1% of filtrate to be typical in the United Kingdom, with a high of 3% and 5% considered to be excessive. Kawamura (2000) states that the ratio of backwash water volume to filtered water volume prior to backwashing should be less than 3% normally, less than 2% for good performance and more than 5% as poor. Also, it is common for the percentage to be less during warmer months compared to colder months.

Montgomery, James M. Consulting Engineers (1985) and Crittenden *et al.* (2012) recommend a filtration efficiency of at least 95% and make mention of a minimum UFRV of 200 m^3/m^2. Kawamura (2000) states anything less than 200 m^3/m^2 as poor filtration, over 410 m^3/m^2 as better performance and over 610 m^3/m^2 as good filter performance for conventional treatment. A typical UBWV is 8 m^3/m^2 (Crittenden *et al.*, 2012; Edzwald, 2011; Montgomery, James M. Consulting Engineers, 1985).

Operators in search of a longer filter run or lower filtrate turbidity will sometimes backwash excessively, which is counterproductive and lowers the filtration efficiency and can also increase the ripening period which will further reduce filtration efficiency (Crittenden *et al.*, 2012).

Other possible reasons for a low filtration efficiency can include “inadequate pre-treatment, filter-clogging algae in the influent water, excessive fines or mudballs in the filter media, too high a filtration rate”, mineral precipitates present in the media or underdrains, “air binding” or hydraulic restrictions between the filter and clearwell (AWWA and American Society of Civil Engineers, 2012).

A different definition of filtration efficiency is the ratio of particle concentration leaving the filter (i.e. in the filtrate) to particle concentration in the water entering the filter (Hendricks, 2010).

CHAPTER 14 : OTHER KEY FILTER DESIGN ASPECTS

14.1 Introduction

Filter design includes for getting water into and out of the filter as well as backwash-related equipment, which can include inlet channels, waste backwash water removal, hydraulic connections as channels or pipework, valves and actuators, backwash pumps and blowers and other ancillaries. These other key filter design aspects are discussed further herein.

14.2 Filter Inlet Channel Design

Any influent channels delivering water to the filters must be designed to prevent settlement within the channel; i.e. velocities of greater than 0.6 m/s but not too high (AWWA and American Society of Civil Engineers, 2012). Kawamura (2000) limits the velocity to 0.9 m/s; however, a maximum of 1.2 m/s is allowable for declining rate filters and for filters with flow rate controllers in the filtered water pipework.

Turbulence and excessive velocities should be avoided to prevent floc shearing. For filters that consist of inlet weirs for equal flow splitting to each filter, the inlet channel design must ensure the same water level at each inlet weir (AWWA and American Society of Civil Engineers, 2012). For this filter type, Kawamura (2000) recommends that the velocity in the filter inlet channel is less than 0.45 m/s.

14.3 Waste Backwash Water Removal

14.3.1 Overview

There are two common mechanisms for removal of waste backwash water from a filter:

- backwash troughs above the filter bed – typical of backwash with full fluidization common in the United States (Kawamura, 2000; Montgomery, James M. Consulting Engineers, 1985)); i.e. not typically used for coarse monomedia filters (Edzwald, 2011), unless air-scouring is included in the backwash sequence (Kawamura, 2000).
- overflow walls on one or both sides adjacent to the filter bed – typical of backwash with partial bed fluidization with air scour common in Europe (Kawamura, 2000; Montgomery, James M. Consulting Engineers, 1985). A central overflow channel that splits the filter bed in two is common in Australia (AWWA and American Society of Civil Engineers, 2012).

14.3.2 Backwash troughs

Sincero and Sincero (2002) depicts backwash troughs installed below filtration water level.

The configuration of a filter with backwash troughs is shown in Figure 50, where:

P = trough depth

H_T = height of trough overflow above rested filter bed

S_T = trough centre-to-centre spacing

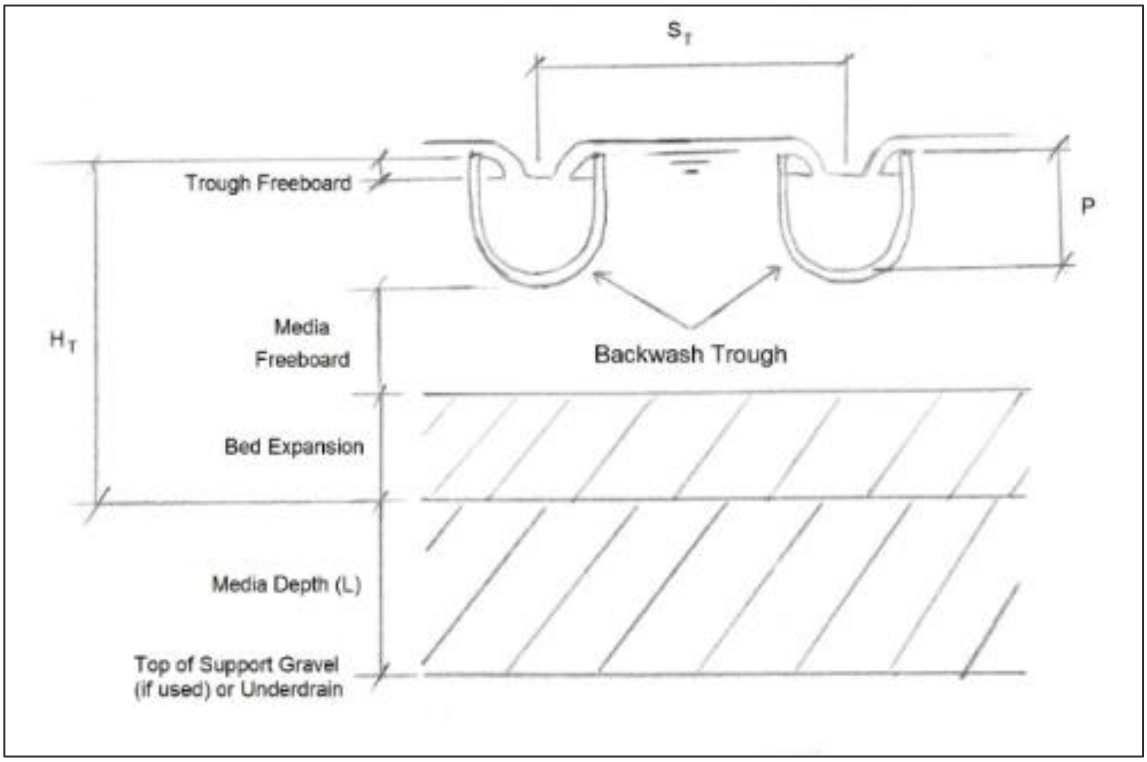


Figure 50 Height and Spacing of Backwash Troughs (adapted from Montgomery, James M. Consulting Engineers (1985))

Trough installation height is critical to ensure that media is not lost, and freeboard is recommended for media that expands during the backwash process.

For installations with multiple troughs, media could be carried over into the troughs due to the increase in flow velocity due to the reduced overall filter area as water passes between the troughs (Beverly, 2011).

AWWA and American Society of Civil Engineers (2012) and Montgomery, James M. Consulting Engineers (1985) present the guideline for trough height position as:

$$(0.75L + P) < H_T < (L + P) \quad (Eq. 14.1)$$

This is more conservative than the proposal by Kawamura (2000), which replaces the 0.75L with 0.5L in the above equation.

Earlier designs allowed the top of the trough at 500 – 600 mm above an unexpanded filter bed (Hofkes *et al.*, 1981). These days, the equation above would result in greater trough height positions relative to the media surface. Furthermore, AWWA and American Society of Civil Engineers (2012) suggests a media freeboard of 150 – 300 mm; whereas Beverly (2011) recommends that this be greater than approximately 200 – 300 mm. For dual media filters, Kawamura (2000) suggests that the top of the troughs be positioned at 1100 – 1200 mm above the media surface.

The number of troughs is dependent on the flow capacity per trough as well as the spacing (Hendricks, 2010), which is an iterative design approach.

The criterion for centre-to-centre spacing is further given by Kawamura (2000) as:

$$1.5H_T < S_T < 2.5H_T \quad (Eq. 14.2)$$

The backwash troughs are generally evenly spaced above the filter media to limit the required horizontal travel to the trough. Earlier designs allowed horizontal travel of 1.5 – 2.5 m (Hofkes *et al.*, 1981); however, Kawamura (2000) limits the horizontal travel at approximately 900 mm; i.e. the maximum spacing between trough edges is 1800 mm. For dual-media filters, Kawamura (2000) indicates that the spacing between trough edges can range up to 3000 mm. Even so, the trough spacing and configuration should still ensure that each trough serves an equal area of the filter (AWWA and American Society of Civil Engineers, 2012).

Trough cross-sectional area can be estimated from supplier sizing diagrams based on water flow. An example of a supplier sizing diagram is shown in Figure 51. Earlier designs indicated that trough design should incorporate at least 5 to 10 cm of trough freeboard at the upper end of any sloping trough (i.e. freeboard within the trough) (Montgomery, James M. Consulting Engineers, 1985). AWWA and American Society of Civil Engineers (2012) prescribes 5 to 8 cm trough freeboard at maximum water flow capacity (i.e. backwash and surface wash flow rates); however, Kawamura (2000) suggests at least 7.5 cm.

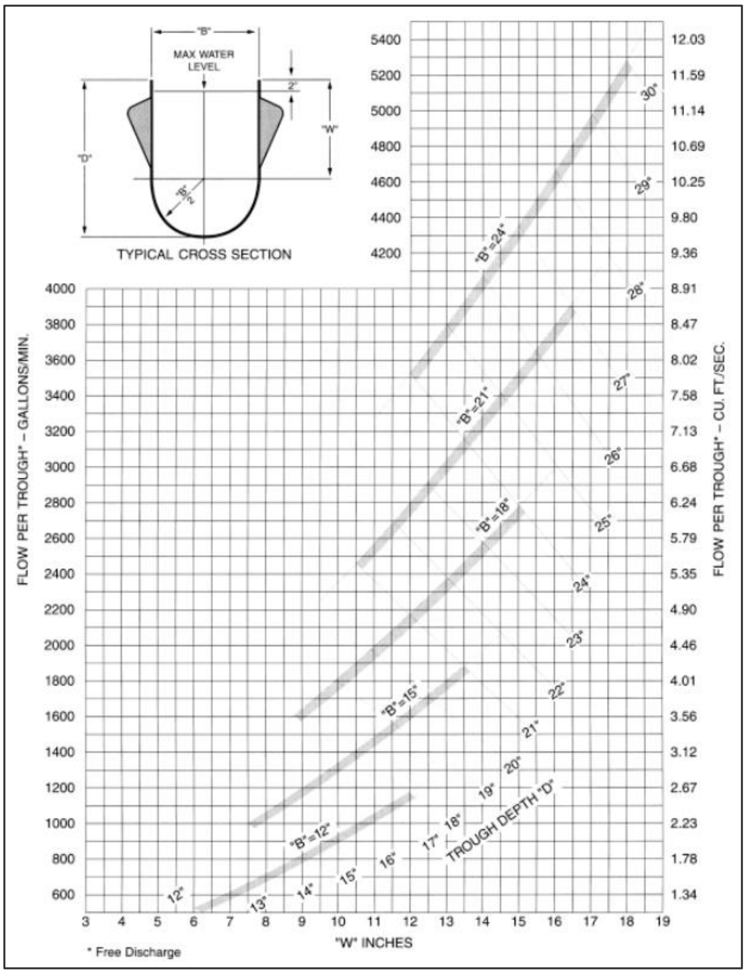


Figure 51 Example Trough Cross-Sectional Area Sizing Chart (AWWA and American Society of Civil Engineers, 2012)

Kawamura (2000) provides a guided estimate of trough depth for a rectangular cross-section with level inverts as follows:

$$P = \left(\frac{q}{1.38B}\right)^{0.667} + trough\ freeboard \quad (Eq. 14.3)$$

Where

q = total rate of discharge per trough in m^3/s

B = trough width in m

Additionally, Montgomery, James M. Consulting Engineers (1985) suggests a configuration with free fall from the trough to the discharge area. Kawamura (2000) suggests that this be 100 – 250 mm and explains that this ensures even flow conditions through each trough.

Assuming there is free fall to the discharge area (i.e. flow will pass through critical depth at the trough outlet) and no wall friction, the water depth at the upstream end of the trough (D_0) can be estimated as follows:

$$D_0 = \sqrt{{d_c}^2 + \frac{2q^2}{gB^2 d_c}} \quad (Eq.\,14.4)\ (\text{Van Duuren, F. A., South Africa Water Research Commission., 1997})$$

With critical depth d_c given by:

$$d_c = \sqrt[3]{\frac{q^2}{gB^2}} \quad (Eq.\,14.5)\ (\text{Van Duuren, F. A., South Africa Water Research Commission., 1997})$$

The calculated water depth (D_0) should be increased by 6 to 16% for frictional losses in the system (Reynolds (1982) as cited in Van Duuren, South Africa Water Research Commission (1997)).

Troughs are generally made of fibre-reinforced plastic (FRP), stainless steel or reinforced concrete, installed with supports to ensure uniform levels throughout (AWWA and American Society of Civil Engineers, 2012).

Fibre-reinforced plastic and stainless steel troughs usually have U-shaped bottoms and concrete troughs consist of V-shaped bottoms. The bottoms should never be flat because “froth and suspended matter are often trapped under the trough bottom and may never be washed out” (Crittenden *et al.*, 2012), capable of falling back down onto the filter bed and forming mudballs (Kawamura, 2000). Some trough designs incorporate baffles on either side of the underside of the trough to assist in limiting media loss, particularly when air scour is implemented. (AWWA and American Society of Civil Engineers, 2012). Figure 52 illustrates the typical cross-sections.

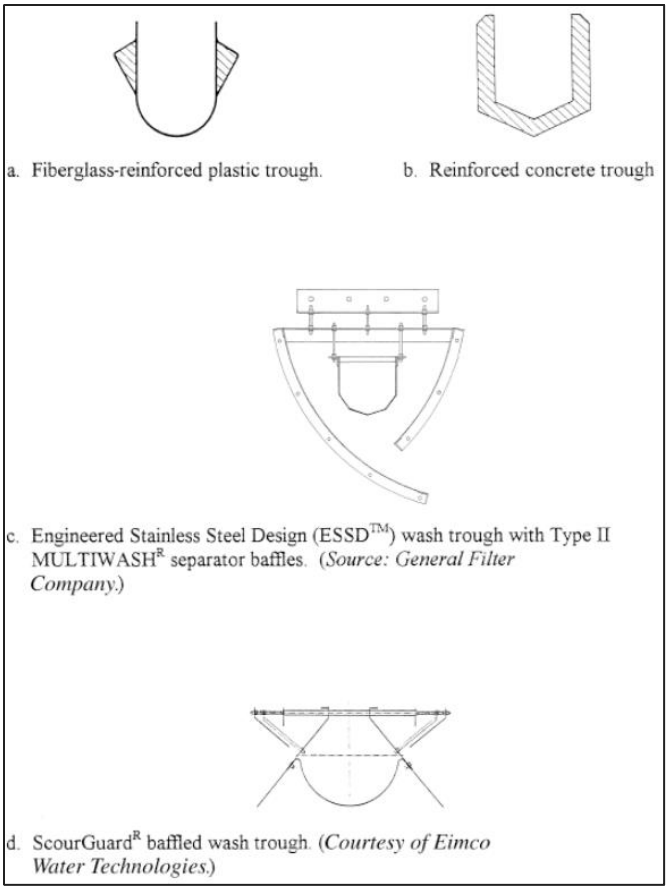


Figure 52 Backwash Trough Examples (AWWA and American Society of Civil Engineers, 2012)

14.3.3 Overflow Walls

Depending on the orientation of the overflow walls, the designer should consider mechanisms to promote horizontal travel of scoured solids towards the overflow areas (to waste), such as tilting side weirs, horizontal water jets or *cross-wash* with the filter influent water entering the filter on the opposite side to the overflow wall. (AWWA and American Society of Civil Engineers, 2012).

Edzwald (2011) and Kawamura (2000) recommend that the overflow is at least 500 mm above the filter bed surface and Edzwald (2011) also recommends that the top of the overflow wall is sloped at 45 degrees towards the filter bay to limit media loss by promoting rolling of media grains back towards the filter bed if these fall on the wall during the backwash process.

14.4 Filter Configurations and Conduits

Hydraulic connections are required for filter inlet, filter outlet, backwash water inlet, air scour inlet and waste outlet(s) as well as drain(s).

It is common for filter-related pipework to be located in a gallery at one end of the filter, which is complex and can be crowded (Binnie, Kimber and Smethurst, 2002). It is typical for pipework in the filter gallery to range from 300 – 900 mm (Hendricks, 2010). Typical filter pipework is shown in Figure 53. The design should rather aim for simplicity and easy access to all valves and actuators, which can be considered as the most important design criteria for filter pipework (Kawamura, 1999), and should be easily visible for inspection purposes (Kawamura, 2000). AWWA and American Society of Civil Engineers (2012) reiterates that accessibility to the pipework is required.

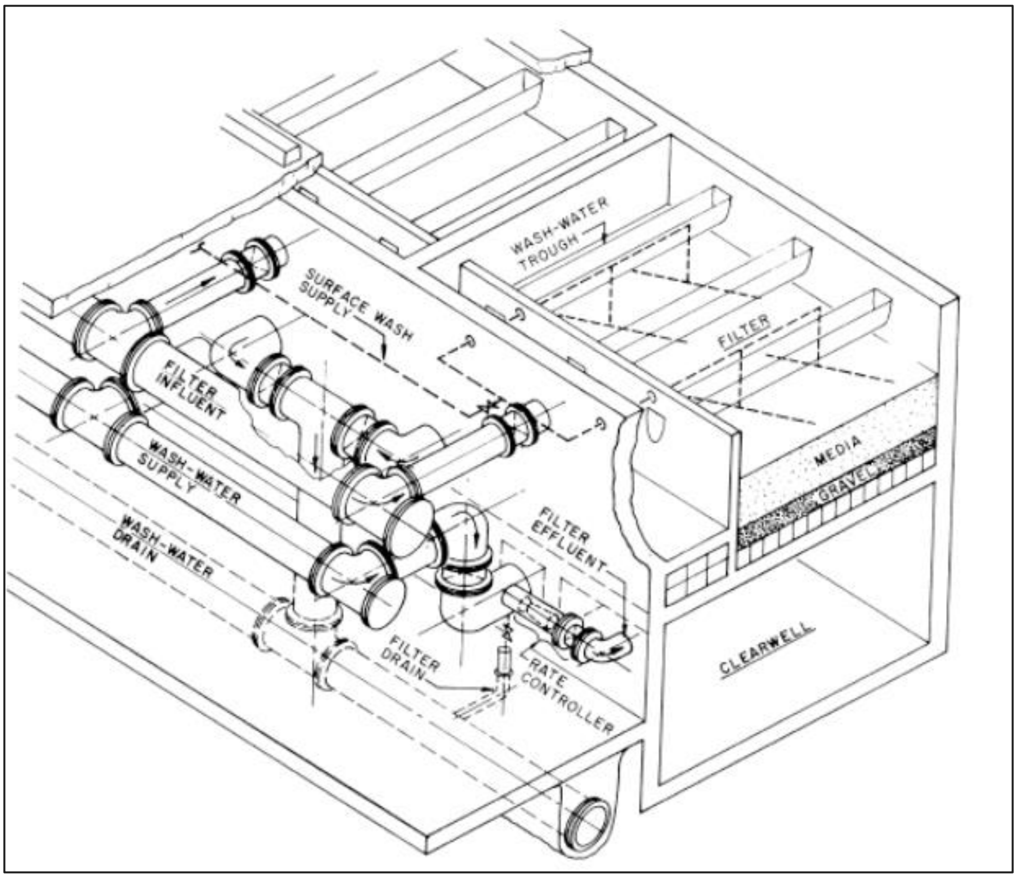
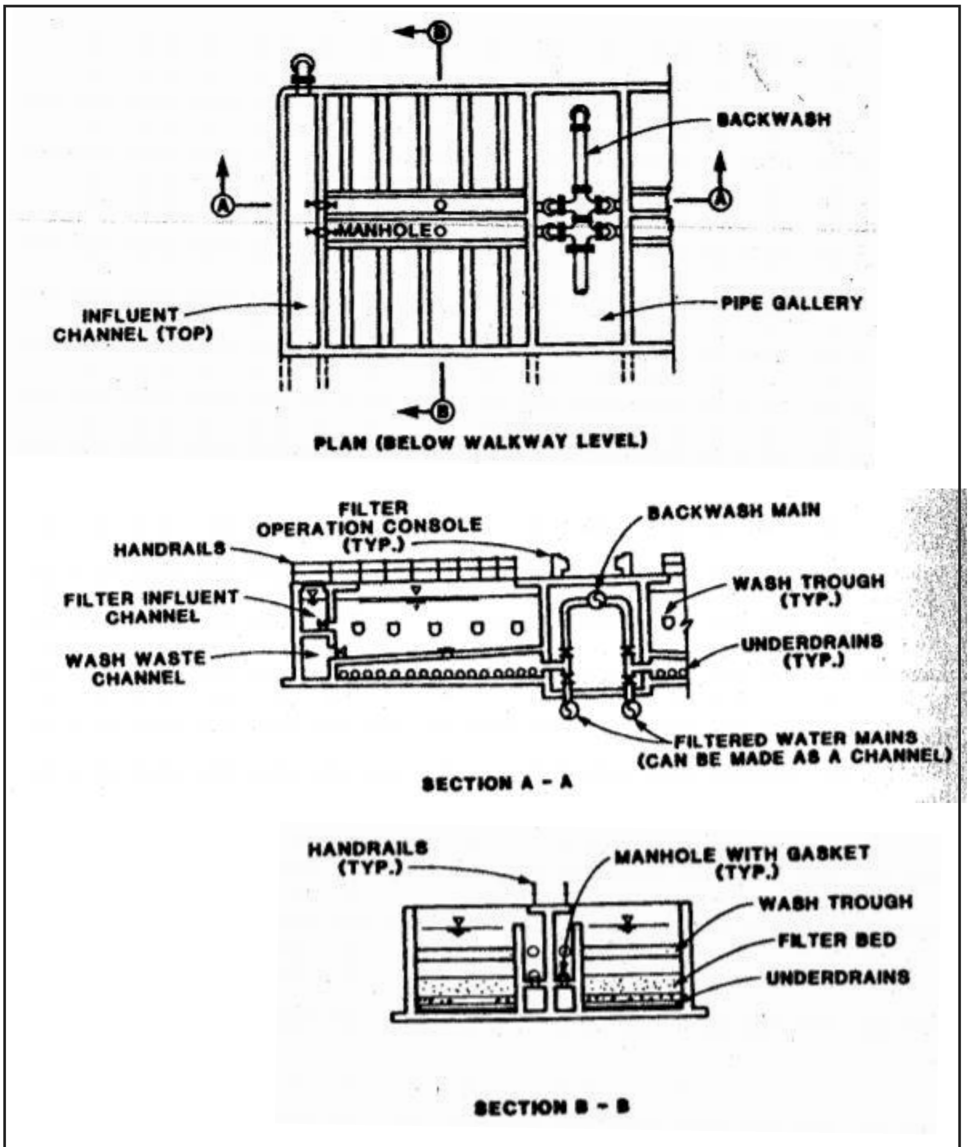


Figure 53 Typical Filter Pipework Configuration (AWWA and American Society of Civil Engineers, 2012)

Pipework should be flanged and provided with mechanical couplings for maintenance purposes (AWWA and American Society of Civil Engineers, 2012). Stainless steel pipework is commonly used for filter pipework and should be labelled and colour-coded (AWWA and American Society of Civil Engineers, 2012). Beverly (2011) recommends EN Grade 1.4301 (304) stainless steel pipework as minimum.

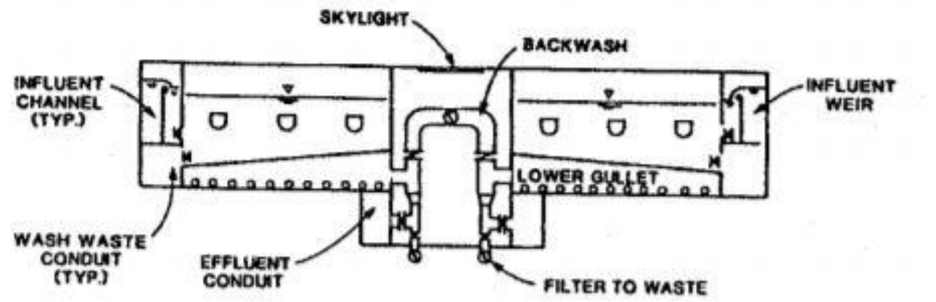
Contamination of filtered and unfiltered should be considered in the design. Kawamura (2000) advises that these pipes should not even be installed in the same pipe gallery. Hendricks (2010) additionally promotes the inclusion of air gaps between waste pipework and flumes/channels to prevent contamination. If the waste and drain pipework are interconnected as illustrated in Figure 53, the installation must prevent backflow from the drain to the filter; e.g. by adding a check valve into the drain lines (AWWA and American Society of Civil Engineers, 2012).

Some other filter layout examples are presented in Figure 54 below. Note that these filter layouts do not indicate filter outlet flow meters or filter-to-waste pipework in every instance; however, these should be provided where required for the design.

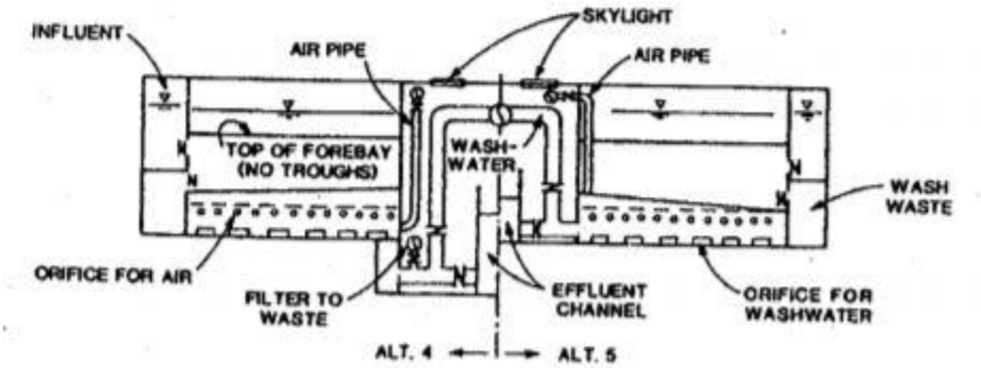


a) basic filter layout

Figure 53 Typical Filter Layout Configurations (Kawamura, 2000)



b) filter layout with surface wash



c) filter layout with air scour alternatives

Figure 54 Typical Filter Layout Configurations (Kawamura, 2000).

The filter layout design should also avoid free fall onto the media or turbulence at the media to prevent disturbing the media, which can be achieved by providing an inlet below the water surface or baffling the stream (AWWA and American Society of Civil Engineers, 2012).

A summary of the velocity considerations for conduits is presented in Table 24, upfront of the discussion thereunder.

Table 24 Velocity Considerations for Filter Conduits

Conduit	Velocity Range (m/s)		Reference
Forebay channel		0.15	(Kawamura, 2000)
Raw water inlet	1	2	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
Flocculated water inlet	0.3	0.6	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
Inlet pipework		0.6	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
		0.9	(Kawamura, 2000)
Inlet valves		1.5	(Kawamura, 2000)
Backwash water inlet pipework	2.5	3.5	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
		2.4 (incl. valve)	(Kawamura, 2000)

Conduit	Velocity Range (m/s)		Reference
	0.9	1.8	(AWWA and American Society of Civil Engineers, 2012)
		1.5	(Beverly, 2011)
Filtered water pipework	2	5	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
	0.9	1.8	(AWWA and American Society of Civil Engineers, 2012)
		1.5	(Kawamura, 2000)
Backwash waste pipework	1.2	1.5	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
		2.4	(Kawamura, 2000)
Filter-to-waste pipework	3.5	4.5	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)
		5.2	Kawamura (1991) as cited in Hendricks (2010)
Air scour pipework		~12	van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)

A maximum velocity of 0.15 m/s is suggested in the forebay channel (Kawamura, 2000); i.e. the inlet to the filters that also serves as a waste backwash water collection chamber and diversion to waste backwash water drain (AWWA and American Society of Civil Engineers, 2012).

Where manifolds (i.e. pipes distributing to and/or collecting from several filters) are used, these must be sized such that head losses therein are small to ensure approximate equal flow at each lateral pipe or orifice (Hendricks, 2010).

Van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997) indicates raw water inlet velocities in the region of 1 – 2 m/s, but 0.3 – 0.6 m/s for flocculated influent water. Inlet pipework velocities should be limited to around 0.6 m/s for the reasons discussed for the inlet channel (AWWA and American Society of Civil Engineers, 2012), although Kawamura (2000) limits this to 0.9 m/s and further suggests a maximum of 1.5 m/s for the inlet valves. Inlet flumes instead of inlet pipework might be more practical for larger water treatment plants (AWWA and American Society of Civil Engineers, 2012).

Despite van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997) indicating that backwash water supply pipe sizing can be based on 2.5 – 3.5 m/s and Kawamura (2000) limiting this to 2.4 m/s, it is normally based on velocities in the range of 0.9 – 1.8 m/s (AWWA and American Society of Civil Engineers, 2012) because higher velocities increase the head losses with water hammer possibilities. Beverly (2011) recommends a maximum of 1.5 m/s for backwash pipework. High points in the backwash water supply pipework should be avoided and air relief valves must be installed if high points exist (AWWA and American Society of Civil Engineers, 2012). These should be release valves and not combined air/vacuum release valves (Beverly, 2011) .

Filtered water pipework velocities can be in the region of 2 – 5 m/s (van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)); although AWWA and American Society of Civil Engineers (2012) indicates 0.9 – 1.8 m/s as normal and Kawamura (2000) limits the velocity in the filtered water pipework to 1.5 m/s.

Kawamura (2000) limits the backwash waste pipe sizing to a velocity of 2.4 m/s, but van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997) provides a guideline of 1.2 – 1.5 m/s.

Filter-to-waste pipework velocities should be limited to 5.2 m/s (Kawamura (1991) as cited in Hendricks (2010)), but 3.5 – 4.5 m/s is a guideline velocity range (van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)).

Air scour pipework is typically sized for a velocity of 12 m/s (van Duuren (1992) as cited in Van Duuren, South Africa Water Research Commission (1997)).

14.5 Gates, Valves and Actuators

The following valves are typically provided for a filter:

- filter inlet
- filter outlet
- backwash water inlet
- waste backwash water outlet
- air inlet
- surface wash water inlet
- filter-to-waste
- drain

Aultman (1959) states that gate valves and sluice gates were used for filters in the past, but that gate valves have been replaced by butterfly valves due to shorter installation lengths, better control capabilities, reduced maintenance and easier operation. However, gate valves might still be favourable in some instances (Hendricks, 2010). AWWA and American Society of Civil Engineers (2012) indicates that larger water treatment plants commonly install sluice gates for filter inlet and waste backwash water outlet.

Butterfly valves are the most common filter valve type (Beverly, 2011; Hendricks, 2010), and are typically still used for rate control valves provided they are selected for operation in the 30 to 60 degree open range (Beverly, 2011). Outside of this range will result in large flow changes for small disc movement, which can result in hunting for the setpoint. Centric rubber-lined butterfly valves are commonly used due to their overall lengths (AWWA and American Society of Civil Engineers, 2012).

Stability and accuracy are essential for modulating filter outlet valves, and careful design is necessary to avoid cavitation. AWWA and American Society of Civil Engineers (2012) indicates operation in the 15 – 65% open region. Kawamura (2000) mentions plug and ball valves for control purposes.

A rate control/pressure reducing valve is required for gravity backwash water sources as shown in Figure 55 (Beverly, 2011).

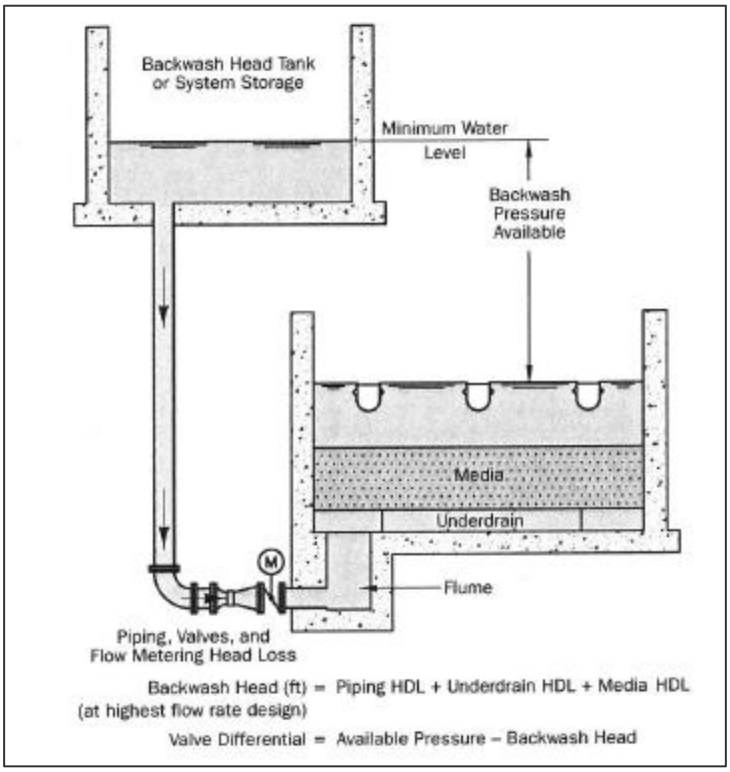


Figure 55 Gravity Backwash Components (Beverly, 2011)

Actuation options include hydraulic, pneumatic and electric actuators. Hydraulic actuation is no longer utilised for this application due to problematic leaks and blocked lines (AWWA and American Society of Civil Engineers, 2012). The advantages and disadvantage of pneumatic and electric actuation are listed in Table 25.

Table 25 Pneumatic versus Electric Actuation of Filter Valves and Sluice Gates

Actuation Type	Advantages	Disadvantages
Pneumatic	Less expensive[1]	Require oil- and moisture-free air[1]
		Hunting issues (over- and under-shooting) for control valves[2]
		Many types open in surges that can cause hydraulic shock[3]
Electric	Good control feature; offer smooth, consistent operation[2,3]	Technical maintenance[1]
	Diagnostic functions[3]	
	Easy to alter in future (programming)[3]	
1 AWWA and American Society of Civil Engineers (2012) 2 Edzwald (2011) 3 Beverly (2011)		

Valve speed of 60 to 120 seconds (Beverly, 2011) is recommended for quarter turn actuators (common for 0 – 90° travel for butterfly and ball valves).

Valve deadbands must be considered. Deadband is the acceptable error/tolerance on a setpoint which must exist and not too be too small which would otherwise cause valve wear and possible failure. Typical tolerance is 5%. (Beverly, 2011).

It is recommended that valves and sluice gate actuators provide local position indication and that the installation promotes easy access for maintenance purposes (AWWA and American Society of Civil Engineers, 2012).

14.6 Blowers and Backwash Pumping Systems

A blower system typically consists of the following components (Beverly, 2011), as depicted in Figure 56 below:

- Blower with inlet and outlet silencers
  - Blowers must be selected to overcome the static water level and head losses through the pipework and valves (and height of j-riser, if applicable), including a safety margin, as shown in Figure 57 below. Additionally, temperature must also be considered for selection and design must be according to the worst-case temperature (i.e. high temperature) (Beverly, 2011).
  - Positive displacement blowers are preferred to centrifugal blowers because the flow rate remains fairly constant irrespective of the pressure changes.
- Inlet filter – protects blower, flow meter and contamination of water in filter underdrain
- Pressure relief valve for positive displacement blowers
- Electrically actuated modulating vent valve for air delivery regulation from positive displacement blowers, particularly for air pipework purging at the start of the air scouring process
  - Sized for operation at 30 – 60° open during purge
  - Minimum 90° rotation period of 1 minute which will assist with reducing hydraulic surges in the manifold/filter
- Pipework (including j-risers for dual parallel lateral underdrains) with optional high loop for water protection of blowers.
  - Bottom entry air:
    - Air pipework installed below the filter's water surface level will commonly be filled with water during filtration. Therefore, the water would need to be purged out of the pipework and is done at a low rate to avoid over-pressurising the air system due to excessive head losses at high rates which would cause surging and could even lead to support gravel upset. Therefore, majority of the air pipework should be installed above the filter water level to reduce the purge period. (Beverly, 2011).
    - For parallel lateral underdrains, the J-risers must also be designed properly as mentioned earlier.
  - Top entry air:
    - The air header can be installed above the filter water surface level and the air distribution pipework can positioned within the filter bed as follows (Beverly, 2011):

      Above support gravel

      Below support gravel between laterals for example

      Drop pipework directly into the underdrain when direct media retention is used
    - This system is more suited to retrofitting existing filters rather than designing of new filters due to the additional opportunity for backwashing issues due to the drop pipework within the media bed (Beverly, 2011).

    - For top entry air, purging is not normally required because the small amount of air in drop pipes does not normally cause problems (Beverly, 2011).
  - Kawamura (2000) suggests headers be located at least 600 mm above the highest water level in the filters.
  - If j-risers are installed, then the required flow rate is crucial to proper operation.
- Check valve for water protection of the blowers
- Actuated filter air inlet valve
- Flow meter – for flow verification and setting of vent valve

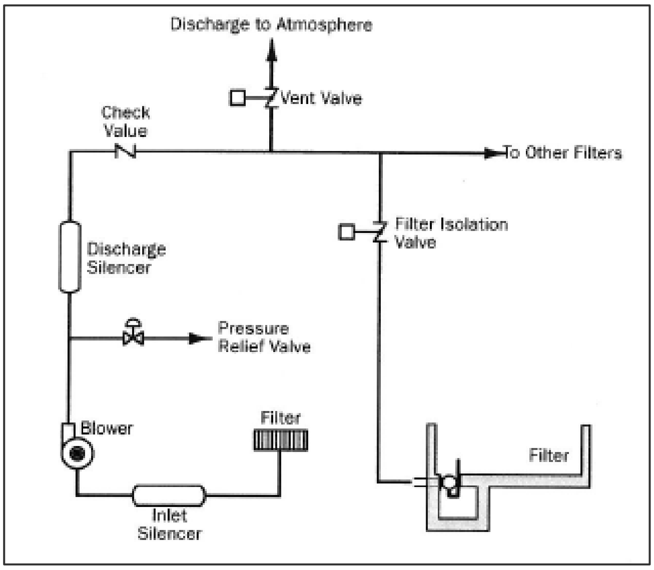


Figure 56 Air Scour Equipment – Blower System (Beverly, 2011)

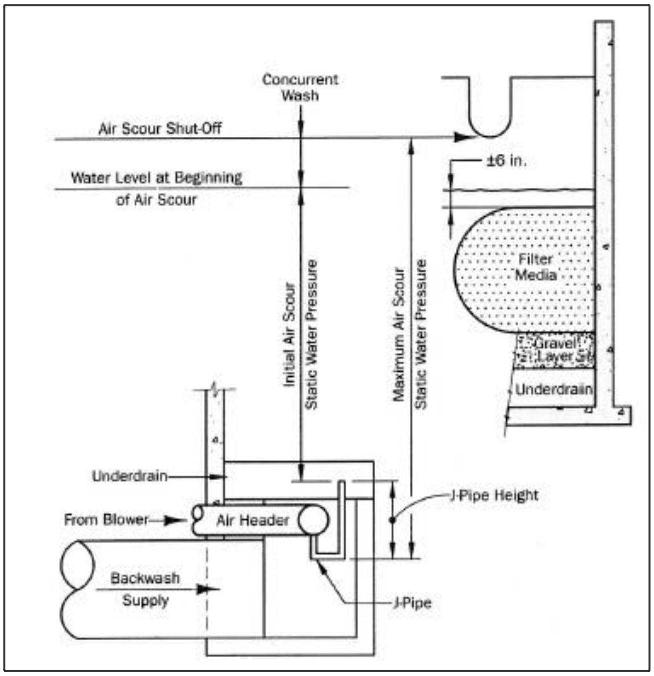


Figure 57 Blower Selection Pressures Based on Parallel Lateral Underdrain (Beverly, 2011)

The literature indicates that backwash pumps are often vertical turbine pumps with the motor and discharge head above floor level (Beverly, 2011) as shown in Figure 58. This configuration is susceptible to air entrainment through the air valve between the source water level and the check valve when not in operation (Beverly, 2011). Careful consideration for air release valve design and backwash sequencing is necessary to release the air prior to reaching the filter is required (Beverly, 2011).

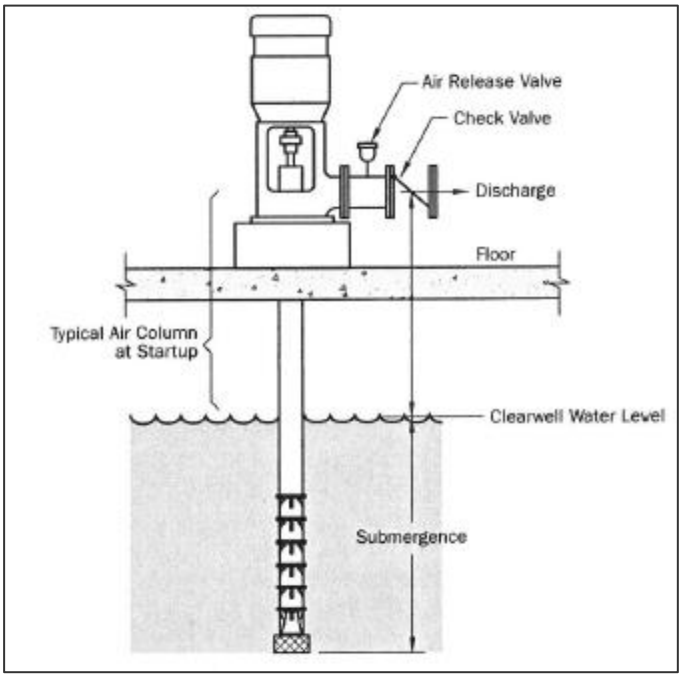


Figure 58 Typical Backwash Pump Installation – Vertical Turbine Pump (Beverly, 2011)

Other pump types in South African observed by the author for backwash pumping systems include end suction and horizontal split case centrifugal pumps.

Backwash pump design must incorporate pressure to overcome the static head (the largest variable in backwash pump pressures), head losses through the pipework, underdrain head loss and media head loss. A required backwash pressure of 6 – 7.5 m is common (Beverly, 2011), but could be up to 20 m (Kawamura, 2000).

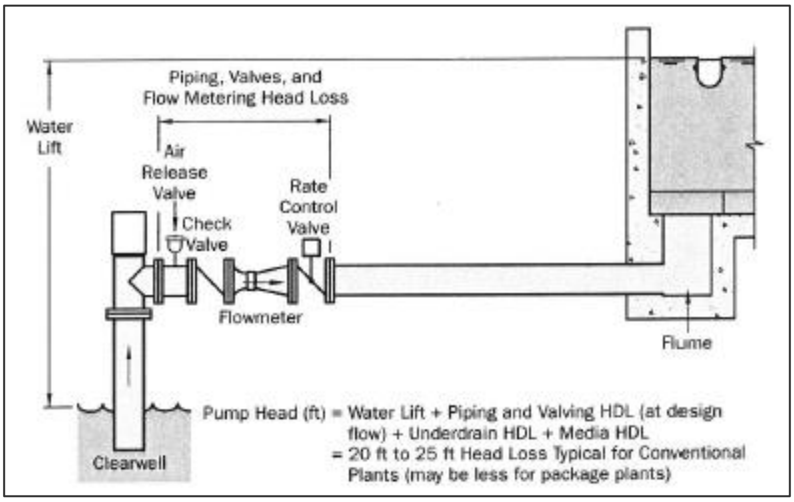


Figure 59 Backwash Pump Selection Pressures (Beverly, 2011)

It may be difficult to obtain a pump for larger filters with a high flow rate and this low head application. A control valve or variable speed drive can assist. This is also an option for the low and high flow rates required by one system, as well as pumps operating in parallel (Beverly, 2011). According to Hofkes *et al.* (1981), pumps should be provided in a two duty, one standby configuration with one duty, one standby only utilized for small installations.

Variable speed drives are also beneficial for backwash pumps for slow backwash flow starting, providing a set backwash rate, altering flows for restratifying media (if applicable) and slow stopping of backwash flow (Hendricks, 2010).

Every high loop or point in backwash pipework must be provided with an air release valve to release any accumulated air in the system. Air release only valves must be used and not combination air release valves that would allow air into the pipework. Additionally, a check valve should be provided for each air release valve to prevent any syphoning possibilities where air will enter the system due to relatively high backwash water velocities. For long high loop sections, an air release valve on either end should be installed. (Beverly, 2011).

Backwash pump operation (and backwash supply volume size) should be level controlled to prevent backwash from starting with insufficient volume available, and to stop backwashing at a low level before entraining air as mentioned in Chapter 15.4 (Beverly, 2011).

14.7 Design Tool Incorporation

Chapter 16.1 details the development of the tool that incorporates a sheet for sizing various components (some of which impact on the head loss calculations); namely "Filter Conduit & Valve Sizing" as per the screenshot below:

	A	B	C	D	E	F	G
1							Section Reference
2			Filter Conduit and Valve Sizing				14.4
3	Parameter	Symbol	Value				Unit
4	Filtration Rate (i.e. all filters online)	Q_f	0.367				m^3/s
5	Maximum Filtration Rate (i.e. x filters	$Q_{f,max}$	0.400				m^3/s
6	Backwash Water Rate 1	v_{bw1}	1.050				m^3/s
7	Backwash Water Rate 2	v_{bw2}	1.167				m^3/s
8	Air Flow Rate	Q_a	1.400				m^3/s

	A	B	C	D	E	F	G	H	I
9	Conduit/Valve	Flow Rate	Recommended Maximum Allowable Velocity	Area	Estimated Diameter	Proposed Diameter	Proposed Velocity		
10		m^3/s	m/s	m^2	mm	mm	m/s		
11	Filter Inlet	0.367	0.9	0.407	720.1	600	1.30	check diameter to limit maximum velocity	0.9 m/s max for pipework; 1.5 max for valves
12		0.400	0.9	0.444	752.1		1.41	check diameter to limit maximum velocity	
13	Air Inlet	1.400	25	0.056	267.0	300	19.81	ok	20-25 m/s is generally acceptable
14	Backwash Water Inlet Rate 1	1.050	1.8	0.583	861.8	900	1.65	ok	0.9-1.8 m/s is generally acceptable
15	Backwash Water Inlet Rate 2	1.167	1.8	0.648	908.4		1.83	check diameter to limit maximum velocity	0.9-1.8 m/s is generally acceptable
16	Filter Outlet	0.367	1.5	0.244	557.8	600	1.30	ok	0.9-1.5 m/s is generally acceptable
17		0.400	1.5	0.267	582.6		1.41	ok	0.9-1.5 m/s is generally acceptable
18	Filter-to-Waste	0.367	5.2	0.070	299.6	300	5.19	ok	3.5 - 4.5 m/s is generally acceptable, with a maximum of 5.2 m/s
19			Flow Rate = Area x Velocity						
20									

Refer also to Appendix section A11 for further guidelines to the designer on how to use this section of the tool.

CHAPTER 15 : COMMON FILTER PROBLEMS

15.1 Introduction

A few references identified during the literature review describe filter problems. Despite these details being more relevant to investigation of existing filter designs, which is not the purpose of this dissertation, the details are included herein for the designer's awareness.

Filter problems can be categorized as follows (Beverly, 2011):

- Process-related, such as inadequate pre-treatment, changing raw water quality and hydraulic overload
- Mechanical/material-related, such as uncontrolled air and hydraulic shock, commonly attributable to quick opening of valves or control sequencing issues

Beverly (2011) warns that all equipment controls for filter operation should always be tested prior to operating the filter so as not to damage the filter, mainly the underdrain system, support gravel and media classification).

AWWA and American Society of Civil Engineers (2012) discusses the following common filter problems:

- inadequate pre-treatment
- inadequate backwashing
- support gravel upset, also referred to as a "blown" filter
- air binding
- restart after shutdown
- filter media replacement

One less common, but interesting issue is liquefaction of the filter bed caused by earthquakes (Kawamura, 2000).

It may be possible to observe various media issues caused by these problems if the water level in the filter is drained to the top of the filter bed during the backwash sequence (Logsdon, 2008).

These problems are discussed further hereunder.

15.2 Inadequate Pre-treatment

Even with a good filter design, a filter will fail if the pre-treatment is inadequate.

AWWA and American Society of Civil Engineers (2012) advises to aim for influent turbidity of less than 2 ntu with a maximum influent turbidity of 4 ntu for pre-treated waters to prevent a short filter run time and uneconomically frequent backwashing. AWWA and American Society of Civil Engineers (2012) also notes that lime-softened water generally has a turbidity greater than 4 ntu, but does not reduce filter run time due to the "compact and incompressible nature of the solids".

Jar tests and pilot studies can inform the design and operation in terms of optimizing chemical dosing.

15.3 Inadequate Backwashing

Inadequate backwashing can be caused by insufficient backwash rates, maldistribution of backwash water and air, underdrain blockages and gravel support layer upsets. This leads to poor filtrate quality and mudball formation within the filter bed.

Mudballing is the agglomeration of residuals sticking to the filter media that remain in the filter bed because they are too large to reach the waste backwash outlets and cannot be removed during the backwash process. The mudballs can grow heavier and can sink into the filter media, creating impassable regions that leads to higher effective filtration rates, poorer filtrate quality and shorter filter run times. (AWWA and American Society of Civil Engineers, 2012).

Although air scour is effective at preventing mudball formation, any formed mudballs may have to be manually removed (AWWA and American Society of Civil Engineers, 2012). Surface mudballs can be removed by using a strainer while backwashing, breaking them up with a rake or high-pressure water jet, whereas deeper mudballs would require chemical soaking of the bed (typically acidic), probing with high-pressure jets, pumping the media through an ejector or even digging out and replacing of the media (Edzwald, 2011).

Maldistribution due to flume problems are commonly identifiable as shown in Figure 60. These are not easily remedied as baffles may be required at the flume entry or orifices added or altered at the entry to the laterals (Beverly, 2011).

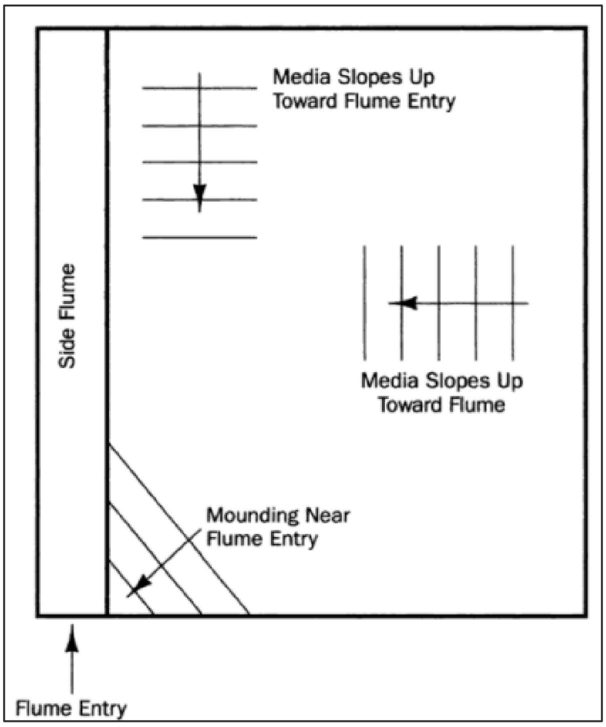


Figure 60 Potential Flume Problems (Beverly, 2011)

15.4 Uncontrolled Air

Uncontrolled air is defined as the accumulation of air in backwash water pipework, which can result in water hammer (i.e. shock waves due to sudden changes in flow) when air enters the filter – see Figure 61.

Uncontrolled air can originate from the backwash pumping system through entrained air or exceeding the minimum pump submergence, or through various fittings in the backwash water pipework such as air release valves, filter valves and other fittings (Beverly, 2011).

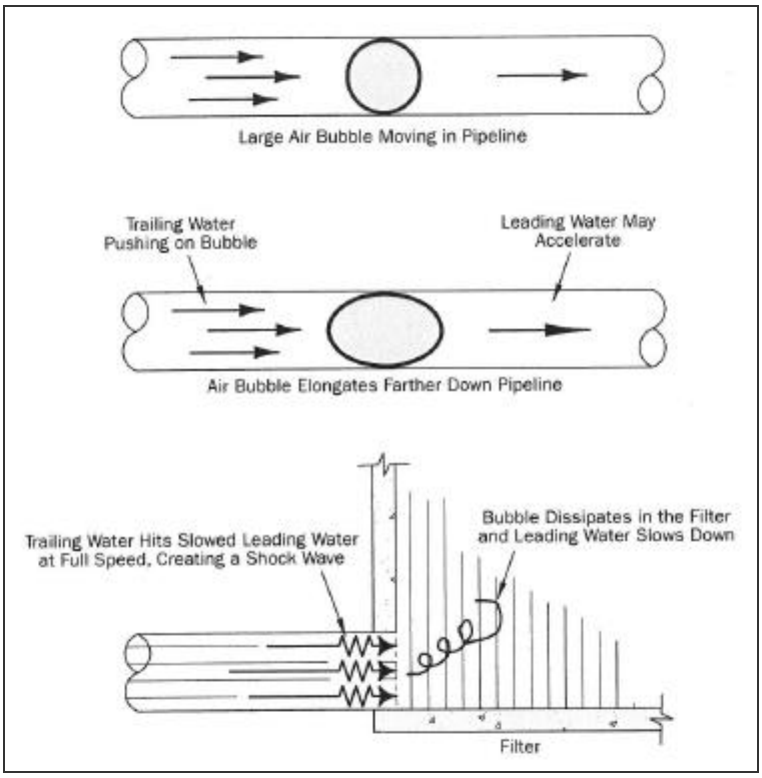


Figure 61 Uncontrolled air and water hammer possibilities (Beverly, 2011)

Air in the backwash system can originate from the backwash pump depending on the type of pump (such as vertical turbine pump) and from rotation at the pump delivery resulting in a central air bubble. The easiest method of relieving a central air bubble is through an air release valve when pumping against a closed valve (Beverly, 2011). Low flow rates may not be capable of purging all the air from the pipework, but the higher flow rate that follows will usually address this issue. If the air release at the higher flow rate is substantial, additional air release valves can be strategically placed to assist. (Beverly, 2011).

Other causes of air in the backwash pipework includes high loops in the pipework where air can accumulate (Beverly, 2011).

Air can be introduced from the water source by exceeding the minimum pump submergence or vortex from a low water level in an elevated water storage tank. Vortex baffles are a solution to this. (Beverly, 2011).

15.5 Support Gravel Upset

Support gravel upset is generally due to uncontrolled air or hydraulic shock (Beverly, 2011). AWWA and American Society of Civil Engineers (2012) notes the following possible causes of support gravel upset:

- backwashing at too high a rate
- (re-)commissioning a dry or drained filter without first filling with a low-rate backwash
- the backwash water inlet control valve is opened too quickly
- air is trapped in the backwash water supply pipework and enters the filter uncontrollably
- air is trapped in the underdrain system and enters the filter uncontrollably
- support gravel installation, gradation and layer thickness are not in accordance with specifications (as per AWWA Standard B100, filter media suppliers' and underdrain suppliers' recommendations)

Typically, a hole is blown through the layers as shown in Figure 62, allowing intermixing and media to leak through to the underdrain. Mudballing and craters where media is lost into the underdrain can result. Other symptoms include poor filtrate quality and shorter filter runs. Underdrain failure could result if left unattended. (Beverly, 2011).

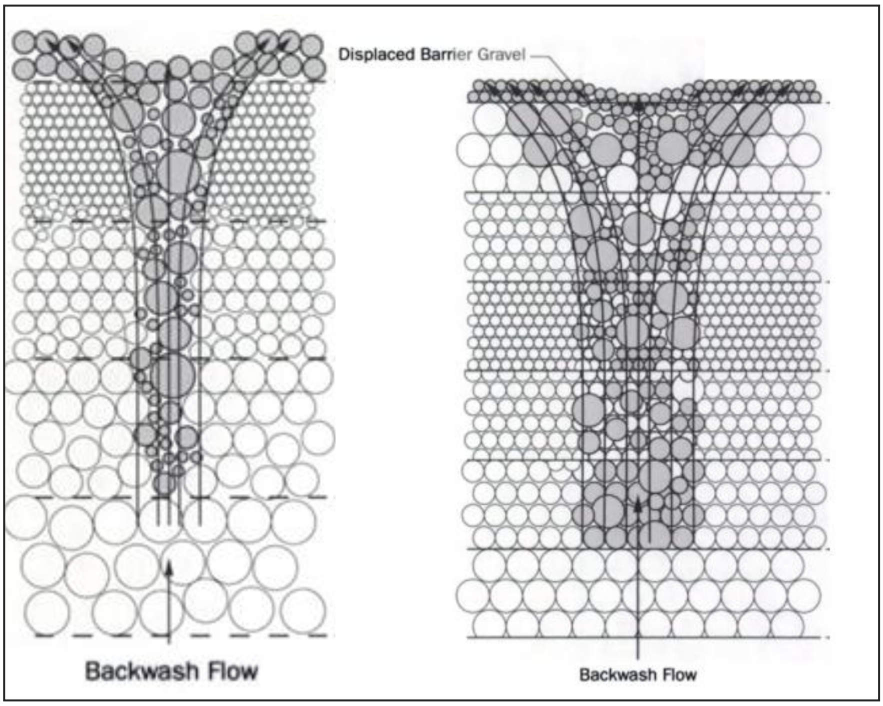


Figure 62 Upset support gravel (Beverly, 2011)

Disrupted gravel can occur slowly after being in operation for 15 to 20 years (Beverly, 2011).

Removing media is time-consuming and therefore the condition of the gravel support should be investigated by probing the filter at least once per year. This is achieved by lowering a pipe through the media during backwash until the gravel is felt gently and the level is marked on the probe. A difference of more than 12 mm between probing locations and changes over time indicate that a problem is developing, whereas 19 – 25 mm is indicative of failure. Upset gravel will feel soft. (Beverly, 2011).

Damage to the support media (or to the underdrain system for that matter) could result in a concentrated high rate through a concentrated location of the filter bed during backwashing, termed “boiling” as shown in Figure 63 (Logsdon, 2008).

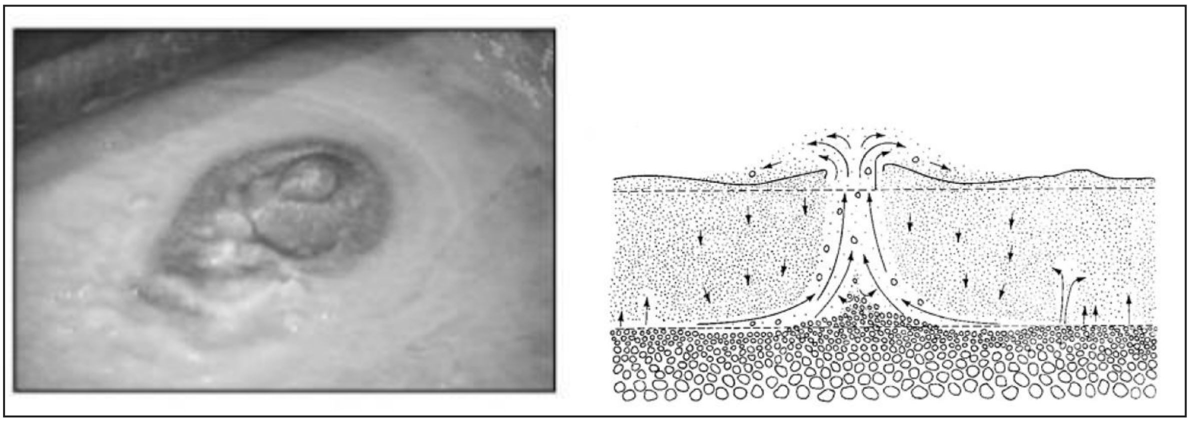


Figure 63 Filter Boil during Backwash Photo (AWWA (2000), as cited in Logsdon (2008)) and Image Showing Displacement of Media (Edzwald, 2011)

Upset support gravel (or a damaged underdrain system) requires manual regradation; i.e. the filter media has to be removed to resolve the issue.

15.6 Air Binding

Air binding occurs when air accumulates in the filter bed and obstructs the water path, thereby increasing the effective filtration rate and increasing head loss (AWWA and American Society of Civil Engineers, 2012).

Air can be introduced to the filter if negative pressures are introduced in the filter with resulting air escaping, and if the source water cascades and is aerated or has a high concentration of dissolved gasses that reaches saturation (particularly possible at increased water temperatures and during algal blooms) (AWWA and American Society of Civil Engineers, 2012).

Air binding commonly occurs a few centimetres into a monomedia filter bed or just below the dual-media bed interface. The air escapes when the filter outlet valve is closed, which can agitate the bed and result in media loss. (AWWA and American Society of Civil Engineers, 2012). Air bubbles can therefore also remove media during backwash (Beverly, 2011).

Air binding can be detected visually by seeing uniform bubbles across the filter when a backwash is started (Beverly, 2011).

The air is formed in the media and will generally not cause damage to any support gravel (Beverly, 2011).

The only easy solution is to backwash more often, but ideally this is a design issue based on positive driving head or negative pressures (Beverly, 2011).

Note that air binding does not occur in declining rate filters due to their design (AWWA and American Society of Civil Engineers, 2012) (see Chapter 7 :).

15.7 Restart After Shutdown

It is preferable that filters operate continuously. However, operators may stop the water treatment plant at night, decommission filters during low-demand periods, cycle the filters or even shut down the plant for long periods, which encourages biological activity or, if the filters are drained, loss of ripened media. (AWWA and American Society of Civil Engineers, 2012).

Restart of a filter with water in the filter bay should include a short backwash or filter-to-waste period to ensure adequate filter performance (AWWA and American Society of Civil Engineers, 2012). Kawamura (2000) suggests backwashing of filters upon restart for filters that have been standing still for more than 3 days. Logsdon *et al.* (2002) lists filter-to-waste as one of the unguaranteed options when restarting a filter without backwashing, and highly suggests that backwashing is implemented.

Restart of a drained filter should follow the commissioning procedure to prevent gravel upset or media separation, by filling with a low-rate backwash and filter-to-waste period for good performance. Note that AWWA and American Society of Civil Engineers (2012) indicate that the initial filling rate for a drained filter should not exceed 5 – 7 m/h to ensure that the rising water does not compress air in the filter bed that can channel the flow through the media and disturb the gravel support.

Filling from the bottom prevents air entrainment.

15.8 Filter Media Replacement

Filter media can be lost during normal operation over the year(s), or due to filter issues such as air binding, low waste backwash water removal systems, improper backwash sequencing of equipment, excessive backwashing including improper surface wash and air scour control, leakage through support gravel (support gravel upset) or underdrain, mismatched media in dual-media and mixed-media filters and low freeboard from inlet to media (AWWA and American Society of Civil Engineers, 2012; Beverly, 2011).

Note that soft and mushy GAC may be the result of the media exceeding its adsorptive capability (Beverly, 2011).

Logsdon *et al.* (2002) suggests that the distance from the top of the filter bay to filter media be recorded to establish the extent of filter media loss and also explains that the levelness of the filter media can be inspected by draining the water to a level that just reaches the filter media. Depressions will fill with water and mounds will protrude above the water level, providing a useful visual indication of the filter media level as well as its condition.

High areas in the filter surface are typically due to a low localized backwash velocity and a low area due to localised high backwash velocity. A disturbed media surface can also be a result of fast influent water onto the media surface and low water level within the filter. A low water level could result in a hole gouged into the media when receiving influent over the inlet weir for example. Fast influent could also cause holes or mounds if the momentum of the flowing water is not dissipated, commonly done by means of baffles or splash plates. (Beverly, 2011).

These conditions could result in gravel upset, but are typically noticed when a filter is put into service.

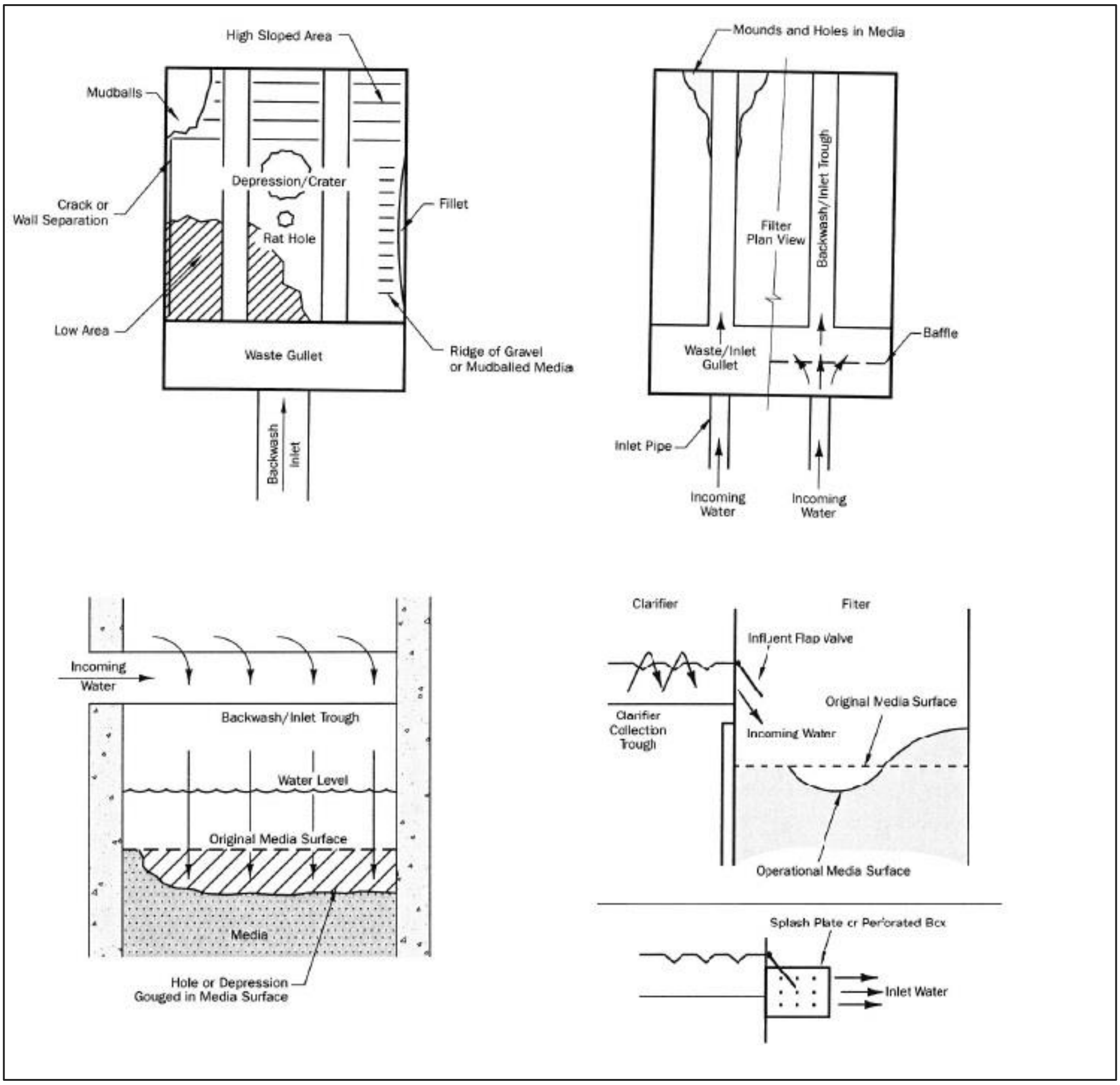


Figure 64 Media Surface Condition Issues (Beverly, 2011)

Large shallow depressions are evidence of a possible failure in progress, whereas a crater typically indicates catastrophic failure. Rat holes can extend from the surface through to the underdrain. (Beverly, 2011).

Cracks in the media and separations of media from the sidewalls can be a result of excessive polymer in the influent or poor backwashing that has not led to complete mudballing (Beverly, 2011).

A sidewall depression is wider than a crack and is typically an indication of support gravel failure (Beverly, 2011).

Filters within buildings prevent debris from entering filters, which would otherwise accumulate in the filter and may even sink to the bottom of the media and cause further issues (Beverly, 2011).

15.9 Liquefaction due to Earthquakes

Kawamura (2000) directs attention to earthquakes with magnitudes greater than 5 that could lead to turbidity breakthrough in filters. If this occurs, the plant should be stopped, or the filtered water should be directed to filter-to-waste. Upon filter restart, it is recommended that the filters are lightly backwashed for 3 minutes.

CHAPTER 16 : DESIGN TOOL DEVELOPMENT, CONCLUSIONS AND RECOMMENDATIONS

16.1 Design Tool Development

The design tool was developed in Microsoft Excel due that best suited the application. The file is included with this dissertation with the name “MEng Filter Design Tool_RWXLAU001_Rev0_202201.xlsm”.

The calculations and outputs of the tool were determined from aspects identified during the literature review. The various sheets within the tool and incorporation of the reviewed aspects are documented in the relevant chapters within this dissertation, including for screenshots from the tool itself.

The design tool additionally includes for the following sheets for reference information to assist in determining parameters as required by other sheets within the tool:

- “Water Properties – reference” that provides the water density and dynamic viscosity related to the design temperature

	A	B	C	D	E	F
1	**Water Properties for Various Temperatures**				ref: (Hendricks, 2010; p778)	
2	**Temperature**	**Density**	**Dynamic Viscosity**			
3	***t***	ρ_w	μ			
4	**°C**	**kg/m³**	**N.s/m²**			
5	0	998.8426	1.79E-03			
6	1	999.9015	1.73E-03			
7	2	999.9429	1.67E-03			
8	3	999.9672	1.62E-03			
9	4	999.975	1.57E-03			
10	5	999.9668	1.52E-03			
11	6	999.946	1.47E-03			
12	7	999.9043	1.43E-03			
13	8	999.8509	1.39E-03			
14	9	999.7834	1.35E-03			
15	10	999.7021	1.31E-03			
16	11	999.6074	1.27E-03			
17	12	999.4996	1.24E-03			
18	13	999.3792	1.20E-03			
19	14	999.2464	1.17E-03			
20	15	999.1016	1.14E-03			
21	16	998.945	1.11E-03			
22	17	998.7769	1.08E-03			
23	18	998.5976	1.05E-03			
24	19	998.4073	1.03E-03			
25	20	998.2063	1.00E-03			
26	21	997.9948	9.78E-04			
27	22	997.773	9.54E-04			
28	23	997.5412	9.32E-04			
29	24	997.2994	9.11E-04			
30	25	997.048	8.90E-04			
31	26	996.787	8.70E-04			
32	27	996.5166	8.51E-04			
33	28	996.2371	8.33E-04			
34	29	995.9486	8.15E-04			

- “Pipework & Fittings – reference” for the designer to estimate pipe roughness and fittings friction coefficients in the sheet named “Other Head Losses”.

	A	B	C	D
1	**Pipe Surface Roughness (e)**			
2	**Material**	**Smooth**	**Average**	**Rough**
3		mm	mm	mm
4	Stainless steel	0.015	-	-
5	Hot dip galvanized steel/iron	0.06	0.15	0.3
6	Lined Steel	0.03	0.06	0.15
7	HDPE	0.015	0.03	0.06
8	PVC	0.015	0.03	0.06
9	Uncoated Cast iron	0.15	0.3	0.6
10	Uncoated Steel	0.015	0.03	0.06
11				

	A	B	C	D	E	F	G	H
12	**Fittings Friction Coefficients and Qauntities**							
13	**Fittings**	**Friction coefficient**	**Filter Outlet Pipework**		**Backwash Pump Pipework**		**Backwash Manifold Pipework**	
14		**K**	**Quantity**	**ΣK**	**Quantity**	**ΣK**	**Quantity**	**ΣK**
15	**Entrances**							
16	Square (small to large)	1		0.0		0.0		0.0
17	Square (large to small)	0.5		0.0		0.0		0.0
18	Square (large to small - inward projecting)	0.8		0.0		0.0		0.0
19	Rounded/Bellmouth	0.05		0.0		0.0		0.0
20	**Exits**							
21	Rounded	0.2						
22	All other exits	1		0.0		0.0		0.0
23	**Bends**							
24	**Miter Bends**							
25	90deg	1.25		0.0		0.0		0.0
26	60deg	0.55		0.0		0.0		0.0
27	45deg	0.26		0.0		0.0		0.0
28	30deg	0.16		0.0		0.0		0.0
29	**90 Degree Bends**							
30	Short Radius	1		0.0		0.0		0.0
31	Medium Radius	0.6		0.0		0.0		0.0
32	Long Radius	0.25		0.0		0.0		0.0
33	**Reducers**	1		0.0		0.0		0.0
34	**Tees**							
35	To branch	1.8		0.0		0.0		0.0
36	From branch	1.5		0.0		0.0		0.0
37	angled branch	1.25		0.0		0.0		0.0
38	straight through	0.6		0.0		0.0		0.0
39	**Valves**							
40	Swing check valve	2		0.0		0.0		0.0
41	Gate valve	0.5		0.0		0.0		0.0
42	Butterfly valve (fully open)	0.5		0.0		0.0		0.0
43	Butterfly valve (50% open)	7.5		0.0		0.0		0.0
44	Globe valve	10		0.0		0.0		0.0
45	Diaphragm valve	2.5		0.0		0.0		0.0
46	Needle valve	0.5		0.0		0.0		0.0
47	Foot valve	1		0.0		0.0		0.0
48	**Sluice gate**	0.5		0.0		0.0		0.0
49	**Flow meter (full bore)**	0		0.0	1	0.0		0.0
50				**0.0**		**0.0**		**0.0**
51								

- “Backwash Channel Width – ref” provides a reference for selecting a Backwash Channel Width to be incorporated in the sheet named “Filter Number, Size & Rate”.

Water Depths at Various Channel Widths				
Safety Factor for Depth	10			%
Backwash Water Rate 1	1.05			m^3/s
Backwash Water Rate 2	1.17			m^3/s
Gravitational Acceleration	9.81			m/s^2
	During Air Scour (Backwash Water Rate 1)		During Rinse (Backwash Water Rate 2)	
Channel Width (BC_{width})	Critical Depth (d_c)	Depth at Upstream End (D_0)	Critical Depth (d_c)	Depth at Upstream End (D_0)
m	m	m	m	m
0.300	1.08	2.05	1.16	2.20
0.350	0.97	1.85	1.04	1.99
0.400	0.89	1.69	0.95	1.82
0.450	0.82	1.57	0.88	1.68
0.500	0.77	1.46	0.82	1.57
0.550	0.72	1.37	0.77	1.47
0.600	0.68	1.29	0.73	1.39
0.650	0.64	1.23	0.69	1.31
0.700	0.61	1.17	0.66	1.25
0.750	0.58	1.11	0.63	1.19
0.800	0.56	1.07	0.60	1.14
0.850	0.54	1.02	0.58	1.10
0.900	0.52	0.99	0.56	1.06

ref: (van Duuren, 1997; p198)

6-16% for friction losses in the system

$$d_c = \sqrt[3]{\frac{Q^2}{gBC_{width}^2}}$$

$$D_0 = \sqrt{d_c^2 + \frac{2Q^2}{gBC_{width}^2 d_c}}$$

- “Backwash Trough Width – ref” provides a reference for selecting a Backwash Trough Width to be incorporated in the sheet named “Filter Number, Size & Rate”.

Water Depths at Various Trough Widths				
Number of Troughs	2			-
Trough Length	7			m
Safety Factor for Depth	10			%
Backwash Water Rate 1	1.05			m^3/s
Rate 1 per trough	0.53			m^3/s
Backwash Water Rate 2	1.17			m^3/s
Rate 2 per trough	0.58			m^3/s
Gravitational Acceleration	9.81			m/s^2
	During Air Scour (Backwash Water Rate 1)		During Rinse (Backwash Water Rate 2)	
Trough Width	Critical Depth (d_c)	Depth at Upstream End (D_0)	Critical Depth (d_c)	Depth at Upstream End (D_0)
m	m	m	m	m
0.300	0.68	1.29	0.73	1.39
0.350	0.61	1.17	0.66	1.25
0.400	0.56	1.07	0.60	1.14
0.450	0.52	0.99	0.56	1.06
0.500	0.48	0.92	0.52	0.99
0.550	0.45	0.86	0.49	0.93
0.600	0.43	0.81	0.46	0.87
0.650	0.41	0.77	0.43	0.83
0.700	0.39	0.73	0.41	0.79
0.750	0.37	0.70	0.40	0.75
0.800	0.35	0.67	0.38	0.72
0.850	0.34	0.65	0.36	0.69
0.900	0.33	0.62	0.35	0.67

ref: (van Duuren, 1997; p198)

decide if along filter length or width and reference Filter Number, Size & Rate Sheet (12m or 7m applies here)

6-16% for friction losses in the system

$$d_c = \sqrt[3]{\frac{Q^2}{gBC_{width}^2}}$$

$$D_0 = \sqrt{d_c^2 + \frac{2Q^2}{gBC_{width}^2 d_c}}$$

Refer to the Appendix for guidance to the designer on how to use/populate information within the filter design tool.

16.2 Conclusions and Recommendations

This dissertation consolidates various filter information and includes a design tool.

The design tool is a computer program developed in Microsoft Excel and is intended for exclusive use by the author’s employer at the time of submission. The tool itself provides a mechanism for the filter designer to easily review the effects of various interlinked parameters in developing and refining a filter solution. Furthermore, this design tool enhances the plant-wide water treatment design tool that incorporates a high-level filter design sheet previously developed by Morrison (2019).

A guideline for the designer on the use of the tool itself is presented in the Appendix. Any results from the tool should still be compared with the designer’s own calculations.

The following further studies and developments are recommended:

- Further enhancements to the tool might include for the following:
  - Filter configuration outputs, including component levels and water operational levels based on the control method
  - Backwash pump system design
  - Blower system design
- A further specific study should be undertaken to test the tool on a practical level and assess its effectiveness on various designs or existing filter systems, and make improvements to the tool.
- Furthermore, the tool can be used to analyse various scenarios particularly when pilot plants are not possible, and hence it would be beneficial to have a record of previously designed filters and related

information, particularly within the South African context. This should include for a record of available media and its properties, as well as the available underdrain systems.

- Additionally, a record of implemented designs with current parameters and operational details would aid filter designers further.
- A laboratory scale pilot plant setup would also benefit filter designers to be able to further investigate parameters and their effects.

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APPENDIX A: FILTER DESIGN SPREADSHEET GUIDANCE NOTES

A1. General

The filter design tool is a computer program developed in Microsoft Excel.

Designer inputs are colour-coded as follows: 

Non-colour coded cells are parameters and/or calculated values that do not require designer intervention.

Various section references are provided in the spreadsheet for easy reference to the applicable sections of this dissertation; for example:

Section Reference 4.3

Various side notes are included in red text for designer review/information.

The following sections present notes for the designer according to the Microsoft Excel Sheets.

A2. Water Properties

This sheet includes the density and dynamic viscosity of water for various temperatures as a reference to define these properties for the selected design water temperatures.

A3. Influent Water Properties

Establish/choose a design temperature for the influent water.

A4. Filter Media – Design

Choose the media bed type from the drop-down list for the filter bed configuration – monomedia, dual-media or mixed-media (Cell B2).

Choose various filter media parameters for the media making up the filter bed. Note that Row 11 will indicate whether the media is included in the design or not, based on the selected media bed type.

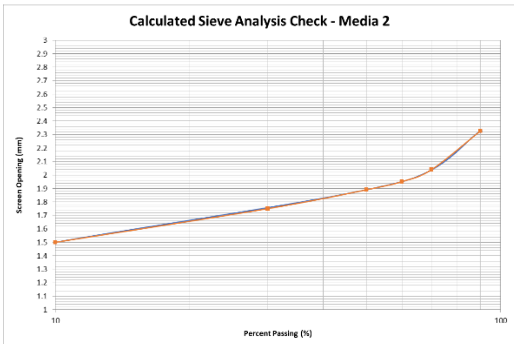
Note that the actual bed depth per media (Rows 25 and 26) will be zero if the media is not included in the design (based on the selected media bed type), irrespective of the selected bed depth per medium (Row 24).

Example inputs and outputs are depicted below.

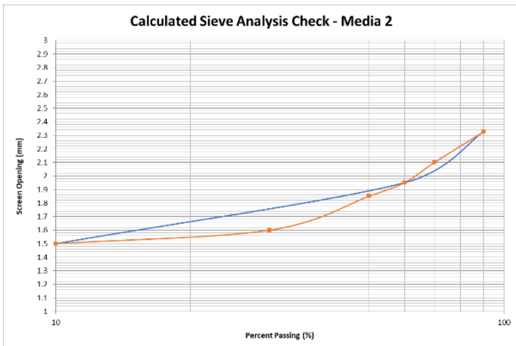
	A	B	C	D	E	F	G
1	**Filter Configuration**		Section Reference				
2	Media Bed Type	Monomedia	4.3				
6	Number of Media	1					
7							
8	**Filter Media Parameters**						Section Reference
9	Parameter	Symbol	Media 1	Media 2	Media 3	Unit	
10	Medium	-	Anthracite	Sand	Garnet	-	
11	Included in Design for Filter Configuration	-	no	yes	no	-	
12	Effective Size	ES or d10	1.3	1.5	0.25	mm	4.2.2 ; 4.3
13			0.0013	0.0015	0.00025	m	
14	Uniformity Coefficient	UC	1.3	1.3	1.3	-	4.2.2
15	Loose/initial bed porosity	ε_0	0.58	0.4	0.58	-	4.2.5
16	Sphericity	ψ	0.53	0.75	0.6	-	4.2.5
17	Medium media grain size	d_{60}	1.7	2.0	0.3	mm	4.2.2
18			0.0017	0.0020	0.0003	m	
19	Large media grain size	d_{90}	2.0	2.3	0.4	mm	4.2.2
20			0.0020	0.0023	0.0004	m	
21	Equivalent Spherical Diameter	deq	1.31	1.69	0.26	mm	4.2.5
22			0.0013	0.0017	0.0003	m	
23	Media Density	ρ_m	1600	2650	3950	kg/m^3	4.2.7
24	Selected bed depth per medium	L medium	450	1500	75	mm	4.3
25	Actual bed depth per medium		0	1500	0	mm	
26			0	1.5	0	m	
27	Bed depth to size ratio per medium	L/ES medium	0	1000	0	-	4.5
28	Total Bed Depth	L	1500			mm	
29			1.5			m	
30	Accumulative Bed Depth to Size Ratio	L/ES	1000			-	4.5

Each media layer is divided into 5 layers for the calculation of head loss and bed expansion. Media grain sizes d_{10}, d_{60} and d_{90} values are automatically populated. These are inserted in a sieve analysis curve to assist in choosing values for d_{30}, d_{50} and d_{70}. Choose these values from the graph, and ensure that the overlay curve is a match.

<u>MATCHING OVERLAY CURVE EXAMPLE</u>



<u>NON-MATCHING OVERLAY CURVE EXAMPLE</u>



A5. Filter Media – Example Analysis

This sheet has been included as a filter media check during filter construction (not specifically necessary for the design).

Insert the screen size and percentage of media retained as received from the laboratory/media supplier. A grading analysis curve is generated. Insert the d_{10} and d_{60} values established from the curve and compare the Effective Size and Uniformity Coefficient to the design values. The bed depth to size ratio is also calculated for comparison with acceptable ranges depending on the filter configuration (see Chapter 4.5).

A6. Filter Number, Size & Rate

This sheet involves an iterative selection process to determine the design number of filters, filter sizing and filtration rate. The design selection is summarised at the top of the sheet. The inputs, calculations and selections are detailed within the sheet.

Choose the number of offline filters for the design. The design flow rate per filter (q_{design}) for various calculations is based on this reduced number of filters receiving the water treatment plant design flow rate.

The filter area, width and length are first estimated based on the filter design flow rate (q_{design}) and a chosen desired filtration rate ($v_{desired}$) and desired filter length-to-width ratio ($r_{l/w\ desired}$).

Obtain various underdrain dimensions from local suppliers. The filter width and length are then further updated (w_{res} and l_{res}, respectively) based on the number of underdrain panels or laterals that can fit within the filter area. Propose the final filter width (w) and length (l).

The actual filtration rate for all filters online (v) and the maximum filtration rate with the chosen number of offline filters (v_{max}) is determined from the chosen filter dimensions. Compare these values to the desired filtration rate and check whether the increase in filtration rate when filters go offline (Δv) is within the typically accepted range.

This sheet also estimates the filter construction size and minimum height, but note that this can only be established once various head losses are defined.

A7. Fluidization

This sheet determines the minimum fluidization velocity (v_{mf}) for the various media and water temperatures. Take note of the safety factor for this critical design parameter.

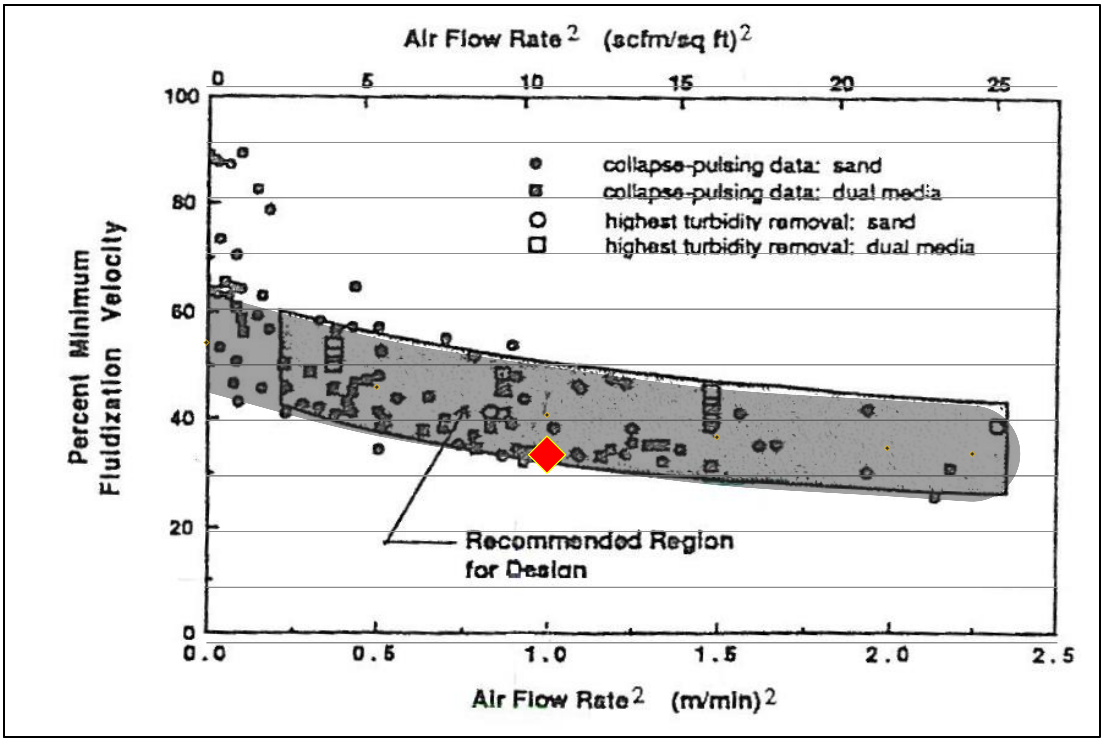
A8. CP Backwash & Design Rates

Collapse pulse backwashing with air and water is the most effective backwash method. This sheet provides an estimate for the backwash rates required for collapse pulse backwashing to occur. There is no estimate for garnet, but note that the garnet has a much lower minimum fluidization velocity compared to sand and anthracite.

Propose the air flow and backwash water rates for the design.

Note that for coarser media that will require a high minimum fluidization velocity, the designer might consider increasing the air flow rate to reduce the required backwash rate and still achieve collapse pulse backwashing. Another consideration is the sizes of pumps, valves and pipework.

A graph is provided to give an indication of the selection compared to the recommended combinations for collapse-pulse backwashing.



A9. Bed Expansion

This sheet determines the percentage bed expansion and expanded bed depth (L_e) for the various media based on the proposed backwash water rate.

An initial expanded bed porosity is inserted in Rows 18, 35 and 52; and then is calculated by clicking the button 

for the respective media (to automatically run an iterative process).

Check whether the total bed expansion is within the typically accepted range and if not, change the backwash rate accordingly.
Note that the expansion correlation must be solved whenever the backwash water rate is changed.

A10. Media Head Losses

This sheet determines the head loss through the filter media for the various design filtration and backwash rates. A summary table presents the calculated head losses based on the original Ergun equation and the modern Ergun equation, should the designer wish to use the more conservative original Ergun equation. The workings follow below the summary table.

A11. Filter Conduit & Valve Sizing

This sheet helps to establish various conduit and valve sizes based on prescribed maximum allowable velocities.

Note that the conduit/valve size must cater for the varying flow rates experienced.

Choose a suitable nominal diameter and confirm the velocity is still acceptable – the note will indicate "ok" if the resultant velocity is below the recommended maximum, and will indicate "check diameter to limit maximum velocity" if not.

A12. Other Head Losses

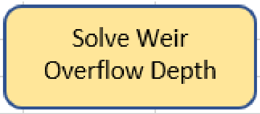
This sheet estimates head losses through the filter, other than filter media, for various flow rates through the filter (filtration and backwash) for the specific filter configuration and includes for:

- Underdrain head losses
- Pipework head losses
- Weir head losses
- Trough head losses

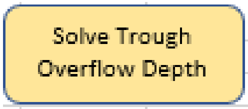
The workings follow below the summary table.

The underdrain head losses include for the underdrain, media retention mechanism, and other minor head losses for entry to the underdrain system

The pipework head losses include calculations for head losses through filter outlet pipework (using the Darcy Weisbach and Barr formulae), including fittings, for filtration and backwash pipework for backwash. The designer can make use of the Sheet named “Pipework & Fittings – reference” Sheet for pipe roughness and fittings friction coefficients to estimate these parameters.

For the weir head losses, an initial weir overflow height (H_{weir}) is inserted in Row 91 and then is calculated by clicking the button 

(to automatically run an iterative process) to determine the actual overflow depth for the various filtration and backwash rates. If a weir is not applicable, enter zero for weir length and do not click the solver button.

For the trough head losses, an initial trough overflow height (H_{trough}) is inserted in Row 105 and then is calculated by clicking the button 

(to automatically run an iterative process) to determine the actual overflow depth for the various filtration and backwash rates. If a trough is not applicable, enter zero for number of troughs and weir length and do not click the solver button. If the trough(s) does not contribute to head loss during filtration, but does for backwashing, then first solve the trough overflow depth and then enter zero for weir lengths for the filtration flow rates.

A13. Pipework & Fittings – reference

This sheet includes various pipe roughness and fittings friction coefficients as a reference to estimate these parameters in the Sheet named “Other Head Losses”.

A14. Backwash Channel Width – ref

This sheet provides a reference for selecting a Backwash Channel Width to be incorporated in the Sheet named “Filter Number, Size & Rate”.

A15. Backwash Trough Width – ref

This sheet provides a reference for selecting a Backwash Trough Width to be incorporated in the Sheet named “Filter Number, Size & Rate”.